NSF/RA-780305

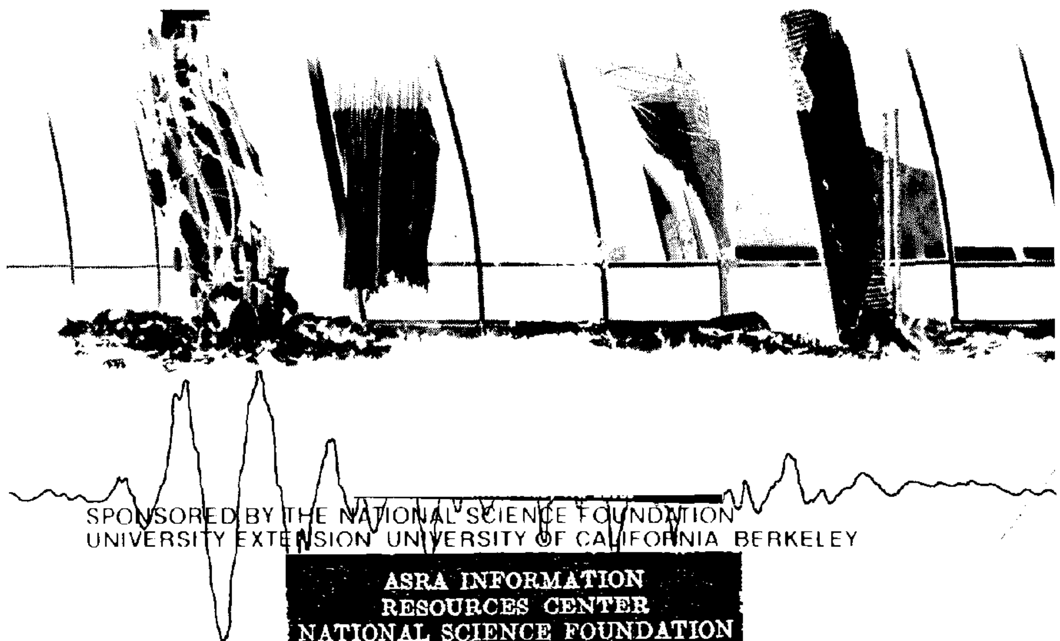
PROCEEDINGS
OF A WORKSHOP ON

EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION

July 11-15, 1977

V. V. BERTERO, ORGANIZER

VOL. III—TECHNICAL PAPERS



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EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION

Proceedings of a Workshop Held at
The University of California
Berkeley, California
July 11-15, 1977

In Three Volumes

Sponsored by the National Science Foundation
Grant No. NSF/ENV76-01923

Vitelmo V. Bertero, Organizer
Stephen A. Mahin, Organizing Secretary

Steering Committee:

William E. Gates
Neil M. Hawkins
John B. Scalzi
Mete A. Sozen
Loring A. Wyllie, Jr.

VOLUME III
TECHNICAL PAPERS

UNIVERSITY OF CALIFORNIA
UNIVERSITY EXTENSION
BERKELEY, CALIFORNIA

Issued June 1978

Preface

The material contained in these three volumes constitutes the proceedings of a workshop on Earthquake-Resistant Reinforced Concrete Building Construction (ERCBC) sponsored by the National Science Foundation, and held at the University of California, Berkeley, July 11-15, 1977. The main purposes of the workshop were to provide a means for the exchange of information related to the state-of-the-art and state-of-the-practice in the design and construction of seismic-resistant reinforced concrete buildings, to evaluate current progress, and to establish research needs and priorities for future work.

The specific objectives and organization of the workshop are summarized in the Introduction to the first volume. The final recommendations of the workshop form the main body of that volume. Four appendixes follow, containing the program, the list of participants, the list of working groups, and, lastly, a research directory.

Volumes 2 and 3 comprise the technical reports and papers that were presented. These furnished the background material for the discussions which ultimately resulted in the final recommendations of the workshop.

It is hoped that these proceedings will help mitigate the destructive effects of earthquakes by encouraging practitioners to implement those recent findings from the research and professional communities that will improve current practice in ERCBC, and by providing researchers and agencies sponsoring research with guidelines for ensuring that future research is oriented toward solving current problems. It is also hoped that the proceedings will serve to stimulate communication and improve cooperation between practitioners, educators, researchers, and representatives from industry and government agencies working in the field of ERCBC.

It is not possible here to thank all the individuals who contributed to the success of the workshop, but a few should be mentioned. The assistance of Dr. John B. Scalzi, Manager of the Earthquake Engineering Program of the National Science Foundation, during the planning of the workshop, and his continuous support and encouragement are gratefully acknowledged. The able assistance of Dr. Stephen A. Mahin, who acted as organizing secretary, throughout all phases of the workshop is greatly appreciated. In addition, thanks must be extended to the members of the steering committee: W. Gates, N. Hawkins, J. Scalzi, M. Sozen, and L. Wyllie, Jr., for their technical assistance; to the session chairmen; the heads and recording secretaries of the working groups; to H. Barry and L. Reid of University Extension for coordinating schedules, arranging accommodations, and making the workshop an enjoyable experience for all the participants; and to L. Tsai, not only for invaluable editorial assistance in the preparation of these volumes, but for her continued help throughout the various phases of the workshop. Finally, special and sincere appreciation goes to the authors of the technical reports and to all the participants, who took time from their busy schedules to collaborate in the workshop. The success of the workshop is the result of their individual and combined efforts.

Funding for this workshop was made possible by grant ENV76-01923 from the National Science Foundation. Their support is gratefully acknowledged. These proceedings constitute the final report to the sponsor. The conclusions and recommendations expressed herein do not necessarily reflect the views of the National Science Foundation.

Vitelmo V. Bertero
Berkeley, California
June 1978

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

DESIGN OF REINFORCED CONCRETE MOMENT-RESISTING FRAMES

by

Donald R. Strand
Structural Engineer
Brandow & Johnston Associates

INTRODUCTION

The design of moment-resisting frames of reinforced concrete for resistance to seismic loads has been upgraded significantly over the last few decades. Prior to that time, frames were designed for code design loads with minimal confinement of concrete and minimal use of continuous reinforcing steel. Present practice requires that the members and their connections be "ductile" in nature.

Prior to 1961, frames were designed for the combined effect of code vertical and seismic loads. Beam members were designed for the moments and shears induced; bottom bars were lapped at the supports; top bars were added through the midspan; and nominal stirrups were added throughout the full span. Reinforcing was sized for the design loads at the particular section considered, and little consideration was given to reversed cyclic loads. Column members were also designed for the moments and shears induced, bars being lapped at the floor, and generally more ties were added immediately above the floor and below the beam. Some engineers designed all of the shear in the elements to be carried in the stirrups or ties.

In 1961, the Portland Cement Association [1] published "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions". The information contained in this book related to provisions for ductile characteristics to the members. Thus, members were to be designed for ultimate capacity criteria and stirrups and ties for the shear induced from the ultimate flexural capacity. Further tests by PCA [2,3] provided data for the design of the beam-column connections.

The provisions for seismic design are generally based on the Structural Engineers Association of California "Recommended Lateral Force Requirements and Commentary" [4] as adopted in the Uniform Building Code [5]. In 1963, SEAOC deleted the 13-story height restriction on buildings and required moment-resisting space frames for buildings over this height. The 1966 revisions required ductile moment-resisting space frames for the taller buildings, and details and criteria for design were presented. The 1971 revisions included ductile provisions for all concrete lateral resisting frames and limitations on member size or reinforcing steel. The major change, however, was to design for shear loads induced by ultimate moments produced by realistic reinforcing steel strengths above the minimum values specified.

The above summary of design provisions for concrete moment-resisting frames is given only for background. This paper will not attempt to evaluate capacity of frames designed under these various provisions because it is recognized that the latest provisions are more realistic and better from the standpoint of resisting seismic ground motions. The purpose of this paper is to present the design of a typical building and, more notably, the design of the members under the 1976 Uniform Building Code. There will be an attempt to show the effect of design assumptions, allowable code provisions, handicaps imposed by codes or present research, and some reconciliation with the dynamic loads. It is intended that this design will point out areas of further research needs for safe and economical concrete building designs.

DESIGN CONCEPTS

The basic concept used for the elements resisting seismic loads is that the members should not fail in a brittle manner. Thus, shear type failures are to be avoided, and yielding of the steel is anticipated provided stability is maintained by proper anchorage and confinement of the steel and concrete. A second premise is that formation of hinges should be restricted to the beam elements.

Procedure

The design of concrete frames for seismic loads is generally under the provisions of the 1976 UBC. In this Code, methods are presented for obtaining design forces and allowable stresses. An area not covered by the Code is the design of the beam-column intersection; and use of the criteria in the Joint ACI-ASCE Committee 352 [6] report is the recommended approach.

The steps used for design can be outlined as follows:

1. Tabulation of dead loads tributary to each floor.
2. Estimation of building period by arbitrary formulas or past experience.
3. Calculation of code design shear forces tributary to each floor.
4. Distribution of forces to each individual frame and sizing of members--usually by judgment.
5. Distribution of forces to the columns and beams in a frame--usually by joint coefficient procedures.
6. Recheck of building period by Rayleigh or similar methods (using 3/4 of calculated because of non-participating elements) and check of drift limitations.
7. Readjustment of forces for large differences of initial and final periods or drift - this may be by use of a computer program using dynamic force levels.
8. Design of beams and columns.
9. Design of beam-column joint.

Beam-Column Tests

The initial tests performed by PCA [2] were to establish means of providing ductility to elements which are capable of many excursions into the inelastic range. Grade 40 reinforcing was used because higher strength bars were found to be somewhat erratic in nature and sometimes brittle in character. The joint tested was an exterior column-beam condition, and no restraint was provided normal to the plane of loading. The tests proved satisfactory when adequate confinement of bars and the joint was provided. The later tests [3] using 60 grade reinforcing steel also included interior joints and stubs to exterior joints, and also proved to be satisfactory. There were restrictions on the ultimate capacity of the steel to assure a long yield plateau on the stress-strain curve.

Since these tests were prepared to force the yield hinges into the beams, the columns were found to be adequate. The columns, however, were not subject to lateral movements during the testing procedures - the beams were moved vertically. In actual seismic loads, loads are transferred through the columns to the beams, and thus P-Δ effects are generated contrary to load tests. Present thinking is that these P-Δ effects may become significant unless drift limitations are imposed.

A primary short-coming of these tests is that torsional restraint of the floor slabs, offset of beams from the centerlines or member restraint normal to the plane of the frame are not being considered. In actual practice, these other constraints may have a significant effect on design or effect.

The procedure of designing by center-line dimensions needs study. Park [7] did some studies of the rotational ductility demand at the joint and found this value to be on the order of three times the displacement ductility demand. This effect has not been considered along with the joint shear displacement and clear span dimensions.

Cracked and Uncracked Members

The analysis of a concrete member can be drastically influenced by the properties used for design. These would include the concrete strength, type of aggregates (rock vs. light-weight), modular ratio of steel and concrete, the assumption of cracked or uncracked properties, and the influence of T-beam action to floor beams.

A design example in the Appendix illustrates the design of a 12-story concrete frame building. As a follow-up of this example, the influence of using a cracked section for members was undertaken. Assumptions as to the degree of cracking which should be used is not well known. ACI 318-71 gives some parameters in Section 9.5.2.2 for cracked sections when computing deflection; Ferguson in his book on page 740 gives curves for a relationship; and ACI publication SP 17-73, Table 4.1, also gives relationships. Since beams can be assumed as T-beams or rectangular in shape, values can vary from 0.33 to 0.55. Columns likewise may vary considerably from nearly uncracked in lower columns with heavy vertical loads to highly cracked in the upper stories, resulting in values for cracked sections of from 0.28 to 0.40. For comparison of loads for a cracked section, this analysis is based on using 50% of the gross moment of inertia for rectangular beams and an average of cracked and uncracked values (70%) for columns. Table 1 is a summary of the effects of these properties on a frame.

TABLE 1 FRAME LOAD COMPARISON

CONCRETE PROPERTIES	PERIOD-sec.		BASE SHEAR kips (kN)		TOTAL DRIFT inch (mm)		DYNAMIC/CODE	
			CODE	DYNAMIC	CODE	DYNAMIC	SHEAR	DRIFT
UNCRACKED	Code min.	1.20	570 (2535)	3803 (16915)	2.69 (68)	10.27 (262)	6.68	3.82
	Calculated	1.98	450 (2000)	2916 (12970)	2.12 (54)	13.75 (349)	6.48	6.49
	3/4 Calculated	1.48	519 (2310)	3803 (16915)	2.45 (62)	10.27 (262)	7.33	4.20
CRACKED	Calculated	2.47	402 (1790)	--	--	--	--	--
	3/4 Calculated	1.85	464 (2065)	3108 (13825)	4.32 (110)	12.89 (327)	6.70	2.98
UNCRACKED / CRACKED	3/4 Calculated	0.80	1.12	1.22	0.57	0.80	1.09	1.40

Many interesting points are apparent from Table 1.

1. The cracked section should not be used for calculating the building period since it over-estimates the period and consequently gives lower design force levels.

2. The calculated building period by the Rayleigh method should be reduced for design to account for non-contributing resisting elements. This recommendation is contained in the SEAOC Recommendations but not in the UBC.

3. The drift of a frame will increase considerably when cracking occurs in the members. Thus, lower allowable drift limitations at design loads would be appropriate for concrete frames.

4. The relation of shears at design load to dynamic load are meaningless since cracking occurs during seismic events and drift controls the building design.

Ductility Demand

The ductility demand defined as the required displacement versus the yield load displacement can only be estimated because of the many variables unknown during design. The design example in the Appendix is compared at the 3rd and 10th floor levels based on the criteria in Table 2. The ductility demand is evaluated on the basis of the capacity of the member with 25% overstrength of reinforcing and without reduction factors--the same criteria used for determining the member shear capacity. Further, the column is not checked since design concepts are to have a column of greater capacity than the beams. The following data illustrates this procedure:

3rd Floor Characteristics -- The negative moments will have a reserve capacity over vertical load of 1082-65=1017$^{k'}$ (1378 kNm). Since the design load was 475$^{k'}$ (644 kNm), the story deflection to yield is 1017/475 x 0.00196=0.00420 radians. This is compared to the cracked section deflection from the dynamic analysis to give a ductility demand of 0.00729/0.00420=1.73. The positive moment capacity is increased to overcome 0.9 M_{DL} to give 937 + 41=978$^{k'}$ (1326 kNm), and the deflection to yield is 978/475 x 0.00196=0.00404 radians. This results in a ductility demand of 0.00729/0.00404=1.80.

10th Floor Characteristics -- In a like manner, this negative moment reserve is 683-65=618$^{k'}$ (838 kNm) with a design load of 260$^{k'}$ (352 kNm). Yield deflection is then 618/260 x 0.00110=0.00261 radians and gives a ductility demand of 0.00762/0.00261=2.92. The positive moment reserve is 540 + 41=581$^{k'}$ (788 kNm), and the deflection to yield is 581/260 x 0.00110=0.00245 radians. This results in a ductility demand of 0.00762/0.00245=3.10.

TABLE 2 FRAME MEMBER COMPARISON

CONCRETE PROPERTIES	FLOOR	BEAM MOMENT kip-ft (kN-m)	BEAM SHEAR kip (kN)	COLUMN MOMENT kip-ft (kN-m)	COLUMN SHEAR kip (kN)	COLUMN DYNAMIC SHEAR kip(kN)	STORY DRIFT DESIGN radian	STORY DRIFT DYNAMIC radian	DYNAMIC/CODE SHEAR	DYNAMIC/CODE DRIFT
UNCRACKED	3	521 (706)	52.1 (232)	532 (721)	81.9 (365)	507 (2255)	0.00196	0.00595	6.20	3.03
CRACKED	3	473 (641)	47.3 (210)	489 (663)	75.2 (334)	417 (1855)	0.00292	0.00729	5.55	2.48
UNCRACKED	10	280 (380)	28.0 (125)	304 (412)	46.9 (209)	292 (1300)	0.00110	0.00574	6.22	5.21
CRACKED	10	257 (348)	25.7 (114)	280 (380)	43.0 (191)	242 (1076)	0.00176	0.00762	5.62	4.32
UNCRACKED / CRACKED		1.09	1.09	1.09	1.09	1.20	0.64	0.78	1.10	1.20

The interesting comparison of the heavily loaded 3rd floor and lightly loaded 10th floor members indicates that the ductility demand is less at the lower floors and that the reinforcing in the bottom of the beam is usually more critical. This confirms some of the recent test results at the University of California [8] wherein bottom reinforcing of approximately three-fourths of the top reinforcing should be available versus the one-half required in the UBC and may add credence to the requirement for ductility demand to be less than 2.5.

Concrete vs. Structural Steel -- An interesting analogy may be noted when comparing the ductility demand of this concrete frame to an equivalent structural steel member of A36 material. First of all, it is noted that concrete members have reinforcing steel designed for the actual loads, as factored, and based on the desired drift requirement. Structural steel members, however, are generally designed for drift limitations and have much more strength capacity to yield. Second, wherein concrete members will be in a cracked section, structural steel members will be in a plastic condition and more analogous to the uncracked condition.

For this design example, it is shown in Table 2 that the cracked section will deflect about 25% more than the uncracked condition. Thus, the structural steel frame would appear to be capable of being designed to a higher drift limitation than a concrete frame or, conversely, that the concrete frame should be designed to a lesser drift. Also, by using an equivalent stiffness to a steel member, a lesser ductility demand is required than for concrete.

Reinforcing Anchorage

The transfer of reinforcing stresses through the beam-column joint has only recently been documented for design in the Joint Committee 352 report [6]. The initial PCA tests [1,2] indicated that by providing proper confinement and bar development length, adequate ductility would be maintained for seismic events. The more recent tests at Berkeley [9] indicate that transfer through interior beam-column joints may cause serious cracking adjacent to the column and result in much higher deformations.

The Committee 352 recommendations relate primarily to development of hooked bars at the exterior joints and are found to be reasonable for design. Recommendations for design of interior joints are not included, but design can be extended for development of the reinforcing bar through the column as shown in the design example in the Appendix of this paper. The Berkeley tests indicate that small bars are preferred and that ultimate shear stresses in the beam should be kept to less than $3\sqrt{f'_c}$. This is impractical in practice since smaller bars are more costly to handle and placement of a number of bars through the column reinforcing could result in bundling for placement--which does not solve the problem.

The primary rebuttal to overly stringent joint requirements is that many buildings with confinement and anchorage less than the recommended design requirements have survived major earthquake loads. Further, the recent tests on the University of California shaking table have indicated good results on only the one test performed [10], when using less than the recommended size-development criteria.

Shear and Confinement Strength

The design of stirrups or hoops for beams and columns quite generally is dictated by the arbitrary code requirements or to resist maximum member flexural capacity. At the present, the hoop requirements for tied columns or for beams in the hinge region are based on only limited tests. The primary concern is whether supplementary ties must engage both the longitudinal bars and hoop tie or either the bar or hoop. The basis of the code is that as long as the stiffer element is restrained the whole will remain intact.

The transfer of longitudinal bar stresses through the beam-column joint has not been completely resolved. The concepts are, that shear is to be transferred or, that confinement must be maintained to allow development of diagonal compression loads. Stirrup-hoops are thus provided for shear and confinement. At the present time, no tests have been undertaken to evaluate a lapped or capped stirrup-hoop. This type of hooping becomes necessary through the joint in actual construction because closed hoops are virtually impossible to place in this zone because of the beam and column longitudinal reinforcing congestion. The restraint of beams normal to the frame members obviously would have a positive effect on any type of lapped hoop.

Detail Drawings

The presentation of design criteria into working drawings for construction of ductile concrete moment-resisting frames is not an easy task. Obviously, details of all conditions throughout the building cannot economically be drawn. The use of typical beam and column elevations with bars presented in a schedule form is generally accepted, and experience for a proper presentation is a necessity. The schedules must show lengths and offsets of bars and ties or stirrups. A large scale drawing of the beam-column intersection must show the beam and column longitudinal reinforcing with the hoop-ties and supplementary ties through this area.

Because of the long lengths of bars and complexity of bar placement in the field, it is often advisable to list a sequence of placing on the drawings. The corner conditions being more complex because of the 90° tails on all the top and bottom bars are illustrative of the sequence as follows:

1. The column hoop-ties through the beam depth are all stacked above the floor level.

2. The beam side forms are left off one side.

3. The beam stirrups are placed on the beam form soffit.

4. The bottom bars in one direction are threaded through the next adjacent column until the bars are in their proper position. The bottom bars normal to the first series of bars are placed likewise.

5. The hoop-ties are dropped into the beam-column zone. Supplementary ties of longer than required lengths are run through

the joint to extend into the beam--the 135° hook end is placed around the outer hoop of the joint. This is an area where full closed hoops may not physically be possible to obtain.

6. The top bars are placed similar to the bottom bars.

7. Design conditions may require higher strength concrete to be placed in the beam-column intersection.

It has become very important that the engineer discuss the bar placing concepts with the reinforcing and concrete superintendents in the early stages of buildings of this type. The deputy inspector required by many codes should also be included in these discussions. Without this early coordination, many unforeseen problems can develop which only lead to delays and extra costs to construction.

RESEARCH TESTING

The surest approach to developing adequate design methods is through full scale testing. Unfortunately, most testing is not consistent from test to test. It would appear that the PCA tests which included both exterior and interior joint conditions were a good start, and the use of a large shaking table with earthquake ground motion input is a good advancement at this stage. The primary fallacy of the tests is that the conditions of the test specimens are not near enough to actual building construction conditions. In actuality, columns have beams normal to the direction of loads; slabs behave as an integral part of the system; and splices occur in the elements. Further, it is unrealistic to provide heavy reinforcing in a slab parallel to the beam to act as top reinforcing to the beam as used in the shaking table tests.

As a start toward providing better interaction between the practicing engineer and the research team, it is suggested that:

1. The design of the test elements should be based on an actual structure conforming to the latest accepted concepts. The modeling techniques for reduced sizes of test members and deviations should also be noted.

2. The cyclic reverse loading procedures should be used as opposed to monotonic methods because of the wide variation of results between these methods.

3. The reinforcing bars to the joints should not be anchored to fixed reactions since this does not allow the joint to develop normally between the beam and column members.

4. The results of the test should be stressed toward design concepts that can be readily used or which result in changes to code requirements for seismic resistance.

Future Tests

There are a number of past research tests indicated throughout this paper which have been discussed. The future research that may become apparent can be summarized as follows:

1. The effect of an adequate evaluation of the cracked section properties on design must be formulated, especially regarding drift limitations and ductility demand.

2. A means of evaluating drift by the sum of the parts, namely, clear beam span rotational effects, beam-column joint shear displacement and column displacement, is not available.

3. There is not a clearly defined set of parameters established as to minimum acceptable displacement or rotational ductility demand of any seismic resisting system.

4. A test of a code designed frame assembly (in all respects including splices) should be undertaken to see if the criteria really works.

5. Tests are necessary for the beam-column joint of a frame, within the building interior, which has high vertical floor loads. This is to determine if full longitudinal bar development is necessary for good results.

6. It is necessary to verify the limitations of light-weight aggregate in frames.

7. The eccentricity of beams offset from the centerline of columns or biaxial effects to columns from seismic conditions has not been tested.

It is quite apparent that some of these projects can be readily undertaken as graduate studies in the universities. Others undoubtedly will require extensive testing.

CONCLUSIONS

The tools for design of concrete ductile frames to resist seismic forces are presently available in the 1976 Uniform Building Code and as discussed in the 1975 SEAOC Recommendations. These provisions are to provide adequate and, hopefully, better capacity for these structures than previous criteria. Essentially, damage control is the present concept rather than past provisions to accept various degrees of damage during an earthquake. A "massaging" of the present values used for forces or for capacities is only to optimize the overall concept and system.

The design of members in a frame has been presented to illustrate the present requirements. Current test data has been included in the discussion of the design process with the intent to lead into future research needs. It is hoped that the future testing will be oriented toward and with the design profession.

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APPENDIX - DESIGN EXAMPLE

A square twelve-story reinforced concrete building is analyzed in conformance to the 1976 Uniform Building Code to illustrate design procedures. Framing consists of light-weight concrete beam and slabs, and the exterior frames are of 4000 psi (27MPa) rock aggregate concrete. Reinforcing steel is 60 grade material. From a preliminary analysis, the exterior frames are determined to resist 70% of the lateral loads and the interior elements the remainder. The beams are all 16" x 36" deep (400 x 915mm) and the columns 28" square (710mm). The applicable requirements of the UBC are Section 2312 for the seismic loads and Section 2626 for the ductile provisions.

$V = ZIKCSW$

$Z = 1.0$

$K = 0.67$

$I = 1.0$

$T_s = 0.5$ sec

$T = 1.25$ sec (Estimated)

$S = 1.0$

$C = 0.060$

$V = 1615^k$ (7185 kN)

$V_t = 140^k$ (625 kN)

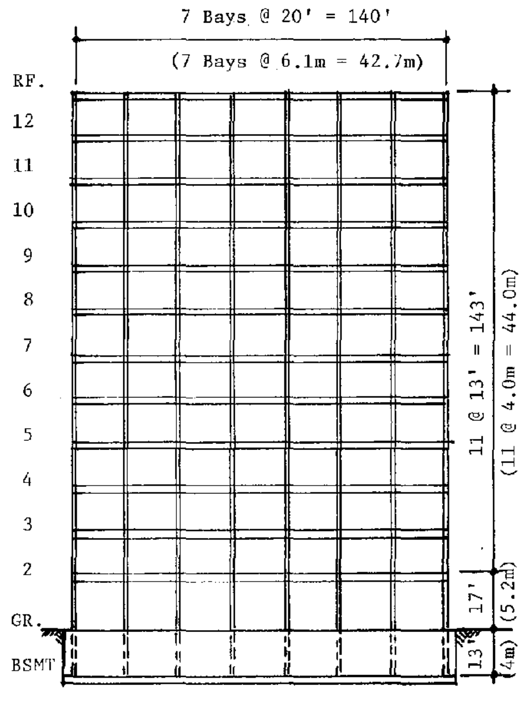
Torsion Increase
$= 3.5^k/fl.$ (15.5 kN)

NOTE: 1" (inch) = 25.4mm

1' (foot) = 0.305m

1^k (kip) = 4.45 kN

$1^{k'}$ (kip-ft) = 1.36 kN-m

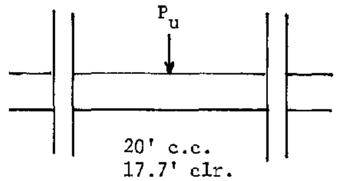


EXTERIOR FRAME ELEVATION

Note- Beams are eccentric to column centerlines

Spandrel Design

Load to Center of Spandrel



$P_{DL} = 104 \text{ lb/ft}^2 \text{x} 10'\text{x}20' = 20.8^k$
$P_{LL} = (50-8) \text{ x}10'\text{x}20' = \underline{8.4}$
29.2

Check for Top Reinforcing

$M_{D+L} = 29.2^k \text{x } 17.7'/8 = 64.6^{k'}$
$M_{Lat} = 53.7 \text{ x } 17.7/2 = \underline{475.2}$
$M_u = 540 \text{ x } 1.4 = 756^{k'}$ — Use 16"x36" Beam

$$K_u = \frac{M_u}{b\ d^2} = \frac{756 \text{ x } 12000}{16 \text{ x } 32^2} = 554;\ a_u = 4.05''$$

$$A_s = \frac{M_u}{a_u d} = \frac{756}{4.05 \text{ x } 32} = 5.88 \text{ in.}^2 < 6.0$$ Use 6-#9 Top Bars

Check for Bottom Reinforcing

$M_{Lat} = 1.4 \text{ x } 475.2^{k'} = 665^{k'}$
$0.9M_{DL} = 0.9 \text{ x } 20.8 \text{ x } 17.7/8 = \underline{-41}$
$M_u = 624$

$$K_u = \frac{624 \text{ x } 12000}{16 \text{ x } 33^2} = 429;\ a_u = 4.03''$$

$$A_s = \frac{624}{4.03 \text{ x } 33} = 4.69 \text{ in.}^2 < 5.08$$ Use 4-#10 Bot. Bars

Check Minimum Requirements

A_s Bottom $\geq 1/2\ A_s$ Top

$b > 10''$; $b/d = 16/36 = 0.44 > 0.3$

$P_{max} = 0.025$ $\quad A_{s\ max} = 0.025 \text{ x } 16 \text{ x } 36 = 14.4 \text{ in.}^2$

$P_{min} = 200/f_y = 0.0033;$ $\quad A_{s\ min} = 0.0033 \text{ x } 16 \text{ x } 36 = 1.9 \text{ in.}^2$

Check Beam Capacity - Based on 125% of f_y and $\emptyset = 1.0$

$M_u = \emptyset\ A_s\ f_y\ (d-a_u/2)$; $a_u = 1.25\ A_s\ f_y/\ 0.85\ f'_c\ b$

Top Bars $a_u = 1.25 \text{ x } 6.0 \text{ x } 60\ /\ 0.85 \text{ x } 4.0 \text{ x } 16 = 8.3''$

$$M_u = 1.0 \left[\frac{1.25 \text{ x } 6.0 \text{ x } 60}{12} (33-8.3/2) \right] = 1082^{k'}$$

Bot. Bars $a_u = 1.25 \text{ x } 5.08 \text{ x } 60\ /\ 0.85 \text{ x } 4.0 \text{ x } 16 = 7.0''$; $M_u = 937^{k'}$

Check Shear Capacity

$$V_u = \frac{M_{ua} + M_{ub}}{L_c} + 1.4\ V_{D+L}$$

$$= \frac{1082 + 937^{k'}}{17.7'} + 1.4 \times \frac{29.2}{2} = 134^k$$

v_u = V / Øbd = 134,000/0.85x16x33 = 299 psi $< 6\sqrt{f'_c}$ = 379 psi

$v_c = 1.9\sqrt{f'_c} + 2500\ p_w V_u d/M_u$; $p_w = A_s/bd$ = 6.0/16x33=0.011

$= 1.9\sqrt{4000}$ + 2500x0.011x134x33/1082x12 = 129 psi

but not to exceed $3.5\sqrt{4000}$ = 221 psi

Because of possible tension - use $2\sqrt{4000}$ = 126 psi

#4 Stirrup Spacing at Supports

$$s = \frac{A_v f_y}{b(v_u - v_c)} = \frac{2 \times 0.20 \times 60000}{16(299-126)} = 8.67''$$

or d/4 = 33/4 = 8.25"

or $A_v f_y/50b$ = 0.4x60000/50x16 = 30.0"

or $A_v d/0.15A_s$ = 0.4x33/0.15x6 = 14.7"

or 8 bar diam. = 8x1.13" = 9.0"

or 24 stirrup diam. = 24x0.5 = 12.0" nor 12" min.

Distance required = 2d = 2x33 = 66"

Use full #4 stirrups @ 8.2"o.c. for 66" from support

#4 Stirrup Spacing at Center

$$s = \frac{2 \times 0.20 \times 60000}{16(299-126)} = 8.67''$$

or d/2 = 33/2 = 16.5"

Use full #4 stirrups @ 8.6"o.c. in center portion of span

Second Story Column Design

$M_y = 1.04^{k}/\text{ft} \times 37.5^2/12 = 122^{k'}$ (Beam moment in weak axis of col.)

M_y @ $e = 0.1t = 2.8''$; $2.8/12 \times 1533^{k} = 358^{k'}$

$M_{uy} = 122^{k'} \times 1.4 = 171^{k'}$

$M_{ux} = 1.4 \times 83.6^{k} \times 10'/2 = 585^{k'}$ (Seismic)

$P_u = 1533^{k}$ w/ Vertical load
$= 1537^{k}$ w/ Seismic
$= 679^{k}$ w/ Uplift

Use 28"x28" Column

$$g = \frac{28''-2\times3''}{28''} = 0.79$$

$P_u/A_g = 1537^{k}/28^2 = 1960$ psi

or $679^{k}/28^2 = 866 \text{ psi} > 0.12f'_c = 480$ psi

$$kl/r = \frac{1.37\times10\times12}{0.3\times28''} = 19.6 < 22$$

No reduction required for slenderness

$$\psi = \frac{2\times0.19}{2\times0.15} = 1.27 \quad \text{Top \& Bottom ; } k = 1.37$$

$$\delta = \frac{C_m}{1 - P_u/\phi P_{cr}}$$

$$EI = \frac{E_cI_g}{2.5(1+P_d)} = \frac{3650\times2.46\times12^4}{2.5(1+104/146)} = 43{,}000{,}000^{k}\ \text{in}^2$$

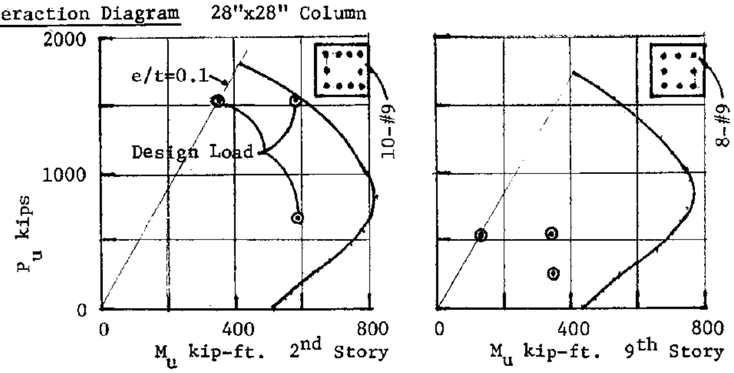
$$P_{cr} = \frac{\pi^2 \times EI}{(kl)^2} = \frac{\pi^2 \times 43000000}{(1.37\times120)^2} = 15700^{k}$$

$C_m = 0.6 + 0.4M_1/M_2$; $M_1 \sim M_2$
$= 0.6 + 0.4(-1) = 0.2$ Use 0.4 Min.

$$\delta = \frac{0.4}{1 - 1537/0.7\times15700} = 0.47 < 1.0$$

No reduction required for moment magnification

Interaction Diagram 28"x28" Column



For Maximum Loading ; p_t = 1.3% required

A_s = 0.013 x 28" x 28" = 10.2 in.^{2} Use 10-#9 Vert.

$p_t = 10/28^2 = 0.0128$

Check for Column Capacity to Exceed Beam Capacity

Reduce for ϕ=0.9 and for 125% of f_y

$$M_{uj} = \frac{1082^{k'} + 937^{k'}}{1.25/0.9} = 1454^{k'}$$

$V_u = 1454^{k'} / 13' = 112^k$

$M_u = 112^k \times 5.0' = 559^{k'}$

With $P_u = 679^k$ or 1537^k & $p_t = 0.0128$

From Interaction Diagram $M_u = 800^{k'}$ or $590^{k'} > 559^{k'}$
(Including ϕ = 0.7)

Column Bending
Capacity ok

Column Shear Capacity

Based on ultimate beam capacity without ϕ
and with 125% of f_y

$M_u = 1082 + 937 = 2019^{k'}$

$V_u = 2019^{k'}/13' = 155^k$

$V_u = 155 \times 0.9/0.7 = 199^k > 112^k$ When adjusted for reduction factors of beam vs column

Column Shear
Capacity ok

Column Ties

$$v_u = \frac{V_u}{\phi\, bd} = \frac{199000\#}{0.85x28''x24''} = 348 \text{ psi}$$

$$\text{or } \frac{V_u}{\phi\, A_c} = \frac{199000\#}{0.85x25''x25''} = 375 \text{ psi} \qquad P_u/A_g > 0.12f'_c$$

$$v_c = 1.9\sqrt{f'_c} + 2500\, p_w\, V_u d/M_u \quad ; \quad p_w = 4.0/28^2 = 0.0051$$

$$= 1.9\sqrt{4000} + 2500x0.0051x0.42 \qquad \frac{V_u d}{M_u} = \frac{112x25}{559\,x12} = 0.42$$

$$= 125 \text{ psi}$$

$$\text{or } 3.5\sqrt{f'_c} = 221 \text{ psi}$$

$$\text{or } \quad 2\sqrt{f'_c} = 125 \text{ psi} \quad \text{Because of possible tension}$$

Spacing 1-#4 Tie Through Column

$$s = \frac{0.20 \text{ x } 60000}{28''(348-126)} = 1.93''x4 \text{ legs} = 7.7''$$

$$\text{or } \frac{0.20 \text{ x } 60000}{25''(375-126)} = 1.93''x4 \qquad = 7.7''$$

$$\text{or } d/2 = 14''$$

Use #4  @ 8" o.c.
through column beyond
confined region

Special Confinement Ties   Max. Spacing = 4"

$$A_{sh} = 0.30\ ah''\ \frac{f'_c}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right)$$

$$= 0.30x4''x25''x\ \frac{4}{60}\left(\frac{28''x28''}{25''x25''} - 1\right) = 0.50 \text{ in.}^2 < 4x0.20{=}0.80 \text{ in.}^2$$

$$\text{or } 0.12\ ah''f'_c/f_{yh} = 0.12x4''x25''x4/60 = 0.79 \text{ in.}^2$$

$$\text{or } l_h p_s s_h/2 \quad ; \quad p_s = 0.45(A_g/A_c - 1)f'_c/f_y$$

$$= 0.45(28^2/25^2 - 1)4/60 = 0.0076$$

$$\text{or } 0.12\ f'_c/f_y = 0.12x4/60{=}\ 0.0080$$

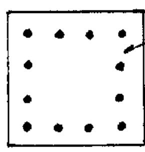
$$= 25''/3 \text{ x } 0.0080 \text{ x } 4''/2 = 0.13 \text{ in.}^2$$

Use 4-#4 @ 4" o.c.
for special confinement

Check for Biaxial Bending Ref. PCA Bulletin #18

Max. Condition	Min. Condition
$P_u = 1537^k$	$P_u = 679^k$
$M_{ux} = 585^{k'}$	$M_{ux} = 585^{k'}$
$M_{uy} = 171^{k'}$	$M_{uy} = 171^{k'}$
$P_o(@\ M=0) = f_y A_s + 0.70 f'_c(A_g - A_s)$ $= 60\text{x}12 + 0.70\text{x}4(28^2 - 12.0)$ $= 2882^k$	
$M_{ox}\ (@\ P_u) = 631^{k'}$	$M_{ox} = 825^{k'}$
$P_u/P_o = 1537/2882 = 0.53$	$P_u/P_o = 679/2882 = 0.24$
$q = p_t f_y/f'_c = 12.0/28^2\text{x}60/4 = 0.23$	
From Fig. #6 $\beta = 0.64$	$\beta = 0.62$
$M_x/M_{ox} = 585/631 = 0.93$	$M_x/M_{ox} = 585/825 = 0.71$
From Fig. #2 $M_y/M_{oy} = 0.23$	$M_y/M_{oy} = 0.53$
$M_{oy}\ (@\ P_u)$ in weak axis $= M_{ox} = 631^{k'}$	
$M_y = 631 \times 0.23 = 145^{k'} \sim 171$ ok	$M_y = 825 \times 0.53 = 437^{k'} > 171$ ok

For Biaxial Bending - Use 4 bars on each side



Use 12-#9 Bars Total

Biaxial Bending Curves
Ref. PCA Bul. #18

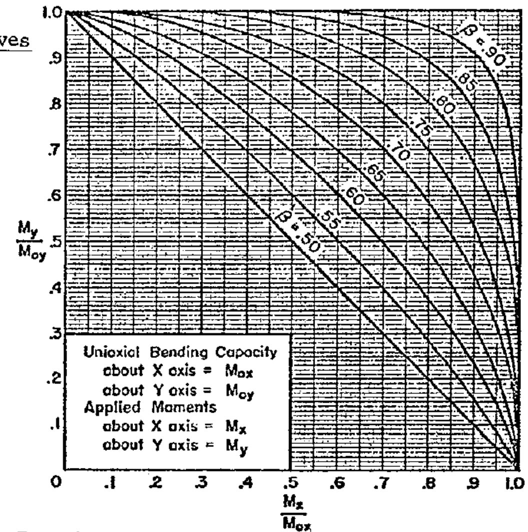


FIG. 2 BIAXIAL MOMENT RELATIONSHIP

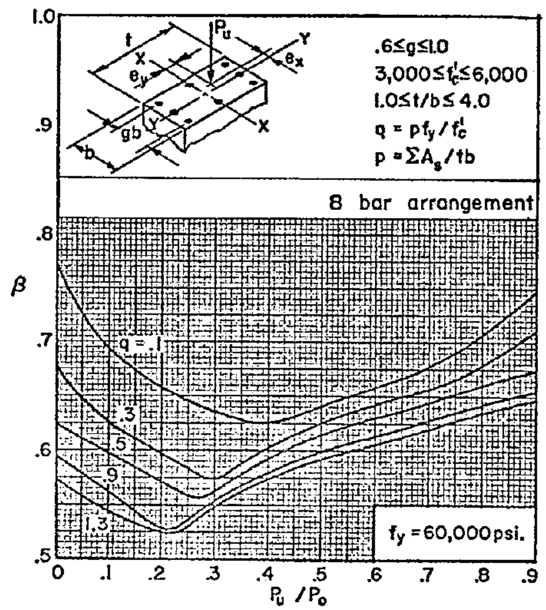
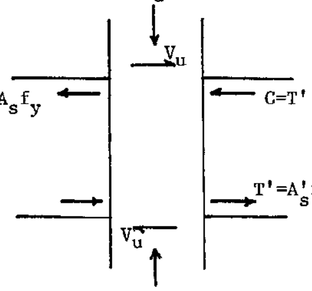


FIG. 6 BIAXIAL BENDING DESIGN CONSTANTS

Beam-Column Joint Design Per Joint Committee 352



Eccentricity to Column

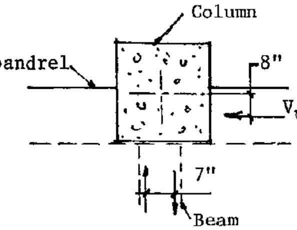
Capacity of girders = $1082^{k'} + 937^{k'}$
(Includes 1.25 factor)

Column shear $V_u = \dfrac{1082+937}{13'} = 155^k$

Min. reinforcing area to cross beam

$${}_tA_s = V_ue/\phi f_y(d-d') = \frac{155x8''}{0.9x60x7''} = 3.3\ in^2$$

Joint Shear



$T = \alpha\ A_sf_y = 1.25x6.0x60 = 450^k$

$C = 1.25x5.08x60 = 381^k$

V_u Joint = T + C - V_u(Column)

$= 450 + 381 - 155 = 676^k$

$T_j = V_ue = 155^kx8'' = 1240^{k''}$

$$v_j = \frac{V_j}{\phi\ A_{cv}} = \frac{676000}{0.85x26''x25''} = 1224\ psi$$

$$v_{tj} = \frac{3T_j}{\phi\sum x^2y} = \frac{3x1240^{k''}}{0.85x28^2x28} = \underline{199}$$

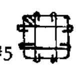
v_u =1423 psi $\sim 20\sqrt{5000}$ = 1414 psi ok

$v_c = 3.5\ \beta\ \gamma\sqrt{f'_c(1 + 0.002N_u/A_g)}$

$= 3.5x1.0x1.0\ \sqrt{5000(1 + 0.002x679000/28^2}$ = 409 psi

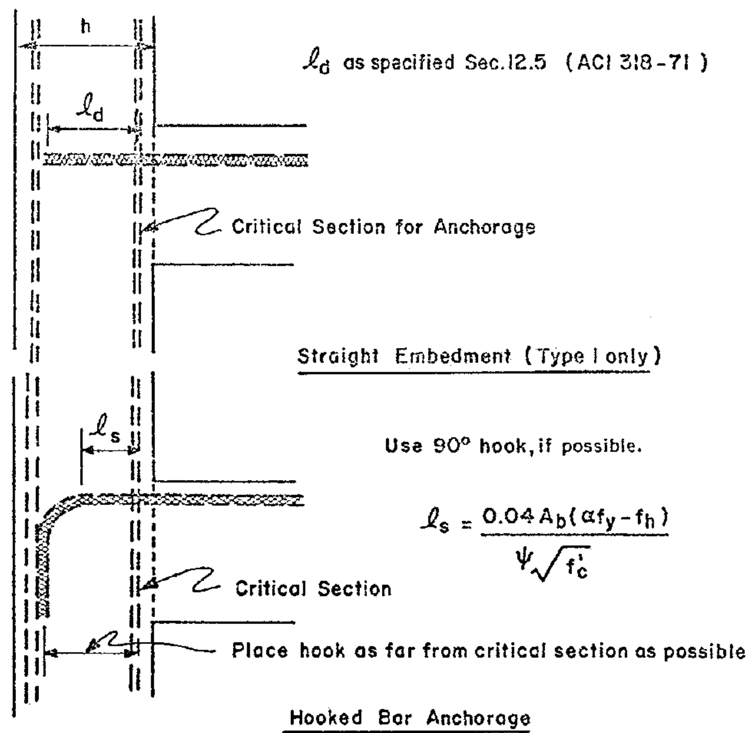
Net Shear = 1423-409 = 1014 psi < $15\sqrt{5000}$ = 1061 psi ok

Spacing 1-#5 = $\dfrac{0.31x60000}{28''x1014}$ = 0.66"x4 legs = 2.6"

Use #5  @ $2^1/2''$ o.c.
through joint w/ 5000 psi
concrete

See Joint Committee Report
for detailing of joints

Minimum Column Embedment Ref. Joint Committee 352



ANCHORAGE REQUIREMENTS FOR JOINTS

Anchorage through interior joint w/ f_h = 0 & #9 bars

$l_d = 0.04x1.0(1.25x60000)/1.8\sqrt{5000} = 23.8"$

Min. Column Size = l_d + 2x min. cover = 23.8"x2x1.5" = 26.8" < 28" ok

Note – The corner condition with bent bars will require less anchorage than the interior condition.

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

CAPACITY DESIGN OF REINFORCED CONCRETE DUCTILE FRAMES

by

Thomas Paulay
Professor of Civil Engineering
University of Canterbury
Christchurch, New Zealand

1 INTRODUCTION

The general acceptance of strength design for reinforced concrete structures has gradually changed our design philosophy, particularly over the past two decades. This change was accelerated by the simultaneous progress in the appreciation of earthquake effects on structures. Instead of adhering to permissible stress limits, increasing attention is being paid to such idealization of material properties and structural behaviour that more likely conform with the laws imposed by seismic disasters. For most structures the emphasis has shifted from the "resistance" of large forces to the "evasion" of the same. Structural failure emerged from the obscurity of hypotheses, characteristic in structures subjected to gravity loading only, and has become a routine concern in its stark reality. Consequently in earthquake resistant design we began to select from types of failures that were acceptable and others that were undesirable. The concepts of hierachy in failure modes, ductility versus brittleness, suitability for possible repair and protection of non-structural components evolved, and presently the precepts of a "capacity design" philosophy are being established.

When formulating a capacity design approach to framed buildings, the major topic of this paper, an important consideration in developing a readily acceptable method must be also simplicity of procedure without adopting overly conservative standards. To this end it is imperative that the plastic rotational capacity of beams and columns in ductile frames be assured, and that the risk of failure due to inadequate assessment of actions is minimised. If these criteria can be achieved with confidence, an upper bound estimation of bending moments and in certain cases that of earthquake induced axial loads in columns is not warranted. However, there should be no risk of a pattern of hinge formation developing that could lead to the formation of storey failure mechanisms in which the major source of energy dissipation would be associated with interstorey column sidesway. There is ample evidence from recent earthquakes that such a mechanism could have disastrous consequences.

The procedure to be presented is intended for the design of regular and rectlinear ductile reinforced concrete multistorey moment-resisting frames in which, in addition to the gravity loads, all earthquake induced forces are resisted entirely by the same frames in one or both principal directions of the building. It is postulated that the intelligent use of the equivalent lateral static forces, prescribed by most building codes [1,2], when combined with utmost care for the detailing of the various reinforced concrete components, will lead to a regular building frame that is likely to perform satisfactorily during a very large disturbance. In the light of our present ability to predict the nature of ground excitations, it is questionable whether a more elaborate non-linear dynamic analysis will impart better properties to a regular space frame, than the modified static analysis to be examined. Because of its simplicity it should have an appeal to a wide section of practising design engineers.

Some of the difficulties in formulating a simple design procedure stem from the facts that:

(i) An attempt needs to be made for rational reconcilation between an elastic analysis for lateral static load and the elasto-plastic response of a frame, exposed to random motions.

(ii) The critical earthquake actions, such as moments, shear and axial forces, to be used in the determination of column strength are interrelated in a complex manner.

(iii) Capacity design represents essentially a deterministic philosophy, whereas the critical interaction of apparently unrelated and random design quantities could only be assessed with the tools of probability.

In the following a compromise is attempted whereby the simplicity and familiarity of the static elastic analysis is retained and at the same time some features of an elasto-plastic non-linear dynamic response are recognised. This is achieved with the use of suitable factors.

2 THE PHILOSOPHY OF CAPACITY DESIGN

When the principle is accepted, that for most buildings the avoidance of irreparable damage under very severe seismic disturbance is not economical, our attention is immediately focused on the various modes of structural failures that could result. Structural failures and consequent irreparable damage to be considered here are not synonymous with structural collapse. Indeed the most important aim in earthquake resistant structural design is to minimise the likelihood of collapse under the most severe excitation that could be expected in the locality during the intended life of the structure. It is therefore necessary to impart to the structure desirable characteristics of behaviour that will ensure an acceptable sequence in the breakdown of the complex chain of resistance. This implies a desirable hierachy in the failure modes and the knowledge of the probable strength of each link.

In spite of the random nature of the displacement patterns applied to a structure during a catastrophic seismic excitation, in the light of our present knowledge, a deterministic allocation of strength and ductility properties, in accordance with the philosophy of capacity design, holds the best promise for a satisfactory structural response and the prevention of collapse. [3]

In the capacity design of earthquake-resistant structures, energy dissipating elements or mechanisms are chosen and suitably detailed, and other structural elements are provided with sufficient reserve strength capacity, to ensure that the chosen energy-dissipating mechanisms are maintained at near full strength throughout the deformations that may occur.

3 RELEVANT CHARACTERISTICS OF MULTISTOREY FRAMES

Before the relationship between two important links in a frame, i.e. beams and columns, can be established it is necessary to examine the nature of overall frame behaviour, to briefly review certain dynamic characteristics and to postulate an acceptable failure mechanism.

3.1 Dominant Modes of Behaviour of Laterally Loaded Frames

The mode of load resistance in laterally loaded moment-resistant multistorey frames is distinctly affected by the relative stiffnesses of the beams and columns. A "shear type" frame is characterised by very stiff beams that provide great restraint against column end-rotations. The dominant deformation is interstorey sway. Under lateral load a particularly advantageous moment pattern along columns indicates a point of contraflexure usually near the midheight of the column. However, the axial loads induced by lateral loads are maxima.

In a "bending type" frame, with very flexible beams, the cantilever action of the columns will dominate. This results in very large moment but smaller axial load demands on the columns. In the extreme, such a frame may degenerate into a coupled shear wall.

This study considers a common type of frame in which the beam and column stiffness are comparable so that under the usual lateral static load deformed columns will exhibit a point of contraflexure in most storeys.

Using Muto's studies [4], it may be shown that such behaviour can be expected in regular building frames when the relationship between the relative stiffness ($k = I/\ell$) of the beams that restrain a column in a storey and the stiffness of that column is such that:

$$\frac{\Sigma k_{\text{upper beams}} + \Sigma k_{\text{lower beams}}}{2\, k_{\text{column}}} > 0.2 \qquad (1)$$

where I is the second moment of area of the cross section and ℓ is the length of the member.

3.2 Modal Participation in Frames

Typical spectra giving the acceleration as a function of the period of vibration for a single degree of linear oscillator, responding elastically to some seismic ground motion, are shown in Fig. 1. Design codes [1,2] assume that the acceleration response of elastic multidegree of freedom systems is similar. Consequently the inertia forces or base shear are taken to be proportional to these accelerations.

The equivalent lateral static load, specified by loading codes [1,2], when applied, will result in deflections and moment patterns in frames that are similar to those generated in the first mode of dynamic response. However, as the natural period of the structure increases the same ground excitation may induce major changes in this response and may cause the structure to respond significantly in one of its higher modes. In terms of

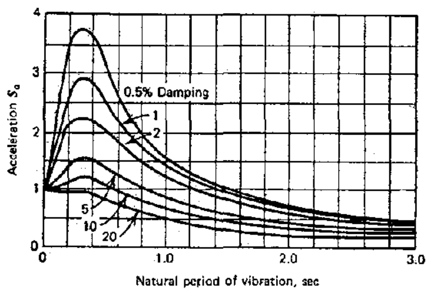


FIG. 1 - Typical acceleration spectra for a single degree linear oscillator [5]

column bending moments over the height of the building, this may be responsible for very considerable deviation from the pattern that was obtained from an equivalent lateral static loading.

Fig. 2 shows the variation of period ratios for frames in the entire range of stiffness ratios, ρ , from "shear type" to "cantilever type" buildings [6]. For a regular frame, that satisfied the criteria of Eq.(1), it will be found that $\rho > 0.1$ and hence, the ratio of periods in the first mode, T_1 , to that in the second mode, T_2, is for all common cases $T_1/T_2 \approx 3$. Similarly $T_1/T_3 \approx 5$. It is seen from Fig. 1 that a building, for example, with a natural period of 1.5 seconds could have considerable acceleration responses for the higher modes because $T_2 = 1.5/3 = 0.5$ sec. and $T_3 = 1.5/5 = 0.3$ sec. The corresponding changes in the modal shapes of the frame will then alter the moment pattern along the columns.

3.3 Preferable Failure Mechanisms

If columns are weaker than the beams, it is likely that a storey sway mechanism develops during an earthquake. This is now generally recognised as an undesirable energy dissipating mechanism. For a given overall displacement ductility very large rotational ductilities in the column hinges will be required. Large interstorey displacements may result in considerable $P - \Delta$ effect and thus the lateral load resistance of the columns may be severely reduced. Where axial compression loads are significant, the required curvature ductilities in column hinges are not readily developed. In general the failure of a column can be expected to have more severe consequences than that of a beam. For the reasons mentioned it is desirable that plastic hinges, well distributed over the entire height of the frame, should develop in the beams rather than in the columns.

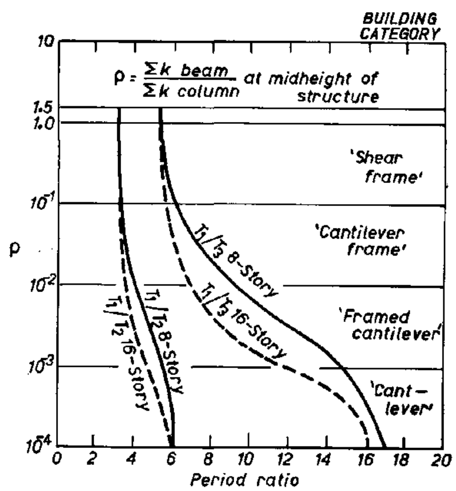


Fig. 2 - Period ratios for various categories of buildings [6]

Transient yielding in an isolated ductile column, while extensive hinging in numerous beams occur, is of no significance. The development of plastic hinges in columns at ground floor or foundation level may not be avoided and the detailing of these requires particular attention. As a rule the possibility of the development storey sway mechanism, with simultaneous column hinges at the top and the bottom of a storey, must be minimised. However, storey mechanisms are acceptable in one storey frames or in the upper storeys of multistorey frames.

4 BEAM DESIGN

4.1 Capacity Relationships

When two plastic hinges with known flexural capacities develop in each span of a continuous beam of a multistorey frame, a statically determinate system results. Therefore the associated shear forces in the beams can be uniquely determined.

Shear mechanisms show only limited ability to dissipate energy. Also they generally exhibit degrading strength with reversed cyclic loading beyond the elastic limit. Therefore a failure in shear must be suppressed. Accordingly the dependable shear capacity of a beam must be equal or larger than the corresponding flexural overcapacity of the beam. [3] The latter must take into account the actual flexural reinforcement that will participate in the flexural resistance, and also the strain hardening of the flexural reinforcement.

4.2 Moment Redistribution in Beams

During a very severe earthquake, when beam flexural overcapacities are developed, large inelastic rotations will occur at selected plastic hinges. For this reason the designer should unreservedly consider a statically admissible redistribution of the combined beam design moments due to gravity and lateral load. When such a moment redistribution occurs it will invariably involve curvature ductilities that are a small fraction of those to be developed during a severe earthquake. In the efficient design of reinforced concrete continuous beams of frames there are three aims that the designer should attempt to achieve:

(a) Reduce the absolute maximum moment, usually in the negative moment region, and compensate for this by increasing the moments in the non-critical (usually positive) moment regions. Thus a better distribution of strength demand is achieved, particularly along prismatic members.

(b) Equalise the critical moment demands in beams at either side of an interior column. This will obviate the necessity of having to terminate and anchor beam bars at interior beam-column joints where a congestion of reinforcement commonly presents construction difficulties. It should be noted that if steel is provided for the larger of the moments on either side of an interior joint, as is often done in non-seismic areas, the column strength will have to be unnecessarily increased so as to enable it to match the beam moment input.

(c) Fully utilise the potential positive moment capacity of beam sections at column faces where this must be at least one half of the negative moment capacity at the same section [1,2]. This aim can be extended and combined with that listed in (a) so as to approach equal positive and negative moment demands at the same section. This would allow the full utilisation of sections with equal top and bottom flexural steel content.

Moment redistribution applied to a subassembly of a multistorey frame, such as shown in Fig. 3(a), must be statically admissible. Moreover it must not result in any decrease of the lateral load resistance of the assembly. Fig. 3(b) shows the moment pattern that resulted from an elastic analysis for a code [2] required combination of gravity loads. For convenience important values are shown in specific moment units. Figs. 3(c) and 3(d) show the prescribed equivalent static seismic load and the resulting moment pattern for both directions of the lateral load application.

The lateral load resistance of the subframe is not affected, and consequently the requirements of equilibrium are not violated, if individual moment increments associated with a particular direction of the lateral loading, such as $\Delta\vec{M}^E$, are altered, provided that the sum of the increments remains the same. Such a change means redistribution of moments between columns and in the beam.

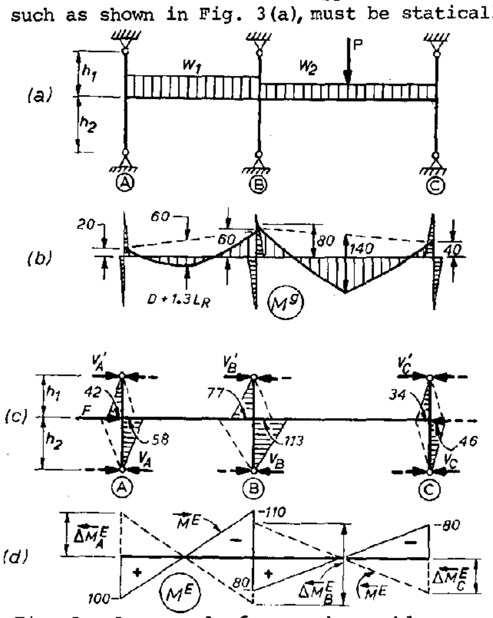


Fig. 3 - An example frame subassembly and the gravity and earthquake moment requirements

The relative distribution of the two components of the moment increment ΔM_B^E at B , (i.e. 110 and 80 units) shown in Fig. 3(d), between the two adjacent beams may be freely altered without affecting the resistance of the subframe. This involves moment redistribution in the beams only.

Moment redistribution between columns means redistribution of the shear forces, such as V' and V shown in Fig. 3(c), between individual columns.

Whether the moment is redistributed from one beam on one side of an interior column to the beam at the other side of the same column or a redistribution of shear forces between columns is intended, in both cases moment redistribution relies on rotations in plastic hinges that form in the beams in question.

Moment redistribution in a frame may alter slightly the ultimate ductility demand of potential plastic beam hinges by increasing the demand in some hinges and at the same time decreasing it in others. The total ductility demand for the structure as a whole remains unaltered. It may be said that the average ductility demand in the localities of potential energy dissipation does not change, and that, with limited redistribution during the design process, the deviation from the average ductility demand during a very large earthquake will be very small.

It was considered in New Zealand [7] that in beams of ductile earthquake resistant frames the magnitude of the moment to be redistributed, ΔM, should be limited as follows:

(i) In any span of a beam ΔM should not exceed at any point 30% of the absolute maximum moment derived for that span from elastic analyses for any combination of earthquake and factored gravity load.

(ii) Moment redistribution between columns should not change the maximum value of the combined end-moments in any column, derived from elastic analyses for any of the load combinations referred to in (i) by more than ±15%. This limitation is satisfied if the redistribution of shear forces between columns is limited to ±15% of the shear force acting on the column in question.

The application of these criteria to the beams shown in Fig. 3 is presented in Fig. 4, where for convenience the combined gravity and earthquake design moments ($E + D + 1.3L_R$) are plotted separately for each direction of the seismic action. The top curves result from the superposition of the moments shown in Figs. 3(b) and 3(d). The dashed lines respresent intermediate steps of moment redistribution in accordance with limitation (i) above. The heavy full lines give the final design moment pattern after the application of moment redistribution between the outer columns, in accordance with (ii) above.

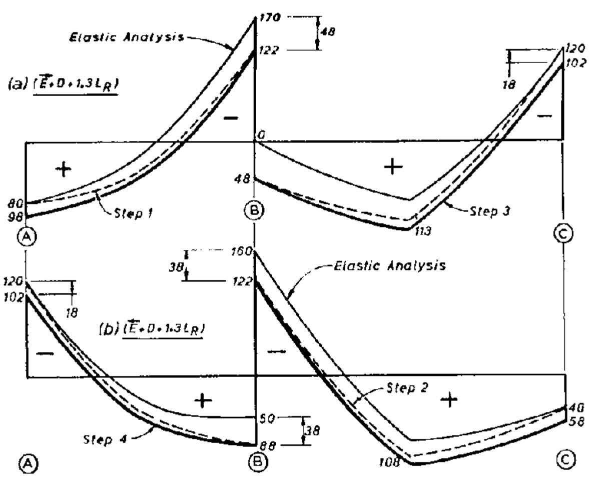


Fig. 4 - Moment redistribution for two loading cases for the beams shown in Fig. 3.

Moment redistribution in beams, particularly when the design gravity moments are comparable with or larger than the design earthquake moments, does not only lead to a more economical beam steel arrangement but it will minimise the unnecessary moment input into columns, to be examined subsequently.

4.3 The Flexural Strength of Beams

The principles of flexural strength are well established and the issues do not warrant detailed examination. However, certain aspects that are considered to be inadequately specified in building codes [1,2,8] and which are particularly relevant to seismic loading are briefly discussed.

4.3.1 Dimensional limitations--It has been customary to limit the width to depth ratio b_w/h, in flexural members to 0.3 because there was insufficient evidence to show that more elongated section would develop the necessary curvature ductilities [1]. The concern should be directed towards the stability of a flexural member when it is subjected to reversed cyclic loading. Therefore the length to width ratio ℓ_n/b_w is at least as important a parameter as the b_w/h ratio. Accordingly relevant proposals are made in the draft recommendations.

As a result of a number of experimental studies it has become evident that beam-column joints are very critical areas in ductile earthquake resistant frames. Therefore certain dimensional limitations must be imposed on the width of beams if the designer wishes to ensure that the beam actions can be effectively transferred to columns. The width of a continuous beam, relative to a column, need not be restricted if gravity load is to be carried only, because usually there is a relatively small moment transfer to the columns. However, under earthquake action beam moments must equilibrate column moments and the exchange of forces must take place in a joint core, as examined in section 6. For this reason the width of a beam should not be much larger than the width of the column into which it frames. Quantitative recommendations are made separately.

4.3.2 Flexural reinforcement participation--It is now generally accepted that the maximum likely moment, that can be developed in a beam when very large curvature ductilities are imposed, need be evaluated. For mild steel (Grade 40) with a specified yield strength of 275 MPa, a 25% increase in strength is considered to be sufficient to account for mean yield strength being higher than the specified value, and for some strain hardening.

It is equally important to realistically assess the amount of steel that will participate in the development of this flexural overcapacity. The difficulty arises in evaluating the effective width of floor slabs in flanged beams in resisting tension. Slab reinforcement placed close to a column will undoubtedly contribute significantly [18] to the total tension force in the top of a beam section. The effectiveness of slab steel placed further away from the web of a beam or the face of a column is likely to depend on the torsional resistance of the beams framing into a column at right angles to the beam, the strength of which is being considered. The load transfer from slab bars near exterior columns is likely to be less effective than at interior columns. These considerations lead to a number of proposals given in the draft recommendations.

If a reasonable estimate can be made with respect to the slab reinforcement, that could participate in negative moment of resistance, then this slab reinforcement should also be considered in the assessment of the dependable strength of the member.

4.4 The Shear Strength of Beams

In accordance with the philosophy of capacity design an attempt must be made to eliminate the possibility of a shear failure. Accordingly the design shear force in a beam must be determined from consideration of the static transverse forces, with the flexural overcapacity being developed at the most probable location of the critical sections within a beam or in adjacent beams, and the gravity load with appropriate load factor [1,3].

It is now well recognised that in potential plastic hinge zones [1,2,8] the entire design shear resistance should be allocated to the web reinforcement. However, no provision so far has been made to control the effects of sliding shear. The problem of reversed cyclic shear is currently being studied by ACI-ASCE Committee 426.

From the limited experimental evidence available it is apparent that the level of shear load is most significant. Shear failures were observed [9,18] along full depth vertical cracks across beams after the abrasion of such crack interfaces, in spite of the presence of web reinforcement that satisfied current code [8] requirements. It is evident that if optimum energy dissipation is to be maintained in a beam, by reducing as much as possible the pinching effect in hysteresis loops,caused by shear, then the maximum nominal shear stress must be limited, unless other precautions are taken [10].

It is evident that a sliding movement across a full depth crack across the plastic hinge zone of a beam can be controlled only by diagonal reinforcement that can respond in tension and compression. Some tests have conclusively shown the improved energy dissipating properties of such beams [11,12]. It appears [10] that if reversed nominal shear stresses in excess of $0.25\sqrt{f'_c}$ MPa ($3\sqrt{f'_c}$ psi) are expected, some diagonal shear reinforcement will be necessary unless loss of energy dissipation is acceptable.

When gravity load is present the intensity of shear, corresponding with each direction of loading will be different. This is recognised by the proposal that the maximum nominal shear stress in a plastic hinge zone be limited to

$$v_u = \frac{(2 + r)\sqrt{f'_c}}{4} \text{ (MPa) } [\, v_u = 3(2 + r)\sqrt{f'_c} \text{ (psi) }] \tag{2}$$

unless diagonal shear reinforcement is provided. In Eq. (2) r is the algebraic value of the ratio of the maximum values of the shear force developed with negative moment hinging, to the shear force developed with positive moment hinging, with the following limits: $0 > r > -1$. More detailed suggestions are made in the draft recommendations.

4.5 Moment Inputs from Beams to Columns

If columns are to be given some reserve strength with respect to the beams that they support, then the maximum likely beam moment input at a beam-column joint must be assessed. When two plastic hinges form in any span of a beam, with the flexural overcapacities developed, as outlined in section

4.3.2, the laws of statics will indicate what the beam moments at the centre lines of the two supporting columns would be. These moments will differ from those derived from the elastic analysis for a code prescribed lateral static loading. The first set of moments represents the maximum feasible beam input, whereas the latter set results from minimum intended strength requirements. Moreover, in the final design a substantial deviation from the initial bending moment pattern may emerge because of the moment redistribution (Section 4.2) allowed for in the design of beams. To simplify routine calculations it is preferable to retain, as a reference, the original bending moment pattern, obtained from the elastic frame analysis for the prescribed lateral load, and to relate the beam flexural overstrength inputs to these reference moments with the aid of a beam overstrength factor ϕ_o .

The beam overstrength factor , ϕ_o , is the ratio of the sum of the flexural overstrengths developed by the beams, as detailed, and the sum of the flexural strengths required by the specified lateral loading, both sets of values being considered at the centre line of the column, i.e. the reference axis of the skeletal frame. This factor need be determined at each beam-column joint for each direction of the loading. Naturally at an exterior column only one beam is considered.

The typical value of ϕ_o is 1.25/0.9 = 1.39, where 1.25 is a generally accepted factor to allow for the maximum developed yield stress in mild reinforcing steel being higher than the guaranteed value, and 0.9 is the customary capacity reduction factor used in determing the dependable flexural strength of a beam [8]. When gravity load considerations govern the strength of a beam, the value of ϕ_o will be larger than 1.39. When moment redistribution is utilized, ϕ_o may be locally less than 1.39. The weighted average of ϕ_o at all beam-column joints of a bent, however, will never be less than the theoretical value of 1.39 unless beams are deliberately underdesigned.

To illustrate the evaluation of ϕ_o, consider the example structure shown in Fig. 4 and let us assume that reinforcement has been carefully provided so that the flexural overcapacities that could be developed are as follows:

(i) At column A: ±140 units.

(ii) At column B: -170 units and + 125 units.

With reference to Fig. 3(d) the values of ϕ_o can now be readily evaluated. For column A ϕ_o = 140/100 = 1.4 and at column B, ϕ_o = (170 + 125)/(110 + 80) = 1.55.

5 THE DETERMINATION OF COLUMN ACTIONS

5.1 General Considerations

The difficulty in formulating a column design procedure, that will give an acceptable degree of protection against premature yielding and excessive hinging during very large disturbances, together with a high degree of protection against the formation of storey mechanisms, arises from the number of phenomena.

(i) Primarily due to higher mode dynamic responses the moment pattern along the height of the column may very considerably deviate from that indicated by the initial analysis for a lateral static load. This is reflected by the movement of the point of contraflexure along the column in any one storey. The phenomena may also be viewed as a disproportionate distribution of beam moment input at any floor between the column section above and below that beam.

(ii) The axial load induced at any level of a column by earthquake load only depends on the sum of the beam shear inputs above that level. The formation of plastic beam hinges above that level, however, is affected by the dominant mode of response of the frame and the extent to which flexural overstrength of the same sense will develop in the beams of the upper floors.

(iii) The probability of the concurrence of maxima in earthquake induced moments and axial loads at the same level, that could determine the design strength of a column, should be considered.

(iv) When frames extend in both principal vertical planes of a building, it is convenient to design the structure to carry the prescribed lateral load separately in each of these principal directions. Designers prefer to deal with plane frames rather than with space frames. A seismic excitation, however, will impose lateral displacements in any direction and may mobilise for example, the full strength of all four beams that join an interior column of a frame. A skew earthquake action will impose biaxial bending and shear on a column.

(vi) When the probable concurrency of critical axial forces and moments in "one-way" or in "two-way" frames is considered, the consequences of possible column hinging need be examined in the light of the intensity of the axial compression likely to be present. This will determine the potential ductility of the section.

(vii) In our attempt to minimise the likelihood of a shear failure, some rational estimate need be made with respect to the maximum attainable moment gradient along a column within a storey. Moreover, the probability of this maximum shear, instantaneously coinciding with the minimum feasible axial compression or perhaps the maximum net axial tension on the column ought to be evaluated.

The complexity of the issues is evident. Unfortunately sufficient data, obtained from experimental or theoretical time history studies of the response of buildings during recorded or artificial ground excitations, is not available yet to serve as a basis for quantitative stochastic studies. In the absence of this one is forced to rely on intuitive probability to arrive at a reasonable degree of protection of columns in earthquake resistant frames.

The term "degree of protection against hinging" can not be defined with exactness. The criterion is subjective and hence it will depend on prevailing engineering views in fashion, which in turn will change with the passage of time and with our experiences gained in future major earthquakes. The proposals put forward here are based on a "reasonable degree of protection" that appears to be palatable in the current technological, environmental and economic climate.

5.2 Dynamic Magnification

The currently used equivalent lateral static load [1,2] results in moments, shear and axial forces in the structure, that are derived from analyses based on linear elastic static behaviour. Traditionally this moment and load pattern is combined with similar patterns resulting from gravity loading and the ensuing maxima are used for the strength allocation to each section of every component.

It must be recognized that a static "seismic" analysis is implicitly intended only to give a distribution of potential strength throughout the structure that is considered to be desirable. The deflected shape of the multistorey frame, that results from the static load, implies only that the moment and force pattern is likely to be similar to that which would result during the dynamic response of the structure in its first mode.

The effect of the higher mode dynamic response, referred to earlier, on the bending moment pattern along six storeys of an exterior column of a 12 storey frame is illustrated in Fig. 5. The first diagram shows the results of an approximate elastic analysis for a code [2] lateral load. The other five moment diagrams represent the loading, in relative moment units, at certain instants, measured in seconds, after the beginnings of the North-South component of the El Centro 1940 ground excitation [13].

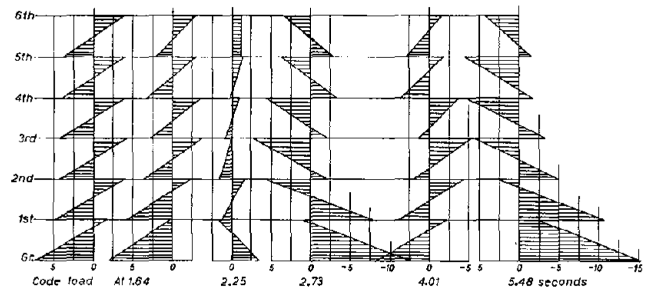


Fig. 5 - Bending moment patterns for the lower six storeys of an exterior column in a 12 storey frame [13].

Limited case studies [13,14] indicated that the deviation from the initial design moment pattern increases with the natural period, T_1, of the frame. This is due to the increased significance of the responses in the second and third mode of vibration, also pointed out in section 3.2. If a reasonable protection against premature plastic hinging in columns is to be provided, then the fact that the moment demand at critical column sections during the higher dynamic modal responses may be large, must be recognised. The dynamic moment demand at a column section may exceed considerably the moment allocated to it by the static analysis, notwithstanding the fact that previously formed beam hinges will greatly limit the load that can enter a column at a beam joint.

To allow in the design process for such column moment increases at the bottom or at the top of a storey, the dynamic magnification factor, ω , is introduced. This multiplier is intended to apply to the peak moments and not to the entire moment pattern. Based on the limited study of a number of frames, subjected to different types of ground excitations, it is proposed that for regular "one-way' frames

$$\omega = 0.6T_1 + 0.85 \tag{3}$$

but not less than 1.2 nor more than 1.8. (see Fig. 6.)

The application of moment magnification is likely to minimise more objectionable damage that could occur in columns before significant yielding in beams would take place.

5.3 Special Cases for Dynamic Magnification of Moments

A disproportionate distribution of beam moments at the restrained base of columns at ground floor is not relevant. At this level the likelihood of hinge formation at a certain strength level, to be examined subsequently, must be accepted. Here the column must be detailed accordingly. Hence the appropriate value of ω at this level should be unity.

When columns are stiff relative to the beams, commonly the column moment pattern obtained from the analysis for code loading at the lower storeys of frames may be such that no point of contraflexure appears in several storeys. This indicates pronounced cantilever action of the column. Such a moment pattern is not likely to be affected significantly at these levels by higher modes of dynamic response. Hence it is suggested that in such cases the minimum value of ω be taken at the first floor level and that its value then be increased linearly to that given by Eq.(3) at the floor below which the point of contraflexure, as indicated by elastic analysis, is located within the middle third of the column height. The interpretation of this approximation is illustrated in Fig. 7.

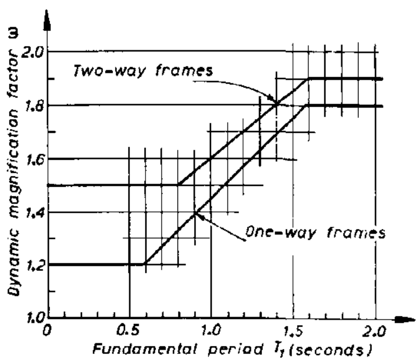


Fig. 6 - The dynamic magnification factor

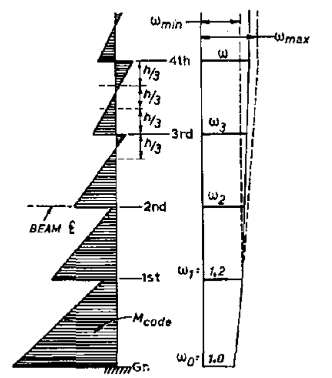


Fig. 7 - The variation of ω when the point of contraflexure is outside the middle third of the column height.

There is no need to give columns the same degree of protection at roof level. Here column hinging and even the formation of a storey mechanism is acceptable. Since the strength of the column need not be larger than the beam load input, the dynamic magnification factor at roof level need not be more than unity.

At the floor immediately below roof level a suitably interpolated value of ω , between unity at roof level and that given by Eq. (3), may be used.

5.4 Consideration of Skew Excitation in Specifying Suitable Dynamic Magnification

It is necessary to make some allowance in the design process for two-directional excitations in "two-way" frames. Moment and axial load input into a column can occur from beams that frame into the column at right angles to each other [2,3,15]. In most overseas codes, however, no reference is made to the phenomena.

The approach currently used in New Zealand considers simultaneous beam overstrength input from both directions, with the critical column section being subjected to corresponding biaxial flexure. The distribution of moments at a joint, between the column sections above and below the floor, follows the proportions indicated by the initial elastic analysis for lateral static load separately along the two principal directions. This procedure intends to recognise the possibility that the building may deform inelastically in its fundamental modal shape in any direction. However, when one considers the relevance of the currently chosen bending moment pattern along a column to its probable response during severe dynamic excitation, than the current method of biaxial code required [2] moment demand, leading to more time consuming section analyses [16,19] appears to exhibit distinct shortcomings.

Once the effect of higher mode responses under unidirectional excitation is appreciated, the probability of the concurrency of corresponding orthogonal excitations must be re-examined. Intuitively one must recognize that the superposition of simultaneous higher mode responses, involving the application at any level of an appropriate dynamic magnification factor, in each of the two directions, would represent a most unlikely event. It should be noted that a column designed for magnified moments, acting separately in each of the two principal directions, will possess very considerable biaxial flexural capacity. Moreover, any probable biaxial flexural demand will need to be considered simultaneously with some probable axial load input from all beams concerned.

It is suggested that columns of two-way frames be made capable of absorbing moments from simultaneously hinging beams during a predominantly first mode response. However, when moments in one direction are being magnified at a floor level, because of participation of the column in the higher modes of dynamic responses also in that direction, a much smaller moment, applied simultaneously from the other direction, should be assumed to act.

To allow for the resistance of biaxial moments, while retaining simplicity in design calculations, it is proposed to magnify further moments used in the design for unidirectional attack on "two-way" frames. A column designed for suitably magnified moments, acting separately along each of the two principal axis of the section, may then be considered to possess acceptable protection against possible hinging during a severe skew attack.

Suitable moment magnifications in one direction, to allow for concurrent earthquake attack, may be arrived at as follows:

(a) At column sections at ground floors, the prescribed code loading should be sustained in any direction [2]. The flexural resistance of the complete set of rectangular reinforced concrete columns of a building along a 45° angle is approximately 90% of that along the principal axes. Hence a 10% increase of the column base design moments caused by unidirectional attack should suffice.

(b) At column sections at upper floors, as a sufficiently severe load combination, one may consider the development of beam flexural overcapacities, M_o (at $f_s = 1.25\, f_y$), in one direction, when beams in the other direction develop simultaneously only their probable strength, $0.9\, M_o$ (at $f_s = 1.12\, f_y$). With an allowance for approximately 10% reduction in moment of resistance at 45° in a square column, the necessary magnification of unidirectional moment, to sustain the biaxial moment input specified above, would be approximately $[M_o^2 + (0.9\, M_o)^2]^{\frac{1}{2}}/0.9\, M_o \approx 1.5$. It is therefore suggested that the design moments, to be used for unidirectional action in columns of two-way frames, that respond primarily in the first mode, be magnified by 50% to accommodate simultaneous moment inputs from adjoining beams. However, when moment magnifications due to higher mode responses become significant, previous consideration of probability indicate that allowances for concurrent attack should be gradually reduced. Accordingly it is proposed that the dynamic magnification factor, ω , applicable to two-way frames be defined as follows:

$$\omega = 0.5T_1 + 1.10 \qquad (4)$$

but not less than 1.5 nor more than 1.9. These values are also shown in Fig. 6.

5.5 Design Axial Column Loads

The maximum earthquake induced axial load in a column will result when the part of the frame above the level to be considered exhibits a predominantly first modal shape, such as indicated by the response to an equivalent lateral static load. Also all beams above that level would need to transmit the maximum earthquake induced shears. These would be associated with the flexural overcapacity of each beam, as outlined in Section 4.4. It has been found [13], however, that with the exception of low rise buildings, beam hinges do not develop simultaneously over a large number of storeys. Moreover, if at any instant beam hinges would occur over a relatively large number of floors, it is unlikely that the majority of these beams would develop their overstrength shear input. A continuous variation of curvature ductility demand must occur over the storeys where the beam hinges form. The formation of typical groups of beam hinges in the frame, referred to in Fig. 5, at differenr instants of its response, is shown in Fig. 8, together with the corresponding shapes of distortion.

It is proposed that in evaluating the maximum likely earthquake induced axial load on a column a reduction of 1.5% be allowed for every floor above the column section considered, when summing the beam over-strength shear forces V_{oe}, up to a maximum reduction of 30% for 20 floors or more. For a six storey building this would correspond with the simultaneous mobilisation of approximately the probable shear strength, based on the mean yield strength of the flexural inforcement, at all floors.

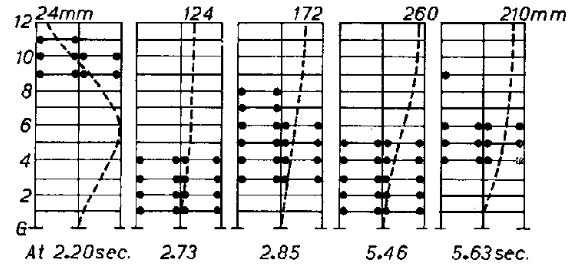


Fig. 8 - The formation of beam hinges in a 12-storey frame at instants of N-S component of El-Centro 1940 earthquake

When higher mode effects are prevalent, recognised in the design by a large value of the dynamic magnification factor ω, beam moment and hence shear inputs into columns may well be of the opposite sense at different floors. (See the frame in Fig. 8 at the instant of 2.20 seconds.) Hence it appears to be justified to consider a further reduction of design axial loads for frames for which the ω factor is large.

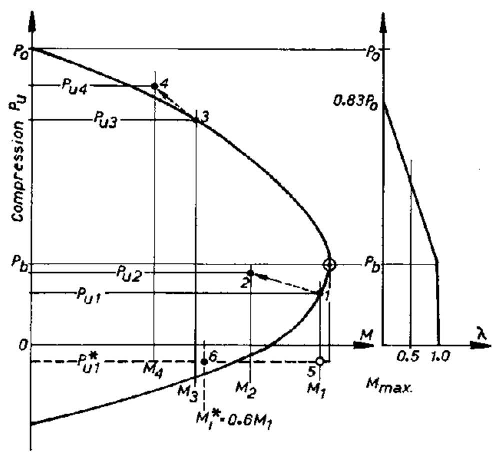


Fig. 9 - Moment-axial load interaction relationship for a column showing different combination of concurrent actions.

It is possible that a frame with a long natural period responds at a particular instant in its first mode. This may then result in an induced axial load in the column that is larger than what we have estimated, using the previous considerations.

The sensitivity of a column section to the accuracy in the estimate of the earthquake induced axial load is examined in Fig. 9. It shows a typical moment-axial load interaction curve for ideal strength. The magnified design moment, M_1, and the total axial load, P_{u1}, that includes gravity load effects with appropriate load factors, is assumed to give point 1 on the curve. In a dominant first mode response, however, the axial load may increase to P_{u2} while the moment demand, now less affected by higher mode distortions, reduces to M_2. (See point 2). It is seen that for any reduction of moment demand there will be ample reserve strength for any feasible increase in axial load demand, provided that the load combination is such that the column hinge formation is tension dominated i.e. $P_u < P_b$. However, if we consider a moment-axial load combination, such as M_3 and P_{u3}, giving point 3 in Fig. 9, it is seen that a possible underestimate of the axial load demand may have more serious consequences. The moment reduction to M_4 during a first mode response may not be sufficient to accommodate the axial load increase to P_{u4}. Moreover, in this compression dominated column (points 3 and 4) particular care should be taken to delay hinge formation because of the limited curvature ductility that would be available.

To give compression dominated column, (i.e. when $P_u > P_b$) increased protection, the reduction in the design axial load due to dynamic magnification should be deliberately underestimated. This is achieved by the introduction of the reduction factor λ, the variation of which with P is also shown in Fig. 9.

By considering now the combined influence of the number of storeys, n above the column section to be designed, the dynamic magnification factor, ω, and the dominance of axial compression, it is proposed that the earthquake induced axial load, P_{eq} be determined as follows:

$$P_{eq} = \{1 - [0.3 + \lambda(0.35\omega - 0.5)] \frac{n}{20}\}\Sigma V_{oe} \tag{5}$$

where the value of ω should not be taken less than 1.4 and that of n not more than 20. The value of λ is given by

$$\text{(a)} \quad \lambda = 1 \text{ when } P_u < P_b \tag{6a}$$

$$\text{(b)} \quad \lambda = \frac{P_o - 1.2P_u}{P_o - 1.2P_b} \text{ when } P_u > P_b \tag{6b}$$

where P_o is the ideal compression load capacity of the column without any moment applied.

P_b is the compression load capacity of the column when the theoretical maximum moment on the given section is developed, and it may be approximated with $0.4f'_c A_g$.

P_u is the design compression load on the column due to earthquake and appropriately factored gravity.

The corresponding relationship between the design axial load due to earthquake and the sum of the maximum beam shear forces from all beams that frame into the column, ΣV_{oe}, is shown in Fig. 10 for the tension dominated columns, i.e. when $\lambda = 1.0$.

5.6 Design Column Moment

It was shown in the previous sections that in order to ensure a reason-

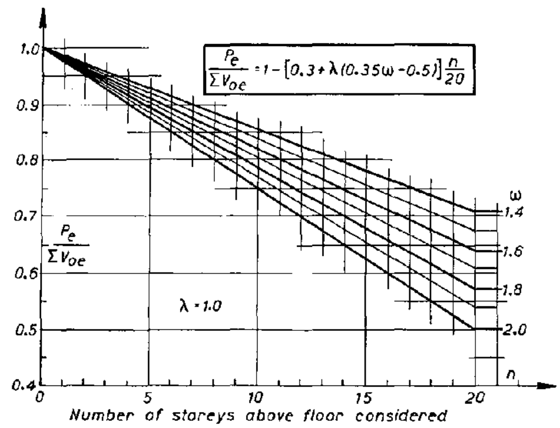


Fig. 10 - The reduction of earthquake induced design axial load with storey number above the level considered.

able protection against column hinging, the moments derived from an elastic analysis for an equivalent lateral static load, M_{code}, need be increased. The minimum ideal strength of the column should not be less than the beam overstrength moment input, as measured by ϕ_o. Also the effect of higher mode responses should be allowed for with the use of ω. Hence the design moment to be used at the critical section of the column, together with the axial load defined in Eq. (5), should be

$$M_{col} = \phi_o \omega M_{code} - 0.3 h_b V_{col} \quad (7)$$

where h_b is the depth of the beam that frames into the column and V_{col} is the column design shear force, to be derived in the next section.

The relationship between these moments (M_{col} and M_{code}) with reference to the centre lines of the members is shown in Fig. 11. The second term in Eq. (7) recognises the reduction of the column moment at the top or the soffit of the beams. To ensure that the moment at this section is not underestimated, when the moment simultaneously developed at the other end of the column is small, only 60% of the probable maximum shear force has been considered to reduce the moment.

It is emphasised that Eq. (7) gives the required ideal flexural strength of the column section and thus the use of capacity reduction factors [8] is not intended.

5.7 Moment Reduction in Ductile Columns

When the axial load on a column produces small compression or net tension, the steel demand for a given moment will rapidly rise. Fortunately the development of the strength of such columns is associated with considerable curvature ductility. For this reason the yielding of such columns, at a level lower than the stipulated lateral load [1,2] on the whole frame, should not be objectionable. It must be remembered that the yielding of one column in a bent is restricted, provided that other columns perform below yield level. Therefore should early yielding occur in one ductile

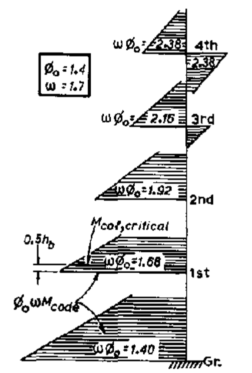


Fig. 11 - Example design moments, with respect to beam centre lines, for the column shown in Fig. 7.

column, where $P_u \ll P_b$, see (Fig. 9) it does not signify hinging during subsequent increase of interstorey drift. Rather the phenomenon is similar to loss of stiffness in one column. If the remainder of the bent is elastic, which may well be the case, a moment redistribution between columns could occur. However, when considering column strength, as part of the capacity design process, we must assume that at the instant under consideration the beam overstrengths in the bent have developed. Because of this other columns would not normally be capable of receiving additional flexural load. Therefore we must accept a certain amount of loss in the lateral load carrying capacity of a bent if one column is permitted to yield prematurely. Provided that the strength loss of the columns in question is not significant, in terms of the overcapacity of the bent, such loss is acceptable.

Consideration of economy, ductility and limited loss in the feasible maximum lateral load carrying capacity of the structure indicate that we should allow reduction in the available flexural strength of certain columns. Such reduction could be considerable if the axial load is tension and if the design moment to be reduced resulted from large magnification because of expected dominance of higher mode dynamic responses. Accordingly it is suggested that when the total design axial compression, P_u , does not exceed $0.1 f'_c A_g$, the design moment, obtained from Eq. (7) be reduced to

$$M_{col,\ reduced} = \left[\left(20 \frac{P_u}{f'_c A_g} + 1\right)(\omega - 1) + 3\right] \frac{M_{col}}{3\omega} \qquad (8)$$

where P_u must be taken as negative if causing tension and provided that:

(a) The value of $P_u/f'_c A_g$ should not be less than $- 0.15$ nor less than $- 0.5\rho_t f_y/f'_c$ where ρ_t is the ratio of the total column reinforcement to the gross sectional area A_g .

(b) The moment reduction used for columns of a bent should not be more than 10% of the sum of the design moments, ΣM_{col}, of all columns of that bent, taken at the same level, and not more than $0.7\ M_{col}$.

The magnitudes of reductions suggested are presented in Fig. 12.

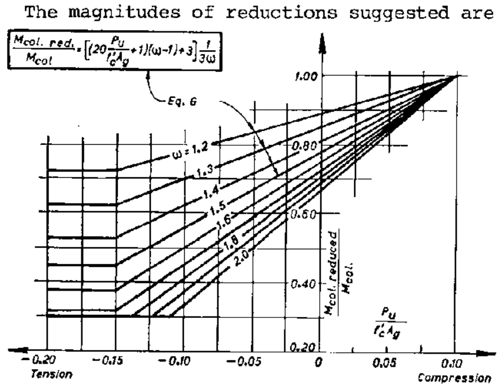


Fig. 12 - Reduction of design moments in tension dominated columns.

Normally there will be only one column in a bent that will qualify for such reduction. For example a column subjected to a net axial tension, P_{u1}^*, that would produce $0.075 f'_c$ nominal tension on the gross concrete sectional area A_g , would qualify for a design moment reduction to $0.6 M_1$, assuming that the required value of ω was 1.5. (See Fig. 12). Let us assume that this symmetrical column is the same as that considered previously and for which the interaction of P_{u1}

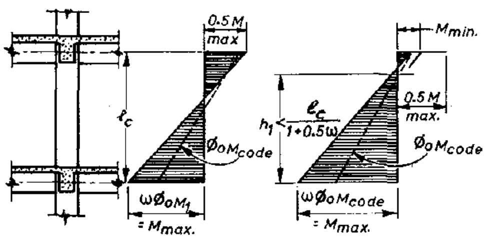
and M_1 gave point 1 in Fig. 9. It is seen that if for an earthquake action in the opposite direction the interaction of P_{u1}* and M_1, shown by the circle at 5, is considered, significantly more reinforcement would need to be provided. However, with the suggested reduction of the design moment, point 6 on the interaction curve is obtained, indicating that this load combination would not govern the design. The loss of moment capacity, $0.4M_1$, is assumed to be less than 10% of the sum of the column flexural capacities in the bent at this level.

5.8 Design Shear Forces for Columns

In the current approach to the design for seismic column shear either the code load shear with an artificially increased load factor [1] is used or simultaneous hinge formation at both ends of a column is assumed. The former approach does not sufficiently recognise the possible variation of shear forces during dynamic excitations, and hence it may be unconservative, particularly for frames with large natural periods. The latter technique commonly results in excessive shear demands because it uses the overly conservative assumption of a storey mechanism hinging in any storey.

The approach proposed here attempts to relate the shear demand to a probable but necessarily severe moment gradient along the column height. It was seen that the column design moments, M_{col}, were magnified primarily to avoid the formation of simultaneous hinges at the top and the bottom of the column. When the yield capacity of a column at one end is approached the simultaneously developed moment at the other end of the column must be small (see Fig. 5), particularly when large dynamic magnification was used in the design.

Column hinging is expected either at the top or at the bottom of a column. Consequently, from considerations of a severe moment gradient that is intuitively considered to be compatible with the moment patterns for the same column at the adjacent storeys, and with reference to Fig. 13(a) it is proposed that for the most common situation $V_{max} = 1.5\ M_{max}/h_c$. This assumes that when the flexural strength of a column is developed at one end, one half of that moment is simultaneously introduced at the other end of the column. To ensure added protection against possibly brittle shear failure, a capacity reduction factor of $\phi = 0.85$ is introduced [8] so that in terms of code load for upper storey columns we have



(a) Case 1 (b) Case 2

Fig. 13 - Moment gradients considered for the determination of column design shear forces.

$$V_{col} = \frac{1.8}{l_c}\,\phi_o\,\omega M_{cod,max} \qquad (9)$$

When the moment pattern, obtained from the initial elastic analysis, indicates that the point of contraflexure is outside the middle third of the column height, Eq. (9) will lead to an unnecessary severe shear load. For such cases it should be sufficient to assume that

$$V_{col} = \frac{1.2}{l_c} (\phi_o \omega M_{code,max} + \phi_o M_{code,min}) \tag{10}$$

This equation will govern the design whenever $\phi_o M_{code,min}/\phi_o M_{code,max} < 0.5\,\omega$, or when the point of contraflexure is located so that $h_1 < l_c/(1 + 0.5\omega)$ as shown in Fig. 13(b).

At the base of the first storey columns considerable plastic hinge rotation must be expected. For this reason the value of $\phi_o \omega M_{code,max}$ in Eq. (9) and Eq. (10) for these columns should be replaced by $M_{o,col}$, the flexural overstrength capacity of the base section, allowing for the axial load on the column that is consistant with the combined loading considered.

The minimum ideal shear capacity of any column should not be less than

$$V_{col,min} = 1.2\phi_o V_{code} \tag{11}$$

for columns of one-way frames and

$$V_{col,min} = 1.8\phi_o V_{code} \tag{12}$$

for columns of two-way frames.

The latter expression takes into account the concurrent effect of shear load on a rectangular section that may be less efficient in resisting shear along a diagonal.

5.9 Reserve Strength in Columns

In the proposed evaluation of the design quantities for columns the demands for the ideal column strength were related to demands specified by building codes. The ideal strength of a section is based on the specified or guaranteed minimum strengths of the materials and the accepted principles of mechanical behaviour. The proposed procedure thus enables the designer to identify the reserve strength of all components in relation to the specified minimum strength demand. This relationship is not obscured by various capacity reduction factors [8] used when gravity and wind loads are considered.

The designer should be aware that the probable strength of a reinforced concrete column, likely to be utilised during severe seismic excitations, will be in excess of its ideal strength. Hence when large inelastic displacements are imposed upon the structure, a column resistance in excess of that relied upon in the proposed design will be available. This may be taken into consideration by the designer when a decision with respect to the degree of protection against damage is being assessed.

One source of strength reserve is the probable strength of the steel and the concrete, that will be in excess of specified values. 5 to 15% excess in yield strength and 20 to 40% excess in concrete compression strength would be typical. Consequently considerable reserve capacity in probable strength of the section would be available, particularly in compression dominated columns.

When considering reserve strength it should be remembered that the proposed beam input load is an upper bound value. Therefore column reserve strengths would be available to absorb moments that, at instants of extreme responses, may exceed those predicted by the $\phi_o \omega M_{code}$ relationship.

Because the splicing of column reinforcement is undesirable immediately above a floor level, the longitudinal steel arrangement in a column will normally be the same below or above a floor. As the larger load demand below or above the floor usually requires more reinforcement, unintentionally the flexural capacity of the less critical section at the floor will be boosted.

The steel content for each column is determined from one particular load combination that is found to be critical for that column. When this critical load combination is exceeded in one column of a bent, yielding of that column would commence. However, the probability of approaching the capacity of all other columns of the same bent under the same load combination is less. A similar relationship may exist between one bent and all the other bents of the whole structure. Consequently additional reserve strength will be available before plastic hinges could develop at one end of all columns of a storey.

6 DESIGN CONSIDERATIONS FOR BEAM-COLUMN JOINTS

It must be recognised that because of the reversed cyclic nature of the loading during inelastic displacements, induced by severe seismic disturbances in reinforced concrete ductile frames, beam-column joints emerge as critical structural components.

One might even suspect that in seismic areas joints might have become the weakest links in a frame. ACI-ASCE Committee 352 recently prepared recommendations [19] for the design of such joints. In the following certain seismic aspects of this report are critically examined. The two major sources of unsatisfactory joint performance under severe seismic conditions, deteriorating shear strength and slip control due to the breakdown of bond, need particular attention.

6.1 Design Criteria

Acceptable criteria for the expected performance of joints in ductile earthquake resistant structures may be formulated as follows:

(1) The strength of a joint should not be less than the maximum strength of the weakest members it connects.

(2) The capacity of a column should not be jeopardised by possible strength degradation within the joint due to inelastic cyclic displacements of a frame.

(3) A joint should not be a prime source of energy dissipation.

(4) During moderate seismic disturbances a joint should preferably respond within the elastic limits so that no repair would be necessitated in inaccessible areas of the structure.

(5) The joint reinforcement, that will ensure satisfactory performance should not present undue construction difficulties.

6.2 Interior Joints

During the most severe loading of ductile frames plastic beam hinges can be expected to develop immediately adjacent to both column faces. The equilibrium condition for this loading situation is shown for a typical interior beam-column joint in Fig. 14(a). The locations and the magnitudes of the internal beam forces, shown in Fig. 14(b) can be determined with a relatively high degree of accuracy. To ensure that the joint core does not become the weakest link in the chain of resisting mechanisms, the beam moments that could be developed during the maximum feasible inelastic frame displacements must be evaluated as outlined in Section 4.5.

The relative magnitudes of the column moments and shear forces that are required to maintain, together with the beam actions, joint equilibrium, as shown in Fig. 14(a), are less certain. During the inelastic dynamic response of a frame there are infinite possibilities for the total beam hinge moment input $(M_1 + M_2)$ to be resisted by the column above or below the floor in question. Moreover, the shear forces in the columns will depend on the column moments generated at the floor adjacent to the one at which the joint in question is located. An acceptable estimate for the mean column shear force can be made, however, as follows

$$V_{col} = \frac{M_1 + M_2 + 0.5(V_b + V_b')h_c}{0.5(l_c + l_c')} \qquad (13)$$

where the notation is that shown in Fig. 14.

With this information the probable maximum horizontal shear force in the joint core can be expressed from Fig. 14(b) as follows:

$$V_{jh} = (A_{s1} + A_{s2})\alpha f_y - V_{col} \qquad (14)$$

where α accounts for the overstrength of the reinforcement, with a typical value of 1.25.

From first principles the vertical joint shear force V_{jv} could also be evaluated using the internal column forces and the relevant beam shear force V_b.

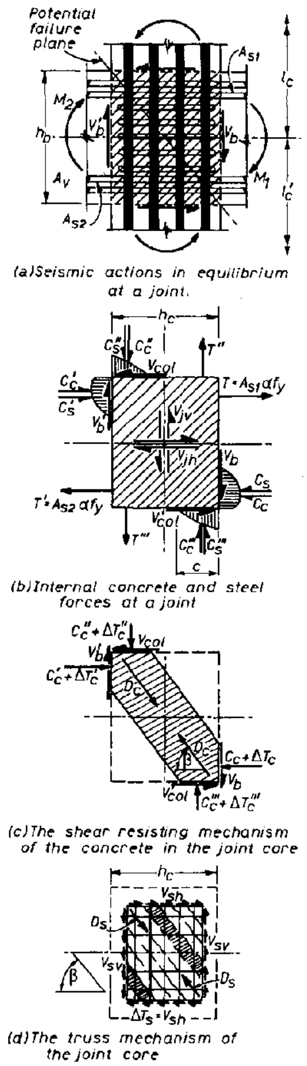


Fig. 14 - Actions at an interior beam-column joint and the mechanism of resistance.

6.3 The Mechanisms of Shear Resistance and their Modelling at Interior Joints

For the case of monotonic loading up to the development of flexural over-capacities, when $f_s = \alpha f_y$, at both beam hinges adjacent to an interior joint core, the internal forces around the joint resisted by the concrete and by the reinforcement, as shown in Fig. 14(b), can be readily identified. In a frame designed for seismic loading, in accordance with Section 5, the critical column sections above and below the joint core will usually respond to the beam actions within the limits of elasticity. Fig. 14(c) shows that for the somewhat idealised example joint, on which for the sake of simplicity no axial column load has been assumed to act, the internal concrete compression forces, together with the column and beam shear and small bond forces, ΔT_c, could form a system in equilibrium. The principal component of this mechanism is a diagonal concrete strut, transmitting the force D_c, as shown in Fig. 14(c). It is thus evident that this concrete mechanism is capable of transmitting a significant fraction of both the horizontal and vertical shear forces across the joint core. It is postulated that the shear resistance of mechanisms associated with aggregate interlock forces along diagonal cracks and those with dowel shear across the reinforcement passing through the joint, are insignificant in comparison with the mechanisms shown in Fig. 14(c). In terms of the forces at the lower right hand corner of the joint (Fig. 14(b)) the horizontal component of the diagonal compression force, D_c, is defined as

$$V_{ch} = C_c + \Delta T_c - V_{col} = D_c \cos\beta \tag{15}$$

where ΔT_c is the bond force transmitted from the beam steel to the concrete approximately within the shaded (Fig. 14(c)) area of the diagonal strut.

If all the remaining steel forces are also to be in equilibrium, then significant bond forces must be induced within the joint core. These will introduce shear stresses to the core concrete. In the majority of cases the diagonal tension capacity of the joint concrete will be overcome at relatively small loads so that the resistance against shear forces, introduced by the longitudinal beam and column bars, by means of shear stresses, would break down. However, as Fig, 14(d) shows, with an effectively anchored horizontal and vertical system of reinforcement a truss mechanism can be developed in which the core concrete supplies the necessary diagonal compression field with a capacity of D_s. By observing in Fig. 14(d) the flow of forces through four node points, the role of two tensions and two compression members becomes self evident. It is to be noted that horizontal shear reinforcement, usually recommended or implied in published studies [19], is insufficient on its own. To sustain a diagonal compression field (Fig. 14(d)) within a joint, it is necessary to maintain horizontal and vertical compression forces at the boundaries of the core [3]. These can be applied to the core concrete by (a) distributed horizontal and vertical reinforcement that is effectively anchored at or beyond the boundaries of the joint core and (b) by external compression forces such as gravity compression on columns and central prestressing [20] in beams. A common and practical solution is to use horizontal stirrup ties and distributed vertical column bars placed so that they pass through the joint core (Fig. 14(a)). It is emphasized that the vertical compression force applied to the joint core by vertical joint reinforcement and compression load on a column is as essential as the horizontal stirrup tie reinforcement, if the truss mechanism (Fig. 14(d)) is to function. It is convenient to denote the horizontal shear resistance of this mechanism by

$$V_{sh} = V_{jh} - V_{ch} = \Delta T_s = D_s \cos\beta \tag{16}$$

where $\Delta T_s = C_s + T' - \Delta T_c$ (Fig.(14b)) is a bond force transmitted from the beam reinforcement to the core concrete.

From considerations of equilibrium and the recognition of a potential diagonal failure plane across the joint, as shown in Fig. 14(a), it is evident that horizontal shear reinforcement needs to be provided so that

$$A_{jh} \geqq \frac{V_s}{nf_y} \tag{17}$$

where n is the number of sets of multilegged stirrup ties, with a cross sectional area of A_{jh}, that are approximately uniformly distributed in the joint core between the top and bottom beam reinforcement, as shown in Fig. 14(a).

6.4 The Interplay between Concrete and Steel Shear Resisting Mechanisms at Interior Joints

The designer's difficulty in satisfying the basic equilibrium requirement of $V_{jh} = V_{ch} + V_{sh}$ is in the allocation of the horizontal shear to be resisted by concrete mechanism only V_{ch} (Fig. 14(c)) and by the truss mechanism V_{sh} (Fig. 14(d)).

ACI-ASCE Committee 352 recommended [19] the use of equations for the determination of safe shear stress sustained by the concrete that have been derived for columns. The components of the concrete shear resisting mechanism in a joint core, however, are significantly different from those encountered in flexural members [3] with or without axial load. For this reason the model given in Fig. 14, rather than that of a beam, will be used to discuss the interplay between the two shear resisting mechanisms.

For this purpose it will be assumed first that the example joint, shown in Fig. 14(a), is subjected to monotonic loading so that the overstrength, with αf_y in tension, of both beams is developed. When, for the sake of simplicity, equal top and bottom beam reinforcement is assumed (i.e. $A_{s2} = A_{s1}$), it is found from Fig. 14(b) that $T' = T = C_c + C_s$. To illustrate both the relative magnitudes and simultaneously the simple equilibrium requirements at say the level of the bottom reinforcement, these forces are plotted for this first yielding as horizontal vector quantities in Fig. 15(a)

Bond transfer is a very significant aspect of joint performance. Therefore some rational assumption needs to be made with respect to bond stress distributions along bars passing through the joint. For the monotonic load stipulated above, a uniform steel stress variation and corresponding constant bond force distribution may be assumed. A part (ΔT_c) of the total steel force ($T + C_s$) (Fig. 15(a)) will be transmitted to the diagonal strut of the concrete shear resisting mechanicsm. It (ΔT_c) combines with the concrete

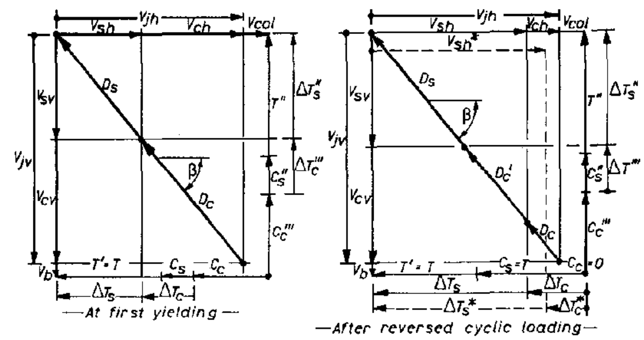


Fig. 15 - The relationship between the internal beam and column forces and the shear resisted by the concrete arch and truss mechanisms respectively (a) at first yielding, (b) after reversed cyclic loading

compression force C_c and the column shear V_{col}, (Fig. 14(c)), to develop, together with similar vertical internal column forces, the principal diagonal compression force D_c Fig.15(a)). The remainder of the total horizontal steel force ΔT_s will be part of the truss mechanism, shown in Fig. 14(d), which when combined with corresponding vertical bond forces from the column reinforcement, will give rise to D_s.

Fig. 15(a) combines these two mechanisms and shows realistic relative proportions of all forces discussed above. Measurements during tests also indicated [21,22] that with negligible axial load on the column the concrete shear resisting mechanisms may account at this stage for over one half of the total joint shear.

After one major excursion in each direction into the inelastic range of behaviour, the moment of resistance in the beam hinges adjacent to the example joint, will be transferred to the beam reinforcement. Permanent large full depth cracks develop across the plastic beam hinge and render the concrete ineffective in compression so that, as Fig. 15(b) shows, $C_s = T$ and $C_c = 0$. At this stage the effective anchorage length of the beam bars through the joint is also reduced.

The total horizontal steel force introduced to the joint core is now larger than in the case of first loading. Moreover, a considerable part of the bond force ΔT_c in the area of transverse vertical compression is absorbed to balance the column shear V_{col}, as can be seen in Fig. 15(b). Consequently only a relatively small horizontal bond force can react with the concrete compression forces from the column (C_c''' and C_s''') to form a much reduced diagonal arch, D_c. The major part of the horizontal steel force ΔT_s will need to be transferred in the unshaded area of Fig. 14(c), necessitating an increased truss action V_{sh}. The transfer of horizontal shear resistance from the linear arch (V_{ch}) to the truss mechanism (V_{sh}), is evident from a comparison of Figs. 15(a) and 15(b).

There is no significant change in the nature of vertical shear transfer. As long as there is no yielding in the column bars, a substantial portion of the flexural compression in the columns is transferred by the concrete to the joint core. This vertical compression ($D_c' \sin\beta$) sustains a significant part of the diagonal compression field of the truss mechanisms and can be considered to replace the role of additional vertical shear reinforcement that might otherwise be needed (Fig. 15(b)).

When several cycles of inelastic reversed load have been imposed on the joint during a severe earthquake, inevitably yield penetration along the beam bars into the joint core occurs. Thereby bond transfer is destroyed and the effective anchorage length of the beam bars is dramatically reduced. The major part of the steel force transfer probably shifts to the centre of the joint, away from the transverse compression exerted by the column. Consequently after yield penetration the major part of the horizontal joint shear V_{jh} must be resisted by the truss mechanism, V^*_{sh} , as shown by the (dashed) vectors in Fig. 15(b).

It is thus evident that reversed cyclic yielding and consequent yield penetration will necessitate more horizontal shear reinforcement. As long as yielding does not occur in the columns no significant change in vertical joint steel demand should be expected.

6.5 The Strength of the Compression Field

A comparison of the loading in the previous example at two stages (Fig. 15) shows that the total diagonal compression force, $D = D_c + D_s$, remains constant and proportional to the total joint shear to be resisted. To fulfil the design criteria, set out earlier, it is necessary to limit the diagonal compression to ensure that a premature and possibly brittle compression failure of the concrete does not occur. Concrete struts, that are formed between diagonal cracks in the core, are subjected to complex loading and distortions which will not permit the crushing strength of the concrete to be attained [3]. One of the major sources of the weakness of the compression field is the formation of two sets of diagonal cracks. When these cracks are permitted to become large because of yielding in the joint shear reinforcement, uneven bearing between faces of the cracks upon closure results, leading to local crushing. It is customary [19] to protect the compression field by limiting the value of the nominal joint shear stress. Confinement is an effective means [3] to strengthen the diagonal compression field. This necessitates ties, transverse to the plane of the frame, that control the possible lateral expansion of the core concrete.

6.6 Bond and Anchorage

Using the simple principles of the diagonal concrete strut and the truss mechanisms (Fig. 14) sufficient horizontal stirrup ties and intermediate vertical column bars can be provided to enable the joint shear to be transferred across the joint core. To provide adequate anchorage for the beam reinforcement, particularly in interior joints, is a more difficult task.

Unfortunately the environment for bond in a joint core may be adversely affected by (a) the condition of the concrete as a result of extensive intersecting diagonal cracks, (b) transverse tensile strains imposed by beam reinforcement that may traverse the joint at right angles to the plane of the frame and (c) yield penetration into the joint from adjacent plastic hinges. Of these the latter is likely to be the most serious.

ACI-ASCE Committee [19] made provisions for the development of reinforcement by invoking Section 12.5 of ACI Building Code [8] and by giving guidance for exterior joint anchorage details. It is strongly emphasised that generally ACI 318-71 development requirements cannot be satisfied for beam bars passing continuously through interior joints that are subjected to severe earthquake loading. For lack of data Committee 352 did not make recommendations [19] for the more critical joints except that it pointed out that smaller bars tend to reduce deterioration under reversed loading. However, the design examples given [19] do not reflect any attempt in this direction. It is implied that #11 (35 mm) Grade 60 (413 MPa) bars passing through a 24 in (610 mm) column could develop over a realistic effective development length of approximately 15 times the bar diameter, in combined tension and compression, 228% of the nominal yield strength of the bar. In the light of the experimental evidence available this appears to be unattainable.

Excellent response to reversed cyclic loading was obtained at the University of Auckland [23] in specimens in which the steel forces were transferred to the core by welded bond (bearing) plates. By increasing the beam steel area within the joint between two bond plates, yield penetration into the joint and hence significant elongation between bond plates, that would have led to hysteresis pinching, was prevented. Although this arrangement cannot be considered as a practical solution to the joint problem, the tests have clearly shown the great significance of proper anchorage within the joint.

It is suggested that when plastic hinges could form adjacent to columns, the diameter of a mild steel beam bar, passing through a joint, should not exceed 1/25th of the column depth in the relevant direction. Experimental evidence obtained so far [21,22] indicates that a large number of excursions with adequate ductility in both direction of seismic loading can be made before slippage of such bars will reduce the strength of the joint.

6.6 Elastic Joints

Two critical aspects of joint behaviour under seismic conditions have been found to result in construction difficulties. Unless the flexural tension reinforcement content in beams is kept small, i.e. less than 1.5%, the horizontal joint stirrup reinforcement may become so large that serious congestion of steel results. The limitation of bar size in beams, to reduce the danger of slippage, may result in the use of an excessive number of bars. Some designers found it necessary to increase member sizes for the sake of steel placement within joint. In spite of these measures in conventionally reinforced joints a satisfactory safeguard does not appear to exist as yet against pull-out of beam bars from joints. For this reason it is suggested that whenever practical the prime cause of these difficulties, beam hinges adjacent to column faces, should be eliminated. This may be achieved by curtailing the beam reinforcement so that a deliberate weakness in flexural resistance results at a more suitable beam section. The relocated potential plastic hinge should be as near as practicable to the column face but far enough to ensure that, as a consequence of reversed cyclic loading, yield penetration will not extend to the column face. In such a beam, when well designed, the steel stresses at the support sections will approach but not exceed the level of nominal yield when simultaneously the overstrength capacity at the relocated plastic hinges is being developed.

As Fig. 16 shows, the disposition of internal forces at such a joint is similar to that of the joint examined previously (Figs. 14(b) and 14(c)). However, a number of distinct advantages become apparent:

(a) Because steel stresses at the boundaries of the joint do not exceed yield, concrete strains are also limited and the concrete compression stresses are relatively low.

(b) The concrete compression forces should not substantially diminish with cyclic reversed loading as all tension cracks should close upon load reversal.

(c) Considering the instantaneous nature of loading a relatively large proportion of the flexural compression force can be expected to be transmitted by the concrete, i.e. there are no significant creep effects.

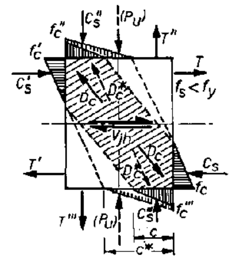


Fig.16 - Internal concrete and steel forces at an elastic joint.

(d) As tensile yielding cannot occur, yield penetration, interfering with the development of the required elastic strength of the flexural reinforcement, cannot take place. The bond conditions will be much more favourable than those in the previous case.

(e) The concrete compression forces and the appropriate proportion of the bond forces from the reinforcement passing through the joint can combine, even after repeated reversed loading, to form an effective diagonal arch D_c similar to that shown in Fig. 15(a). Therefore the share of this mechanism, V_{ch}, in resisting the total horizontal joint shear V_{jh} could be maintained.

(f) As a corollary to this, a smaller shear force V_{sh} need be allocated to the truss mechanism. This results in both a considerable reduction in the joint shear reinforcement and an easing of the steel congestion in the joint core. The relative proportions of the internal forces and the corresponding shear resistance for the previously discussed interior joint will be similar to but more favourable than those shown in Fig. 15(a).

(g) Because yield penetration along the beam bars cannot occur and because the environment for bond in the joint region in general is much better, the use of larger size beam bars should be possible and thereby the number of beam bars could be reduced.

Preliminary tests at the University of Canterbury and at the University of California at Berkeley [24] verified the superior behaviour of elastic joints.

6.7 The Effect of Axial Load

It is to be expected that axial compression will increase the shear strength of a beam-column joint. Hence, for the same beam moment input less joint shear reinforcement will be required. The simple mechanisms examined previously need only be extended to explain how axial compressive column load can contribute to shear resistance.

Fig. 16 shows (with dashed lines) that, as a result of vertical compression load on the column, the neutral axis depth at the boundary section will increase from c to c*. Consequently a larger proportion of the development of beam bars will be in the zone of transverse compression. Equilibrium considerations will require that the main diagonal compression force, D_c^*, becomes steeper and that it engages an appropriate horizontal force ($C_c + \Delta T_c^* - V_{col}$) to maintain its inclination β^* shown in Fig. 17. Thus the share of the horizontal steel force ΔT_c^*, that will combine with the total column compression stresses, must become larger. Fig. 17 shows qualitatively the distribution of the horizontal joint shear components V_{ch}^* and V_{sh}^* with axial compression P_u, while exactly the same beam moments are maintained as in the previous example cases shown in Fig. 15(a).

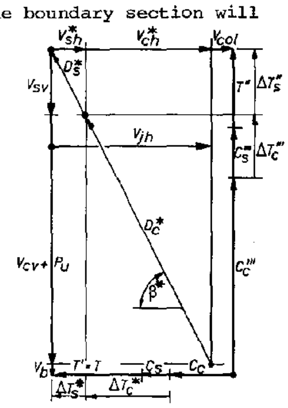


Fig.17 - The relationship between the internal forces of an elastic joint also subjected to vertical compression.

The principles, relevant to axial compression, apply equally to the case when plastic hinges in the beams form at the column faces. Axial load on the column, however, is not likely to reduce significantly yield penetration. For this reason the benefit of axial compression in "inelastic joints" is likely to be less significant than in "elastic joints".

6.8 Exterior Joints

With few exceptions the principles of behaviour discussed also apply to exterior beam-column joints. Because of space limitations only a few important aspects of behaviour and design are stated here.

Generally conditions at exterior joints are less critical because the joint absorbs actions from one beam only and because beam bars can be anchored more favourably by bending them toward the core at or beyond the remote face of the column. After bond penetration into the joint core, standard or extended 90° hooks may still be capable of providing full anchorage. At first loading beyond yield a diagonal strut, similar to that shown in Fig. 14(c), can find full support at the bend of correctly detailed beam bars. However, high hoop stresses, exerted by large diameter bars against the core concrete, that has been damaged by intersecting diagonal cracks, have been observed to have led to excessive hoop deformations and consequent slip [25].

One critical aspect of exterior joint behaviour is that of bond transfer from column bars that pass through the core near the face of the column which is remote from the beam that enters the joint. The cover concrete over these column bars tend to spall relatively easily, particularly when heavy horizontal joint stirruping is used [3]. Moreover, the spalling of the cover concrete may extend beyond the joint area and it may significantly reduce the flexural strength of the columns. [26].

The difficulties may be overcome at exterior joints, particularly when relatively small columns are used, if the bars are anchored in a beam stub, such as shown in Fig. 18. This way the hoop bearing stresses induced by beam bars can be introduced to a mass of concrete that is not subjected to joint shear. Moreover, the anchorage of the column bars adjacent to this stub is greatly improved. In a series of tests the superior performance of specimens with beam stubs was identified [3].

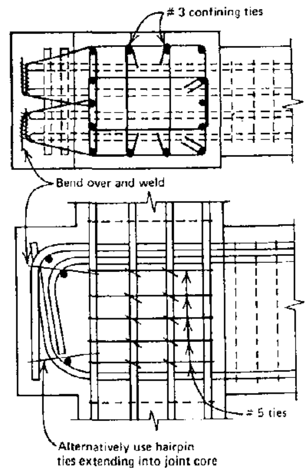


Fig.18 - Suggested arrangement of joint reinforcement at an exterior joint with a stub-beam [3].

7 CONCLUSIONS

The principles of the "capacity design" philosophy, as applied to reinforced concrete ductile frames, have been presented. The hierarchy in the preferred failure mechanisms was emphasised. A simple technique, that uses the quantities of an elastic analysis for a code prescribed equivalent lateral static load, has been presented to determine the design moments, axial loads and shear forces for columns. The mechanisms of shear resistance in joints have been examined in some detail.

Several of the suggested design approaches are the outcome of informal discussions sponsored in 1976 by the New Zealand National Society for Earthquake Engineering. They represent the current trend which is likely to influence the drafting of the new New Zealand Code of Practice for Reinforced Concrete Structures.

Specific suggestions mainly for those aspects of reinforced concrete design for which no provision have been made in currently used codes [1,8], are presented separately in the Draft Recommendations.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

REINFORCED CONCRETE DUCTILE FRAMES
THE USE OF DIAGONAL REINFORCING
TO SOLVE THE JOINT PROBLEM

by

Russell A. Poole
Principal
Holmes Wood Poole & Johnstone
Consulting Engineers
Christchurch, New Zealand

1 INTRODUCTION

This paper describes the design and detailing of 2 multi-storey reinforced concrete buildings in Wellington, New Zealand, in which diagonal reinforcing has been adopted to solve the joint problem. Both buildings are described briefly and the design method is outlined. The design and detailing of the diagonal reinforcing is discussed. Prefabrication of beam cages is essential when using details of this complexity but this in turn results in considerable site economies.

Williams City Centre was designed in late 1972 and construction was completed in early 1976. It is a tube within tube structure, 28 storeys high but the outer tube is pierced or frame only in the top 20 storeys and is virtually solid shear walls below. The plan dimensions are 36.6 metres (120'3") x 18.8 metres (61'9"). Columns in the outer pierced tube or frame are close spaced, the gravity moments are very small so the outer tube frame moments are virtually completely due to lateral loads.

Diagonal reinforcing is used in the joints to transfer a proportion of both the horizontal and vertical shears and to limit diagonal cracking and consequent deterioration of the truss action by which the balance of the shear is transferred. At the time this design was completed the joint problem was identified but little research had been done and no design guidelines evolved. Assumptions were necessary in the design, many of which now appear doubtful. Nevertheless the design evolved represented a considerable improvement on previous details and it is likely that in the event of a major earthquake the building's performance will reflect this improvement.

Lambton Square has been designed during the last year and construction has not yet started. It is 18 storeys high and although the lateral load resistance is by means of peripheral frames the columns are well spaced and beam flange action is negligible. The plan dimensions are 34.5 metres (113'6") x 25 metres (82'0"). Because of the floor system and the concentration of lateral load resistance in the heavy peripheral frames the gravity loads on the frames are small in relation to the seismic loads.

Diagonal reinforcing is employed in the midspan of the beams to form a single elongated plastic hinge per bay. The remaining beam stubs and beam-column joints are detailed to ensure they remain elastic and thus the concrete can be utilised to provide arch action in both zones. This results in considerably reduced secondary reinforcing in these areas. A test programme of this system is planned at the University of Canterbury for late 1977 and it is likely that this will confirm that this system is satisfactory and a major improvement over conventionally reinforced frames.

2 WILLIAMS CITY CENTRE

DIAGONAL JOINT REINFORCING

2.1 Building Description

The tower of Williams City Centre is 28 storeys high and is the main building in a complex of 3 adjoining buildings. The plan and elevations of the tower are shown in figures 1 and 2.

The lowest 3 floors are retail space, the next 5 car parking and the top 20 office accommodation. The car parking floors are connected to the previously constructed adjoining car parking building. To allow satisfactory movement and parking of cars and to satisfy other architectural and town planning requirements a floor plan 36.6 metres (120'3") x 18.8 metres (61'9") was adopted. Economy dictated that floor to floor heights be kept to a minimum and this reason together with the recommendations of Reference (2) dictated the structural system adopted.

2.2 Structural System

The floor slabs are insitu flat plates with drop panels over the internal columns. The internal walls surrounding the service core are insitu concrete and carry both gravity loads and lateral loads particularly in the north-south direction. The external walls are solid reinforced concrete in the bottom 8 storeys, penetrations being kept to a minimum to allow the car park to function. The top 20 storeys of the external walls are a peripheral frame or pierced tube. 1.0 metre (3'3") wide columns are spaced at 3.0 metre (9'9") centres and the beams are 675 mm (27") deep in a floor to floor height of 2.9 metres (9'6"). These member proportions were chosen to encourage beam hinging and were accepted as satisfactory by the architect.

2.3 Lateral Load Behaviour

In the long east-west direction lateral loads are resisted by tube action on the external tube. The internal shear walls are not interconnected by lintels and have minimal stiffness in this direction.

In the short north-south direction tube action of the outer tube is considerably assisted by the internal shear walls for the first few levels of pierced external tube. The internal shear walls are virtually fixed through diaphragm action to the outer box of shear walls and are consequently very stiff relative to the external frame in this area.

2.4 Frame Design

A capacity design approach was followed. Gravity actions on the beams were insignificant compared with seismic actions and equal top and bottom reinforcing resulted. The maximum beam shear associated with beam overstrength capacity was limited to $0.50\sqrt{f'_c}$ MPa ($6\sqrt{f'_c}$ psi). Reinforcing in the form of double rectangular spiral stirrups was provided to carry all of this shear for a distance d adjacent to the column face.

Columns were designed for moments associated with the development of over-strength capacity moments in the beams and axial loads associated with simultaneous hinging of a proportion of the beams above the column in question. Typical maximum beam and column reinforcing is indicated in sections 1 and 4 of figure 3.

2.5 Joint Design and the Use of Diagonal Reinforcing

At the time this building was designed in late 1972 the main problems inherent in beam-column joints had been identified but design methods had not been evolved and few joints had been tested. Common practice was to reinforce the joint with horizontal ties only as required by ACI 318-71 and the 1968 SEAOC code. The problems identified were:

(a) To form an effective truss mechanism to transfer horizontal or vertical shear across the joint both vertical and horizontal reinforcing are necessary within the joint.

(b) The transfer of forces from the longitudinal bars, particularly the beam bars, to the concrete struts, which involves extremely high bond stresses.

(c) Yield penetration into the joint from the face of the column thus causing a further increase in the required bond stress.

(d) The development of diagonal cracking which when associated with yielding of joint reinforcement and reversal of loads causes deterioration of the concrete strut action.

Initially joint reinforcement was calculated assuming that both horizontal and vertical reinforcing would be used in the joint and on the basis of the following assumptions:

(1) Zero axial load in columns. This is conservative in most cases but probably unconservative where tension occurs.

(2) Beam moments were calculated on the assumption that beam bars developed stresses of 350 MPa (50 ksi) compared with the guaranteed minimum of 280 MPa (40 ksi).

(3) Midpoint contraflexure was assumed in both beams and columns.

(4) Beam compression forces are carried 50% on steel, 50% on concrete.

(5) It was assumed that compression steel forces decrease to zero over the first 190 mm (7½") and 140 mm (5½") in the beam and column bars respectively. This area of the joint is under biaxial stress which improves bond transfer.

(6) Bond stresses are assumed to be zero over the first 190 mm (7½") and 140 mm (5½") in the beam and column tension bars respectively.

(7) Tension bar forces are assumed to be transferred by bond over the central 600 mm (24") and 400 mm (16") of column and beam respectively and hence by truss action that requires joint stirrup reinforcement.

In effect it was assumed that compression forces would be transferred by arch action and tension reinforcing forces by truss action.

These calculations indicated that, with the maximum beam steel of 8 - 24 mm bars both top and bottom, joint stirrups, both vertically and horizontally, would have to be 20 mm dia. at 55 mm centres. This is absolutely impractical and so the diagonal alternative was considered and adopted.

2.6 The Diagonally Reinforced Joint

The joint reinforcing, adopted is shown in Figure 3. The 4 interior bars of the 8 total bars both top and bottom are cranked through the joint. To allow the bars to pass it is necessary to offset these bars as indicated in section 4 of figure 3. This has the secondary effect of offsetting the bars from the associated stirrup which was considered to be acceptable in the body of the beam.

In the simplest terms a beam-column joint can be considered to be subject to a principal tension across the diagonal and a principal compression across the other diagonal. Thus the diagonal bars can be considered to resist principal tension and compression directly.

In the joint adopted truss action is assumed to transfer approximately 50% of the horizontal and vertical shears and the diagonal bars are assumed to carry the balance. The bond stresses will be little changed from the conventional alternative considered but improved behaviour seems likely since only 50% of the shear is required to be transferred to the concrete struts. Because of the direct resistance of the diagonal bars to the principal tension force it is anticipated the control of diagonal cracking will be much improved. Joint stirrup reinforcement to effect truss action transfer of 50% of the joint forces was calculated at a maximum of 16 mm dia. at 75 mm centres both vertically and horizontally. This proved to be quite practicable with careful detailing and fabrication.

2.7 Reinforcing Detailing and Prefabrication

When a complex reinforcing detail such as the diagonal reinforcing of the beam-column joint is adopted prefabrication becomes a necessity for reasons of tolerance alone. Building height, and in this case climate, were other compelling reasons.

Columns were prefabricated in single storey lifts. Beams were fabricated in 2 bay cages 5.95 metres (19'6") long and lowered over the column cages. Beam cages were butted at midspan and lapped with separate lapping bars. The use of rectangular spiral stirrups and ties was adopted to avoid having to anchor each one separately and thus the congestion problem created. In addition considerable placing labour was saved. For tolerance in erection the joint horizontal steel was detailed in the form of pairs of hairpin bars which had to be temporarily tied to the vertical joint steel during erection and finally tied in place around the column bars through the joint as indicated in section 3 of figure 3.

3 LAMBTON SQUARE

DIAGONAL BEAM REINFORCING

3.1 Building Description

The Lambton Square building will be 18 storeys high and has a floor plan measuring 34.5 metres (113'6") x 25.0 metres (82'0") as indicated on figure 13. Typical floor to floor height is 3.15 metres (10'6").

The lowest two floors are retail shopping, the 3rd level provides car parking and levels 4 to 18 are office space. Podium floors which are separate structures extend from level 1 to 4. The planning requirements particularly at the retail and car parking floors, the interconnection with the podium areas and the need to functionally separate the office space from the 3 lower levels dictated a frame structure. Architectural requirements and the need to keep floor to floor heights to a minimum so as to achieve the maximum floor area within a strict height limit dictated that the frames be restricted to the periphery of the tower structure.

3.2 Structural System

A metal deck floor system with insitu topping spans between precast internal beams and exterior insitu beams. Both internal and external columns are insitu. Internal precast beams are wide and shallow consistent with the minimum floor height and have insignificant stiffness compared with the exterior east and west frames.

Lateral loads are resisted by the frame action of the external wall frames. Despite the relatively stiff beams flange action under lateral loads was shown by the analysis to be relatively insignificant and was disregarded in the design.

3.3 Frame Design

The capacity design approach was adopted using the recommendations of the N.Z. National Society of Earthquake Engineering Workshop on Ductile Moment Resisting Frames. These recommendations will be published in the Society's Bulletin during the latter half of 1977 and the basic principles are presented in Paulay's paper (2).

Beam design for loads specified by the N.Z. Loading code NZS 4203 : 1976 (3) is discussed in detail in section 3.4. Column actions were evaluated and designed for using the recommendations of Professor Paulay. The beam column joints which remain elastic were also designed using the recommendations outlined by Paulay from Reference (4).

3.4 The Diagonally Reinforced Midspan Plastic Hinge

Detailed design of this building was begun in mid 1976. By that time research and development of design philosophy indicated that the solution of the joint problem could best be achieved by keeping the plastic hinges away from the column faces and hence keeping the joints elastic. In addition the plastic hinge shear stress limit had been halved to $0.25 \sqrt{f'_c}$ MPa ($3\sqrt{f'_c}$ psi).

Initially the solution suggested by Bertero and Popov (5) and illustrated in figure 4 was considered. However it was decided that a longer zone 1 was prudent to protect the column and this combined with 45° diagonals in zone 2 meant zone 3 became fairly small and the hinges were located a significant distance from the column face in relation to the maximum moment. Initial investigation also indicated difficult problems in fabricating the reinforcement.

At this stage Paulay made the suggestion to adopt a single long central diagonally reinforced hinge similar to those used in shear wall coupling beams. The suggestion originated from work by Paulay and Spurr (6). Figures 5, 6 and 7 illustrate the superiority of diagonally reinforced hinges as opposed to conventionally reinforced hinges.

Figure 8 is a schematic representation of the beam reinforcing adopted and Figure 9 shows moment capacity and moment demand diagrams. This diagram is presented in terms of pure seismic moment demand for simplicity. Providing gravity moments are relatively minor, as in this case, they have little effect on the argument. Figure 10 illustrates the internal forces in the beam and joint.

Essential features of this design are

(a) Yielding is confined to the central diagonally reinforced "hinge" region which behaves like a diagonally reinforced coupling beam. Shears and moments are carried by a reinforcing tie and strut. The concrete and secondary reinforcing in the hinge region is required only to prevent buckling of the strut as a whole and of the individual reinforcing bars of the strut.

(b) The beam stubs and beam-column joints remain elastic hence allowing reliance on concrete arch or strut action.

(c) To achieve the same displacement ductility an increase in curvature ductility is required. However in this hinge system yielding can be spread over a considerably greater length and thus reinforcing strains and crack widths can be reduced.

(d) There is no possibility of sliding shear failure in the plastic hinge because shear and moment are carried directly by the reinforcement. There is a slight possibility of sliding shear failure at the end of the stub but this is unlikely because the moment gradient encourages spread of yielding and additional secondary reinforcement has been provided, as described in section 3.5, to further encourage distribution of yielding and reduced crack widths.

(e) Beam geometry has several influences. A long stub means steeper diagonal reinforcement which is more efficient but the resulting short hinge has greater rotational ductility demand thus requiring greater steel strains and resulting in greater crack widths. A longer stub also gives greater protection to the joint but requires greater steel content at the column face.

Consideration of possible reinforcing layouts such as shown in figure 11 indicated that an arrangement resulting in column face reinforcing equal to twice the diagonal reinforcing would be satisfactory. From a consideration of beam geometry it can be shown that

$$l_s/l_n = 0.5 - \frac{K \cos \alpha}{2n}$$

where l_s = length of stub from column face
l_n = clear span of beam
K = Factor of Safety against yielding at column face
n = ratio of steel area at column face to area of the diagonal reinforcing
α = angle to the horizontal of the diagonal reinforcing.

Typical values for K (1.25) and n(2.0) gave $l_s = 0.194\ l_n$ when $\alpha = 12^{\circ}$.

3.5 Beam Design

From equilibrium it follows that the vertical component of the tension and compression forces at yield equal the applied design shear.

Secondary steel is provided to prevent buckling of the compression strut and to hold concrete in the body of the beam during an earthquake.

A modest allowance is made for the contribution of concrete to shear transfer in the stub and conventional beam stirrups carry the balance of the shear.

Steel is provided at the end of the stub to resist the vertical component of the compression strut force based on its over-strength capacity.

3.6 Joint Design

As outlined above the design ensures that the joint remains elastic. Proposals by Blakeley (4) suggest that more than 50% of horizontal joint shear can be carried by concrete arch action in an elastic joint providing it carries no tension. Horizontal joint steel is reduced accordingly.

Vertical shear is resisted by the same concrete arch action with assistance from vertical column reinforcing passing through the joint.

Design of joint reinforcing followed the recommendations of Reference 4.

3.7 Reinforcing Detailing and Fabrication

Details of the beam reinforcing are shown on Figures 13 and 14. This system was designed in close consultation with both Paulay and the builders, Civil and Civic (N.Z.) Limited. Other systems indicated schematically on figure 11 were considered. Options b and c were rejected because of concern regarding anchorage and congestion with the beam-column joint. Option d solves the anchorage problem but is not practical because of access problems and is extravagant in terms of construction time. A system using option e was developed but ultimately rejected because it involved a set placing sequence and was likely to occupy too much site placing time.

The system adopted involves only 3 basic prefabricated beam cages viz. the male, female and smaller end types shown in plan on figure 13. There are minor variations in the basic types to accommodate the corner interlapping. Apart from the corners where the cages have to be slid horizontally into position all other cages are lowered vertically into position. This ability considerably eases tolerance problems. Internal columns are prefabricated in single storey lifts to facilitate the above beam erection system. Corner columns are prefabricated in 2 storey lifts.

With reference to figures 13 and 14 the following points of interest are noted

(a) The light reinforcing within the diagonally reinforced hinge area.

(b) The concentration of vertical stirrups at the end of the beam stubs to carry the vertical component of the diagonal compression strut.

(c) Horizontal joint reinforcing consisting of column ties carried through the joint in the corner columns. This was possible because of the need to slide the corner beam cages horizontally into position.

(d) In interior column joints straight bars are used to carry the horizontal shear and these are anchored in the elastic beam stub.

In addition they are extended into the hinge region as shown to assist in spreading the yielding and cracking in this area thus reducing the possibility of sliding shear failure.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 The Diagonally Reinforced Joint : Williams City Centre

The adoption of diagonal joint reinforcing in Williams City Centre represented a considerable improvement on conventional reinforcing in the light of research at the time. Some of the assumptions made in the design of the joint and the plastic hinge shear stress limit of $0.5 \sqrt{f'_c}$ MPa ($6 \sqrt{f'_c}$ psi) have subsequently been shown to be doubtful. Nevertheless it is difficult to envisage these joints seriously deteriorating or full depth shear cracks developing at the faces of columns except perhaps in limited areas.

With improvements to minimise the possibility of sliding shear failure such as central prestressing or those suggested by Fenwick and Irvine in Reference (7) and the moving of hinges sufficiently far away from the columns to reduce the possibility of yield penetration this type of joint reinforcement warrants further research.

Despite the apparent complexity of the reinforcing layout with careful detailing and the extensive use of prefabrication construction problems were minimal and considerable on site economies were effected.

4.2 The Diagonally Reinforced Midspan Plastic Hinge : Lambton Square Building

The solution to the joint problem in this building has been effected in a time honoured manner viz. to remove the problem. The diagonally reinforced midspan plastic hinge despite increased rotational ductility demand, is expected to have much superior behaviour because of increased hinge length and hence distributed yielding and cracking. Hinge behaviour is dependent solely on the diagonal reinforcement except for secondary reinforcement to prevent buckling of the compression strut. With minor additional secondary reinforcement the likelihood of sliding shear failure at the end of the beam stub is reduced.

The beam stub and joint are designed to remain elastic and consequently the concrete can be relied upon to provide substantial arch action to transfer shears. Secondary reinforcement is thus greatly reduced particularly in the joint.

A test programme on this beam reinforcing arrangement and the associated joint is planned at the University of Canterbury during the latter half of 1977.

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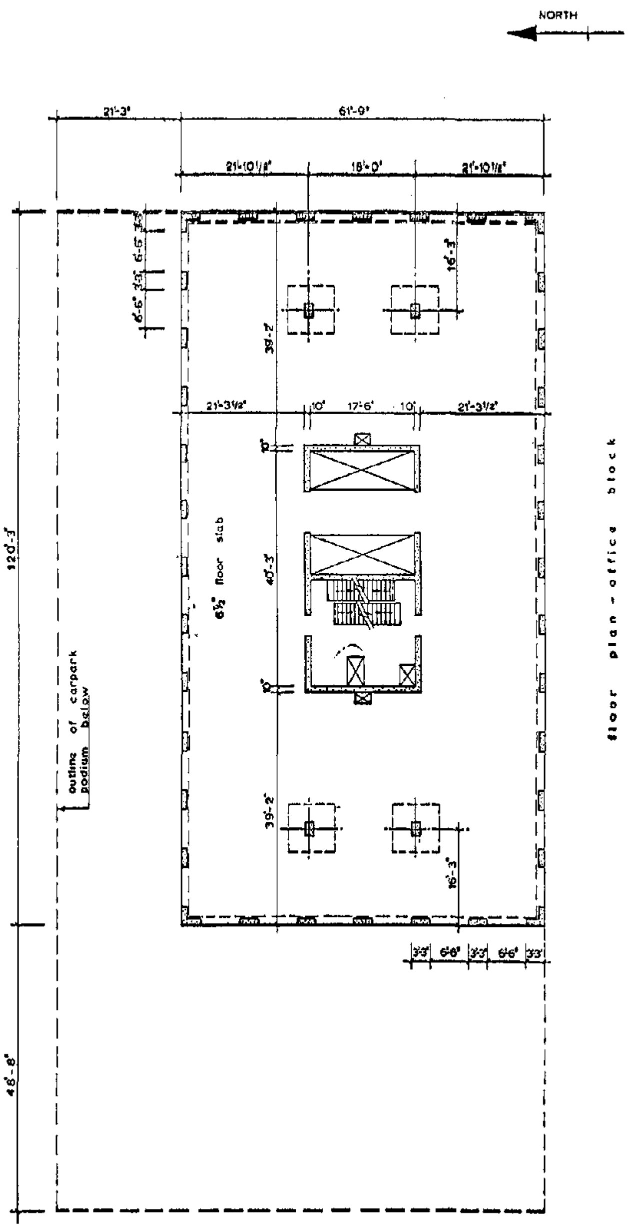
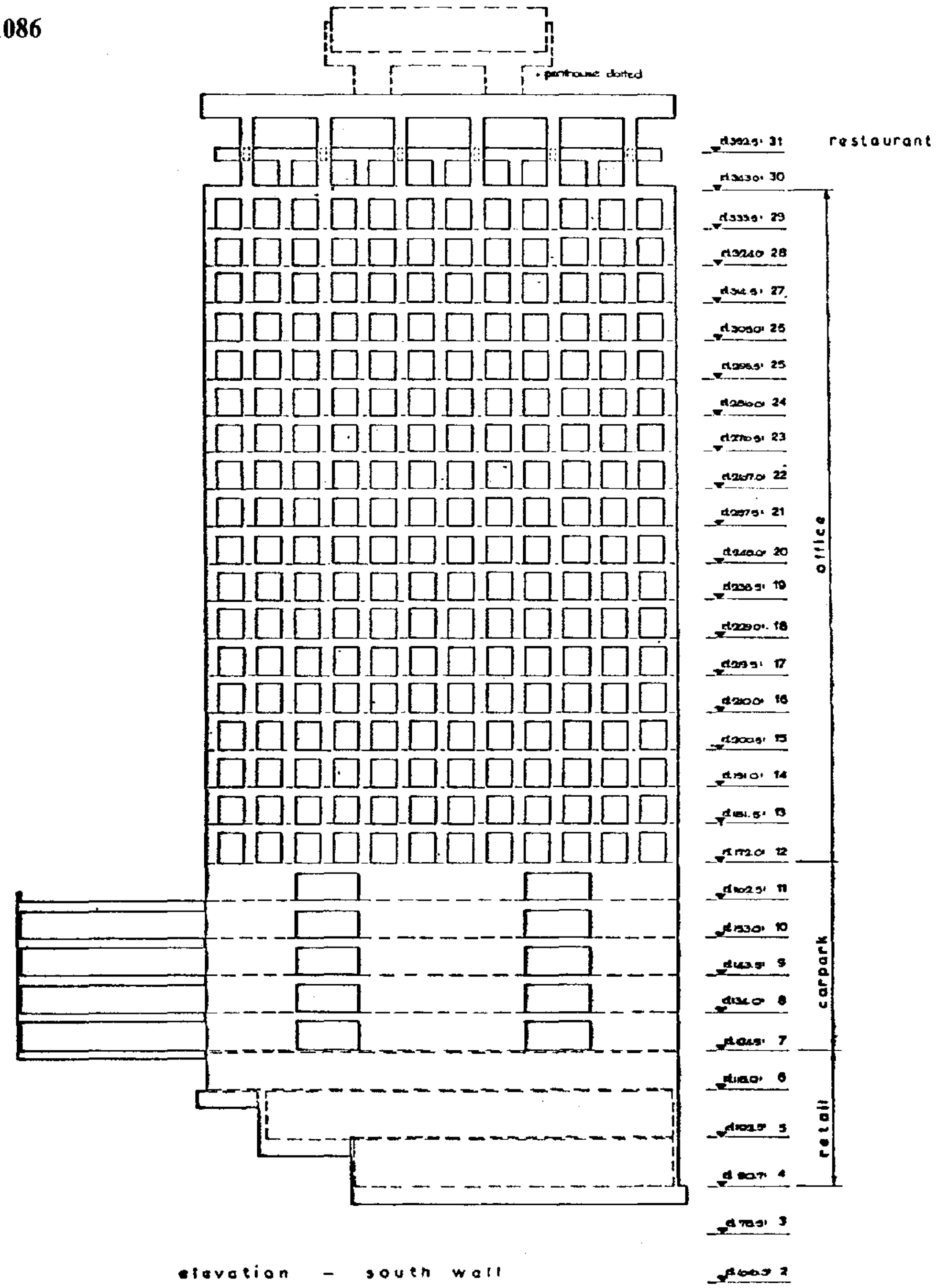
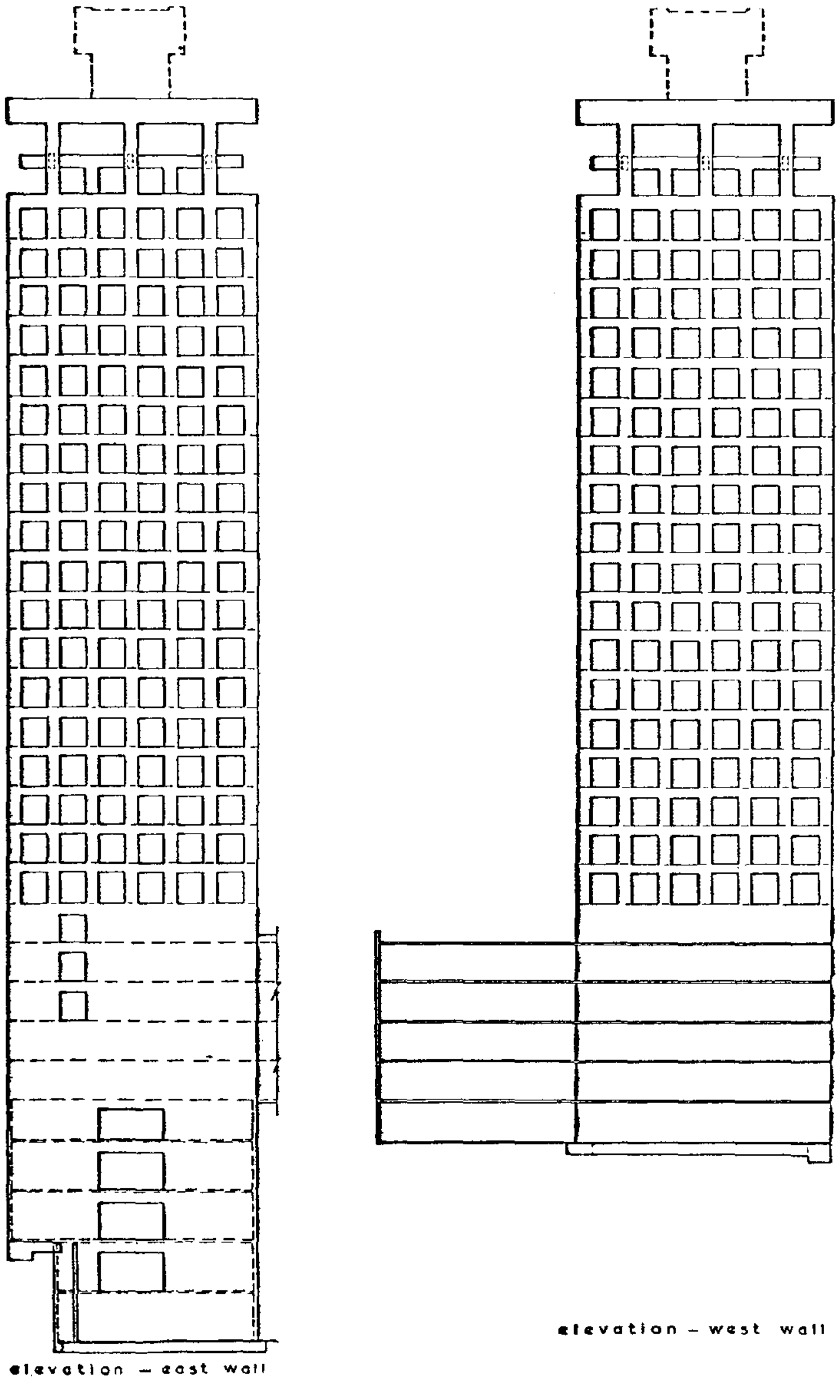


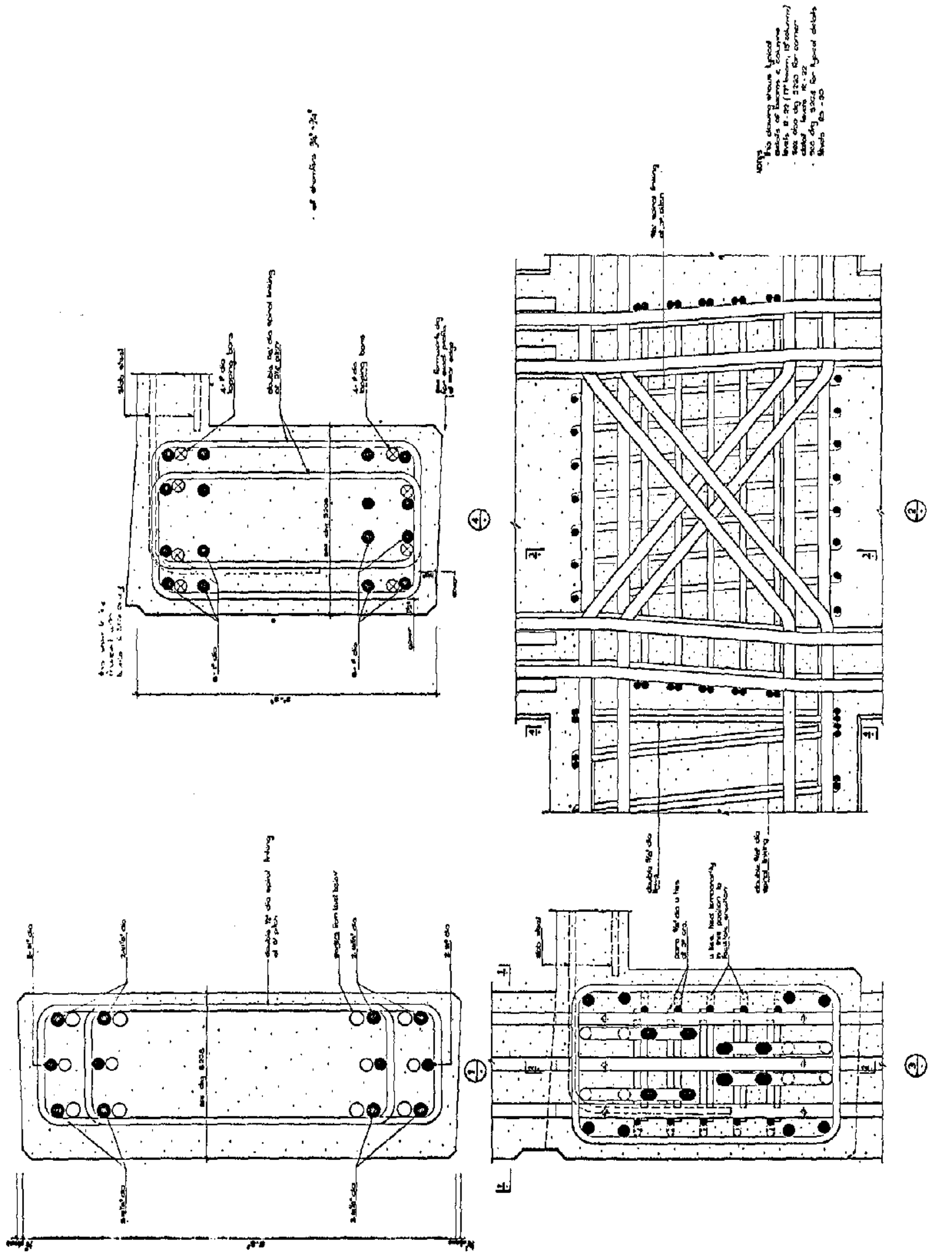
Williams City Centre
Floor Plan
Fig 1



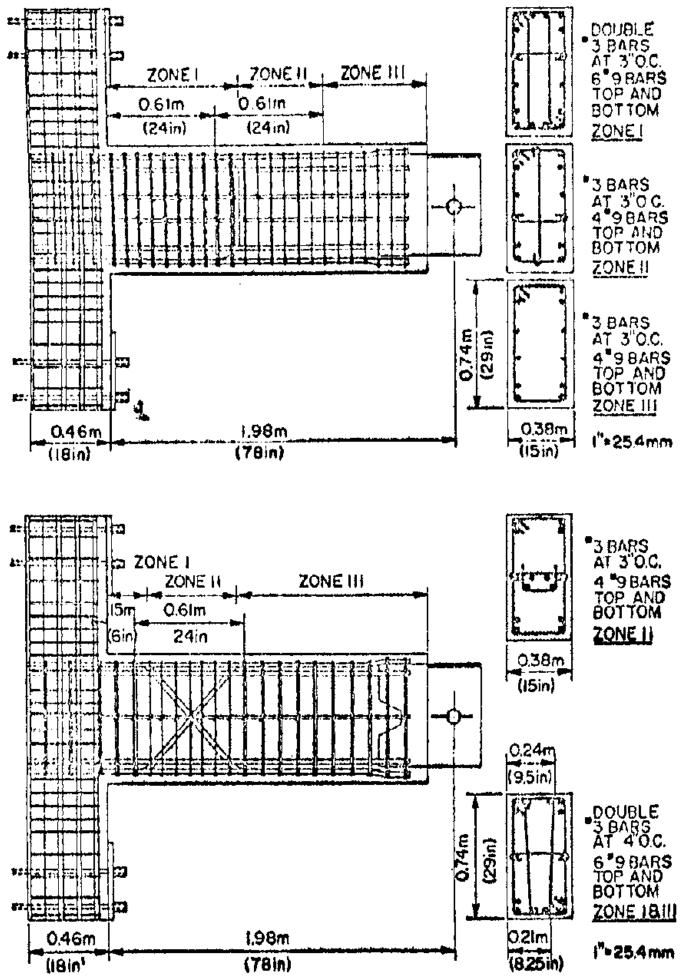
Williams City Centre Wall Elevation
Fig 2



Williams City Centre Wall Elevation
Fig 2 (cont'd)



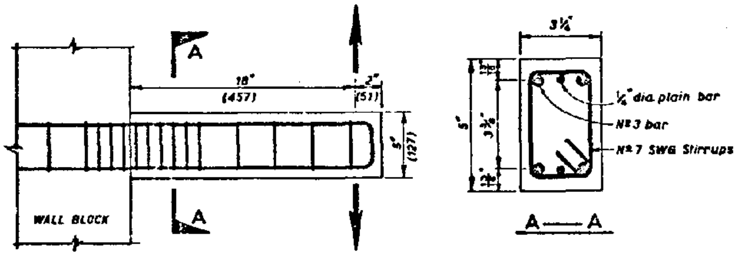
Williams City Centre
Diagonal Joint Reinforcement
Fig 3



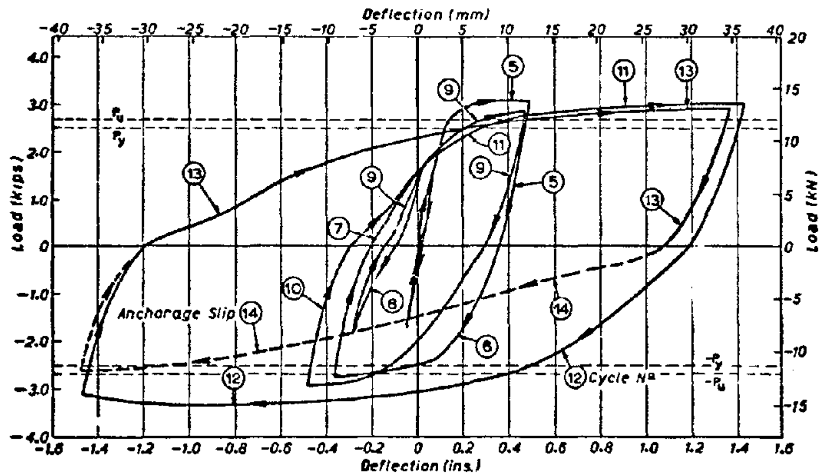
Alternative Beam Hinge Reinforcement

Fig 4 :

(5)



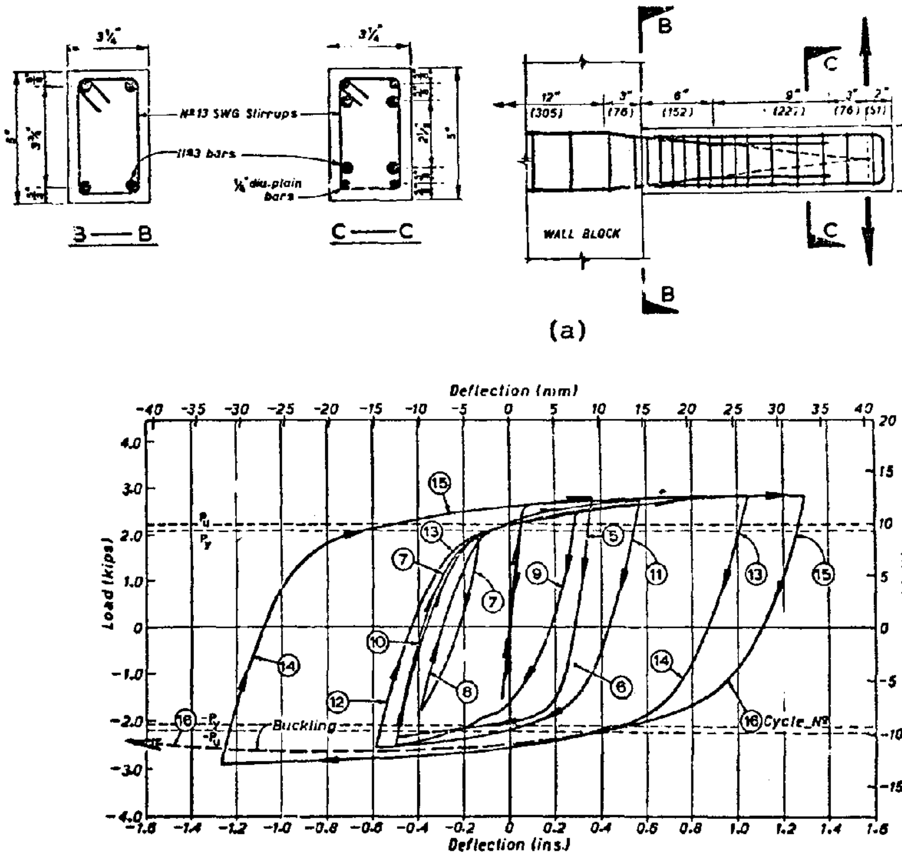
(a)



(b)

Fig. 5 : Details of a Conventionally Reinforced Beam-Wall Junction Specimen.
(a) Dimensions and Reinforcement.
(b) Load-Deflection Relationship.

(6)



(b)

Fig. 6 : Details of a Diagonally Reinforced Beam-Wall Junction Specimen.
(a) Dimensions and Reinforcement.
(b) Load-Deflection Relationship.

(6)

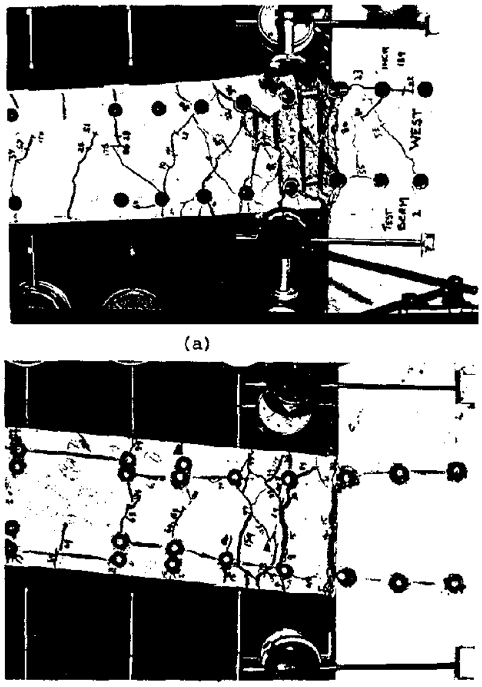


Fig. 7 ; The Plastic Hinge Zones of Beam-Wall Junction Specimens.

(a) Conventionally Reinforced Beam.
(b) Diagonally Reinforced Beam. (6)

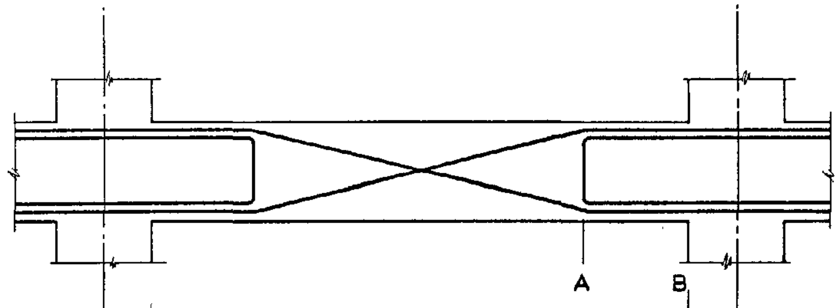


Fig 8 : Elevation of Beam Reinforcing

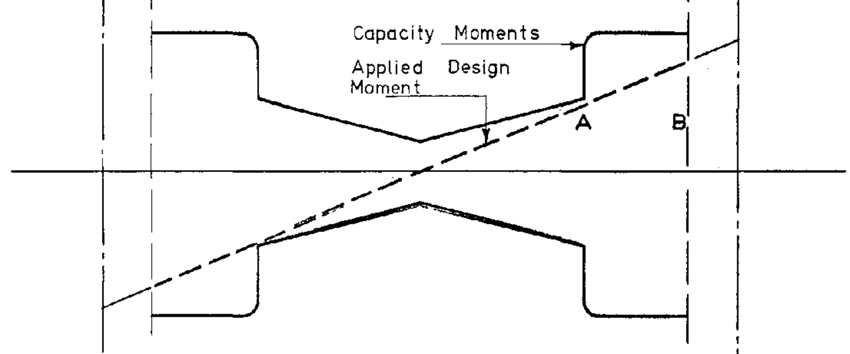


Fig 9 : Bending Moment Diagram

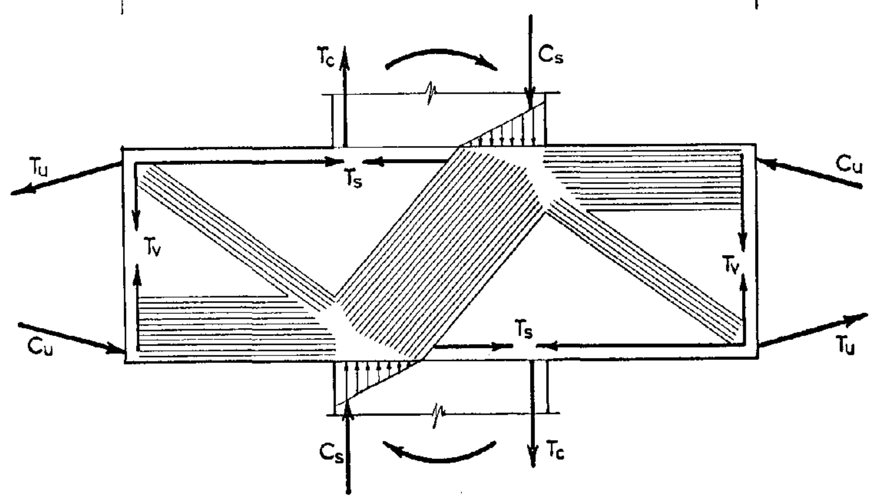


Fig 10 : Concrete arch action in beam stub and beam-column joint

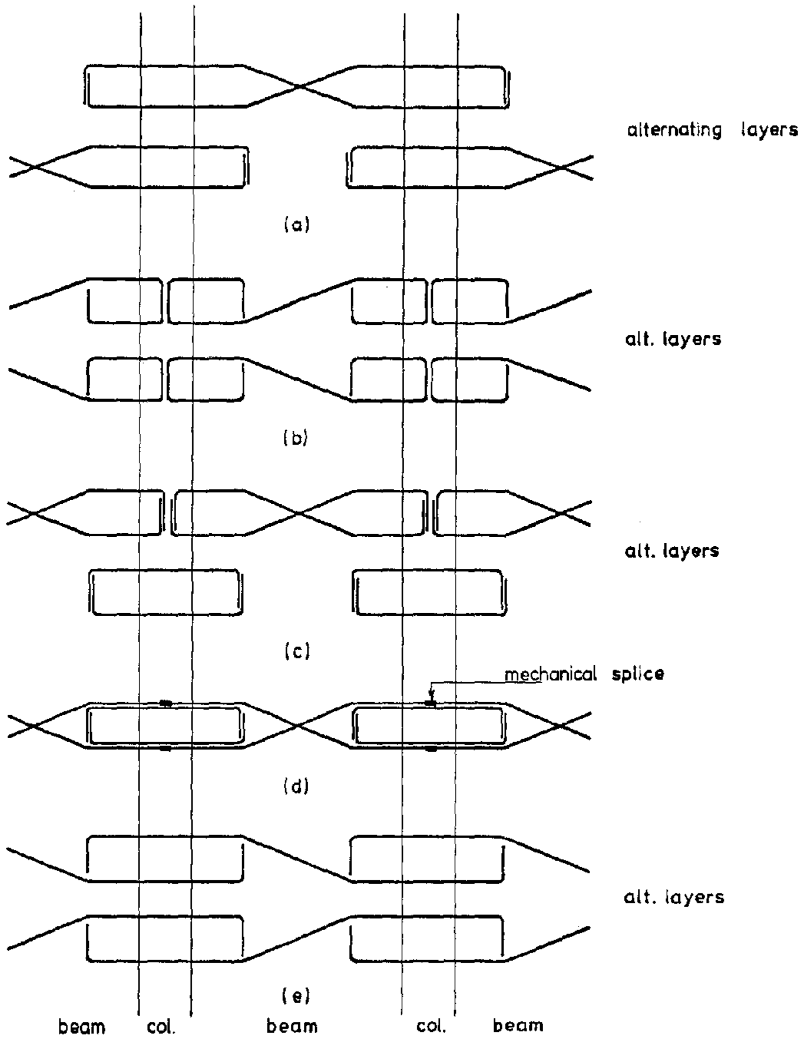
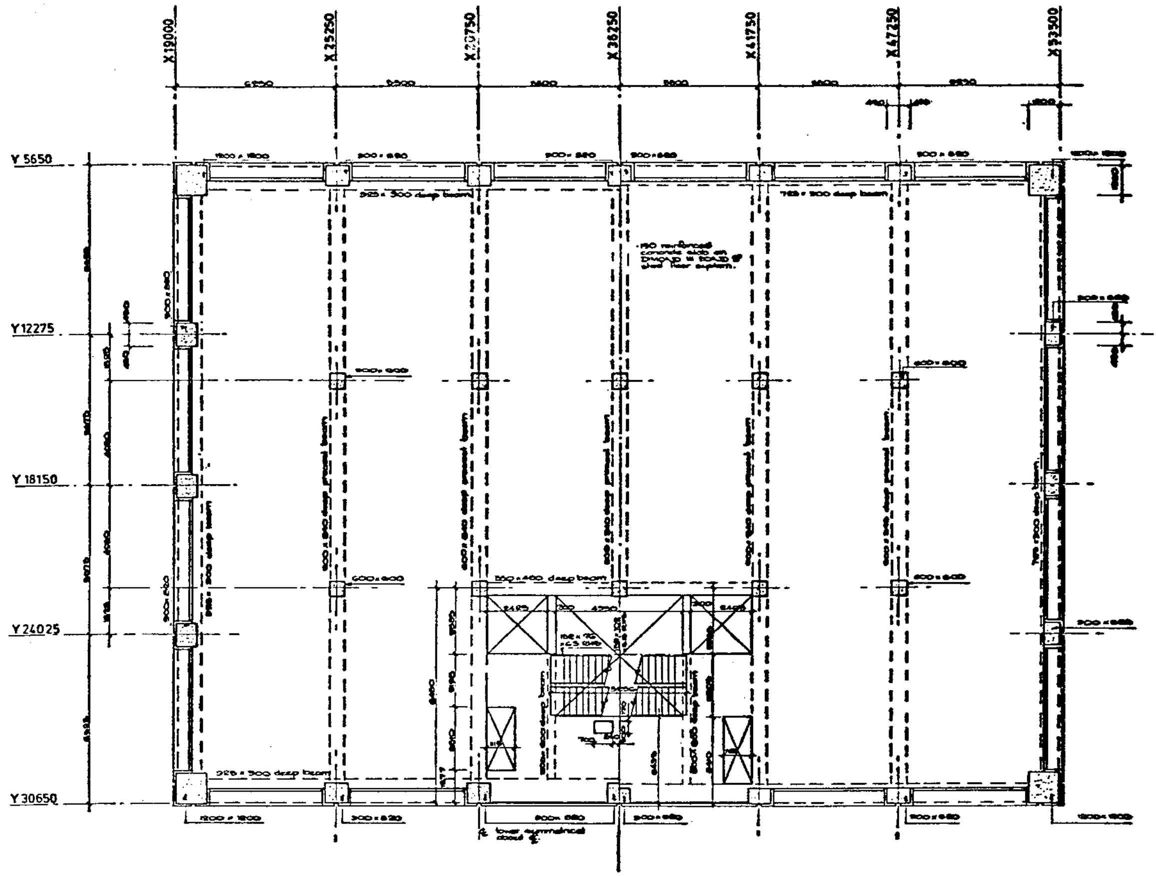
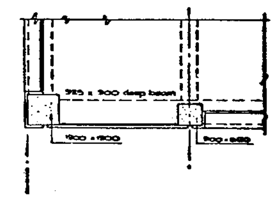


Fig 11 : Alternative reinforcing layouts



level 6 - 12 level 13 - 17



level 6, 7, 8.

OFFICE BUILDING 89-93 THE TERRACE FOR CHALLENGE CORPORATION

Fig 12

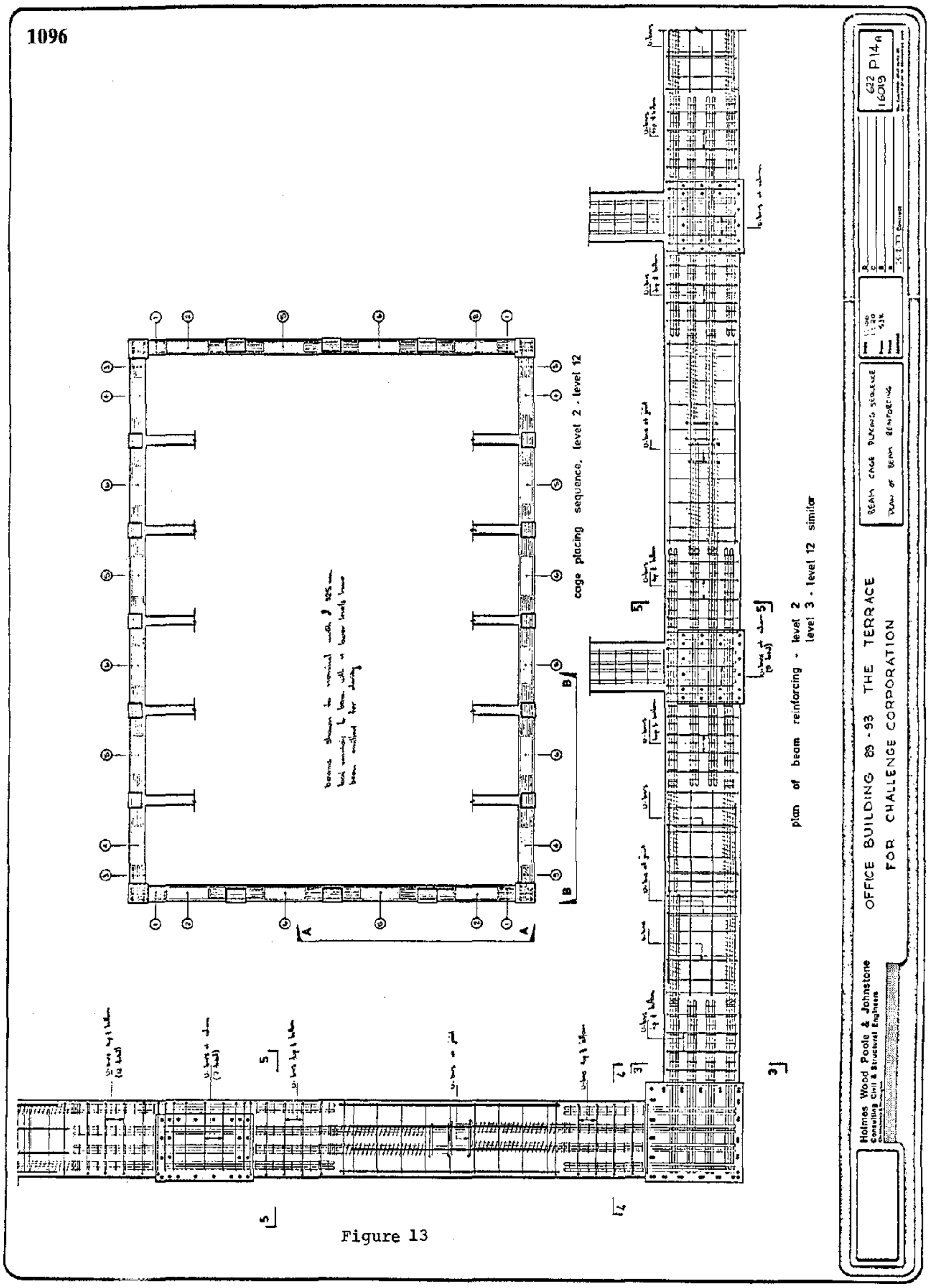
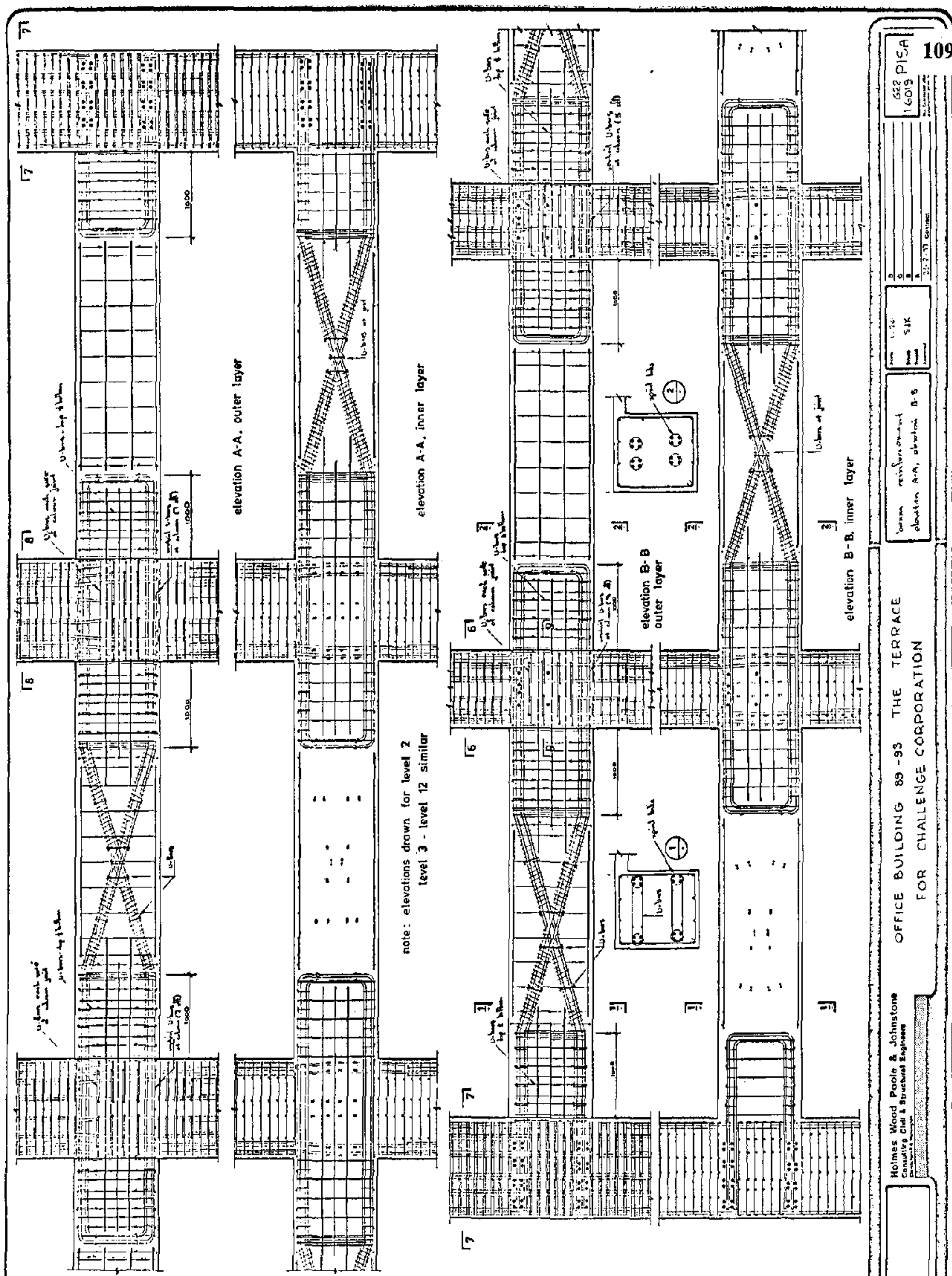

cage placing sequence, level 2 - level 12
plan of beam reinforcing - level 2
level 3 - level 12 similar
OFFICE BUILDING 89-93 THE TERRACE
FOR CHALLENGE CORPORATION
Holmes Wood Poole & Johnstone
Consulting Civil & Structural Engineers
BEAM CAGE PLACING SEQUENCE
PLAN OF BEAM REINFORCING
622
16019
P14A

Figure 13



elevation A-A, outer layer
elevation A-A, inner layer
note: elevations drawn for level 2
level 3 - level 12 similar
elevation B-B
outer layer
elevation B-B, inner layer
OFFICE BUILDING 89-93 THE TERRACE
FOR CHALLENGE CORPORATION
Holmes Wood Poole & Johnstone
G22 PISA
16019

Figure 14

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

THE PROBLEM OF DAMAGE TO NONSTRUCTURAL COMPONENTS AND EQUIPMENT

by

K. L. Merz
Earthquake Research Engineer
Ayres Associates
Los Angeles, California

INTRODUCTION

The nonstructural components of a building include facades, curtain walls, ceilings, partitions, elevators, lights, electrical power systems, plumbing, ventilation, exhaust and air conditioning systems, heating and refrigeration systems, fire protection systems, telephone and communication systems, storage racks, and even large pieces of owner-supplied furniture or portable equipment. In the past, the usual structural design procedure has been based on the philosophy that to design a building to avoid all damage during a major earthquake is not economically justifiable; the structural system of the building is intended to be deformed by strong ground motion, and damage to some of the nonstructural elements is expected. However, recent major earthquakes (Alaska 1964, San Fernando 1971, Managua 1972, and Guatemala 1976) have caused considerable damage to the nonstructural elements and electrical/mechanical equipment of buildings sustaining only moderate structural damage. The investigation [1-7] of the damage caused by these earthquakes has indicated the need for architects and designers of nonstructrual building systems to acquire background and additional skills in the analysis and design [8] of these systems for the building dynamic environment caused by the structural response to earthquake ground motion. An even greater emphasis is provided by the fact that approximately 70 percent of the construction cost of a building is for equipment and nonstructural elements. An increasing concern over the life-safety aspects of building design is also apparent. Thus, not only must the substantial monetary investment in nonstructural elements and equipment be protected, but also the systems concerned with insuring life-safety must be made seismic resistant. A building is not safe if, during an earthquake, light fixtures and ceilings fall, elevators do not operate, emergency generators do not come on, loose objects block exits, and broken glass falls into the street. A building is not properly designed if an owner sustains huge losses due to nonstructural damage. The lessons learned by detailed studies of damage sustained by earthquake-tested buildings must be carefully reviewed by both architects and engineers. One lesson from past earthquakes is clear: The amount of damage sustained by nonstructural building components could have been greatly reduced by relatively inexpensive corrective measures.

The participation of nonengineered filler walls and other nonstructural elements in the total structural response is an increasing concern of structural engineers. The distribution of nonstructural walls can force a torsional response in symmetric buildings, alter the system frequency response and damping characteristics, and create loading conditions on structural elements for which they were not designed. Comparisons of nonstructural damage noted in recent post-earthquake damage studies [3,7] between reinforced concrete ductile frame and shear wall building construction have been striking. The control of inter-story drift in the design of ductile frame structures is a critical problem, both from a structural standpoint and from an architectural detailing standpoint.

CURRENT NONSTRUCTURAL DESIGN PRACTICE AND CODE REQUIREMENTS

The development of plans and specifications for a modern building is a team effort. An architect acts as a coordinator and general manager of the project as it moves from concept to design and into the field and is finally erected. The primary outside consultants on the design team are the structural, mechanical, and electrical engineers. An additional outside consultant is usually retained to design the elevators. The structural engineer and the architect require the services of foundation and soils engineers, and continual liaison with material manufacturers and governmental agencies by all team members is necessary in the development of the design. This outside group of consultants often controls the design of 75% of the total construction cost of a building.

The first concept of the size and shape of a building are developed by the architect from his knowledge of the client's needs. In most instances, the fundamental decisions are rendered before the structural engineer is called to develop a structural frame to meet an architectural design and before mechanical and electrical engineers are called in to design their systems. Thus, the architect has the initial responsibility of advising the client of the necessity for considering the seismic design of nonstructural components within the proposed building. This must be done at the earliest possible time to insure that the costs for such considerations are included in the preliminary cost estimates.

Most building owners, and unfortunately their architects and engineers, consider building code minimum requirements as adequate protection against earthquake damage, and they will not increase their capital costs to improve occupant safety or reduce future repair costs. This firm belief in the infallibility of building codes is usually badly shaken after each earthquake. But memories are short and the magnitude of repair costs and other post-earthquake difficulties with buildings are not made public, so owners usually resist added costs for earthquake resistive features that are not spelled out in a code. Thus, recent legislative efforts have been concerned with upgrading codes, especially for "critical" facilities.

After the 1971 San Fernando earthquake, in which several modern hospital buildings and equipment were seriously damaged, the California Legislature enacted The Hospital Seismic Safety Act of 1972. The implementing regulations [9] which have been adopted are the first government code to link geology, seismology, structural engineering, and nonstructural building design. The regulations, which are the most complete concerning nonstructural building components to date, require that nonstructural components and equipment resist the application of an equivalent lateral static force which can be equal to the equipment weight (i.e., 1.0 G acceleration). The dynamic design of equipment is allowed as a "footnote" type option. The nonstructural requirements of Title 17 are summarized in Appendix Table 1. Considerable experience has been gained in the administration of the regulations in current California hospital construction. A comprehensive document [10] is under preparation which will give design guidelines for acceptable nonstructural detailing practice consistent with the intent of the regulations.

The concern over hospital earthquake resistance is not limited to California. The military services and the Veterans Administration have standard requirements [11, 12] for the seismic design of hospital facilities, including nonstructural elements. Recent changes to the Uniform Building Code [13] have also upgraded the lateral force coefficents for nonstructural components. These new UBC requirements are summarized in Appendix Table 2. The current efforts of the Applied Technology Council (ATC-3 Project) to review the state-of-the-art in earthquake engineering and develop comprehensive seismic design recommendations [14] including nonstructural components, should also be noted.

The existence of a good building code, however, does not in itself ensure that its provisions will be properly applied by the design team or installed by the contractors. Most of the problem stems from the traditional divisions of responsibilities between design professionals, and a construction industry that does not require careful detailing of nonstructural elements. It is during the preparation of the working drawings and specifications that the final decisions are made regarding the detailing -- or lack of detailing -- of the nonstructural components for seismic resistance. Often the mechanical and electrical drawings are schematic only, with the design and installation requirements contained within the written specifications. Because the documents are prepared for competitive bidding, alternative equipment and materials must be accepted if they are equal in quality and perfomance to those specified. Shop drawings prepared by the successful contractors or materials manufacturers must be submitted to the design team for approval before installation. These shop drawings contain the actual installation details and become the final guide to the execution of the design. This shift from the plans prepared by the design team to the shop drawings and then to the work of the installer at the building requires careful supervision if the intent of the design is to be executed properly. Many of the installation details of nonstructural elements are deliberately omitted from drawings, because of long standing trade practices that have left many of these decisions to product manufacturers and installers. To overcome these problems, all members of the design team must see that all nonstructural elements are detailed or carefully described in the specifications. The entire design team must then vigorously defind these details from contractor proposed alternates that do not meet the design intent, and then demand that they be properly executed in the field.

It should be noted that there is a considerable gap between the equipment qualification procedures used in normal commercial building design, including hospitals, and those utilized in critical military and utility facilities [15]. Building equipment design requirements are based upon application of an equivalent static force to insure proper anchorage and enclosure or support strength. The problem of functional performance is not addressed. Equipment items deemed critical, such as life safety system components (fire pumps, smoke ventilation, elevators, etc.) are simply designed for higher levels of equivalent static force in an attempt to obtain performance. This philosophy is valid for non-critical equipment, given the damage patterns noted during past earthquakes, which indicates that a great majority of damage can be prevented simply by expedient restraint of building service system equipment. But critical equipment, whose function is mandatory, cannot be qualified by application of anchorage requirements. Some code work (NFPA, ASME) in this area is currently under development.

BUILDING DYNAMIC ENVIRONMENT

Usually, the structural engineer is the only member of the design team to analyze the effect of dynamic building forces induced by earthquakes. All members of the design team, however, must inform themselves of the nature of earthquake-induced forces in buildings and of the manner in which the stress paths occur between the structural and nonstructural elements of a building. The structural frame may absorb the earthquake forces without significant damage, but the movement of the building induces significant secondary damage to nonstructural elements. In addition, the net resistance of the nonstructural elements with floor-to-floor connections contributes to the overall stiffness of the structural system, thus influencing the dynamic response of the building. The resulting damage to nonstructural components shows a lack of knowledge among nonstructural designers of building response characteristics due to an earthquake. Since the majority of building service equipment is located both on the ground floor and roof, the nonstructural designer must understand the characteristics and response effects of both ground motion and floor motion.

Building Amplification

Since 1965, the City of Los Angeles has required placement of three strong-motion accelergraphs in all new structures greater than six stories in height. Subsequently, adjacent municipalities have adopted similar requirements. These instruments are placed in the basement (base level), mid-portion (intermediate level) and near the top (upper level) of buildings. The 1971 San Fernando earthquake may be viewed as a full-scale experimental test of a wide variety of building types to strong ground motion. Forty-nine buildings, ranging in height from 7 stories to 43 stories, recorded motion in three component directions at the base level and at least one higher level. These buildings were located at distances from the epicenter ranging from 20 km to 83 km and were exposed to peak horizontal base (ground) accelerations ranging from 0.030G to 0.255G and peak vertical base accelerations ranging from 0.019G to 0.171G (1G = 980.6 cm/sec/sec). The resulting peak horizontal upper level floor accelerations ranged from .08G to .50G while peak vertical upper level floor accelerations ranged from .04G to .36G. The uniformly processed, digitized, corrected, and analyzed data set for these recorded accelerograms has been published [18-20].

A structural system acts as a mechanical narrow-band filter for earthquake ground motion, amplifying and filtering at approximately the modal frequencies of the building. The resulting floor motion becomes the input base motion for anchored (and unanchored) equipment. The severity of floor motion is usually measured by the peak or maximum floor acceleration. While the use of peak acceleration as measure of damage is often unsatisfactory, the maximum acceleration parameter is physically understood as a measure of the inertial force that must be resisted by a rigid, anchored object. Recent studies [16, 17] have characterized the amplification of building motion by the ratio of peak output (floor) acceleration to peak input (base) acceleration. This comparison yielded average values for a large sample of building types, heights, and construction of recent design. The understanding of the response behavior of a building subjected to ground motion is complicated by the effects of three dimensional motion, coupled torsional-lateral response, and non-linear behavior. A great many parameters influence the response of a particular structure including the frequency content of the ground motion at the building site, soil-structure interaction, discontinuities in structural framing, the detailing of the structural connections, and even the stiffness of the nonstructural components. In addition, the recorded motion represents the response of a singular point within the structure, thus a wide range of values should be expected when using the extreme or maximum peak values as a measure of response severity.

An example of a recorded reinforced concrete frame building response is shown in Figure 1 for a duration of 35 seconds. The nonlinear filtering behavior of the building is easily noted by the comparison of the base level accelerograph record with the upper level. The frequency of response during the first 10.7 sec. has considerable higher frequency content than the latter 24.3 sec. of record. A more detailed evaluation of the recorded floor motion reveals that the average period of response during the first portion of the record is approximately 0.6 sec. and then lengthens to 1.5 sec. for the remainder of the record. For this presentation, the record was filtered by eye to remove the higher frequencies present in the recorded accelerogram (the larger scale plot contained in [18] was used as a guide; of course the record could also be numerically filtered). This particular building was the subject of a detailed post-earthquake study [22,23] which noted that the interior partitions and exterior cement plaster walls reduced the design response period by 30% and accounted for approximately 60% of the initial lateral force resistance of the structure prior to cracking. It should be noted that, in this case, the peak floor acceleration occurs prior to the peak ground acceleration which implies that the ratio of the two values is at best only a qualitative measure of amplification.

The horizontal amplification characteristics [16] of the buildings during the 1971 San Fernando earthquake are given in Figure 2 which compares the computed FAF (Floor Amplification Factor) over the range of story heights reported [21] for the 49 buildings. The steel frame and reinforced concrete buildings have been identified in this presentation. As can be noted from Figure 2, the amplification behavior of the buildings is relatively independent of story height with an average horizontal FAF value of 2.3. The steel structures, when compared to the reinforced concrete buildings within the building sample, exhibit a characteristic lower amplification.

In Figure 3, the largest horizontal FAF is compared to the largest peak ground acceleration, disregarding component direction. This comparison indicates a trend of decreasing building amplification with increasing peak ground acceleration. Assuming that peak recorded base acceleration is a measure of the overall strength or intensity of ground motion, we observe that the amplification decline may be attributed to the increased equivalent damping level caused by accumulated structural damage. Curves have been fitted [17] to the initially reported [21] San Fernando data which distinguish geologic conditions and reinforced concrete/structural steel construction.

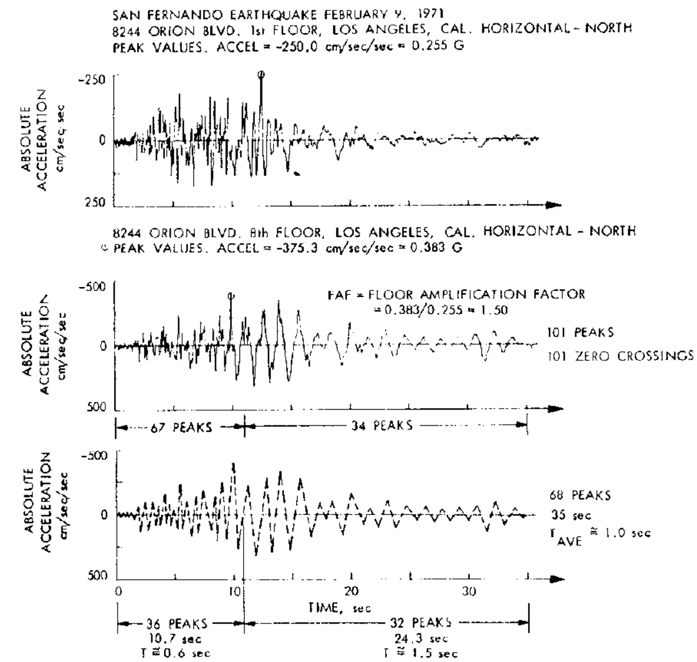


FIGURE 1 . EVALUATION OF RECORDED BUILDING MOTION.

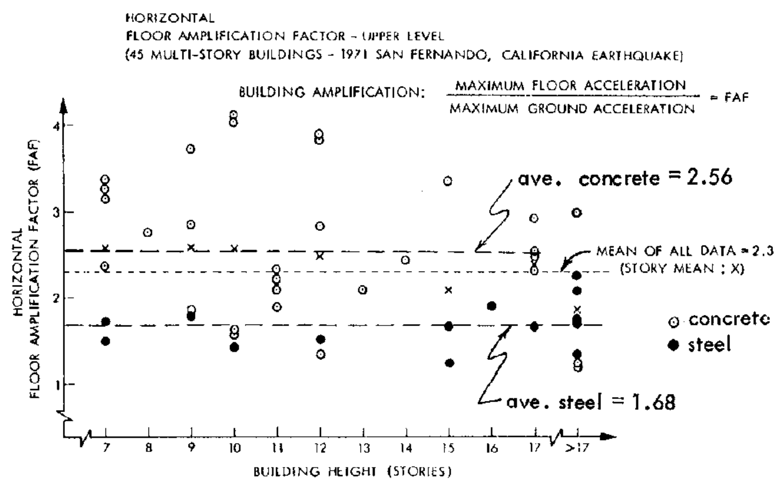


FIGURE 2. HORIZONTAL UPPER LEVEL BUILDING AMPLIFICATION.

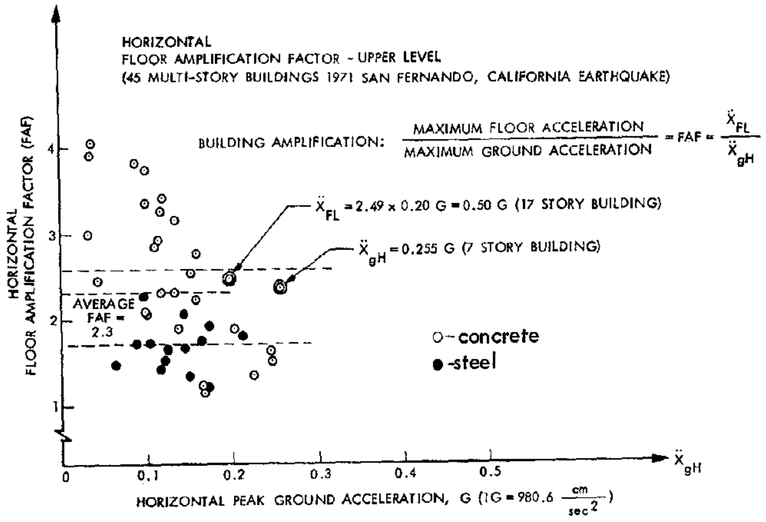


FIGURE 3. HORIZONTAL BUILDING AMPLIFICATION COMPARED TO PEAK GROUND MOTION.

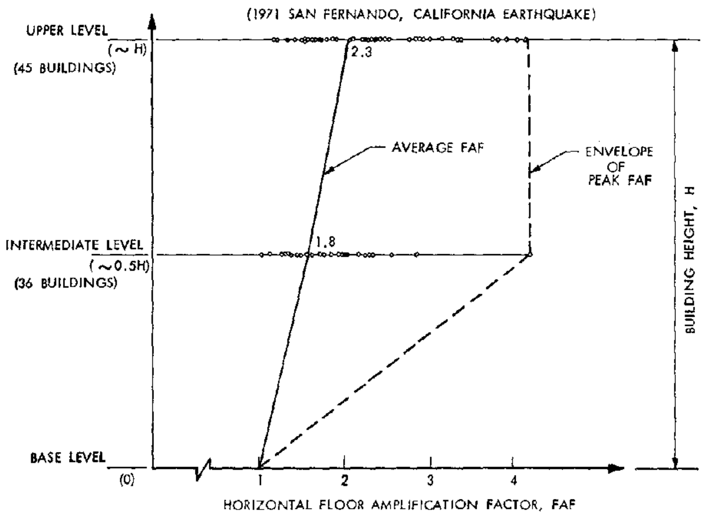


FIGURE 4. DISTRIBUTION OF HORIZONTAL BUILDING AMPLIFICATION OVER BUILDING HEIGHT.

However, any use of the data to indicate a definite trend should be viewed with caution due to the few data points greater than 0.20G horizontal and 0.10G vertical.

Figure 4 indicates the distribution of horizontal amplification over building height. Vertical amplification of ground motion is another important consideration for nonstructural components and equipment. Figures 5 - 7 present the vertical amplification characteristics of the group of buildings with recorded motion. Again, the amplification behavior of the buildings is relatively independent of story height with an average vertical FAF value of 2.6. Both horizontal and vertical components have the same average amplification, FAF = 1.8, for the intermediate levels. The distinction between steel and reinforced concrete construction is not apparent for vertical amplification behavior.

Spring Mounted Equipment Response

Given that an equipment item is properly anchored (i.e., not susceptible to overturning or sliding), the equipment with attached components will respond to the floor motion as an independent structural or mechanical system. Rigid equipment, such as motors, pumps, etc., which are directly mounted to the floor will not experience additional amplification. However, building service equipment is often placed on spring mounts, or vibration isolators, to reduce the transmission of equipment vibration to the structure (and tenants). The failure of vibration mounts and the resulting overstress of connecting pipe and conduit is a frequent observation during post-earthquake damage surveys. The specification of spring mounts with 1.0 in. (2.54 cm) static deflection and equal vertical and lateral stiffness is a common practice for building service equipment, resulting in equal vertical and horizontal natural frequencies of 3.1 cps (Hz) (ignoring for the present discussion the effect of rotatory inertia due to a base mount configuration). The response of such flexible equipment within a building is best evaluated using floor response spectra. The analysis [20] of the recorded building motions provides computed floor response spectra for each component of building motion recorded. Using the ratio of response spectrum ordinate to maximum ground acceleration, the computed [16] RSAF (Response Spectrum Amplification Factor) for the lateral response of spring isolated equipment with 1.0 in. static deflection mounts and equal vertical and horizontal stiffness are compared in Figure 8 for equipment located at the upper levels of a building. The largest computed [20] response spectrum ordinate (acceleration) within a frequency band, f = 3.1 $\pm$0.4 cps, for a system with 5% critical damping was used in computing the RSAF of each component of motion. The range of upper level horizontal response spectrum ordinates was 1.85 G to 0.123 G. Figure 10 compares the RSAF for equipment located at the base, intermediate, and upper levels of a building. The average amplification for a spring mounted equipment item (f = 3.1 cps, 5% damping) was 3.3 at the base level, 5.0 at the intermediate level, and 6.2 at the upper level of the buildings with recorded motion. Consistent with the lower amplification of the steel structures noted in Figure 3, the steel structures appear to input less energy to the 3.1 cps spring mounted system.

Another study [17] of earthquake-induced in-building motion criteria has presented average design spectra based on simulation results. Table 3 has been prepared for comparative purposes with Figure 9. The suggested design criteria [17] appears to be conservative when compared to average amplification of the buildings with recorded motion.

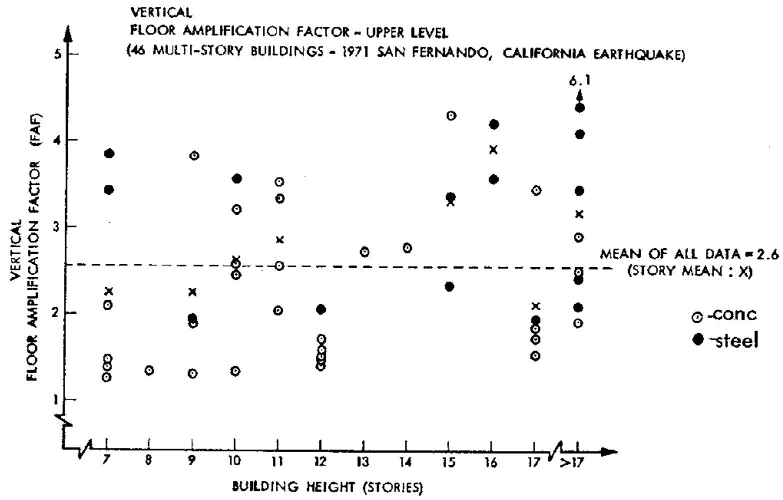


FIGURE 5. VERTICAL UPPER LEVEL BUILDING AMPLIFICATION.

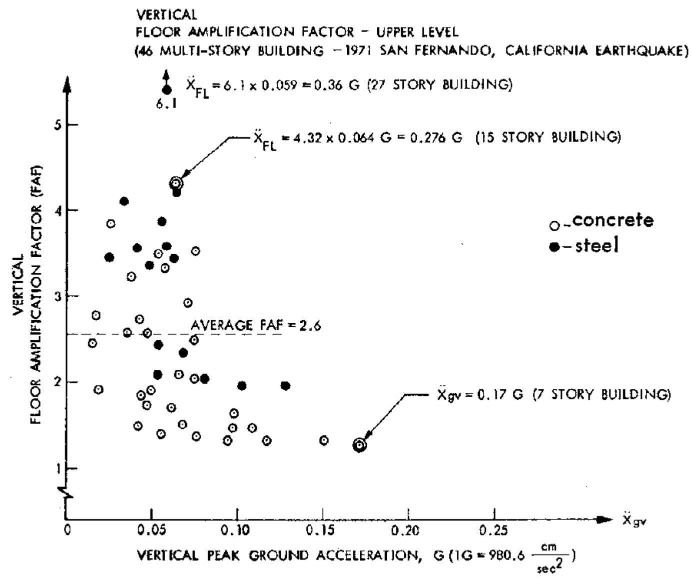


FIGURE 6. VERTICAL BUILDING AMPLIFICATION COMPARED TO PEAK GROUND MOTION.

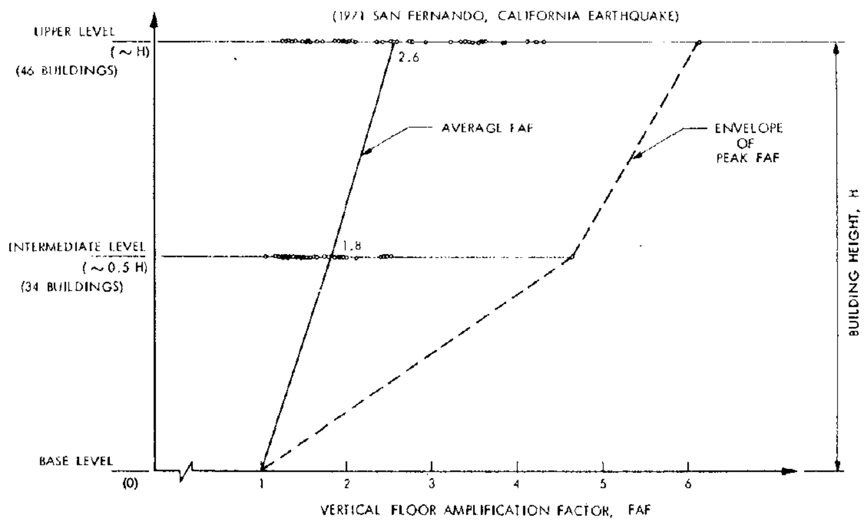


FIGURE 7. DISTRIBUTION OF VERTICAL BUILDING AMPLIFICATION OVER BUILDING HEIGHT.

TABLE 3
RESPONSE SPECTRA AMPLIFICATION (Ref. 17)

f = 3 cps, 5% Damping, $\ddot{X}_{Floor} = 0.7G$, $\ddot{X}_{Ground} = 0.3G$

No. Stories	Roof $\ddot{X}$, G	$\ddot{X}/\ddot{X}_F$	$\ddot{X}/\ddot{X}_G$ = RSAF
2	6.0	8.6	20
5	4.5	6.3	15
10	3.0	4.3	10
20	2.5	3.6	8.3

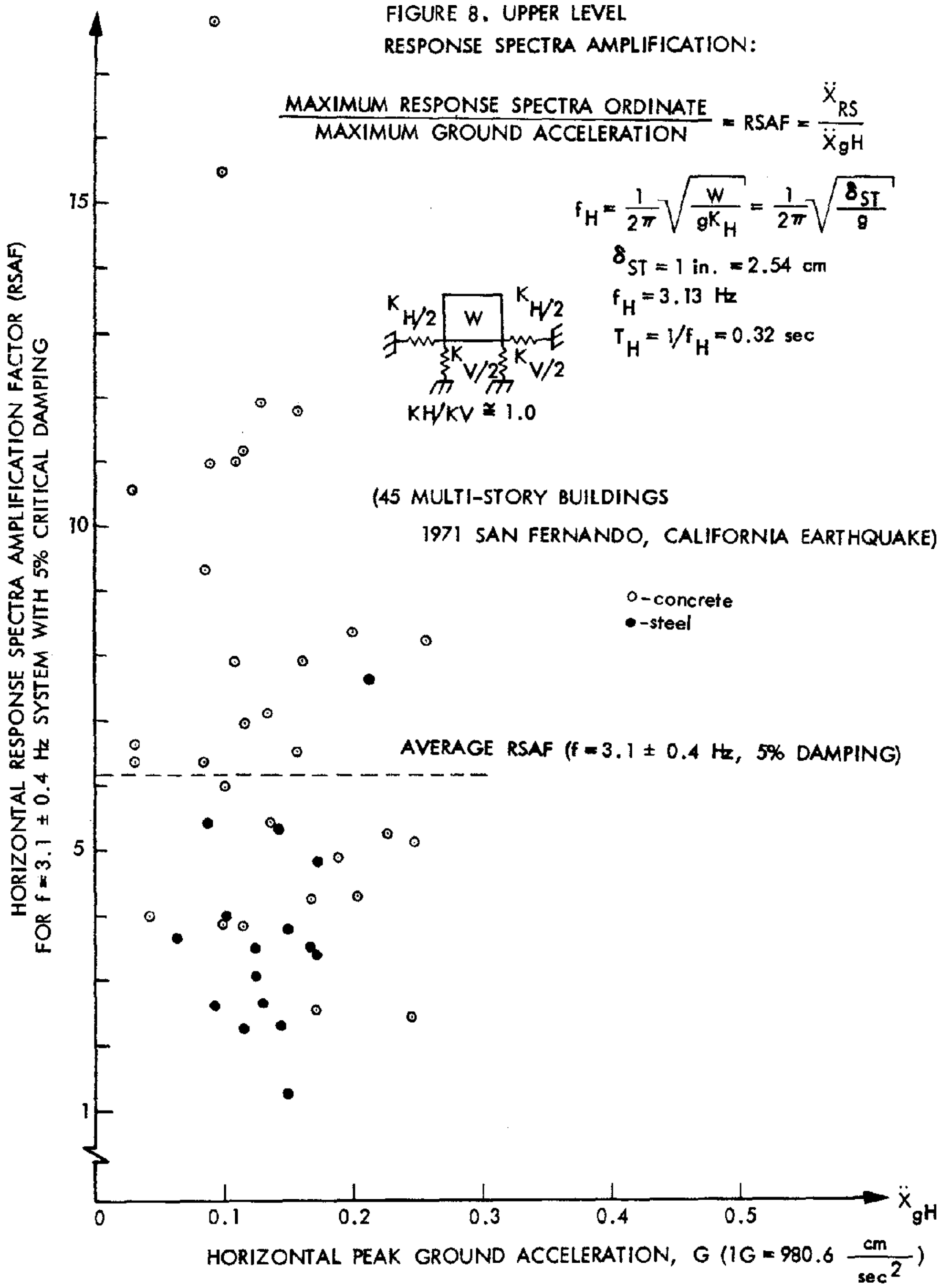


FIGURE 8. UPPER LEVEL
RESPONSE SPECTRA AMPLIFICATION:
MAXIMUM RESPONSE SPECTRA ORDINATE / MAXIMUM GROUND ACCELERATION = RSAF = $\ddot{X}_{RS} / \ddot{X}_{gH}$
$f_H = \frac{1}{2\pi}\sqrt{\frac{W}{gK_H}} = \frac{1}{2\pi}\sqrt{\frac{\delta_{ST}}{g}}$
δ_{ST} = 1 in. = 2.54 cm
f_H = 3.13 Hz
$T_H = 1/f_H$ = 0.32 sec
$K_{H/2}$
W
$K_{V/2}$
KH/KV ≅ 1.0
(45 MULTI-STORY BUILDINGS
1971 SAN FERNANDO, CALIFORNIA EARTHQUAKE)
o-concrete
•-steel
AVERAGE RSAF (f = 3.1 ± 0.4 Hz, 5% DAMPING)
HORIZONTAL RESPONSE SPECTRA AMPLIFICATION FACTOR (RSAF)
FOR f = 3.1 ± 0.4 Hz SYSTEM WITH 5% CRITICAL DAMPING
15
10
5
1
0
0.1
0.2
0.3
0.4
0.5
$\ddot{X}_{gH}$
HORIZONTAL PEAK GROUND ACCELERATION, G (1G = 980.6 $\frac{cm}{sec^2}$)

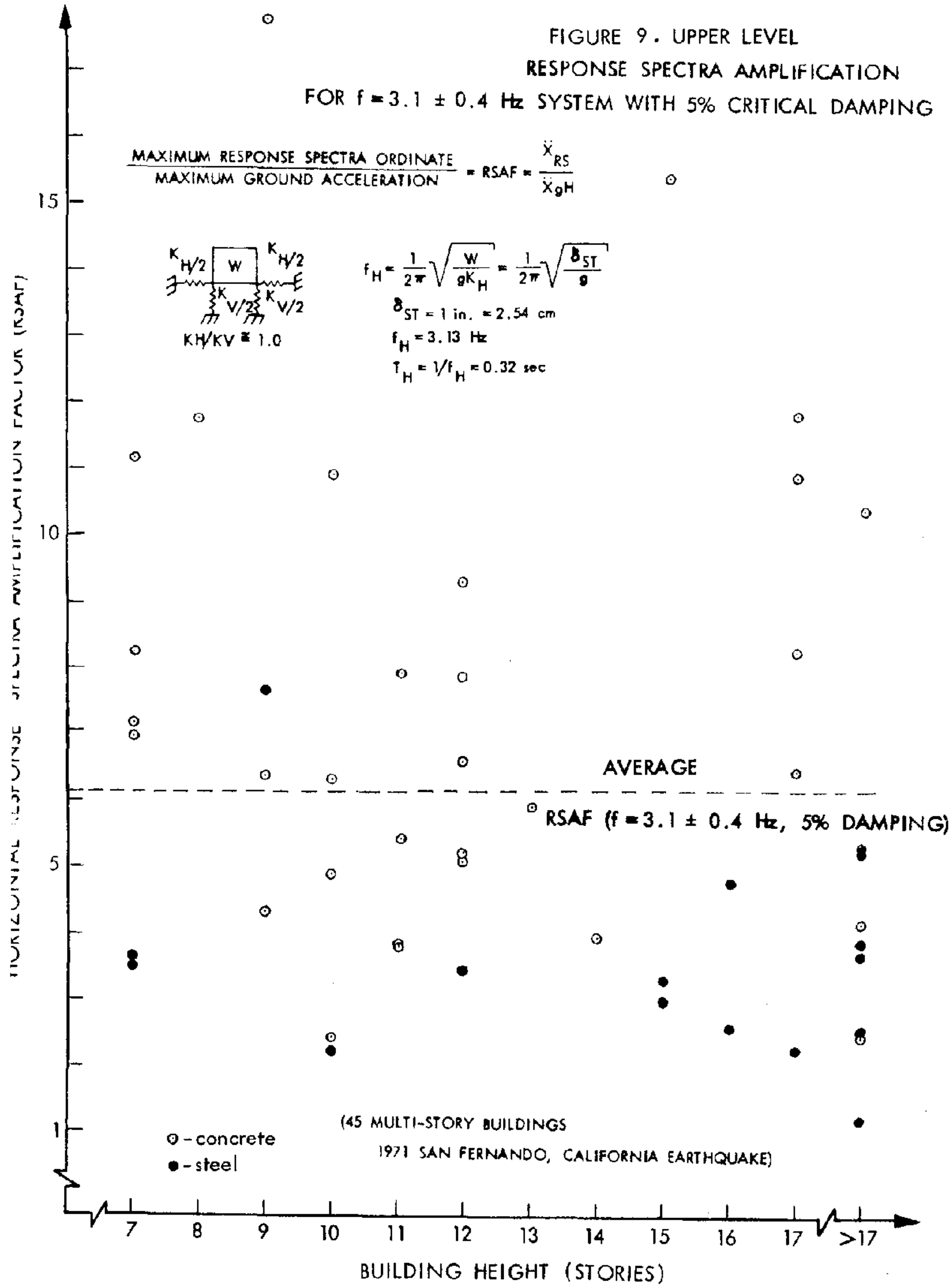


FIGURE 9. UPPER LEVEL
RESPONSE SPECTRA AMPLIFICATION
FOR f = 3.1 ± 0.4 Hz SYSTEM WITH 5% CRITICAL DAMPING
MAXIMUM RESPONSE SPECTRA ORDINATE / MAXIMUM GROUND ACCELERATION = RSAF = $\ddot{X}_{RS} / \ddot{X}_{gH}$
$K_{H/2}$ W $K_{H/2}$ $K_{V/2}$ $K_{V/2}$
KH/KV ≅ 1.0
$f_H = \frac{1}{2\pi}\sqrt{\frac{W}{gK_H}} = \frac{1}{2\pi}\sqrt{\frac{\delta_{ST}}{g}}$
δ_{ST} = 1 in. = 2.54 cm
f_H = 3.13 Hz
$T_H = 1/f_H$ = 0.32 sec
AVERAGE
RSAF (f = 3.1 ± 0.4 Hz, 5% DAMPING)
HORIZONTAL RESPONSE SPECTRA AMPLIFICATION FACTOR (RSAF)
15
10
5
1
o - concrete
• - steel
(45 MULTI-STORY BUILDINGS
1971 SAN FERNANDO, CALIFORNIA EARTHQUAKE)
7
8
9
10
11
12
13
14
15
16
17
>17
BUILDING HEIGHT (STORIES)

Research to define the measured response amplification at other building equipment frequencies of interest is being conducted by the author.

Building Drift

The response of structures (in terms of structural element stress) at earthquake levels which exceed the design capacity are mitigated by nonlinear behavior but at the expense of large yielding displacements or drifts. Often, drift is the cause of the majority of damage sustained by buildings during an earthquake. The review of actual recorded building motion provides a realistic estimation of building drifts which are the result of the ductile behavior of buildings during moderate earthquakes. Figure 11 gives the drift determined from recorded data [19] obtained from the example instrumented multi-story concrete frame building. The relative displacement response of the building upper level was obtained by graphically subtracting the plotted [19] absolute displacement of the base level from the absolute displacement of the upper level (assuming that both instruments were triggered at the same time). The long period (11 sec.) fluctuation is due to a processing error caused by misalignment of sectional enlargements of the record during the digitization process [24] (see also [19], Part G). Since the relative displacement response is dominated by the fundamental mode, the response of the floors are in phase thus the peak story drift distribution over the building may be estimated by plotting the upper and intermediate peak relative displacements as shown in Figure 11. The peak story drift for this example building was of the order of one inch or 0.01 foot drift per foot of building height which is in accordance with the observed [23] nonstructural damage resulting from the 1971 San Fernando earthquake. Estimates [25, 29] of approximate damage levels of drift for buildings during the San Fernanco earthquake have been made for comparison of frame and shear wall construction performance. Studies [30, 31] have attemped to correlate the damage statistics gathered after the San Fernando earthquake, but definitive design criteria have not been developed which consider damage limitation due to drift.

ARCHITECTURAL SYSTEMS

A primary function of architectural systems is to enclose and subdivide the interior space of a building. A wide variety of enclosure and finish systems are utilized within the building construction industry. Nonstructural architectural components which have floor-to-floor connection such as slab-to-slab partitions (fire-walls), curtain walls, and stairs must accommodate story drifts or be damaged by the imposed forces. Exterior and interior glazing, doors, and hung ceilings, while normally not directly connected between floors must accommodate the deformation imposed by the exterior panels or interior partitions. The cost of repairing plaster, drywall, glass, and other drift damage is often the most costly post-earthquake repair item due to the labor manhours required. Often, the mode of interaction between the enclosure or finish system and the primary structure is not apparent. Research [25,29] is continuing in the identification and recommended detailing required to accommodate such interaction. Design guidelines and acceptable architectural details [10] within hospitals are currently under preparation. The design guidelines utilized for GSA buildings [32] should also be noted.

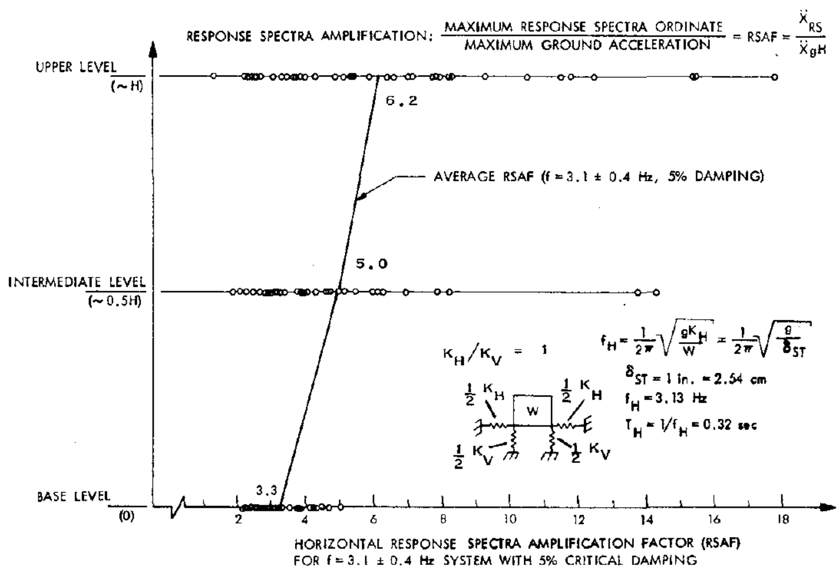


FIGURE 10. SPRING MOUNTED EQUIPMENT RESPONSE AMPLIFICATION.

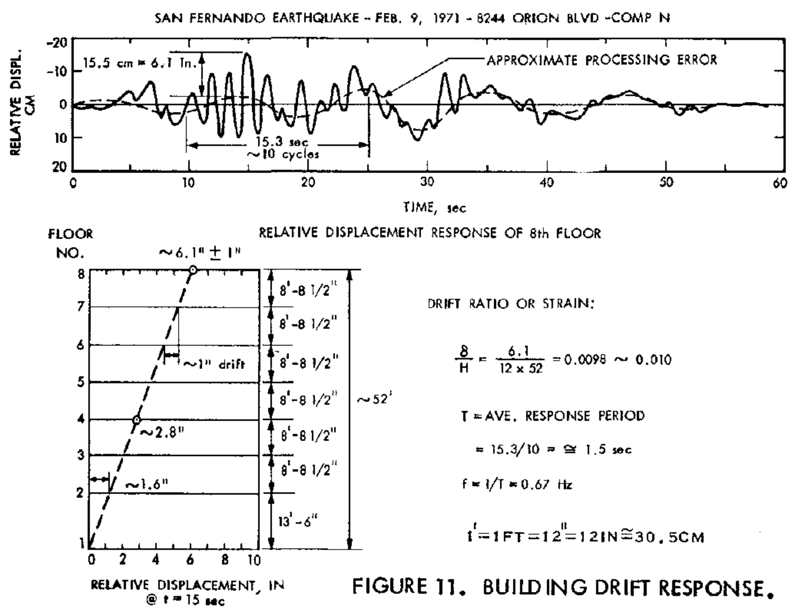


FIGURE 11. BUILDING DRIFT RESPONSE.

Suspended ceilings which are hung with wire, and yet attached to a partition at the room periphery, will accommodate drift. However, the presence of knee braces (extended metal studs) in long walls and the occurence of firewalls will retard this flexibility in some areas. Some peripheral damage will occur either by buckling or tearing away of the suspended ceiling. In order to prevent ceiling collapse, additional wire hangers should be provided, especially at the periphery of rooms and corridors. Recommendations which suggest that diagonal crossbracing wires be used to insure that ceilings remain rigid and move with the above slab require that the peripheral details of such installations accommodate the drift or simply be a covered gap. The current use of "structural" ceilings for California hospital construction should be noted.

The behavior of stairs within an enclosed stairwell which is distorted by building drift is critical. The detailing of door frames which provide building egress, or access through firewall partitions and structural walls must be carefully examined to insure that doors are not jammed shut.

The proper detailing of exterior enclosure systems, glazing, and connections for architectural precast panels, stone veneer, and sheet metal panels are areas which require investigation. Considerable research and design effort in these areas have been expended for the design of individual large buildings, but little formal documentation can be found in the published literature.

The critical design parameter for architectural systems is the inter-story drift expected during a moderate earthquake; not the drift determined from application of design lateral forces required by code. Racking tests [26,27] of various types of interior partitions have indicated that incipient damage, in terms inter-story drift, begins at about 0.0025 times the story height. Inter-story drifts that would require repair to the partitions would be in the range of 0.005 to 0.010 times the story height. These values are in general accordance with the observed damage [29] resulting from the San Fernando earthquake. In a building which has not been specifically designed to limit excessive seismic drift damage, little can be done to prevent such damage. An understanding of the structural behavior of a building during moderate earthquakes is necessary. The question of how much drift allowance to provide in the detailing of architectural components must be decided upon by the architect in consultation with his structural engineer. Cost trade-off studies are necessary to determine whether construction dollars should be placed in increased structural resistance (stiffness) or architectural details which allow for drift.

The design of frame structures with masonry infill walls or other types of nonstructural infill panels which act as a shear diaphram is an area of concern for structural engineers. Such walls affect the strength of reinforced concrete frames and must be included in the structural analysis and design. Recent earthquake damage has identified [3, 7] the problems of masonry infill wall construction. Procedures and design recommendations for such construction are available in the literature [28] .

ELEVATOR SYSTEMS

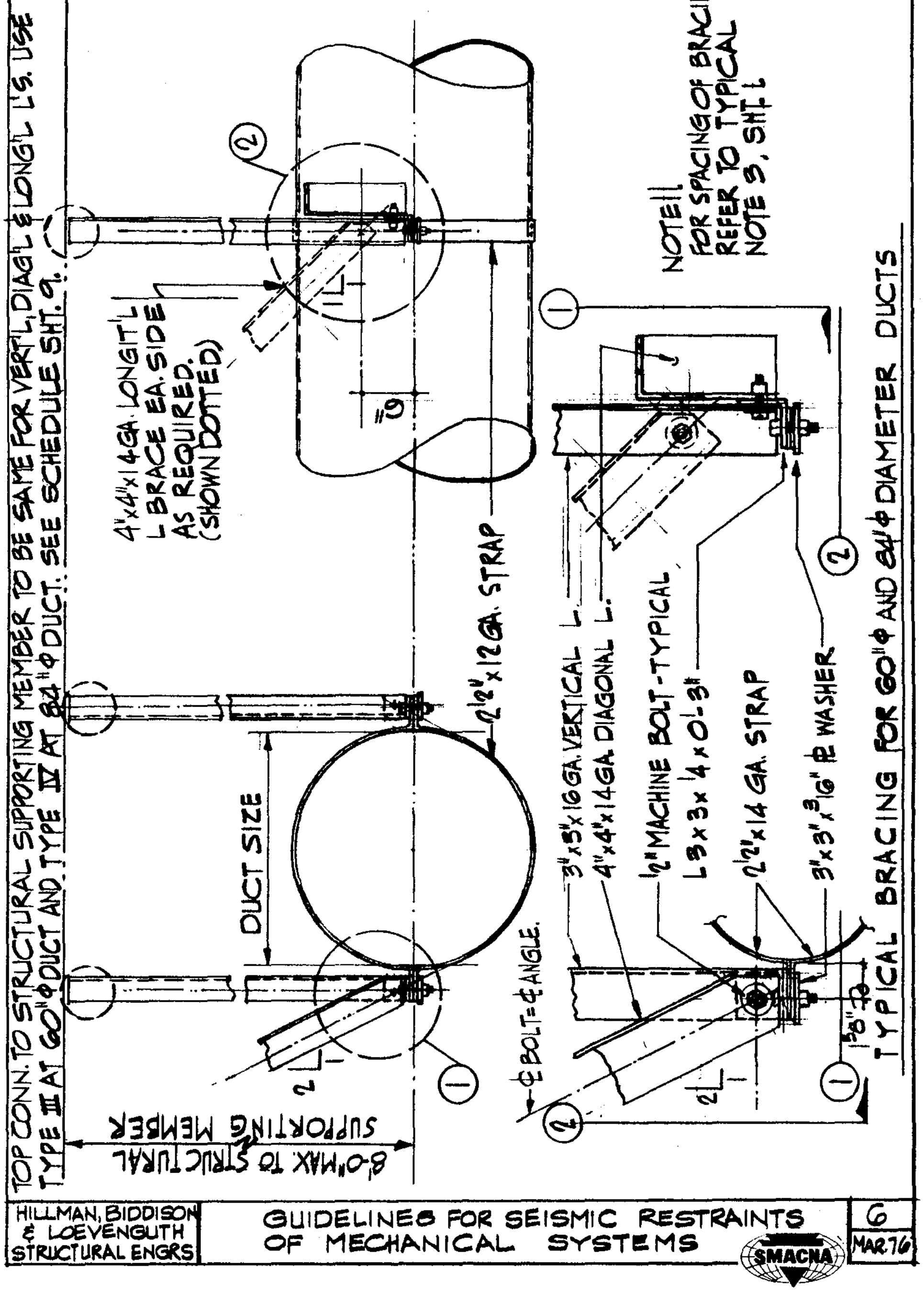
The vulnerability of building elevator systems to earthquake damage has been well documented in earthquake damage studies and reports [1,2,3]. The damage statistics (over 674 inoperable) for elevator damage due to the 1971 San Fernando earthquake [2,8] provide an indication of the expected elevator damage that will occur when an earthquake of moderate magnitude occurs near a major metropolitan area. The occurance of a large magnitude earthquake near an urban area would damage and impair an even greater number of building elevator systems due to the larger area experiencing significant ground motion. Regulatory code changes have been proposed and adopted by a few government plan check and review agencies to mitigate some of the past earthquake damage modes for new elevator construction. The question of retrofiting existing elevator systems has been discussed and a state-wide code recently adopted. These codes require that equipment be anchored and that rails and support framing be designed to resist specific lateral forces. In addition, these codes include provisions for automatic controls which shutdown the elevators following an earthquake, after allowing passengers to exit at the nearest floor, and prevent use until inspection and repair occur. Thus, the purpose of these code requirements is to minimize physical elevator damage, and provide for shutdown of the elevators in the case of damage to prevent entrapment and further elevator damage.

However, the failure of elevators to operate after an earthquake has a more serious aspect than the loss of a means of egress for the occupants of buildings. Current Life-Safety Codes for high-rise buildings (greater than 75 feet in height) require that elevators, in the case of fire, operate under the control of the Fire Department. It is not practical to get people out of a large, tall building in emergencies, and current practice is to design places of refuge within the building where the occupants will be safe from fire. But the elevators must function so that fire rescue teams can have immediate access to the floors involved and must continue to function, even when the occupants are protected by firewalls and other emergency devices, to allow the necessary fire fighting and smoke removal equipment to be rapidly brought up to the floors as required. Thus, given the increased probability of fire following an earthquake, the elevator systems of a building are the "weak-link" of the Life Safety System of a modern high-rise building located in an earthquake prone area. Current elevator code provisions do not consider the necessity for functional requirements following an earthquake. A comprehensive review [34] of the current seismic design considerations for elevator systems is currently under preparation. The primary problem in elevator design, from a structural standpoint, is providing sufficient framing and anchor points within the hoistway to allow restraint (and adequate connections) for the car and counterweight rails.

MECHANICAL/ELECTRICAL SYSTEMS

The mechanical/electrical systems of a building are an extremely complex network of equipment and distribution of required services. The level of detailing for these systems contained within the construction documents has, in the past, been minimal. The construction drawings are schematic with great emphasis placed upon the written specifications. Thus, the requirement of seismic details on mechanical and electrical drawings for California hospital construction caused some initial confusion among designers. The development of guidelines and acceptable common details [33] greatly eased this problem (see Figure 12). A more comprehensive set of guidelines [34] (and commentary)

FIGURE 12. EXAMPLE GUIDELINE DETAIL(REF.33)



is currently under development. It is anticipated that these guidelines will greatly simplify the seismic design of mechanical/electrical systems within all buildings. The mechanical/electrical service systems of a building may be logically identified as:

Mechanical Systems
- HVAC
- Plumbing

Electrical Systems
- Power
- Lighting
- Communication and Signal

Life-Safety Systems

These systems are basically equipment systems. Equipment components may be classified as either rigid or flexible. A suggested definition of rigid equipment is a system having a natural frequency greater than 12 hz, which is about the upper limit of amplification for a floor mounted simple damped oscilator. Anchored rigid equipment transfers the inertial (acceleration) forces directly to the anchor points. As discussed previously, a study of the recorded building motion obtained during the 1971 San Fernando earthquake from a sample of 49 instrumented high-rise buildings indicates that amplification factors which range from 2 to 4 should be expected for peak horizontal and vertical floor accelerations in the upper levels of a multi-story building. For base mounted equipment, the anchor points must resist the combined effects of both base shear and overturning forces. In determining the overturning moment, the effect of vertical accelerations must be considered. While an insert anchor can be installed which will resist such forces, the connection of an anchor bracket to a minimum gage sheet metal enclosure can be difficult and require localized stiffening. This problem can be avoided by restraining the equipment at the top by a diagonal brace or wall attachment. However, a diagonal brace requires additional clearance adjacent to the equipment and if wall or partition attachment is considered, a nonstructural partition will not be capable of sustaining a large attachment force.

A vast majority of mechanical equipment within a building is supported on vibration isolation mounts to eliminate noise transmission through the structure. All major manufacturers of vibration mountings offer an "earthquake mount" or "earthquake snubber" restraints. Numerous articles and design details on this subject are available [8,33] ; most have been published in trade magazines.

The support of tanks must be carefully considered, particularly vertical tanks on legs.

Mechanical service systems require extensive piping systems. It has been generally observed that piping systems within a building sustain little damage despite significant structural and nonstructural damage suffered by the building due to an earthquake. Earthquake damage to piping systems, when damage occurs, is caused by excessive pipe movement and differential deflection between piping and connected equipment. Few pipes are actually broken or sheared; most failures occur at fittings. Often failures occur due to excessive swaying of long pipe runs flexing smaller intersecting branch lines or short vertical risers which are clamped to the structure. Ordinary piping systems are suspended from floor slabs with vertical hangers which, in effect, forms a pendulous system. The frequency of this effective system is quite low which essentially isolates the piping system from lateral inertial forces. This flexibility, which is due to pendular behavior, accounts for both the few instances of failure and the failures due to excessive displacements. Fire sprinkler systems are the only piping which in the past has been designed to resist earthquake loading. Due to the few instances of damage to fire-sprinkler piping caused by earthquake, it is often suggested that all major piping within a building be braced in the same manner as sprinkler piping. Another criteria utilized is to place braces at intervals such that the piping system moves with the slab from which it is suspended. This practice, required by code in some instances, is highly controversial since problems with noise transmission and thermal expansion arise. A more reasonable criteria would be to provide bracing only at points which would prevent the type of piping failures which have been noted in earthquake damage surveys. This would require restraining pipe runs in order to prevent overstress of branch lines or where piping changes direction and passes through a fire-wall. Attention should also be given to the manner in which the pipe riser weight is supported vertically. The above comments on piping apply also to ductwork and conduit. Ducts must be prevented from excessive swaying which can damage ceiling support systems. The crossing of building seismic joints by piping, ductwork, and conduit should be avoided.

Lighting fixtures must be properly secured to the structure or architectural components. Recessed light fixtures which are supported by exposed T-bar ceiling systems are potential personnel hazards. Each fixture must have at least two independent hanger wires per fixture at diagonal corners which are anchored to the floor slab above.

Electrical equipment is usually placed within sheet metal enclosures. The specification and anchoring of such equipment requires careful attention. Often, the most significant source of flexibility in equipment enclosures is due to local deformation of the equipment base near the anchor points. In such cases, the dynamic behavior is simply a rigid body rocking on effective base springs.

Elements of the life-safety systems such as emergency power, emergency lighting, alarm systems, and smoke removal systems require concentrated attention during the design process. These systems must be secure and functional after a major earthquake. Emergancy battery rack failures are one of the most common observations in post-earthquake studies, yet the cost for strengthening and securing such racks are minimal.

Evaluation of equipment subjected to dynamic environments requires consideration of operational and functional aspects as well as structural or enclosure strength. Unless specific requirements have been included in equipment procurement specifications, the ability of equipment to survive a dynamic environment, such as the building response to an earthquake, will be quite uncertain. The basic information required for an evaluation of flexible items are the equipment dimensions, sizes and arrangement of basic stress (structural) resisting elements, anchor details, distribution of weight (mass), equipment functional tolerances, and information concerning the previous dynamic environments sustained by similar equipment (such as transportation).

BUILDING CONTENTS

Free-standing equipment is susceptible to sliding or overturning due to floor motion. Since coefficients of friction vary widely, static friction cannot be relied upon to restrain equipment and supplies. Experience has indicated that furniture, cabinets and unanchored equipment within a building can undergo considerable displacement during an earthquake, especially in the upper levels of a building. Large, rigid architectural components, such as heavy artwork, heavy fixtures, shading devices, etc., must be anchored to the structure. The Architect should provide recommendations to the owner for the restraint of heavy furnishings. Face bars on shelves are suggested. It should be noted that overhead mutual bracing of shelves and cabinets is an expedient means of preventing tip-over. The attachment of shelving and cabinets to drywall partitions with toggle bolts is acceptable for lightly loaded shelves. For heavy shelving, positive attachment should be provided to the partition studs. Often, the distribution of weight on shelving is overlooked; heavy or fragile items should be located in the lower half of the shelf. A comprehensive study outlining restraint of hospital equipment, furniture, and supplies has been prepared [35].

CONCLUSIONS & RECOMMENDATIONS

1. An understanding of building response to earthquake ground motion is necessary for nonstructural designers to prepare specifications for equipment manufacturers and suppliers. The proper design of equipment anchorage and the interconnection of nonstructural components within a building requires knowledge of the magnitude of the forces induced in equipment and the deformations imposed upon the components.

2. Architectural designers need accepted criteria, consistent with structure types (i.e., frame, shear wall, box, etc.), so that adequate attention can be devoted to seismic design during conceptual design development.

3. Additional research, including testing, is required to allow modeling of infill wall, panel and interior partitions for incorporation in structural analysis procedures.

4. The development of code requirements which require dynamic testing to demonstrate function of critical life-safety system equipment components should be encouraged.

5. Considerable information concerning nonstructural design provisions included in recent designs is contained in design office files. Architects and consulting engineers should be encouraged to publish this data.

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TABLE 1: Extract from Title 17, Safety of Construction of Hospital, State of California

(ℓ) Lateral Forces on Non-Structural Components.

Architectural, mechanical and electrical components and systems in hospital buildings, essential equipment necessary for the complete functioning of the hospital operations and critical components located outside of buildings shall be anchored for lateral forces in accordance with Section 2312(g), formula (12-8) and the exception thereto. Where Cp in Table No. T17-23-J is less than 1.0 the product of IS need not exceed 1.5. The values of Cp for the anchorage of architectural, mechanical and electrical components and systems in buildings and critical components outside of buildings shall be as set forth in Table No. 23-J and Table T17-23-J.

Where the provisions of these tables do not specify Cp values for the anchorage of particular components which in the opinion of the Office of the State Architect should be anchored to resist lateral forces for the safety of the occupants, the office may assign Cp coefficients with the advice of the architect or engineer based on coefficients specified for similar components listed in these provisions.

The design of mechanical and electrical equipment, machinery, cabinets, etc., and the provisions incorporated in its manufacture for anchorage to supports or connection to seismic restraints should provide for these same lateral forces. However, the Office of the State Architect will not review the design or construction of such manufactured items except for their anchorage to the building structure or to a supporting foundation.

TABLE NO. T17-23-J
Horizontal Force Factor "C_p" for Elements of Structures and for Anchorage of Non-structural Components

Category	Direction of Force	Value of C_p [1]
1. Interior nonbearing walls and partitions over 5 feet in height	Normal to flat surface	0.20
2. When not part of a building and over 5 feet in height, masonry or concrete fences and walls	Normal to flat surface	0.20
3. When part of a building, cantilever walls above the ground floor (except parapets)	Normal to flat surface	0.30
4. Penthouses (except where framed by an extension of the building space frame)	Any horizontal direction	0.20

TABLE NO. T17 - 23 - J
(Con't)

5. When connected to, part of, or housed within a building:		
(a) Storage racks with the upper storage level more than 5 feet in height (plus contents)	Any horizontal direction	0.20 [2,4]
(b) Floor supported cabinets, files, and bookstacks more than 5 feet in height (plus contents)	Any horizontal direction	0.20 [2,4]
(c) Wall hung cabinets, shelving, and television racks(plus contents)	Any horizontal direction	0.20 [2,4]
(d) Suspended or surface mounted light fixtures [5]	Any horizontal direction	1.00
(e) Piping, electrical conduit, cable trays, and air handling ducts: [3]		
(1) Rigidly supported	Any horizontal direction	0.33 [4]
(2) Flexibly supported	Any horizontal direction	1.00 [4,6]
(f) Equipment and machinery such as boilers, chillers, pumps, tanks, cooling towers, engines, generators, motors, air handling units, transformers, switchgear, and control panels:		
(1) Rigidly supported (fundamental period of vibration of equipment with its supports less than 0.05 seconds)	Any horizontal direction	0.33 [4]
(2) Flexible or flexibly supported	Any horizontal direction	1.00 [4,6]
(g) Hospital equipment permanently attached to building utility services such as: Surgical, morgue and recovery room fixtures, radiology equipment, food service fixtures, and laboratory equipment	Any horizontal direction	0.20 [4]

(h) Communication equipment and emergency power equipment such as motor generators, battery racks, and fuel tanks necessary for the operation of such equipment[7]	Any horizontal direction	1.00 [4]
6. Power-cable driven elevators or hydraulic elevators with lifts over 5 feet:		
(a) Car and counterweight guides, guide rails and supporting brackets and framing	Any direction	0.33 [8]
(b) Driving machinery operating devices and control equipment:		
(1) Rigidly mounted	Any direction	0.33 [4]
(2) Flexibly mounted (a fundamental period of vibration of the installation greater than 0.05 seconds)	Any direction	1.00 [4,6]

Footnotes

1. Cp shall be not less than the ratio of Fx/Wx for floor or roof level under consideration. Where a dynamic analysis is used in the design of the building, the forces so determined may be used in the design of the elements or components with appropriate resistance criteria. Where a dynamic analysis is not used the minimum Cp values given should provide reasonable protection, but the use of higher Cp values is suggested for unusually important or expensive equipment or for equipment located in the upper levels of multistory buildings. See Section 2312(g) and Section T17-2312(e) for maximum values of the product of IS in formula (12-8).

2. Wp for storage racks, cabinets and bookstacks shall be the weight of the racks plus contents. The value of Cp for storage racks over two storage support levels in height shall be 0.16 for the levels below the top two levels.

3. Seismic restraints may be omitted from the following installations,

   (a) Gas piping less than 1 inch inside diameter.

   (b) Piping in boiler and mechanical equipment rooms less than 1¼ inch inside diameter.

   (c) All other piping less than 2½-inch inside diameter.

TABLE NO. T17 - 23 - J
(Con't)

(d) All piping suspended by individual hangers 12 inches or less in length from the top of pipe to the bottom of the support for the hanger.

(e) All electrical conduit less than 2½ inch inside diameter

(f) All rectangular air handling ducts less than 6 square feet in cross sectional area.

(g) All round air handling ducts less than 28 inches in diameter.

(h) All ducts suspended by hangers 12 inches or less in length from the top of the duct to the bottom of the support for the hanger.

4. The component anchorage shall be designed for the horizontal "Cp" force acting simultaneously with a vertical seismic force taken as one third of the horizontal "Cp" value used.

5. Suspension systems for light fixtures which have passed shaking table tests approved by the Office of the State Architect or which, as installed are free to swing a minimum of 45° from the vertical in all directions shall be assumed to comply with the lateral force requirements of Section T17-2312(ℓ).

   Unless of the cable type, free swinging suspension systems shall have a safety wire or cable attached to the fixture and structure at each support capable of supporting 4 times the support load.

6. Because of the possibility of resonant response of flexible equipment systems in the upper stories and roofs of buildings, consideration should be given to the use of higher values of Cp when the predominant period of response of structure and equipment systems are the same or nearly the same. Under the situation values of Cp twice as large as those indicated above are suggested.

7. Emergency equipment should be located where there is the least likelihood of damage due to earthquake. Such equipment should be located at ground level and where it can be easily maintained to assure its operation during an emergency.

8. Wp for elevator cars shall be the weight of the car plus 0.4 times its rated load. The lateral forces acting on guide rails shall be assumed to be distributed 1/3 to the top guide rollers and 2/3 to the bottom guide rollers of elevator cars and counter weights.

HOSPITALS

SUBMITTAL REQUIREMENTS

ANCHORAGE AND BRACING OF MECHANICAL AND ELECTRICAL SYSTEMS

1. Provide complete seismic anchorage and bracing details for the lateral and vertical support of piping, duct work, conduit, mechanical and electrical equipment, etc. as required by T17-2314(d)(2). See Table T17-23-J for applicable force factor "Cp". Consider the effect of temperature change in the preparation of anchorage and bracing details. Equipment anchorage details may be submitted and approved subsequent to plan approval by means of the "deferred approval" procedure, providing the equipment and procedure are noted in the plans and specifications.

2. The seismic bracing and anchorage of piping and ducts may in part be specified by reference to "Guidelines for Seismic Restraints of Mechanical Systems" (sheets 1 - 13), published by SMACNA and approved by OSA. Details for systems excluded from this standard shall be detailed on plans per note 1 above prior to approval (no deferred approval).

3. The seismic bracing for fire sprinkler systems may be specified by reference to NFPA No. 13, Installation of Sprinkler Systems 1972. Anchorage of bracing to structure must be detailed on plans (no deferred approval).

4. Provide details for piping, ducts and conduit that cross structural separations between buildings or building units. Note on plans the longitudinal and transverse displacements that must be accommodatedat each floor and roof level separation (2 x sum of drifts). Refer to T17-2314(m) and consult with structural engineer for drift criteria. Flexible connections which will accommodate the specified displacements without damage shall be detailed on the plans prior to approval (no deferred approval).

APPENDIX TABLE 2: Extract of 1976 Uniform Building Code

(g) **Lateral Force on Elements of Structures.** Parts or portions of structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = ZIC_p SW_p \ldots\ldots\ldots\ldots\ldots (12\text{-}8)$$

EXCEPTION: Where C_p in Table No. 23-J is 1.0 or more the value of I and S need not exceed 1.0.

The distribution of these forces shall be according to the gravity loads pertaining thereto.

(h) **Drift and Building Separations.** Lateral deflections or drift of a story relative to its adjacent stories shall not exceed 0.005 times the story height unless it can be demonstrated that greater drift can be tolerated. The displacement calculated from the application of the required lateral forces shall be multiplied by $(1.0/K)$ to obtain the drift. The ratio $(1.0/K)$ shall be not less than 1.0.

All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflection from seismic action or wind forces.

(k) **Essential Facilities.** Essential facilities are those structures or buildings which must be safe and usable for emergency purposes after an earthquake in order to preserve the health and safety of the general public. Such facilities shall include but not be limited to:

1. Hospitals and other medical facilities having surgery or emergency treatment areas.
2. Fire and police stations.
3. Municipal government disaster operation and communication centers deemed to be vital in emergencies.

The design and detailing of equipment which must remain in place and be functional following a major earthquake shall be based upon the requirements of Section 2312 (g) and Table No. 23-J. In addition, their design and detailing shall consider effects induced by structure drifts of not less than $(2.0/K)$ times the story drift caused by required seismic forces nor less than the story drift caused by wind. Special consideration shall also be given to relative movements at separation joints.

TABLE NO. 23-K
VALUES FOR OCCUPANCY IMPORTANCE FACTOR I

TYPE OF OCCUPANCY	I
Essential Facilities[1]	1.5
Any building where the primary occupancy is for assembly use for more than 300 persons (in one room)	1.25
All others	1.0

[1]See Section 2312 (k) for definition and additional requirements for essential facilities.

APPENDIX TABLE 2: Extract of 1976 Uniform Building Code

TABLE NO. 23-J—HORIZONTAL FORCE FACTOR "C_p" FOR ELEMENTS OF STRUCTURES

PART OR PORTION OF BUILDINGS	DIRECTION OF FORCE	VALUE OF C_p
1. Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions. Masonry or concrete fences	Normal to flat surface	0.20
2. Cantilever parapet	Normal to flat surface	1.00
3. Exterior and interior ornamentations and appendages.	Any direction	1.00
4. When connected to, part of, or housed within a building: a. Towers, tanks, towers and tanks plus contents, chimneys, smokestacks and penthouse	Any direction	0.20[2]
b. Storage racks with the upper storage level at more than 8 feet in height plus contents		0.20[2,3]
c. Equipment or machinery not required for life safety systems or for continued operations of essential facilities		0.20[2,4]
d. Equipment or machinery required for life safety systems or for continued operation of essential facilities		0.50[4,5]
5. When resting on the ground, tank plus effective mass of its contents.	Any direction	0.12
6. Suspended ceiling framing systems (Applies to Seismic Zones Nos. 2, 3 and 4 only)	Any direction	0.20[6]
7. Floors and roofs acting as diaphragms	Any direction	0.12[7]
8. Connections for exterior panels or for elements complying with Section 2312 (j) 3C.	Any direction	2.00
9. Connections for prefabricated structural elements other than walls, with force applied at center of gravity of assembly	Any direction	0.30[8]

See also Section 2309 (b) for minimum load on deflection criteria for interior partitions.

When located in the upper portion of any building where the h_n/D ratio is five-to-one or greater the value shall be increased by 50 percent.

W_p for storage racks shall be the weight of the racks plus contents. The value of C_p for racks over two storage support levels in height shall be 0.16 for the levels below the top two levels. In lieu of the tabulated values steel storage racks may be designed in accordance with U.B.C. Standard No. 27-11.

Where a number of storage rack units are interconnected so that there are a minimum of four vertical elements in each direction on each column line designed to resist horizontal forces, the design coefficients may be as for a building with K values from Table No. 23-I, $CS = 0.20$ for use in the formula $V = ZIKCSW$ and W equal to the total dead load plus 50 percent of the rack rated capacity. Where the design and rack configurations are in accordance with this paragraph the design provisions in U.B.C. Standard No. 27-11 do not apply.

For flexible and flexibly mounted equipment and machinery, the appropriate values of C_p shall be determined with consideration given to both the dynamic properties of the equipment and machinery and to the building or structure in which it is placed but shall not be less than the listed values. The design of the equipment and machinery and their anchorage is an integral part of the design and specification of such equipment and machinery.

For Essential Facilities and life safety systems, the design and detailing of equipment which must remain in place and be functional following a major earthquake shall consider drifts in accordance with Section 2312 (k). The product of IS need not exceed 1.5.

Ceiling weight shall include all light fixtures and other equipment which are laterally supported by the ceiling. For purposes of determining the lateral force, a ceiling weight of not less than 4 pounds per square foot shall be used.

Floors and roofs acting as diaphragms shall be designed for a minimum force resulting from a C_p of 0.12 applied to w_x unless a greater force results from the distribution of lateral forces in accordance with Section 2312 (e).

The W_p shall include 25 percent of the floor live load in storage and warehouse occupancies.

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

PROBLEM OF DAMAGE TO NON-STRUCTURAL COMPONENTS AND EQUIPMENT : WALLS AND STAIRS

by G H McKenzie
Assistant Chief Structural Engineer
New Zealand Ministry of Works and Development

INTRODUCTION

In a modern earthquake resistant building damage in earthquakes is more likely to occur to non-structural components than to the structure. In many cases these non-structural components represent a large portion of the total value of the building, and, consequently measures to lessen the extent of such damage will have a worthwhile effect on reducing the monetary loss caused by earthquakes. Damage to non-structural components is also likely to represent the major risk to life, if the structure is adequately designed against earthquakes. Exterior cladding components breaking loose and falling down the face of a high rise building present an obvious risk to life, while pieces of masonry fire walls and other components falling into stairwells and other means of egress are liable to cause death or injury, and are liable to impede the evacuation of a building.

Most types of modern buildings are very susceptible to non-structural damage, because they are generally flexible structures with low damping valves and they are designed to sustain large excursions into the inelastic range when responding to the largest earthquakes. Even shear wall buildings are likely to sustain large interstorey deflections in the upper levels, where the angle of deflection from the vertical is large, and, although elements very close to the shear walls may be protected by the walls against deformation, frames running parallel to the walls will be subjected to large deformations, and non-structural elements on or near such frames lines are likely to be severely damaged.

Good seismic design and detailing of the non-structural components can greatly reduce the likely level of damage, and the extra cost involved is usually comparatively small.

SCOPE OF STRUCTURAL ENGINEER'S RESPONSIBILITIES

If the problems in this field are to be adequately dealt with, it must be accepted that the structural engineer has responsibility for the non-structural components, as well as for the structure, and the professional fee structure should be based accordingly.

In some countries the structural engineer's fee is based almost solely on the value of the structure, and he is likely to receive inadequate reimbursement for any work he does on the non-structural elements. The scope of the considerations which he has to take into account is extensive, including the avoidance of unnecessary expense and concerns of the architect, such as avoidance of unsightly details where provision for large movements has to be made, and waterproofing of movement joints in exterior cladding. Close co-operation between structural engineer and architect is essential if details are to be produced that will satisfy the needs of both.

REDUCTION OF RESPONSE MOVEMENT

The problems involved and the cost of measures to minimise non-structural damage could be reduced if the magnitude of the interstorey response deflections were made lower.

The first approach which one examines is that of stiffening the building structure. Unfortunately, this also results in higher seismic forces, both on the structure and on the non-structural components and their fixings. On low buildings, with a height to width ratio of one to one, changing from a frame structure to a shear wall structure can considerably reduce response deflections. However, on higher buildings even shear wall structures can show large deflections in the upper stories, even with height to width ratios as low as 2 to 1.

The approach which appears likely to offer the best prospects in the future, is that of incorporating energy absorbing devices in building structures. This is still in the experimental stage of development, but we are designing our new Wellington District Office building with such devices included in the structural system. Their effect is totally beneficial, because they lengthen the response period of the building as a whole but reduce the interstorey deflections and response forces at all levels above that of the devices. In justifying to clients the extra cost of the energy absorbers by presenting corresponding cost benefits, the engineer can cite reduction in the cost of seismic provisions for non-structural components, as well as reduction in cost of the structure due to lower seismic forces.

CLEARANCE FOR SEISMIC MOVEMENT

The first line of defense against damage to non-structural components is to provide adequate clearance for the computed relative seismic response movement, both at the fixings and around the margins of the components where they are adjacent to other elements.

Basis for Computed Deflection

This immediately brings up the question of what the basis for the computed deflection is. The deflection computed for the design earthquake loading will be slightly less than that corresponding to yield point stresses in potential hinging areas in flexural members. However, a dectile structure will be designed to respond in the inelastic range to major earthquakes, and its maximum response may be more than 4 times the deflection causing general flexural yielding. The desirable criteria for clearances is thought to be that

1 *They should avoid damage in the moderate earthquakes which may be expected several times in the life of a building.*

2 *They should limit damage in the most severe earthquakes and should avoid loss of life.*

3 *They should prevent non-structural components from having adverse effects on the seismic properties of the structure.*

New Zealand Code Provisions for Seismic Movement

The New Zealand Code of Practice for General Structural Design and Design Loadings for Buildings, N.Z.S. 4203:1976, requires non-structural components in ductile structures to have clearances for releative movements corresponding to the deflection of the structure under an applied horizontal load of

$$\frac{2.0\ CI}{C_d}\ C_d\ W_t$$

$$= 2.0\ CI\ W_t$$

where C is the basis seismic coefficient corresponding to the seismic zone and the period.

I is the importance factor, which varies from 1.0 to 1.6

C_d is the design seismic coefficient, and is given by:

C_d = CISMR.

S is the structural type factor, which varies from 0.8 for the most ductile type of frame structure to values greater than 2 for some diagonally braced structures.

M is the structural material factor which varies from 0.8 for structural steel to 1.0 for reinforced concrete and 1.2 for reinforced masonry.

Hence the deflection of the structure which has to be provided for is twice that which would be produced by the basic seismic coefficient weighted by the importance factor. The structural type factor and the material factor are not taken into account in calculating the deflection, because it is assumed that the constant displacement hypothesis approximately applies. i.e that the total of elastic plus inelastic displacements is independent of the level of load at which the structure goes into the inelastic range and the ductility factor capacity of the structure.

N Z S 4203 requires that, where inter-storey deflections, calculated as above, exceed 0.0006 of the storey height, non structural elements shall have clearances provided for the calculated deflections when they come into the following categories:

a) *Elements, such as stairways, rigid partitions, and infillings, that are capable of altering the intended structural behaviour.*

b) *Precast concrete claddings and other claddings of similar mass.*

c) *Glass windows and other rigid brittle exterior claddings, except in the case of claddings on class III buildings in seismic zone C that in the case of failure cannot fall through a height greater than the storey in which they were installed.*

The separation provided for the above categories shall not be less than 12 mm between vertical surfaces of structure and element.

The calculated inter-storey deflections must not be more than 0.010 of the storey height in any case.

PROVISIONS FOR THE PROBABLE MAXIMUM EARTHQUAKE

Limitations of New Zealand Code clearance provisions

Dynamic response calculations will fairly quickly disclose that the New Zealand code provisions do not require large enough clearances for the probable maximum earthquake level. For example, in typical six storey building with ductile shear wall structure, the maximum inter-storey deflection under design load was 0.1 inches. The clearances required by the code would provide for an inter-storey deflection of 0.1 inches x 2/1.2 = 0.16 inches (since S for single cantilever ductile shear walls is set at 1.2 by the code). However, the inelastic response displacements computed for the building for 30 sec. of the artificial A_2 earthquake record gave maximum inter-storey deflections of ½ inch, approximately three times the deflection which the code clearances would provide for. Investigations of ductile frames and ductile coupled shear walls gave similar results.

Other effects and contingencies could make the code clearances even more inadequate. For example, in the plastic hinge area, full scale tests have resulted in the formation of open cracks up to ½ inch wide. If the fixings for a component were on either side of such a crack, the crack width would add to the clearance requirement. Again, defects in workmanship, such as not centering components between clearances in opposite directions or allowing architectural finishes to partly fill clearance gaps, can reduce effective clearances below the design levels.

Measures to Provide for Movements Greater than Code Clearance Requirements

It is apparent from the above that the clearance requirements in the New Zealand code will provide for small displacements into the inelastic range in medium earthquakes, but will be inadequate for high intensity earthquakes. Hence, additional measures are required to cater for the larger displacements that occur in more severe earthquakes. These can be classified under the following two main ogjectives, *(a) avoiding adverse effects on the seismic properties of the structure, and (b) minimising damage to the non-structural components and resultant risk of injury and loss of life.*

Measures to avoid adverse effects on the seismic properties of the structure

These are of primary importance, because failure to take such measures could seriously reduce the effectiveness of the seismic provisions in the main structure. Even sacrificial measures, such as reducing the strength round the margins of infill panels, would be justified as a last resort in some situations in this category.

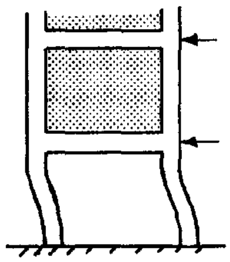

fig 1

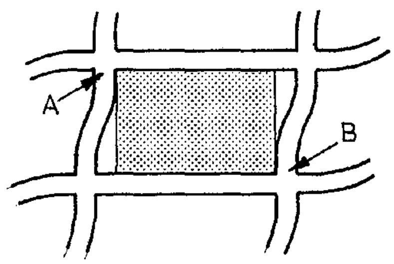

fig 2

Examples of such situations are shown in fig 1 and fig 2. In fig 1, infill panels in the upper storeys cause all the ductility demand to be concentrated in the bottom storey, with very large plastic hinge relations resulting in that storey, and make conditions even worse by forcing a column hinge mechanism. In fig 2, an isolated infill panel sets up shear failure conditions in adjacent columns at points A and B. Panels that do not go full storey height can also have serious effects. In fig 3, the effective storey height of the column available to absorb the inter-storey deflection in bending has been reduced to CD. This induces very severe hinge rotations in the column, places the lower column hinge at a level where there is no confining reinforcement and may make the column weaker in shear than in flexure. In fig 4 it can be seen that a large window opening will result in a similar situation, with the effective storey height of the column in flexure being reduced to EF.

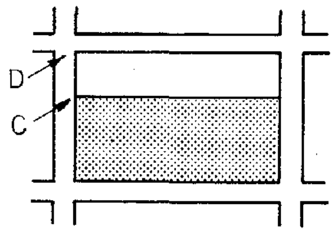

fig 3

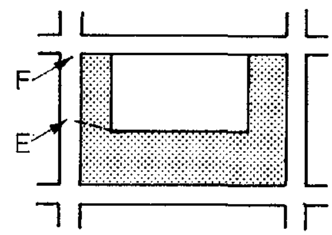

fig 4

A different type of undesirable condition occurs where stiff panels are placed very unsymmetrically and severe torsion effects can result.

It is obvious that there is a variety of detail arrangements which can result in adverse effects on the structure and the designer must use his imagination and keep on the alert to recognise these. Problems in this category are usually associated with masonry infill panels, although occasionally stairs with inadequate clearances may have undesirable effects on adjacent structural members.

Measures to remedy such effects can include one or more of the following:

a) *Change the material of the wall from masonry to one with more flexibility. For example, for exterior wall claddings use narrow vertical precast panels or curtain walls with clearances in the light metal framing sections.*

b) *Arrange the form of the building, so that the panels are kept outside the lines of the frames as far as possible. For example, exterior walls can be arranged to run completely outside the columns or completely inside them.*

c) *Use details which will allow considerably more than the calculated movements, such as a channel that the wall can slide along for the top fixing and a deep channel that the wall can slide into at each end of the panel.*

d) *Introduce vertical joints into each panel, to break it up into vertical strips which will not have sufficient strength and stiffness to cause damage to the structure.*

Measures to Minimise Damage to Non-Structural Components

These include some of the measures listed in the previous section to protect the structure from adverse effects. One of the most important requirements is to have ductile fixings which can accommodate movements greater than those for which clearances have been provided by yielding without breaking. Heavy exterior cladding panels and pieces of window glass must be prevented from breaking loose and falling, to the danger of people in the street below. Similarly, pieces of non-structural components must not break loose in a position that allows them to fall into stairwells and egress corridors. Fire rated walls must, as far as is practical, retain their fire rating after the earthquake. Some damage to waterproofing of joints between exterior cladding components can be accepted in a severe earthquake, but the water-proofing components must be readily accessible for repair.

Seismic provisions to minimise damage are dealt with more specifically in the next section on component details.

COMPONENT DETAILS

Exterior Cladding Panels

In general simpler details can result and larger inter-storey deflections can be accommodated if the exterior panels are not located on the lines of the columns, because relative movement between each column and adjacent panels does not have to be provided for.

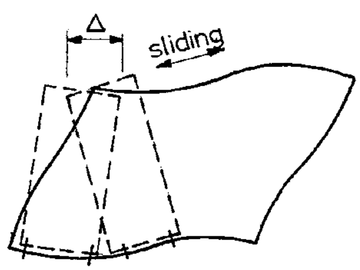

fig 5

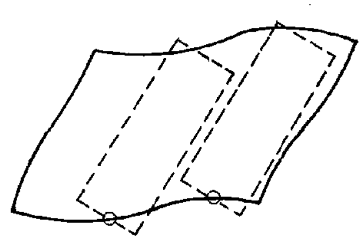

fig 6

Fig 5 illustrates the system where the bottom of each panel is bolted to the floor below, while the top is free to slide in a channel running in the plane of the panels, which is fixed to the underside of the floor or beam above. The connection details have enough clearance or flexibility to allow the panels to rock at right angles to their plane, for that movement component.

Fig 6 illustrates the system where the bottom of each panel has a connection which acts as a pivot for movement in the plane of the panel, and the top connections are equivalent to a pivot connection to the floor above. Thus the panels remain parallel to each other as they move.

It is obvious that the system of fig 5 will involve large relative horizontal movements between the vertical edges of adjacent panels, between panels and adjacent columns, and at corners between the end panels running in the two different directions. This will tend to cause damage at the vertical edges of panels, it will impose severe conditions on the weathering components across the vertical joints, and it will tend to cause damage at corners of exterior walls. Calculations indicate that in some conditions the deflection Δ between adjacent panels can be more than twice the inter-storey deflection. It is also obvious that the system is not suitable for walls that run along column grid lines, infilling the spaces between columns.

The system of fig 6, on the other hand, involves practically no relative horizontal movement between adjacent vertical panel edges or between adjacent panels in different planes at building corners. The weathering components across the joints will only have to withstand a vertical sliding movement parallel to their height, which will involve practically no damage to a suitably designed system. Further, the relative horizontal movements between columns and adjacent panels are small, so the system would perform reasonably well for walls that run along column lines.

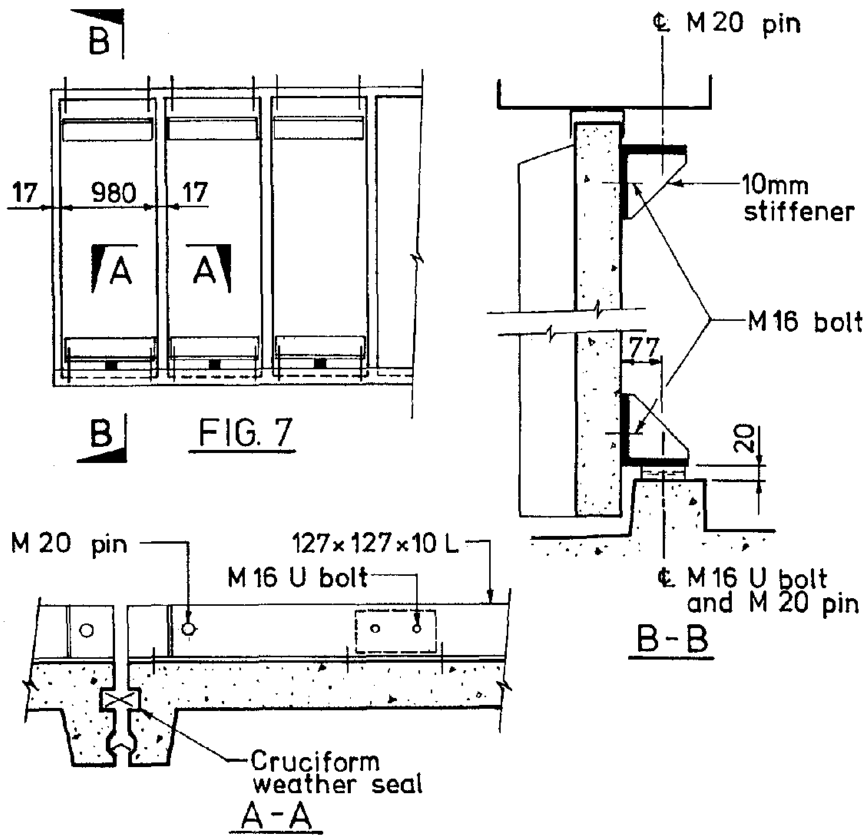

Fig 7 and sections A-A and B-B show typical details for the panels and connections for the system of fig 6. The panels are vertically supported by the centre fixing at the bottom, and are laterally supported by pins at the four corners, which allow the panel to rotate in its own plane. To facilitate erection, the metal connections for the pins are initially provided with oversized holes, but after rection and adjustment, close fitting washers are placed over the pins and welded in position.

The weathering problems are minimised if floor slabs project beyond the exterior faces of the slabs. A concrete upstand immediately behind the panel provides a weathering rebate at the bottom, while a channel or flashing on the underside of the floor above leads the weather down the exterior face of the panels. This weathering system is not liable to damage by panel movement.

For the system of fig 5, the upper channel should be positioned below the level of any transverse beams, to avoid the complication of providing for horizontal movement relative to vertical beam faces. Storey claddings above that level are fixed to and move with the upper storey. The bottom connection details should give some freedom or flexibility for yielding both vertically and horizontally. If deformation of the beam below forces the tops of adjacent panels into contact, further movement could strain the connection upward, requiring vertical yielding, while cracks opening in the beam hinge area could cause horizontal movements between connections. Panel connections generally should be ductile and capable of being strained into the yield range without failing in shear or in the anchorage to the structure or the anchorage to the panel. The most dependable form of anchorage to the panel is to weld the connection to the panel reinforcing bars or provide a positive fixing between connection and reinforcement.

It is not desirable to depend for support of vertical panel loads on bolts in shear, if the same bolts can be deformed laterally by seismic movements.

Weathering components across joints need to be appropriate for the type of movement. Those depending on the bending and unbending of a fold can perform well for lengthening and shortening of the joint gap, but give no freedom for relative vertical sliding movements. If the components are liable to require repair or replacement after a severe earthquake, they should be readily accessible.

Exterior Glass Cladding

Window assemblies can be given freedom for large movements in their plane if they are fixed to the floor below along their bottom margin and slide freely in a channel at their upper margin. The channel supports the window assembly laterally against loads at right angles to the plane of the windows. For inter-storey deflections at right angles to the plane of windows, the assembly follows the deflection by rocking, and the bottom fixings and the top channel have clearances or flexible packings to allow this rocking. The channel is fixed below the underside of any transverse beams, and all the glass cladding above this level is fixed to the floor above. This avoids troublesome details for providing clearance for horizontal movement relative to vertical faces of beams.

If the window plane runs into columns or other vertical members, provision for the horizontal movement of the vertical margin of the window assembly can be made by using vertical light metal sections which incorporate channels that the window frame can slide in and out of.

Fairly neat details can be provided in this manner, but they involve extra cost and waterproofing problems can arise. It is still preferable to try to avoid placing windows on column lines, in order to eliminate the need for large clearances to vertical elements.

Provision of clearance between the glass and the window frame is an important source of freedom for movement for window assemblies. It can be relied upon if neoprene or other elastomeric window gaskets are used, but if ordinary glazing putty is used, the freedom for movement will progressively reduce as the putty hardens with age. This clearance is always desirable, even if the sliding channel top fixing is used, because there are other sources of in-plane deformation in window assemblies, such as deflection and plastic hinging of the beams supporting the floor below.

Interior Walls

One of the main problems in providing for inter-storey movement in interior partition walls is that there is usually a large number of right angle intersections between walls. Having the top of a partition held laterally by a channel in which it can slide would work well for a wall running in one direction, but the system presents difficulties for walls intersecting at right angles. Each wall accommodates the inter-storey deflection component normal to its plane by rocking in that direction, and the resultant relative movement between intersecting walls can be in any horizontal direction, which makes it very difficult to provide satisfactory junction details. The writer believes that a vertical light gauge rectangular hollow aluminium section at the junction, connected at the bottom to the floor below and at the top to the floor above may provide a solution, with a sliding fit channel enclosing the end of each of the intersecting walls.

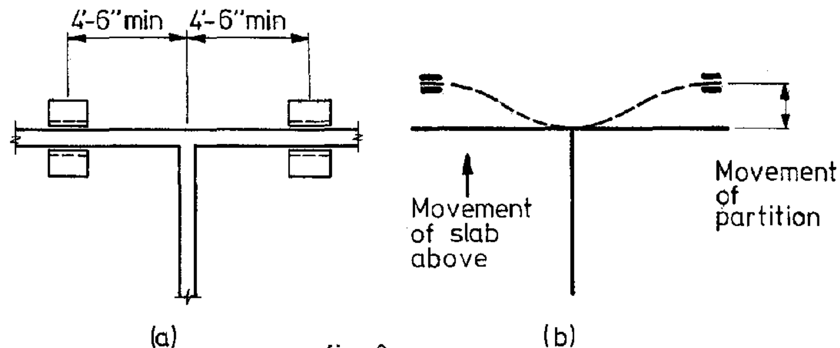

fig. 8

The post would always remain in the plane of each wall, even when both walls rocked, and each channel attached could slide horizontally over the wall end that it enclosed, to accommodate the corresponding component of inter-storey deflection. However, the detail is fairly expensive and would not please all architects.

In one building, a good economical solution was found by fixing the partitions solidly together at the intersection and placing the angle brackets for top lateral support 4'6" away from the junctions, thus allowing the partition to bend out of its plane.

Tests showed that, for this distance to brackets, a deflection of the partition of ½" could be obtained without any damage to the partition, which was lined both sides with ½" particle board. The arrangement is shown in fig 8.

Once again, the simplest details result from making the level of sliding support at ceiling level, below the undersides of all beams, to eliminate the complication of trimming for movement against vertical faces of beams.

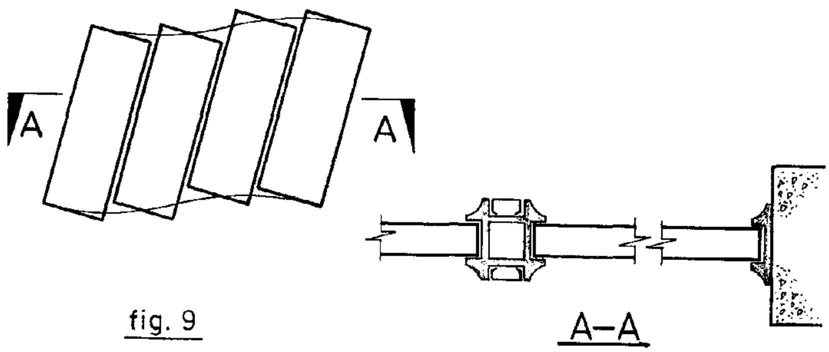

fig. 9

Fig 9 shows a system which satisfies practically all movement requirements for a light type of partition. The wall is made up of fairly narrow vertical sheets of gypsum, faced with vinyl on both sides. The sheets are laterally supported along their vertical edges by vertical extruded aluminium members which are fixed at the top to the floor or ceiling above and at the bottom to the floor below. The advantage of this system is that all wall elements in one wall rock through exactly the same angle as all wall elements in any wall running at 90 degrees to the first wall. Hence there are no problems at wall corners or tee junctions, and even where wall junction with columns the relative movement is sliding in the plane of the wall, which involves simple trimming details.

Where partitioning can conveniently form an assembly of rigid boxes, a good solution can be provided by connecting the partitions and ceilings to form rigid boxes. These boxes follow the movement of the floor below, and the only connections to the floor above are flexible vertical hangers to take the vertical load of the ceiling. This eliminates all relative movements between elements of the boxes and makes for simple detailing and little seismic damage. Clearance does have to be provided round columns where they pass through the ceilings, but the details involved are simple. However, if any of the walls run into columns, awkward detailing can arise, due to the necessity to provide for relative horizontal movement in any direction, and it will be advantageous to keep walls off column lines where possible.

The box system described above is certainly the most attractive solution for concrete or reinforced masonry partitions. Where walls of the boxes intersect rocking exterior walls, dowelled connections across a movement gap to the exterior wall can be used, and trimming details to cover the gap are simple.

The box system is also a good solution for fire rated walls round stairwells, as it avoids the necessity for having open gaps for movement which might lower the fire rating. The only movement joint required is a horizontal sliding joint right round the box, which can be detailed to a good fire rating standard. The joint can be against the underside of the slab above, or can be at the level of the underside of the beams. In the latter case, a shallow box connected to the floor above extends down to the movement joint, and lines up with the lower box extending up from the floor below. This has been used by our designers for timber framed fire walls which are sheathed with fire rated sheets of duratherm.

A typical fire rated shear movement joint detail is a ½ inch horizontal gap packed with asbestos rope. On one recent building the gap was filled with a ½ inch thick strip of fibrous gypsum plaster, with the top surface graphite coated.

Where a column comes on the line of a fire-rated wall, provision for movement is difficult. The movement gap must be packed with a fire-rated compressible filler that has a low resistance to compression and can elastically spring back to its former volume.

Of the materials that we have investigated to date, the most suitable appears to be Kaowool, a ceramic fibre, but we have to carry out further tests.

Where reinforced masonry or concrete walls run along the line of a frame beam below, the bottom connections of the wall should preferably be by sliding vertical dowels rather than anchored starter bars, unless the wall panels have been specifically designed to act as part of the structure.

Stairs

Stairs tend to act as diagonal bracing between floors, and can have damaging loads induced in them by inter-storey deflections. Hence effective provisions to free them must be made.

One architecturally attractive solution is to design stairs as two flight or three flight free-standing staircases, spanning from the floor above to the floor below as a self contained structure, without any outside support to the landing. The flexibility for inter-storey movement at right angles to the main flights must be checked.

The arrangement most frequently adopted is to put a separation gap through the mid storey height landing, so that each half of the landing is connected to only one flight of stairs. The support for the vertical load of the landing is arranged so that the landing is free to move laterally. Such support can be by flexible hangers, flexible struts or sliding support on a beam.

Where a stairway consists of single flights between floors, each flight can be fixed at one end by a movement gap and sliding support, or freed at the top end by providing flexible strut support.

Separation gaps can be covered by metal plates with provision for sliding.

CONSTRUCTION PROBLEMS

Provisions for seismic movement involve features that are contrary to normal trade practice.

If the site work force do not understand the reasons for some of the details, they are liable to place packers in movement gaps, connect elements that move relative to one another by fixings and lock up sliding joints with sealers. We have adopted the practice in recent jobs of including explanatory notes with the specification, setting out how the various separation provisions are intended to work.

CONCLUSIONS

The non-structural portion of a building requires seismic design. This can require a very large amount of design effort, and should not be regarded as less important than the design of the structure. Poor design of the non-structural components can result in very costly damage, risk to life and even adverse effects on the structure. Minimising damage potential requires a high level of co-operation and understanding through the whole design and construction period between engineer, architect and site construction forces.

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

COMPUTER-AIDED OPTIMUM DESIGN OF DUCTILE R/C MOMENT-RESISTING FRAMES

by

Stan W. Zagajeski
Research Assistant
University of California
Berkeley, California

Vitelmo V. Bertero
Professor of Civil Engineering
University of California
Berkeley, California

INTRODUCTION

General Goals and Current Practice of Earthquake-Resistant Design

The general philosophy of earthquake-resistant design for buildings other than essential facilities has been well established and proposes to (1) prevent nonstructural damage in frequent minor earthquake ground shakings in the service life of the structure, (2) prevent structural damage and minimize nonstructural damage in occasional moderate earthquake ground shakings, and (3) avoid collapse or serious damage in the rare major earthquake ground shakings. This philosophy is in complete accord with the concept of comprehensive design [1], but current design methodologies fall short of realizing its objectives.

Although it is recognized that for buildings located in regions near active faults where there is the possibility that very severe earthquake ground shaking might occur during the service life of the structure, the most critical limit states are the ultimate, most of the seismic-resistant design procedures presently used in practice are based on (1) the use of equivalent (or effective) static seismic lateral forces defined at service or first significant yielding level, (2) determination of internal design forces by linear-elastic analyses, and (3) proportioning of members using either working (service) stress methods or by considering the ultimate strength of their critical sections. Only recently has design practice in regions of high seismic risk begun using procedures based on ultimate limit states, focussing on safety against collapse of the main structure as the controlling ultimate limit state. Reference 1 discusses the desirability of introducing into seismic-resistant design practice a new group of limit states based on damageability to bridge the serviceablity and collapse limit states.

The authors believe that structural design should be based on the limit state that actually controls it. If an ultimate limit state (damageability or collapse) controls the design and a fictitious linear-elastic limit state is adopted for preliminary design, the resulting design should be checked at ultimate states using realistic models.

The advantages of developing a design method based on two failure stages have been discussed by Sawyer [2], and for simple structures subjected to standard loading a design method based on two behavioral criteria (collapse and loss of serviceability) and on four optimizing criteria has been developed by Cohn [3]. Application of the latter method to the seismic-resistant design of ductile moment-resisting steel frames seems feasible and practical [4].

The ultimate objective of the designer is to have an economical, serviceable, and safe building. To achieve this aim, an efficient preliminary design is necessary. Sophisticated and efficient computer programs recently developed for the analysis of complex structures do not necessarily guarantee an efficient design, particularly for the case of seismic-resistant design. Regardless of how sophisticated the computer programs are, repeated analyses of a poor preliminary design will usually only lead to an improved "poor final design."

Recognizing the importance of the overall design concept and the need for a sound preliminary design, the authors have developed the seismic-resistant design procedure described herein. It should be emphasized that the proposed procedure has been developed for the design of R/C framed structures of buildings located in regions near active faults where there is the possibility that very severe earthquake ground shaking might occur during the service life of these buildings.

Main Objectives of Proposed Design Procedure

The principal objective of this procedure is to develop the most economical and practical design consistent with serviceability requirements under all possible service excitations--a design which will at the same time minimize economic losses due to damage (primarily nonstructural) and minimize the danger of collapse under a possible, but highly unlikely, severe earthquake ground shaking.

The procedure should be versatile enough to permit the inclusion of new and more reliable data as they become available as well as new design requirements and/or practical design constraints.

In addition, the procedure should be automated as much as possible to produce a preliminary design in a relatively short time.

Scope

The proposed seismic design procedure was developed specifically for R/C ductile moment-resisting frames. It represents an extension of the procedure developed by Bertero and Kamil [4] for steel frames. To achieve the above objectives, the procedure developed employs a computer-aided iterative techniqe in five basic steps which are carried out in two main phases: a preliminary design phase and a final design phase. In this paper great emphasis is placed on the preliminary design phase

GENERAL DESCRIPTION OF DESIGN PROCEDURE

Preliminary Design Phase

The objective of this phase is to obtain efficiently a preliminary design which is as close as possible to the final "optimum" design. This is deemed essential in obtaining a true optimum design. The preliminary design phase consists in three basic steps, which form an iterative loop to be repeated until an acceptable design is obtained. These steps are preliminary analysis, preliminary design, and analyses of the preliminary design.

Preliminary analysis--The final objective of this first step is to obtain the design story shear forces. To this end, the given data regarding the function of building and building site are studied in order to establish serviceability, damageability, and safety requirements and to select a realistic design earthquake. The design earthquake is best defined by a smooth ground spectrum. An inelastic design spectrum is then constructed by selecting appropriate damping ratio and displacement ductility factors. Finally, the design story shear forces are obtained from the inelastic spectrum by a modal superposition analysis based on estimated values of periods of vibration and mode shapes. Expected P-Δ effects are estimated and included in the design story shears.

Inherent in the use of the modal analysis technique is the assumption that a sufficient number of plastic hinges form simultaneously to transform the frame into a mechanism. In other words, the frame is assumed to behave as an elastic, perfectly plastic, single degree-of-freedom system (Fig. 1). The likelihood of this happening, especially in response to an earthquake excitation, is very small. First, it should be recognized that the proportioning of members is based on envelopes of internal forces that include all possible load combinations. Thus, design of the different critical regions must be governed not only by different load combinations, but also by forces that do not occur simultaneously. In addition, during an earthquake ground motion plastic hinges typically migrate from the lower to the upper stories during the response. Many of the plastic hinges which formed in the lower stories during initial stages will close before a sufficient number of hinges can form in the upper stories to transform the structure into a mechanism. Because the plastic hinges form gradually, the change in stiffness at "yield" is more gradual than in the idealized case, and a more realistic generalized force displacement relationship would be that indicated by the dashed curve in Fig. 1. As indicated in Fig. 1, overstrength would be expected not only because of the gradual hinge formation, but also due to the fact that actual member yielding strength will be different from the computed required design capacity (typically greater) because of the finite number of combinations of member sizes and reinforcement arrangements, and due to strain-hardening of the reinforcement.

Preliminary design--The preliminary design consists of a story-wise weak girder - strong column limit design using an optimization procedure. Linear programming techniques are employed to find the beam design moments which minimize an objective function proportional to the required volume of flexural reinforcement. The beam design moments must satisfy equilibrium constraints derived from the kinematic theorem of simple plastic theory. Additional constraints are imposed to include serviceability requirements and practical design considerations. The merit function combined with the equilibrium, serviceability, and practical constraints comprise a standard linear programming problem. A solution for the beam design moments is obtained using a Simplex algorithm. The beams are then proportioned to provide these design moment capacities and the columns are subsequently designed to satisfy the weak girder--strong column design criterion. The member sizes and reinforcement are found by using a computer program which is based on the 1973 UBC [5] ultimate strength requirements for reinforced concrete members.

An iterative loop exists within the preliminary design step (Fig. 2).

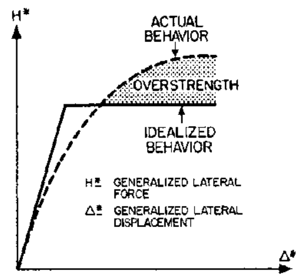

FIG. 1 LATERAL FORCE DISPLACEMENT BEHAVIOR

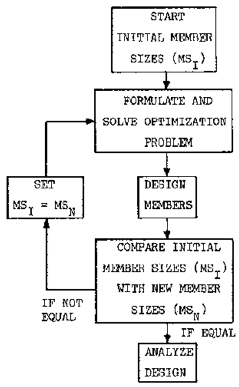

FIG. 2 ITERATIVE LOOP IN PRELIMINARY DESIGN STEP

Analyses of Preliminary Design--In this final step the preliminary design is analyzed to determine if it is acceptable. The dynamic characteristics of the designed structure are determined using standard procedures and are compared with those selected in the preliminary analysis. The behavior under service load conditions is determined to check serviceability requirements.

An inelastic static analysis of the designed frame subjected to the lateral force pattern corresponding to the seismic design story shears is carried out to determine displacement ductility factors and static overstrength factors and to locate any apparent weaknesses in the design.

Finally the structure's response to different earthquake ground motions is obtained using a nonlinear dynamic analysis program. In this program all members are represented by a two-component element which effects an elasto-plastic moment curvature relationship with linear strain hardening. The P-Δ effect, the influence of axial force on the column yielding strength, and the influence of the floor slab on the frame stiffness are included in the analyses.

Unfortunately both the static and dynamic nonlinear analyses programs used in the current study are limited to the analysis of the planar behavior of frames. Consequently a three-dimensional structural analysis of the entire structural system, which would include torsional effects, cannot be carried out with these programs.

Maximum values, as well as time histories, of the main dynamic response parameters are examined to determine if they are acceptable with respect to: (1) the established design criteria for damageability and safety; (2) known member deformation capacities; and (3) the assumptions made in the first step (preliminary analysis).

If these analyses prove that the designed structure meets the established design criteria (that the design characteristics are similar to those assumed in the preliminary analysis, and that the required inelastic deformations are compatible, that is, can be developed by the members), then the preliminary design process is complete, and a final optimum design is attempted. If any characteristics of the designed structure are unacceptable, the design is modified, either by starting at the first step or by making the adjustments necessary to eliminate the observed shortcomings.

Final Design Phase

The final design phase consists of two steps. In the first step a final optimum design is obtained. The design procedure is similar to that employed in the preliminary design phase, with the exception that a more sophisticated subassemblage is used in the formulation of the optimization problem. The seismic design forces are determined from the inelastic design spectrum by using the dynamic characteristics of the accepted preliminary design. As in the preliminary phase, a weak girder-strong column design criterion is established. In the second step, the optimum design is analyzed to evaluate its overall reliability under service and ultimate loading conditions.

Summary of Design Procedure

A flow chart of the design procedure is shown in Fig. 3. The steps in the preliminary design phase are repeated until an acceptable preliminary design is obtained, at which point the final optimization it attempted.

The proposed design procedure will be illustrated by presenting a detailed discussion of the design of a ten-story three-bay frame (Fig. 4). Throughout the presentation emphasis is placed on the methodology of the design procedure rather than on detailed computations.

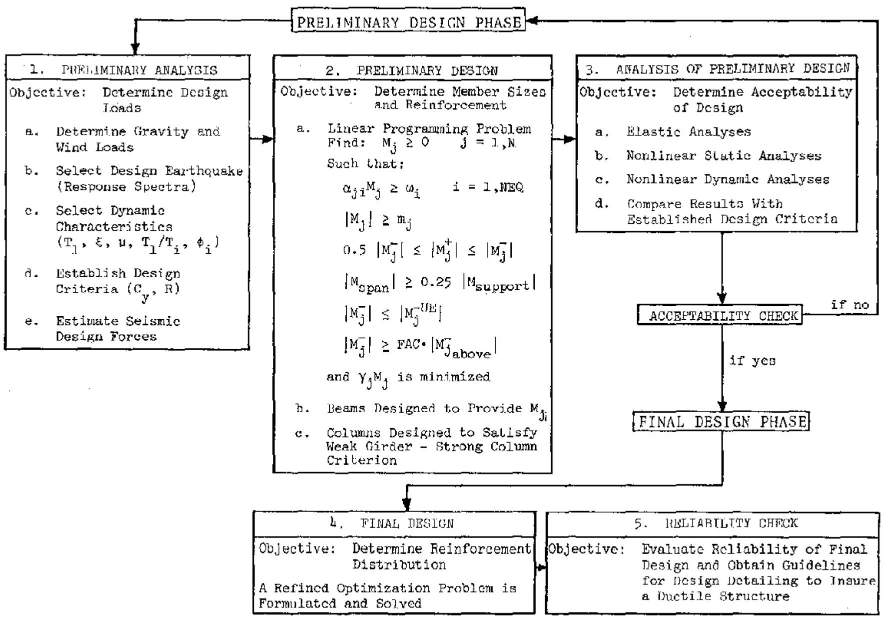

FIG. 3 SUMMARY OF DESIGN PROCEDURE

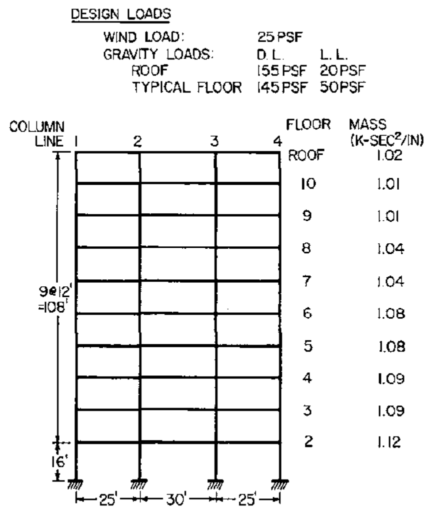

FIG. 4a FRAME ELEVATION AND DESIGN DATA

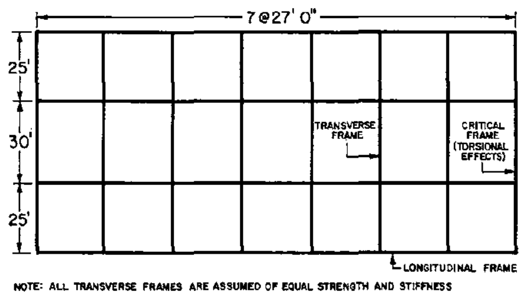

FIG. 4b TYPICAL FLOOR PLAN

PRELIMINARY DESIGN PHASE

General Design Criteria

As already noted, the objective of the preliminary design phase is to efficiently obtain a design which is as close as possible to the final desired design. In seismic-resistant design, the following general design characteristics are considered desirable.

1. A weak girder-strong column design should result. In other words, it is desired to force inelastic deformations to occur in the girders and to

limit as much as possible the inelastic deformation demands in the columns.

2. Abrupt transitions in mass, stiffness, strength, and ductility should be avoided throughout the height as well as the plan area of each story of the structure. If a change in stiffness is necessary, for example when the beam or column size is changed, a corresponding change in strength should be included in the members in this area. This is required to prevent early yielding in a particular region which might result in large localized inelastic deformations and create the possibility of a failure, as defined by the damageability or ultimate limit states. The importance of a smooth transition in stiffness and strength cannot be overemphasized.

The above characteristics have been established as design criteria in both the preliminary and final phases of the design procedure.

Preliminary Analysis

The objective of this step is to obtain the lateral story shears corresponding to a given or selected design earthquake. It involves the following.

Analysis of given data--The frame geometry, standard design loads (dead, live, and wind), and story masses are given in Fig. 4(a). The most difficult task in the first step is to select the proper design earthquake. For the present application it is described quantitatively by the inelastic response spectrum shown in Fig. 5. This spectrum is constructed from given values of effective peak ground acceleration, 0.4g, ground velocity, 48.6 cm/sec. (19.2 in./sec) and ground displacement, 36.6 cm (14.4 in.) that are expected to occur at the building site, following the method suggested by Newmark [6]. It should be noted that the ground motion spectrum in Fig. 5 represents a very severe ground shaking which might occur only at regions near active faults.

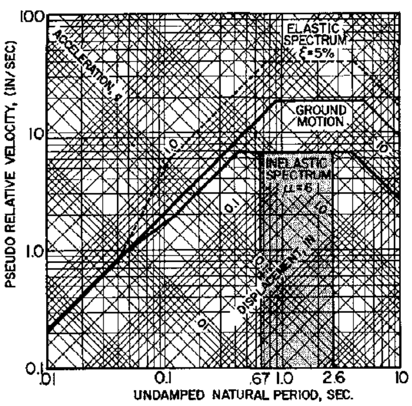

FIG. 5 DESIGN SPECTRA

Selection of main seismic design parameters--The seismic design parameters are the seismic coefficient (C_y), acceptable drift indices (R), period ratios T_1/T_i (T_1 is the first mode period, and T_i is the ith mode period), and mode shapes, ϕ_i. Acceptable limit values for C_y should be assigned according to present design and construction experience and economic considerations. Acceptable values for R should be selected on the basis of acceptable damage at the service load limit state and on the basis of damageability and safety against collapse at the ultimate limit state. The acceptable damage levels should derive from the functional and economic implications of this damage. However, reliable quantification of damageability limit states is still unavailable.

For the example it was decided that C_y should be less than 0.2, and that R should be less than 0.002 at service load conditions and less than 0.015 at the ultimate load state. R at the ultimate load state defines in a very simplistic way the damageability limit state.

The frequency ratios and mode shapes can be found from available tables [7], from previous experience, or from a frequency analysis of an initial design. The latter method is used for this example.

Selection of values of T_1, μ, and ξ--Initially it is necessary to assume a set of values for the first mode period, T_1; the displacement ductility factor, μ; and the damping ratio, ξ. Based on an analysis of the frequencies of similar structures, T_1 was assumed equal to 1.0 sec. From previous experience with similar structures, μ was assumed equal to 6, and ξ was 5%. It should be noted that the value of ξ is generally found to vary little with the natural frequency and seems to depend almost exclusively on the structural material, structural system and nonstructural components and their interaction, and on the degree of damage expected which in turn is a function of μ. The final selection of the proper values for these three factors usually requires an iterative procedure which includes a series of computations described in the next step.

Estimation of first mode maximum response--The inelastic design spectrum for a single-degree-of-freedom system (SDOFS) is obtained from the selected ground motion spectrum in two steps. The elastic response spectrum is first constructed by multiplying the ground motion spectrum by the amplification factors suggested by Newmark [6] for the assumed value of the damping ratio, ξ. The inelastic spectrum is then constructed by dividing the elastic spectrum by appropriate functions of the assumed displacement ductility [6]. The elastic and inelastic response spectra for $\xi = 5\%$ and $\mu = 6$ are shown in Fig. 5. Maximum response parameters and the range of periods over which the established limitations on C_y and R_{ult} can be satisfied may be obtained from the first period mode shape and the assumed value of T_1, as indicated below.

The maximum lateral displacement,

$$Y_1 = \frac{\underline{M}\,\underline{I}\ \underline{\phi}_1}{\underline{\phi}_1^T\ \underline{M}\underline{\phi}_1} \cdot S_{d_{inelastic}} = \frac{L_1^*}{M_1^*} \cdot S_{d_{inelastic}} \tag{1}$$

can be used to give an idea of the expected story drift index

$$R_1 = \frac{Y_1}{H} = \frac{L_1^*}{M_1^* H} \cdot S_{d_{inelastic}} = 0.00092\ S_{d_{inelastic}} \tag{2}$$

where H is the total structure height.

The base shear,

$$V_1 = \frac{(L_1^*)^2}{M_1^*} \cdot PS_{a_{inelastic}} = C_1 \cdot W_{1_{effective}} \quad (3)$$

can be used to give an idea of the expected seismic coefficient,

$$C_1 = \frac{V_1}{W_{1_{effective}}} = \frac{PS_{a_{inelastic}}}{g} \quad (4)$$

From Fig. 5 it can be seen that the design requirements $C_y \leq 0.2$ and $R_{ult} \leq 0.015$ can be satisfied for the following range of T_1.

$$0.67 \text{ sec} \leq T_1 \leq 2.6 \text{ sec}$$

A similar check could have been performed for the serviceability limit state if a design spectrum had been established for this limit state.

For the assumed value of $T_1 = 1.0$ sec, the spectrum of Fig. 5 gives $C_1 = 0.12$ and $R_{1,ult} = 0.0063$. Although the value of C_1 is considerably lower than the acceptable limit of 0.20, the higher modes will increase the response, and the current values of T_1, μ, and ξ can be accepted for carrying out the preliminary design.

Estimation of lateral story shears--The displacement and base shear modal participation factors, λ_i^y and λ_i^v respectively, can be estimated from the selected mode shapes by the expressions:

$$\lambda_i^y = \frac{L_i^*}{M_i^*} \quad (5) \qquad \text{and} \qquad \lambda_i^v = \frac{(L_i^*)^2}{M_i^*} \quad (6)$$

The contributions of each mode to the maximum displacement and base shear are then found, using the selected values of T_1/T_i and T_1, from the expressions:

$$Y_i = \lambda_i^y \cdot (S_{d_{inelastic}})_i \quad (7) \qquad \text{and} \qquad V_i = \lambda_i^v \cdot (PS_{a_{inelastic}})_i \quad (8)$$

By examining these modal contributions, the number of modes contributing significantly to the response may be determined. For the design example, only the first three modes were significant.

Once the story shears for each of the significant modes are computed, the maximum probable story shear, $S_{j_{max}}$ is estimated by computing the square root of the sum of the squares of the modal maxima (SRSSMM).

Although in this example it was assumed that μ_j was constant throughout the height of the building, it would generally be more rational to use different values for the ductility through the height. This is because the state of stress in the girders at the upper stories usually permits the development of large ductility, and the consequences of large story drifts are less detrimental in the upper than in the bottom stories.

The P-Δ effect has been included in the design forces by estimating an additional story shear

$$(\Delta S_{P-\Delta})_j = P_j \cdot \frac{\delta_j}{h_j} \tag{9}$$

where P_j is the total dead load + the reduced live load of levels above level j; δ_j is the maximum relative story deflection at level j (this value should be estimated considering the expected inelastic response which depends on the value of μ_j at that story); and h_j is the story height of level j.

For the design example, the values of δ_j/h_j were assumed constant and equal to:

$$\frac{\delta_j}{h_j} = \frac{\Delta_{roof}}{H} \tag{10}$$

where Δ_{roof} is the square root of the sum of the squares of modal maximum displacements at the roof, and H is the total height of the frame.

The final design story shears are obtained from the expression:

$$S'_{j_{max}} = S_{j_{max}} + (\Delta S_{P-\Delta})_j \tag{11}$$

The $S'_{j_{max}}$ obtained for the design example are shown in Fig. 6.

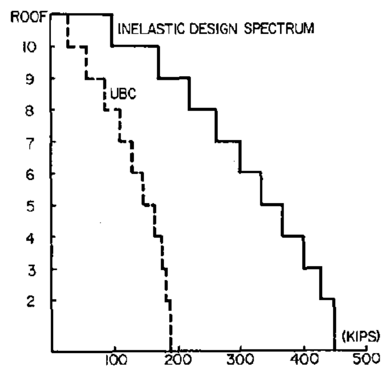

FIG. 6 DESIGN SEISMIC SHEAR FORCES

PRELIMINARY DESIGN

The basic problem in this step of the procedure may be stated as follows:

Given:

1. Gravity and wind loads
2. Seismic lateral story shears obtained in the preliminary analysis (Fig. 6).

3. Critical load combinations. Those considered were
   0.75 (1.4 DL + 1.7 LL + 1.7 W)
   1.4 DL + 1.7 LL
   1.2 DL + 1.0 LL ± 1.0 E
   0.8 DL ± 1.0 E

4. Mechanical characteristics of the construction materials. The nominal compressive strength of concrete was taken as 4000 psi, and the nominal yield strength of the reinforcement was taken as 60000 psi.

Find:

The sizes of beams and columns as well as the distributions of beam flexural reinforcement and column longitudinal reinforcement.

This problem is solved by a simplified story-wise weak girder-strong column optimum limit state design.

Design subassemblage--The single-story subassemblage used in the preliminary design is shown in Fig. 7. Use of this subassemblage and the weak girder-strong column design criterion simplifies the design problem because it reduces the number of design variables to the selected girder moments in a given story. In a typical intermediate story the use of this subassemblage is justified by the presence of large seismic shear forces which force the column inflection points to be very close to mid-height. In the design procedure, both the negative and positive design moments at a given section are considered independent design variables. If the design moments are assumed to be symmetric about mid-span of the center bay, 8 independent design moments may be identified (Fig. 7). Determining the optimum value of these design moments is the objective of the optimization procedure presented below.

Design procedure--Linear programming techniques are used to obtain an optimum inelastic design. The optimization process attempts to minimize the volume of flexural reinforcement. Equilibrium constraints obtained from the kinematic theorem of simple plastic theory form a physical basis for the optimization, with additional constraints imposed to satisfy serviceability, as well as practical, requirements.

More realistic objective functions than the volume of flexural reinforcement (such as the total cost of construction which would include the cost of concrete, steel reinforcement, and form work [8]), might be formulated as part of the optimization procedure. However it is generally difficult to formulate realistic linear relationships between cost variables and design variables (the beam moment capacities), and since an approximate linear relationship between the area of steel and the design moment capacity exists, the volume of flexural reinforcement was chosen as the

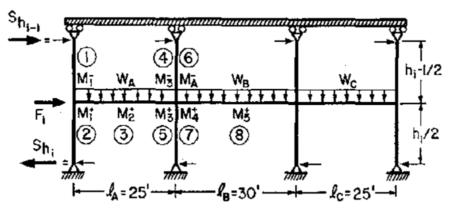

FIG. 7 SUBASSEMBLAGE FOR PRELIMINARY DESIGN

objective (merit) function. The possibility of considering a nonlinear total cost function by employing a different mathematical programming technique, such as the method of feasible directions used by Walker and Pister [9], should be studied.

Starting design--Elastic analyses for the service and ultimate load states must be carried out before starting the above design procedure because: 1) the merit function and a set of practical constraints are based on ultimate load elastic moment envelopes; and 2) the serviceability constraints are based on the service load envelopes. As a result a starting preliminary design (starting relative sizes of members) is needed in order to carry out these elastic analyses. This presents a problem. How can a good starting design be obtained in the first iteration of the design process? Although upper and lower bound approaches can be used, the following procedure is suggested.

1. Assume that the moment capacity in a given span is constant.

2. Formulate an optimization problem based only on the equilibrium constraints at collapse, thus eliminating the need for elastic analyses.

3. Use the computed moment capacities to size the beams.

4. Use beam capacities found in 3 to size the columns to insure a weak girder-strong column design.

The sizes of beams and columns in all stages of the design process, were based on the permissible percentage of reinforcement, ρ, bounded as follows. In the beam design:

$$200 \leq \rho \leq 0.025 \text{ or } \leq 0.75\ \rho_b\text{, whichever is smaller}$$

The lower bound is a code requirement for the minimum amount of flexural reinforcement. The upper limit of 0.025 is that recommended by the UBC (2626) requirements for ductile moment-resisting reinforced concrete space frames. The upper limit of 0.75 ρ_b is that recommended by the UBC (2610) requirements for the design of flexural members. In this design example the upper bound, $\rho \leq 0.75\ \rho_b$, controlled the design.

In the column design:

$$0.01 \leq \rho \leq 0.04$$

The upper limit is chosen to obtain more ductile (tough) columns and also to relieve congestion of reinforcement at beam-column joints. The lower bound is a code requirement.

Details of the design relationships used and the relationship between beam moment capacities and column design moments will be presented after a discussion of the optimization procedure.

Formulation of the design problem--In order to use a linear programming technique to obtain the stated optimization objective, it is necessary to formulate a linear function in the desired moment capacities which is proportional to the volume of flexural reinforcement, (Vol). To obtain such a function it is assumed that the moment capacity and area of steel are related by the expression:

$$M_i = A_{s_i} \cdot f_y \cdot jd \tag{12}$$

Consequently the merit or objective function may be expressed as:

$$Vol \sim \Sigma\, M_i\, \gamma_i \tag{13}$$

where γ_i is the effective length over which area A_{s_i} is required.

The quantity γ_i is an effective length because it includes required development lengths at columns and the effect of bar cutoffs. As a consequence it is dependent on the bar size used in design. Typically larger bars will result in larger values of γ_i. In order to arrive at the smallest amount of reinforcement and also to minimize significant bond deterioration use of the smallest possible bar size is recommended.

The contribution of column reinforcement to the volume of flexural reinforcement should be considered in construction of the merit function. Since a weak girder-strong column design criterion is imposed, the sum of the column moment capacities at a given joint can be expressed in terms of the beam moment capacities at that joint. This expression can be multiplied by an appropriate length factor and added to the merit function. This length factor should include the effect of axial load on the column moment capacity and also any slenderness effects which may be considered in the actual member design.

A typical merit function then consists of two components; the beam contribution, γ_i*, and the column contribution, γ_i**. An investigation into the formulation of the γ_i* component indicated that the solution of the optimization problem is sensitive to the size of the bar used to define development lengths, particularly in the intermediate and lower stories. As a result, the construction of the merit function warrants further investigation.

The equilibrium constraints used in this design example can be obtained from the mechanisms given in Fig. 8 and are represented by the expression

$$\alpha_{ji}\, M_j \geq \omega_i \tag{14}$$

where α_{ji} is the coefficient of the jth design moment, M_j, in the ith equilibrium constraint and ω_i is the work done by the external forces in the ith equilibrium constraint.

The serviceability constraints place a lower bound on the design moment capacities. The lower bound used in this design procedure is one suggested by Cohn [3] and it is imposed to prevent yielding, wide cracking, and large deflection under service load conditions.

$$|M_j| \geq \lambda_o |M_j^{SE}| \tag{15}$$

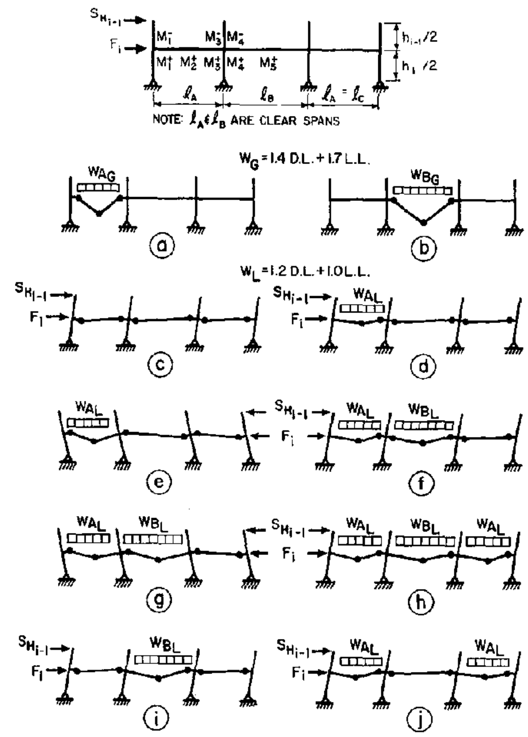

FIG. 8 MECHANISMS CONSIDERED IN FORMULATION OF EQUILIBRIUM CONSTRAINTS

where M_j is the jth design moment, M_j^{SE} is the ordinate of the elastic moment envelope under service load conditions and λ_0 is a factor that depends on the serviceability requirements (a value of 1.2 was used)

The remaining constraints are imposed to meet code requirements, to achieve a desired inelastic redistribution of moments and to limit detailing problems, thus arriving at a practical design. The adopted optimization procedure has the advantage of providing an experienced designer opportunity to use additional constraints based on his many years of design experience. The following constraints were used in the design example.

At the beam ends

$$0.5|M_j^-| \leq |M_j^+| \qquad |M_j^+| \leq |M_j^-| \tag{16}$$

This constraint bounds the positive moment capacity at a given support section with respect to the negative moment capacity at that section. The lower bound is based on code requirements (UBC 2626). It not only recognizes the severity and cyclic (with reversal) characteristics of the seismic excitation, but also represents an attempt to include in member design the beneficial effect of compressive reinforcement on the inelastic deformation characteristics of the member. The upper bound is based on practical considerations.

At beam midspan

$$|M_{span}| \geq 0.25|M_{support}| \tag{17}$$

This constraint is based on code requirements (UBC 2626 (ℓ)2) that at least one quarter of the larger amount of support reinforcement be continued through the girder.

$$|M_j| \leq |M_j^{UE}| \qquad (18)$$

where M_j^{UE} is the ordinate of the elastic moment envelope for the ultimate load condition at section j.

This constraint represents an upper bound on the design moment capacity and was imposed only on the negative support moment capacities. It was imposed to relieve steel congestion at the beam-column joint.

$$|M_j^-| \geq FAC \cdot |M_{j_{above}}^-| \qquad (19)$$

where FAC is the ratio $M_j^{-UE}/M_{j,above}^{-UE}$ and $M_{j,above}^-$ is the design moment obtained in the solution of the optimization problem for the story above the current story.

This constraint represents an attempt to arrive at a smooth transition in member strength and stiffness through the height of the structure.

A last practical constraint is imposed in conjunction with the serviceability constraint. If the product $\lambda_o\ M_j^{SE}$ was less than the moment capacity corresponding to the minimum allowable percentage of reinforcement ($M_{\rho,min}$), then the latter value was used as a lower bound constraint.

Summary of optimization problem--The optimization problem may be summarized as follows.

Find $M_j > 0$ $\qquad$ j = 1, number of desired moment capacities (N)

which satisfy:

a) equilibrium constraints:

$$\alpha_{ji}\ M_j \geq \omega_i \qquad \begin{array}{l} j = 1,\ N \\ i = 1,\ \text{number of equilibrium constraints (NEQ)} \end{array}$$

b) serviceability constraints

$$|M_j| \geq m_j \qquad j = 1,\ N$$

where m_j is the larger of $|\lambda_o\ M_i^{SE}|$ or $|M_{\rho,min}|$

c) practical constraints

$$0.5|M_j^-| \leq |M_j^+| \leq |M_j^-|$$

$$|M_{span}| \geq 0.25|M_{support}|$$

$$|M_j^-| \leq |M_j^{-UE}|$$

$$|M_j^-| \geq FAC \cdot |M_{j_{above}}^-|$$

and minimizes the linear function

$$\gamma_j M_j \geq 0 \qquad j = 1, \ldots, N$$

An optimization problem is formulated for each story and then solved with the aid of a standard Simplex algorithm. The 'optimum' girder moment capacities, M_j, obtained are then used to design the frame members.

Member design--The beams and columns were designed using available computer programs based on the following design relationships.

a) beam design

The beams were designed according to the equation:

$$\frac{M_u}{\phi} = bd^2 \{\rho f_y[1 - (\frac{f_y}{f'_c}) 0.59 \rho]\} \tag{20}$$

M_u is the optimized beam design moment multiplied by an amplification factor, α_F, to account for column slenderness effects. The value of α_F is determined from column moment amplification factors corresponding to member sizes of the starting design.

After the girders have been designed, their moment capacities are evaluated and used as the basis of column design.

b) column design

The columns are designed to insure that the weak girder-strong column criterion is satisfied. The following expression, derived from equilibrium at a typical beam-column joint, is used to relate column design moments to beam moment capacities (Fig. 9).

$$M_c^T \cdot f(\ell_c^T, h_b) + M_c^B \cdot f(\ell_c^B, h_b) > F[f(M_b^D) + f(\frac{M_b^D}{\ell_b}, h_c) + f(W_b\ell_b, h_c)] \tag{21}$$

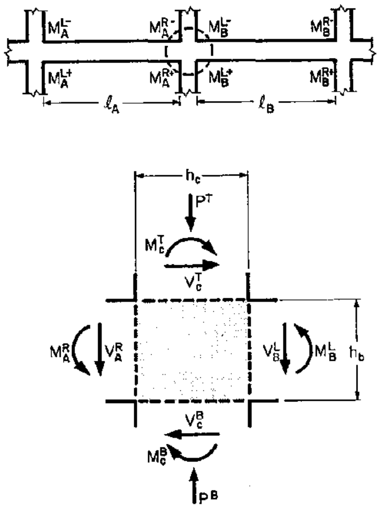

FIG. 9 FORCES AT A TYPICAL BEAM COLUMN JOINT

where the subscript c indicates column and b indicates beam.

The 3 functions on the right-hand side of the above inequality should be chosen to give the maximum possible column design moments. That is, all possible combinations should be considered, and the one which results in the largest value for the right-hand side should be selected. It should be noted that slenderness effects are included in the above expression by the use of amplified beam moments.

The following example will illustrate how the above functions might look.

Consider an exterior beam-column joint (Fig. 10). For simplicity it is assumed that in span A there are two design moments, M_1 and M_2. In other words, the positive and negative moment capacities are equal. The joint equilibrium equation is:

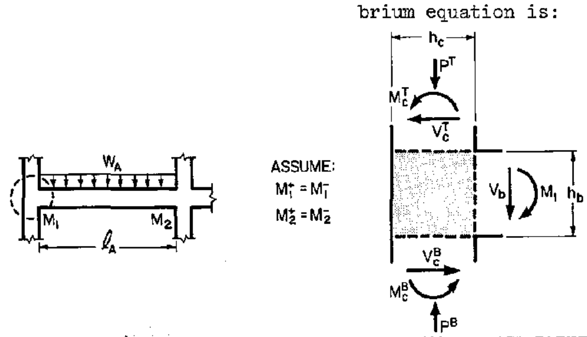

FIG. 10 EQUILIBRIUM AT AN EXTERIOR BEAM COLUMN JOINT

$$M_c^T + V_c^T \frac{h_b}{2} + M_c^B + V_c^B \frac{h_b}{2} = M_1 + V_b \frac{h_c}{2} \tag{22}$$

From the equilibrium of beam A:

$$V_b = \frac{W_A \ell_A}{2} + \frac{|M_1| + |M_2|}{\ell_A} \tag{23}$$

If the column inflection point is assumed at mid-column height:

$$V_c^T = \frac{2M_c^T}{\ell_c^T} \quad \text{and} \quad V_c^B = \frac{2M_c^B}{\ell_c^B} \tag{24}$$

where ℓ_c^T and ℓ_c^B are the clear story heights above and below a joint.

If we now assume that $\ell_c^T = \ell_c^B = \ell_c$, Equation (22) can be written as:

$$\left(\frac{\ell_c + h_b}{\ell_c}\right) \sum M_c = |M_1| + \frac{|M_1| + |M_2|}{2\ell_A} h_c + \frac{W_A \ell_A}{2} h_c \tag{25}$$

and

$$f(\ell_c^T, h_b) = f(\ell_c^B, h_b) = \frac{\ell_c + h_b}{\ell_c}; \; f(M_b^D) = |M_1|$$

$$f\left(\frac{M_b^D}{\ell_b}, h_c\right) = \frac{h_c}{2\ell_A} \left(|M_1| + |M_2|\right) \; ; \; f(W_b \ell_b, h_c) = \frac{W_A \ell_A}{2} h_c$$

In the column design, the reinforcement which defines the column moment capacity is chosen to be the same above and below a joint. That is, all column splices occur near mid-column height. If the lengths and boundary conditions of the top and bottom columns at a given joint are assumed to be the same, then the distribution of ΣM_c^D between the top and bottom columns can be based on the following equation $M_c^T = \beta M_c^B$ where β is a moment distribution factor obtained from the distribution of elastic stiffness at the beam-column joint.

When a change in column section occurs through a joint, each section is designed to resist its own critical load combinations, and the larger amount of reinforcement obtained is used for both columns.

In order to consider all critical column load combinations, a bound on the axial force due to overturning effects (tension or compression) must be obtained. For an exterior column such a bound is easily established. The maximum beam shears induced by lateral shear forces can be expressed in terms of the beam design moments by: (See Fig. 7).

$$V_{tension} = \frac{|M_1^+| + |M_3^-|}{\ell_A}$$
$$V_{compression} = \frac{|M_1^-| + |M_3^+|}{\ell_A} \tag{26}$$

The bound for a given story is obtained by summing the maximum beam shear for that level with those for all higher levels.

Such bounds cannot be so easily obtained for an interior column, however, and the overturning moment axial forces obtained in the ultimate load elastic analysis used to formulate the preliminary design problem were considered in the interior column design.

In the column design, the most critical combination of axial load and bending moment was considered. The moment corresponding to the code minimum eccentricity[1] was also checked to see if it controlled the design. The computer program used to design the columns was based on the ACI ultimate strength design code [10]. A standard capacity reduction factor of 0.7 was used, in the column design and this value was increased for small axial loads as allowed in UBC 2609(c) 2D. The selection of column size was guided by the desire to keep column axial loads below the balance point of the axial force moment interaction relationship. This requirement would provide some ductility in case the column should yield.

Final remarks on preliminary design--Preliminary design involves two iterative loops. The first loop is illustrated in Fig. 2. Iteration is required because initial member sizes are needed to formulate the optimization problem, the solution of which will generally result in different member sizes.

A second iterative loop exists in the actual design. Column design moments depend on beam moment capacities, which depend on the moment amplification factors due to slenderness, which depend on column sizes. Since designed members generally will have different column sizes than those used in evaluating the initial slenderness effects, iteration is required to arrive at a design in which the initial and final moment amplification factors are in close agreement.

Results of Preliminary Design

Results of the preliminary design are summarized in Figs. 11-14. The design defined by these figures was obtained after two iterations of the optimization procedure (Fig. 2). The beam design moments obtained from the optimization problem are shown in Fig. 11 and the resulting member sizes are shown in Fig. 12. Selection of member sizes was based on the design criterion of a smooth transition in stiffness through the height of the structure with consideration of the economics involved in formwork changes. The final beam moment capacities[2] are shown in Fig. 13 and the variation of column reinforcement is shown in Fig. 14.

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[1] UBC 2610(d) 6 specified $e_{min} > 1$ in. or, for a tied column, $e_{min} > 0.1\ h$, where h is the larger column dimension.

[2] The T-beam effect of the slab on beam moment capacity was not considered.

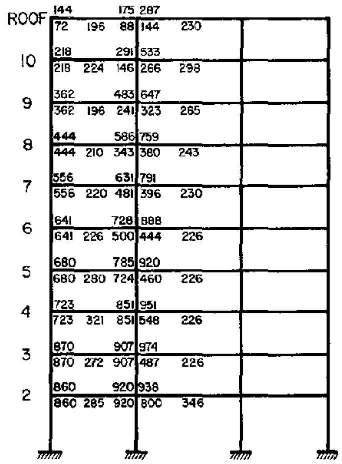


FIG. 11 OPTIMUM BEAM DESIGN MOMENTS (K-FT)

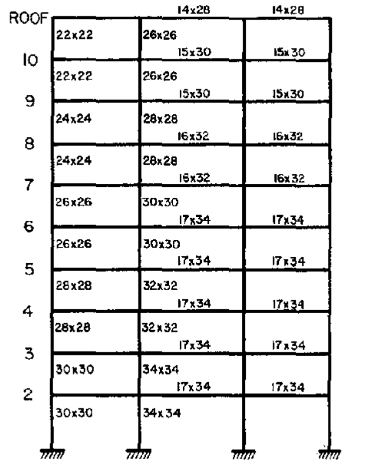


FIG. 12 MEMBER SIZES (INCHES)

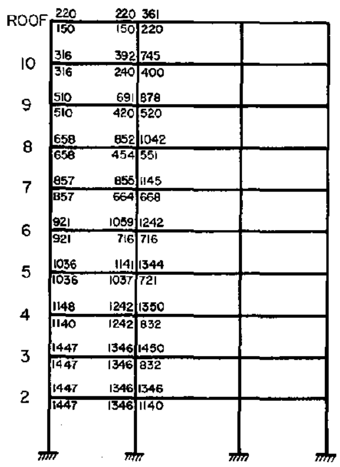


FIG. 13 AS DESIGNED BEAM MOMENT CAPACITIES (K-FT)

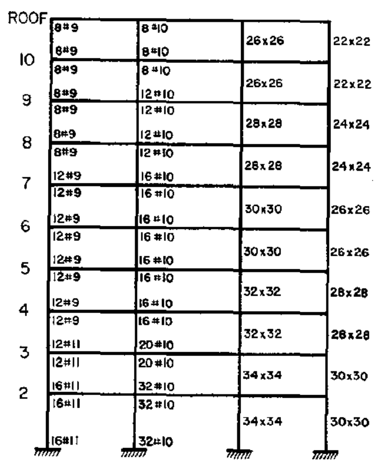


FIG. 14 COLUMN SIZES AND REINFORCEMENT

The significant difference between the preliminary optimized moments (Fig. 11) and the final beam moment capacities (Fig. 13) is due to three factors. First, a code capacity reduction factor for flexural members of 0.9 was used in member design. Second, the beam moments shown in Fig. 11 were amplified to account for column slenderness effects. Amplification ranged from 1.06 in the upper stories to 1.25 in the lower stories. Finally the use of practical formwork sizes and a limited number of available bar sizes caused the selected beam size and reinforcement arrangements to provide moment capacities greater than actually required.

Analyses of Preliminary Design

Results of elastic analyses--An elastic frequency analysis of the designed structure resulted in a first mode period of 1.21 sec, in good agreement with the value of 1 sec assumed in determining the seismic design forces. The period ratios (T_1/T_i) and the mode shapes (ϕ_i) obtained in this analysis were also similar to those chosen initially. In this frequency analysis and subsequent elastic and nonlinear static and dynamic analyses, the effect of the floor slab on the frame's stiffness was included by using a method proposed by Edgar and Bertero [11].

The results of an elastic analysis for service load conditions (D.L. + L.L. + W.L.) yielded a maximum story drift index of 0.00026, which is well under the established design criteria of 0.002.

The nonlinear static behavior of the designed frame was investigated using a modified version of the program ULARC (12). The frame was subjected to the design gravity loads and a monotonically increasing seismic base shear which was distributed through the height of the frame according to the lateral force pattern obtained from the spectral modal analysis.

Results of the nonlinear static analysis are summarized in Figs. 15-18. Variations of roof and first story lateral displacements with the increasing value of the base shear are shown in Fig. 15. Two analyses were carried out in order to investigate the P-Δ effect. A comparison of the two roof displacement responses demonstrates the 'negative stiffness' contribution of the P-Δ effect. This is particularly evident at large displacements, 25-38 cm (10-15 in.) where the applied lateral force decreases with increasing displacement.

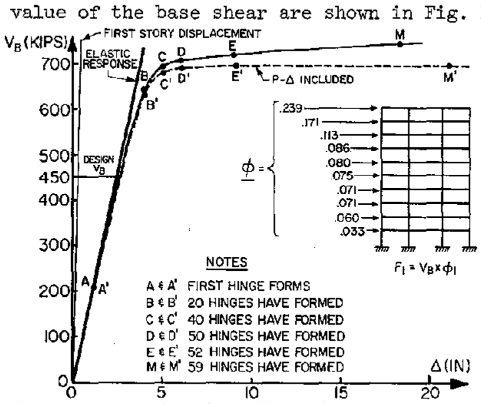


FIG. 15 LATERAL LOAD-DISPLACEMENT RELATIONSHIP

A significant overstrength is observed whether or not the P-Δ effect is included. For the case in which the P-Δ effect is considered the maximum base shear was greater than the design base shear by 55%.

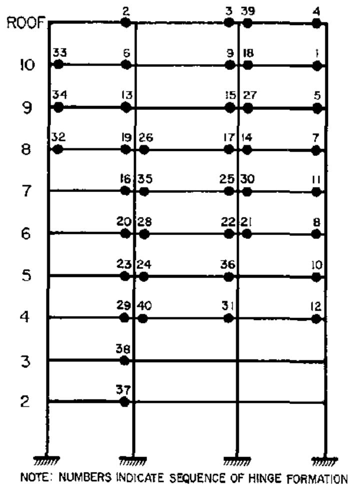


FIG. 16 PLASTIC HINGE PATTERN
AT STAGES C-C' IN FIG. 15

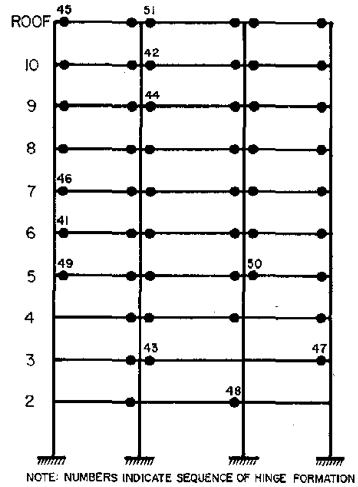


FIG. 17 PLASTIC HINGE PATTERN
AT STAGES E-E' IN FIG. 15

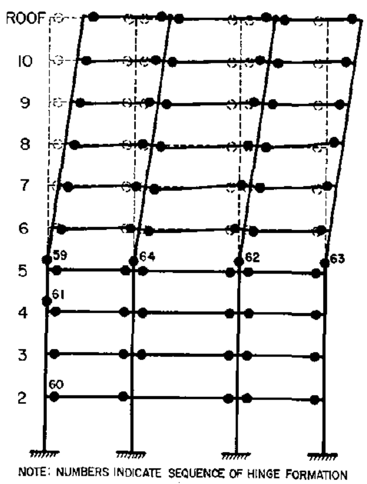


FIG. 18 PARTIAL SWAY MECHANISM

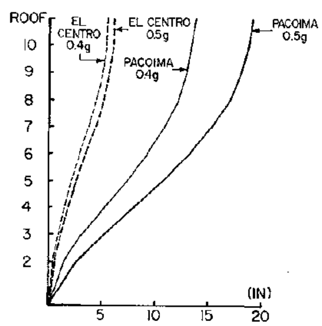


FIG. 19 ENVELOPE OF MAXIMUM
FLOOR DISPLACEMENT

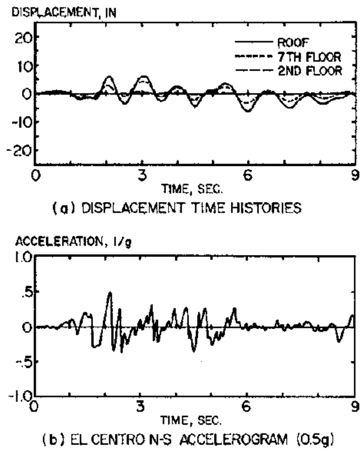


FIG. 20 STORY DISPLACEMENT TIME HISTORIES - EL CENTRO MOTION

Two factors contribute to this overstrength. First the final beam strengths were, as discussed previously, significantly larger than required (Figs. 11 & 13). Second, the design forces were based on the assumption that the entire frame would be transformed into a mechanism simultaneously. An examination of the sequence of hinge formation indicated that this did not occur. The first plastic hinge formed at stage A-A' which corresponds to a base shear of approximately one half the design value. Subsequent hinge formation was gradual and is depicted in Fig. 15.

Changes in the deformation pattern through the height of the frame with increasing roof displacement level and the associated plastic hinge patterns are of interest. From Fig. 15 it is evident that the first story displacement response remained essentially elastic throughout the analysis. This is consistent with the observed hinge formation sequence (Figs. 16 & 17). Plastic hinges gradually progressed downward. Consequently the initial elastic stiffness of the lower stories is affected at a much later stage than the stiffness in the upper stories. In this particular example the sequence of hinge formation is such that the stiffness of the first story remains essentially unchanged.

In the static analysis yielding was limited to the beams until point M-M' at which point the columns of the fifth story yielded (Fig. 18). Subsequent increases in displacement resulted in a partial sway mechanism in this story at a roof displacement of 68 cm (27 in.). Column yielding is due to a change in the distribution of the beam moments between the columns above and below a typical beam column joint. This change, caused by beam yielding, was such that almost the entire joint moment was resisted by one column.

The column yielding demonstrates the difficulty in insuring an efficient weak girder-strong column design. Overdesigning the columns by a factor of 1.7 with respect to the beams failed to prevent column yielding prior to complete beam yielding. As is discussed in subsequent sections, this problem becomes more prominent in the dynamic response.

Results of nonlinear dynamic analyses--The nonlinear dynamic response of the designed structure to the El Centro N-S component and the Derived Pacoima Dam ground motions was obtained using SERF, a program developed by Mahin and Bertero [13]. The accelerations of both ground motions were scaled to have peak values of 0.4g and 0.5g. It should be noted that while the El Centro ground motion had dynamic characteristics represented by the ground spectrum selected as the design earthquake the Derived Pacoima Dam did not [14]. In the analyses the following assumptions are introduced:

1. Rayleigh-type damping with a 5% damping ratio in the first two modes.

2. Both the beams and columns have a bilinear M-ϕ relationship with linear strain hardening. Strain hardening values of 5% and 2% were investigated to evaluate the influence this parameter had on the response.

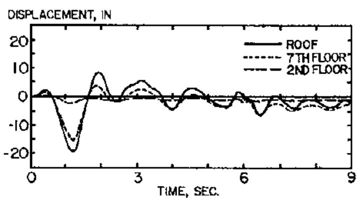
3. Column yielding is determined from the corresponding axial force bending moment interaction.

4. P-Δ effect is included.

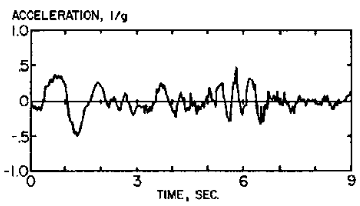
5. Beam column joints are rigid.

Based on the results for 5% strain hardening the following observations are made.

1. There was a significant difference in response for the two ground motions considered. Though the maximum ground acceleration of the two input motions was the same, the maximum displacements during the response to the Derived Pacoima ground motion were approximately three times those recorded during the El Centro motion (Fig. 19). This demonstrates the need to consider all possible ground motions at a given site and also all characteristics of these ground motions (not just the peak ground acceleration) when selecting a design earthquake [14]. The long duration pulses in the initial portion of the Derived Pacoima ground motion caused severe inelastic behavior in one direction (Fig. 21). The displacement time histories at various floor levels are shown in Figs. 20 and 21.



(a) DISPLACEMENT TIME HISTORIES



(b) DERIVED PACOIMA DAM ACCELEROGRAM (0.5g)

FIG. 21 STORY DISPLACEMENT TIME HISTORIES - DERIVED PACOIMA DAM MOTION

Results of the nonlinear static analysis may be used to obtain an estimate of the dynamic floor displacement ductilities. The static base shear-roof displacement relationship is reproduced in Fig. 22. If the first significant lateral yield displacement of the frame is defined by the Δy_{roof} indicated in Fig. 22 and the displacement pattern corresponding to this roof displacement is assumed as the yield displacement pattern, the floor displacement ductilities given in Table 1 may be computed. From these ductility data it is evident that the Derived Pacoima ground motion causes significantly (three times) larger inelastic deformations than the El Centro ground motion. In addition it appears that the displacement ductility assumed in determining the seismic design forces is exceeded during the response to the Derived

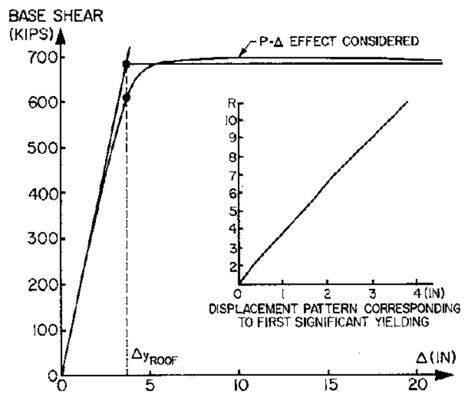


FIG. 22 DEFINITION OF LATERAL YIELD DISPLACEMENT UNDER STATIC LATERAL LOADING

Pacoima ground motion. It should be noted, however, that the displacement ductilities given in Table 1 are just estimates because of the different nature of the responses during which the yield and maximum displacements were determined and these ductility demands should be used as guideline values only.

2. Comparison of the envelopes of actual and design story shears (Fig. 23) indicates that the dynamic story shears are greater than the design forces. This is due to a number of factors. First, the members had been overdesigned and an increase in shear capacity had already been revealed by the static analysis. Second, strain hardening increased the member capacities. Third, the story shear forces given for the dynamic response were absolute maximum values at each story and did not occur simultaneously during the response. Finally, the design shear forces were determined based on the assumption that the entire frame was transformed into a

TABLE 1. FLOOR DISPLACEMENT DUCTILITIES

FLOOR	LATERAL DISPL. AT FIRST SIGNIFICANT YIELDING Δ_y (in.)	LATERAL DISPLACEMENT DUCTILITIES, μ, UNDER			
		DERIVED PACOIMA		EL CENTRO	
		0.5g	0.4g	0.5g	0.4g
ROOF	3.79	5.04	3.63	1.64	1.47
10	3.41	5.49	3.92	1.76	1.58
9	3.00	6.24	4.27	1.90	1.67
8	2.58	6.57	4.65	2.00	1.71
7	2.18	7.02	4.95	2.04	1.69
6	1.80	7.30	5.10	2.00	1.62
5	1.43	7.44	5.10	1.90	1.52
4	1.07	7.33	4.86	1.76	1.41
3	.70	7.10	4.50	1.61	1.31
2	.33	7.20	4.24	1.53	1.34

mechanism simultaneously. As in the static analysis, this did not occur in the dynamic response either. Detailed analysis of plastic hinge formation during the dynamic response to ground motion indicates that there is a migration of plastic hinges from the base to the top of the building, and that as plastic

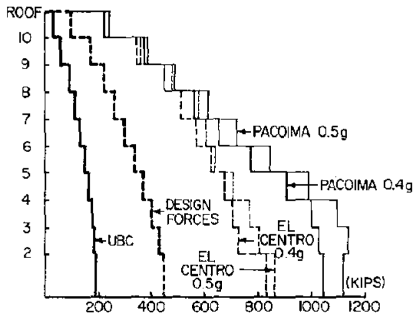


FIG. 23 ENVELOPES OF STORY SHEARS

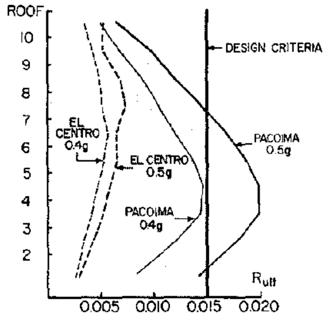


FIG. 24 ENVELOPES OF STORY DRIFT INDEX, R_{ult}

hinges form in the upper stories the ones in the lower stories close.

3. Comparison of the story shear envelopes for the two ground motions considered (Fig. 23) indicates that the Pacoima ground motion resulted in larger story shears, particularly in the lower stories. This difference is attributed to strain hardening considered in the assumed M-ϕ relationship for the member. Strain hardening increased the beam moment capacities beyond those computed on the basis of the yielding of the reinforcement. Since the columns were overdesigned with respect to the beams (by a factor of about 1.7), story shears greater than those corresponding to the designed beam capacities could result. The increase in beam moment capacity associated with strain hardening is directly proportional to the magnitude of inelastic deformation. Since the inelastic deformations recorded during the Pacoima response were larger than those for El Centro, the increase in beam moment capacity was greater during the Pacoima response. Since the story shears are proportional to the beam moment capacities, greater story shears were recorded during the Pacoima response.

4. Envelopes of the story drift index (Fig. 24) indicate that the design criterion for R_{ult} is violated during the response to the Pacoima ground motion with a maximum acceleration of 0.5 g. The large story drifts demanded by the Pacoima ground motion indicates a high probability of severe nonstructural damage.

5. Examination of the required column ductilities (Figs. 25-26) indicates that the weak girder-strong column design criterion is satisfied during the El Centro response. However, column yielding does occur at various locations during the response to the Pacoima ground motion, and is attributed in part to the increase in beam moment capacities caused by strain hardening. In addition, the distribution of the beam moments between the column sections above and below a given joint was typically different from that assumed in design. Not only are the end conditions of the upper and lower column at a given joint not the same as assumed in the preliminary design, but also the formation of beam plastic hinges above, below, and at a given joint can alter the moment distribution at that joint to the extent that the sum of the beam moment capacities

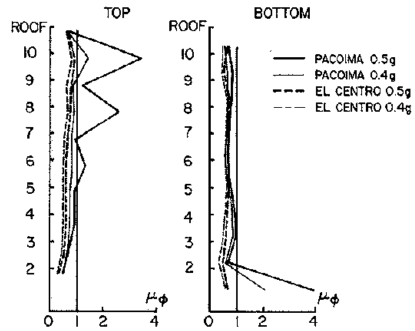


FIG. 25 MAXIMUM COLUMN CURVATURE DUCTILITY, μ_ϕ, INTERIOR COLUMN

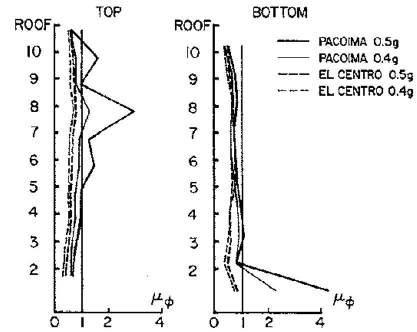


FIG. 26 MAXIMUM COLUMN CURVATURE DUCTILITY, μ_ϕ, EXTERIOR COLUMN

is resisted by only one of the columns at the joint.

An attempt is made in the final design to eliminate the problem of different column end conditions by designing the columns based on a moment distribution found from an elastic analysis.

The magnitude of the inelastic column deformation (plastic hinge rotation) in the upper stories was small, being less than 0.003 radians. The accumulated rotation was essentially the same as the maximum value indicating one large inelastic excursion. The maximum plastic rotation in the columns at the ground level was approximately 0.01 radians and could be tolerated with proper detailing.

6. The effects of the different ground motions on the individual members are illustrated by the beam inelastic deformation behavior. For the El Centro ground motion, the maximum cyclic curvature ductility is less than 9 (Fig. 27). However, in the response to the Pacoima ground motion, cyclic ductility values greater than 17.5 were obtained. This trend is also indicated by the maximum plastic hinge rotations, $\theta_{p,max}$ and the accumulated plastic hinge rotations $\theta_{p,acc}$ (Figs. 28 and 29) where

$$\theta_{p_{acc}} = \sum_N |\theta_p| \qquad N = \text{Number of yield excursions}$$

For the El Centro ground motion $\theta_{p,max}$ was 0.0065 radians and $\theta_{p,acc}$ was 0.033 radians while during the Pacoima ground motion the values were 0.020 and 0.060 radians, respectively. The large difference

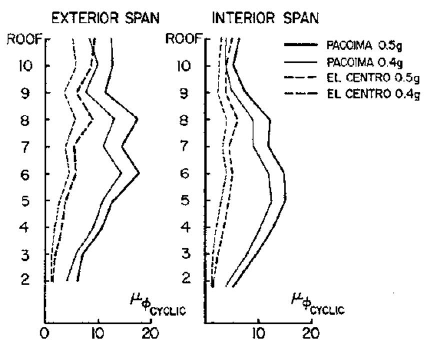


FIG. 27 MAXIMUM BEAM CYCLIC CURVATURE DUCTILITY, $\mu_{\phi_{cyclic}}$

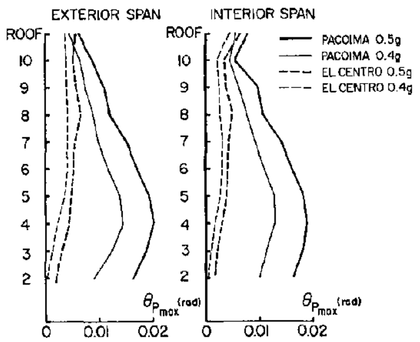


FIG. 28 MAXIMUM BEAM PLASTIC ROTATIONS, $\theta_{P_{max}}$

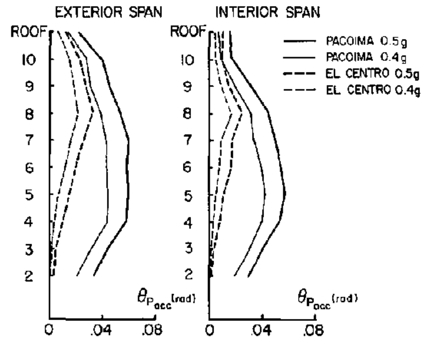


FIG. 29 ACCUMULATED BEAM PLASTIC ROTATIONS, $\theta_{P_{acc}}$

in $\theta_{p,max}$ between the two ground motions demonstrates the effect of the long acceleration pulses contained in the Pacoima ground motion on the structure response. Because of the severe long duration pulses the structure was forced to deform in one direction for a long period of time (0.56 sec) during which it experienced large inelastic deformations.

Comparison of the plastic hinge rotation demands with experiemental results obtained by several investigators [15, 16] indicates that the frame should be able to resist the El Centro earthquake safely. However, because of large inelastic deformation requirements, the current structure might experience severe structural damage and possible structural failure if it were excited by a ground motion with characteristics similar to the Pacoima record.

7. The ratios of maximum story displacements, story drift indices, story shears, and maximum beam plastic hinge rotations recorded at peak ground accelerations of 0.5g and 0.4g are compared for the two ground motions in Table 2. An examination of the data in Table 2 indicates that the effect on the story shear of increasing the ground acceleration is approximately 10% for each of the ground motions and is mainly due to an increase in the beam moment capacities caused by the larger amount of strain hardening which occured when the peak acceleration is increased from 0.4g to 0.5g.

From the data in Table 2 it is evident that the effect on the floor displacement response of increasing the ground acceleration is significantly different for the two ground motions. For the Pacoima ground motion the floor displacements typically increased by 40% as the ground acceleration was increased from 0.4g to 0.5g. This is approximately two times the increase observed for the El Centro ground motion. This difference is attributed to the degree of inelastic behavior caused by the Pacoima ground motion, in particular yielding in the lower story columns.

8. The extent of the effect of the strain hardening ratio varied between the two ground motions, being more significant during the response to the Pacoima ground motion. This is expected, however, since the inelastic deformations during the response to this ground motion were substantially

larger than those which occurred during the El Centro ground motion.

TABLE 2. RATIOS OF MAXIMUM VALUES FOR DIFFERENT RESPONSE PARAMETERS FOR 0.5g AND 0.4g PEAK GROUND ACCELERATIONS

FLOOR	EL CENTRO				PACOIMA			
	FLOOR DISPL.	DRIFT INDEX	STORY SHEAR	$\theta_{p_{max}}$ BEAM	FLOOR DISPL.	DRIFT INDEX	STORY SHEAR	$\theta_{p_{max}}$ BEAM
ROOF	1.12	1.47	1.12	1.51	1.39	1.34	1.09	1.41
10	1.12	1.22	1.04	1.31	1.40	1.42	1.04	1.37
9	1.14	1.36	1.07	1.47	1.40	1.34	1.08	1.45
8	1.17	1.40	1.10	1.58	1.41	1.38	1.10	1.33
7	1.20	1.14	1.06	1.30	1.42	1.38	1.11	1.52
6	1.22	1.20	1.05	1.30	1.43	1.35	1.10	1.42
5	1.24	1.25	1.06	1.32	1.46	1.35	1.09	1.38
4	1.25	1.26	1.09	1.91	1.54	1.39	1.10	1.40
3	1.23	1.27	1.10	2.38	1.58	1.49	1.11	1.50
2	1.13	1.14	1.04	4.75	1.70	1.70	1.07	1.80

Note: Maxima do not always occur in same directions for the two peak ground accelerations.

Summary of Preliminary Design

The results of the nonlinear dynamic analysis indicate that the current structure will be able to resist a ground motion with characteristics similar to the El Centro record without suffering extensive nonstructural or structural damage. However, significant nonstructural and possibly structural damage would be expected in the lower and intermediate stories if a ground motion with characteristics similar to the Pacoima record should shake the building. Long acceleration pulses such as those in the Pacoima ground motion cause large inelastic beam deformations which lead to a number of design problems. First the large inelastic deformation requirements may lead to beam failures. Second, the strain hardening associated with the inelastic beam deformation increases the beam moment capacities which contributes to column yielding. The inelastic deformations in the upper story columns are small ($\theta_{p,max}$ < 0.003 rad) and well within the deformation capabilities of ductile reinforced concrete columns. A problem may arise in the first story columns where plastic rotations of approximately 0.01 radian are required.

A final design problem associated with the large inelastic deformations is the large story drifts which result. The drift indices recorded during the Pacoima response exceeded the limiting value of 0.015 in many stories indicating the possibility of significant nonstructural damage.

Design modifications should be made to eliminate the above shortcomings in the current design. It is apparent that the structure should be made stronger in order to decrease the required inelastic deformations, i.e., the design forces should be increased. But how can they be increased within the context of the proposed design procedure? In Reference 14 it is shown that the Pacoima ground motion is not accurately represented by the chosen ground motion spectrum, in particular with respect to the effective peak ground velocity. Redefining the ground motion spectrum with a ground velocity more representative of the characteristics of the Pacoima ground motion will permit a systematic increase in design forces. These new design forces should result in a structure which does not have the shortcomings of the current design.

The acceptable behavior during the El Centro ground motion, whose characteristics are precisely the ones considered in the formulation of the response spectra used in the design, is an indication that the general procedure works and the current preliminary design will be used to formulate a final design.

The significantly different responses to the El Centro and Pacoima ground motions clearly demonstrate that the problem that remains to be solved is the development of more reliable design earthquakes for ultimate state design.

FINAL DESIGN PHASE

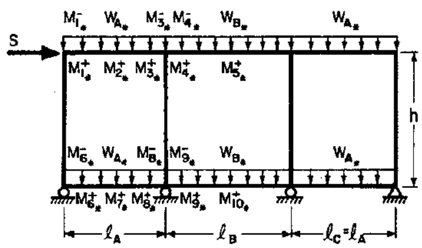
Introduction

The objective of the final design phase is to arrive at the 'optimal' solution to the seismic design problem. Seismic design forces are determined utilizing characteristics of the structure found in the preliminary design phase. These forces are then used in conjunction with a more sophisticated subassemblage to formulate the optimization problem from which the final design is obtained. Once a design has been obtained a series of elastic and nonlinear analyses are carried out to check the overall reliability of the design and to provide guidelines for proper detailing to insure ductile behavior. The final design procedure is illustrated in the following sections by applying it to the example frame.

Final Design

Design subassemblage -- The subassemblage selected for the final design is shown in Fig. 30. It has been used by El-Hafez and Powell in a nonlinear static analysis program [17]. They have investigated the reliability of this subassemblage in analysis and have concluded that good results can be obtained if the structure does not contain radical changes in stiffness. Since a smooth variation in stiffness is one of the basic principles for seismic-resistant design, this subassemblage should be applicable to the proposed seismic design procedure.

As in the preliminary design, a weak girder-strong column design criterion is established and the only design variables are the beam moments. If the design



NOTE: * INDICATES 1/2 ACTUAL VALUES

FIG. 30 FINAL DESIGN SUBASSEMBLAGE

symmetry assumed in the preliminary design is also assumed in the final design, 16 design variables may be identified in a typical subassemblage of the example frame (Fig. 30).

The solution of the optimization problem based on this subassemblage yields one-half the moment capacity of a given section, i.e., it is assumed that one-half of the moment capacity of each beam section goes to the story subassemblage above the beam and the other half to the story subassemblage below it. When the two subassemblages are rejoined the total moment capacity for that beam is recovered.

There are two advantages in using this subassemblage. First, it involves more design parameters than the subassemblage used in the preliminary design and consequently should provide a better distribution of moment capacities throughout a particular story and through the height of the structure. Secondly, the assumption that the points of inflection occur at mid-height of the columns, which is inherent in the preliminary design subassemblage, has been eliminated. As a result this type of subassemblage should be more realistic than the one used in the preliminary design.

Estimation of lateral story shears--The dynamic characteristics of the structure found in the preliminary design are used in conjunction with the design spectra to evaluate the seismic story shears by the modal analysis procedure discussed in the preliminary design phase.

Final optimization--The final design optimization problem is formulated by a procedure identical to that used in the preliminary design.

In addition to the serviceability and practical constraints imposed in the preliminary design, the practical constraint defined by the inequality

$$\left| M^{\text{support}}_{\text{below}} \right| \geq \left| M^{\text{support}}_{\text{above}} \right|$$

is imposed in the final design. This constraint is an attempt to insure a smooth transition in support design moments through the height of the structure.

Member design--Final member design follows the same procedure outlined in the preliminary design phase. Only the reinforcement distribution is determined in the final design, since the member sizes were established in the preliminary design phase. The relationships used to design the members are the same as those used in the preliminary design with one exception. The distribution of the sum of the beam moments at a given joint to the column sections above and below the joint was based on the results of an elastic analysis. This represents an attempt to eliminate one of the factors thought to be responsible for the column yielding observed in the analyses of the preliminary design.

Reliability of Final Design

Results of final design--The beam design moments found from the solution of the optimization problem are shown in Fig. 31. Comparison of these moments with the preliminary design moments (Fig. 11) indicates the following differences:

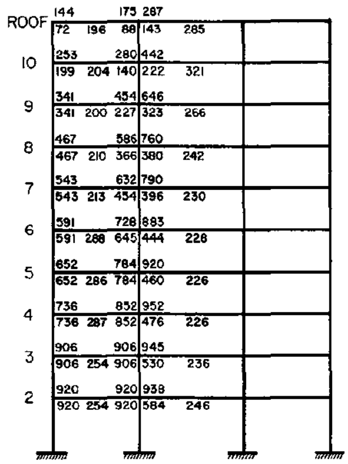


FIG. 31 OPTIMUM BEAM DESIGN MOMENTS IN K-FT, FINAL DESIGN

1. The subassemblage used in the final design does result in a smoother variation in exterior span design moments in going from story seven to story four. This is attributed to the additional practical constraint imposed in the final design.

2. Notable differences between the two designs occur in the tenth, third, and second stories. Typically the left exterior support moments were larger and the interior support moments smaller or the same at these stories in the final design. Differences at these stories would be expected, however, because the assumption inherent in the preliminary design subassemblage that column inflection points are at mid-story height is less likely to be true at these story levels.

The final column design differed only slightly from that obtained in the preliminary phase (Fig. 14). The most notable differences are in the lower two stories where the exterior column reinforcement increased and the interior column reinforcement decreased, due to changes in the beam design moments (Fig. 31 & 11).

Analysis of final design--Results of these analyses indicate that the gross structural behavior of the final design was virtually identical to that of the preliminary design. There were some significant differences in the member inelastic deformation demands, however. In general, the peak demands obtained in the preliminary design which are illustrated in Figs. 25 through 29, were eliminated in the final design, thus indicating an improved design. The main reason for this improvement is that the story subassemblage used in the final design resulted in a smoother transition in strength. The dynamic response of the final design to the Pacoima ground motions still shows some demand above acceptable levels.

CONCLUSIONS AND RECOMMENDATIONS

The proposed design procedure permits the inclusion of most of the important factors affecting and/or controlling selection of design criteria which are in accordance with the accepted general philosophy of seismic resistant design and consequently it provides an efficient and rational basis for the seismic design of multistory moment-resisting frame structures. The procedure is very versatile. Present design constraints can be changed and/or new constraints added in order to obtain several preliminary designs in a relatively short time which can be used as guidelines for the final design. The subassemblage suggested for the final design provides smoother transitions in strength than can be obtained in the preliminary design. Consequently it results in an improvement over the preliminary design.

In the development of this seismic design procedure, a number of problems were identified which warrant further study.

a) Significant differences between the responses to the Derived Pacoima Dam and El Centro ground motions demonstrate the need to establish better design earthquakes for inelastic design. These design earthquakes should take into account the severe long duration acceleration pulses contained in the Pacoima ground motion.

b) The relatively recent concept of damageability limit states creates problems with regard to its inclusion in the design process. The precise definition of this limit state (the story drift index was used here) and how to incorporate it rationally in design have still to be decided.

c) Selection of the displacement ductility factor for a multiple degree-of-freedom system should be examined more closely. In particular, the consequences of varying the ductility factor through the height of the structure should be studied.

d) The use of additional and/or different constraints in the optimization procedure in order to obtain more practical designs should be studied, particularly in regard to the most efficient way of redistributing the moments (internal forces) that are obtained from linear elastic analyses.

e) There is a need for improved expressions for the merit function which will require the formulation of more practical rules defining the cut-off point of flexural reinforcement than those derived from the present code requirements.

f) The significance of the considerable overstrength obtained by using present recommended code equations and values for the different factors (ϕ, α_f) should be investigated. More rational values than those presently used for column overstrength factors, F, should also be obtained.

g) Participation of the floor system in the strength of the beam critical sections (regions) should also be investigated.

h) A better model than the 2-component model is needed to predict the inelastic deformation of members. A model to account for beam column joint deformations (concrete as well as steel slippage) should be developed.

i) The current dynamic analysis program should be extended to three dimensions in order to investigate the effect of torsion on the reliability of the design.

j) A comparison should be made between the proposed seismic design procedure and those used in current practice, as well as those suggested recently in the literature, with respect to the volume of materials (cost) required and to the structural response to critical design excitations.

ACKNOWLEDGMENTS

This paper is based on studies conducted under the sponsorship of the National Science Foundation under Grant No. ENV76-01419.

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EXPERIMENTAL AND ANALYTICAL INVESTIGATIONS OF REINFORCED CONCRETE FRAMES SUBJECTED TO EARTHQUAKE LOADING

by

Peter Gergely
Professor of Structural Engineering
Cornell University

INTRODUCTION

Significant advances have been made during the past decade in understanding the behavior of reinforced concrete framed structures subjected to ground excitation. A parallel and nearly as fundamental development has occurred in the refinement of dynamic analysis capabilities for reinforced concrete structures.

It has become clear that the behavior of reinforced concrete structures under reversed cyclic loading is entirely different from that under monotonic loading. The knowledge obtained from unidirectional monotonic tests cannot be transferred to or extrapolated for seismic loading; the latter renders reinforced concrete a different type of material. Reversed cyclic loading fundamentally alters behavior, especially in shear and bond.

Several aspects of dynamic behavior have required much more sophisticated investigation than what has normally been used for the static case. One motivation for the extra effort is the relatively high probability of loading beyond the yield level during an earthquake; other loads rarely reach or exceed their design values. Another reason for the intensity of research is the advancement in the sophistication of testing facilities which has permitted more realistic loadings and much better instrumentation.

Of special significance is the fact that most designers of earthquake-resistant structures have also realized the need for research in the dynamic behavior of structures, and many are ready to incorporate the latest research results in design practice. Clearly that is not the case in most other areas of structural design.

Scope of Report

This report summarizes only the most important and interesting research results on the response of reinforced concrete frames to seismic forces. Clearly, time has not, space does not, and capabilities of a reviewer will never permit an exhaustive or complete review of such a rapidly advancing and often perplexing subject.

Shear and anchorage problems in beams and columns are discussed, with only minor reference to flexural behavior. Sliding shear behavior is emphasized in beams and to a limited degree in columns and walls because this has received relatively little attention. Analytical representations and some analysis results are also reviewed, but in less detail than experimental investigations.

The behavior of joints and hinges, and general shear problems are reviewed by James O. Jirsa in another report at this meeting. General questions of frame and wall behavior, analytical studies, and design are discussed by others, therefore only minor references are made to these topics in this report.

The following are the major section headings:

1. Review of Selected Research Topics on Frame Behavior
2. Anchorages and Splices
3. Shear in Beams and Columns
4. Sliding Shear
5. Research Needs
6. Summary and Conclusions
7. References

Acknowledgement

The author is grateful to Richard N. White for his help and perceptive comments during the preparation of this report and for his guidance and contributions in a continuing investigation of interface shear transfer at Cornell University.

REVIEW OF SELECTED RESEARCH TOPICS ON FRAME BEHAVIOR

The main thrust of both experimental and analytical research has been concerned with the energy dissipation capacity of reinforced concrete elements. The studies reveal that the ductility ratio is not always a satisfactory and complete measure of maximum inelastic response, rather the shape of the complete hysteresis curve is often required. The hysteresis curve is the best measure of energy dissipation of elements or connections and its knowledge permits realistic analysis of an assembly of elements such as frames and wall structures. It is usually established for moment-curvature, load-deflection, or shear-slip of members or subassemblies of structures. At the end, however, it must be known for the entire story or similar subdivisions of the structure.

Experimental Studies of Hysteretic Behavior

Many researchers have studied the effects of various parameters on the shape of the hysteresis curve. The most important factors are: shear force, bond slip, strain hardening of longitudinal steel, degree of confinement, axial load, and the number of load cycles.

High shear and high axial compression produce a pinching of the hysteresis curve which means a reduction in energy dissipation capacity and possibly an unstable loop leading to premature failure. The effects of bond deterioration are discussed in the next section and the effects of high shear and the use of special reinforcement are examined in the subsequent two sections. The influence of axial force is covered only briefly since it has been studied mainly in joints and at hinges (see the report by James O. Jirsa). Very little is known about the behavior of members with axial tension together with cyclic shear and flexure.

One aspect of the hysteretic behavior of reinforced concrete structures is of particular interest and is emphasized in this report. The load-deflection or moment-rotation curve may contain a segment with a very low slope

(Fig. 1) followed by a rather stiff portion which is representative of hardening systems. This kind of behavior, which develops only after a few cycles of loading, may be caused by bond slip, sliding shear, dowel deformation of bars, or

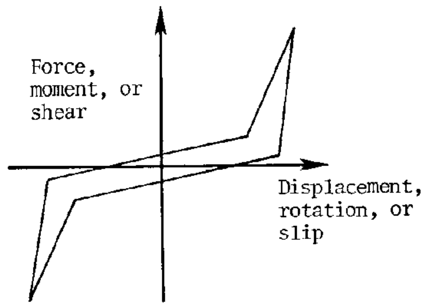
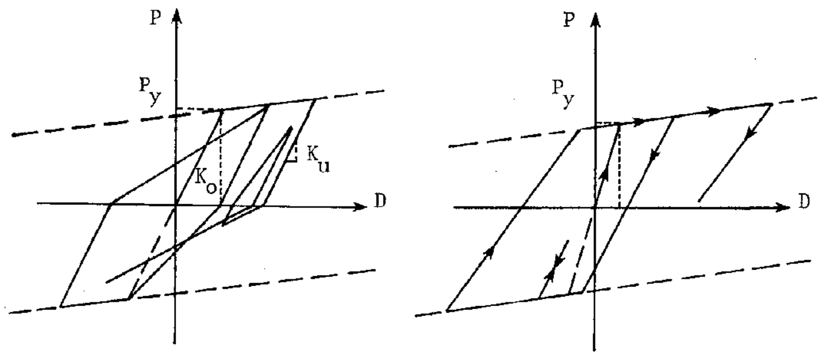


Figure 1 Hysteresis Curve with Low Stiffness Segment

slip between frame and wall. It is quite different from the customary elasto-plastic hysteresis and could cause significant differences in dynamic response. Sliding shear and the use of the corresponding hysteresis curve in dynamic analysis is discussed in a later section in this report.

Several typical load-deflection hysteresis loops have been proposed for reinforced concrete cantilever members in which flexural deformation dominate; some of these are sketched in Fig. 2 [12,54]. D_y and D_{max} are the yield and maximum displacements, respectively, and α is about 0.4 to 0.6. The slope of the reloading line is oriented toward the previous peak displacement point and thus decreases with increasing prior displacement. The application of these curves in analysis requires a set of rules that is ordinarily used to represent all forms of energy dissipation. The degrading bilinear system is relatively simple and is recommended for use with 5% damping. Nonlinear analysis with such curves can be rather complex as low-level cycles may occur between the extreme amplitude excursions defined by the main hysteresis curve. Furthermore, various elements of a frame are ordinarily not at the same point on the loop and the bookkeeping during analysis is complex. A degrading stiffness representation, rather than bilinear or simple elasto-plastic stiffness, is necessary to model intense response, especially for relatively stiff structures [10,21].

Several researchers have demonstrated that previous loading history has a great effect on the hysteresis loop. For example, one large amplitude excursion may reduce subsequent energy dissipation capacity significantly in comparison with monotonically increasing amplitude in each cycle [30]. This behavior complicates the problem so much that simplifying assumptions are necessary to reduce the number of permutations or amplitude sequences both for testing and for analysis.



$K_u = K_o$ [12]

$K_u = rK_o$ [54]

$K = rK_o$ [21]

$r = (D_y/D_{max})^{\alpha}$

Figure 2 Hysteresis Curves Idealized for Analysis

Hysteretic Representations in Analytical Studies

Many researchers have used various types of idealizations and hysteresis rules in nonlinear analyses and have shown that good results can be obtained when the idealizations directly correspond to the system being modeled. In most cases, however, not all modes of stiffness deterioration were included in the analysis and in the corresponding tests. Significant advances have been made in system identification techniques that allow the determination of damping and stiffness properties from test results, or enable linearization of nonlinear systems. Most nonlinear analyses are too complex for design use but they are helpful in identifying the effects of various factors as well as in aiding in the planning of test programs.

Many factors affecting nonlinear response have not yet been isolated or studied sufficiently. Therefore, most analyses are reasonably accurate only for the test program for which they were derived. If other factors modify the behavior or if a different type of loading is applied, the agreement between analysis and test is generally poor, especially after two or more load cycles. Some of the factors that have recently been identified are mentioned in the following paragraphs.

In one program it was found that strain hardening must be included in the analysis [37]. The frequency content and the time variation of the spectral content of the ground motion also affect the response of hysteretic systems [30]. The loading rates occurring during earthquake excitation influence primarily the flexural strength, though the increase in moment capacity demands a corresponding increase in shear capacity if shear failures are to be avoided.

When cracking and bond-slip are not the dominating degradation effects, the nonlinear cyclic behavior of plain concrete should be considered. It was found that compressive softening of concrete under cyclic loading may have significant effect on the load-deflection response of panels or walls [13].

Hysteresis loops are usually incorporated in stiffness analysis approaches. One logical and relatively convenient formulation is to treat halves of beams and columns between frame joints and midspan inflection points as cantilevers and assign empirical flexibility properties to them. Several researchers used this or similar idealizations. Members may also be idealized as rigid links with parallel linear and elasto-plastic elements and with rotational springs at their ends [37].

Analytical studies, coordinated with experimental research, are important in reducing the number of variables or in studying their importance. Analytical approaches can also use information from basic tests to study complex systems, albeit with considerable simplifications, to reveal the interaction of members, and to pinpoint problem areas. For example, in one analysis it was found that inelastic interaction of motions in two directions greatly increases response [38]. Another study [3] showed that vertical acceleration and gravity effects increase the demand for energy dissipation in multistory frames. This tandem approach of experimental and analytical research, each aiding and benefiting from the other, is an effective and productive approach.

ANCHORAGES AND SPLICES

Two questions have primarily concerned researchers: anchorage failure and the effects of bond slip on the flexibility of structures. Few anchorage failures during earthquakes have been reported but this is possibly due to the fact that shear distress or collapse usually follows anchorage failures and it is often hard to identify the primary cause. Anchorage failures occur where insufficient web reinforcement is provided but then shear failure is also likely.

Several column steel anchorages failed at the top of columns during the San Fernando earthquake. Major inclined cracks developed at bar cut-off points during the Mexico City quake of 1957. Similarly, shear failures occurred at cut-off points in long beams during the Guatemala earthquake and insufficient column splicing caused column failures. Beam splices are ordinarily not subjected to large stress reversals from lateral loads, but the superposition of vertical acceleration could create significant forces in beam splices.

Experiments have clearly demonstrated that bond and splice capacity may be greatly reduced under cyclic loading [7,18,22,47]. Bond slip and the crack width at highly stressed sections increase with cycling, especially when tensile and compressive yield forces alternate in the bar. Overload creating large strains in the bars also causes irreversible damage and reduces subsequent bond capacity [7,22].

Most studies of bond fatigue have been concerned with relatively low forces and large number of cycles. It was found that fatigue failure does not occur for load amplitudes less than half the static pull-out load [47]. However, high-level reversing loads can cause bond problems [1]. A thorough review of bond and material behavior for rapid loading was recently completed

[20], but the load levels considered were generally smaller than those generated in a major earthquake.

Little information is available on the behavior of lapped splices. Impact type tests of splices showed an increase in splice capacity even for reversed loading not greater than the static capacity. Stirrups enhance the toughness and ductility of splices [48]. Research is needed on the behavior of lapped splices and mechanical splices at high-level load reversals.

Bond-slip curves have been proposed for analysis purposes [8,33,50]. Again, load reversals drastically change the shape of the curves; unloading is stiff and a sliding portion follows [29]. It was found that the hysteresis loop is similar to that in Fig. 1, and it is necessary to use such curves to obtain good analytical results for beams or for cyclic deformations of concrete panels [49].

Perhaps the most interesting effect, noted by several researchers, is the large contribution of bond slip to frame deformations [5,22,30,52,54]. This effect increases with cycling and may cause as much as 50% of the total tip deflection of cantilever beam specimens. Thus bond slip is an important attribute of the hysteresis loop and influences the dynamic behavior of beams, joints, and columns. The width of the transverse crack increases with bond slip, and therefore the interface shear transfer capacity across the crack is reduced significantly, as is discussed in a subsequent section. Bond deterioration also greatly decreases the effectiveness of stirrups [26].

Bond slip introduces a low stiffness region in the hysteresis loop of a flexural or tensile member and the resulting shape is similar to that shown in Fig. 1. This decreases the reliability of dynamic analysis during low-amplitude vibration. It is questionable whether such a complex slip-type hysteresis model should be used in building analysis, but it is an important research topic.

Although several researchers have studied the problem, insufficient information is available on the effect of alternating tensile and compressive yield forces on bond deterioration [7,8,22]. Most cyclic tests were performed at working load levels. Dowel shear impairs the bond performance of bars; this is but one of the several variables important in this behavior.

SHEAR IN BEAMS AND COLUMNS

Shear failures have been especially conspicuous in reinforced concrete buildings hit by major earthquakes. The shear force created by ground motion is very large, especially when there is little energy dissipation in the structure.

Columns have failed due to shear alone or in combination with bending, torsion, axial load, or the P-delta effect (for example in the San Fernando, Tokachi-Oki, Mexico City, and Caracas earthquakes). Short members with high shear-moment ratios are most vulnerable to shear failures. This occurs in short beams spanning openings (San Fernando earthquake) or in columns free between spandrel walls or light partitions (Guatamale earthquake). These "captive" columns, shortened by lateral support as far as bending is concerned, receive high shear forces.

Damage studies and research have prompted progressively more and more conservatism in design for shear; now the limit has been reached whereby many design for the maximum possible shear in a member, corresponding to actual flexural capacities and actual support conditions.

For an unsupported length L_u the maximum shear in a member is

$$\frac{(M_u)_t + (M_u)_b}{L_u}$$

where $(M_u)_t$ and $(M_u)_b$ are the top and bottom flexural capacities at the ends of the unsupported length L_u. These moments should be calculated considering all factors that could increase them, such as higher than specified yield strength of the bars, strain hardening (a factor of about 1.3), high rate of loading (also a factor of about 1.3), contribution of slab or panel, and additional longitudinal splice steel or longitudinal web reinforcement. Sometimes L_u is quite small because of architectural elements attached to the column. In these cases the shear may be so high that it is impossible to design for it and the structural configuration must be changed. Often full and ductile shear walls offer the best solution.

Unfavorable research results have forced some researchers to recommend calculating the shear capacity of concrete members based on the confined core only when axial compression is present and relying on transverse reinforcement only when significant axial compression is not present [55]. In any case the maximum shear stress should be considerably less than $10\sqrt{f_c'}$. The question of shear capacity with axial tension has not been studied except at transverse cracks (see the next section).

One of the most significant developments has been the realization that inclined steel is more effective than transverse stirrups in preventing shear failures under reversed cyclic loading; this has not been found for static loading [5,25,42]. This is especially critical where sliding shear occurs. Inclined stirrups or bars prevent excessive slip along the crack. Web steel in an x-shape is particularly effective; when these bars are confined to avoid buckling of the compression bars, a 75% increase in the energy dissipation was achieved [30].

Researchers in Japan have found that a shear span to member depth ratio of at least two and an axial force less than a third of the axial load capacity are necessary to have flexural rather than shear failure and sufficient ductility. Closed ties, with every longitudinal bar ties (in the corners or with tie pins), or spiral steel are equally good if the shear reinforcement ratio is at least 0.6% [55]. The energy dissipation in such a case increases by about 75% [30]. Rectangular hoop ties, that envelop all bars but leave some bars not in a corner of the stirrups or not secured with tie pins are not satisfactory.

Closely spaced stirrups are frequently necessary to provide adequate energy dissipation capacity, but sliding failure is still possible along transverse cracks between stirrups; that problem is examined in the next section.

A number of engineers have proposed the soft first story concept for the design of buildings. However, based on the relatively poor performance of columns in earthquakes and considering the many factors affecting their behavior, that avenue does not seem prudent. In usual structural engineering practice there is no harm overdesigning members and most of the knowledge developed and formulated in codes are typically on the "safe side". Yet, a soft first story must not be overdesigned or underdesigned; it has to be just right or within narrow limits as far as stiffness is concerned. Furthermore, analyses showed that very low yield levels and elasto-plastic column resistance are necessary to shield the rest of the structure from attracting seismic energy and that the ductility demand in the soft first story is high [9].

SLIDING SHEAR

An important failure mode involves shear sliding at open cracks produced by flexure or tension. The resisting mechanism has been named "interface shear transfer" or IST [2]. It is made up of aggregate interlocking and shear friction. Dowel forces in embedded bars normal to the crack and axial and dowel forces in bars crossing the crack at an angle also contribute to the shear stiffness at open cracks.

Sliding shear failures or at least significant slips along cracks have been observed in earthquakes. In Skopje a 5 to 8 cm (2 to 3 in.) displacement occurred at a wall [46]. Shear walls slipped relative to beams along wide cracks between them in Caracas. The box structure of the Nethercutt Museum displaced about 10 mm (3/8 in.) at a construction joint in the 200 mm (8 in.) wall during the San Fernando earthquake. Slips at construction joints of shear walls were noted in the Alaska earthquake.

Sliding is greatly facilitated by the fact that static friction may be greatly reduced or even eliminated during earthquake excitation. The combined effects of horizontal and vertical vibrations may exhaust the static friction and then relatively small shearing forces can cause sliding. This question has not been studied and all information on interface shear transfer is based on static tests or low frequency dynamic tests.

Sliding shear may occur in beams at column faces if alternating flexural yielding opens the crack; this is especially critical in coupling beams. In cyclic shear tests of cantilevers, transverse cracks formed at every stirrup and sliding shear deterioration developed [8] for deflection amplitudes five times the yield deflection or more. Sliding shear can develop along horizontal cracks in shear walls and columns. Such cracks were observed in earthquakes and in tests of single- or multistory models [36]. Tension cracks were observed even in beams.

Sliding shear failures or distress was observed in frame-shear wall tests [35] and in short beam tests [6]. A complete tension crack was noted at the base of the bottom column of a three-story model structure subjected to ground acceleration [36] which was not predicted by elasto-plastic analysis. Local crushing at the same plane increases the width of the crack and causes poor interface shear transfer capacity during cyclic loading.

Sliding shear distress may also develop in walls. Several tests [4], field observations, and analyses showed net tension and even yielding in walls. Sliding along horizontal joints is an important problem in the design of shear walls [41]. Sliding between a steel frame model and infill wall also resulted in a hardening stiffness model [14].

The importance of interface shear transfer in columns in tension has not been examined. Yet, for example, for a 400 mm by 400 mm (16 in. by 16 in.) column with eight #9 bars the crack width is about 0.5 mm (0.02 in.) when pure tension produces stresses close to yield in the bars (from overturning effects and vertical acceleration). Such cracks usually form at ties and therefore the ties closest to a crack would be one tie spacing away. As is shown subsequently in this section, in the absence of transverse reinforcement close to the crack the reversed cyclic interface shear transfer capacity for such crack widths is of the order of 1.4 to 2.1 MP_a (200 to 300 psi), and considerable slip and crack deterioration may occur at lower stresses. The dowel capacity of the bars is equivalent to only about 0.35 MP_a (50 psi); thus sliding shear distress is possible in columns subjected to tension. For a given interstory relative displacement of, say 13 mm (0.5 in.), the compression columns (or walls) would deform in shear and flexure but the tension columns (or walls) would also have to distort the same amount and could undergo damaging sliding shear displacements. A slip of less than about 1 mm (0.05 in.) is already harmful; such a slip is quite conceivable considering the relative magnitudes of approximate sliding shear and column lateral stiffnesses.

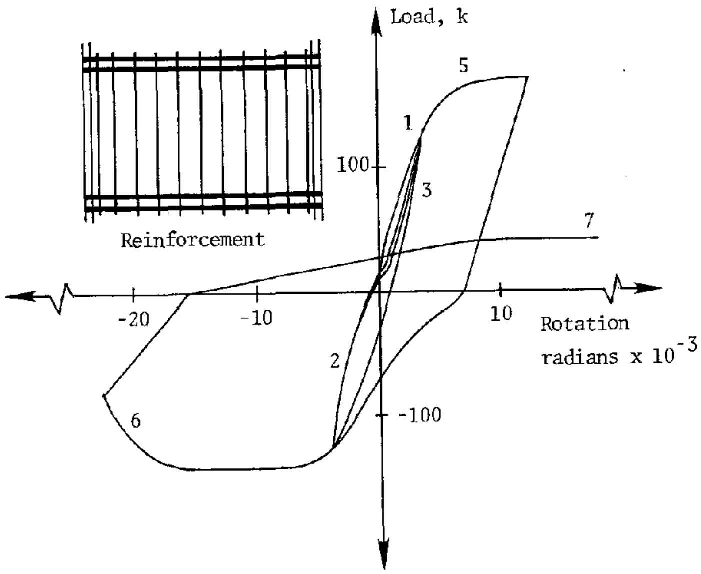
Several approaches have been developed for the study of cyclic interface shear transfer across open cracks in concrete. In some experimental investigations [16] the crack width was held constant during shear cycling. In another study the bars were yielded first to produce the desired initial crack width [32]. In a third approach the bars crossing the crack were stressed in tension during the shear cycling; the tension was applied to obtain the desired initial crack width [27]. An important factor affecting the behavior of interface shear transfer is the amount of steel near the crack. Reinforcement parallel and close to the crack delays or prevents shear deterioration because the concrete is confined. In addition, diagonal cracks are not allowed to propagate from the crack plane, and dowel splitting is restrained. The mechanism of interface shear transfer for various types of experiments is described in several papers [16,27,32].

A significant contribution has been the demonstration that heavy diagonal reinforcement is an effective means of controlling sliding shear in beams and walls [6,25,40,42]. The diagonal bars must be confined to avoid their buckling, and in applications where progressive straining of the normal bars opens the crack, the diagonal cross reinforcement is expected to take the entire shear.

The cyclic behavior of two beam specimens, one with normal web reinforcement and another with inclined steel, is sketched in Fig. 3; only some of the hysteresis loops are shown. The hysteresis loop for the member with diagonal steel does not have a low-load soft portion because little sliding occurs even if the crack does not close during cycling.

In another study the use of diagonal crossing struts at the fixed end of cantilever specimens resulted in a 50% increase in energy dissipation capacity compared with specimens generously reinforced with transverse stirrups [6].

The hysteresis curve for shear forces versus shear slip produced by interface shear transfer is again similar to the one shown in Figure 1. The



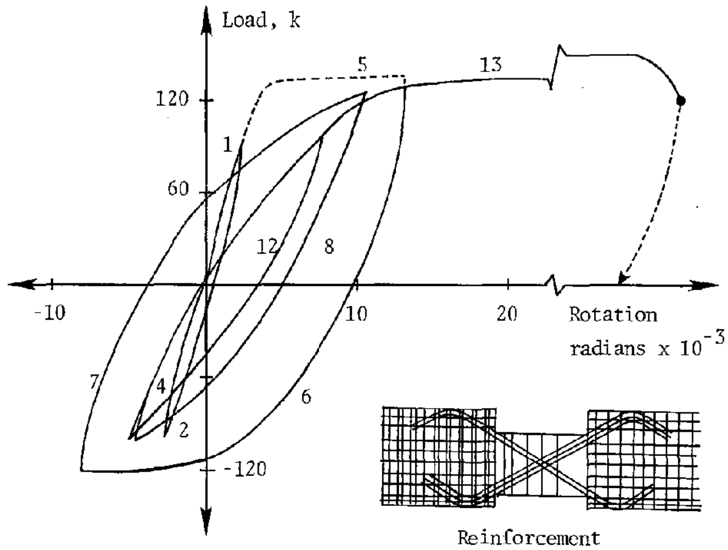


Figure 3 Effect of Web Reinforcement Geometry on Hysteresis [42]

mechanics of slip is illustrated in Figure 4. There is a nearly free slip between the two extreme positions where crack irregularities touch and bearing develops in both directions. The surfaces wear down during cycling and the slip increases. As bearing between the concrete surfaces develops, the stiffness increases and further slip results from a combination of bearing compression and sliding of the sloping contact surfaces leading to a widening of the crack. The relative magnitudes of the bearing and frictional resistances depends greatly on the axial stiffness of the reinforcing bars across the crack and on the dowel effect of the same bars.

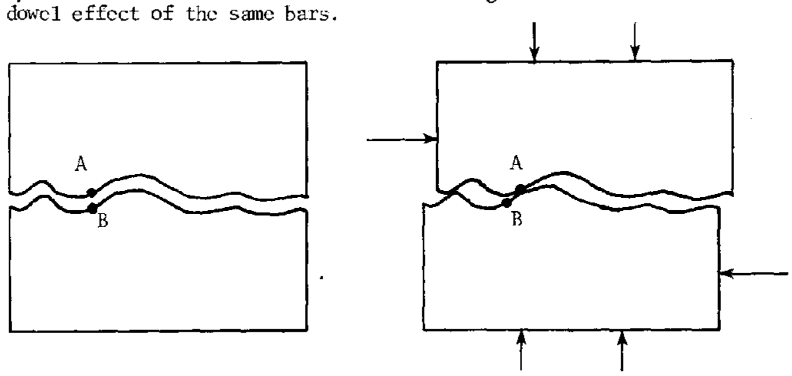


Figure 4 Shear Displacement Along Crack

A large number of variables affect interface shear transfer, especially its degradation with load cycling: initial crack width, parallel reinforcement near the crack, axial bar stiffness, shear force magnitude, dowel contribution, type of aggregate, and frequency content of loading. Only some of these factors have been studied sufficiently to permit reliable conclusions [16,27,32].

A typical set of hysteresis loops is shown in Figure 5 for a 0.15 m^2 (225 in.^{2}) shear area reinforced with two #14 bars having 214 MP_a (31 ksi) initial axial stress [24]. The crack width increased continuously from an initial value of 0.5 mm (0.02 in.) to about 0.7 mm (0.027 in.) and the slip to about 0.5 mm (0.02 in.) during 40 cycles of loading with applied shear of ± 1.4 MP_a (± 200 psi). In the 38th cycle the stress in the bars was increased to 280 MP_a (41 ksi) and cracks were observed forming planes through the bars parallel to the applied shear. The specimen failed in a brittle manner at a shear stress of about 1.6 MP_a (230 psi) during the 42nd cycle. The hysteresis loop did not change much after about the tenth cycle.

In another test four #9 bars were tensioned to 227 MP_a (33 ksi) and subjected to cyclic shear stresses of 1.1 MP_a (160 psi). The crack width and slip increased very little from their values (0.5 mm or 0.02 in. and about 0.13 mm or 0.005 in., respectively) during the first cycle. Little distress was noted even after 30 cycles. The essential difference between these tests is dowel effect of the bars; the two #14 bars have greater dowel stiffness and dowel splitting capabilities. Note that the steel areas were approximately equal (about 2% steel) and therefore the kinking of the bars did not play a role as

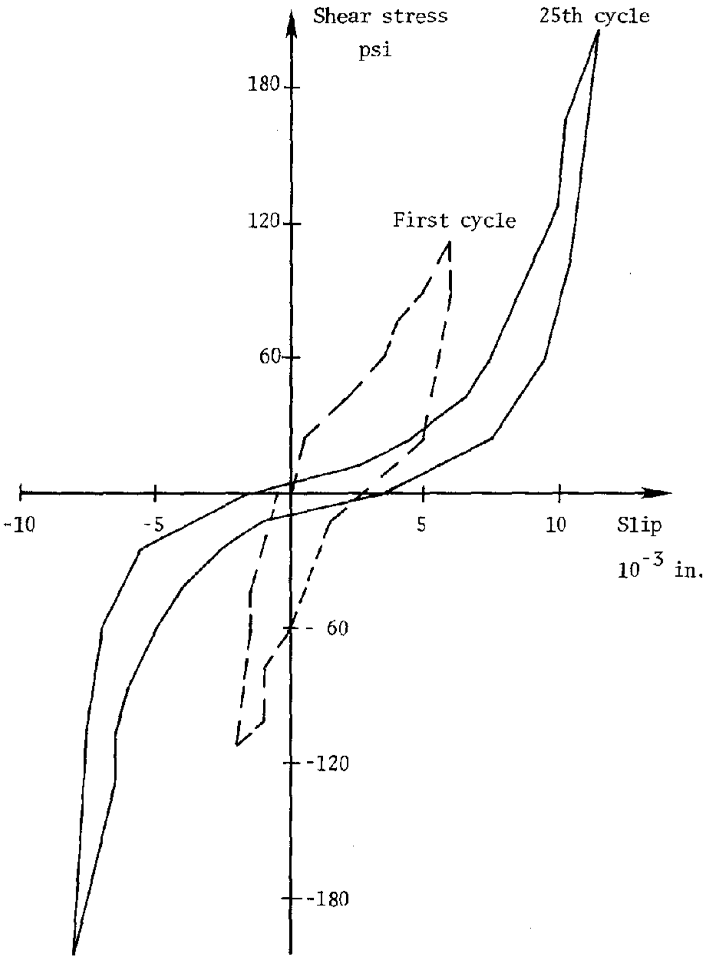


Figure 5 Shear Stress versus Slip [24]

it might with much smaller bars [43], consequently it was not necessary to reach appreciable slip before mobilizing dowel forces.

Typical shear stress values that produced stable or only slightly degrading hysteresis loops were 1.7 MP_a to 2.1 MP_a (250 to 300 psi), for initial crack width of 0.25 to 0.64 mm (0.01 to 0.025 in.). Single-directional shear gave significantly higher capacities, though dowel splitting is possible at a stress of less than 2.8 MP_a (400 psi) in members with large bars. Dowel splitting can develop in the direction of the shear force or normal to it.

In the other types of tests, where parallel steel was provided near the crack (within about 50 mm or 2 in.), or where the transverse steel was not highly stressed during the test, the unidirectional shear capacity was much larger and the reduction in shear capacity due to shear cyclic was only about 20% [32]. A typical set of load-slip curves is shown in Figure 6, illustrating the effect of initial crack width.

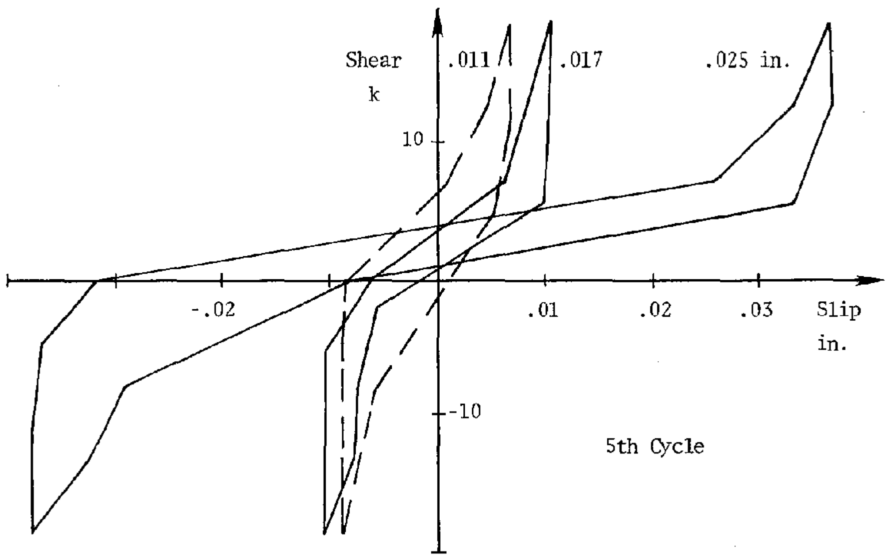


Figure 6 Hysteresis Curves for Various Crack Widths [32]

Significant sliding shear contributes to the bond deterioration and crack width increases, and all of these factor decrease stiffness and energy absorbtion capacity. For example, a shear stress of about 2.4 MP_a (350 psi) led to a stiffness that was only 50% during the first cycle and only 12% after loading four times to the yield level in both directions, compared with another specimen with a shear stress of 1.6 MP_a (230 psi) and loaded similarly [32].

It is evident, that such loss of stiffness at modest shear stresses should be avoided wherever open cracks and shear might occur. It was suggested that hinge formation in beams should be forced a distance away from the column face [6], mainly to reduce bond deterioration.

Shear transfer across construction joints was also studied [43] and design equations were developed for the case where vertical compression acts. Only in heavily reinforced joints did cyclic loading at 4.8 MP_a (700 psi) cause continuing loss of stiffness. Joints that were prepared only by trowelling showed large slips once the bond was broken. Open cracks, tension, and the deterioration of interface shear transfer were not considered.

Analysis with IST

Only a few analyses were explicitly considering interface shear transfer. Most studies, mentioned in the following paragraphs, predicted open tension cracks and simultaneous shear and thus identified instances where IST may be important. Other investigations included IST effects by assuming a hysteresis curve of the kind shown in Fig. 1, though in most cases the assumed or measured hysteretic behavior represented several effects (bond slip, high shear, dowel splitting, high axial force, and IST). Interface shear transfer was directly considered in at least one study.

Dynamic analysis of a coupled shear wall building resulted in a shear force of 28% of the total force in a wall that was yielding in tension [53]. It was necessary to consider inelastic axial stiffness of the wall even though IST and dowel degradation of the wall were not included. In this and in most other studies, stiffness degradation was assumed to be proportional in various members of the structure or proportional in flexure and shear. However, shear stiffness reduces faster than flexural stiffness [44], and this may affect the results in certain applications.

Hysteretic behavior with IST was considered in an analytical study of seismic shear transfer in nonprestressed nuclear containment shells [51]. Although the stiffness properties of such a structure are not similar to those in most buildings, the lessons learned are applicable. Horizontal cracks in the cylindrical shell result from internal pressurization which may develop concurrent with the earthquake. Shear is transmitted by a combination of IST and dowel action if diagonal bars are not employed. A parallel experimental investigation, mentioned above, provided the information necessary for the analysis.

The hysteresis loop was idealized as shown in Figure 7, based on a test having two #14 bars (Figure 5). The hysteresis loop is 1-2-3-4-5-6 for shear stresses $\pm$1.4 MP_a ($\pm$ 200 psi). For lower shear stresses the rules for using the hysteresis curve are: unloading from A is parallel to 2-3 and reloading from C (line C-D) is parallel to 1-2 and it has greater stiffness whenever loading starts from a stress level higher than point 1. Unloading from a point higher than point 2 is along E-F which is parallel to 2-3. Thus the hysteresis loop changes with each cycle and is different for each crack in the structure. A linear stiffness was used in the first cycle, as indicated by tests. The crack flexibilities were incorporated in the regular flexibility matrix of the structure. Great care had to be taken at sudden changes of stiffness (such as at B or 3) to represent the change accurately during the numerical integration.

System identification procedures [15] resulted in an equivalent linear system that gave response close to the nonlinear analysis prediction even for such a highly nonlinear case, at least for the particular synthetic earthquake

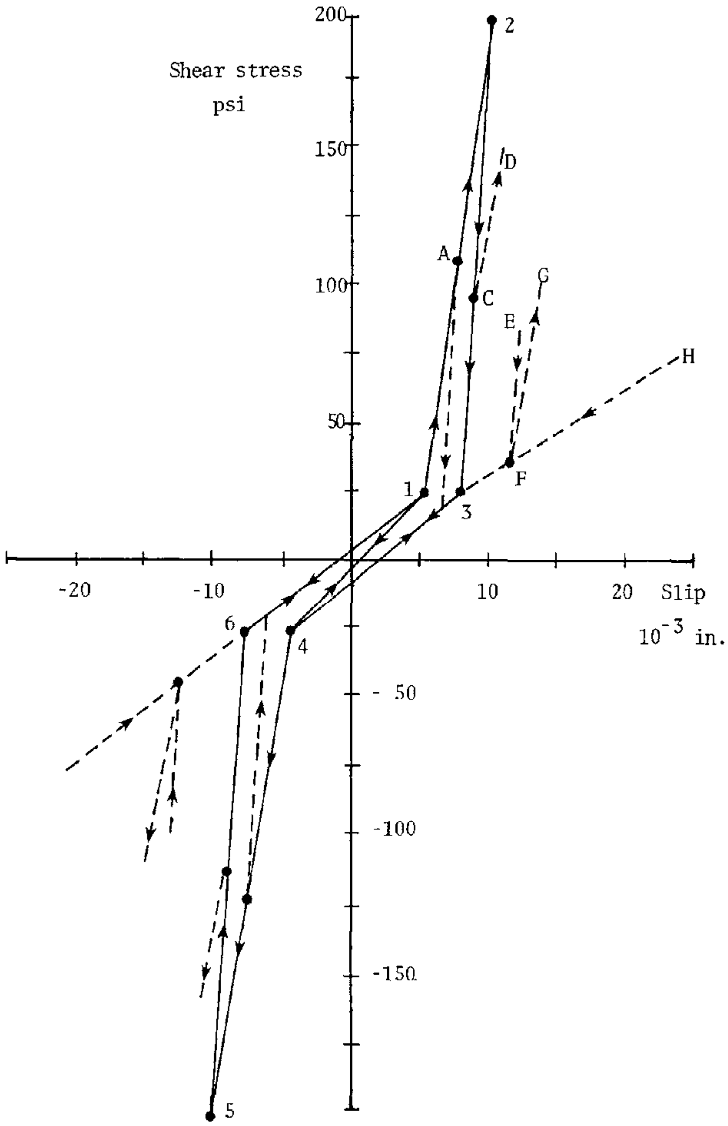


Figure 7 IST Hysteresis Used in Analysis [51]

used. This indicates that the approach may be viable in the development of earthquake-resistant design procedures. However, the generality of system identification methods is questionable as other types of ground excitations may result in different response.

It should be pointed out that experimental evidence is needed for a wider variety of geometric and loading condition to assure that the interaction of all factors is properly assessed. In none of the IST studies were the forces in the transverse bars cycled and thus bond deterioration was not great. (The forces in these bars do vary as the crack width changes during cycling, but this variation is not consequential). In actual structures the transverse bars may reach yield in both tension and compression and this could seriously reduce interface shear transfer stiffness and capacity.

RESEARCH NEEDS

Based on the information reviewed in preparing this report the following questions need further investigation:

1. Column behavior considering alternating high level tension and compression, combined with reversed cyclic shear, flexure, and torsion.
2. Interface shear transfer degradation considering dowel splitting, frequency content of excitation that alters the friction, and the amount of reinforcement near the crack.
3. Effect of earthquake duration and load history on response, damage, and hysteretic behavior and the establishment of a few classes of time histories of motion to provide realistic guides to researchers using static and simulated earthquake loading.
4. Sliding shear between walls and beams, and the ties between walls and beams or grade beams.
5. Sudden load transfer from walls to frames.
6. Behavior of short (captive) columns supported by walls over part of their lengths.
7. Behavior and design of grade beams and walls tied to them using vertical or diagonal steel.
8. Relationship between energy dissipation capacity, stiffness variation, and maximum displacement for various structural elements.
9. The flexural behavior and ductility requirements of beams near midspan, considering gravity effects, vertical acceleration, and splices.
10. Study of the feasibility of standard preassembled beam-column joint bar cages for typical applications and for several types of concrete.
11. Various rates and types of degradation at different parts of a structure.

SUMMARY AND CONCLUSIONS

This report reviews recent progress in the study of the behavior of reinforced concrete frames subjected to earthquake forces. Shear and bond effects are emphasized outside beam-column joints and away from flexural hinges. The following are the most significant findings, conclusions and ideas:

1. Reversed cyclic loading completely alters the behavior of reinforced concrete elements, especially when bond, shear, high level flexure, or interface shear transfer across a crack are important factors in the load-carrying mechanism. Information obtained from unidirectional monotonic loading can rarely be extrapolated or used in the cyclic reversed loading case.

2. Great care must be taken in design to assure energy dissipation capacity in columns under the combined action of compressive or tensile axial force, shear, flexure, and torsion.

3. Bond-slip of longitudinal bars is a significant source of flexibility and may account for as much as 50% of the total deformation of members. It may also lead to increased crack width that can produce sliding shear distress along the crack if the loading is reversed to high levels.

4. Low beam shear (less than about 1.4 MP_a or 200 psi) may be transmitted at the column face if closely spaced stirrups are used. Higher cyclic shear (about 2.4 MP_a or 350 psi or more) seriously decreases beam stiffness and energy absorption capacity unless diagonal crossing main reinforcement is used. Even higher shear (of the order of $10\sqrt{f'_c}$) seriously impairs the earthquake resistance of the member and should not be permitted in seismic design.

5. Nonlinear analysis procedures using degrading hysteresis curves can predict the dynamic response of structures if bond-slip, sliding shear deterioration, and strain hardening are included. This is currently possible only in the first few cycles. In some cases additional factors, such as gravity loading, vertical acceleration, and degrading concrete response must also be included in the analytical approach.

6. There are many problems that have not been studied sufficiently, the most important ones are listed in the previous section.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

BEHAVIOR OF ELEMENTS AND SUBASSEMBLAGES--R. C. FRAMES

by

J. O. Jirsa
Professor of Civil Engineering
The University of Texas at Austin

INTRODUCTION

Studies of the behavior of reinforced concrete members or frames under simulated earthquake loadings have expanded our knowledge rapidly in the past ten years. It is quickly apparent that the amount of work that has been done cannot be reviewed adequately in one short paper. The purpose of this report will be to call attention to those studies which appear to be the most applicable to the problem of design of earthquake resistant construction of reinforced concrete frames.

Categorization of Investigations

The approaches used by various investigators may be quite different, but generally fall into one of several categories:

(1) Material properties
(2) Isolation of behavior of one aspect of member response--e.g., bond and anchorage tests, flexural strength and ductility of sections, or shear strength of members
(3) Behavior of frame subassemblages consisting of multiple members and the connections or joints between them
(4) Studies of complete structures

It is evident that as one proceeds down the list, the response obtained at any level is related to the response obtained from tests at lower or preceding levels. Under monotonic loading a certain hierarchy can be established where the material properties, relationships between steel to concrete bond and other elemental response characteristics can be combined to predict member response in the form of moment-curvature or load-deformation characteristics. These can then be combined to predict the response of frames or frame subassemblages. At each level it is possible to run tests to check the validity of the model used to predict response at that level. Once satisfactory agreement between model and test is obtained, the model can be used to develop design recommendations. At any level the response may not be possible to predict using lower level response characteristics and empirical relationships must be relied on as in the case of shear strength.

Cyclic Loading

When the structure is subjected to reversed or cyclic loadings producing large inelastic deformations, the hierarchy described previously becomes difficult, if not impossible, to apply. The response at any level becomes path oriented--that is, the response is a function of that which has happened before. Many attempts have been made to use material properties to predict section, member, and finally structure response under cyclic loading, but have met with varying degrees of success. The models proposed by Ma, et al.

[18] are one example. Much more success has been achieved by modeling force-deformation relationships for members or structures under cyclic loads without attempting to model lower level response characteristics. The approaches used by Takeda, et al. [27] or by Atalay and Penzien [5] are examples of such models.

The discussion here will be limited to studies in categories 2 and 3. Other reporters will discuss material properties (category 1) and investigations of total structure response (category 4). It should also be noted that this discussion will be confined to response of members and subassemblages under static loadings.

LOADING HISTORY

If the design of frame structures for seismic loadings is to advance, some attention must be given to the loading history imposed on the structure, member, or section during an earthquake and to model this loading in the experimental program. It is quite possible to produce different answers to questions regarding structural behavior by varying the loading history. Examples of various loading history [6,11,16,31] used in experimental investigations are shown in Fig. 1.

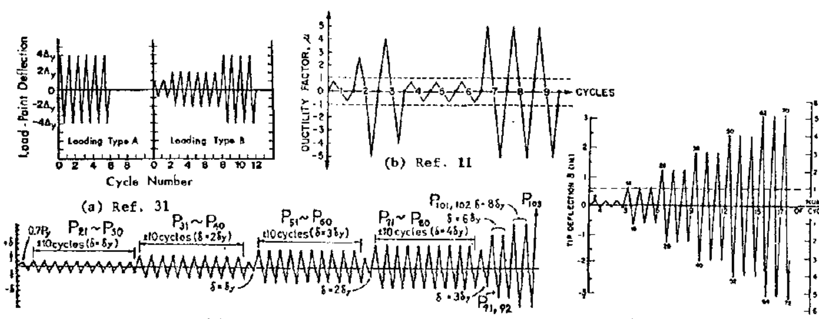

Fig. 1. Various Types of Loading History

The differences are readily apparent. In some cases the load is cycled between prescribed deflection limits until failure or severe distress is observed. In other cases, the deformation limit increases after the application of a number of load cycles at a given limit--a steadily worsening earthquake. A further complication arises when deformations used to control the loading history are not directly comparable, such as deflection in one case and member rotation or curvature in another.

Influence of Loading History on Response Characteristics

It may not be necessary to have "standard" loading routines used by all investigators, but it is important that users of experimental results are cognizant of the influence of loading history on response in evaluating experimental data. In fact, there may be merit in diversity because there will be a greater need for more thoughtful comparison of test results. Load-deflection curves are shown in Fig. 2 [16] for two specimens subjected to the loading history shown in Fig. 1(c). The performance can be compared visually by noting the "pinching" of the curves toward the origin in Fig. 2(a) indicating less satisfactory energy-absorbing characteristics. The specimen in Fig. 2(b) exhibits fairly stable hysteretic behavior at each series of loading at a given deflection limit. Note that it is farily clear in the early cycles of loading that the response of the specimen shown in Fig. 2(a) is inferior to

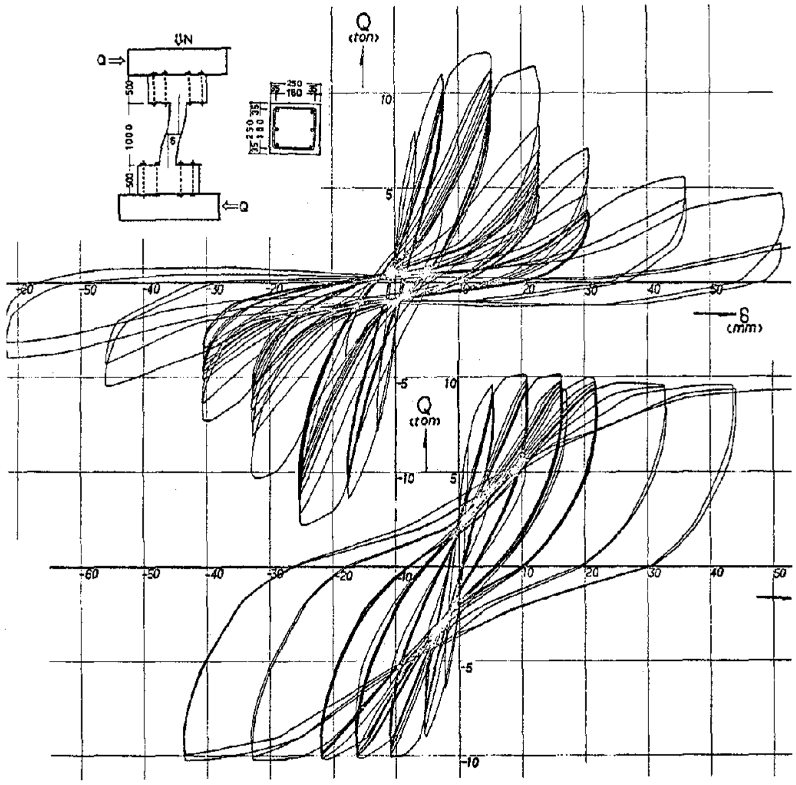

Fig. 2. Lateral Shear-Deformation Curves [Ref. 16]

that of Fig. 2(b). From the response shown for the specimen [30] in Fig. 3 [load history in Fig. 1(a)], it is evident after the second cycle that the performance of the specimen is unsatisfactory. Poor performance will generally be evident after a relatively few load applications and subsequent loading only confirms the trends evident initially. Higashi, Ohkubo, and Ohtsuka [15] conducted a series of tests using loading patterns, shown in Fig. 1(c), on companion specimens subjected to either three or ten load reversals at each deflection level. The results indicate that increasing the number of cycles did not alter the response substantially. Of greater significance was the severity of the reversal. Where specimens were subjected to equal deformation levels in each direction (CL loading), the strength degradation was more rapid and severe than when the deflection in one direction was limited (AL loading), as shown in Fig. 4. Both types of loading caused a more rapid decay of strength than for monotonic loading (SL).

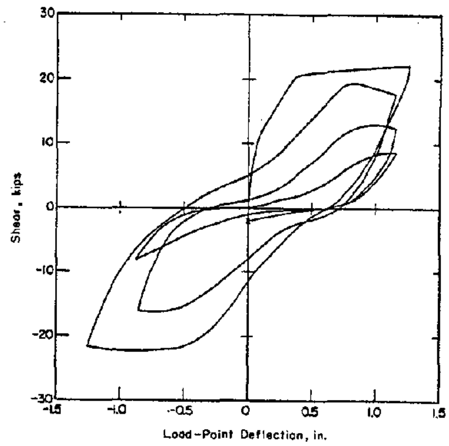

Fig. 3. Shear-Deflection Curves [Ref. 31]

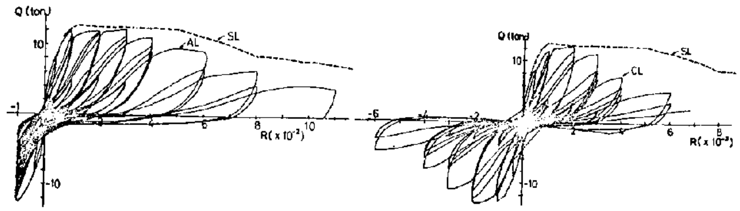

Fig. 4. Influence of Load History [Ref. 31]

FRAME DESIGN CONCEPTS

Weak Beam-Strong Column

The approach to experimental work in reinforced concrete frames reflects, to a great extent, design philosophy. There is a general consensus that hinging and inelastic deformation should be concentrated in flexural members. Columns and joints should be designed to force hinging in the beams. Some fairly complex reinforcing arrangements have been devised [7] to promote the development of hinges at desirable location (Fig. 5).

Biaxial Loading

Frames are generally designed considering that each principal direction resists lateral forces independently of the orthogonal direction. As a result, research has been limited to tests of members or subassemblages of

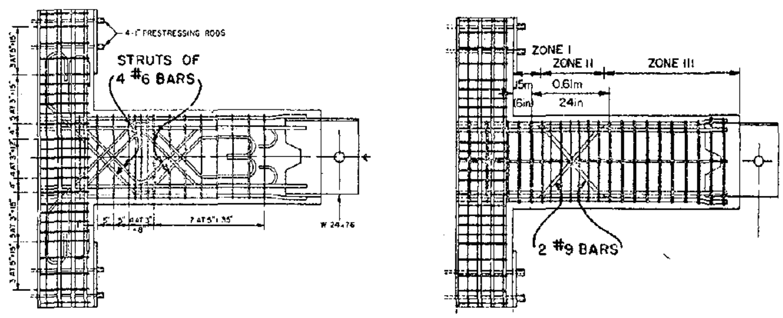

Fig. 5. Special Reinforcement for Development of Hinging [Ref. 7]

planar frames. Analytical studies indicate that two-dimensional response characteristics may be considerably more severe than when only one direction of motion is considered. For example, planar analysis of the Olive View Medical Center [3] did not adequately explain the extent of damage and deformation. Aktan, Pecknold, and Sozen [2] compared 1D and 2D response of a reinforced concrete column subjected to various ground acceleration records. A comparison of the relative displacements of the column for one record (Taft 1952) is shown in Fig. 6. From this study it was concluded that calculations based on one horizontal component of ground motion were unconservative compared with the displacement obtained from a consideration of both components. Takizawa [28] and Selna, et al. [25], have undertaken similar studies to determine the influence of biaxial deformations on reinforced concrete column response. However, there is very little experimental work available to complement the analytical studies. Extensive research, both experimental and analytical, is needed to ascertain the importance of bidirectional loadings and to evaluate the strength of frame structures under such loadings.

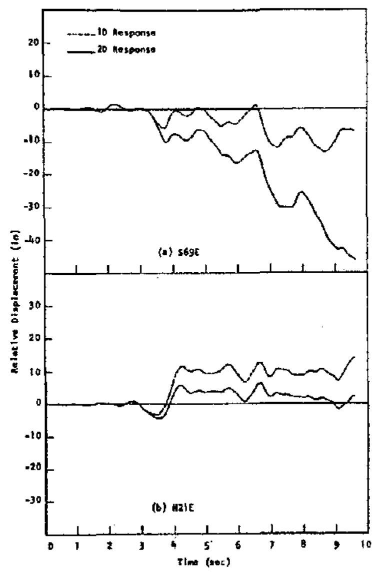

Fig. 6. Comparison of Computed 1D and 2D Response [Ref. 2]

General comments regarding the relationship between design and experiment and the differences produced by test procedures or analytical studies serve to set the stage for a discussion of specific measures of structural performance. Design eventually reduces to questions for which specific answers are needed; e.g., What area of transverse steel and at what maximum spacing?

SHEAR IN BEAMS AND COLUMNS

The performance of members failing in shear under planar lateral forces and axial compression has been studied fairly extensively. Some of the types of test specimens utilized are shown in Fig. 7. An extensive test program conducted in Japan, summarized by Higashi and Hirosawa [16], provides a summary of the types of behavior or mode of failure to be expected in members subjected to load reversals. Using load histories, as shown in Fig. 1(c),

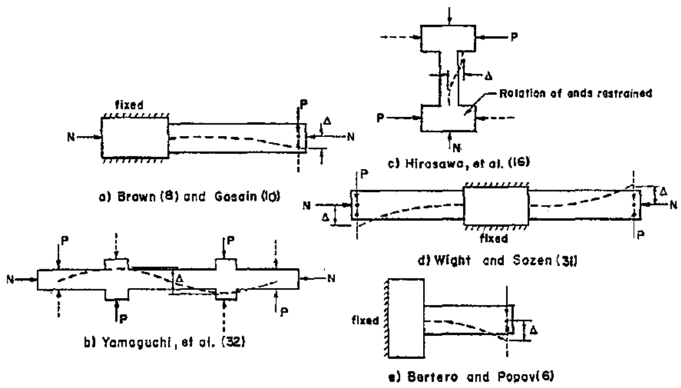

Fig. 7. Type of Test Specimen

the characteristics of the member performance were summarized as follows:

A. Very ductile members failing in shear or buckling of compression bars at large lateral deformations.
B. Ductile members failing in shear, bond deterioration, or bar buckling at ductility ratios (Δ/Δ_y) between 4 and 6.
C. Members reaching yield but failing in shear, bond, or bar buckling in early stages of loading.
D. Members failing in shear or bond before flexural yielding is reached.

Hinging Regions Failing in Shear

Members failing in Cases A, B, and C reach flexural yield strength and then, depending on the severity of loading, fail in shear, bond, or bar buckling. Generally, such members would perform satisfactorily under unidirectional loading. Gosain, et al. [9], proposed an approach for estimating the relationship between severity of loading and shear resistance in hinging regions using the area under the load-deflection curves. Rather than compute the actual area under the curves, a simplified procedure was used. First, the load-deflection curves were normalized with respect to yield values (P/P_y and Δ/Δ_y). From the normalized load-deflection curves, the maximum deflection ratio was determined for each cycle. In the tests considered, the deflection ratio in each direction from the origin was about equal. The work index I_w for each cycle in which the load reached at least $0.75P_y$ was calculated. For the entire load history, the work index was expressed as

$$I_w = \sum_1^n \frac{P_n}{P_y} \times \frac{\Delta_n}{\Delta_y} \approx \sum_1^n \frac{\Delta_n}{\Delta_y} \tag{1}$$

where n = number of cycles with $P/P_y \geq 0.75$ and for simplification P/P_y was taken as unity since values likely will fall between 0.75 and 1.25.

Examination of load-deflection curves indicated that the work index was sensitive to the shear span-to-depth ratio (a/d_c) and to the level of axial load (N/A_c) on the member. Figure 8 shows the pinching of load-deflection curves toward the origin with low shear span-to-depth ratios and with low axial load. To reflect these effects, the work index was modified

$$I'_w = I_w(1 - d_c/a)(1 + 0.0005N/A_{core}), \tag{2}$$

Only core dimensions were used because the outer shell tends to spell away at early stages of loading leaving the core to carry all forces.

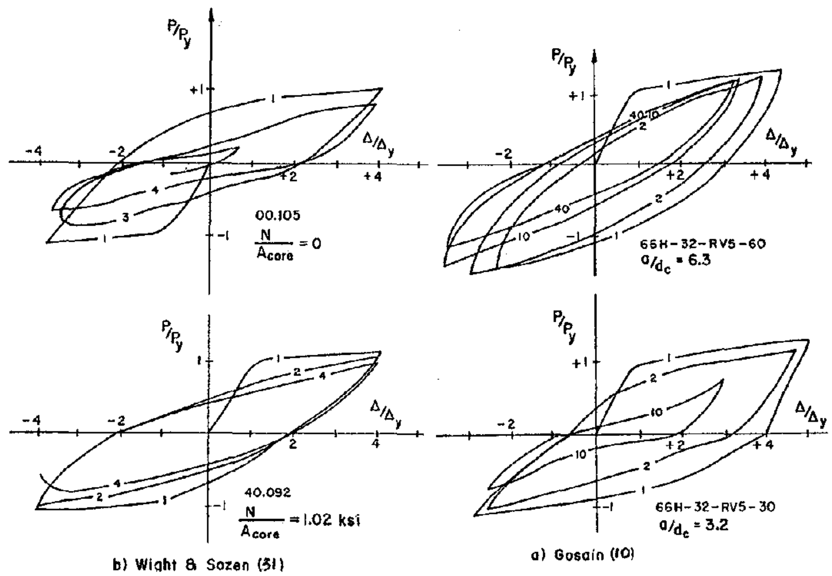

Fig. 8. Influence of Axial Load (N/A_{core}) and Shear Span (a/d_c) on Response

Maximum Allowable Shear Stress. Figure 9 shows the relationship between modified work index and measured ultimate shear stress. A fairly clear trend is observed and a straight line with 90% confidence limits has been drawn to highlight the trend. A similar plot for the relationship between the amount of transverse steel and work index revealed a trend toward higher work indices with high amounts of transverse steel (but with greater scatter).

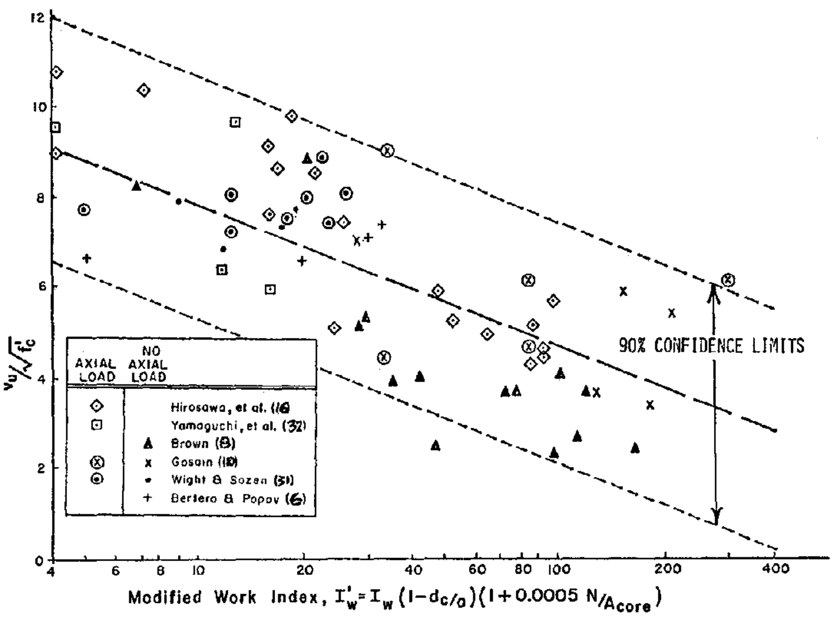

Fig. 9. Variation of Modified Work Index with Ultimate Shear Stress [Ref. 9]

Depending on the performance required, the maximum allowable shear stress on the core can be estimated. Assuming that a performance equivalent to 5 cycles at 5 times the yield deflection is required, shear stresses on the core should not exceed about $6\sqrt{f'_c}$. In addition, the study showed that the transverse steel should provide a capacity approximately equal to that provided by the concrete so that

$$\rho_w f_y = \frac{A_v}{sb_c} f_y \geq 6\sqrt{f'_c} \tag{3}$$

where s is the spacing of stirrups, b_c is the core width, and A_v is the area of transverse steel. These values are similar to those recommended by the other investigators [6].

Buckling of Longitudinal Reinforcement. Failure of hinging regions is often a complex interaction between shear deformation, concrete crushing, bond deterioration, and longitudinal bar stability. Because the shear deformation is primarily across flexural cracks almost normal with the direction of bending [8,10], the reinforcement serves more as confinement for the core than as a shear-carrying element. In addition to confining the core, it binds the longitudinal steel to the core and reduces the unsupported length of compression bars. To provide adequate lateral support, Gosain, et al. [9]

recommended spacings not exceeding 6 longitudinal bar diameters, Bertero and Popov [6] recommend 6 to 8 bar diameters, and Higashi and Hirosawa [16] recommend 8 bar diameters.

Bond Failures. The influence of bond and anchorage will be discussed by Prof. Gergely in another report, but brief mention is made here of problems associated with such failures. Higashi and Hirosawa [16] indicate that using the concept of bond stress was not adequate to explain the failures observed. Rectangular hoops were not as effective as spiral hoops in improving bond characteristics. One approach to improvement of bond in the hinging region is to limit the flexural capacity of the section to some fraction of the shear capacity at the end of the hinge (a distance from the face of the support equal to the effective depth) where bond failures were observed to start. It should also be noted that bond deterioration within the joint aggravates distress in the hinging region. As the bars slip within the joint, flexural cracks in the member widen and reduce the effectiveness of shear transfer across the crack.

It should be noted that where the section is subjected is subjected to tension, similar failures may be produced but very little experimental work has been done regarding sections in tension.

Biaxial Load Reversals. Figure 10 shows some results of a recent study by Okada, et al. [21], on reinforced concrete members under biaxial load reversals and constant axial load. The specimens developed flexural hinges. Under biaxial loading the specimen deteriorated more rapidly, as indicated by the restoring force history.

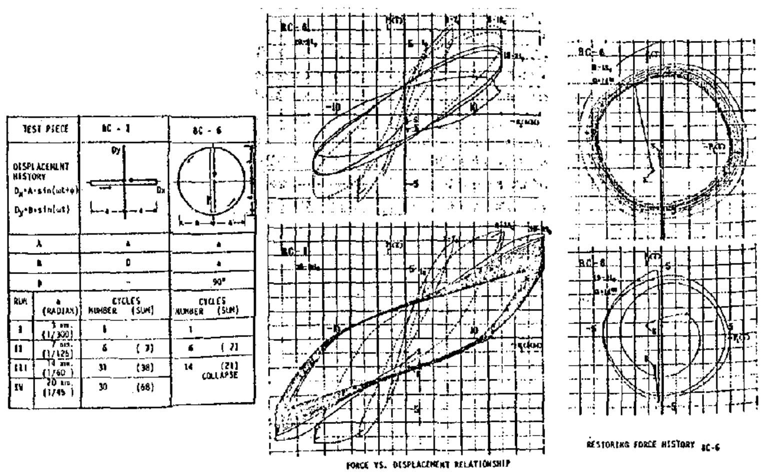

Fig. 10. Biaxial Lateral Loading of Columns [Ref. 21]

Columns Failing in Shear. Where failure occurs prior to the achievement of flexural yielding, the failure is generally of a brittle nature and, under reversed loading, is characterized by a rather rapid degradation of shear strength. Where flexural yielding occurs, adequate ductility is generally obtained; however, hinging in columns is to be avoided in most designs. Therefore, it is essential that the designer also prevent shear failure in the column from occurring before flexural hinging in the beam occurs. Where this cannot be done, the frame will have to be designed for lateral loads based on the maximum shear strength of the columns. Unfortunately, the M-P-V interaction for columns (Fig. 11) has not been adequately described and estimates of shear strength under various combinations of M and P cannot be made at present. Wakabayashi [30] has discussed this problem in some detail.

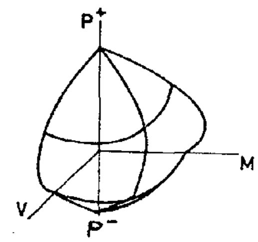

Fig. 11. Interaction of M-P-V

A special problem with regard to shear strength is the scale of the specimen. Large-scale tests are needed but the difficulties in applying large axial loads have prevented investigations of such magnitude. Virtually all existing tests on the shear strength of columns have been conducted on relatively small columns with axial compressive loadings. Because frame structures may be subjected to overturning effects, or vertical accelerations, tensile loads may be produced which will likely reduce the shear resistance of the column. When scale effects, tensile loadings, and bidirectional lateral loadings are all considered, it is clear that this area needs to be studied in depth for an understanding of shear strength which will lead to the development of design recommendations covering a variety of load cases.

BEAM-COLUMN JOINTS

As indicated previously, the basic premise underlying the design of frame structures is that inelastic deformations should occur in the members and the joint integrity should be maintained throughout the loading history. A number of investigators [11,12,14,20,22,29] have examined the problem of beam-column joint behavior under cyclic loadings and current design procedures are based on those studies. It is interesting to note that the approach used for design varies considerably depending on the interpretation of test results and on the joint performance required.

Shear Strength

The design of connections between the beam and the column requires that the joint shear strength be sufficient to fully develop the moment capacities of the member at the joint. If planar action is assumed, the moments and shears acting on a freebody of the joint are as shown in Fig. 12. The shear acting on a horizontal plane through the joint can be expressed as

$$V_j = T_1 + T_2' + C_2 - V_{col}$$

The forces T_1 and $T_2' + C_2$ are a function of the area of longitudinal beam reinforcement provided and the stress reached in that reinforcement. In a design recommendation by ACI Committee 352 [1], a stress of $1.25f_y$ is suggested to reflect the increase in stress due to strain hardening when the beams are subjected to large inelastic deformations. The increase in stress at hinging regions of members subjected to moment gradients has been well-documented [1]. The ACI 352 report suggests that the shear in the joint is carried by the sum of the shear capacities of the concrete and the transverse reinforcement.

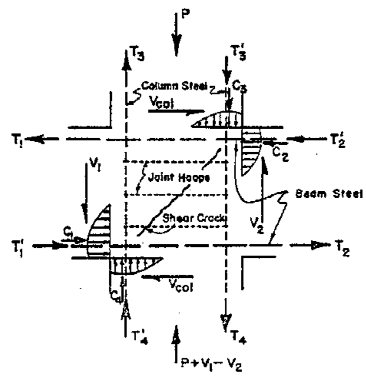

Fig. 12. Freebody of Joint

Concrete Contribution. In the ACI 352 recommendations, the concrete is considered to provide a unit shear stress

$$v_c = 3.5\sqrt{f_c'(1 + 0.002N_u/A_g)} \tag{4}$$

where N_u/A_g represents the compressive stress on the section from axial loads. The equation was originally derived from tests of members and the applicability of the equation to joints is questionable. A review of the test results suggests that joint strength under load reversals is not significantly affected by the magnitude of column load--a conclusion which is reflected in a shear strength equation for the concrete proposed by Meinheit [20].

$$v_c \leq 12\sqrt{f_c'} \tag{5}$$

Park and Paulay [22] recommend that the concrete not be considered in shear strength calculations and that the transverse reinforcement be designed to carry the total shear. This recommendation is based on the behavior of joints under large reversals in which the concrete in the joint becomes severely cracked and distorted and the shear capacity decays rapidly under load reversals finally becoming ineffective.

The ACI 352 report [1] increases the shear stress from Eq. (4) by a factor of 1.4 for the beneficial influence of lateral intersecting beams which cover not less than 50% of the side face of the joint. Meinheit proposes that Eq. (5) be increased by a factor of

$$1 + 0.25\ [(\text{width of beam})/(\text{width of column})] \tag{6}$$

which results in a maximum adjustment of 1.25.

It should be noted that the equations for shear carried by the concrete and for the influence of lateral beams are based on tests in which only planar forces were applied. No information is available on the effect of biaxial loadings nor on the influence of axial tensile forces on the column.

Transverse Reinforcement. The ACI 352 recommendations require that the transverse reinforcement provide the difference between the joint shear and the shear carried by the concrete.

$$v_s = v_j - v_c = \rho_w f_y \qquad (7)$$

where ρ_w is the percentage of transverse reinforcement (A_v/bs) and f_y is the yield strength of the transverse steel. Sugano and Koreishi [26] also suggest an additive approach, but reduce the importance of transverse reinforcement by using $v_s = 2.7\sqrt{\rho_w f_y}$. Meinheit [20] suggests a departure from the additive procedure by adjusting the shear carried by the concrete in Eq. (5) to reflect the confinement from the transverse reinforcement. The shear stress v_c is multiplied by $1 + 6\rho_s$, where ρ_s represents the volumetric ratio of the transverse reinforcement. Some minimum value of ρ_s is needed in all cases, probably about 0.01. (For a square column with perimeter transverse hoops, $\rho_s \approx 2\rho_w$.)

Comparison of Design Approaches. Figure 13 shows the shear strength of the joint relative to the amount of transverse reinforcement. As can be seen, the additive approach in which the strength of the joint increases linearly with an increase in transverse reinforcement produces very high shear strengths. The Japanese approach is additive, but with diminished reliance on transverse reinforcement. The approach proposed by Meinheit leads to values about equal to those suggested by Sugano and Koreishi.

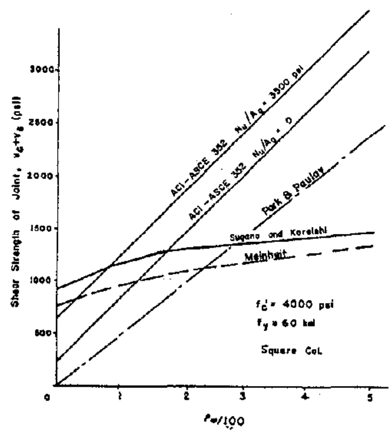

Fig. 13. Shear Strength of Joint vs. Transverse Reinforcement

The differences reflect the manner in which the equations were derived. The linear relationships, ACI 352 [1] and Park and Paulay [20], are extensions of beam shear equations, modified for the effects of reversed loadings. The beam shear equations reflect classical shear theory--truss analogies. The equation described by Sugano and Koreishi is an extension of empirical studies carried out by Arakawa [4] adjusted for beam-column test data. The shear strength proposed by Meinheit is based on a regression analysis of test data.

Anchorage of Reinforcement

In beam-column joints anchorage of reinforcement becomes a concern because of the adverse effects of loading reversals on stress transfer between steel and concrete. At interior joints, anchorage is a problem in relation to stiffness and energy absorption, while at exterior joints strength, as well as stiffness and energy absorption, may be affected by anchorage characteristics.

Interior Joints--Straight Bars. Figure 14 shows the forces acting on a freebody of the joint if strain compatibility is maintained. Figure 15 is a record of strains in the longitudinal beam bars at the column face of a beam-column joint under cyclic loading [20]. Note that at load stage 27 the #8 bottom bars, which should be in compression, are in tension in the east beam as a result of a loss of bond through the joint produced by the large inelastic tensile deformations of the #8 bars on the opposite face (west beam). The freebody for the section considering the loss of bond is shown in Fig. 14. Figure 16 shows the inability of the east beam to develop yield moment at load stage 27 because the #8 bottom bars were not carrying compressive stresses.

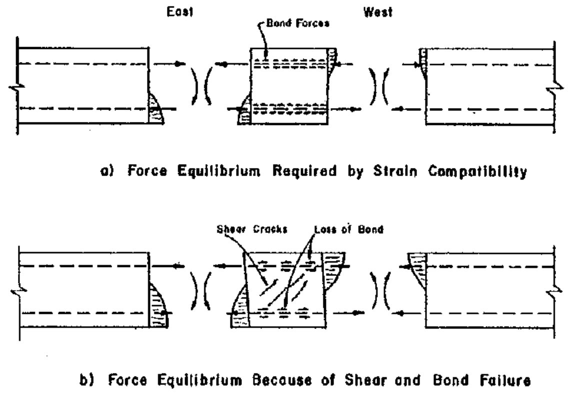

Fig. 14. Forces on Joint-Bond Failure

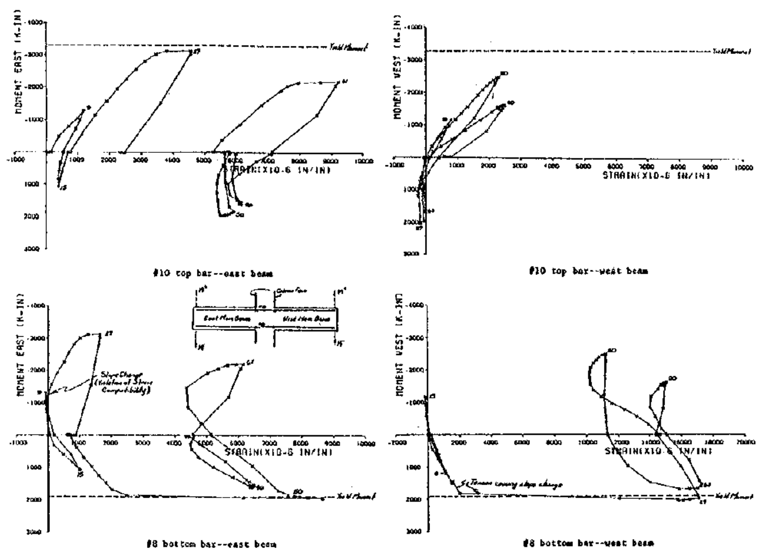

Fig. 15. Measured Strains in Longitudinal Bars through Joint [Ref. 20]

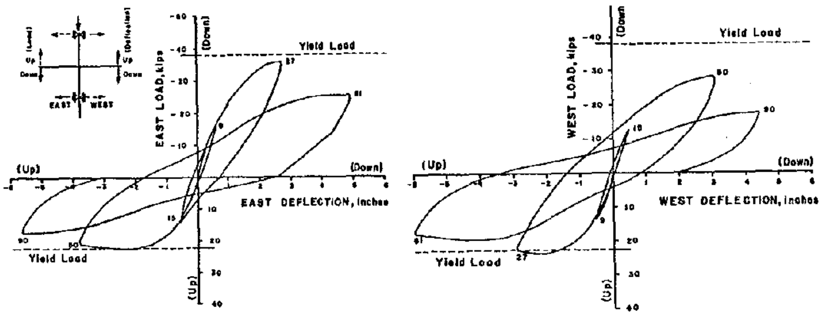

Fig. 16. Load-Deflection Curves [Ref. 20]

Popov and Bertero [6] have observed the same phenomena and have measured bar slip in the joint versus applied beam moment (Fig. 17). Ma, et al. [18] describe an approach for modeling bond behavior under cyclic loading which explains some of the observed response. Popov, et al. [24] have tested special reinforcing arrangements to reduce the problem of bond loss through beam-column joints. Hinges were forced away from the column providing greater anchorage and development lengths. Figure 5 shows some of the reinforcing arrangements. Additional work on this topic is continuing at the University of California-Berkeley.

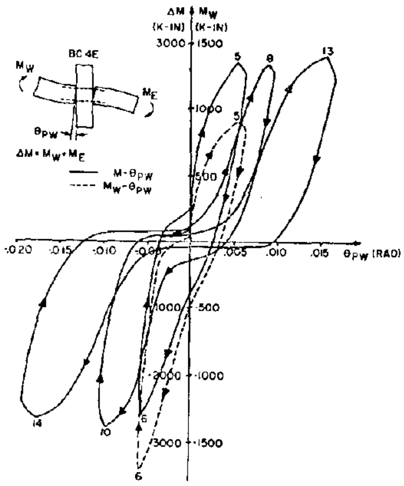

Fig. 17. Beam Moment vs. Bar Slip [Ref. 6]

Studies by Hassan and Hawkins [13] on simplified models also exhibit progressive bond deterioration under cyclic loading and stress the importance of load history on response.

Exterior Joints--Hooked Bars. The strength of beams at exterior joints is dependent on the development of stresses at critical sections using hooked bar anchorages. A series of tests simulating exterior beam-column joints has been reported [19,23] which provide data for the derivation of design equations for hooked bar anchorages. The proposed embedment length ℓ_{dh} for a hooked bar is

$$\ell_{dh} = 0.2 d_b f_y \sqrt{f'_c} \tag{8}$$

where ℓ_{dh} includes straight lead embedment and radius of bend, and d_b is the diameter of the anchored bar. Adjustments to ℓ_{dh} reflecting the depth of concrete cover or transverse steel are proposed. The equation is based on tests in which the load was monotonically increased to failure; however, limited additional work was done in which the bars were subjected to cyclic loading, Fig. 17. Hawkins and Hassan [13] tested hooked bars embedded in blocks of concrete simulating a joint and concluded that the hook adversely affects energy absorption because the hook tends to break up the concrete around it and destroys the integrity of the joint. The use of mechanical anchors to replace hooked bars has also been studied [17], as shown in Fig. 18.

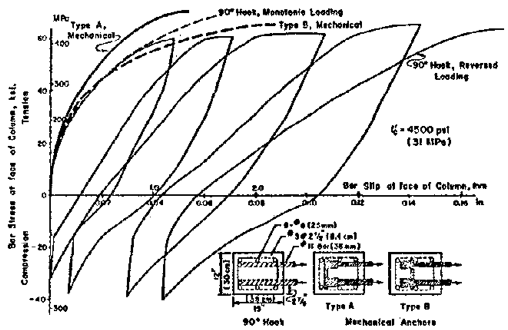

Fig. 18. End Anchorage Behavior [Ref. 17]

Interaction of Shear and Anchorage

Because the behavior of the joint under severe load reversals is heavily influenced by the amount of cracking and spalling which occurs in the joint, it is essential that future research be done on full-scale specimens which are realistic models of structural joints. Studies have been conducted examining one aspect of behavior--shear or anchorage. It is clear, however, that the severity of shear cracking will influence anchorage behavior. Likewise, spalling and crushing produced near a hooked bar will adversely influence shear strength. Experimental work is needed to explore the interaction between shear and anchorage in the beam-column joint.

EXTANT PROBLEMS

In the preceding discussion, a limited review of the studies of the behavior of elements and subassemblages of reinforced concrete frames provides an indication of current understanding. It is likely that this writer has missed or omitted some studies which help elucidate, and in some cases obfuscate, problem areas. Notwithstanding these shortcomings, the following research areas appear to offer the most promise of returns (in the form of design guidance) in the near future.

Loading Criteria

Top priority should be given to experimental studies of subassemblages or frames which are correlated with analytical studies to determine response requirements under all components of ground motion. Much attention has been given to isolation of specific problems by simplifying test specimens and analytical models. Tests have been conducted on simplified specimens under simplified loadings to determine bond-slip relations, for example, and the observed behavior expressed in mathematical terms. However, when the bond-slip relationships are incorporated into an analytical procedure for complex inelastic behavior of subassemblages, the model may no longer represent the behavior observed.

The use of simplified loadings is a necessity, because no well-defined "earthquake loading" exists for a member or a subassemblage. At one location in a structure, a hinge may need to withstand five load reversals at five times the yield displacement, but it may not be necessary for all hinges to meet the same requirement and if they do not, the consequences of such inadequacies need to be examined.

A prime concern in this area is the incorporation of biaxial lateral loadings into loading histories. Since horizontal ground motion will not coincide with a principal axis of the structure, tests should reflect the skewed nature of the loading.

As mentioned previously, very little work has been done on the behavior of members, especially columns, under tensile loadings or under variable axial loadings ranging from tension to compression.

Realistic Test Specimens

Large-scale tests of frame subassemblages and complete frames simulating real structures must be conducted. For example, planar frames rarely exist but a number of planar beam-column joints or portions of joints have been tested with little attention to the effects of cast-in-place slabs or orthogonal frames on the behavior of the specimen. Loads on the test specimens must reflect the three-dimensional nature of loading likely on the real structure. Where shear and anchorage govern behavior, results of reduced scale tests cannot always be extrapolated to reflect full-scale inelastic behavior.

Specific Problem Areas

Within the broad categories listed above, specific problem areas have been identified which deserve special mention: (1) shear strength of columns under cyclic biaxial lateral loads and varying levels of axial load, and (2) interaction of shear and anchorage in joints and connections under three-dimensional loadings.

CONCLUDING REMARKS

In this report the vast amount of research done on member and subassemblage behavior has been skimmed to provide some indication of progress to date and to outline areas needing attention. It is suggested that future work emphasize higher order tests--that is, frame subassemblages or complete frames subjected to three-dimensional cyclic loadings. Such tests will provide response characteristics which can be used to gauge the applicability of results obtained from simplified models and loading histories and to judge the adequacy of current design procedures.

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A METHOD FOR DELAYING SHEAR STRENGTH DECAY OF RC BEAMS

by

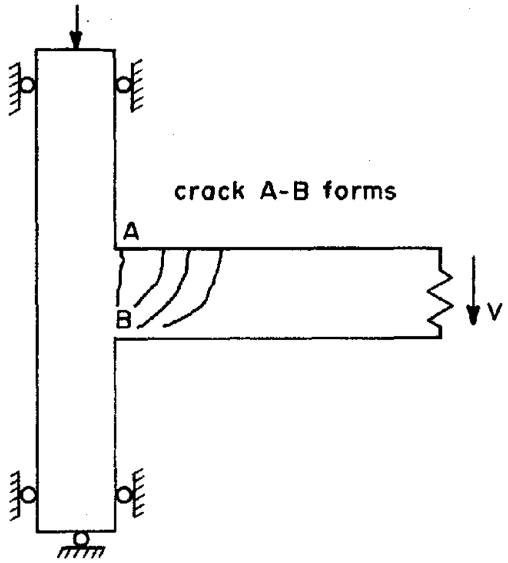
Charles F. Scribner, Graduate Student, and
James K. Wight, Assistant Professor
Department of Civil Engineering
University of Michigan, Ann Arbor

INTRODUCTION

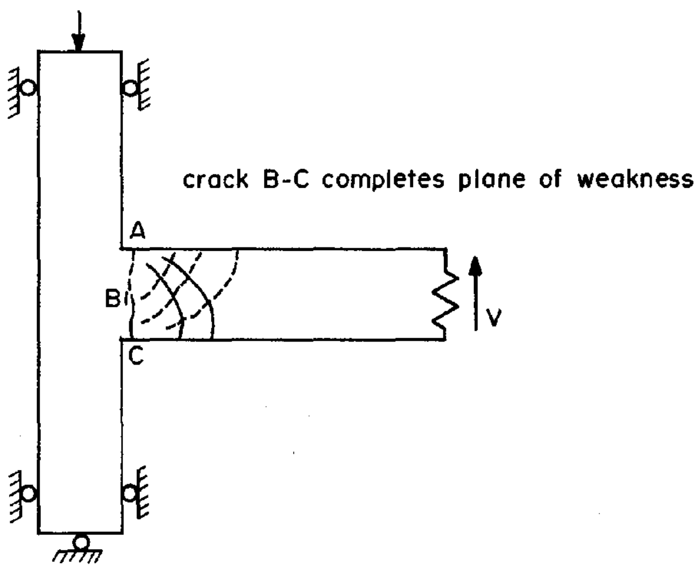
Buildings situated in regions of frequent seismic activity must be capable of dissipating seismic energy without experiencing severe structural damage. Because it is usually not possible or practical to design main structural members for elastic behavior at all levels of earthquake excitation, inelastic action in frame members must be anticipated. To reduce the severity of damage inelastic rotations of frame members should occur in beams or girders rather than colums. Current building practice [1,2] reflects this logic with a "strong column-weak beam" design philosophy. In addition current practice requires that members be capable of developing their flexural capacity through several cycles of load reversal in the inelastic range.

The analysis of this problem has been only recently considered. It was not until the mid 1960's that researchers subjected reinforced concrete members to large flexural reversals. At first efforts were directed toward analysis and design of such members to maintain suitable flexural capacity, but it has been shown [3,4,5] that shear strength decay becomes the dominant factor in beam performance. The primary goal of a research program in progress at The University of Michigan is to investigate the use of intermediate longitudinal reinforcement in flexural hinging zones to delay or prevent a reduction of shear strength and stiffness during large load reversals.

The present UBC [1] attempts to prevent shear failures by imposing very close spacing of ties and stirrups in regions of high shear. Nevertheless, flexural cracks may form a plane of shear slippage perpendicular to the member axis after only a few cycles of inelastic deflection reversal. The formation of such a slip plane is illustrated in Fig. 1. When loaded in one direction, a flexural crack may form. As loading direction is reversed, a second flexural crack may form, intersecting the first crack. The resultant plane of weakness may not intersect a vertical tie or stirrup, regardless of spacing. As a result, shear is now resisted at this point by sliding friction between adjacent sections and by dowel action of reinforcing bars. Intermediate longitudinal bars will provide an additional tension force which will reduce the width of such cracks and will consequently increase the sliding friction.



Formation of Cracks during First Inclastic Loading



Additional Cracks Formed during Load Reversal

Fig. 1 Typical Crack Pattern

A second problem develops due to the formation of intersecting inclined cracks in this zone of inelastic rotation. The intersecting cracks essentially divide the region into a matrix of rectangular blocks and the stability of this region in successive cycles will be governed by the confinement provided by reinforcement. The intermediate longitudinal bars will definitely provide additional confinement within the hinging region.

DESCRIPTION OF SPECIMENS

A T-shaped specimen representing an exterior beam-column connection was used in testing. Figures 2(a) and 2(b) show general specimen shape and reinforcement pattern for critical sections. The type of deformations the subassemblage may be subjected to during an earthquake was approximated by holding the column at its points of contraflexure above and below the beam and deflecting the beam tip up and down several times to maximum deflections of four to seven times its yield deflection. Various physical parameters of the specimens are given in Tables 1 and 2.

Specimen size and shape were dictated by two factors. First, this general specimen size had been used by previous researchers and any results obtained could be compared to previous results. Second, beam sizes were chosen to provide a range of shear span to depth ratios to test the importance of gross shear stress intensity in controlling shear behavior.

Design and placement of shear reinforcement was the most important phase of specimen design. For each beam size tested, one specimen used shear reinforcement as recommended by the Building Code Requirements for Reinforced Concrete (ACI 318-71) [6]. A second specimen used supplementary intermediate longitudinal shear reinforcement in addition to the reinforcement used in the standard beam. As there were no guidelines for size of intermediate reinforcement, bars were chosen using stirrup and beam sizes as guides. Size and placement parameters for all reinforcement are given in Table 2.

SPECIMEN TESTING

Each specimen was tested in accordance with a predetermined loading pattern. The pattern was chosen to simulate the amount of inelastic activity the beam-column joint might encounter during a severe earthquake. This pattern is shown schematically in Fig. 3. The first two cycles to approximately half the yield moment were included to check the operation of various data gathering instruments.

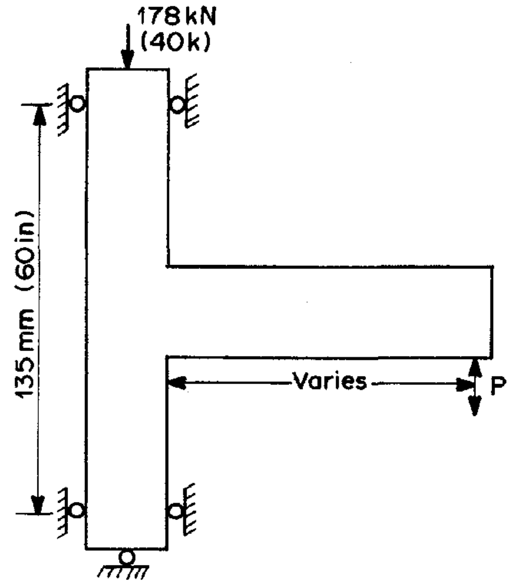
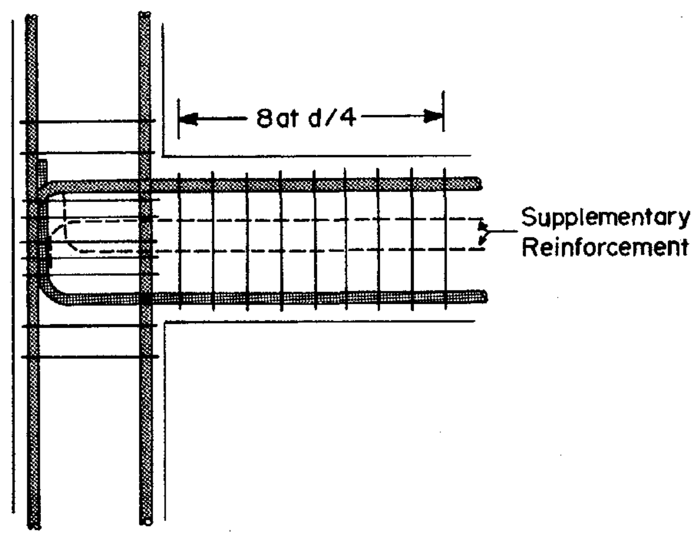


Fig. 2a General Testing Configuration



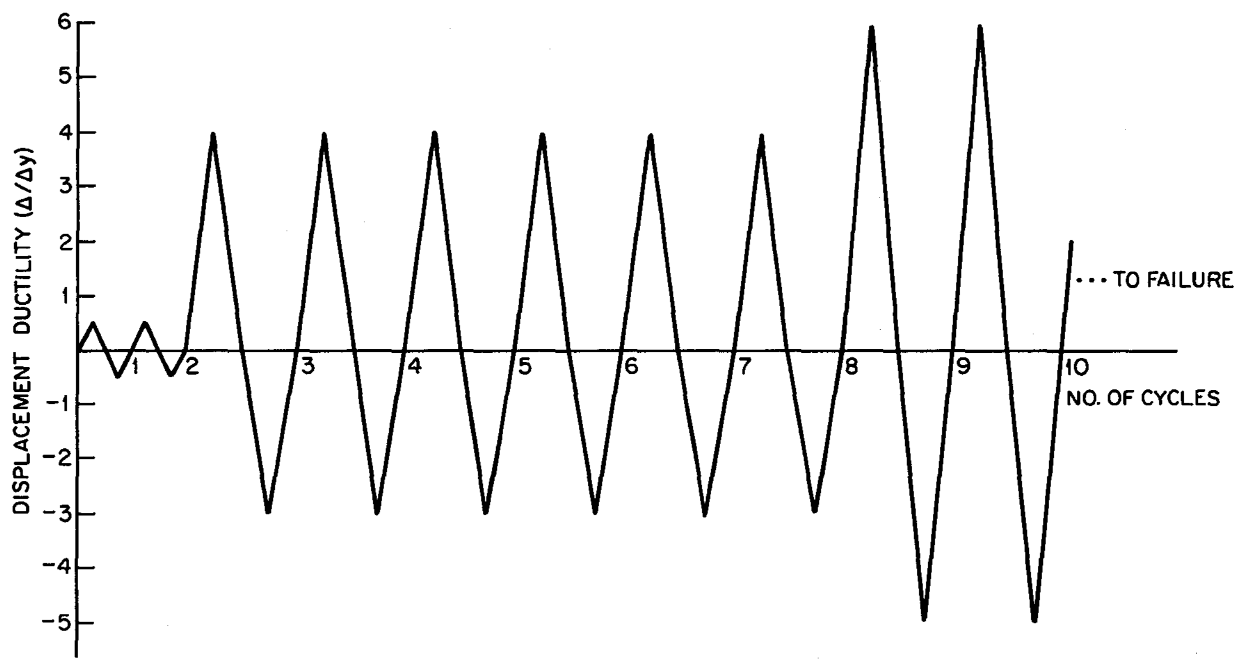


Fig. 3 Loading Pattern Specimens 1 - 4

After imposing six cycles of load displacement ductilities of 4 in the positive direction and 3 in the negative direction, larger displacements were imposed. For these larger cycles a maximum travel of the loading ram was used as a limit in the positive direction (approximately a ductility of 6.5) and 5 times yield deflection was used as a negative limit. Cycling continued until the beam failed or it became clear that the specimen's behavior had stabilized. In every case a column axial load of 178 kN (40 kips) was applied prior to any loading of the beam and this column load was held constant throughout testing.

SOURCES OF TEST DATA

In order to quantitatively evaluate the performance of each beam, several types of instrumentation and sampling were used. Beam tip load and deflection were continuously recorded on an X-Y plotter.

Strain gages were also used to determine steel strains at selected points during loading. Figure 4 shows the position of strain gages. The pattern of gage placement was designed to indicate stress levels at various points within the hinging zone as well as to monitor strains within the beam-column joint and indicate the effectiveness of main reinforcement anchorage.

Four linear variable displacement transducers (LVDTs) were positioned in the region of anticipated beam hinging to monitor shear strain, flexural rotation, and elongation within the hinging region. The location of these four transducers is shown in Fig. 5.

In addition to these three methods of measuring beam response during testing, it was important to test samples of the materials used in construction of the specimens. Three standard compression cylinders were tested in conjunction with each specimen to determine concrete ultimate compressive strength. Samples of each reinforcing bar were also tested to determine yield stress and ultimate tensile strength. Average results of these tests are given in Table 1.

TEST RESULTS

Most of the results discussed here are for specimens 1 through 4. The testing of specimens 1A through 4A has just been completed and the data is not yet in a usable form. Additional specimens 5 through 10 are to be tested in July, 1977.

Specimens 1 and 2 are cycled through the entire predetermined loading pattern and both specimens showed stable

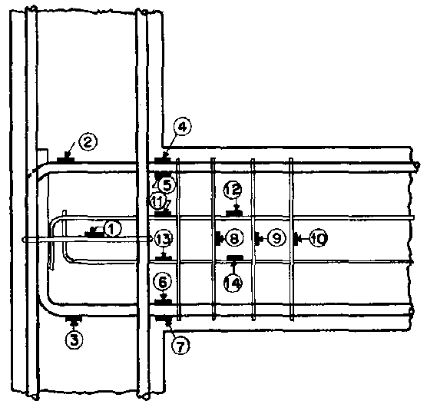


Fig. 4 Strain Gage Location

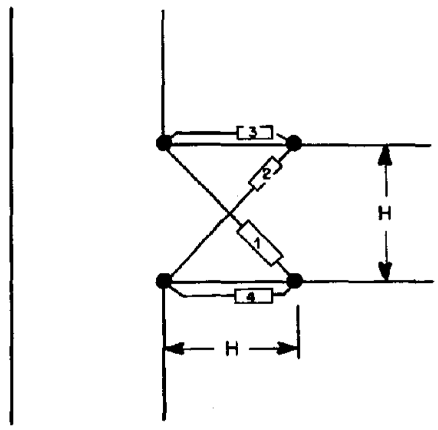


Fig. 5 LVDT Placement to Monitor Deformations in Beam Hinging Zone.

behavior at the conclusion of testing. Specimen 2 dissipated more energy for a comparable number of cycles at comparable displacements, but this could have been anticipated because the intermediate reinforcement augmented the moment capacity. Load-deflection curves for these specimens are shown in Figs. 6 and 7.

Specimens 3 and 4 experienced much more shear strength decay. Specimen 3 deteriorated rapidly and was able to sustain only nine inelastic cycles before failure in shear and torsion. Figure 8 shows the load-deflection curve for this specimen. Two important relations can be observed in this figure.

First, shear strength deterioration with accompanying loss of energy dissipation potential took place even at a relatively low displacement ductility (4) as shown by significant "pinching" of the load-displacement curves.

Second, the load necessary to produce any given displacement dropped gradually during the first six cycles of loading. In contrast, stiffness drops rapidly. The hysteresis curve shows that very little force is required to move the beam tip near the zero load position even in the third and fourth cycles.

Torsional instability was evident in the final stages of testing of specimen 3. This was a byproduct of the loss of shear strength. Several planes of weakness had formed perpendicular to the axis of this specimen and all components of load resistance had been lost parallel to such planes. As torsion is resisted by shearing forces along such planes, any loss of shear strength will result in a loss of torsional strength as well.

Specimen 4 experienced the largest shear stress of any beam tested, but was able to undergo eleven inelastic load cycles before failure. Figure 9 shows the load-deflection curve for this beam. The behavior is far more stable in each cycle than for the comparable beam without intermediate reinforcement (specimen 3). Here, the hysteresis loops formed by the load-deflection curves are stable during cycles to four times yield deflection, i.e., cycles three through eight. There was very little drop in load required to provide a given deflection and more important there was very little drop in stiffness near zero load position of the beam. When larger deflection cycles were initiated, stiffness was maintained for one full cycle. Stiffness degraded at approximately the same rate in this beam as it did in specimen 3 during the final load cycles.

After testing, free or disintegrated concrete was removed from the hinging region to determine the character of concrete breakup in this region. Specimens 1 and 2 preserved large intact blocks which were interspersed with much smaller chips. Intact blocks in the hinge of specimen 3 were somewhat smaller but still on the order of size of the beam core. Specimen 4 was unique

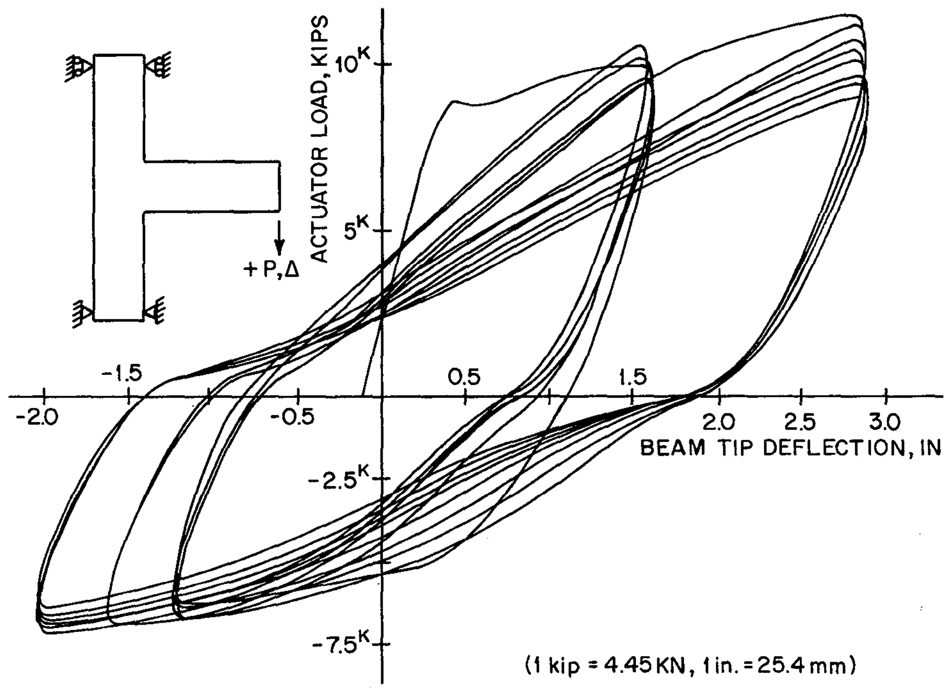


Fig. 6 Beam Shear vs Displacement Curve - Specimen 1

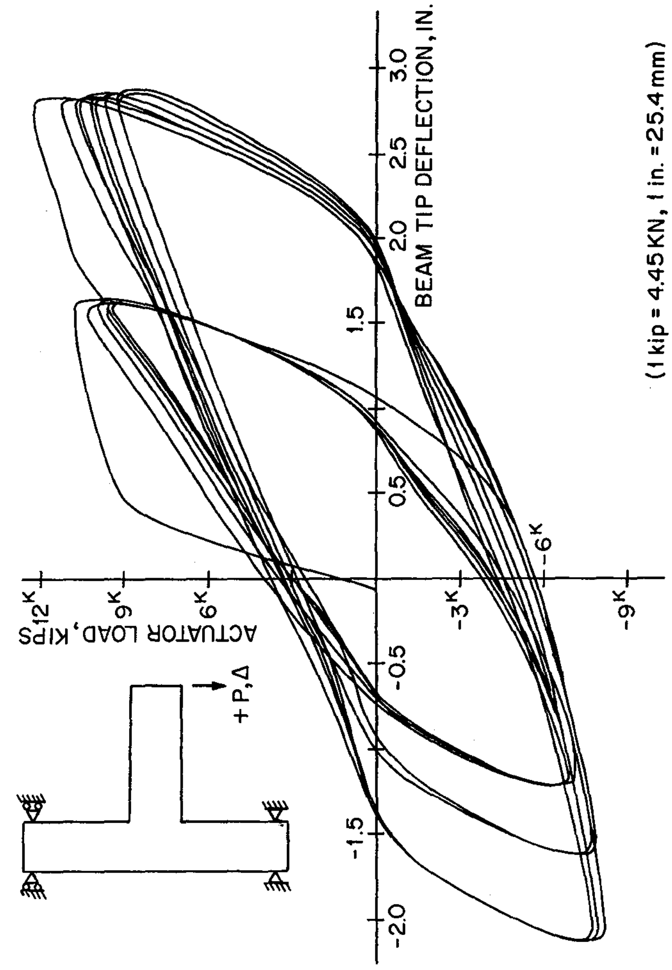


Fig. 7 Beam Shear vs Displacement Curve - Specimen 2

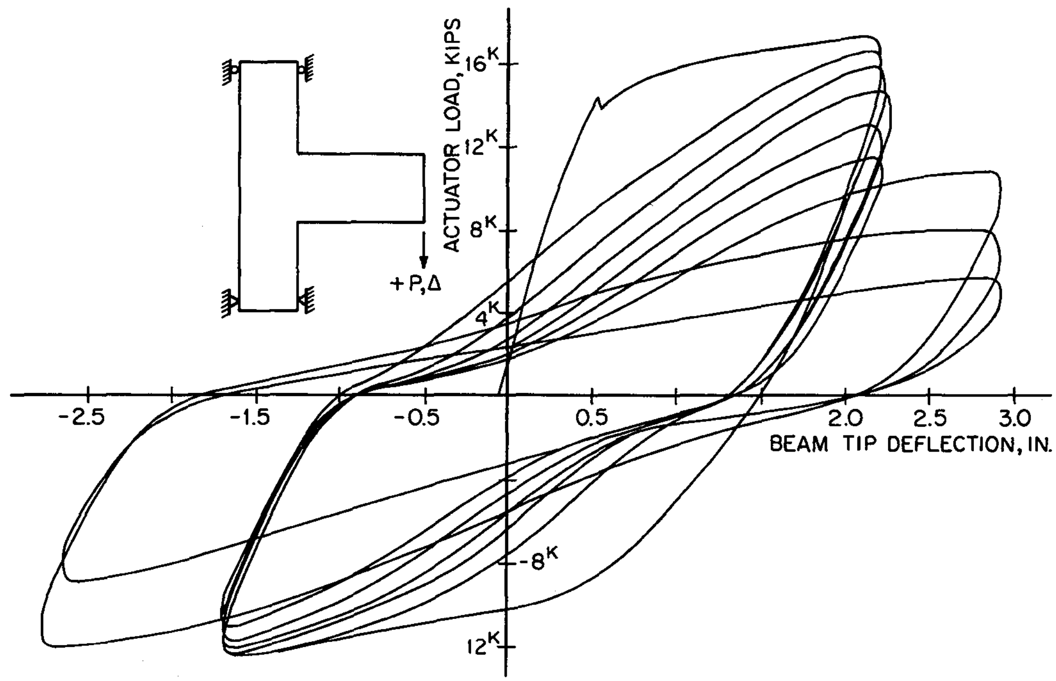


Fig. 8 Beam Shear vs Displacement Curve - Specimen 3

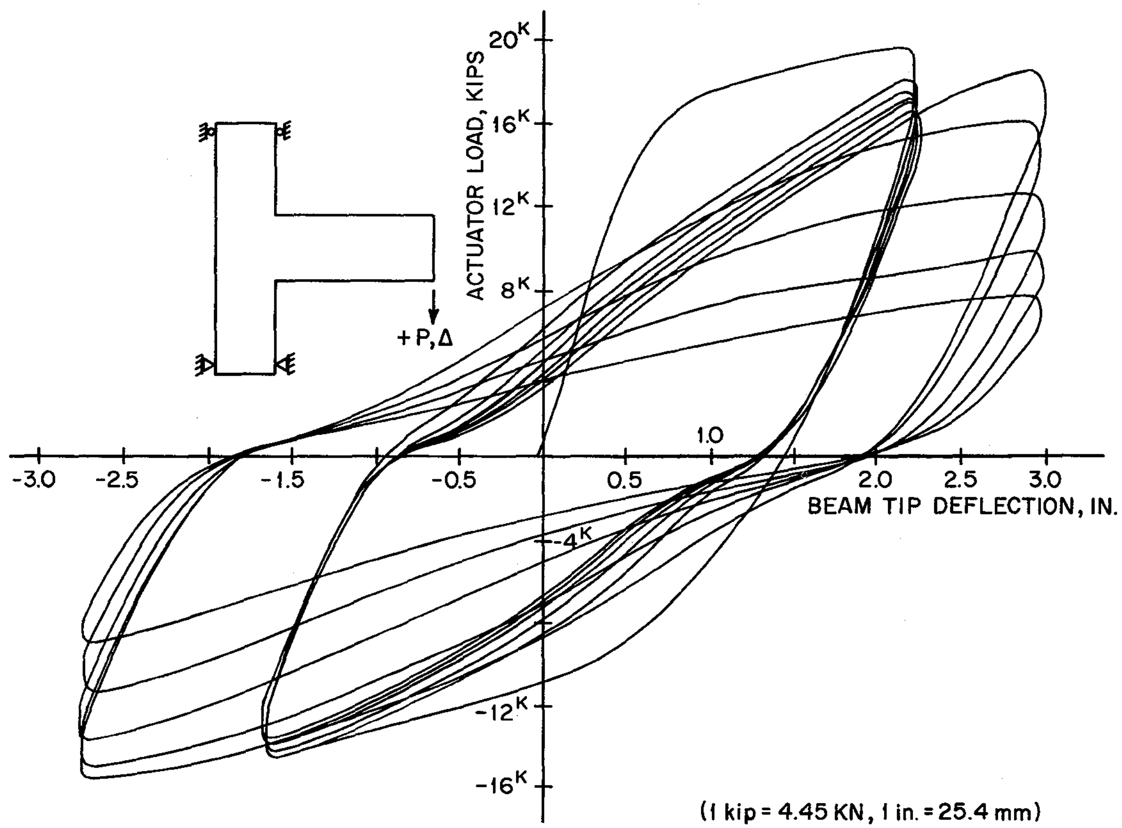


Fig. 9 Beam Shear vs Displacement Curve - Specimen 4

among all specimens, with almost all concrete in the hinging region having been crushed small enough to remove through the ties. This indicated that the confinement provided by the supplementary intermediate reinforcement had been important in prolonging satisfactory shear behavior.

When testing each specimen, the yield point was assumed to be the point during the first large cycle when a sudden change in slope occurred. A comparison between measured and calculated yield moments is given in Table 3.

Strain gage data, in conjunction with results obtained from reinforcing bar tensile tests, helped to explain several aspects of specimen behavior. Strain measured in gages attached to main reinforcement at the column face (gages 4,5,6&7) indicated that the visual estimation of yield point was very accurate for all beams. As expected, strains measured at the visually selected point were higher than the actual yield strain because straining of bars could not be stopped between the time the yielding trend was observed and strain readings were taken. Figure 10 shows the load-strain plot for specimen 3, gage 4. The estimated yield point corresponds to a point somewhat higher than the actual yield strain.

Strain gage data also gave an indication of how the reinforcement carried flexurally induced loads. Figures 10 and 11 show beam load vs strain plots for gages 4 and 7 respectively of specimen 3. Gage 4 represents strains measured in top reinforcement (3-#6 bars) and gage 7 represents bottom bar strains (3-#5 bars). The results from gage 4 indicate that after top bar yielding occurred, a permanent elongation was present in those bars until the largest cycles of negative deflection were imposed. Unclosed concrete cracks must be assumed to have been present in this case. The absolute strain level at maximum positive deflection decreased with each loading cycle, indicating that shear rather than flexure was responsible for an increasing amount of beam tip deflection.

Because smaller bars were used for bottom reinforcement, higher strains were expected in these bars than in top bars. Figure 11 shows that such strains were present. Maximum strains reached by bottom reinforcement were nearly double those reached by top bars.

The results shown in Figs. 10 and 11 clearly indicate that after the first few cycles the entire beam moment at the face of the column was carried by a couple between the top and bottom layers of longitudinal steel. The curves in both Figs. 10 and 11 are smooth with no indication of a flexural crack closing and the smaller bottom bars (Fig. 11) are strained to higher levels than the larger top bars as required for equilibrium.

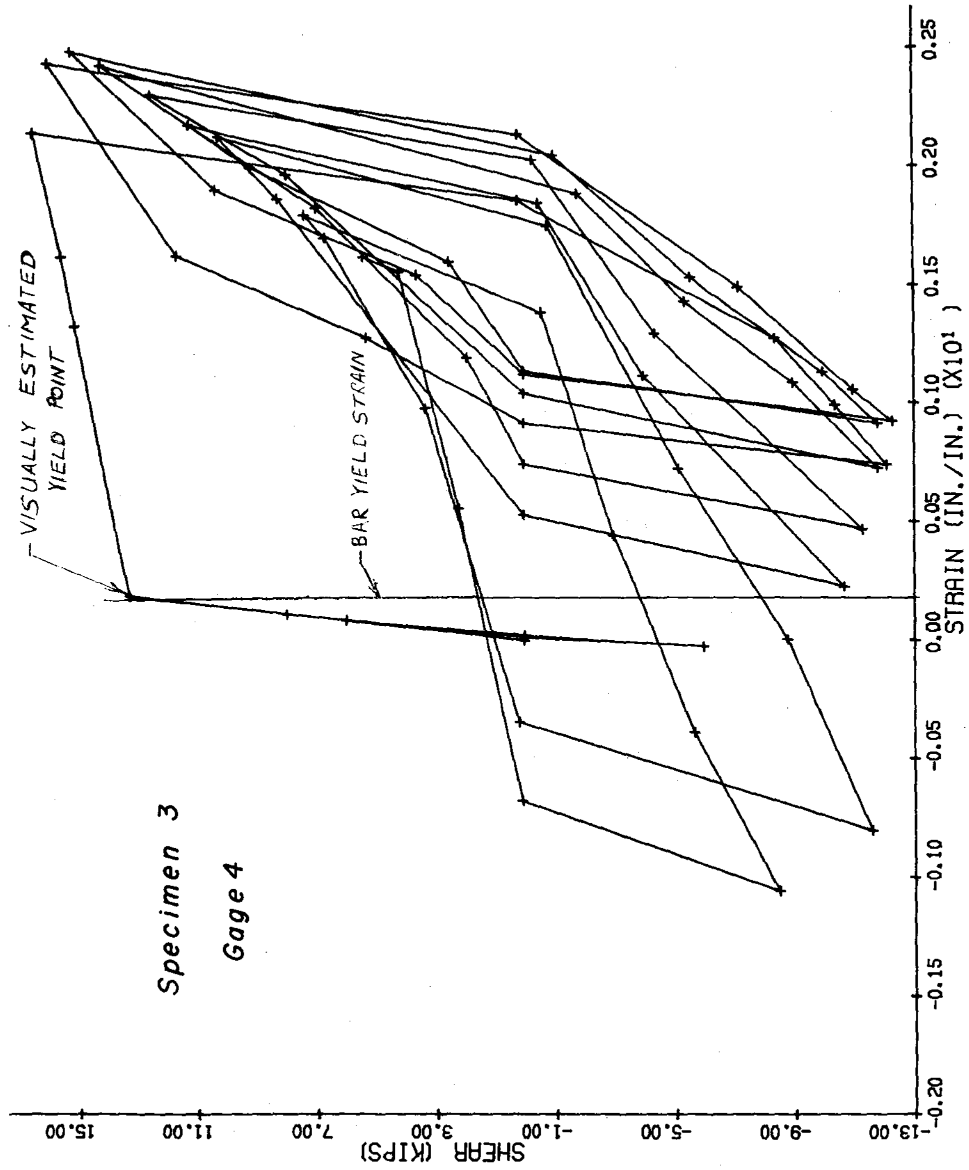


Fig. 10 Beam Shear vs Beam Longitudinal Bar Strain - Specimen 3, Gage 4

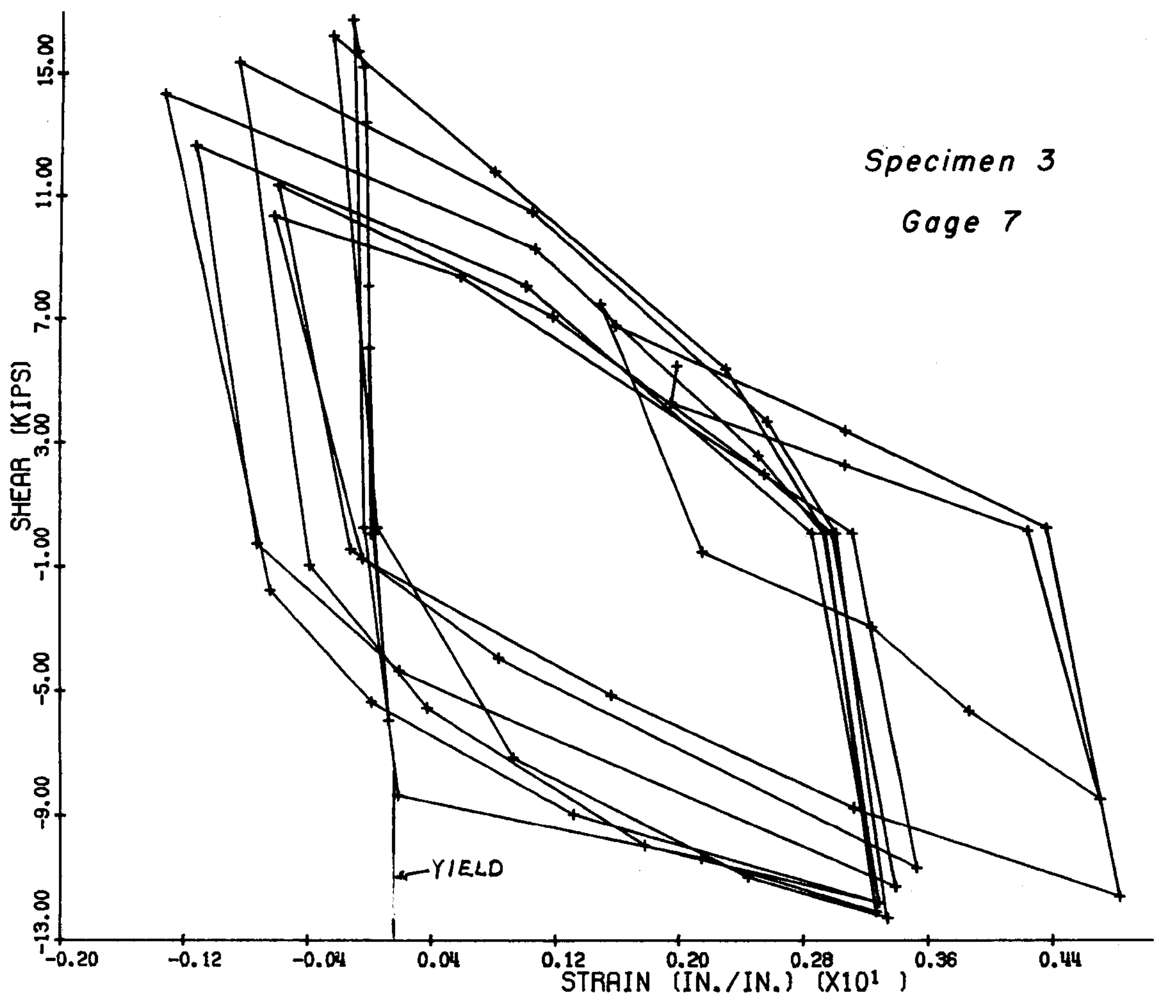


Fig. 11 Beam Shear vs Beam Longitudinal Bar Strain - Specimen 3, Gage 7

Strains measured in gage 1, located on the center beam-column joint tie, gave an indication of shear stress in the joint at first cracking. Figure 12 shows the beam displacement vs joint hoop strain for specimen #4, which was subjected to the greatest shear. It is clear from the jump in strain that first cracking took place at loads between 8 and 11.3 kips (loads obtained from Fig. 9), or shear stresses in the joint between 459 and 649 psi. These stresses were computed using the Recommendations [2] for computing area over which joint shear should be assumed to act, width having been taken between the outsides of joint ties and depth taken as the distance from column face to the centerline of steel near the opposite column face. A survey of gage 1 data in all specimens showed that in no case did the tie reach its yield stress and in no specimen did any detectable deterioration take place in the beam-column joint. A summary of the shear stress in the joint at first cracking is given in Table 3.

Strain gages placed on beam ties in the hinging region are less easy to interpret, but are important in describing the problems encountered in attempting to carry large shears in cyclically loaded members. In only a few cases was yield strain exceeded in these ties (gages 8,9&10). This was expected because ties were not required for strength in any of the beams tested but were placed in accordance with ACI seismic provisions. Figure 13 shows an example plot for specimen 3 gage 10. Although this tie did reach yield during the first cycle of load, it remained at strains less than the yield strain for subsequent larger cycles. Other ties in the hinging region showed similar response.

Load vs flexural rotation and load vs shear strain plots derived from LVDT data provided a measure of the contribution of flexural rotation and shear strain in the hinging zone to overall tip deflection. Figure 14 shows the load vs flexural rotation plot for specimen 3. During the first six cycles of load, flexural rotations were stable. When displacements were increased, however, greater flexural rotations were not obtained in the positive direction. The increase in deflections for these cycles resulted primarily from shearing, not rotation in the hinging zone.

Figure 15 shows the load vs flexural rotation plot for specimen 4. Rotations at all levels of displacement were stable. When displacements were increased to their maximum values, rotations increased as well. Deflections at all levels resulted from flexural rotation as well as shearing strain.

Load vs shear strain diagrams for specimens 3 and 4 are shown in Figs. 16 and 17. It is apparent that shear strains at maximum deflections increased with each cycle of loading for specimen 3 and shear stiffness decreased with each cycle. In contrast, specimen 4 showed stable levels of shear strain for

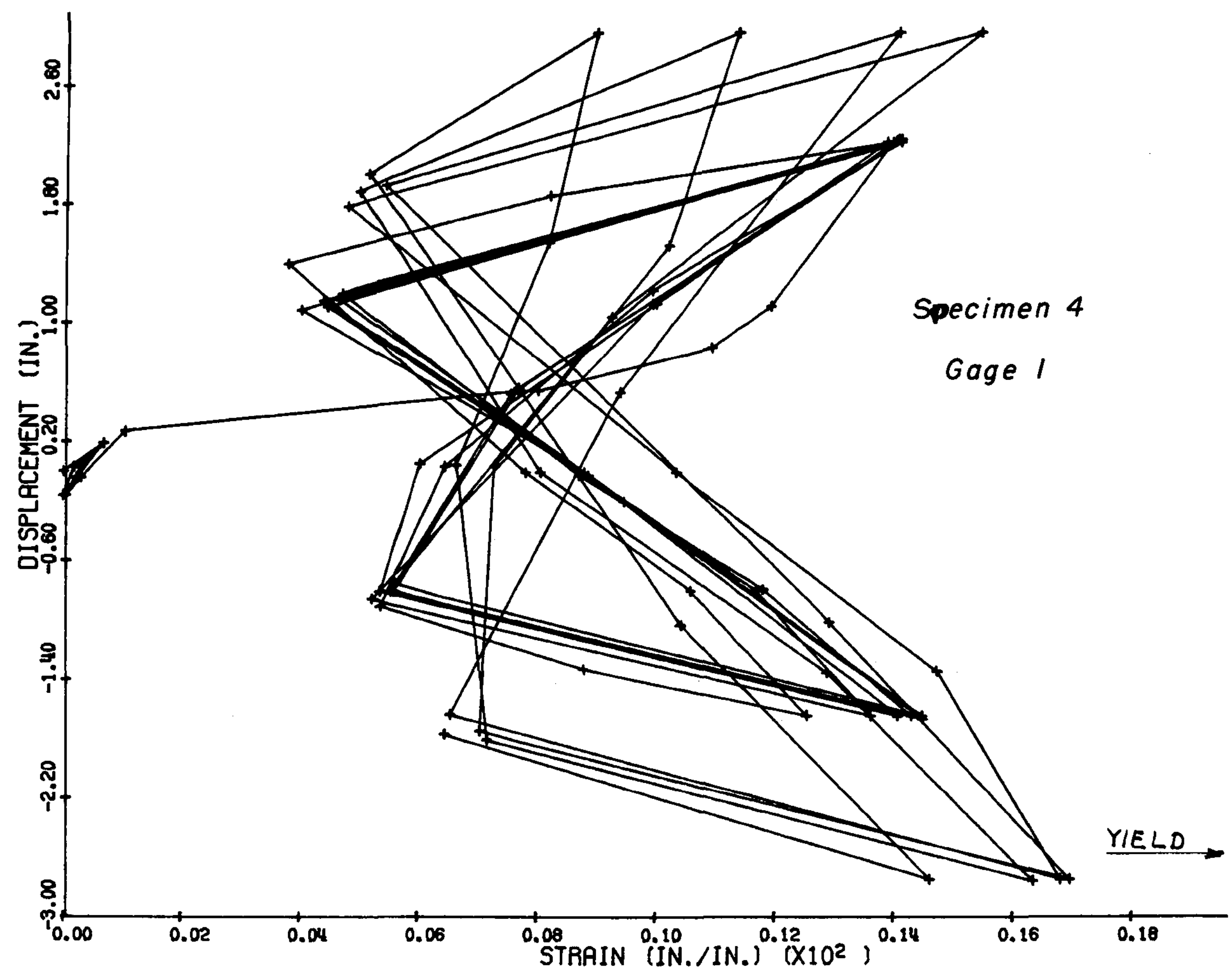


Fig. 12 Beam Displacement vs Joint Hoop Strain - Specimen 4, Gage 1

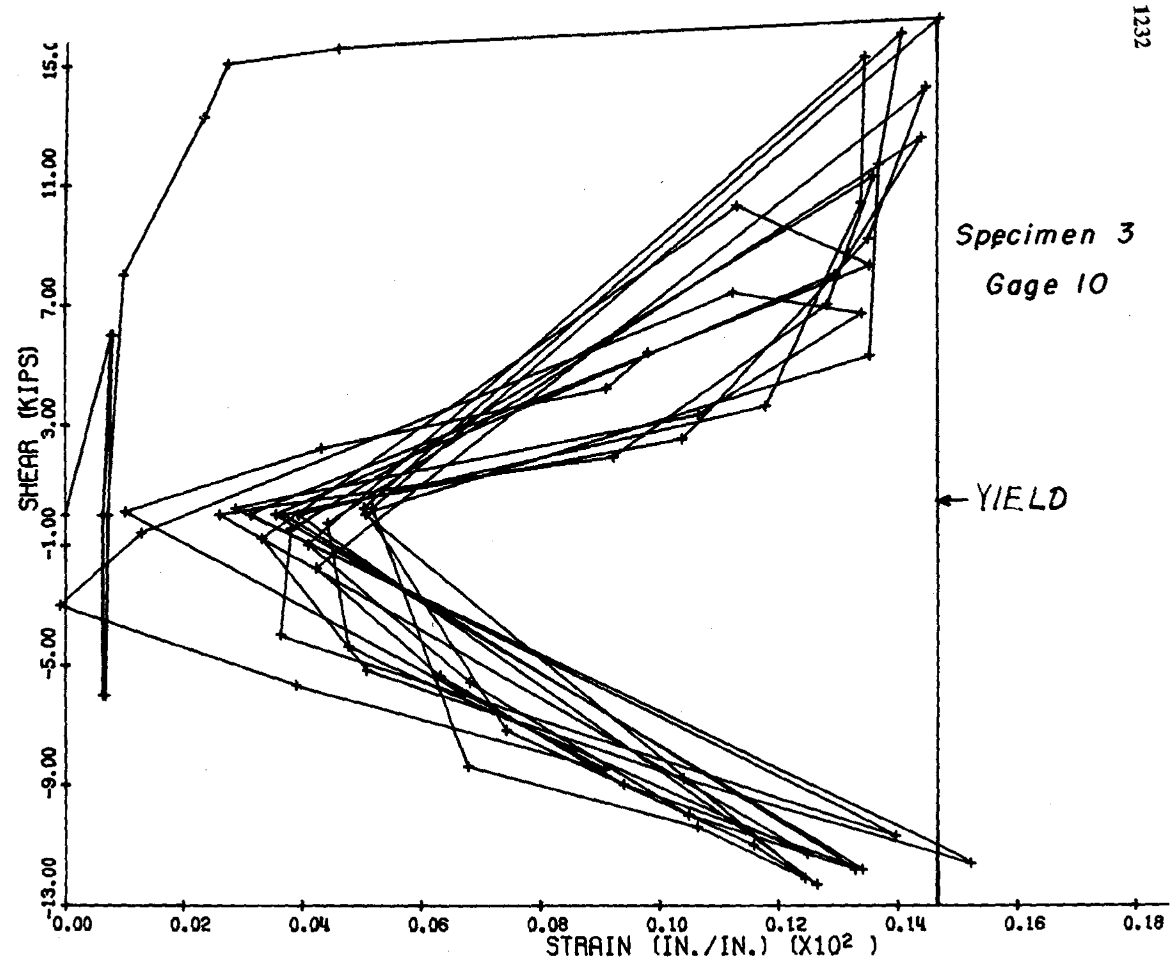


Fig. 13 Beam Shear vs Beam Stirrup Strain - Specimen 3, Gage 10

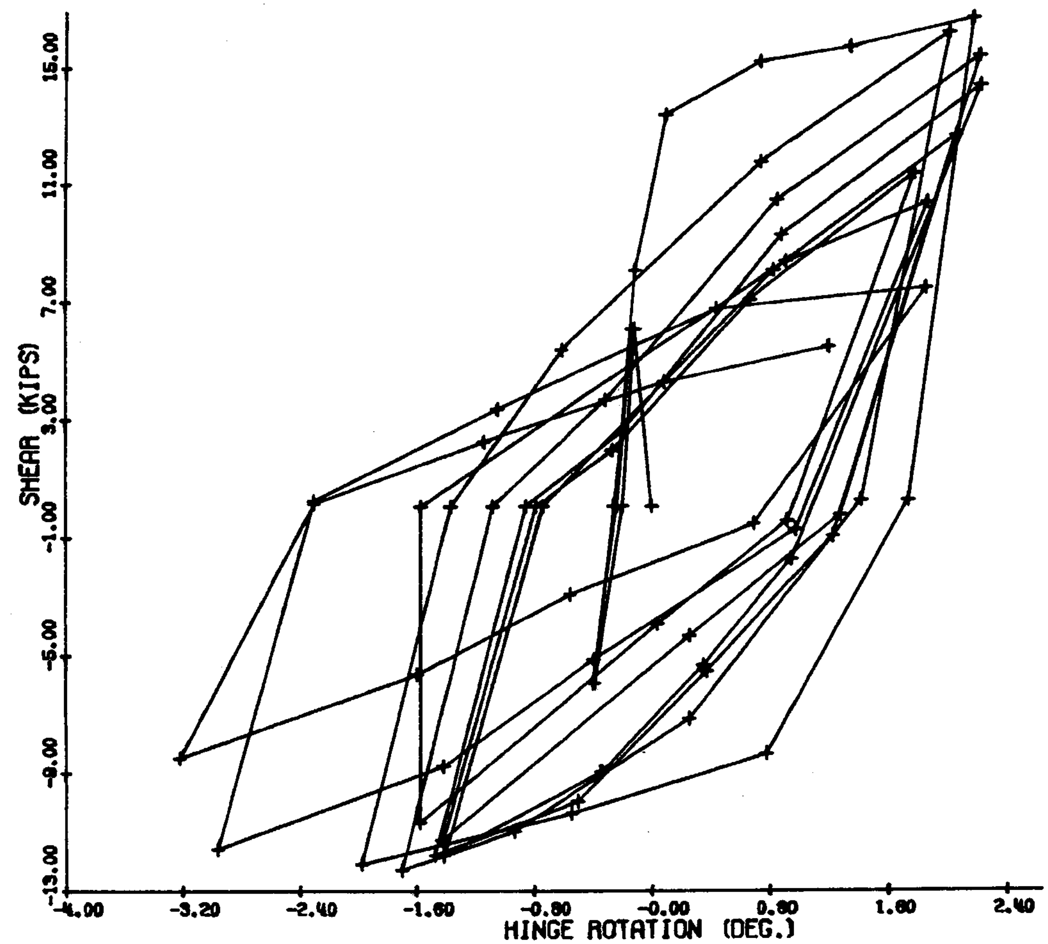


Fig. 14 Beam Shear vs Flexural Rotation in Hinging Region - Specimen 3

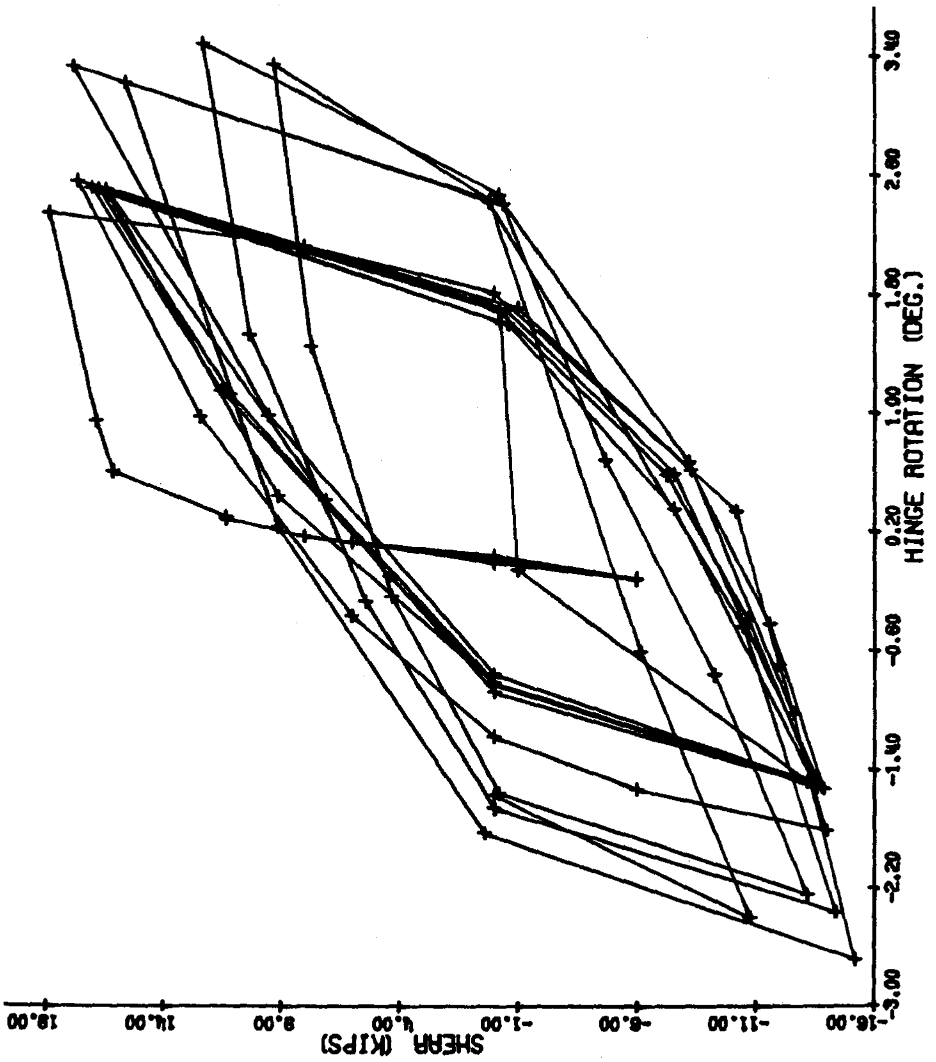


Fig. 15 Beam Shear vs Flexural Rotation in Hinging Region - Specimen 4

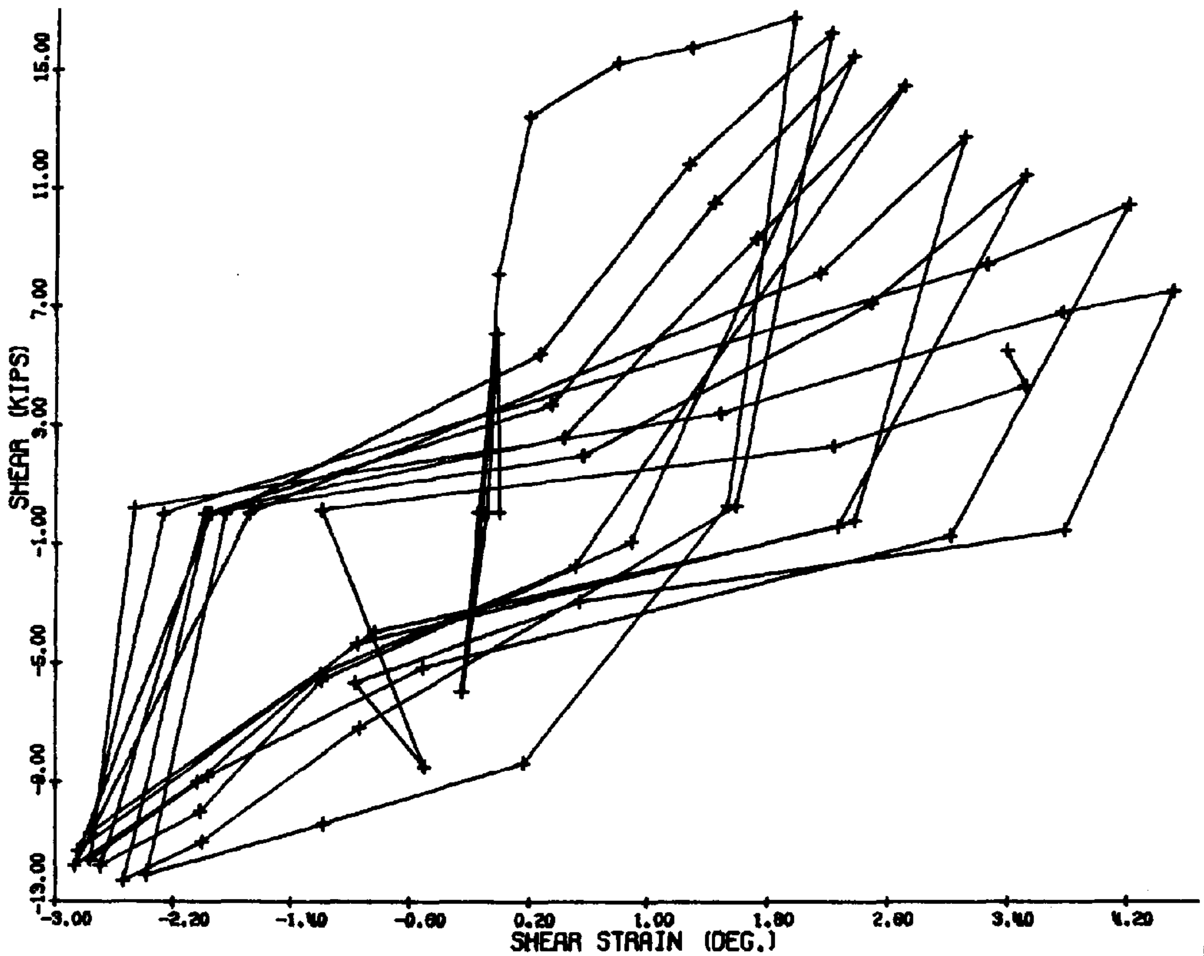


Fig. 16 Beam Shear vs Shear Strain in Hinging Region - Specimen 3

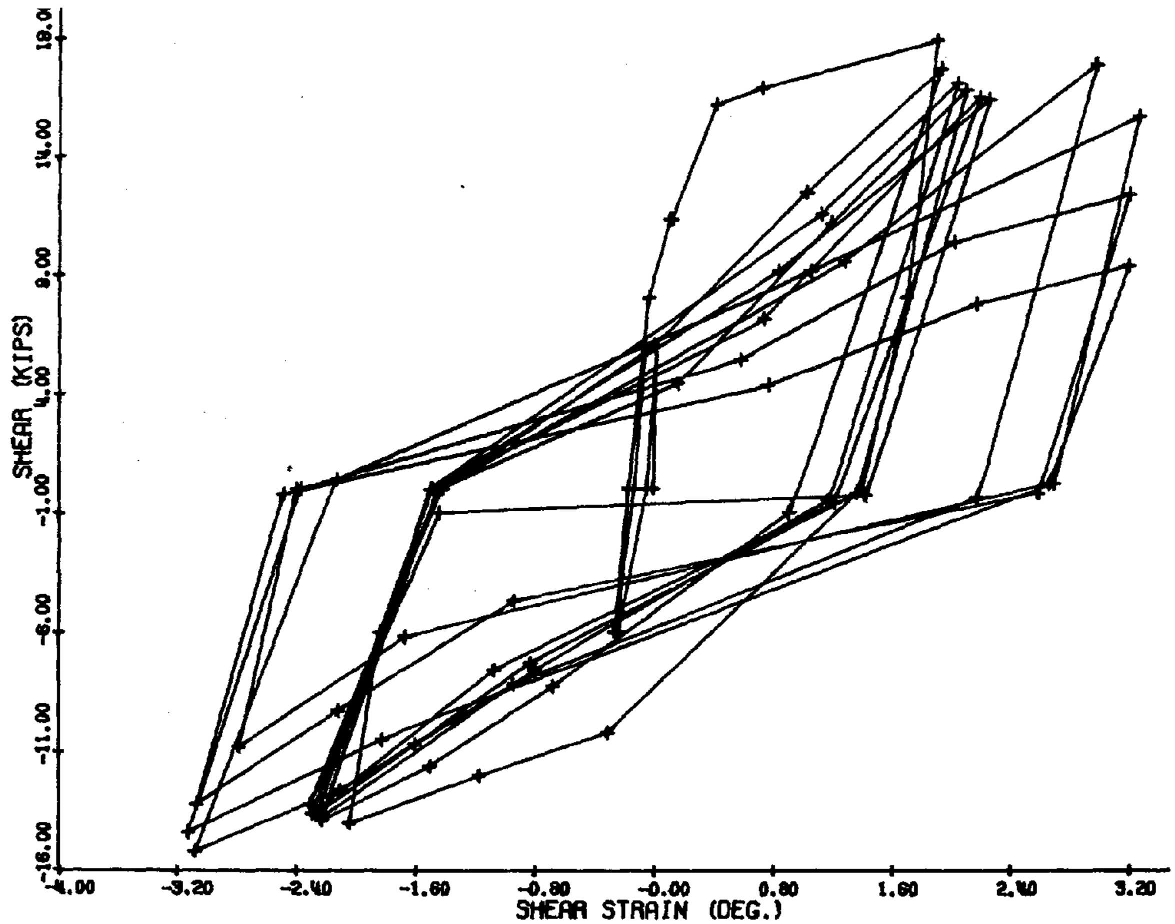


Fig. 17 Beam Shear vs Shear Strain in Hinging Region - Specimen 4

deflection ductilities of four. When displacements were increased, shear strains increased also, but restabilized at a new level.

SUMMARY AND CONCLUSIONS

The first four tests conducted in the present research effort may most effectively be considered as two series of tests. The first series included specimens 1 and 2. These two tests allowed bending of the beams without shears as high as those encountered in specimen 3 or 4 and as a result shear deterioration was negligible in both beams.

The second series included specimens 3 and 4, comparitively deeper beams which resulted in higher shear stresses. These higher stresses caused severe damage to the conventionally reinforced beam, with loss of stiffness very soon after commencement of testing and ultimately caused failure in shear and torsion after nine inelastic cycles. Specimen 4 contained supplementary shear reinforcement which was designed to intersect shear cracks regardless of their orientation and provide greater overall confinement of core concrete. The evidence of improvement is provided by comparison of beam shear vs deflection curves, plastic hinge shear strain and rotation plots, and strain gage data from the two specimens. Based on this data it is possible to draw two conclusions.

First, from consideration of data from speciments 1 and 2, it is clear that special reinforcement was neither necessary nor effective in providing shear resistance greater than that provided by conventional seismic tie provisions for the level of shear stress encountered in these two specimens.

Second, from analysis of results of tests of specimens 3 and 4 it is clear that these beams experienced loss of ductility and energy dissipation potential due to the shear stresses they encountered. It was possible to design shear reinforcement to delay shear strength decay. Such reinforcement was placed in specimen 4. This longitudinal reinforcement was able to delay shear strength decay by intersecting vertical planes of weakness and providing increased confinement of core concrete. The specially reinforced beam showed less tendency to increase shear strain with large displacement cycling and maintained greater flexural rotation.

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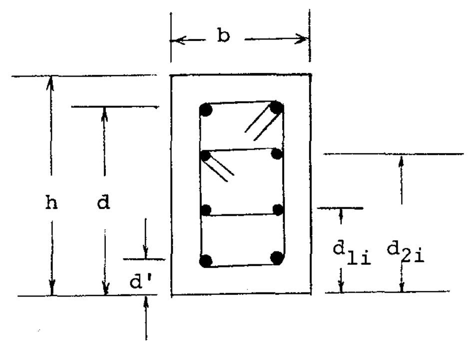
TABLE 1 PHYSICAL SPECIFICATIONS

Specimen No.	Column Load	Shear Span (mm)	h	b	d	d'	d_{1i}	d_{2i}	Shear Span / Depth	f'_c x10^4 KN/m^2
			all mm							
1	178kN	1050	254	206	219	34	-	-	4.80	3.42
1A	"	787	"	"	"	"	-	-	3.59	2.82
2	"	1050	254	206	219	34	95	159	4.80	3.48
2A	"	787	"	"	"	"	"	"	3.59	2.82
3	"	1050	305	206	258	46	-	-	4.09	3.40
3A	"	"	"	"	"	"	-	-	"	2.65
4	"	1050	305	206	258	46	113	171	4.09	3.42
4A	"	"	"	"	"	"	"	"	"	2.70



NOTE: 1) See Table 2 for complete schedule of reinforcement.

2) Average yield stress for all longitudinal bars is 3.45 x 10^5KN/m^2(50 psi)

3) 4.45 kN = 1k, 25.4 mm = 1 in., 6.9 KN/m^2 = 1 psi.

TABLE 2 BEAM REINFORCEMENT SCHEDULE

Specimen No.	A_s No.-Bar	A'_s No.-Bar	A_i No.-Bar	Ties
1	2 - #6	2 - #5	None	#2 @ 51 mm
1A	"	"	"	"
2	2 - #6	2 - #5	4 - #2	#2 @ 51 mm
2A	"	"	"	"
3	3 - #6	3 - #5	None	#2 @ 64 mm
3A	"	"	"	#3 @ 64 mm*
4	3 - #6	3 - #5	4 - #3	#2 @ 64 mm
4A	"	"	"	#2 @ 64 mm**

* Grade 60

** No ties on intermediate bars

Note: 1) 25.4 mm = 1 in.

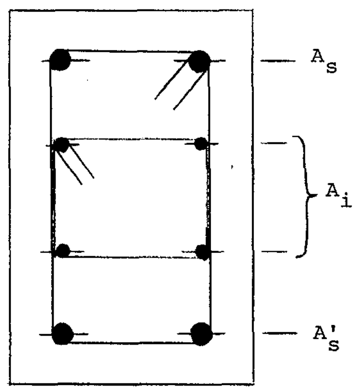


TABLE 3 SELECTED TEST RESULTS

Specimen No.	Max. Shear (KN)	Vu/bd $\times 10^3$ KN/m^2	$\frac{Vu/bd}{\sqrt{f'_c}}$	Cycles to Failure	Actual Yield Moment (KN-m)	Calculated Yield Moment (KN-m)	Beam-Column Joint Cracking Stress Range $\times 10^3$ KN/m^2	$\frac{\text{Stress Range}}{\sqrt{f'_c}}$
1	51.2	1.13	2.33	14*	37.8	37.6	2.20 - 4.43	4.5 - 9.1
	65.9	1.46	3.31	12*	40.7	37.2	3.53 - 4.60	8.0 - 10.4
2	54.7	1.21	2.46	14*	38.9	39.3	2.95 - 4.15	6.0 - 8.5
	66.8	1.48	3.35	14*	42.4	38.9	2.27 - 3.29	5.2 - 7.5
3	77.4	1.46	3.01	9	62.4	65.3	3.77 - 6.28	7.8 - 13.0
	80.5	1.52	3.55	12	70.3	64.8	3.77 - 6.99	8.8 - 16.3
4	87.7	1.65	3.40	11	75.5	69.2	3.73 - 5.26	7.7 - 10.8
	86.8	1.64	3.79	14	75.1	68.7	2.79 - 4.66	6.5 - 10.8

Note: 1) 4.45 kN = 1k, 25.4 mm = 1 in,
6.9 KN/m^2 = 1 psi, 0.113 kN-m = 1 k-in.

* Stable behavior at end of testing

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

REINFORCING BARS IN EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION

by

William C. Black, Chief Engineer
Reinforcing Bars, Piling and
Construction Specialty Sales
Bethlehem Steel Corporation

Our paper will be limited to the detailing, fabrication and placing of reinforcing bars as it applies to earthquake resistant reinforced concrete building construction.

The architect and engineer establish the concrete outlines and reinforcement requirements. The reinforcing bar detailer must interpret these requirements and prepare placing drawings that show (1) the engineer how the detailer interpreted his requirements, and (2) the ironworker (placer) how to place the reinforcing bars into the forms. This is a function normal to all reinforced concrete structures and we would like to only discuss those things that are more or less peculiar to seismic designs. Generally speaking, these designs are characterized by such things as staggered lap splices, heavy confinement reinforcement (especially at joints), non-standard hooks in some instances, including special "seismic" hooks for confinement hoops, and standard hooks where straight ends would usually be satisfactory in non-seismic designs.

Ductile moment resistant frames are of course heavily reinforced and require special detailing consideration. Splices are usually not permitted at, or adjacent to, beam-column joints. This then requires that the horizontal beam bars and vertical column bars be extended some distance beyond the joint before splicing. At exterior joints the horizontal reinforcement must be properly anchored. Within the joint itself confinement reinforcement consisting of closely spaced hoops is necessary.

Shear walls require special heavy edge reinforcement, tied as for columns, with the horizontal wall reinforcement adequately anchored to these "columns."

The detailer is obligated to follow the design details as established by the architect/engineer. A common problem with these heavily reinforced elements is one of physically finding room for the reinforcing, especially where bars intersect, as at joints. The detailer should not be responsible for determining whether the reinforcing bars as designed can be detailed so that the ironworker can place them in the forms - this is the designers responsibility. The detailer, in the course of preparing the reinforcing details, may discover that a "fit" problem exists and, even though it is not one of his responsibilities, will probably approach the customer and/or engineer so that the engineer can work out a solution.

Fabrication of reinforcing bars for structures designed for seismic forces does not necessarily differ from the fabrication for non-seismic construction. However, the percentage of bent reinforcing to total requirements is significantly larger and the types of bending may differ. For instance, there may be rather large quantities of ACI Type T1 (Figure A) ties with seismic 135 degree hooks, that is, with 10 bar diameter hook extensions. These are more difficult to fabricate than ACI Type T2 ties as would be used on most non-seismic structures. One of the problems is the overlapping 135 degree hooks. Normally, ties are fabricated in multiples of three or more. With the 135 degree hooks it becomes necessary to "lift" the final hook into position. Where ties are fabricated in multiples this means that the hook must be lifted, or offset the height of the pile. With smaller bar sizes it is possible to pull the hooks together in the field but with the heavier sizes this cannot be easily accomplished. This restricts the number of ties that may be fabricated at one time.

Fabricating tolerances become increasingly critical in seismic designs, mostly because of the high percentage of bent reinforcing bars and the problems of placing in the forms. Standard industry fabricating tolerances as covered in the CRSI Manual of Standard Practice and ACI 315-74 apply to all fabrication, including bars for seismic application. However, as will be discovered when placing is discussed, this does not assure that the ironworker can put all the pieces together.

Placing of reinforcing bars is one of the most critical elements in the whole chain of events. The design may have been meticulously accomplished, the detailers have faithfully and accurately followed the designers intentions, the fabrication all within the permitted industry tolerance and properly bundled and delivered to the jobsite. But, if the ironworker (placer) does not place the rebars in their correct final position all the foregoing does not really "count."

One of the elements that affects the placing is the tolerances permitted in construction. This includes both the form tolerances and the rebar fabricating tolerances. [1] [2] When these are combined with the permissible placing tolerances it is found that they are, in many instances, incompatible. ACI Committee 117 Tolerances has been studying this for some years, conducting symposia, etc., but with no real progress towards a final solution.

Joints in ductile moment resisting frames, as mentioned earlier, are difficult to design, detail and to place. Refer to Figure A-1 which represents an intermediate exterior joint at an intermediate floor level. This example detail is a composite of the design information, including "fit" details for the reinforcing bars and a tabulation of the rebar placing sequence as the ironworker would be expected to put it together. In this example the designer has solved the anchorage problem for the girder bars by establishing the bend diameter larger than ACI Standard and using the total bar extension around the bend, as permitted by ACI 318-71 Commentary Section 12.3.

Note also that the girder top bar hook extends well below the bottom of the girder spandrel beam. This creates a problem for the contractor if he wishes to have a construction joint at the underside of the spandrel beam as would be usual. The #11 top girder bar would have to be placed and supported while the columns were being cast. This is especially "tricky" as these bars extend at least to mid-span of the girder plus splice. The rebar placing sequence would also be affected as the sequence shown in Figure A-1 is based on monolithic construction up to top of slab.

The stirrups in the girder are shown as two piece elements to facilitate placing as follows:

stirrup plus cap tie to form a closed stirrup-tie.

The spandrel stirrup-ties have been shown in this instance as one piece. The single leg cap ties and intermediate single leg column ties, shown with one 90 degree and one 135 degree hook, would have these hooks alternated. This is something that should be specified by the designer.

The detail as presented is based on theoretical dimensions. One may quickly observe that any deviations from the theoretical in either form dimensions or rebar dimensions would probably create interference problems.

Figure A-2 shows a corner column joint detail at an intermediate floor level. Note that the spandrel beams are each on the column center-line and of the same depth. The #9 top bars in each direction extend well below the beam soffits and create the same contractor construction joint problem as with Figure A-1. Note how the bars must be interlaced and canted at the corner. Note that cap ties are used in the spandrel beams to make closed stirrup-ties.

Figure A-3 also shows a corner column joint detail at an intermediate floor level but with the spandrel beams flush with the outside faces of the column. The bar placing is complicated by the fact that the top and bottom spandrel bars on the exterior face interfere with the column vertical bars. Note how bars have been "pulled in" to clear. It is more of a problem with the top bars as they must clear the 135 degree hook of the stirrup. With these fairly heavy bars the net result would probably be relocating these bars for the length of the spandrel. One then must consider whether an extra corner bar is required to meet the torsion criteria.

Figure A-4 covers an interior joint at an intermediate floor. The floor beams are of such size that confinement reinforcement is required throughout the joint. This joint is fairly straight foreward from a placing viewpoint. The sets of column ties must be placed in the sequence noted. Note that the beam bars, both top and bottom, happen to clear the column verticals but this would not usually be the case. Note also that the top beam bars in the East-West beam have been dropped to fit under the North-West top beam bars. The two piece stirrup ties have been shown to have the standard clearance at top of beam so that the top beam bars are not in the upper corners of the ties. Note that the cap tie has been shown with two 90 degree hooks because a 135 degree hook would foul the outside beam bar. An alternate solution could have been to lower the top of the stirrup-tie if permitted by the design.

Figures A-5 and A-6 show current thinking in seismic joint design and are taken from the ACI-ASME 352 Report on Joints that appeared recently in the ACI Journal. [3]

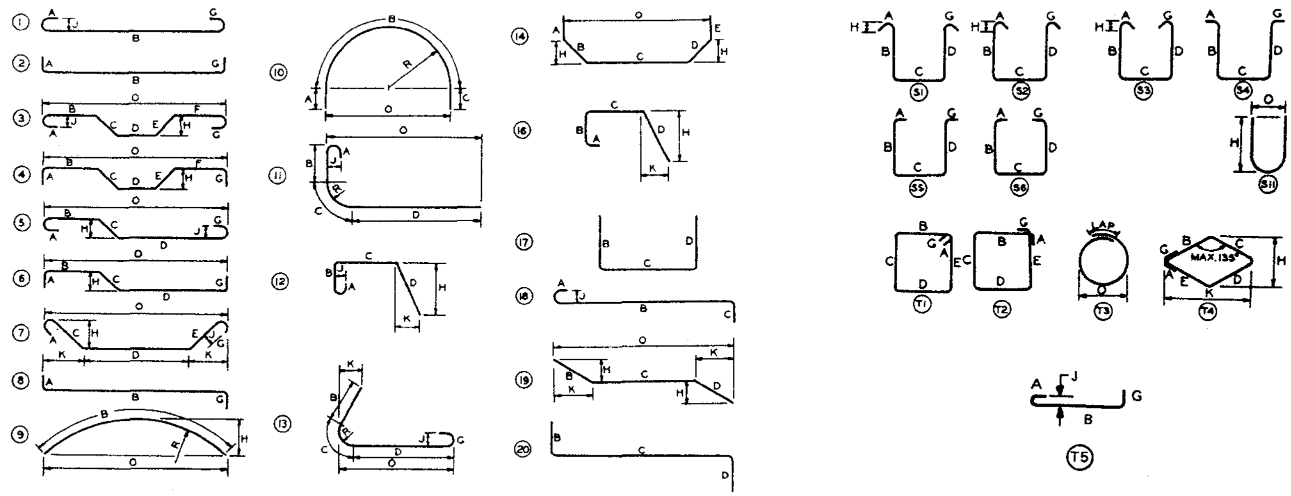
The problem that must be considered is whether it is economically feasible, or even practicable, to build joints such as have been illustrated. Dr. Jirsa and others have recently reported on research work at the University of Texas at Austin that may alleviate the congestion at joints. The latest thinking is that the quantity of confinement reinforcement in the joint does not have to be nearly as great as that recommended by ACI-ASCE 352.

We trust that all the foregoing will be discussed in greater detail in the Workshop on Reinforcing.

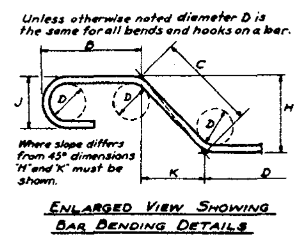
[1] Tolerances for Reinforcing Bars, by W. C. Black, ACI Journal, October 1970

[2] Tolerances by A. E. Fisher, ACI Journal, February 1977

[3] Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures by ACI-ASCE Committee 352, ACI Journal, July 1976



NOTES:
1. All dimensions are out to out of bar except "A" and "G" on standard 180 and 135 deg. hooks.
2. "J" dimension on 180 deg. hooks to be shown only where necessary to restrict hook size, otherwise standard hooks are to be used.
3. Where "J" is not shown "J" will be kept equal to or less than "H" on truss bars. Where "J" can exceed "H," it should be shown.
4. "H" dimension on stirrups to be shown where necessary to fit within concrete.
5. Where bars are to be bent more accurately than standard bending tolerances, bending dimensions which require closer working should have limits indicated
6. Figures in circles show types.
7. For recommended diameter "D," of bends, hooks, etc., see tables.



ACI 315-74

FIGURE A

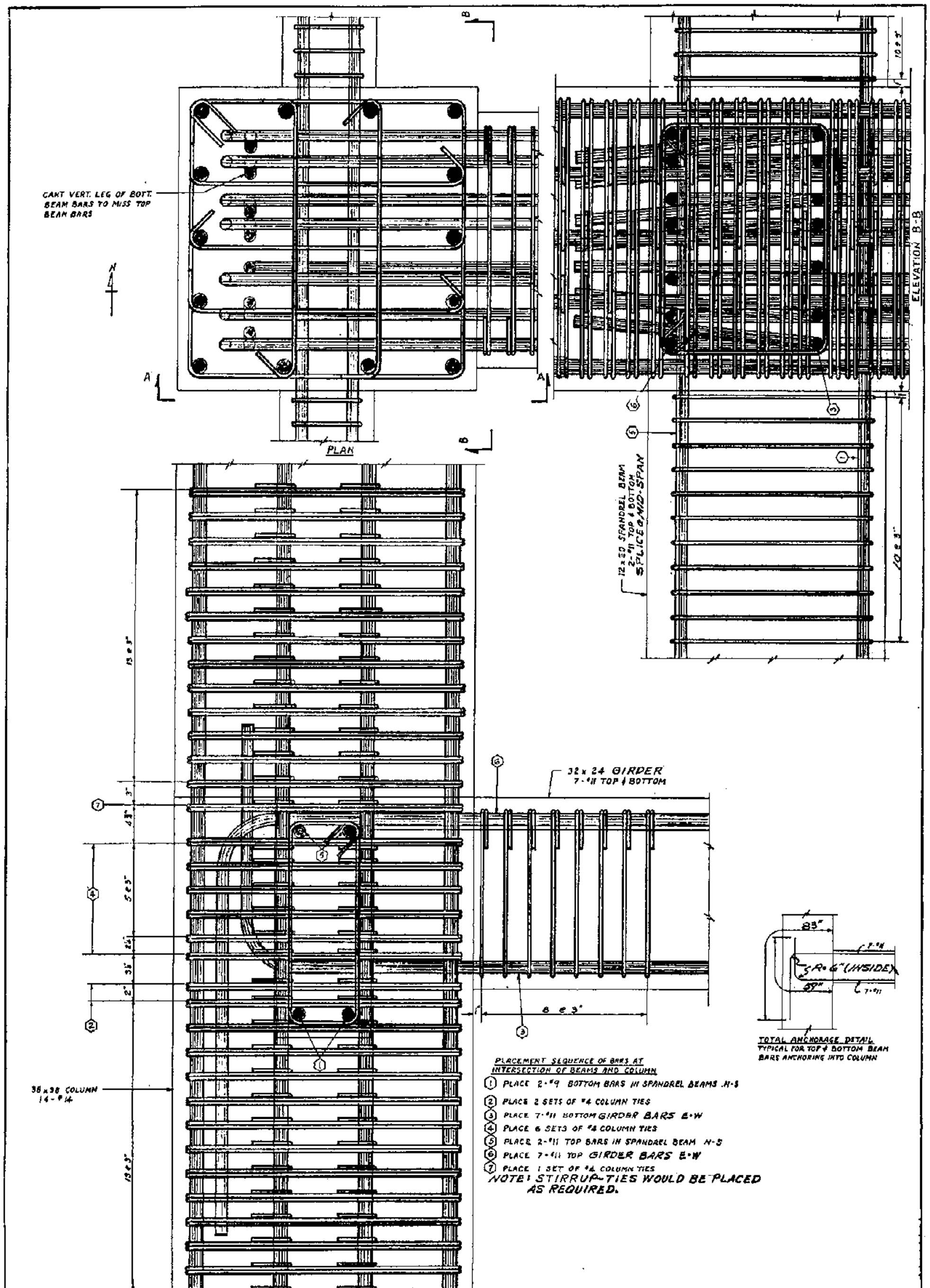

CANT VERT. LEG OF BOTT. BEAM BARS TO MISS TOP BEAM BARS
PLAN
ELEVATION B-B
12 x 20 SPANDREL BEAM 2-#11 TOP & BOTTOM SPLICE @ MID-SPAN
32 x 24 GIRDER 7-#11 TOP & BOTTOM
38 x 38 COLUMN 14-#14
8 @ 3"
TOTAL ANCHORAGE DETAIL
TYPICAL FOR TOP & BOTTOM BEAM BARS ANCHORING INTO COLUMN
R = 6" (INSIDE)
PLACEMENT SEQUENCE OF BARS AT INTERSECTION OF BEAMS AND COLUMN
1 PLACE 2-#9 BOTTOM BARS IN SPANDREL BEAMS N-S
2 PLACE 2 SETS OF #4 COLUMN TIES
3 PLACE 7-#11 BOTTOM GIRDER BARS E-W
4 PLACE 6 SETS OF #4 COLUMN TIES
5 PLACE 2-#11 TOP BARS IN SPANDREL BEAM N-S
6 PLACE 7-#11 TOP GIRDER BARS E-W
7 PLACE 1 SET OF #4 COLUMN TIES
NOTE: STIRRUP-TIES WOULD BE PLACED AS REQUIRED.
ELEVATION A-A

FIGURE A-1

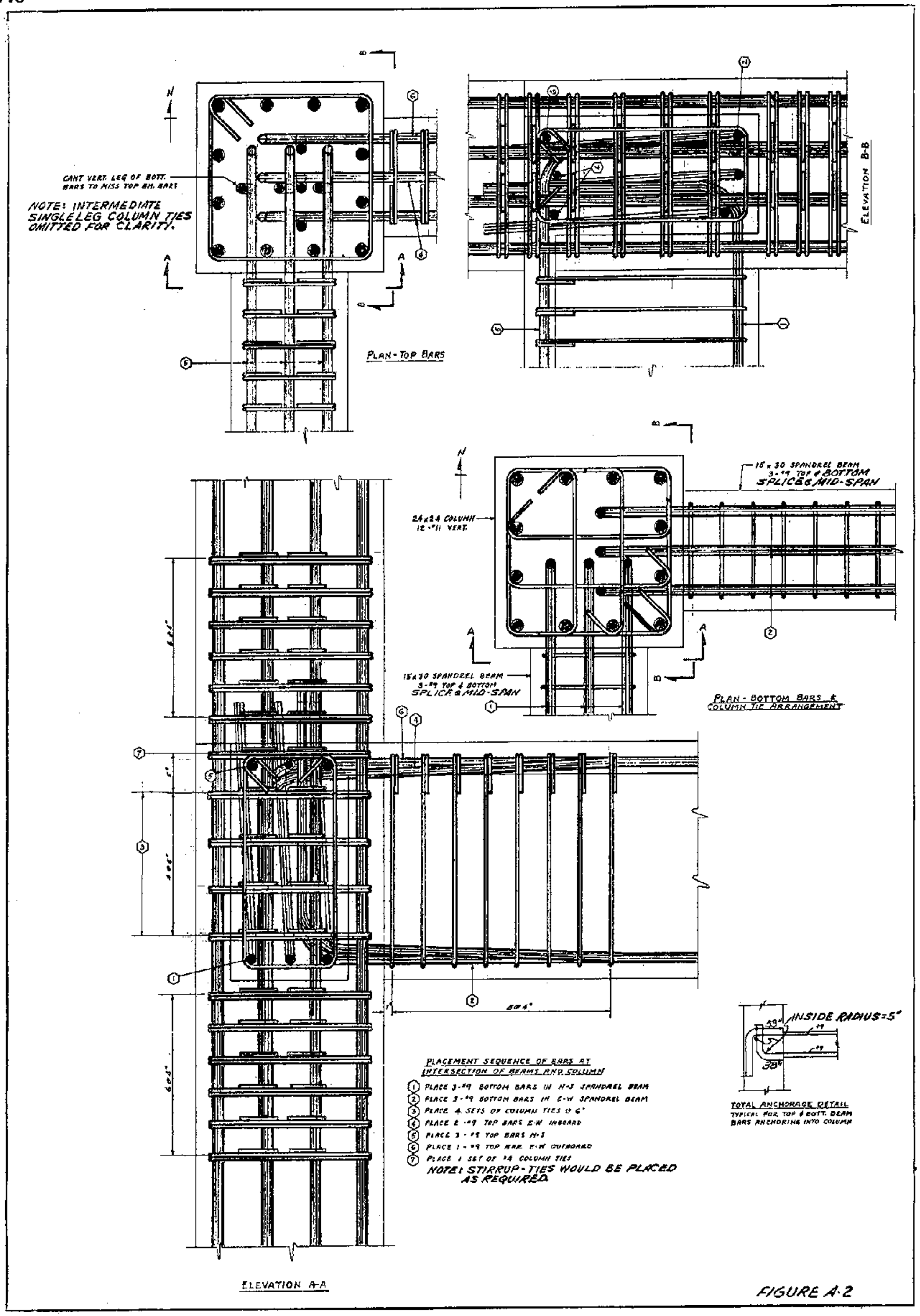

CANT VERT. LEG OF BOTT. BARS TO MISS TOP BM. BARS
NOTE: INTERMEDIATE SINGLELEG COLUMN TIES OMITTED FOR CLARITY.
PLAN-TOP BARS
ELEVATION B-B
24x24 COLUMN 12-#11 VERT.
15x30 SPANDREL BEAM 3-#9 TOP & BOTTOM SPLICE & MID-SPAN
15x30 SPANDREL BEAM 3-#9 TOP & BOTTOM SPLICE & MID-SPAN
PLAN - BOTTOM BARS & COLUMN TIE ARRANGEMENT
INSIDE RADIUS=5"
TOTAL ANCHORAGE DETAIL
TYPICAL FOR TOP & BOTT. BEAM BARS ANCHORING INTO COLUMN
PLACEMENT SEQUENCE OF BARS AT INTERSECTION OF BEAMS AND COLUMN
(1) PLACE 3-#9 BOTTOM BARS IN N-S SPANDREL BEAM
(2) PLACE 3-#9 BOTTOM BARS IN E-W SPANDREL BEAM
(3) PLACE 4 SETS OF COLUMN TIES @ 6"
(4) PLACE 2-#9 TOP BARS E-W INBOARD
(5) PLACE 3-#9 TOP BARS N-S
(6) PLACE 1-#9 TOP BAR E-W OUTBOARD
(7) PLACE 1 SET OF #4 COLUMN TIES
NOTE: STIRRUP-TIES WOULD BE PLACED AS REQUIRED
ELEVATION A-A

FIGURE A-2

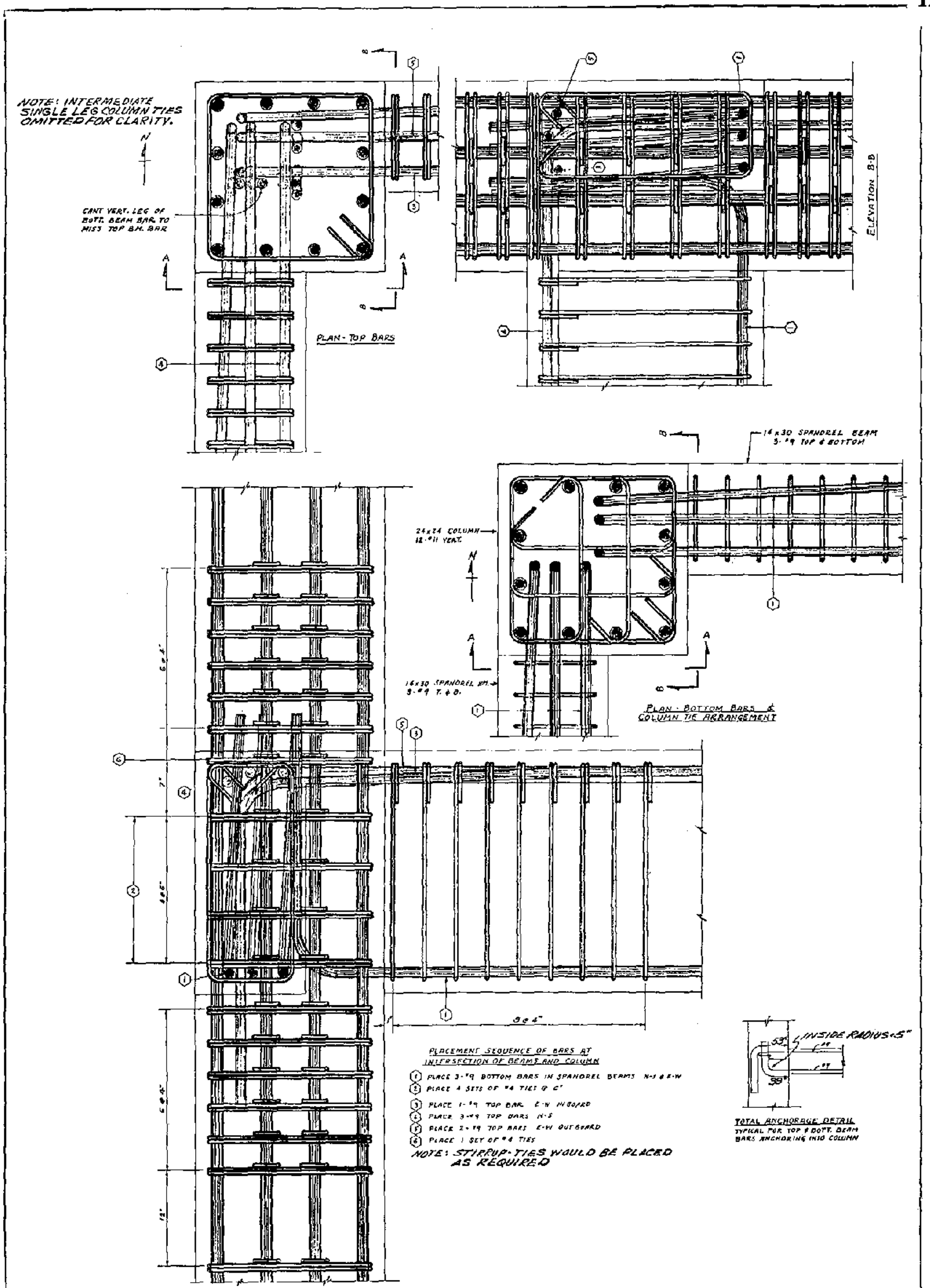

NOTE: INTERMEDIATE SINGLE LEG COLUMN TIES OMITTED FOR CLARITY.
CANT VERT. LEG OF BOTT. BEAM BAR TO MISS TOP BM. BAR
PLAN - TOP BARS
ELEVATION B-B
14 x 30 SPANDREL BEAM 3-#9 TOP & BOTTOM
24 x 24 COLUMN 12-#11 VERT.
14 x 30 SPANDREL BM. 3-#9 T. & B.
PLAN - BOTTOM BARS & COLUMN TIE ARRANGEMENT
PLACEMENT SEQUENCE OF BARS AT INTERSECTION OF BEAMS AND COLUMN
1 PLACE 3-#9 BOTTOM BARS IN SPANDREL BEAMS N-S & E-W
2 PLACE 4 SETS OF #4 TIES @ 6"
3 PLACE 1-#9 TOP BAR E-W INBOARD
4 PLACE 3-#9 TOP BARS N-S
5 PLACE 2-#9 TOP BARS E-W OUTBOARD
6 PLACE 1 SET OF #4 TIES
NOTE: STIRRUP-TIES WOULD BE PLACED AS REQUIRED
INSIDE RADIUS=5"
TOTAL ANCHORAGE DETAIL
TYPICAL FOR TOP & BOTT. BEAM BARS ANCHORING INTO COLUMN
ELEVATION A-A

FIGURE A-3

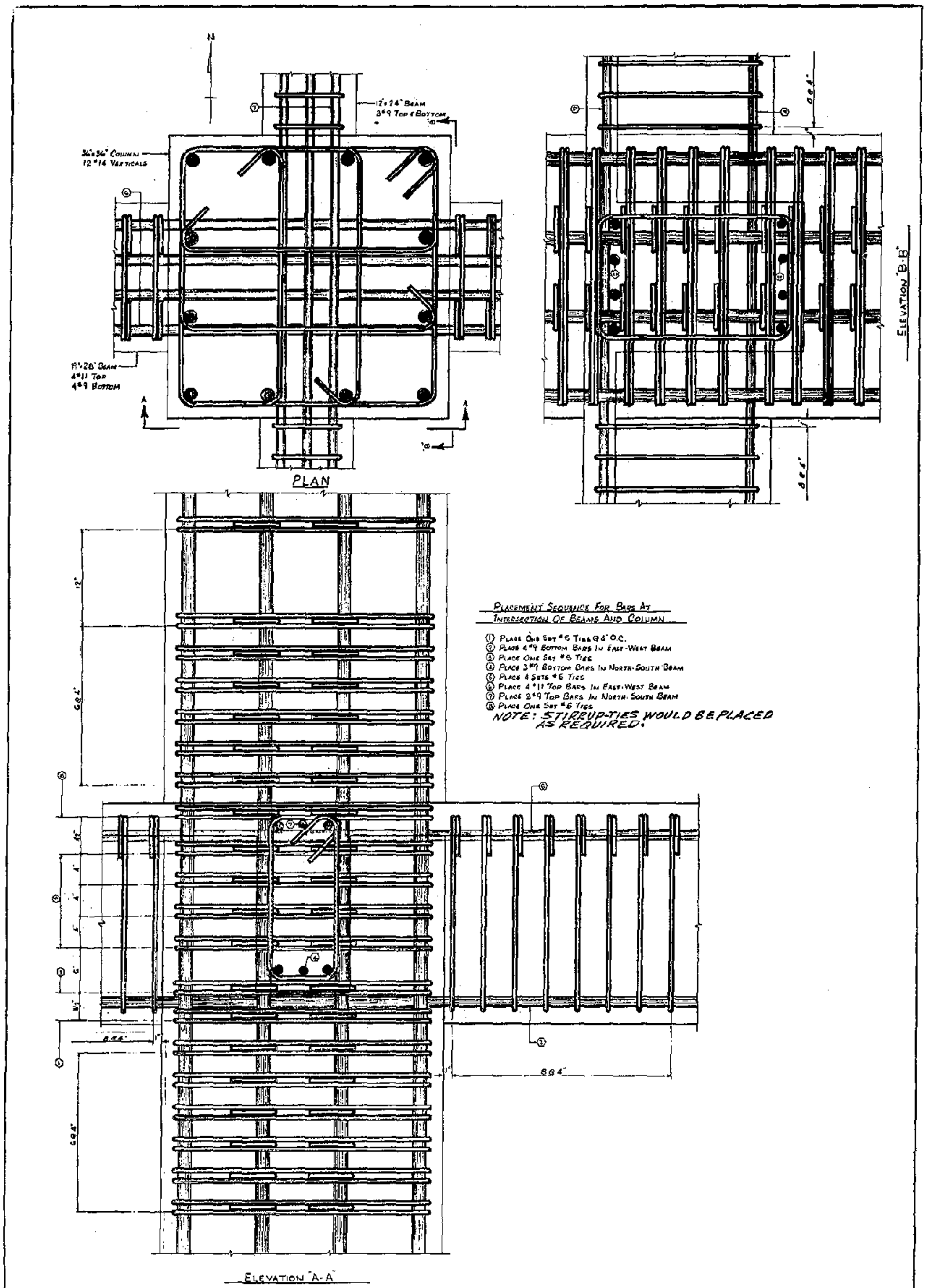

N
12"x24" Beam
3#9 Top & Bottom
36"x36" Column
12 #14 Verticals
19"x28" Beam
4#11 Top
4#9 Bottom
PLAN
Elevation "B-B"
Placement Sequence For Bars At Intersection Of Beams And Column
① Place One Set #6 Ties @ 6" O.C.
② Place 4#9 Bottom Bars In East-West Beam
③ Place One Set #6 Ties
④ Place 3#9 Bottom Bars In North-South Beam
⑤ Place 4 Sets #6 Ties
⑥ Place 4#11 Top Bars In East-West Beam
⑦ Place 3#9 Top Bars In North-South Beam
⑧ Place One Set #6 Ties
NOTE: STIRRUP-TIES WOULD BE PLACED AS REQUIRED.
Elevation "A-A"

FIGURE A-4

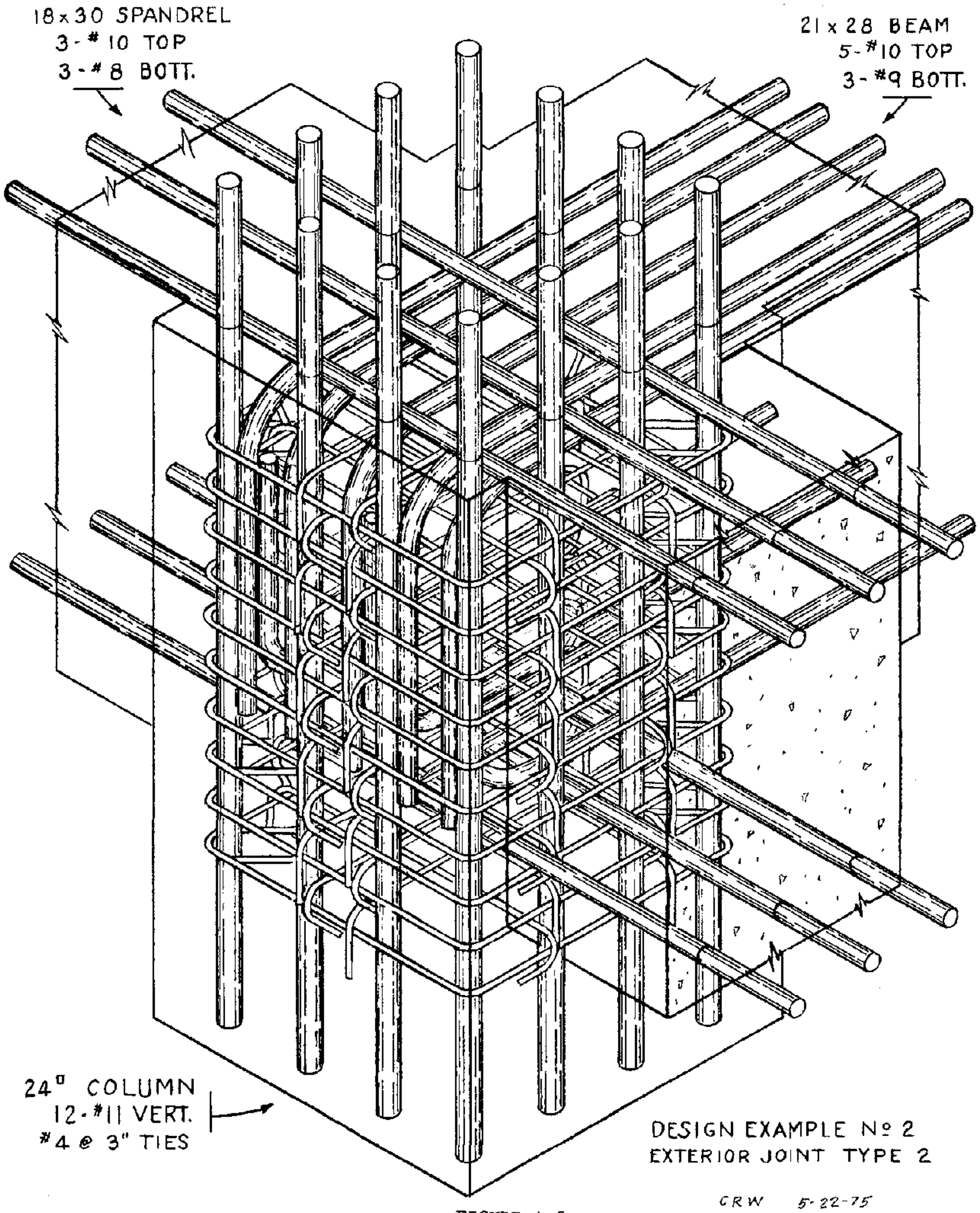

18x30 SPANDREL
3-#10 TOP
3-#8 BOTT.
21x28 BEAM
5-#10 TOP
3-#9 BOTT.
24" COLUMN
12-#11 VERT.
#4 @ 3" TIES
DESIGN EXAMPLE No 2
EXTERIOR JOINT TYPE 2
CRW 5-22-75

FIGURE A-5

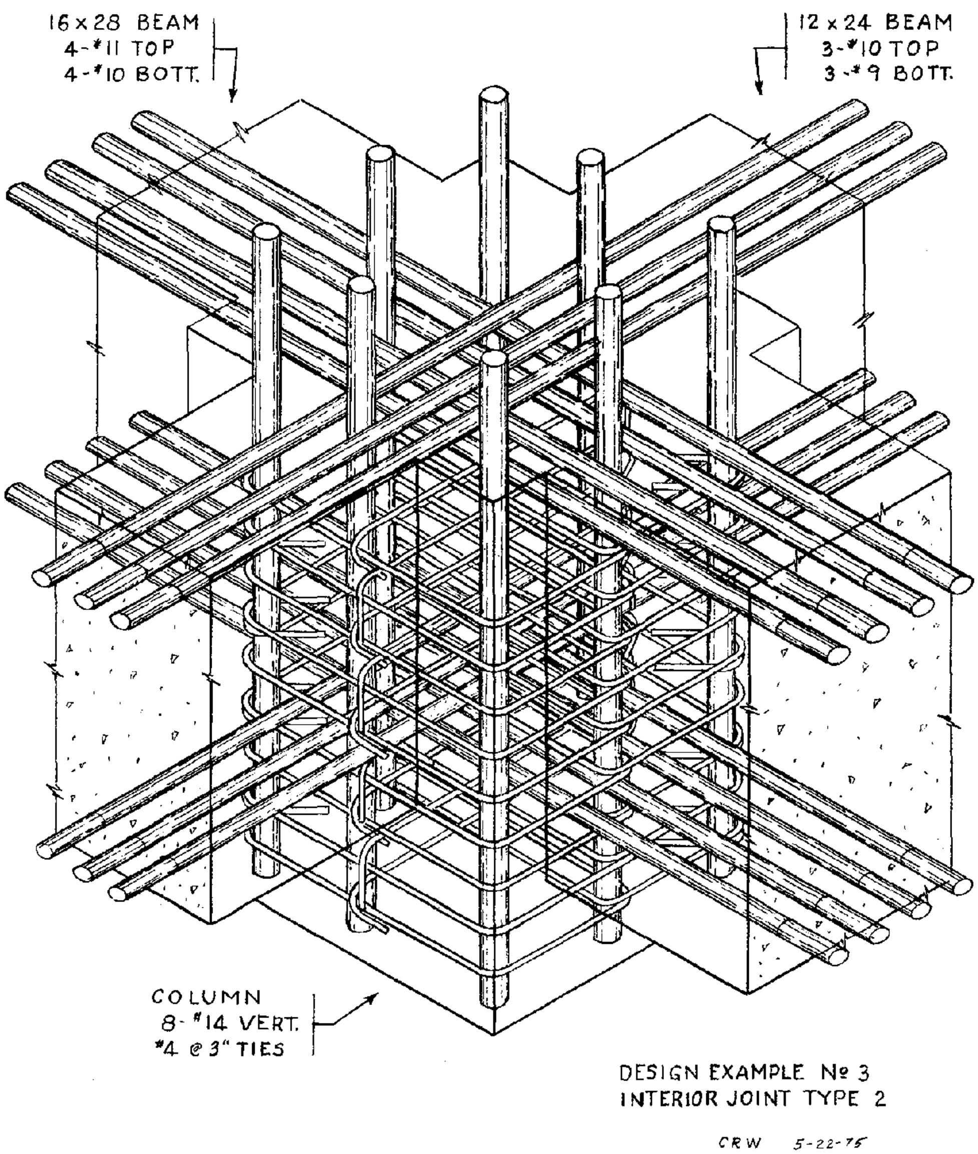

16 x 28 BEAM
4-#11 TOP
4-#10 BOTT.
12 x 24 BEAM
3-#10 TOP
3-#9 BOTT.
COLUMN
8-#14 VERT.
#4 @ 3" TIES
DESIGN EXAMPLE № 3
INTERIOR JOINT TYPE 2
CRW 5-22-75

FIGURE A-6

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

SEISMIC RESPONSE CONSTRAINTS FOR SLAB SYSTEMS

by

Neil M. Hawkins
Professor of Civil Engineering
University of Washington

INTRODUCTION

For the earthquake resistant design of concrete buildings there are three important problems associated with slabs: (1) the use of flat plate construction as a vertical or lateral load carrying system in seismic zones; (2) the effectiveness of slabs as coupling members between shear walls; and (3) the diaphragm action of the slab as it transmits seismic forces to the lateral load resisting elements. This paper summarizes the current state of knowledge for each of those problems.

FLAT PLATE CONSTRUCTION

General Considerations

From the architect's and general contractor's viewpoints, flat plate construction is an ideal structural form. Flat plate frames provide maximum flexibility for the architect's expression of his creativity both in the exterior form of the structure and its interior spatial relationships. Flat plate framing is more economic than beam and column framing because the rental volume per story is more for flat plate, formwork costs are less, the turnaround time per floor for multi-story construction is less, and, most important, the structural frame provides minimum impediment to the placement of mechanical and electrical services. These considerations ensure a continuing demand for flat plate framing in seismic zones.

Code restrictions-- The Uniform Building Code [1] requires all buildings designed with a horizontal force factor, K, of 0.67 or 0.8 to have ductile moment resistant space frames and such frames are required around the perimeter of all frame buildings in Seismic Zones No. 2, No. 3 and No. 4. If the building exceeds 160 ft. in height those frames must be capable of resisting not less than 25 percent of the required seismic forces for the structure as a whole. All framing elements not required by the design to be part of the lateral force resisting system must be adequate for vertical load-carrying capacity and induced moment at a distortion 3/K times that resulting from the Code required lateral forces. The UBC Code provisions are intended [2] to exclude the use of flat plates as ductile moment resistant space frames. Such framing is, however, widely used in the eastern and central parts of the United States which include many Zone 2 and 3 regions.

Lessons from recent earthquakes--The restrictions on the use of flat plate framing in seismic zones stem partly from the poor performance of structures with such framing in recent earthquakes. The collapse of the J. C. Penney building in the 1964 Alaska earthquake [3] involved numerous failures at slab-column connections. The collapse of the San Jose Building in the 1967 Caracas earthquake [4] is also commonly attributed to an inadequacy of the flat plate type framing in that structure and other adjacent badly damaged structures. Flat slab and plate failures also occurred in the 1971 San Fernando and 1972 Managua earthquakes. The large amount of non-structural damage to the Orion Avenue and Marengo Street Holiday Inns during the San Fernando earthquake [5] highlights the lack of stiffness in structures containing such framing even when the over-all structure has adequate strength. There is a need to more definitively delineate limitations to the use of flat plate framing for the differing Seismic Zones.

Laboratory Investigations

Approximately 40 high intensity reversed cyclic lateral loading tests on specimens simulating slab-interior column connections are reported in the literature [6-10]. In addition there are data on some 15 similar specimens [9, 10, 11, 12] subjected to high intensity cyclic shear forces or non-reversing moment and shear forces [13]. That information can provide reasonable answers to most questions concerning the likely strength, stiffness and ductility of such connections for seismic loading. By contrast little information is available on the likely cyclic lateral load performance of slab-exterior column connections [9, 14, 15].

Slab-interior column connections - cyclic lateral loadings--In the cyclic lateral loading tests the variables examined have been as follows: monotonic as opposed to cyclic loadings to failure, [7, 8]; differing types of shear reinforcement including structural steel shearheads, closed stirrups, hairpin stirrups and bent bars, [6, 8, 9, 10]; differing amounts of negative moment column strip reinforcement with values ranging from 0.25 to 1.0 times the balanced reinforcement ratio, ρ_b, [6, 9, 10]; differing distributions for the negative moment column strip reinforcement [6, 9, 10]; differing amounts and extent for integral beam closed stirrup reinforcement [8, 10]; the side length to slab depth ratio for square columns with values ranging from 2.5 to 3.5 for most specimens and equal to 8.0 in one investigation [7]; the aspect ratio for rectangular columns with values ranging from 1.0 through to 6.0 and moments transferred in either the long or short directions of the column [9, 10]; and the shear stress transferred simultaneously with the reversing moment with values ranging from $-0.5\sqrt{f'_c}$ to $3.3\sqrt{f'_c}$ [9, 10]. Other less significant variables have been the reinforcing bar size, deformations and yield strength, and the concrete strength.

Typical moment-rotation envelopes for differing connection characteristics are shown in Fig. 1a. The envelopes are idealizations based on typical experimental results such as those shown in Fig. 1b. There is a linear response range OA, the extent of which is dependent on the intensity of the simultaneous gravity loading. The rate at which rotations develop then increases markedly once the reinforcement passing through the column yields at A. All specimens without shear reinforcement fail suddenly at B in Fig. 1 by punching. Those with shear capacities considerably greater than their flexural capacities (low ρ values in Fig. 1) show little ductility as compared to similar specimens loaded monotonically to failure. The stiffness prior and subsequent to yielding at A, the moment for yielding, the maximum capacity and the rotation at that capacity increase as the reinforcement within lines two slab thicknesses either side of the column increases. The concentration of column strip reinforcement within that region up to the maximum ratio of ρ_b is markedly beneficial. The strength of a specimen loaded cyclically to failure can be up to

10 percent less than that of a similar monotonically loaded specimen. The reduction in strength increases as the severity of the cyclic loading increases. Rotations of the slab relative to the column for monotonic and cyclic loading show similar variations.

Shear reinforcement in the form of shearheads increases the moment that could be transferred to the column but does not improve the ductility. Shear reinforcement consisting of properly anchored hairpin stirrups [9] or closed hoops [6, 8, 9, 10] increases both the moment capacity and the ductility. For ductility the shear reinforcement must hold the top and bottom reinforcing mats together and delay the splitting off of the concrete cover from the tension bars. With such reinforcement crushing at the slab-column interface is delayed until edge deflections about five times those for first yielding. The imposition of deflections greater than those for crushing causes a loss in strength but not a decrease in energy absorption. The rate of decrease in capacity increases as the amount by which the shear capacity exceeds the flexural capacity decreases. To minimize the rate of decrease and maximize the hysteretic damping the shear capacity should exceed the flexural capacity by at least the shear contribution of the concrete in the compression zone between the inside of the stirrup and the compression surface. That value is typically about 20 percent. Specimens with stirrup reinforcement still fail suddenly by punching in the post-maximum capacity range unless that reinforcement is carefully detailed. Testing has developed proper detailing requirements for integral beam closed stirrup reinforcement only [10]. If loading reversals cause the slab to crack through its depth the stirrup spacing should not exceed d/3, every longitudinal bar passing through the column should be located at the corner of a stirrup and stirrups should terminate around longitudinal bars with standard 135° hooks. For the connection shown in Fig. 2, the integral beam stirrups must extend far enough from the column that the distance ed exceeds the slab depth and the nominal shear stress caused by the lateral load shear plus one quarter of the panel shear does not exceed $1.6\sqrt{f'_c}$ on the critical section abcd.

As apparent from Fig. 1b, before the negative moment reinforcement passing through the column yields hysteresis loops are narrow and spindle shaped. Damping values are typically 10 percent in the first cycle to a new peak and 8 percent for subsequent cycles to the same or lesser peaks. Prior to yielding there is little shakedown in capacity with cycling. After yielding shakedown is significant with most of the decrease occurring between the first and second cycles. That rate of decrease doubles once crushing develops at the slab-column interface. Hysteresis loops remain spindle-shaped in the post-yield range only if the reinforcement ratio in the column head area is 0.9% or less, the shear stress does not exceed $3\sqrt{f'_c}$ for a specimen without shear reinforcement and $6\sqrt{f'_c}$ for a specimen with shear reinforcement, and the column dimension in the direction of moment transfer is significantly greater than the development length for the column head reinforcement. Typically post yield damping values for connections with non-S shaped hysteresis loops are 14 percent for the first cycle to a new peak deflection and 12 percent for subsequent cycles to the same peak. Those values increase with decreasing ρ values, with lower shear stresses and with greater embedment lengths for the slab reinforcement within the column.

An increase in the shear transferred simultaneously with the moment causes a reduction in the maximum capacity approximately equal to the magnitude of the shear stress caused by that shear divided by $4\sqrt{f'_c}$. The stiffness for a given increment in moment and the degeneration in capacity with cycling are similar for specimens with and without simultaneous shear transfer. As illustrated by the results in Fig. 3 [9], an increase in the shear causes a marked reduction in ductility unless properly detailed stirrup reinforcement is used. As the rectangularity of the column in the direction of moment transfer increases the capacity decreases at approximately the rate predicted when shear only is transferred [16]. The ductility is correspondingly reduced unless properly detailed stirrup reinforcement is used [9, 10].

Edge deflection caused by column and slab rotations have agreed closely with those calculated assuming cracked or uncracked sections as appropriate for the combinations of axial load and moment acting on the given section. However, some 50 to 60 percent of the edge deflection is caused by a concentrated rotation that occurs at the column perimeter. That rotation is a result of bond slip of the reinforcement within the column combined with torsional cracking at the side faces of the column.

Interior column connections - cyclic shear loadings--In contrast to the behavior for laterally loaded connections, yielding of the reinforcement passing through the column does not cause a marked change in the stiffness of a connection subject to cyclic shear loadings only [10]. There must be general yielding of the reinforcement across the width of the specimen before such a change occurs. For connections without shear reinforcement and shear capacities equal to their flexural capacities, the decrease in the limiting shear stress of $4\sqrt{f'_c}$ caused by cycling does not exceed 10 percent. If, however, the shear strength is made less than the flexural strength in order to provide some ductility, cycling markedly reduces that ductility. Further, while for monotonic loading the limiting shear stress becomes progressively greater than $4\sqrt{f'_c}$ as the ratio of the shear to the flexural capacity decreases, cycling largely eliminates that increase. Hairpins, closed hoops [10][13] and bent bars [11, 12] can all increase the shear capacity and provide adequate ductility for cyclic loadings. The maximum capacity is reached when the concrete at the slab-column interface crushes. After that the capacity decreases with increasing edge deflections and converges for large amounts of shear reinforcement on a limiting value of approximately $6\sqrt{f'_c}$. For shear reinforcement providing capacities of less than $6\sqrt{f'_c}$ there is only a 20 percent or less decrease in the post-maximum capacity with cycling and increasing edge deflections. Post-maximum unloading and reloading curves are essentially linear and coincident. The stiffness does not change significantly with cycling between constant load limits and does not decrease markedly with increasing peak edge deflections. If the cyclic loading does not crack the slab through its depth the reinforcement must extend far enough from the column that the shear stress on section abcd in Fig. 2 does not exceed $2\sqrt{f'_c}$.

Exterior column connections - cyclic lateral loadings--In flat plate construction there is always moment transfer at exterior column connections and therefore both horizontal and vertical components of the earthquake motion cause cyclic moment transfer effects. While edge beams are required by UBC 1976 all tests conducted to date have been for specimens without edge beams. The over-all behavior of the test specimens has been very similar to that for specimens simulating interior connections. For specimens without shear reinforcement failure has always been by punching. However, for the same geometry and reinforcment the ductility developed prior to failure and the hysteretic damping are greater for edge connections.

Although uni-directional cyclic loading tests [17] have been conducted on eight specimens with moment transferred in the direction of the discontinuous edge only one such specimen has been subjected to reversed cyclic loading [9]. In the later case the slab projected beyond the column centerline by an amount equal to the column side length. Although the capacity of that specimen was only about 10 percent less than that of a similar interior column connection, the stiffness in the elastic range was about 25 percent less. In the uni-directional cyclic loading tests the column was attached to the slab on one face only. The maximum capacity increased for increasing flexural reinforcment ratios in the column vicinity. However, adding closed hoop stirrup reinforcement had negligible effect on the capacity and the ductility. These tests proved that torsional cracking is a major factor affecting the stiffness of the slab-column connection. The stiffness after torsional cracking was only one sixth of that before cracking and that stiffness decreased continuously with increasing maximum rotations.

For specimens with moments transferred perpendicular to the discontinuous edge and no shear reinforcement severe torsional cracking develops at the discontinuous edge at relatively low loads [14]. While the stiffness for changing lateral loads decreases slowly with increasing edge deflections the deflection under gravity loads increases rapidly especially after yielding of the reinforcement passing through the column. That rate of increase in deflections is markedly reduced by increasing the column size and by concentrating flexural reinforcement in the column region. The provision of hairpin stirrups inserted perpendicular to the discontinuous edge and at a spacing of d/2, prevents the torsional cracks from opening wide but does not markedly improve the stiffness, strength and ductility characteristics of the connection. Integral beam stirrup reinforcement proportioned by the same rules as for interior connections is necessary to prevent a punching failure.

Data for corner columns are limited to two tests reported by Zaghlool [15]. Reversals of loading at 25 and 75 percent of the expected capacity had no significant effect on the ultimate capacity or ductility as compared to similar specimens loaded monotonically to failure. Prior to yielding of the reinforcement anchored within the column the percent damping was less and the hysteretic curves considerably more S-shaped than for the edge column connections reported in Reference [14].

Response Predictions

Strength--Cyclic loading strengths can be assessed using the same procedure as those for monotonic loading. Beam analogies [18], [19] provide the best estimate of the local strength of connections transferring moment. They permit proper recognition of slab flexural reinforcement effects and the relative contributions of shear reinforcement at the transverse and side faces of the column. A ten percent reduction in strength should be made if loading repetitions are expected stressing the connection beyond 85 percent of its predicted capacity. The ACI 318-71 procedure over-estimates the shear strength of connections with less than one percent flexural reinforcement in the column region, it can be very conservative for high reinforcement ratios and large amounts of moment transfer, and does not provide a realistic model for torsional effects. However, for slabs whose design is controlled by gravity loading considerations, its use will generally lead to reasonable shear strength and moment cut-off predictions. Where ductility is required, shear reinforcement should be provided to take all shear stresses in excess of $1.6\sqrt{f_c'}$ for a slab cracked through its depth and $2\sqrt{f_c'}$ for a slab not cracked thorugh its depth. The post-maximum shear strength should be taken as $(d - d')/d$ times the maximum strength. Collapse in flexure is possible due either to a local fan mechanism centered on the column or a folding mechanism with yield lines extending across the slab [18]. The local mechanism effect is generally suppressed in complete structures by in plane effects, and it is adequate to consider only local moment transfer effects and the folding mechanism for the complete structure.

Stiffness--While there is general concensus on strength predictions [17], [18], [19] there is no similar agreement for stiffness assessments. Designers commonly use a frame with only part of the slab width effective for lateral loadings. Most studies have been performed on the idealized slab-column element shown in Fig. 4a. In one of the early studies [20] that element was divided into six intersecting beams in each direction and the simultaneous equations for deflection and slope at all points of intersection due to a centrally applied moment M_o were solved. The effective slab width, ℓ_e, was defined as that resulting in the same central rotation, θ_o, in a beam subject to a central moment M_o. Those theoretical results were checked and compensation made for column size by testing a crude elastic model. The resulting design recommendations are shown as solid lines in Fig. 4b. More recent studies have shown that predictions are sensitive to the stiffening effect provided by the finite size of the column [21][22]. When that rigidity is taken into account the effective width increases substantially. Shown in Fig. 4c are the effective widths recommended by various investigators for square columns and square panels. It has also been found that (i) longitudinal boundary conditions, (L direction in Fig. 4), have little effect while transverse boundary conditions have a pronounced effect especially for high c/L values [21], (ii) even ignoring column rigidity effects the equivalent frame method of ACI 318-71 predicts too low a stiffness for c/L values less than 0.12 [21], (iii) stiffnesses are insensitive to the rectangularity of the column and the column dimension in the loaded direction should be used with Figs. 4b and 4c [22], and (iv) the aspect ratio, ℓ/L is an important variable only for ℓ/L values less than 0.75.

Analytical predictions differ widely from experimental results. While the equivalent frame method of ACI 318-71 was not intended for lateral loading, it does allow for some softening at the slab-column junction by considering torsional effects in accordance with the theory of elasticity. Even for the elastic range of behavior the lateral loading stiffness measured in the slab-column tests has been less than 50 percent of that predicted using the ACI equivalent frame and a cracked section for the slab. To correctly predict measured rotations for monotonic loading proper account must be taken of the torsional stiffness of cracked concrete sections [23], [24]. The ratio of the cracked to uncracked stiffness for torsion is much less than that for bending. Finite element, finite difference and theory of elasticity approaches ignore that fact. In addition to correctly predict cyclic loading effects account must also be taken of bond slip of the reinforcement within the column.

When the column dimension in the direction of moment transfer is considerably greater than the anchorage length for the reinforcement in that direction, the model shown in Fig. 5(a) can be used to correctly predict the stiffness for virgin loading for both the elastic and post-yield ranges [10]. The slab is assumed attached to the column by cantilevering cracked flexural elements F_1 and F_2 and cracked torsional elements T_1 and T_2 each having the properties of those sections in the real slab. The elements F_1 and F_2 have the loading and unloading stiffnesses shown in Fig. 5(b) and the torsional elements have the stiffness $(GJ)_{cracked}$ shown on Fig. 5. Compatability for points A, B, C and D determines the twist, ϕ, of the torsional elements and effectively builds in a concentrated rotation at the connection. The moment for yielding should be taken as $(c_2 + h/c_2)$ times that given by the model in order to recognize redistribution effects that occur at incipient yielding [9]. The post-yielding stiffness is that obtained when the front face element F_1 is given zero bending stiffness. For elastic behavior the simpler but less accurate equivalent frame model shown in Fig. 6 will predict reasonable and generally slightly lower stiffnesses. The effective slab widths resulting in the same deflections as those predicted by the model of Fig. 6 are plotted in Fig. 4c. In most flat plate structures c/L ratios lie between 0.06 and 0.15 and in that range the stiffnesses predicted by the model of Fig. 6 lie below those established by elastic theoretical studies even ignoring the effects of column stiffening. For reversed cyclic loadings in the elastic range the stiffnesses will be slightly greater than that predicted by the model shown in Fig. 6. Use of a modified version of the model shown in Fig. 5 is then appropriate for both elastic and inelastic behavior predictions [23]. The models shown in Figs. 5 and 6 are not appropriate for two way floor systems. The effective slab width varies widely depending on the aspect ratio for the side lengths of the slab, the ratio of the flexural stiffness of the boundary beams to that of the slab and the ratio of the torsional stiffness of the transverse beam to the flexural stiffness of the flexural beam [39].

COUPLING OF SHEAR WALLS BY SLABS

General Considerations

Popoff [25] has graphically demonstrated the importance of the effective width of the slab on the design of coupled shear walls. He points out that small differences in the effective slab width greatly influence the rigidity of the system. For a 12 in. wall, 4 ft. corridor, 8 in. slab and two 18 feet deep walls the relative rigidity changes from 18 for uncoupled walls through to 100% for a solid 40 ft. wall. If the effective slab width equals the wall width the relative rigidity is 50%. If the slab width equals 10 times the wall width the rigidity is 87% and almost equal to that for a solid wall. In addition, if the actual rigidity is greater than the design rigidity the axial force on the wall can be badly underestimated. If the reverse is true the moment can be badly underestimated.

Lab Investigations

Very few experimental investigations have been made. Three have used extremely small scale models. In one the epoxy sheets for the slab and shear wall have been glued together [26]. In another an asbestos sheet was used for the slab and steel plates for the shear walls [27]. In the third perspex sheet was used for the slab and steel plates for the walls [28]. In the first case the 22 story high coupled walls were rigidly fixed at their base and the effects of different slab widths examined. It was found that with Roxman's theory [29] the entire bay was effective for the coupling slab width. In the second case the walls were pivoted at their base and the slab bolted to the wall. The method of bolting had no significant effect on the coupled stiffness and the results verified the writer's theory. The third study employed a set-up similar to the second and the effects of differing wall configurations; planar, T-section and box, were examined. A flange on the wall appreciably increased the effectiveness of the coupling slab. Since models of this type employ elastic materials they can only verify elastic analyses. They provide no information of the effects on behavior of flexural and torsional cracking, yielding and bond slip.

There have been two investigations using reinforced concrete models. However, in neither case have the walls been reversed cyclically loaded. In the micro-concrete model [30] precast walls and slabs were epoxy glued together and the wall reinforcement cast into a rigid anchor block. The main variable was the coridor width separating the walls. A gradual variation from cantilever to frame action occurred as the corridor width decreased from 1.67 times a wall depth to 0.33 of a wall depth. In the other [31], one third scale investigation, the model represented three pairs of coupled shearwalls with two of the pairs located along the exterior edge of the model. The load-deflection behavior for the model is shown in Fig. 7a. Cracking at the corridor ends of the walls occurred at A, transverse cracking across the full panel width between walls at C, punching at the interior wall ends at D and at the exterior wall ends at F. Lines L_1 and L_2 in Fig. 7a are the theoretical stiffnesses for an uncracked and cracked slab respectively with an effective width equal to half the corridor width. Line L_3 is the theoretical stiffness calculated as for L_2 on a span equal to the corridor width plus the wall thickness. The force required to displace the end walls was about 40 percent of that for the same displacement for the center walls and in agreement with elastic theory [27]. It is recommended that (i) the effective width of the coupling slab be taken as half the corridor opening, (ii) the shear capacity be predicted using a limiting shear stress of $4\sqrt{f_c'}$ and the U-shaped critical section abcd in Fig. 7b, and (iii) the flexural strength be predicted using the slab width c + t in Fig. 7b with that reinforcement extending the corridor opening beyond the end of the wall.

Response Predictions

Stiffness--Qadeer and Smith [27] used finite difference techniques to study the effects of bay width, corridor opening and wall depth on the effective width of slabs coupling rectangular walls. They presented their results in a series of graphs and concluded that the effective width was less than the corridor width plus the wall thickness. In contrast, Black et al [32] used finite element methods to study wall thickness effects and found stiffnesses 33% higher than those reported by Qadeer and Smith. More recently, finite element techniques have been used to study variations in the effective slab width for coupled planar, T-section and box walls [33]. In the later case for planar walls findings were similar to Black et al and it was concluded that (i) the effect of coupling was only significant when the wall opening was less than 0.3 of the total coupled wall depth (ii) for T-sections only half the flange width instead of the wall width should be used in calculations, and (iii) if the T-section is at the opposite end from the corridor,the wall should be treated as planar for estimation of the coupling width.

Lines L_4, L_5 and L_6 on Fig. 7a are the theoretical elastic stiffnesses for a cracked coupling slab having effective widths defined by the recommendations of References [27], [33] and Fig. 6. The elastic thoeretical analyses considerably over-estimate the measured stiffness. Use of the Fig. 6 recommendations results in a stiffness closely paralleling the slope of the load-displacement curve for its cracked section range. Even before the application of any lateral load flat plate slab-column connections are generally cracked by the slab's dead weight. In that case use of the cracked section properties is obligatory. For shear walls coupled by slabs, gravity loads are less likely to crack the slab along the corridor opening. However, in a real structure cracking at the ends of the shear wall is likely due to shrinkage effects and use of the cracked section properties is probably appropriate.

Strength--While the procedure for shear strength evaluations recommended by Schwaighofer and Collins and shown in Fig. 7(b) is easy to apply, their test result is also predictable using ACI 318-71 shear strength recommendations for moment transfer situations provided the moment cut-off limitation of ACI 318-71 is ignored and the reduced shear stress recommended in References [16] and [19] for essentially one way action is used. The depths of the Reference [31] shear walls were considerably greater than the column depths on which the ACI 318-71 moment cut-off limitation is based, and the width within which the reinforcement is effective for transfer of the portion of the moment not transferred by shear is undoubtedly greater for the shear wall case. Most of the specimens on which the ACI 318-71 limitation is based had column side lengths to slab over-all depth, h, ratios of about 2.0. Thus, the ACI 318-71 limitation of 1.5h could also have been interpreted as three quarters of the column depth. In that case a limitation for shear walls of the lesser of that value or half the corridor width, as suggested in Reference [31] is appropriate. If those additional limitations and interpretations are correct, lateral load strength and stiffness evaluations for coupling slabs can be made consistent with similar evaluations for flat plate construction.

DIAPHRAGM ACTION

General

In reinforced concrete construction diaphragm action refers to the transmitting of shear forces through the roof or floor of the structure to the lateral load resisting system. Diaphragms are usually classified as rigid or flexible. Rigid diaphragms transmit loads to resisting elements in proportion to the relative rigidity ofthose elements and cause torsional effects when the center of mass is eccentric from the center of rigidity. Flexible diaphragms transmit loads in proportion to the area tributary to each element and do not transmit rotational forces. Between these two limits there is a wide range of flexibilities where the behavior depends on the rigidity of both the diaphragm and the lateral load resisting system [34]. In reinforced concrete structures diaphragms are really shear walls oriented horizontally. They can usually be treated as rigid and often must transfer large shear forces between parallel lateral load resisting sustems. Particular points of concern in rigid diaphragm design are : (1) assessment of distribution effects; (2) identification of compatability distortion effects, and (3) proportioning of connections between diaphragms and the lateral load resisting system.

Distribution of Shears by Diaphragms

Both flexural and shear effects should be considered in assessment of the relative rigidity of lateral load resisting elements [34]. Ramakrishnan [35] conducted elastic behavior tests on six micro-concrete models with differing heights and wall configurations and found that the shear rigidities determined the distribution of forces up to wall height to depth ratios of one and a quarter and that flexural rigidity effects governed above height to depth ratios of one and three quarters. Lateral deflections were two to three times the theoretical and the neutral axis shifted within the wall towards the compression side with increasing load. The changing behavior with height to depth ratios and the measured deformations indicate relative flexural to shear rigidity ratios less than those based on gross section theories. Another important lateral load distribution question is the effectiveness of transverse flanges for both open and closed core wall sections. Ramakrishnan found that for a closed section, a flange width half the core depth was always fully effective. However, a width one and one half times the core depth, had no greater strengthening effect until flexural rigidity considerations clearly governed. Coull and Adams [36] have presented a simple elastic method for assessing diaphragm effects considering both bending and torsion. They show that for a tall building a considerable redistribution of load occurs throughout the height of the building. While planar walls resist most of the shear in the upper stories, shears flow from those walls through the diaphragms and are resisted predominagely by the core at lower levels. Taranath [37] has studied the effects of warping on inter-connected shear wall-flat plate structures using finite element methods that recognized both the warping displacement of open sections and the warping associated with the flat plate. Neglect of warping effects introduced large errors, resulted in marked under-estimations of the core torques and therefore the longitudinal stresses in the core. The effect of taking account of the out-of-plane stiffness of the slab, although significant, was of an order of magnitude less than warping effects. Recently, Stephen and Bouwkamp [38] reported forced vibration tests on an eleven story reinforced masonry structure with a structural discontinuity at the mid-length of the diaphragms. They concluded that for structures where the in-plane stiffness of the floor system is less than or comparable to the stiffness of the lateral load resisting system it is incorrect to assume that floors are rigid in their own plane.

The dynamic response of moment resistant space frames is very sensitive to the assumed participation of the floor slab as it restrains the rotation of the columns at the floor levels. Edgar and Bertero [39] have observed that no single parameter can be used to define an effective slab width for two way floor systems. The effective width varies with the aspect ratios for the side lengths of the diaphragm, the ratio of the flexural stiffness of the boundary beams to that of the slab, the point of application of the slab loads over the depth of the boundary beams and the ratio of the torsional stiffness of the transverse beam to the flexural stiffness of the flexural beam.

None of the research to date has addressed problems of how the distribution of shears by diaphragms changes with cyclic loading and cracking. The latter was probably an important effect in the tests by Stephen and Bouwkamp. Analyses of large shear panels loaded inelastically and reversed cyclically have shown that prediction of the results requires consideration of compression softening of concrete stressed above $0.7f'_c$, the non-closing of previously formed cracks under load reversals and possibly bond slip effects [40].

Compatability Distortion Effects

Wilby [41] conducted tests on two six story, one fifth scale, one bay by one bay, three dimensional, reinforced concrete structures. The simulated seismic loading was applied in one direction only (the longitudinal direction) of the bi-directionally designed structure. One of the most startling features of the tests was the severe damage caused to the lateral beams by torsional effects ignored in the bi-directional design. The rotations of the longitudinal beam-column joints rotated the ends of the lateral beams. The slab restrained those rotations causing large torsional cracks in the lateral beams and diagonally across the corners of the floor slabs. Those cracks did not close with loading reversals and concrete spalled off the lateral beams after only a few cycles to a lateral force of about 0.4 g. As predicted theoretically in Reference [39], the interaction of the slab and the lateral beam caused a significant increase in the effective stiffness and flexural strength of the longitudinal beams. While the decrease in flexural and torsional stiffness and flexural strength, but not torsional strength of the lateral beams was significant, it was not critical because the lateral loading was perpendicular to those beams. However, in an actual structure the more skew loading likely in a real earthquake could cause torsional cracking and strength and stiffness degradation of all perimeter beams.

Connections Between Diaphragms and Lateral Load Resisting Systems

Popoff [25] has asked what should be the allowable in-plane unit shear stress at the connection between a diaphragm and a wall. Since diaphragms are shear walls oriented horizontally much of their likely shear behavior remote from the connections should be predictable using data for shear walls. However, at the connection with the lateral load resisting system the in-plane shears are imposed on a region where gravity loads may already create high negative moments and high shears. Reinforcement should be adequate to prevent sliding on that connection plane. Based on extensive research Mattock [42] has proposed design procedures to prevent sliding shear failures in reinforced concrete corbels including situations where the plane is also subject to moment and direct tension. Testing has shown that approach to also be valid for in-plane shear transfer between slabs and structural steel or precast concrete beams. Reinforcement should be provided to independently resist the shears, moment and normal forces acting on the plane of potential sliding. Flexural and direct tension reinforcement is provided according to ACI 318-71 concepts. Shear reinforcement is provided according to ACI 318-71 shear friction concepts or "modified shear-friction" equations in which the ultimate interface shear stress, v_u, is taken for normalweight concrete as

$$v_u = 0.8\rho_v f_y + 400 \text{ psi} \ngtr 0.3f'_c \qquad \text{--(1)}$$

where ρ_v is the shear transfer reinforcement ratio. In addition the total reinforcement ratio ρ on the interface times f_y should be not less than 200 psi and care must be taken to properly bond the shear reinforcement on either side of the interface. If the connection is also a construction joint roughness at that joint is vital if the shear stresses are to reach values given by Eq. (1). Otherwise the ultimate stresses drop dramatically to $0.6\rho_v f_y$. If previous reversed cyclic loading [43] has not caused separation at the interface, the shear strength for subsequent monotonic loading is unaffected. However, if the cyclic loading causes separation the shear strength decreases as the width of the crack increases dropping at ultimate to 0.8 of that given by Eq. (1). For cyclic loadings and a given stress level the slip is greater and the separation less than for monotonic loading. Shear transfer across interfaces is not a good energy absorber since the shear-slip loops become extremely S-shaped even when that maximum stress is first increased to as little as half that for failure.

ACHIEVEMENTS OF PAST DECADE AND RESEARCH NEEDS

Flat Plate Construction

Over the past decade there has been research adequate to reasonably define the likely strength, stiffness and ductility characteristics of normal-weight reinforced concrete slab - interior column connections for both cyclic vertical and lateral loads and for connections both with and without shear reinforcement. By contrast little information has been generated on the likely seismic load performance and detailing requirements for slab-exterior and corner column connections especially those incorporating edge beams as required in most current seismic codes. Future experimental research should be concentrated in those areas and possibly interior connections incorporating slabs and beams so that relevant information is also generated for two way slab systems, pan joist and waffle slab construction. In particular the portion of the slab that acts with a beam to resist torsion should be defined. Experimental work is also needed on all types of connections for lightweight and prestressed concrete construction. Future analytical research should be concentrated on developing mathematical models that take account of cracking and inelastic actions, are built on sound reinforced concrete principles and can define the initial and cyclic loading stiffnesses, the hysteretic damping characteristics and the limiting ductilities of connections. Once such models are generated they should be translated into simplified forms suitable for use with existing computer programs.

Slab and Wall Construction

In spite of the widespread use in apartment buildings in seismic zones of slab and wall construction little experimental research has been conducted into design constraints for the slab. Further, analytical studies have been entirely elastic with no account taken of the inelastic actions that must occur in actual structures even under gravity loads. While the analytical studies conducted to date provide general guidelines for assessment of behavior they are inadequate for use with current and proposed methods for determining earthquake forces and are likely to result in unsafe structures. Research is needed on the stiffness of coupling slabs, in appropriate methods for determining the shear and flexural strength of those slabs and on the detailing of reinforcement for those slabs.

Diaphragms

Little experimental and analytical research has been conducted on reinforced concrete diaphragms. More precise information is needed on the factors dictating their flexibility and in particular any restrictions on their flexibility relative to that of the lateral load resisting system. Experiments should be undertaken to define the changes in flexibility likely with cracking and inelastic action, and with differing boundary elements and configurations. Particular attention should be given to the three-dimensional nature of a building's response to earthquakes and how compatability constraints imposed by diaphragms affect that response. Constraints on the connection of diaphragms to the lateral load resisting system should be identified and rules developed to ensure that such connections are not the weak point of the structural system.

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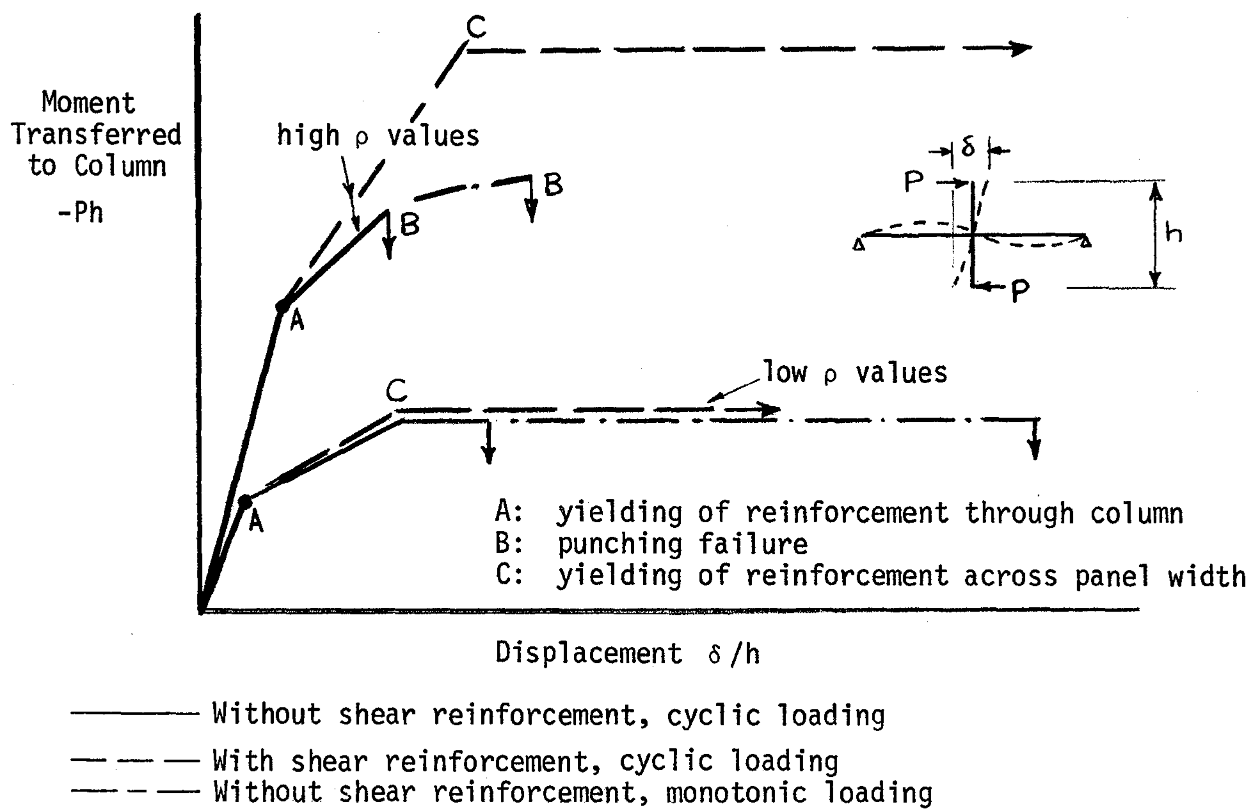
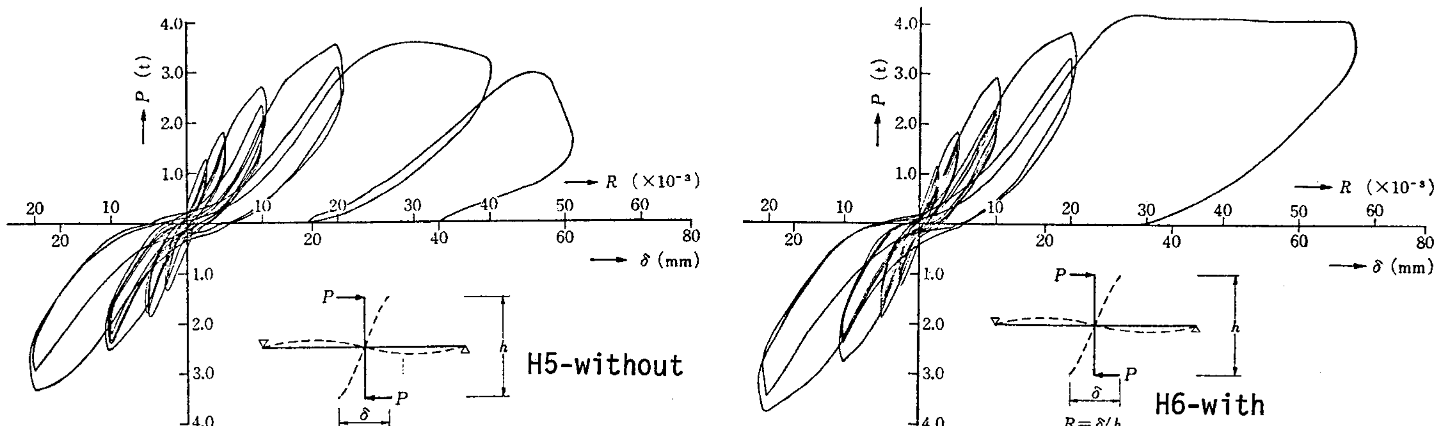
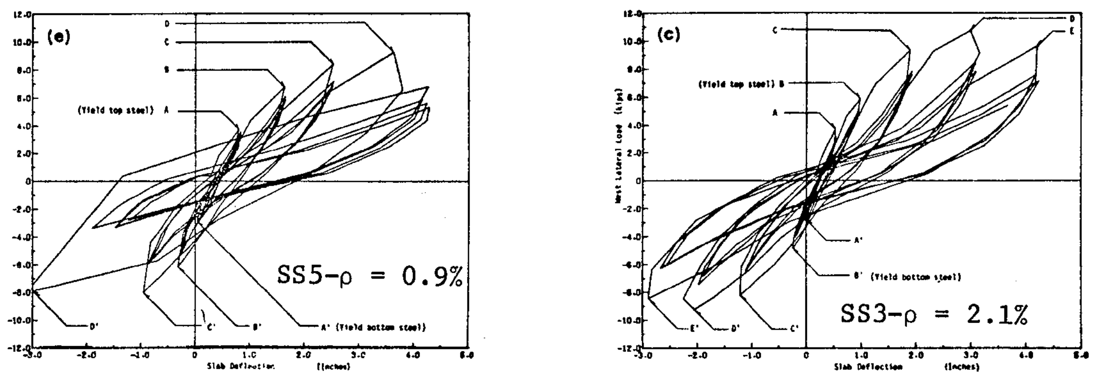


Fig. 1a IDEALIZED MOMENT-DISPLACEMENT ENVELOPES



Effect of shear reinforcement [9]



Effect of ρ value - specimens with shear reinforcement [10]

Fig. 1b LATERAL LOAD-DISPLACEMENT TEST DATA

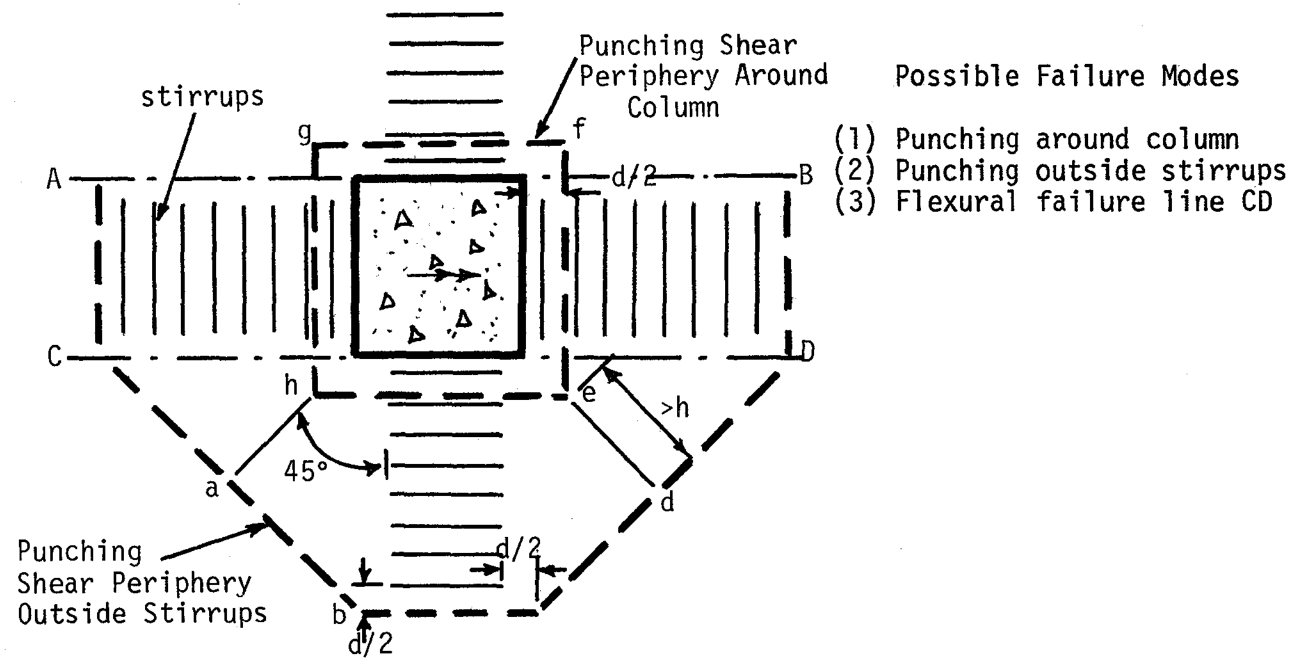
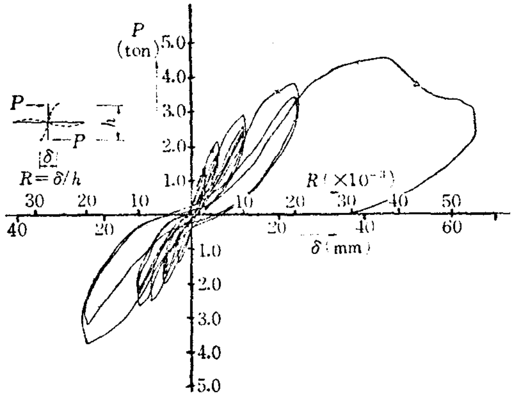
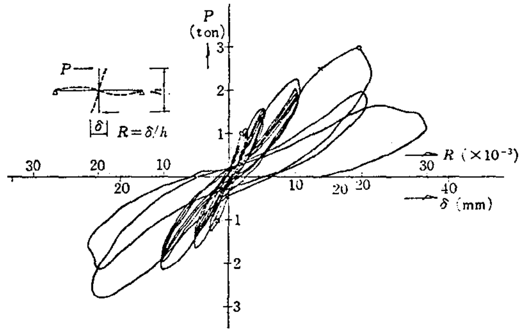


Fig. 2 DETAILING OF INTEGRAL BEAM STIRRUP REINFORCEMENT

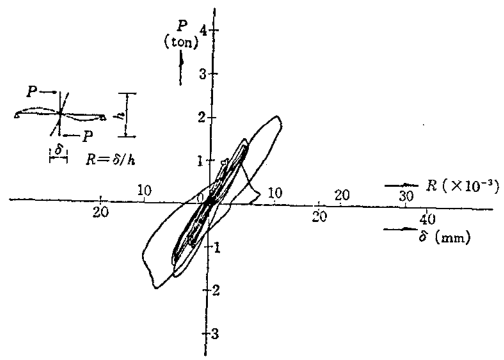


H6 - No Shear Transfer



H10 - H6 + shear of $\sqrt{f'_c}$

Fig. 3 EFFECT OF INCREASING SHEAR TRANSFER [9]



H11' - H6 + shear of $2\sqrt{f'_c}$

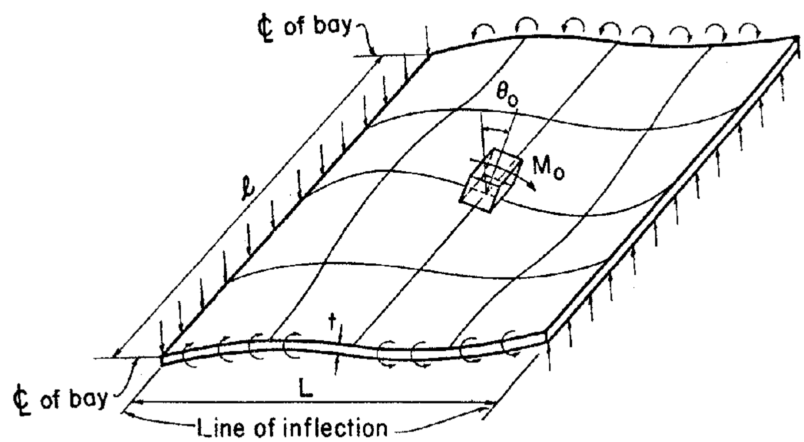


Fig. 4a IDEALIZED SLAB-COLUMN ELEMENT

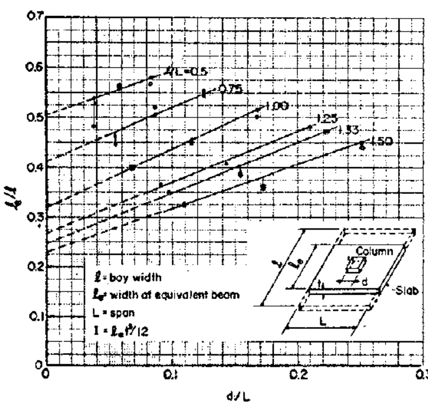


Fig. 4b EFFECTIVE WIDTH OF SLAB [20]

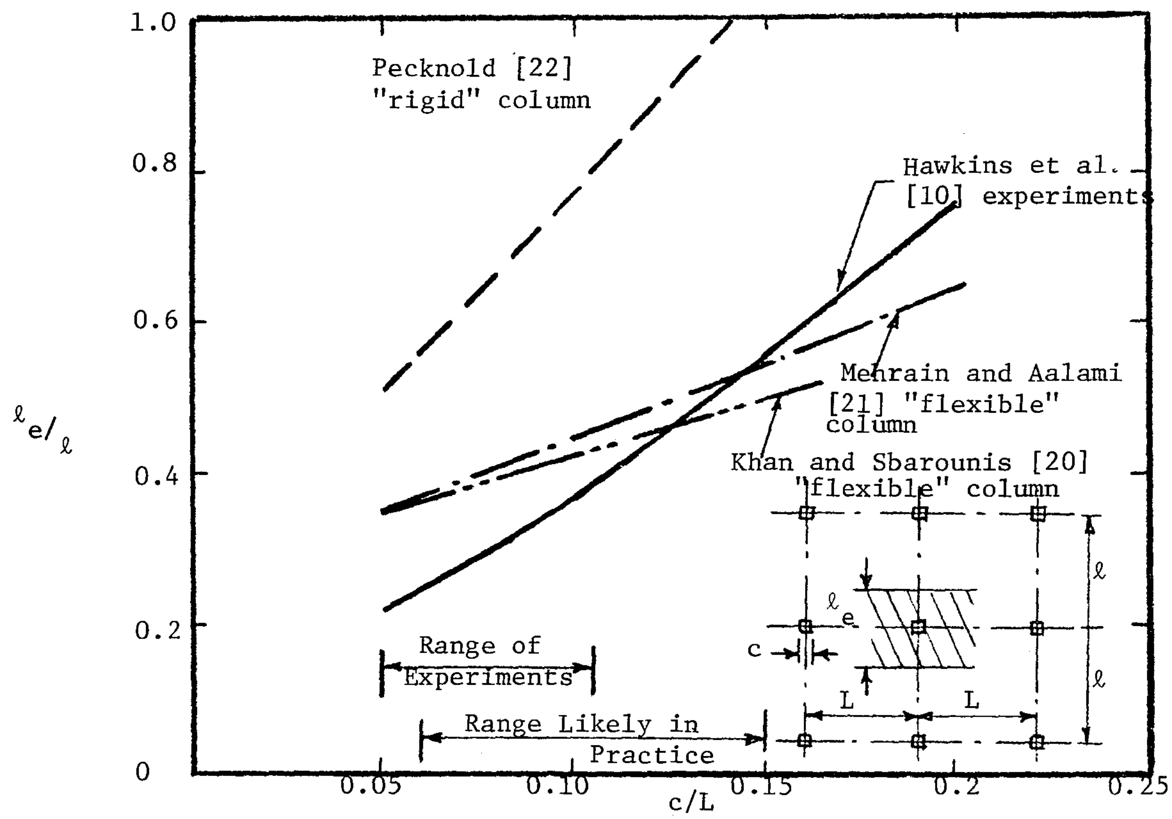
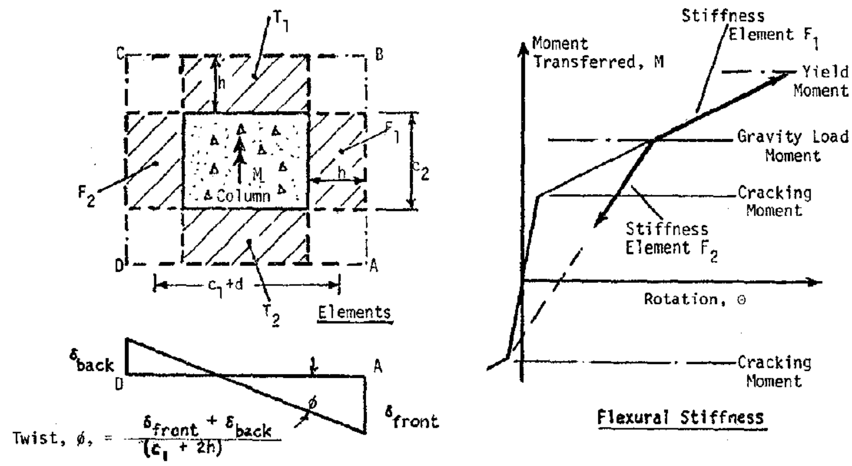


Fig. 4c EFFECTIVE WIDTH OF SLAB REFERENCES [10][20][21][22]



For T_1 and T_2 $\quad (GJ)_{cracked} = \dfrac{[2A_t(c_1+2d-d') + A_\ell s]\ 2.1(GJ)}{100(c_1+d)(h)(s)}_{uncracked}$

where A_t, A_ℓ = cross-sectional area of top bars of slab in c_1, c_2 directions
s = spacing of bars within elements T_1 and T_2

Fig. 5a LATERAL LOAD-STIFFNESS MODEL

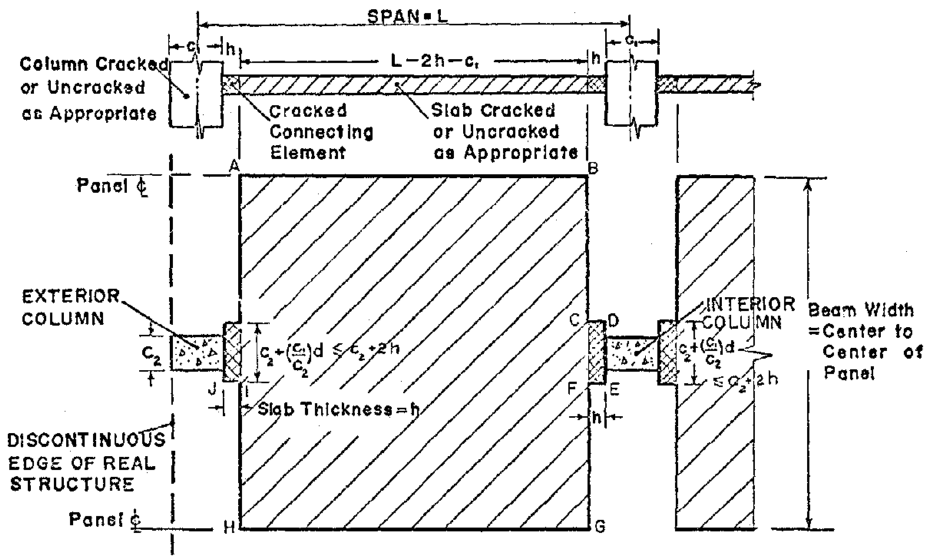


Fig. 6 SIMPLIFIED ELASTIC EQUIVALENT FRAME FOR LATERAL LOADINGS

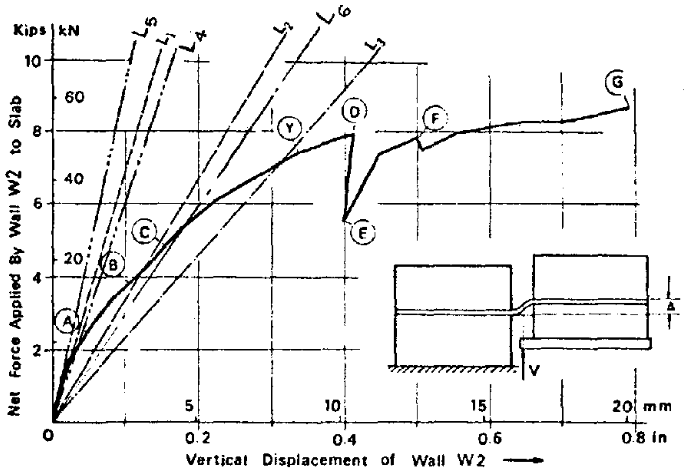


Fig. 7a LOAD-DISPLACEMENT DIAGRAM AND STIFFNESS PREDICTIONS FOR COUPLED SHEAR WALLS

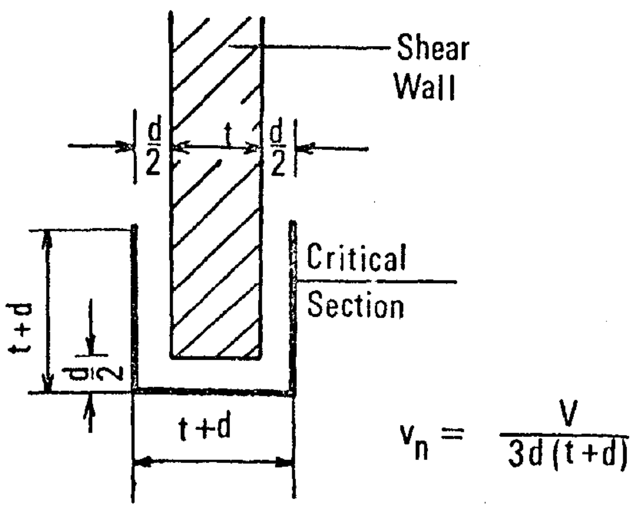


Fig. 7b CRITICAL SECTION FOR SHEAR FOR COUPLED SHEAR WALLS [31]

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

HYSTERETIC BEHAVIOR OF INFILLED FRAMES

Richard Klingner
Assistant Professor of Civil Engineering
The University of Texas at Austin

Recent research has indicated that properly designed infilled frames have several advantages over comparable bare frames, particularly if they may be subjected to severe ground motions (Ref. 1, Fig. 1).

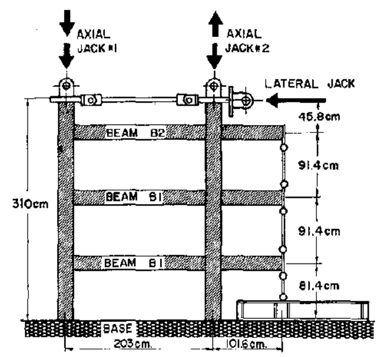


FIG. 1a BARE FRAME SPECIMEN

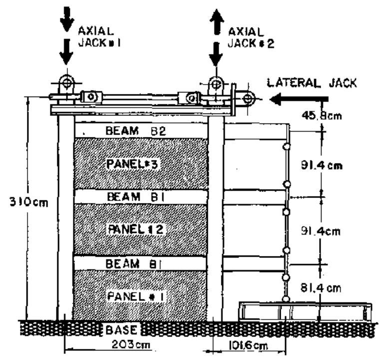


FIG. 1b INFILLED FRAME SPECIMEN

As shown in Fig. 2, infilled frames offer significant increases with respect to strength, stiffness, and energy dissipation capacity. This performance can be achieved using the following design guidelines:

(1) Frame members (particularly the columns) are designed for high rotational ductility and resistance to degradation under reversed cyclic shear loads.
(2) Gradual panel degradation is achieved by using closely spaced horizontal and vertical infill reinforcement.
(3) Panel thickness is limited so that the infill cracking load is less than the available column shear resistance.

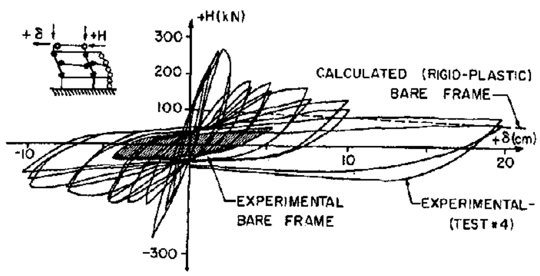


FIG. 2 COMPARISON OF HYSTERETIC BEHAVIOR BETWEEN BARE FRAME (TEST #1) AND INFILLED FRAME (TEST #4)

The ductile behavior of this type of infilled frame is considerably different from ductile shear wall behavior. A ductile shear wall is designed to fail in flexure. Under complete load reversals, this type of failure often results in the opening of cracks which run through the cross section of the wall. Rotational ductility is then generally limited by resistance to sliding shear failure, which is particularly likely to occur at the base of the wall, or at horizontal construction joints. While such failure may be prevented or delayed by diagonal steel, it may be difficult to place and anchor a sufficient amount of diagonal steel throughout the height of the wall.

In contrast, the type of infilled frame considered herein is designed to respond inelastically as a braced frame. Its failure is governed by crushing of the equivalent diagonal compressive struts in the infill panels. To ensure that an infilled frame subassemblage will fail as a braced frame rather than a ductile shear wall, it must be designed so that the lateral shear necessary to cause flexural failure considerably exceeds that required to produce infill crushing.

Although the high strength and stiffness of flexural shear walls result in good earthquake resistance, it may be difficult to repair a flexural shear which has been badly damaged by a severe earthquake. On the other hand, a comparably damaged infilled frame can be repaired by removing the damaged infill panels and replacing them with new ones.

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DESIGN METHODS AND EXPERIMENTAL AND ANALYTICAL INVESTIGATIONS RELATED TO THE EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION OF FRAME-WALL STRUCTURES; CORRELATION WITH FIELD OBSERVATIONS OF EARTHQUAKE DAMAGE

DESIGN OF FRAME-WALL STRUCTURES

by

Arnaldo T. Derecho
Manager, Structural Analytical Services
Portland Cement Association, Skokie, Illinois

INTRODUCTION

General

Observations of the performance of buildings subjected to earthquakes during the last decade have focused attention on the need to minimize damage in addition to ensuring the general safety of buildings during strong earthquakes [1-3]. The need to control damage to both structural and nonstructural components during earthquakes becomes particularly important in buildings such as hospitals and other facilities which must continue operation following a major disaster. Damage control, in addition to life safety, is also economically desirable in tall buildings designed for residential and commercial occupancy, since the nonstructural components in such buildings usually account for from 60 to 80 percent of the total cost. For the purpose of this discussion, a strong earthquake is that which can reasonably be expected to occur several times during the life of a structure.

While reinforced concrete rigid frame structures have performed reasonably well in earthquakes, particularly with respect to the primary performance criterion of life safety (i.e., no collapse), their inherent flexibility usually results in lateral displacements that cause significant damage to nonstructural components in a building. Reinforced concrete structural walls (or shear walls) have long been used to stiffen tall buildings against wind. When properly designed, walls offer one of the most logical and economical means of minimizing damage in buildings subjected to strong ground motion.

There is little doubt that structural walls offer an efficient way to stiffen a building against lateral loads. When proportioned so that they possess adequate lateral stiffness to limit interstory distortions to acceptable levels and designed to maintain their strength under the earthquake-induced motions, walls effectively reduce the likelihood of damage to the nonstructural elements in a building. When used with rigid frames, walls form a structural system that combines the gravity-load- carrying efficiency of the rigid frame with the lateral-load-resisting efficiency of the structural wall.

In its simplest form, the frame-wall structure consists of an unperforated wall linked to a rigid frame. The linkage may consist of beams rigidly connected to the wall or just the floor slabs. Often, the 'wall' in a frame-wall system takes the form of coupled walls, i.e., walls in the same plane connected by beams. This is typical in the corewalls of so-called 'hull-core' or 'tube-in-tube' systems. As mentioned, the structural walls in frame-wall systems, whether consisting of single unperforated walls or of coupled walls, are generally used in multistory buildings when the stiffness of the frame alone (as designed for gravity loads) is not sufficient to limit the lateral displacements due to wind or earthquake motions to tolerable levels. It is mainly this application of walls in multistory structures which will be discussed here. The behavior of short walls, i.e., walls with a height-to-depth ratio of less than about 2, is governed by slightly different considerations [4] than those applying to tall, relatively slender, walls and will not be discussed here.

Distinguishing Feature

A major distinction between the typical frame-wall system and the rigid frame structure is the interaction that takes place between the frame and the wall under lateral loading (Fig. 1). This interaction, which results from the tendency of the basic elements to deflect in different modes under lateral load, often gives rise to horizontal story shears acting on the frame columns at the top stories which are greater than the corresponding total applied story shears. In the presence of major discontinuities in stiffness, particularly in the wall, this same interactive behavior can result in horizontal story shears acting on the wall and the frame which, separately, can be appreciably greater than the corresponding total applied story shears. This is illustrated in Fig. 2, which shows the horizontal story shears resisted by the wall and the frame columns in a statically loaded frame-wall structure where the wall is discontinued at the first story [5]. The story shear shown as corresponding to the wall at the first story level is actually resisted by the columns supporting the wall. Under strong ground motion, high ductility or deformation requirements tend to be associated with such discontinuities.

Note that horizontal interactive forces due to lateral loads can also occur between coupled walls which have different stiffness distributions along their height, as shown in Fig. 3.

BASIC PLANNING AND DESIGN CONCEPTS

Typical Plan Configurations

The general objective in the design of frame-wall structures for strong ground motions is the provision of sufficient stiffness, strength and deformation capacity to withstand the induced forces and deformations while limiting the overall displacements to acceptable levels. In planning multistory frame-wall structures to meet this objective, certain general features are desirable. Among the more important of these are plan symmetry, the avoidance of significant discontinuities in mass, stiffness or geometry and the location of stiffening elements where they are most effective in resisting displacements parallel to the plan axes as well as torsional motions. This third consideration requires that structural walls be located close to the plan periphery. Because torsion (whether due to the non-coincidence of the centers of mass and resistance or to phase differences in the excitation of various points at the base of a structure by seismic waves propagating at finite speed [6]) can induce significant forces in corner vertical elements, an effort should be made early in the design stage to minimize its effects. The above three basic desirable features are intended to minimize torsional effects and the force concentrations and associated deformation requirements that occur at regions of major discontinuity in a structure. An example of a plan for a rectangular building illustrating the above plan features is shown in Fig. 4.

A frame-wall plan configuration that is commonly used in tall office buildings is the so-called 'hull-core' or 'tube-in-tube' system, consisting of a centrally located service core and a closely spaced grid of frame elements

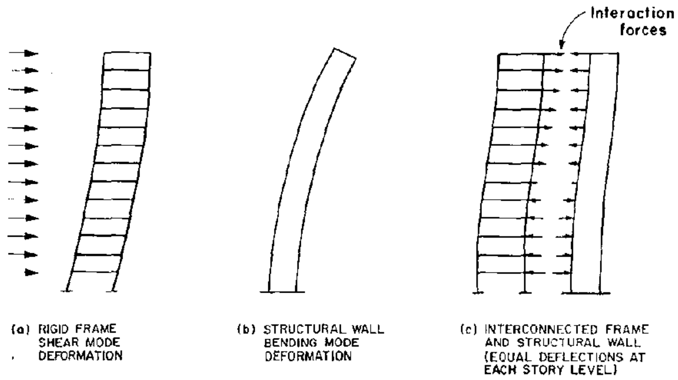


Fig. 1 Structural Wall-Frame Interaction Under Lateral Loading

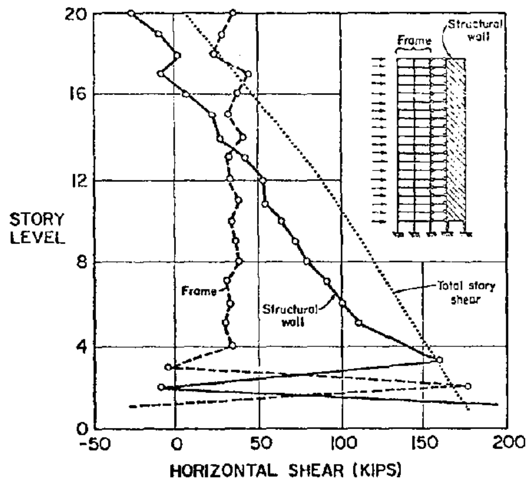


Fig. 2 Distribution of Horizontal Story Shears Between Wall and Frame Under Statically Applied Lateral Load

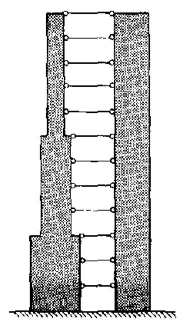


Fig. 3 Linked Structural Walls with Different Stiffness Distribution Along Height

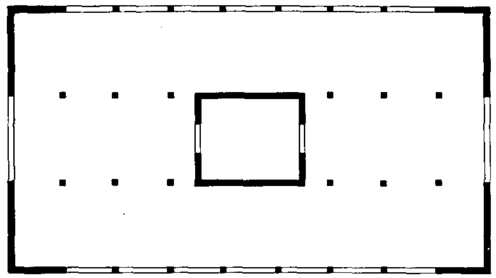


Fig. 4 Example of a Frame-Wall Plan

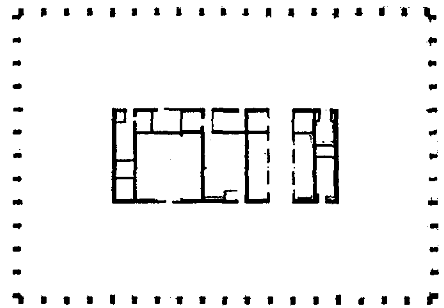


Fig. 5 Typical Tube-in-Tube Plan

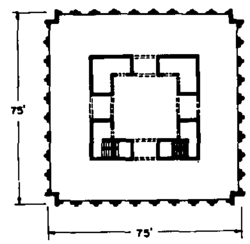


Fig. 6 Banco de America, Nicaragua

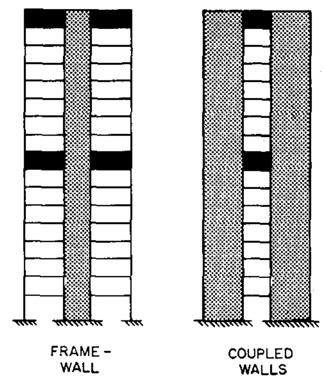


Fig. 7 "Belted" Systems

(Fig. 5). In this system the corewalls are usually perforated for elevator doorways and other openings and thus function as coupled walls in one or both directions.

A more effective disposition of the stiffening walls, particularly with respect to torsional resistance, would have narrower walls located closer to the plan periphery. The walls can then be coupled by beams to increase the overall stiffness of the system and provide the desirable energy-dissipating mechanism in the event of a strong earthquake. The Banco de America building in Managua is a good example of this arrangement (Fig. 6). For certain plan proportions and building uses, however, this type of layout may not be too welcome from the architectural point of view nor too efficient from the mechanical/electrical services standpoint.

Belt Courses

A device used to enhance the coupling between the different vertical elements, and hence increase the overall lateral stiffness, of relatively tall structures is the so-called 'belt course'. This is a one- or two-story-deep beam extending across the width of the structure (Fig. 7). The principal purpose of such heavy beams is to allow the resulting structure to resist the overturning moment due to lateral loads more by cantilever action, that is, by mobilizing to a greater degree the axial resistance of the connected vertical elements. Belt courses are usually located at the top of the structure and at one or more intermediate floors where mechanical equipment and other services can be placed. Studies on the optimal location of belt courses are reported in References 7 and 8.

The use of reinforced concrete belt courses has proved quite effective in systems subjected primarily to wind loading. Their use in earthquake-resistant frame-wall systems, however, may require special attention. The fact that they represent regions of discontinuity along the height of the structure with accompanying high shears, and their appreciably greater stiffness and strength relative to the connected frame columns will almost ensure significant yielding in the columns. Also, the tensile forces that may be developed in the connected vertical elements will tend to reduce their shear (and deformation) capacity. Because of these considerations, the increase in overall lateral stiffness obtained by greater coupling may be outweighed by the negative effects that the use of belt courses may have on the behavior of frame elements attached to it.

Earthquake-Resistant Design Concepts

In addition to the general layout of the building in plan and elevation, there are considerations relating to the manner in which each structural element making up the system is to function under progressively increasing amplitudes of deformation associated with response to strong ground motion.

A major advantage of frame-wall systems, as compared to isolated walls, i.e., parallel walls not in the same plane and connected by floor slabs, is their structural redundancy. This redundancy allows the engineer the option of designing into a structure a hierarchy of elements such that inelastic action occurs first in secondary elements and progresses up the scale to primary elements as the overall deformation of the structure increases. The term secondary is used here to denote elements which are not critical to the overall stability of the structure, i.e., elements in which distress caused by excessive inelastic action cannot seriously undermine the overall gravity-load-carrying capacity of the structure. Secondary elements will generally take the form of beams but may also be vertical walls specifically designed for this purpose. The desirable condition would be to have most of the inelastic action and energy dissipation occur in secondary elements.

Thus, coupled wall systems can be so proportioned that significant yielding under strong ground motion occurs in the coupling beams before inelastic action takes place at the bases of the walls. The so-called "strong column-weak beam" concept used in proportioning moment-resisting frames also serves the same purpose of forcing most of the inelastic action to take place in elements that are less critical to the overall stability of the system. The same general concept applies to frame-wall structures where the wall is designed to be the principal lateral-load-resisting element while the frame carries most of the gravity loads. Figure 8 shows an example of a plan where the walls need not be relied on to carry the gravity loads.

Structural Wall and Frame -- In a frame-wall system consisting of a single (i.e., not coupled) wall connected to a frame, yielding under strong ground motion is most likely to occur first at the base of the wall (unless the beams connecting the wall to the frame are very stiff, i.e., deep or have short spans). In many cases, the wall serves not only as the major lateral-load-resisting element but also carries a significant portion of the gravity loads. The axial compressive forces produced by gravity loads on the wall tend to increase the shear capacity of the wall and help reduce tensile stresses at the foundation level. However, because yielding can occur early at the base of the wall, it is important to design the wall so that its vertical-load-carrying capacity is not impaired as a result of hinging at the base.

Coupled Walls and Frame -- A preferable configuration, and one that occurs often in practice, is a system consisting of coupled walls connected to a frame. In such a system, most of the inelastic action (energy dissipation) can be made to occur in the coupling beams before yielding occurs at the bases of the walls [9,10]. The strength (i.e., yield level) of the coupling beams can be varied along the height of the structure to permit most of these to yield at a predetermined deflection, if desired. Because of the feasibility of controlling the hinging sequence and the relative ease and economy with which the coupling beams in a coupled wall system can be repaired, this type of structure stands out as a most appropriate subsystem for earthquake-resistant reinforced concrete structures. Its superior behavior is such that, even when a solid wall is called for, it may be desirable to deliberately design and detail the wall as a coupled wall system, with nonstructural filler panels used to cover the spaces between coupling beams. This may require compensating for the stiffness lost by introducing coupling beams in place of the solid web of the wall. However, it is believed that this is a reasonable price to pay for an improved performance which allows most of the inelastic action to take place in secondary, easily repairable, elements (i.e., coupling beams) rather than at the critical section near the base of the wall.

In all of the above schemes, it is assumed that the individual structural elements making up the frame-wall system will be designed to provide the necessary strength and deformation capacity. The results of recent tests conducted in various laboratories on large-size coupled wall [4, 11], and isolated wall specimens [12-14], as well as coupling beams [15-18], beam- and

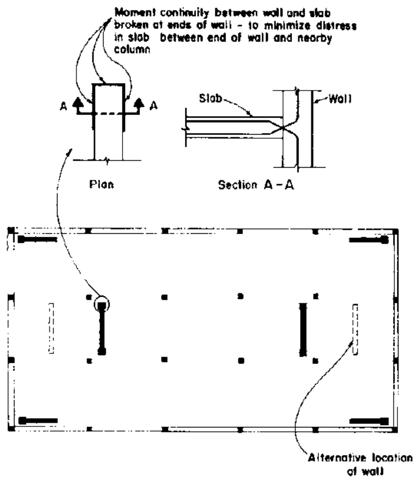


Fig. 8 Walls Functioning Primarily as Lateral-Load-Resisting Elements

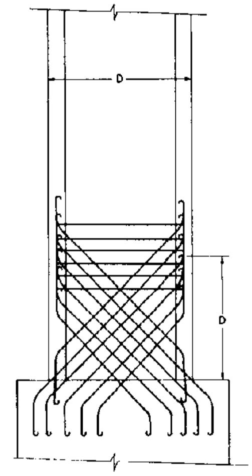


Fig. 9 Diagonal Web Reinforcement at the Base of a Wall

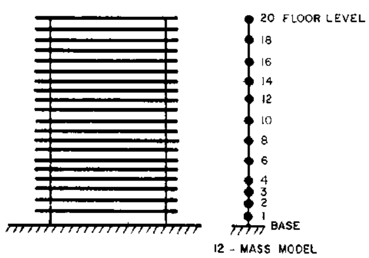


Fig. 10 Isolated Structural Wall Model

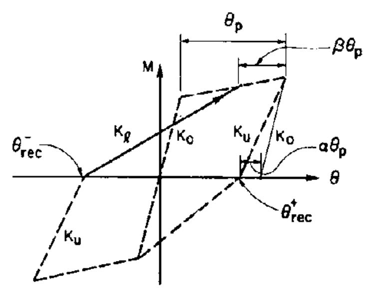


Fig. 11 Decreasing Stiffness Model Parameters α and β

slab-column connections [20-30], and columns [31-33], subjected to slowly reversing loads indicate that it is economically feasible to design frame-wall structures with capacities equal to or greater than the expected demands associated with response to strong ground motions.

Special Details -- In planning a structure for earthquake resistance, certain features (e.g., symmetry, avoidance of major discontinuities, etc.) have been pointed out as being desirable in order to reduce the forces induced in the structure. This objective of reducing the forces induced by ground motions can logically be pursued further by introducing special devices or mechanisms into the structure.

Among a number of proposals advanced to improve the response characteristics of a structure through the use of special devices or mechanisms are: (a) isolation devices to limit the magnitude of the earthquake forces transmitted to the superstructure. These may take the form of ball-bearing or Teflon pads combined with dampers [34, 35] or columns at the base designed to yield at a predetermined deflection, i.e., soft story concept [36, 37]; rocking ball mechanisms [38] and similar devices, and (b) mechanisms designed to provide additional energy dissipation through yielding, either in bending [39] or direct tension [40] of specially mounted steel rods. A method of increasing the lateral deformation capacity of walls by introducing slits into it (i.e., the "slitted wall"), thereby converting it essentially into a series of closely spaced columns has also been used [41].

While these special devices, if reliably designed and properly maintained, can provide some attenuation in response and in a sense increase the margin of safety in design, the basic problem should still be recognized as that of determining reliable estimates of the demands, corresponding to any particular configuration, and the correlation of these with available capacities. Obviously, where conventional systems can be shown to provide the necessary capacity economically, these would be preferred.

THE DESIGN PROBLEM

Basic Requirements

As in all structures to be designed for earthquake resistance, the basic design requirements for frame-wall structures consist of:

(1) Estimates of the force and deformation demands in critical regions of structures corresponding to different combinations of the significant structural and ground motion parameters. These data on demands deal primarily with the requirement of life safety, i.e., the prevention of collapse under the design earthquake.

An auxiliary consideration is the combination of stiffness and strength needed to minimize damage to both structural and nonstructural components by limiting the overall structural displacements.

(2) Estimates of the strength and deformation capacity of typical structural elements corresponding to different values of the significant design parameters, i.e., element cross-section, reinforcement details, axial load, level of shear, etc.

The compilation of comprehensive data on force and deformation demands which can serve as bases for a design procedure will require extensive dynamic inelastic analyses of realistic models of the basic structure. The desired information should show the variation of demand with the significant structural and ground motion parameters. In a similar manner, design data which can be used for proportioning members must be obtained through a systematic test program using large-size specimens subjected to realistic loading conditions.

At present, there is a lack of design information relating to frame-wall systems reflecting a correlation between force and deformation demands with corresponding capacities. There has been no systematic compilation and correlation of data aimed specifically at developing design information for use in everyday practice.

Typical Design Approach

Apart from a straightforward adherence to standard code requirements, the usual approach to the design of multistory buildings which justify a more-than-usual investigation, i.e., beyond that normally required by codes, consists in carrying out elastic time-history analyses of appropriate models using a few input accelerograms [42,43] . Engineering judgment is then used to arrive at design values by allowing for inelasticity developing in critically stressed members on the basis of the calculated elastic forces and displacements.

In other cases, estimates of the maximum overall displacements and the associated forces are obtained by modal superposition using smoothed or averaged response spectra. The most common practice is to take the square root of the sum of the squares [44-46] of the response corresponding to the first few significant modes (assuming the modal frequencies to be spaced far enough from each other). Where the calculated elastic moment is greater than the known yield moment of a member, the ductility requirement is sometimes estimated on the basis of the overstress ratio, i.e., the ratio of the maximum elastic moment to the yield moment.

Comparisons [9, 47] of the results of linear and nonlinear dynamic analyses, however, have shown that while an elastic analysis may provide fair estimates of the maximum overall structural displacements, it can grossly underestimate the magnitude of the inelastic deformations in critical regions of structures. In order to obtain reliable data on deformation demands, inelastic time-history analyses of realistic models are required.

A design procedure proposed by Shibata and Sozen [48] replaces the planar model of a structure by a "substitute (elastic) structure" with reduced stiffness and an equivalent viscous damping based on assumed tolerable damage levels. The design forces are then obtained by a modal superposition analysis of the substitute structure using linear response spectra. The procedure allows, in an approximate and indirect way, for the local concentration of inelastic deformations in critical members.

A more elaborate analysis and design procedure employing both linear and nonlinear time history response analyses was discussed by Bertero and Kamil in Reference 49. The approach includes a logical progression from linear dynamic analysis for preliminary proportioning of elements to a verification of the final design by inelastic dynamic analysis, a procedure clearly desirable for major projects. The method allows an examination of the deformations in critical regions of structures. However, because the procedure requires the use of dynamic analysis programs, its use on moderate-sized projects by the average engineer - who may not have access to the necessary computing facility or even the time to familiarize himself with the programs - may be limited.

The Need for a Simple Rational Design Procedure -- From the point of view of broad application, it would be desirable to have at the disposal of the average engineer relatively simple and practical design information which provides reliable estimates of the force and deformation demands in critical regions of structures as well as guides on the proper proportioning of elements to provide the required capacity. Such information should cover the practical range of variation of the significant design parameters. The development of this design information will obviously require a comprehensive and integrated analytical and experimental program of investigation.

PROPOSED PROCEDURE FOR DEVELOPMENT OF DESIGN INFORMATION

Generation of Data on Demand Through Dynamic Inelastic Analysis

It has often been noted that although our structural analytical capabilities have advanced considerably during the last two decades - mainly as a result of the electronic digital computer - this advance has not been matched by a corresponding improvement in the overall bases for the design of structures, particularly with respect to strong ground motions. While this condition may be typical of scientific progress in general, that is, of theoretical analysis spearheading the development of rational design procedures, it would seem desirable at this point to seek to narrow this gap by taking full advantage of our vastly improved analytical capability to further the aims of structural design.

It is worth noting that except for the investigation of individual structures, most analytical studies on dynamic earthquake response have been concerned mainly with either examining the validity of certain proposed mathematical models of structures or with parametric studies of response. Relatively little effort has been spent in a systematic compilation of force and deformation demands corresponding to different combinations of the significant design variables.

There is no doubt that the development of adequate mathematical models constitutes the first step in the preparation of the necessary tools for dynamic analysis. In assessing the validity of a proposed model - developed to account for an action or mechanism judged to be significant in a structure - the results of dynamic analyses using the model are usually compared with data obtained from shaking table tests of specimens designed specifically for this purpose [10, 50-52], or with observed damage of actual structures subjected to earthquake motions [9,47,53]. The accuracy of the analytical prediction (and hence the validity of the proposed mathematical model) with respect to the observed experimental behavior is generally determined by a comparison of time-history response curves for nodal displacements. In the case of structures damaged by earthquakes, the damage which may be inferred from calculated deformations is compared with the extent of observed damage. In this connection, the importance of using the proper criteria in establishing the equivalence between analytical and experimental results should not be overlooked. For instance, from the design standpoint, it is more important to have reliable estimates of the critical force and deformation demands in local regions of primary elements than of overall or gross structural displacements, which are generally not as sensitive to parameter variations. Thus, agreement between analytical and experimental results in terms of rotational ductilities in critical regions, rather than in terms of overall or top displacements, may be the more significant criterion in such comparisons.

In developing information for use in design practice, a slightly different approach must be taken to utilize our dynamic inelastic analysis capabilities. In contrast to the basic use of analysis to assess the validity of certain mathematical models or modelling techniques, the estimation of critical force and deformation demands in primary elements of typical structural configurations requires the systematic compilation of response data for practical ranges of values of the significant parameters. It is in the generation of comprehensive data on demand to serve as bases for a design procedure that dynamic analysis can find one of its most useful applications. This, of course, assumes the use of an adequate mathematical model as a basic analytical tool. While there will always be room for improving our models, particularly as our knowledge of structural behavior improves, it is believed that we at present have the necessary tools to determine reasonably good estimates of earthquake demands in structures.

Because of the need for simplicity in the design procedure, only the most significant parameters can be considered in the formulation of the design methodology. Therefore, a parametric study to determine the relative importance of the different variables affecting dynamic structural response is necessary.

For the particular case of frame-wall structures, the relative influence of the following basic structural parameters on the force and deformation requirements in critical regions may have to be examined:

1. fundamental period of structure (as affected primarily by stiffness)
2. yield level in flexure of walls
3. yield level in shear of walls
4. coupling beam-to-wall stiffness ratio } where coupled
5. coupling beam-to-wall strength ratio } walls are used
6. frame-to-wall stiffness ratio } 'wall' may be single wall(s) or
7. frame-to-wall strength ratio } coupled walls
8. foundation rocking

Once the significant variables have been isolated, a comprehensive series of analyses can be undertaken to compile data on estimated demands corresponding to measures of available capacity obtainable from experiments. The generation of design data will involve analyses using several input accelerograms of reasonable duration and having frequency characteristics designed to excite a structure critically [54]. Furthermore, analyses using input motions of varying intensity will have to be carried out to obtain data corresponding to varying ranges of expected ground motion intensity. In order for the dynamic response data to fulfill the requirements implied in this application, the analyses must obviously be quite comprehensive.

Development of Experimental Data on Capacity

It is clear that any advance in design capability will have to rely heavily on experimental data concerning behavior of elements and structures, in addition to analytical results. Until relatively recently, little in the way of experimental data has been generated relating to the behavior of typical structural configurations, and particularly of structural walls and wall systems, subjected to earthquake or earthquake-type loading. Whether this is a reflection of funding priorities - given the generally greater cost of experimental programs - or an indication of the preference on the part of many researchers to undertake analytical studies rather than experiments, is not clear. However, it is clear that if a significant advance is to be accomplished in the area of design, a systematic approach combining both analysis and experiment must be considered. Such an effort must be specifically aimed toward the development of design procedures covering the more important structural types.

The development of design data to guide the proportioning and detailing of structural elements and systems for a specified strength and deformation capacity will require the systematic determination of the effects of different structural and loading variables through testing of large-size specimens under representative loading conditions. Such tests, designed to isolate, to the extent possible, the effect of each major variable, would obviously have to be fairly extensive.

Experimental investigation of the effects of the following variables on the strength and deformation capacity of structural walls and wall systems is needed:

1. wall cross-section
2. concrete strength
3. confinement reinforcement
4. shear reinforcement
5. level of applied shear
6. moment-to-shear ratio
7. axial load.

In the process of obtaining experimental information on capacity for different element types, the feasibility of utilizing special reinforcement details not normally used in conventional reinforced concrete construction should be explored [4,11,15,16,19,27] . The effectiveness of alternative details designed to enhance the resistance of elements to cyclic inelastic deformations under high shears should be examined. For instance, the effectiveness of diagonal web reinforcement at the base of structural walls deserves consideration (Fig. 9). It is believed that with proper detailing of the anchorage of the diagonal bars and confinement of the region in the web near the corners of the base, such a detail would prove more effective than horizontal bars. Construction of such a detail need not cause undue difficulties if it is prefabricated and used only in potential hinging regions, especially at the bases of walls.

Correlation of Data on Demand and Capacity

The response data from dynamic inelastic analyses should provide estimates of the stiffness requirements to limit distortions in structures to tolerable levels as well as force and deformation demands corresponding to particular combinations of the significant structural and ground motion parameters. The force and deformation demands in regions of elements which become inelastic are of particular interest, since design attention will have to be focused on these inelastic regions. Such analytically derived data on demand, when correlated with experimental data on capacity, can serve as bases for determining appropriate force levels to be used in design.

The development of practical design information based on a correlation of analytically determined demands and experimentally derived capacities can be tedious but otherwise fairly straightforward. However, relatively little has been done to generate the necessary information and carry out the correlation to the point where results useful to the design engineer can be formulated. An effort along the lines suggested here has been initiated and is now in progress at the Portland Cement Association, for the particular case of isolated structural walls [55]. The project is sponsored in major part by the National Science Foundation. An indication of what can be done for the case of frame-wall structures may be obtained by considering a few of the results of this particular study.

Determination of Design Force Levels (for Isolated Structural Walls)

Figures 12 to 16 illustrate the results of the dynamic analyses of 20-story isolated structural walls (Fig. 10) subjected to input motions having a spectrum intensity* equal to 1.5 times the spectrum intensity of the N-S component of the 1940 El Centro record (= SI_{ref}). The graphs shown in these figures represent the maximum response to six different input motions. Similar graphs have been prepared for walls of different heights and input motion intensities equal to 0.75 and 1.0 (SI_{ref}). The intent in determining the critical dynamic response quantities for different input motion intensities was to have such values available in anticipation of the development of seismic regionalization maps defining zones in terms of the maximum spectrum intensities of the expected motions - or some quanitity related to these - and their corresponding return periods or recurrence intervals.

The structural models used in obtaining Figs. 12 to 16 had the following common characteristics: viscous damping coefficient for first and second modes = 0.05; yield stiffness ratio, i.e., the ratio of the slope of the second, post-yield branch to the slope of the initial 'elastic' branch of the bilinear M-θ curve, = 0.05; parameters characterizing the 'decreasing stiffness' hysteretic loop (Fig. 11): unloading parameter, α = 0.10, reloading parameter, β = 0; stiffness of wall uniform throughout height; strength, i.e., M_y, uniform throughout height except for adjustments to reflect effect of axial load; and, wall fully fixed at base, with the input motion applied directly to base.

Figures 12 and 13 show the variation of the maximum top displacement and interstory displacement, respectively, with the initial fundamental period for different values of the available ductility, μ_a. The essentially identical maximum displacements of structures having different available ductilities (for the same period), a behavior observed earlier with respect to single-

*defined as the area under the 5%-damped velocity response spectrum corresponding to 10 seconds of the ground motion, between periods 0.1 sec to 3.0 sec.

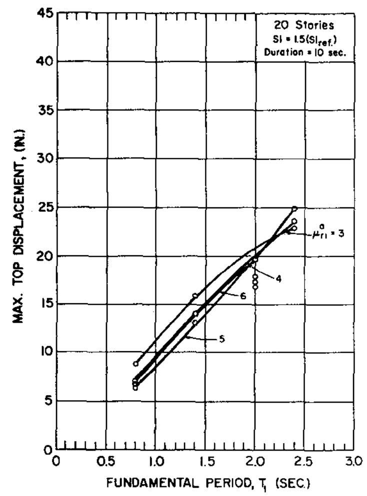


Fig. 12 Max. Top Displacement as a Function of Fundamental Period and Available Ductility - Isolated Structural Walls

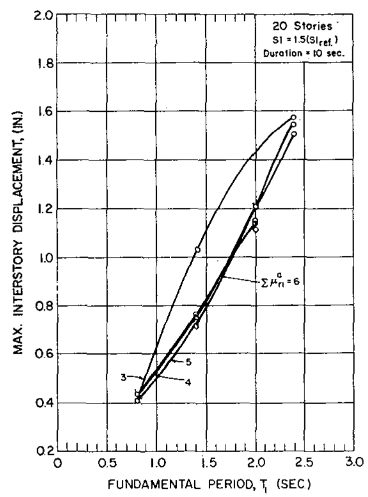


Fig. 13 Max. Interstory Displacement as a Function of Fundamental Period and Available Ductility - Isolated Structural Walls

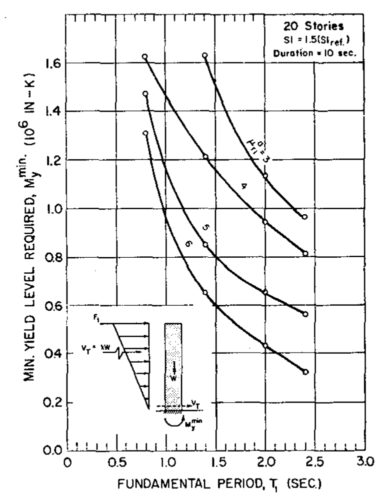


Fig. 14 Min. Yield Level Required at Base of Wall as a Function of Fundamental Period and Available Ductility - Isolated Structural Walls

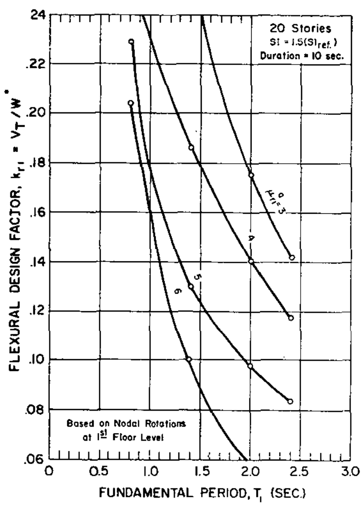


Fig. 15 Flexural Design Factor, k, as a Function of Fundamental Period and Available Ductility - Isolated Structural Walls

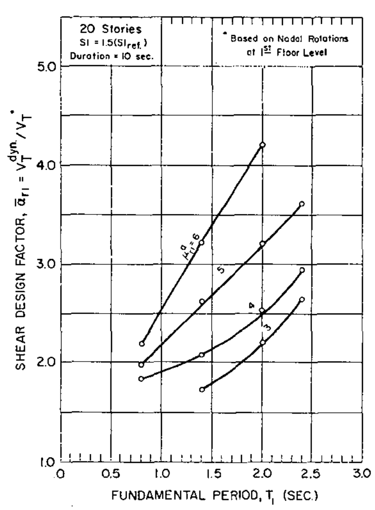


Fig. 16 Shear Design Factor, $\overline{\alpha}$, as a Function of Fundamental Period and Available Ductility - Isolated Structural Walls

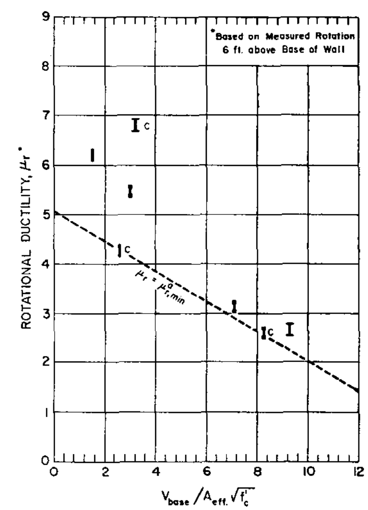


Fig. 17 Measured Available Rotational Ductility, μ_r^a, as a Function of the Applied Maximum Shear Stress - Isolated Structural Walls [see Ref. 14]

degree-of-freedom systems [56] as well as frames [47], will be noted in these figures. The variation of the minimum yield level, M_y, required at the base of the wall with the fundamental period, for different values of the available ductility, is shown in Fig. 14. As might be expected, the figure shows that for a given fundamental period, a higher available ductility implies a lower minimum required strength (yield level) at the base.

In determining ductility requirements for use in design, a study was conducted [55] to assess the relative magnitudes of the different measures of deformation in the hinging region that have been used in the literature. The measures of deformation considered are shown in Fig. 18. These include three measures of rotation and one of rotational energy. The study involved the calculation of all four measures for both dynamic analysis results and test specimens. A comparison of these different measures of deformation indicated that, at least for the samples considered, the satisfaction of the deformation requirement in terms of rotational ductility, μ_r, generally ensures the satisfaction of the other measures of deformation.

Figures 15 and 16, based on dynamic analysis results, show examples of charts that can be used in actual design work. These charts, together with Fig. 17, which summarizes the essential results of tests of isolated structural walls of varying cross-section and detail [14], form the basis for establishing the design force levels to be used in proportioning the structure. The use of the charts is best explained by describing the steps in the design procedure. A similar general procedure can be applied to frame-wall systems, with appropriate modifications to cover the additional considerations involved in the more complex systems.

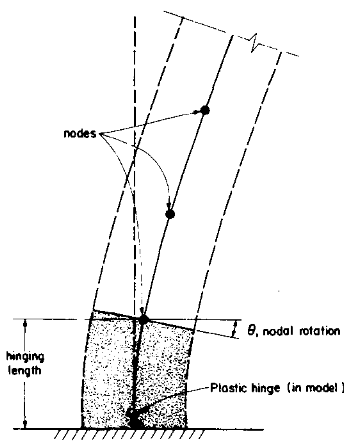
(1) Preliminary Design -- A logical first step is a design satisfying gravity and wind loading requirements. Here the proper disposition of stiffening elements in plan, with particular regard to symmetry and torsional resistance, cannot be over-emphasized.

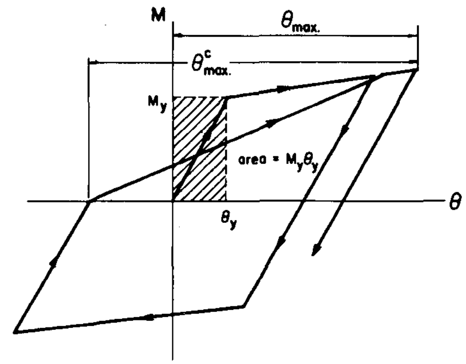
From the preliminary design an initial effective stiffness can be assumed and the corresponding initial fundamental period, T_1, determined.

(2) Stiffness Design for Damage Control -- As far as stiffness and the associated displacements due to ground motion are concerned, the major design considerations are (a) the stability of the structure, and (b) damage control. Generally, the considerations related to damage control govern, i.e., the damage control criteria are more stringent than those related to stability.

The maximum tolerable deformation, whether expressed in terms of the ratio of the maximum top displacement to the total height or of the maximum interstory displacement to the story height, which can be considered acceptable in order to limit damage to nonstructural components of buildings has not been clearly defined. Obviously, this will depend on the material of which the critical component is made and the mounting or attachment details used.

Figures 12 and 13, or similar ones for other structure heights and earthquake intensities can be used as guides in selecting the appropriate fundamental period, and hence stiffness, once the tolerable maximum displacement has been selected or assumed.





Measures of Deformation:

rotational ductility, $\mu_r = \dfrac{\theta_{max.}}{\theta_y}$

cyclic rotational ductility, $\mu_{rc} = \dfrac{\theta^c_{max.}}{\theta_y}$

cumulative rotational ductility, $\Sigma\mu_{rc} = \dfrac{\Sigma\theta^c_{max}}{\theta_y}$

cumulative rotational energy, $\Sigma A_r = \dfrac{\text{cumulative area under } M-\theta \text{ loops}}{M_y\theta_y}$

Fig. 18 Different Measures of Flexural Deformation in Hinging Region of Walls Considered in Reference 55

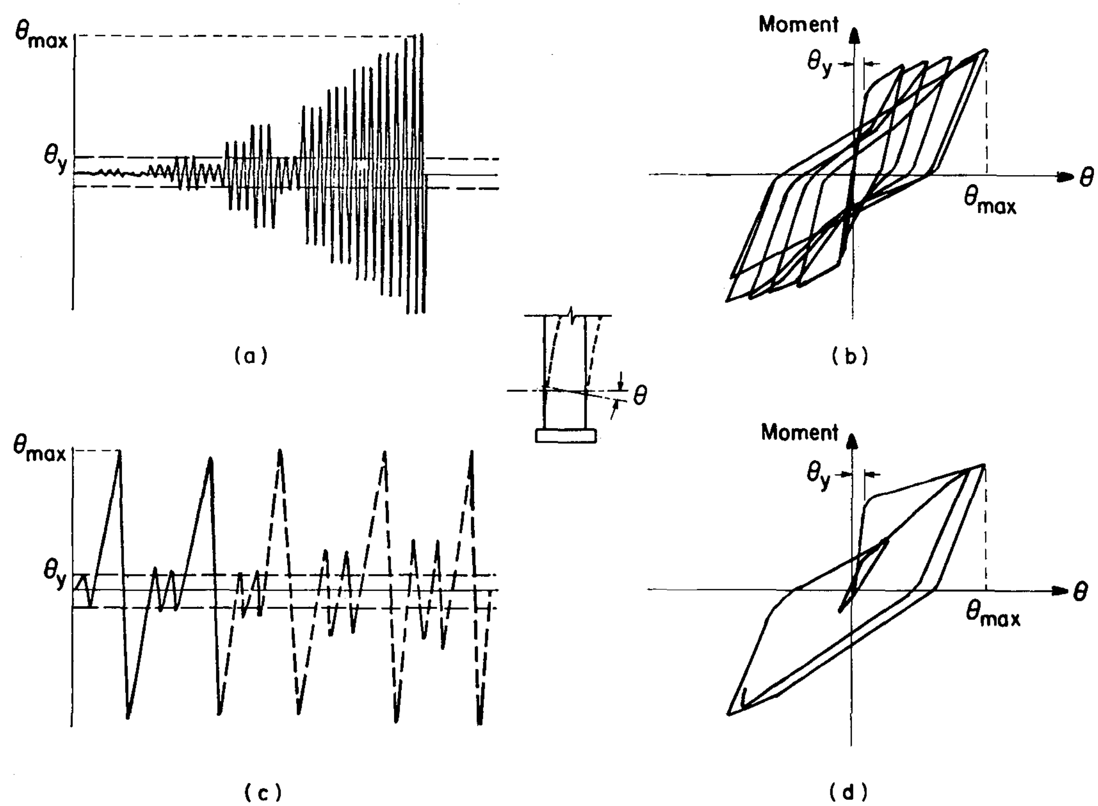


Fig. 19 Sequence of Deformation in Experimental Loading Program
(a) and (b) - Most Commonly Used Program
(c) and (d) - Program Derived on Basis of Study Reported in Ref. 55

(3) Design for Strength and Deformation Capacity: Base of Wall --

(a) Assume an available rotational ductility at the base of the wall, μ_r^a. A trial value may be obtained from a chart such as shown in Fig. 17 (based on experimental data) showing available rotational ductility as a function of the maximum nominal shear stress, by entering the chart with an assumed value of the maximum shear stress.

(b) Determine the minimum yield level required at the base, M_y^{min}, using a chart such as is shown in Fig. 16, giving M_y^{min} as a function of the fundamental period, T_1, and the available ductility, μ_r^a. This value of M_y^{min} can be used to determine the required flexural reinforcement at the base of the wall, if the value of the nominal shear stress assumed in (a) is verified as correct or acceptable.

Also determine the flexural design factor, k, from chart such as is shown in Fig. 15, giving this factor as a function of T_1 and μ_r^a. Then calculate the total horizontal design force, V_T = kW, where W is the total effective weight of the structure.

(c) Determine the shear design factor, $\bar{\alpha}$, from a chart such as Fig. 18, showing this factor as a function of T_1 and μ_r^a. Then calculate the effective static design shear for proportioning the shear reinforcement at the base, $V_{design} = r_v \bar{\alpha} kW = r_v \bar{\alpha} V_T$, where r_v is an appropriate reduction factor intended to account for the over-conservatism inherent in the critical dynamic shears shown in Fig. 16 when compared to the shear capacity obtained from the experimental program.*

(d) Using the experimentally derived chart shown in Fig. 17, or a similar chart, check if the available ductility, μ_r^a, assumed in Step (a) can be developed under the design shear stress determined in Step (c).

If the assumed ductility can be developed, then determine the required shear reinforcement - using design and detailing recommendations developed on the basis of the experimental investigation. If the assumed ductility cannot be developed under the calculated design shear stress, adjust the assumed available ductility value, μ_r^a, and/or modify the wall section dimensions to reduce the shear stress (a recalculation of the period, T_1, may be required in the latter case), and repeat Steps (a) through (d) until a reasonable agreement between assumed and developable ductility is obtained.

The above comparison between assumed and developable values can alternatively be carried out in terms of the shear stress instead of ductility, which can then be assumed as fixed.

(4) Design of Upper Portions of Wall -- Determine flexural and shear reinforcement required in upper portions of wall on the basis of the distribution of V_T (= kW). (The appropriate distribution of V_T is still being studied.)

A check on the ductility requirements in upper portions of the wall, in a manner similar to that used for the base of the wall, may have to be considered.

*The reasons for this over-conservatism are given in Reference 55.

A major distinction between the above-described procedure and current simplified design procedures is the explicit relationship established between the principal structural parameters, i.e., the fundamental period and yield level, and the force and deformation requirements in the critical regions of walls as well as the manner in which these have been correlated with experimental data to yield design forces. A design procedure for frame-wall systems can be developed along similar lines, with appropriate modifications to reflect the effect of other structural parameters characterizing the more complex systems.

Some Questions Concerning Loading

In correlating capacity values obtained from experiments with demands estimated from dynamic inelastic analyses, it is essential that the capacity values be derived under conditions closely approximating those prevailing under dynamic conditions. This is particularly important for those conditions or factors which have significant influence on the behavior of reinforced concrete elements. The validity of any correlation between demand and capacity will depend on how representative the loading conditions used in the laboratory are of actual dynamic response. While there are many aspects to this problem [57], only two factors will be discussed here.

Representative Loading History; Effect of Sequence of Deformation -- For the purpose of obtaining detailed data on specimen behavior for design applications, the most common loading program used in quasi-static tests of large-size specimens under cyclic reversed loads consists of imposing deformation cycles of progressively increasing amplitudes until failure occurs [11-33] (Fig. 19(a) and (b)). The maximum forces and deformations sustained are then noted as indicating capacity. It has been suggested by Bertero [57] that such a loading program may not be as conservative as a program in which the peak deformation is imposed early in the test.

The development of a 'representative' loading history for critical regions in structures which can be used in testing large-size specimens under slowly applied reversing loads is one of the more important results that can be obtained from dynamic inelastic analyses. Such a loading program would have to be defined in terms of the maximum amplitude of deformation, the number of cycles of large amplitude and the sequence in which the large-amplitude cycles occur, with particular reference to the first large-amplitude cycle. The deformation of interest in most cases will be the total rotation that occurs in the hinging region of elements. In addition, the intensity of the accompanying shear and axial load and the variation of these relative to the deformation will have to be noted.

In application, laboratory tests using a loading history developed on the basis of dynamic analyses will have the character of proof tests. A specimen that sustains such a loading program without significant loss of strength can be said to be adequate with respect to design and details for the particular combination or range of values of the significant design variables represented by the loading program.

A study of loading history for isolated structural walls now underway at PCA, for example, indicates that the maximum number of large-amplitude (i.e., 0.75-1.0 of the maximum) cycles of deformation at the critical section near the base rarely exceeds six for a 20-second duration input motion [58]. The input accelerograms used were synthesized by repeating the first ten seconds of strong motion to give a total of twenty seconds. Samples of the composite 20-second accelerograms used in the study are shown in Fig. 20.

Figure 21 shows histograms indicating the number of "fully reversed cycles" and the number of inelastic cycles of rotational deformation calculated at the base of the wall. For the purpose of Fig. 21(a), a "fully reversed" cycle was defined as a cycle with at least one peak value between 0.75 and 1.0 of the calculated maximum amplitude and the other - on reversal - between 0.50 and 1.0 of the maximum. A total of 170 cases are represented in Fig. 21, covering wall heights from 10 to 40 stories, fundamental period values from 0.8 to 3.0 seconds, yield level values ranging from 33,890 to 338,940 kN.m (300,000 to 3,000,000 in-kips) and spectrum intensity values for the input motions from 0.75 to 1.5 times that corresponding to the first 10 seconds of the N-S component of the 1940 El Centro record (Imperial Valley earthquake). A total of 10 different input motions were used, including one artificially generated accelerogram. Further details of the study are reported in Reference 58.

The inelastic cycles plotted in Fig. 21(b) include "large" and "small" amplitude inelastic cycles, a large amplitude being defined as an inelastic rotation between 0.75 and 1.0 of the corresponding maximum. The amplitude of a wave in all cases was measured from the initial (zero) position. Thus, in a rotation-vs.-time plot such as is shown in Fig. 22, a rotation cycle that exceeds the horizontal line representing the initial yield value was considered inelastic.

Of particular interest insofar as sequence of loading is concerned is the fact that in many cases, a deformation equal or close to the maximum occurs quite early in the response, with hardly any inelastic cycle preceding it. This is indicated, for example, in Fig. 22 which shows the history of rotational response of the node at the first floor level (representing the total rotation occurring in the segment between the fixed base and the first floor level) of 20-story walls subjected to the first 10 seconds of the E-W component of the 1940 El Centro record. A plot summarizing the information on this particular aspect of response for the 170 cases studied is shown in Fig. 23. This figure clearly demonstrates that for the particular type of structure considered, it is reasonable to expect a deformation amplitude equal or close to the maximum occurring early in the response, with no inelastic cycle preceding it.

Preliminary results of tests on isolated structural walls conducted at PCA and designed to verify the effect of sequence of loading indicate that a loading program in which the design maximum deformation is imposed early in the test, with only an elastic cycle preceding it, as shown in Fig. 19(c) and (d), can be much more severe than a program consisting of reversed cycles of loading with amplitudes progressively increasing to the maximum (Fig. 19(a)).

Effect of Character of Shear Loading -- In addition to deformation history, it is important, in simulating earthquake response through quasi-static tests, to impose on specimens the forces that analyses indicate may reasonably accompany the maximum deformations. This is particularly important in the case of shear because of its significant influence on behavior.

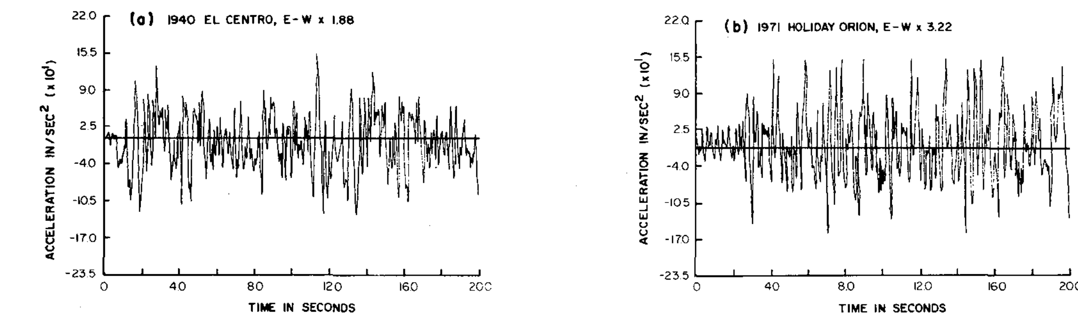


Fig. 20 Composite 20-Second Accelerograms

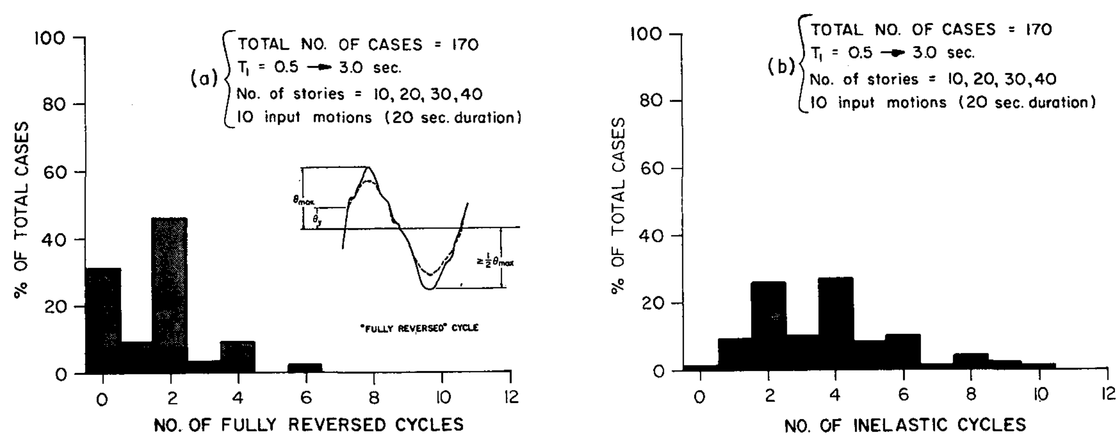


Fig. 21 Histograms Showing Distribution of "No. of Fully Reversed Cycles" and "No. of Inelastic Cycles" - Isolated Structural Walls

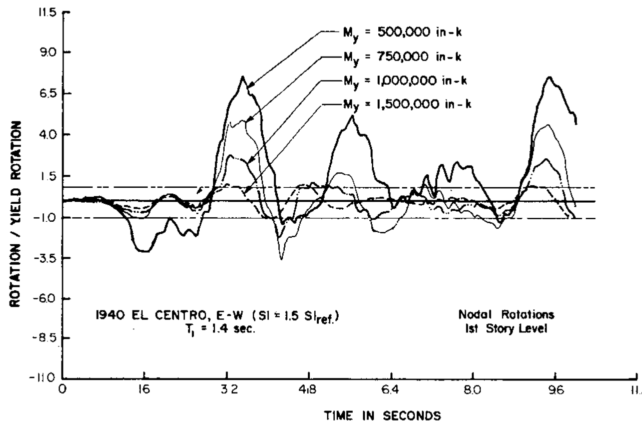


Fig. 22 Variation of Rotation in First Story with Time - 20-Story Isolated Structural Walls

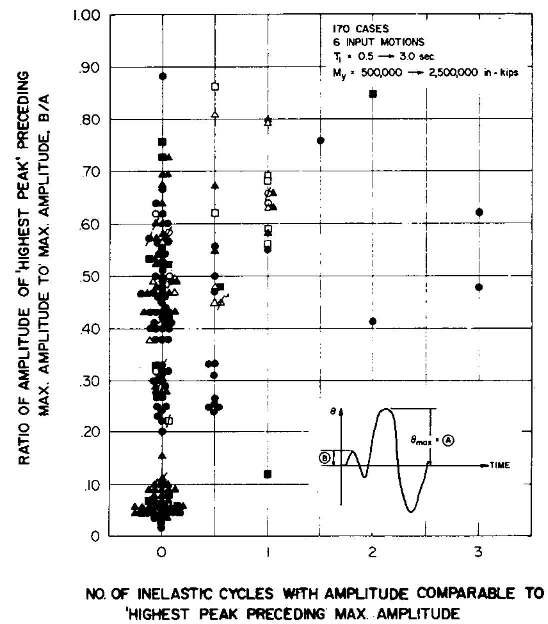


Fig. 23 Number of Inelastic Cycles Preceding Maximum Amplitude Deformation

As far as the shear force used in tests is concerned, two aspects have to be considered. First is the magnitude of the maximum shear force. The second is its variation with time, and particularly in relation to the accompanying moment and deformation. Most of the quasi-static tests that have been conducted to date have been concerned mainly with the magnitude of the expected shear forces. The loading imposed on test specimens [11-33] has been characterized by the moment, shear and the deformation in the critical region being all in phase.

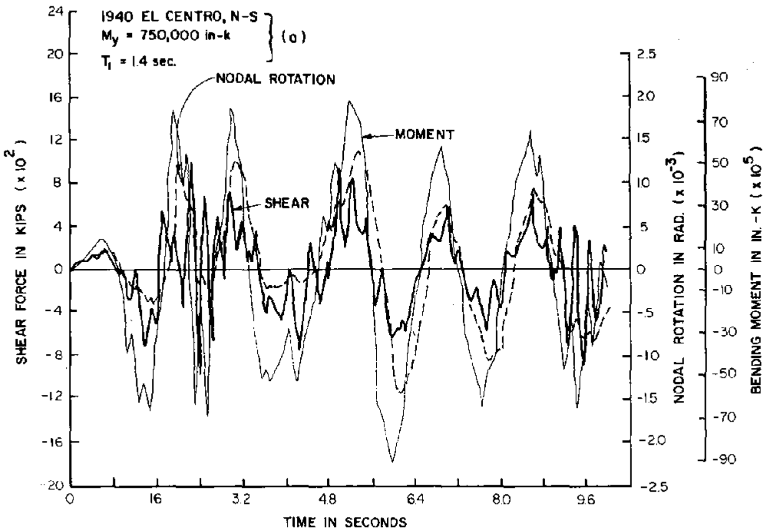
The response studies of isolated structural walls undertaken at PCA[57], however indicate that the shear in the critical region at the base is more sensitive to higher mode response and thus fluctuates more rapidly with time than either the moment or the rotation. This is illustrated in Fig. 24(a) and (b) which show time-history plots of the shear, rotation and moment in the first story of an isolated wall subjected to two different input motions.

The behavior of the shears shown in Fig. 24 may be partly due to the fact that the hinging region in the model used allowed yielding in flexure only, while remaining linearly inelastic with respect to shear throughout the response. Experimental studies [12,14] have shown that this is generally not the case. Whatever the effect of this modelling assumption may be*, it is important in correlating experimental data on capacity with analytical data on demand to allow for possible differences in the manner in which shear is induced under dynamic response conditions and in the typical quasi-static test. It is believed that a shear force that fluctuates rapidly and reaches its peak value only for very short durations relative to the associated moment and rotation does not represent as severe a loading condition as one in which the shear, moment and deformation are all in phase.

SUMMARY

The introduction of reinforced concrete structural walls or coupled walls into frames to form frame-wall structures combines the gravity-load-carrying efficiency of the rigid (open) frame with the lateral-load-resisting efficiency of the structural wall. In planning such structures, a conscious effort can be made to take full advantage of the redundancy in such systems by allowing most of the inelastic action under strong ground motion to take place in elements that are not too critical to the overall stability of the system. By providing sufficient stiffness, strength and deformation capacity in a hierarchy of elements such that a logical sequence of inelastic action occurs under progressively increasing deformations, a reliable energy-dissipative mechanism can be provided while at the same time ensuring the overall integrity of the structure. In this respect, the coupled wall-frame system offers the most effective configuration. The performance of frame-wall structures during recent earthquakes has shown that such a system, when properly conceived and designed, provides an efficient solution to the twin requirements of life safety (i.e., no collapse) and damage control in earthquake-resistant buildings.

*a model which will allow yield in shear, based on uncoupled behavior relative to moment, has been developed to study this and related questions concerning shear yielding.



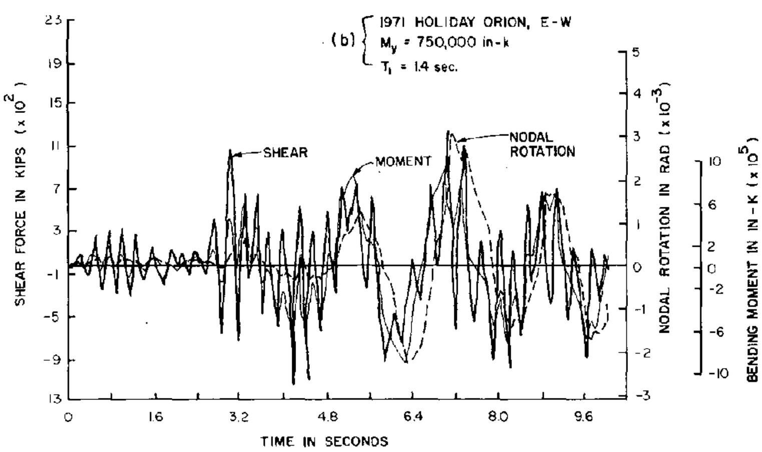


Fig. 24 Variation of Shear and Moment at Base and Rotation in First Story with Time - 20-Story Isolated Structural Walls

The question of sufficiency of design, however, depends primarily on the availability of reliable estimates of demand as well as capacity. At present, there is a lack of information concerning both the force and deformation demands in critical regions of frame-wall structures and the capacity of typical elements (particularly walls) subjected to reversed cycles of loading. A systematic compilation of data on both demand and capacity in frame-wall systems will be required before a practical and reliable design procedure can be developed. Such information should cover a reasonably wide range of values of the major design variables.

A design procedure for earthquake-resistant isolated structural walls has been discussed briefly. It is suggested that a similar approach can be adopted, with appropriate modifications, for the case of frame-wall systems.

Acknowledgment. Work on this report has been supported in part by the National Science Foundation through a Grant (No. GI-43880) for the investigation of "Structural Walls in Earthquake-Resistant Structures."

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

DESIGN OF REINFORCED CONCRETE FRAME-WALL STRUCTURES:

CRITERIA AND PRACTICAL CONSIDERATIONS

by

Eric Elsesser
Structural Engineer
Forell/Elsesser Engineers, Inc.

INTRODUCTION

The use of reinforced concrete frame-wall structural systems for seismic load resistance in building construction has grown in acceptance by structural engineers in recent years. Cast-in-place wall systems have several potential advantages over frame systems: good lateral drift control; high overload capacity; stability in the inelastic load range; and reasonable cost. Current design procedures for shear walls are based on elastic behavior concepts, even though the need for good wall performance in the inelastic range has generally been acknowledged for many years. Engineers must use intuition, knowledge gained through observatons of earthquake damage, and data from laboratory load testing to design walls for the ultimate behavior range. The vast number of variables which determine shear wall performance under seismic loads remains to be systematically studied to provide the engineer with a rational basis for design of walls with predictable behavior in the non-linear, inelastic range.

FRAME-WALL STRUCTURES

Frame-wall structures can be defined as lateral load resisting wall and frame systems which resist lateral loads principally by shear resistance of the walls. The significant characteristic of frame-wall systems is the existence of dual resistance: a vertical and lateral load carrying building frame, and coupled shear walls. Wall-frame systems may act primarily as: (1) independent shear walls; or (2) as infill walls within moment frames; or (3) as walls coupled to moment frames. Figure 1 illustrates the basic forms of frame-walls. The strength of shear wall systems is governed by shear capacity of the wall section, especially in low, long walls; or by flexural capacity in tall, slender walls, which is governed by the reinforcing steel and confinement of the vertical boundary members. Figure 2 illustrates this general behavior.

From the structural engineer's standpoint, the goal of seismic resistant design is to develop a building system which will perform well during earthquakes (i.e., without collapse, without major damage, and be repairable for a functional life after an earthquake). To accomplish this goal, the engineer desires a system with the following characteristics:

1. High energy absorption.
2. Controlled inelastic, post-yield behavior.
3. Good level of ductility.
4. Framework stability.
5. Control damage.

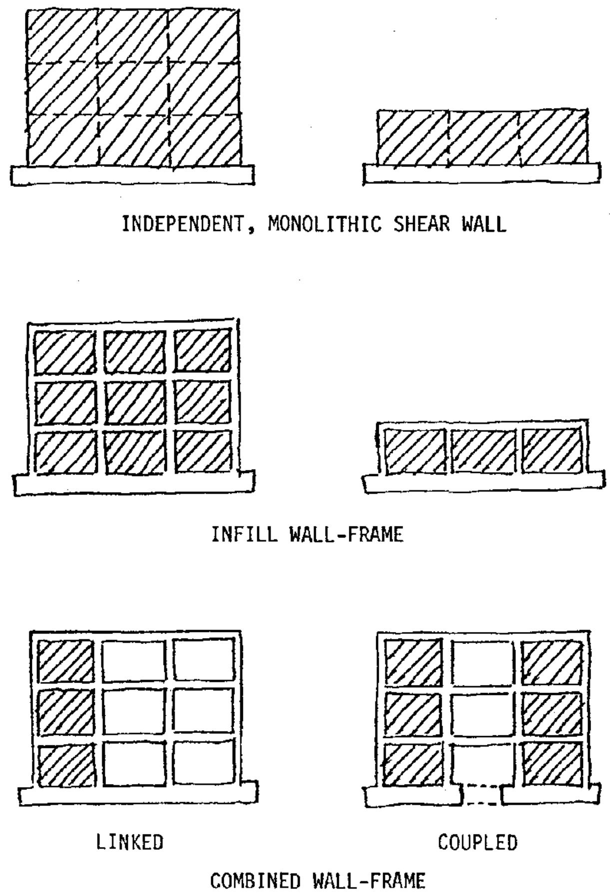


FIGURE 1 - BASIC FORMS OF FRAME-WALL SYSTEMS

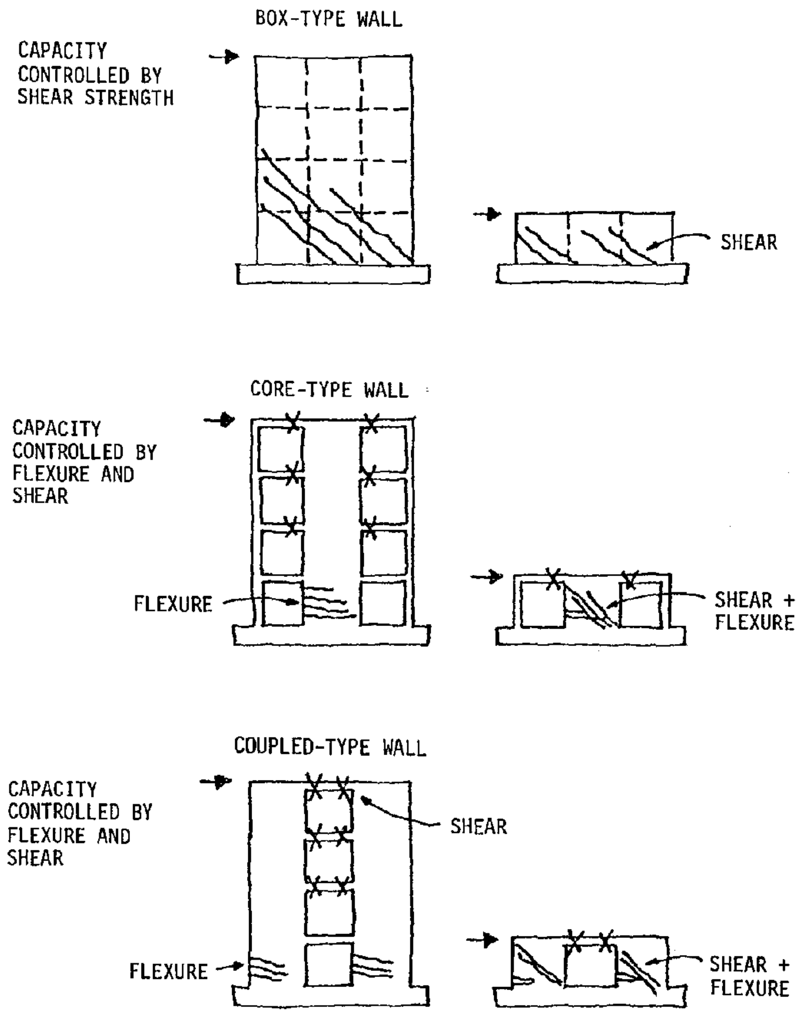


FIGURE 2 - BASIC SHEAR WALL YIELD BEHAVIOR

Frame-wall buildings can potentially satisfy these criteria, provided they are well tied together so that the entire structure (diaphragms, frames, walls, and foundations) can respond as a single system.

The frame-wall system can be designed using judgement to provide: (1) high shear strength; (2) yielding in the flexural mode before shear yielding; (3) stability during inelastic behavior; and (4) continuity of superstructure and sub-structure. Shear wall performance in the elastic range is directly effected by concrete strength, reinforicng steel quantity and strength and by the general wall proportions. Behavior of walls in the inelastic, non-linear range is particularly sensitive to the following additional parameters:

1. Building configuration (i.e., the form of the shear system within the 3-dimensional building envelope).
2. Shear wall size and proportions, boundary frame, and percentage of reinforcing in wall flanges and web.
3. Discontinunities of strength and stiffness in the wall-frame system, such as holes, offsets, and abrupt change in profile.
4. Horizontal diaphragm and collector anchorage to the wall.
5. Stiffness of the frame relative to the wall.
6. Foundation rigidity and ability to transfer loads to adjacent vertical load-carrying elements, and/or to allow rocking of the individual wall.

Figure 3 illustrates several basic shear wall systems with general performance characteristics

SELECTION OF A WALL-FRAME SYSTEM

The selection of a seismic resistant system is usually dependent on the building size, shape and form which normally results from development of the architectural and user program. The feasibility of a shear wall system for a particular building is dependent on the occupancy type and a building concept that will allow significant walls to be properly located, from the structural standpoint. A building which requires large open spaces, flexibility of interior spaces, and substantial perimeter exposure will not generally lend itself to a shear wall lateral load resisting system. A reasonable shear wall solution is appropriate for a building with either a program for fixed interior spaces which can be separated by walls, such as residential, hotel, or hospital occupancies; or appropriate for occupancies which do not demand significant exterior exposures, such as commercial stores, warehouses, auditoriums, and special buildings without the need for windows. Figure 4 indicates several building plans with shear walls advantageously located.

Advantages of a Shear Wall System

The benefits of a shear wall system relative to alternative systems such as ductile moment frame, or braced frames are commonly understood by structural engineers to be:

1. Economy of construction.

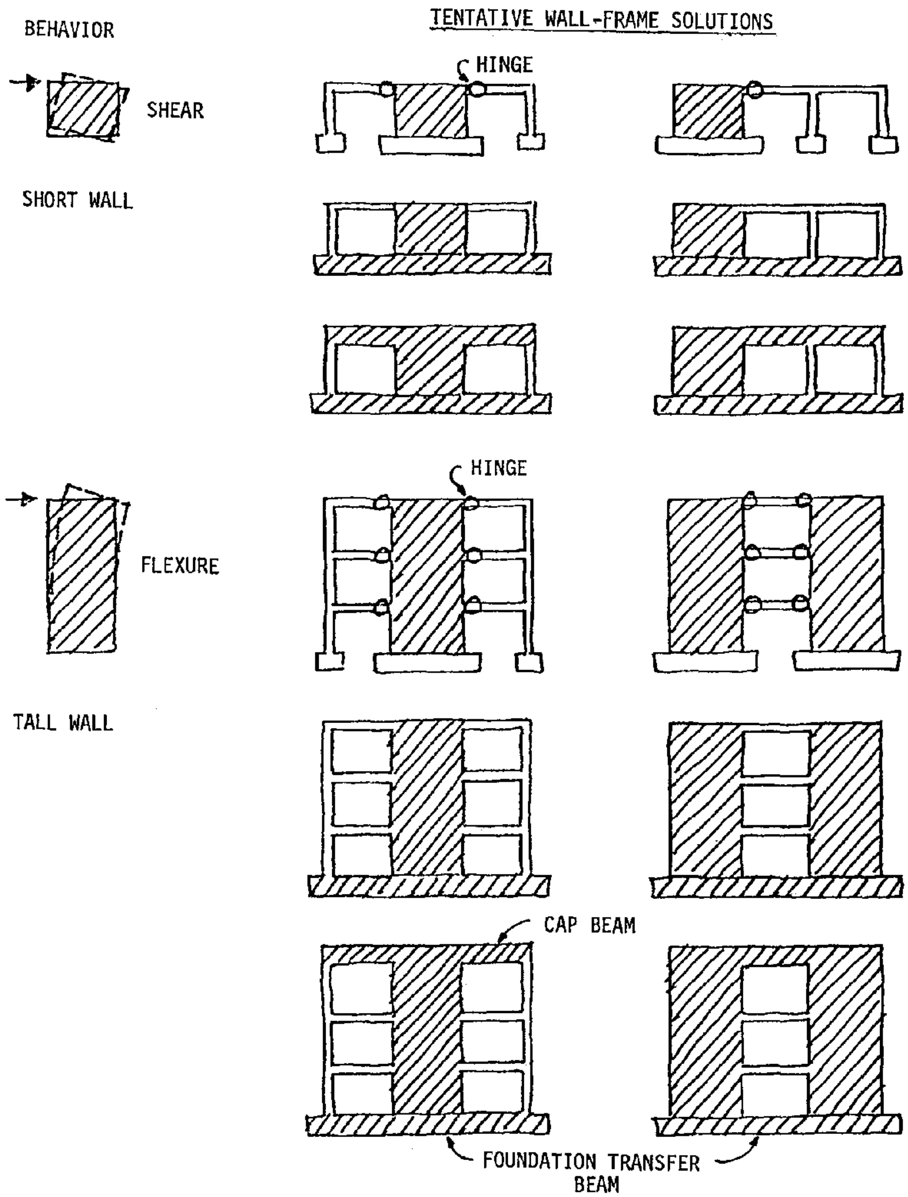


FIGURE 3 - SHEAR WALL BEHAVIOR AND TENTATIVE SOLUTIONS TO PROBLEMS

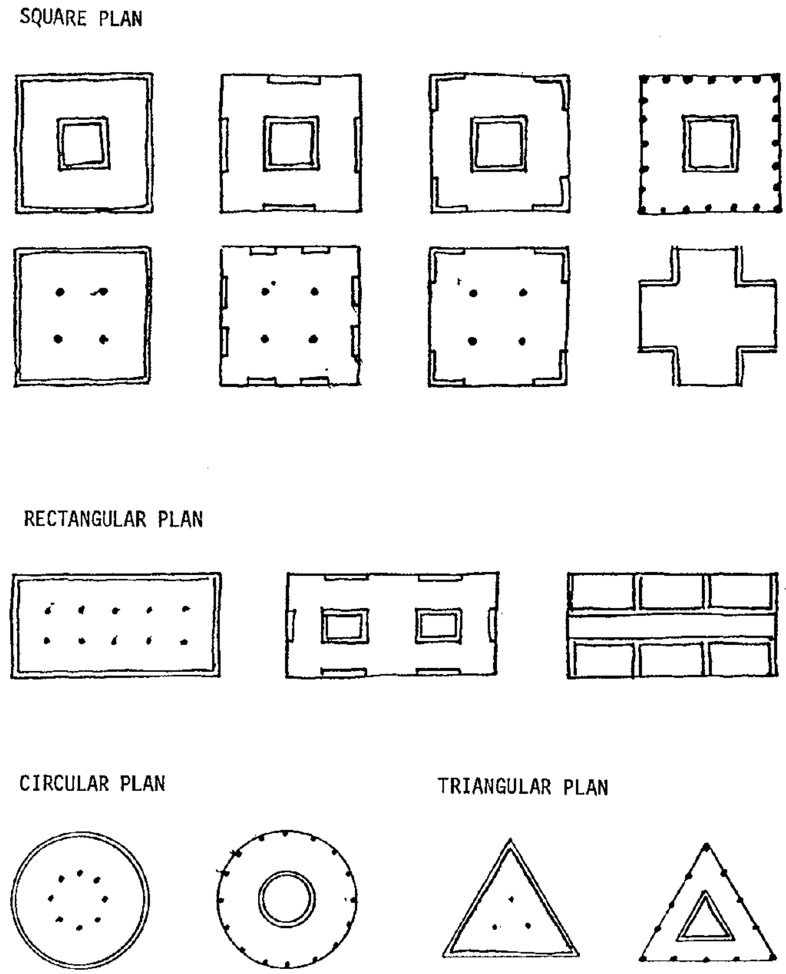


FIGURE 4 - BASIC SHEAR WALL VARIATIONS FOR TYPICAL BUILDING PLANS

2. Use of wall elements for multiple structural and non-structural functions (i.e., weather enclosure, security, thermal protection, load resistance, and separation of occupancies).
3. Ease and speed of construction.
4. Reliability of construction, because of non-critical elements in the construction.
5. Excellent lateral-load drift control of the building.
6. Redundancy of lateral load systems, available with wall systems because wall configurations usually provide excess capacity with corresponding low stresses.

Additional benefits of frame-wall systems not yet acknowledged by all engineers are:

7. Good ductility levels, provided wall shear strength and flexural capacities in the inelastic range are well developed by selection of proper proportions and reinforcement.
8. Good seismic energy absorption, provided shear wall flexural capacity is developed and load resistance does not degrade rapidity.
9. Energy absorption capacity due to rocking of wall systems, available in some wall configurations.
10. Repairability of post-earthquake damage, provided redundant elements are provided to share loads.

Difficulties With Shear Wall Systems

Although shear walls have significant benefits when used properly as part of a structural system, they do possess several disadvantages most of which are directly related to the rigidity of walls relative to other structural elements. The engineer is confronted with the following problems:

1. Determining elastic behavior patterns of shear wall systems, particular when walls are not uniform and regular.
2. Modeling walls to reflect the inelastic behavior expected during earthquakes.
3. Mass-stiffness discontinunities created by irregularities of building configurations, both in plan and elevation, cause problems of balancing shear wall strengths and responses.
4. Shear wall rigidities frequently create excessive restraint for non-seismic loads (i.e., thermal expansion and contraction, and concrete shrinkage), with resultant pre-seismic distress.
5. Modeling and analysis of shear wall foundation rocking during actual earthquake motion.
6. Selection of the appropriate method of analysis and design for the variety of frame-wall configurations, shapes and sizes is one of the most significant engineering problems.

Figures 5, 6, and 7 illustrate several of the more basic wall forms which represent problems and for which design methods must be evolved by the designer, since conventional methods do not take into account all factors affecting the wall performance.

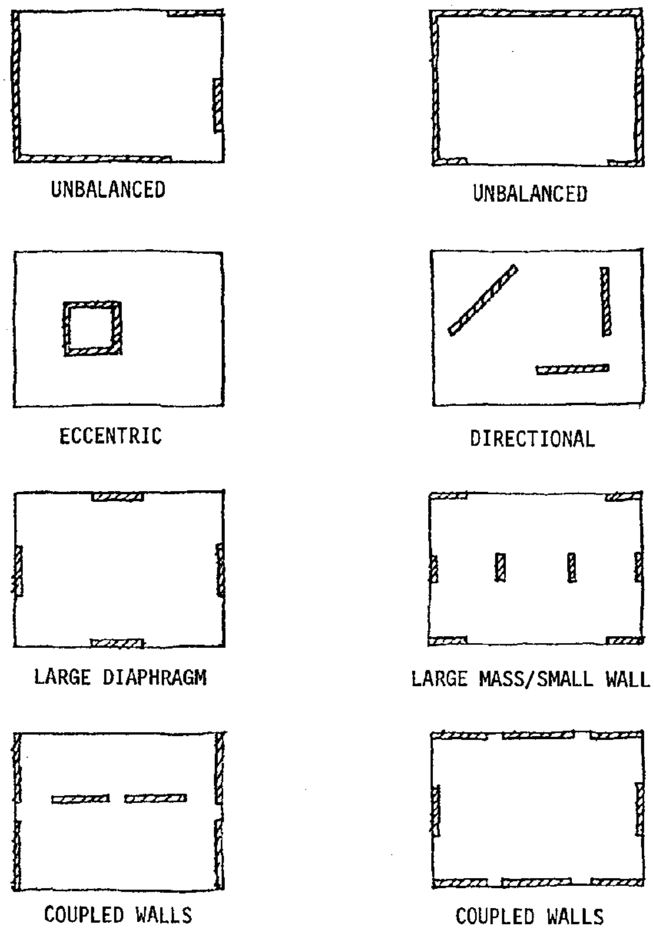


FIGURE 5 - PLAN VIEWS OF TYPICAL BUILDINGS SHOWING
SHEAR WALLS "IRREGULARITIES"

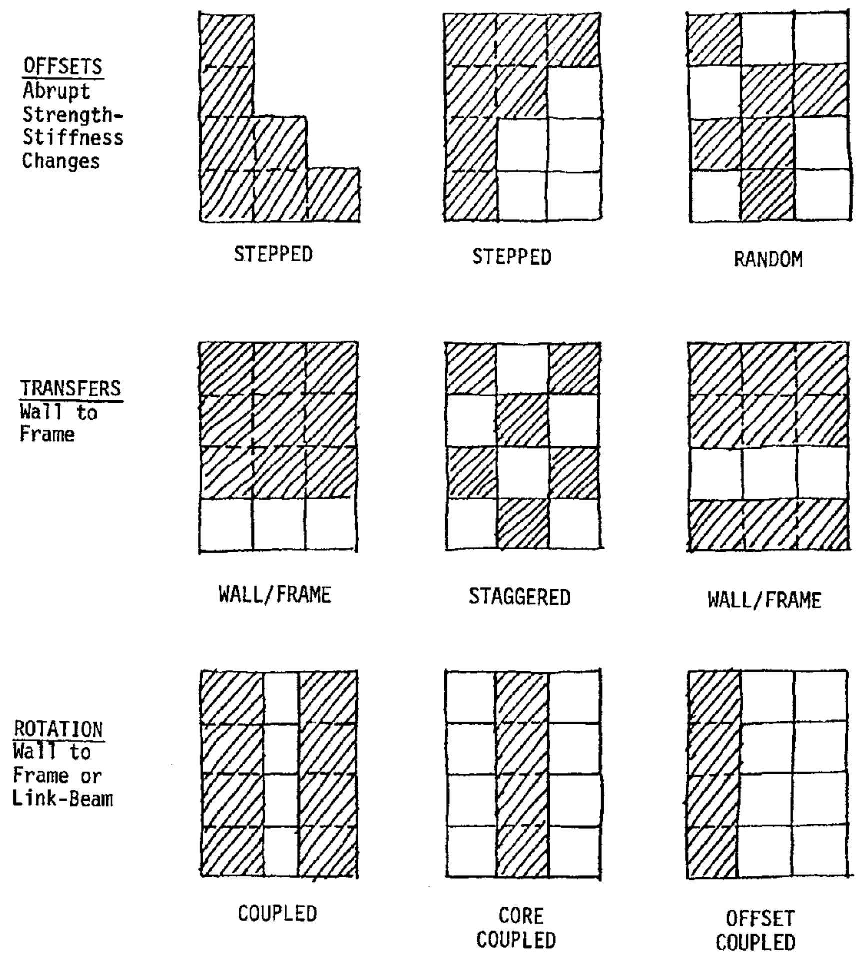


FIGURE 6 - TYPICAL BUILDING ELEVATIONS SHOWING FRAME-WALL POTENTIAL VERTICAL DISCONTINUITIES

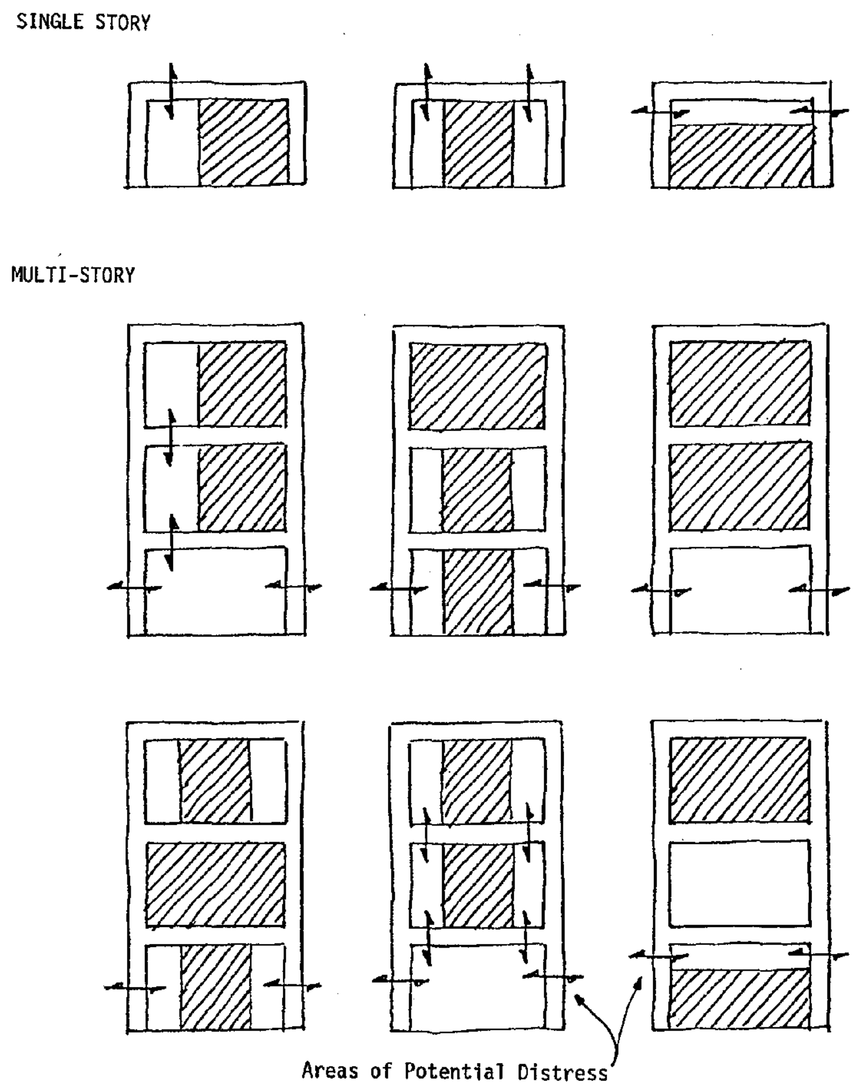


FIGURE 7 - COMMON SHEAR WALL-FRAME CONFIGURATIONS WITH STIFFNESS/STRENGTH PROBLEMS

TABLE 1 - UBC SEISMIC PROVISIONS FOR SHEAR WALL SYSTEMS* - 1949 TO 1976

CODE YEAR	SEISMIC LOADS (ZONES 3 OR 4)	STRESS/STRENGTH AND REINFORCING REQUIREMENTS
1949 to 1955	$F = CW$ $C = \frac{.20}{N + 4\text{-}1/2}$	Shear Stress: $= \frac{V}{bjd}$ Allowable Shear: $c = 0.05\ f'c$ Minimum Reinforcing: $A_s = 0.0025A_c$ (each direction)
1958	Same as 1949	Same as 1949, except: Minimum Reinforcing: $A_s = 0.0025A_c$ (horizontal) $A_s = 0.0015A_c$ (vertical)
1961 to 1964	$V = ZKCW$ $C = \frac{0.05}{T^{1/3}}$ $K = 1.33$ (wall) $K = 0.8$ (dual)	Same as 1958
1967 to 1970	Same as 1961	Shear Stress: $= \frac{Vu}{\phi hd}$ $\phi = 0.85$ $U = 2[1.4(D+L+E)]$ $U = 2[0.9+1.25E]$ Minimun Reinforcing: $A = \frac{Vu's}{\phi fyd\left(\frac{H}{D} - 1\right)}$ $A = \frac{Vu's}{\phi fyd}$ $A_s = 0.0025A_c$ (horizontal) $A_s = 0.0015A_c$ (vertical) Maximum Shear Strength: Shear Resisted by Concrete: $c = (3.7 - \frac{H}{D})\ 2\phi \sqrt{f'c}$ For $H/D < 1$, max. $Vc = 5.4\phi \sqrt{f'c}$ For $H/D > 2.7$, min. $Vc = 2\phi \sqrt{f'c}$ Shear Resisted by Concrete & Reinforcing: $u = \left(0.8+4.6 \frac{H}{D}\right)\phi \sqrt{f'c}$ For $H/D < 1$, min. $Vc = 5.4 \sqrt{f'c}$ For $H/D > 2$, max. $Vc = 10 \sqrt{f'c}$
1973	Same as 1967	Shear Stress: $= \frac{Vu}{\phi hd}$, $d = 0.8\ L_w$ Ultimate Strength Factors: $\phi = 0.85$ $U = 1.4(D+L)+2.8E$ $U = 0.9D+2.8E$ Minimum Reinforcing: $A_s = 0.0025A_c$ (each direction) $P_h = 0.0025+0.5\ (2.5-\frac{hw}{Lw})(P_h-0.0025)$ Ductile boundary members for shear walls with K=0.8 Maximum Shear Strength: Lesser of $Vc = 3.3\sqrt{f'c} + \frac{Nu}{4L_wh}$ $Vc = 0.6\sqrt{f'c} + \frac{Lw\left(1.25\sqrt{f'c} + 0.2\left(\frac{Nu}{Lwh}\right)\right)}{\frac{Mu}{Vu} - \frac{Lw}{2}}$ Maximum on any Section: $v = 10\sqrt{f'c}$
1976	$V = ZIKDSW$ Same as 1961, except: $C = \frac{1}{15\sqrt{T}}$ $S = 1.0$ to 1.5 $I = 1.0$ to 1.5	Same as 1973, except: Ultimate Strength Factors: $U = 1.4\ (D+L)+2.0E$ $U = 0.9D+2.0E$

*Refer to Respective Editions of UBC for Notations.

DESIGN CRITERIA

The practicing structural engineer has at his disposal three sources for design assistance. The first source is building codes and recommendations such as The Uniform Building Code [1], ACI-318,[2] and the Recommended Lateral Force Requirements of SEAOC [3]. The second source is published data from full-size or scaled tests of shear wall assemblies. The third source is actual observation or published observation of behavior of structures subjected to real earthquakes. Building codes generally provide design criteria assuming elastic performance, while cyclic and shaking-table tests and earthquake observations can provide the basis for an understanding of the actual behavior during an earthquake.

Buidling Codes

Our codes provide a minimum value for design within the elastic level of performance; code values form the basis for most design efforts by engineers. These minimum values do not necessarily assure adequate performance under real seismic loads; however, recent provisions for confined boundary members (UBC 1967 to 1976) acknowledge the need for ductility and good post-cracking performance. Table 1 summarizes the provisions of the UBC from 1949 to 1976, which apply to concrete shear walls. Several items are worthy of special emphasis:

1. The allowable unit shears have almost doubled from 1949 (with 0.05 f'c allowable) to 1976 with 10 f'c (ultimate strength, and U factor of 2).
2. The minimum reinforcement for walls has remained basically unchanged.
3. A recent attempt has been made to acknowledge that wall behavior is dependent on wall proportions (H/D radios), flexural effects, and axial loads.
4. Recent revisions to code formulas have served to clarify a range of elastic behavior, but have not addressed the area of non-linear, inelastic behavior, consequently, with the exception of confinement requirements, no formal provisions exist for real seismic performance.
5. The basic code provisions cover uniform, regular walls without a caution to the designer as to the limitations of the code for special wall configurations.

Load-Tests

A variety of load tests on representative shear wall assemblies have been undertaken over the past 25 years. The first significant tests were monotonic tests of shear wall panels, both with and without openings carried out at Stanford University in the 1950's [4] [5]. The most recent tests have been at the University of California, Berkeley [6], the PCA Laboratories in Illinois [7] [8] [9] and in New Zealand [10] [11] and Yugoslavia [12], all using cyclic loads. Tests of significance to the designer have also been conducted in Japan [13] [14] [15], in Canada [16], the University of Illinois [17] and in Portugal [18]. Single wall tests are summarized in Table 2A, coupled walls in Table 2B, and multistory wall tests in Table 2C.

FIGURE 2A - SUMMARY OF SEVERAL LOAD TESTS OF SINGLE PANEL SHEAR WALL

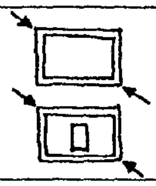
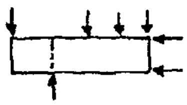
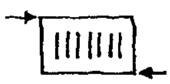
WALL TYPE	REFERENCE	CONFIGURATION	LOADING TYPE
SINGLE PANELS SOLID	[4] (STANFORD)		MONOTONIC
SINGLE PANELS WITH OPENINGS	[5] (STANFORD)		MONOTONIC
SINGLE PANELS SOLID AND WITH OPENINGS	[14] (JAPAN)		MONOTONIC
SINGLE PANELS WITH OPENINGS	[15] (JAPAN)		CYCLIC
SINGLE CANTILEVER WALL SOLID	[9] (PCA)		MONOTONIC
SINGLE PANEL SLITTED	[13] (JAPAN)		CYCLIC

FIGURE 2B - SUMMARY OF SEVERAL LOAD TESTS OF COUPLED SHEAR WALLS

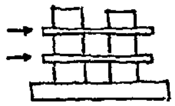
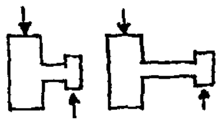
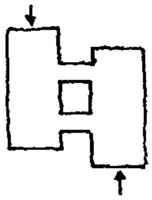
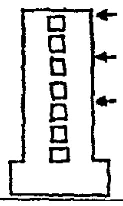
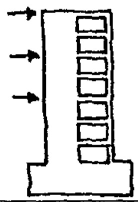
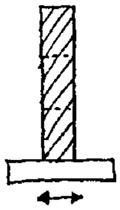
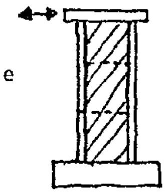
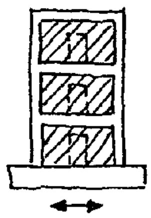
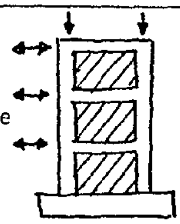
WALL TYPE	REFERENCE	CONFIGURATION	LOADING TYPE
COUPLED WALLS SLAB LINK	[16] (CANADA)		MONOTONIC
COUPLED WALLS BEAM LINK	[12] (YUGOSLAVIA)		CYCLIC
COUPLED WALLS BEAM LINK	[8] (PCA)		REVERSING
COUPLED WALLS BEAM LINK	[10] (NEW ZEALAND)		CYCLIC
LINKED WALL-FRAME	[11] (NEW ZEALAND)		CYCLIC

FIGURE 2C - SUMMARY OF SEVERAL LOAD TESTS OF MULTI-STORY SHEAR WALLS

WALL TYPE	REFERENCE	CONFIGURATION	LOADING TYPE
MONOLITHIC 3-STORY WALL	[17] (ILLINOIS)	$\frac{1}{6.25}$ Scale 	SHAKING TABLE
MONOLITHIC 3-STORY WALL	[7] (PCA)	$\frac{1}{3}$ Scale 	CYCLIC
3-STORY FRAME WITH INFILL WALL	[18] (PORTUGAL)	$\frac{1}{4}$ Scale 	SHAKING TABLE
MONOLITHIC 3-STORY FRAME-WALL	[6] (UBC)	$\frac{1}{3}$ Scale 	CYCLIC

The significance of tests for the design engineer confronted with aseismic design is the availability of real inelastic range performance data. Parametric test studies enable the engineer to develop an understanding of response to overload conditions and to develop his intuition for the real behavior problems which generally are not codified. Tests results from shaking tables and cyclic load applications are the most significant.

Because of the testing expense of recreating real conditions experienced during earthquakes, shear walls have generally been tested with "fixed" base conditions, a modeling condition which is not found in real buildings, except perhaps for large monolithic multi-story wall systems. Moreover, most tests again, for reasons of economy, are performed on test models which do not represent the restraint of floor framing, large vertical loads, or of coupled beams. Tests are also generally performed on a single shear wall element abstracted from the complete 3-dimensional structural system, which distorts the results especially for core-wall or tube-wall configurations. Nevertheless, test data is extremely important for insight into behavior, and is the only reasonable source the design engineer has for understanding of complex building forms.

Earthquake Damage Observation

The performance of structures during real earthquake motion is the ultimate test and the most significant laboratory for study of the numerous concepts proposed by the structural engineering profession. The single most important problem with observations of damage is the variable nature of real structures and the lack of a datum or reference by which to compare data collected from several earthquakes for a single structural type, or data collected from several buildings in a single earthquake. Nevertheless, accurate reporting of earthquake performance during the last 30 years has enabled substantial progress to be made in the evaluation of structures.

Significant shear wall damage was recorded in the following earthquakes (as well as others not listed: Kern County, Calif. (1952); Mexico DF (1957); San Francisco, Calif. (1957); Agidir, Morroco (1960); Skopje, Yogoslavia (1963); Alaska (1964); Venezuela (1967); Tokachi-Oki, Japan (1968); Peru (1970); San Fernando, Calif. (1971); Nicaragua (1972); and Italy (1976).

Failure has generally been observed in the following elements of reinforced concrete shear walls:

1. Compression failure of wall vertical boundary members.
2. Shear failure of walls.
3. Shear failure or sliding of wall horizontal construction joints.
4. Shear failure of spandrel and pier elements in shear walls with openings.
5. Shear failure of link-beams in coupled shear walls.

With few exceptions no collapse has been observed in shear wall buildings, provided the continuity of the shear wall system was not interrupted, or discontinuities created. Most shear wall systems have also been capable of post-earthquake repair.

Only study of past earthquake behavior of frame-shear wall systems can enlighten the design engineer as to the failure mechanisms of the next frame-wall system he has selected; and only diligent study can inform the engineer as to how and where to provide a satisfactory amount of reinforcing steel, and how to proportion the structure. The Building Code can not, and should not, cover the information that can be gained from informed observation.

METHODS OF ANALYSIS AND DESIGN

Conventional methods of analysis of structural shear wall systems envision elastic behavior and this is the predominant assumption made when analyzing and designing structures to resist earthquakes, even though it is known that the structure must perform in the inelastic range to satisfy energy demands imposed by seismic motions. Consequently, any discussion of analysis and design methods begins with a contradiction between practice and reality.

The methods of analysis and design become interdependent in the actual process of developing a structure because the assumptions of the analysis method must be compatible with the design and in turn with the actual structural performance. Any procedure utilizing static loads (or equivalent static loads), allowable stresses or strengths, and elastic behavior is compatible. Numerous methods have been developed to analyze frame-wall systems from the simpliest portal method and manual calcualtion of wall element rigidities to the more complex finite-element concepts [19], all of which are reasonable for elastic level loads. The use of loads generated by a dynamic analysis, using response spectrum techniques and modal superposition is a significant breakthrough in the description of realistic seismic loads, but the loads generated are only valid with compatible inelastic behavior modeling of the structure.

Practical analysis methods for non-linear behavior of buildings do not yet exist for the engineering design office, nor do predictable models for shear wall yield mechanisms; and only limited data and methods are available for designing and detailing for the post yield range. The resulting design process which uses sophisticated inelastic range loads and precise elastic analysis methods produces an obvious conflict which can only be overcome by a resort to load test data for appropriate factoring of the input, but this is an approximate and temporary solution at best, for the engineer.

The conventional design and analysis procedure for a shear wall system can be summarized as follows:

1. Develop structural system concept.
2. Develop tentative shear wall locations and sizes.
3. Analyze overall system for lateral load distributions, based on rigidity of resisting elements, based on elastic behavior.
4. Check components for shear stresses, combined stresses, diaphragm and collector stresses, wall-flange requirements and stresses, stresses at openings, stability due to overturning effects, lateral drift, rotation, and deflection compatibility, and capacity of the foundation system to develop loads assigned to the shear walls.

5. Detail the frame-wall components to be consistant with the assumptions made in the conceptional analysis and design phases; and in conformance with the Building Code requirements.

Additional special procedures are available for refined analysis or for special or unusual features:

1. Consider the 3-dimensional aspects of the structural system in both elastic and inelastic ranges, so to develop the ultimate load carrying potential for all structural elements.
2. Analyze the system for load distribution based on dynamic-modal effects.
3. Check components for non-linear behavior using finite-element methods, and prediction of yield mechanisms.
4. Detail components for non-linear behavior (i.e., ductility, degrading stiffness and strengths.

Even these special procedures do not yet provide the engineer with the capability of predicting seismic behavior of frame-walls with the reliability of predicting elastic behavior. The problem of how to develop a model to represent the non-linear behavior of a variety of shear wall-frame systems remains.
Figure 8 shows several of the variables (shapes, size openings, coupling, chord size, and base condition) which should specifically be considered in design provisions.

CONSTRUCTION DETAILS

Conventional details for concrete frame-wall systems are generally devised to satisfy elastic-level loads (normally prescribed by building codes) and with no significant ductility, or increased capacity for behavior in the inelastic range. The actual performance level of conventionally detailed frame-wall systems is unpredictable and dependent on many variables.

Shear wall systems with special details to provide ductility have been devised in recent years to provide satisfactory behavior in the non-linear load range (substantially above current building code loads). Ductility is provided in boundary or flange elements, with corresponding capacity for significant energy absorption. To date only boundary members are detailed to provide confined concrete for increased load capacity. The actual performance level of specially detailed frame-wall systems is as unpredictable for the design engineer, as it is with conventionally detailed walls, and is also dependent upon many variables.

Individual Walls

Figures 9, 10 show a comparison of typical construction details for both single-story and multiple-story shear wall systems, respectively. All confined boundary and edge members are extended for the full height or length of the panel to avoid abrupt transitions. Laps of flange or boundary longitudinal reforcing steel is critical and staggered, long laps should also be considered.

TYPICAL ELEVATIONS OF WALL PANELS

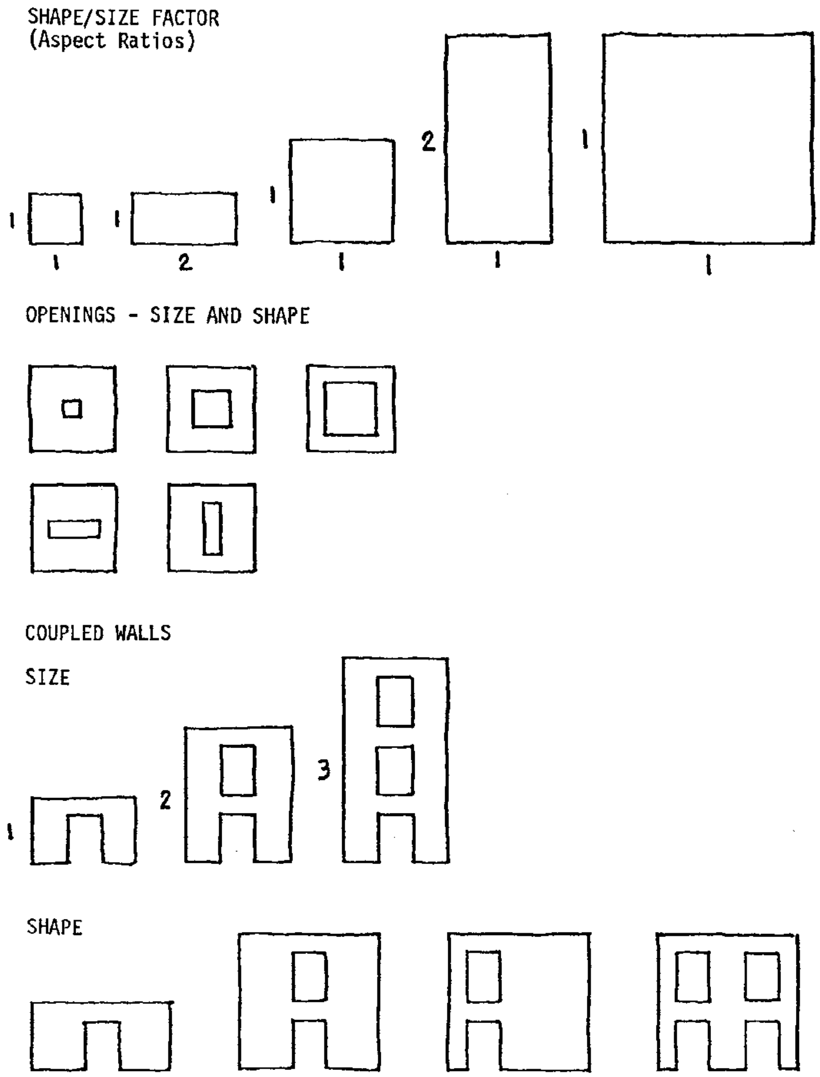


FIGURE 8 - SEVERAL FACTORS TO CONSIDER WHEN SELECTING A DESIGN METHOD

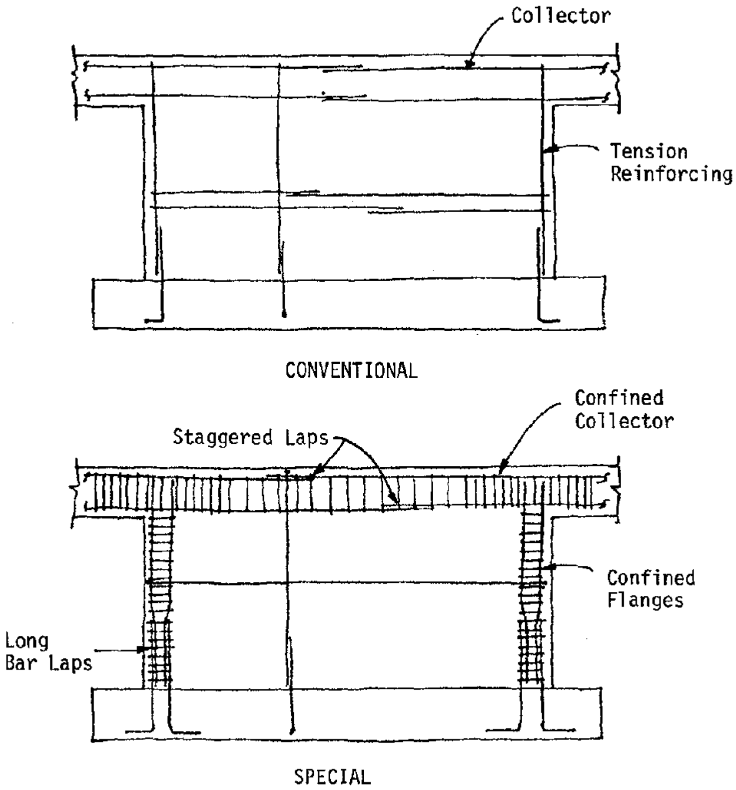


FIGURE 9 - COMPARISON OF CONVENTIONAL AND SPECIAL REINFORCING FOR 1-STORY SHEAR WALL

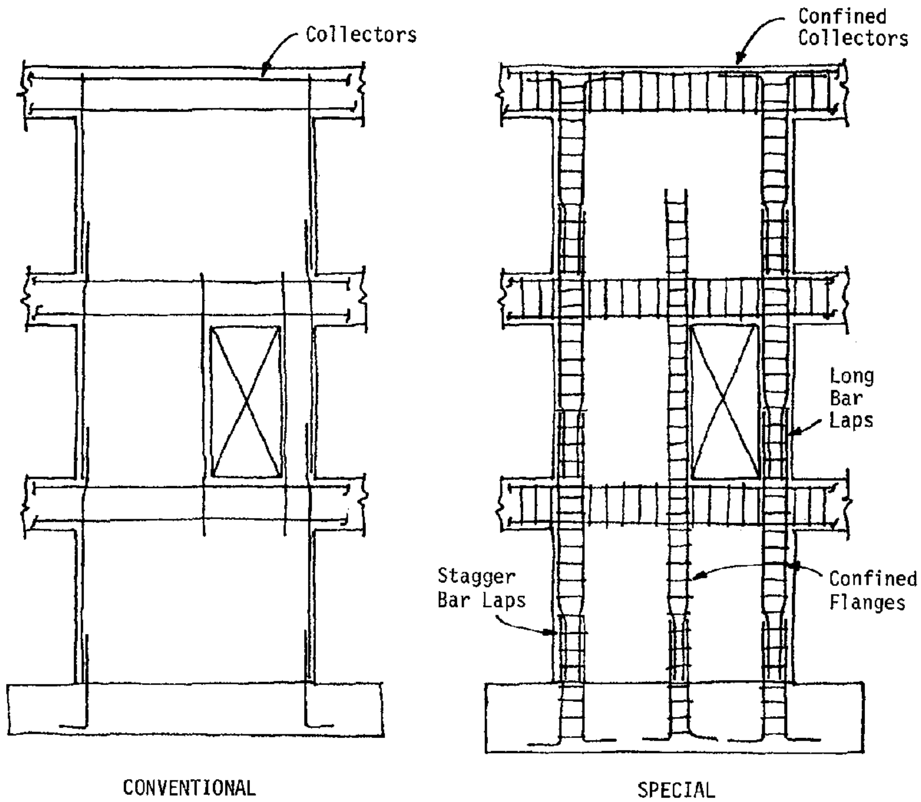


FIGURE 10 - COMPARISON OF CONVENTIONAL AND SPECIAL REINFORCING
FOR MULTI-STORY SHEAR WALL

Figurell provides a comparison of conventional and special details at the junction of the horizontal diaphragm to wall. Figure 12 illustrates several plan views of vertical wall boundary elements (ends, corners and tee-intersections). In all transition details which are attempted within the confines of the wall or slab profile, that is without an enlarged boundary member, significant reinforcing conjestion results when confining transverse ties are introduced. Generally both steel and concrete placement difficulties are experienced when wall boundary members are not increased in size to accommodate the increased reinforcing steel required by special or ductile detailing. Anchorage of horizontal wall bars within the confined vertical boundary members is essential; however, placement of bars with hooks at ends is difficult within the confinement ties.

Coupled Walls

The system of coupling two rigid walls together with relatively weak or flexible link-beams presents significant problems to the designer. The incompatibility of wall and link-beam rigidities and rotations under lateral loads cannot easily be overcome with special reinforcing details. The link will generally crack severely under earthquake motions. The only practical solution is creation of a structural element which will yield and yet not be completely destroyed during seismic loading, so that repair or rebuilding is feasible. Figure 13 shows several walls and details at the typical link beam. Extension of beam steel and confinement well into the wall is an attempt at maintaining continuity. When total coupled wall system deformations are considered, the designer cannot accurately predict the ultimate load behavior of the line beams, or of the walls. There are numerous variables affecting the behavior of coupled walls, especially the proportions of the link beams to the walls themselves, and the quantity and placement of reinforcing steel, especially the effect of layering of wall and beam steel.

Foundations

The development of the ultimate load capacity of shear wall system or wall-frame system is frequently dependent on an adequate foundation to restrain the bases of frame columns and shear walls. Both conventional and special details for typical walls are shown in Figure 14. The large transfer element, at either the base of the wall or at its top, or at both locations, is a useful system for reducing shear wall rotations and transferring the resulting shears and moments to other resisting elements.

FUTURE DIRECTIONS

Shear wall-frame systems can be used with confidence when the design engineer possesses complete understanding and data on the performance of the system. Moreover, with an increased understanding of the systems currently in use, we can expand our thinking to develop other, perhaps more favorable wall-frame configurations.

The ability to accurately predict behavior in the inelastic range is essential to creative structural engineering. Tests are needed for many types of systems; these tests must have cyclic loading or shaking table loading. After understanding behavior then methods for design can be developed. The future of frame-walls is potentially a most interesting and challenging one.

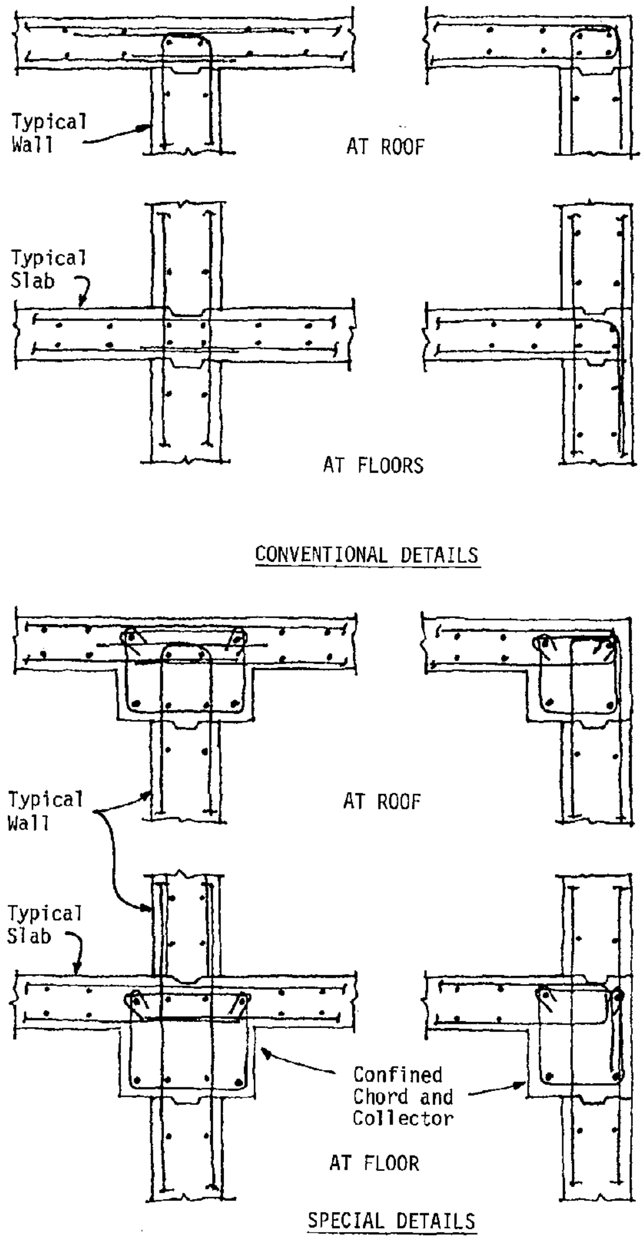


FIGURE 11 - COMPARISON OF CONVENTIONAL AND SPECIAL DIAPHRAGM TO SHEAR WALL DETAILS

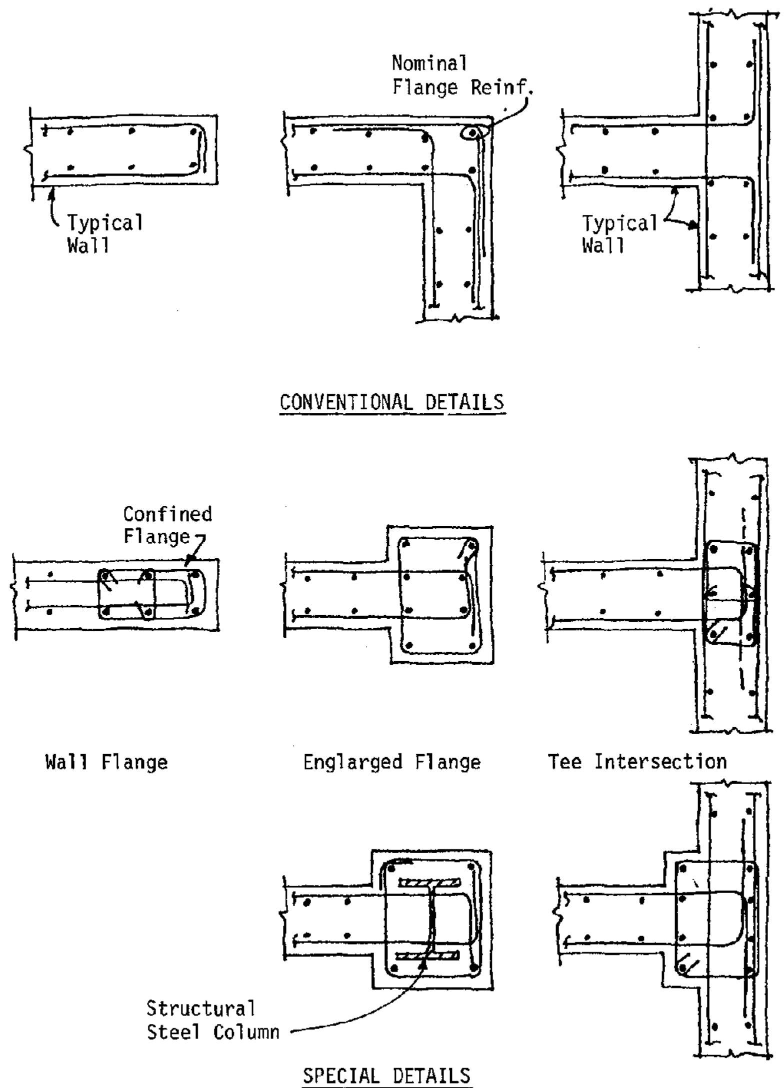


FIGURE 12 - COMPARISON OF CONVENTIONAL AND SPECIAL PLAN DETAILS OF SHEAR WALL FLANGES

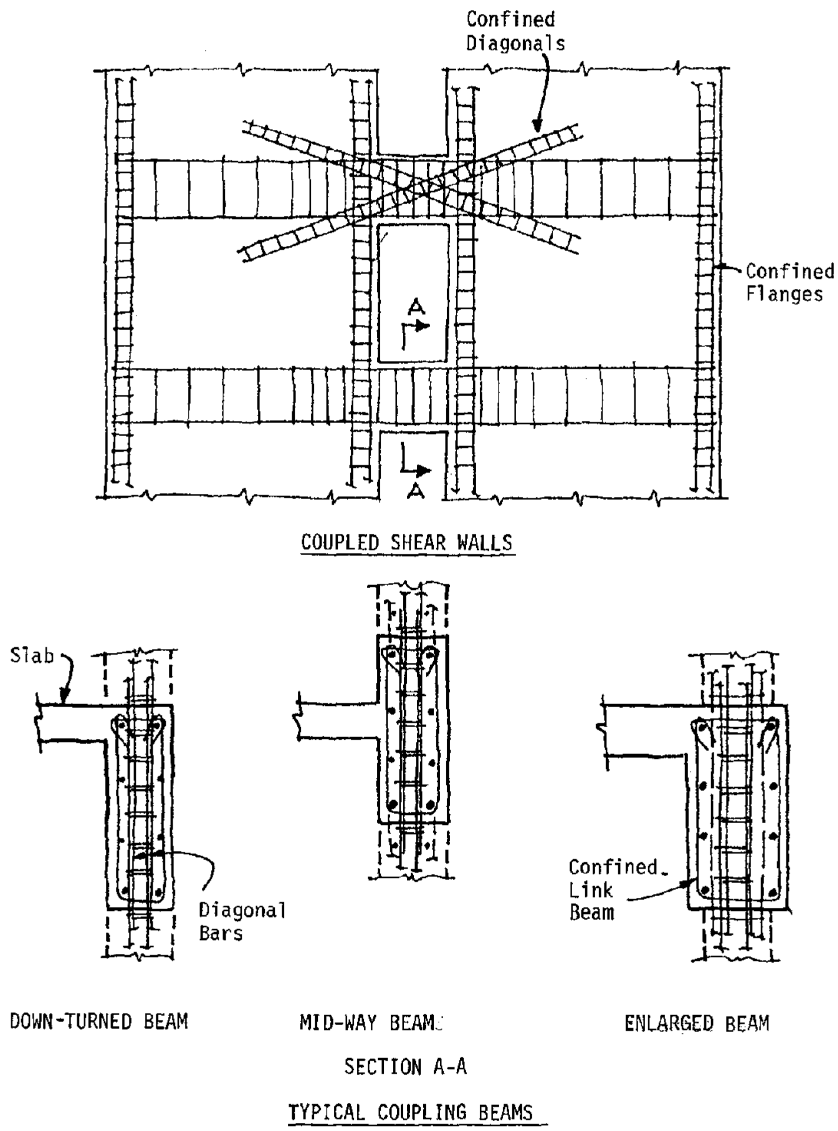
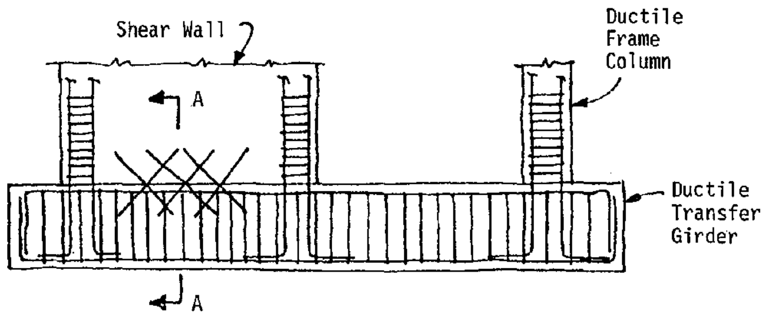
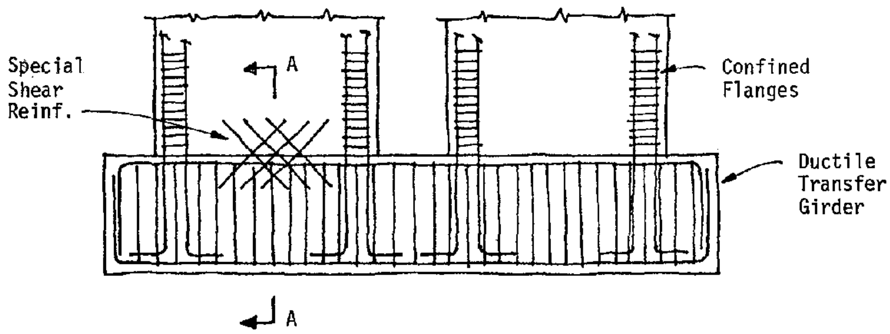


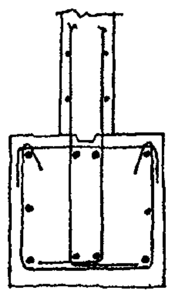
FIGURE 13 - COUPLED WALL LINK BEAM DETAILS



FRAME-WALL



COUPLED SHEAR WALLS



SECTION AA

FIGURE 14 - SPECIAL SHEAR WALL FOUNDATIONS

Recommended Studies

Some of the many areas requiring systematic investigation are listed below; they have been separated into three basic categories: configuration studies; system behavior studies; and basic development of data, or parametric studies.

Configuration Studies:

1. 3-dimensional box or tube frame-wall systems.
2. 2-dimensional, planar frame-wall systems.
3. Establish limits of irregular systems.
4. Development of non-abrupt strength/stiffness systems. (Non-rectangular, tapered sections.)
5. Development of post-earthquake repairable assemblies.

Behavior Studies:

1. Tabulation of basic wall configurations, with corresponding definition of elastic and inelastic behavior patterns.
2. Tabulation of ultimate stresses, ductilities, damping, and energy absorption for basic frame-wall systems.
3. Foundation restraint requirements for basic wall systems. (Flexible, semi-fixed or fixed.)
4. Development of progressive resistance systems. (Structural tuning.)

Parametric Studies (for Frame-Wall Performance):

1. Size of wall panel (absolute size).
2. Shape of wall panel (height to length ratio).
3. Size of wall panel openings (relative proportions).
4. Location of wall panel openings.
5. Effect of axial load.
6. Size, shape and proportion of wall flanges.
7. Flange reinforcement (quantity configurations, laps).
8. Wall reinforcement (patterns and quantities).
9. Coupled wall link beams (rotation capacities, sizes, assemblies, limitations).
10. Coupled wall or frame-wall transfer beams (at top and/or foundation).
11. Frame-wall anchorage to foundations.

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EARTHQUAKE RESISTANT STRUCTURAL WALLS

by

Thomas Paulay
Professor of Civil Engineering
University of Canterbury
Christchurch, New Zealand

1. INTRODUCTION

The usefulness of structural walls in the planning of multistorey buildings has long been recognized. When walls are situated in advantageous positions in a building, they can become very efficient in lateral load resistance, while also fulfilling other functional requirements.

Because a large portion of the lateral load on a building, if not the whole amount, and the horizontal shear force resulting from it, are often assigned to such structural elements, they have been called shear walls. The name is unfortunate because shear should not be the critical parameter of behaviour.

The basic criteria that the designer will aim to satisfy when using structural walls in earthquake resistant structures are as follows:

(a) To provide adequate stiffness so that during moderate seismic disturbances complete protection against damage, particularly in non-structural components, is assured.

(b) To provide adequate strength to ensure that the seismic excitation envisaged by building codes [1,2] does not result in permanent structural damage. Even though during such an event some non-structural damage is expected, it is unlikely that in buildings with well designed shear walls this will be serious.

(c) To provide adequate structural ductility and capability to dissipate energy for the case when the largest disturbance to be expected in the region does occur. Extensive damage, perhaps beyond the possibility of repair, is accepted under these extreme conditions, but collapse must be prevented.

The subsequent sections concentrate on those aspects of the design and response of structural walls that are relevant to this third design criterion. Consequently the inelastic response of structural walls , when subjected to simulated cyclic reversed loading, together with various parameters that must affect this response, will be examined in some detail for various types of structures. It will be assumed that in all cases adequate foundations can be provided so that uplift or rocking will not occur and that energy dissipation, when required, will take place in the structural wall above foundation level. Also it will be assumed that inertia forces at each floor can be introduced to the structural wall by adequate connections, such as collector beams, from the floor system.

2. CANTILEVER WALLS

Most cantilever structural walls in multistorey buildings are slender enough to be treated as ordinary beams. There is no reason to suspect that in their behaviour such walls would disobey the familiar principles of reinforced concrete theory. Therefore the analogy to beams or beam-columns is appropriate. The performance of squat shear walls may be dominated by shear and hence they require special treatment.

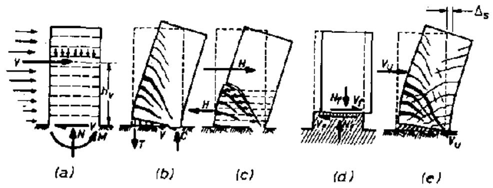

Fig 1. Failure modes in laterally loaded reinforced concrete cantilever shear walls

It is a prerequisite for any ductile structure that its strength shall not be limited by insufficient anchorage of the reinforcement or by the instability of any of its components, including reinforcing bars subjected to extensive compressive yield strain.

Fig. 1a, shows a prototype cantilever wall in a multistorey building, subjected to gravity and lateral loading, and the corresponding critical actions at the base. In the following, the possible failure modes of this structure, together with critical aspects of its behaviour are examined briefly.

2.1 - Flexure

Because of load reversals wall sections necessarily contain substantial compression reinforcement. Gravity loads commonly produce only small axial stresses. Consequently the available curvature ductility at the critical section, shown in Fig. 1b, is usually ample.

It has been customary to distribute the vertical reinforcement uniformly over the length of lightly loaded structural walls. It is obvious that this is not an efficient arrangement when large overturning moments require considerably more than minimum (ρ = 0.25%) reinforcement content. In such cases this arrangement of wall reinforcement, as shown in Fig. 2, will also result in reduced curvature ductility in the potential plastic hinge zone [3]. When more than minimum reinforcement is required, it will be more advantageous to concentrate the reinforcement, that is in excess of the minimum, at the extremities of the wall section, as shown in Fig. 2. It may be necessary to increase the thickness of the wall to accommodate such reinforcement.

The most efficient way to increase both the stiffness and the yield capacity of a cantilever structural wall is by providing flanges. Parametric studies of symmetrical flanged I and H sections [4] indicated that:

(a) The additions of flanges will increase the curvature ductility of the section.

(b) The increase of flange thickness, while giving additional stiffness and increased yield resistance does not favourably affect the ductility of the section.

(c) For obvious reason, the most inefficient way to boost stiffness and strength is by increasing the thickness of the web.

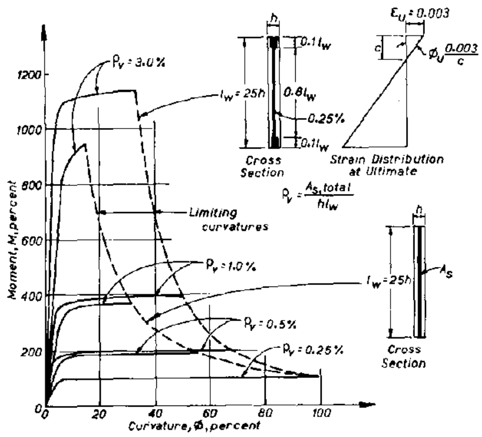

Fig 2. Effect of amount and distribution of vertical wall reinforcement on ultimate curvature [3].

If and when required, the concrete in the vicinity of maximum concrete strains may be confined and thus the ductility of a section may be further increased. Because the flexural failure mode in a cantilever wall, illustrated in Fig. 1b, is usually associated with adequate ductility, every attempt must be made by the designer to force upon the structure this energy dissipating failure mechanism.

Because of the repeated and reversed nature of the loading the flexural (vertical) reinforcement at the extreme fibres of wall section may be subjected to large tensile yielding. This implies that after load reversal considerable compression yielding must occur before a previously formed large crack can close and the concrete can again contribute to carrying compression. It is essential therefore that compression bars that could possibly yield be restrained against buckling. Near the extreme fibres of the section, where concrete strains may well be in excess of 0.003, it is advisable to disregard the contribution of the cover concrete, and to rely entirely on transverse ties to give lateral support to the compression bars. The spacing of such ties over the potential plastic hinge length should not exceed $6d_b$, where d_b is the diameter of the principal bar to be laterally supported [5]. It seems reasonable to assume that under extreme load conditions where the computed strain is less than 0.0015 the concrete cover will not spall and yield strain in the vertical bars will not be exceeded. Consequently in such areas special transverse reinforcement to stabilise vertical bars should not be required. Draft recommendations for corresponding provisions are made subsequently.

2.2 - Instability of Structural Wall Sections

The instability of a cantilever wall, as a structural member, seldom need be considered in practice because the floors, which introduce the lateral load to the wall, will normally provide ample lateral support. However, because in buildings relatively thin walled sections are common, some precaution must be taken to ensure that in the potential plastic hinge region the repeated and reversed full load can be sustained within the horizontal diaphragms (floors) that provide lateral support.

A theoretical or experimental study of "compactness" of structural wall sections has not been reported. However, using existing code [6] requirements for slender columns some crude but conservative guides can be established to limit slenderness in shear walls. It may be conservatively stipulated that any part of a thin wall element, which may be subjected to large compression and which does not receive lateral support between floors, should be considered as an isolated column. Hence it is suggested that the thickness, b, of any part of a structual wall, two storeys or higher, in which the combination of flexure and axial load can produce a compression strain of 0.0015 or more, should not be less than $l_n/10$, where l_n is the clear vertical distance between floors or other effective horizontal lines of lateral support.

Similar limitations can be formulated for outstanding flanges of structural walls with various sectional shapes. Detailed suggestions are made in the draft recommendations.

2.3 - Shear Strength

It is now more generally recognized that the shear strength of tall walls can be assessed the same way as that of beams. The behaviour of short shear walls requires special considerations. Shear failures are generally associated with limited ductility. When shear dominates the response of a shear wall the undesirable features of degrading stiffness and strength become distinct consequences of simulated seismic cyclic loading. Therefore every attempt must be made to suppress a shear failure.

In a cantilever wall a distinction must be made between the potential plastic hinge area and the remainder of the structure which, even though extensively cracked, is likely to remain in the elastic domain of response. Diagonal cracks are usually extensions of flexural cracks, as illustrated in Fig. 1b. Therefore the yielding of the flexural (vertical) reinforcement affects also the widening of the diagonal cracks. It is advisable therefore that over the length of the potential plastic hinge the contribution of all mechanisms to the shear strength, except that of the web reinforcement, be neglected. This implies that in order to ensure a ductile flexural response the plastic hinge region must have sufficient (horizontal) stirrup reinforcement to carry below its yield strength level the entire shear force, shown as H in Fig. 1c, associated with the maximum possible flexural strength of the shear wall. In the remainder of the wall the shear strength may be assumed to be the sum of the contributions of the concrete, V_c, and that of the web reinforcement, V_s.

It is to be noted that for cantilever shear walls the code specified equivalent lateral static load does not necessarily give satisfactory protection against a shear failure when, during a severe excitation, the maximum flexural strength at the base is being developed. During certain combinations of the different modes of vibrations the centre of the lateral inertia forces, located with h_v in Fig. 1a, may be lower than that indicated by customary code prescribed load patterns, such as an inverted triangular load. Consequently considerably larger shear forces may be generated when the moment capacity at the base is attained. [7] It has been shown in case studies that for a number of standard ground motion inputs the ratio of shear forces induced during the combined higher mode responses to the shear force derived from statics and the base moment capacity increases with the fundamental period of the cantilever structure. [8] To guard against a shear failure under such circumstances the design shear envelope for the cantilever wall could be the shear force diagram, corresponding with a code specified static load, magnified by a factor, which will depend on the class of the building and the fundamental period of the structure. This, together with other aspects of dynamic response, is discussed further in section 2.7.

2.4 - Sliding Shear Phenomena

There are two potential locations in cantilever walls where a failure by sliding could occur. One is a horizontal construction joint which is sensitive to the quality and nature of surface preparation. The other is the plastic hinge zone, usually immediately above foundation level, where, because of the yielding of the flexural reinforcement in both faces and consequent residual strains, continuous cracks across the full depth of the shear wall section will occur.

2.4.1 -- Sliding at construction joints. The sliding along construction joints, illustrated in Fig. 1 d, often observed in shear walls damaged by earthquakes, [9] can be suppressed if, in accordance with the concepts of shear friction, adequate distributed vertical reinforcement is provided over the length of the wall to supply, together with the available gravity load, the necessary clamping force, N_f. [10] The inelastic response of the mechanisms associated with sliding shear indicates drastic loss of stiffness and strength with reversed cyclic loading. [11] Therefore sliding shear must be considered as being an unsuitable energy dissipating mechanism in earthquake resistant structures. It is relatively easy to provide the necessary clamping force across construction joints below yield strength level of the vertical wall reinforcement. The increased shear forces due to higher mode responses, as outlined in the previous section should, however, be considered. Provisions are made in the draft recommendations for the determination of the required longitudinal reinforcement.

2.4.2 -- Sliding across plastic hinge zones. The possibility of sliding shear is much more serious in the plastic hinge zone where, as a result of reversed cyclic load and the ensuing residual plastic strains in the flexural reinforcement of the wall, large continuous cracks may form, as shown in Fig. 1 e. The interlock between two serrated faces of such a full depth crack is greatly diminished. Moreover concrete blocks, bound by wide intersecting diagonal and flexural cracks, are subjected to eccentric diagonal compression forces and consequently will be gradually broken up into smaller blocks. If the axial compression across the section due to gravity loads is small, a sliding shear failure may occur across the precracked compression zone (see Fig. 1 e) at nominal shear stresses that are below values permitted by codes [6]. If the foundation is massive, it will provide some lateral restraint at the section of maximum moment and the section of failure will move away from the force of base fixity [7,12], as shown in Fig. 3, usually to a line where a set of horizontal stirrups are located.

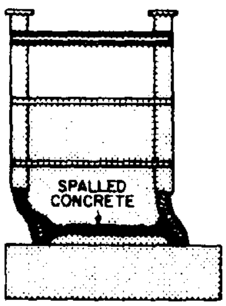

Fig 3. The mechanism of sliding shear failure in a wall specimen. [7]

Shear transfer by dowel action of the

vertical reinforcement is mobilized only after a substantial slip, shown by Δ_s in Fig. 1 e, has occurred. A particularly severe situation can arise in wide flanged shear walls when the neutral axis of the section, at the development of the full flexural capacity, is in or near the flange. Under such circumstances the previously cracked compression area, over which the major part of the shear force must be transmitted, is very small and consequently the interface shear stress can become excessive. Sliding shear displacements are largely responsible for the loss in energy dissipation in structural walls, as evidenced by the pinching in histeresis loops. This was demonstrated also by some recent tests (Fig. 4) of the Portland Cement Association. [12] When the nominal shear stress at the attainment of the flexural capacity approaches the maximum value specified by the ACI Code [6], i.e. $v_{u,\ max} \approx 0.83\ \sqrt{f'_c}$ MPa ($10\ \sqrt{f'_c}$ psi), shear deflections may equal or exceed the simultaneously imposed flexural deflections, measured at the tip of a cantilever. [7] With large nominal shear stresses, shear distortions will certainly dominate in the plastic hinge zone. [7] From the study of a large number of experiments the effect of shear on the pinching of histeresis loops, and its representation by a concave Ramberg-Osgood relationship, was examined by Celebi. [13]

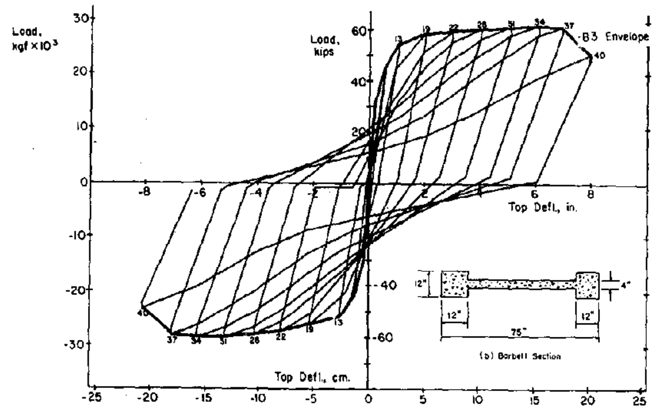

Fig 4. Load-top deflection relationship for a cantilever shear wall [12].

To control shear distortions in potential plastic hinge zones when the nominal shear stress is significant, diagonal reinforcement may need to be provided so that the shear force is transmitted, from one side of a wide horizontal crack to the other side, by the horizontal components of diagonal tension and compression steel forces. Some relevant suggestions are made in the draft recommendations.

2.5 - Squat Structural Walls

In many low-rise buildings, the height of cantilever walls is less than their length (i.e., their structural depth). Clearly, in such situations the assessment of the flexural and shear strength and appropriate reinforcement cannot be based on the conventional techniques applicable to taller walls. Rather, the principles established in connection with the behaviour of deep beams are relevant. It is no longer possible to discuss separately flexure and shear, since the two are more intimately interrelated in squat walls.

Low-rise structural walls normally carry only very small gravity loads and for this reason their beneficial effect, derived at least for shear strength, is best ignored. The flexural steel demand will also be small in most cases because of the relatively large available internal lever arm. It will be more practical, therefore, to distribute the vertical (i.e., flexural) reinforcement uniformly over the full length of the wall, allowing only a nominal increase at the vertical edges.

For seismic loading the corresponding loss of ductility is not likely to be of great importance for two reasons. First, the low steel requirement is often satisfied by near-minimum steel content (i.e., 0.25%), which provides sufficient energy absorption in the postelastic range (see Fig. 2). Second, properly detailed squat shear walls can be made to absorb all or most of the seismic shock in the elastic range without demand for great reinforcement contents.

For want of other and better information, it has been the practice to attempt to predict the likely behaviour of low-rise shear walls from tests carried out on deep beams. Geometric similarities suggest such a procedure. Most tests on deep beams have a common feature - the load is directly applied to the top and bottom faces of the simply supported specimens in the span and at the supports, respectively. It must be pointed out that this form of load application considerably enhances the effectiveness of arch action. Stirrups crossing the main diagonal crack, forming between load point and support, are not engaged in efficient shear resistance because no compression struts can form between stirrup anchorages. The arch disposes of the shear along the shortest possible route, and this is associated with smaller deformations. It is not surprising, therefore, to find from experiments that additional stirrups did not improve shear strength.

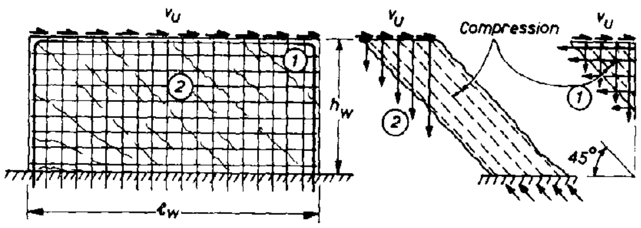

Fig 5. The shear resistance in low-rise structural walls.

The crack pattern, likely to occur in a low-rise shear wall, is sketched in Fig. 5. From considerations of equilibrium of the triangular free body, marked 1, it is evident that horizontal stirrups are required to resist the shearing stress applied along the top edge. The diagonal compression forces set up in the free body also require vertical reinforcement. In the absence of external vertical compression, the horizontal and vertical steel must be equal, to enable 45° compression diagonals to be generated. In the free body bound by two diagonal cracks and marked 2, on the other hand, only vertical forces, equal to the shear intensity, need be generated to develop the necessary diagonal compression. This steel is often referred to as shear reinforcement, even though its principal role is to resist the moment that tends to overturn free body 2. Figure 5 thus illustrates the role of vertical and horizontal bars in resisting shear forces in low-rise shear walls.

Squat structural walls are sometimes used, particularly in Japan, as elements of slender cantilever walls or as infill panels in rigid jointed frames with the intention of providing increased stiffness during small seismic excitations and damping when large interstorey displacements are expected [14]. They are commonly referred to as "framed shear walls". A typical test panel after loading is shown in Fig. 6. [15].

Such walls are expected to remain uncracked till a shear strain of 0.25×10^{-3} rad. is reached. After diagonal cracking the boundary frame is expected to restrain the panel against further expansion. [14] Thereafter the panel is to carry the applied shear by means of a diagonal compression field, all other necessary tensile forces being supplied by the boundary elements. Also the boundary members are assigned the total gravity load while the web panel is to provide the desired damping. It is envisaged that after extensive diagonal cracking the panels could be replaced because the boundary elements remain free of damage. In Japanese practice "frameless walls" are expected to fail in shear and not in flexure [14]. Therefore their use is discouraged. It is interesting to note that the recommended thickness of such framed wall panels can be as little as $h_n/30$, where h_n is the clear height between the horizontal boundary members.

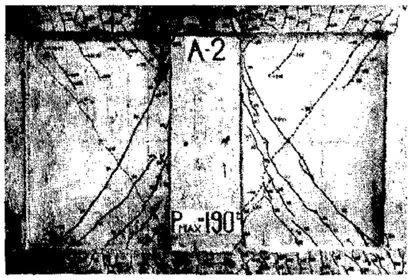

Fig 6. Framed shear wall studies in Japan. [15]

As shear dominates the response of such elements, it is not surprising that dramatic stiffness and strength loss results as a consequence of repeated cyclic loading, particularly with increasing amplitude. Such a response is shown in Fig. 7. [16] The applied load usually consits of compression along one

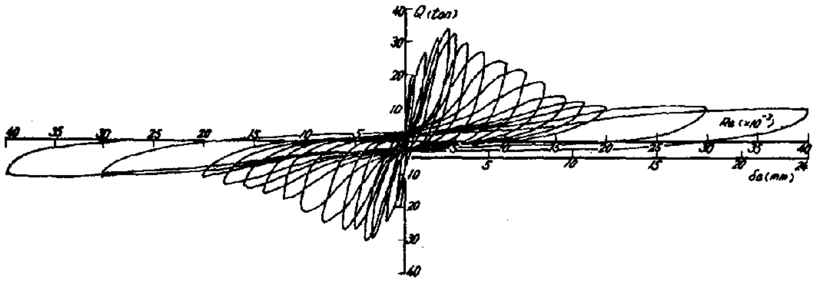

Fig 7 Shear load - shear displacement relationship for a "framed shear wall" panel [16].

of the diagonals. Better energy dissipation can be attained if the mesh reinforcement in the wall panel is arranged along the 45° diagonals [14].

A suggested application for the use of "framed shear wall" panels is illustrated in Fig. 8. The intention is to block, in a particular pattern, the panels, the dimensions of which are also shown in Fig. 8. [17]

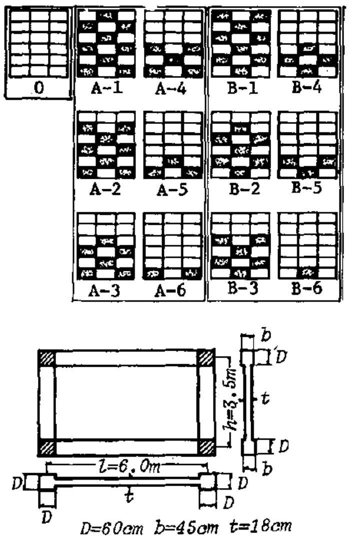

Fig 8. A suggested use of "framed shear wall" panels. [17]

Another Japanese proposal relates to diagonal members in a rectangular boundary frame shown in Fig. 9. It is claimed [18] that in addition to the stiffness, which is comparable to that of "framed shear walls" considerable ductility can also be obtained. From the detailing of such a unit and the failure patterns shown in Fig. 9, it is evident that considerable care would need to be taken with the confining and slenderness of compression members and the detailing of the joint regions. It is doubtful whether the improvement in response would outweigh the difficulties involved in the construction.

In order to increase the flexibility of structural walls the Kajime construction Co. of Japan [19] introduced the concept of "slit shear walls". By inserting asbestos cement plates into the panel, more widely distributed cracking can be induced in a series of vertical beam-like members, as shown in Fig. 10, and thus improved damping can be attained.

It is doubtful whether the superposition of the observed response of individual "framed shear wall" panels will correctly predict the overall response of a reinforced concrete shear wall structure. In monolithic cast in situ construction, the wall will attempt to act as one integral unit in resisting combined gravity and seismic forces. Every attempt should be made by the designer to encourage such action, which suits best the natural behaviour of a structural wall. In doing so it will be relatively easy to ensure a predominant flexural mode when energy dissipation is required. When a shear failure mechanism is suppressed strength and stiffness degradation under reversed cyclic load will be minimised and hence energy dissipation will be approximately proportional to the imposed inelastic displacement on the structual wall.

2.5 - Moment - Axial Load Interaction

Cross sections of flanged, angle, or channel shapes often appear in shear walls, forming the core of multistory buildings. These may be subjected to axial loads of varying intensity, including net tension, together with bending moments about one or both principal axes. For practical reasons the

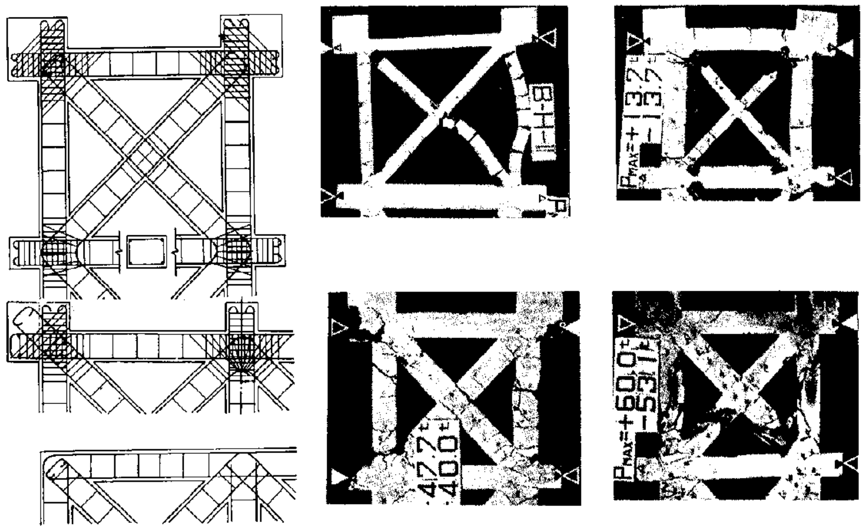

Fig 9. Reinforced concrete truss frame intended for earthquake resistance. [18]

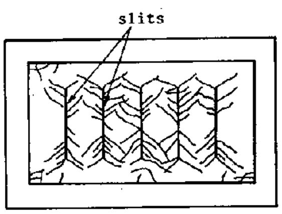

Fig 10. Slit shear wall in its boundary frame. [19]

cross sections remain reasonably constant over the full height of the structure. It is possible, and it may be advantageous, to evaluate the interaction relationship between flexure and axial force for such cantilever shear walls. It can be rather cumbersome to work out the required reinforcement for a particular load interaction, but is is relatively easy to determine the possible load combinations for given arrangements and amount of reinforcement, particularly with the aid of a computer. The results can then be used to allocate the required reinforcement at any level along the full height of the structure.

When a channel-shaped cross section is subjected to axial load and flexure about its weak principal axis, interaction curves of the type illustrated in Fig. 11 result. In this particular section the reinforcement was assumed to be uniformly distributed along the center of the wall thickness. The load eccentricity is with reference to the plastic centroid of the section. A positive moment is considered to cause compression at the tips of the flanges and tension in the web of the channel. For pure flexure, this would be an over-reinforced section with about 3% total steel content. For a reversed (negative) moment, causing compression in the web of the section, a marked increase in moment capacity follows the application of compression forces. The wall section in Fig. 11 is suitable to re-

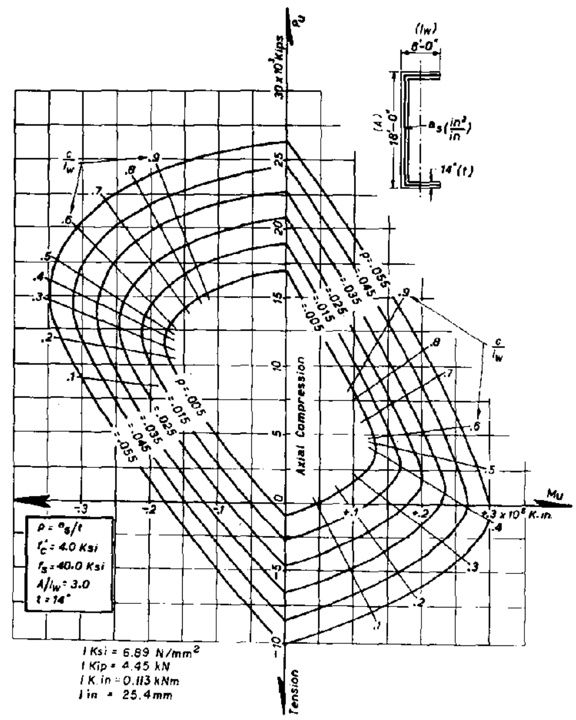

Fig 11 Typical moment-axial force interaction relationship for a channel-shaped wall. [11]

sist moderate axial tension in combination with positive moments, and considerable axial compression with negative moments. These are typical load combinations occurring in coupled shear wall structures.

The position of the neutral axis, shown by the radiating straight lines, is indicative of the curvature ductility that is involved at the development of the ideal strength of such a section. It is evident that in estimating the available ductility, the configuration of the cross section is more significant than the intensity of the axial compression. For example, if a possible moment is applied to the structural wall, shown in Fig. 11, with only negligable axial compression, a considerable portion of the flanges will need to be provided with confining reinforcement to ensure that adequate curvature ductility will be available. Provisions for such situations are made in the draft recommendations.

It may be noted that the ductility potential of such walls is considerable if positive moments occur with net axial tension,and compression forces are applied only with negative moments. This is the case in coupled shear walls, to be examined in section 4, where channel shaped walls can be efficiently employed.

In spite of an exhaustive treatment of the subject, some misconceptions still exist with respect to the relationship between curvature ductility at the critical section, μ_ϕ, and the overall (displacement) ductility, μ_Δ, of a structural wall. Therefore it is emphasized that the curvature ductility demand for a cantilever wall at its base will be considerably larger than the displacement ductility, particularly when the height to length ratio of the wall is large [20]. The relationship between these two ductility factors μ_ϕ and μ_Δ, is shown in Fig. 12 for cantilever walls subjected to a single lateral point load at the top. The intersecting boundaries of the bands

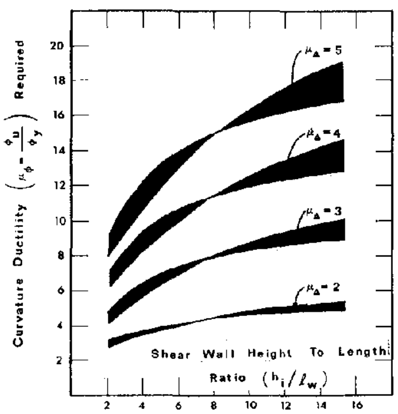

Fig 12. The relationship between curvature and displacement ductility factors in cantilever structural walls [20].

represent two different assumptions with respect to the length of the plastic hinge at the base of the wall.

2.7 - Some Implications of Dynamic Response

From a limited number of theoretical and experimental studies a few design features emerge which are important. An extensive parametric study of the dynamic response of 20 storey cantilever walls, with various fundamental periods and base yield properties, was carried out by the Portland Cement Association [21, 22, 23], and some of the conclusions were as follows:

(a) the magnitude of the ductility demand at the base hinge generally increases with decreasing fundamental period, when the moment of resistance provided at the base section remains the same.

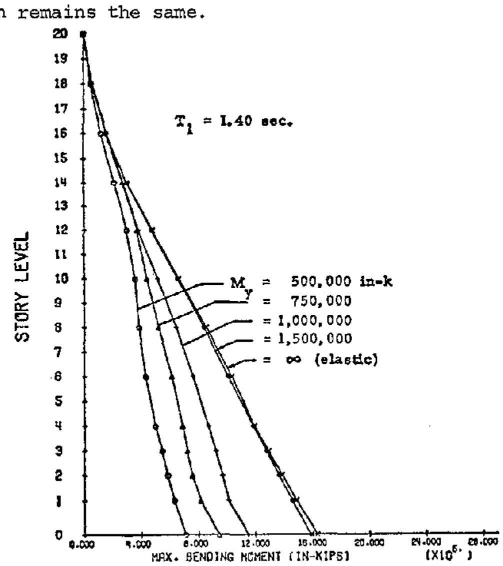

Fig 13. Variation of flexural demand along the height of a twenty storey cantilever subjected to different dynamic excitations [23].

(b) The variation of shear load with height does not significantly change in walls of different fundamental periods ($0.8 < T_1 < 2.4$ sec.) and identical flexural capacity at the base. This contradicts with the results of other studies particularly those using modal superposition. [8]

(c) For constant stiffness i.e. fundamental period T_1, the shear demand increases considerably at every level, as expected, with increasing flexural capacity of the base.

(d) As the flexural capacity of a wall, with a given period, is reduced the extent of yielding (length of plastic hinge) progresses from the base, i.e. from one to six storeys in a 20 storey cantilever wall.

A particularly interesting result of this study [23], shown in Fig. 13, indicates that the flexural moment demand in the upper storeys is larger than what one would obtain from a bending moment diagram constructed for a code prescribed equivalent lateral static load. [1, 2] This then signifies that if yielding is to be avoided in the upper storeys of a cantilever wall, then the flexural strength i.e. curtailment of the principal longitudinal flexural reinforcement, should follow a linear variation rather than a more rapidly decreasing moment pattern, such as a third order parabola. Yielding at upper storeys is not objectionable while extensive rotational ductility is expected at the base. However, it must be remembered that the flexural yieldings strongly affects the shear resistance of a wall. In particular the contribution of the concrete shear resisting mechanisms would be reduced and consequently shear reinforcement would need to be provided for the entire expected shear force.

The intense shaking of small scale wall models at the University of Illinois [24] drew attention to the phenomena, observed also in static tests, that sliding shear failure presents a potential hazard in the plastic hinge zone of conventionally reinforced structural walls.

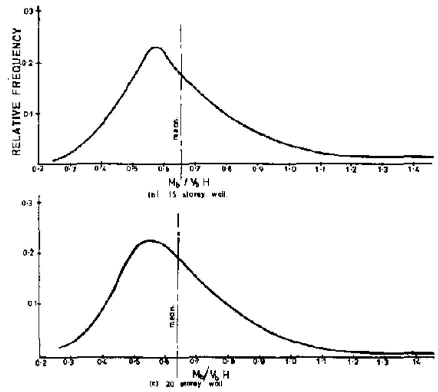

Fig 14. Frequency distribution of centre of applied initial load for two cantilever walls. [8]

Inelastic time history studies of cantilever walls from 6 to 20 storeys, using various earthquake acceleration records, made comparisons [8] with flexural and shear demands specified by the New Zealand loading code. [2] These studies also verified previous findings, discussed in section 2.3, that both induced moments and shear forces will exceed at various floors the quantities indicated by equivalent static loads. The ratio of the base moment to base shear, M_b/V_b, represents a convenient measure for assessing the variability of the modal combinations at flexural capacity. [8] Frequency curves for two of the structures are reproduced in Fig. 14 showing all values of M_b/V_b recorded while the structures were in the strain hardening range for all earthquake analyses. The frequency distribution shows skewness with the mode less than the mean. Wherever the value of M_b/V_b is less than that implied by the static code loading, which is approximately 0.72H, the shear forces near the base will be higher than that resulting from the code specified distribution of the loading. Conversely where the values of M_b/V_b are larger, the shears near the top of the structure will be larger. This observation resulted in the incorporation of dynamic shear magnification factors in the New Zealand Loading Code [2], shown in Table 1, which are to be applied to the shear forces derived for the specified lateral static loading and scaled up to correspond with the strain hardened flexural capacity of the wall base.

Number of Storeys	Building Class* I	II	III
1 to 5	1.0	1.2	1.3
6 to 9	1.2	1.3	1.5
10 to 14	1.4	1.5	1.7
15 to 20	1.4	1.6	1.8

* Class I: Essential facilities required to be completely functional immediately after a seismic disaster, with an importance factor of I = 1.6.
Class II: Public buildings not included in Class I with I = 1.3.
Class III: All other buildings with I = 1.0.

Table I: Dynamic Shear Magnification Factors [2]

3. FRAME-WALL INTERACTION

Analytic techniques currently in use, when suitably programmed enable the accurate assessment of the elastic interaction of structural walls and rigid jointed frames to be made. Certain problems of behaviour arise from the interaction of two structures that respond in distinctly different manners to lateral load.

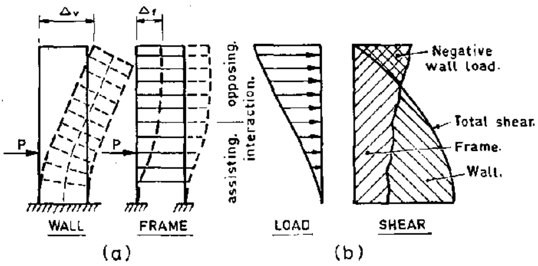

Fig 15. Interaction of a cantilever wall with rigid jointed frames

Fig. 15a illustrates a cantilever wall and a frame, both carrying the same load at a certain height. This causes the shear wall to suffer bending distortions and to assume a constant slope above the level of load applications. The originally horizontal sections at each floor tilt. The frame experiences mainly translatory displacements and it tends to become vertical above the load level. When column shortening is neglected, which is justified in most reinforced concrete buildings of moderate height, the floors remain horizontal. Because of this incompatibility of deformations, a cantilever wall can oppose an open frame in its load resistance at the upper floors. Only at the lower floors do the two structures assist each other in carrying the external lateral load. A typical distribution of the lateral load between a relatively slender shear wall and a frame, in terms of shear forces, is illustrated in Fig. 15b.

Because of the large difference in stiffness with respect to lateral load, the walls are likely to yield at their base before any yielding in the beams or columns of the interacting frames would occur. After the formation of a base hinge subsequent rigid body rotations of the wall or walls will enable the full plastifications of the interacting frames to develop. If the walls remain elastic above the base hinge then they are likely to absorb most of the higher mode effects during the subsequent nonlinear dynamic response of the building. This could be beneficial,for it is likely to ensure a more even distribution of ductility demand in the beams of the interacting frames. Also there is likely to be a much more even distribution of beam input moment among the columns above and below a particular floor. Thereby the likelyhood of hinge formation in the columns at upper storeys is virtually eliminated and the restoring force characteristic of the frames are enhanced.

For certain types of buildings it is customary to allocate the entire specified lateral strength to structural walls and to provide back up frames with a potential strength at least equal to 25% of the required lateral load. [1] This implies lack of faith in structural walls because of their presumed inability to supply adequate ductility with only insignificant loss of strength. There is ample evidence available, however, to show that structural walls can be designed and detailed so that ample ductility will be available during several reversed excursions into the postelastic range of response without significant loss of strength. With properly detailed walls, particularly in the potential plastic hinge regions, there should be no need to provide for a flexible back up framing system that would be mobilised when the much stiffer wall system has failed, presumably in a brittle manner.

The behaviour of cantilever structural walls, that are designed to interact with rigid jointed frames does not appear to be any different from those discussed in section 2. For this reason the draft recommendations for earthquake resistant structural walls are equally relevant.

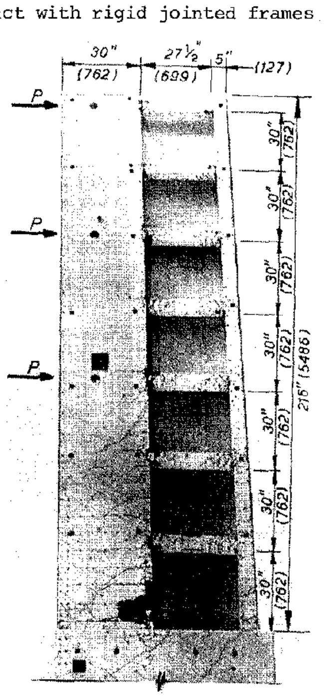

Fig 16. One quarter full size reinforced concrete frame-shear wall assembly after testing. [25]

The weakness in a seven storey, one quarter scale reinforced concrete wall-frame model, tested at the University of Canterbury under reversed static cyclic loading, was found to be in the beams of the frame rather than in the structural wall. [25]

The relatively short beams, shown in Fig. 16, developed plastic hinges at both ends after the onset of plastic rotations at the base of the wall. In spite of the low nominal shear stresses induced ($0.028\sqrt{f'_c}$ MPa, $2.8\sqrt{f'_c}$ psi) the hinges adjacent to the walls showed excessibe slidings shear displacements. When this mode of failure was controlled by the placings of diagonal reinforcement at the potential beam hinges, a very satisfactory overall response to several cycles of reversed loading was obtained, as shown in Fig. 17.

Small scale walls and frames that were coupled at three floors during intense shaking, showed that the contribution of frames became significant after extensive damage (crushing and sliding shear) occurred at the wall base. [24] However, the upper part of the wall remained effective in stiffening the upper storeys and the observed base shear wave form was still very considerably larger than in an identically excited frame without walls.

4. COUPLED SHEAR WALLS

4.1 - Concept of Behaviour

Many shear walls contain one or more vertical rows of openings. A particularly common example of such a structure is the "shear core" of a tall building, which accommodates elevator shafts, stairwells, and service ducts. Access doors to these shafts pierce the walls. Thus the walls on each side of openings may be interconnected by short, often deep beams. It is customary to refer to such walls as being "coupled" by beams. A typical structure is illustrated in Fig. 18c.

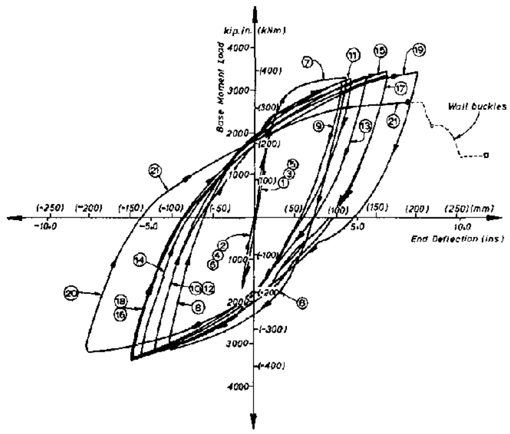

Fig 17. Load-roof deflection relationship for a frame shear wall assembly with diagonally reinforced beams. [25]

Although a coupled shear wall may be examined with standard open frame programs, that take the unusual relative dimensions into account, it is preferable to discuss behaviour in terms of a coupling, because generally the response of the walls will dominate the overall response of such structures.

The coupling system, consisting of a number of short coupling beams, transmits shearing forces from one wall to another (see Fig. 18c), subjecting coupling beams to flexure and shear. Because of the small span/depth ratio of these beams, shear deformations may become very significant.

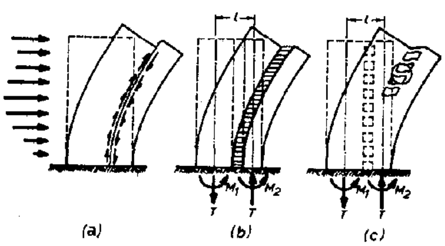

Fig 18. Laterally loaded coupled shear walls.

Owing to their large stiffness, the coupling beams are sensitive to relative movements of their built-in supports. For this reason the axial deformations of the coupled walls, which are responsible for such movements, may have a considerable effect on the overall behaviour.

In a technique specifically devised for the study of such structures, referred to as "laminar analysis" the discrete connecting beams are replaced by an equivalent elastic continuum. [26, 27] With the extensive use of this analysis, numerous parametric studies have been carried out, making allowances for different boundary conditions, so that the elastic response of such structures to static load may be considered as being well understood.

The particular advantages of coupled shear walls in earthquake resistant design may be better appreciated if their behaviour is compared with that of a simple cantilever. In a homogeneous, isotropic cantilever beam the maximum shear stresses, which may be critical, will be induced along the fibre of the neutral axis. If this critical shear fibre, or some other fibres nearby, are potentially weak, as may be the case in precast panel construction, a sliding shear failure along the fibre, as illustrated in Fig. 18a, is a possibility.

However, if this failure mechanism could be made to be ductile and also to possess good energy dissipating properties under reversed cyclic loading, then it could be used as a viable component of the total load resisting system when shaking of extreme intensity is to be survived. By assuming that such ductile shear transfer mechanism can be constructed, the previously discussed cantilever shear walls could be transformed into coupled shear walls, illustrated in Fig. 18b. It is seen that the total over-turning moment, produced by the external lateral load, is now resisted by flexure in each of the two cantilever walls, M_1 and M_2, and by the axial force, T in each of the walls, which operates on an internal lever arm l. The axial force, T, is simply the sum of the shearing forces across the continuous coupling system.

The potential of this structure, as an efficient earthquake resistant construction, would stem from its ability to dissipate energy, when required, within this shear transfer (coupling) system over its full height. This would be in addition to the energy dissipated in the plastic hinge that would be expected to form eventually at the base of each wall. The behaviour of the individual coupled walls would be similar to that of cantilevers, discussed in the previous sections, except that the effect of the axial force, T, would have to be also considered.

With a skillful selection of relative stiffness and strength properties, it is possible to reinforce the various components in such a way that under continuously increasing lateral static loading the strength of the coupling system is developed before the onset of yielding at the base of the walls. This would imply that considerable energy could be dissipated by the coupling system, involving yielding and consequent damage, before damage and permanent misalignment of the coupled walls would occur. Under catastrophic conditions the major part of the total energy to be dissipated could be derived from the coupling system, thus reducing the ductility demand at the wall hinges.

In comparison with cantilever walls coupled shear walls offer more than one line of defence when energy dissipation is required. The wider dispersal of energy dissipating devices is likely to result in improved structural damage control. When a coupling system with a reasonable stiffness is chosen, which is relatively easy to achieve in practice, the reduction of the stiffness of the coupled shear wall structure, in comparison with a cantilever wall with the same overall dimensions, is insignificant. Consequently the protection against damage of the non-structural components of the building within the elastic response of the coupled shear walls can still be assured.

It appears that a deliberate introduction of a weakness to a cantilever shear wall in the form of a ductile shear fibre, as shown in Fig. 18a, or its equivalent, illustrated in Fig. 18b, may result in a structure which possesses the features that are so desirable in earthquake resistant construction, i.e. adequate stiffness to give overall damage control during moderate disturbances and ample ductility and energy dissipating ability for the catastrophic situation. For obvious reasons, the coupling system, which normally is an insignificant component when gravity loads are considered, lends itself much better for repair than cantilever walls.

The axial force in the walls results from the accumulation of the shearing forces across the coupling system of beams or laminae. The larger the stiffness of the coupling system relative to the walls, the more efficient the coupling, the larger the induced vertical shearing forces in the beams, and the larger the consequent axial force in the walls. The interplay between the modes of internal moment resistance depends on the strength and stiffness of the coupling between the two walls. Clearly it is more efficient to resist the external moment predominantly by internal forces T, which operate with a large lever arm l, than by component internal moments M_1 and M_2.

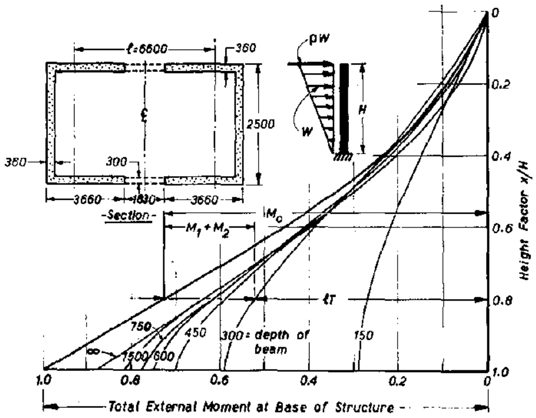

Fig 19. The mode of internal moment resistance in a coupled shear wall structure.

The relative proportions of the contributions of the internal couple, lT, in resisting the external moments M_o, at various levels of an elastic 20 storey shear core are presented in Fig. 19. It is seen that the coupling is efficient for the top half of the structure for all but the shallowest beams. At the base, little difference in behaviour is indicated for 600 mm deep or infinitely stiff beams. The latter represents the case when no distortions occur in the process of shear transfer from one wall to another; that is, a continuous linear strain distribution occurs across the entire shear wall structure. The low effficiency of 150 mm deep coupling beams shows the approach to the other limiting situation when $lT = 0$; that is, when the entire external moment is resisted by flexure in the component cantilever walls M_1 and M_2.

The role of cracking in the elastic behaviour of shear walls has been examined theoretically and experimentally. [28] Because of the very large differences between the stiffnesses of the components, and the drastic loss of stiffness in the coupling system after diagonal cracking, a 75 to 100% increase in both the deflection and the wall moments has been obtained in case studies in which allowance was made for cracking. It is best to make a number of assumptions with regard to loss of stiffness caused by cracking in the walls and the coupling system and to carry out an analysis for each.

4.2 - Ductility Demands

The strength of two coupled shear walls, subject to seismic-type lateral loading, is reached when a collapse mechanism is formed. Two plastic hinges in each coupling beam are required to terminate its ability to accept additional shear. In addition, one plastic hinge needs to be developed in each of the cantilever walls, normally at their base, to complete the collapse mechanism. The sequence of hinge formation for a given loading will depend on the relative strength and stiffness of the components. The mechanism is similar to that for a multistory frame.

The behaviour of some coupled shear walls that were exposed to severe earthquakes, indicated that all or most coupling beams failed before the ultimate strength of the coupled walls was attained. Classic examples are some of the end shear wall frames of two 14 story buildings, severly damaged during the 1964 Alaska earthquake. [29] It is possible, however, that in some structures the ultimate strength of the walls will be exhausted before plastic hinges form in the coupling beams.

The order of ductilities that need to be developed in the coupling beams and the walls can be estimated if an equivalent static load, such as specified by codes [1, 2], is applied to the structure. With a monotonic increase of the load, till the development of the complete failure mechanism, the subsequent deformations in all elements can be computed. Such studies show that for a given displacement ductility, as measured for example by the lateral deflection at roof level, very large ductilities may be required to be developed in the coupling beams. Typically, for a system displacement ductility factor of 4, we may require coupling beam distortions corresponding with a member ductility factor of 10 to 12. [11] Because shear distortions in coupling beams may be very significant, it is not convenient to express distortions in terms curvature ductilities. Rather the term "member ductility factor" is used which relates to the absolute rotations and displacements at the boundaries of such coupling beams, and hence takes into account all the distortions that may occur in the entire body of the beam. In particular, it should be noted that the plastic rotations that occur in the wall base hinges, θ_w, usually cause greatly magnified additional plastic rotations θ_p, The magnification is evident from the relationship: $\theta_p^* = l\theta_w/l_n$, where l is the distance between the centroidal axes of the two walls and l_n is the shear span of the coupling beams i.e. the distance between the inner forces of the coupled walls.

It is evident that the design of coupling beams warrants careful considerations if the desired ductility demand is to be maintained without significant loss of shear carrying capacity.

The strength and available ductility of the walls can be assessed with the aid of moment-axial load relationships such as shown in Fig. 11. It should be noted that, as opposed to normal column sections, the interaction relationship for rectangular coupled shear wall sections will not be symmetrical. For obvious reasons considerably more reinforcement will need to be provided

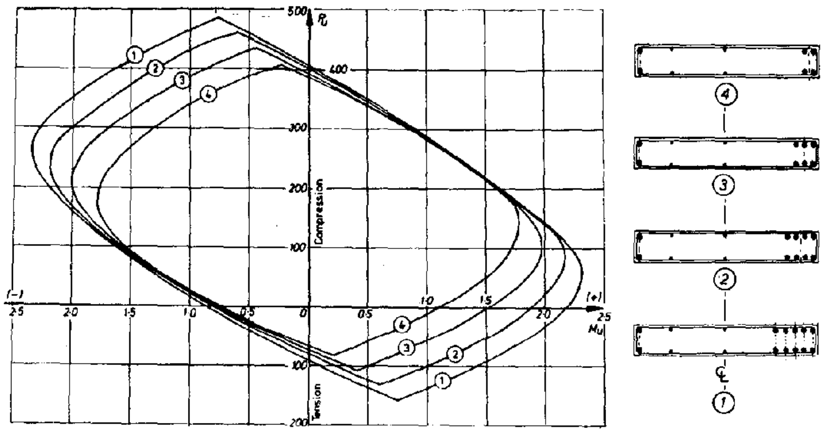

Fig 20. Moment-axial load interaction relationships related to the reference axis of non-symmetrical wall sections. [30]

near the outer edges than at the inner faces of the walls because of the moment-axial tension combination. This arrangement will also cause the plastic centroid not to coincide with centroid or reference axis of the wall. It is convenient to establish the interaction relationship with reference to the axis of the wall rather than its plastic centroid. Such relationships are shown for a model structure in Fig. 20.

4.3 - The Strength and Ductility of Coupling Beams

A more detailed examination of the observed performance of the coupling beams is given in another state-of-the-art report for this ERCBC workshop. Therefore only the main issues relevant to the design of coupled shear walls, are stated here.

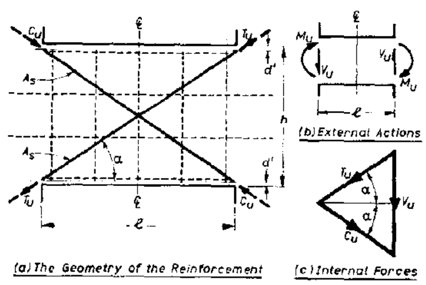

Fig 21. Model of diagonally reinforced coupling beam.

It is evident that diagonal tension failure, such as frequently observed in buildings damaged by recent earthquakes [29] must not be permitted to occur. Therefore the entire shear force that could be generated in such beams, when the flexural reinforcement provided develops its strain hardened strength, must be resisted by web (stirrup) reinforcement. The probability of yielding in stirrups must be minimised.

It has been observed that

when the nominal shear stress across the beam section becomes large a sliding shear failure is likely to occur. To eliminate such a relatively brittle failure it is recommended that the entire flexure and shear be resisted by diagonally arranged reinforcement. The simple model that obviously satisfies equilibrium requirements and which can be used as a basis of design, is shown in Fig. 21. A suggested practical arrangement of the reinforcement for a beam of particular dimensions is given in Fig. 22. The most important aspect of the detailing is the prevention of buckling of the diagonal bars that will be subject to Bauschinger effect after one excursion of the beam into the postelastic range of response.

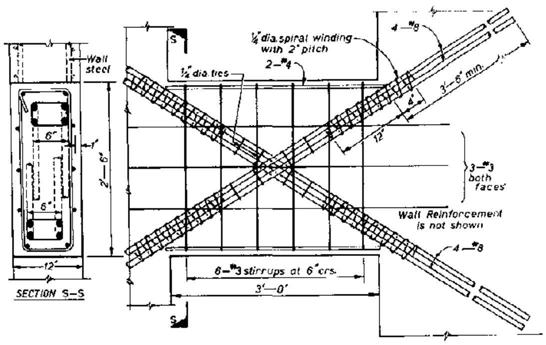

Fig 22. Suggested steel arrangement in a diagonally reinforced coupling beam

It may be noted that with inelastic reversed cyclic loading all the forces must gradually be transferred to the diagonal reinforcement. Consequently the expected response and the supply of ductility is that of a steel member. This was confirmed. [31]

4.4 - The Behaviour of Coupled Shear Walls

To verify the contribution of conventionally and diagonally reinforced coupling beams to the overall elasto-plastic response of coupled shear walls, two one quarter full size seven storey reinforced concrete coupled shear wall models were constructed, instrumented and tested under simulated cyclic loading at the University of Canterbury. The current code requirements in New Zealand [2] are such that a shear wall structure of this type is expected to be able to sustain a load which would cause a lateral displacement, usually measured at roof level, at least 4 times as much as the displacement at yield. Moreover, the load, when applied in this manner at least four times in each direction, must not diminish by more than 20%. Briefly, a displacement ductility of 4 must be sustained at least 4 times in each direction with a strength loss not exceeding 20%. The tests were carried out in such a way that the cumulative ductilities, imposed during progressive and increasingly severe loadings, were at least 16 (i.e. 4 x 4) in each direction of the load application. Only the highlights of the results, most relevant to the design of coupled shear walls, are presented here. Details of the study may be obtained from other reports. [11, 32]

Both model structures contained identical longitudinal wall reinforcement. The beams in wall A were conventionally reinforced while those of wall B contained diagonal bars, as shown in Fig. 23.

It is seen in Fig. 23 that at the end of the test all coupling beams of

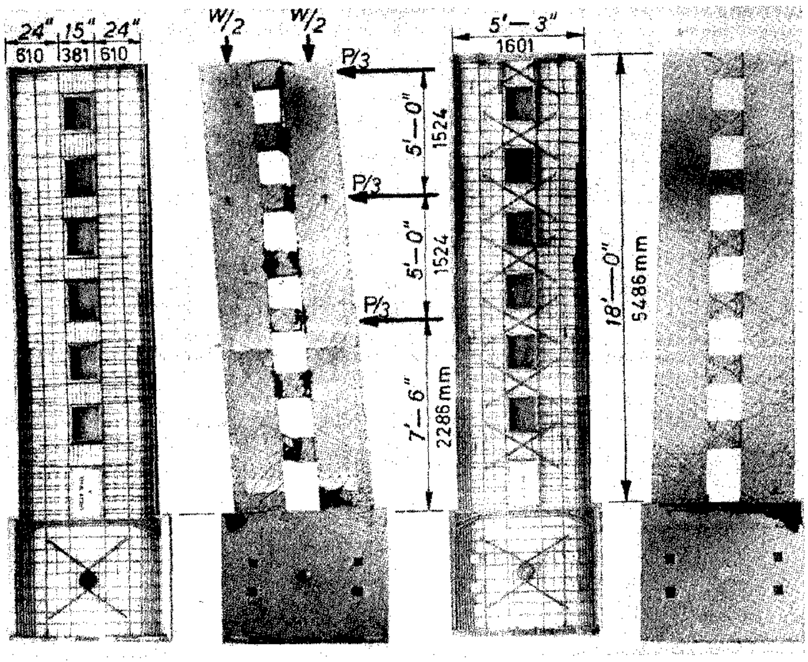

Fig 23. Reinforcing details and crack patterns of two model coupled shear walls subjected to simulated cyclic loading [32]

shear wall A failed at one or both ends by sliding shear. The crack pattern in the beams of shear wall B on the other hand show considerably less distress in spite of large imposed lateral displacements which correspond with a displacement ductility factor, μ_Δ, of more than 10.

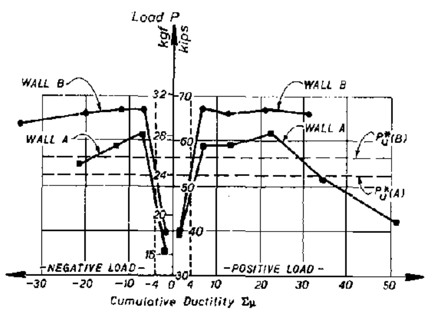

Fig 24. A comparison of the load sustained with imposed cumulative ductility in two coupled shear walls [30]

Because of strain hardening of the Grade 40 (f_y = 300 MPa) reinforcement, both walls developed a total lateral load capacity, P_u, which exceeded by up to 20% the theoretical load capacity, P_u^*, based on the observed strength properties of the steel and the concrete. Fig. 24 shows that with progressive loading, as measured by the cumulative displacement ductility, the strength, P_u, of wall A was gradually reducing. However, no significant strength loss was observed in wall B in spite of the severe loading sequence which imposed a cumulative ductility of approximately 32 (i.e. 2 x 16) in both directions.

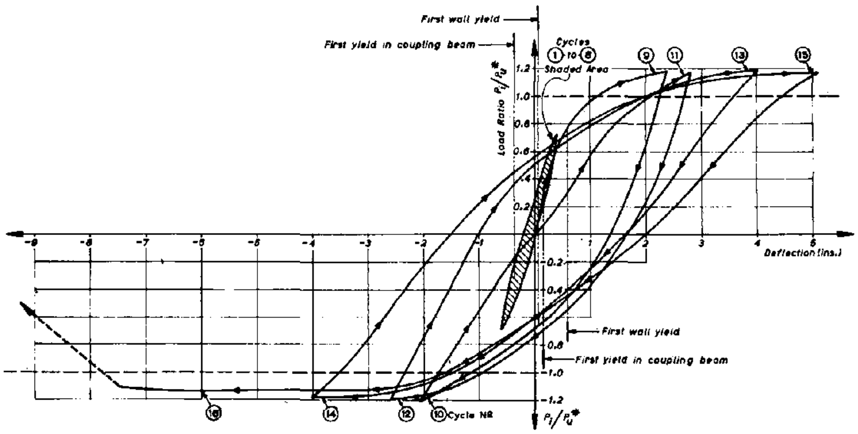

Fig 25. The load-roof displacement relationship for a model coupled shear wall with diagonally reinforced coupling beams.

Fig. 24 thus shows the excellent performance of the coupled shear wall model with diagonally reinforced beams in terms of the repeatedly sustained ultimate load, P_u.

The stable hysteretic response of wall B, resembling that of a steel structure, is shown in Fig. 25.

4.5 - Design Considerations for Coupled Shear Walls

The proportioning and detailing of walls in coupled shear wall structures should follow the same principles as outlined for cantilever structural walls.

Particular attention must be paid to the fact that the earthquake induced axial load intensity in these walls can be very large.

Limited number of theoretical studies of the nonlinear dynamic response of coupled shear wall structures [33] indicated that as in cantilever, the dynamic moment demand along the height of one wall is likely to exceed the intensity predicted by an elastic analysis for code required [1, 2] lateral static load. It seems advisable to curtail the longitudinal reinforcement to correspond with a linear moment variation between the base at the top of the structure.

As was suggested for cantilever structural walls, the shears derived from code prescribed loading should be magnified in order to recognise the increased dynamic shear demand, particularly in the lower parts of the structure. This was to preserve the ductile energy dissipating mechanisms in these structures. It appears to be justifiable to retain this procedure also for coupled shear walls.

It may be easily shown that the shear strength of the plastic hinge zone in a wall, subjected simultaneously to large axial tension, will greatly diminish. Indeed the very small flexural compression zone at the base of such a wall would not be capable of transmitting significant shear in friction. Fortunately shear failure in one wall cannot occur alone and therefore shear redistribution, particularly at 1st floor level, can take place. Therefore the compression wall can attract the bulk of the external applied shear force. This indicates that coupled shear walls must be amply reinforced for shear in the potential plastic hinge zones.

The treatment of horizontal construction joints should be the same as for cantilever structural walls, discussed previously.

The proportioning of the coupling system should be such that the major part of the overturning moment is resisted by the internal axial forces (shown as T in Fig. 18 and Fig. 19). This will not only lead to efficient use of reinforcement but it will assure that a large if not a major part of the required energy will be dissipated in the coupling beams. These are repairable and non-essential in carrying gravity loads.

With a suitable selection of stiffnesses it is possible to ensure that the majority of coupling beams will need to yield before the onset of yielding at the base of the walls. This desired response was also observed and predicted in a model structure studied at the University of Illinois [34]. This sequence assures that the walls, though more difficult to repair, will benefit from a greater degree of protection during moderately strong seismic disturbances. The New Zealand Loading Code [2] recognises this by requiring coupled shear walls to be designed for the same base shear as an open ductile rigid jointed frame, both having the same fundamental period, provided that the lT component of the internal moment of resistance, shown in Fig. 19, is at least two thirds of the external overturning moment.

Some additional and specific design proposals are made in the draft recommendations.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

DESIGN OF R/C FRAME-WALL STRUCTURES

discussed by

Toshikazu Takeda
Manager, Structural Section
Engineering Research Institute
Ohbayashi-Gumi, Ltd.

INTRODUCTION

On observation of actual low- and medium-height concrete buildings, even buildings which are purported to be pure frames are seen to have reinforced concrete walls around cores without exception, and in general, walls are provided in fairly random manner, while this is often ignored in the process of structural design. It is quite a problem to painstakingly perform elastic stress analysis of a structure having walls, while on the other hand, it is extremely difficult to properly carry out elastic stress design because of factors such as stress concentrations produced due to high shearing rigidities of walls and undetermined foundation rotation rigidities, and moreover, a question remains whether a building designed in such a manner will be sound.

In the recently published "Earthquake Loads and Earthquake Resistance of Building Structures" [1] examinations are made of the behaviors of reinforced concrete structures under earthquake loads (Method No. 1). This is one way of checking in regard to seismic resistance of buildings subjected to earthquakes of the maximum magnitude where all stories yield almost simultaneously. In effect, a standard velocity response spectrum is first given as the velocity response value for examination, upon which type of ground, ratio between period of building and critical period of ground, regional reduction, damping of the building, etc., are considered. The design shear force is obtained by modal analysis — root mean square method — to which the concept of energy equivalence indicated by Newmark concerning one-mass system displacement response having elastoplastic hysteresis is applied to obtain ductility factor and story deflection and to examine the earthquake resistance, and the ultimate strength of the building becomes involved in this case. Further, regarding the period of the structure, it is pointed out that determinations should be made with the yield seismic coefficient as the parameter considering the restoring force characteristic peculiar to reinforced concrete. In any event, resistance of a building to a major earthquake cannot be considered irrespective of ultimate strength.

The earthquake resistance of a structure having walls consisting of strength, deformation properties, energy absorption properties, etc. is greatly influenced by the boundary frames connected to the walls in addition to the properties of the walls themselves including their configurations. In particular, not much can be expected of tensile failure and compressive plasticity of concrete for plastic energy absorption in reinforced concrete structures, and it is thought that the energy absorption properties due to plasticization of reinforcing steel are great, and moreover, stable. At present, when stable yielding in shear of walls cannot be obtained with structures not having large wall quantities, it is thought bending failure of boundary beams or walls of bending failure type should be actively promoted.

This paper, based on the above background and recent research situations and the results thereof, discusses matters such as the ultimate strengths of frames which have walls.

PRESENT DESIGN METHODS

In earthquake-resistant design of ordinary structures in Japan today, computations of member cross sections are made in principle for the sum of earthquake stress and long-term load stress with seismic coefficient at 0.2 (increased by increments of 0.01 for each 4 m above height of 16 m) based on elastic rigidities of members. In case of design being based on the elastic theory, horizontal forces are concentrated at lower stories because of high shearing rigidities if there are shear walls so that shear stresses are increased, while uplift of foundations is apt to occur.

When a high shear stress is anticipated, there are cases in which designing is done taking into account reduction in shearing rigidities of walls. And, when loads are carried in exact accordance with elastic theory, the shear walls would bear the greater part of the horizontal forces, and considering that the strengths of frame portions are lowered, designs are often made so that horizontal forces suitably increased in proportion are carried by structural planes of frames.

The designs of members for bending stresses are according to the allowable stresses of materials and the straight line theory. With respect to shear design, allowable shear forces are specified for beams, columns and walls, and designing is done so that design shear forces are not exceeded. In this case, it is permissible for the design shear forces for beams and columns to be taken as the stresses for bending yield load of a frame including these members or 1.5 times the shear force during horizontal loading. It is said that, in principle, designing should be done in a manner that members will not show shear failure even if the frame reaches a state of collapse, but there is no special mention regarding external force distribution and ultimate strength of a structural body having walls and a distinct design philosophy has not been established.

SURVEY OF PAST RESEARCH

Strengths and Deformations of Beams and Columns

Eq. (1) has been proposed for ultimate shear force Q_u of a beam or column based on the results of many experiments [1], and this is Arakawa's formula to which 0.1 N/bh has been added inside the brackets.

$$Q_u = \left\{\frac{0.092\ k_u \cdot k_p\ (180 + F_c)}{M/Qd + 0.12} + 2.7\sqrt{P_w f_{wy}} + 0.1\ N/bh\right\} bj \qquad (1)$$

where

- b : width of member (cm)
- j : distance between resultant compression force and tension force (cm)

k_u : correction factor based on cross-sectional dimensions
$= 0.72$ ($d > 40$ cm)
k_p : correction factor based on tension reinforcement ratio
$= 0.82\ P_t^{0.23}$ ($P_t = 100\ A_s/bd$)
F_c : concrete strength (kgf/cm²)
M/Qd : shear-span ratio
P_w : area of shear reinforcement within a distance s, shear reinforcement spacing, divided by $b \cdot s$
f_{wy} : yield strength of shear reinforcement (kgf/cm²)
N : axial load to be taken as positive for compression (kgf/cm²)
h : overall thickness of member (cm)

For ultimate flexural strength of a column the following equations are proposed [1].

$$\left.\begin{aligned} M_u &= \{0.8A_s f_y h + 0.12bh^2 F_c\}\left(\frac{N_{max} - N}{N_{max} - 0.4\ bhF_c}\right), \\ &\quad \text{for } N_{max} \geq N > 0.4bhF_c \\ M_u &= 0.8A_s f_y h + 0.5Nh\ (1 - N/bhF_c), \text{ for } 0.4bhF_c \geq N \geq 0 \\ M_u &= 0.8A_s f_y h + 0.4Nh, \text{ for } 0 > N \geq N_{min} \end{aligned}\right| \qquad (2)$$

where, $N_{max} = bhF_c + (A_s + A_s')f_y$, $N_{min} = -(A_s + A_s')f_y$

A_s : area of tension reinforcement
A_s' : area of compression reinforcement
f_y : yield strength of reinforcement

For determining approximately the deformation of a member beyond elasticity there is the method based on inelastic rigidity [2]. With the relation between end moment and rotation angle of a member subjected to inverse symmetrical moment as αS (S: elastic rigidity), α is obtained by the following equation:

$$\left.\begin{aligned} 1/\alpha &= 1 + (1/\alpha_y - 1)(1 - M_{cr}/M)/(1 - M_{cr}/M_y) \\ \alpha_y &= (0.043 + 1.64\ nP_t + 0.043a/D + 0.33N/bh)(d/h)^2 \end{aligned}\right\} \qquad (3)$$

where
M_{cr} : cracking moment
M_y : yield moment
n : modular ratio $= E_s/E_c$
P_t : A_s/bh
a/D : shear-span ratio
d : distance from extreme fiber to centroid of tension reinforcement

For exact calculations, elastoplastic bending deformation of the member, additional deformation due to pull-out of main reinforcement at the end of the member, and deformation of the beam-column connection panel may be considered. Suitable data do not exist with respect to shearing deformation, but for long columns it should be permissible to consider them as elastic.

Strength and Deformation of Wall

Shear behavior of plane shear wall -- Experiments on plane isolated walls have been carried out in large number from the past with emphasis on shear strength. Just as the so-called shear-span ratio M/QD influences ultimate strength in the case of a beam, it cannot be ignored in the case of a wall either. Hirosawa et al., have examined shear strength based on the results of many experiments modifying Eq. (1) [3], [4].

Meanwhile, Sugano, with respect to the results of experiments on so-called shear of walls showed that calculated shear force in resisting bending and experimental shear strength were fairly close to each other when wall reinforcement was disregarded and only column reinforcement was considered as effective [5], and this is a point which will require great care in order to cause stable bending failure of the wall.

Regarding deformation of a wall at maximum load, Hirosawa [4] indicated the frequency distribution of rotation angles of walls at maximum load from the results of experiments on 175 specimens as shown in Fig. 1. As has been said from the past, it is thought the shear deformation angle is around 4×10^{-3}.

Okada reduced the shear-deformation relationships of walls to a model in tri-linear form as shown in Fig. 2 [6]. This was a slightly conservative evaluation of strength based on the shearing rigidity reduction rate equations of Kokusho and Tomii, and on the relationship between shear cracking strength τ_c and shear resistance τ_u indicated by Sugano. The points at which shear cracks are produced correspond to shear stress of $\tau_c = 0.1F_c$ and shear deformation of $\gamma_c = 0.2 \times 10^{-3}$ in the equations of Kokusho [7] and Tomii [8].

Further, Hirosawa has suggested that if $F_c < 300$ kgf/cm^2, around $\tau_c = 0.1F_c$ would be satisfactory as a simplified equation [3]. There is still room for consideration concerning shear strength, however.

Shiga et al., conducted dynamic shear failure experiments of walls and concluded that except for the first application of load the hysteresis loops of repetitive stressing become extremely small and are about 4% in terms of equivalent viscous damping constants [9].

FEM analysis of walls on which efforts have recently been made in many quarters is an effective tool for clarifying the relationship between strength and deformation, but it is felt this has not progressed to the extent of predicting shear failure modes or failure produced immediately after bending yield.

Bending failure type wall -- While the resistance of a wall to bending can be computed by the straight line theory, the following approximate equation gives results which are in accordance roughly with experimental values [1].

$$M_u = A_s f_y \ell + 0.5\Sigma(A_{wh} f_{wy})\ell + 0.5N\ell \qquad (4)$$

where

b : width of column on compression side

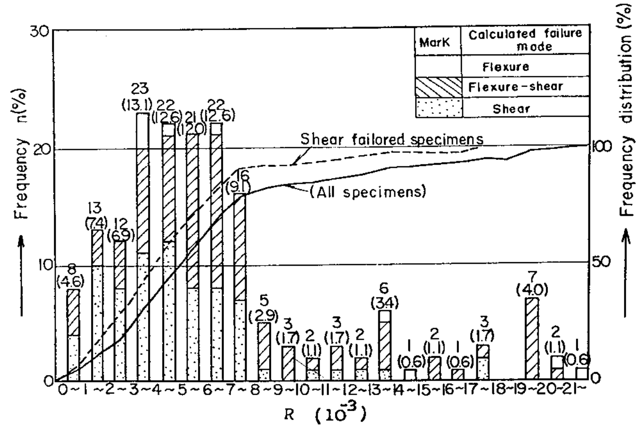


Fig. 1 ROTATION ANGLE OF WALL AT MAX. LOAD

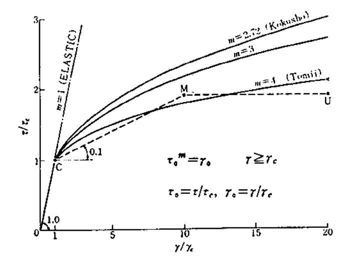


Fig. 2 NORMALIZED SHEAR STRESS τ - DEFORMATION γ RELATIONSHIP

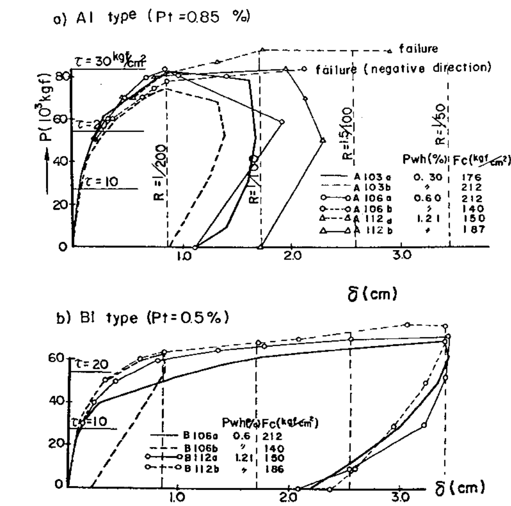


Fig. 3 ENVELOPE OF LOAD-DEFLECTION

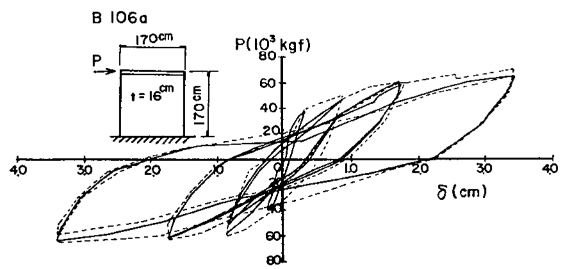


Fig. 4 LOAD-DEFLECTION CURVE

ℓ : length of wall (distance between column centers)
A_{wh} : total cross-sectional area of vertical bars in wall
f_{wh} : yield strength of vertical bars in wall

Hirosawa indicated from comparisons of shear force during bending resistance and ultimate shear resisting capacity that a relatively stable restoring force of bending yield precedent type is obtained for a wall of rectangular section and $M/QD = 1$. Examples are given in Figs. 3 and 4. It is pointed out in this case that the average shear stress of walls should be about 30 kgf/cm^2 or under.

Box type wall, others -- In actual buildings, walls around cores often are in box form, but there have been extremely few experimental studies on this shape. Umemura, Aoyama et al. [11], conducted experiments on box-type and cylinder-type walls and developed a method of calculating ultimate shear force. In essence, the effective cross-sectional area of a web is determined from the neutral axis location during bending resistance, and this is multiplied by the ultimate shear stress by Arakawa's formula applied for beams, and it is indicated that bending failure and shearing failure types can be distinctly separated.

Higashi, Okubo and others carried out experiments on columns having small walls on both sides and deduced an equation for ultimate shear strength and an approximate equation for ultimate flexural strength [1].

Elastoplastic Behaviors of Frame-Wall Structures

There are comparatively few cases of experiments having been made on frame-wall structures. Recently, experiments were individually made on the structural planes of a 2/3-scale model of the bottom two stories of a 5-story apartment house building [12], and the average ultimate shear stress of structural planes having openings was 16.9 kg/cm^2, failing at a relatively low value perhaps due to existence of the openings.

Matsushima applied horizontal load to the 5-story apartment house full scale, and converting the entire framework into line members, calculations were made through tracing of hinges [13].

Nakayama, Eto et al., conducted horizontal loading experiments on a three-story, three-span structure (scale, 1/2) having walls at the two end spans, while in calculations the walls were split into elements perpendicular to the wall axes to make comparison studies through analyses giving the moment-curvature relations and reductions in shearing rigidity of the elements. Further, beams all had tri-linear type restoring forces of hinges on analysis by the method of tracing hinges [14].

Sonobe, Imai et al., conducted experiments on a 4-span, 5-story steel and reinforced concrete structure with walls arranged in checkerboard fashion [15], and showed that with walls arranged in this manner there are wall portions produced which do not carry any shear forces due to tensile forces of columns because of overall bending.

Although a steel frame, Shotaka, Omote et al., took a 3-span, 30-story structure, and converted pure frame structural planes into multiple-mass shear-type elastoplastic systems and structural planes having walls at centers into direct elastoplastic frame models, and assuming that the horizontal displacements of the various structural planes were identical, carried out elastoplastic earthquake response analyses [16].

Kuno and Okada took a parallel structure of a frame represented by degrading tri-linear type hysteresis in structures and walls having restoring forces of original point-oriented type which have no resisting capacity after critical deformation, carried out response calculations of one-mass systems changing the wall ratio, and pointed out that it is necessary for frame strength to be decided in accordance with the plasticity ratio required from response results of the frame only when there is a possibility of the wall failing [17].

Ultimate Strengths of Frame-Wall Structures

Okada and Bresler took out walls including boundary beams and determined ultimate resisting capacity applying lateral forces [18].

Aoyama and others have proposed a method of obtaining ultimate strength including walls [1], and one example is indicated in somewhat detail in Figs. 5 through 8. The structure was a 3-storied building of 3 spans transversely and 5 spans longitudinally with external forces distributed in the form of triangles for the frame structural planes A and D and structural planes B and C containing walls, all four being in the longitudinal direction.

Regarding the frames A and D, the sizes of the sums of yielding moments M_y of beams and columns at the various joints were compared, and assuming that hinges are produced at the smaller sides, the principle of virtual work was applied to determine external force. Methods not based on virtual work were also studied, for example, a case of hinges produced in beams and no hinges produced in columns above and below the joints was taken, and distributing joint moments to the columns above and below, it was concluded that in general the former is superior.

With respect to the B and C frames containing walls, the effects of orthogonal frames connected to the walls were taken up in the form of yielding of boundary frames, and the ultimate resisting capacity at bending yield of the walls and at uplift were obtained from the principle of virtual work, and the lesser was taken as the ultimate resisting capacity. In case of bending yield of walls, foundation beams will not have yielded, while when there is uplift, orthogonal frame foundation beams will have yielded.

The ultimate resisting capacity of the structure will be the sum of the ultimate resisting capacities of the frames A and D, and B and C. In this case, the ultimate resisting capacities were the values given below, indicating that the ultimate resisting capacity of a frame having walls can be considerably greater than in the case of a frame.

Ultimate shear strength of one story in frames A and D
0.767×10^5 kgf
Ultimate shear strength of one story in frames B and C
6.036×10^5 kgf at uplift of wall

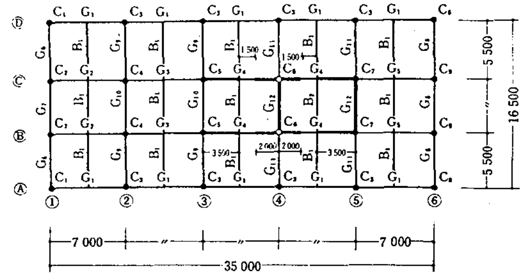


Fig. 5 PLAN

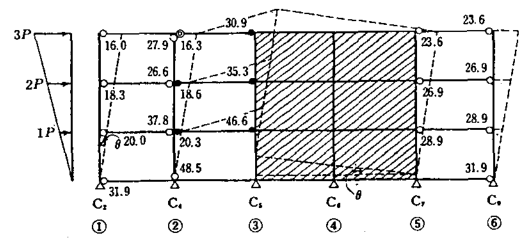


Fig. 7 B, C FRAME: WALLS YIELD

A,D FRAME

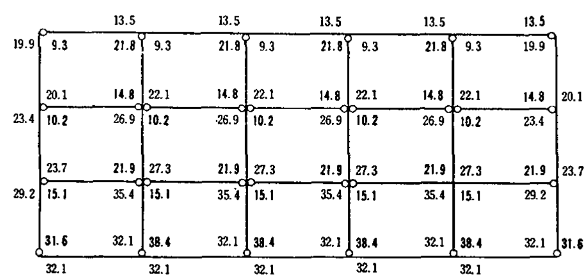
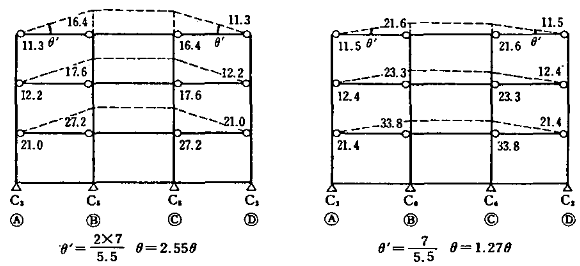


Fig. 6 YIELD MOMENT (10^3kgf m) AND MECHNISM



3 FRAME

4 FRAME

Fig. 8 YIELD MOMENT AND MECHANISM OF PERPENDICULAR FRAMES WHEN WALLSIN B, C FRAME YIELD

Ultimate shear strength of one story in frames B and C
7.416×10^5 kgf at bending yield of wall

The shear force applied to a wall is obtained with external force distribution on walls having boundary beams given as triangle distribution. This method is that which seeks the kinematically admissible state as mentioned in limit analysis and points out that the evaluation is on the risky side for ultimate strength at real collapse, or that with frames of collapsing type conditions of external instability and internal indeterminateness, moments cannot be obtained unless shear force distribution is assumed for each individual column.

PROPOSALS AND DISCUSSIONS ON DESIGN OF FRAME-WALL STRUCTURES

In case of low- and medium-height structures where there are walls in large quantity in random fashion, it is thought permissible to perform so-called strength design in which external forces are resisted by the total sum of wall and column quantities. In doing so, such items as structural planning including the plan arrangement of walls and continuity in the vertical direction, material strength, quantities and placement arrangements of main and hoop reinforcing bars, ratios between resisting capacities of ductile frames and brittle walls and short columns, boundary beam effects, etc. should be taken into account in evaluating the ultimate strength of a building. This would mean introducing the method of judging earthquake resistance of existing buildings which is actively being studied in Japan of recent to the stage of design in reverse.

Ultimate Strength by Limit Analysis and Design Shear Force of Wall

In general, in cases where there are not many shear walls, it is thought effective for design of frame-wall structures to consider ultimate strengths of the entire buildings in accordance with the method of virtual work described earlier. The method of absorbing vibration energy entering a structure by rotation of yield hinge, or, hysteresis energy, is quite excellent as a design method. In addition, since the hinge location can be predicted and that part reinforced, it may be considered as a superior design concept in this sense also.

The upper bound theorem in limit analysis was applied in the earlier case. Naturally, it would be better to evaluate the total external force applied to the building adding the lower bound theorem according to the statically permissible state. There are various techniques of limit analysis and the following may be said to be one method.

(1) The distribution pattern of total external force of a building is determined and the ultimate load is calculated by computer [19] or by hand. As external force distribution, a triangle distribution is practical, but it is advisable to refer to elastic response results obtained with various earthquake waves or to literature. As a natural consequence, it is desirable for external force mode and collapsing mode not to be very different, and for response modes from elastic to plastic to be similar.

(2) Generally speaking, in an ordinary structure, the plastic mechanism is internally indeterminate and shear design of walls becomes impossible.

Therefore, moment distributions of the columns of the bottom story are determined in a manner that shear forces of the walls at the bottom story will be maximum without destroying the plastic mechanism. This, for example, is as in Fig. 10, when yield of a column base and beam of the first story is in the state of the assumed plastic mechanism, the moment of a second-story column base is taken at the yield value as in the figure, this is made to be propagated to an upper story, and if yielding moments are not exceeded at the various parts, the shear force obtained from the moment distribution is the minimum for this column. If the yielding moments are exceeded, the moment distribution of the column at the bottom story is changed and a similar procecure is repeated.

When there is a plural number of walls, the maximum shear force of the bottom story which it is possible to be applied to those walls is obtained in the same manner as above, following which individual walls are taken out including boundary beams, external force distribution is applied to each, and design shear forces of the respective walls are determined in accordance with the shear force ratios.

With joints causing the mechanism to be internally indeterminate, for example, where beams yield but the columns above and below do not, and the difference between the sum of the yielding moments of the columns and the sum of the yielding moments of the beams at the joint is small, the design shear force of the wall will not be very excessive, but on the other hand, the columns should be designed to possess adequate deformation capability.

Inelastic Analysis

Failure modes can be understood through limit analysis, but stress distributions such as moments of the various parts, and the load-deformation relationships as a whole are obscure. In order to learn about the deformation properties from elasticity to plasticity and the ultimate force distribution, it will be sufficient to carry out calculations appropriately preparing models of the structural bodies based on bending crack loads, bending yield moments, and especially with walls, shear cracking loads, ultimate shearing loads and the corresponding deformations.

It would be optimum if such factors as deformations at joints between columns and beams, and additional deformation due to pull-out of reinforcement at member ends were to be considered, but it should be permissible for these to be disregarded for the purpose of designing. In this sense, although depending on stress conditions, it should be possible, for example, to consider expediences such as performing calculations using elastic rigidities for columns and cracking rigidities for beams from the beginning.

Design

When the proportion of ultimate strength of a wall in the ultimate strength of a structure is extremely large, the design shear force of the shear wall can be the total lateral force, but with further examination as mentioned above, the design shear force for the shear wall can be made smaller.

For shear reinforcement of a portion where yield hinge is anticipated, it will be necessary at this stage to provide ample allowance in reinforcement, not taking average shear stress at a very high level.

With regard to beams and columns also, it is important to reinforce portions where yielding is expected in a manner that energy absorption will be provided.

CASE STUDIES

Limit Analysis

Fig. 9 shows a 7-story, 3-span building which is a frame with a fixed foundation having story height and span of 3 m and 5.4 m respectively, and a wall at the middle. Wall thickness is 20 cm, columns are 60-cm squares, tensile reinforcement ratio P_t = 0.7%, while beams are 40 x 70 cm with P_t = 0.9%. The stresses N/bh in columns at the bottom story due to axial forces are 40 kgf/cm^2 and 50 kgf/cm^2 for exterior and interior columns, respectively. These stresses are reduced in the direction of height in accordance with story number.

Assuming external force to be triangularly distributed and using the principle of virtual work, the maximum shear forces of the first and second stories will be obtained as 2.72×10^5 kgf and 2.62×10^5 kgf, respectively. The mechanism, excepting column heads of exterior columns at the top story, and column and wall bases at the bottom story, is that of yielding of beams in all cases, but the stresses are unknown because the mechanism is internally indeterminate.

If here it is chosen for the shear force applied to the wall of the first story to be maximum, in effect, for the shear forces carried by the columns of the first story to be minimum, the maximum shear force applied to the wall will be ${}_1Q_{wp} = 2.40 \times 10^5$ kgf, meaning that about 88% of the entire shear force is carried by the wall. Following the same procedure for the second story, ${}_2Q_{wp} = 2.80 \times 10^5$ kgf, which is larger than for the first story. This happens because the shear forces on the columns of the second story can be in the opposite direction from external force. The figures in parentheses in Table 1 are for cases of negative shear produced being disregarded. Furthermore, it has been ascertained that the structure in this case is in a statically permissible state.

The discussion above has been on one frame possessing a wall. The results for a structure where one or two rows of 3-span frames having columns identical to the exterior columns of the above frame are arranged parallel to the frame-wall are indicated in Table 1. It is assumed that horizontal displacements of all of the frames are equal and external force of triangular distribution is applied to the entire structure. In this case, Q_{wp} becomes larger as the number of frames, in effect, the number of columns becomes larger.

In the case of Fig. 5, if it is assumed that the wall shear force of the first story obtained by the method described earlier is ${}_1Q_{wp} = 5.743 \times 10^5$ kgf and loads of triangular distribution are applied to frames A and B, then ${}_1Q_{wp} = 5.732 \times 10^5$ kgf, while when only the wall including boundary beams is taken out and a triangularly distributed load is applied, ${}_1Q_{wp} = 5.58 \times 10^5$ kgf.

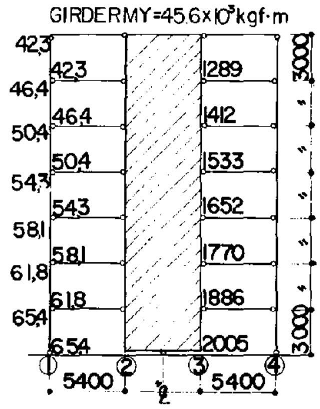


Fig. 9 YIELD MOMENT (10^3kgf m) AND MECHANISM

unit ($\times 10^3$ kgf)

Story		W	W	W+F	W+2F
7	Q_T	69*	68	102	137
	Q_{wp}	22*	12	66	143
6	Q_T	129*	126	190	255
	Q_{wp}	102*	128 (126)	136 (132)	145 (139)
5	Q_T	179*	175	263	352
	Q_{wp}	148*	181 (175)	221 (209)	262 (244)
4	Q_T	218*	213	322	431
	Q_{wp}	188*	225 (213)	270 (246)	315 (279)
3	Q_T	248*	243	366	489
	Q_{wp}	215*	259 (243)	354 (322)	449 (401)
2	Q_T	268*	262	394	529
	Q_{wp}	238*	284 (262)	398 (354)	515 (449)
1	Q_T	278*	272	409	548
	Q_{wp}	214*	240	281	324

( ): negative shear is neglected
Q_T: Total shear Q_{wp}: shear for wall
*Qmax by inelastic analysis

W W+F W+2F

TABLE 1 PROBABLE MAX. SHEAR Q_{wp} OF WALL AND TOTAL SHEAR Q_t

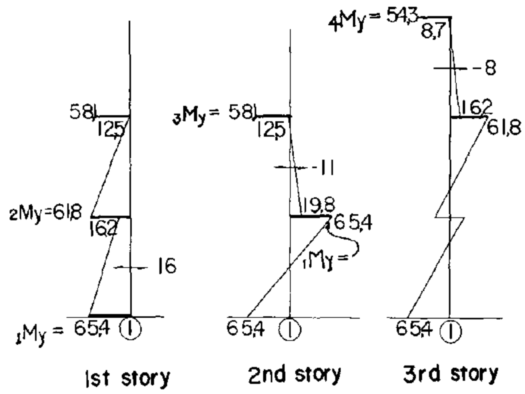


Fig. 10 PROBABLE MIN. SHEAR OF EXTERIOR COLUMNS

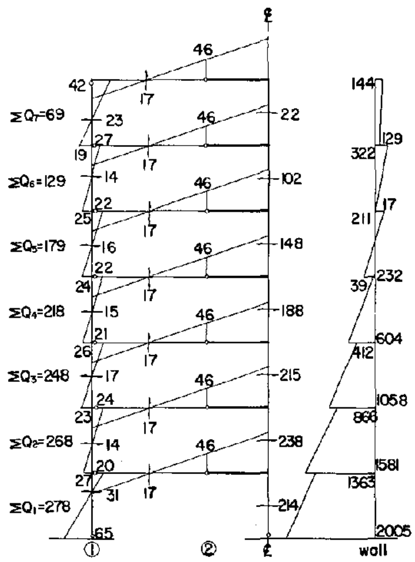


Fig. 11 MOMENT (10^3kgf m) AND SHEAR (10^3kgf) AT ULTIMATE LOAD (INELASTIC)

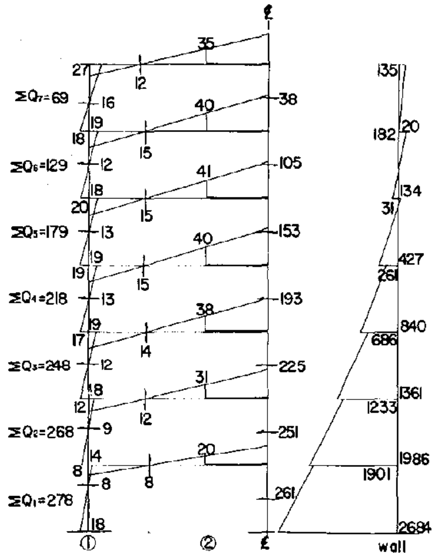


Fig. 12 MOMENT (10^3kgf m) AND SHEAR (10^3kgf) AT ULTIMATE LOAD (ELASTIC)

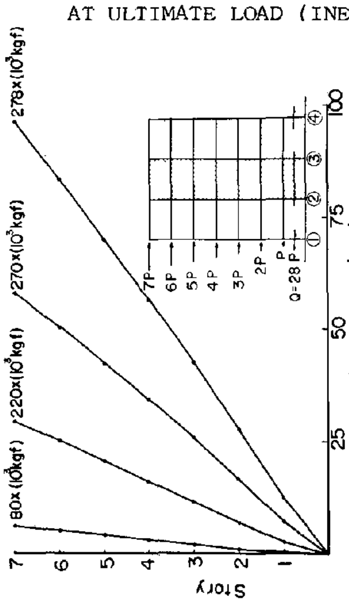


Fig.13 DISPLACEMENT MODE (INELASTIC ANALYSIS)

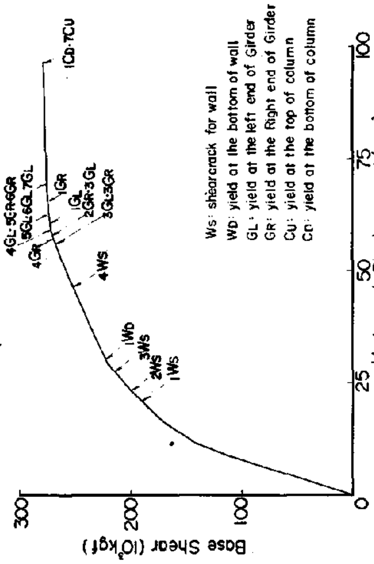


Fig. 14 LOAD-TOP DISPLACEMENT (INELASTIC ANALYSIS)

As indicated above, it is unavoidable for excessive shear force to be considered if shear design of a wall is to be done through simple calculations only. Problems will remain in case frames with large differences between column moments and beam moments at joints are included.

Inelastic Analysis

The results of inelastic analysis in case of applying triangular distribution external force to one structural plane of the frame-wall of Fig. 9 are indicated in Figs. 11 through 14 and Table 1. This method of analysis divides a member into a number of elements in a direction orthogonal to the axis of the member and gives a tri-linear moment-curvature relationship considering cracking and yielding to each of the elements for calculation of bending deformation, but does not consider inelastic axial deformation as a structure and variations in axial forces of columns caused by lateral forces.

Regarding shearing rigidity of the walls, it was assumed that rigidity was decreased at the point of average shear stress equal to $0.08F_c$(F_c = 240 kgf/cm^2) and that this rigidity was 1/10 of elasticity. The shear force Q_{max} in plastic mechanism according to this inelastic analysis shows a difference of about 2% as indicated in the table, but this is due to a slight amount of rigidity considered after yielding. The shear force at the wall of the first story is 2.14×10^5 kgf which is about 10% smaller than the previously-mentioned ${}_1Q_{wp} = 2.40\times10^5$ kgf.

The moment distribution at the time of plastic mechanism according to inelastic analysis and the results of elastic analysis under the same external force are indicated in Fig. 11 and Fig. 12. Compared with the results of elastic analysis, the moments are approximately 4 times greater at the first-story column, approximately 1.5 times at the top story column, and 0.75 times at the wall base. Fig. 13 shows displacement modes according to load level, where bending modes are predominant in the elastic range, but a linear trend is indicated as load is increased to become close to triangular distribution.

On looking at the load-displacement curve for the top-story floor of Fig. 14, it may be seen that the wall base first yields and plastic mechanism is about reached at displacement approximately double that at the time of yielding of the wall base.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

A PRACTICAL METHOD TO EVALUATE SEISMIC CAPACITY OF EXISTING MEDIUM- AND LOW-RISE R/C BUILDINGS WITH EMPHASIS ON THE SEISMIC CAPACITY OF FRAME-WALL BUILDINGS

by

Hajime Umemura
Professor and Dean
Faculty of Engineering
University of Tokyo

Tsuneo Okada
Associate Professor
Institute of Industrial Science
University of Tokyo

INTRODUCTION

In order to develop a practical method to evaluate the seismic capacity of existing medium- and low-rise R/C buildings, a specific committee sponsored by the Ministry of Construction of Japan was established in July 1976.* A method was proposed in April 1977 [1].

The primary purpose of this paper is to describe the basic concept of the method with emphasis on the unified seismic index to evaluate the seismic capacities of ductile moment-resisting frame, shear wall, and wall-frame buildings.

SEISMIC INDEX OF STRUCTURES

The seismic index of structures (I_s) is used to evaluate the seismic capacity, which is calculated by Eq. (1) at each story and in each direction.

$$I_s = E_o \cdot G \cdot S_D \cdot T \tag{1}$$

where

E_o = basic structural index calculated by ultimate horizontal strength and ductility of structure

G = local geological index to modify the E_o-index

S_D = structural design index to modify the E_o-index due to the grade of the regularity of the building shape and the distribution of stiffness and strength

T = time-dependent index to modify the E_o-index due to the grade of the deterioration of strength and ductility

The overall method consists of three independent methods: first, second, and third evaluation methods, for the engineers to be able to choose any of them. The first evaluation method is the simplest, but least reliable, of the three, while the basic concept is common for all three.

BASIC STRUCTURAL INDEX (E_o)

Since the G-, S_D-, and T-indices are the modification factors less than or

*The first author served as the chairman, and the second author was Chairman of the Task Committee assigned to make draft proposals.

equal to 1.0 and the E_o-index usually predominates, the outline for evaluating the E_o-index is described here.

The E_o-index consists of the strength index (C), the ductility index (F), and the story index (β). The evaluation starts from categorizing the failure type of each column and wall. The types of failure used for the first, second, and third evaluation methods are shown in Tables 1 and 2. Then, all columns and walls at each story level are assigned into one of three groups; Group-1, Group-2, and Group-3, according to their F-indices shown in Tables 1 and 2. The minimum value in a group is assumed as the F-index of the group and the F-index of Group-1 should be the smallest. The number of the groups should not be more than three, and the smaller the number the better.

The C-index of each group is calculated by Eq. (2).

$$C_j = \sum Q_j \Big/ \sum_i^n W_k \tag{2}$$

where

$\sum Q_j$ = story shear of Group-j at ultimate stage

W_k = weight at k-th story level

n = total number of stories

i = story level under consideration; i=1 designates first story

The E_o-index is calculated either by Eq. (3) or Eq. (4) according to the adopted criterion.

$$E_o = \beta \sqrt{E_1^2 + E_2^2 + E_3^2} \tag{3}$$

$$E_o = \beta \cdot (C_1 + \alpha_2 C_2 + \alpha_3 C_3) \cdot F_1 \tag{4}$$

where

$E_j = C_j \cdot F_j$

β = (n+1)/(n+i). For particular case in the third evaluation method, β becomes 2(2n+1)/3(n+i)

α_2, α_3 = values given in Table 3

Equation (3) is used when the seismic capacity is determined by the failure of the group having the largest F-index and Eq. (4) is used when the failure of the group having the smallest F-index (Group-1) determines the criterion.

COMMENTARY

A group of the single story buildings consisting of shear walls and ductile frames is used as an example. The shear wall is categorized as Group-1 and the ductile frame is Group-2. Since the third group does not exist, the

E_3-index and C_3-index in Eqs. (3) and (4) become zero. The relationship of the story shear and the story drift of the buildings is assumed as illustrated in Fig. 1. A quarter of the circle in Fig. 2 shows Eq. (3), and the seismic capacities of the buildings on the line are considered equal in this method. The decision criterion is at the ultimate stage of the frames. The broken line in Fig. 2 shows Eq. (4), and the criterion is at the ultimate stage of the shear walls.

The results of the computer simulation [2] were used to assess the feasibility of Eqs. (3) and (4). The earthquake response of the structural models representing the wall-frame R/C single story buildings to the recorded ground motions [2] were expressed in the E_1-E_2 domain as shown in Figs. 3, 4, and 5. In the simulation, one-mass models supported on the nonlinear parallel spring system consisting of the origin-oriented hysteretic model, which represented the shear walls and the degrading trilinear model which represented the frames, were used. The variables were the strengths and the earthquake ground motions: El Centro 1970 (NS), Taft 1952 (EW), and Hachinohe 1968 (NS). The maximum ground accelerations were modified to 30% of the acceleration of the gravity. Since the ultimate displacement of the wall and the yield displacement of the frames were assumed constant, the initial natural periods of the systems were proportional to their strengths: 0.1 sec - 0.6 sec.

As recognized by the figures, the use of Eqs. (3) and (4) in evaluating the seismic capacities of the frame-wall R/C buildings seems feasible for practical purposes, while more detailed investigation is necessary to refine the method.

REFERENCES

[1] The Japan Special Building Safety Center Foundation, "Evaluation of Seismic Capacity of Existing R/C Buildings," April 1977.

[2] Hisano, M. and Okada, T., "Earthquake Response of R/C Frame-Wall Buildings," Proceedings of the A.I.J. Annual Convention, October 1975.

Table 1 Type of Failure and F-index for First Evaluation Method

Type	F-index
Column	1.0
Wall	1.0
Extremely Short Column	0.8

All columns and walls are assumed brittle members

Table 2 Type of Failure and F-index for Second and Third Evaluation Methods

Type	F-index	
Ductile Column-1 (Bending column)	1.27-3.2 1)	Second & Third
Ductile Wall-1 (Bending wall)	1.0 -2.0	
Brittle Column-1 (Shear column)	1.0	
Brittle Wall (Shear wall)	1.0	
Extremely Brittle Column	0.8	
Ductile Column-2 (Column in beam bending type of frame)	3.0	Third
Brittle Column-2 (Column in beam shear type of frame)	1.5	
Ductile Wall-2 (Wall fails in overturning)	3.0	

1) $$F = \frac{\sqrt{2\mu-1}}{0.75(1+0.05\mu)}$$

μ = Ultimate ductility factor

Table 3 The Values of α_2 and α_3 in Eq.(4)

First Group / Second and Third Groups	Extremely Brittle column	Brittle Column,or Brittle Wall
Ductile Column	0.5	0.7
Ductile Wall	0.7	1.0
Brittle Column or Brittle Wall	0.7	-

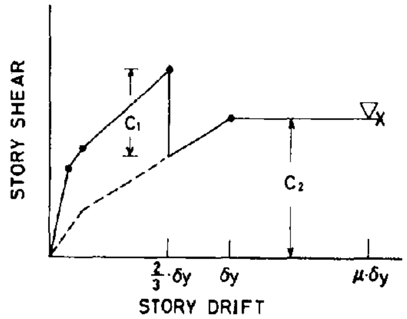


Fig.1 Assumed Story Shear vs. Story Drift

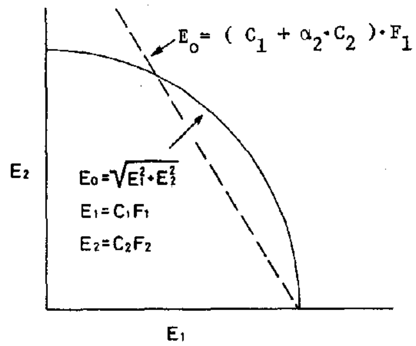


Fig.2 Seismic Capacity of Frame-Wall Buildings

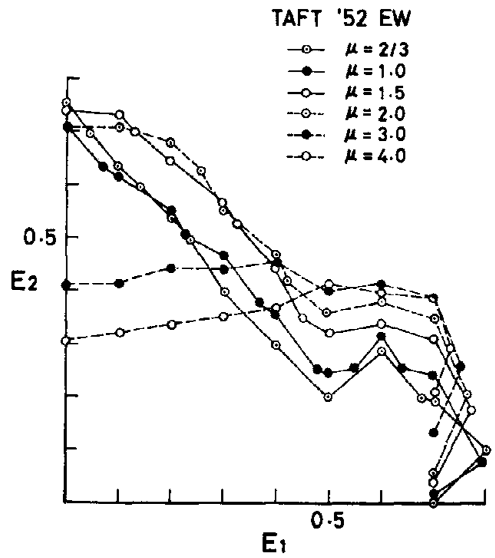


Fig.3 Earthquake Response vs. E_o-index (1)

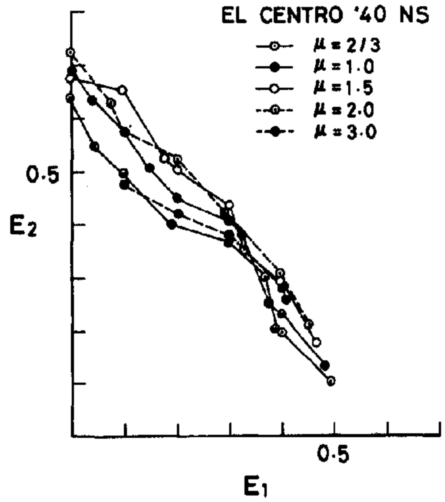


Fig.4 Eathquake Response vs. E_0-index (2)

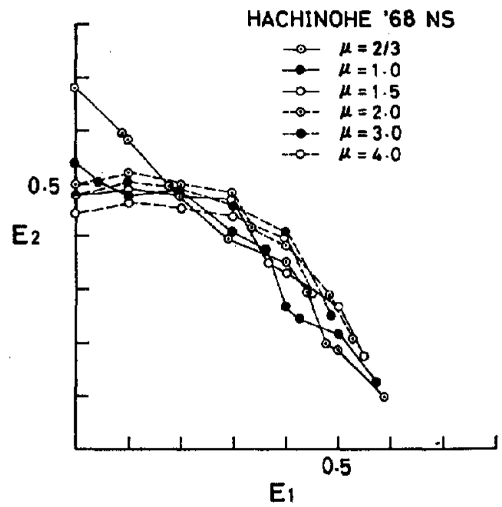


Fig.5 Earthquake Response vs. E_0-index (3)

SHEAR WALL RESEARCHABLE ITEMS

by

John F. Meehan
Principal Structural Engineer,
Research Director
Structural Safety Section
Office of the State Architect
State of California

Current research on reinforced concrete shear walls has been performed on test specimens where the vertical perimeter reinforcing has been in the form of wall tied columns and the shear wall horizontal bars are well anchored into the columns. This construction is usually found in high-rise construction. In most low-rise construction the perimeter of the shear wall does not terminate in an enlarged section such as a column but merely ends without any enlargement. Further, the horizontal bars seldom have any special treatment with respect to the vertical bars, i.e. the horizontal bars are not normally bent or hooked around the vertical bars. The influence of the perimeter conditions of shear walls should be clearly established.

In small buildings columns are frequently designed by taking a portion of the wall and reinforcing it to meet the requirements of columns. That is, a portion of an 8-in.-thick wall is reinforced like a column. The performance characteristics of such columns should be determined.

Current codes establish the "shear friction" concept to determine the allowable shear stresses across horizontal construction joints in shear walls. Horizontal construction joints have been shown to be weak and allow slip to occur in innumerable earthquakes. Competent construction techniques and deatils must be determined for various lateral load levels.

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

LABORATORY TESTS OF EARTHQUAKE-RESISTANT STRUCTURAL WALL SYSTEMS AND ELEMENTS

by

A. E. Fiorato
Assistant Manager
Structural Development Section
Portland Cement Association
Skokie, Illinois

W. G. Corley
Director
Engineering Development Department
Portland Cement Association
Skokie, Illinois

INTRODUCTION

Reinforced concrete walls are frequently used to provide lateral stiffness for buildings. Such walls can be used in a variety of building plan configurations. Selected examples are illustrated in Fig. 1. In particular, for tall buildings structural walls have become more important as buildings have become more slender and less massive. The walls are used to keep lateral drift within reasonable limits by resisting horizontal forces in the plane of the wall.

Although much of the development of structural wall systems can be related to design for wind forces, there is currently increased interest in the earthquake resistance of reinforced concrete walls. For severe earthquakes it is not practical to design tall buildings to respond elastically [27,50,53, 54]. Therefore, walls must possess ductility for energy dissipation. Much of the recent experimental research on earthquake resistance of structural walls has been directed toward evaluation of inelastic deformation capacity as well as load carrying capacity.

Objective and Scope

In this paper available laboratory tests of reinforced concrete structural walls are reviewed. Major findings are indicated and future research needs are identified. While the intent has been to be complete, it is probable that investigations have been missed.

Only laboratory tests with lateral loadings in the plane of the wall are included in this paper. Types of specimens include construction joints, coupling beams, isolated walls, coupled wall systems, and frame-wall systems.

Definitions

For many years the term "shear wall" has been used to describe reinforced concrete walls used as lateral load resisting elements in buildings. The term originated from the fact that horizontal shears are carried by the walls. The term has been misinterpreted to refer to behavior dominated by shear [50]. Except for short, wide walls, shear is seldom the critical mode. Therefore, the term "shear wall" is a misnomer [50,63]. For this reason, the term "structural wall" is becoming more common.

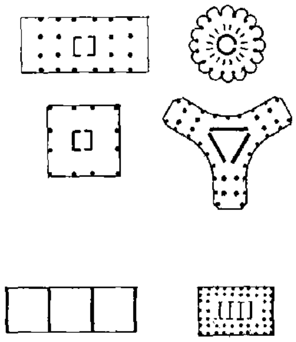


Fig. 1 Building Plans Illustrating Wall Layout

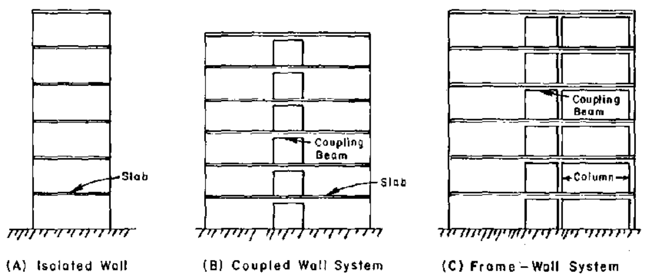


Fig. 2 Structural Wall Systems

Depending on the plan of the building, a structural wall may act individually as an isolated wall, together with one or more others in coupled wall systems, or with a frame in a frame-wall system. These types of walls are illustrated in Fig. 2. For a very small span-to-depth ratio of the coupling beams, the coupled wall system shown in Fig. 2b can approach the proportions of an isolated wall pierced by openings.

Tests on structural walls can be classified as those on elements, those on isolated walls and those on systems. Elements include tests of wall construction joints and coupling beams. In addition, an isolated wall can be considered as an element of a coupled wall or frame-wall system.

Tests of isolated walls generally fall into one of two categories: low-rise or high-rise walls. Low-rise walls with height-to-horizontal length ratios of one or less have been tested. High-rise walls with height-to-horizontal length ratios of two or greater have also been tested.

To date there are few tests on coupled wall and frame-wall systems.

LABORATORY TESTING

Background

Most of the research on structural walls has been carried out within the past 15 years. If the number of publications written per year is taken as a rough measure of the interest in this topic, some interesting results are obtained. Figure 3 is a plot of number of publications versus the year of publication based on several sources.

Two literature surveys of analytical and experimental publications on structural walls have recently been made at South Dakota School of Mines and Technology [64] and at the University of Toronto [71]. These surveys provide the data for two of the curves in Fig. 3. The third curve, covering only laboratory tests of reinforced concrete walls is based on the publications surveyed for this paper.

The total number of analytical and experimental publications increased rapidly between 1960 and 1972 and then began to decline. The number of papers on laboratory tests, however, really only started to increase in 1970. Apparently, the experimental effort has lagged the analytical effort.

The absolute numbers of publications can be argued, as can be seen by comparing the South Dakota and the Toronto surveys. However, the overall trends indicated are consistent.

Figure 3 does not give the complete picture with regard to laboratory tests of reinforced concrete walls. Tomii [76] has pointed out that as early as 1926 Dr. Tachu Naito reported on tests of walls carried out in Japan. Numerous tests followed in Japan, but it appears that many of these results have not been utilized by U.S. investigators.

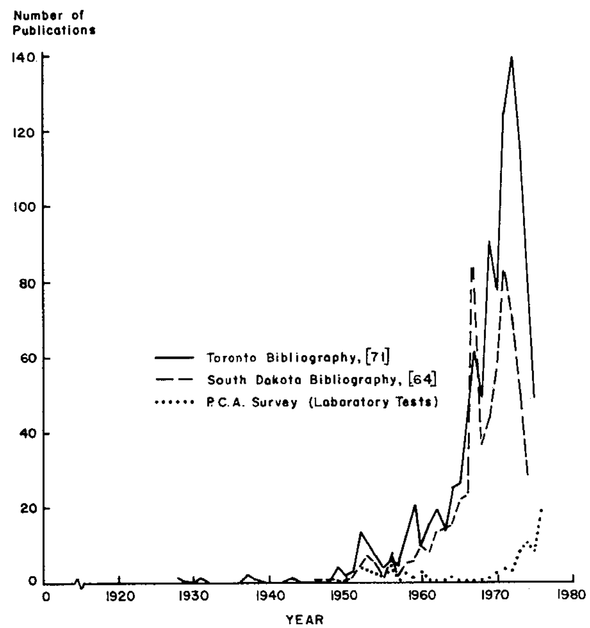


Fig. 3 Publications on Structural Walls

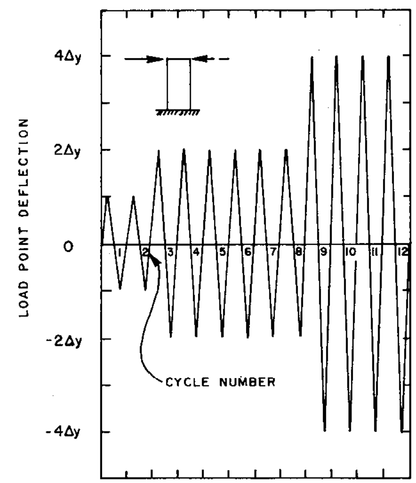


Fig. 4 Laboratory Loading History

Prior to 1966 the most significant experimental program in the United States was that carried out by Benjamin and Williams [9,10,11] in the 1950's. Their work consisted primarily of monotonic load tests of low-rise walls. Tests at MIT [3], also in the 1950's, included dynamic loads to evaluate blast resistance.

In April 1966 a symposium on "Tall Buildings with Particular Reference to Shear Wall Structures" was held at the University of Southampton (England). This was probably the first conference concerned solely with wall behavior and design. The conference proceedings [23] consist almost entirely of papers on elastic methods of analysis and on design and planning. This appears to have resulted because of an emphasis on wind loadings. The few experimental papers were on infilled frame research.

A number of investigations of infilled frames have been made since 1950. Most of this research has been on masonry infilled steel or reinforced concrete frames. This topic is not covered in this paper.

Subsequent to 1966, interest in structural walls as earthquake-resistant elements has increased in the United States. This interest has stemmed primarily from the generally good performance of walls in recent earthquakes. While research on elastic performance of structural wall systems had advanced considerably, research on inelastic performance had not. Experimental data was especially lacking. Consequently, inelastic behavior became an important topic for research.

Since 1966, major experimental programs on structural walls have been initiated at the Portland Cement Association [20,26], the University of Canterbury [50,54], the University of Illinois [47,74], and the University of California, Berkeley [15]. These programs are providing data on the strength, ductility, and energy dissipation capacity of reinforced concrete structural walls.

Laboratory tests of structural walls can be characterized by the type of applied loading. Two general types of loading have been used: static and dynamic [13,22,73].

Static Monotonic Load Tests

Test Method. Static monotonic load tests are carried out by applying load or deflection in increasing increments until the specimen is destroyed. Prior to 1970 most wall specimens in the U.S. were tested under monotonic loads.

Advantages. The primary advantage of the monotonic test is that it is simple and relatively inexpensive. It also represents, to some extent, a "control" or "reference" for the other types of tests.

Disadvantages. Monotonic response is not sufficient to describe the response of structures for earthquake type loadings [73]. A knowledge of the hysteretic response of the structure is essential.

Static Reversing Load Tests

Test Method. Static reversing load tests are carried out using load or deflection control. With the reversing load test, the specimen is subjected to a predefined series of load cycles so that the sequence from elastic behavior through inelastic behavior to destruction of the specimen can be monitored. This type of loading, though not as realistic as dynamic base motions, permits detailed evaluation of the hysteretic behavior of the test structure.

Advantages. Complete control of the loading sequence is possible. Because the test can be stopped, decisions about subsequent loading can be made based on the then current status of the test specimen. It is also possible to closely observe overall behavior, crack patterns, and the physical condition of the specimen at any time during the test.

Useful information regarding the hysteretic response of the structure is obtained from the test. For example, based on measured force-versus displacement curves, changes in the stiffness and energy dissipation characteristics of the test specimen can be determined. Generally, the data obtained provide a lower bound for strength and stiffness.

Disadvantages. Because loads are applied slowly, characteristics sensitive to the rate of loading are neglected. In addition, the application of the applied loads does not completely simulate the inertia forces on the structure.

The primary disadvantage to the reversing load test is related to selection of the applied load history. Since the results obtained are dependent on load history, they must be carefully interpreted. The common method of testing has been to subject the specimen to gradually increasing levels of force or deformation as illustrated in Fig. 4. Razani [65] has proposed a standard sequence of gradually increasing increments to be used for comparison of test results.

Another approach [22] has been to select a specific prototype structure and to model it in the laboratory. Then the loading history can be based on dynamic response analyses of the prototype. This approach is complex because the critical load combinations change with input motions, type of analysis, and time during the "earthquake".

The question that exists is whether hysteretic response based on static reversing load tests can be generalized. Japanese investigators [30,70] have been looking at this question. Experimental work is also underway in the United States [17,26].

Dynamic Earthquake Simulator Tests

Test Method. This test requires an earthquake simulator consisting of a servo-equipped hydraulic system, a power supply, a test platform, an electronic control and monitoring equipment [22,73,75]. Specimens attached to the test platform are subjected to base motions simulating particular earthquake records.

Advantages. A significant advantage of the earthquake simulator test is that the test structure responds to base motion, rather than an attached loading system. In addition to the monitored response data, the observed response of the test structure is of significant engineering value [73].

Disadvantages. The primary drawbacks of earthquake simulator tests are the cost of the test facility, both initial costs and life cycle costs, and limits on the size of test specimens.

TESTS OF CONSTRUCTION JOINTS

Lateral forces generated during earthquakes must be transferred across horizontal construction joints in walls. These joints must be capable of transmitting the shear under repeated reversals of loading.

Tests prior to 1960 relating to construction joint behavior have been reviewed by Hanson [29]. Hanson [29] also describes a series of "push-off tests" to evaluate shear transfer behavior under monotonic loading. Generally, his results indicated that a joint where the fresh concrete was cast against roughened dry concrete performed well. Keyed joints were no more effective than rough bonded joints.

Brook [18] has reviewed field practice, research, and code specifications for construction joints. His paper provides a useful summary of techniques for making satisfactory joints.

With regard to seismic performance, there are few tests of joints subjected to reversing loads. Significant work has been carried out by Mattock [41]; Paulay and Loeber [57]; and Paulay, Park, and Phillips [58]. Results of their tests are summarized below.

Construction Joint Behavior Under Reversed Cyclic Loading

The mechanism of shear transfer across a construction joint consists of several components. The most important are bond, aggregate interlock, and dowel action [41,58]. Bond is the "adhesive capacity" developed between the hardened concrete surface and the fresh concrete. A cracked joint is unbonded. Aggregate interlock is that mechanism resulting in shear transfer by interface friction and by direct bearing of particles along a crack. Dowel action results as slip along a joint is resisted by the reinforcement crossing the joint.

Mattock's tests [41] evaluated the effects of shear reversals on initially cracked monolithic concrete. The investigation was directed toward the behavior of precast concrete connections during earthquakes, but is also pertinent to construction joint behavior. Figure 5 illustrates selected shear versus slip curves obtained by Mattock. Development of pinching in the curves is evident after the initial cycles. This occurs as abrasion of the crack surfaces reduces shear transfer by direct bearing of interface particles. As loading continues more slip must occur to bring particles into contact. Abrasion caused by the load reversals smooths the surfaces and reduces the fric-

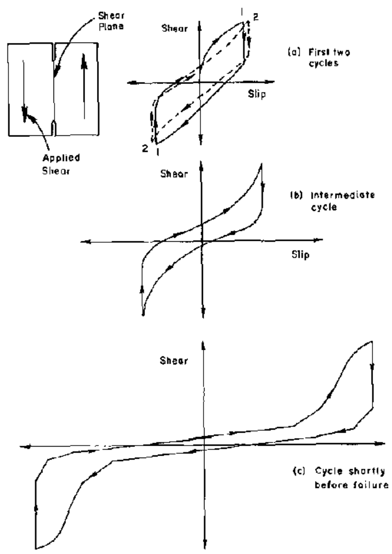


Fig. 5 Shear Versus Slip Curves [41]

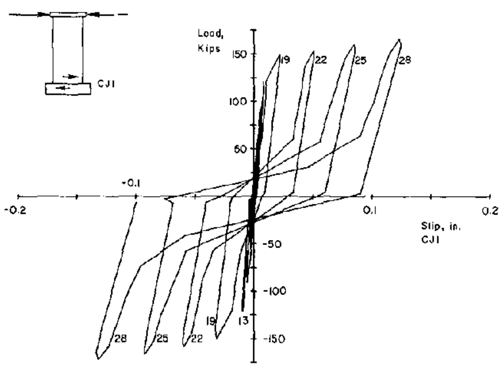


Fig. 6 Shear Versus Slip Curve from Isolated Wall Test [45]
(1.0 kip = 4.448 kN; 1.0 in. = 25.4 mm)

tional coefficient between the surfaces. At larger slips, tensile stresses in the reinforcement crossing the cracks increase as slip increases. This results in larger frictional forces. In addition, dowel forces increase.

Mattock's results indicate that the utimate shear transferred across a cracked shear plane under cyclic reversed loads is approximately 20% less than that for monotonic loads. Also slip is greater at equivalent levels of shear. Mattock also observed that larger initial crack widths resulted in a reduced shear stiffness.

Paulay, Park, and Phillips [58] observed hysteretic shear versus slip curves similar to those in Fig. 5. They found that efficient shear transfer is obtained if the crack along the joint is restrained from opening by reinforcement crossing the joint. With regard to shear transfer by doweling, Paulay [54,58] has concluded that the slip required to mobilize doweling forces is so large that there is no practical advantage to the utilization of doweling resistance.

Design Capacity of Construction Joints

Mattock [41] and Park and Paulay [50] have developed design recommendations based on shear friction theory. For shear transfer across cracks, Mattock recommends taking the strength for reversed loading as 80% of the monotonic strength, that is:

$$V_u = 0.8 \quad (\varphi A_{vf} f_y \mu) \tag{1}$$

where:

- V_u = shear transfer strength
- A_{vf} = area of shear friction reinforcement crossing the joint
- f_y = specified yield strength of reinforcement
- μ = coefficient of friction, taken as 1.0 for concrete placed against hardened concrete
- φ = capacity reduction factor, taken as 0.85 for shear

Park and Paulay do not specify the reduction for cyclic loading, but indicate that any beneficial effects from axial compression forces across construction joints should be reduced by 20% to account for possible upward accelerations.

Experience From Wall Tests

While shear transfer tests provide basic data for evaluation of construction joint performance, they do not fully simulate stress conditions at a horizontal construction joint in a wall. In laterally loaded walls, moment reversals applied to the wall can completely yield all reinforcement across the joint. Therefore, it is useful to supplement shear transfer data with observations made during tests of walls.

Tests of isolated cantilever walls being conducted at the Portland Cement Association [45] provide information on construction joints. The tests indicate that adequate performance will be realized if joints are made following "standard practice". This practice includes roughening and cleaning the hardened surface to remove laitance and loose particles prior to placing fresh concrete.

In the tests, slip across cracks at construction joint locations was no larger than that across structural cracks at other locations. A representative shear versus slip curve is shown in Fig. 6. Similarity to the curves in Fig. 5 is clear. The presence of vertical compressive loads tends to reduce the maximum slip for an equivalent shear and also reduces the amount of "pinching" of the curves. Confined boundary elements also reduced slip at construction joints.

Capacities of specimens subjected to shear stresses of $7\sqrt{f'_c}$ psi ($0.68\sqrt{f'_c}$ MPa) or larger were limited by web crushing prior to any breakdown of shear transfer at construction joints.

TESTS OF COUPLING BEAMS

Coupling beams are used to join adjacent structural walls. During an earthquake load is applied to the beams as the walls deflect laterally. The resulting deformations can be quite severe, with repeated rotations well beyond yield [40]. This is illustrated in Fig. 7 where the deformations involved are exaggerated. Investigations of coupling beam behavior require simulation of these boundary conditions.

Observed Behavior.

One of the primary observations from coupling beam tests is the progressive decrease in stiffness and strength resulting from repeated inelastic reversals of high shear forces [8,14,19,37,38,51,52,56,79]. Much of the experimental work carried out has been aimed at developing reinforcing details to reduce decay of stiffness and strength.

Tests at University of Canterbury. Tests on relatively deep coupling beams at the University of Canterbury [51,52,56] provide significant results. Conventionally reinforced beams, with span-to-depth ratios less than 1.5, failed in "sliding shear" under large shear reversals. This mechanism resulted as intersecting cracks formed by load reversals led to a breakdown in shear transfer across a vertical section at the intersection of the beam and wall. Because the slip plane was perpendicular to the beam span, conventional web reinforcement was not effective in controlling shear displacements. Shear slip led to significant loss of stiffness under inelastic load reversals.

To prevent sliding shear Paulay and Binney [56] used diagonal reinforcement as illustrated in Fig. 8. With this arrangement the deep beam acts much as a Mesnager hinge. In the University of Canterbury tests, the performance of the diagonally reinforced beams was excellent. They sustained their theoretical capacity over a greater number of load cycles and under much larger

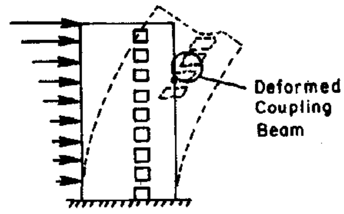


Fig. 7 Deformed Shape of Coupled Wall System

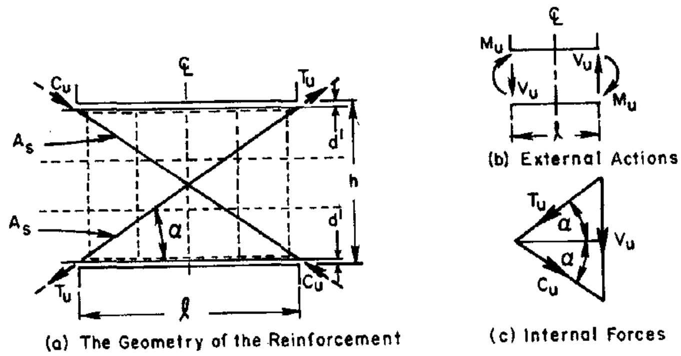


Fig. 8 Diagonal Reinforcement in Coupling Beam [56]

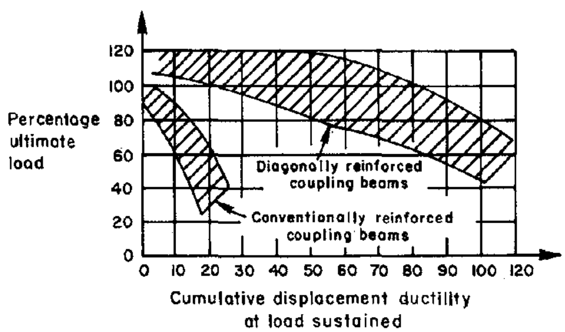
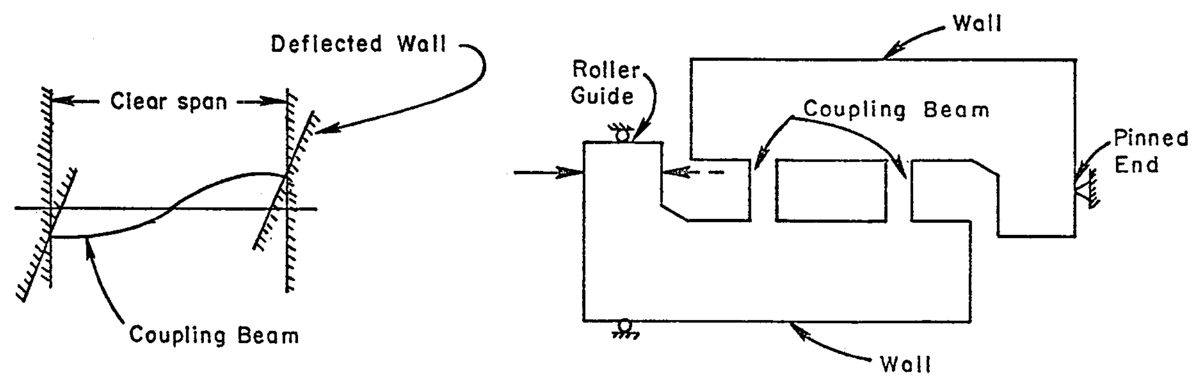


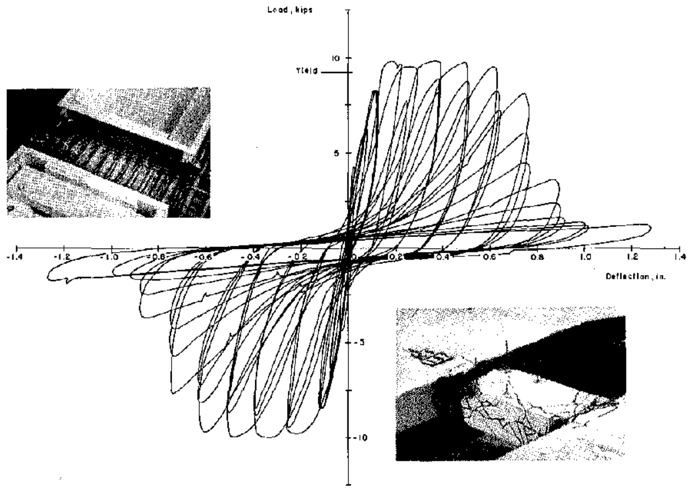
Fig. 9 Cumulative Displacement Ductilities for Deep Coupling Beams [56]



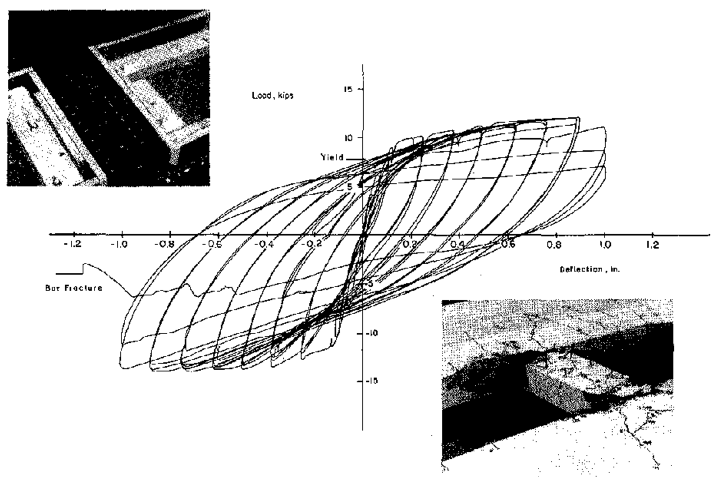
(a) Idealized Beam Deformation

(b) Test Setup

Fig. 10 Setup for Reversing Load Tests [8]



(a) Specimen with Conventional Reinforcement



(b) Specimen with Diagonal Reinforcement

Fig. 11 Load Versus Deflection Curves for Coupling Beams with a Span-to-Depth Ratio of 2.5 [8] (1.0 kip = 4.448 kN; 1.0 in. = 25.4 mm) ductilities than could be obtained by conventional reinforcement. Figure 9 is a comparision of the cumulative displacement ductilities obtained for conventional and diagonally reinforced beams.

Tests at the Portland Cement Association. Recent tests at the Portland Cement Association [8] have extended Paulay's work to more slender coupling beams. Span-to-depth ratios for these tests are 2.5 and 5.0. Reversed loads were applied through simulated wall segments shown in Fig. 10. Eight pairs of beams were tested.

Load versus deflection curves for two specimens are shown in Fig. 11. These beams had a span-to-depth ratio of 2.5. Specimen C5 was conventionally reinforced while Specimen C6 was reinforced with diagonal bars. The maximum shear stress developed was 7 $\sqrt{f'_c}$ psi (0.6 $\sqrt{f'_c}$ MPa) for Specimen C5 and 11 $\sqrt{f'_c}$ psi (0.9 $\sqrt{f'_c}$ MPa) for Specimen C6. The shear reinforcement for C5 was designed to carry the entire shear based on the theoretical ultimate moment capacity of the beam section.

Comparison of the load versus deflection curves indicates the improvement in behavior obtained using diagonal reinforcement. For beams with span-to-depth rations of 5.0, the improvement was not as dramatic. In addition, for longer span beams gravity loads within the span take on greater importance. These loads cannot be resisted efficiently using diagonal reinforcement.

The performance of the beams with diagonal reinforcement was limited by buckling and subsequent fracture of the bars. At late stages in the test severe spalling of concrete at the beam-wall intersection exposed the diagonal steel within the structural wall. This region did not contain confinement reinforcement to support the bars. To insure adequate performance, diagonal bars must be "confined" to inhibit buckling.

Additional Tests. Several other investigators [14,19,37,38,79] have indicated the importance of the level of shear on the ability of beams to sustain repeated excursions into the inelastic range.

Although other reinforcement arrangements have been tested [14], at present only the Mesnager hinge arrangement for short coupling beams has practical appeal.

An aspect of observed behavior not previously mentioned is that of reinforcement anchorage. Several investigators [1,34,37,38] have shown that slip of the main reinforcement anchored in the wall contributes significantly to beam deflections during reversals. Anchorage of reinforcement is of paramount importance in assuring satisfactory performance.

Design Recommendations

The following represents a summary of recommendations from the experimental investigations considered.

For structural members subjected to monotonic loads, present code requirements are adequate. However, for coupling beams designed to resist repeated large shear reversals in extreme inelastic ranges of response, the web reinforcement should be designed to carry the entire shear. Design shear should be based on actual section capacities including strain hardening of the reinforcement. Web reinforcement should be provided by closed hoops. Beam reinforcement details should be designed to effectively confine and contain the concrete within the core of the section. All longitudinal reinforcement should be adequately anchored within adjoining walls.

In certain situations the proportions of the structure may be such that high shear stresses on the coupling beams cannot be avoided. This could occur, for example, when beam span-to-depth ratios are less then 2.0. Such stresses would be on the order of $7\sqrt{f'_c}$ psi ($0.6\sqrt{f'_c}$ MPa) to $10\sqrt{f'_c}$ psi ($0.8\sqrt{f'_c}$ MPa). If repeated inelastic reversals at these stress levels must be resisted, diagonal reinforcement may be considered. Design recommendations for this arrangement are given elsewhere [50,54,56].

TESTS OF LOW-RISE ISOLATED WALLS

Isolated structural walls traditionally have been "classified" as low-rise or high-rise walls. A low-rise wall has been defined as having a height-to-horizontal length ratio of 1.0 or less [50]. This ratio, however, may not be the best indicator since wall behavior also depends on loading conditions, section geometry and section reinforcement. A more important design consideration for low-rise walls is that the shear capacity rather than flexural capacity may govern.

Observations During Earthquakes

A survey of the 1968 Tokachioki earthquake damage to low-rise buildings has led to the results shown in Fig. 12 [69,73] . These results provide a simple index for evaluating the likelihood of damage. For the buildings surveyed, it is evident that those having a larger wall area relative to their floor area had less damage.

Observed Behavior of Walls with Boundary Elements

A series of monotonic tests by Benjamin and Williams [9,10,11] was the first significant investigation of structural walls in the United States. Low-rise walls with column boundary elements were tested. For these tests the initial stiffness of the walls was reasonably well predicted by elementary strength of materials expressions for elastic flexural and shear deformations. This finding applied to walls with openings as well as solid walls.

At the Portland Cement Association, Barda [5,6,7] tested eight low-rise walls with flange boundary elements. The test program was designed to determine the effect of load reversals on shear strength.

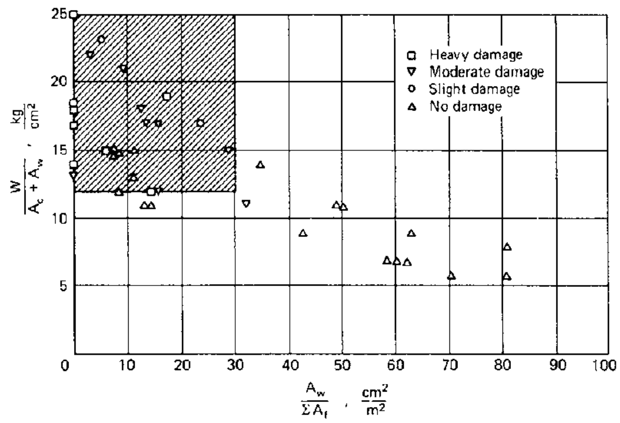


Fig. 12 Damage in Low-Rise Reinforced Concrete Buildings [69,73] (W = weight of building above level considered; A_c = total column area at level; A_w = total wall area at level; A_f = total floor area above level)

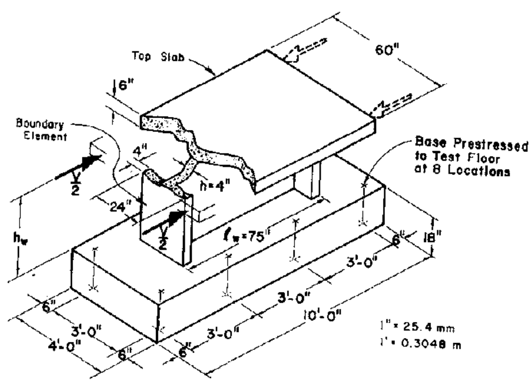


Fig. 13 Test of Low-Rise Wall with Boundary Element [7]

Principal variables included amount of flexural reinforcement, amount of vertical wall reinforcement, and height-to-horizontal length ratio. Flexural reinforcement was varied from 1.8 to 6.4% of the boundary element area; horizontal wall reinforcement and vertical wall reinforcement were varied from 0% to 0.5% of the wall area; and height-to-horizontal length ratio was varied from 0.25 to 1.0.

The test specimen and position of loading are shown in Fig. 13. Loads were applied to the wall through the top slab. Maximum measured forces corresponded to nominal shear stress from 8 $\sqrt{f'_c}$ psi (0.7 $\sqrt{f'_c}$ MPa) to 16 $\sqrt{f'_c}$ psi (1.3 $\sqrt{f'_c}$ MPa).

Principal findings were as follows:

1. Shear strength of the test specimens was not affected by differences in the amount of flexural reinforcement (vertical boundary element reinforcement) as long as all bars were properly anchored in the foundation. The design of these walls was such that vertical reinforcement in the boundary element did not yield.

2. Specimens subjected to load reversals had a shear strength about 10% less than similar specimens subjected to monotonic loading.

3. For specimens with a height-to-horizontal length ratio of 0.5 or less, horizontal wall reinforcement did not contribute to shear strength. However, horizontal bars distributed the cracks and reduced crack widths.

4. Vertical wall reinforcement was effective as shear reinforcement in specimens with height-to-horizontal length ratios of 0.5 and 0.25. It was less effective for a height-to-horizontal length ratio of 1.0. Vertical bars distributed the cracks and reduced crack widths.

5. Shear strength of a specimen with a height-to-horizontal length ratio of 1.0 was about 20% lower than the strength of comparable specimens with ratios of 0.5 and 0.25.

6. Load-carrying capacity beyond the maximum load depended primarily on the ability of the boundary elements to act as a frame. Frame action provided a mode of failure that was gradual.

Shiga, Shibata, and Takahashi [70] have tested low-rise walls with column boundary elements. Their tests were similar to Barda's [7] . However, they also provided for application of vertical loads. The walls had a height-to-horizontal length ratio of 0.6. The results obtained from these tests were comparable to those found by Barda. Both sets of tests indicate loss in stiffness of the walls under repeated load reversals.

Observed Behavior of Rectangular Walls

Relatively few reversing load tests of low rise rectangular walls have been reported. A series of seven tests on walls with a height-to-horizontal length ratio of 1.0 were carried out at the Portland Cement Association [20]. This series included one wall subjected to load reversals. The strength of the wall under load reversals was 93% of the monotonic strength. Maximum loads on these walls corresponded to about 10.0 $\sqrt{f'_c}$ psi (0.8 $\sqrt{f'_c}$ MPa).

Tests at the University of Canterbury [50] on rectangular walls with a height-to-horizontal length ratio of 1.0 indicated that sliding shear may limit capacity. For a wall with a flexural capacity corresponding to a shear stress of about 5.6 $\sqrt{f'_c}$ psi (0.5 $\sqrt{f'_c}$ MPa), the full calculated flexural strength was developed under reversals. The wall contained shear reinforcement greater than that required to develop the flexural capacity.

At McMaster University [2] the effects of axial loads on low-rise rectangular walls have been investigated. Results indicated that compressive axial stress increases load capacity, reduces the rate of deterioration in stiffness, and reduces attainable ductility.

Design Recommendations

For walls with height-to-horizontal length ratios of 1.0 or less, Barda [6,7] recommends that the design shear capacity be calculated as:

$$v_u = \varphi \left(8\sqrt{f'_c} - 2.5\sqrt{f'_c}\ \frac{h_w}{\ell_w} + \frac{N_u}{4\ell_w h} + \rho\ f_y\right) \qquad (2)$$

where:

v_u = shear capacity of wall
φ = capacity reduction factor, taken as 0.85 for shear
f'_c = specified compressive strength of concrete, psi
h_w = height of wall
ℓ_w = horizontal length of wall
N_u = axial force normal to cross section
h = wall thickness
ρ = ratio of area of wall reinforcement to gross concrete area
f_y = specified yield strength of reinforcement

The ratio is taken as the lesser of the value of the vertical or horizontal wall reinforcement. The term f_y should not exceed $6\sqrt{f'_c}$. In addition, the minimum ratio of vertical or horizontal wall reinforcement should be 0.0025.

In Fig. 14 strengths calculated using Eq. 2 (with φ = 1.0) are compared with measured strengths. Tests included are those from References [7], [70], and [20]. Only tests where shear was indicated as the mode of failure are included. Calculated strengths provide a reasonable lower bound to those measured.

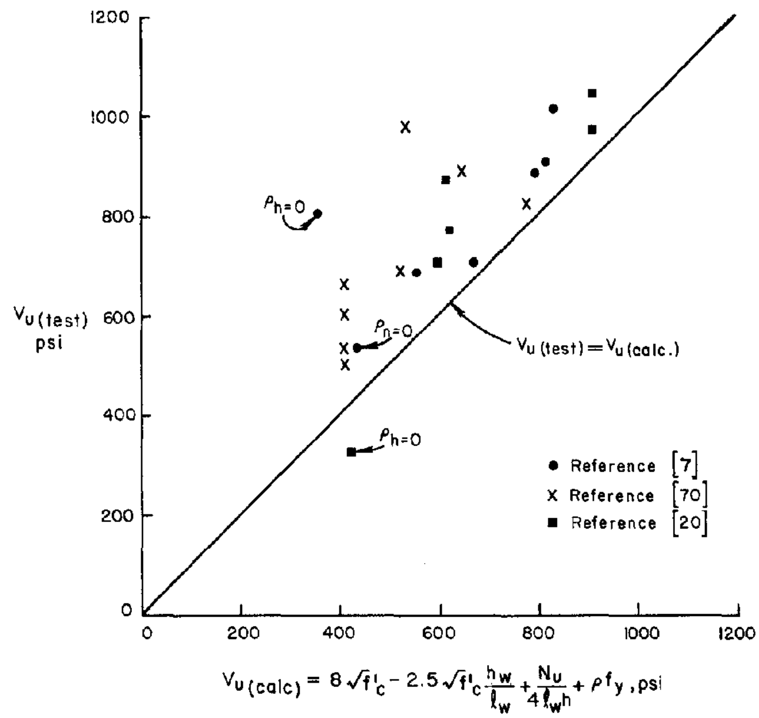


Fig. 14 Comparison of Calculated and Measured Strengths of Low-Rise Walls (1000 psi = 6.895 MPa)

Based on the University of Canterbury tests on rectangular walls, Park and Paulay [50] suggest:

1. If ductile flexural wall behavior is desired maximum nominal shear stresses on the wall should not be greater than 6 $\sqrt{f'_c}$ psi (0.5 $\sqrt{f'_c}$ MPa).

2. The entire shear force should be resisted by the web reinforcement.

While these suggestions could lead to larger attainable ductilities under reversing loads, such ductilities may not be demanded for low-rise walls. In many situations low-rise walls can be designed to respond "elastically" to earthquake forces. Repeated inelastic load reversals may not be necessary or required.

TESTS OF HIGH-RISE ISOLATED WALLS

Observations following recent earthquakes have indicated that properly designed structural walls used as lateral bracing in multistory buildings can significantly improve performance. However, there is considerable debate over design provisions for walls. This has resulted in part because of the lack of experimental information on strength and deformation capabilities. As mentioned previously, tests of high-rise structural walls for seismic loadings have been started in the United States only within the past few years. One objective of these investigations has been to find reinforcing details that will provide ductility and energy dissipation capacity as well as strength. In addition, tests on isolated walls have been conducted to determine hysteretic behavior so that better analytical procedures for determining dynamic response can be developed.

Observed Behavior-Monotonic Tests

A series of tests at Stanford [9,10,11] along with tests started in 1967 at the Portland Cement Association [20,21] formed the basis for current North American code provisions for structural walls. Virtually all of these were monotonic tests on rectangular isolated walls.

The test setup for tests at the Portland Cement Association is shown in Fig. 15. Walls were tested as horizontal cantilever beams. The setup included provisions for applying compressive axial load. Specimens had a thickness of 3 in. (76 mm) and a horizontal length of 6.25 ft (1.90 m). The height of the specimen was either 21 ft (6.40 m), 12 ft (3.66 m), or 6.25 ft (1.90 m).

These tests indicated that the strength of a tall rectangular wall subjected to monotonic loading is generally governed by flexure rather than shear. The flexural capacity of high rise walls can be predicted by methods used for ordinary reinforced concrete beams. Shear capacity of the walls was adequately predicted by 1971 ACI Building Code provisions.

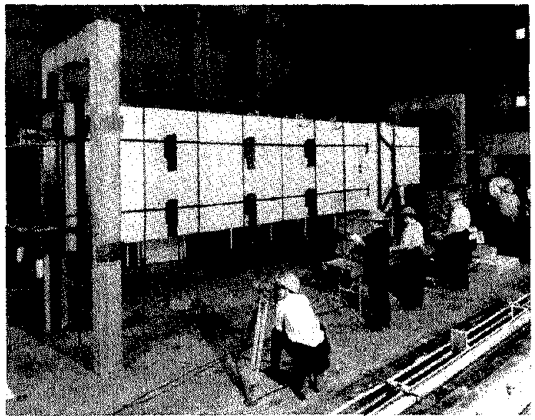


Fig. 15 Test Setup for Monotonic Test of Wall [20]

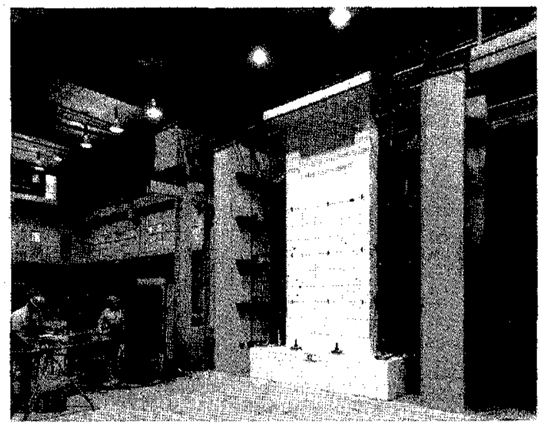


Fig. 16 Isolated Wall Test Setup [45]

Influence of the amount and distribution of vertical reinforcement on the deformation characteristics of walls was determined in these tests. Concentration of vertical reinforcement near the ends of the walls resulted in a higher moment capacity and a greater ultimate curvature.

Axial compression on the walls increased the moment capacity, but reduced the ultimate curvature.

While these tests were a significant advancement of our knowledge, only one of the specimens was subjected to load reversals. Thus, hysteretic behavior of the walls could not be evaluated completely.

Observed Behavior - Reversing Load Tests

Major programs for evaluation of the behavior of structural walls under static reversing loads are in progress at the University of California, Berkeley [15,16,17,63] and the Portland Cement Association [24,25,26,44,45]. A primary objective of these programs is the development of design criteria for structural wall systems. To meet this objective both programs have included tests of isolated walls.

Tests at the Portland Cement Association The large test specimen being used at the Portland Cement Association is shown in Fig. 16. Height of each wall is 15 ft (4.57 m). Horizontal length of the wall is 6 ft 3 in. (1.91 m) and its thickness is 4 in. (102 mm). Walls are subjected to reversing in-plane loads applied through the top slab. The base block is clamped to the test floor.

The test program is a partial parametric study with the specimen representing a basic element of a structural wall system. Controlled variables have included shape of the wall cross section, amount of main flexural reinforcement, amount of hoop reinforcement around the main flexural reinforcement, amount of horizontal shear reinforcement, load history, and axial compressive load. The amount of main vertical flexural reinforcement controls the moment capacity of the wall and, thus, the intensity of shear applied to the wall. To date, 13 walls have been constructed and tested. Two of these walls have been repaired and retested.

In proportioning the walls, the design moment was calculated following 1971 ACI Building Code provisions [45], neglecting strain hardening of the reinforcement. In most specimens shear reinforcement was provided so that the calculated design moment would be developed. In one of the walls, shear reinforcement was provided to carry the total shear based on the full calculated moment capacity rather than the design moment capacity.

Figure 17 shows that in all cases the measured capacity of the walls exceeded the design capacity. The design strength plotted is the lesser of the flexure or shear strength with the capacity reduction factor $\phi = 1.0$.

The capacity of walls subjected to low shear stress, $v_{max} < 3\sqrt{f'_c}$ psi ($0.3\sqrt{f'_c}$ MPa) was limited by alternate tensile yielding and compressive buck-

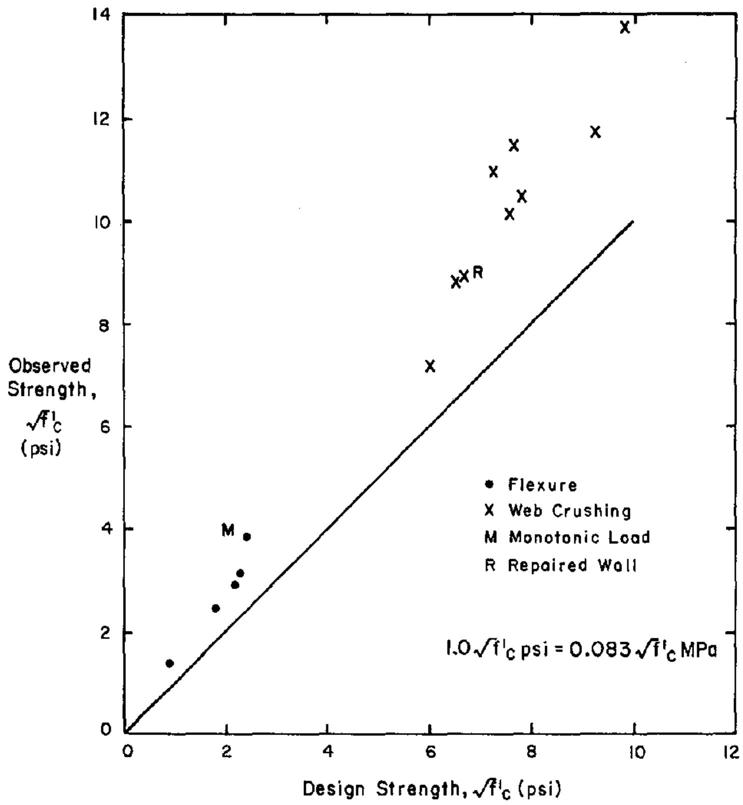
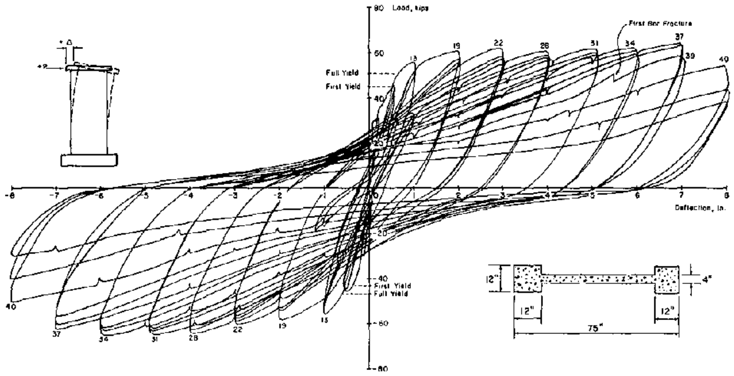
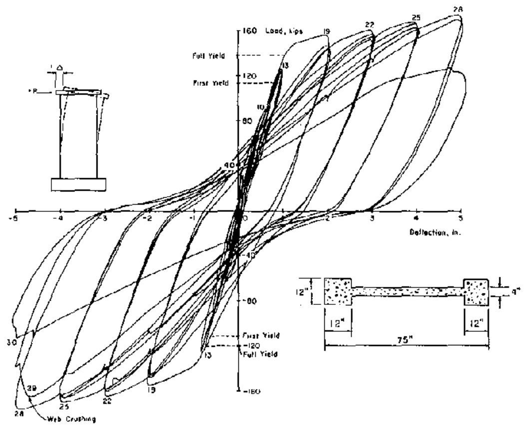


Fig. 17 Comparison of Observed and Design Strengths for Isolated Wall Specimens [45]



(a) Specimen with Maximum Nominal Shear Stress
$V_{max} = 3.1 \sqrt{f'_c}$ psi $(0.3 \sqrt{f'_c}$ MPa)



(b) Specimen with Maximum Nominal Shear Stress
$V_{max} = 8.5 \sqrt{f'_c}$ psi $(0.7 \sqrt{f'_c}$ MPa)

Fig. 18 Load Versus Deflection Curves for Isolated Wall Specimens [45] (1 in. = 25.4 mm; 1 kip = 4.448 kN) ling of the main vertical reinforcement. This buckling was accompanied by loss of concrete not contained by the reinforcement. Buckling of the vertical steel was followed, after several cycles, by bar fracture. One or two bars fractured at a time, with a corresponding decrease in load. Confinement hoops around the main vertical reinforcement delayed bar buckling and contained the core concrete.

In one rectangular wall with low shear stress, large out-of-plane displacements of the compression zone of the wall were observed as the specimen was cycled in the inelastic range.

The capacity of walls that developed high shear stress, $> 7\sqrt{f'_c}$ psi ($0.6\sqrt{f'_c}$ MPa) was limited by web crushing. The reversed loading resulted in a system of inclined cracks that crisscrossed the web forming relatively symmetrical compression strut systems for each direction of loading. The web crushing that occurred was associated with progressive deterioration of the strut during the load reversals. Loss of load capacity at web crushing was usually sudden.

A measure of the hysteretic behavior of isolated walls is given in Fig. 18. This figure shows measured load versus deflection relationships for two of the test walls. The specimen with low nominal shear stress sustained a greater number of load cycles at higher deflections. Pinching of the loops is evident for both walls.

The relationship between specimen strength and ductility as observed in the Portland Cement Association tests is illustrated in Fig. 19. Deflection ductility is the maximum stable deflection at the top of the wall divided by the full yield deflection. For this comparison, selection of the definition of ductility was arbitrary. Since the walls were subjected to similar load histories, the ductility measure chosen provides a basis for comparison of inelastic response. This definition should be kept in mind if the results are extrapolated for other uses [12,62].

Figure 19 shows that for higher levels of shear, ductility is reduced. As mentioned previously the failure mode for walls with shears $> 7\sqrt{f'_c}$ psi ($0.7\sqrt{f'_c}$ MPa) was web crushing.

One of the problems in interpretation of measured ductilities is that insufficient information exists on ductility demands. This makes it difficult to evaluate the "goodness" of a test result. For example, experiments show "pinched" hysteresis loops, but there is no absolute measure of how much pinching can be tolerated.

Tests at the University of California. The University of California [15, 16,17,63] has developed an experimental facility to simulate the loading and boundary conditions on the bottom three stories of a prototype structure. The test facility is illustrated in Fig. 20. The loading system is synchronized to simultaneously apply shear, overturning and axial forces.

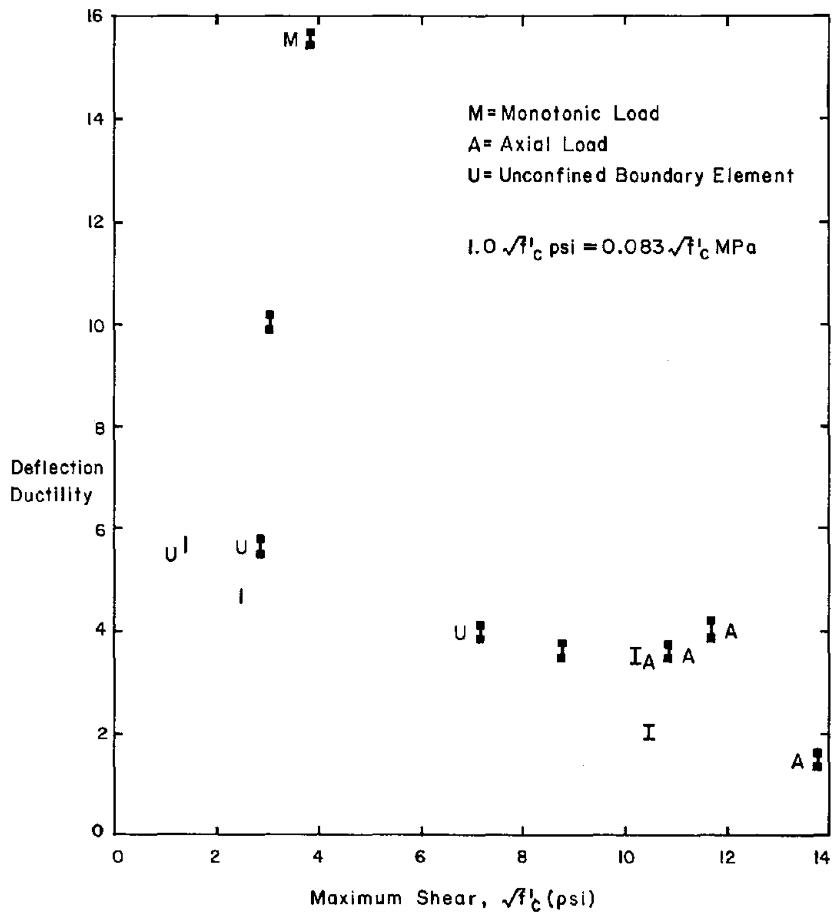


Fig. 19 Relationship Between Ductility and Maximum Shear for Isolated Wall Specimens [45]

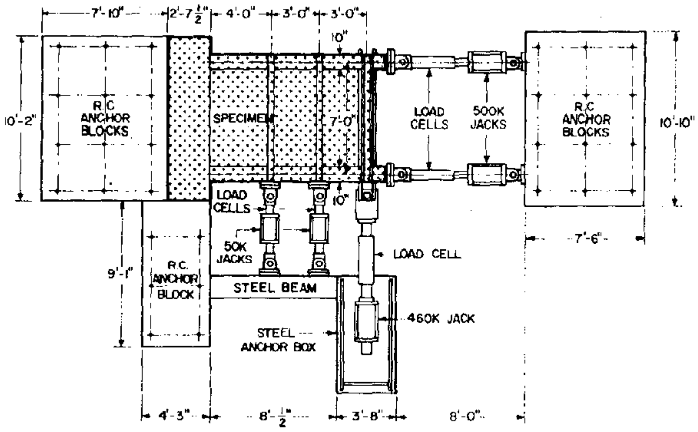


Fig. 20 Structural Wall Test Facility [17]
(1.0 in. = 25.4 mm)

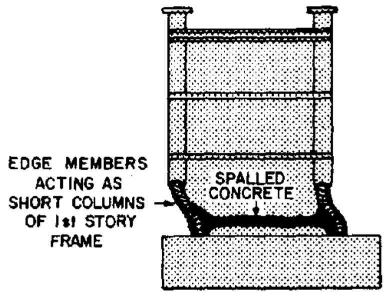
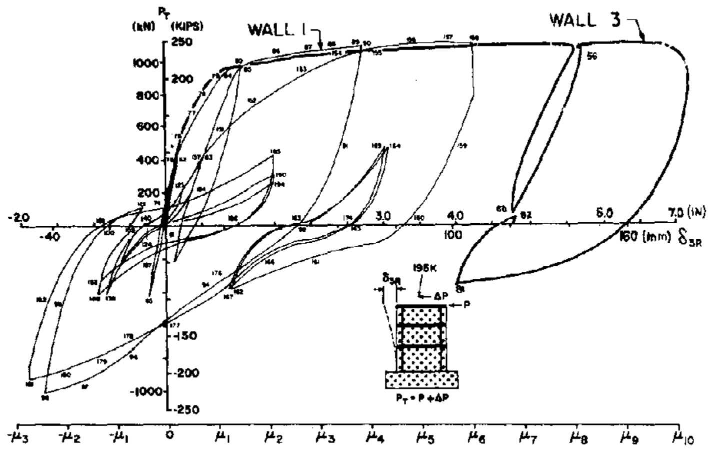
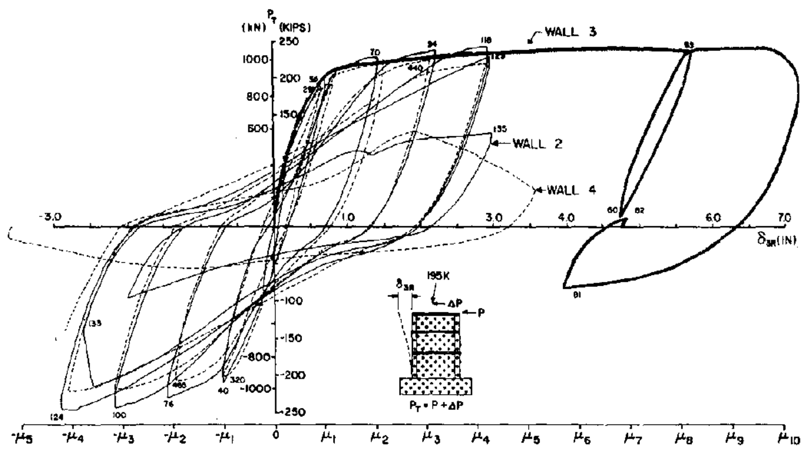


Fig. 21 Observed Mode of Failure [17]



(a) Wall Specimens 1 and 3



(b) Wall Specimens 2, 3, and 4

Fig. 22 Load Versus Deflection Curves for Isolated Walls [17]

Four tests have been carried out on 1/3-scale walls with column boundary elements. The walls are 4-in (102-mm) thick and have a horizontal length of 7 ft (2.13m). Two walls were tested with spiral reinforcement in the boundary elements while the other two had closely spaced square ties. In addition to column confinement, the primary variable was the applied load history.

The tests ended with web crushing at levels of shear corresponding to about 11 $\sqrt{f'_c}$ psi (0.9 $\sqrt{f'_c}$ MPa). This behavior, illustrated in Fig. 21, is similar to that observed in the PCA tests [45] of walls subjected to relatively high shear stress.

The hysteretic behavior of the walls tested at the University of California is shown in Fig. 22. Dependence of the ductility on load history is particularly evident in these curves. However, load history did not affect the yield strength or the maximum strength. The different types of confinement of the vertial boundary element reinforcement had no significant effect on overall response.

Both the University of California [17] and the Portland Cement Association [45] tests have shown that shearing distortions are a major component of lateral displacements.

Design Recommendations

The following is based on the tests reviewed:

1. Strengths of the test specimens exceeded present code design strengths for flexure and shear. However, the designer must be aware that present provisions underestimate flexural capacity because strain hardening of the reinforcement is neglected. For inertia loadings, shear forces developed are related to actual flexural capacity not the design flexural capacity. Thus, the level of shear can be significantly higher than anticipated.

2. Structural walls capable of developing large ductilities can be designed for reversing loads corresponding to shear stresses as high as 10 $\sqrt{f'_c}$ psi (0.8 $\sqrt{f'_c}$ MPa).

3. The maximum level of shear stress that can be developed in a wall may be limited by web crushing capacity. Addition of horizontal shear reinforcement beyond present code provisions does not significantly improve strength for this mode of failure.

4. Use of confinement reinforcement in the boundary elements within the hinging region significantly improves inelastic behavior. It appears that there is no major difference between the use of spirals or closely spaced rectangular hoops.

5. Stiff boundary elements help to limit shear distortions and construction joint slip.

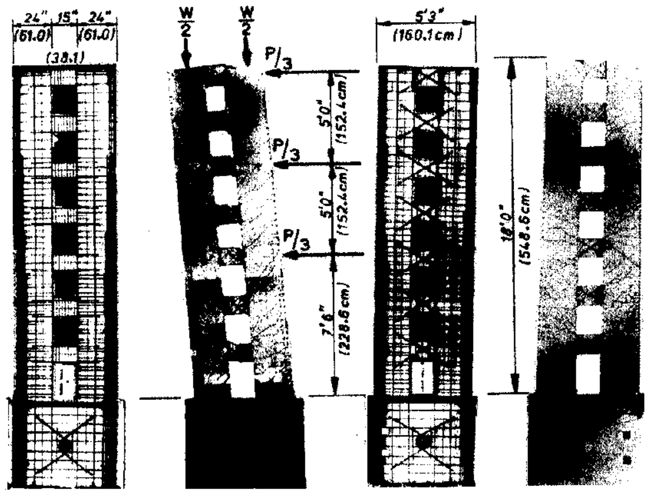


Fig. 23 Coupled Wall Test Specimens [59]

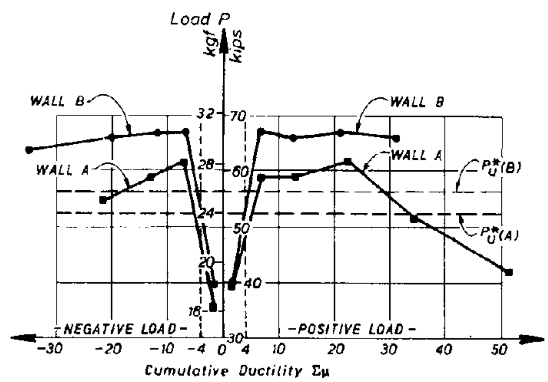


Fig. 24 Comparison of Cumulative Ductilities of the Coupled Walls [59]

TESTS OF COUPLED WALL SYSTEMS

Only a few tests on coupled wall systems have been reported. These include reversing load tests at the University of Canterbury [55,59,60,66,67] and earthquake simulator tests at the University of Illinois [4,74].

Observed Behavior - Static Tests

Tests carried out at the University of Canterbury were designed to confirm anticipated performance of diagonally reinforced deep coupling beams in coupled walls [59,60]. In the basic design, the coupled systems were to utilize the coupling beams to dissipate as much of the input energy as possible. Thus, the walls were designed so that yielding at the base of the wall could occur only after all beams had yielded.

Test specimen details are shown in Fig. 23. The specimens were 1/4-scale models of a seven-story coupled wall. The only difference between the models was the coupling beam reinforcement. Coupling beams in Wall A had "conventional" reinforcement while those in Wall B had diagonal reinforcement.

Beams with conventional reinforcement developed sliding shear failures after several cycles of reversed loading in the inelastic range. The diagonally reinforced beams did not deteriorate under similar loadings.

Overall performance of the specimens based on stiffness degradation, ductility, and energy dissipation was better for the walls coupled by diagonally reinforced beams. This is illustrated by the comparison of cumulative ductilities in Fig. 24.

Observed Behavior - Dynamic Tests

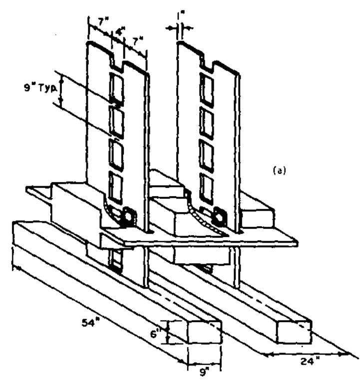
Dynamic tests at the University of Illinois [4,74] on small-scale walls shown in Fig. 25 have led to the following observations.

The natural frequencies were progressively reduced as the models were subjected to a series of test runs of increasing intensity. Consequently the dynamic response of the structures was related to the reduced stiffness rather than the initial stiffness.

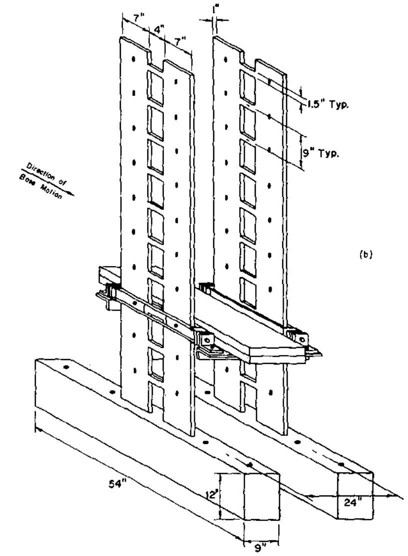
Design of the models was such as to limit yielding to the coupling beams for a particular intensity of motion. Observed responses indicated that this design concept was realized.

TESTS OF FRAME-WALL SYSTEMS

As was the case for coupled walls, there are very few tests of frame-wall systems. Again, static reversing load tests have been carried out at the University of Canterbury [61] and earthquake simulator tests have been carried out at the University of Illinois [47,48,49].



(a) Series 1



(b) Series 2

Fig. 25 Coupled Wall Specimens for Dynamic Tests [74]
(1.0 in. = 25.4 mm)

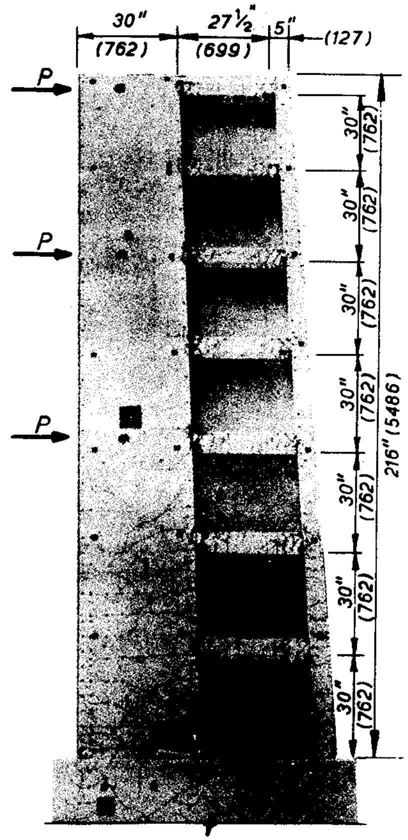


Fig. 26 Frame-Wall Specimen after Static Reversing Load Test [61]

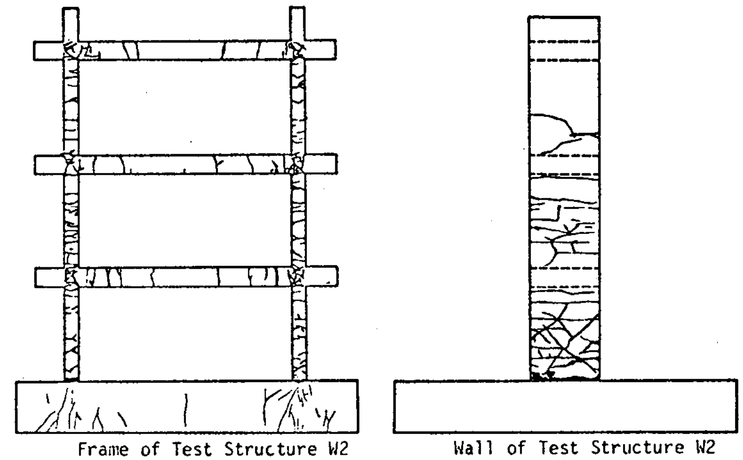


Fig. 27 Frame-Wall Specimen after Dynamic Test [48]

Observed Behavior - Static Tests

Two frame-wall tests at the University of Canterbury [61] were similar to the coupled wall tests in that the primary variable was beam reinforcement. Figure 26 shows the test specimen.

The tests showed that sliding shear developed in the plastic hinges of the beams near the wall in the conventionally reinforced beams. This did not occur in the specimen with beams having special diagonal reinforcement.

Observed Behavior - Dynamic Tests

Small-scale models tested on the earthquake simulator at the University of Illinois [47,48,49] showed that test structures carefully designed to prevent shear or anchorage failures can withstand a series of intense base motions without collapse.

Coupling of a structural wall to a frame was effective in reducing lateral displacements. This was true even after the development of flexural damage at the base of the wall. Wall damage resulted as wide flexural cracks formed at the base of the wall when concrete crushed. Eventually, sliding shear damage occurred at the base of the wall. A specimen after testing is shown in Fig. 27.

RECOMMENDATIONS FOR FUTURE RESEARCH

Within the past ten years significant effort has gone into experimental work on structural walls. Much basic information on the behavior of these systems is now available. However, there are a number of areas where additional effort is needed.

1. Perhaps the most pressing need for research is in the correlation of analytical and experimental results. In particular, most experimental effort is directed toward developing reinforcement details to give beams and walls more ductility, less stiffness loss, and more energy dissipation. However, little is known about how much is enough. Effort should be expended to relate attainable ductility to demanded ductility.

2. Additional experimental data on the response of coupled wall systems and frame-wall systems must be developed.

3. Experimental data on the effects of openings on the behavior of walls must be developed.

4. Experimental data on use of lightweight aggregate in earthquake-resistant buildings with structural walls is needed.

5. Tests have indicated that after a number of large inelastic load reversals it is possible to encounter stability problems in the compression zone within the hinging region of a rectangular wall.

Floor slabs may not provide adequate lateral support because the "softened" wall may displace laterally between floors. Research in this area is needed.

6. Confinement of vertical reinforcement within the boundary elements of walls has been shown to be of significant benefit. Much of the design information developed for this type of reinforcement is from tests intended to determine needed information on limiting concrete strains. A primary benefit of confinement in walls, however, is to limit bar buckling and to contain concrete under load reversals. Additional research should be initiated to develop effective confinement details and design criteria for conditions encountered in wall boundary elements.

7. Most tests of isolated walls have been on specimens subjected either to very low or very high nominal shear stresses. Information on the mid-shear range needs to be developed. This would be in the range from 6 $\sqrt{f'_c}$ psi (0.5 $\sqrt{f'_c}$ MPa) to 8 $\sqrt{f'_c}$ psi (0.7 $\sqrt{f'_c}$ MPa).

8. Requirements for detailing of reinforcement should be reviewed with the goal of developing effective and economical detailing practices. In particular, information on the use of tension lap splices in earthquake resistant structures is needed.

ACKNOWLEDGMENTS

This paper was written at the Structural Development Section of the Portland Cement Association, Dr. H. G. Russell, Manager. Mr. R. G. Oesterle reviewed the manuscript and contributed a number of valuable suggestions. The tests conducted at the Portland Cement Association and reported in References [24,25,26,44,45] were sponsored in part by the National Science Foundation under Grant No. ENV74-14766.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

IMPORTANCE OF REINFORCEMENT DETAILS IN EARTHQUAKE-RESISTANT STRUCTURAL WALLS

by

A. E. Fiorato
Assistant Manager
Structural Development Section
Portland Cement Association
Skokie, Illinois

R. G. Oesterle
Structural Engineer
Structural Development Section
Portland Cement Association
Skokie, Illinois

W. G. Corley
Director
Engineering Development Department
Portland Cement Association
Skokie, Illinois

INTRODUCTION

Post-earthquake damage investigations over the past 25 years have provided valuable lessons on the importance of detailing. For severe earthquake loading it is inevitable that neglected details lead to major problems. The designer must be aware of the importance of proper detailing for seismic resistance. In addition, the contractor must be aware of the importance of proper construction practices so that the structure is built according to the design.

This paper gives examples of detailing practices related to design and construction of reinforced concrete structural wall systems. These are based primarily on experience gained in laboratory tests. They are supplemented by findings from post-earthquake damage investigations.

The areas covered include confinement reinforcement in vertical boundary elements and anchorage of horizontal wall reinforcement.

CONFINEMENT REINFORCEMENT IN VERTICAL BOUNDARY ELEMENTS

Figure 1 illustrates several wall cross sections encountered in buildings. Each of these configurations can be designed with vertical boundary elements. For box-sections, flanged, and intersecting walls, the boundary element may be located at intersections. Barbell walls have column boundary elements at each end. For rectangular walls, the boundary element may be concealed within the thickness of the wall.

To perform effectively during severe earthquakes, vertical reinforcement in boundary elements must be confined by properly detailed transverse reinforcement. Transverse confinement reinforcement serves four primary functions:

1. It increases limiting strain capacity of the concrete core;
2. It supports vertical reinforcement against inelastic buckling;
3. Along with the vertical bars, it forms a "basket" to contain concrete within the core;
4. It increases the shear capacity and stiffness of the boundary elements.

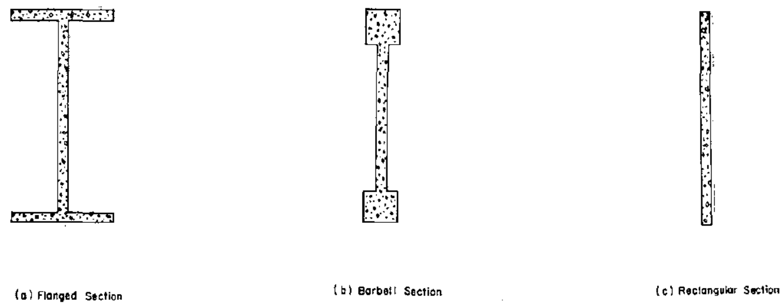


Fig. 1 Structural Wall Cross Section

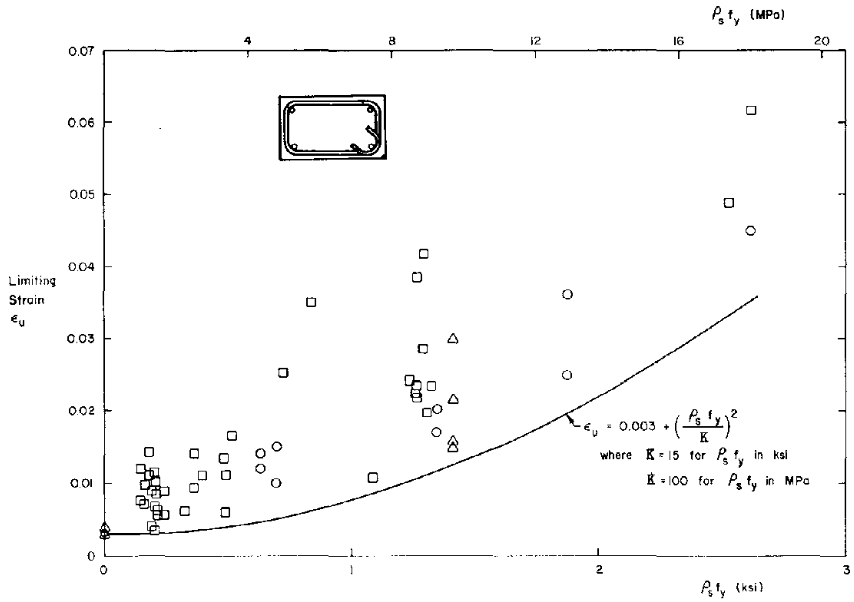


Fig. 2 Effect of Transverse Hoop Reinforcement on Limiting Strain Capacity of Concrete [5]

Confinement to Increase Limiting Concrete Strains

The effectiveness of rectangular hoops as confinement reinforcement to increase compressive strain capacity of concrete has been investigated in tests of relatively large scale elements. 5 Rectangular hoop reinforcement meeting or exceeding the confinement requirements of Appendix A of the 1971 ACI Building Code 3 extended the limiting concrete strain beyond 0.015. This is considerably greater than the value of 0.003 for plain concrete.

A summary of results is shown in Fig. 2. The observed limiting strains, ε_u, are plotted as a function of the product the volumetric hoop reinforcement ratio, ρ_s, and the yield strength, f_y, of the transverse reinforcement. The curve represents a lower bound to the test results. All arrangements of rectangular hoops were effective in increasing limiting concrete strains.

Reversing load tests of isolated structural walls [7] have also indicated that confinement reinforcement provided in accordance with the 1971 ACI Building Code, [3] or the 1976 Uniform Building Code [9] is adequate to maintain the compressive strength of boundary elements under large rotational strains.

Design of confinement reinforcement according to a limiting strain criteria is not always necessary for structural walls. In many cases, the geometry of walls is such that they are considerably under-reinforced in flexure. Therefore, fracture of reinforcement in tension rather than concrete in compression is the limiting criteria. However, confinement is necessary for support of vertical reinforcement and containment of concrete in the compression zone.

Confinement to Support Vertical Reinforcement and Contain Concrete Core

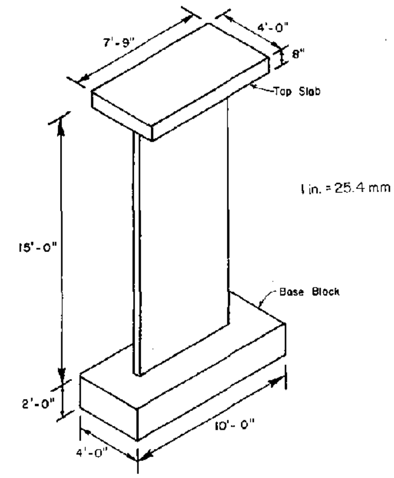
The functions of transverse reinforcement to restrain vertical bars against inelastic buckling and to contain the concrete core are of considerable importance. Comparison of two tests of isolated structural walls clearly illustrates this function.

The isolated walls tested were approximately 1/3-scale models of full-size walls. [7] Nominal dimensions of the specimens are given in Fig. 3(a). Each specimen was tested as a vertical cantilever with reversing loads applied through the top slab. The test set-up is shown in Fig. 3(b).

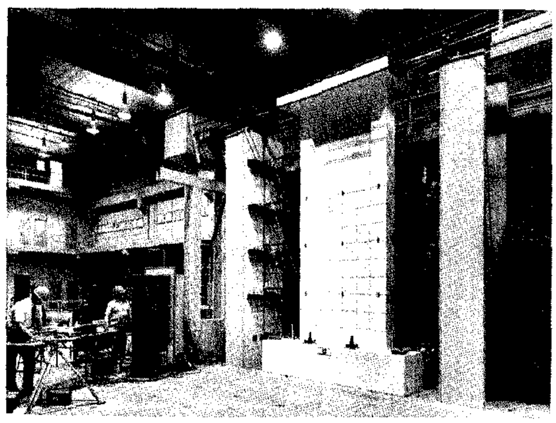
Reinforcement details for two of the specimens, B1 and B3, are shown in Figs. 4 and 5, respectively. These walls had barbell cross sections with vertical reinforcement in the boundary elements corresponding to 1.1% of the column areas. The walls were nominally identical except for the transverse reinforcement in the boundary elements.

Specimen B1, the unconfined wall, contained ordinary column ties designed according to Section 7.12 of the 1971 ACI Building Code. [3] The resulting tie spacing was 8 in. (203 mm), corresponding to 16 vertical bar diameters.

Specimen B3, the confined wall, had special transverse reinforcement designed according to Section A.6.4 of the 1971 Building Code. [3] This confinement was placed at a spacing of 1.33 in. (34 mm) over the first 6 ft (1.83 mm) of



(a) Nominal Dimensions of Test Specimens



(b) Test Set-Up

Fig. 3 Tests of Isolated Walls

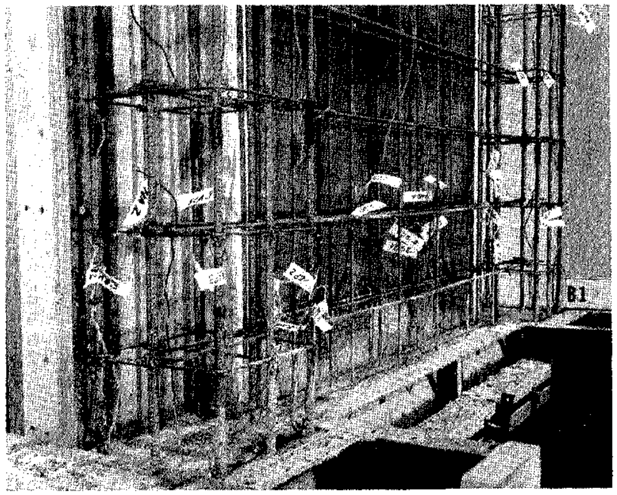


Fig. 4 Reinforcement for Specimen B1

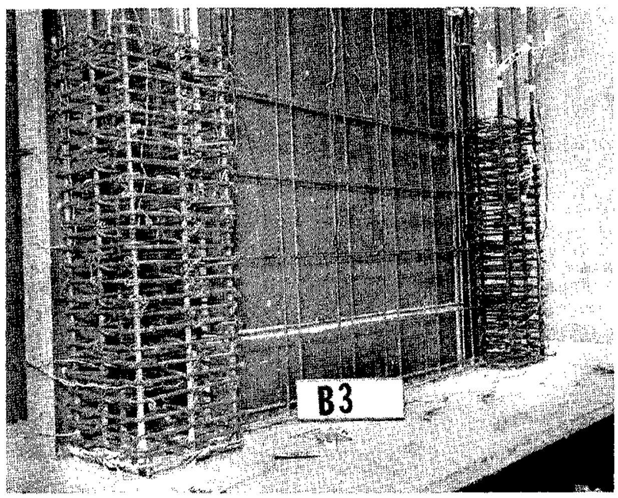


Fig. 5 Reinforcement for Specimen B3

the wall. Ordinary column ties were used over the remaining height of the wall. Confinement reinforcement spacing corresponded to 2.7 vertical bar diameters.

The hysteretic response of Specimens B1 and B3 is illustrated in the load versus top deflection relationships in Figs. 6 and 7. The maximum loads sustained by these walls corresponded to a nominal shear stress of $[3]\sqrt{f'_c}$ psi ($0.3\sqrt{f'_c}$ MPa).

Deterioration in strength and stiffness of Specimen B1 was caused by damage to the boundary elements by alternate tensile and compressive yielding. This led to buckling of the main vertical reinforcement. Because of the reversing inelastic loads, buckling of vertical reinforcement was more critical than it would be for monotonic loading. In addition, shear distortions resulted in relatively large eccentricities in the compressive force on each bar. Buckling was accompanied by loss of concrete not contained by the vertical and transverse reinforcement when the boundary element was in tension. A photograph of the buckled reinforcement is shown in Fig. 8.

The confinement hoops in Specimen B3 did not significantly increase the strength or maximum rotation as compared to Specimen B1. However, the hoops maintained the integrity of the boundary elements by delaying bar buckling and containing the concrete core. Photographs of the two walls at the same load increment in Figs. 9 and 10 clearly show the effectiveness of the confinement. For equivalent levels of load, the confined wall suffered less damage and thus could have been reparied more easily.

The development of criteria for transverse reinforcement as a function of inelastic buckling of vertical reinforcement requires additional investigation. For example, Bresler and Gilbert [2] have considered tie requirements for columns subjected to monotonic compression. However, no work has been done on the effects of reversing stresses in the inelastic range.

Confinement to Provide Shear Capacity

Transverse hoop reinforcement in vertical boundary elements improves shear capacity and stiffness. This function was also observed in the tests of isolated walls described previously.

Two specimens, B2 and B5, were constructed with nominally identical reinforcement except for the transverse confinement. Both walls had barbell cross sections with vertical reinforcement in the boundary elements of about 3.7% of the column area. Photographs of the reinforcement are shown in Figs. 11 and 12.

The unconfined wall, Specimen B2, had ordinary column ties at a spacing of 8 in. (203 mm) or 10.7 bar diameters. The confined wall, B5, had hoops spaced at 1.33 in. (34 mm) or 1.8 bar diameters over the first 6 ft (1.83 m) of the wall. Ordinary column ties were used over the remaining height of the wall.

Load versus top deflection relationships for the two specimens are given in Figs. 13 and 14. The capacity of both walls was limited by web crushing. Specimen B2 reached a capacity corresponding to a nominal shear stress of $7.2\sqrt{f'_c}$ psi ($0.60\sqrt{f'_c}$ MPa). Specimen B5 reached $8.8\sqrt{f'_c}$ psi ($0.73\sqrt{f'_c}$ MPa).

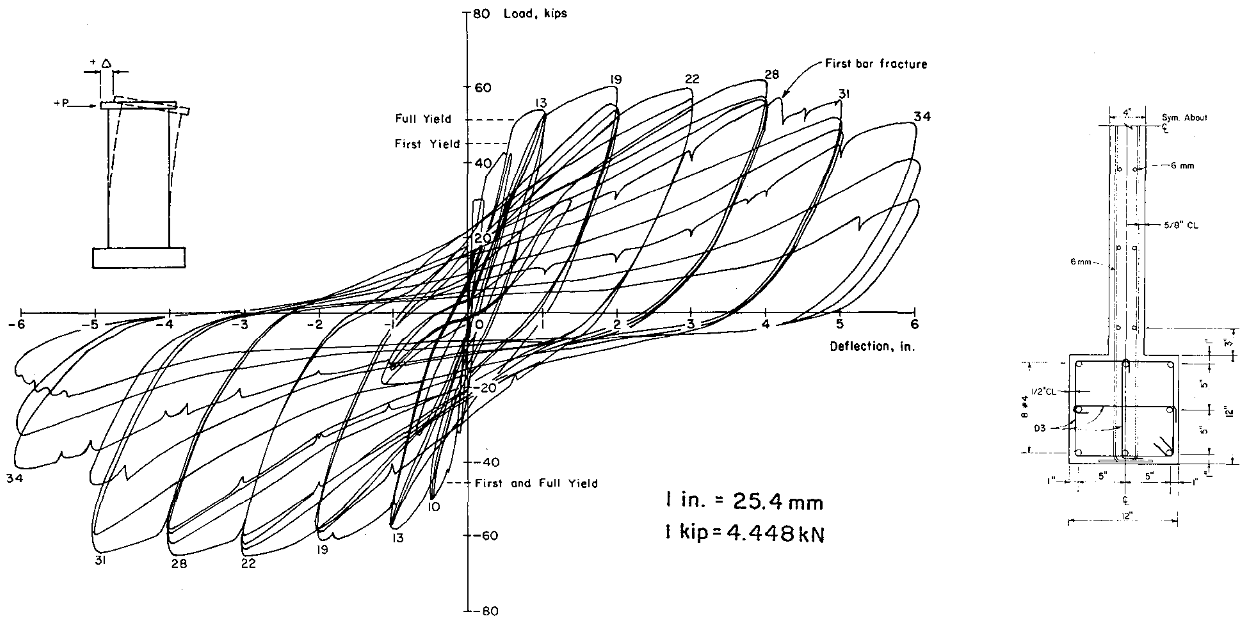


Fig. 6   Load Versus Deflection Relationship for Specimen B1

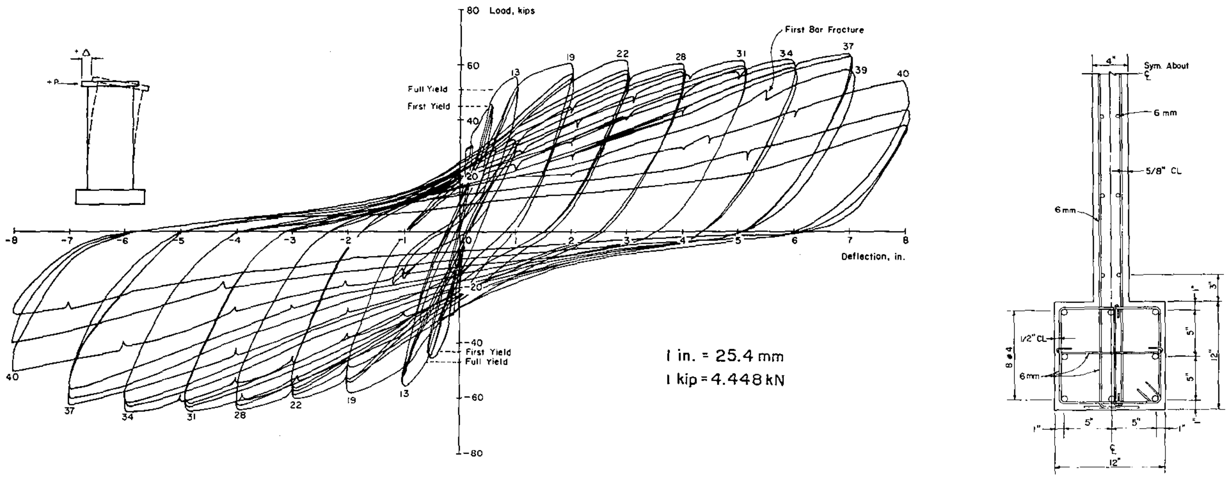


Fig. 7 Load Versus Deflection Relationship for Specimen B3

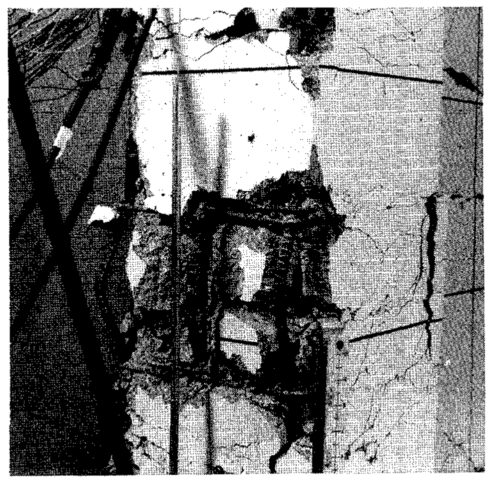


Fig. 8 Buckled Reinforcement in Specimen B1

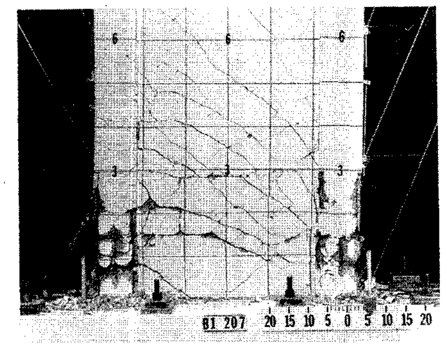


Fig. 9 Specimen B1 During Load Cycle 34

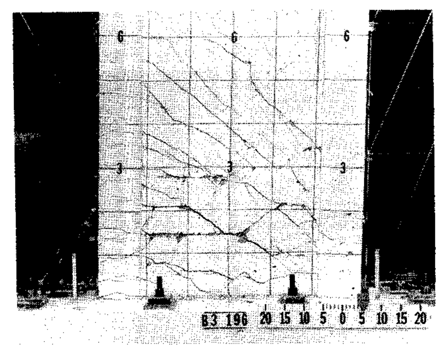


Fig. 10 Specimen B3 During Load Cycle 34

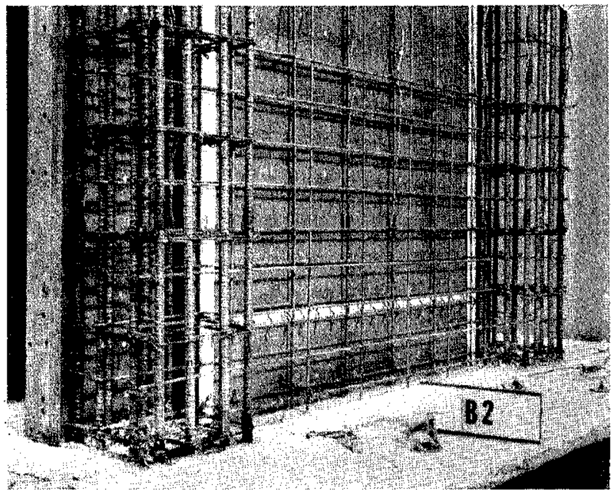


Fig. 11   Reinforcement for Specimen B2

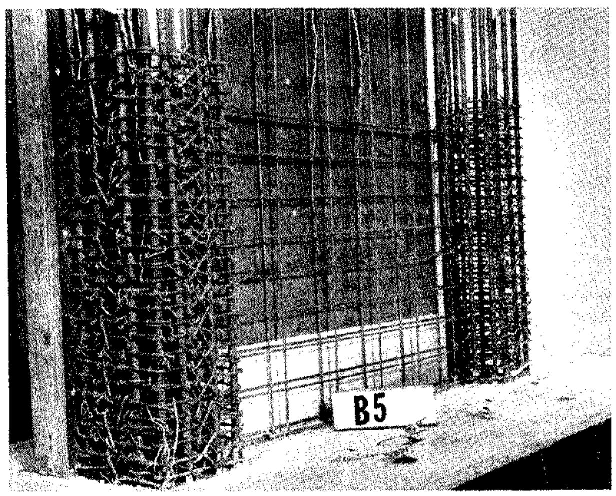


Fig. 12   Reinforcement for Specimen B5

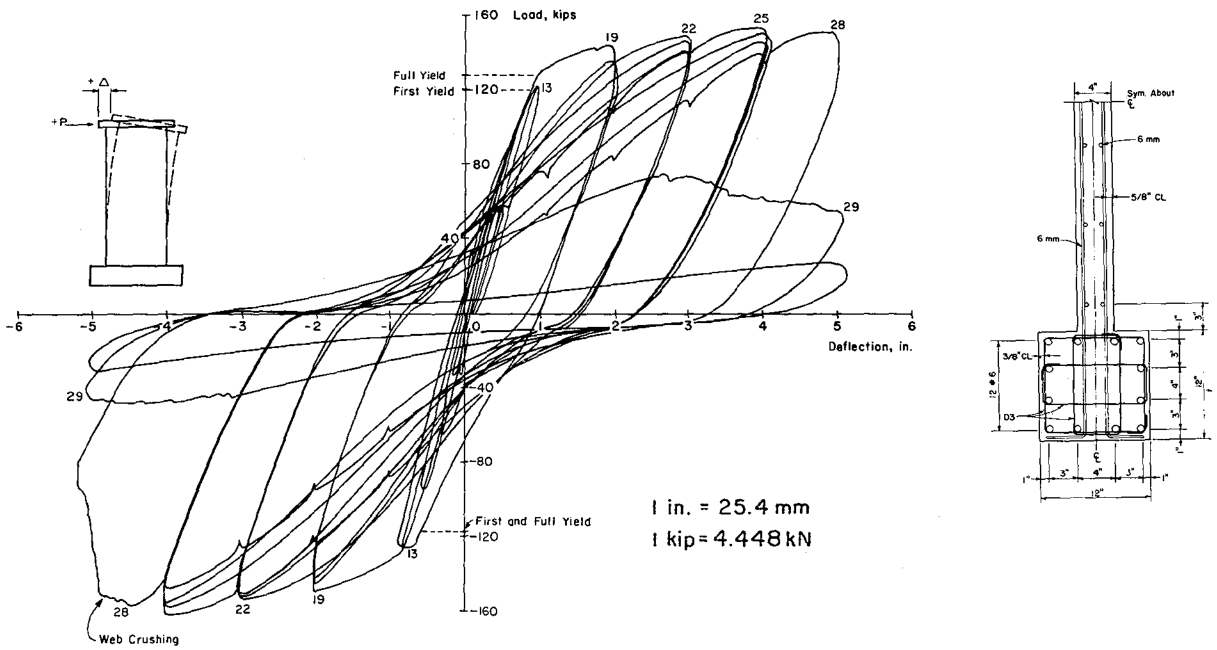


Fig. 13 Load Versus Deflection Relationship for Specimen B2

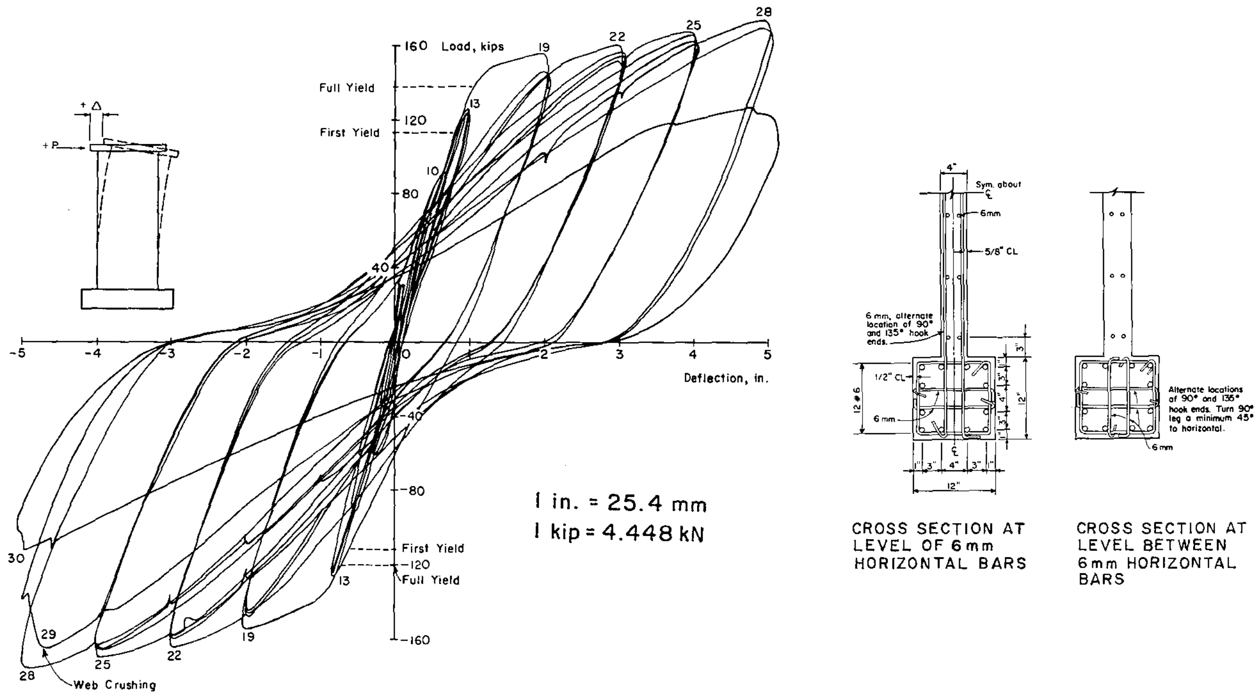


Fig. 14 Load Versus Deflection Relationship for Specimen B5

In Specimen B2, without confinement, the boundary elements deteriorated prior to web crushing. Several bars buckled and concrete was lost from the core of the columns as loads were reversed. In the last load cycle, the boundary elements were badly damaged near the base as shown in Fig. 15. Subsequently, web crushing occurred and the column was destroyed. Specimen B2 after web crushing is shown in Fig. 16.

In Specimen B5, confinement hoops prevented bar buckling and loss of concrete from the core of the boundary elements. They also reinforced the boundary elements for shear as can be seen in Fig. 17. Because of the confinement, Specimen B5 could be repaired simply by replacing the damaged web concrete.

Comparison of observed deformations in Specimens B2 with those of B5 indicated that confinement reinforcement decreased shear distortions for equivalent horizontal deflections. The improvement in shear stiffness was attributed to the confined boundary elements acting as stiff dowels.

Although only the lower 6 ft (1.83 m) of the boundary elements were confined, the primary zone of damage did not extend above this level. Strain gage data indicated that the only hoops stressed significantly were in the lower 3 ft (0.91m).

Confinement in Specimens B3 and B5 was provided by rectangular hoops. No spiral reinforcement was used. Tests at the University of California, Berkeley [1], indicate that, for vertical boundary elements of structural walls, closely spaced ties were as effective as spirals.

The benefits of transverse reinforcement for supporting vertical reinforcement, containing concrete, and improving shear resistance have also been illustrated under "field conditions". Figure 18 shows photographs of two columns in the grount story of the same building taken after the 1971 San Fernando earthquake. The effects of confinement provided by the spiral reinforcement are apparent.

Recommended Details

Recommended Details of confinement reinforcement for columns of ductile moment resisting frames are shown in Fig. 19. The detail shown for the square column was used for isolated wall Specimen B3 shown in Figs. 5 and 7.

The use of supplementary crossties with 180° hooks at each end caused numerous construction problems. Hoops and crossties had to be fabricated as a unit that was then slipped over the vertical reinforcement. To alleviate this assembly problem, the supplementary crossties for Specimen B5 were detailed with one 135° hook and one 90° hook as shown in Fig. 14. This arrangement permitted placement of crossties after the hoops were in place. The crossties were alternated end for end as construction progressed up the wall. Also, crossties parallel to the plane of the web were not provided at levels where the horizontal web reinforcement was anchored into the columns.

Reinforcement for Specimen B5 performed well, consequently it appears suitable for use as boundary element confinement.

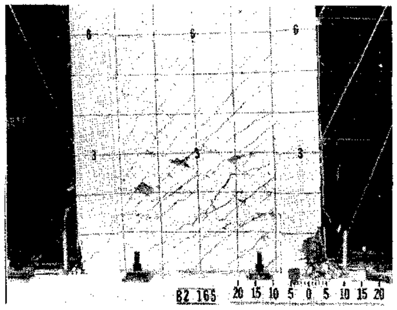


Fig. 15   Specimen B2 Immediately Prior to Web Crushing

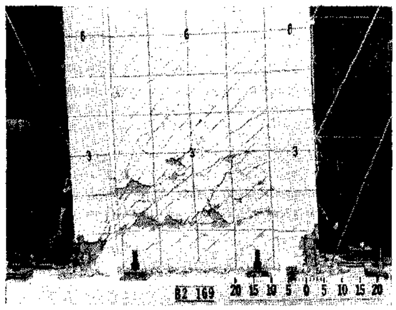


Fig. 16   Specimen B2 After Web Crushing

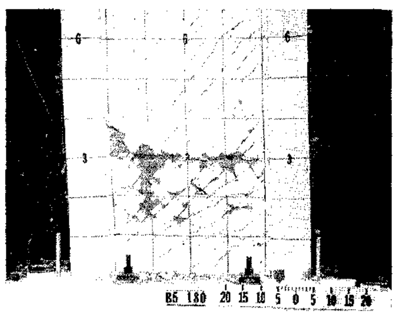


Fig. 17   Specimen B5 After Web Crushing

(a) Ordinary Tied Column

(b) Spiral Column

Fig. 18 Performance of Columns in 1971 San Fernando Earthquake

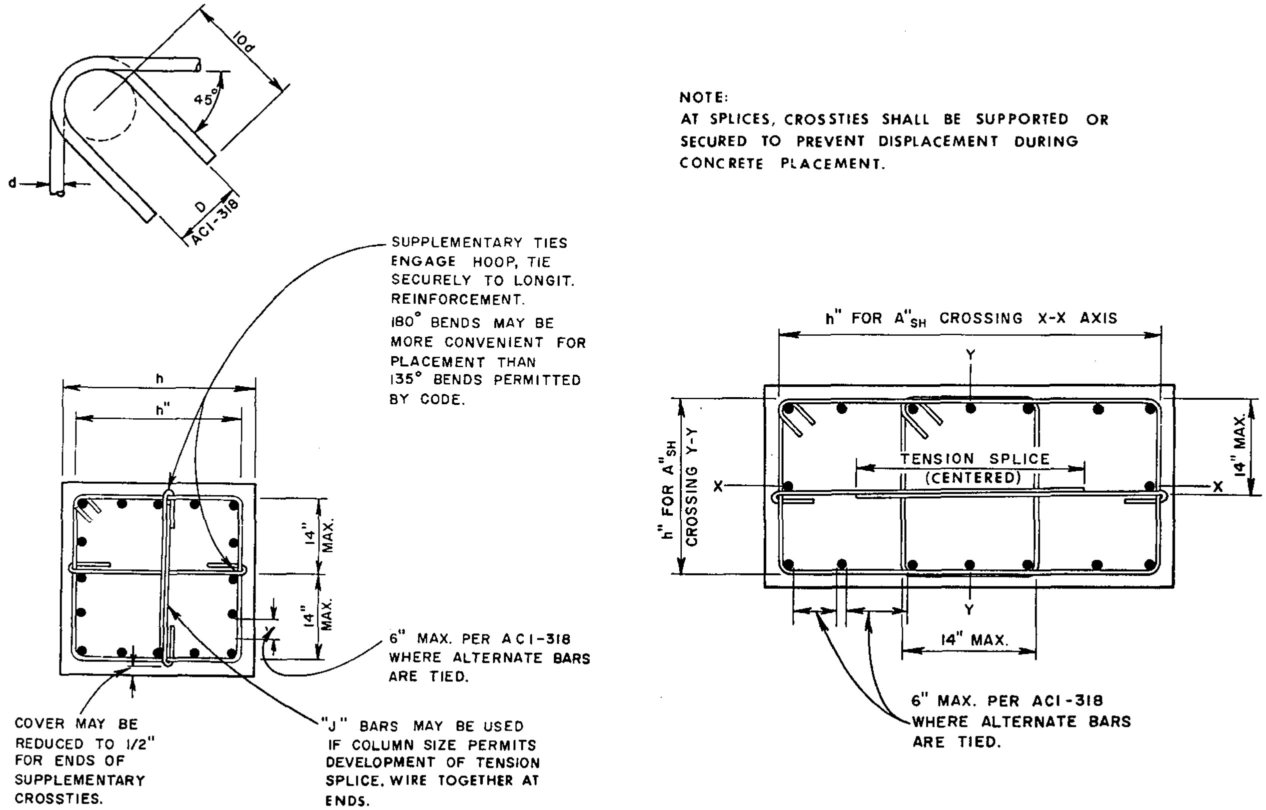


Fig. 19 Column Details for Ductile Moment Resisting Frames [8]

Based on observations from the isolated wall tests, a tension splice detail such as that shown for the rectangular column in Fig. 19 should not be used for vertical boundary elements in this hinging region. Because of severe cracking that can develop in the boundary elements under inelastic load reversals, it is likely that tension splices in the crossties would not be effective. This is particularly important if a spliced supplementary tie parallel to web of the wall is considered for shear resistance. Lap spliced crossties are not recommended for use in structural wall boundary elements within a hinging region.

Not all walls have column boundary elements. For rectangular walls and for intersecting or flanged walls other details are required. Figures 20 and 21 show examples of confinement details that can be used to build in boundary elements.

ANCHORAGE OF HORIZONTAL WALL REINFORCEMENT

Current code provisions permit horizontal reinforcement in the web of the wall to extend staight into the vertical boundary element. No hook is required on the end of horizontal bars. This type of horizontal reinforcement anchorage is shown in Fig. 22.

Reversing load tests of isolated walls indicate that straight bars may not provide adequate anchorage. The crack pattern developed in an isolated wall test specimen is shown in Fig. 23. Horizontal cracks in the tension boundary element propogate into diagonal web cracks. The horizontal cracks usually form at the levels of the horizontal bars because these bars form a weak plane against tension in the column. If the horizontal web reinforcement had been anchored into the column without an end hook, it is doubtful that it would have been as effective in resisting the shear forces. This is indicated by the observation that the hooks tended to open at later stages in the tests.

Within hinging regions of structural walls, it is recommended that horizontal web reinforcement be extended across the boundary element and terminated with a standard 90° bend. This was done for Specimen B3 shown in Fig. 7.

For walls subjected to levels of shear corresponding to $8\sqrt{f'_c}$ psi ($0.66\sqrt{f'_c}$ MPa) to $10\sqrt{f'_c}$ psi ($0.83\sqrt{f'_c}$ MPa), consideration should be given to the detail used for Specimen B5 shown in Fig. 14. With this detail, the wall reinforcement is anchored with either a 90° or a 135° hook. These are alternated end for end over the height of the wall.

As an alternative to the details given above, it appears that the horizontal bars could be terminated in the core of the boundary element with 90° bends in a vertical plane. However, the horizontal bar could not be considered to act as a supplementary crosstie if this detail is used.

RECOMMENDATIONS FOR RESEARCH

1. Provisions for confinement reinforcement in the 1971 ACI Building Code [3, 4] and the 1976 UBC Building Code [8,9] are based on criteria primarily related to increasing the strain capacity of the concrete and retaining the compressive strength of the core. The volume of required hoop

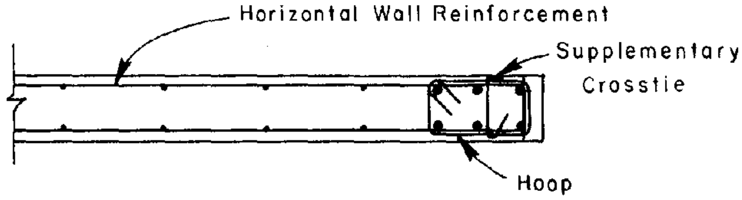


Fig. 20 Confined boundary Element Concealed in Rectangular Wall

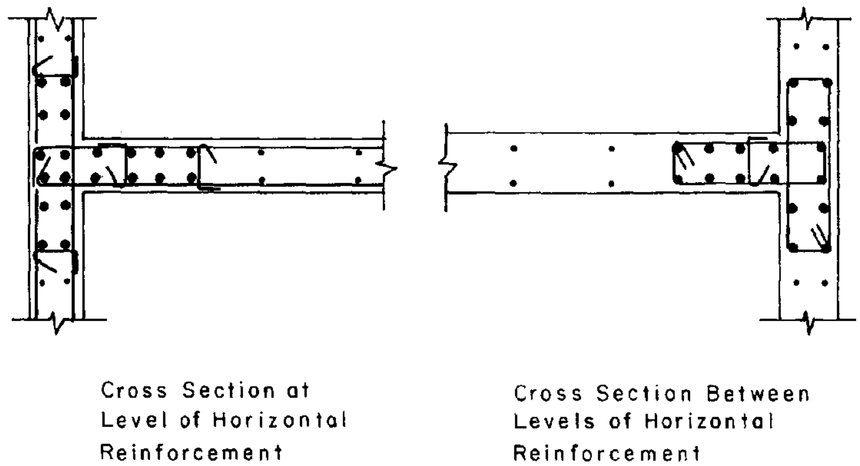


Fig. 21 Confined Boundary Element Concealed at Intersection of Walls

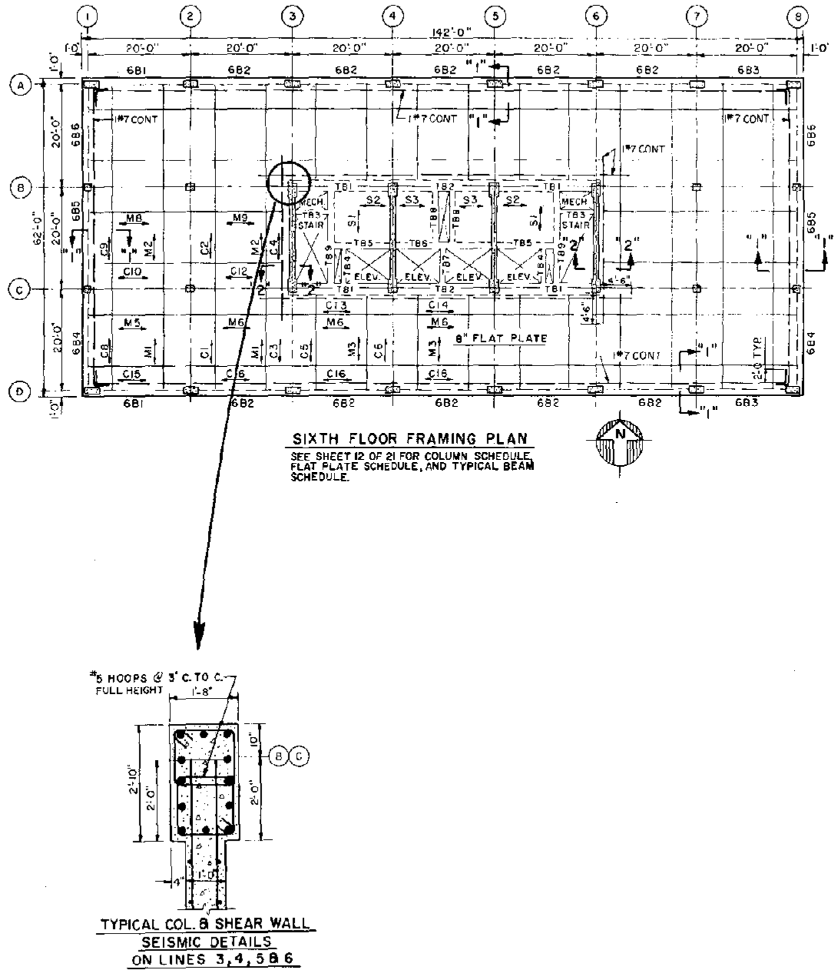


Fig. 22 Example of Structural Wall with Vertical Boundary Elements [6]

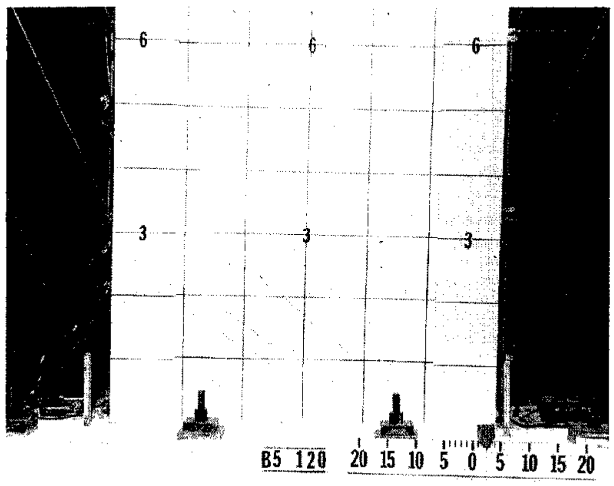


Fig. 23 Crack Pattern for Specimen B5

reinforcement was devised to provide the same average compressive stress in the rectangular core as would exist in the core of an equivalent circular spiral compression member. Research should be carried out to determine design criteria for required hoop size and spacing to delay inelastic bar buckling and to contain the concrete core.

2. Criteria for confinement reinforcement based on a limiting concrete strain can be important for walls with boundary elements having a high percentage of vertical reinforcement. Research is needed to determine the adequacy of confinement details for walls with a maximum six percent vertical reinforcement in the boundary elements.

3. Current codes require transverse confinement reinforcement over the full height of the vertical boundary element for structural walls under certain conditions. This provision should be investigated both analytically and experimentally. Tests of isolated cantilever walls indicate that confinement is only needed within the hinging region of the wall. If first mode effects dominate response, significant savings in reinforcement could result.

4. Research is needed on practical reinforcement details, especially for confinement reinforcement. Tests should be carried out to develop simple, economical and effective details. In addition, field trials should be made.

5. Tests of reinforcement splices for earthquake resistant construction are needed. Specifically, little information exists on the reliability of lap splices under severe seismic loading.

ACKNOWLEDGMENTS

This paper was written at the Structural Development Section of the Portland Cement Association, Dr. H. G. Russell, Manager. The wall test results are based upon research supported by the National Science Foundation under Grant No. ENV74-14766 and the Portland Cement Association. M. Fintel is Overall Project Director. Any opinions, findings and conclusions are those of the authors and do not necessarily reflect the views of the National Science Foundation.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

COUPLING BEAMS OF REINFORCED CONCRETE SHEAR WALLS

by

Thomas Paulay
Professor of Civil Engineering
University of Canterbury
Christchurch, New Zealand

Coupled shear walls of multistorey reinforced concrete buildings, when suitably placed, can offer particularly advantageous resistance against earthquake attack. Their stiffness will minimise damage during moderate disturbances. To meet the demands of very large earthquake excitations, suitably detailed coupled shear walls can be expected to be ductile enough to assure survival without collapse [1]. The major source of energy dissipation should be the coupling system because this will give some protection to the walls, which are more difficult to repair and in which large curvature ductilities may be difficult to develop without generating large compression strains in confined concrete.

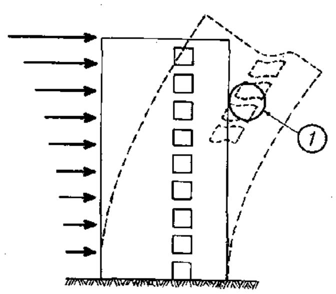

Fig. 1 Deformations in beams of coupled shear walls

Fig. 2 Coupling beams of a 14-storey building that failed during the 1964 Alaska earthquake [4]

Theoretical [1,2] and experimental studies indicated that very large ductilities may need to be developed in coupling beams if a substantial overall displacement ductility for the entire structure is to be assured. This becomes evident from the deformations that laterally loaded walls, such as shown in Fig. 1, will impose on such beams.

Coupling beams, formed between window or door openings, are often deep relative to their span. Because of this a relatively small amount of flexural reinforcement, usually arranged as shown in Fig. 9, will generate large shear forces that may dominate the behaviour of such beams. Typically the $V_u d/M_u$ ratio, a parameter commonly used in the evaluation of shear strength [3], is equal to or less than unity.

The response of coupled shear walls to recent earthquakes demonstrated clearly that the conventional approach to the design of such beams will result in brittle diagonal tension failures (Fig.2).

The first task of the designer is clearly to suppress a shear failure. However, even with web reinforcement resisting the entire shear that may be imposed on such beams, adequate ductility may not be assured. Because of the relative dimensions of such beams and the possible dominance of shear stresses, significant interaction between shear and flexure, traditionally disregarded in the routine design process, may be present. The subsequent sections briefly summarise the findings of a research programme with respect to the behaviour and design of coupling beams, undertaken at the University of Canterbury.

Conventionally Reinforced Coupling Beams

Conventional reinforcement in coupling beams normally consists of top and bottom flexural bars that are carried through the beams without splices or curtailment and are well anchored into adjacent walls. Because of the great depth some nominal intermediate horizontal bars would also be used for better crack control. Shear resistance would be provided by stirrups as shown in Fig. 9.

Fig. 3 Crack pattern in a conventionally reinforced coupling beam subjected to monotonic loading.

Flexural behaviour-- As Fig. 3 shows, diagonal cracks dominate the behaviour. They considerably affect the distribution of steel stresses and hence the deformation of the member.

When stirrups are provided in accordance with current code provisions [3], whereby some of the shear is assumed to be resisted by mechanism other than web reinforcement, inevitably a diagonal tension failure will result along the principal diagonal, seen in Fig. 3. The internal forces may be evaluated with the aid of the model shown in Fig. 4 where T is the force in the flexural reinforcement, D_1, D_2 are dowel shear forces, s is a stirrups force, P is the total shear to be transferred by the beam, g represents aggregate interlock stresses and p_s is the equivalent stirrup force per unit length.

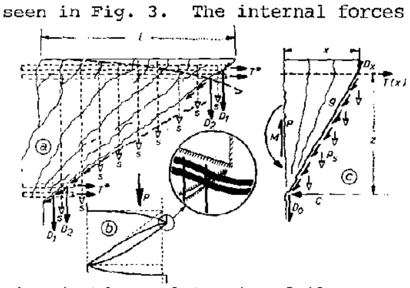

Fig. 4 Diagonal tension failure mechanism and internal forces in a coupling beam beam.

Fig. 4c shows a free body bound by a typical diagonal crack and this enables the following simple expression to be written to give the variation of the flexural tensile force, as a function of the distance, x, from the built in support thus:

$$T(x) = T_{max}\,[1 - \eta(\frac{x}{\ell})^2] \qquad (1)$$

where $\eta = V_s/P$ is the ratio of the contribution of the stirrup reinforcement to shear resistance, V_s, and the total transverse load P. In this equation the contribution of the dowel forces has been neglected.

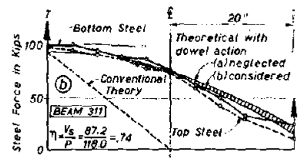

Fig. 5 Theoretical and observed tension force distribution along flexural reinforcement

It is evident that the steel force does not vary linearly with the imposed bending moments. When in Beam 311, shown in Fig. 3, the stirrups resisted 74% of the applied shear, the measured tensile forces were as shown in Fig. 5. The smooth curve shows the theoretical prediction of Eq. (1). Some consideration was also given to the likely contribution of dowel action, but as seen in Fig. 5, this was found to be insignificant.

Certain conclusions may be drawn from this finding:

1. For coupling beams with a small aspect ratio, the flexural reinforcement can be expected to be in tension over the entire span of the beam. A low stress area in the vicinity of zero bending, at midspan, does not exist and this should be noted when it is intended to splice bars near the point of contraflexure.

2. The design or analysis of the critical support sections can not be based on the customary assumptions of doubly reinforced concrete beams. Both the top and bottom reinforcement are in tension after diagonal cracking. For this reason the beneficial effect of the compression reinforcement upon ductility is not available.

3. The unconventional distribution of the internal forces suggests that a different approach to the assessment of distortions and stiffness characteristics of coupling beams is warranted.

4. In spite of the high intensity of shearing forces, the flexural bond stresses are not likely to be critical because the rate of change of the internal tension is considerably less than that of the bending moment.

5. Because the flexural reinforcement is in tension over the entire clear span the length of the beam increases with the load.

6. Because both the top and the bottom flexural bars are in tension, the internal tension resultant will be located between the levels of the top and bottom reinforcement. Fig. 5 indicates that the maximum total tension will occur near midspan at the region of zero moment. Consequently at this point the internal lever arm, z, must be zero. This suggests that such beams attempt to resist the variable moments along the span by means of near constant internal forces T = C operating on a variable internal lever arm, z.

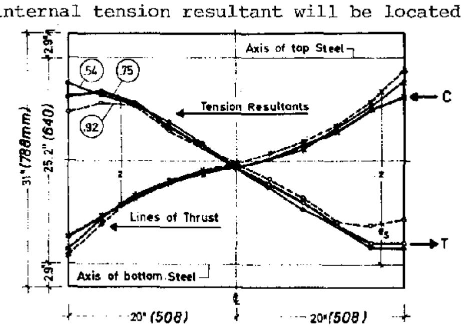

Fig. 6 The location of the internal stress resultant in a coupling beam at different stages of the loading.

Fig. 6 shows the location of the internal forces in a coupling beam, as determined from steel strain measurements, at various levels of load intensities. (The circled figures give the ratio of the applied load to the theoretical flexural failure load.)

It is also seen that the internal lever arm at the critical section is considerably smaller than what one would obtain from conventional flexural analysis of those sections. It is not surprising therefore, that the theoretical flexural capacity of deep coupling beams could not be attained.

Behaviour in shear-- The basis of the design of shear reinforcement in reinforced concrete beams is the traditional assumption that diagonal cracks form at 45° and that stirrups crossing these cracks are capable of resisting that fraction of the transverse force which is not resisted by the concrete. The shear allocated to the concrete is transferred through the compression zone of the beam, through dowel action of the flexural reinforcement and by means of aggregate interlocking across cracks [2]. In beams with small shear span to depth ratio a considerable portion of the transverse force may be resisted, after diagonal cracking, by arch action.

These concepts of shear resistance need to be modified when applied to coupling beams, such as shown in Fig. 3. The crack pattern indicates that the stirrups form part of a truss, the compression members of which radiate from the compression corners of the beam. The analogous truss so formed is highly indeterminate. From considerations of strain compatibility and the distribution of bond forces it is evident that in the elastic range the stirrup stresses along the beam can not be uniform. The most highly stressed stirrups are situated near the center of the beam. This is also evident when the deformations of the two triangular halves of the beam, shown in Fig. 4b, are compared. Numerous strain measurements along several points of a number of stirrups substantiated this behaviour.

With increasing load and consequent yielding of the stirrups at the center portion of the beam, the contribution of the aggregate interlock forces is diminishing. This situation is shown in Fig. 4a. Ultimately, only the stirrup and the dowel forces contribute towards shear resistance. However, the dowel displacements and the discontinuity of the flexural reinforcement at the corners, shown in detail in Fig. 4b, contribute towards the destruction of the compression area of the beam.

Note that; in spite of the small span to depth ratio, no arch action can develop in such beams after the yielding of the web reinforcement, because the reactive shear forces at the boundaries are applied over the full depth and not in concentrated form at the top and bottom surface of the beam.

To prevent a separation failure along a main diagonal, i.e. a diagonal tension failure, it is essential that the shear be transferred entirely by by web reinforcement and that no reliance be placed on other mechanisms which might assist in shear resistance.

The effect of cracking on stiffness -- The effect of cracking in reducing the stiffness of structural members is well recognised, even though it is seldom allowed for in routine design calculations. In elastic analyses leading to the evaluation of actions, usually relative stiffnesses will suffice. Since the effect of cracking is of the same order in beams and columns of moment resisting frames, relative stiffnesses will not be affected significantly.

In shear wall structures much larger differences in the stiffnesses of components can exist and the effect of cracking, in assessing, for example, the elastic response of a coupled shear wall, may be more important.

It must be appreciated that diagonal cracking has a much larger effect on shear stiffness than has flexural cracking on flexural stiffness [2]. Thus diagonal cracking will have a major influence on the overall stiffness of members in which shear distortions will dominate. This is the case for coupling beams with small span to depth ratios.

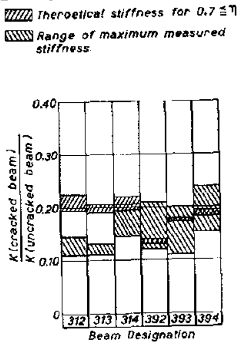

Fig. 7 The reduction of stiffness of coupling beams as a result of flexural and shear cracking.

With the aid of mathematical models, based on observed crack patterns, overall deformations in cracked coupling beams can be predicted with a satisfactory degree of accuracy [5]. However, such analysis would seldom be warranted because in earthquake resistant design the distribution of actions, that results from an elastic static analysis, should only serve as a guide with respect to the desirable distribtion of strength.

The order of the loss in stiffness of coupling beams, as a consequence of flexural and diagonal cracking, is shown in Fig. 7, where this loss is expressed as the ratio of the stiffness based on cracked sections to that based on uncracked sections. A comparison with theoretical predictions [5] is also made. The dimension of the beams designated 312 to 314 are as shown in Fig. 6. Those designated 392 to 394 are 39 in (990 mm) deep. It is seen that the loss of stiffness due to cracking is of the order of 85%, a quantity significant enough to be considered in the design process.

Fig. 8 Diagonal tension failure in coupling beams subjected to reversed cyclic loading.

The effects of reversed cyclic loading beyond the elastic limits -- Because of relatively large shearing forces that are usually present, diagonal tension failure in coupling beams with web reinforcement in accordance with traditional code requirements [3], is accentuated. The complete failure of such beams was observed in Achorage (Fig. 2 and Fig. 8a) and in tests, such as shown in Fig. 8b.

When the strength of the stirrups, crossing the crack along the main diagonal of a deep spandrel beam, is equal to or larger than the external load, then failure is confined to one of the support sections. Such beams can repeatedly attain their flexural capacity. However, in beams with span to depth ratio of less than 2, the ultimate moment capacity, predicted by a Whitney analysis, cannot be developed. The high shear force causes the flexural reinforcement to be in tension at the top and the bottom of the beam (Fig. 5). Therefore the internal lever arm of the stress resultants is smaller than in a conventional beam possessing the same sectional properties (Fig. 6). With cyclic loading this is further accentuated. Experiments indicated that after high intensity load reversals the attainable flexural strength is only about 85% of that predicted by a Whitney analysis [6] (Fig. 9).

Prior to failure the entire shear force must be carried by the concrete through the compression zone and by means of dowel action of the horizontal reinforcement across a vertical section, adjacent to the support. This shear force is to be transmitted by aggregate interlock friction because the compression zone is already severely cracked. In the process of closing a wide crack, particularly at low loads, shear displacements occur. These may be enhanced by the yielded flexural reinforcement which can delay the closure of the crack. The shear displacement prevents a perfect fit between the two faces of a crack. Very high local bearing stresses, dislocations of aggregate particles, numerous small new cracks and, later, the filling of the crack with debris ensue. These phenomena reduce the frictional resistance of the compression zone to such an extent that large shear displacements may occur. Such a local slip is evident at the right hand support of the beam shown in Fig. 9. At this stage no stirrup or perhaps one or two, cross the failure crack, along which the shear slip takes place. These stirrups and the dowel contribution of the flexural reinforcement, cannot arrest a failure once sliding across the pulverized plane of the compression zone commences.

The load displacement relationship for a beam containing adequate web reinforcement, shown in Fig. 9, demonstrates that only limited rotational ductility could be developed and that after one large flexural yield excursion, such as applied in the 6th cycle of loading, a sliding shear failure is imminent. The shear sustained after substantial sliding, as in cycle 7 of the test beam, is primarily due to the kinking (dowel) action of the flexural reinforcement. The rotational ductilities observed in such beams (Fig. 9) fall considerably short of the magnitudes predicted from theoretical studies of coupled shear walls.

It must be concluded that in conventionally reinforced coupling beams, with a span to depth ratio of less than 2, the strength and ductilities, desired in earthquake resistant coupled shear walls, are not likely to be attained.

In more slender beams two distinct plastic hinges will appear. The behaviour of such beams is the same as that of beams of moment resistant frames. However, the transfer of shear across plastic hinges in such relatively slender beams is also likely to be critical and hence it deserves particular attention.

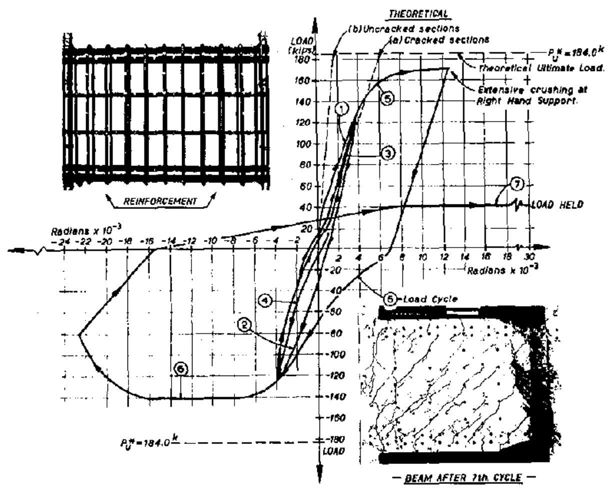

Fig. 9 Load-rotation relationship for a conventionally reinforced coupling beam designed to fail in flexure.

Diagonally Reinforced Coupling Beams

Experiments at the University of Canterbury revealed [2, 7] that the ductility and useful strength of coupling beams can be considerably improved if instead of the previously described conventional steel arrangement, the principal reinforcement is placed diagonally in the beam. The design of such a beam can be based on the premise that the shearing force resolves itself into diagonal compression and tension forces, intersecting each other at midspan where no moment is to be resisted, as in Fig. 10. Initially the diagonal compression is transmitted by the concrete, and the compression steel makes an insignificant contribution. After the first excursion of the diagonal tension bars into the yield range, however, large cracks form and remain open when the load is removed. When the reversed load is applied, as during an earthquake, these bars are subjected to large compression stresses, perhaps yielding, before the previously formed cracks close. Accordingly, at the development of the yield strength, Figs. 10a and 10b give

$$T_u = C_u = A_s f_y \quad \text{and} \quad V_u = 2T_u \sin\alpha$$

hence

$$A_s = \frac{V_u}{2f_y \sin\alpha} \tag{2}$$

where

$$\tan\alpha = \frac{h-2d'}{\ell} \tag{3}$$

The resisting moment at the supports of the beam (Fig. 10b), may be found if desired, either from the shear force, that is,

$$M_u = \frac{V_u \ell}{2} = \ell\, T_u \sin\alpha \tag{4a}$$

or from the horizontal components of the diagonal forces, that is,

$$M_u = (h-2d')\,T_u \cos\alpha \tag{4b}$$

Since equal amounts of steel are to be provided in both diagonal bands, the loss of the contribution of the concrete is without consequence, provided the diagonal compression bars do not become unstable. For seismic-type loading it is therefore important to have ample ties around the diagonal compression bars, to retain the concrete around the bars. The main purpose of the retained concrete is to furnish some lateral flexural rigidity to the diagonal strut, thus to enable compression yielding of the main diagonal bars to take place. Where beams only 6 in (150 mm) thick were studied, buckling failures were clearly identified [7].

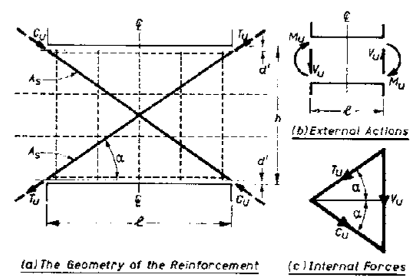

Fig. 10 Geometry of diagonally reinforced coupling beams.

Because the concrete, apart from stabilizing the compression bars, has no influence on the behaviour of diagonally reinforced coupling beams, no degradation in strength or stiffness is to be expected during alternating cyclic loading that imposes moderate ductility. Fig. 11 gives the load-rotation relationship for a beam having the same overall dimensions as that in Fig. 9.

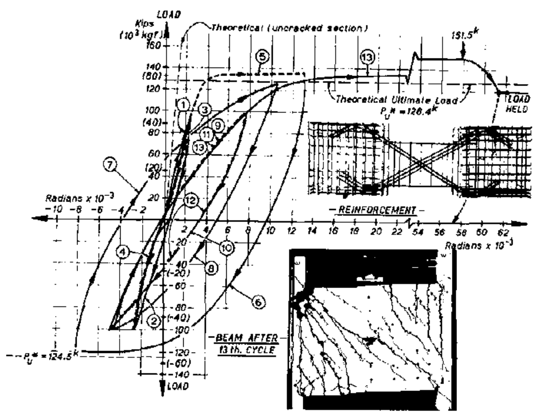

Fig. 11 Load rotation relationship for a diagonally reinforced coupling beam.

The hysteresis loops for this beam have the characteristics of a steel member. Strength degradation occurs only when the buckling of the compression bars commences. When load reversals occur, however, these bars can take up tension and straighten themselves. The process leads eventually to the complete breaking up of the concrete around the compression bars, hence to further loss of restraint against buckling and consequent loss of strength.

The superior response of these diagonally reinforced beams under high-intensity alternating loading can be seen in a comparison of the reduction in strength with the cumulative ductility imposed during cyclic loading on beams tested, shown in Fig. 12. The span depth ratio ℓ/h of these beams varied between 1.03 and 1.29. Fig. 13 illustrates the reinforcement for such a beam used in a building in New zealand.

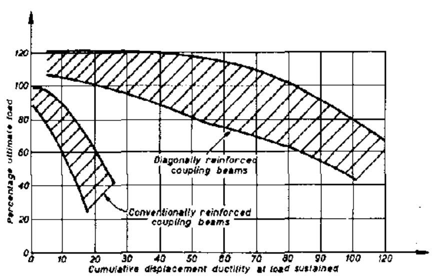

Fig. 12 Cumulative ductilities imposed on conventionally and diagonally reinforced coupling beam.

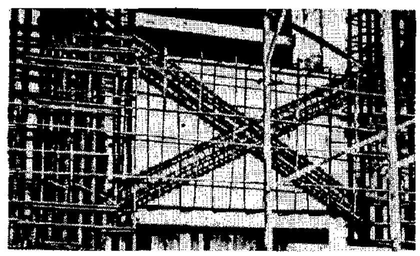

Fig. 13 A diagonally reinforced coupling beam as constructed.

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FOUNDATIONS AND RETAINING STRUCTURES

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

SEISMIC ROCKING PROBLEM OF RIGID COMPENSATED FOUNDATIONS

by

Leonardo Zeevaert
Professor at the Graduate School of Engineering
Universidad Nacional Autónoma de México
Consulting Engineer

I. INTRODUCTION

In earthquake areas the response of a building foundation to the strong ground motion is important in the design of the foundation structure and building structural frame. To achieve a rational design it is necessary to learn with reasonable accuracy on the subsoil conditions and on the earthquake characteristics at the site in question.

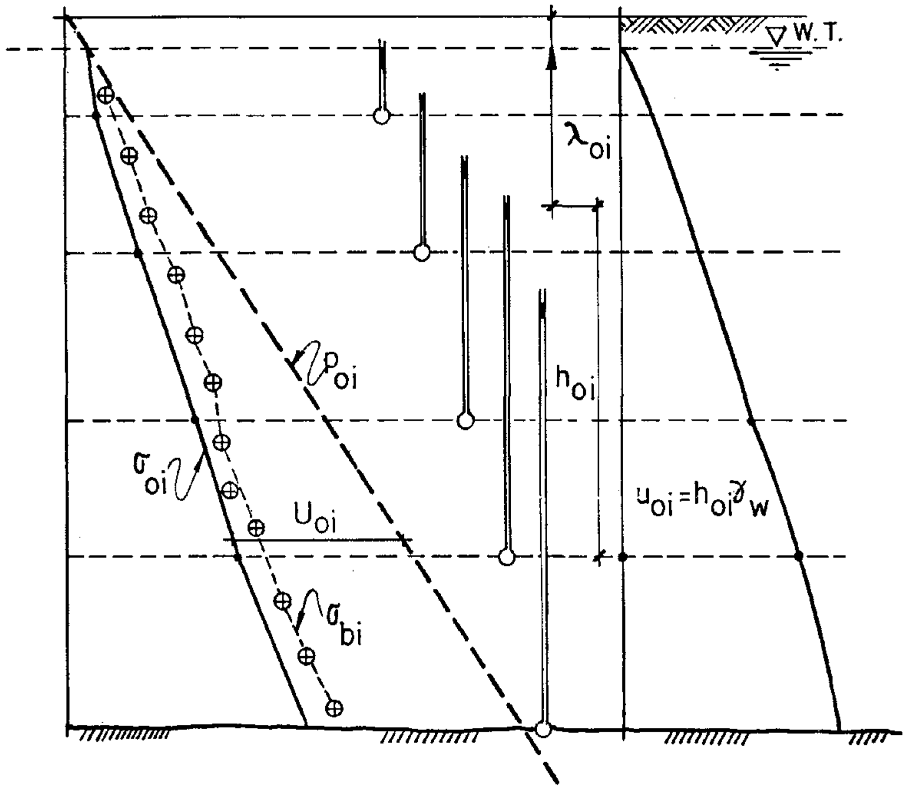
The site investigation is performed from the physiographical and geological point of view and learning on the hydraulic conditions prevailing in the area where the building will be constructed and at the time the mechanical properties of the subsoil materials are investigated. Compensated foundations are only indicated in deep and highly compressible sediments usually of lacustrine or marine origin in which case, the stratigraphy may be found concordant in large areas [a].

The hydraulic conditions at the site are investigated by means of piezometers installed at different depths covering the total thickness of the deposit to a depth where firm ground may be encountered, or to a depth at which the influence of the construction of the foundation may be negligible. Using the piezometric readings and the index properties of the materials, the total and effective stresses at the site are determined, Fig. 1. The hydraulic conditions in the environment of the site are also investigated for future possible changes. There are areas where large pumping is taking place and the drawdown of the piezometric water levels produces ground surface subsidence that should be taken in consideration when designing a compensated foundation.

The vertical water flow component increases the effective overburden stresses in proportion to the piezometric water elevation drawdown. The total pressures and the effective stresses are indicated in Fig. 1. They may be calculated from the piezometric water heads determined at different depths within the soil deposits and from the unit weight of the soil determined from the index properties.

In clay sediments the shear strength is estimated continuously along the entire soil profile by means of the natural consistency of the soil. The coefficient of permeability is investigated for the pervious layers that may be considered drainage fields to assign the thickness of the strata for consolidation purposes, and to estimate the water flow during excavation. The elastic response strain modulus under static loading and the elasto-plastic coefficient of the soil are important. The first one is used to evaluate the elastic heave during excavation, therefore, it shall be determined in every layer, where different response may be expected. The elasto-plastic properties of the soil should be investigated to evaluate permanent deformation induced in the soil mass by the foundation subjected to large overturning moments [a].

The compressibility of the soil is achieved by means of compressibility tests from



STRATIGRAPHY, HYDRAULIC CONDITIONS, EFFECTIVE AND TOTAL STRESSES, AND CRITICAL STRESS

FIG.- I

STRATUM	I	II	III	IV	TEST
Depth					
Classification					
Water content					w
Specific gravity					s_s
Degree of saturation					S%
Unit weight					γ_n
Unconfined compression strength					q_u
Coefficient of permeability					k
Response elastic strain modulus					M_e
Elasto-plastic coefficient					$\mathcal{K}_{ep}$
Compressibility test C_u and consolidation parameters					m_{ep}
					m_t
					c_v
Dynamic shear modulus of elasticity					μ
shear wave velocity					v_s
Overburden stress					σ_{oi}
Critical stress					σ_b

MECHANICAL PROPERTIES OF SEDIMENTS AND RELATED INDEX PROPERTIES

FIG-2

which the consolidation parameters for primary and secondary consolidation are investigated. The secondary consolidation is explained by the phenomenon of intergranular viscosity [a]. The consolidation properties are carefully investigated since the main object of a compensated foundation is to forecast the settlement with time after the soil mass is recompressed.

The dynamic shear modulus of elasticity and corresponding shear wave velocity is determined to investigate the dynamic behavior of the ground. With this information the structural engineer and the foundation engineer evaluate the earthquake effects on the structure and its foundation.

The mechanical properties of the soils mentioned above are determined with accuracy in undisturbed soil specimens representative of each one of the strata that may be considered different from each other. In general it is found that the soil changes its mechanical properties with depth. The compressibility reduces and the elastic properties increase. The strata at greater depth show more favorable mechanical properties. A table showing the engineering characteristics of soil required to design a compensated foundation are tabulated in Fig. 2.

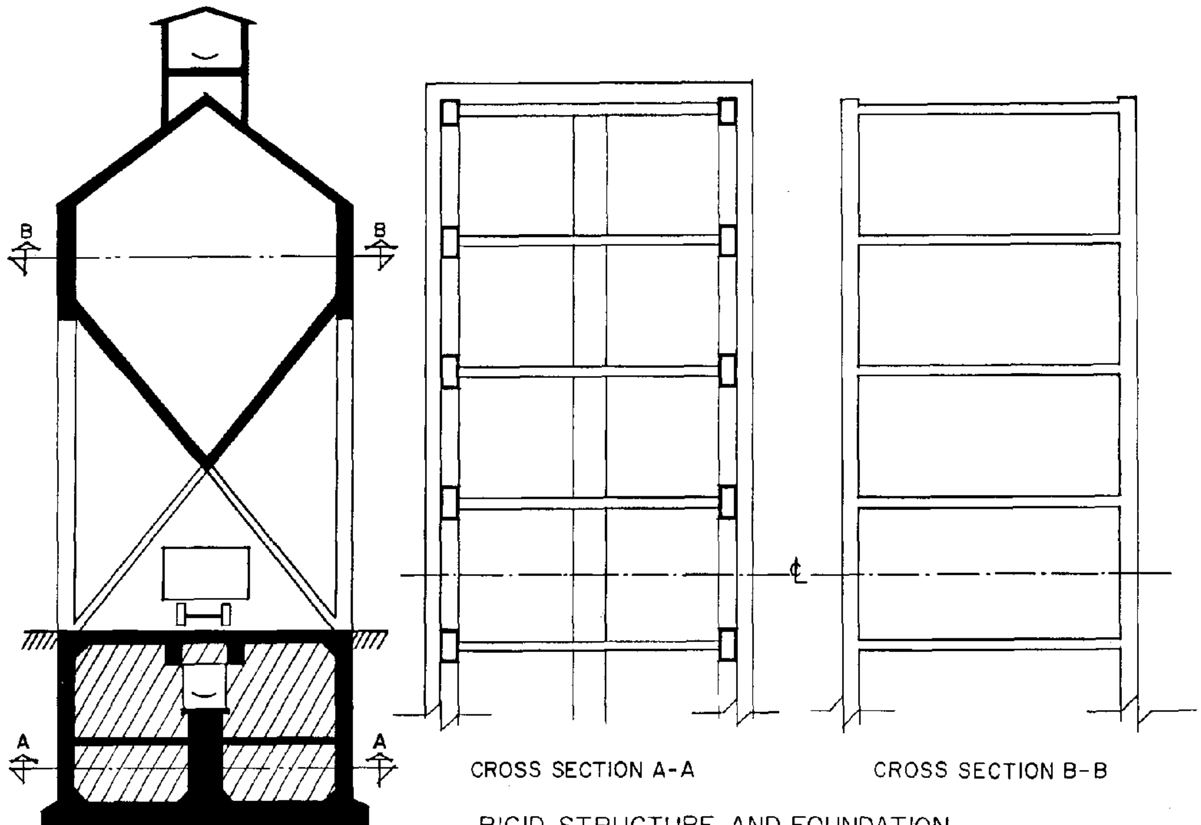
The earthquake characteristics are investigated from previous experience of recorded strong ground motions in the area where the building is going to be erected. The main problem is to assign a proper maximum ground acceleration to design the building. In many instances there are no previous records that may be used to investigate the probable ground accelerations, and it is necessary to make use of equal earthquake intensity charts that may be available for that region. On the other hand, general atenuation empirical equations of acceleration against distance may be used that may apply in the region under consideration[b].

Another important item, yet in state of investigation is the magnification factors for different degrees of critical damping in the soil mass [c] [g]. As it is well known that the response acceleration is a function of the equivalent period of vibration of the structure and of the damping phenomenon.

With the knowledge of the earthquake characteristics at the site and the dynamic mechanical properties of the soil, the foundation engineer is able to estimate the subsoil dynamic behavior and its implications or interaction on the foundation structure of the building. The seismic response of the foundation is then calculated to determine the soil reactions on the foundation structure and from there on using the building loads, the shears and bending moments induced by the seismic action may be determined.

There are many structures on compensated foundations deep seated in the ground that are very rigid in their structural frame including the foundation structure designed as a monolithic box, as the one shown in Fig. 3, for a grain bin. When such is the case, the period of vibration of the structure in itself is very small and the important factor is the rocking phenomenon. This paper is devoted to this important problem in the aim to analyze such a rigid structure and estimate the seismic response of its foundation. However, the general short-hand procedure given in this paper may be used for any other type of structural frame, except that the structural dynamic properties should be known to be included in the calculations, [h].

When the seismic response of the foundation structure is known the contact stresses against the retaining walls and base slab are calculated and the stability from the soil



ELEVATION

CROSS SECTION A-A

CROSS SECTION B-B

RIGID STRUCTURE AND FOUNDATION

FIG.- 3

mechanics point of view revised. That is to say, the factors of safety should be determined for the passive earth pressure developed by the strong ground motion against the sides of the foundation box and the ultimate bearing capacity at the edges of the foundation base slab. At these places large stresses induced by the overturning moments may produce soil failure or plastic action in a limited zone of the foundation contact area. When these plastic zones are large the foundation structure and thus the building will suffer an important permanent tilt during the earthquake.

II. EARTHQUAKE CHARACTERISTICS

The proper rating of an earthquake at the particular site is very important for the economy and design of a foundation. The location of the source of the seismic motion may be fairly well known as reported by the seismological stations in terms of their latitude, longitude and depth at the epicentral zone. The fault zones are better known every day and the studies on the probable Richter Magnitude to be expected is progressing rapidly. More difficult, however, is to assign a seismic intensity in areas where previous records are not available. Nevertheless, for this purpose charts of equal seismic intensity may be used or statistical atenuation formulas for acceleration vs distance, as the one developed by W. K. Cloud (b), giving the upper bound of destructive earthquakes

$$\log a = 6.17 - \log (3900 + D^2) \tag{1}$$

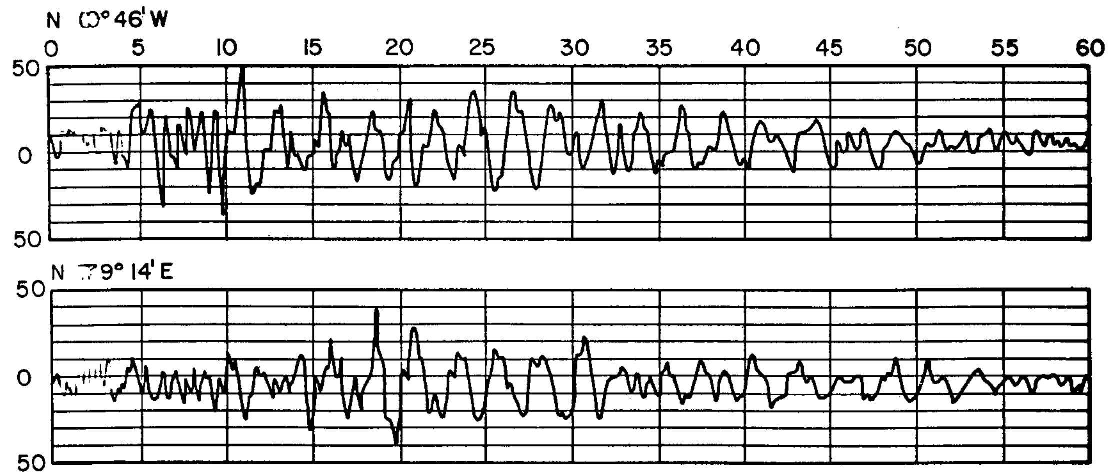
in which the acceleration a is given in cm/sec^2 and D the distance in miles from the epicenter. The table I, was prepared to give an idea of the importance of the seismic intensity with the distance from the epicenter in miles (d).

TABLE I

Distance D Miles	Maximum ground acceleration in cm/sec^2
5	377
20	344
40	269
80	144
100	106
150	56
(165)	(47.5) Mexico, D. F. July 28, 1957
200	34

For foundation design purposes it is important to know the maximum ground acceleration and the characteristics of the response spectrum. The surface ground acceleration may be recorded by means of accelerographs and the maximum acceleration determined, Fig. 4. The accelerogram so obtained shows a series of random impulses with a few large peaks.

The maximum response of a vibrating system resting on the ground surface can be determined integrating these impulses to obtain the maximum by means of the well known expression

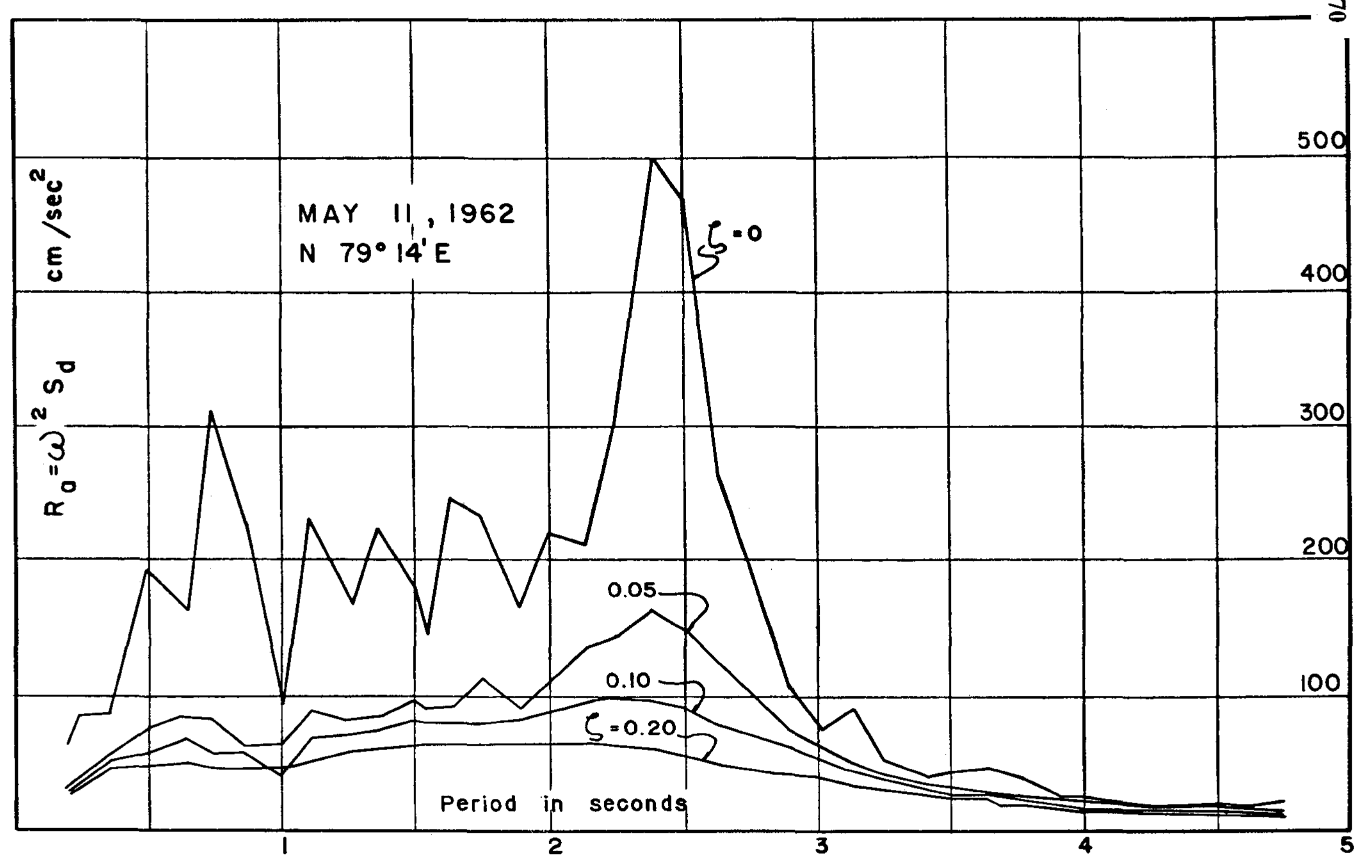


EARTHQUAKE: MAY. 11, 1962

The origin corresponds to 33 seconds after the accelerograph started.
Data from a photo copy from the original

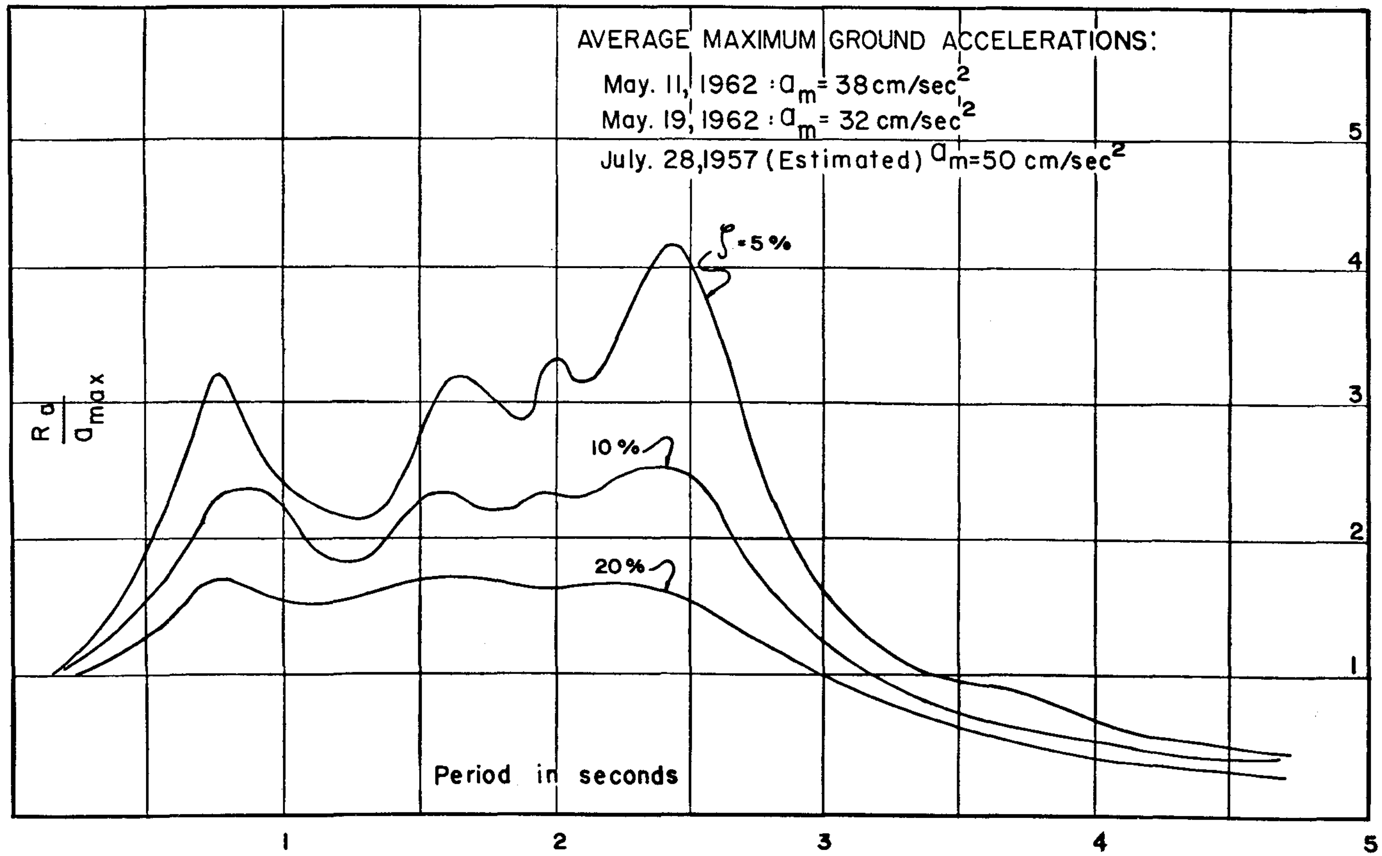
ACCELEROGRAM MAY 11, 1962 MEXICO CITY

FIG. - 4



PSEUDO-ACCELERATION SPECTRUM MAY 11, 1962
MEXICO CITY

FIG.-5



ENVELOPE RELATIVE ACCELERATION RESPONSE RATIO SPECTRUM
MEXICO CITY

FIG.-6

$$R_v = \int_0^t a(\tau)\, e^{-\zeta\omega(t-\tau)} \sin\omega_d (t-\tau)\, d\tau \Big/_{max} \tag{2}$$

The above expression[e] [f] gives the maximum response in terms of the relative pseudo- velocity as a function of the equivalent frequency of one degree of freedom systems. Here ω is the circular frequency and ζ a fraction of the critical damping of the system. The response acceleration

$$R_a = R_v \cdot \omega \tag{3}$$

The pseudo - acceleration spectrum for the accelerogram shown in Fig. 4 is given in Fig. 5 for the response of one degree of freedom systems, and for the case of Mexico City, May 11, 1962 strong earthquake. Hence, entering with the natural period of vibration of the system the response acceleration R_a may be obtained and the seismic shear calculated. If the mass of the system is called M, then the seismic horizontal force at the center of mass of the structure is

$$V_m = M \cdot R_a \tag{4}$$

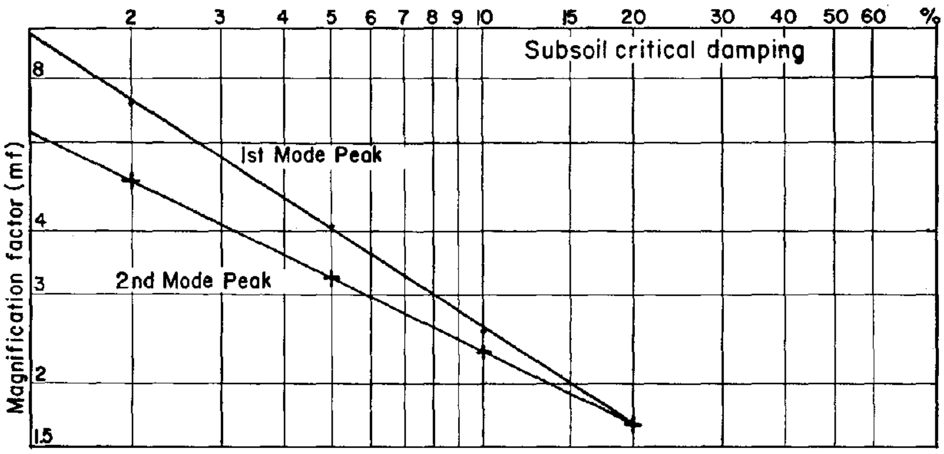
Furthermore, for that particular pseudo - acceleration response spectrum a maximum ground acceleration was recorded in the accelerogram, let us call it a_m, and write

$$V_m = M \left(\frac{R_a}{a_m}\right) a_m \tag{5}$$

hence, the spectrum may be visualized in terms of the ratio R_a/a_m representing an acceleration magnification factor, as shown in Fig. 6 for the envelope of pseudo-acceleration response spectra for Mexico City center.

Every spectrum has certain characteristic values at the peaks representing an important resonance of the vibrating systems with the soil mass. For instance in case of Mexico City the magnification factors observed for different degrees of critical damping [c] [g] at definite periods representing the 1st and 2nd vibration modes of the soil mass are shown plotted in Fig. 7. The author has compared these values with other records and it appears that the magnification ratios are practically independent of the subsoil characteristics provided the subsoil is fairly uniform from the ground surface to firm ground. These values may have characteristics representative of certain seismic area. Calling T_{s1} the dominant period of the ground the envelope response spectra may be constructed in terms of R_a/a_m vs T_o/T_{s1} for different degrees of critical damping ζ, as shown in Fig. 8. Therefore, to learn approximately on the response acceleration ratio R_a/a_m or acceleration magnification factor it will be necessary to calculate from the stratigraphical and dynamical properties of the soil mass the dominant periods of the ground T_{s1}, and T_{s2}[a], and make an estimate of the maximum ground acceleration a_m. Moreover, it is required the determination of the equivalent periods of vibration and critical damping of the foundation and building system T_o and ζ_o respectively, [h]. Hence, computing the ratio: T_o/T_{s1}, and entering the design envelope spectrum for corresponding critical damping ζ, the value of R_a/a_m may be obtained, Fig. 8.

Using the maximum assigned ground acceleration a_m the seismic shear V_m is



MAGNIFICATION FACTORS OBSERVED FOR MEXICO CITY SUBSOIL

FIG.-7

SEISMIC DESIGN ENVELOPE SPECTRA

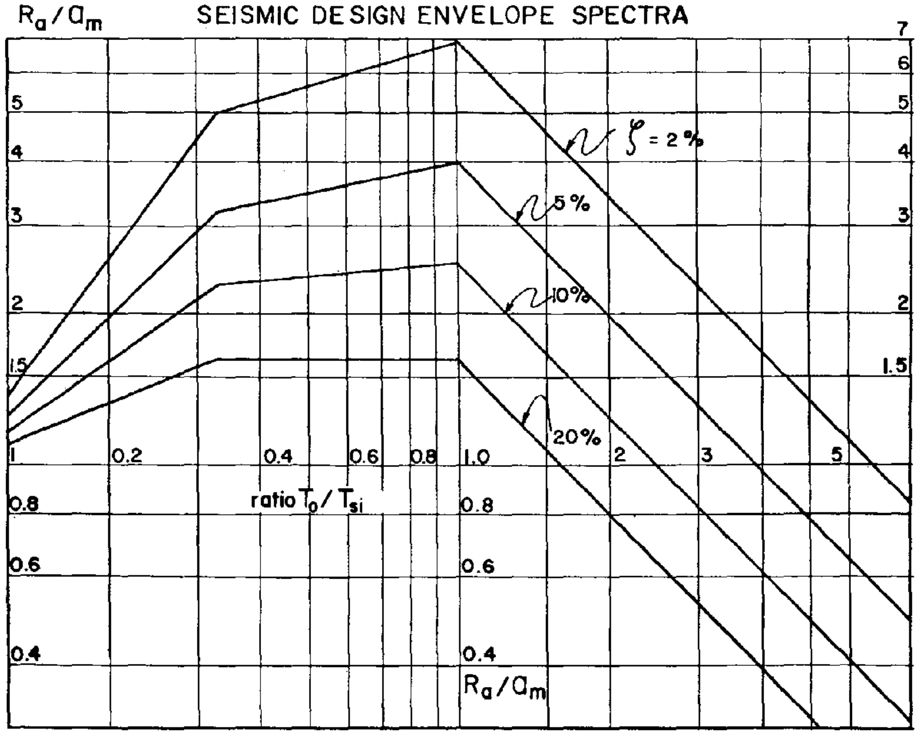


FIG.-8

calculated by means of formula (5). The value so obtained is used to investigate the forces taken by the foundation structure as discussed in further paragraphs.

III. SUBSOIL CONDITIONS

A rational seismic foundation design requires the knowledge of the stratigraphical, hydraulic and dynamical properties of the subsoil to a depth at which the soil properties will not influence the physical behavior of the foundation. The soil profile is determined as a minimum to a depth equivalent to one and one half times the average dimension of the foundation area or to a depth at which it is considered that the influence of the foundation action is negligible. The soil weight is required to calculate the total pressure in the soil mass p_{oi} . The hydraulic pressures in the ground are determined by means of sufficient piezometers installed at the strata interfaces, $U_i = \gamma_w \cdot h_i$ other values may be interpolated considering the permeability properties of the strata. The effective stress profile is determined by means of

$$\sigma_{oi} = p_{oi} - U_{oi} \tag{6}$$

The vertical effective stress is important because it is necessary to calculate the average volumetric confining stress

$$\sigma_c = \frac{1}{3}(1 + 2K_o)\sigma_{oi} \tag{7}$$

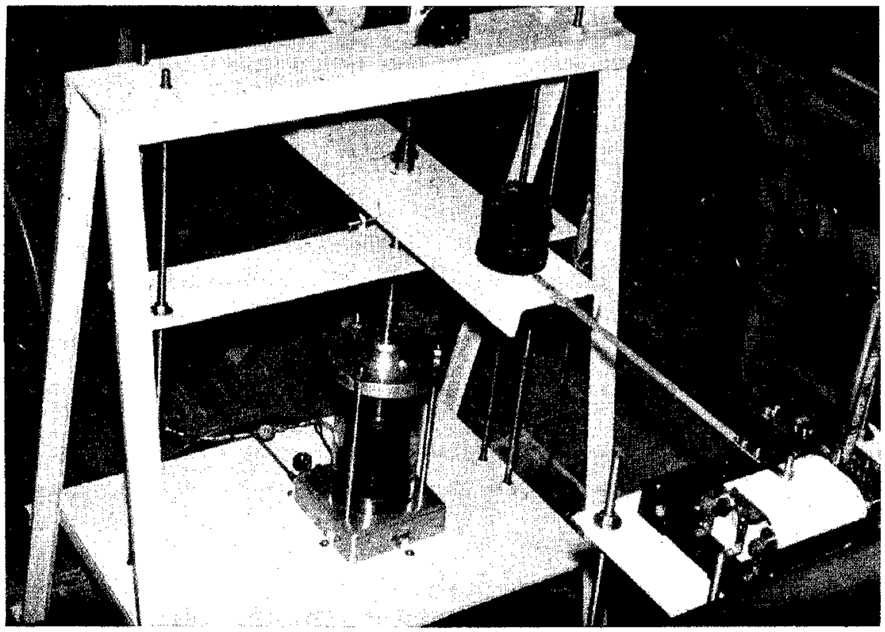
In this expression K_o represents the ratio of the horizontal to the vertical stresses at rest. To estimate the soil mass behavior during an earthquake it is required to know the value of the dynamic shear modulus of elasticity of the soil. This mechanical property is a function of the confining stress in the ground. The phenomenological laws the author has found governing this property (a) (i) may be stated as follows:

1) For Clays $$\mu = \mu_o\, e^{n_s \sigma_c} \tag{8}$$

2) For Sand $$\mu = C_s\, \sigma_c^{\,n_s} \tag{9}$$

in which σ_c is the volumetric confining stress given by (7). The first of these equations (8) for clay soils states that when the soil is unconfined the initial shear modulus of elasticity is μ_o , in contrast with the second law equation (9) for cohesionless soil that when the confining stress σ_c is very small the value of the dynamic shear modulus of elasticity is also very small.

The author has designed a simple instrument known as the "Free Vibration Torsion Pendulum" with which the value of μ may be investigated for different volumetric confining stresses (i), however, undisturbed samples of the soil are required. The sample is placed in a triaxial chamber and subjected to a volumetric confining stress close to the one it has in the ground, Fig. 9. Once the sample is consolidated, that is to say, after the hydrostatic excess pore water pressure is dissipated, the sample is induced to vibrate freely, the response due to the elastic elements is measured and also the damping properties of the soil. From this information the value of the shear modulus of elasticity is determined with enough accuracy to be applied in practical foundation problems.

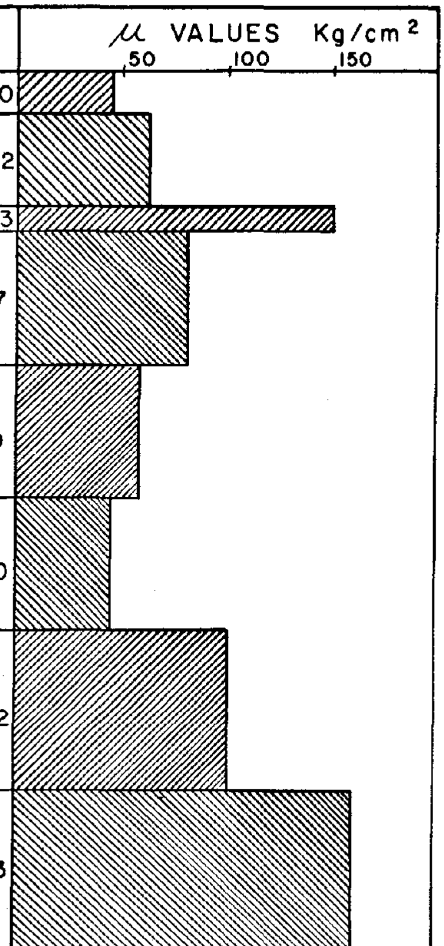


FREE VIBRATION TORSION PENDULUM

FIG.-9

DYNAMIC SHEAR MODULUS OF ELASTICITY PROFILE

	STRATUM	Z	d	W	S_s	γ
A	Fill		1.5	–	–	1.70
B	Brown Silty Clay	5	3.5	60	2.58	1.62
C			1.0	80	2.63	1.53
D	Olive Brown Silty Clay	10	5.0	320	2.52	1.17
E	ditto	15	5.0	270	2.38	1.19
F	ditto	20	5.0	250	2.38	1.20
G	ditto	25	6.0	230	2.45	1.22
H	ditto	30	6.0	230	2.54	1.23
		35	FIRM GROUND			



Z Depth in meters (m)
d Thickness in m.
W Water content
S_s Specific gravety
γ Unit weight

FIG.-10

The investigation of μ is performed for each different soil strata and its value determined for the confined state of stress existing at the depth where the sample was taken. A profile of the μ values is then made as that shown in Fig. 10, where the thickness of the soil strata is recorded, and the average values of the water content, specific gravety and unit weight for each stratum are also recorded.

IV. SUBSOIL DYNAMIC BEHAVIOR

The earthquake epicenter is usually located deep in a fault zone where the potential energy is accumulated due to the distortion of the earth crust, and it is suddenly liberated into kinetic energy. The result is the generation of two important body waves, that reach the ground surface at the site with different ground surface accelerations (a).

a) The compressional waves require change in volume of the soil, and travel with a velocity

$$v_\alpha = \sqrt{\frac{1-\nu}{(1+\nu)(1-2\nu)} \cdot \frac{E}{\rho}} \tag{10}$$

here E is the linear modulus of elasticity and ν Poisson's ratio.

b) The shear or constant volume waves travel with velocity

$$v_s = \sqrt{\frac{\mu}{\rho}} \tag{11}$$

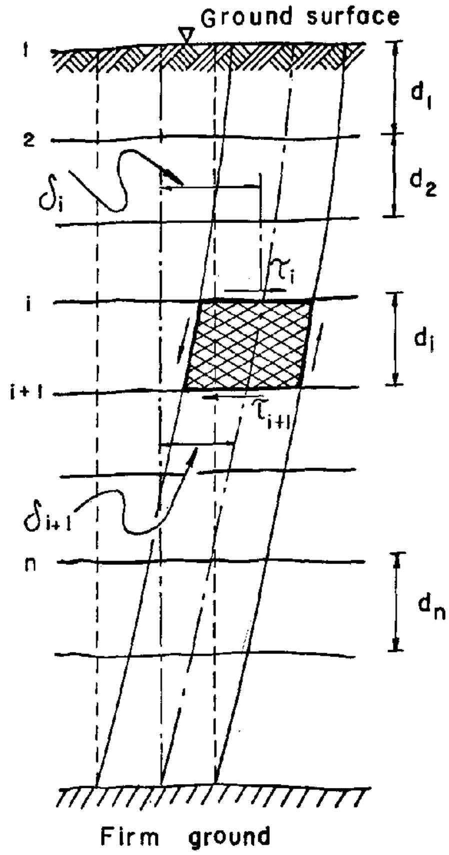
in which μ is the dynamic shear modulus of elasticity of the soil and ρ the unit soil mass. The shear wave velocity is independent of Poisson's ratio.

Notice that the ratio $v_\alpha / v_s > 1$, therefore, the compressional waves have a higher velocity. When seismic body waves hit the interphase between firm and soft ground, the compressional and shear waves travel upward to the ground surface. These two types of seismic waves produce in the soil mass different physical phenomena. Their effect may be treated separately.

When the response pseudo- acceleration spectrum for a strong earthquake is not known close to the site in question, it will be necessary to perform a soil investigation to construct a semi-empirical envelope response spectrum for design purposes. When achieving this one calculates the fundamental period of vibration, the horizontal displacements and shear stresses in the soil mass. The calculation is performed using the mechanical and index properties of the soil for each one of the strata encountered from the ground surface to firm ground.

The shear stresses in the subsoil are investigated and compared with the soil shear strength. The shear stresses induced by the earthquake strong ground motion added to the static shear stresses induced by the weight of the building in the subsoil, may result high and may run over the shear strength of the soil, giving as a result a partial or total failure of the foundation (j).

Consider a soil column subjected to the strong ground motion induced by the earth-



SOIL COLUMN SUBJECTED TO STRONG GROUND MOTION

FIG.-11

quake, Fig. 11. The average horizontal relative displacement with respect to firm ground of a soil element of thickness d_i at any depth $z = i$, is $(\delta_i + \delta_{i+1})/2$. Assuming ω_n is the free angular frequency of the soil mass, then the maximum inertia force of the element will be

$$(\tau_{i+1} - \tau_i) = (\rho_i \; d_i)\, \omega_n^2 \cdot \frac{1}{2}\, (\delta_i + \delta_{i+1}) \tag{12}$$

This force will be balanced by the elastic response of the soil elements under shear strain, hence

$$\frac{\delta_i - \delta_{i+1}}{d_i} = \frac{\tau_i + \tau_{i+1}}{2\mu} \tag{13}$$

Combining these two equations the algorithms for the calculation of the problem in question may be obtained

$$\delta_{i+1} = A_i \delta_i - B_i \tau_i \tag{14}$$

$$\tau_{i+1} = C_i(\delta_i + \delta_{i+1}) + \tau_i \tag{15}$$

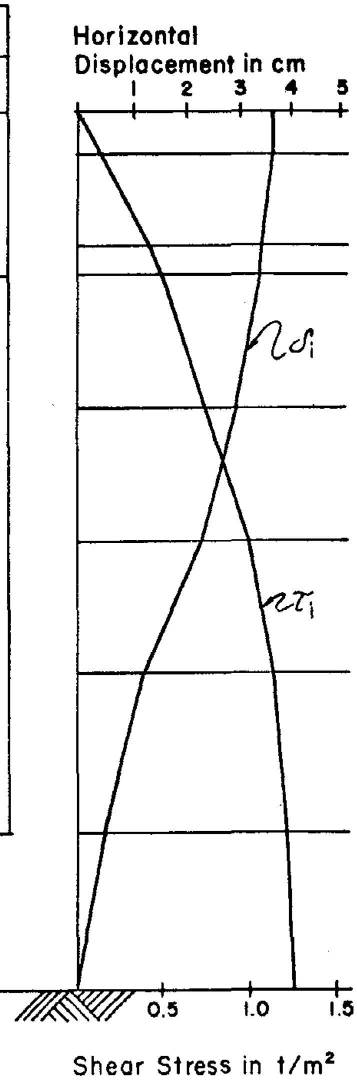
in which

$$A_i = \frac{1 - N_i}{1 + N_i} \qquad B_i = \frac{1}{1+N_i} \cdot \frac{d_i}{\mu}$$
$$C_i = \frac{1}{2}\, \rho_i \; d_i \; \omega_n^2 \qquad N_i = \frac{\rho_i \; d_i^2 \; \omega_n^2}{4 \mu_i} \tag{16}$$

The coefficients A_i, B_i and C_i are a function of the geometrical and mechanical properties of the strata and may be calculated for an assumed value of the circular frequency ω_n. The integration of (14) and (15) is performed step by step knowing that at the ground surface $Z = O$, $\tau_i = O$ and $\delta_{so} = a_m/\omega_n^2$, in which a_m is the maximum acceleration at the ground surface. The integration follows with depth using equation (14) obtaining δ_{i+1} then entering in (15) to obtain τ_{i+1}. Entering again in (14) with τ_{i+1} to obtain δ_{i+2} and then in equation (15) to find τ_{i+2}, and so on. When the value of the natural frequency is well chosen, the calculation will give at the firm ground a zero relative displacement; any other mode of vibration may be investigated by the same method [a]. The calculation so performed gives the maximum shear stresses and horizontal displacements induced in the soil mass because of the strong ground motion.

STRATUM		t/m³	t sec/m⁴	t/m²	m/sec	sec	sec
N°	d	γ	ρ	μ	v_s	$\sum\Delta t$	$\sum\Delta t$
A	1.5	1.70	0.170	450	51.4	0.117	
B	3.5	1.62	0.165	640	62.3	0.342	
C	1.0	1.53	0.156	1500	98.1	0.383	
D	5.0	1.17	0.119	820	83.0	0.624	
E	5.0	1.19	0.121	600	70.4	0.908	1.12
F	5.0	1.20	0.122	460	61.4	1.23	
G	6.0	1.22	0.124	1050	92.0	1.49	
H	6.0	1.23	0.125	1620	113.8	1.70	

T_{s1} = 1.70 sec T_{s2} = 1.12 sec



SEISMIC SUBSOIL BEHAVIOR

FIG.-12

The fundamental period of vibration of the ground and other resonant periods corresponding to partial addition of strata may be investigated using the shear wave velocity method(a). In fact, waves are generated at the firm ground with different wave lengths, and whenever one quarter of the length of a shear wave is coincident with a stratum or sum of strata, amplification of horizontal displacements takes place. The waves reflect up and down in the strata as they hit the interfaces. The fundamental period of vibration is obtained adding through the entire soil deposit the times the shear wave takes to travel each stratum. For one stratum $\Delta t = d_i / v_{si}$.

Therefore, the fundamental period is

$$T_{si} = 4 \sum \frac{d_i}{v_{si}} \tag{17}$$

Here, T_{s1} is the largest period producing resonance, v_{si} is the shear wave velocity for the stratum with thickness d_i. As stated before, each stratum may suffer magnification of stresses and horizontal displacements when the shear wave passing through has a length four times the thickness of the stratum or sum of soft soil strata. Therefore, several possible resonant periods should be investigated that may affect the building. In Fig. 12 a case is presented where the dominant period of vibration corresponding to the maximum ground acceleration is calculated in 1.70 sec. Nevertheless, another important period is the resonance of strata D, E, F and G with 1.12 sec. The acceleration at the ground surface, however, produced by a stratum or sum of strata is smaller than the acceleration obtained from the fundamental free period of vibration of the ground when all strata are considered. Therefore, when the period T_{s1} is considered the largest, it is taken as giving the peak response acceleration at the ground surface. The second mode of vibration of the entire deposit should be also investigated. For uniform subsoil conditions the higher harmonics have values on the order of 1/3, 1/5... of the fundamental period.

V. FOUNDATION SEISMIC RESPONSE

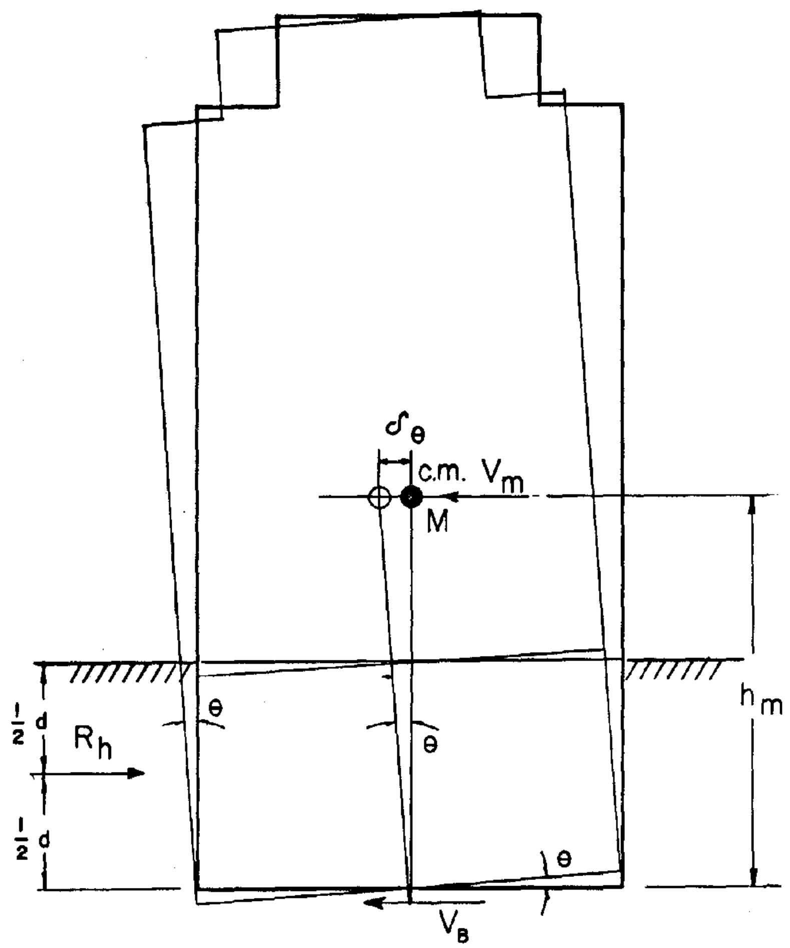
With the ratio of the rocking free period of vibration T_θ of the rigid compensated foundation and the fundamental free period of vibration of the ground T_{s1} and assigning a critical damping of the soil, the envelope acceleration response spectrum may be entered Fig. 8, and the response acceleration magnification factor determined R_a/a_m from which the seismic shear is calculated

$$V_m = M\left(\frac{R_a}{a_m}\right) a_m \tag{5}$$

here a_m is the maximum measured or assigned ground surface acceleration.

The strong ground surface motion will push the rigid box- type foundation, in such a way that a seismic horizontal force V_m will be developed at the center of mass of the building, Fig. 13. The foundation during the ground motion will compress the soil with a force R_h, assumed to be applied at the middle depth of the foundation structure. If the soil does not fail due to passive earth pressure then the base shear at the foundation grade elevation will be

$$V_B = R_h - V_m \tag{18}$$



ROCKING OF BUILDING

FIG.-13

and the overturning moment on the foundation base slab will be

$$O_{TB} = V_m \cdot h_m - \frac{1}{2} R_h \cdot d \qquad (19)$$

Calling ω_θ the free natural frequency of the foundation system and δ_θ the displacement at the center of mass the total overturning moment due to the inertia force may be written

$$O_T = M\delta_\theta \cdot \omega_\theta^2 \cdot h_m \qquad (20)$$

Calling K_θ the rocking foundation modulus for the rigid box-type foundation, defined as

$$K_\theta = \frac{O_T}{\theta} \qquad (21)$$

here θ is the amplitude angle of the rocking phenomenon, hence

$$M\delta_\theta \omega^2 \cdot h_m = K_\theta \frac{\delta_\theta}{h_m}$$

from which the rocking free natural frequency may be obtained:

$$\omega_\theta^2 = \frac{1}{h_m^2} \frac{K_\theta}{M}$$

and the rocking free period of vibration of the foundation will be

$$T_\theta = 2\pi h_m \sqrt{\frac{M}{K_\theta}} \qquad (22)$$

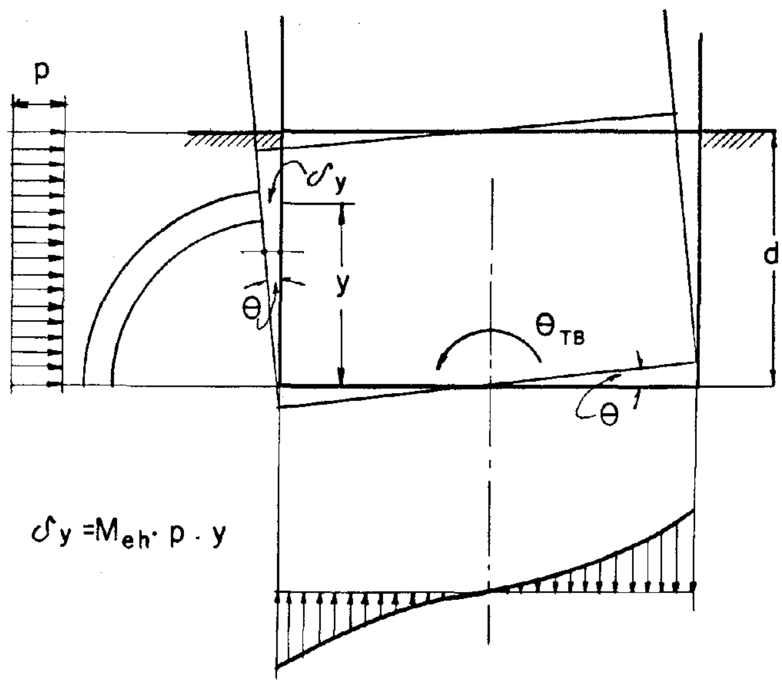
The next problem will be to investigate the value of K_θ to be able to compute the period T_θ .

VI. SOIL REACTIONS ON THE FOUNDATION STRUCTURE

In the preceding section it was stated that the overturning moment of the rigid compensated foundation may be expressed as:

$$O_T = K_\theta \cdot \theta \qquad (23)$$

in which K_θ is the foundation modulus due to rocking and θ the amplitude angle of the rocking motion, Fig. 14. Let us call O_{TW} the part of the overturning moment taken



SOIL REACTIONS DURING ROCKING OF FOUNDATION

FIG.-14

by the wall or sides of the foundation rigid box, and O_{TB} that part of the overturning moment taken by the base slab at the foundation grade elevation. Therefore, the foundation moduli for the wall and base may be expressed respectively

1) for the wall
$$k_{\theta W} = \frac{O_{TW}}{\theta} \tag{24}$$

2) for the base
$$k_{\theta B} = \frac{O_{TB}}{\theta} \tag{25}$$

and since
$$O_T = O_{TW} + O_{TB}$$
then
$$k_\theta = k_{\theta W} + k_{\theta B} \tag{26}$$

The problem will be now to estimate the values of the foundation moduli $k_{\theta W}$ and $k_{\theta B}$.

To obtain a value for $k_{\theta W}$, the assumption is made that the reaction force on the wall is uniformly distributed when the wall rotates in a plane and compresses the soil, Fig. 14, hence

$$R_h = p \cdot d \tag{27}$$

Moreover, it is further assumed that the horizontal wall displacement at any height from the foundation grade elevation may be expressed as follows

$$\delta_y = M_{eh}\, p \cdot y \tag{28}$$

where M_{eh} is the horizontal average lineal strain modulus of the soil in contact with the wall, and p the unit maximum pressure exerted on the soil by the wall during the rocking phenomenon, therefore

$$M_{eh} \cdot p = \theta \tag{29}$$

In terms of the dynamic shear modulus of elasticity

$$M_{eh} = \frac{1}{2(1+\nu)\mu} \tag{30}$$

On the other hand
$$O_{TW} = \frac{1}{2}\, p\, d^2$$

and substituting values in $\sigma_{T_W} = K_{\theta W}\,\theta$ an approximate value for the wall foundation modulus may be obtained; hence

$$K_{\theta W} = (1+\nu)\, d^2 \mu \tag{31}$$

The determination of the base foundation modulus $K_{\theta B}$ requires a more elaborate calculation to be able to obtain more accuracy.

The dynamic effect at the foundation grade elevation induced by the earthquake, requires the determination of the deformation of the strata from the point of view of the dynamic shear modulus of elasticity or soil rigidity μ , (l) , hence

$$\alpha_N = \left(\frac{d}{2\,(1+\nu)\,\mu}\right)_N \tag{32}$$

The value of α_N represents the deformation of the stratum N of thickness d , or change in thickness due to a unit stress.

To study the compatibility of deformation at the interface of the foundation structure and soil (k) (l), the contact surface is divided into equal size tributary areas $\bar{a}$, and so many as required for accuracy. Further assume four subsoil strata to be considered. Hence, the deformation of the strata matrix is formed

$$\begin{Bmatrix} \alpha_A \\ \alpha_B \\ \alpha_C \\ \alpha_D \end{Bmatrix} \tag{33}$$

The unit reactions influence matrix may be formed for each unit loaded tributary area as shown in Fig. 15. The influence values I_{ji} should be determined taking in consideration approximately the stratigraphical conditions of the subsoil. For this purpose a special stress net may be used based on Frölich subsoil considerations (a). The matrices so obtained are transposed and multiplied by the deformation strata matrix to calculate the unit influence vertical displacement in all points at the foundation grade elevation due to the unit load in tributary areas $\bar{a}$, hence for the unit load applied on any tributary area

$$\{\bar{\delta}_{ji}\} = [I_{ji}]^T \{\alpha_N\} \tag{34}$$

in the same manner the unit influence matrices are formed for other unit loaded tributary areas, (l).

The settlement or vertical displacements matrix may be calculated multiplying the influence coefficients transposed matrix formed for the $\bar{\delta}_{ji}$ values, Fig. 16, by the

UNIT LOAD AT TRIBUTARY AREA $\bar{a}_1$

Stratum	1	2	3	4	5	6	α
A	I_{11}^{A}	I_{21}^{A}	I_{31}^{A}			I_{61}^{A}	α_A
B	I_{11}^{B}	I_{21}^{B}	I_{31}^{B}			I_{61}^{B}	α_B
C	I_{11}^{C}	I_{21}^{C}	I_{31}^{C}			I_{61}^{C}	α_C
D	I_{11}^{D}	I_{21}^{D}	I_{31}^{D}			I_{61}^{D}	α_D

UNIT STRESS INFLUENCE MATRIX

FIG.- 15

1	2	3	4	5	6	q
$\bar{\delta}_{11}$	$\bar{\delta}_{12}$	$\bar{\delta}_{13}$	$\bar{\delta}_{14}$	$\bar{\delta}_{15}$	$\bar{\delta}_{16}$	q_1
$\bar{\delta}_{21}$	$\bar{\delta}_{22}$	$\bar{\delta}_{23}$	$\bar{\delta}_{24}$	$\bar{\delta}_{25}$	$\bar{\delta}_{26}$	q_2
$\bar{\delta}_{31}$	$\bar{\delta}_{32}$	$\bar{\delta}_{33}$	$\bar{\delta}_{34}$	$\bar{\delta}_{35}$	$\bar{\delta}_{36}$	q_3
$\bar{\delta}_{41}$	$\bar{\delta}_{42}$	$\bar{\delta}_{43}$	$\bar{\delta}_{44}$	$\bar{\delta}_{45}$	$\bar{\delta}_{46}$	q_4
$\bar{\delta}_{51}$	$\bar{\delta}_{52}$	$\bar{\delta}_{53}$	$\bar{\delta}_{54}$	$\bar{\delta}_{55}$	$\bar{\delta}_{56}$	q_5
$\bar{\delta}_{61}$	$\bar{\delta}_{62}$	$\bar{\delta}_{63}$	$\bar{\delta}_{64}$	$\bar{\delta}_{65}$	$\bar{\delta}_{66}$	q_6

UNIT DISPLACEMENT MATRIX

FIG.-16

columnar unit reactions matrix for tributary areas 1 to n, hence the vertical displacements at the center of each tributary area

$$\{\delta_n\} = [\bar{\delta}_{ij}]\{q_n\} \tag{35}$$

The vertical displacement or settlement matrix is necessary to calculate the interaction of the foundation structure with the soil mass.

The problem however, may be divided into a symmetric action for no moment on the foundation, and into an antisymmetric action to consider an overturning moment. It may be found that the unit displacement matrix shown in Fig. 16 is a symmetric matrix

$$[\bar{\delta}_{ij}]^T = [\bar{\delta}_{ij}]$$

and if the vector $\{q_n\}$ is also symmetric the matrix settlement equation may be reduced to

FOR SYMMETRIC REACTIONS

$\bar{\delta}_{11} + \bar{\delta}_{16}$	$\bar{\delta}_{12} + \bar{\delta}_{15}$	$\bar{\delta}_{13} + \bar{\delta}_{14}$	$q'_1 = q'_6$
$\bar{\delta}_{21} + \bar{\delta}_{26}$	$\bar{\delta}_{22} + \bar{\delta}_{25}$	$\bar{\delta}_{23} + \bar{\delta}_{24}$	$q'_2 = q_5$
$\bar{\delta}_{31} + \bar{\delta}_{36}$	$\bar{\delta}_{32} + \bar{\delta}_{35}$	$\bar{\delta}_{33} + \bar{\delta}_{34}$	$q'_3 = q'_4$

FIG.- 17

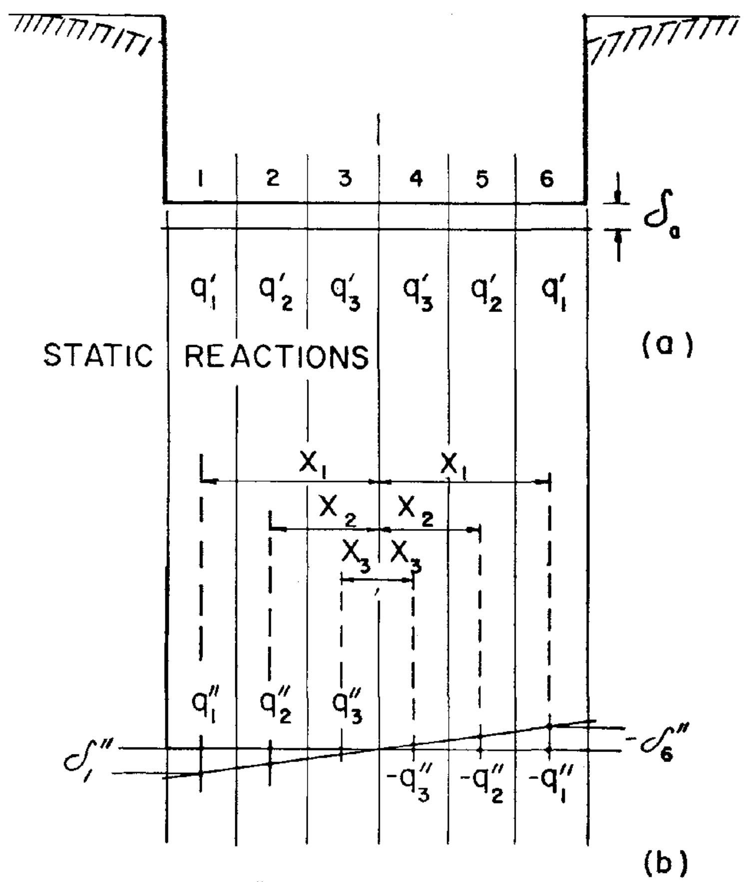
hence one can write the symmetric reactions vertical displacement equation in the form

$$\{\delta'_n\} = [\bar{\delta}_{ij}]\{q'_n\} \tag{36}$$

Equation (36) may be used to calculate the symmetric reactions q'_n. Since δ'_n has a constant value δ'_a for a rigid foundation on a compressible soil, the equations system given by (36) may be solved for reactions q'_n corresponding to the static loading (I), Fig. 19a.

The antisymmetric unit displacements matrix may be formed to solve the problem of the seismic overturning moment, as follows

RIGID FOUNDATION



$$\delta''_1 = -\delta''_6 = \theta' \cdot X_1$$
$$\delta''_2 = -\delta''_5 = \theta' \cdot X_2$$
$$\delta''_3 = -\delta''_4 = \theta' \cdot X_3$$

SEISMIC REACTIONS

FIG.-19

FOR ANTISYMMETRIC REACTIONS

$\bar{\delta}_{11} - \bar{\delta}_{16}$	$\bar{\delta}_{12} - \bar{\delta}_{15}$	$\bar{\delta}_{13} - \bar{\delta}_{14}$	q_1''	$= - q_6''$
$\bar{\delta}_{21} - \bar{\delta}_{26}$	$\bar{\delta}_{22} - \bar{\delta}_{25}$	$\bar{\delta}_{23} - \bar{\delta}_{24}$	q_2''	$= - q_5''$
$\bar{\delta}_{31} - \bar{\delta}_{36}$	$\bar{\delta}_{32} - \bar{\delta}_{35}$	$\bar{\delta}_{33} - \bar{\delta}_{34}$	q_3''	$= - q_4''$

FIG.- 18

and calling the antisymmetric reaction vector $q_1'' q_2'' q_3'' - q_3'' - q_2'' - q_1''$ then the following vertical displacements matrix equation is formed

$$\left\{\delta_n''\right\} = \left[\bar{\delta}_{ij}''\right]\left\{q_n''\right\} \tag{37}$$

in which the values δ_n'' are the vertical displacements at the foundation grade elevation due to a rocking moment.

In this case the deformation condition imposes a plane displacement configuration at the foundation grade elevation due to an overturning moment O_{TB}', Fig. 19b, hence

$$\left\{\delta_n''\right\} = \begin{Bmatrix} \theta' X_1 \\ \theta' X_2 \\ \theta' X_3 \end{Bmatrix} \tag{38}$$

a value of θ' may be assigned and the equation (37) solved for the antisymmetric reactions q_n'', corresponding to an overturning moment

$$O_{TB}' = \sum \bar{a}\, q_n'' X_n \tag{39}$$

The rocking foundation modulus is then determined

$$K_{\theta B} = \frac{O_{TB}'}{\theta'} \tag{40}$$

and the total rocking foundation modulus including the depth of the foundation will be now

$$K_\theta = K_{\theta W} + K_{\theta B}$$

from which the rocking free period of vibration of the foundation is obtained,

$$T_\theta = 2\pi h_m \sqrt{\frac{M}{K_{\theta W} + K_{\theta B}}} \tag{41}$$

Since $K_{\theta W}$ is proportional to the square of the foundation depth by formula (31), notice the important effect in the free period of vibration of the foundation when it is a surface foundation or when it may be deep seated in the ground.

The envelope for the response spectrum, Fig. 8, may be entered now with T_θ / T_g, that is, with the ratio of the rocking free period of vibration and that of the ground, and the acceleration magnification factor R_a/a_m determined. Knowing the maximum ground surface acceleration the seismic shear may be computed:

$$V_m = M(R_a/a_m)\,a_m$$

and the total overturning moment

$$O_T = V_m \cdot h_m$$

The rocking angle amplitude of the system will be

$$\theta = \frac{O_T}{K_{\theta W} + K_{\theta B}} \tag{42}$$

and since

$$O_{TW} = K_{\theta W} \cdot \theta \qquad and \qquad O_{TW} = \frac{1}{2}\, p d^2$$

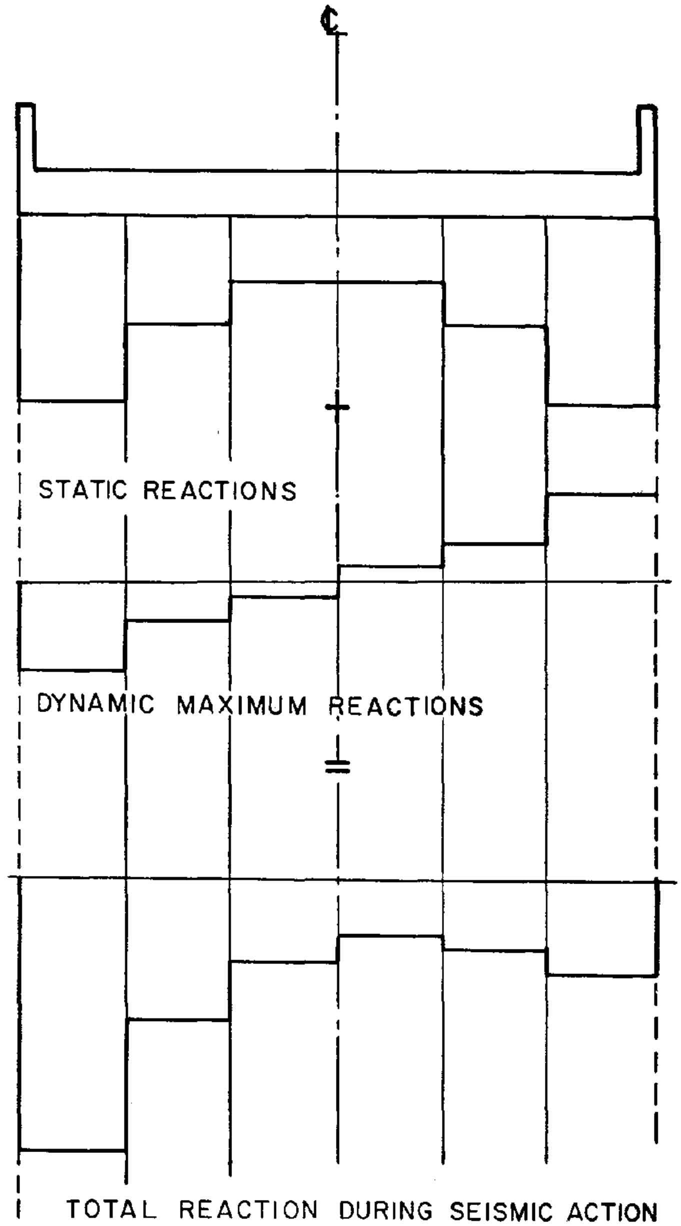
we may find the unit reaction to the wall from

$$p = \frac{K_{\theta W}}{K_{\theta W} + K_{\theta B}} \cdot \frac{2\,O_T}{d^2} \tag{43}$$

The overturning moment taken by the base slab will be

$$O_{TB} = \frac{K_{\theta B}}{K_{\theta W} + K_{\theta B}} \cdot O_T \tag{44}$$

The antisymmetric soil reactions due to the rocking effect obtained from (37) may be determined correcting the values q_n'' in proportion to σ'_{7B} and σ_{7B}. The reactions so obtained are added to the static contact stresses at the foundation grade elevation, Fig. 20, to obtain the final interacting stresses during the seismic action. The stresses at the edge of the foundation are examined to pass the local bearing capacity with a reasonable factor of safety. Once the soil reaction stresses are determined the members of the rigid foundation structure may be calculated by current methods of reinforced concrete design.



CONTACT STRESSES DISTRIBUTION
AT FOUNDATION GRADE ELEVATION

FIG.-20

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

COMMENTS ON STRUCTURE-SOIL INTERACTIONS DURING EARTHQUAKES

by

Loring A. Wyllie, Jr.
Structural Engineer
H.J. Degenkolb & Associates
San Francisco, California

INTRODUCTION

The intent of this paper is to present some comments and observations from a structural engineer's viewpoint on interactions between structure and soil during an earthquake. The paper purposely will not address traditional soil-structure interaction, which consists of the soil deformations caused by the building motion which in turn modifies the response of the structure. Considerable research and published results are available in that area, although the research in this area appears to have only dented the surface.

The paper will discuss buried structures and building basements and the ranges of their seismic performance, as well as foundations themselves. A brief discussion will follow on how earthquake forces or displacements get into buildings and then how they get out. Lastly, some comments will be offered on the importance of soil effects on building response. In all of these areas the need for well thought-out research will be emphasized.

It is emphasized that this paper does not intend to cover all areas of structure-soil interaction.

UNDERGROUND AND BASEMENT STRUCTURES

The traditional approach to the design of underground structures or basements is to determine a soil pressure on the exterior surfaces and design the structure accordingly. In recent years considerable work has been done by geotechnical researchers and soil pressures recommended for walls in seismic areas have increased. In California we rarely see the faithful old 30 pcf equivalent fluid pressure any more, its always at least 40 or 50 pcf. I do not mean to discuss soil pressures for either cantilever or restrained walls, as they are a matter of research for the geotechnical people. Let me discuss areas of interest to structural researchers.

In designing underground structures in seismic areas, we need to consider more than just soil pressures. We need to consider the stiffness and other properties of the soil, the stiffness and geometry of the buried structure, the location of the structure within the ground and,of course, such factors as earthquake intensity, etc. When the earthquake occurs, the ground shakes and the soil deforms. In rock and dense rock-like materials, the ground tends to move as a mass and the ground displacement will be approximately equal to the displacement at depth. In soft soils, the surface displacement will tend to be greater than the displacement at depth. This is somewhat like shaking a bowl of jello.

The buried structure in rock or rock-like soils will go along for the ride and probably not be damaged severely because there have not been any major distortions in the soil surrounding the buried structure. However, the structure buried in soft soils will be subjected to displacements in the soil layers, as schematically illustrated in Figure Number 1. If the structure is very flexible and able to deform with damage, which is unlikely for underground concrete structures, it will go along for the ride. In the normal case, the structure will try to resist the soil displacements and failure may occur if the structure is not strong enough. If the structure is very stiff compared to the soil, then it will alter the soil deformations and win the battle of stiffness. Allow me to cite a few examples:

Several underground structures were damaged at the Joseph Jensen Filtration Plant in the San Fernando earthquake of 1971. Several of these structures were reported elsewhere. [1] A four channel box culvert in a stream bed was heavily damaged because the soil displaced more than the culvert could, as seen in Figure Number 2. The culvert was half full of silt and the culvert walls either failed and sheared at the top due to restraint of the silt, or the walls hinged both top and bottom. The soil in this case moved more and was and stronger than the structure. What soil pressure or criteria should one use to design this structure? Incidentally, the repairs consisted of adding buttresses on each side to stiffen and strengthen the structure.

Many underground box conduits laced the Joseph Jensen Filtration site and some were damaged in the earthquake. These conduits were often covered with ten to twenty feet of earth. The conduits in firmer,dense soil had no damage nor permanent displacement at expansion joints. In loose soils the conduit racked laterally in places as seen in Figure Number 3 and had offsets at expansion joints. The wall failed in shear as seen in Figure Number 4. In other areas of the site conduits were underlain with loose saturated sands which liquefied and caused landsliding. Conduits in this landslide area generally had minimal concrete damage but had considerable movement and opening of expansion joints. Ignoring the liquefaction problem, what design criteria should one use for these conduits?

The underground finished water reservoir at the Jensen plant was structurally complete at the time of the earthquake and was 500 feet by 520 feet in plan and 37 feet deep. Damage was described in Reference [1] . A good correlation between response and damage was obtained by considering the reservoir lifted up out of the soil and placed on grade. In other words, this structure was so large in plan that lateral earth pressures were a minor factor in estimating its structural response.

Allow me to cite one additional example. The Banco Central de Nicargua Building was heavily damaged in the 1972 Nicaraguan earthquake. Subsequent investigations revealed that the building was located within a fault zone which displaced a small amount in the 1972 earthquake. Detailed trenching around the building documented that a surface fault trace which offset in the earthquake passed through the building [2] . However, the 30 foot deep basement with heavy vault walls was so rigid that surface fault displacement was offset to the west of the building. [3] Here the structure was stronger than the ground.

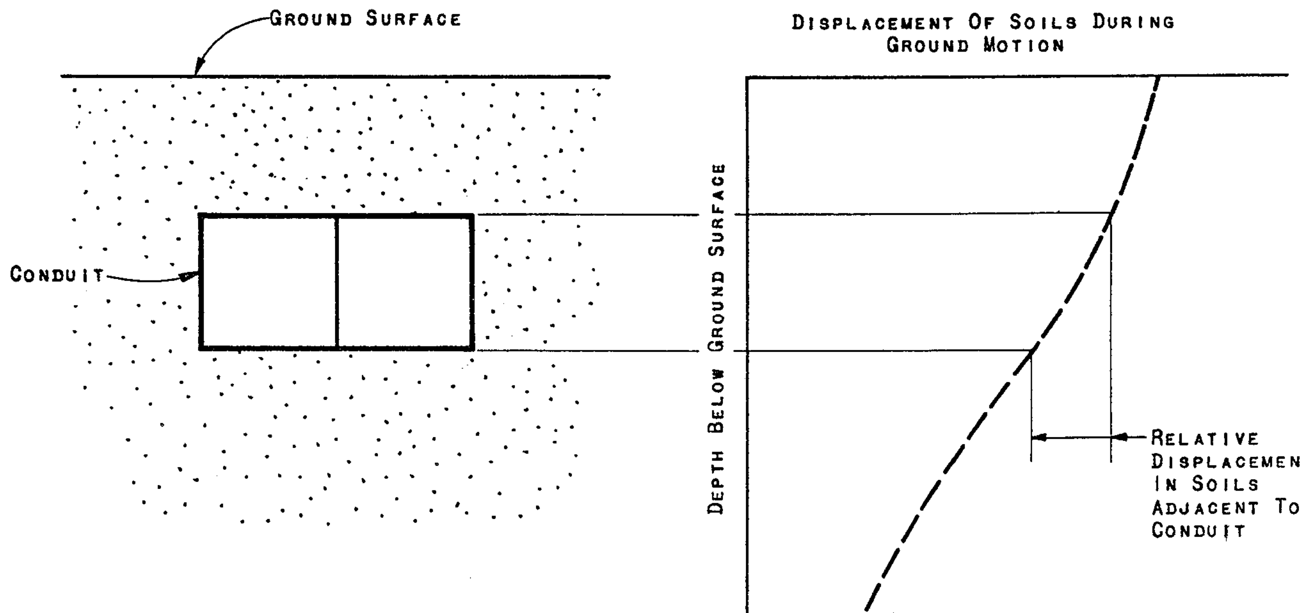

FIGURE NUMBER 1. SCHEMATIC ILLUSTRATING RELATIVE DISPLACEMENT IN SOILS ADJACENT TO CONDUIT OR BURIED STRUCTURE DURING GROUND MOTIONS.

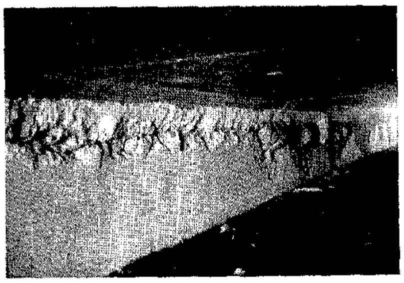

Figure Number 2. Damage to box culvert. Note bent reinforcing bars where wall top has failed in shear and moved laterally.

Figure Number 3. Box conduit which racked laterally in San Fernando earthquake of 1971. Note offset from plumb measure.

Figure Number 4. Failed box conduit shown in Figure Number 3. Note shear failure at top of conduit wall.

Similar arguments can be made for piles and caissons. Piles are usually used for buildings in poor or soft soil sites. Thus, we can expect significant differential horizontal displacements (pile bending) along the length of a pile during an earthquake. The usual pile is flexible and thus it must have sufficient ductility to prevent failure and enough corresponding strength to support the structure. Large diameter piers or caissons may be stiffer than the soil and require significant reinforcement and ties to prevent shear or flexural failures. In all cases, reinforcement greater than typical building code minimums is probably required. How much is needed? What are the ranges for different piles and different soil conditions? We must remember that it is difficult to impossible to repair piles or caissons after an earthquake, let alone inspect them, so a greater degree of conservatism in design is perhaps in order.

Rational, simplified, design criteria is needed for buried structures and buried structural elements. Valid research in this area cannot come solely from geotechnical engineers, but needs the teamwork of structural engineers. Some good work is needed in this area.

TRANSMITTING FORCES BETWEEN SOIL AND BUILDING

Traditional seismic design of buildings assigns equivalent static forces at each floor level and then the building is analyzed similar to wind conditions. Spectral dynamic analysis of a building establishes periods of vibration and mode shapes and eventually ends up with values treated somewhat similar to equivalent static forces. The results of these "loadings" by either approach result in shears and overturning forces to be resisted by the foundation and the supporting soils.

This basic approach is contrary to what actually happens in an earthquake. In an earthquake, the ground moves. As the ground moves, the foundations also move a similar amount depending on the degree of coupling between building and soil. This movement of the foundation with certain dynamic characteristics in turn moves the building in accordance with its flexibility and inertia forces are generated.

The real forces on the foundation are the earth forces trying to move the building, not building forces trying to be resisted by the ground. Furthermore, at each instance of time, a point on the ground surface will be moving at a certain pattern. At a nearby point at the same instant of time, a different motion must be expected. Therefore, the input motion to the structure is not the free field ground motion but the summation of all the free field motions affecting the building at each instant of time, plus any slip that occurs. This will tend to be less than the free field motion. This situation is undoubtedly one contributing reason why structures perform better than some dynamic studies would indicate.

Little research has been done in this area and no extensive recommendations are known to the author. Japanese studies [4] after the Tokachioki earthquake revealed completely out-of-phase motions at two stations 36 meters apart in some aftershocks. Observations at the Rio Blanco underground nuclear explosion [5] also found an out-of-phase relationship. Theoretically, if ground motion was steady state harmonic motion and the foundation and its ties were perfectly elastic, we could design a building of a certain specific size for a specific harmonic motion such that the building would never move, its foundations would simply stretch and contract. This is obviously not the case of real earthquakes, but it offers an interesting comparison.

In addition to a reduction of building response, this out-of-phase motion beneath a building emphasizes the need for substantial ties within the foundation. The present 10% tie requirement for pile foundations is an arbitrary value needing confirmation. Spread footings should perhaps also be tied, at least in less competent soils, as old codes used to require and as still practiced in Central and South America. Alternate systems should be rationally studied by valid research.

Together with studies in the area mentioned, the whole question of passive pressure and/or friction to take static forces out of a building needs appropriate investigation. There are many engineers, myself included, that do not believe that this is a valid concept and does not have to be calculated nor provided. If the force can't get in, it doesn't have to get back out into the soil. However, others, including some enforcement agencies, feel differently. The classic case is the building with batter piles, which has a firm coupling to the ground and generally experiences increased response and pile distress.

One more area of possible concern warrants brief mention. Surface waves cause undulations of the ground surface. Seismologists and others are not apparently sure of the amplitude and wave length of these waves. However, still frightened observers after recent earthquakes have sworn to mountainous waves. If wave lengths are relatively short and amplitudes significant, the resulting foundation rotations on sizable concrete structures could be significant. Stratta and Griswold [6] have outlined this problem but it is basically an unknown area, needing joint input from the fields of seismology and structures.

EFFECTS OF FOUNDATION ROTATION ON STRUCTURAL RESPONSE

In the usual structural analysis and design in earthquake areas, considerable analysis is done to determine relative rigidities of bracing elements. Computers work to great precision, usually assuming that the foundations are fixed. The end result is that meaningless calculations often result and that the important effects are neglected. Allow me to illustrate with several examples taken from Reference [7] which has not been widely distributed outside the San Francisco Bay Area.

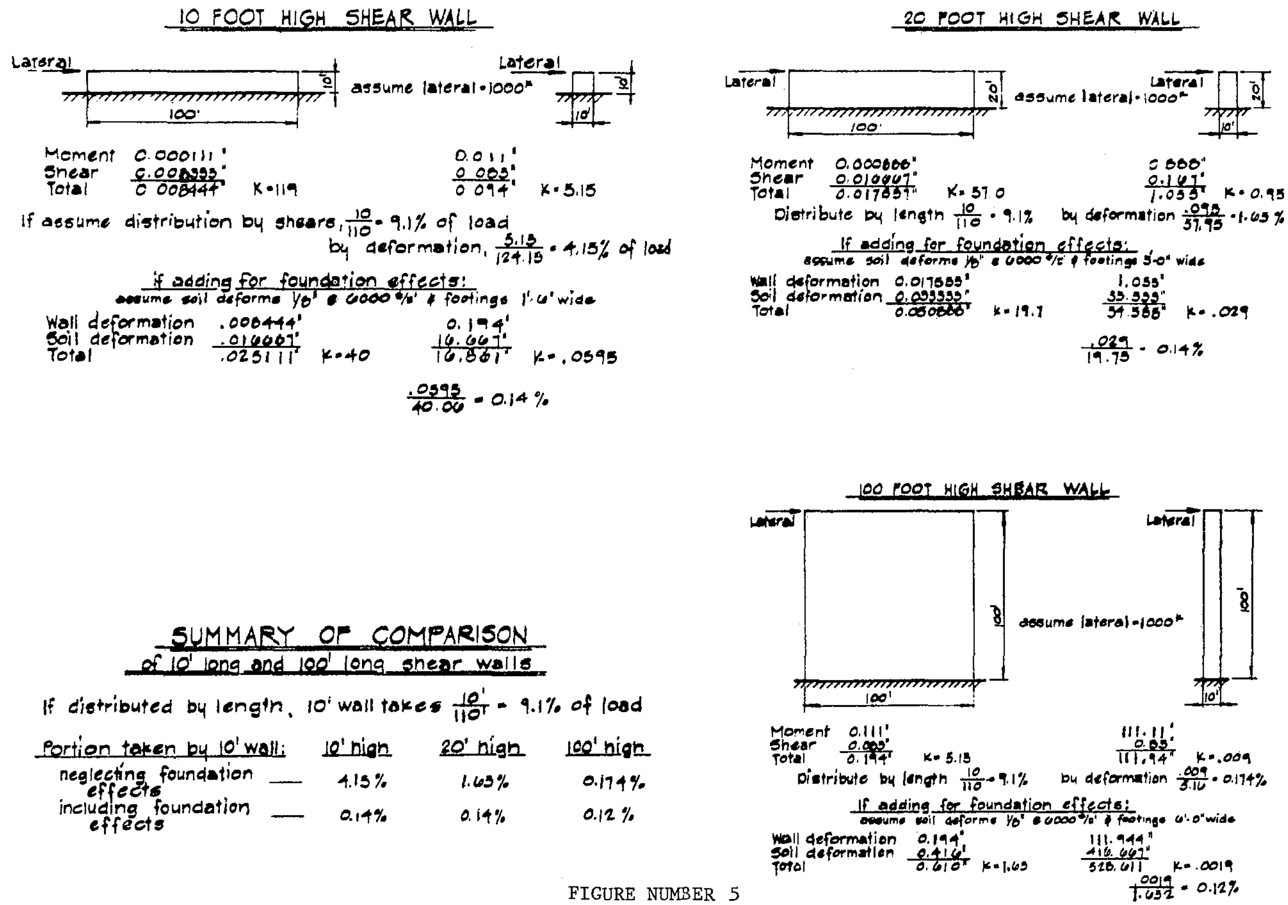

FIGURE NUMBER 5

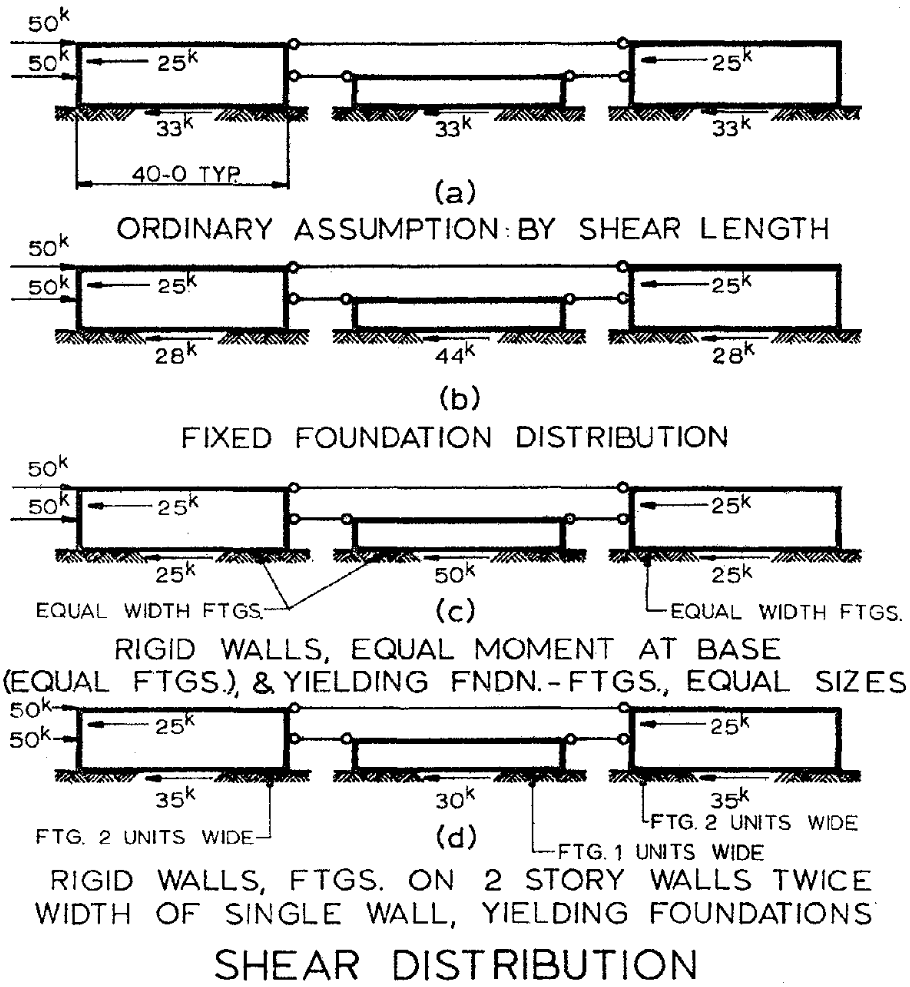

FIGURE NUMBER 6

Let us assume two shear walls, one 10 feet long and the other 100 feet long. Let us also consider three heights of shear walls - 10 feet, 20 feet and 100 feet and calculate the relative rigidities that determine the proportion of the load that each one takes, both neglecting foundation rotation and considering it. For the foundation rotation, let us assume a linearly elastic very good soil - one that has about 1/4 inch deflection for 6000 pounds per square foot soil pressure.

Figure Number 5 shows the relative deformations - the inverse of which measures rigidity - for the 10 foot high wall. If we consider shear deformations only - length of wall - the 10 foot long wall takes 9.1% of the shear. Considering shear and bending deformations, it takes 4.15% of the load, but when considering foundation rotation it only takes 0.14% of the load.

Similar relationships are shown for the 20 foot high wall and the 100 foot high wall in Figure Number 5. All the results are summarized. A study of these relationships shows that we are calculating with great precision the unimportant items - shear and moment deflections but neglecting the most important item (by a factor of several times) - the foundation effect. Note in Figure Number 5 that even in the 100 foot long, 10 foot high wall, the foundation deformation (0.0167 inches) is double the concrete deformation (0.00844 inches). Yet we neglect it!

The above concerns walls of equal height. Another example has come to attention recently. Figure Number 6 shows three 40 foot long walls in a two story building. Two walls extend to the roof and are two stories high. One identical wall only extends up one story. In the top floor there is no question of shear distribution. The two identical walls must each take one-half the load and consequently they have 25 kips shear each. When we get below the second story, however, the distribution of shears depends on the assumptions made. The ordinary assumptions, going from story to story shown in Figure Number 6(a) would indicate 33 kips shear in each of the three walls.

An analysis of this relationship on a computer was made assuming no diaphragm elongation and fixed foundations. That distribution of shears is shown in Figure Number 6(b) and you will note that the center wall takes more shear than would be assumed by the ordinary methods. This would be your ordinary answer if you were using the usual computer programs.

However, we assumed the foundations as fixed and we saw before that the foundation has a large effect. Let us now assume that the walls are rigid - they have very little deflection which is true. Assume also that each wall has a footing of the same width so that the foundation rotation is equal in all cases. Since the base moments of all walls must be identical, statics tell us that the shear distribution must be as shown in Figure Number 6(c) and the center wall takes even more load than the computer told us.

However, it is probable that the two story high walls will have more vertical load on the footings than the one story high wall. In this case, the footing on the two story high walls could be twice as wide as the footing on the low center wall. If the soil is identical and the foundation rotations are equal, then the overturning moment resisted by the high walls must be double that of the low wall and the shear distribution is shown in Figure Number 6(d) which gets quite close to the ordinary assumptions and different from the computer result.

In these examples we have neglected any diaphragm deflections, collector bar elongation, variation in concrete properties and probably more important, a variation in the foundation properties or the effects of concrete shrinkage and temperature. It is obvious that the precise figures of conventional analysis are meaningless - it is more important to realize the overall actions and limitations and to provide a logical system with reasonable allowances for variations.

One additional example is appropriate from a recent hospital project in our office. Two shear walls connected by coupling beams occurred at each end of the building. The walls were about 27 feet long and 118 feet high with coupling beams approximately 7 feet deep and 21 feet long at each of seven levels. If we assumed the walls all fixed at the base, the first mode period of vibration was 0.42 seconds. If we pinned the base and forced the spandrels to work, the period was 0.68 seconds. However, since the building was pier supported, we modeled foundation springs simulating the elastic properties of the piers which yielded a building period of 0.56 seconds, which we used in design. If we eliminated the coupling beams and relied on the tall cantilever walls, the period calculates at 0.66 seconds for fixed bases and 0.96 seconds for the same elastic spring model used before. The result in applied forces is a reduction of code static forces by one-third if the extreme values are compared. Furthermore, from our design, elmination of the coupling beams could have reduced code static force requirements by one-fourth, however, overturning forces and drift would increase and redundancy would be greatly decreased. For a hospital structure where continued functioning after an earthquake is required, we chose the stiffer, redundant structure as being far superior in performance. However, many engineers would eliminate the coupling beams to reduce code forces, equating lower code forces with improved performance. This appears to be an idiosyncrasy in our code which undoubtedly misleads the less knowledgeable practitioner.

My reason for presenting these examples is to emphasize the need for considering soil effects in structural analysis. We need better tools and models to estimate these effects, as well as education of the engineering profession to implement such methods. Above all, simplified methods are essential as normal structural design fees do not permit unending complex formulation and study. Furthermore, the codes must lead us towards better buildings, not ones which will have increased damage.

CONCLUSIONS

In conclusion, this paper has attempted to illustrate several research needs in the area of interaction between structure and soil in major earthquakes. Briefly, these can be summarized as follows:

1. A thorough understanding and simplified design procedures need to be established for underground and buried structures.

2. The interaction of piles and caissons with the soil needs further study to develop design procedures and details that will insure satisfactory performance.

3. The coupling between structure and soil during an earthquake needs to be thoroughly understood and documented. Out-of-phase ground motions and their effects on structural response require considerable study.

4. Foundation tie requirements need a rational basis and thorough understanding

5. The need for passive pressure capacity and/or friction to transfer shears between soil and foundation needs to be documented or soundly discredited.

6. The effects of foundation rotation from surface waves needs study to determine its relevance.

7. Simplified design procedures to incorporate accurately foundation soil deformations are needed for the profession.

Most of these areas of potential study need to be interdisciplinary teams with structural researchers working together with geotechnic engineers and seismologists.

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COMMENTS ON STRUCTURE-SOIL INTERACTIONS DURING EARTHQUAKES

Discussed by

William T. Holmes
Structural Engineer
Rutherford & Chekene
San Francisco, CA

INTRODUCTION

This discussion will amplify and emphasize design problems and research needs pointed out in the subject paper. It should not be surprising to find that the areas of foundations and underground structures have design technique deficiencies and need significant research. Interdisciplinary problems are always the last to be tackled because research is normally confined to a single discipline or to a narrow specialty. Elements of buildings occurring in the architectural-structural and mechanical, electrical-structural interfaces in the past have also been initially neglected in earthquake resistant design with research being confined to easily defined pure structural problems.

The "If it can't get in, we don't have to get it out" rationale for foundations is true enough, but it is important to realize that the mass of the building will attempt to respond to earthquake motions through the stiffest connections to the ground. If that connection is not strong or tough enough, long term vertical support for the building may be weakened; the potential for immediate more catastrophic failures is also present.

UNDERGROUND AND BASEMENT STRUCTURES

The primary problem for most structures in this category is displacement. The work in seismology defining actual movements should be coordinated with structural capabilities into usable criteria. References [3] and [6] are pertinent to this area.

FOUNDATION TIES

Horizontal axial force ties between isolated footings in structures have been required in some form or other by most seismic codes. The purpose of such ties has never been defined however, and the requirements have therefore been inconsistent. There are three possible purposes for tying foundations together:

1 - Lateral Buckling at Column-Foundation Joint

As illustrated in Figure 1a, deep foundations such as piles and caissons could buckle at the column-foundation joint if the joint is forced out of the load line by earthquake motion. This is clearly a danger in very soft soils using tip bearing piles. However, present codes require ties (or equivalent restraint) at all "pile caps and caissons". Is the same buckling potential present for short piles with moment capacity at the column foundation joint? for friction piers in competent soils?

2 - Out of Phase Motions

As illustrated in Figure 1b, ties could help mitigate damaging effects from out of phase motions within a building. If this is the case, why are isolated spread footings presently exempted?

The presently accepted requirement for foundation ties is 10% of the higher column load. Cohesionless soils have relative low ultimate passive pressures but can generate large friction (.65N) forces whereas cohesive soils often exhibit low friction but ultimate passive pressures as high as vertical foundation design values. In either case, spread footings can often generate resistence to significant lateral movement within the soil approaching their design vertical load. Even allowing the tie its capability at ultimate strength, a 10% (or 20% for an interior column) lateral tie could hardly restrain differential spread footing movement caused by out of phase motions, except in extremely soft soils. Massive caps required for most pile and caisson installations in addition to passive pressures at the tops of the piles themselves can also be shown to often generate resistances far in excess of 10%. If the cap-pile interface cannot take the same 10% or 20% of the vertical load in shear, the restraint of the tie could actually cause failure at that location should out of phase motions be in direct opposition. The extent of this problem must be defined before rational tie criteria can be developed.

3 - Differential Lateral Foundation Movements Due to Differential Structural Stiffnesses

It is often stated that movement at the foundation level is beneficial because of energy absorbtion and reduced response. This is true but the movement should be small and equal throughout the building. Since the seismic motion enters the building through the stiffest elements, those elements will have the strongest tendency to move within the soil mass; the more flexible elements will move with the soil. This differential motion, when translated to the second floor, could cause unexpected distress in the flexible elements. Referring to Figure 1c, the total motion at the second floor would be A, the structural deformation of the wall, plus B, the movement within the soil of the wall foundation. Stresses in the wall would be due to A alone. However, at the more flexible column, where the foundation has not moved within the soil, the stresses would be due to the total movement A + B. Lateral ties at the foundation level, if properly designed, may reduce B (all foundations would be anchoring the building rather than just the wall footing), but would maintain A as the stress producing deformation in all elements. The extent of this problem is directly related to the magnitude of movement B, rather than to foundation type.

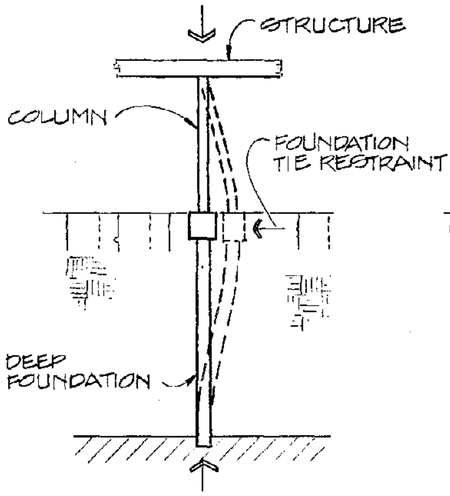

FIGURE 1a
Lateral Buckling at
Column-Foundation Joint

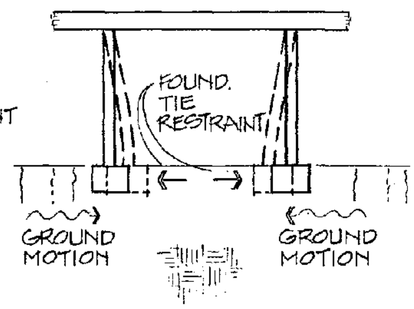

FIGURE 1b
Out of Phase Motion

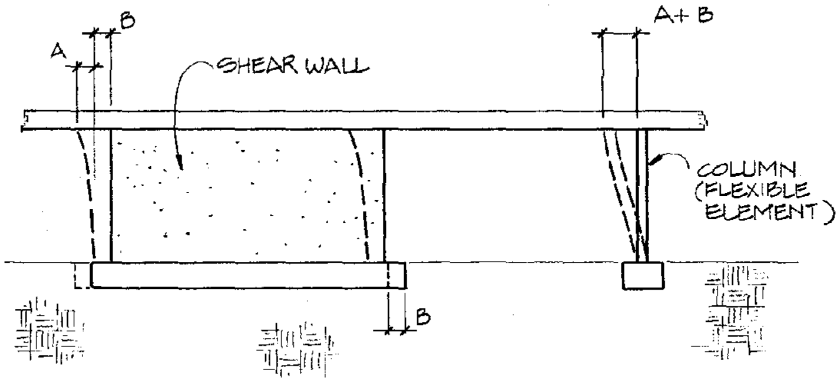

FIGURE 1c
Differential Lateral Foundation Movements
Due to Differential Structural Stiffnesses

Clearly the present code requirement for foundation ties dependent on foundation type, regardless of soil conditions, is irrational. The 10% capacity requirement is a continuing carry over, and is not based on any real criteria. Research is needed to determine which of the three possible purposes for ties are important and under what conditions. More realistic code requirements would undoubtedly follow.

"IF IT CAN'T GET IN . . ."

Increasing code forces and dynamic analysis have made us realize the disparity between structural design and geotechnical "allowables". In structures we have attempted to consider actual motions, realistic forces and yield of elements. Geotechnical input often has factors of safety for design and realistic predictions of behaviour are difficult to obtain. We have come to depend on relatively accurate predictions of what will yield and under what force levels. Inaccurate or conservative soil values could lead to unexpected results in foundations similar to overstrength concrete or reinforcing in ductile concrete. Varying soil or foundation stiffness can drastically change shear distributions within a building or could cause local failures in the stiffer foundation elements.

These problems can best be illustrated by an example. The building was approximately four hundred feet square in plan and seventy feet tall. Vertical loads were taken by a steel frame and lateral support was supplied by concrete shear walls symmetrically placed throughout the building. The building was supported by drilled cast in place friction piers. Because of the high seismicity of the site and the critical nature of the building (a regional hospital), extremely high lateral forces were used in the design [2].

During the design it became apparent that the lateral forces distributed to the shear walls could not be taken by the wall foundation system alone in any reasonable manner. It could be argued that the design motions therefore could never get into the structure. However, the points of resistance to the motion would be under the shear walls. The nature of the failure of the wall foundation system to accept the motion had to be investigated. If the piers yielded prior to the soil yielding, ductility had to be provided at the expected hinge points and shear failure had to be prevented. In addition, the effect of the inelastic motion at the walls on the rest of the structure had to be determined. It was found that with reinforcing required for vertical loads, the piers in general yielded both at the pier cap interface and in contraflexure prior to significant lateral soil failure. This mechanism at the walls would then create additional movement and unknown moments and shears in the balance of the foundation system and first floor structure, at best causing unacceptable damage levels and at worst endangering the vertical load carrying capacity of the structure.

It was decided it was necessary to determine the relative rigidity of the entire pier system and attempt to produce a uniform yielding of all piers at approximately the same force level. Using modulus of subgrade reactions from Terzaghi [5], Reese [4], and advice from the soils consultants, and considering varying top of pier restraint by the structure as well as reduction in stiffnesses due to group action, eight different pier types were identified. Using the slab on grade as a diaphragm, shear was distributed to the piers and the piers reinforced for moment accordingly. Using a procedure developed by Broms [1] utilizing top and contraflexure yield moments, ultimate shear envelopes of the various piers were calculated and shear reinforcing provided for this limiting case. A system was thus created that forced uniform pier yielding throughout and prevented shear failures within the piers. It was found that, because all piers were used, this procedure raised the overall capacity of the pier system at yield to a level consistent with the design of the structure.

It is not suggested that such an analysis is necessary for all buildings, but this example points out the necessity of at least qualitatively considering the effect at all load levels on the structure of foundation systems that "don't let the force in". The benefits of loose coupling in terms of reduced response should be taken advantage of with full awareness of all ramifications.

EFFECTS OF FOUNDATION TRANSLATION ON ELEMENT STIFFNESSES

Mr. Wyllie has shown the large effect that foundation rotation can have on relative wall rigidity and therefore on shear distribution. In a stiff shear wall building, pure translation or sliding at the foundation level can have the same effect unless the slippage magnitude happens to be proportional to the wall rigidities. This effect merely adds more uncertainty to the results of the highly sophisticated computer analysis in common use.

CONCLUSIONS

It is clear that presently the effects of foundation coupling, both translational and rotational, and possible out of phase motions casts grave doubts on the accuracy of our structural analyses. These effects might also explain much of the selectivity of earthquake in damaging buildings. Unfortunately, foundations are not visible after an earthquake and, barring serious failure, they are seldom excavated and inspected. In many cases, even such inspections would not reveal the real effect the foundations had on building response. In terms of improving the accuracy of structural analysis and also raising the earthquake resistance of buildings through code improvement, Mr. Wyllie's research suggestions should be seriously considered as top priority.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

CAST-IN-FIELD REINFORCED CONCRETE SYSTEMS FOR NEW BUILDINGS DESIGN OF FOUNDATIONS

by

Stanley E. Teixeira
Structural Engineer
H. J. Brunnier Associates

INTRODUCTION

The design of foundations for buildings in zones of high seismic activity requires certain special considerations. There are also certain matters of judgment exercised in the design; research in certain areas would enhance the confidence in this judgment.

FOUNDATIONS

The subject of foundations for earthquake-resistant reinforced concrete buildings is equally applicable to foundations for buildings constructed of any material, whether structural steel, wood, or concrete.

What makes foundation design for buildings in zones of strong seismic activity different from foundation design in zones of low activity? There are a few fundamental differences that must be recognized, but recognized more in a qualitative sense than quantitative.

Basically, designing a foundation system in a zone of strong seismic activity involves designing a system that can act as a unit by resisting being displaced either horizontally or vertically by strong ground shaking. At the bottom of relatively narrow shear walls, for instance, adequate overturning resistance must be provided by engaging sufficient weight at the foundation level. Frequently it is necessary to design a system of foundation tie beams that are capable of engaging the weight that is available from neighboring footings and columns.

One real void in our knowledge of earthquake response is a quantitative evaluation of overturning forces. Some of our codes now demand, in the judgment of many, consideration of unrealistic overturning forces; and this is in the face of the fact that in the history of earthquake failures, no buildings have been known to overturn, except as the result of ground failure.

It may also be necessary to provide a system of ties at foundation level to distribute the horizontal shears among several footings.

Common types of foundations include piles, both drilled and driven, spread footings, and mat foundations. Driven piles may be structural steel, cast-in-place concrete, or precast (usually prestressed) concrete.

The mechanics of the transfer of horizontal forces between soil and piles in various classifications of soil should be a consideration in the design of the piles, yet there seems to be little known about this phenomenon; this matter could be considered a subject for research involving the expertise of the soils engineer as well as the structural engineer. In the past, whenever this phenomenon has been recognized in the design of the piles, it has been answered purely by judgment. In the case of concrete piles, the design results have varied widely, from placing relatively light reinforcing in the top few feet of the piles to extending rather heavy reinforcing the full length of the piles.

Interconnecting individual pile footings with ties, as required by most codes, seems prudent in order to force a unified response of all footings; however, is the arbitrary code force requirement of 10% of the column load appropriate for the design of the ties in all soil classifications?

Basement slabs which have been designed for hydrostatic uplift and mat foundations inherently provide strong ties among columns.

A similar requirement and question is applicable to foundation ties interconnecting spread footings on ground of relatively low bearing capacity.

The design of mat foundations involves considerable judgment in estimating the parameters of contact soil pressures that may occur during the life of the structure; for instance, while a certain distribution of pressure can be assumed for the early life of the structure, it must be recognized that the pressure distribution will change as settlement occurs. The distribution of pressure under a mat after several years may differ markedly from the distribution in the early life; the redistribution is influenced by the relation between the stiffness of the mat and the compressibility of the soil, among other influences.

Having been involved in the design of several heavy mat foundations, some on piles, but most soil bearing that involve judgment in establishing the parameters of the pattern of soil contact pressure, I have long believed that useful and informative research could result from instrumenting the soil under the mat to record the contact pressures as they change over a period of years.

Mat foundations are usually heavily reinforced; therefore, serious practical considerations must be given to simplifying the placing of reinforcing steel and developing a pattern of placing rebar that will facilitate the placing of concrete. Concrete is normally placed rapidly and in large volume; space must always be available between rebars to accommodate chutes, hoppers, "elephant trunks", vibrators, and other placing appurtenances.

Another useful area of research would involve determining criteria for the seismic design of basement walls, retaining walls, and other earth retaining structures.

DESIGN CODES

In my judgment, the most desparate need, now, for structural engineers involved in reinforced concrete design, in seismic zones or not, is a simple, workable concrete design code. ACI 318 is absolutely contrary to this need.

Concrete is basically an unsophisticated product and the design of reinforced concrete does not warrant the involved mathematical exercises of ACI 318; especially when considering the many variables that cannot be quantitatively recognized in design; e.g., variations in load, concrete properties and rebar properties, construction tolerances, variations in curing, reshoring and other variables too numerous to mention. Stresses in reinforced concrete structures, furthermore, are time dependent and are also very sensitive to differential settlements in foundations.

The fact that one of the marvels of our age, the electronic computer, is available to solve complex mathematical expressions is no justification for the complexity of ACI 318.

Structural engineers must be liberated from the needless and repugnant mathematical enigmas of ACI 318 that give a deceptive impression of the sophistication of reinforced concrete and of the analytical precision required for satisfactory designs. Successful designs do not result from the solutions of these enigmas, but do result from a qualitative recognition of the properties of concrete and from devoting ingenuity and creativity to the development of appropriate framing schemes, details, and the presentation of them in thorough and workable contract documents.

The need of structural engineers for a simple, useable reinforced concrete design code cannot be exaggerated, in my opinion.

1515

EXPERIMENTAL INVESTIGATIONS OF REAL BUILDINGS, MODELS OF COMPLETE BUILDINGS, AND LARGE SUBASSEMBLAGES OF BUILDINGS; CORRELATION WITH ANALYTICAL INVESTIGATIONS AND WITH DATA FROM FIELD OBSERVATIONS OF EARTHQUAKE DAMAGE

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

DYNAMIC RESPONSE INVESTIGATIONS OF REAL BUILDINGS

by

Sigmund A. Freeman, Kenneth K. Honda, and John A. Blume
URS/John A. Blume & Associates, Engineers
San Francisco, California

INTRODUCTION

During the last decade there has been a great advance in our understanding of the response of real buildings to earthquake type motion. Some of the major contributing factors to these achievements include the development of high speed computer programs, increased development and use of seismic recording instrumentation, the abundance of data obtained from the San Fernando earthquake of February 1971, and the contribution of the structural response project of the United States Energy Research and Development Administration (formerly the United States Atomic Energy Commission) through its Nevada Operations Office (ERDA-NV).

Computers have been an aid to analyzing buildings, analyzing instrumentation records, and in correlating building motion and ground motion.

Because of the requirements of the Uniform Building Code (UBC) [39], requirements of local building officials (e.g., Los Angeles building department), and programs of government agencies such as the United States Geological Survey (USGS) [37], more instruments are being installed and presumably maintained in buildings that have been or will be subjected to strong earthquake caused ground motion.

During the San Fernando earthquake of February 9, 1971 many records were obtained from buildings that had instruments located on the roof, at mid-height and on the ground floor [29]. Through the work of many individuals and engineering firms plus the support of some government funding, several of the buildings were analyzed to correlate measured with calculated results [33].

Since 1964, URS/Blume has been conducting a structural response project, under contract to ERDA, which includes the developmental effects prediction guidelines for structures subjected to ground motion caused by underground nuclear explosions (UNE). Two of the by-products of this project have been the collection of structural response data for Las Vegas high-rise buildings subjected to ground motion caused by UNE's at the Nevada Test Site and the detailed studies made on two four-story reinforced concrete test structures constructed at the Nevada Test Site. [2,3,4,5,10,11,12,15,16,17,19,20,23,26 & 27]

In addition to the above, there have been many contributions to this subject in prior decades by some of the pioneer individuals and organizations in the field of earthquake engineering. [1,7,9,13,14,18,28,31,32,34,35 & 40] Time and space do not permit acknowledgement of all the contributors, past and present, to this subject; but, by means of this paper we hope to encourage participation from other members of this workshop by reviewing the state-of-the-art, discussing some projects we have been affiliated with, and presenting an outline for future needs.

MEASURING RESPONSE

Instrumentation

Considerable improvements have been made during the last decade in the seismic measuring devices and recording equipment, particularly as they apply to building response measurements. These measurements were, of course, necessitated by the improvements in the analysis techniques and in the better understanding of the response of buildings to earthquakes.

It has generally been the practice to use a self-contained tri-axial accelerograph located at roof, mid-height, and basement of buildings to satisfy code requirements. [37,39] Since they are self-contained, there is no accurate time correlation between the recordings at the various levels. Because of the bulkiness of the equipment, it cannot be placed at an ideal location in the building. The locations were most often dictated by the building owner, depending on the availability of space. Oftentimes, one set may be placed in a closet and another on a stair landing, at opposite ends of a building. A new system, which has been displayed by USGS and California Division of Mines and Geology, is a multi-channel accelerometer cabled to a central recording unit. With this system, accelerometers can be attached to the structural framing at normally inaccessible and remote locations, space requirements are minimal, and recording equipment can be centrally located for easy maintenance and record retrieval. It is therefore possible to obtain simultaneous time correlated recordings of ground motion and building motion. With these records, techniques permit determination of complete dynamic properties of buildings. Another improvement is the seismic equipment's automatic change in sensitivity. When the motion exceeds the limits of the recording system, the sensitivity is automatically changed to accommodate the change; thereby enabling recordings of a wider range of amplitude of motion.

Parallel improvements have also been made in the recording equipment coupled with the data processing equipment. It was a major undertaking by the California Institute of Technology to digitize and process the ground and building motion records after the San Fernando earthquake of 1971 [29]. Although new recording equipment cannot completely replace the data processing procedure, it certainly improves the processing. One of the recent developments is that of recording the motion on film and on magnetic tape (analog or digital). Data can be converted to computer use very quickly by passing the time consuming manual digitization of paper records.

The above discussions were on instrumentation and recording systems in general. Of course, instrumentation has been used for specific projects by various research facilities for quite sometime. The instrumentation program in the Earthquake Research Institute building at the University of Tokyo is a prime example. Seismic instruments as well as strain and story-drift measuring devices were placed in this building. In addition, seismic instruments were placed at several locations on and below the ground.

Another example of specific projects is the instrumentation and data processing used in our work with ERDA-NV program. The instrumentation was similar to that discussed earlier; seismometers placed at various locations on the building and cabled to a central recording system. Data are recorded to analog tape which can record up to four days. With proper hardware and software, data can be quickly converted to digital format for computer application.

Sources of Motion

The types of motion that may be recorded can range from the response to large earthquakes to the measurement of ambient motion [9,24,29,33]. In addition to recording motion caused by natural events, lateral response of buildings can be induced by man-made events such as underground explosions [3,15, 26,38], building vibration generators [1,10,15,30], pull tests [15], and human excitation [16].

Processing and Analysis of Records

The time-history recordings of the motion are either graphically represented on paper or magnetically represented on tape. The paper records can be used (1) for a quick estimate of peak amplitudes and predominant periods; (2) for a detailed manual analysis to estimate modal characteristics of the structure; or (3) for digitizing to a computerized format [29]. The magnetic tape records can be reproduced on paper for visual inspection or they can be digitized for computer use. When records are digitized care must be taken to account for instrument characteristics and to make zero base line corrections. Once the data are properly digitized for computer use, the data can be used to compute response spectra or for time-history analysis. Response spectra of the ground motion can be used to correlate building motion with damping [3]. Spectral acceleration contours and profiles of recorded building motion can be used to determine the change in period during high amplitude motion [24] as illustrated in Figure 4 and discussed later in this presentation. Time-history methods which determine transfer functions between ground motion and building motion recordings can be used to determine building characteristics [25,36].

DYNAMIC TESTING

Some of the earliest test data consisted of measuring ambient motion and using a building vibrator [1,9]. Beginning in the early 1960's synchronized vibration generators with electronic speed control were developed [28,30] that added more sophistication to the testing procedures. During the 1960's a number of buildings were tested and analyzed for dynamic characteristics [7,13,31,32,34,35,40]. Since 1964, URS/Blume has been under contract to ERDA-NV to conduct structural response studies and provide other services related to safety. Following are summaries of three studies that were conducted under that contract that relates to the subject of this presentation.

Las Vegas High-Rise Building Response to UNE at NTS

As part of the structural research program being conducted for the U.S. Energy Research and Development Administration, Nevada Operations Office (ERDA-NV), by URS/John A. Blume & Associates, Engineers, the response behavior of high-rise buildings in Las Vegas, Nevada, due to ground motion caused by underground nuclear explosions (UNE's) at Nevada Test Site (NTS) has been measured and analyzed for the past 13 years [3,4,5,26,27]. There are approximately 40 buildings in Las Vegas which range in height from 9 to 32 stories and are located approximately 130 to 170 km from UNE's. Responses of these buildings to UNE generated ground motion have been measured on one occasion or another. Measurements range from a single three component accelerograph on a roof to a multi-channel recorders with seismic signals from various locations on a building, depending upon the availability of instruments. On one building over 40 measurements of building response have been obtained. On several occasions responses to earthquake generated ground motion have also been measured.

The usefulness of the measured response depends on the types of data obtained and the availability of analysis technique. For the response measurements only on the roof, the data are limited to a time-history of roof motion; extracted data include amplitudes of motion, natural periods of building, and duration of building response. Any change in response behavior can be determined by comparing period measurements from one event with those from other events.

With a multi-location system, measurements of building response at various locations on the building as well as the input ground motion can be monitored simultaneously on a single magnetic tape. From these data, techniques permit determination of complete dynamic characteristics of buildings [25,36]. Building characteristics thus obtained were used among others to evaluate the accuracy of structural modeling techniques, to observe change in building characteristics over a period of time, and to evaluate the influence of architectural partitions and elements.

An Example of a 21-Story Building--Figure 1 shows the displacement-period relationship observed on one of the buildings in Las Vegas [26]. Note the dispersion of periods, which vary from 1.4 to 2.1 sec. The building is a 21-story reinforced concrete frame system with nonstructural concrete block filler walls around the elevator and stairs. Although the period of the bare frame system was not measured, it was calculated to be about 3 sec. It is quite evident that the architectural filler walls, although not designed as such, are acting as a lateral force resisting system. When the walls were first installed, at a small amplitude of motion the stiffness contribution of the walls was equal to the stiffness of the structural frame system. As time elapsed, the contribution of the filler wall was reduced causing the period to lengthen Minor visible evidence of the separation of the concrete block walls from the structure system was observed. The separation of the wall from the structural system did not take place uniformly throughout the height of the building, but concentrated on a few lower stories. Relative horizontal movement between the soffit of the concrete beam and the top of the filler wall at one of these lower stories was measured to be about 10% of the total roof motion during response to UNE's. Thus, the relative interstory drift at this story was substantially greater than it would have been if the walls were reacting uniformly throughout the height of the structure or if the walls were removed. This means that the distribution of member forces throughout the building could be considerably different than that for which it was designed.

Other Las Vegas High-Rise Buildings--Data from other buildings show how periods change with amplitude of motion, previous exposure to ground shaking, and effects of the passage of time in addition to the effects of nonstructural elements. Some of this data has been published [3,4,5,19,26,27] and an effort is being made to publish more in the future.

Four-Story Test Structures

The two four-story reinforced concrete test structures, which were constructed in 1965 at the Nevada Test Site, are part of the ERDA structural response program associated with the detonation of underground nuclear explosions.

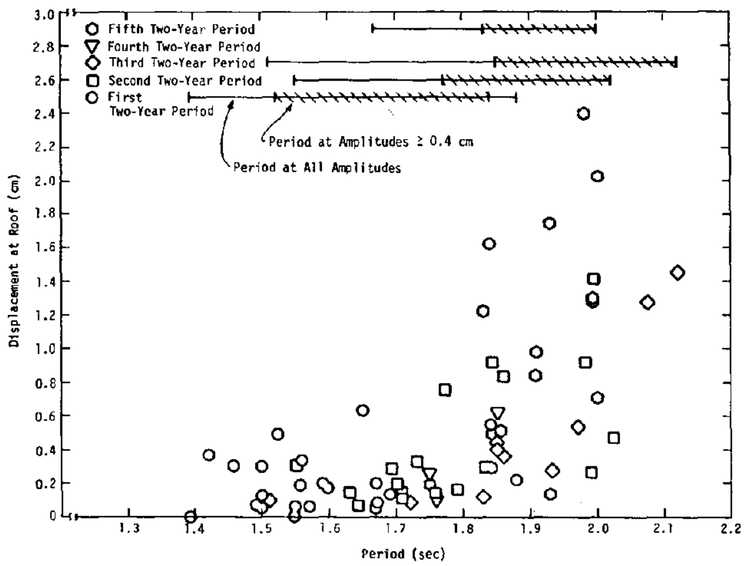


FIG. 1.--Displacement-Period Relationship, Building A (Longitudinal Direction)

The structures were built to obtain experimental data on the dynamic response characteristics of high-rise concrete buildings [10,11,12,15,17,20,23].

Description--The test structures are 3.7 m (12 ft) by 6.1 m (20 ft) in plan and consist of four 2.7 m (9 ft) stories. Four rectangularly tied corner columns are 41 cm (16 in) by 36 cm (14 in). Spandrel beams are 36 cm (14 in) by 30 cm (12 in) in the 3.7 m direction and 41 cm (16 in) by 38 cm (15 in) in the 6.1 m direction. The floor slabs are 15 cm (6 in) thick, reinforced for two-way action. The design of these structures was consistent with the 1963 edition of the American Concrete Institute Building Code [8]; design for lateral loads was based on horizontal static forces recommended by the 1961 version of the Uniform Building Code (UBC) for Seismic Zone 3 [39]. Some provisions for ductility and reserve energy absorption capacity [6] were also incorporated into the design of these structures. In anticipation of the possible additional weight of testing equipment and nonstructural partitions, the actual dead load plus 100 psf live load was used in computing the weight of each story for the design lateral force calculations. Thus, when loaded with dead load plus live load, the structures satisfied UBC Seismic Zone 3 requirements; but, when only dead load was present (the most common configuration), these structures had nearly twice the capacity of the 1961 UBC Seismic Zone 3, or approximately the capacity required by the 1976 UBC Seismic Zone 4 [39]. The design and construction of the structures is discussed in detail in Reference 15.

Dynamic Tests--In the course of 10-years, various methods of dynamic excitation were used to test the 4-story structures. The most frequent source of dynamic excitation was the ground motion generated by the detonation of underground nuclear explosions at the test site. The ground zeros or epicenters of these simulated earthquakes were located from less than 3 km (2 miles) to more than 50 km (30 miles) from the test structures, producing ground motion signals at the structures with a variety of amplitudes and frequencies. The maximum roof displacement observed during over 40 underground nuclear detonations was approximately 2.5 cm. References 15 and 20 give additional data.

Another method of dynamic excitation was the pull-release procedure, also known as a pull test. This method imposes a static horizontal deflection on the structure by pulling with a predetermined force on a steel cable attached to the building and releasing this force suddenly, causing the structure to experience free vibration. Although all modes were initially excited, the fundamental mode dominates the response after the first few cycles of motion. In the course of these tests, forces up to 40,000 N (9000 pounds) were applied at various floor levels of the test structures, causing maximum dynamic roof displacements up to 0.5 cm. The principal advantages of this procedure are ease of field implementation and ease of data reduction to determine the period or damping ratio under the recorded response motion. The pull-release procedure was used during Test Series A through N [10,15,20]. Some sample results are shown in Figures 2 and 3, and Table 1.

The 4-story test structures were also tested using counterrotating-mass vibration generator built by URS/Blume [15] and reciprocating-mass vibration generator built by Sandia [10]. These mechanical devices can produce approximately steady-state harmonic motion and enable in-depth study of the response of structures over a wide range of frequencies and amplitudes. Through the use of these devices, it was possible to isolate and excite four structural modes of vibration and determine the dynamic response characteristics of each.

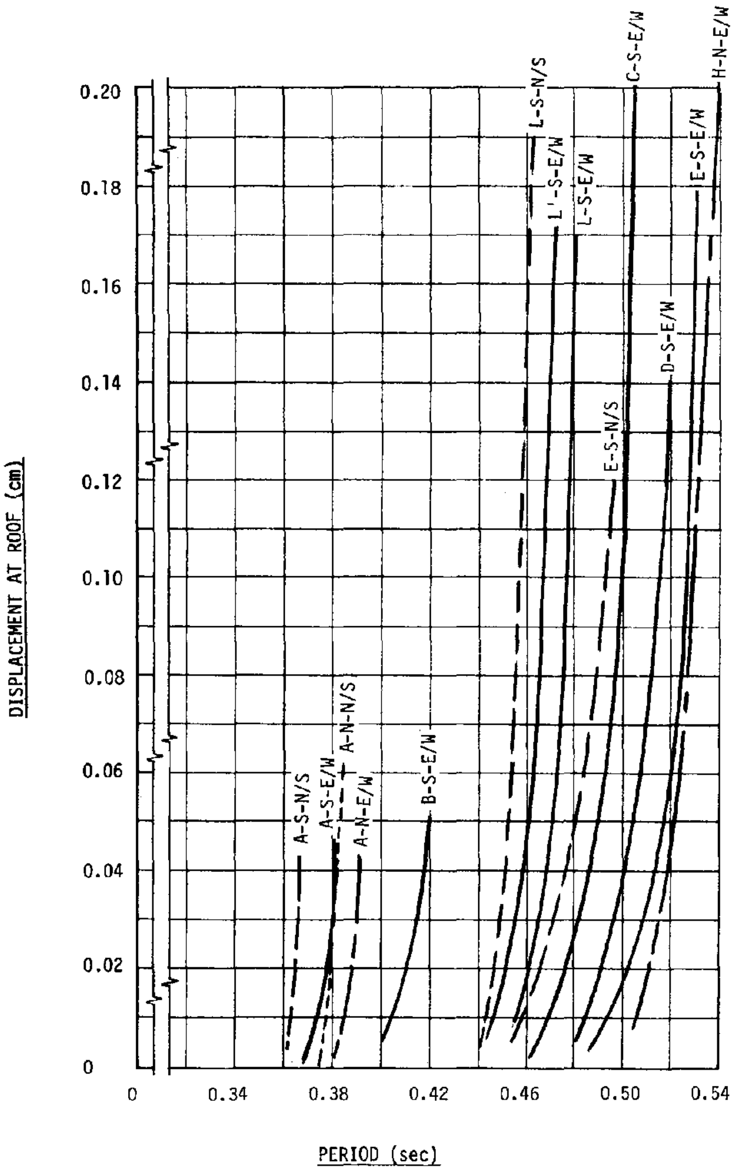


FIG. 2.--4-Story Test Structures-Period Data.
From Pull-Release Tests A through L, North (N) and South (S) Structures, in the Longitudinal (N/S) and Transverse (E/W) Directions. No Partitions Parallel to Direction of Motion.

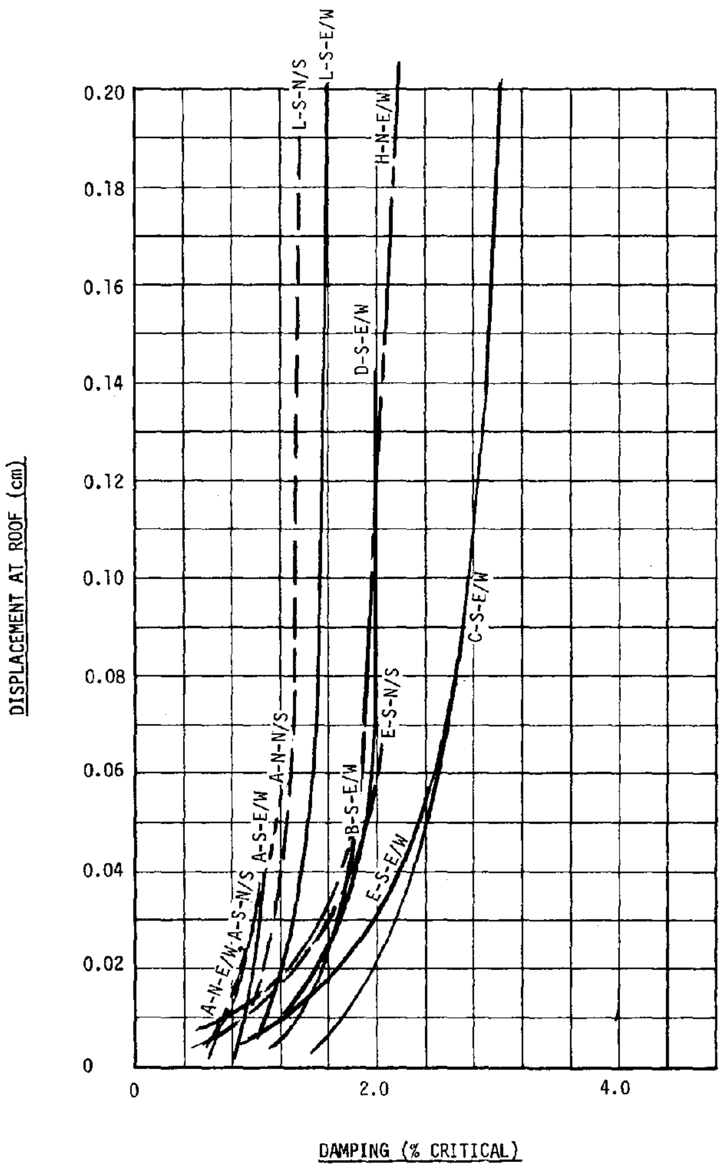


FIG. 3.--4-Story Test Structures, Damping Data.
From Pull-Release Tests A through L, North (N) and South (S) Structures, in the Longitudinal (N/S) and Transverse (E/W) Directions. No Partitions Parallel to Direction of Motion.

TABLE 1.--4-Story Test Structure Stiffness Characteristics at Lateral Roof Displacement = 0.10 cm. From Pull-Release Tests A through L.

TEST SERIES	SOUTH STRUCTURE														
		EAST-WEST							NORTH-SOUTH						
	M_r	T	ζ	K_r	K_f	K_p	Partitions	Rocking	T	ζ	K_r	K_f	K_p	Partitions	Rocking
A	1.00	0.385*	1.1	1.08	1.08	1.0	--	5%	0.37*	1.2	1.17	1.17	1.0	--	3%
B	1.02	0.425*	1.9	0.91	0.91	1.0	--		0.34*	1.6	1.41	1.15	1.22	P	
C	1.02	0.50	2.7	0.65	0.65	1.0	--		0.40	3.0	1.02	0.83	1.22	P	
D	1.02	0.51	2.0	0.63	0.63	1.0	--		0.43	2.6	0.88	0.72	1.22	P	
E	1.00	0.53*	2.7	0.57	0.57	1.0	--		0.49*	2.1	0.67	0.67	1.0	--	
F	1.14	0.34±*	3.5±	1.58±	0.57	2.8	CB	9%	0.47	1.5	0.82	0.67	1.22	--	2%
G	1.14	0.44±	2.2	0.94±	0.57	1.65	CB		0.51	1.9	0.70	0.67	1.05	--	
H	1.14	0.48	2.2	0.79	0.57	1.4	CB	3%	0.50	1.8	0.73	0.67	1.09	--	3%
J	1.17	0.36±*	2.8	1.45±	0.57	2.5	CBP	8%	0.47	1.7	0.85	0.67	1.27	--	1%
K	1.17	0.42	1.9	1.06	0.76	1.4	CBP	6%	0.44	1.5	0.97	0.85	1.14	--	2%
L	1.00	0.47	1.5	0.72	0.72	1.0	--	3%	0.46	1.3	0.76	0.76	1.0	--	1%

TEST SERIES	NORTH STRUCTURE														
		EAST-WEST							NORTH-SOUTH						
	M_r	T	ζ	K_r	K_f	K_p	Partitions	Rocking	T	ζ	K_r	K_f	K_p	Partitions	Rocking
A	1.00	0.393*	1.1	1.04	1.04	1.0	--	9%	0.388*	1.4	1.07	1.07	1.0	--	5%
B	1.10	0.385*	3.3	1.19	0.85	1.40	G		0.35*	3.7	1.43	1.03	1.39	G	
C	1.10	0.48	3.9	0.76	0.62	1.23	G		0.44	4.9	0.93	0.75	1.24	G	
D	1.10	0.49	3.9	0.73	0.61	1.20	G		0.42	4.7	1.00	0.75	1.33	G	
E	--	--	--	--	--	--			--	--	--	--	--		
F	1.10	0.48±	3.6	0.76±	0.60	1.27	G	6%	0.45	4.4	0.87	0.70	1.24	G	2%
G	1.10	0.51	4.3	0.68	0.60	1.13	G		0.44*	5.0+	0.93	0.70	1.33	G	
H	1.06	0.53	1.9	0.60	0.60	1.0	--	3%	0.45	4.5	0.84	0.70	1.20	G	2%
J	1.17	0.30±	4.1	2.08±	0.60	3.5	HCT	12%	0.40*	4.6	1.17	0.70	1.20x 1.40= 1.67	G	3%
K	1.17	0.33±*	2.7	1.72±	0.70	2.5	HCT	12%	0.40	4.6	1.17	0.85	1.20x 1.15= 1.38	G	5%
L	--	--	--	--	--	--	--		--	--	--	--	--		

*Extrapolation

M_r = relative generalized mass
T = fundamental period of vibration
ζ = % of critical damping
K_r = relative generalized stiffness for the overall building
K_f = relative generalized stiffness for the bare frame
K_p = K_r/K_f and represents the stiffness effects of the partitions. K_r/K_f is equal to 1.0 when there are no partitions
Partition Content: CB = Concrete block, CBP = Concrete block with plaster coat, P = Plywood, G = Gypsum wallboard, and HCT = Hollow clay tile

The counterrotating-mass device was used in addition to pull tests during Test Series C, D, F, H, L, and N [15,20], and the reciprocating-mass device was used during Test Series O [10,11,12].

For very quick and approximate determination of the fundamental mode period and damping ratio, the man-induced vibration technique was used. In this method, a man sways his body back and forth in approximate resonance with the fundamental mode. Because of the large dynamic amplification factor of these lightly damped structures, motion at and above the human perceptibility level can be produced [16].

High Amplitude, Destructive-Level Tests--The vibration tests described above have provided much information on the dynamic response of concrete frame structures at pre-yield amplitudes. Maximum stresses experienced by these structures during the vast majority of the tests conducted through 1973 were well below working stress levels; however, at least one UNE may have generated sufficient structure motion to cause yield stresses in some of the reinforcing steel. Although no serious damage to the structure was observed, there was minor damage to some nonstructural partitions. In 1973 planning began for high-amplitude, destructive-level vibration testing to gather data on the dynamic response of a full-scale reinforced concrete structure. The lack of experimental data in this area suggested that such a study would be especially valuable because it could yield new information about structural response as structural damage and inelastic behavior were experienced. Data on the Sandia vibration generator and the high amplitude testing are given in References 10, 11, and 12.

Results--In the course of the vibration tests of these structures, a great variety of instrumentation and recording procedures were used to measure the horizontal and vertical motion of the floors. Velocity and acceleration time histories of motion were recorded both on paper strip charts and analog tapes. Many analog records have been digitized and transferred to computerized magnetic tapes for further analysis. In the early years of the testing program, hand analysis of the strip charts was the only reliable method for determining the response characteristics of the structure (i.e., period, damping ratios, mode shape, and maximum amplitude of response). With the improvement of digitization and computerized analysis techniques, the manual methods have played a less significant role and a time domain analysis technique [36] has been used to compute response characteristics.

The results of the testing programs have demonstrated the time- and amplitude-dependent nature of the dynamic characteristics of the reinforced concrete structures. Prior to the high-amplitude testing, measured fundamental periods of vibration ranged from 0.37 second to 0.55 second for the bare frame structures. When nonstructural partitions were installed, periods were reduced in some cases to less than 0.30 second. During the high amplitude testing, where yielding and damage were induced, the period lengthened to 0.9 second. Prior to the high amplitude tests, damping was approximated at values ranging from 1% to 3% of critical for the bare frame and up to 5% with partitions. The high amplitude testing indicated damping up to 4% of critical for the bare frame structure.

San Fernando Earthquake

There were 66 high-rise buildings in the major Los Angeles area that were instrumented with strong-motion accelerographs at the time of the February 9, 1971, San Fernando earthquake [33]. About one-half of these had valid records at the three instrumented floor levels (roof, mid-height, and ground floor). Reports on the investigation and analysis of eleven of the instrumented buildings are included in Reference 33. Six of these buildings have reinforced concrete structural systems. One is a ductile moment resisting frame, four are moment resisting frame systems, and one has a shear wall system.

Description of Two Identical Holiday Inn Buildings--Two of the above reinforced buildings are Holiday Inn Motor Hotels, one located in Van Nuys (Orion) about 20 km (13 miles) south of the epicenter, and the other located in Los Angeles (Marengo) about 40 km (26 miles) south of the epicenter [21,24,33].

Both of the buildings are seven-story reinforced concrete frame structures that are approximately 19 m (62 ft) wide and 46 m (151 ft) in length. The buildings were designed by the same engineer and architect and were built for the same client. Both sets of drawings are dated late 1965. The only differences are in building orientation and elevator location. With the exception of the framing at the elevator openings, the reinforcing and dimensions of the buildings are identical. The differences due to the elevator location have a negligible effect on the structural characteristics of the buildings.

Column spacing is typically at 6.1 m (20 ft) centers in the transverse direction and 5.8 m (19 ft) centers in the longitudinal direction. Spandrel beams, typically 36 cm (14 in) wide and 58 cm (23 in) deep, are located around the perimeter of the structure. The floor system is a reinforced concrete flat slab, 20 cm (8 in) thick at the roof, 21 cm (8-1/2 in) thick at the 7th through 3rd floors, and 25 cm (10 in) thick at the 2nd floor.

The structures are constructed of regular weight reinforced concrete. Lateral forces in each direction are resisted by the interior column-slab frames and by the exterior column-spandrel beam frames. Each exterior frame is roughly twice as stiff as each interior frame because of the added stiffness afforded the exterior frames by the spandrel beams.

The interior partitions, in general, are gypsum wallboard on metal studs. Cement plaster of 2.5 cm (1 in) thickness is used for the exterior facing at each end of the building and at the stair and elevator bays on the long side of the building. The cement plaster is supported by double 16-gage metal studs. Four bays of brick masonry filler walls, as well as some additional cement plaster walls, are located at the 1st story. Although none of these wall elements are designed as part of the lateral force-resisting system, they do contribute in varying degrees to the stiffness of the structure.

Recorded Motion of the Holiday Inn Buildings--Peak horizontal ground acceleration was measured at about 0.25g at Orion and 0.15g at Marengo. The peak roof acceleration was measured at about 0.40g for both structures; however, at the peak building response the Orion building was responding at a longer fundamental period (1.6 sec) than the Marengo building (1.1 sec). The maximum horizontal roof displacements were approximated at 19 cm (7 in) at Orion and 11 cm (4 in) at Marengo. The fundamental periods were observed to vary during the earthquake motion ranging from 0.6 sec during the early portion of the records to the longer periods at the later portion of the recorded motion. An illustrative representation of the change in period is shown in the spectral acceleration contours and profiles of Figure 4.

These periods were compared with values obtained from previously recorded ambient motion as well as motion recorded after the earthquake, and are summarized in Table 2 (V.N. is Orion and L.A. is Marengo). Table 3 summarizes calculated fundamental periods using bare frame mathematical models as well as mathematical models that approximate the participation of nonstructural elements [21,22,24]. A graphical representation of displacement versus period is shown in Figure 5 and a graphical representation of acceleration versus displacement is shown in Figure 6. A detailed explanation of this data can be found in References 21, and 24.

Results of the Holiday Inn Study--The investigation of these two structures has been fairly extensive; fortunately there was a good supply of data, the buildings have relatively uncomplicated features for ease of analysis [21, 24], and there was a reasonable amount of funds available to perform the investigation. The results indicated that both structures exceeded their elastic limit threshold during the earthquake and have suffered permanent losses of stiffness. On the basis of the analysis, one would have expected more damage than was actually observed. Peak accelerations were 4 to 5 times design levels and displacements were estimated at 8 to 15 times design level. The cost of damage repairs was about 11% of the initial construction cost for the Orion building and 7% for the Marengo building; however, essentially all this cost was for nonstructural damage such as partitions, bathtubs, bathroom tile, etc. Structural repair amounted to only about 1% to 2% of the total repair costs. The results of the investigation illustrated how the period changed with amplitude of motion and how the stiffness degraded. Ambient period measurements should be used with caution, the participation of nonstructural elements must be considered, and methods of modeling stiffness characteristics (e.g., cracked section, uncracked section, slab participation) must be established.

FUTURE NEEDS

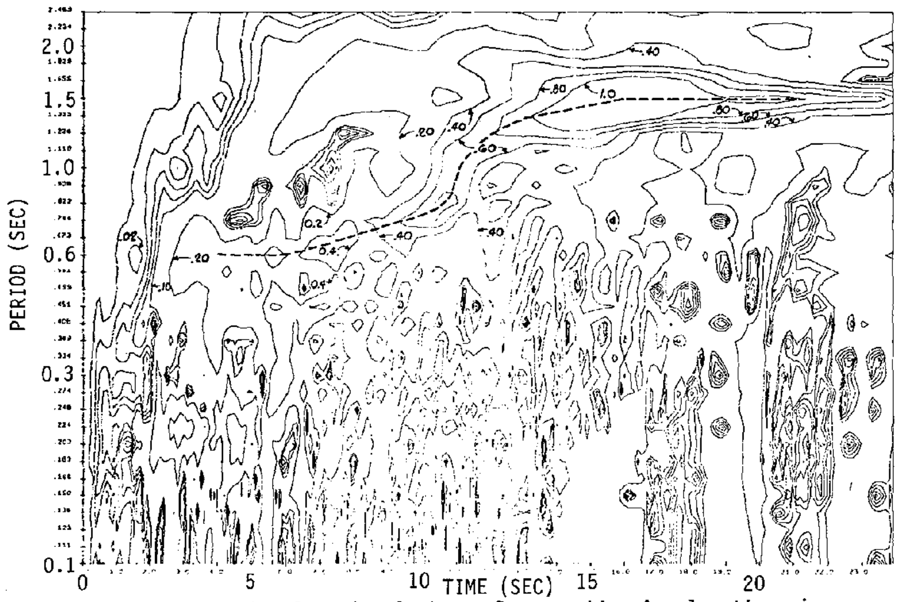
Following is an outline that might be used as a guide for a discussion on future needs in the field of instrumentation and dynamic testing of real reinforced concrete buildings.

A. Consolidation of Existing Response Data

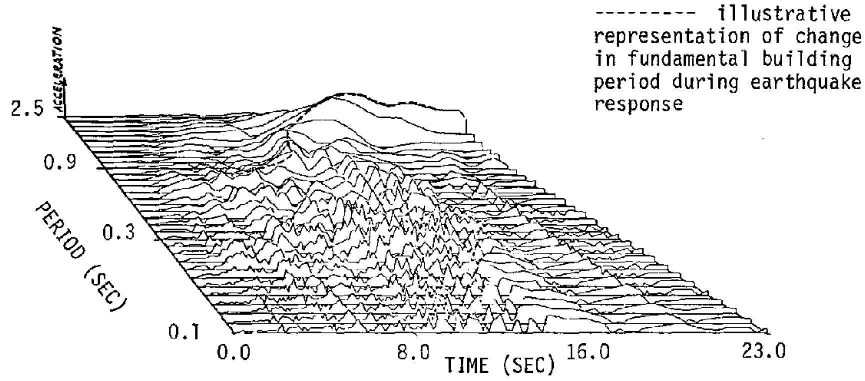
1. Instrumentation data
2. Analysis data

B. Normalization of Data for Direct Comparison

1. Methods of modeling reinforced concrete structures
2. Amplitudes of motion versus period of vibration
3. Damping
4. Other dynamic response parameters



a. Spectral Acceleration Contours Representing Accelerations in g



b. Spectral Acceleration Profiles

FIG. 4.--Response Envelope Spectra (5% Damped) Using Transverse Direction of Roof Acceleration Record of the Holiday Inn Orion (V.N. Building

TABLE 2.--Observed Fundamental Periods (Sec) for the Holiday Inn Orion (V.N.) and Holiday Inn Marengo (L.A.)

		Transverse		Longitudinal	
		V.N.	L.A.	V.N.	L.A.
Pre-Earthquake V.N. 2/16/67 L.A. prior to 4/8/68		0.48	0.49	0.52	0.53
Earthquake 2/9/71	Early Portion	0.70	0.63	0.70	0.60
	Late Portion	1.6	1.15	1.5	1.1
Post-Earthquake 2/71 and 3/71		0.68	0.63	0.72	0.64
Aftershock 3/31/71		1.1	--	1.2	--
Post-Repair 5/27/71		0.58	--	0.64	--

TABLE 3.--Calculated Fundamental Periods (Sec) for the Holiday Inn Orion (V.N.) and Holiday Inn Marengo (L.A.)

	Transverse	Longitudinal
Structural Frame Only*	0.88 (T-2)**	0.79 (L-2)
With Partitions Modeled as Diagonal Struts	0.83 (T2-P)	
Partitions as Struts and Cement-Plaster as a Shear Wall	0.54 (T2-PW)	0.68 (L2-PW)
Additional Cement-Plaster Wall Included	--	0.63 (L2-PW2)

*Based on beams and slabs @ gross concrete section, slab width @ 60% tributary width, and columns @ gross transformed section.

** Letters in parentheses identify the model being used.

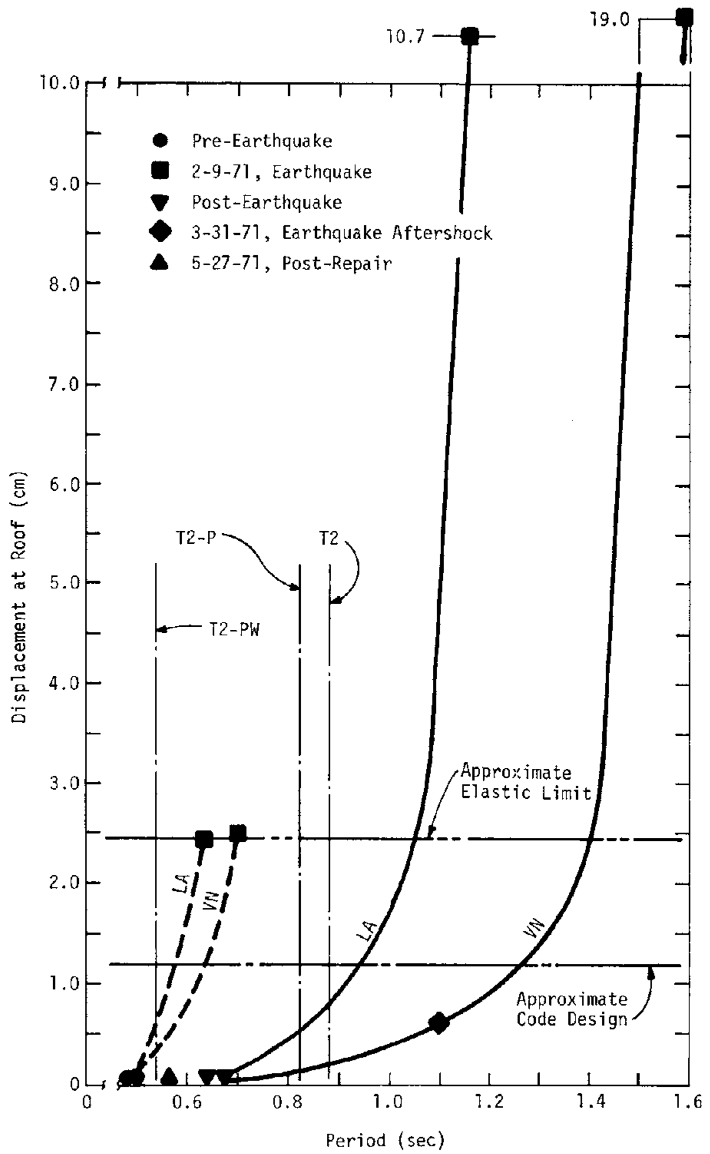


FIG. 5.--Peak Response Observations, Transverse Direction for the Holiday Inn Orion (V.N.) and Holiday Inn Marengo (L.A.) Buildings [24]

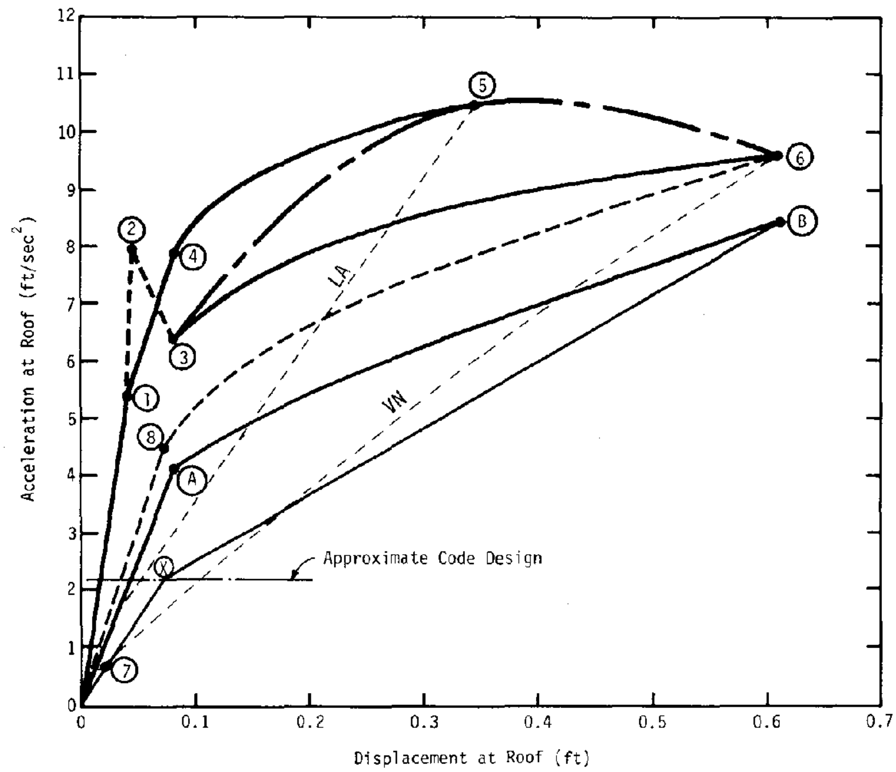


FIG. 6.--Roof Acceleration Versus Roof Displacement, Transverse Direction for the Holiday Inn Orion (V.N.) and Holiday Inn Marengo (L.A.) Buildings [24]

C. Review of Existing Instrumentation and Make Recommendations for Instrument Locations and Arrays [37]

D. Evaluate Present Analysis Procedures and Make Recommendations for Standardization of Analytical Procedures

E. Standardizing Terminology (e.g., relating to amplitude of motion with the following)

1. Elastic response
2. Elastic limit
3. Period
4. Damping
5. Ultimate strength
6. Nonlinear response

F. Recommendations for Future Test Programs

1. Structures
   a. Existing buildings
   b. Test structures
2. Testing
   a. Nondestructive testing
   b. Destructive tests
   c. Repairing damage and retesting

G. Goals

1. Develop criteria and procedures for determining dynamic characteristics of reinforced concrete structures
2. Develop procedures to identify and determine the various stages of load deflection cycles (e.g., elastic limit and ultimate strength) of reinforced concrete structures
3. Develop procedures to determine the effect of prior loading on future responses
4. Damage-cost relationships

CONCLUSIONS

A great amount of test data and analyses are available. The challenge is how to make the best use of this information and to direct future work towards a goal that will help us better understand how reinforced concrete structures respond to earthquakes so that we can improve design and construction practices.

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EXPERIMENTAL INVESTIGATIONS - CORRELATION WITH ANALYSIS

by

Robin Shepherd
Visiting Professor of Civil Engineering
California Institute of Technology
On Leave from the University of Auckland, New Zealand

and

Paul C. Jennings
Professor of Applied Mechanics
California Institute of Technology

SUMMARY

In an attempt to improve understanding of the earthquake response of reinforced concrete buildings, about one hundred such structures throughout the world have been tested or instrumented in some manner. Predictions of dynamic characteristics have been compared with observed behavior when excitation is provided by wind or other ambient excitations, by a controlled forced vibration exciter or by strong earthquake motions. For some buildings experimental results are available from more than one type of excitation.

In this article references to the various testing techniques and their results are given and, in particular, the trends exhibited by forced vibration tests are summarized. In addition, interpretations are made of response to strong ground motion and suggestions are advanced for further experimental testing of full-scale structures. The greatest needs are for the quantification of the capacity of reinforced concrete structures to resist strong ground motion and for additional experimental investigations of soil-structure interaction in buildings of the shear-wall type.

INTRODUCTION

In the last twenty years the improved availability and capabilities of high speed digital computers, which have allowed increasingly sophisticated mathematical models of structures to be used in the seismic design process, has been paralleled by the development of finely controlled exciting apparatus and sensitive response detection equipment. This modern instrumentation has made possible detailed experimental investigations on prototype structures, with the object of verifying assumptions made at the design stage and increasing our understanding of dynamic response in general.

Although the principles of vibration measuring instruments have been described thoroughly in classical texts in mechanical vibrations, only relatively recently have reliable field instruments suitable for testing of buildings been available commercially. Similarly, effective steady-state testing only became a practical possibility when speed control systems were developed, to ensure a satisfactory level of performance in the adverse conditions near structural resonance. These developments have been fostered, in part, by an increasing interest in full-scale testing that has developed over the last fifteen years [1]. At the present time well-proven excitation systems and measuring apparatus are available from commercial sources and increasing activity in the area of dynamic testing of full-scale buildings is expected.

EXCITATION SOURCES

Provided that sensitive transducers are available, a convenient technique used to establish the small amplitude natural periods and corresponding deflections of structures is to measure the response to wind or microtremor excitation. Difficulties arising from the presence of "noise" from traffic or mechanical equipment, from the complexity of the building, and from the effects of the non-stationary, gusting, characteristics of the wind have been recognized, and good correlation usually has been obtained between observed and predicted responses of reinforced concrete buildings [2-7]. However, in the field of earthquake engineering the relationships between small amplitude vibration properties and strong motion seismic response is not sufficiently well established that ambient testing is preferred to alternative procedures in which larger amplitudes of motion occur. It is difficult to extrapolate from the amplitudes of an ambient test to that of earthquake response and the testing method giving the largest allowable amplitudes is recommended.

Spring-back tests involve the imposition of an initial deflection on a structure, typically by way of a tensioned cable or rocket thrust generator, and the subsequent recording of the free vibrations of the system when the additional load is suddenly released. This technique has been used with success on certain categories of structures, including tall chimneys and towers, but the magnitude of the forces necessary to apply significant distortions to reinforced concrete buildings has discouraged its application in this field.

Forced vibration testing using variable frequency exciters enables steady-state conditions to be achieved; the steady-state response permits determination of natural frequencies, mode shapes and damping. The latter variable is particularly hard to determine from ambient tests. Several rotating mass shakers have been developed, those having a vertical axis being restricted to generating horizontal loads, whereas machines with horizontal axes can be used to excite either horizontal or vertical oscillations [1,8,9]. Although the capacity of available shakers is insufficient to vibrate full-scale reinforced concrete structures into the range of damage or collapse, sinusoidal forces of a few tons magnitude can be generated and building deflections several times greater than those arising under ambient conditions can be imposed. Synchronous operation of more than one shaker can facilitate investigations of torsional dynamic properties, as well as simplifying the study of translational modes of vibration.

Artificially generated strong ground motion from explosions has been used on several occasions to vibrate reinforced concrete buildings [10,12]. Good correlation between the observed elastic response and that determined using mathematical modelling techniques has been reported.

The most realistic tests, conducted by recording the actual behavior of buildings when subjected to strong ground motion generated by a natural earthquake are necessarily infrequent. It is only in the last decade that arrangements have been successful to install strong motion recording instruments in over one hundred structures throughout the world. To the time of writing only the 1971 San Fernando earthquake has provided more than a few samples of response; even on this occasion no instrumented building was close to that area which experienced the most severe intensity of ground shaking, and the several structures that were severely damaged, or collapsed, were not instrumented [11]. It is to be expected, however, that the continuing programs of instrument installation and maintenance will ultimately yield reliable evidence of the loads and movements associated with the earthquake response of severely damaged structures.

MEASUREMENT

The difficulties encountered in attempting to measure absolute deflections in a building vibrating with frequencies in the one to ten hertz range have resulted in detection equipment being selected with careful regard to the object of the investigation. Where relative motions are the primary requirement, as in determining natural modes frequencies and mode shapes, velocity type instruments such as standard Willmore or Ranger seismometers have frequently been used. These units have the advantage of high sensitivity and convenient operation but are difficult to calibrate if absolute amplitudes of displacement are required. For moderate levels of responses, Statham accelerometers capable of convenient absolute calibration have proven useful [13, 14]. Where sufficiently large responses are available, as in a major earthquake, specially developed instruments [15] may be used and the displacements calculated from the measured accelerations.

The apparatus used to record building vibration has included chart recorders, multichannel oscilloscopes, film, magnetic tape recorders and integrated data processing systems providing digital output directly. The increasing sophistication of the equipment provides a vastly improved data handling capability, in return for much larger initial costs.

RESONANCE TESTS

The objectives of forced vibration testing are basically two-fold. The experimentally determined mode shapes, frequencies and dampings can verify the accuracy of the assumptions made at the design stage and lead to improvement in future designs through better understanding of the dynamic behavior of buildings. Secondly, if the dynamic properties are established experimentally both before and after a structure experiences strong ground shaking, some of the effects of this experience may be inferred from changes in these measured properties.

A Brief Review of Resonance Testing Abroad

Japanese experience in resonance testing has included shaking tests on reinforced concrete and reinforced-concrete, steel-composite framed buildings. From these tests, average values of modal damping were determined and expressions for the modal periods as a function of building height were proposed [16]. Efforts to clarify the soil-structure interaction effect, which is of particular importance for the many relatively stiff Japanese buildings founded on relatively soft ground, has included the forced vibration testing of a stiff six story building, which showed that rocking vibration can predominate [17, 18]. Interested readers are referred to the literature for details of the extensive Japanese testing program.

Investigations in New Zealand have included comparison of predicted dynamic properties with measured ones for a framed building with spandrel beams [19, 20], for four shear-wall buildings [21, 22] and for a simple framed structure [23].

Hand shaking excitation of a sixteen floor apartment building enabled the two first mode translation frequencies to be established. The measured values lay between those predicted in the design when plane frame axial deformation effects were firstly neglected and secondly included. Correlation better than 10% was obtained when the first mode periods were recalculated using an increased value of elastic modulus and including the flange action of the exterior frames transverse to the direction of movement when determining the structural stiffness [19]. The requirement to include flange action in buildings of the type tested, in which virtually all the lateral resistance is provided by the perimeter frames, was probably the most significant result derived from this test. The necessity for including the effects of shearing deformations in deep beam reinforced concrete frames - as distinct from shallow girder frames - had been recognized at the design stage [20].

Steady state resonance testing has been undertaken on one six story [21] and three adjacent eight story buildings [22], each being of the pierced shear-wall type. Five modes of vibration were examined in the lower building and a total of seventeen in the eight story complex (Figure 1). It was established that whereas the dynamic characteristics of the structural system may be predicted reliably, the effects of soil-structure interaction proved difficult to assess with any degree of certainty. Damping of up to 10% critical was measured and foundation compliance contributions of almost 40% of the maximum top story deflection were noted.

Forced vibration into the post-elastic range of a two story full-scale reinforced concrete frame has been accomplished using a variable speed exciter mounted on the frame [23]. The structure was designed and erected to the standards applicable some ten years ago, consequently it is anticipated that the observed performance of the test frame will be representative of that which could be expected of a typical building erected in the same period. As no recent New Zealand earthquake has resulted in even moderate shaking of reinforced concrete buildings, no comparisons with seismically loaded frames are available locally. However, the damage sustained by the tested frame corresponds well with that observed as a result of earthquake activity elsewhere. Specifically, structural damage in the form of slip across construction joints, column bending cracks and shear cracks in the beam-column joints and significant stiffness degradation was observed. The dynamic load at which definite inelastic behavior was recorded was found to be very much less than the equivalent static lateral load on which the elastic seismic design had been based. Additionally, the frame was found to exhibit marked torsional response despite symmetric disposition of the excitation force and the assessment that the intended symmetry of the structure was achieved satisfactorily in erection.

Resonance Testing in the United States

Resonance testing in the United States began with a series of tests in the 1930's by Jacobsen, Blume and Carder [24]. The primary purpose of these early tests was to determine the natural periods of buildings. Another test, using basically the same equipment, was performed by Alford and Housner in the early 1950's [25]. More extensive and detailed resonance testing, which included the accurate determination of damping and mode shapes, began in the early 1960's. This advance was made possible by the development of vibration generators capable of very accurate frequency and phase control, and with the feature of being able to operate in concert [1]. Although there has been continued activity in full-scale resonant testing of structures for the past fifteen years, most of the tests have been conducted on steel-framed buildings, and apart from nuclear reactor structures, which are excluded from the present discussion, the number of resonance tests of reinforced concrete or reinforced masonry buildings is limited. The structures tested include the four-story reinforced concrete building built in 1923 that was studied by Alford and Housner [25] and tests of the Pioneer building, an old (1910) reinforced concrete building in downtown Los Angeles, by Englekirk and Matthiesen [26]. Tests of modern structures include studies by Nielsen [27] of a five-story reinforced concrete building, two series of tests of Caltech's Millikan Library [13, 14, 28-30], and an investigation of an eleven-story reinforced masonry building by Stephen, et al. [31].

A common feature of all of the more recent tests has been the experimental determination of the natural frequencies, mode shapes and damping for the fundamental modes of the structure, i.e., longitudinal, transverse and torsion. In addition, similar properties of as many of the higher modes that could be excited have been measured. The limitation of the equipment usually employed to frequencies less than 10 hertz prevents the study of modes with frequencies above this number. The resonance response of the structures during typical tests far exceeds normal wind response, and the motions, if not limited, can easily be felt by the occupants. The acceleration levels vary from less than .005 g to as much as .02 g. These accelerations are associated with maximum displacements of the buildings on the order of 0.3 mm to 2 mm. It is seen then that these motions, although much stronger than achieved in ambient testing, are on the order of 20 to 100 times smaller than measured earthquake response, and still considerably lower than the response levels that would be associated with major structural damage.

Selected results from several of the vibration tests are shown in Figures 2 through 5; these figures illustrate some of the major features of the experiments. Perhaps the most significant result is the extent of soil-structure interaction that can occur in the fundamental modes of the buildings; interaction appears to be much more important for concrete or masonry structures, particularly those that are essentially shear wall buildings, than it is for steel buildings. The most detailed studies of soil-structure interaction are those done on Millikan Library [13, 14, 28-30]. For this building, Kuroiwa and Jennings [13, 28] reported that soil-structure interaction was negligible, accounting for about 3% of the total roof motion during resonance in the fundamental E-W and N-S modes. Later, after the San Fernando earthquake of February 9, 1971, foundation translation and rotation were found to contribute much more to the roof motion, 30% in the case of the N-S fundamental mode [14, 29, 30]. Because the accompanying changes in period are consistent with the changes in mode shapes, and because the amount of interaction now present is consistent with simple analytical models of the building-soil system [14], it is concluded that the change in the degree of interaction is probably due to fracturing of some of the brittle elements surrounding the structure, which include retaining walls, walkways and a small plaza. The amount of interaction now shown by Millikan Library is consistent with that noted for a concrete shear-wall structure by Reay and Shepherd [22], for a reinforced masonry shear wall structure by Stephen, et al. [31], and by Kawasumi and Kanai [18]. Significantly less interaction, contributing about 6% to the roof motion, was reported by Nielsen for a concrete-framed building [27].

Another result seen in the tests is the change of resonant period with amplitude. This is shown clearly in the result by Kuroiwa [13, 28] shown in Figure 2. This effect has also been noted by Nielsen [27] and by other investigators. The amount of change of period is typically of the order of 3 or 4% over the range of amplitude of testing, which is a factor of 5 to 10. This amount of change is itself not significant, but it does indicate the possibility of substantial decreases in period during the much larger response levels expected during earthquake response. The feature was confirmed by records obtained during the San Fernando earthquake.

The measured response of buildings is the only way to determine values of damping, as methods to calculate it are still in the earliest stages of development. For concrete structures, the forced vibration tests typically show values for fundamental modes of vibration ranging from 1 to 3% of critical, with a clear tendency toward increasing values of damping with increasing amplitudes of response. Exception to this general trend are higher modes, which usually show somewhat higher values. Also, one feature of the tests reported by Stephen, et al. [31] was the high values of damping, approaching 9%, that were observed for modes with relatively large amounts of interaction. It should be realized that these values of damping are only representative of the amplitude levels of the tests. It is generally not possible to extrapolate these values with confidence to the higher amplitudes of earthquake motion; such values of damping are best obtained from the analysis of measured earthquake response.

The presently used techniques of resonant testing are capable of defining in significant detail the deformation of a structure as it vibrates in its resonant modes. To date, the most detailed results have been presented by Foutch, et al. [14, 29], who measured three components of motion at each of 50 points, for each of six levels of Millikan Library. With these measurements he was able to measure the out-of-plane and in-plane deformations of the floor slabs, the nature of the interaction between shear-walls and the frame, and other details of structural response. The amount of in-plane motion of floor slabs was also measured by Stephen, et al. [31] who noted that for structures in which the in-plane stiffness of the floor is comparable to or less than the stiffness of the lateral load resisting system, the assumption of in-plane rigidity of floor slabs may be invalid.

MEASUREMENTS OF EARTHQUAKE RESPONSE

The instrumental records of the response of buildings to strong ground shaking come almost exclusively from the San Fernando earthquake [11, 32]. Over fifty multistory buildings provided sets of accelerograms from the earthquake, and many of these structures were of reinforced concrete construction. The records included some buildings shaken strongly enough to receive structural damage, although none of the few concrete buildings that collapsed or were irreparably damaged were instrumented. The measured levels of maximum acceleration varied from a few percent of gravity to over 40% g, with many buildings receiving shaking of the order of 20% g. The maximum deflection of the structures varied, of course, but interstory displacements of 10 to 20 mm were achieved by many instrumented structures. Selected samples of accelerograms and derived displacement records obtained in reinforced concrete structures during the San Fernando earthquake are shown in Figures 6, 7 and 8 [33, 34].

One of the most noted results from the building records obtained during the San Fernando earthquake was the clear evidence of lengthening of the natural periods of vibration. This increase in period took two forms. First, there was a general increase in periods shown in earthquake response over those found from ambient or forced vibration tests. This had been anticipated from the behavior of structures during resonance tests, but the magnitudes of the changes, some as great as 50% or more, were larger than most engineers anticipated. Secondly, in some records it is clear that there is a lengthening of the natural period during the earthquake response itself. This behavior can be seen upon careful examination of the responses of the Holiday Inn and Millikan Library, shown in Figures 6 and 8. It is thought that this decrease in stiffness during the earthquake is the result of a combination of cracking, non-structural damage and, in some cases, minor structural damage. This conclusion is supported by the tendency of the periods shown during earthquake response to be in agreement with those calculated considering only the structural resistance of the building. The rather large changes in period that a structure can show under the forces of wind, resonance testing and different levels of earthquake excitation point to the necessity of considering the natural periods of individual buildings as temporal quantities associated with substantial degrees of uncertainty.

As mentioned previously, the increase in damping values with increasing amplitudes of response observed in resonance tests indicates that the damping observed in the tests may be much lower than that shown in the earthquake response of structures. This fact was demonstrated clearly by the analyses made of individual building responses to the San Fernando earthquake [32, 34-36]. These analyses showed that the buildings typically exhibited effective viscous damping values ranging from 5 to 10 percent or more, with the higher values associated with the larger amplitudes of response. This trend in the effective damping values exhibited by reinforced concrete buildings is seen also in the result shown in Figure 9 which is taken from a recent study by Hart and Vasudevan [37]. This figure shows the relation between damping values and the level of the undamped response spectrum of the accelerogram recorded at the base of the building. The results show that the damping values developed in the San Fernando earthquake ranged from 2 to 12%, with a tendency for concrete structures to show larger values than steel buildings. As a final note, it is emphasized that the concept of equivalent viscous damping is a convenient way to describe what is, in fact, a very complex, poorly understood process by which buildings dissipate energy. The concept appears adequate for earthquake response up to levels associated with minor structural damage, but it is generally agreed that it is not adequate for larger levels of response. For larger response, in which significant damage is expected, a more direct consideration of the energy dissipation in the structure is recommended.

An examination of the analytical investigations of the earthquake response of eleven instrumented buildings, including six of reinforced concrete, was made by Gates [35] in the same volumes where these studies are presented [32]. In addition to some of the items noted above, Gates makes some important points applicable to reinforced concrete structures, which are paraphrased below.

1. Modern analytical techniques can reproduce adequately the recorded earthquake motions of the buildings, and these same techniques can be applied effectively in design.

2. All of the buildings, including those of reinforced concrete, experienced force levels greater than the design values of the code, in some cases the response was large enough to cause significant yielding and nonlinear response.

3. Building displacements ranged from two to four times the values computed for the design forces prescribed by the code. These large story drifts produced significant cracking of partitions in all the buildings.

The response of reinforced concrete buildings during the San Fernando earthquake and the comparison of earthquake response to that under code forces makes it clear that the relation between the seismic design criteria for a building and the actual capacity of the structure needs to be investigated. The results of some research in progress on this question at the California Institute of Technology are shown in Figure 10. The figure is limited to reinforced concrete buildings and shows three quantities for each lateral direction of vibration of several structures. The abscissa of the figure is the period of vibration observed during the earthquake. The ordinates plotted are the base shear used in design, expressed as a percentage of gravity, the base shear experienced during the earthquake as determined from the displacement of the building in the fundamental mode, and the maximum acceleration at the top of the building, given by the strong-motion accelerometer. Buildings which showed structural damage are indicated by ticks. The data in Figure 10 are too few to define trends precisely, but there is clear evidence that, on average, reinforced concrete structures are capable of withstanding base shears three or more times the values used in design before structural damage becomes significant. It is expected that the structures could have withstood substantially larger responses without being in danger of collapse. As more data accumulates on the response of buildings to strong earthquake motion, plots such as these should help quantify the degree of conservation that is implicit in modern building codes and design procedures. This knowledge is essential to a realistic determination of seismic hazard.

A major earthquake strongly shaking a large number of instrumented buildings has not yet occurred in Japan, but records obtained on instrumented structures in Japan during moderate ground shaking indicate that the measured acceleration at the base of one reinforced concrete building was less than that recorded on the ground surface nearby, whereas that at the top of the building was two to four times larger than that recorded at the base [17]. A tendency for longer modal periods and larger damping to be exhibited by buildings shaken by ground motion rather than dynamic exciter is also reported [16], as is the fact that in earthquakes the maximum acceleration does not necessarily occur at the top floor. The reliability of response predictions based on carefully selected mathematical models was confirmed by studies of measurements made in two different earthquakes [38]. A similarly satisfactory result has also been reported from Yugoslavia [39].

DISCUSSION

This brief summary of experimental results for reinforced concrete structures reveals a number of areas where research on full-scale structures is needed to clarify the ability of such buildings to withstand strong earthquake motions. Foremost among these is the fact that the response of a building subjected to extremely strong shaking has not yet been measured. Thus, we are lacking the full-scale verification that modern methods of analysis and design are leading to structures which perform as intended under severe loading. Although many buildings are now instrumented to give useful data if subjected to strong shaking, the well-known problems of instrumenting for earthquake response have so far precluded the detailed instrumentation of any single building. It may be possible, however, in the near future to do this, and recover from the earthquake response information concerning the mechanics of plastic hinge formation, the behavior of seismic joints, the progress of deterioration, the load redistribution during the progression of damage, and other needed data.

Another related topic concerns the amplitude dependent characteristics of reinforced concrete structures in the range of response up to and including minor structural damage. This response can be described for many purposes by linear mathematical models, in particular such models are useful in analyses for design. It is important in using such models, however, to employ realistic values of natural periods and damping. The present data is sufficient to establish general trends of these quantities with amplitude, but more data is required to determine the most appropriate values to use for a given structure.

Reinforced concrete shear-wall buildings, reinforced masonry buildings, and other structures such as in-filled frames which respond to ground motion as shear-wall structures show a much larger degree of soil-structure interaction than has been observed in steel-frame or concrete-frame buildings. The data are too few to draw definite conclusions, but it does appear fairly clear that when such structures are founded on alluvium, the amount of soil-structure interaction can be large enough to influence significantly the earthquake response. Interaction of this amount should be considered in design, as the periods, deflections, and drifts can be affected substantially. The amount of soil-structure interaction and the details of the rotational and translational compliances of the foundation are recommended as central parts of future tests of reinforced concrete buildings.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to the National Science Foundation for financial support under Grant No. ENV74-19135 A01. The effort of Albert Ting, who performed some of the calculations reported above, is also acknowledged. Any opinions, findings, and conclusions or recommendations expressed in this paper are those of the authors and do not necessarily reflect the views of the National Science Foundation.

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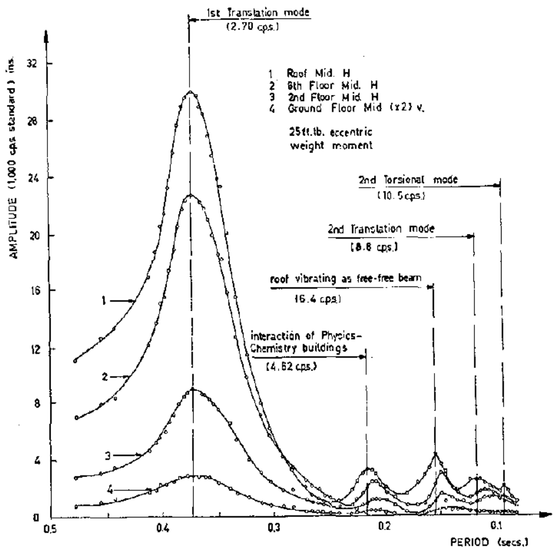


Figure 1. Building Response vs. Period of Excitation.
Peaks Indicate Resonances as Noted [21, 22].

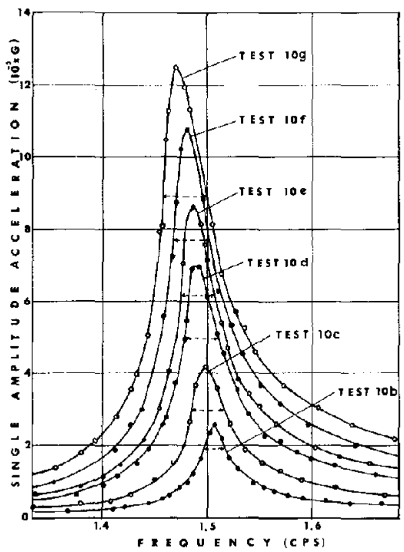


Figure 2. Steady-state Response of Millikan Library Measured at Different Levels of Forced Vibration [13, 28].

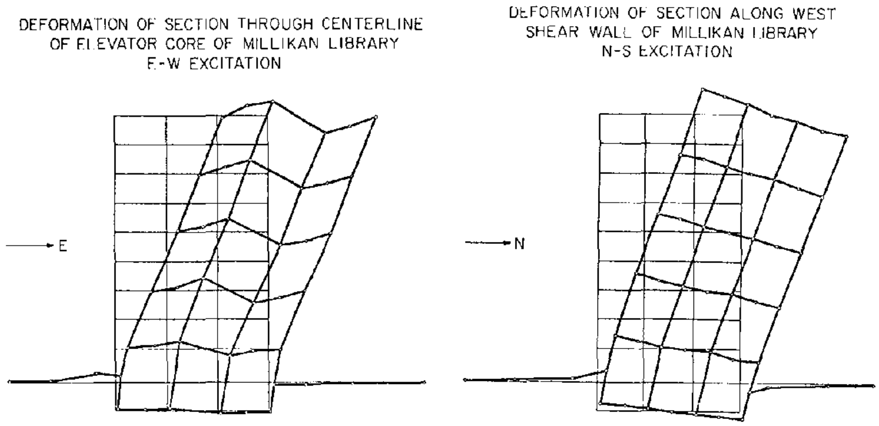


Figure 3. Fundamental Translational Modes of Millikan Library. Including Vertical Motion and Soil-Structure Interaction [14, 29].

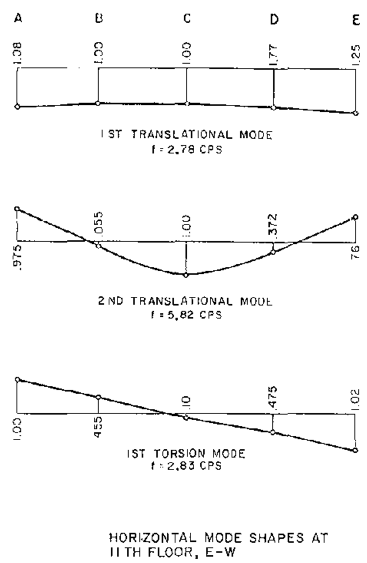


Figure 4. Top View of Mode Shapes of an 11-Story Reinforced Masonry Building. See also Figure 5 [31].

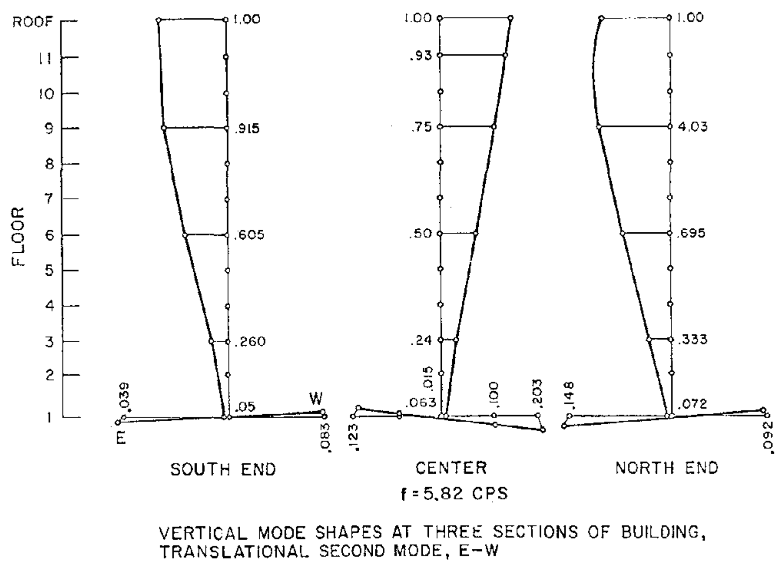


Figure 5. Vertical Components of the Second Translational Mode of an 11-Story Reinforced Masonry Building [31].

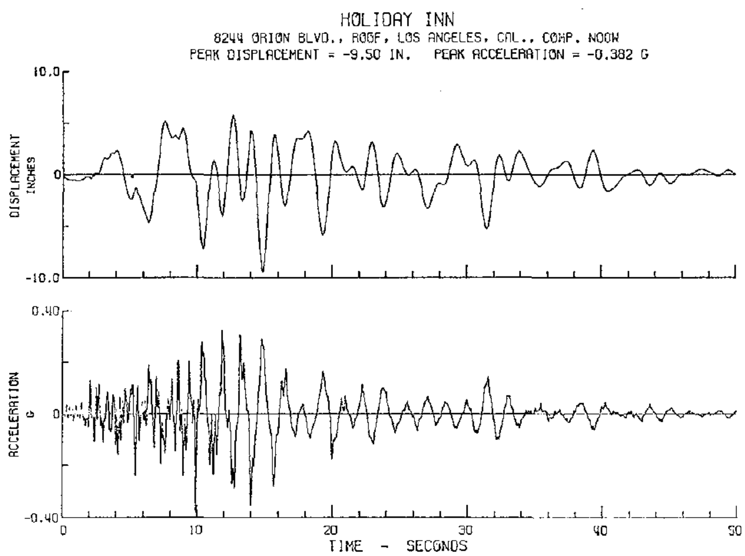


Figure 6. Measured Acceleration and Calculated Displacement for the Roof of the Orion Ave. Holiday Inn. This was the Instrumented Building Closest to the Epicentral Area in the San Fernando Earthquake of February 7, 1971 [11,32,33].

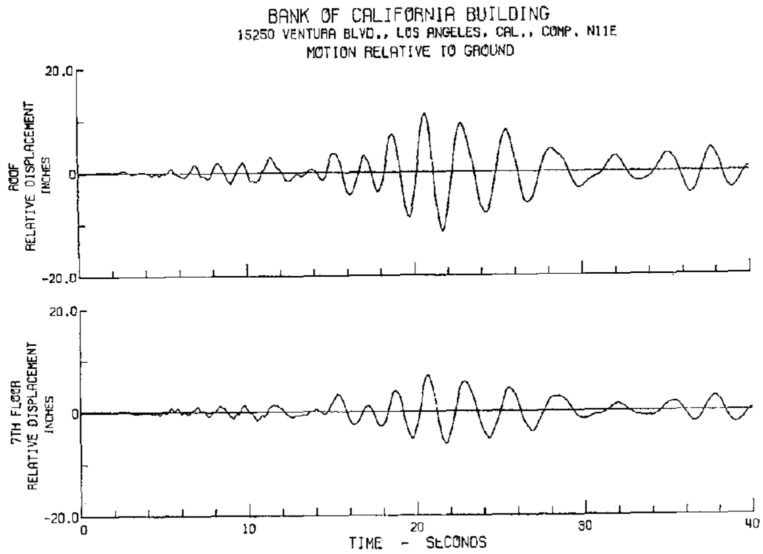


Figure 7. Calculated Values of Displacement Relative to the Ground for the Bank of California during the San Fernando Earthquake [33].

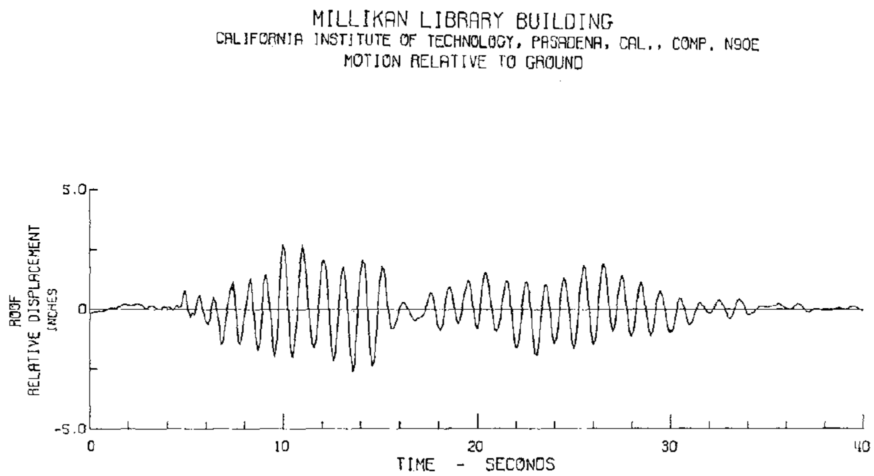
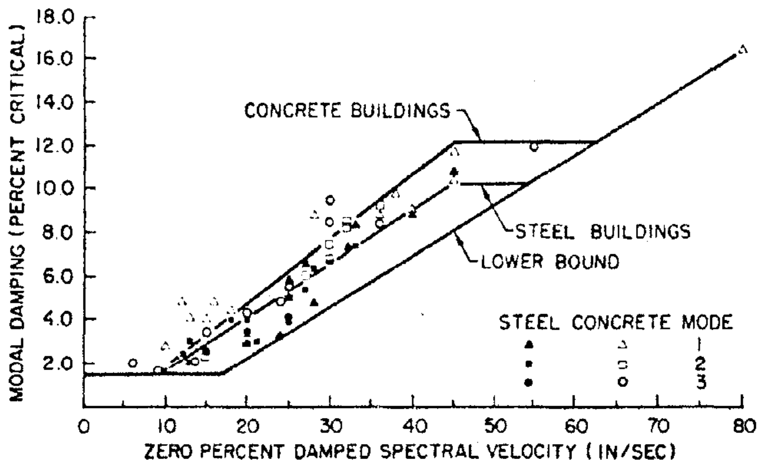


Figure 8. Calculated Relative Displacement of the Ninth Floor of Millikan Library during the San Fernando Earthquake [33, 34].



Damping Versus Magnitude of Spectral Velocity

Figure 9. Relation of Damping Values of Buildings during the San Fernando Earthquake to the Amplitude of Ground Motion [37].

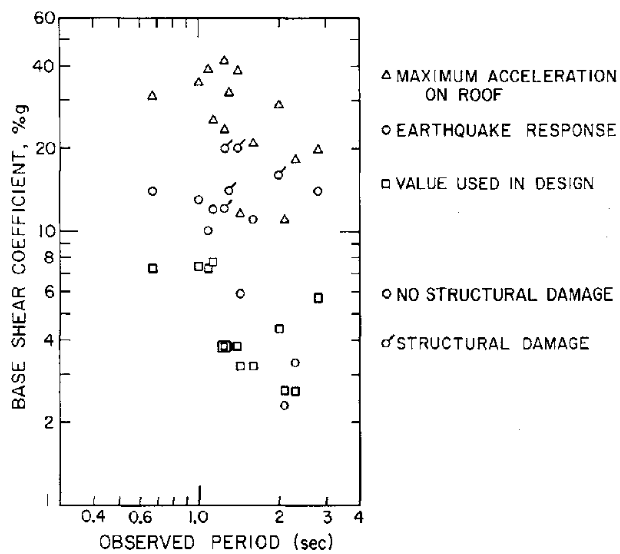


Figure 10. Capacity of Reinforced Concrete Structures Demonstrated during the San Fernando Earthquake.

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

LARGE-SCALE DYNAMIC SHAKING OF 11-STORY REINFORCED CONCRETE BUILDING

Ronald L. Mayes[1] and Theodore V. Galambos[2]

(1) Principal, Computech, Berkeley; Technical Director, Applied Technology Council, Palo Alto, California

(2) Professor and Chairman of Civil Engineering Department, Washington University, St. Louis, Missouri

1.0 INTRODUCTION

During 1976 the demolition of 30 or more eleven-story apartment buildings, Fig. 1, of contemporary reinforced concrete construction in St. Louis, Missouri, provided a unique opportunity to subject a portion of one of these structures to large amplitude dynamic excitation and to observe the effects of this motion on its behavior.

The test-building was an approximately 40 ft. x 40 ft. square eleven-story tower structure, Fig. 2, which was separated from a much larger building by severing the connecting beams and slabs with jackhammer and torch, Fig. 3. The larger portion of the building was then demolished by explosive charge. The test-building had 13 columns (9 on the periphery and 4 near the center), beams, slabs, and a stairway, Fig. 5. The periphery and the stairwell were enclosed by block and/or brick walls which were intermittently interrupted by windows or doors. The construction was of good quality conventional reinforced concrete, built around 1958 and designed in 1953 in accordance with the then effective ACI Code. No special provisions were made, according to contemporary requirements, to resist seismic loads.

The on-site research work was performed in the period of June through mid-November of 1976. No access was possible prior to June, and the remnants of the structure were demolished and removed within days of the completion of the last experiment. The work consisted on the following three phases:

(1) Survey of material and dimensional properties of the building through sonic and magnetic measuring devices and by taking samples of the concrete, the brick, and the steel.
(2) Small amplitude dynamic excitation to determine the dynamic characteristics of the structure as it existed before the large amplitude tests.
(3) Large amplitude dynamic tests to study the change of dynamic characteristics as the building was progressively damaged.

The large amplitude motion was induced by the sinusoidal horizontal motion of a 60 kip mass of lead which was placed on hardened steel balls. The mass was moved through an amplitude of up to $\pm$ 20 in. by a servo-controlled hydraulic actuator. One end of the actuator reacted on the building frame, and it was driven by a large motor-pump assembly. The lead mass was placed in the center of the eleventh floor, where also the motor-pump assembly was located, Fig. 4. Control of the equipment was by remote control from a trailer. This trailer also contained the apparatus for recording the data from the accelerometers placed in fixed or movable locations within the building. The shaker was supplied by Boeing Construction Co. of Seattle and the data acquisition system by McDonnel Douglas Aircraft Co. of St. Louis.

2.0 SMALL AMPLITUDE TESTS

The small amplitude tests were performed by Applied Nucleonics Corporation of Los Angeles. The objective of these tests was as follows:

a) To determine the dynamic characteristics of the structure by methods that have been extensively used over the past decade.
b) To compare the results obtained in a) with those obtained from the large amplitude forced vibration tests.
c) To use the information obtained in a) to plan the large amplitude forced vibration tests.

The tests were performed with an eccentric mass vibrator mounted in the southwest corner of the eleventh floor. The nine lowest natural frequencies were determined and the corresponding response shapes of the first six of these frequencies were mapped. A summary of all the tests performed and their respective results is given in Table 1.

3.0 LARGE AMPLITUDE TESTS

The large amplitude tests were performed with the moving mass vibrator described in the introduction and mounted on the eleventh floor. The objective of the tests were as follows:

a) To determine changes in mode shapes, frequencies, and damping values as the force level of excitation increased.
b) To determine the resistance capability of the non-seismically designed building as the force level of excitation increased.
c) To determine the effect of soil-structure interaction on the large amplitude vibration of the building.

Large amplitude testing was performed in the E-W direction with the external cladding (infill walls) in place and in the N-S direction with the external cladding removed. Two different types of tests were performed. Damping tests were performed to determine resonant frequencies and damping values of modes of interest at various input force levels. Mode shape tests were performed to determine the mode shapes at various resonant frequencies.

3.1 DAMPING TESTS

The damping tests were performed such that the force level during a given test varied approximately as a function of the frequency squared. The theoretical force level generated by the moving mass vibrator is given by:

$$F(t) = M \omega^2 X \sin \omega t$$

where M is the mass of the bucket weight, ω is the frequency of excitation, and X is the single amplitude of motion of the moving mass. The purpose of using this forcing function was to enable the data, whenever possible, to be analyzed by techniques normally used for eccentric mass forced vibration tests. The required input force level, F_r, from the moving mass vibrator was determined from the relationship

$$F_r = M \omega_r^2 X$$

where ω_r is the resonant frequency of interest.

The approximate resonant frequency at a required force level was determined by a slow continuous sweep beginning at approximately 1.0 cps. above the estimated resonant frequency and sweeping down below the actual resonant frequency.

During this sweep, two recordings were made. First, the signal of the reference accelerometer on the eleventh floor was analyzed by a spectrum analyzer. The peak of the resulting curve was used to identify the resonant frequency. Second, the force signal from the moving mass vibrator was plotted against the signal from the reference accelerometer on a two channel oscilloscope. Theoretically for an elastic system at resonance the two signals are 90° out of phase and the resulting plot traces a circle on the oscilloscope. Both methods were used to identify the resonant frequency during the sweep.

In addition to identifying the resonant frequency, the continuous sweep enabled the structure in most cases to achieve a stable structural condition at a particular force level. This was helpful, because as the input force level increased in increments of 5,000 lbs. the structural system changed. An example of this was hinging that occurred in the beams and stairwell of the lower levels. At higher force levels in the first modes, the continuous sweep was not performed because the structural changes were more significant and consequently, data was required as these changes occurred.

Once the resonant frequency was identified, a step-wise sweep was performed at appropriate frequency intervals to determine the damping and resonant frequency at the particular input force level. At each frequency step the structure was vibrated until steady state was achieved and the data was recorded. In all except two tests, the step-wise sweep was performed by sweeping from a frequency above the resonant frequency and sweeping down below the resonant frequency.

At various stages throughout the large amplitude test program a standard damping test was performed. The standard damping test consisted of the damping test described above at a nominal input force level of 5,000 lbs. The objective of the standard damping test was twofold. First, it provided the means of comparing changes that occurred in the damping and resonant frequencies of the building during various stages of the test sequence. Second, it provided the means of determining whether or not changes that occurred in the damping and resonant frequencies at larger input force levels remained the same at lower force levels.

3.2 MODE SHAPE TESTS

Mode shape tests were performed at various phases of the test program to determine in detail the response of the building at resonance. Prior to each mode shape test a continuous frequency sweep, described in subsection 3.1 was performed to determine the resonant frequency. The building was then vibrated at the resonant frequency and data was recorded. The response of the structure was measured at the 11th, 9th, 7th, 5th, 3rd, 1st, and basement levels. At each level the response at 25 grid points was recorded. At each point a triaxial accelerometer was used enabling the three-dimensional response to be obtained.

3.3 SEQUENCE OF TESTS

The sequence of tests performed in the E-W direction with the cladding in place is listed in Table 2. Also given in the Table are the resonant frequencies and damping values obtained for each of the tests to date. After the first series of standard damping tests (Test Nos. 1E-SD to 3E-SD) the mass of the bucket of the moving mass vibrator, was increased from 5,800 lbs. to 19,000 lbs. to improve the performance of the vibrator. Following this increase in bucket weight, a first mode frequency sweep was performed at a force level of 10,000 lbs. No damping data was recorded during this sweep although it was observed from the oscilloscope that the resonant frequency dropped from 1.41 cps. to approximately 1.15 cps. Following this 10,000 lb. sweep test, two standard damping tests were repeated (Test Nos. 5E and 6E-SD). After these two tests, the bucket weight was increased to its moving mass vibrator. From this point on the bucket weight remained at 57,700 lbs.

The sequence of tests performed in the N-S direction with the cladding removed is listed in Table 3. Also listed in the table are the resonant frequencies and damping values available to date.

4.0 TEST RESULTS

The test results available to date are presented in Tables 4 to 10 and Figures 6 to 10. These test results consist of the changes in the period and damping, the changes in the mode shapes and the forces induced in the building during one of the tests with the cladding in place.

4.1 PERIOD AND DAMPING

A summary of the changes in the damping and the period for the first and second translational modes of the building with the external cladding in place (E-W direction) are presented in Tables 4 and 5 respectively. A summary of the changes in the damping and the period for the first and second translational and torsional modes of the building with the external cladding removed (N-S direction) are presented in Tables 6 to 9 respectively. It should be noted that the direction of the sweep was down for all except three of the tests. Down refers to a sweep from a frequency above (or a period below) resonance down to a frequency below (or a period above) resonance. In tests 1E-SD, 19N-D, and 20N-D the direction of the sweep was in the reverse direction.

The damping and resonant frequency results presented in Tables 4 to 8 were obtained from the frequency response curves that were obtained from the step-wise sweeps. The "half-power" point method was used to calculate the damping from the frequency response curves. This consists of determining the width $\Delta\omega$ of the frequency response curves at points where the response is equal to 0.707 of the peak values. The damping β is obtained from the relationship

$$\beta = \frac{\Delta\omega}{2\omega_r}$$

where ω_r is the resonant frequency.

Damping results not included in the tables were not able to be obtained by this method. These are currently being obtained by a least squares curve fitting method. It should be noted that results obtained from the least squares curve fitting method are not in good agreement with those obtained from the "half-power" point method. This discrepancy is being investigated and will be discussed in a subsequent report.

4.2 MODE SHAPES

The mode shapes corresponding to the first and second translational resonant frequencies of the building with the cladding in place are plotted in Figures 6 and 7. The two plots included in each figure are the results of the mode shapes before and after the large amplitude tests. These are tests 11E-M and 27E-M for the second translational mode and 14E-M and 28E-M for the first translational mode. The motion of the floor slab in each of these modes was pure translation. No torsional component was present.

The translational mode shapes of the building with the cladding removed contained torsional components. The displacement of the floor slab at the ninth level corresponding to the first translational and torsional resonant frequencies are shown in Figures 8 and 9 respectively. A plot of the N-S translational component of the external west side beams of the first translational mode before and after the large amplitude tests (Test Nos. 5N-M and 32N-M) is given in Fig. 10.

4.3 CODE EQUIVALENT OF THE BASE SHEAR FORCE INDUCED DURING TESTING

In order to provide a frame of reference for the magnitude of forces induced during the large amplitude shaking, the base shear force generated in the tests was compared with design base shear forces of the Uniform Building Code (UBC). The base shear force, V_T, generated in the structure during testing was calculated as:

$$V_T = \sum_{i=1}^{11} m_i \ddot{x}_i$$

where m_i is the mass at the i^{th} level and $\ddot{x}_i$ is the maximum acceleration of the i^{th} level. The UBC design base shear is calculated from

$$V_c = ZKCW \qquad 1974 \text{ UBC}$$

or

$$V_c = ZKICSW \qquad 1976 \text{ UBC}$$

where W is the total weight of the structure, Z is a zone factor and varies from 1 to 1/4 for the 1974 UBC and 1 to 3/8 for the 1976 UBC. K is a factor dependent upon the framing system and for this example was taken as 1.0. I is an importance factor and is taken as 1.0. S is a factor dependent on the soil conditions and was taken at its maximum value of 1.5. C is a function of the period of the building and differs for the 1974 and 1976 codes. In the 1974 UBC,

$$C = \frac{0.05}{\sqrt[3]{T}}$$

and in the 1976 UBC,

$$C = \frac{1}{15\sqrt{T}}$$

where T = 0.1N in both and N is the number of stories.

A comparison of V_T and V_C is presented in Table 10 for Test No. 23E-D where the input force level at resonance was 18,800 lbs. Note that this test is the second highest force level test performed on the structure with the external cladding in place.

5.0 DISCUSSION OF TEST RESULTS

5.1 PERIOD

There were large changes in the period of the building in most of the modes. The largest changes occurred in the first translational modes.

With the external cladding in place the period of the first translational mode increased by a factor of 2.6 from 0.74 secs to 1.92 secs as the input force increased from 4,760 lbf. to 25,000 lbf., respectively. The period of the second translational mode increased approximately 50 percent from 0.22 secs to 0.31 secs, as the input force level increased from 4,310 lbf. to 29,920 lbf. Following the large amplitude first mode tests the period of the second mode had increased to 0.39 secs.

With the external cladding removed the period of the first translational mode doubled from 1.23 secs to 2.44 secs as the input force level increased from 3,610 lbf. to 15,000 lbf. The test performed at an approximate input force level of 20,000 lbf. was not completed because of the apparent impending collapse of the building. The period of the first torsional mode increased almost 50 percent from 1.04 secs to 1.47 secs as the input force level increased from 5,170 lbf. to 15,000 lbf.

With the external cladding removed the period of the second translational mode increased by 40 percent from 0.31 secs to 0.42 secs as the input force level increased from 3,530 lbf. to 22,840 lbf. The period of the second torsional mode increased 15 percent from 0.28 secs to 0.32 secs as the input force level increased from 4,500 lbf. to 26,940 lbf. The period of the second translational and torsional modes after the large amplitude first mode tests was 0.53 secs and 0.36 secs, respectively.

It should be noted that the changes in the period associated with the large amplitude tests were permanent in that all lower input force level tests performed after a series of large amplitude tests had periods very close to the last large input force level test. This indicates that permanent changes occurred in the lateral force resisting system. These changes which consisted on beam hinging, joint shear cracking, and hinging of the stairwell are discussed in detail in section 6.

5.2 MODE SHAPES

The changes in mode shapes associated with the large changes in period described in the preceeding subsection were generally small as shown in Figures 6, 7, and 10. The most significant changes were in the first translational mode without external cladding. The values plotted in Fig. 10 indicate significant differences at the first and eleventh floor levels.

5.3 DAMPING

The damping results presented in Tables 4 to 9 contain many interesting results even though the large amplitude first mode tests are not yet available. The discrepancy referred to in Section 4.1 indicates that the results presented in Tables 4 to 9 may be higher than those that will be obtained from the least squares curve fitting method. Caution is therefore suggested in using the absolute values of damping presented in Tables 4 to 9.

1) For the second translational modes, both with and without external cladding, there was a significant increase in damping as the input force level increased. With the external cladding in place the damping increased from 2.6 percent to 8.2 percent as the input force level increased from 4,310 lbf. to 29,920 lbf. (Table 5). With the external cladding removed the damping increased from 2.8 percent to 5.6 percent as the input force level increased from 3,670 lbf. to 22,840 lbf. (Table 8).

2) The large amplitude tests caused an increase in the damping associated with the second translational modes measured as low input force levels. This increase was by a factor of 2.1 with the external cladding in place and 1.4 with the external cladding removed. With the external cladding in place, Table 5, the damping of the second translational mode prior to the large amplitude second mode tests was 2.6 percent at an input force level of 4,500 lbf. Following the large amplitude second mode tests the corresponding value at an input force level of 4,860 lbf. was 6.3 percent. With the external cladding removed, Table 8, the damping prior to and after the large amplitude second mode tests was 2.8 percent and 3.4 percent at input force levels of 3,670 lbf. and 3,290 lbf., respectively. Note that after the large amplitude first mode tests the damping values of the second translational mode decreased from 6.3 percent to 5.6 percent with the external cladding in place and increased from 3.6 percent to 3.9 percent with the external cladding removed.

3) With the external cladding removed there was little or no increase in the damping associated with first and second torsional modes of the building as the input force level increased, Tables 7 and 9.

4) In the two tests performed, the damping was a function of the direction of the sweep. For the second translational mode without external cladding (Test Nos. 18N-D and 19N-D) the damping obtained from a frequency sweep down was 3.7 percent, while for a sweep up the damping was 2.0 percent. For the second torsional mode without external cladding (Test Nos. 17N-D and 20N-D) the corresponding damping values were 2.8 percent and 1.6 percent respectively. It should be noted that the direction of sweep was down for all except three of the tests i.e., a sweep down is from a higher to lower frequency. Consequently the damping values given in Tables 4 to 9 are all higher than those that would have been obtained if the direction of the sweep was in the reverse direction.

5.4 CODE EQUIVALENT OF THE BASE SHEAR FORCE INDUCED DURING TESTING

The base shear force comparison V_T/V_C, presented in Table 10 is made for both the code calculated period and for the period of the structure measured during the test. For zone 3 of the 1974 UBC the measured base shear force of 250 kips with the external cladding in place (Test No. 23E-D) was 2.4 times the code calculated design base shear force. For the 1976 UBC the corresponding factor was 1.2 for zone 4. The damage to the building following this test is described in section 6. Although the exact figures are not available for a comparison with the external cladding removed, the factors are estimated to be two to three times greater than those presented in Table 10.

It is clear that the non-seismically designed building both with and without external cladding was able to withstand a base shear force greater than that required by the Uniform Building Code when subjected to the sinusoidal type of motion induced by the moving mass shaker. Until further analysis of the results is performed it cannot be inferred from these results that the building would have resisted an earthquake that induced a base shear force of the same magnitude.

6.0 DISCUSSION OF DAMAGE TO THE BUILDING

6.1 STATUS BEFORE THE START OF THE LARGE AMPLITUDE SHAKING TESTS

At the beginning of October, prior to the commencement of the large amplitude shaking the building was essentially undamaged from the structural point of view. There were a few hairline cracks in the beams and the columns of the top floor (at level 11) which were induced by the small amplitude shaking performed in July by the Applied Nucleonics Company (Fig. 11), and there were some diagonal cracks in the E-W filler walls around the stairwell (Fig. 12) on the 4th, 5th, and 11th floor. The outside E-wall (Fig. 13) was already damaged by the blast when the center portion of the building was removed in 1972, and the small amplitude shaking loosened some of the blocks on the top layers further. The stairs were completely whole, and the outside brick facade was essentially intact except for a small part of the S-W corner (Fig. 14) which was inadvertently hit by a headache ball when the adjacent building was demolished. The only modification to the structure consisted in removing a portion of the roof slab at the end of August to facilitate placing the equipment (Fig. 15). It should be noted that the bottom beam steel at the exterior columns was embedded from 9 to 12 inches into the column with no hook. The top beam steel had a 90° bend, 12 inches long in the joint.

6.2 DAMAGE AFTER MODERATE E-W SHAKING (UP TO TEST NO. 12E-D)

After the fully clad structure was subjected to a series of test-runs (up to Test No. 12E-D) with 5 and 10 kip force levels the damage to the slab and structural frame was slight, consisting mainly of hairline cracks at the column tops (notably in Col. 33, 1st story) and at the ends of some E-W beams (notably in Beams B4 at levels 1 and 2). Some blocks fell off the E block wall in the 1st story, and cracks developed all across the joints between the stairs and the stairway landing at the 1st story and the ½ level landings. Those cracks became quite large later and subsequent photos will show them in a more developed stage. The most interesting feature of these moderate level shake tests was the behavior of the E-W block infill wall around the stairway. These wall panels moved with the frame above level 3, but in stories 2 and 3 the panels remained essentially stationary while the frame moved back and forth, leaving a gap of up to 1/8 inch between the wall and column at maximum amplitude, and knocking against the wall on the opposite side (Fig. 16). There was noise due to friction as the beams rubbed against the top face of these walls.

6.3 DAMAGE AFTER THE COMPLETION OF THE E-W TESTS

The major amount of testing in the E-W direction on the fully clad structure was performed during the period October 9 through October 13. Some 10K force-level tests were performed on October 15, and finally the most severe E-W shaking, Test No. 24E-D, took place in the evening of October 15 just before this phase of testing was discontinued.

All of the damage to the structure above level 8 was restricted to hairline cracks at some column tops and beam ends, and no new cracks were discovered between levels 5 and 8. It can be noted that no substantial structural damage was discernible above level 5. Major structural damage occurred at level 1, with damage dimishing with height. All of the E-W beams on levels 1 and 2 had cracks at their ends, and most of the beams on levels 3 and 4 had hairline cracks at their ends. Typically the most severe cracking and spalling took place on level 1, and the following figures illustrate this:

Typical interior joint X-cracks are seen in Fig. 17, which show the joint of Col. 36 at the top of story 1. The top of Col. 11 (S-W corner) in story 2 is shown in Fig. 18. This same column, at one level below, exhibits a crack through the beam and into part of the column (Fig. 19). A typical 1st level beam end (Beam 56) shown in Fig. 20 illustrates the crack at the column face. This crack opened and closed during the cyclic motion of the building. The most severe column damage occurred at the top of column 37 in the 2nd story (Fig. 21). The most severe beam cracking occurred at the E end of beams B4 in level 1, and Figs. 22 through 26 illustrate the progression of damage, including the fracture of the reinforcing steel. During the final E-W test it appeared from the observed motion of the structure that the E-W beams connected to the exterior columns on levels 1 and 2 acted essentially as hinges when the bottom beam steel was in tension. The slab on levels 1 and 2 cracked through from N to S across the building.

The stairway up to the fourth level was severely cracked at each joint between the stairs and the landing. This joint heaved up and down during each cycle of loading. The lowest joint (between levels 0 and 1) is shown during the early tests (after Test No. 17E-D) and after the E-W tests in Figs. 25 and 26 respectively. Top and bottom stairway joints are shown in Figs. 27 and 28.

A considerable portion of the E block wall fell out during the tests (Fig. 29; compare with Fig. 13 to see the extent of wall damage), and some of the outside brick walls fell off also (Fig. 30; compare with Fig. 1 to see the damage on the N brick wall). A portion of the lower part of the S wall, shown in Fig. 31, demonstrates the horizontal fracture lines at the top of the window. The adjacent parts of the wall rubbed against each other during load cycling.

On levels 1 and 2 at the end of the E-W tests the E-W beam ends were cracked through, some columns were cracked and one (Fig. 21) was moderately damaged, the slab was cracked across the building, the stairway was behaving as a mechanism with hinges at the stair-landing joints, the E block wall had fallen out, the E-W brick faces were severely damaged or had fallen out, and the E-W block infill walls next to the stairway lost their capacity to act as infill walls. The N-S beams and the N wall were essentially undamaged. Damage to columns, beams, stairs and walls diminished progressively from the third to the fifth story, with the structure-stairwall system intact and acting as a unit above the fifth story. Below that level the walls and the stairway system were broken up and the beams were hinging. As testing continued damage seemed to be confined to the lower three to four floors, the top riding along as the softening and damaged lower floors swayed back and forth. Little damage was observed on the eleventh floor where the heavy moving machinery subjected the frame and the slab to continued severe impacts. It appears that once softening started on level 1 the damage became isolated on the lower part of the building.

6.4 DAMAGE DURING THE N-S TESTS

During the period between October 15 and October 27 the cladding was removed from all but the upper two floors of the building and the shaking apparatus on the 10th floor was rotated 90 degrees to produce forces in the N-S direction, Fig. 32. The removal of all of the brick and block cladding did not result in any additional damage to the structure.

The N-S testing commenced on October 27 and continued through November 4 when the experiment was terminated. During October 27, 28, 29, and 30 tests up to Test No. 16N-D were performed at mainly low force levels, 5 to 10 kips, and only one 15 kip level test of the second mode was performed. The new damage due to this shaking was slight, consisting of the development of flexural cracks between the column faces and the N-S beam ends. The motion of the floor of the building during the first mode N-S tests is shown in Figs 8 and 9. The rotational motion of the first translational mode, Fig. 8 tended to wrack the W face considerably more than the E face, and damage was mainly confined to the beams and columns on this face.

The shaking tests on November 1 through November 4 consisted of the larger input force level tests, Test Nos. 17N-D to 41N-D, and severe damage was inflicted on the W portion of the structure. This damage occurred in essentially two ways: (1) with continued shaking more and more joints in the N-W (Col. 9) and S-W (Col. 11) failed, and (2) columns 37 and 38 crushed in compression. There was also damage in the joints of the center columns on the W face (Col. 10). A typical damaged joint of this center column tier (Col. 10, level 4) is shown in Figs. 33 and 34, where the damage prior to Test No. 23N-D, Fig. 33, consists of spalling and after Test No. 30N-D part of the lower beam reinforcing bar is exposed. The extent of damage to this joint did not increase with later tests. The other joints of this column experienced similar damage, with all joints losing some concrete from level 1 through level 8. The corner columns (Cols. 9 and 11) were damaged rather more severely, all joints from level 1 through level 8 losing almost all the concrete from the joints, leaving the beam flexural reinforcing fully exposed.

The photo in Fig. 35 shows a portion of the N-W corner, illustrating the severe damage at the end of the tests at the joint. All columns appeared to be a series of hinged elements between stories, with the reinforcing holding them in place. Progression of damage prior to Test No. 21N-D through Test No. 41N-D for one typical joint (Col. 9, level 4) is shown in the sequence of pictures given in Figs. 36 through 44. All but one of these figures show the outside of this joint, and Fig. 43 shows the inside of the corner, from below.

The deterioration of the interior column (Col. 38, 4th story) is seen in Figs. 45, before Test No. 28N-D, and 46, after all the tests, and a close-up of the crushing failure is seen in Fig. 47.

During the last test, Test No. 41N-D, the following damage was evident: All the joints below the 9th level in the two W corner columns (C 9 and C 11) had lost almost all of the concrete from the joints (see Fig. 48), and the column in the N-W corner of the 6th story was visibily pushed out (Fig. 49). The joints in the center columns of the W face (Col. 10) were also damaged up to level 9, but not as severely. Interior columns 37 and 38 were severely crushed on the 2nd, 3rd, and 4th floor, with Col. 38 completely crushed in the 4th floor. The other columns showed little additional damage, except that during the last run X cracks began to develop in the joints of the 5th level in the two E central columns (Cols. 33 and 34).

During the last test there was very large deformation of the top of the W face ($\pm$ 28 inch), the W face appeared to be just flopping back and forth, there was a lot of noise (groaning, cracking) and damage progressed apparently toward the columns which appeared to be holding up the structure. For the sake of safety and equipment recovery it was decided to stop the testing. When all motion stopped the structure was to all appearances straight. No additional structural or cladding damage occurred in the enclosed top two stories.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided by the National Science Foundation under Grant No. ENV. 76-08244. They also extend their most sincere thanks to all who participated in this most interesting project, and to all who helped on other ways to make the program a success. The following individuals are especially singled out for their significant contributions: D. S. Hatcher, J. M. McKelvey, M. P. Mustaine, L. R. Brown, A. J. Bletch, D. Armstrong, R. Gardiner, E. M. Gregory, J. R. Vosper, A. Monsey of Washington University; T. Costello of the St. Louis Housing Authority; P. Ibanez, R. S. Keowen, W. E. Gundy of Applied Nucleonics; E. L. Smith, L. D. Russell, R. T. Jensen of McDonnell-Douglas; J. Hess, D. Burlingame, W. I. Lathrop, J. D. King, D. E. Marshall, G. Ehlenfeldt of Boeing; M. H. Rose, J. B. Crane, T. B. Laws, M. Mitchell of the Cleveland Wrecking Company; and M. M. Hochman of the AALCO Wrecking Company.

TABLE 1

SUMMARY OF RESULTS OF SMALL AMPLITUDE TESTS

Peak Acc. cm/Sec^2	Resonant Frequency Hz	Dominant Response	Damping Ratio (%)
62.22	1.43	E-W First Mode	0.88
37.59	1.44	E-W First Mode	1.52
40.92	1.47	E-W First Mode	1.34
9.42	1.53	E-W First Mode	1.45
70.37	1.56	N-S First Mode	0.98
59.77	1.58	N-S First Mode	1.40
47.01	1.61	N-S First Mode	1.28
13.94	1.64	N-S First Mode	1.52
40.92	2.22	First Torsional	1.70
23.55	2.28	First Torsional	1.26
60.75	4.68	E-W Second Mode	1.87
55.84	4.94	N-S Second Mode	1.77
21.30	7.15	Second Torsional	1.74
13.44	7.35	Second Torsional	2.04
25.52	12.70	E-W Third Mode	3.94
10.89	14.05	N-S Third Mode	
	17.4-18.5	Third Torsional	

TABLE 2

TESTS PERFORMED IN THE EAST-WEST DIRECTION
WITH EXTERNAL CLADDING IN PLACE

TEST NO.	TYPE OF TEST	NOMINAL FORCE LEVEL (LBF)	FORCE LEVEL AT RESONANCE (LBF)	RESONANCE FREQUENCY (CPS)	DAMPING (%)
1E-SD	Standard Damping	5,000*	4,760	1.36	1.4
2E-SD	Standard Damping	5,000	5,050	2.32	---
3E-SD	Standard Damping	5,000	4,310	4.60	2.6
4E-S	Sweep	10,000	---	1.15	---
5E-SD	Standard Damping	5,000	5,190	1.23	3.6
6E-SD	Standard Damping	5,000	4,620	4.55	3.3
7E-SD	Standard Damping	5,000	2,770	1.28	3.0
8E-SD	Standard Damping	5,000	6,400	4.50	2.9
9E-D	Damping	10,000	8,940	4.34	2.5
10E-D	Damping	10,000	7,800	1.19	3.4
11E-M	Mode Shape	10,000	8,900	4.32	---
12E-D	Damping	10,000	10,430	2.32	---
13E-M	Mode Shape	10,000	10,500	2.32	---
14E-M	Mode Shape	10,000	10,500	1.14	---
15E-SD	Standard Damping	5,000	3,960	1.12	---
16E-SD	Standard Damping	5,000	4,920	4.32	---
17E-D	Damping	15,000	15,880	3.94	3.7
18E-D	Damping	20,000	18,090	3.63	5.2
19E-D	Damping	25,000	29,920	3.23	8.2
20E-SD	Standard Damping	5,000	4,590	1.08	4.0
21E-SD	Standard Damping	5,000	4,860	3.45	6.3
22E-D	Damping	15,000	10,740	0.85	4.5
23E-D	Damping	20,000	18,800	0.60	---
24E-D	Damping	25,000	----	0.52	---
25E-SD	Standard Damping	5,000	4,990	2.59	5.6
26E-D	Damping	10,000	11,250	0.52	3.6
27E-M	Mode Shape	10,000	9,200	2.46	---
28E-M	Mode Shape	10,000	10,800	0.54	---

*The direction of sweep for this sweep was up, i.e., from a lower to higher frequency.

TABLE 3

TESTS PERFORMED IN THE NORTH-SOUTH DIRECTION WITH EXTERNAL CLADDING REMOVED

TEST NO.	TYPE OF TEST	NOMINAL FORCE LEVEL (LBF)	FORCE LEVEL AT RESONANCE (LBF)	RESONANCE FREQUENCY (CPS)	DAMPING (%)
1N-SD	Standard Damping	5,000	5,170	0.96	3.1
2N-SD	Standard Damping	4,000	3,610	0.81	3.5
3N-SD	Standard Damping	5,000	4,500	3.52	2.6
4N-SD	Standard Damping	4,000	3,530	3.23	3.7
5N-M	Mode Shape	5,000	3,800	0.82	---
6N-M	Mode Shape	5,000	4,600	0.94	---
7N-M	Mode Shape	5,000	4,300	3.14	---
8N-M	Mode Shape	5,000	4,100	3.47	---
9N-SD	Standard Damping	5,000	4,680	0.93	2.5
10N-SD	Standard Damping	4,000	2,920	0.80	3.5
11N-SD	Standard Damping	5,000	5,000	3.51	2.1
12N-SD	Standard Damping	4,000	3,670	3.16	2.8
13N-D	Damping	10,000	9,910	3.41	2.5
14N-D	Damping	7,500	7,640	2.97	3.0
15N-D	Damping	15,000	15,140	3.31	3.2
16N-D	Damping	10,000	10,570	2.84	3.4
17N-D	Damping	20,000	20,160	3.25	2.8
18N-D	Damping	13,000	12,930	2.72	3.7
19N-D	Damping	13,000*	13,180	2.70	2.0
20N-D	Damping	20,000*	19,410	3.21	1.6
21N-D	Damping	25,000	26,960	3.15	2.8
22N-D	Damping	15,000	15,260	2.51	3.2
23N-D	Damping	20,000	22,840	2.36	5.6
24N-SD	Standard Damping	5,000	5,190	3.30	2.5
25N-SD	Standard Damping	4,000	3,290	2.69	3.4
26N-SD	Standard Damping	5,000	5,320	0.92	3.0
27N-SD	Standard Damping	4,000	3,430	0.76	3.0
28N-D	Damping	10,000	8,480	0.81	3.1
29N-D	Damping	10,000	9,610	0.53	---
30N-D	Damping	15,000	---	0.68	---
31N-D	Damping	15,000	---	0.41	---
32N-M	Mode Shape	5,000	5,100	0.40	---
33N-M	Mode Shape	5,000	5,800	0.75	---
34N-M	Mode Shape	5,000	5,600	2.93	---
35N-M	Mode Shape	5,000	5,800	1.91	---
36N-SD	Standard Damping	5,000	5,470	2.91	2.8
37N-SD	Standard Damping	5,000	5,050	1.90	3.9
38N-SD	Standard Damping	5,000	4,720	0.72	3.1
39N-SD	Standard Damping	5,000	5,900	0.40	5.8
40N-D	Damping	10,000	8,[illegible]30	0.69	2.3
41N-D	Damping	15,000	---	---	---

*The direction of the stepwise sweep was up, i.e., from a lower to higher frequency. For all other tests the direction of the sweep was down, i.e., from a higher to lower frequency.

TABLE 4

FIRST TRANSLATIONAL MODE DAMPING TESTS WITH EXTERNAL CLADDING, EAST-WEST DIRECTION

TEST NO.	FORCE LEVEL AT RESONANCE (LBF)	PERIOD (SECONDS)	DAMPING (PERCENT)
1E-SD	4,760*	0.74	1.4
4E-S	10,000	0.87	---
5E-SD	5,190	0.81	3.6
7E-SD	2,770	0.78	3.0
10E-D	7,800	0.84	3.4
15E-SD	3,960	0.89	---
	SECOND MODE DAMPING TESTS		
20E-SD	4,590	0.93	4.0
22E-D	10,740	1.18	4.5
23E-D	18,800	1.67	---
24E-D	25,000	1.92	---
26E-D	11,250	1.92	3.6

*In all tests except 1E-SD the direction was down, i.e., from a higher to lower frequency. In 1E-SD the direction of the sweep was the reverse.

TABLE 5

SECOND TRANSLATIONAL MODE DAMPING TESTS WITH EXTERNAL CLADDING, EAST-WEST DIRECTION

TEST NO.	FORCE LEVEL AT RESONANCE (LBF)	PERIOD (SECONDS)	DAMPING (PERCENT)
3E-SD	4,310	0.22	2.6
6E-SD	4,620	0.22	3.3
8E-SD	6,400	0.22	2.9
9E-D	8,940	0.23	2.5
16E-SD	4,920	0.23	---
17E-D	15,880	0.25	3.7
18E-D	18,090	0.28	5.2
19E-D	29,920	0.31	8.2
21E-SD	4,860	0.29	6.3
	FIRST MODE DAMPING TESTS		
25E-SD	4,990	0.39	5.6

TABLE 6

FIRST TRANSLATIONAL MODE DAMPING TESTS WITHOUT EXTERNAL CLADDING, NORTH-SOUTH DIRECTION

TEST NO.	FORCE LEVEL AT RESONANCE (LBF)	PERIOD (SECONDS)	DAMPING (PERCENT)
2N-SD	3,610	1.23	3.5
10N-SD	2,920	1.25	3.5
	SECOND MODE TESTS		
27N-SD	3,430	1.32	3.0
29N-D	9,610	1.90	---
31N-D	15,000	2.44	---
39N-SD	5,900	2.50	5.8
---	20,000	---	---

TABLE 7

FIRST TORSIONAL MODE DAMPING TESTS WITHOUT EXTERNAL CLADDING NORTH-SOUTH DIRECTION

TEST NO.	FORCE LEVEL AT RESONANCE (LBF)	PERIOD (SECONDS)	DAMPING (PERCENT)
1N-SD	5,170	1.04	3.1
9N-SD	4,680	1.08	2.5
	SECOND MODE TESTS		
26N-SD	5,320	1.09	3.0
28N-D	8,480	1.24	3.1
30N-D	15,000	1.47	---
38N-SD	4,720	1.39	3.1
40N-D	8,130	1.45	2.3

TABLE 8

SECOND TRANSLATIONAL MODE DAMPING TESTS WITHOUT EXTERNAL CLADDING, NORTH-SOUTH DIRECTION

TEST NO.	FORCE LEVEL AT RESONANCE (LBF)	PERIOD (SECONDS)	DAMPING (PERCENT)
4N-SD	3,530	0.31	3.7
	MODE SHAPE TESTS		
12N-SD	3,670	0.32	2.8
14N-D	7,640	0.34	3.0
16N-D	10,570	0.35	3.4
18N-D	12,930	0.37	3.7
19N-D	13,180*	0.37	2.0
22N-D	15,260	0.40	3.2
23N-D	22,840	0.42	5.6
25N-SD	3,290	0.37	3.4
	FIRST MODE TESTS		
37N-SD	5,050	0.53	3.9

*In all tests except 19N-D the direction of sweep was down, i.e., from a higher to lower frequency. In 19N-D the direction of the sweep was the reverse.

TABLE 9

SECOND TORSIONAL MODE DAMPING TESTS WITHOUT EXTERNAL CLADDING, NORTH-SOUTH DIRECTION

TEST NO.	FORCE LEVEL AT RESONANCE (LBF)	PERIOD (SECONDS)	DAMPING (PERCENT)
3N-SD	4,500	0.28	2.6
11N-SD	5,000	0.28	2.1
13N-D	9,910	0.29	2.5
15N-D	15,140	0.30	3.2
17N-D	20,160	0.31	2.8
20N-D	19,140*	0.31	1.6
21N-D	26,940	0.32	2.8
24N-SD	5,190	0.30	2.5
	FIRST MODE TESTS		
36N-SD	5,470	0.34	2.8

*In all tests except 20N-D the direction of sweep was down, i.e., from a higher to lower frequency. In 20N-D the direction of the sweep was the reverse.

TABLE 10

UNIFORM BUILDING CODE EQUIVALENT OF FORCES INDUCED DURING TEST NO. 23E-D

CODE	PERIOD (SECONDS)	V_T/V_C - BASE SHEAR			
		ZONE 4	ZONE 3	ZONE 2	ZONE 1
74-UBC	MEAS. = 1.67		2.7	5.4	10.8
74-UBC	CODE = 1.1		2.4	4.8	9.6
76-UBC	MEAS. = 1.67	1.5	2.0	4.0	8.0
76-UBC	CODE = 1.1	1.2	1.6	3.2	6.4

NOTE: 1. The measured base shear was 250 kips.
2. The input force at resonance was 18,800 lbf.

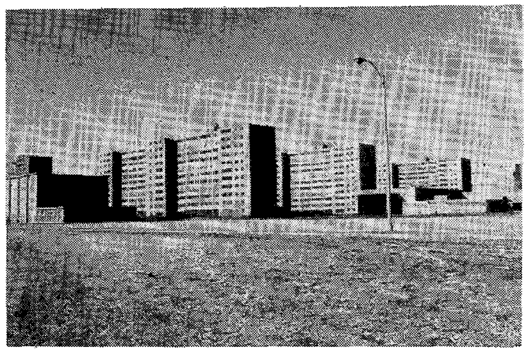


Fig. 1 Pruitt-Igoe Complex

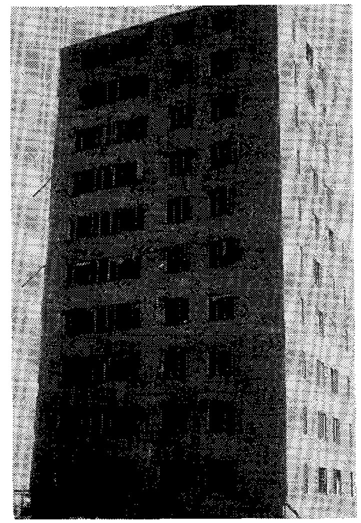


Fig. 2 Test Building, North Face

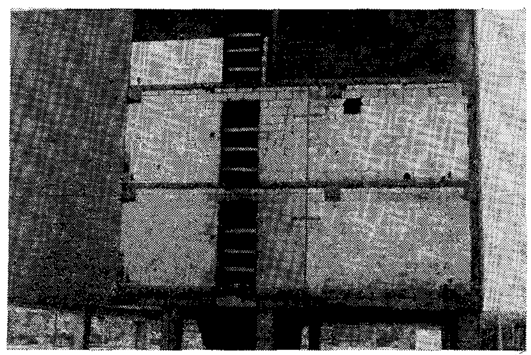


Fig. 3 East Face, Severed Connections

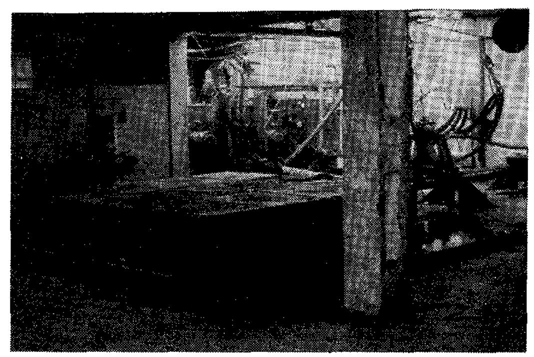


Fig. 4 Test Equipment

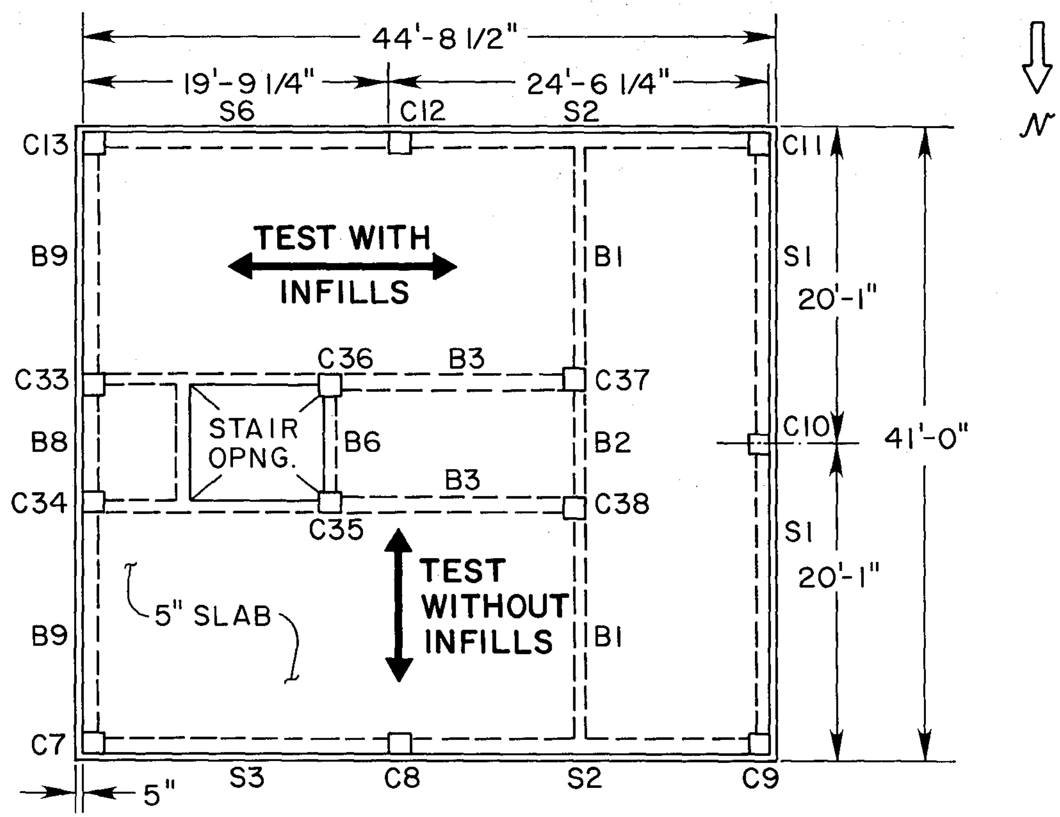


FIGURE 5. TYPICAL FLOOR PLAN WITH BEAM AND COLUMN NUMBERS.

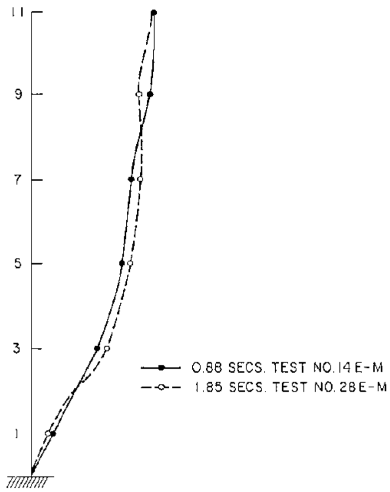


Fig. 6 Normalized Mode Shapes for 1st Translational Mode with External Cladding

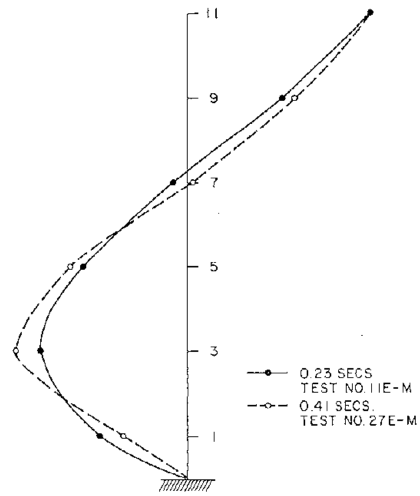


Fig. 7 Normalized Mode Shapes for 2nd Translational Mode with External Cladding

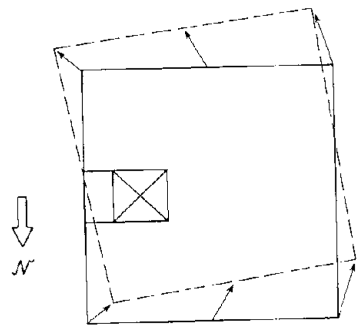


Fig. 8 Response of 9th Floor for 1st Translational Mode without External Cladding

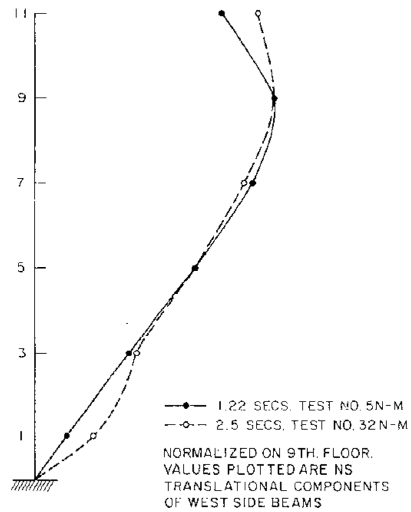


Fig. 10 Normalized Mode Shapes for 1st Translational Mode without External Cladding

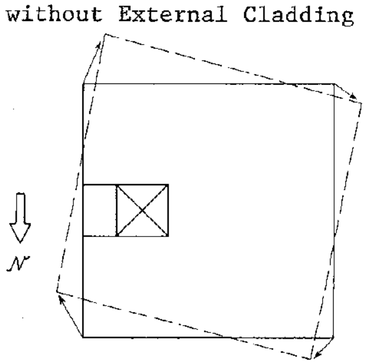


Fig. 9 Response of 9th Floor for 1st Torsional Mode without External Cladding

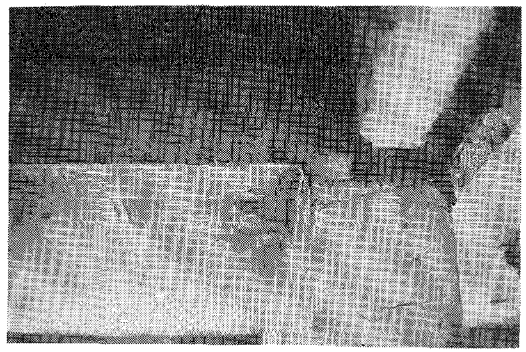


Fig. 11 Level 11, Column 38,
Prior to Testing

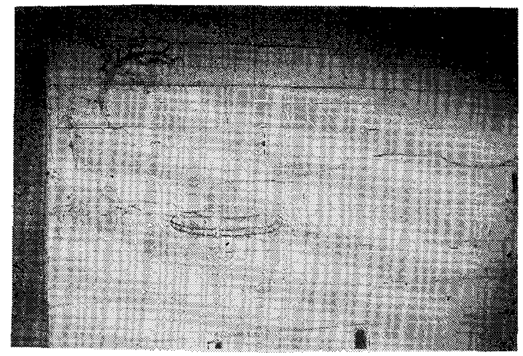


Fig. 12 Stairwell Wall, Level 11,
Prior to Testing

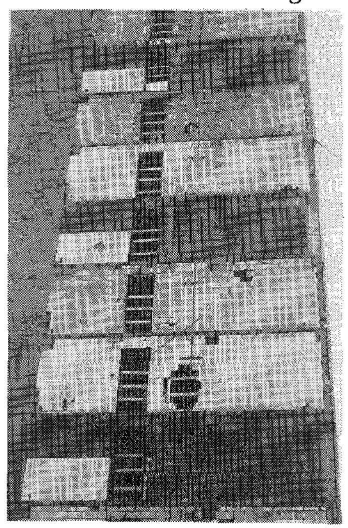


Fig. 13 East Face,
Prior to Testing

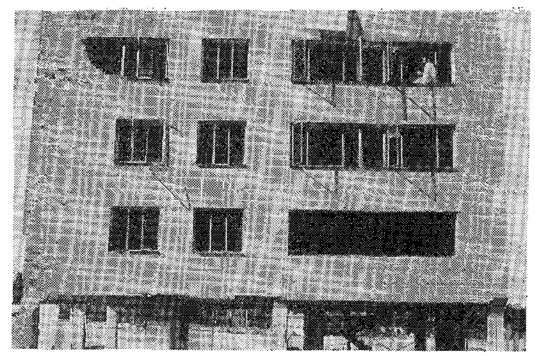


Fig. 14 South Face, Prior to Testing

Fig. 15 Southeast Corner, Prior to Testing

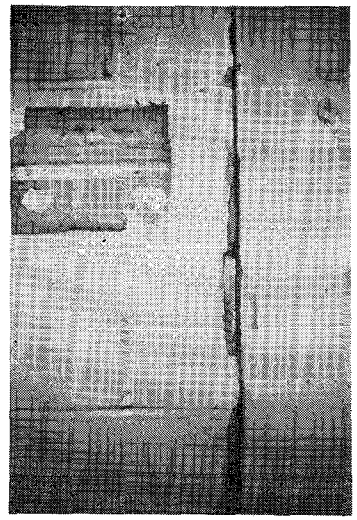


Fig. 16 Stairwell Wall, Level 2, After Test No. 12E-D

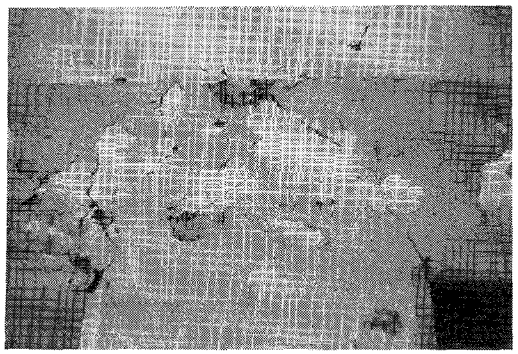


Fig. 17 Column 36, Level 1, after EW Tests

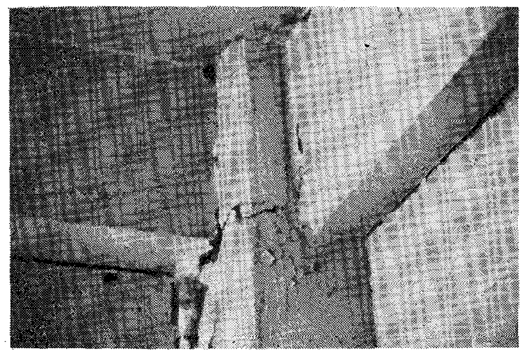


Fig. 18 Column 11, Level 2, after EW Tests

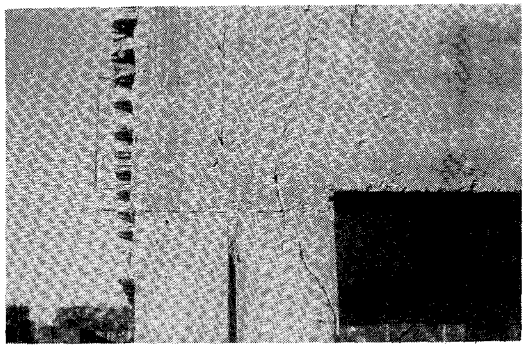


Fig. 19 Column 11, Level 1, after EW Tests

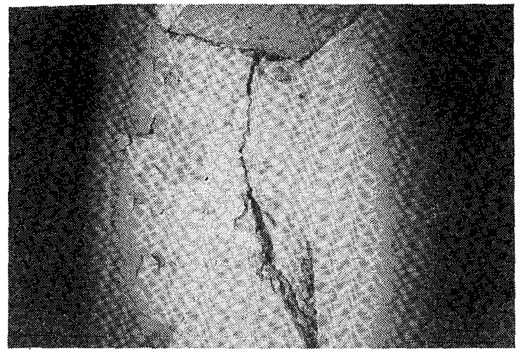


Fig. 21 Column 37, Level 2, after EW Tests

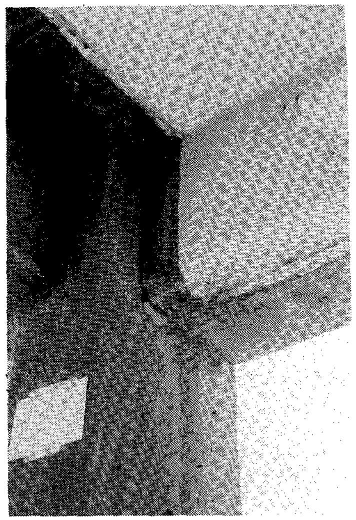


Fig. 20 Beam S6, Level1, after EW Tests

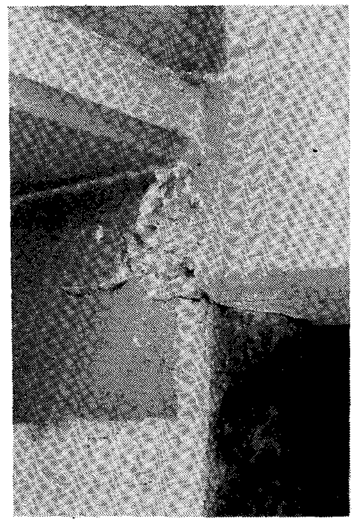


Fig. 22 Beam 4 to Column 33, during EW Tests

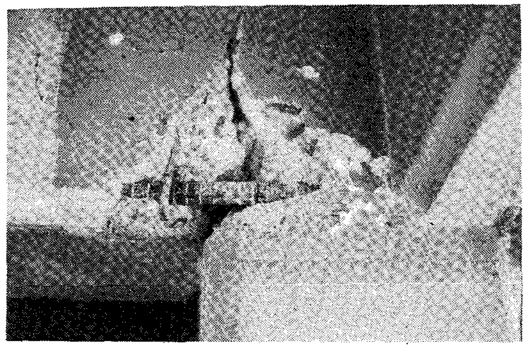


Fig. 23 Beam 4 to Column 33, during EW Tests

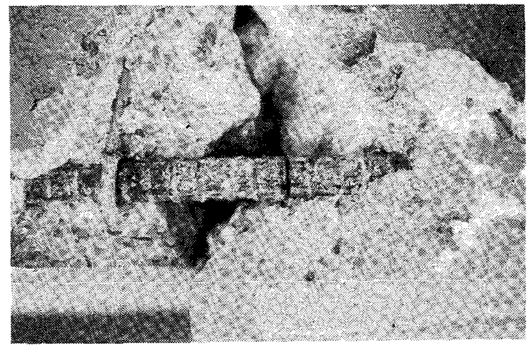


Fig. 24 Beam 4 to Column 33, after EW Tests

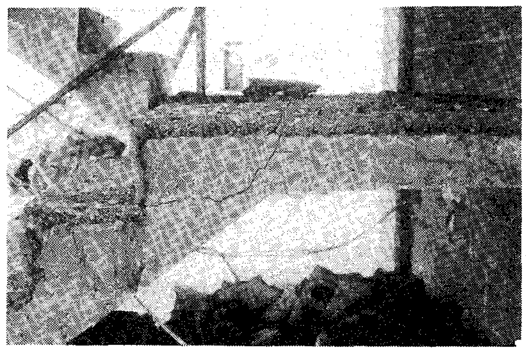


Fig. 25 Side of Stairwell Landing between Levels o and 1 after Test No. 17E-D

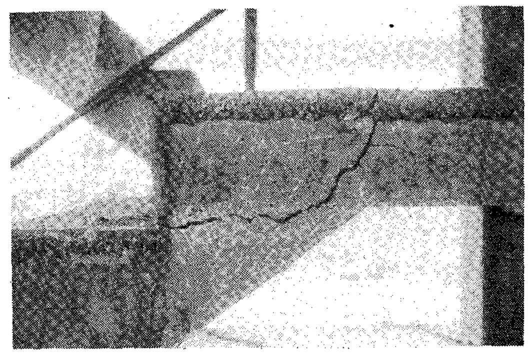


Fig. 26 Side of Stairwell Landing between Levels 0 and 1 after EW Tests

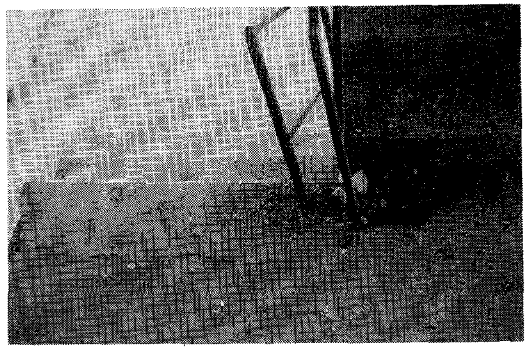


Fig. 27 Top of Stairwell Landing between Levels 0 and 1 after EW Tests

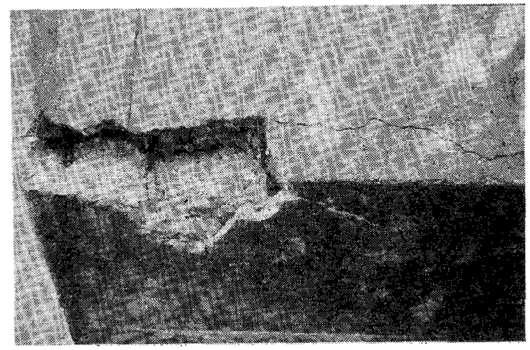


Fig. 28 Bottom of Stairwell Landing between Levels 0 and 1 after EW Tests

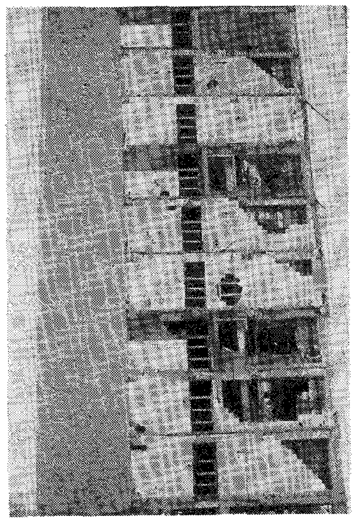


Fig. 29 East Face, after EW Tests

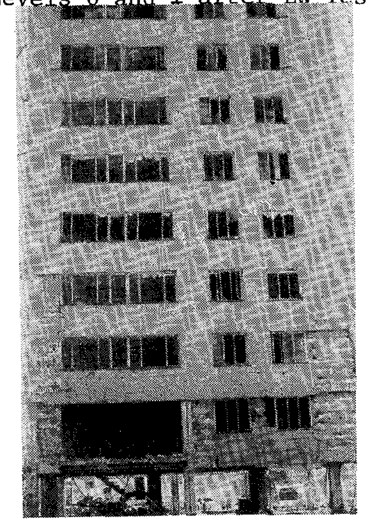


Fig. 30 North Face, after EW Tests

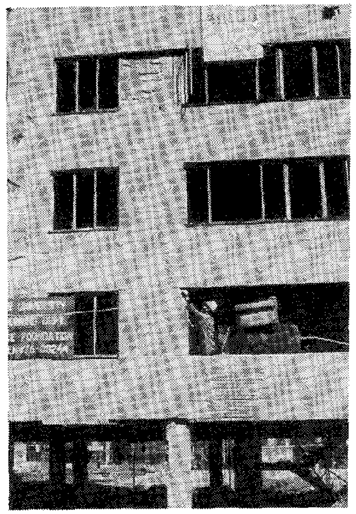


Fig. 31 South Face, after EW Tests

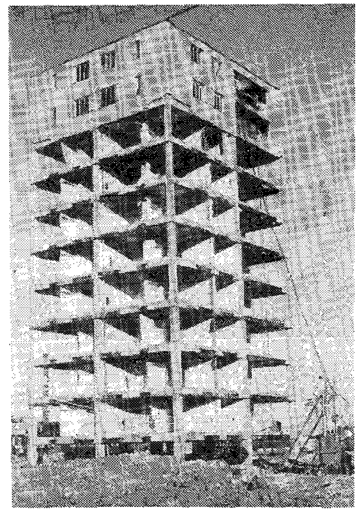


Fig. 32 Test Building, prior to NS Tests

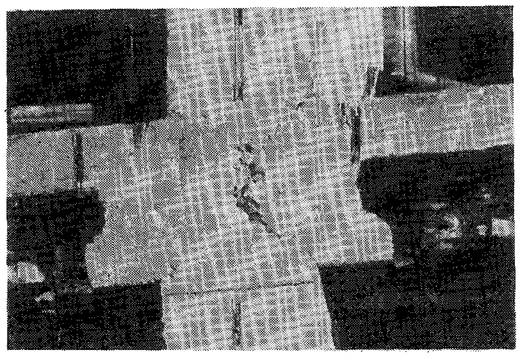


Fig. 33 Column 10, Level 4, after Test No. 23N-D

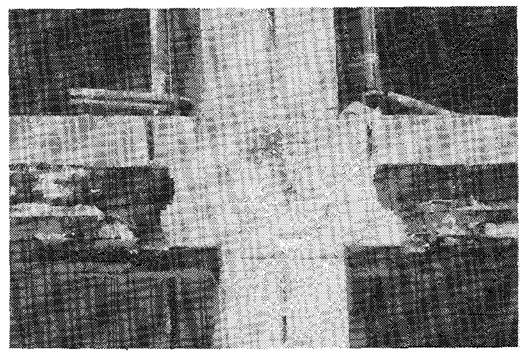


Fig. 34 Column 10, Level 4, after Test No. 30N-D

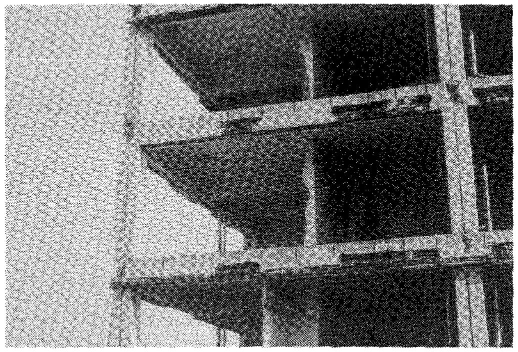


Fig. 35 Column 9, Level 4,
after NS Tests

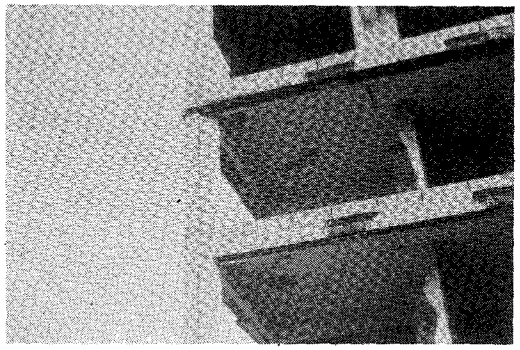


Fig. 37 Column 9, Level 4
after Test No. 21N-D

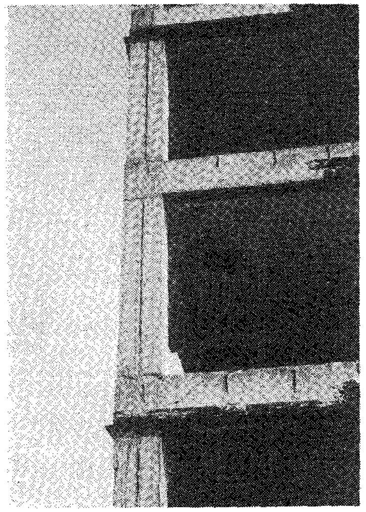


Fig. 36 Column 9, prior to
Test No. 21N-D

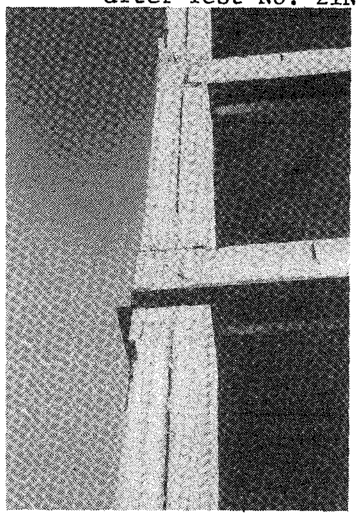


Fig. 38 Column 9, Level 4,
after Test No. 22N-D

Fig. 39 Column 9, Level 4, during Test No. 23N-D

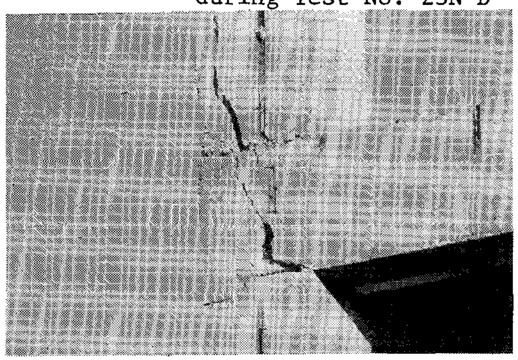


Fig. 40 Column 9, Level 4, during Test No. 23N-D

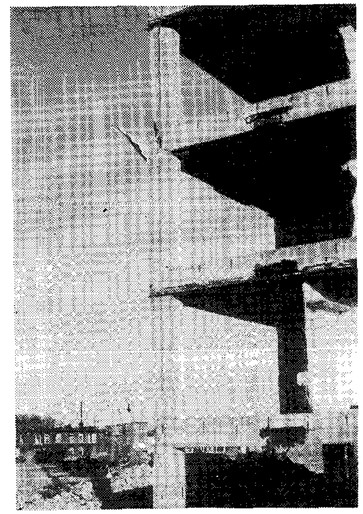


Fig. 41 Column 9, Level 4, during Test No. 23N-D

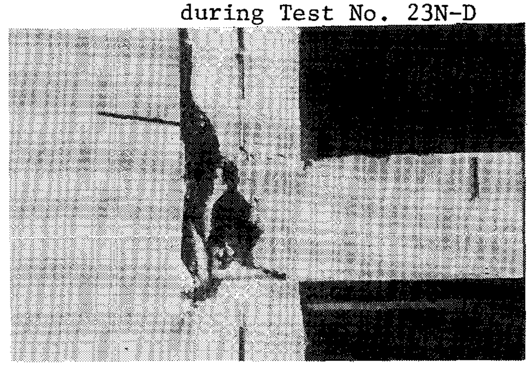


Fig. 42 Column 9, Level 4, after Test No. 23N-D

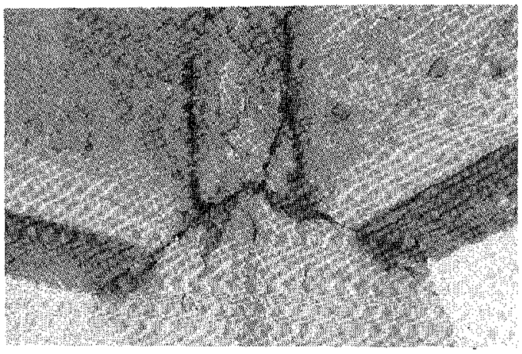


Fig. 43 Column 9, Level 3, after Test No. 29N-D

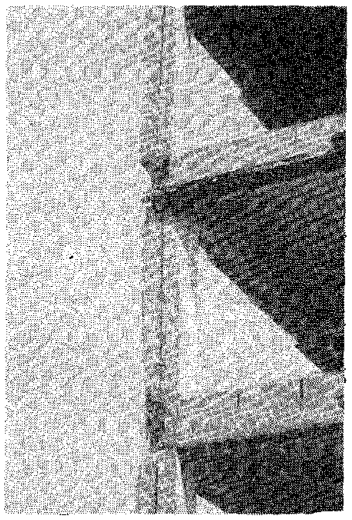


Fig. 44 Column 9, Level 4, after Test No. 41N-D

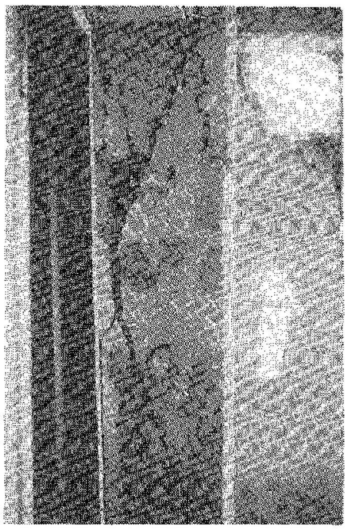


Fig. 45 Column 38, Level 4, after Test No. 29N-D

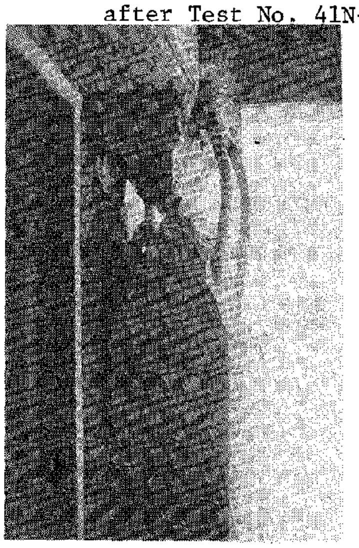


Fig. 46 Column 38, Level 4, after Test No. 41N-D

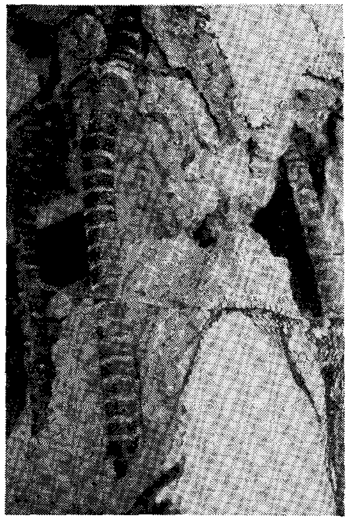


Fig. 47 Column 38, Level 4, after Test No. 41N-D

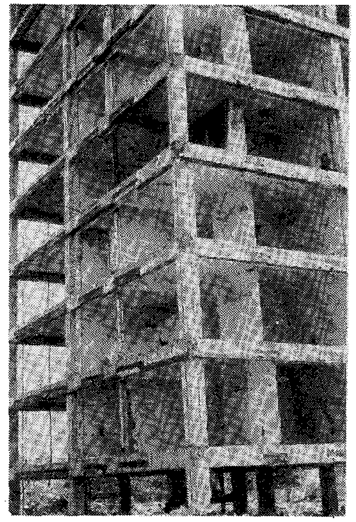


Fig. 48 Southwest Corner, after Test No. 41N-d

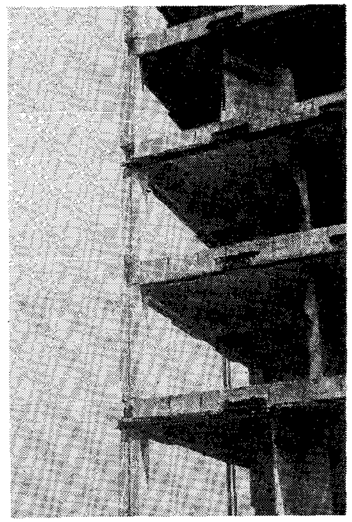


Fig. 49 Northwest Corner, after Test No. 41N-D

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

DYNAMIC BEHAVIOR OF AN ELEVEN-STORY MASONRY BUILDING

by

R. M. Stephen
Principal Development Engineer
University of California, Berkeley

J. G. Bouwkamp
Professor of Civil Engineering
University of California, Berkeley

INTRODUCTION

The design of multistory structures subjected to dynamic forces resulting from foundation motions requires a consideration of both the characteristics of the ground motion and the dynamic properties of the structure. The availability of high-speed digital computers and the sophistication of the idealization of structures and the computer model formulation of the structure have made available the elastic, and in certain structural systems, the inelastic response of structures when subjected to earthquakes. However, the accuracy of the results in large measure depend upon the computer model formulation of the structure and its foundation. In order to determine the accuracy of the calculated results and to accumulate a body of information on the dynamic properties of structures, especially when these structures have novel design features, a number of dynamic tests have been conducted on full-scale structures [1].

For the above reasons a dynamic test was performed on the Oak Center Towers, Oakland, California [2].

DESCRIPTION OF BUILDING

The Oak Center Towers is an eleven-story structure located in Oakland, California. The 100-foot-high building has an overall plan of 85 feet by 200 feet. The building is offset in the middle by approximately 16 feet so that it does not have a pure rectangular plan. It is constructed with reinforced concrete block shear walls and prefabricated prestressed concrete slab elements. The elevator shaft is located in the center south section of the structure with stairwells at either end. Figure 1 shows the east elevation of the building.

The building is designed as a housing development for the elderly and is therefore modular in concept. The building is serviced by two elevators in the center south section of the structure. Stairwells are located on either end of the building. Figure 2 shows a typical floor plan for the second through eleventh floors.

The vertical and horizontal load carrying systems are reinforced concrete masonry shear walls in both the transverse and longitudinal directions. These walls rise from the first floor and run up to the roof except in one section on the south end of the first floor where the dining room is located. In this location the walls terminate at the second floor and a reinforced concrete frame system carries the loads to the foundations.

The foundations are in general spread footings under each of the walls from 4 feet to 6 feet in width and 18 inches thick.

The compressive strength of the masonry unit is 3000 psi below the eighth floor and 2000 psi above the eighth floor. All of the cells in the masonry were grouted with 4000 psi hard rock concrete.

The transverse walls are made up of eight-inch-wide blocks from the first floor to the roof. The longitudinal walls, which basically run down each side of the corridor, are twelve-inch-wide block up to the fifth floor and eight-inch-block from the fifth floor to the roof.

The minimum reinforcement consisted of two number 4 bars at 24 inches on centers vertical and the same horizontal for the twelve-inch block and in the eight-inch block one number 4 bar at 24 inches on center, both vertically and horizontally. Special reinforcement is added at wall ends, corners and where two walls connect. This consisted mainly of number 8 bars up to the eighth floor then number 6 bars from the eighth to the tenth floors and number 5 bars from the tenth floor to the roof.

The floor system consists of precast prestressed plans 6 inches deep and 40 inches wide spanning between the transverse walls. A two-inch lightweight concrete topping is placed over these planks.

EXPERIMENTAL PROGRAM

Forced vibrations were produced by two rotating-mass vibration generators or shaking machines mounted on the eleventh floor of the building and oriented so as to induce the maximum forces in the east-west and north-south directions as shown in Figure 2. A complete description of the vibration generators is given elsewhere [1,3].

The transducers used to detect horizontal floor accelerations of the building were Statham Model A4 linear accelerometers, with a maximum rating of $\pm$ 0.25g. The electrical signals for all accelerometers were fed to amplifiers and then to a Honeywell Model 1858 Visicorder. For the translational motions the accelerometers were located near the center of the floor and oriented so as to pick up the appropriate east-west or north-south accelerations. For recording, the torsional motion accelerometers were properly oriented near the north and south ends of the building. To determine the resonant frequencies of the building the accelerometers were located on the eleventh floor. In addition vertical accelerations were taken at six locations on the first floor to determine the foundation motion. The mode shapes were evaluated from records taken at all of the floors, including the roof.

MATHEMATICAL MODEL

A mathematical computer model of the Oak Center Tower building was formulated to assess its dynamic characteristics. The model was formulated using both a rigid base and a flexible base. TABS, a general computer program developed by the Division of Structural Engineering and Structural Mechanics of the Department of Civil Engineering at the University of California, Berkeley, was used to calculate the frequencies and mode shapes of the building. A complete description of this program is given in reference 4.

The program considers the floors rigid in their own plane and to have zero transverse stiffness. All elements are assembled initially into planar frames and then transformed, using the previous assumption, to three degrees of freedom at the center of mass for each story level (2 translational, 1 rotational). Coupling between independent frames at common column lines is ignored. The basic model of the building was formulated as a system of independent frames and shear wall elements interconnected by floor diaphragms which were rigid in their own plane and fixed at the first floor level.

The story masses were obtained from the approximate dead loads per floor. These lumped weight values include the floor slabs and masonry walls for each floor level.

During the experimental phase of the work, significant vertical motion was recorded at the first floor level in the building. Therefore, as a second basic approach the model was allowed to have a flexible base.

Based on the measurements of the ground accelerations at the first floor of the building the following approach was used.

It was assumed that the accelerations measured were due primarily to first mode response. As such, a first mode shape was assumed and acceleration values at each mass point computed. These were used to calculate an effective overturning moment which, when compared with the measured ground acclerations, allowed an assessment of the base rotational and translational stiffness. An additional basement story was added to the structure with the stiffness values as determined above, assigning to the elements as a means of modeling the rotational and translational flexiblity.

RESULTS

In the forced vibration tests, two translational modes in the east-west and north-south directions were excited, as well as the one torsional mode. Typical frequency response curves in the region of the resonant frequencies are shown in Figure 3. The typical vertical and horizontal mode shapes are shown in Figure 4. The resonant frequencies and damping factors evaluated from the experimental data along with the analytical results are summarized in Table 1.

CONCLUSIONS

In comparing the forced vibration as well as the analytical solution it is noted that there is reasonable agreement in the first two modes, although this does not hold as true for the higher modes. The analysis indicated that in the first E-W mode there was a significant contribution of torsion in this mode. This is also noted in the forced vibration study where the first E-W mode and the first torsional mode (2.78 and 2.83 cps, respectively) were very close together.

The predominant feature which came out of the analytical solutions was the effect of the foundations on the response of the structure. The fundamental frequency was almost half for the flexible foundation as for the rigid foundation (2.45 versus 4.13 cps, respectively). The analysis of very rigid structures on flexible foundations must consider the soil-structure interaction phenomena or the solution could be as much as 100 percent off.

It is apparent that for structures where the in-plane stiffness of the floor system is less or comparable to the stiffness of the lateral load resisting system, the assumption that the floors are rigid in their own plane does not seem to hold true.

This same response regarding the flexibility of the foundation and the in-plane bending of the floor system was also noted in some recent forced vibration studies carried out on a building in Sarajevo, Yugoslavia [5].

The damping values determined from the forced vibration studies varied from about 2 percent to almost 9 percent. The higher damping values could be due to the flexibility of the foundations and their contribution to the response of the building.

TABLE 1 COMPARISON OF RESONANT FREQUENCIES AND DAMPING RATIOS

EXCITATION	MODE							
	1				2			
	FORCED VIBRATION		ANALYSIS		FORCED VIBRATION		ANALYSIS	
	FREQ cps	DAMP %	FIXED BASE	FLEX BASE	FREQ cps	DAMP %	FIXED BASE	FLEX BASE
E-W	2.78	6.4	4.50	2.45	5.82	2.1	14.49	4.13
N-S	3.30	8.8	4.98	2.98	5.93	2.8	5.08	--
Torsional	2.83	2.6	--	--	--	--	--	--

ACKNOWLEDGMENT

The authors gratefully acknowledge the financial support provided by the National Science Foundation under Grant NSF GK-31883. They also wish to thank the owner, the Episcopal Homes Foundation; the architect, Kennard and Silvers; and the contractors, Williams and Burrows and F. M. Taylor and Son, Inc., for their help and cooperation during the tests.

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2. Stephen, R. M., et al., "Dynamic Properties of an Eleven Story Masonry Building," Report No. EERC 75-20, Earthquake Engineering Research Center, University of California, Berkeley, July 1975.
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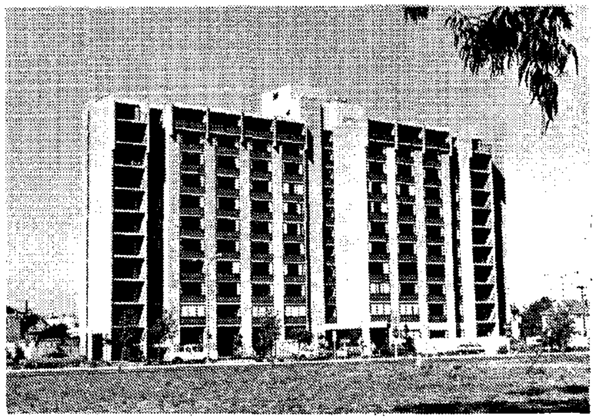


FIG. 1 OAK CENTER TOWERS, OAKLAND, CALIFORNIA (EAST ELEVATION)

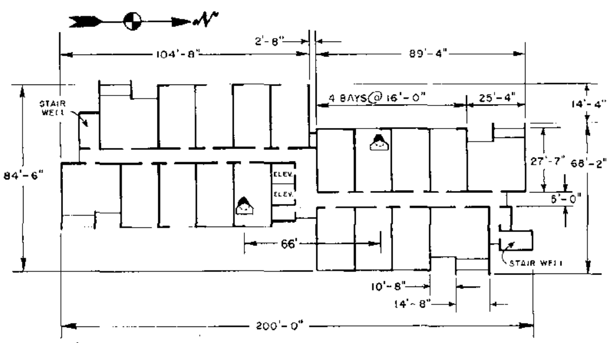
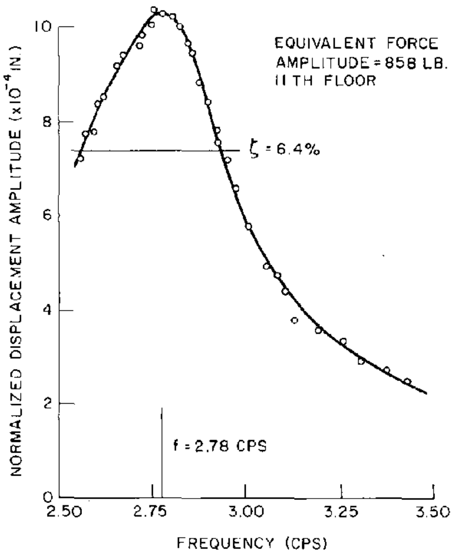
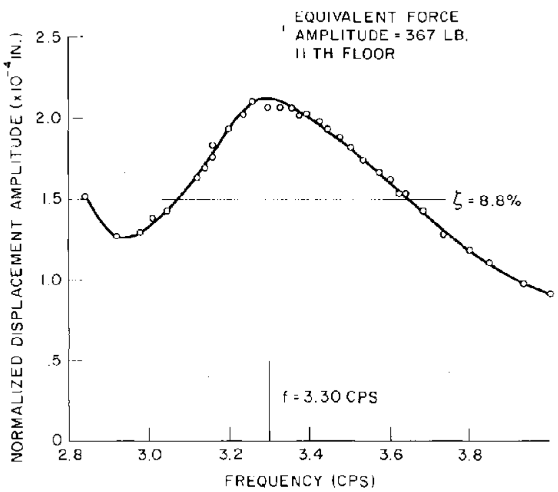


FIG. 2 TYPICAL FLOOR PLAN



FIRST MODE EAST-WEST



FIRST MODE NORTH-SOUTH

FIG. 3 TYPICAL FREQUENCY RESPONSE CURVES

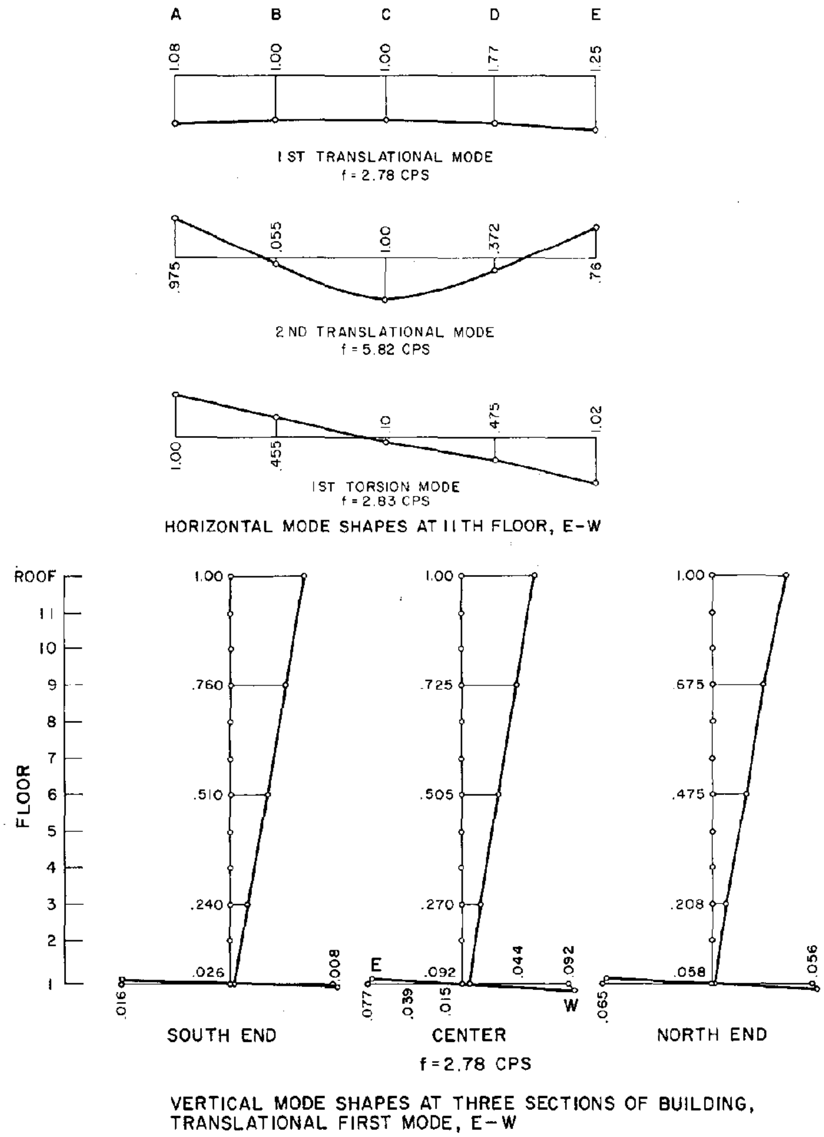


FIG. 4 TYPICAL MODE SHAPES

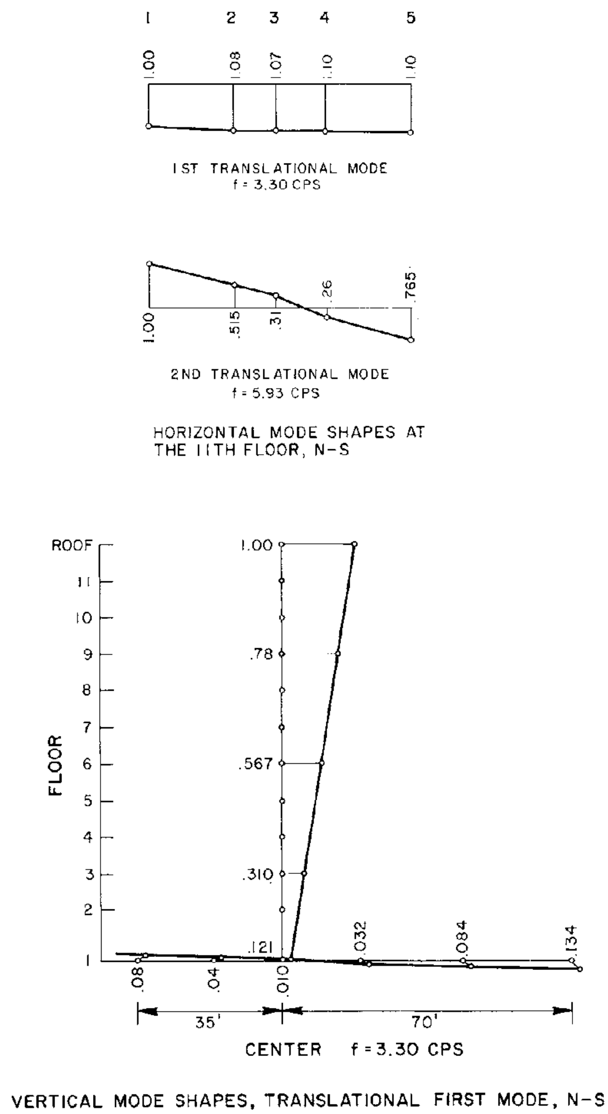


FIG. 4 (CONT'D); TYPICAL MODE SHAPES

STRONG-MOTION INSTRUMENTATION OF REINFORCED CONCRETE BUILDINGS

by

Christopher Rojahn
Research Civil Engineer
U.S. Geological Survey
Menlo Park, California

INTRODUCTION

Currently there are three programs in the U.S. that provide for the strong-motion instrumentation of buildings. The U.S. Geological Survey (USGS), with funding from the National Science Foundation and other cooperating Federal and State agencies, operates a national network of strong-motion instruments, the California Division of Mines and Geology (CDMG) operates a statewide strong-motion program that is funded through a tax on building permits, and municipal building codes in some cities in the most active seismic areas of the U.S. require accelerographs in most buildings over six stories in height [2, 5]. Eighty to ninety percent of the approximately 300 buildings that have been instrumented to date in the U.S. have been instrumented as a result of building code requirements; most of the remainder have been instrumented under the CDMG program.

This paper describes current USGS guidelines for the strong-motion instrumentation of buildings in general, and reinforced concrete buildings, in particular. These guidelines, developed by Rojahn and Matthiesen [7] and implemented both by USGS and CDMG [6] differ radically from those specified in existing building codes. The codes simply require three triaxial accelerographs in each instrumented building (one in the basement, one at mid-height, and one near the top), whereas the techniques prescribed here utilize state-of-the-art remote recording instrumentation located in accordance with known or expected mode shapes and so as to record separately both translational and torsional response. The guidelines are intended to overcome some of the major weaknesses of the code requirements, both with regard to type and location of instrumentation, as well as the selection of buildings to be instrumented. The underlying objective is to obtain data that can be used to improve engineering design practice.

BUILDING SELECTION

The two most important considerations in selecting buildings for strong-motion instrumentation are structure location and type. Structure location is crucial because the selection of an inappropriate location may result in noncritical or, at worst, no data; structure type is important because it dictates the usefulness of any derived data and resulting analyses.

It is essential that buildings selected for instrumentation be located in areas where damaging or severe non-damaging levels of strong ground motion can be expected to occur at least once within the life of the instrumentation, currently estimated to be 20 to 40 years (oral commun., H. T. Halverson, 11/4/77). To do otherwise would be to lower or negate the probability of obtaining data from damaged or heavily loaded structures--the most interesting, critical, and difficult information to obtain. Furthermore, if the probability of obtaining significant strong-motion data is low (or nonexistent), then the probability of wasting the entire financial outlay for instrumentation, currently estimated to average $15,000 per structure for instrument purchase and installation and $600 per year for maintenance (oral commun., T. M. Wootton, 11/7/77), is high (or assured). This possibility is surely economically undesirable.

In selecting structures for instrumentation, consideration should also be given to the estimated natural frequencies of predominant response of the structure in comparison with the frequency range of expected high-amplitude long duration ground motion. This is important because damage potential is related to the frequency content of ground motion. High-frequency structures, those with fundamental frequencies above 2 Hz, should be instrumented in areas where high-amplitude motion is expected to be in the same high frequency range; conversely, lower frequency structures should be instrumented in areas where predominant strong ground motion is expected to be in the lower frequency range. In the western U.S., for example, where high-frequency motion tends to attenuate more quickly with distance from the source of energy release than does lower frequency motion, those low-rise (1- to 6-story) and other high-frequency buildings that are selected for instrumentation should be located near the potential source of energy release, say 10 km or less. Low-frequency motion, on the other hand, can be expected to be significant at all locations between those reasonably close to the source of energy release and those as far away as 150 km or more.

Buildings selected for instrumentation should be typical in terms of age, type of construction, and number of stories, as well as simple in framing and design. If the structural design concept is simple, both the required instrumentation and the assumptions necessary for interpretation of any derived strong-motion data are minimized. The advantage in selecting typical buildings is that the results are transferrable, i.e., the results from analyses of derived data can be applied to other similar structures.

RECOMMENDED INSTRUMENTATION

It is recommended that remote recording instrumentation, consisting of single-axis or multiaxial accelerometers connected via data cable to a central recorder or recorders, be used rather than a system of three triaxial optical-mechanical self-contained accelerographs, as is presently required by the City of Los Angeles and other municipalities that adopted similar ordinances. The remote recording systems record data on a common time basis and are recommended because: the accelerometers, which are available in one-, two-, or three-component configurations, require less space than the triaxial self-contained accelerographs; the accelerometers can be attached directly to the building's structural system at all locations of interest including columns; the recorder or recorders can be located at one convenient location for easy maintenance and record retrieval; and there need be no loss in the frequency range of flat response for the system, which is equivalent to that of the modern triaxial self-contained accelerographs—0 to 20 or 25 Hz. Furthermore and perhaps more important, triaxial systems like those presently required in code-instrumented buildings do not provide enough data to isolate translational and torsional response, a capability that forced-vibration tests as well as analyses of the 1971 San Fernando earthquake records indicate is vital even in highly symmetrical buildings [7].

GUIDELINES FOR ACCELEROMETER PLACEMENT

As a minimum, accelerometers should be placed on the lowest and/or ground level, at the main roof level, and at one or two intermediate levels. The quantity and arrangement of instrumentation at these levels are dependent upon foundation conditions and type, building size, structural framing system, known dynamic characteristics, and the location of seismic joints.

Accelerometers are placed on the lowest level in order to record the input motion at the base of the structure. As a minimum, it is recommended that three orthogonal accelerometers (two horizontal and one vertical) be attached firmly to the foundation or floor near the center of plan with the horizontal accelerometers oriented parallel to the transverse and longitudinal axes of the buildings. If the foundation conditions are such that differential horizontal motion may occur, one or more additional horizontal accelerometers are recommended. In a building that is large and relatively square in plan, two additional accelerometers should be positioned along and parallel to two adjacent outside walls (figure 1), whereas in a building that is very long in comparison with its width, one additional accelerometer positioned along and parallel to one of the outside end walls

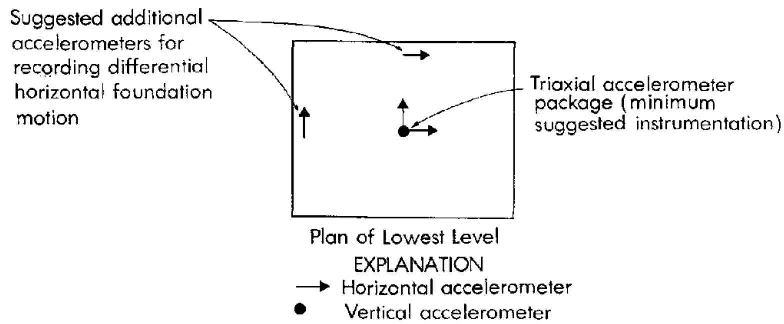


Figure 1.--Suggested strong-motion instrumentation scheme for recording differential horizontal foundation motion in a building that is large and relatively square in plan.

may be sufficient. If the building has a rigid mat foundation and rocking motion is expected, or if rocking of a shear-wall or steel bracing system is expected to be significant, additional vertical accelerometers are recommended. For the mat foundation case, one vertical accelerometer should be positioned in each of three corners of the building so that rocking motion can be recorded along any azimuth. In the shear-wall or steel bracing system case, one vertical accelerometer should be positioned at each end of the wall or bracing system.

In buildings without basements, the lowest level would of course be the ground level and should be instrumented in accordance with the above guidelines. In buildings with one or more basements, however, it would be desirable to instrument both the ground and lowest level if it is likely that the motion will be significantly different at the two locations. In buildings without shear-walls between the ground and lowest level, for example, instrumentation of both levels is particularly important, primarily because the effect of the surrounding soil and retaining walls on the building's mode shapes is unknown. In this case, it is recommended that the ground level be instrumented in the same general fashion as the roof and other above-ground levels.

Above ground level, significant building response to earthquake motion is primarily horizontal in nature. Appropriate instrumentation, therefore, would consist of an array of remote horizontal accelerometers located at the roof level and at one or more intermediate floors. The arrangement of horizontal accelerometers at each of these levels is influenced by the presence of seismic joints and the stiffness characteristics of the floor (or roof). If there are seismic joints, each isolated section should be treated as a separate building and instrumented accordingly. If the floor (roof) is very stiff and is expected to be rigid in the horizontal plane, three accelerometers are recommended (figure 2). A biaxial pair should be

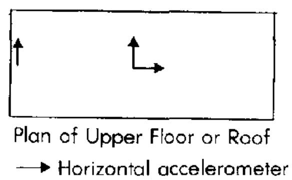


Figure 2.--Suggested strong-motion instrumentation scheme for roof (or floor) expected to be rigid in the horizontal plane.

located at the predicted or known center of rigidity so as to record, if possible, purely translational motion. The accelerometers should be oriented parallel to the transverse and longitudinal axes of the building (parallel to the biaxial pair on the lowest level). The third accelerometer should be positioned along and parallel to the most distant outside end wall so as to obtain, after analysis, torsional motion. If the floor (roof) is not expected to be rigid in the horizontal plane, one or more additional horizontal accelerometers are recommended. The location of each of these will be dependent upon the expected response of the floor (roof). For example, in the case of a rectangular-plan exterior shear-wall building with a floor (roof) diaphragm flexible in the transverse direction and not in the longitudinal direction, one additional accelerometer is recommended. It should be positioned (figure 3) so as to facilitate the interpretation of relative motion in the transverse direction between the end walls and the center of the floor (roof) diaphram. Other flexible floor (roof) diaphragm systems should be instrumented similarly, that is, so as to facilitate interpretation of potentially significant relative motion(s).

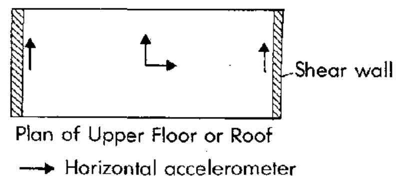


Figure 3.--Suggested strong-motion instrumentation scheme for roof (or floor) where relative diaphragm motion is expected in the transverse direction.

The number of intermediate levels at which instrumentation should be installed is a function of the structural framing system, number of stories, and known dynamic characteristics of the building. As a minimum, it is essential to instrument the roof covering the main portion of the building (not the roof of a penthouse that is significantly smaller in plan) to obtain maximum amplitude data on first mode response. In general, additional instrumentation is recommended at as many intermediate floors as is economically feasible although the instrumentation of more than three intermediate floors may be unnecessary. This suggested upper limit of three instrumented intermediate levels is based on the observation that predominant building response to strong earthquake motion generally occurs in one or more of the first four modes [7]. Since it is necessary to instrument only one above-ground level for every mode of response about which information is desired, instrumentation would be required only at a maximum of four levels above ground, regardless of building height. In general, it is recommended that at least two intermediate levels be instrumented in buildings having more than six stories above ground, and at least one level be instrumented in buildings having two to six stories. If the mode shapes of predominant response of the building have been determined by ambient-vibration or forced-vibration tests, however, it is probably necessary to instrument only one intermediate level (in buildings of any height). When only one intermediate level is to be instrumented, it should not coincide with a nodal point of any of the modes of predominant response, rather, it should coincide with what we shall call an "anti-node" area. Similarly, if two or more intermediate levels are to be instrumented, at least one should coincide with an "anti-node" area. The other instrumented intermediate level or levels, however, could be located at a nodal point of one or more of the modes of predominant response, and in fact, it may be desirable to do so in order to filter one or more specific modes of response out of the recorded data, and thereby facilitate data reduction and analysis. A recent optimization study has implied that the second floor (first level above ground level) is always an "anti-node" area in multistory frame buildings, regardless of their height or number of stories [8]. This level, then, is definitely a good candidate for instrumentation as the primary "anti-node" level in frame buildings. Close examination of the mode shapes in figure 4 suggests that other "anti-node" areas for buildings uniform in plan with height are located at about 20%, 40%, and 70% of the above ground building height. Nodal points for the same mode shapes are located at approximately 30%, 50%, 60%, 80%, and 90% of the above ground building height. Unless mode shapes based on a computerized model, on an ambient-vibration study, or on a forced-vibration study of a building are available, these generalized mode shapes are recommended for use in determining optimal locations for intermediate level instrumentation.

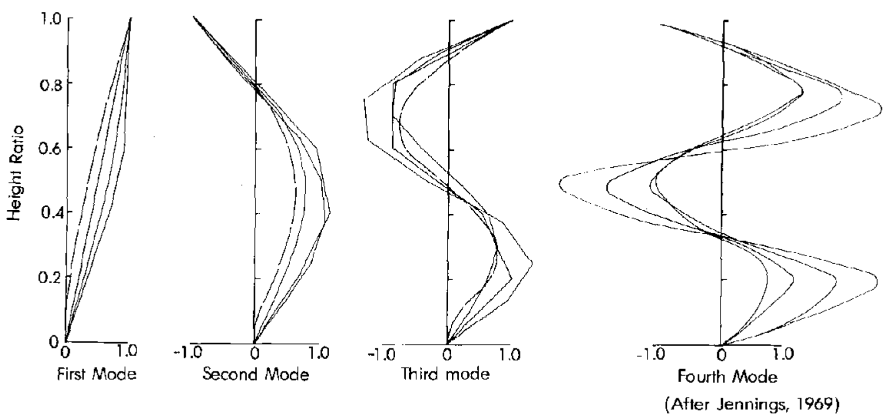


Figure 4.--Experimental and analytical mode shapes for buildings.

INSTRUMENTATION OF REINFORCED CONCRETE BUILDINGS

Below are specific recommendations on strong-motion instrumentation schemes for reinforced concrete frame buildings and reinforced concrete shear-wall buildings. The schemes are based on the assumption that ambient-vibration or forced-vibration tests have not been performed on the buildings and little is known about their dynamic characteristics.

Reinforced Concrete Shear-Wall Buildings

In general and as a minimum, it is recommended that strong-motion instrumentation be placed at the main roof level, second level, and ground level. If the building has more than six stories above ground, one additional intermediate level should be instrumented, and if there is one or more basement levels and it is likely that the motion at the two locations will be significantly different, the lowest basement should also be instrumented. Instrumentation of the second level is particularly important. In addition to the fact that the second level is always an "antinode" area, it is the level at or near which the most severe structural damage is likely to occur in most buildings. This expectation is based on the fact that earthquake loads are input at the base of the building, shear loads are greatest near the base, and the maximum inter-story displacements are likely to occur between the ground and second level. Moreover, field observations of earthquake damage strongly suggest that the onset of severe structural damage, particularly in buildings that are uniform in plan with height, is likely to occur in the vicinity of the second level. Of the five reinforced concrete frame buildings and one reinforced concrete frame and shear-wall building that were structurally damaged by the 1971 San Fernando earthquake [1, 4] all sustained their most severe structural damage at the second level or between the ground and second level (in most cases one or more beam-column joints and/or columns were damaged). Similar trends in damage were observed by the author, and others in Bucharest, Romania after the March 4, 1977 earthquake.

Shown in figure 5 is a suggested strong-motion instrumentation scheme for a typical 13-story moment-resistant reinforced concrete space frame

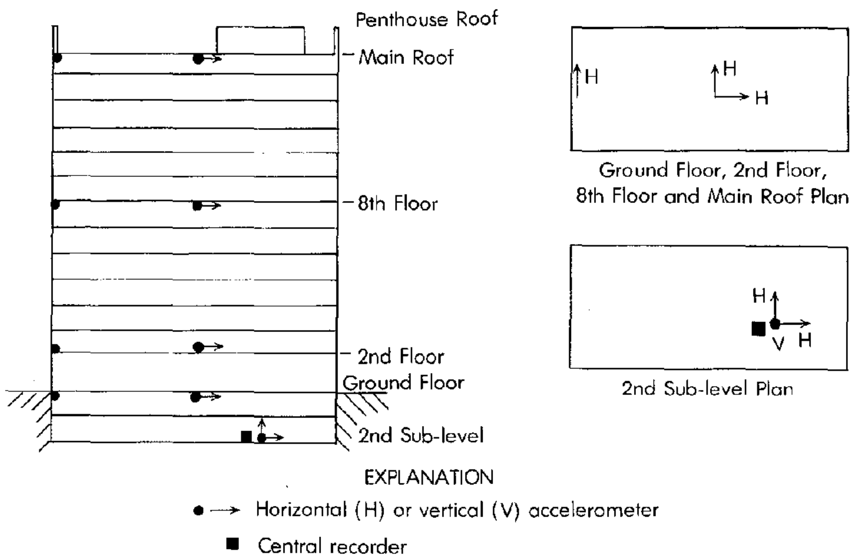


Figure 5.--Suggested strong-motion instrumentation scheme for a typical 13-story moment-resistant reinforced concrete frame building.

building with rigid concrete floor slabs. The scheme has been designed in accordance with the above guidelines and has been installed in a building in Los Angeles under the California Strong-Motion Instrumentation Program. Instrumentation is located at the main roof, eighth floor, second floor, ground floor, and lowest basement level. With the exception of the eighth floor, the reasons for selecting each level are described in the above guidelines. The eighth floor was selected for instrumentation because it is located near the mid-height of the structure, the region where second mode response of the building would be maximized and where third-mode response, which will be recorded at the roof and second floors, is not likely to appear. In essence, third-mode response is intentionally being filtered from the recorded data in order to facilitate data reduction and analysis. The fourth mode, of course, would be recorded at all three above-ground locations.

Reinforced Concrete Shear-Wall Buildings

In general and as a minimum, it is recommended that strong-motion instrumentation be placed at the main roof level, one or two intermediate levels (one level in buildings with six or less stories and two in buildings with more than six stories above ground), and on the lowest level. When only one intermediate level is to be instrumented, the instrumentation should be located at the second level, or level nearest 20%, or 40%, of the above-ground building height. When two (or more) intermediate levels are to be instrumented, the instrumentation should be located at any two (or more) of the following four: second level, or level nearest 20%, 40%, or 70%, of the above-ground building height.

Shown in figure 6 is a suggested strong-motion instrumentation scheme

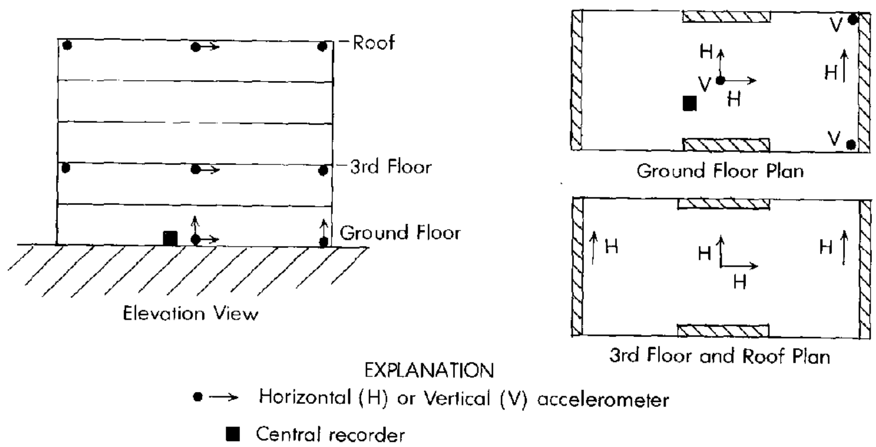


Figure 6.--Suggested strong-motion instrumentation scheme for a typical five-story reinforced concrete shear-wall building.

for a typical five-story building with reinforced concrete shear-walls in both directions along the perimeter walls. In-plane bending of the floor slabs is anticipated in the transverse direction, differential horizontal foundation motion is possible, and potential rocking motion of the end-wall shear-walls is of interest. In accordance with the above guidelines, four horizontal accelerometers have been located at the roof level and third floor (the intermediate level nearest 40% of the above-ground building height) and a triaxial accelerometer package, an isolated single-axis horizontal accelerometer, and two special-purpose vertical accelerometers have been located at the ground level. The purpose of the lone horizontal accelerometer is to obtain information on differential horizontal ground motion, whereas the purpose of the vertical accelerometers at each end of the shear-wall is to obtain rocking motion data.

ACKNOWLEDGMENTS

The author gratefully acknowledges the review comments of Drs. R. B. Matthiesen, G. N. Bycroft, and A. G. Brady of the U.S. Geological Survey, Menlo Park, CA; the information provided by Mr. T. M. Wootton of the California Division of Mines and Geology, Sacramento, CA, and Mr. H. T. Halverson of Kinemetrics, Inc., Pasadena, CA; and the financial support provided by the National Science Foundation under Inter-Agency Agreement CA-114.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

EARTHQUAKE SIMULATION IN THE LABORATORY

Mete A. Sozen
Professor of Civil Engineering
University of Illinois, Urbana

Generation of earthquake-like motions in the laboratory to shake test structures is not new [2,3,5,8,13]. However, the earthquake simulator has not yet been accepted as the ordinary and limited experimental research tool that it is. Most of those who have thought about it seem to be divided into two opposing camps [4] with essentially the same viewpoint. Some believe that total simulation can and should be achieved, with the results directly and incontrovertibly applicable to practice. Thus, the simulator becomes the total tool for which no expense is beyond contemplation. Others believe that total simulation should but cannot be achieved, concluding therefore that if the simulator cannot do all, it should do nothing.

This paper attempts to discredit the myth of the earthquake simulator as the total and final arbiter and develop a basis for its use as a laboratory tool for structural research related to reinforced concrete.

Earthquake simulation in the laboratory may be defined as the process of moving, at prescribed varying rates of speed, the base of a test structure through a displacement program representing one or several components of a representative or particular earthquake motion. The energy and control requirements for such an undertaking tend to limit the size of the test structure and the number of base motion components in a given test. This central restraint leads to a series of compromises which reduce the test structure, not necessarily in size, to a model with all the attendant doubts about its relevance to the real thing. One conclusion from this predicament is readily apparent. The simulator can simulate conveniently only a segment of the total phenomenon. Consequently, trends observed in the simulated environment may not be transferred to practice without assimilating them in terms of strain geometry, forces, and explicit force-displacement relations (as it ought to be done for static testing). This is a severe limitation. It suggests that the controlling resistance mechanisms of a complex test structure must be limited to those reasonably well understood in terms of transferable index values (such as cylinder strength), virtually limiting the reinforced concrete test structure to failures in axial load and flexure.

If the above limitation is accepted, the role of the simulator model in the range of nonlinear response becomes subservient to that of the analytical model. The simulator may be thought of simply as an experimental device to test an analytical model, for a particular combination of basic controllable variables. This definition may be considered too narrow in view of the possibility of establishing, through a series of simulator tests, the effect of a specific variable. But such an undertaking is feasible only for simple specimens, and answers which can be obtained from simple specimens can, in almost all cases, be obtained more economically without the use of the earthquake simulator. Parametric studies of any reasonable extent using the simulator become economically unreasonable for complex test structures.

Considering the use of the simulator as a tool for general structural research in reinforced concrete (as distinguished from proof testing), the following sections discuss the nature of the base motion, the design of experiments, and a sample of results obtained using the University of Illinois Earthquake Simulator. The physical attributes of various current-generation earthquake-simulator facilities are described in references [3,4,12,15,18,19].

SIMULATED EARTHQUAKE MOTION

The first and obvious question about a simulator facility is the quality of the earthquake-motion component that it reproduces. Before considering this question, it is helpful to remember that how precise the reproduction must be depends on the particular model and the phenomenon being studied. Otherwise one can go to the absurd as in the case of investing in loudspeakers which can reproduce frequencies of sound quite inaudible to the human ear.

Assuming that the main role in structural research of the earthquake simulator is to provide an experimental test of an analytical model, the critical question is not whether the system will produce an earthquake motion -- as this question becomes embroiled in the problems of defining a universal earthquake motion -- but whether it will reproduce a motion, with "typical" characteristics, being used as input for an analytical model. Contained in this question is the assumption that the "input motion" for the simulator precedes that for the analytical model because reversing this order would make the concern trivial. For the simulator to function as an experimental tool, it has to be able to reproduce a particular motion on command. Totally random motion, even if it qualified every time as an earthquake motion, would prohibit direct comparisons of the behavior of different systems.

Perfect fidelity in reproducing an input motion is certainly not undesireable. Lacking that level of attainment, this section reconsiders a procedure for judging the goodness or acceptability of the reproduced motion suggested in reference 15. The tests of goodness are limited to the relationship between the input and the reproduced motions, and do not entail questions about what constitutes an acceptable base motion.

Figure 1 shows two acceleration-time records. The "input" motion is, in effect, the desired prototype. It was modeled after the North component of the record obtained at El Centro during the Tehachapi earthquake of 1940, with the time and acceleration scales modified to suit a particular test. The "base" accelerations are those measured at the base of a ten-story test structure mounted on the platform of the simulator.

The acceleration ordinates for the base are the actual measured values. The input accelerations were normalized so that the spectrum intensity calculated for the input record would be equal to that for the measured record. (The magnitudes of the input accelerations are significant only in relation to each other. The absolute accelerations on the platform are controlled by the amplification system of the simulator, ideally without distortion.) The time (horizontal) scale was modified by a factor of 2.5 (platform time was 2.5 as fast as real time) in order to excite the small-scale structural model.

A visual comparison of the two records will reveal that the reproduction was not perfect. Besides developing a high-frequency low-amplitude noise toward the end, the acceleration record measured on the test platform does not exhibit the same relative magnitudes of consecutive peaks as the input record.

As the second broad-brush test of goodness, Fig. 3 compares the acceleration and displacement spectra, at damping factors of 0.02, 0.05, 0.10, and 0.20, for the input and measured acceleration records. It should be noted that both acceleration and displacement values have been plotted to an arithmetic scale. The abscissas are given as frequencies to 10 Hz and as periods from 0.1 sec. in order to expand the frequency range of interest in the model environment. The logarithmic plot (Fig. 2) is not of as convenient use in comparing test motions as the arithmetic plot because of the distortion of the magnitude scales.

In relation to the test of a reinforced concrete test structure excited into the nonlinear range of response, it is more meaningful to evaluate the acceptability of the reproduced motion by comparing response spectra calculated for a damping factor of 0.10, because the equivalent damping factor for the test structure is usually in that range.

The fact that the calculated responses to the input and reproduced motion are in general of comparable magnitude is not quite significant. The input acceleration record was normalized on the basis of the spectrum intensity. However, the fact that the shapes of the curves indicating response at a particular damping factor are in conformity is significant and positive. The acceleration response (damping factor = 0.10) is reasonably flat from 25 Hz to 0.25 sec (4 Hz) for both the input and reproduced motions. Acceptable agreement between the two motions may also be inferred from comparison of the displacement responses.

Because the response spectra provide no information on the sequence of events, agreement of the linear response spectra for the input and reproduced motions is essential but not sufficient for judging the acceptability of the reproduced motion. A convenient measure of the goodness of the motion with respect to sequence as well as content of frequency components of the reproduced motion may be provided by comparing the responses of a number of SDOF oscillators having natural frequencies covering the range of frequencies in which the model to be tested is expected to respond.

As an example, compare the response histories in Fig. 4, calculated for the first six seconds of the two motions in Fig. 1, of three linearly elastic SDOF oscilators having natural frequencies of 8.0, 4.0, and 2.0 Hz with damping factors of 0.02 and 0.10. Although a quantitative index value could be devised, it is preferable to conduct the comparison visually if only to force the test designer to develop a better perspective.

Comparing the pertinent response histories, it is seen that (a) the response maxima occur at comparable times for the input and reproduced motions in all cases and (b) the waveforms are reasonably close. Least favorable comparisons of the waveforms occur at the two extremes of the frequency-damping plane, at 8 Hz for a damping factor of 0.02 and at 2 Hz for a damping factor of 0.10.

On the basis of the comparisons included in Fig. 4, it may be inferred that if the desired responses of the test structure are dominated by frequencies in approximately the range 8 to 2 Hz, the reproduced motion is satisfactory even if it is not a perfect copy of the input motion. The inference could be strengthened by using a complex analytical model of the test structure rather than a collection of linear oscillators, but such an approach is expensive, can be done properly only after the fact, and invites a question as to the necessity of the testing effort if the analytical model is that reliable.

TEST STRUCTURE

Foregoing the use of earthquake simulation as a "proof test" frees experimental design from the rather rigid requirements of similitude which can get to be virtually impossible to satisfy completely in the case of nonlinear response of small-scale test structures. The experiment is designed to test a theoretical construct rather than a particular structure. Used in this context, the simulator study is comparable to computer simulation and is susceptible to similar pitfalls of distortion in the construction or loading of the model.

An important consideration in designing the test structure is that what should be modeled is the structural response rather than the geometry of the structure. Although it is an added asset from the viewpoint of disseminating the results, visual similarity is a negotiable requirement.

Within the current state of the art, the basic principle for proportioning structural elements is that the response of the reinforced concrete model be dominated by internal stresses parallel to the axes of the elements, because the interrelationship among axial load, bending moment, and rotation can be related to material properties through reasonably explicit procedures. Even within those bounds, conclusions from studies of test results may be crippled if observed phenomena are dependent on effects difficult to rationalize, such as the effect of lateral confinement of the concrete by model transverse reinforcement.

Given the overall objective of establishing the nonlinear response characteristics of a structural system or concept to a particular type of earthquake motion, the design of the experiment follows certain general restraints created by (a) the characteristics of the material, (b) dimensional minima related to fabrication requirements, and (c) the operating limits of the simulator. Because the "earthquake" can be tailored to fit the structure as well as the structure can be proportioned for the earthquake, the freedoms may often outweigh the restraints. The optimum experiment is more of a function of the traditions and data handling capabilities of the particular laboratory than of the physical restraints.

Material

Whatever the size of the test structure, there is little incentive or justification for using a material other than some form of reinforced concrete. If the quest is learning about the nonlinear response of reinforced concrete, ordinary or small-scale reinforced concrete is the most efficient way of obtaining elements with the desired moment-rotation characteristics.

Independently of the size of the concrete and the reinforcing bars used, it is inadvisable to plan experiments which will be influenced by shear or bond failures or distribution of flexural cracks. Experimental study of such phenomena require extensive replication and ought to be handled using static loading and specimens as close to "full scale" as possible. The possibility of, for instance, a shear failure occurring as a result of dynamic loading -- from an increase in the yield stress of the longitudinal reinforcement -- can be avoided with trivial increase in cost of construction in the realm of typical strain rates associated with earthquake response.

If the size of the test structure demands the use of small-scale concrete, then it is essential that bond and shear stresses do not have major influence on the observed behavior. In relation to this point, it should be mentioned that reinforcing details, of which effects are not to be investigated in small scale, need not conform to those in full scale structures. For convenience, transverse reinforcement may be provided by a continuous "spiral" and anchorage by mechanical attachements.

If the stiffness of the system at steel stresses well below yielding is critical for the test results, small scale structures should be used only after exploratory experimental work with static loads demonstrates that the flexural-crack formation and distribution are comparable to those in full scale elements.

Initial Stiffness and Weight

The choices of initial stiffness (or dimensions and reinforcement) and weight of the structure are governed by the fabrication limitations of the test structure including the added masses, the necessity to maintain the critical strain rates in the reinforcement within an acceptable range [14,16], and the operating limites of the simulator.

The typical operating limits in one direction of a simulator system may be illustrated in terms of the variation of maximum acceleration with frequency of motion as shown qualitatively in Fig. 5. Quantities are used for the x-axis to signal the distortion of the scale as well as to establish a correspondence between this plot and the response acceleration plots.

The bound represented by line A refers to the acceleration limit and would be likely to be controlled by the ratio of the moving effective mass to the capacity of the hydraulic ram or the overturning moment on the test platform even though the system may have a limit of acceleration independent of the load on it.

Curves B and C refer to the velocity and displacement limits of the system interpreted in terms of acceleration.

The bounds of a particular simulated earthquake motion may be represented by the solid curve identified by the numeral (1). The location of these bounds in Fig. 5 below the limiting bounds A, B, and C depend on choices made by the experiment designer. Curve 1 can be shifted to the left in Fig. 5 by increasing the ratio of "real time to platform time" (by compressing uniformly the time axis of the input record), or it can be shifted to the right by decreasing the time ratio. Also, the amplitudes of curve (1) may be increased by amplifying the input accelerations. In effect the experiment designer has a certain amount of freedom to "adjust" the earthquake to fit the demands of a particular specimen.

The effective frequencies of the test structure, which can be controlled by selecting dimensions, floor masses and reinforcement, should have a relationship to the bounds of the simulated earthquake motion comparable to the relationship of the effective frequencies of the full scale structure (or class of structure) to the bounds of the full scale earthquake motion. This condition can be satisfied by modifying either the test structure or the simulated motion, or both.

For example, assume that the initial design of a ten-story frame model to be tested using a motion with the bounds represented by curve (1) resulted in a calculated natural frequency of 10 Hz. Typically, the lowest mode frequency of such a structure would be expected to be in the nearly-constant-velocity range of the spectral response curve (which may be related to the base-motion bounds illustrated in Fig. 5 by an amplification constant).

To satisfy that condition, the test structure or the test motion has to be changed. If convenient, the masses on the test structure may be increased or the effective stiffness may be decreased to reduce the frequency. Alternately, the time ratio of the input acceleration record may be decreased to develop the base-motion bounds represented by the broken curve (2) in Fig. 5. If the response to this motion is not sufficiently high to damage the test structure, the accelerations may be amplified to develop the bounds indicated by curve (3).

Given the general types of the test structure and the base motion, the pivotal criterion in the design of the test is the relationship of the effective frequencies of the test structure to the test motion as outlined above. Additional restraints are set by the strain rates and the necessity to prevent large net tensile forces at the base of the test structure.

EARTHQUAKE SIMULATOR STUDIES OF REINFORCED CONCRETE STRUCTURES

Although there are several laboratories with earthquake simulation-facilities in the United States, to date only two have reported experiments of reinforced concrete structural systems with a view to structural research.

Figure 6 shows the two-story reinforced concrete frame tested at the Richmond testing facility of the University of California by Hidalgo and Clough [7]. There was virtually no "modeling" involved for this test except that only one horizontal component of the prototype acceleration record was used for the base motion (the time scale was not changed). The frame was "half-scale" but the dimensions and the materials used would qualify the structure as a "full scale" or, at least, an unwarped model, except for the transducers, spliced into the columns at midheight, which could have been left out and which would not be expected to have had critical influence on the failure conditions of the columns. Any failure phenomenon observed in such a structure could be analyzed directly and on the basis of phenomenological approaches if necessary. For example, forces corresponding to shear failure of the columns (had the columns failed in shear) or bond failure of another element could have been normalized in terms of pertinent parameters and applied directly in design. Obviously that strategem would have required a large number of tests which would have been costly. The investigators did not choose such an approach. Rather, they used the experimental results primarily as tests of the analytical model which would then serve as a vehicle for generalizing the observations. Even with a large-capacity simulator, the optimum use of the system is again as a device to provide physical tests of analytical models.

The University of Illinois simulator has been used, because of its limitations, almost exclusively for testing of planar small scale models. A summary of the types of structures tested is provided in Fig. 7. A sample of results is shown in Fig. 8 and discussed below.

The acceleration records in Fig. 8 were obtained during an initial test run of the ten-story wall shown in Fig. 7 e, described in detail in reference 1. The first record indicates the base accelerations. The succeeding ones describe the acceleration-time histories measured at each of the ten levels.

It will be noted that the acceleration record is relatively clean of second-mode components at level eight which level happened to be very close to the node point for the second mode considering the structure as a linear system. For the purpose of this discussion, it is not important what this observation indicates. The observation itself is important. Despite all the constraints of the experiment, the observation provides an insight into the behavior of a complex reinforced concrete structure responding in the nonlinear range. To obtain data of this type within a reasonable time span, the simulator is essential.

CONCLUDING REMARKS

Although attempts at earthquake simulation in the laboratory started as early as at the turn of the century, the simulator has become a convenient laboratory tool for structural research only within the last two decades because of its dependence on advanced technology in electronic controls, data acquisition, and data management.

In this observer's view, the role of the earthquake simulator in research is narrower than but very much like that of the digital computer. The simulation expands the possibilities of what can be studied without the necessity of a large number of buildings having to undergo a large number of earthquakes but it can be as deceptive as it can be revealing.

In studying the response of building structure systems in reinforced concrete, the admissibility of concentrating the masses at discrete levels and the conjecture that energy dissipation is dominated by hysteresis in the nonlinear range (which can be simulated satisfactorily in small or large scale models) rationalize experiments outside the limits set by conditions of similitude. Those phenomena which can be understood explicitly on the test platform may be transferred to the real world with a reasonable level of confidence. This approach has its own restraints and, in effect, makes the earthquake simulator simply a device to provide physical tests of an analytical model. In this role, the simulator becomes an irreplaceable technique in improving knowledge on response of complex structures. Certainly it should not be necessary to await a major earthquake to have a physical check on whether the distribution of lateral strength, indicated by a proposed method of analysis, over the height of a building with an irregular mass or stiffness arrangement is satisfactory.

Using the earthquake simulator in the spirit of a proof test is plausible in the case of, say, industrialized buildings or particular standard types of construction. In that case, the simulation must be complete, a requirement which unfortunately cannot yet be met by any facility in the United States.

The use of the earthquake simulator in studying the behavior of reinforced concrete structures is still in its primitive stage. Considering that it takes five to ten years to develop a simulator facility (development includes more than the assembly of the physical devices) and accepting that it takes a critical mass of at least half a dozen laboratories producing results to enrich and define a research technique, it may be another decade before the proper function of the earthquake simulator is established. However, there is no doubt even now about the fact that earthquake simulation does and will fulfill an important supporting role in structural research.

ACKNOWLEDGMENT

The work at the University of Illinois to which this paper refers was supported through a series of grants starting in 1965 by the National Science Foundation. The work is currently supported by NSF Grant ENV 76-22252.

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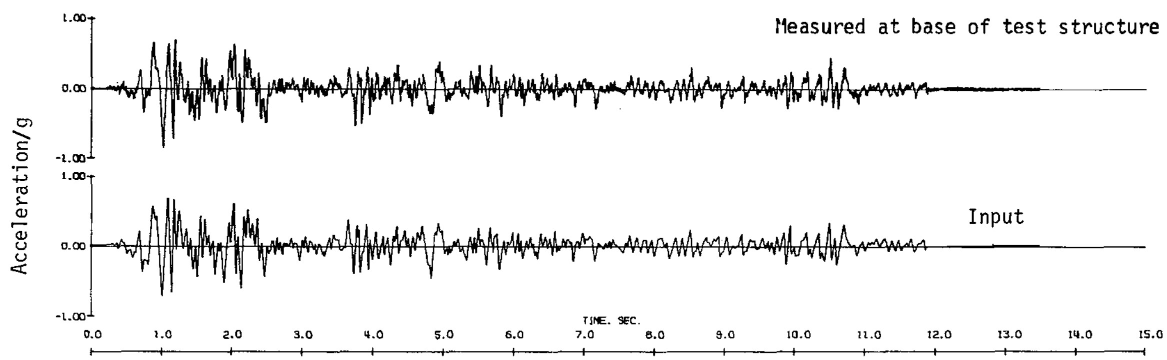
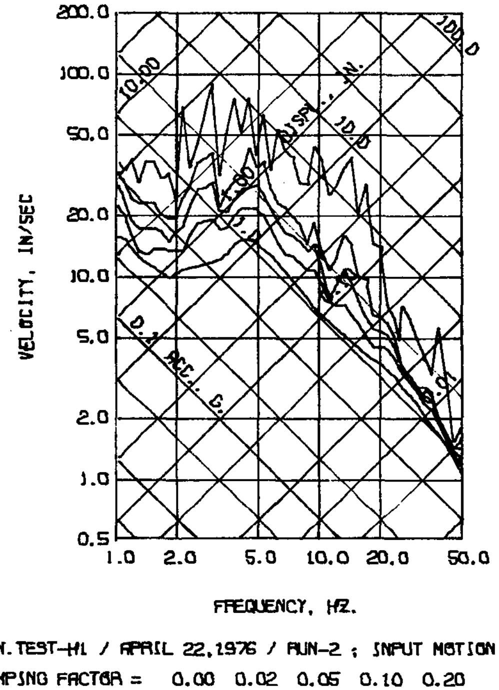
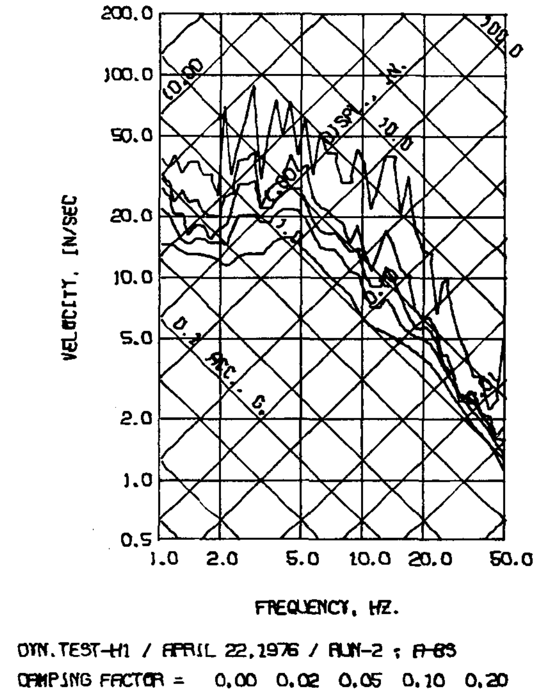


Fig. 1. Acceleration-Time Records

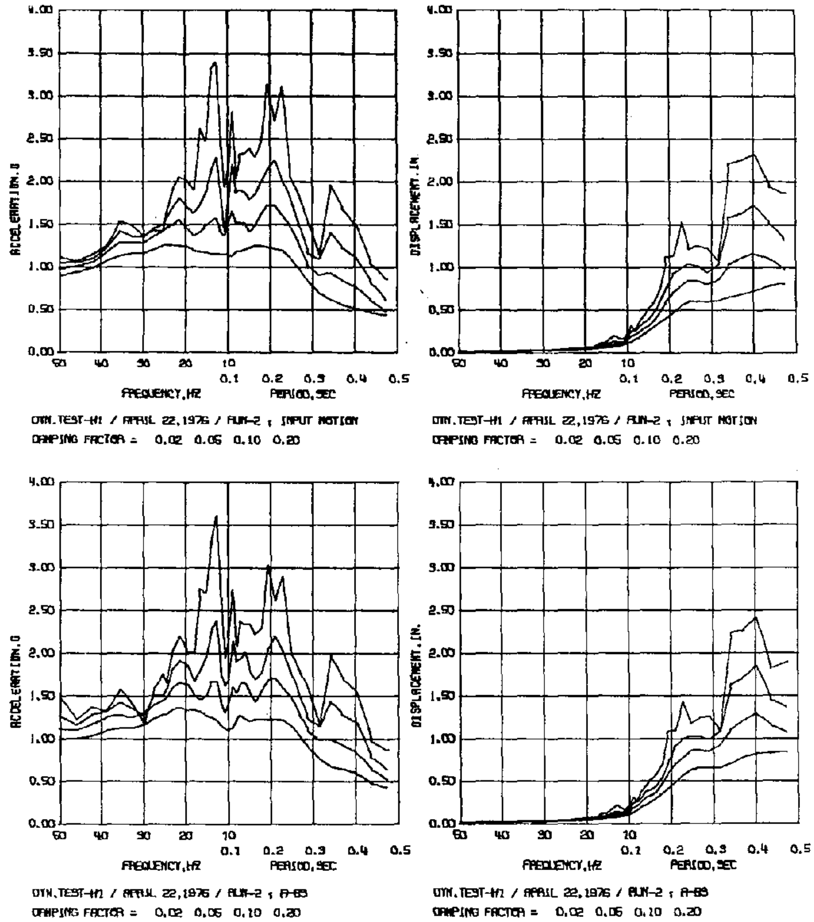


(a) Response to Input Motion



(b) Response to Base Motion

Fig. 2. Response Spectra Plotted to Logarithmic Scale



(a) Response to Input Motion

(b) Response to Base Motion

Fig. 3. Response Spectra

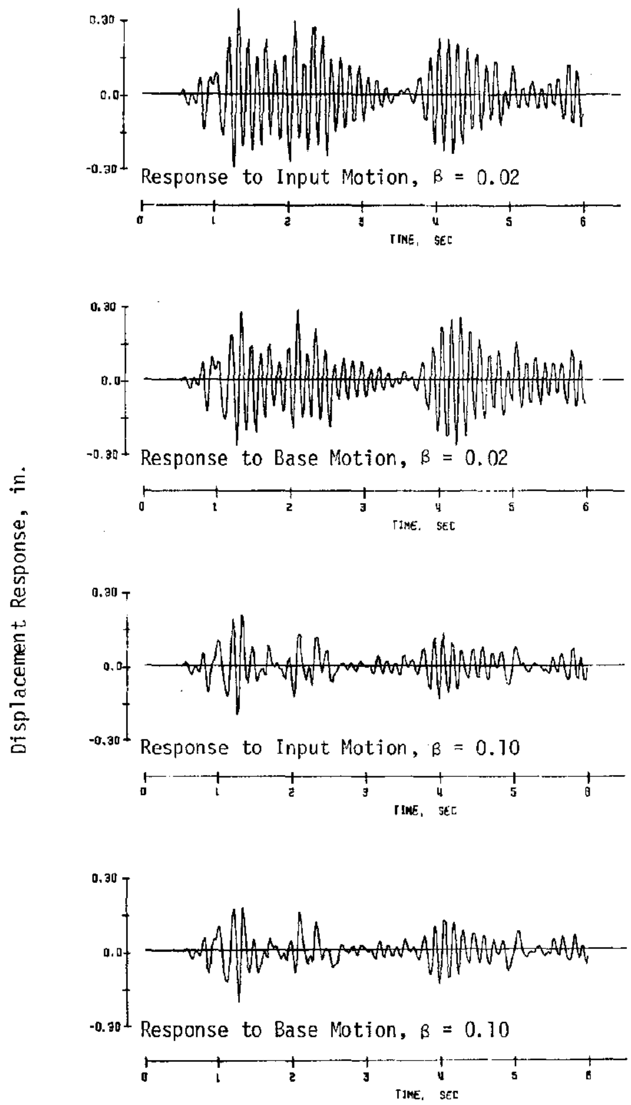


Fig. 4a. Calculated Displacement Response, f = 9.0 Hz

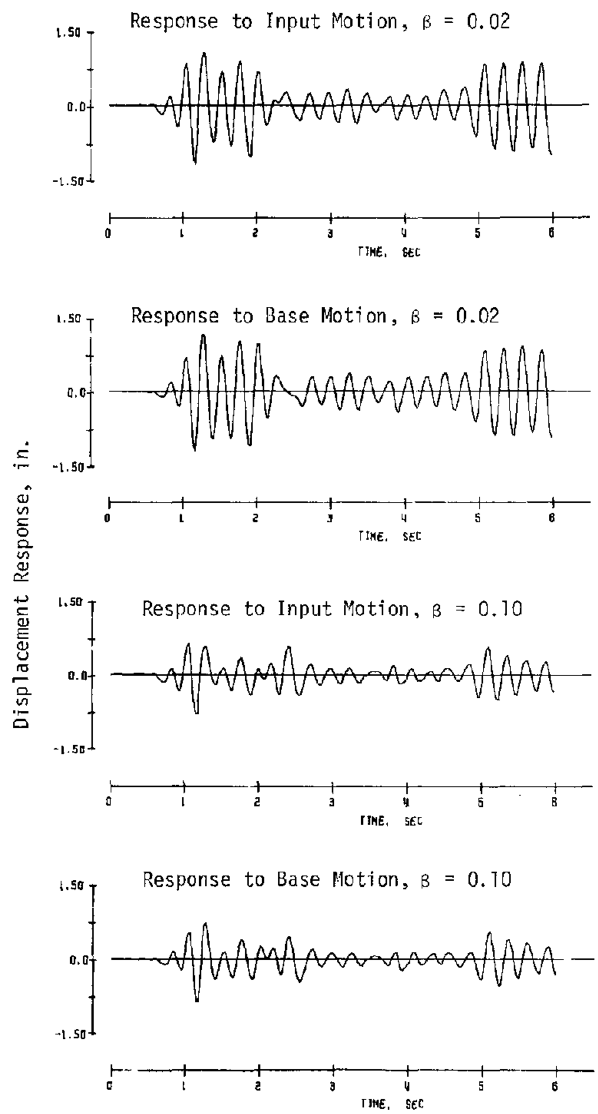


Fig. 4b. Calculated Displacement Response, f = 4.0 Hz

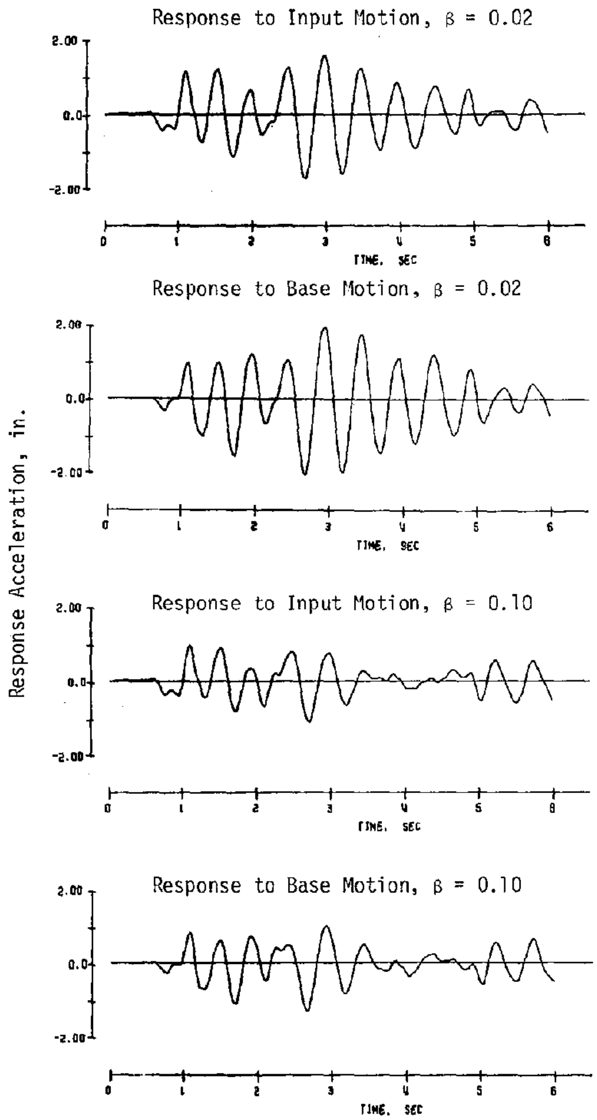


Fig. 4c. Calculated Displacement Response, f = 2 Hz

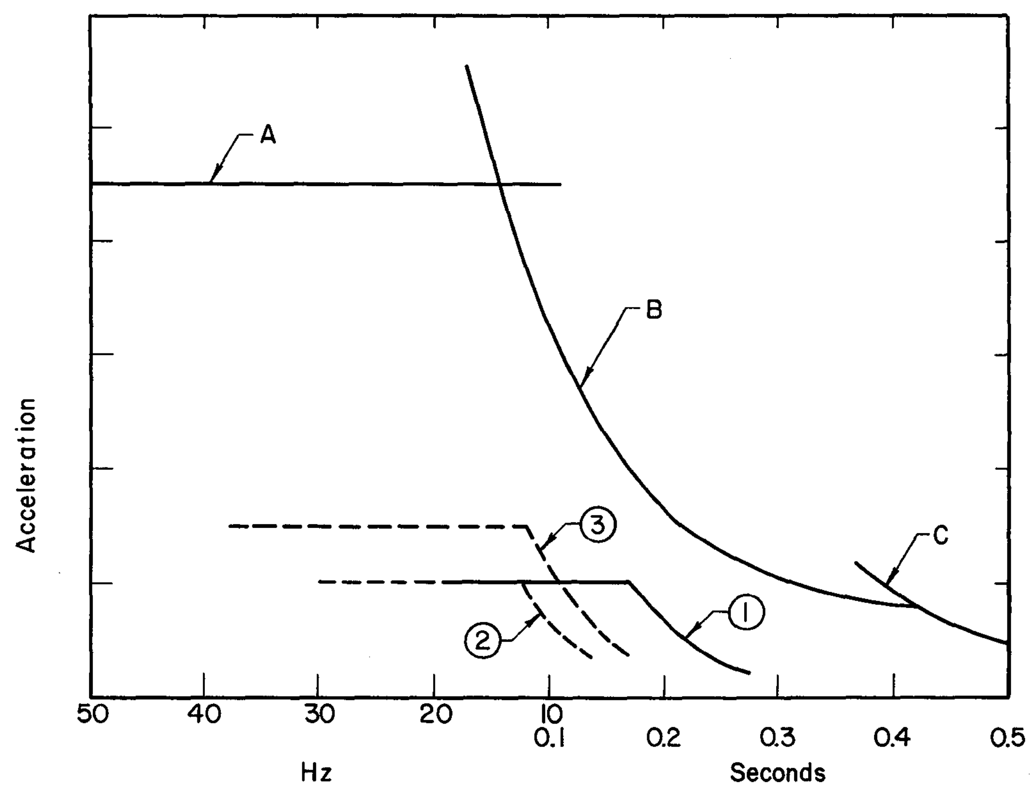


Fig. 5. Operating Limits of an Earthquake Simulator (Qualitative)

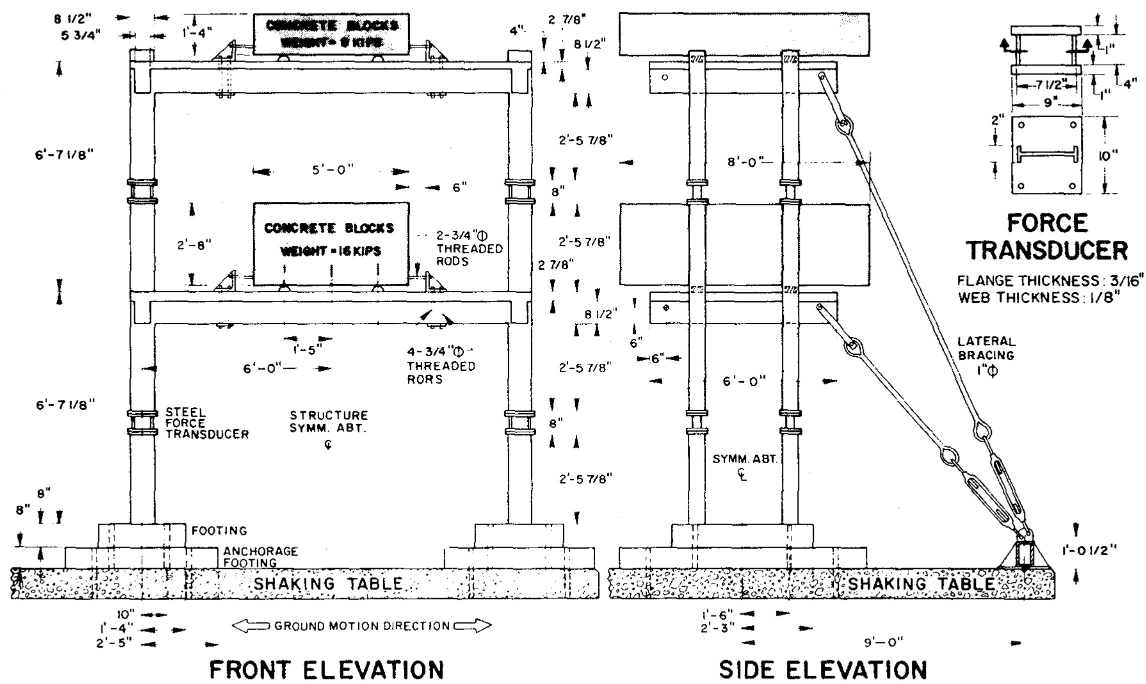


Fig. 6. Two-Story Reinforced Concrete Frame Tested by Hidalgo & Clough

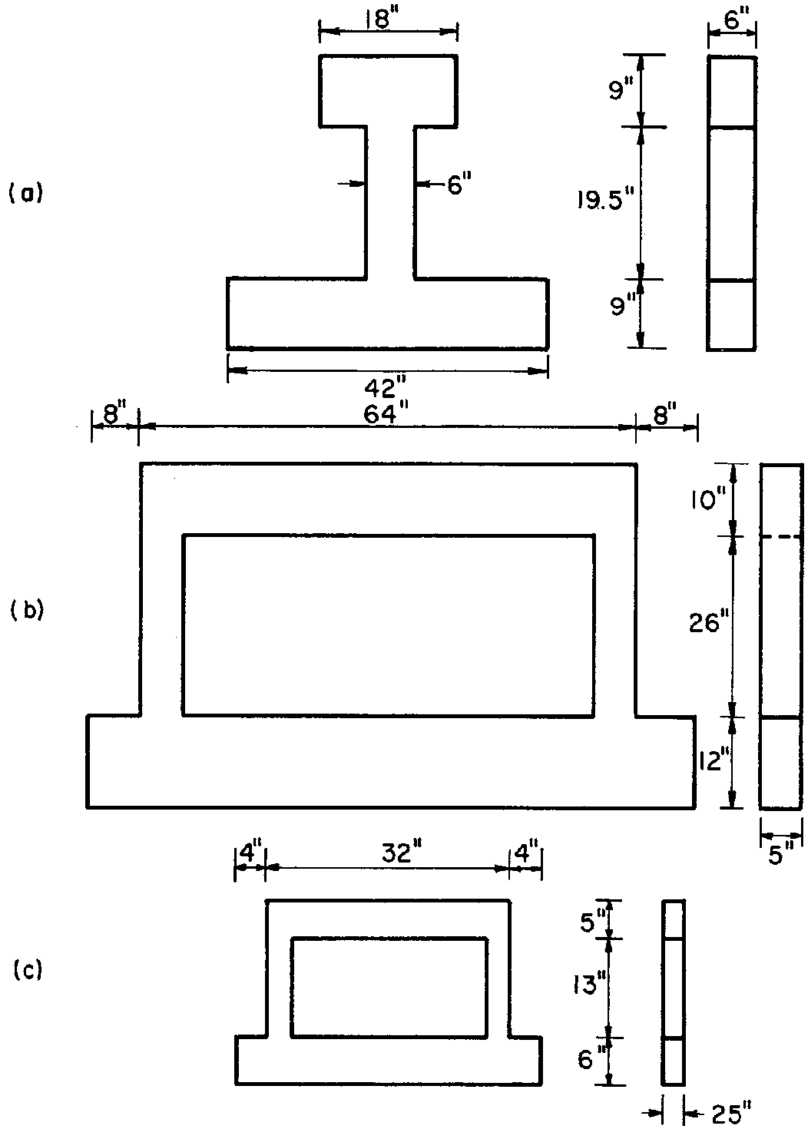


Fig. 7. Types of Small-Scale Elements and Structures Tested Using the University of Illinois Earthquake Simulator

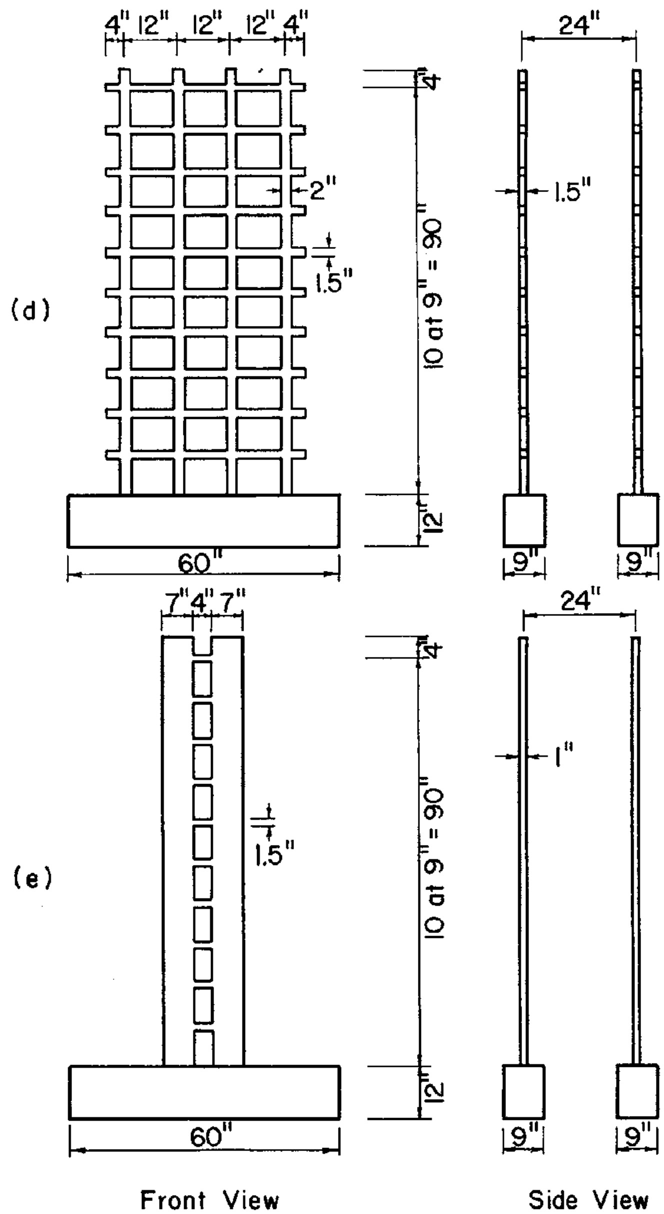


Fig. 7. Continued

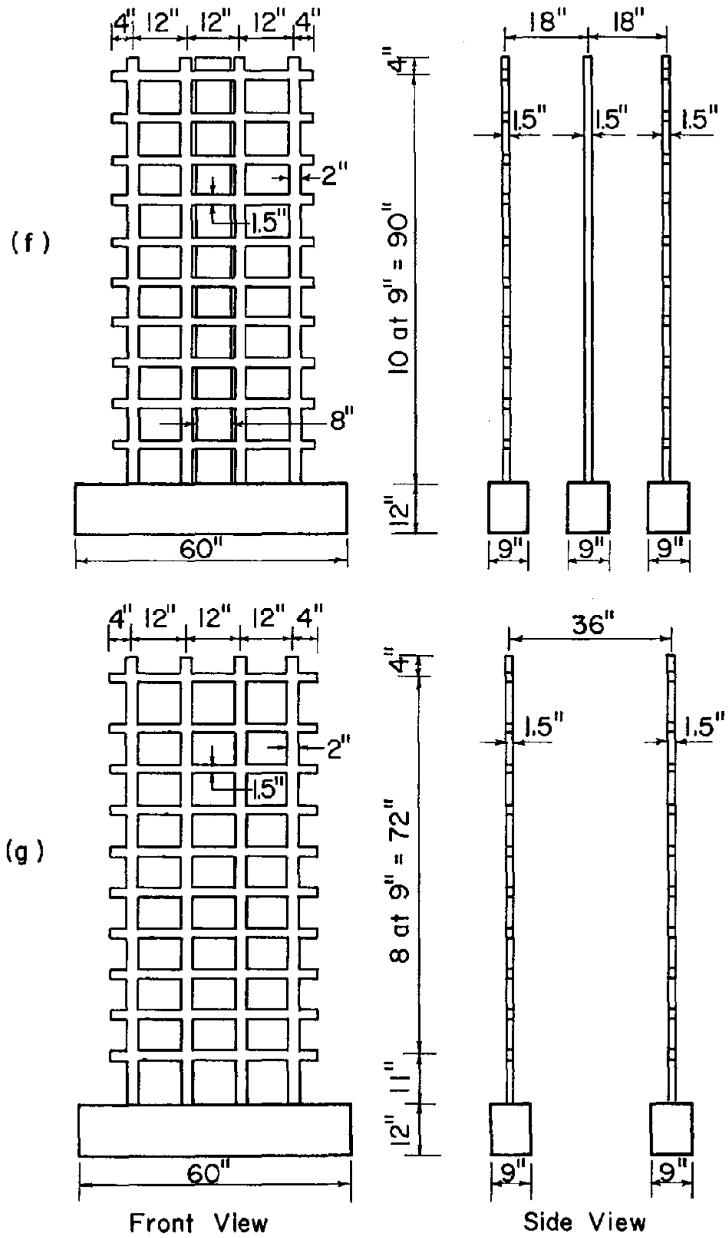


Fig. 7. Continued

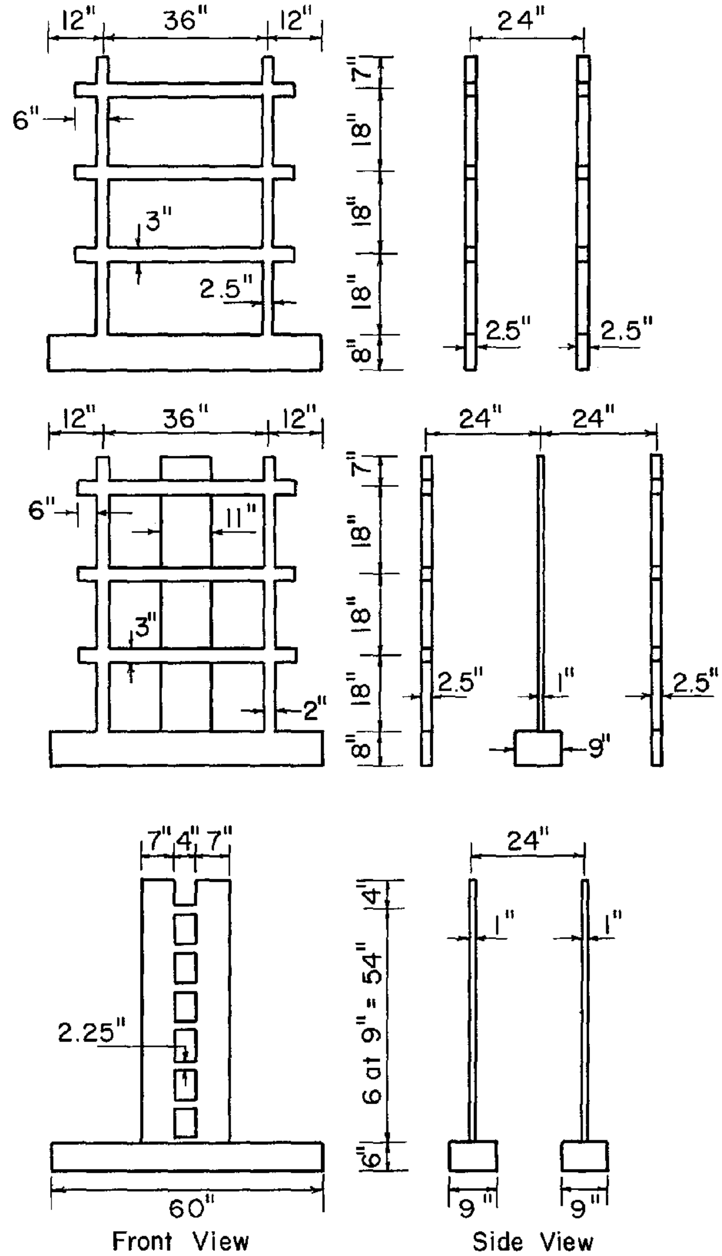


Fig. 7. Continued

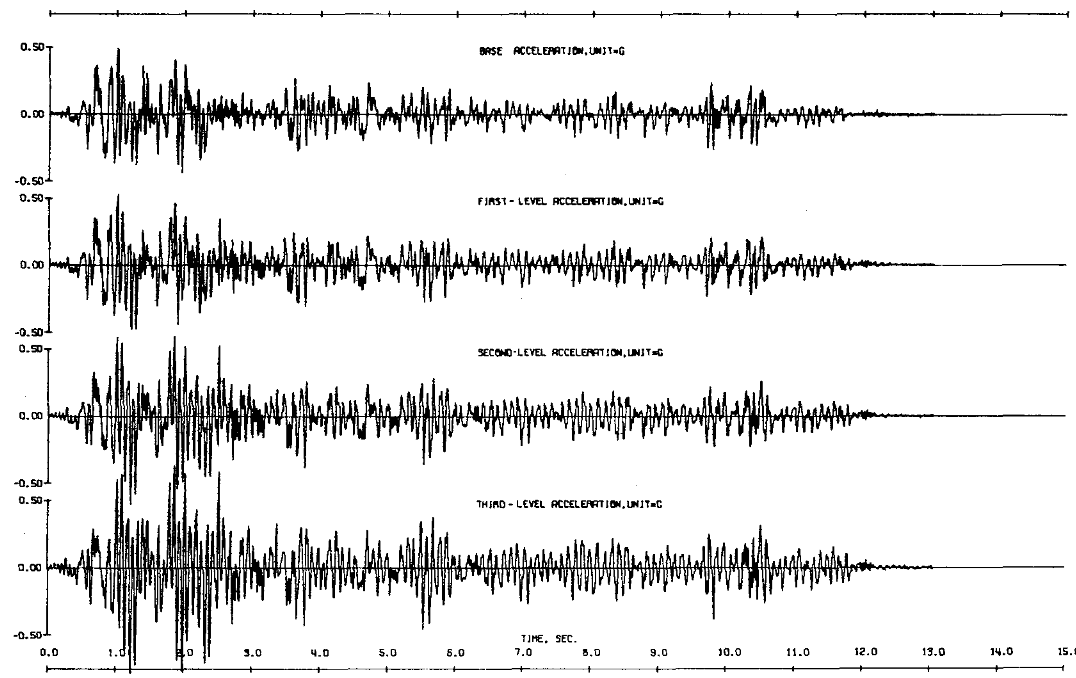


Fig. 8a. Acceleration Response Measured at Various Levels of a Ten-Story Test Structure [After Reference 1]

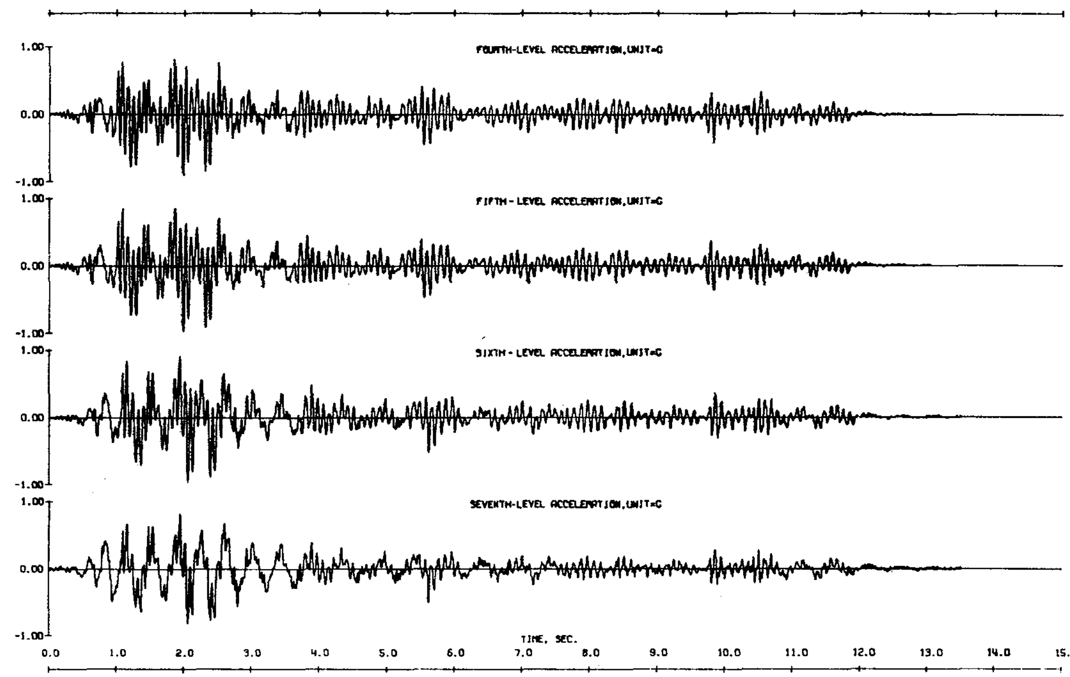


Fig. 8b. Acceleration Response Measured at Various Levels of a Ten-Story Test Structure [After Reference 1]

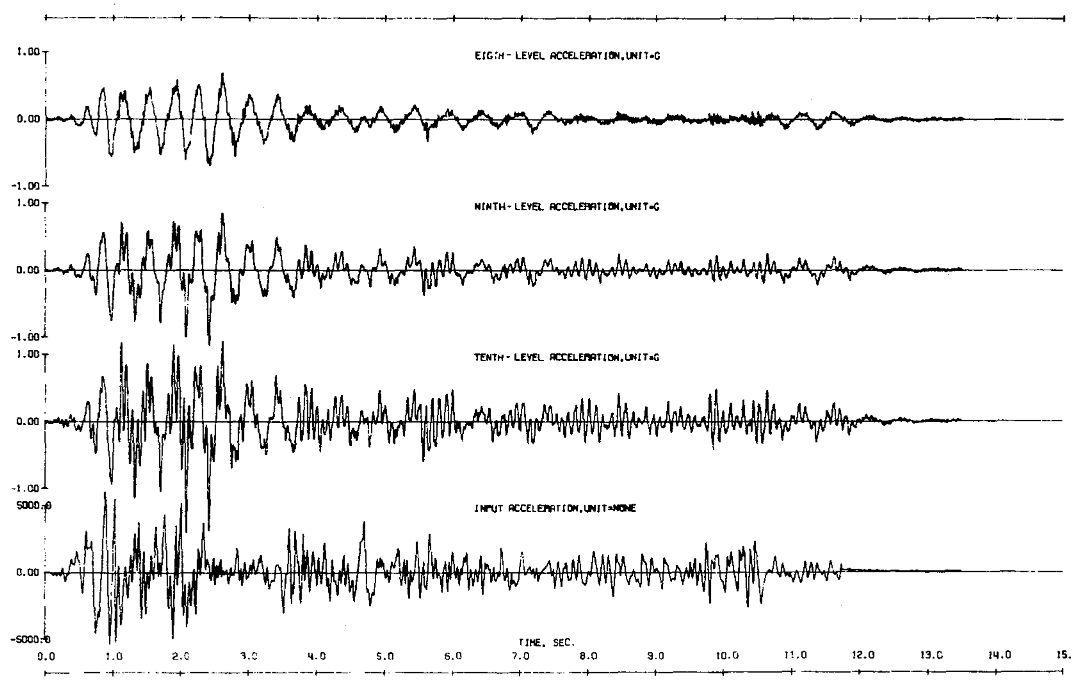


Fig. 8c. Acceleration Response Measured at Various Levels of a Ten-Story Test Structure [After Reference 1]

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

THE EXPERIMENTAL INVESTIGATION ON ERRCBC WITH EMPHASIS ON THE USE OF EARTHQUAKE RESPONSE SIMULATORS IN JAPAN

by

Tsuneo OKADA

Associate Professor
Institute of Industrial Science
University of Tokyo, Tokyo, Japan

INTRODUCTION

The importance of the dynamic testing on reinforced concrete building constructions for developing more rational earthquake resistant design method has been widely recognized and a number of experimental investigations have been carried out especially in the last decade. The useful data have been obtained, however, the incomplete problems still remain to apply the results to practice. The objective of this report is to review the experimental investigations on the Earthquake Resistant Reinforced Concrete Building Constructions, with emphasis on the use of earthquake response simulators*, carried out mostly in the last decade in Japan and to point out the specific problems that need to be solved in the future.

METHODOLOGY FOR EXPERIMENTAL SIMULATION OF EARTHQUAKE RESPONSE

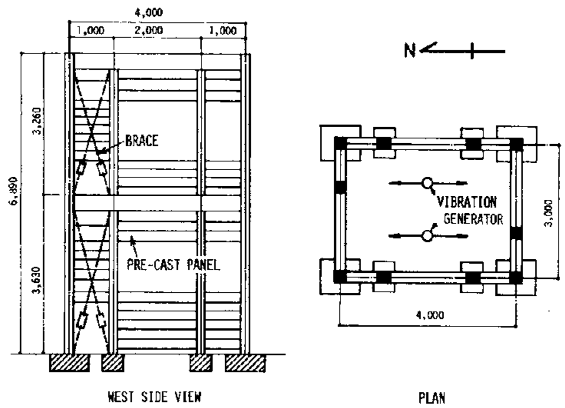
In reviewing the development of the experimental investigation on ERRCBC in Japan, the research work done by the specific Committee, called TSI, organized in Architectural Institute of Japan in the late 1940's should be referred first, while it was accomplished nearly thirty years ago[1]. Because the experimental investigations in Japan have been much influenced by the Committee's work.

The Committee was established in 1948 to develop the method for evaluating seismic capacity of existing buildings and the report was published on the experimental studies associated with the analysis in 1950. The report consisted of two parts. In the first part, the experimental techniques to evaluate the seismic capacity of building constructions and the basic theory to analyze the test data were discussed including the law of similarity between the prototype building and the building model. The recommended methods for testing are;

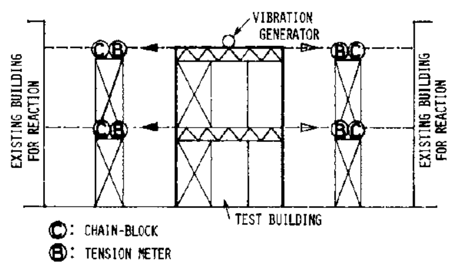
1) Pseudo-dynamic loading test
2) Dynamic tests
   a) Vibration table test
   b) Vibration generator test, and
   c) Artificial ground motion test

In the second part, the test results were described. Two full size reinforced concrete buildings and a half scale building model were tested by the pseudo-dynamic and/or dynamic loadings(Figs.1,2 and 3). Besides them, one-tenth scale model of reinforced concrete walled type apartment house was tested on the vibration table. The examples of the loading system are shown in Figs.1 and 3. The lateral load was applied by the chain blocks and alternated. The dynamic load was applied by the unbalanced mass type of vibration generator

* Tools to simulate experimentally the earthquake response of structures.



(a) Section and Plan



(b) Loading System

Fig.1 Test on Precast Reinforced Concrete Building [1]

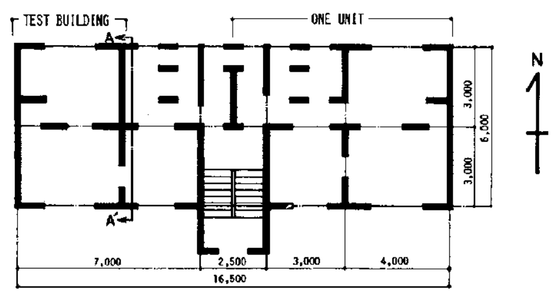


Fig.2 (a) Three-story Reinforced Concrete Block Apartment House
(A part of the building was tested [1])

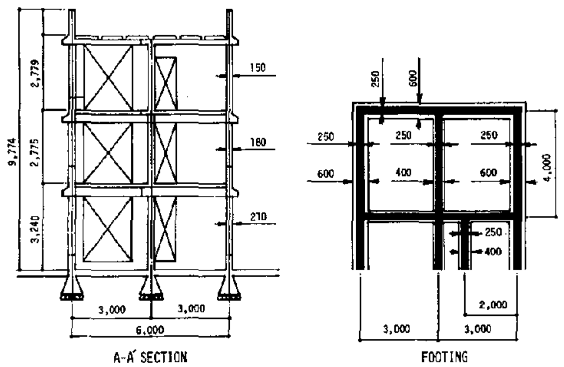
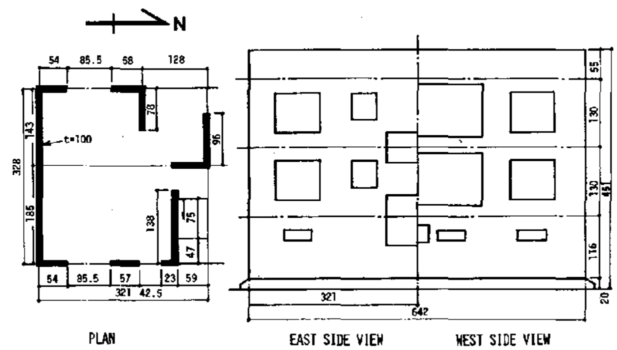


Fig.2 (b) Test Building-- A part of the building in Fig.2(a) [1]



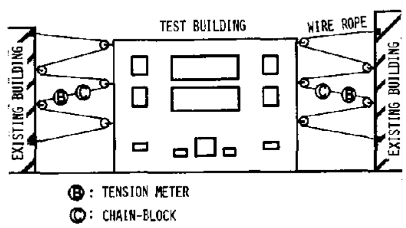


Fig.3 Test on A Half Scale Model of Three-story Reinforced Concrete Walled Type Apartment House [1]

driven by an electromotor. All test buildings were loaded until about twice of the design lateral force which was 0.2 in terms of the base shear coefficient. Although the testings were stopped before collapse stage due to the limitation of the capacity of the loading system, some of them reached nearly at the collapse stage. It should be worth to be mentioned (1) that the methodology for seismic testing on full size buildings was developed thirty years ago and (2) that it was applied to the building models including the real buildings to evaluate their seismic capacities.

The experimental techniques applied to the simulation of earthquake response of reinforced concrete buildings and/or building models after the TSI Committee's work including the last decade in Japan are;

a) Shaking Test on Earthquake Simulator
b) Shaking Test by Vibration Generator, and
c) Simulation by Computer-Actuator On-line System

A significant progress in this field is an electro-hydraulic actuator has become available for the power of the shaking table as well as the pseudo-dynamic loading test. In this paper hereafter, the shaking table driven by the electro-hydraulic actuator is defined as the earthquake simulator for its high reliability to simulate the command earthquake ground motion, while the traditional type of table driven by an electromotor is called the vibration table.

SHAKING TESTS ON EARTHQUAKE SIMULATORS

There may be no doubt in theory that a shaking test on the earthquake simulator is the most reliable technique to investigate experimentally a response of buildings to a simulated earthquake ground motion. However, fewer shaking tests on reinforced concrete buildings or building models have been carried out than the pseudo-dynamic loading tests, while more than twenty earthquake simulators have been installed in the last decade. It seems to the author that the main reasons are;

1) A shaking test costs much time, care and money, and
2) A shaking model is usually very small due to a limitation of the driving power that makes difficult to satisfy the law of similarity between a prototype and the shaking model.

In other words, it may have been considered that a shaking test has lower benefit-to-cost ratio than a pseudo-dynamic loading test and/or a computer analysis. Indeed, a numerous number of cyclic or alternate loading tests on reinforced concrete members and frames including the existing buildings have been performed in Japan, which is called the pseudo-dynamic loading test in this paper. Computer analyses using the analytical models derived from the pseudo-dynamic loading tests have been also done and it appears to the author that the combination of the pseudo-dynamic loading test and the computer analysis has been considered an optimum choice to simulate the earthquake response of the reinforced concrete building constructions. However, one of the limitations of this technique is due to the difficulty to derive a general hysteretic rule to represent the real nonlinear characteristics from the test data obtained under a specific loading path. Because a nonlinear characteristics of reinforced concrete structure is very sensitive to the loading path. Another limitation is that the effect of strain rate can not be considered in the pseudo-dynamic loading test.

Consequently, to investigate the real behavior of the reinforced concrete building constructions during earhtquake and to develop more realistic analytical model, more efficient use of the earthquake simulator should be considered. Among the shaking tests on the reinforced concrete building models in Japan [2,3,4,5,6,7,8], two successful examples done at the Tohoku University and at the Technical Research Institute of the Obayashi-gumi Ltd. are referred here.

Shaking Test at The Tohoku University [2]

Professors T.Shiga, A.Shibata, J.Ogawa and their colleagues performed the shaking test on the reinforced concrete frame models in the late 1960's. They tested one-story one bay frame models on the vibration table driven by the unbalanced mass type of the vibration generator (Fig.4). The shaking table of 5.2 m x 4.2 m was supported on the flexible steel plates and was excited by the generator fixed on the table. The size of the model structures were about 1/3 of the real frames and they were designed so as to fail in bending at the column ends. The emphasis was laid upon investigating the hysteretic loop during the vibration beyond the yielding stage. The stiffness degradation and the equivalent viscous damping ratio were estimated and the analytical models; Degrading Stiffness Model (DS Model) shown in Fig.5 and Cubic Model, were developed. The rule of the DS Model is;

1) The enveloping curve is assumed as elasto-plastic,
2) An equivalent stiffness at a certain deflection is defined as a secant modulas on the elasto-plastic enveloping curve, and
3) An equivalent viscous damping ratio is used to consider the hysteretic damping which is determined from the maximum experienced deflection. The suggested equivalent viscous damping ratio to the critical damping corresponding to the maximum experienced story displacement by the story height (R) is; 1 % to R=0.5/1000, 3% to R=2/1000, 8% to R=20/1000 and 13% to R=30/1000.

It should be noted that such useful data were obtained by the use of the simple vibration table and the work would become the basis of the development of the substitute structure method proposed by Professor A.Shibata in cooperation with Professor M.A.Sozen at the University of Illinois, Urbana-Champaign in 1974 [9].

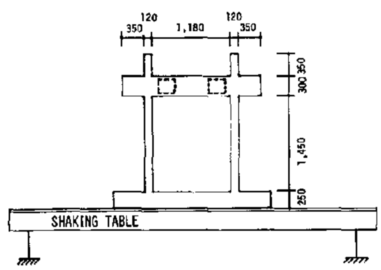


Fig.4 Shaking Test at Tohoku University[2]

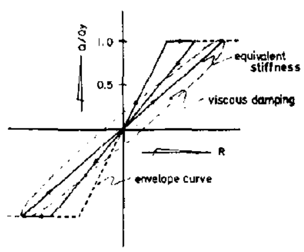


Fig.5 Degrading Stiffness Model[2]

Shaking Tests at The Obayashi-gumi Ltd. [3,4]

A couple of the shaking tests on reinforced concrete building models were carried out by Dr.T.Takeda and his colleagues. The first test series was a shaking test by the impact loading and the second was a shaking test on the earthquake simulator.

Impact Loading Test -- The reinforced concrete three dimensional building models with four columns were tested on the impact loading table. The impact loading table was hung from the steel frame and the impact load was given by the pendulum weight (Fig.6). The maximum weight of the pendulum was thirty tons and the intensity of the impact loading was controlled by the weight and the number of the shock absorbers made of steel and synthetic rubber.
An example of the building models is shown in Fig.7. The intensity of the shock was increased progressively until the response displacement became three times of the yield displacement. The maximum response acceleration was 1.5 times of

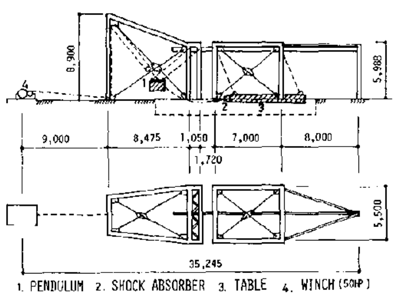


Fig.6 Impact Loading System[3]

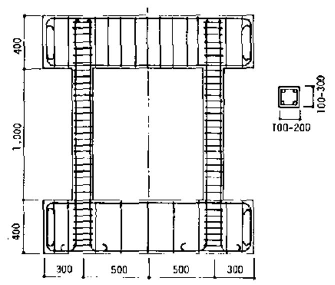


Fig.7 A Example of Frames Tested on Impact Loading Table[3]

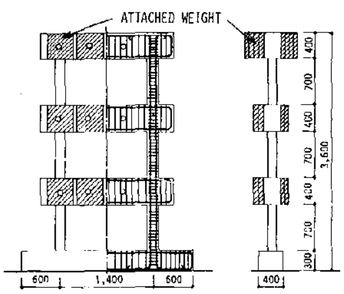


Fig.8 Tested Frame on Earthquake Simulator[4]

the gravity. One of the building models was tested until the collapse stage of which response displacement was more than nine times of the yield displacement. The measured response was compared with the results of the computer simulation. A bi-linear model, Tri-linear model and the Takeda model were used for the computer simulation. They pointed out that the feasibility of the computer simulation using the Tri-linear model and the Takeda model was approved while the further investigation on the viscous damping ratio was necessary. In their simulation, 5% and 2% to the critical damping were used for the Tri-linear and the Takeda model, respectively.

Shaking Test on Earthquake Simulator -- A three-story one bay reinforced concrete frame model was tested on the earthquake simulator(Fig.8). The earthquake motion simulating the NS component of El Centro'40 was used. The intensity of the table acceleration was increased progressively from 40 gals to 1000 gals. The yielding of the column at the first story was observed at the table acceleration of 600 gals. The response displacement to the 1000 gals table acceleration was 2.8 times of the yield displacement at the first story, 1.56 times and 1.48 times at the second and the third story, respectively. The test results were compared with the shear model analysis and the pseudo-dynamic loading test performed on another frame model. A fairly good agreement between the shaking test and the shear model analysis was obtained, however, they pointed out that the shear model analysis was very sensitive to the assumption of the viscous damping ratio and the further investigation was necessary to develop more feasible analytical model.

Future Use of The Earthquake Simulator

Although the useful data were obtained by the shaking tests, the tests were limitted on the simulation of the small models to the uni-directional ground motion. In order to design and more rational earthquake-resistant reinforced concrete building constructions, the followings are pointed out on the future use of the earthquake simulators;

1) Shaking test on full size buildings should be carried out. A number of the tests does not need to be large. Three dimensional large size earthquake simulator is desirable, but, one dimensional large size may be sufficient.
2) The shaking tests on small size building models having various parameters are also necessary. Three dimensional small size simulators are necessary.
3) Pseudo-dynamic loading tests should be associated with them.
4) The comparative study should be done on the data obtained from the shaking tests on full size and small size building models and the pseudo-dynamic loading tests. Emphasis should be on the development of the feasible analytical models.

SHAKING TESTS BY VIBRATION GENERATORS

A forced vibration test by the vibration generator has been recognized as one of the most acceptable techniques to investigate the dynamic properties such as natural period, mode shape of the vibration, damping coefficient etc.. Especially for high rise buildings, it has been often used to assess the feasibility of the analytical model used in the original design. However, the disadvantage of this method is usually due to the difficulty to control the amplitude of the building response, which causes a different vibration from the earthquake response and requires the modification of the data to simulate the earthquake response. As far as the building remains within the elastic range, the modification is possible [1], however, it is usually difficult if the response exceeds the elastic limit. Therefore, in order to investigate the overall behavior of the building during earthquake, other appropriate method should be associated together. Development of the generator having the acceleration control system is also desirable.

From this point of view mentioned above, several forced vibration tests on reinforced concrete existing buildings and the real size building models were carried out associated with the pseudo-dynamic loading test up to the collapse stage in Japan. For an example, the test on the five-story full size apartment house of reinforced concrete walled frames was carried out at the Building Research Institue in 1970 [10]. The testing method were pseudo-dynamic lateral loading test and the forced vibration test by the vibration generator (Fig.9). A lateral force was applied at each floor level and alternated with a progressively increased amplitude of the displacement. After a completion of an alternate loading, a forced vibration test was carried out by the unbalanced mass type of the vibration generator. The change of the natural periods, damping coefficients and the mode of vibration due to the increase of the damage by the pseudo-dynamic loading test were measured and considered in making the analytical model to simulate the earthquake response of the building.

A demolishment of the existing building is a good chance to test the full size buildings. Several tests were performed by Professors H.Umemura, H.Aoyama and their colleagues [11,12]. One of them was the test on the reinforced concrete three-story school building suffered from a severe damage due to the 1968 Tokachi-oki earthquake. Since the damage was concentrated at the first story, a part of the upper two stories was chosen for the test (Fig.10). The damage at the first story was repaired and strengthened so that it had

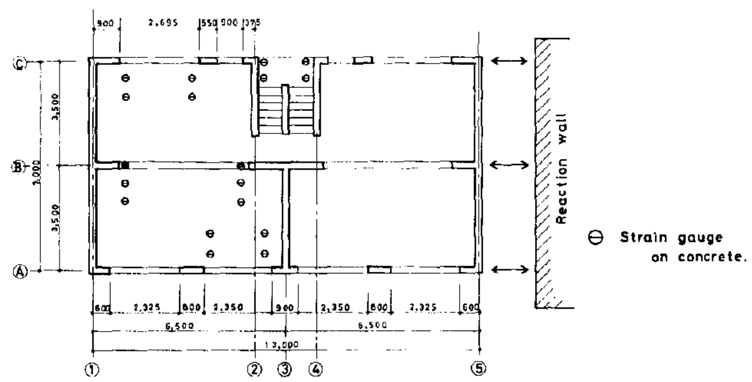


Fig.9(a) Plan of Five-story Apartment House [10] enough strength and stiffness. The pseudo-dynamic loading test and the forced vibration test were carried out on two frames and a block of the building. The ultimate strength, the deterioration of the strength and stiffness in the plastic range, the modes of vibration and the shape of the hysteretic loop were examined and the overall consideration on the cause of the damage of the building was made.

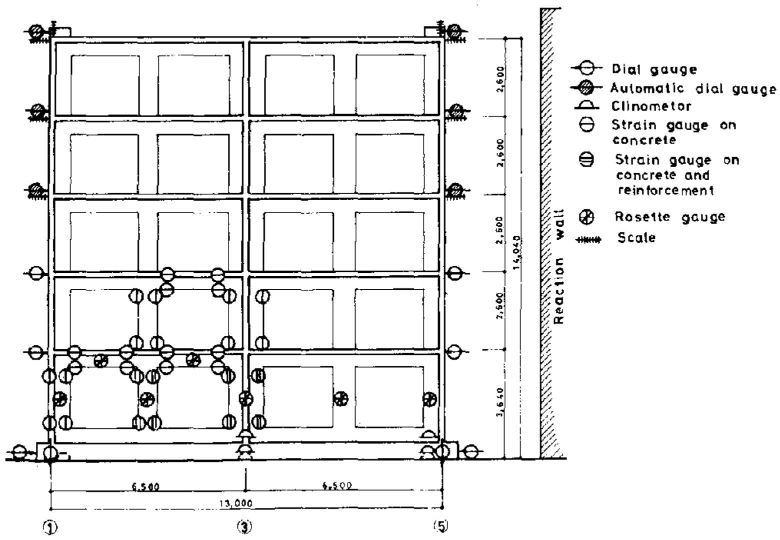
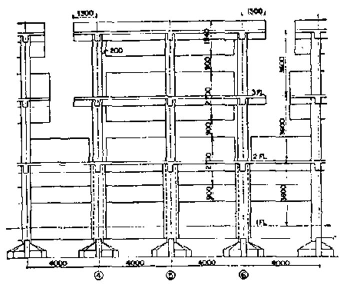
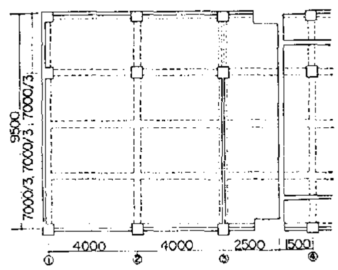


Fig.9(b) Section of Five-story Apartment House [10]





(a) An Example of Test Frames (b) A Test Block of the Building

Fig.10 An Example of Test Frames and Test Building [11]

SIMULATION BY COMPUTER-ACTUATOR ON-LINE SYSTEM

The computer-actuator on-line system is a newly developed technique to simulate the earthquake response of structures. A principle of the simulation by the computer-actuator on-line system is to solve the nonlinear differential equation expressing the earthquake response of the structural system to the earthquake ground motion by the computer taking into account the real restoring force characteristics obtained by the pseudo-dynamic loading test performed in parallel with the computer analysis. Therefore, it is a kind of the hybrid system of the computer analysis and the pseudo-dynamic loading test. As far as the author knows, the first trial to simulate earthquake response by a computer-actuator on-line system was made by Dr.M.Hakuno in 1969 at the Earthquake Research Institute, University of Tokyo [13]. He made the system consisting of an electro-magnetic actuator and an analogue computer and simulated the earthquake response of a single degree of freedom system supported on a small centilevered steel beam. In 1970, another trial to make the on-line system consisting of the electro-hydraulic actuator system and a digital computer started at the Institute of Industrial Science, University of Tokyo[14]. The system was completed in 1973 [15] and applied to the simulation of earthquake response of single story reinforced concrete frame models [16]. The system has been recently extended to be able to simulate the response of multi-degree of freedom system and the simulation of the response of the reinforced concrete single story frame models to bi-directional components of the earthquake ground motion was carried out [17].

Since the system has been developed recently, only a few tests were performed. However, it would be one of the most useful techniques to simulate the earthquake response of the reinforced concrete building constructions. The advantages of the on-line system are summarized as follows;

1) Earthquake response simulation can be performed taking into account the real nonlinear restoring force characteristics of structures or structural elements without assuming the analytical model.
   This advantage may help not only to realize the simulation of the earthquake response of the structure of which nonlinear restoring force characteristics is quite uncertain but also to assess the feasibility of the simplified analytical models derived from the pseudo-dynamic loading tests and to improve them so that they can represent more realistic behavior of the structures.
2) Pseudo-dynamic loading test following the realistic loading path on large size structure can be performed by the electro-hydraulic actuator having a comparatively small capacity. Because, the test specimen is not always required to be a whole structure but a part of the structure or the structural elements. This may solve the problem on the loading path in the pseudo-dynamic loading test and the problem on the law of similarity between the model structure and the prototype structure which is usually rather difficult to satisfy in the shaking test on the earthquake simulator.
3) The observation of the failure mechanism of the structure and the collection of the data can be easily done by the modification of the time axis. Because the test does not need to be performed on real time but can be performed on an enlarged non-real time controlled by the computer, i.e., the response simulation can be paused at any moment and as long as necessary unless the effect of creep becomes significant.

4) Gravity load can be easily applied to the model structure by actuator previous to the on-line test to consider the stress due to the gravity load such as axial stress in column, bending stress in beam, etc., while a difficulty is sometimes due to a heavy weight attached to the model structure for shaking test on the earthquake simulator.

Since the computer-actuator on-line system a kind of the hybrid system, it is innevitable that the system has some of the disadvantages both in the pseudo-dynamic loading test and the computer analysis. As mentioned before, the disadvantage of the pseudo-dynamic loading test is due to the difficulty to consider the effect of the loading path and the strain rate. The on-line system can improve the former but the latter. The choice of an adequate numerical integration method as well as an acurate measuring system should be carefully done to ensure the accuracy of the simulation.

Application of the Computer-Actuator On-line System

The earthquake response of single story reinforced concrete frame models has been simulated by the on-line system by the author and his colleagues since 1973 [16,17,18,19]. Two test series on the earthquake response simulation; one was to uni-directional component of earthquake ground motion and another was to bi-directional components, were performed.

Frames, Test Specimens and Loading System -- The column yielding type of the reinforced concrete single story frame models having strong and stiff beam were analyzed (Fig.11). The test specimens were made so as to represent the column of the frame model (Fig.12). An example of the test setup is shown in Fig.13. Another actuator located in perpendicular to the actuator #2 was also used for the simulation to the bi-directional components of the ground motion. Axial load was applied at the beginning of the test and kept constant during the test. Each component of the response displacement was enforced by the actuator controlled by the computer system. Both ends of the column were fixed to bending and shear, but, an end could slide to the vertical direction.

Flow Diagram of the On-line System -- The flow diagram of the on-line system is shown in Fig.14. The computer system-I and the on-line system were used for the on-line simulation. Each component of the response displacement of the frame to the ground motion at a certain step was calculated and transformed to that of the specimen in the computer system. Each component of the displacement was applied to the specimen through the D-A convertor and each component of the restoring force was measured in the loading system. The measured force was returned to the main system through the A-D convertor and transformed to the restoring force of the frame. This procedure was repeated until the record of the ground motion was terminated.

Variables and Numerical Analysis -- The variables were the initial period, the lateral strength, the axial stress due to the gravity load and the number of the horizontal components and the intensity of the earthquake ground motion. For numerical integration, the central difference method; Lumped-impulse method [21], was used. However, since the method was not self-strating, the linear acceleration method was used until the response displacement reached at a certain limited value within linear range. The acceleration record of the Hachinohe 1968; NS component and/or EW component, were used for the ground motion. The duration time was 12 seconds with zero data at the last two seconds.

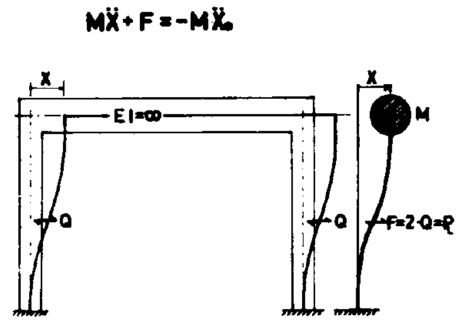


Fig. 11 Analyzed Frame [16-20]

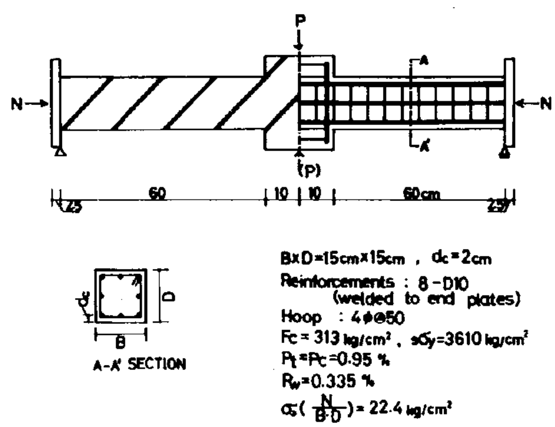


Fig. 12 Test Column [17-20]

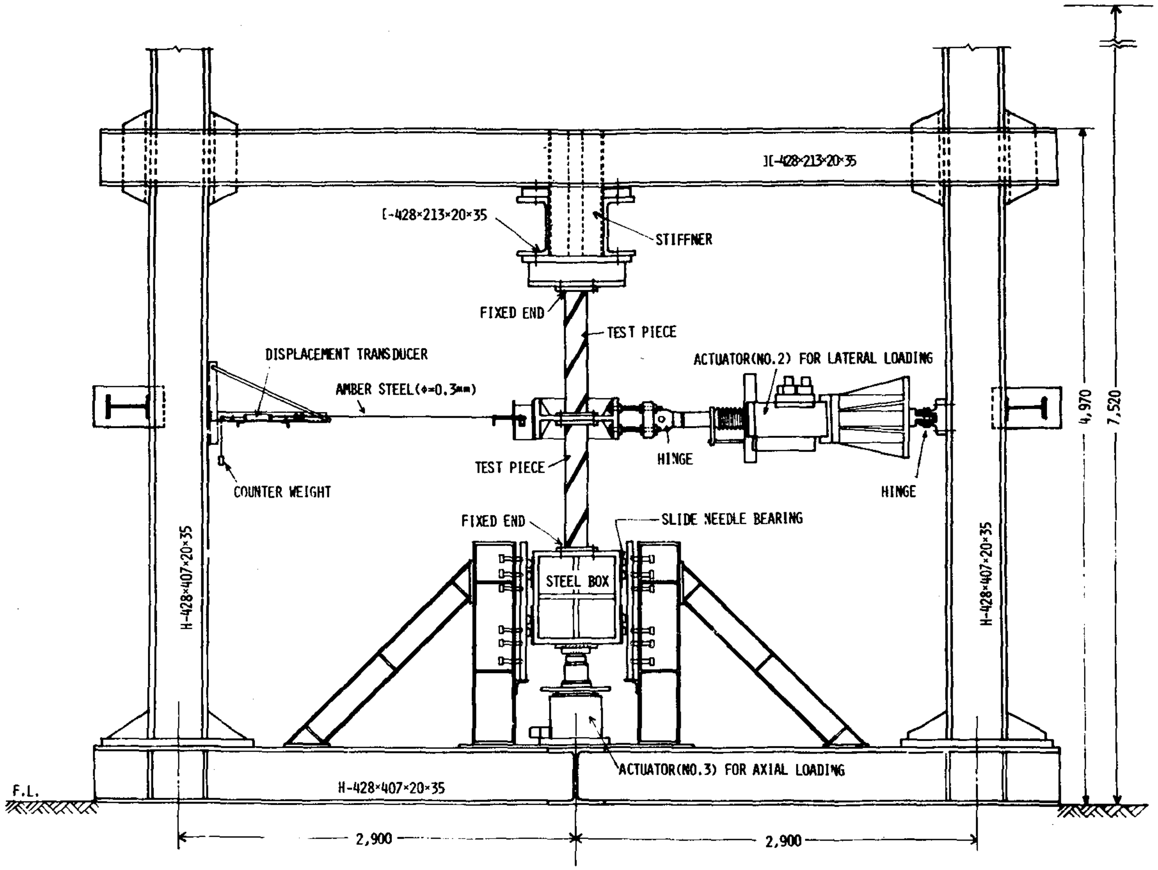


Fig. 13 Test Setup [17-20]

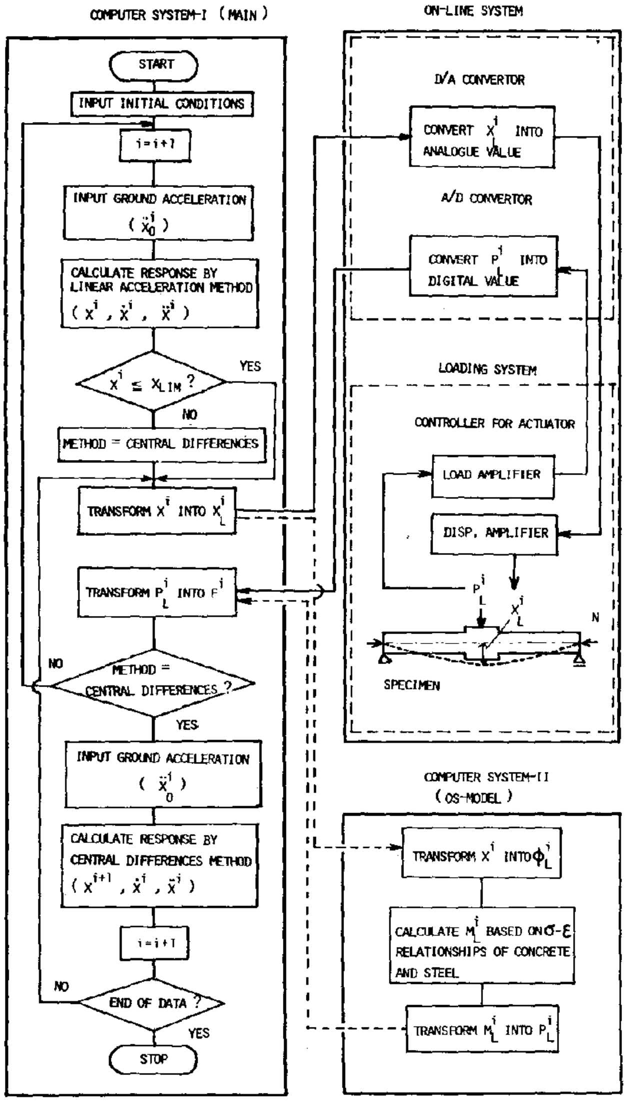


Fig.14 Flow Diagram of On-line System [17-20]

Results -- The examples of the results are shown in Figs.15.16,17 and 18. The results of the computer simulation by the fibre model analysis; the computer programs OS-1D and OS-2D, are also shown there. The stress-strain relationships for concrete and steel are shown in Fig.19 [19,22], and the flow diagram of the computer simulation is shown in Fig.14.

The concluding remarks were;

1) The developed on-line simulation system appeared to be useful for the nonlinear earthquake response simulation on the reinforced concrete building constructions,
2) Response to bi-directional ground motion could be simulated by the on-line system,
3) A good correlation was obtained between the results by the on-line simulation system and by the developed fibre model analysis, and
4) The degrading tri-linear model and the origin-oriented model would be the lower and the upper models , respectively, to simulate the response displacement of reinforced concrete buildings.

CONCLUDING REMARKS

The experimental earthquake response simulations on reinforced concrete building constructions in Japan have been reviewed. The concluding remarks are;

1) In order to investigate the real behavior of the reinforced concrete building constructions during earthquake and to develop more rational earthquake resistant design method, more efficient use of the earthquake response simulators should be considered,

2) The feasible experimental facilities are a) Earthquake Simulator, b) Vibration Generator and c) Computer-Actuator On-line System,

3) The shaking tests on full size specific reinforced concrete buildings should be done. The systematic investigations on the small size building models are also necessary. The emphasis should be on the investigation of the law of similarity and the development of the realistic analytical models,

4) Development of the vibration generator controlled by the acceleration is desirable to realize the earthquake response test of existing buildings, and

5) The computer-actuator on-line system is one of the most feasible method. Since it is a hybrid system of the computer system and the pseudo-dynamic loading system, it is desirable to assemble the existing computer systems and the pseudo-dynamic loading systems, while the development of the soft-ware is necessary.

ACKNOWLEDGEMENTS

Sincere appreciation is due to Messrs. M.Seki, K.Okada and A.Park for their assistance in preparing the drawings referred in this report.

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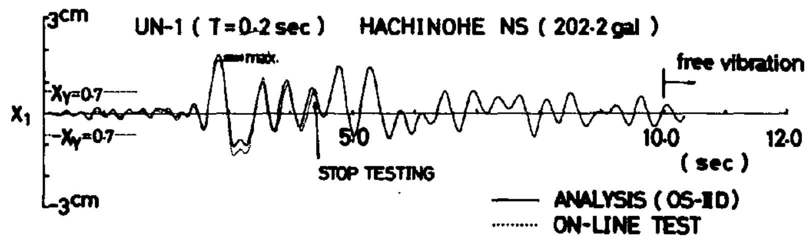
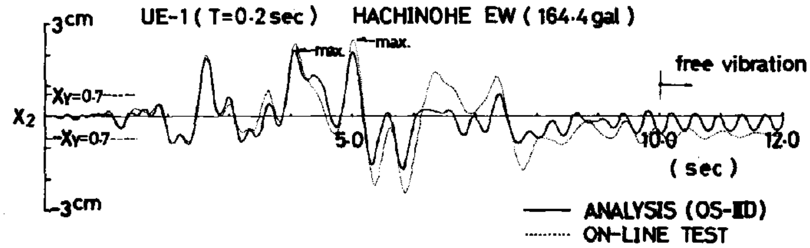
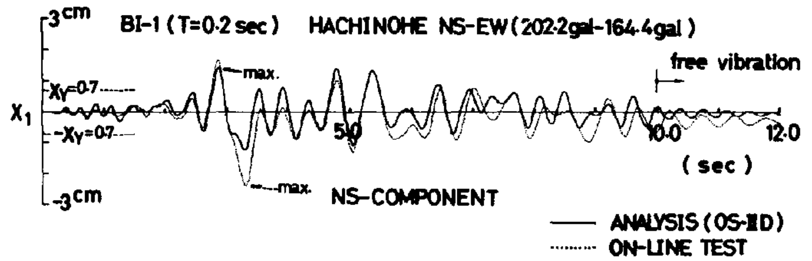


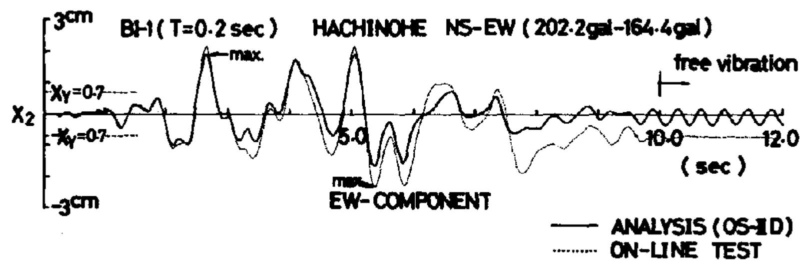
(a) Response Displacement to NS Component of Ground Motion



(b) Response Displacement to EW Component of Ground Motion



(c) NS component of Response Displacement to Bi-directional Ground Motion



(d) EW Component of Response Displacement to Bi-directional Ground Motion

Fig.15 Response Displacements by On-line Simulation and Computer Simulation [17]

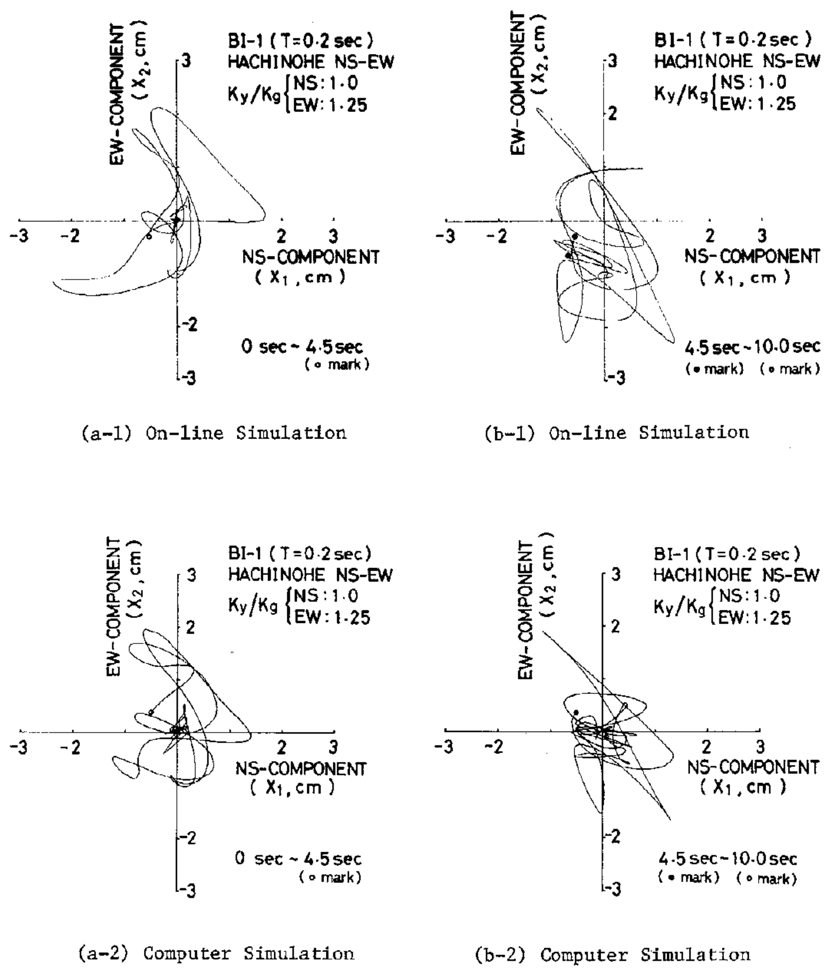


Fig.16 Horizontal Traces of Response Displacements to Bi-directional Ground Motion [17]

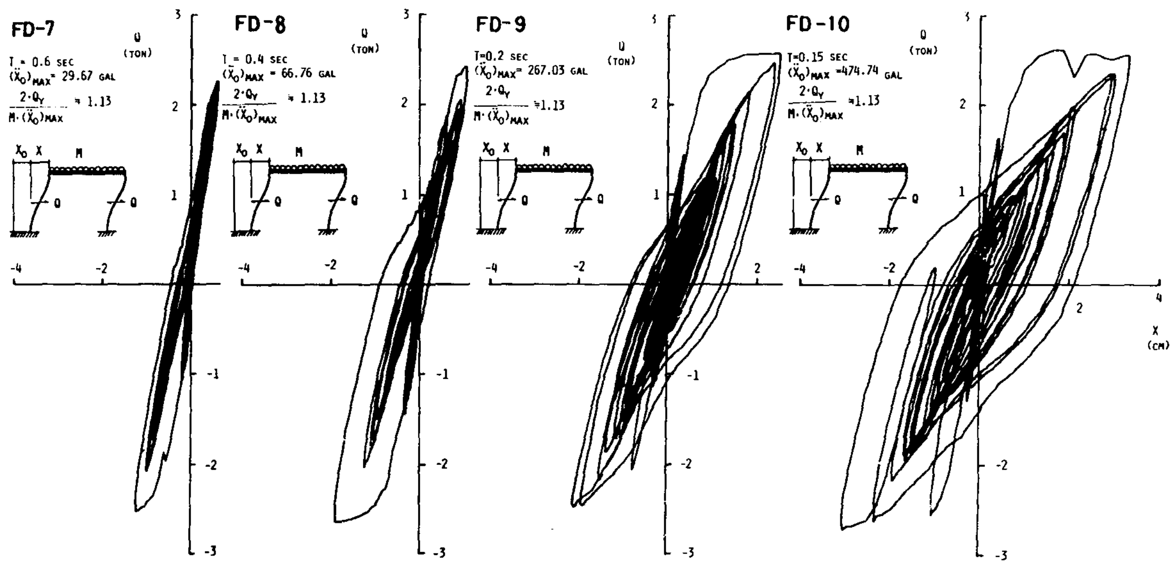


Fig. 17 Restoring Force Characteristics by On-line Simulation [18,19]

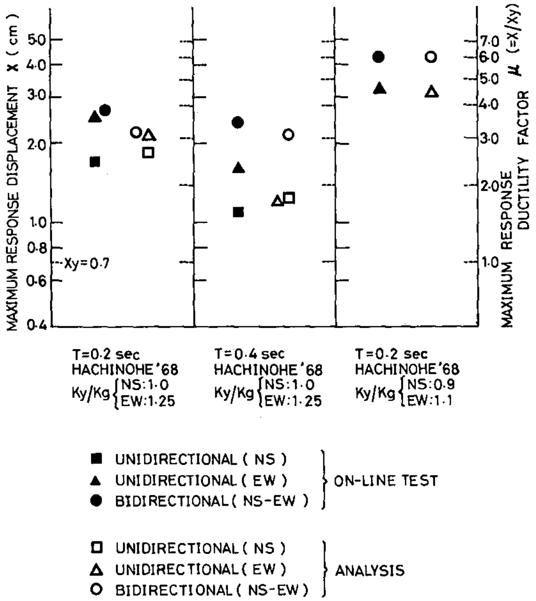


Fig. 18(a) Maximum Response Displacements [17]

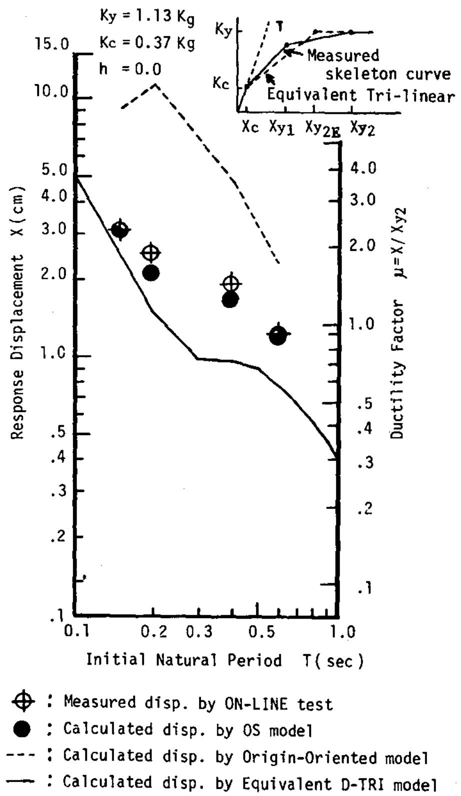
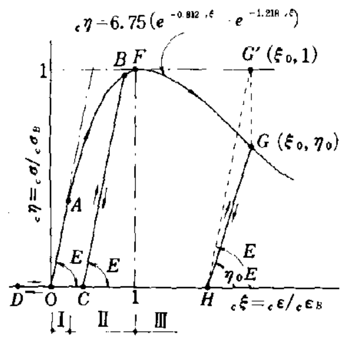
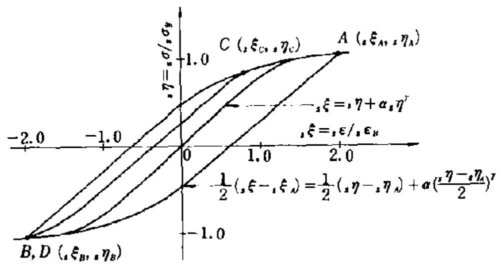


Fig. 18(b) Maximum Response Displacements [19]



(a) Concrete



(b) Steel

Fig. 19 Stress-Strain Relationships
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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

USE OF EARTHQUAKE SIMULATORS AND LARGE-SCALE LOADING FACILITIES IN ERCBC

by

V. V. Bertero
Professor of Civil Engineering
University of California, Berkeley

R. W. Clough
Professor of Civil Engineering
University of California, Berkeley

M. Oliva
Research Assistant
University of California, Berkeley

INTRODUCTION

The ultimate objective of research in Earthquake-Resistant Reinforced Concrete Building Construction (ERCBC) should be to develop methods of design and construction to produce seismic-resistant buildings which are both functional and economical. Achievement of this goal entails considerable analytical and experimental studies, research which must be fully integrated. Ideally, the end result should be a capability to design buildings to behave satisfactorily under various specified earthquake input (design earthquakes), and to predict analytically their seismic performance. Development of this analytical capability requires experimental study on the behavior of buildings when subjected to seismic loads so that appropriate mathematical models may be formulated. This task is more complicated than it appears since prediction of a building's response to earthquakes necessitates study of not only the building's structural system, but of the entire soil-building system as well. This problem is discussed in greater detail in reference 1.

Moreover, because seismic excitations usually induce large nonlinear deformations, it is not possible to isolate the effects of these excitations from those induced by either normal excitations or cumulative damage. Cumulative damage can be due to previous moderate seismic excitations or any other consequential or independent, abnormal events likely to occur during the service life of a building.

The most effective experiment for this purpose would be one in which actual buildings, whose current states of stress and deformation are known, were subjected to actual earthquakes, and where the nonlinear response of the buildings could be measured, and the mechanisms associated with the observed behavior could be identified. Unfortunately, this approach is unfeasible for effecting immediate improvement in ERCBC, not only because of the difficulties involved in predicting the current state of buildings, but, mainly, because of the high cost of providing adequate instrumentation in a building and the low probability that the instrumented structure will be subjected to a severe earthquake within a reasonable period of time. The 'elastic' vibration properties of selected buildings can be determined effectively by field experiments. However, to study the strength and nonlinear damage mechanisms of structures, which are the most important aspects of structural response for design of buildings against severe earthquake ground motions, it is necessary to resort to laboratory experiments with model structures.

Objectives and Scope

The purpose of this report is to describe laboratory procedures for studying the response of buildings under seismic excitations, with emphasis on the response of structures and structural components, using earthquake simulators and large-scale loading facilities. After a brief discussion of the different experimental techniques that are used in ERCBC research, the report briefly describes these two types of facilities. Some examples of tests using the two facilities are offered. The advantages and disadvantages of each system are discussed, followed by suggestions for future development and experimental research.

All of the above items are reviewed, discussed, and evaluated in terms of their implications for seismic-resistant design and construction. The experimental data can be used in several different ways to work toward this objective: (1) in investigating new concepts and new structural layouts, (2) in verifying or improving mathematical modeling concepts for use in computer analyses, and (3) in improving design details to increase the toughness of the structure in response to seismic excitations.

EARTHQUAKE SIMULATORS

Although no innovative experimental techniques have been developed within the last six years, there have been major advances during this time in the area of seismic-resistant design of buildings in the U.S. The occurrence of the San Fernando earthquake of February 1971 was the primary reason for the numerous studies in this area which followed. Investigations of this event have not only produced valuable data but have led to post-earthquake laboratory studies, all of which have resulted in considerable improvement in seismic-resistant design and construction.

At the ASCE-EMD Specialty Conference on Dynamic Response of Structures, several authors [2,3] summarized the present status of experimental research on earthquake-resistant structures. Experimental techniques for tests of actual buildings in the field have been discussed by Hudson [2] and Bertero [1] and are also discussed in the reports presented in this ERCBC workshop by Shepherd and Jennings [4] and Freeman, Honda, and Blume [5]. Several experimental laboratory facilities and techniques for pseudo-static and dynamic testing of building structures are discussed by Bertero [1] and Clough and Bertero [3]. The different experimental techniques that are used in Japan have been reviewed by Okada [6], and the problem of earthquake simulation in the laboratory has been discussed by Sozen [7].

In this paper a review of the use of earthquake simulators and large-scale loading facilities is presented.

Existing Earthquake Simulator Facilities

There are presently several laboratory or institutions using small- to medium-scale earthquake simulators. A brief description of the largest simulators presently in use and those that have been designed in the U.S. and Japan follows.

Largest U.S. simulator--The largest simulator being used in this country is located at the Earthquake Simulator Laboratory of the University of California, Berkeley. The facility consists of a 20-ft-square (6.1-m-square) reinforced concrete slab whose weight is 100 kips (444.8 kN) and which is driven by servo-controlled hydraulic actuators to produce displacements in the vertical direction and in one horizontal direction [8,9]. Test structures which are fastened to the slab can be subjected to any desired base motion in these two components; thus, ground motions which were recorded during past earthquakes can be applied to the test structures, and their seismic response can be measured directly. The capacity of the simulator is ±5 in. (±12.7 cm) displacement and .75 gravity acceleration, horizontally, and ±2 in. (±5.1 cm) displacement and 0.5 gravity, vertically, with a full pay load of 100 kips (444.8 kN). Hydraulic oil at an operating pressure of 3,000 psi (20.7 MPa) is supplied to each of the actuators from four 30 gpm (0.114 m^3pm) pumps with on-line accumulators available to supply momentary peak flows. The flow rate of the servo-valves, 200 gpm (0.76 m^3pm) horizontal and 90 gpm (0.342 m^3pm) vertical, limits the maximum velocities in the horizontal and vertical directions to 25 in./sec. (63.5 cm/sec.) and 15 in./sec. (38.1 cm/sec.), respectively. Under harmonic action, the table capacity might thus be governed by either displacement, velocity, or acceleration, depending on the frequency of excitation (as indicated in Fig. 1) for the unloaded table.

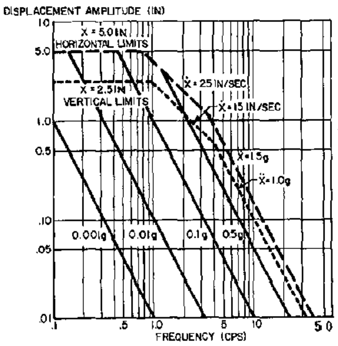


Fig. 1 Capacity of Unloaded Shaking Table as a Function of Frequency (1 in.= 2.54 cm)

The simulator is computer-controlled to follow a prescribed earthquake history, and the same computer also controls the data acquisition system. The latter has a capacity of 128 channels, each of which is normally sampled at a rate of 50 readings per second, with the values recorded in digital form on a magnetic disk. After a test has been completed, the data is transferred to magnetic tape for further processing, plotting, and permanent storage. The earthquake simulator was first put into operation in 1973, and has been used to test a wide range of structural types [3]. A brief discussion of the experiments that have been, are, and will be conducted on reinforced concrete structural systems follows. The main objective of this discussion is to point out the advantages and limitations of this facility with regards to research in the area of ERCBC.

Reinforced concrete frame program [10,11]--One of the earliest structures tested on the earthquake simulator was the two-story reinforced concrete frame shown in Fig. 2. This was the first in a series of four similar frames that have been tested to date in a continuous program of reinforced concrete research at the Earthquake Simulator Laboratory.

The frames, all of similar general geometry, were not intended to be exactly scaled models of any particular type of structure but could be representative of a portion of a small office building, and were designed and constructed according to standard seismic codes. The frame was built to about 0.7 scale for reasons of convenience and economy. This scale is large enough to permit use of normal deformed reinforcing bars and construction procedures, and was treated as a small, full-scale structure in interpreting its observed performance. Twenty to thirty-six kips (88.9 to 160.1 kN) of concrete blocks (Figs. 2 and 3) have been loaded on the floors to give the test structures a typical period of vibration and also to induce significant seismic forces during the tests. Lateral bracing was provided to constrain the building against any lateral or torsional motions.

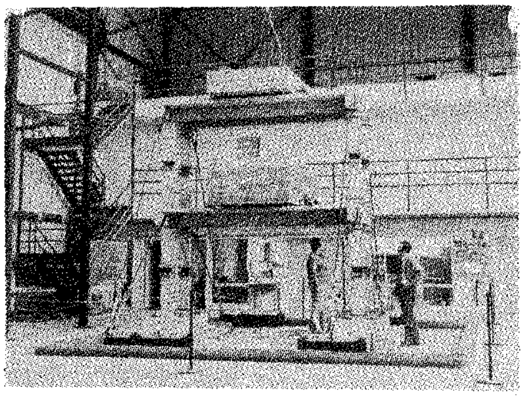


Fig. 2 Two-Story R/C Frame on Shaking Table

The basic instrumentation required to interpret the overall response was similar throughout the testing program. Accelerometers, displacement measuring potentiometers, and DCDT's (Direct Current Differential Transformers) were located at each story level and on the shaking table to measure lateral and torsional motions. Strain gages have been located on column and beam reinforcing bars at points of maximum expected strains along with externally mounted displacement gages to measure relative rotations (curvature). Force transducers, calibrated to indicate moment and shear forces, with stiffness properties

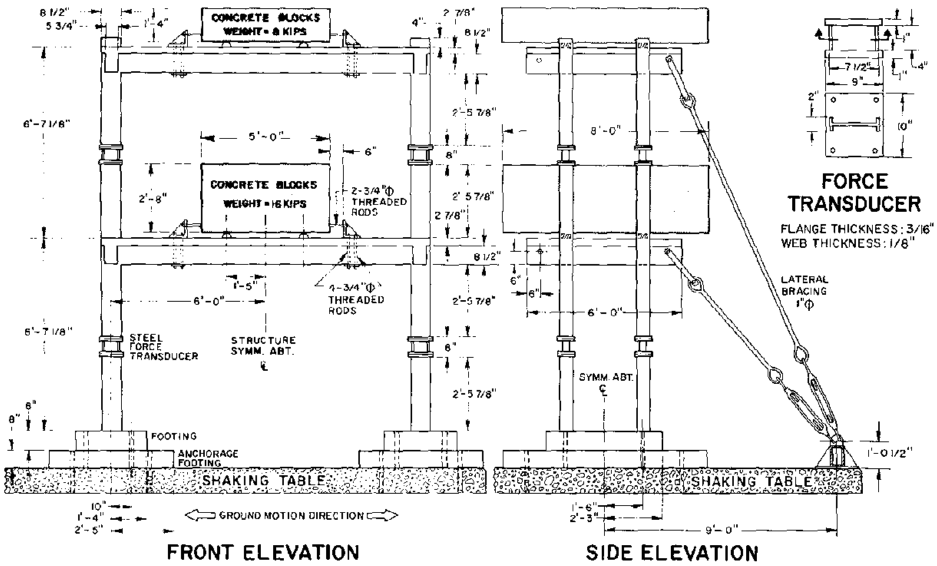


Fig. 3 Test Structure and Test Arrangement on Shaking Table
(1 in. = 2.54 cm; 1 ft = 0.3048 m ; 1 kip = 4.448 kN)

similar to the columns, were built into the columns at midheight near the expected curvature inflection points (Figs. 2 and 3). In each test, approximately 100 channels of data were interpreted and recorded.

Each particular test frame was designed with certain variations in individual member sizes so as to vary the effect of different structural mechanisms on the overall structural response. Thus each individual structure had special instrumentation, in addition to that mentioned above, to measure those response quantities of particular interest in the structure.

Throughout the test program the main earthquake input signal was derived from the 1952 Taft N-69°-W component (Fig. 4) excluding vertical motion, although some structures were exposed to other excitations as will be noted later. The excitations have been applied with increasing intensities on each test structure to introduce initial cracking, to test an initially cracked structure in the elastic range under low excitations, and to induce strong motions forcing the structure into the inelastic range. After each earthquake test the flexibility, free vibration frequency, and damping were determined by suddenly releasing approximately 1,000 lbs (4.448 kN), applied either at first or second story levels. The successive changes in these properties, illustrated in Fig. 5, demonstrate the extent to which the structure was damaged in each test run. The results depicted by this figure also point out the difficulty of assessing realistic values for the dynamic properties of an actual structure when a severe earthquake occurs, since these values would depend on the cumulative damage induced by previous events.

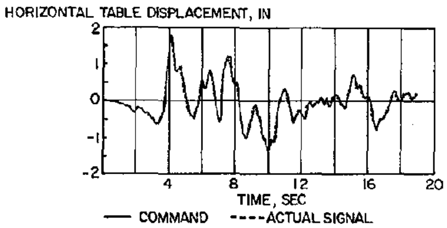


Fig. 4 1952 Taft N-69°-W Component: Comparison of Command and Actual Table Signal (1 in. = 2.54 cm)

The test results have been used to evaluate the ability of available nonlinear analysis procedures to predict response and to improve the mathematical modeling of structural mechanisms used in such analyses. Structural displacements, particularly floor displacement time histories, were used as the basic quantity in the above evaluations.

The first reinforced concrete frame tested at the Simulator Laboratory exhibited a resistance function primarily affected by flexural plastic hinging at the column and beam ends [10]. Reinforcement detailing was responsible for the poor anchorage of the slab reinforcement at its support along the transverse girders. Early slippage of this reinforcement created a weak critical section at the column interface which resulted in considerable flexural cracking [maximum width = 0.5 in. (1.27 cm)] at the ends of the bottom story girders. A global stiffness degrading mechanism based on first mode amplitude was developed for modeling in combination with a bilinear yielding hinge mechanism at member ends. Correlation between analytical and experimental results (Fig. 6) is considered adequate although further improvements could, undoubtedly, be made with refinements in the mathematical model.

The second concrete frame was nearly identical to the first, differing

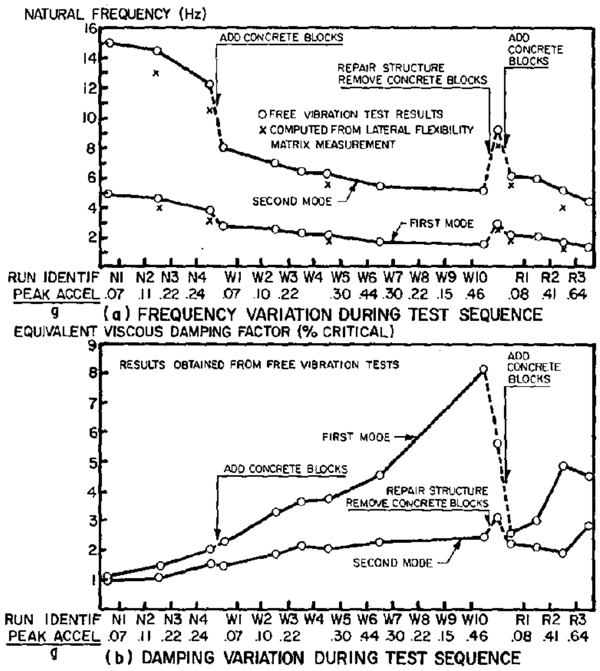


Fig. 5 R/C Frame 1: Changes in Dynamic Characteristics of Structure During Testing

only in that the poor anchorage of the slab reinforcement previously noted was not apparent. Damages induced in the frame were similar to those seen in the previous frame. Analytical correlation with observed results was obtained using the same modeling concepts developed for the first frame with results as indicated in Fig. 7 [11].

After testing two frames whose response was dictated mainly by the flexural hinging of column members, a third frame was redesigned to result in higher shear stresses with a reduced shear resistance vs. moment capacity ratio by using heavier longitudinal flexural reinforcement with increased yielding strength. During testing, yielding of the main reinforcement and crushing and spalling of concrete at the column ends (Fig. 8) again resulted in a hinging action, causing a predominant first mode type of response. High cyclic straining in

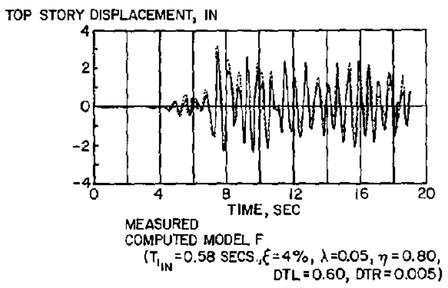


Fig. 6 R/C Frame 1: Correlation of Computed and Measured Top Story Displacement (1 in. = 2.54 cm)

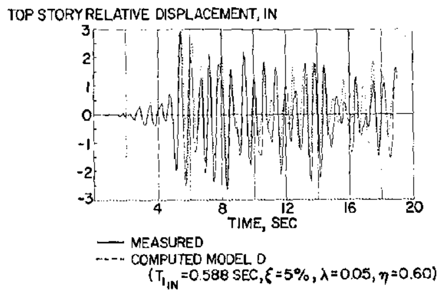


Fig. 7. R/C Frame 2: Correlation of Computed and Measured Top Story Displacement (1 in. = 2.54 cm)

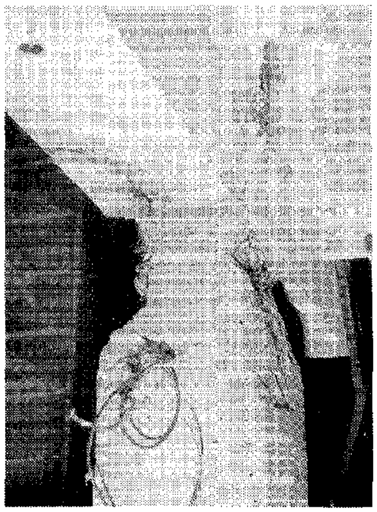


Fig. 8 Column Hinging at Top of First Story Column

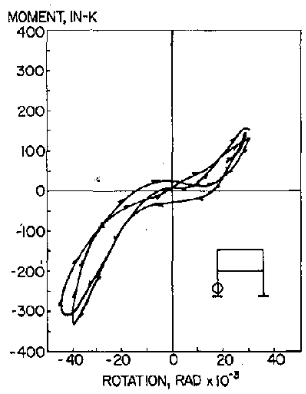


Fig. 9 R/C Frame 3: Actual Moment-Rotation at Base of First Story Column (1 in. = 2.54 cm; 1 kip = 4.448 kN; 1 rad = 57.3°)

the column, especially at beam-column and column-footing joints, caused significant bond deterioration and bond slippage, resulting in significant pinching of the hysteretic loops as exhibited in the moment-rotation plots of Fig. 9, where a seemingly zero stiffness condition existed at low load level. Mathematical models have been developed which duplicate local response, including pinching effects in the moment-rotation relation of Clough's Bilinear Stiffness Degrading Model. The models exhibit good correlation in the local moment-rotation behavior in the earlier part of the signal as indicated in Fig. 10. Correlations between overall structural behavior and that predicted have not yet been computed.

Special detailing for column and beam longitudinal bars, decreasing of column lengths and sizes, increasing of stirrup spacing and addition of extra concrete mass blocks resulted in fairly different mechanical characteristics for the fourth reinforced concrete frame structure. All of the modifications were intended to further accentuate shear effects on the response by decreasing shear strength relative to column flexural strength and by increasing the total base shear. In this case, the most striking visible failure after testing was the occurrence of sizable diagonal shear-induced cracks [maximum width ≈ 0.4 in. (1.0 cm), Fig. 11] with anchorage failure in the stirrups. Hinging action was evident from rotational data at the column bases where high steel strains indicated definite bond loss. With the expectation of increased shear effects, special instrumentation, consisting of multiple diagonal displacement measurements to determine shear deformation, and new force transducers were incorporated.

The increase in flexural strength of the structure due to the modifications mentioned resulted in a frame too strong to fail under the maximum intensity Taft earthquake that the facility could apply; hence, a special signal composed of multiple cyclic square acceleration pulses

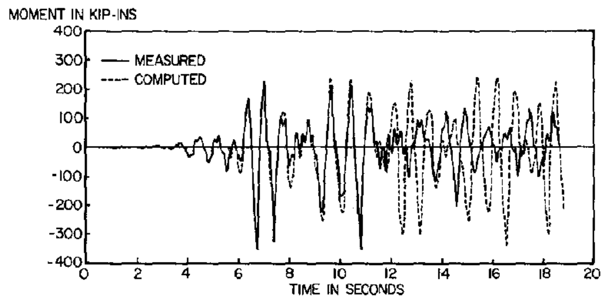


Fig. 10 R/C Frame 3: Computed vs. Measured Moments at Base of First Story Column (1 in. = 2.54 cm; 1 kip = 4.448 kN)

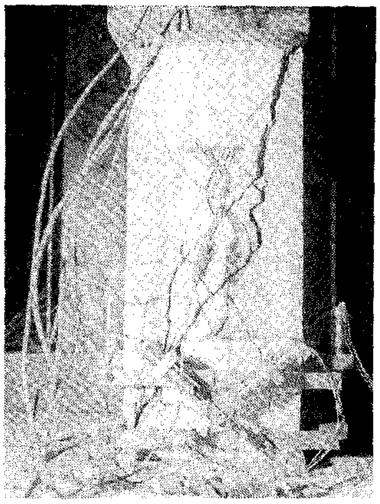


Fig 11 R/C Frame 4: Column Failure

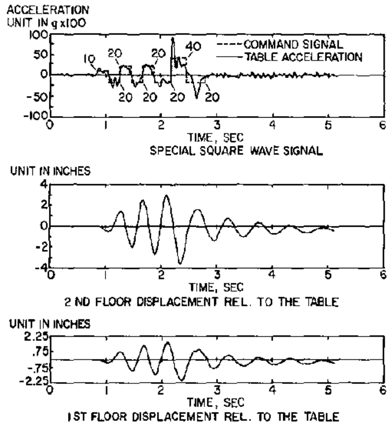


Fig. 12 R/C Frame 4: Square Wave Input Acceleration Motion and Floor Displacement Responses (1 in. = 2.54 cm)

with a frequency near the natural frequency of the structure (Fig. 12) was developed to initially build up a resonant response with low amplitude input. A pulse large enough in one direction was then applied to induce a significant inelastic displacement excursion. The large pulse was designed to bring the table to a peak 25-in./sec. (63.5-cm/sec.) velocity. The actual table accelerations differed from the desired square wave due to the depletion of oil in accumulators and limited pump capacity, but were sufficient to induce the shear failure mentioned. The limited velocity [25 in./sec. (63.5 cm/sec.)]

and displacement [± 5 in. (± 12.7 cm)] of this earthquake simulator makes it very difficult to bring the testing structure into a collapse state or even to introduce severe damage.

Modeling is presently being attempted using the moment rotation degrading models in the Drain-2D inelastic analysis program [12] coupled with a new element developed to match the shear force - shear deformation behavior with the pinching characteristics evident in Fig. 13.

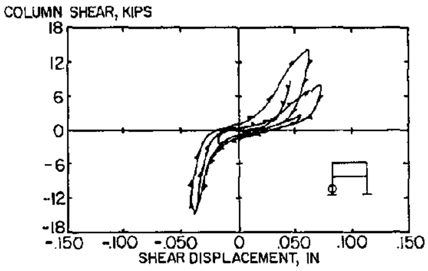


Fig. 13 R/C Frame 4: Column Shear vs. Shear Displacement (1 in. = 2.54 cm; 1 kip = 4.448 kN)

Future research programs--

1. Effects of horizontal ground motions inclined with respect to main axes of the structure: Future testing is being planned to include the skew placement upon the simulator of a frame similar to that of Fig. 3 but square in plan. This will permit the one horizontal motion of the simulator excitation to induce lateral deformations in both of the frame's major axes, causing biaxial bending in columns. A series of specimens with different floor systems will be investigated under this type of diagonal excitation.

2. Effects of rotation (flexibility), uplift and/or sliding of the foundation on the seismic response of reinforced concrete buildings: At present it is generally recognized that at sites close to active faults which can be the source of large magnitude earthquakes, the intensity of a seismic ground motion can be considerably higher than that which has been considered in past seismic codes. Dynamic analyses of buildings, based on linear-elastic theory using recorded or artificially obtained severe ground motions, have shown that the resulting base shear coefficient and, therefore, the equivalent static lateral load that would be required, are considerably higher than those recommended in codes. The use of these higher design forces leads to larger overturning moments, which, as currently required, should be resisted in their totality by the structural system. This requirement may often necessitate a costly supplementary foundation anchorage system since the dead weight alone cannot balance the required overturning effects to be incorporated into the foundation design. However, it should be recognized that these forces and overturning moments are usually obtained assuming a fixed, rigid foundation, that is, one that will not uplift and/or settle, and, in some cases, is also fixed against rotation.

For certain types of structures, the construction cost of foundations that will not rotate (rock) or uplift can be prohibitive. This is especially true for certain cases of shear wall structures. According to the results obtained in experimental and analytical studies of such structures [13], before the walls can develop their capacity, the foundation will tilt and part of it will uplift. Although some studies [14,15] have indicated that allowing the foundation to tip and uplift under dynamic lateral loading can have a desirable effect on strength and ductility requirements, these studies should be considered of an exploratory nature only. Comprehensive integrated experimental and analytical studies in this area are necessary. Earthquake simulators can be used with advantage in conducting such experimental studies. This has been confirmed in part by the results obtained in two experimental studies carried out on two different steel frames at the Simulator Laboratory.

Three-story steel frame program [16]--This test structure is pictured in Fig. 14 and the uplift mechanism is shown in Fig. 15. The column bases were

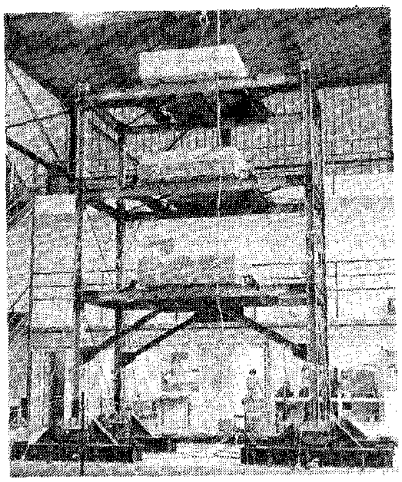


Fig. 14 Three-Story Steel Model on Shaking Table

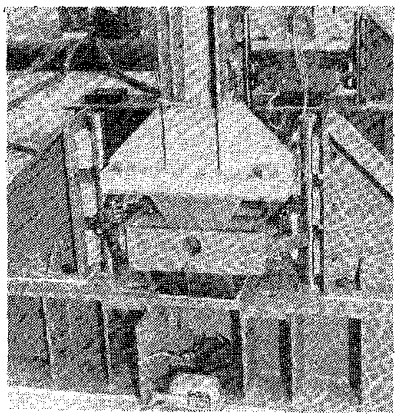


Fig. 15 Uplift Mechanism

pinned to stiffened steel footing blocks. These blocks were rotationally constrained but allowed to move vertically on roller bearings riding on 1.5 in. (3.8 cm) rounds. Longitudinal motion (horizontal translation) of the blocks relative to the shaking table was constrained by the brackets shown in Fig. 15. These brackets were bolted to steel beams prestressed to the shaking table. Neopreme impact pads were provided beneath the footing blocks. The effects of two pads with different amounts of stiffness were investigated.

The experimental results are briefly described in reference 16. A detailed description of these results and their correlation with analytical predictions are given in reference 17. The response of the uplifting frame was vastly different from that of the constrained conventional frame. The results show that the concept of allowing a structure to uplift during an earthquake is a potential means of reducing strength and ductility requirements. Therefore, it was decided to further investigate its use in a nine-story frame.

Nine-story steel frame program [18]--The primary objective of this combined experimental and analytical program was to study the effect of allowing column uplift in response to seismically induced overturning moments. The experimental phase included tests under both uplifting and fixed base conditions, responding to identical base excitations. The analytical phase included evaluation of current nonlinear numerical analysis techniques in predicting both the uplifting and fixed base responses. In order to accomplish this evaluation, experimental table motions were used as input to nonlinear dynamic analyses, with the resulting analytical results compared directly to corresponding experimental results. Local element nonlinear behavior was modeled by concentrated bilinear plastic hinges; the uplift phenomenon was modeled by the use of bilinear elastic foundation springs with zero tensile capacity and stiffness in the upward direction.

The test model mounted on the shaking table is shown in Fig. 16. As can be seen, it is a three-bay, nine-story steel moment frame. Two of the bay widths are 78 in. (198.1 cm), and the third is 60 in. (152.4 cm); the first floor height is 48 in. (121.9 cm); and the remaining floors are 36 in. (91.4 cm). A dead weight of 10 kips (44.5 kN) was added to each floor, making the total weight of the model 102 kips (453.7 kN). All columns are W4x13, and all girders are W6x8.5; A36 steel was used throughout.

The table motion and response spectra for an uplift test signal based on the 1940 El Centro N-S record are shown in Fig. 17. The original record was time-scaled by a factor of 1.732 to account for the scale of the model, and amplified to produce a peak acceleration of about 0.5g. For the fixed base test the excitation was shifted about 1 sec. to the left on the time axis but was essentially identical in form.

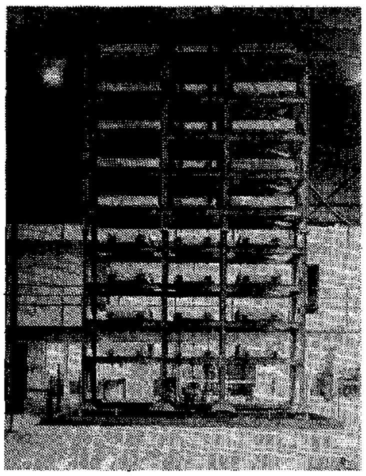


Fig. 16 Nine-Story Steel Frame on Shaking Table

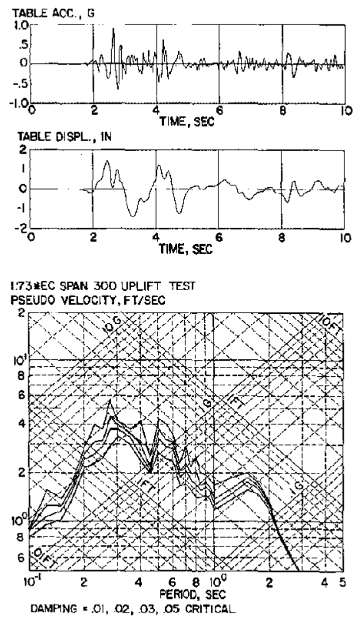


Fig. 17 Table Motion for El Centro Span 300 Test (1 in. = 2.54 cm; 1 ft = 0.3048 m)

Selected relative floor displacements are shown in Fig. 18. As indicated, experimental response is shown by the dashed line and analytical response by the solid line; the agreement between the two is quite good. The amplitudes of relative displacement are also seen to be similar for the two base conditions.

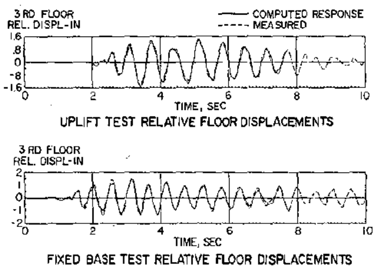
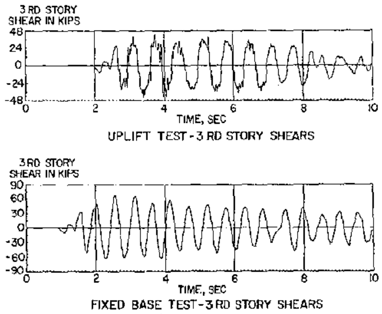


Fig. 18 Comparison of Third Floor Relative Displacement (1 in. = 2.54 cm)

Selected story shears and base overturning moments are shown in Figs. 19 and 20. As can be seen, column uplift has a 'fuse' effect on structural response. The overturning moment was about twice as great for the fixed base test as compared to the uplift test. The maximum exterior column uplift which occurred was about 1 in. (2.5 cm). Column uplift is thus seen to have a significant beneficial effect. The cost of supplying anchorage for the columns is eliminated; the loads and subsequent ductility demand on the structure are reduced for a major seismic loading. Current analytical techniques are adequate to predict the resulting nonlinear response.



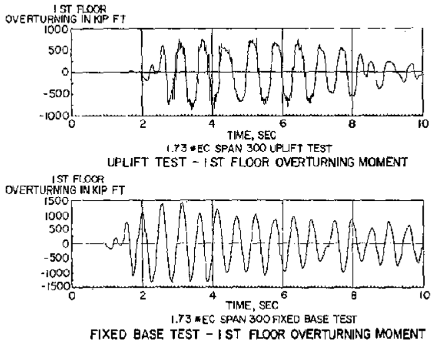


Fig. 19 Nine-Story Steel Frame - Comparison of Third Story Shears (1 in.=2.54 cm; 1 kip = 4.448 kN)

Fig. 20 Nine-Story Steel Frame - Comparison of First Floor Overturning Moments (1 in. = 2.54 cm; 1 ft = 0.3048 m; 1 kip = 4.448 kN)

Concluding remarks--The experimental results show that the allowing of a model structure to uplift from its foundation during earthquake response is a potential means of significantly reducing the strength and ductility requirements, thus enabling more economic design and construction of structures. Results from preliminary analytical studies confirm the experimental findings.

The applicability of these research results in seismic-resistant design and construction of actual buildings can be assessed from two different points of view. In one view, the experiments conducted are intended to simulate a prototype situation in which the steel superstructure frame is supported by a rigid concrete foundation mat or basement structure. Although the use of uplifting column base devices is a significant departure from current practice, the simplicity of these mechanisms will enable them to be reliably reproduced in the field. Therefore, in this case the device can be interpreted as being an exact replica, on a small scale, of a practical isolation or input controlling mechanism [1]. Before adopting this partial isolation technique in practice, however, it will be necessary to conduct comprehensive studies on the behavior of the uplift devices under all kinds of ground motions, particularly those with severe pulse-like excitations.

Alternatively, these tests can be considered to represent a simplified idealization of an extreme case of uplift in situations where standard types of individual column footings are embedded in the soil. The promising results obtained here demonstrate the feasibility of permitting footing uplift to take place, but additional studies will be needed to examine the behavior under more realistic foundation conditions.

In performing model studies on standard footing systems, new techniques will have to be developed for simulating three-dimensional soil-foundation interaction problems, since the capacity of available earthquake simulators is not adequate to permit testing of a large volume of soil. The importance of this problem points out the need for a large earthquake simulator where actual soil-structure systems can be properly simulated and tested.

<u>Feasibility study of 100 ft x 100 ft (30.5 m x 30.5 m) earthquake simulator [19,20]</u>--In 1966 Penzien, et al. [19] began studying the feasibility of designing and constructing a large-scale earthquake simulator facility capable of subjecting large-scale structures to strong seismic motions. During the course of this study the following basic criteria were arbitrarily established, and these are indicated in Table 1.

Note that the selected test structure weight of 4,000 kips (17 792 kN) corresponds roughly to that of a full-scale, three-story reinforced concrete building, 100 ft x 100 ft (30.5 m x 30.5 m) in plan. From the studies carried out it became evident quite early that the technical feasilibity of a large shaking table system would depend primarily upon the development of an adequate electrohydraulic servo-controlled system. Preliminary studies conducted by Penzien and the MTS Systems Corporation [19] showed that a servo-controlled system could indeed be designed which would provide adequate control for the table. The construction and performance of the present 20 ft x 20 ft (6.1 m x 6.1 m) facility at Berkeley has proven the technical feasibility of building a much larger table which could provide three components of simultaneously simulated earthquake motions [20].

<u>Critical observations regarding above design</u>--The authors believe that experimental research needs in ERCBC and in other areas of earthquake engineering justify the final design and construction of a large earthquake simulator facility. However, the experience gained through the use of the 20 ft x 20 ft (6.1 m x 6.1 m) facility at Berkeley, as well as through other medium- and large-scale facilities available elsewhere, and analysis of results obtained in analytical investigations of earthquake damage indicates that some of the basic design criteria established above must be modified. Particularly, it will be necessary to increase the velocity and displacement requirements. The

TABLE 1 EARTHQUAKE SIMULATORS

CHARACTERISTICS		FACILITIES IN						
		U. S. A.		JAPAN				
		EXISTING (BERKELEY)	FEASIBLE	EXISTING (TSUKUBA)	UNDER CONSTRUCTION (TSUKUBA)	UNDER DESIGN (TADOTSU)	FEASIBLE	
TABLE DIMENSIONS		20'x20' (6.1x6.1 m)	100'x100' (30.5x30.5 m)	49.2'x49.2' (15x15 m)	19.7'x19.7' (6x6 m)	49.2'x49.2' (15x15 m)	65.6'x65.6' (20x20 m)	98.4'x98.4' (30x30 m)
TABLE WEIGHT		100 kips (444.8 kN)	2,002 kips (8 906 kN)	352 kips (1 568 kN)				
TEST STRUCTURAL WEIGHT	H	100 kips (444.8 kN)	4,000 kips (17 792 kN)	1,100 kips (4 893 kN)	120 kips (534 kN)	2,205 kips (9 810 kN)	4,405 kips (19 595 kN)	11,013 kips (48 987 kN)
	V			441 kips (1 962 kN)				
NUMBER OF MOTIONS COMPONENTS		2 simult. 1 H & 1 V	3 simult. 2 H & 1 V	2 independ. 1 H & 1 V	2 simult. 1 H & 1 V	2 simult. 1 H & 1 V	2 simult. 1 H & 1 V	1 H
MAXIMUM ACCELERATION	H	0.75 g	2/3 g	0.55 g	1.2 g	1.8 g	0.8 g	0.5 g
	V	0.5 g	2/9 g	1.0 g	0.8 g	0.9 g	0.4 g	
MAXIMUM VELOCITY	H	25 in/sec (63.5 cm/sec)	25 in/sec (63.5 cm/sec)	14.6 in/sec (37 cm/sec)	23.6 in/sec (60 cm/sec)	29.5 in/sec (75 cm/sec)	13.8 in/sec (35 cm/sec)	7.8 in/sec (20 cm/sec)
	V	19 in/sec (38.1 cm/sec)	19 in/sec (38.1 cm/sec)		15.7 in/sec (40 cm/sec)	14.8 in/sec (37.5 cm/sec)	6.9 in/sec (17.5 cm/sec)	
MAXIMUM DISPLACEMENT	H	± 5 in (±12.7 cm)	± 6 in (±15.2 cm)	± 1.1 in (±3 cm)	± 3.9 in (±10 cm)	± 7.9 in (±20 cm)	± 6 in (±15 cm)	±11.8 in (±30 cm)
	V	± 2 in (± 5.1 cm)	± 3 in (± 7.6 cm)		± 1.9 in (± 5 cm)	± 3.9 in (±10 cm)	± 3 in (±7.6 cm)	

main reason for building such a facility is to obtain data on the nonlinear behavior and collapse mechanisms of different soil-building systems. According to results of recent investigations [21] and those of the test of reinforced concrete frames conducted in the earthquake simulator at Berkeley, to induce significant structural damage and impending collapse to models of real structures it is necessary to have ground motions with peak velocities and displacements considerably higher than the limits selected above for the horizontal direction, i.e., 25 in./sec. (63.5 cm/sec.) and ±6 in. (±15.2 cm).

Largest Japanese simulator--Of the several medium-scale earthquake simulator facilities in use in Japan, the largest has been in operation since 1970 and is located in the laboratory of the National Research Center for Disaster Prevention at Tsukuba [22,23]. The table is 49.2 ft x 49.2 ft (15 m x 15 m) and can support test structures weighing up to 1,100 kips (4 893 kN) for horizontal motions and 441 kips (1 962 kN) for vertical vibrations.

The table is driven by eight actuators (vibrators). Four of them are used to produce horizontal displacements in one direction and the other four for producing vertical displacements. These two groups of actuators are controlled by an electrohydraulic servo-controlled system. The testing structure can be subjected to only base motions in one of the above two directions. The maximum force that can be applied is 793 kips (3 527 kN) in either horizontal or vertical direction, the maximum velocity is 14.6 in./sec. (37 cm/sec.), and the maximum displacement is 1.1 in. (3 cm). As postulated by Watabe [23], this limited stroke of 1.1 in. (3 cm) is the most serious drawback of this facility. Due to the stroke and velocity limitations, the maximum possible accelerations are those indicated in the graph of Fig. 21. As can be seen for a test structure of 11,020 kips (49 017 kN), the maximum acceleration that can be delivered is about 0.55g. Because of the limited stroke, this shaking table cannot be used effectively to test structures to collapse.

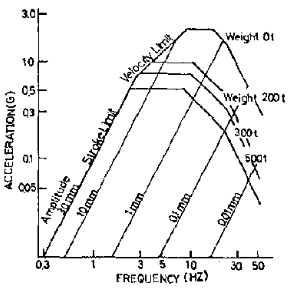


Fig. 21 Acceleration Capacity of Existing Shaking Table at Tsukuba, Japan

Earthquake simulators under construction or design in Japan [24,25,26]--A two-dimensional shaking table, 19.7 ft x 19.7 ft (6 m x 6 m) in plan, has been designed and is being constructed at the National Research Center for Disaster Prevention. The table is designed to have a maximum test weight of 120 kips (534 kN) with a capability of simultaneously simulating components of ground motion vibrations up to maximum displacements of ±3.9 in. (±10 cm), in the horizontal direction, and ±1.9 in. (±5 cm), in the vertical direction, and delivering maximum accelerations of up to 1.2g and 0.8g, respectively.

Feasibility studies on very large earthquake simulators were conducted in Japan in 1973 [25]. The two types indicated in Table 1 were concluded to be feasible. The design of one large facility for testing structures for nuclear power plants is presently underway at the Center for Nuclear Safety Engineering Research is and expected to be fully operational by 1980. The facility will be built at the Engineering Test Center in Tadotsu Town. Initial construction costs were estimated at 80 million U.S. dollars, although latest estimates are closer to 200 million.

The main objective in constructing such a huge facility is to conduct experiments on large-scale models of large equipment and components for nuclear power plants to determine their safety against earthquakes. As summarized in Table 1, this shaking table will have dimensions of 49.2 ft x 49.2 ft (15 m x 15 m), with a maximum test weight of 2,205 kips (9 810 kN), and maximum strokes of $\pm$7.9 in. ($\pm$20 cm) and $\pm$3.9 in. ($\pm$10 cm) in the horizontal and vertical directions, respectively. This table will be capable of simultaneously reproducing components of ground motions in both vertical and one horizontal direction up to an acceleration of 1.8g, horizontally, and 0.9g, vertically, and with velocity limits of 29.5 in./sec. (75 cm/sec.) and 14.8 in./sec. (37.5 cm/sec.) in the horizontal and vertical directions, respectively.

It is believed that the construction and functioning of this or a similar facility will permit investigations to be conducted on the behavior of whole soil-building systems so as to study the behavior of soil-foundation and soil-structure interaction, which is one of the areas where reliable data are needed to improve seismic-resistant design and construction of buildings.

ADVANTAGES AND DISADVANTAGES OF EARTHQUAKE SIMULATOR FACILITIES FOR ERCBC

The obvious advantage of using earthquake simulators is that test structure models can be subjected to 'true' earthquake types of ground excitations so that the actual distribution of forces can be measured and the nonlinear behavior and damage mechanisms of the model can be observed. To discuss the limitations of earthquake simulators it is convenient to classify these experimental facilities by their scale.

Small- and Medium-Scale Facilities

The principal limitation of all existing small- and medium-scale facilities is that only relatively small-scale models of building components can be tested. The dynamic testing of models in the nonlinear range in compliance with the requirements imposed by the laws of dimensional similarity is difficult and costly. Sozen, in a paper presented to this workshop [7], has discussed in detail the advantages and disadvantages of existing medium- and small-scale facilities. Although the 20 ft x 20 ft (6.1 m x 6.1 m) shaking table at Berkeley permits testing of basic components of engineered and non-engineered buildings of one or, at the most, two-story buildings in full-scale, the limited velocity and displacement capacity of the table do not, in most of the cases, permit severe damage to be induced in the structure. Furthermore, none of the existing medium-scale facilities can be used to carry out studies of the behavior of actual soil-building systems.

Large-Scale Facilities

There is no doubt that a facility such as the one being designed by the Center for Nuclear Safety Engineering Research in Japan would be superior to any of the small- or medium-scale simulators presently available. However, it would be highly desirable to develop a facility which would permit testing of structures weighing up to 4,408 kips (19 620 kN) and capable of developing velocity in the horizontal direction of up to about 59.1 in./sec. (150 cm/sec.). A simulator of this type would facilitate investigations on soil-structure interaction since it will permit large numbers of soil layers to be built up on the shaking table. For example, such a facility could accommodate soil layers 40 ft x 40 ft (12.2 m x 12.2 m) in plan, having a total depth of 15 ft (4.58 m), weighing about 2,900 kips (12 899 kN), and yet still maintain an available 1,500 kips (6 672 kN) for the building model. Tests could therefore be performed on full-scale models of single bay structures up to 10 stories tall or one half-scale models of two-bay structural prototypes.

If one or more of these large earthquake simulators becomes available in the near future, its use will still be restricted to proof-type testing and to the study of specific problems such as soil-structure interaction under actual seismic excitations for which no other facilities are available. For most structural types, parametric studies of their mechanical behavior may be more efficiently carried out using large-scale loading facilities, since the use of earthquake simulators is not only very expensive for such studies, but, as in the case of dynamic testing, has the basic disadvantage that the input motion and/or the recording instruments have a high probability of malfunctioning due to their complexity. These limitations, coupled with the difficulty of observing the sequence of damage during any test due to its short duration, indicate that it would also be convenient to have other facilities available in which the dynamic excitations are replaced by equivalent pseudo-static excitations.

LARGE-SCALE CONTROLLED LOADING TEST FACILITIES

Detailed parametric studies of the seismic behavior of selected structural elements or complete units of components of buildings can be performed more effectively and at less cost through the use of controlled loading systems. Such facilities will enable a programmed history of external forces or deformations to be directly applied throughout the test structure to simulate the effects of earthquake shaking. Depending upon the time rate at which these forces or deformations are applied, the facility can be classified as being either dynamic or pseudo-static. An inherent problem in the use of all controlled loading facilities is the selection of the test loading. In principle, the applied forces should induce stresses and deformations in the test component which are similar to those caused in the real structure by seismic excitations. However, it is very difficult to predict the combination of loads needed to produce the actual state of stress. Usually, the only recourse is to simulate the critical combination of forces that could develop at a certain time and then to vary the intensity of these forces according to simple time functions. Rational selection of this critical combination requires integrated experimental and analytical studies because this combination will vary depending upon the specific subject of the study.

Furthermore, even if a rational combination of forces could be selected, the problem of appropriately varying the magnitude of these forces would remain. It is well known that the behavior of reinforced concrete is very sensitive to the loading path [27]. To select the proper load sequence, a simplified mathematical model is first assumed, and the earthquake response of the test component is calculated. Based on test results obtained by using this calculated load sequence, a new mathematical model is formulated and the analysis is repeated, this sequence being followed until satisfactory convergence is achieved. A significant new development in controlled loading tests has been reported by Okada and Seki [28]. It consists of a computer-actuator on-line system which allows the displacement command signals to be consistent with the test structure's response to a prescribed earthquake excitation. Thus, the command signals are coupled to the actual force-deformation characteristics of the structure being tested. This new technique has been applied successfully to a simple case where one simple actuator was required to simulate the effect of earthquake motions.

Dynamic and Pseudo-Static Loading Facilities

If loads are applied according to the time variation rate expected during the response of a structure, the testing facility can be classifed as being dynamic. If the load sequence is applied slowly, however, the facility can be denoted as being pseudo-static. In the latter case, the effects of strain rate, $\dot{\varepsilon}$, and velocity-dependent damping, $\xi(t)$, will be negligible. The main advantage of experimentally studying the hysteretic behavior of structures under pseudo-static excitations is that the test can be stopped at any time to check the instrumentation, recording devices, state of the specimen's damage, etc. Thus, it is possible to change the program of applied excitations according to the results obtained during the tests. Furthermore, a better picture of the actual mechanisms of stiffness and strength deterioration and of failure of the structure can be obtained. Before accepting the results of pseudo-static tests, however, it is necessary to study the time-rate effects. This is accomplished through the combined use of dynamic and pseudo-static loading tests.

The hysteretic behavior of the following critical regions of reinforced concrete structures have been investigated using different rates of strain: (1) regions subjected to pure bending moments [29]; (2) flexural regions subjected to different degrees of shear force [30]; and (3) regions subjected to combined flexural, shear, and axial forces [31]. From the results obtained to date, it can be concluded that the principal effect of an increase in strain rate on the hysteretic behavior of reinforced concrete flexural regions subjected to different magnitudes of shear and axial forces is to increase the moment capacity at first yielding of the reinforcement. Although studies of the effect of strain rate should be continued to determine the actual increase in strength, to avoid economically undesirable overconservatism in design and construction, according to the results available it is clear that comprehensive experimental studies utilizing pseudo-static testing procedures can be carried out on the behavior mechanisms for stiffness and strength degradation as well as for failure of critical regions of reinforced concrete elements subjected to severe seismic actions. However, some precautions should be noted. In interpreting the results so obtained it is necessary to recognize that although neglecting the observed increase in flexural capacity is conservative from the point of view of the bending capacity design, it is not so for detailing of the reinforcement required to resist the shear or axial forces that can be developed in the same regions or members. Moreover, the effect of strain rate on bond deterioration and on the behavior of the anchorage and splicing of the main reinforcement should be investigated.

Static and Dynamic Tests on Subassemblages

Comprehensive studies of the hysteretic behavior of large buildings by means of destructive pseudo-static testing is very costly. Thus, such studies should be conducted only on the basic subassemblage of such buildings. The type of subassemblage to be studied depends on the structural system used. There have been significant and steady advances in knowledge of the hysteretic behavior of moment-resisting frames, infilled frames, braced frames, and wall-frame systems within the past five years by testing planar subassemblages of these systems. Versatile loading facilities have been developed [3] which permit highly sophisticated and precise pseudo-static, and even dynamic, loading tests to be conducted on such planar subassemblages.

Perhaps the larger and more complex testing facilities that have been developed in the U.S. are those used for studying braced frames and structural wall systems. A brief description of the facilities developed at Berkeley for this purpose follows.

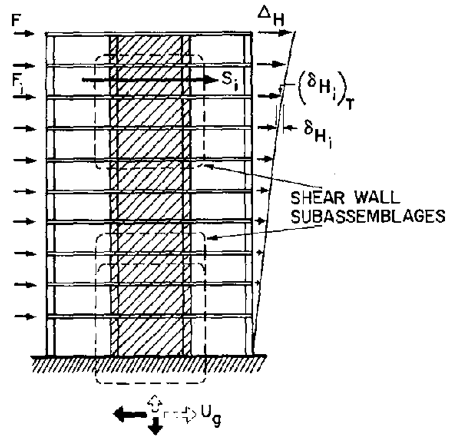
Testing facility for braced frames and structural wall systems--Adequate lateral strength and stiffness for tall slender buildings can be economically achieved using braced frames or structural wall systems. To obtain basic information for predicting the in-plane seismic behavior of these two systems, it is necessary to test subassemblages such as those indicated in Fig. 22. The first problem encountered in selecting these subassemblages is the simulation of actual boundary conditions. Solution of this problem usually requires the use of specimens at least two, three, or four stories high. Another problem requiring careful consideration is the selection of the loading conditions to be applied to the specimen [13,32-34].

The testing facility used at Berkeley for studying the in-plane seismic behavior of these wall subassemblages is shown schematically in Fig. 23. The principal feature of this system is its ability to simulate pseudo-statically the significant loading conditions which are present due to gravity and changes in environmental conditions, as well as those that are induced during earthquake ground shaking. The specimens are tested in a horizontal position. The bottom part of the specimen is anchored to reaction blocks and vertically supported by the floor slabs, which slide on low-friction pads. As indicated in Fig. 23, servo-hydraulic actuators, which are positioned between the specimen and a set of reaction blocks anchored to the laboratory floor, are arranged so as to apply axial loads, which simulate both static gravity effects and dynamic alternating forces due to seismic overturning moments. A lateral actuator is provided at the top of the test specimen to apply shear forces simulating the seismic shear transmitted from the stories above; two other lateral actuators attached to the floor slabs simulate the inertial forces developed at these levels. All the lateral actuators are electronically coupled with the axial actuators so that dynamic shears and overturning moments will act in phase. The test setup is described in detail in reference 13. With very slight modifications, this test setup can also be used for dynamic testing.

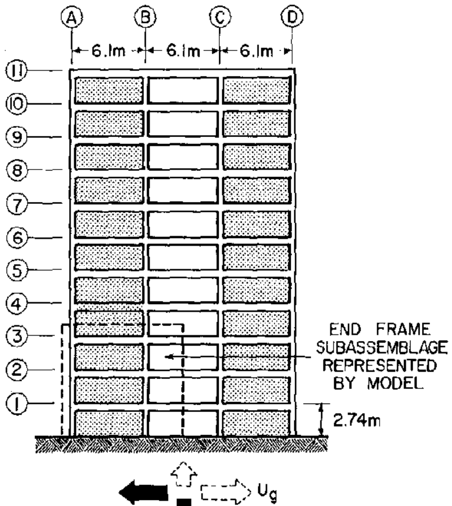
A high-speed data acquisition system has been developed to be used with this as well as the other controlled loading facilities at Berkeley. This system is a computer-controlled multichannel analog data system. A maximum of

88 low-level transducers (strain gages, load cells, etc.) and 40 high-level transducers (linear variable differential transformers, linear potentiometers, etc.) can be read on command. All 128 channels can be read in about 6.4 milliseconds. Extensive data reduction and plotting facilities have also been provided.

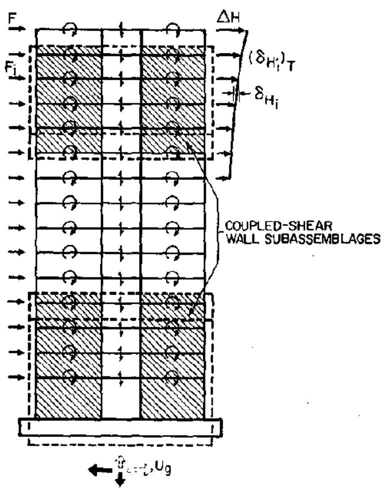
A series of tests has already been conducted on one-third-scale models of wall subassemblages of a ten-story, reinforced concrete frame-wall structural system. Results obtained in these tests are described in references 13 and 32. By carrying out additional free vibration tests at different stages of damage, it has been possible to obtain valuable information regarding variations of the period of vibration and of damping with increased



(a) Shear Wall



(b) Infilled Frame



(c) Coupled Shear Wall

Fig. 22 Shear Wall, Infilled Frame, and Coupled Shear Wall Subassemblages

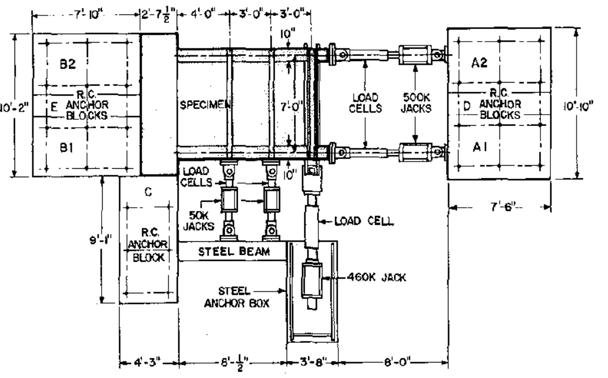


Fig. 23 Test Setup for Wall and Frame
(1 in. = 2.54 cm; 1 ft = 0.3048 m;
1 kip = 4.448 kN)

damage. The facility has also been used for testing one-third-scale models of both reinforced concrete frames infilled with reinforced masonry [33] and braced steel frame planar subassemblages.

A large controlled loading facility has been designed and is being constructed to test coupled-wall subassemblages [34]. The test setup is illustrated in Fig. 24. In this facility the subassemblage specimens are tested in their actual vertical position.

The possibility of using these controlled loading facilities together with the computer on-line technique developed by Okada and Seki [28] enhances the future of this type of testing facility. Now that the technology has been developed and applied to loading facilities for testing planar subassemblages, its application should be extended to the development of a large, three-dimensional, pseudo-static testing facility. This facility should permit

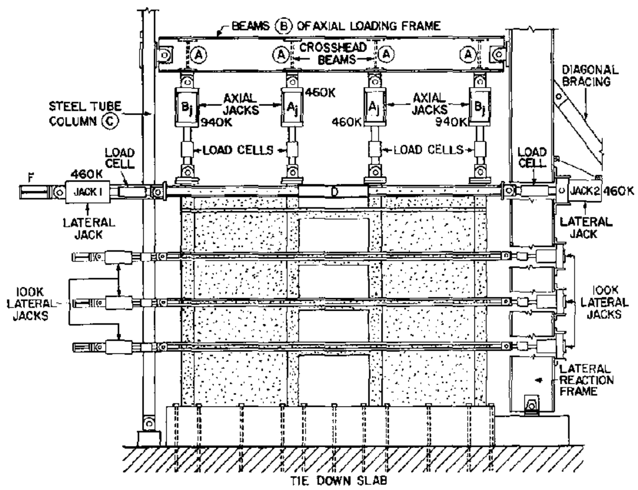
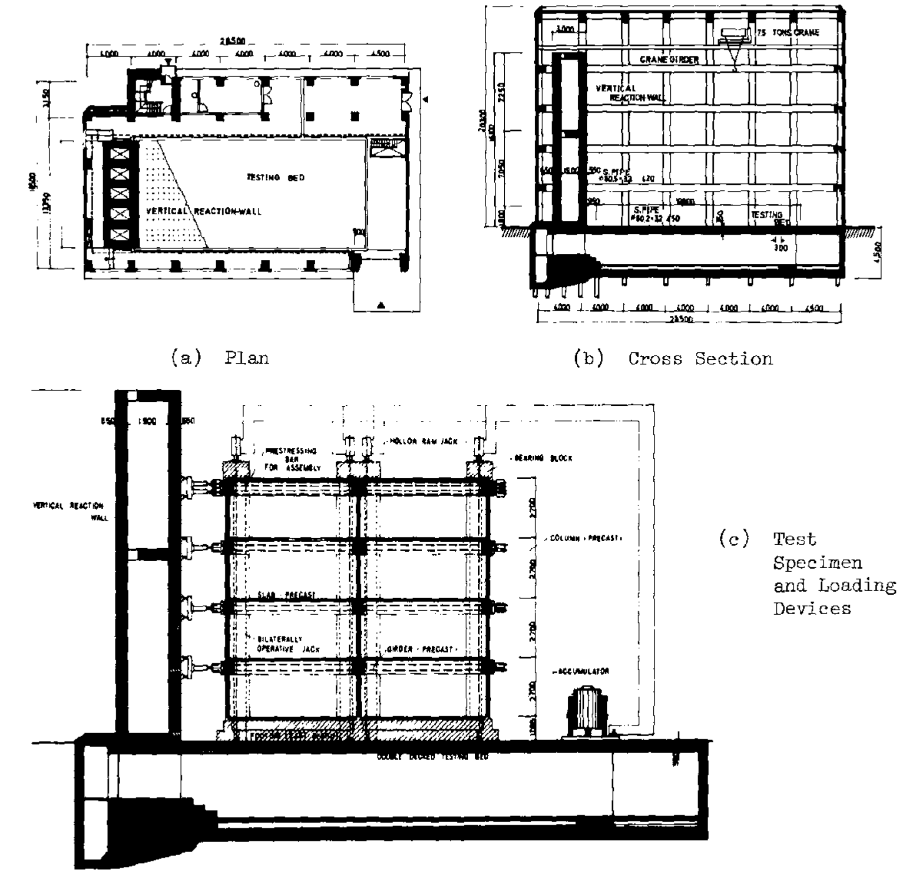


Fig. 24 Test Setup for Coupled Walls
(1 kip = 4.448 kN)

single and multiple story space subassemblages to be tested by subjecting them to forces in the vertical and two horizontal directions. The hysteretic behavior of columns under biaxial bending and associated shear and that of joints under three-dimensional actions; the effect of the interaction between perpendicular wall elements and floor systems in the lateral stiffness and strength of the whole building; and the interaction between structural and nonstructural elements to determine what controls the amount of acceptable ductility--these are just some of the problems that need to be investigated and which require such a large, three-dimensional loading facility.

Full-size buildings or large-scale models--Since 1967, Japanese researchers have been carrying out pseudo-static tests on full-size apartment buildings up to five stories high [1] using the facility illustrated in Fig. 25.



(a) Plan

(b) Cross Section

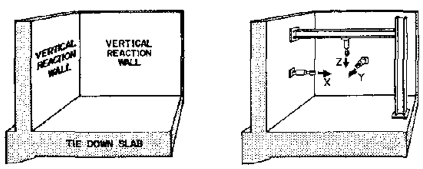
(c) Test Specimen and Loading Devices

Fig. 25 Large-Scale Pseudo-Static Facility of the Building Research Institute, Tokyo

In most of the tests, repeated reversed lateral forces of a preselected fixed pattern were used. The magnitude of the forces was increased in steps. The advantage of using this method is that after each step, the building can be subjected to free and/or forced vibration by means of shakers, thereby making it possible, at each time step, to obtain the variation of period and of damping with the amount of damage induced in the building. The results of these tests have clarified the probable seismic behavior of highly complex structures fabricated from cast-in-place reinforced concrete, precast reinforced concrete, and precast concrete with prestressed construction systems.

It is doubtful that the observed interaction between the different components of these structures could have been predicted analytically or by means of separate tests of their individual structural components. Problems similar to these are being confronted by researchers throughout the world. In the U.S., for example, large panel precast concrete buildings are now considered economically and architecturally viable systems of construction. Although these types of buildings are potentially able to resist severe ground motions with controllable damage, realization of this potential will require extensive research. Researchers at the Massachusetts Institute of Technology, who are involved in the development of advanced dynamic modeling techniques capable of estimating the full range of potential seismic response of these panelized structures, have concluded, after preliminary studies [35], that the successful evolution of these techniques depends on the availability of reliable test data.

It is believed that only tests on full-size or large-scale models of this industrialized type of building and on its components can produce the required data. The need for large- rather than small-scale models is due to the fact that the inelastic behavior of structure--particularly when reversal of deformation occur--is very sensitive to the detailing, which is very difficult to simulate at reduced scales. Thus, a large pseudo-static facility that will permit the applications of multidirectional deformations or loadings as discussed above should be developed. This can be accomplished with the arrangement illustrated schematically in Fig. 26. This type of facility would permit the application of horizontal biaxial deformations, as well as vertical loading by simply attaching auxiliary steel frame elements to the permanent walls and the tie-down slab. The variation of the dynamic characteristics at the different levels of damage induced during the pseudo-static test of a model can be determined by conducting ambient and forced vibration tests. To obtain the variation of dynamic characteristics with large amplitude vibrations, it is necessary to develop shakers more powerful than those presently available.



(a) (b)

Fig. 26 Pseudo-Static Facility for Testing Large-Scale Specimens under Three-Dimensional Deformations

Large-Scale Three-Dimensional Controlled Loading Facilities

U.S.--At present there is only one facility that will permit three-dimensional controlled loading experiments to be conducted on large-scale models of subassemblages or buildings. This facility, which is being constructed at the Civil Engineering Research Laboratory, Balcones

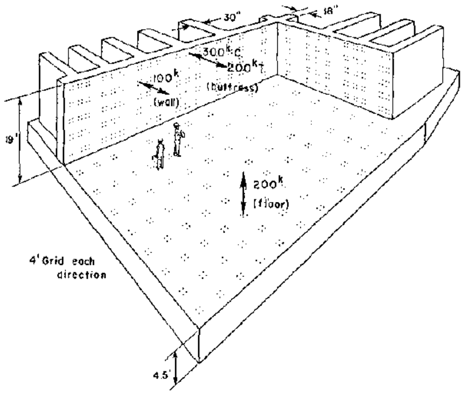


Fig. 27 Structural Floor-Wall Reaction System at the University of Texas, Austin (1 in. = 2.54 cm; 1 ft = 0.3048 m; 1 kip = 4.448 kN)

Research Center, University of Texas, Austin, consists of a structural floor-buttressed wall system illustrated in Fig. 27. The anchorage capacities in the walls and floor are also indicated. The floor-wall system will be served by a computer-controlled data acquisition and closed-loop hydraulic loading system. This test facility has been developed to conduct a comprehensive investigation on the behavior of reinforced concrete frame elements under biaxial loads [36]. The use of floor-buttressed wall systems, together with specially constructed loading frames such as the one illustrated in Fig. 26(b), will enable studies of multidirectional loading histories to be conducted on large three-dimensional subassemblages or up to two stories of full-scale three-dimensional structural systems. It would be highly desirable to develop a similar facility with a large height capacity, particularly for studies of tall panelized buildings.

Japan--The present facility shown in Fig. 25, does not permit application of three-dmensional loads or deformations. A new facility is being constructed in Tsukuba. This facility occupies in plan 210 ft x 164 ft (64 m x 50 m) and has a total height of 123 ft (37.5 m) above the ground level and 22.5 ft (6.85 m) below the ground level. The plan and sections of the floor-reaction wall system is illustrated in Fig. 28. The loading floor is 168 ft x 65.6 ft (51.2 m x 20 m) in plan and has a thickness of 22.5 ft (6.85 m). This loading floor is divided into two zones (I and II) by one of the reaction walls (A). Zone II, which has a clear floor area of 65.6 ft x 50.5 ft (20 m x 15.4 m), has two reaction walls (A and B) at right angles, with a height above the loading floor of 82 ft (25 m) to allow full-scale building units of up to nine stories to be tested. Zone I has a clear floor plan of 80.7 ft x 65.5 ft (24.6 m x 20 m) and has only reaction wall (A). The loading, deformation, and data acquisition will be directly controlled by a computer system.

RESEARCH AND DEVELOPMENT NEEDS

The ultimate objective of research in ERCBC should be to develop methods of design and economic construction which produce functional seismic-resistant buildings. Two types of buildings should be considered: non-engineered (low-cost) and engineered. Achievement of this goal entails considerable development and analytical and experimental studies which should be fully integrated.

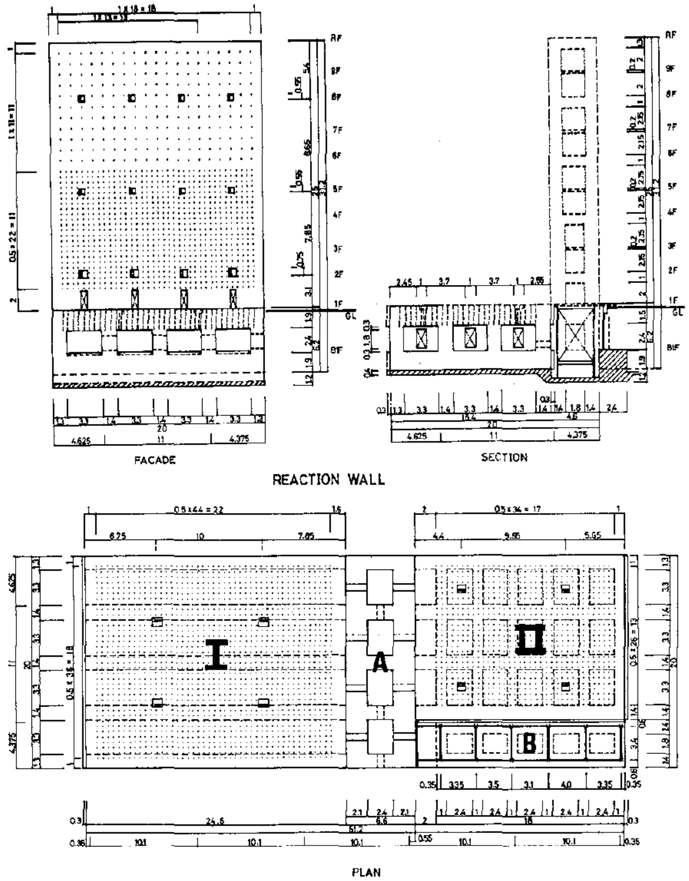


Fig. 28 Large-Scale Testing Facility at Tsukuba, Japan

Nonengineered Buildings

The construction of low-cost earthquake-resistant housing is one of the most pressing problems for engineers. The main problem is to provide adequate continuity, i.e. anchorage, to the various components of a building. This involves proper detailing of the connections (joints) of the different components. New methods of anchoraging should be developed through full-scale testing of building components in loading facilitiies. The reliability of these new techniques and possible improvements of established methods can be studied by final tests of the whole building using medium- or large-scale earthquake simulator facilities.

Engineered Buildings

In these cases involving defined structural systems, prediction of building response to earthquakes necessitates study of not only the bare building's structural system, but also of the entire soil-building system.

To predict analytically the hysteretic behavior of a building, it is necessary to study experimentally the behavior of the soil-building system and the building's components subjected to seismic actions so that appropriate mathematical models may be devised. Despite increased knowledge on the hysteretic behavior of structural elements and planar subassemblages, data are insufficient to predict the three-dimensional inelastic behavior of most buildings.

For rapid improvement in the field of ERCBC, there is a need for a national laboratory for large-scale experimentation which should include both the largest possible earthquake simulator and the largest possible three-dimensional loading facility. The earthquake simulator is needed for:

1. Studying the soil-building system. This is to include soil-structure interaction and the effect of rotation, translation, settlement, uplift, and/or sliding of the foundation on the dynamic response of the building.

2. Studying up to collapse new building systems whose analytical behavior is difficult to predict. One example of such a system would be precast concrete panel buildings.

3. Studying the response of components and equipment and their effects on the response of a structure. There is an urgent need for quantifying the damage indices required for establishing damageability limit states.

4. Testing nonengineered buildings up to collapse.

The three-dimensional loading facility is needed for:

1. Carrying out comprehensive parametric studies on the hysteretic behavior of three-dimensional subassemblages or complete basic units of reinforced concrete buildings under multidirectional loading, particularly of precast concrete panel buildings.

2. Carrying out comprehensive parametric studies on the interaction between structural and nonstructural components to different types of buildings.

While the development, design, and construction of a large-scale, three-dimensional controlled loading facility could be carried out immediately, it is suggested that the final design and construction of the earthquake simulator be carried out only after performing a thorough and detailed feasibility study of the largest possible facility of this type. The need for this study is dictated by the need for a large shaking table on which the above studies can be conducted, using realistic models of soil-building systems. The development of this earthquake simulator will be so costly that the economic restrictions should be carefully weighed. It is also believed that the two large facilities--the controlled loading and the earthquake simulator--should be built in the same location, the desired location to be selected considering the technicians needed for its functioning and maintenance as well as the research and professional environment for its efficient use.

To this end the result should be part of an integrated effort on the part of researchers, practicing engineers, manufacturers, and government and building developers.

ACKNOWLEDGMENTS

Appreciation is expressed to Dr. M. Watabe for supplying the information pertinent to the facilities in Japan, and to A. Hucklebridge for performing some of the data reduction on the uplift tests. The financial support of NSF is gratefully acknowledged.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

EXPERIMENTAL RESEARCH NEEDS FOR EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION

by

Helmut Krawinkler
Assistant Professor of Civil Engineering
Stanford University

INTRODUCTION

Much experimental work has been done during the last 15 years on existing structures and in laboratories of universities, government and industry, directed towards achieving a better understanding of the response of structures to seismic excitations. All of this experimental work, whether basic or applied research or seismic proof testing, is part of a concerted effort to develop - as expressed in NSF guidelines - methods and techniques that can provide more cost-effective protection for man and his works from the life loss, injury, property damage, social dislocation, and economic and ecological disruption caused by natural hazards and disasters.

The ERCBC workshop provides an excellent opportunity to look back and ahead and evaluate what has been and what needs to be done towards achieving the aforementioned objective. Reading the preprints for this workshop, it is clear that extensive well documented experimental work has been done and abundant qualitative as well as much quantitative information can be extracted from research reports and technical papers. However, surprisingly little continuity and coordination can be found between individual research efforts, with much duplication in some areas and visible gaps in others. Also very few of the recent research findings have lead to significant changes in regulatory documents. Perhaps this is acceptable to some degree, since large segments of the engineering profession repudiate the drafting of elaborate codes which could be considered restrictive to good engineering judgement.

Nevertheless, so much experimental and analytical research has been done in the recent past that the individual user cannot keep abreast with important recent development. Workshops, such as this ERCBC workshop, are essential at this time to organize and disseminate research information for professional use. Ideally, such workshops should lead to the dissemination of recommendations and comprehensive but concise state of the art reports drafted by working groups. It is equally important that these recommendations and state of the art reports be updated at regular intervals.

As far as experimental research is concerned, this workshop should provide an opportunity to coordinate individual research efforts, explore the feasibility and limitations of experimental techniques, identify those topics where experimental research is most needed, and suggest guidelines for future research which ultimately should result in a complete set of information needed for the design of serviceable and safe structures in seismic environment.

Although individual research projects and specific experimental facilities are discussed extensively in the preprints of this workshop, not much is said about the overall scope and objective of experimental research in ERCBC. To my knowledge, a research report by Professor Bertero [1] is the only publication available in U.S. literature which addresses itself fully to this subject. In this paper I would like to add a few subjective thoughts to this comprehensive discussion of research needs.

Conceptually, the scope of experimental research in ERCBC appears to be well defined. Information on the constitutive properties of the component materials and the interactions between the component materials (bond and crack propagation) theoretically should suffice to solve mathematically any problem in ERCBC by continuum mechanics principles. Our inability to formulate simple and reliable mathematical models based on continuum mechanics principles makes it necessary to derive models for the load-deformational response of individual elements from experimental measurements. Uncertainties in predicting the interactions between individual elements necessitates experimental research on subassemblies and complete structures. The importance of structural detailing on the performance of ERCBC adds another dimension to experimental research. If we now add cyclic response effects, time dependent phenomena, the random nature of seismic events, and the interaction effects between structures and the supporting soil medium, it is evident that we are faced with a multitude of rather complex problems.

Personally I cannot see how these problems can be solved with every researcher doing his own thing. As much as individual initiative and ingenuity is needed, it is equally important at this time to coordinate experimental research efforts, define research needs and objectives, develop a national or perhaps international research program and set several definite research goals.

As an attempt to raise interest and stimulate discussion on this subject, presented below is rough, subjective and certainly incomplete outline of research objectives and a brief discussion of feasible experimental means for achieving these objectives.

OBJECTIVES OF EXPERIMENTAL RESEARCH ON ERCBC

Much of earthquake engineering research is directed towards defining each term in the discretized equation of motion

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = [M]\{\ddot{u}_g\}$$

where $[M]$ ideally should represent the proper mass distribution throughout the structure and the supporting soil layers; $[C]$ ideally should represent all damping contributions which cannot be included with confidence in the formulation of the $[K]$ matrix; $[K]$ ideally should represent all stiffness characteristics of the structure and the supporting soil layers, including geometry and material nonlinearities, stiffness degradations, and time dependent effects, and $\{\ddot{u}_g\}$ ideally should represent the random nature of the six components of ground motion at the boundary of the soil-structure system.

Experimental research is needed to evaluate each of the aforementioned terms and also to arrive at decisions directly applicable to the design process for ERCBC. In particular, the following problems need further study.

Basic Dynamic Characteristics of Structures

Not enough research information is available in the literature on natural frequencies and damping in actual structures. From a designer's point of view, the strength design of structures is very sensitive to period and damping estimates. Presently available equations for estimating the fundamental period (0.1N and $0.05h_n/\sqrt{D}$) are inadequate, and period determination based on stiffness characteristics of resisting elements will rarely do justice to structure-floor system -non structural element interaction. Also, the soil stiffness should be considered in the period determination, particularly for shear wall type structures.

To complicate matters further, we have to live with the fact that natural periods and damping are severly dependent on the vibration amplitude. To my knowledge nobody has yet taken a close look at the consequences of different designs which can result from period and damping estimates based on either low amplitude vibration (uncracked section) or vibrations causing design level forces. Perhaps our presently accepted philosophy of basing period calculations on uncracked sections is too conservative. Or are we trying to balance one conceptual error - that of ignoring the stiffening effect of nonstructural elements - with another conceptual error - that of overestimating the stiffness of the structural elements? A well planned testing program on existing structures together with an analytical study could shed significant light on this problem.

Soil-Structure Interaction Effects

Many different effects of major importance on the structural response can be lumped together under this name. To name a few, the increase in the fundamental natural period of the structure due to the soil compressibility (as mentioned in the previous paragraph), the increase in effective damping due to energy dissipation through radiation and hystertic action in the soil medium, and the effects of overturning and uplift tendencies could be included in this topic. Very little is known about each of these effects and reliable mathematical models cannot be formulated without experimental evidence. Thus, as difficult as it may seem at this time, rational experimental means for studying these effects should be developed.

Effects of 3-D Motion

In the past, laboratory experimentation has been limited to response studies for one or two translational components of input motion. Interestingly, most dynamic testing of structures in the post-elastic range was also carried out on symmetric structures which prevented torsional modes of vibration. From damage observations in past earthquakes it is evident that torsional modes of vibration can be critical in ERCBC, particularly at corner columns and re-entrant corners. More experimental work is needed on investigating torsional effects and, in general, the effects of all six components of ground motion.

Properties of Materials and Interaction between Component Materials

Much experimental research has been carried out on low strain rate (static) properties of concrete and reinforcing steel, on bonding between steel and concrete, on crack initiation and crack propagation, and on the effects of confining concrete through closely spaced hoop reinforcement. Most of this work has been done under monotonically increasing loading, some under quasi-static cyclic loading, and very little under dynamic loading. If we ever are to achieve a reliable correlation between material and element behavior under dynamic actions, much more research work needs to be done at the material level under strain rate and loading histories similar to those expected under seismic actions. The importance of such material testing is also evident from the fact that much of the element testing is done under quasi-static loading and reliable conclusions have to be drawn from these tests on the dynamic behavior of elements.

Load-Deformational Response of Structural Elements, Subassemblies, and Complete Structures

There are two main purposes for experimental research in this area: to develop and/or verify mathematical models for the load-deformational response as needed in the formulation of the stiffness matrix $[K]$, and to formulate rational design and detailing criteria for structural elements.

An abundance of experimental data is available on the cyclic response of individual elements, usually obtained from quasi-static cyclic load tests. In this area of significant overlap between individual research projects, it may be time to take a pause in experimental work and take a close look at the accumulated data. An evaluation of these data, which should result in a comprehensive set of user oriented recommendations, is urgently needed.

At this time, a consistent evaluation will be somewhat difficult since the load-deformational response is highly sensitive to the applied loading history and individual experiments were carried out based on rather subjective decisions on load cycling. It is essential to perform a thorough study of loading histories which possibly may occur in various types of structural element under realistic seismic excitations, and which will be critical for the ductility supply of the elements. Such a loading history may be of the incremental collapse type (primarily unidirectional loading) for elements governed by flexure, and of the low cycle fatigue type (symmetric cycling about undeformed position) for elements governed by shear. Perhaps it is possible to develop standardized loading histories such as those used in the New Zealand Loading Code.

Past experimental studies on structural elements have given us much information on how individual elements behave under cyclic loading. Future research, where needed, should be primarily concerned with design improvements which must be economically and constructionally feasible. For instance, ductile moment resisting space frames designed to take 100% of the lateral loads are so difficult to construct that only few have been built in Northern California. Perhaps there are alternatives to conventional means of achieving ductility, such as possibly the use of fiber reinforced concrete at critical regions; or perhaps changes in conventional configurations of frames may lead to a decrease in ductility demands. Very little research has been done on possible alternatives to conventional design.

More experimental research is needed on interaction effects between individual elements, such as three-dimensional framing action, frame-shear wall interaction, and interaction between vertical lateral load resisting units and floor systems. Ideally, such studies should be done on three-dimensional substructures or complete structures, with extensive instrumentation in every element to permit a correlation between element and structure behavior. The feasibility of dynamic or quasi-static testing on prototypes or scale models should be investigated. A thorough conceptual study should be performed of realistic loading histories representing gravity load effects and three-dimensional seismic effects. The effects of boundary conditions in substructure testing must be closely studied and the strain rate effects must be investigated in more detail if we are to gain confidence in predicting the dynamic response from quasi-static cyclic testing.

Determination of Acceptable Damage Levels

It appears that design philosophy is headed towards a two-level seismic design; a design for an earthquake of a certain small probability of occurrence which the structure should resist with "acceptable" damage, and a design for the maximum credible earthquake under which collapse must be prevented. Experimental work is needed for assessing acceptable damage, which will require a thorough study of repairability of structural as well as nonstructural damage.

Seismic Proof Testing

Certain critical facilities such as reactor components, and standardized units and details such as elements and connections in the prefabricated concrete industry will require experimental verification of their integrity and safety under various levels of ground motions. The development of recommendations for standardized seismic test procedures should prove very useful to industry and regulatory agencies. Such recommendations should include information on required material testing as well as on the types of loading histories and the simulation of boundary conditions for component testing.

Many other experimental research needs have not been included in this brief summary, such as work on vibration isolation systems, damping devices, foundations, special structural problems such as interaction between frames and infill walls, strengthening of existing buildings, and reliability of experimental results. Most of these topics are discussed elsewhere in the proceedings to this workshop.

EXPERIMENTAL PROCEDURES AND FACILITIES

Let us assume that we have established the research needs, scope and objectives for experimentation on ERCBC. Then we are faced with the problem of trying to achieve these objectives in an optimum manner. This means, obtaining a complete and reliable set of information by taking advantage as much as possible of already available test facilities and well established test procedures, and creating new facilities as well as developing new test procedures - where needed.

The previously discussed objectives could be achieved through the following types of tests.

Existing Structures - Field Testing

Such structures are gold mines which yet have not been fully exploited. Ambient vibrations, man or vibration generator induced motions, or motions induced by explosions can reliably be used to measure the dynamic characteristics of structures - unfortunately in most cases under small vibration amplitudes. Correlations between measured and computed frequencies and mode shapes should give important information on mathematical modeling techniques and effects of nonstructural elements. Measurements of rotations at the foundation level can be used to evaluate rocking effects. If explosives are used, a comparison between free-field motion and foundation motion can result in valuable information on soil-structure interaction effects.

Ideally we should go treasure hunting for structures which are ordained for demolition and which are designed according to modern codes - with all the design information available. Such a structure was recently tested by Professor Galambos in St. Louis. Experiments on prospective structures of this type are most promising for acquiring a complete set of information on amplitude dependent frequency and damping variations, soil-structure interaction effects (if underground explosions can be used), and structural response.

Test Structures - Field Testing

The two four-story reinforced concrete tests structures tested by John Blume/URS at the Nevada Test Site proved the significant value of field testing of test strcutures excited by vibration generators and underground expolosions. Such experiments are most desirable from the standpoint of design and construction control, instrumentation planning, and realistic representation of soil conditions, but can hardly be included in a general research program due to their excessive cost.

Test Structures and Substructures - Laboratory Testing

Since most field studies are limited to low amplitude testing, it is expected that laboratory experimentation will remain the main source of physical information on ERCBC. The most widely used tools for reproducing seismic effects in the laboratory are earthquake simulators and hydraulic actuators which are used for quasi-static or dynamic load applications at discrete points. Only these two types of motion generators are briefly discussed below, although other viable alternatives such as vibration generators should be considered in laboratory experimental programs.

Earthquake Simulators. In the United States several research laboratories are equipped with earthquake simulators, usually of small size with the largest table measuring 20 ft. x 20 ft. Due to these size constraints, shaking table testing is presently limited to either very simple prototypes, small substructures, or models of prototype structures. We could propose to build larger simulators, but economic considerations (it has been estimated [2] that the total cost of shaking table experimentation increases roughly with the square to cube of the linear table dimension) seem to outweigh the prospective benefits. I trust that the experimental evidence, which can be produced with medium size simulators (20 ft. x 20 ft. and smaller), together with field testing and quasi-static cyclic as well as dynamic laboratory testing, is sufficient to meet our near future research objectives.

To obtain a complete set of information on the seismic response of specific structures we would have to reproduce the complete prototype structure-soil system in the laboratory, which is physically nearly impossible even on large shaking tables. Hence, laboratory earthquake simulation is bound to be a compromise which limits the usefulness of shaking table experimentation.

The purpose of experimentation on earthquake simulators is to investigate - within the limitations of rational table sizes - seismic response phenomena which cannot be studied more reliable and cost efficient by other experimental means. Such phenomena could include rate of loading effects, dynamic response characteristics under realistic seismic excitation ranging from low amplitude vibrations to excitations producing inelastic response and failure, failure mechanisms, effects of mass and stiffness irregularities, torsional effects, overturning effects, dynamic instability, idealized soil-structure interaction effects, and interaction between structural and nonstructural elements. Clearly, shaking table experimentation as a means of verifying mathematical modeling techniques needs also to be emphasized.

I have purposely omitted studies on large and complex structural systems. Such structures are usually of a unique nature and it would be difficult to draw general conclusions from a specific test.

It appears that all of the aforementioned topics (perhaps with the exception of interaction effects between structural and nonstructural elements) can and in many cases have to be studied by means of structural models. It has been accepted practice for many years to carry out static and dynamic structural testing on approximately half scale models. Such "large scale" model tests have been the basis of most of our present design recommendations and very few objections have been raised in regard to scaling effects. I only wonder at what scale factors researchers and professionals start losing confidence in model testing.

A thorough study of scale modeling effects is urgently needed to clarify questions regarding strain rate effects, strain gradient effects, dimensional tolerances, detailing requirements, and many more. For instance, it is well established that model concrete (at any scale factor) exhibits a relative increase in tensile strength which will lead to improper simulation of cracking. Bond characteristics are almost impossible to simulate in small scale models. But what are the scale factors at which these effects become important? Perhaps we can use smaller than half scale models without losing our confidence in experimental results. If we also start paying close attention to dynamic model similitude requirements, we may find out that reliable and sufficiently complete information can be obtained from model experiments at scales which permit testing on presently available simulators.

If we need to set priorities, at this time it appears most critical to look into medium size tables which are capable of simultaneously reproducing several components of ground motion. Research on the effects of three-dimensional ground motion is urgently needed.

Hydraulic Actuators. Quasi-static cyclic load testing by means of hydraulically driven actuators has been for decades the standard technique for investigating the load-deformational behavior of structural elements and element assemblies. Synchronized electronic control has made it possible to apply predetermined loading (or deflection) histories to several loading points and thus simulate more closely the anticipated loading path of seismically induced action. This experimental technique is particularly attractive from the standpoint of instrumentation as well as load and deformation control. It probably will remain our main source of information on strength and deformation characteristics since methods can be developed to overcome its two main drawbacks.

The most obvious drawback is that time dependent effects are not simulated in this testing procedure. However, recent studies [3] have indicated that these time dependent (strain rate) effects appear not to be of major importance. Still insufficient data are available at this time to draw definite and general conclusions on strain rate effects. If more research would be done on these effects at the material and element level, we should be able to reliably predict the dynamic response from quasi-static cyclic load tests. It should be possible to achieve this at the element level through tests on pairs of identical speciments subjected to identical displacement histories, one applied quasi-statically, the other at rates similar to those expected under seismic excitations.

The second drawback of cyclic load testing of elements, subassemblies and structures is the rather subjective decision which must be made on the usually predetermined loading history to be applied to the specimen. Analytical studies preceding the experiment will aid in making this decision, but it seems to defeat the purpose of the experiment to predict loading histories from analytical studies based on strength and stiffness properties which are not known a priori.

An alternative which shows much promise is presented in the paper by Professor Okada. He reports work done in Japan on simulation by means of a "computer-actuator on-line system". This technique uses a feedback system between computer and actuators, where the computer receives the electronically measured stiffness characteristics of the test specimen, computes the incremental response of the specimen to a predetermined ground motion, and feeds the response increment back to the electronically controlled actuators. In this manner the loading (or deformation) history will follow closely that which is expected when the test specimen is subjected to the assigned earthquake. This feedback system appears to be quite feasible but may become rather complex for three-dimensional structural assemblies with several translational degrees of freedom, since it may require a series of actuators and extensive instrumentation in every element for which stiffness must be defined in the computer analysis segment.

As mentioned previously, the future needs for actuator operated load testing will be primarily in the area of three-dimensional structural assemblies with one, two, or three-dimensional load application. With the present state of the art in instrumentation and synchronized loading equipment it is feasible to construct suitable facilities of any size. To gain the types of information needed for design improvements in ERCBC, much thought should be given to the loading histories for two and three-dimensional load application. It will be very difficult to draw general conclusions from tests with specific loading histories.

CONCLUSIONS

1. Guidelines are needed for the scope, objectives, and priorities of future experimental research on ERCBC. This workshop is an appropriate forum to develop such guidelines.

2. Existing experimental facilities and techniques need to be assessed as to their capability to achieve research objectives in an efficient and reliable manner.

3. Where presently available facilities and techniques are found to be inadequate to meet the research objectives, recommendations should be made on the types of facilities and techniques which need to be further developed.

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DESIGN METHODS AND EXPERIMENTAL AND ANALYTICAL INVESTIGATIONS RELATED TO THE EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION OF PRESTRESSED AND PREFABRICATED STRUCTURES; CORRELATION WITH FIELD OBSERVATIONS OF EARTHQUAKE DAMAGE

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

DESIGN OF EARTHQUAKE-RESISTANT, PRESTRESSED CONCRETE STRUCTURES

by

T. Y. Lin, Felix Kulka, and James Tai
Board Chairman, President, and Associate respectively
T. Y. Lin International

I. INTRODUCTION

The design of earthquake-resistant prestressed concrete buildings is quite similar to that of reinforced concrete ones. In fact, the main features of aseismic design are generally identical for the two types of buildings, with some differences due the peculiar features of prestressing. The purpose of this report is to point out the differences between prestressed and reinforced concrete insofar as earthquake-resistant design is concerned.

As a total structural system, whether the components are made of reinforced or prestressed concrete, the response and behavior of the system as a whole remains practically the same. In fact, most building systems are composed of a combination of reinforced and prestressed elements so that the prestressed portion may be only a small part of the entire structure. Hence, the presence of prestressing in these components may or may not affect the system as a whole. None-the-less, it is important to realize the differences as well as the similarities between them.

While research on seismic design has been centered on reinforced concrete, and some of the findings are equally applicable to prestressed ones, there has not been too much research done directly on prestressed concrete. The same is true for the design of prestressed as versus reinforced concrete structures for seismic resistance. But the incorporation of prestressed components into buildings can be properly accomplished, provided the peculiarities of prestressing are considered in the components and in the details.

II. ELASTIC BEHAVIOR OF PRESTRESSED CONCRETE MEMBERS UNDER MOMENT REVERSAL*

Because earthquakes act on buildings with varying and alternating directions, a member that is eccentrically prestressed may seem to be incapable

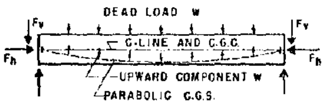


FIG. 1.—A C-LINE LOCATION IN A SIMPLE BEAM

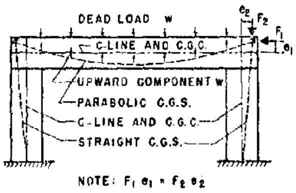


FIG. 2.—LOCATION OF C-LINE IN A RIGID FRAME

*Sections II through V are taken from paper "Design of Prestressed Concrete Buildings for Earthquake-Resistance," by T. Y. Lin, Journal of Structural Division, ASCE, October 1965.

of resisting moment applied in an unfavorable direction. For example, the simple beam shown in Fig. 1 prestressed with a parabolic c.g.s (centroid of steel) would seem to have little resistance for upward loadings produced by earthquakes. In actuality, however, this is not true, because under the action of dead load a prestressed member would usually possess a line of pressure (the C-line) close to the c.g.c. centroid of concrete section. In fact, in an ideal case, such as shown in Fig. 1, the upward component of the tendons balances the gravity action of the dead load, and the C-line will pass exactly through the c.g.c. Then any additional load would simply move the C-line away from the c.g.c. and the beam is able to resist a sizeable loading whether upward or downward.

Sometimes a prestressed member is subjected to the moment reversal of earthquake loadings with its C-lin already quite far away from the c.g.c.; in this case, the resisting capacity is relatively low and additional nonprestressed steel may be required. This is similar to a reinforced concrete member under flexure when reinforcement may be required on both sides in order to provide sufficient resistance to reversal under earthquake actions.

The elastic behavior of a prestressed rigid frame is shown in Fig. 2. An ideal design for earthquake resistance is again to locate the C-line through the c.g.c. before the application of earthquake forces. For example, the c.g.s. in the beam should be located to supply an upward component to balance the dead load, so that we will have a C-line in the beam coinciding with the c.g.c. If the horizontal component of the prestress, F_1, should act with an eccentricity, e_1, at an end of the beam, that moment, F_1e_1, should be counter-balanced with another moment supplied by the tendons in the columns, such that $F_2e_2 = F_1e_1$. If this is done, the columns will be under concentric prestress with the C-lines coinciding with the c.g.c. although the c.g.s. is physically located away from the c.g.c.

In an actual building, it is not always possible to achieve the ideal location indicated in Fig. 2. But if the C-line is not too far from the c.g.c., the elastic capacity of the members to resist moment reversal can be sufficient to meet code requirements and the action of moderately heavy earthquakes. It is emphasized that the resistance of the prestressed members to moment reversal is not indicated by the physical location of the c.g.s., but by the location of the C-line before the application of earthquake forces.

III. ALLOWABLE STRESSES AND LOAD FACTORS FOR PRESTRESSED CONCRETE MEMBERS UNDER EARTHQUAKE LOADINGS

For buildings of conventional materials, such as steel or reinforced concrete, most building codes permit a one-third increase in the allowable stresses when considering earthquake loadings. This increase is justified because of the infrequent occurrence and the short duration of the design earthquakes. It is also convenient because this procedure avoids the necessity of re-proportioning all members for earthquake effects. While the above reasoning is equally applicable to prestressed concrete, the value of one-third increase cannot be directly applied.

Consider the case of allowable tension for extreme fiber stresses in a prestressed concrete member. A one-third increase for zero tension would still culminate in zero tension; a one-third increase for an allowable tension of 400 psi would add to the moment capacity by only approximately 5%. Consider the case of allowable tension in prestressed steel, which is usually set at 0.60 f_s'; a one-third increase would raise this to 0.80 f_s' which is clearly much too high. Hence, a straight one-third increase for allowable stresses cannot be specified when considering earthquake effects in prestressed concrete.

In order to preserve the original intent of the one-third increase in allowable stresses, without encountering the technical difficulties mentioned above, it is suggested that the loads, shears, and moments be modified by a reduction factor of 3/4 when considering earthquake effects while the allowable stresses remain unchanged. This method, sometimes used for earthquake design of conventional structures, is indeed the sensible solution for prestressed concrete. This method simply permits the overloading of all parts by one-third of the normal loadings when considering earthquake effects. It assumes that all members would be able to carry a one-third increase in load for a short duration without signs of distress; this is a valid assumption for prestressed concrete, as in the case of other conventional materials when designed according to the usual standards.

When ultimate strength method is used for seismic design of prestressed concrete, most building codes specify the same load factors as for conventional materials. The 1958 Uniform Building Code and the 1961 Building Codes Requirements of Prestressed Concrete Institute both call for a load factor of 1.4, which means equating ultimate strength U as

$$U = 1.4\ (D + L + W) \tag{1}$$

in which W = earthquake effects.

This, when compared to the requirement of U = 1.8 (D + L), indicates that for loadings including earthquake load, W, an increase of 1.8/1.4 - 1 = 1.3 - 1 = 0.3 = 30% is permitted. This is again in line with the one-third increase in allowable stresses for conventional structures, or the reduction factor of 3/4 suggested above. The 1963 ACI Code calls for 1.25 (D + L + W), together with an understrength factor of 0.9, so that the actual load factor requirement for ultimate strength is,

$$\frac{1.25\ (D + L + W)}{0.9} = 1.39\ (D + L + W) \tag{2}$$

which is close to the previous requirement of 1.4 (D + L + W).

IV. DUCTILITY OF PRESTRESSED MEMBERS

The ductility of prestressed concrete is dependant on (1) the tensile strength and elongation of the prestressed steel and (2) the compressive strength and shortening of concrete. Before the cracking of concrete, the entire concrete section absorbs the energy with the strain energy of steel sharing only a small part of the work. After cracking, however, the steel participates fully in the energy absorption, while only the uncracked part of the concrete remains active. This post-cracking behavior of prestressed concrete is indeed quite similar and comparable to that of reinforced concrete.

The energy absorption capacity of prestressed steel is typically described in Fig. 3. During the process of prestressing a sizeable amount of energy is stored in steel, although part of that energy is lost as a result of the loss of prestress. Under the application of external loading, the change in steel strain is quite small, and the energy is essentially absorbed by the concrete. After cracking, however, the reserve energy capacity of steel is exceedingly high. It is noted that the steel can seldom be stretched to its ultimate strain which has a 4% specified minimum but generally extends to more than 6% for the ASTM A416 seven-wire strands now prevalent in the USA. Hence, there is no question of the sufficiency of the prestressed steel to absorb an unusual earthquake, e.g., 4 to 5 times heavier than the design earthquake. Although the ultimate strain of 4% to 6% is much lower than that for reinforcing bars, or for structural steel, the ductility is already more than enough to meet the requirements and can seldom if ever be fully utilized in any case.

To illustrate the energy absorption capacity of prestressed concrete beams, two typical examples are shown in Fig. 4--one for a post-tensioned beam B with a tendon bonded to the concrete and two nonprestressed steel bars in addition and another for an unbonded beam, U, with two greased tendons [1]. Both beams were loaded at the third points and the middle third part was measured for moment-curvature relationships. Because moment, M, multiplied by curvature, ϕ, is a measure of the energy stored in the beam, the area under the curve before cracking is a measure of the elastic energy while the remaining area indicates the energy absorption capacity in the plastic range. It can be shown that the Beam U, the ratio of the plastic energy absorption capacity to the elastic capacity is approximately 22, while that for Beam B is about 70. In both cases, there is plenty of reserve capacity. The bonded beam, B, with lower percentage of steel and some nonprestressed bars, exhibited especially high ductility.

While curves in Fig. 4 are typical of under-reinforced slender beams failing in flexure, beams with short shear spans and thin webs may fail in shear, especially if no web reinforcement is provided. The two lower curves in Fig. 5 indicate the lack of ductility of such beams, and clearly bring out the necessity for web reinforcement in these cases [2]. The two top curves, on the other hand, confirm the ductility of beams with proper web reinforcement. Generally speaking, the amount of web reinforcement called for by the 1971 ACI Code will be sufficient to assure a reasonable amount of ductility for the purpose of earthquake resistance.

Fig. 6 gives the results of some tests for rectangular beams without web reinforcement subjected to high shear in combination with moment [3]. The low shear span to depth ratio of 2 was the main reason for the low ductility. It will be noticed that the ductility, measured by the ratio of the deflection at ultimate load, P_u, to that at 0.6 P_u ranged from 2 to 8, depending on the extent of the effective prestress in the steel, indicating that higher prestress results in higher ductility. This is also confirmed by tests at the University of Illinois (Fig. 7), which indicated higher resilience for higher levels of prestress [4].

The energy absorption capacity of prestressed beams subjected to external moment producing compression on the precompressed side is not too well known. But from available data, it can be said that the use of nonprestressed reinforcement on the opposite side will greatly increase the strength and the ductility of the beam [5].

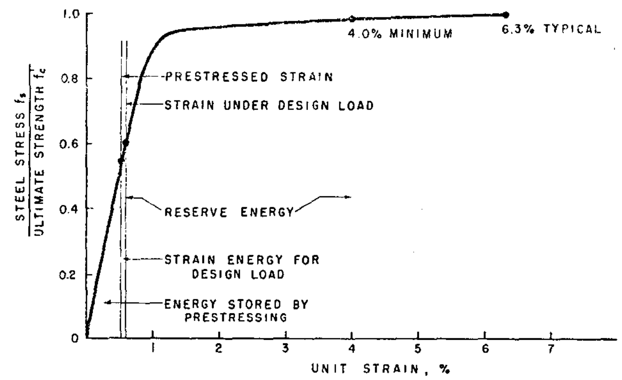


FIG. 3.—STRESS-STRAIN DIAGRAM FOR 7-WIRE STRAND

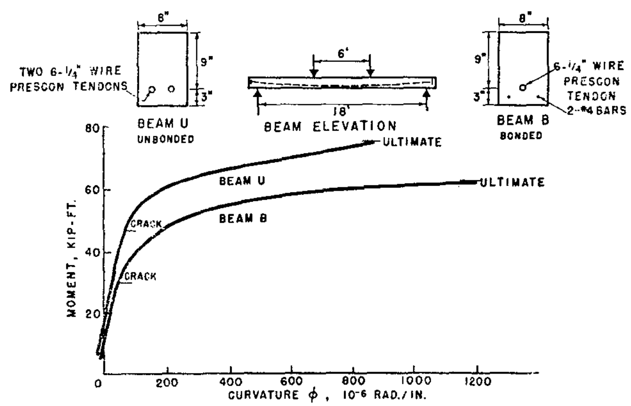


FIG. 4.—MOMENT-CURVATURE RELATIONSHIP FOR PRESTRESSED BEAMS

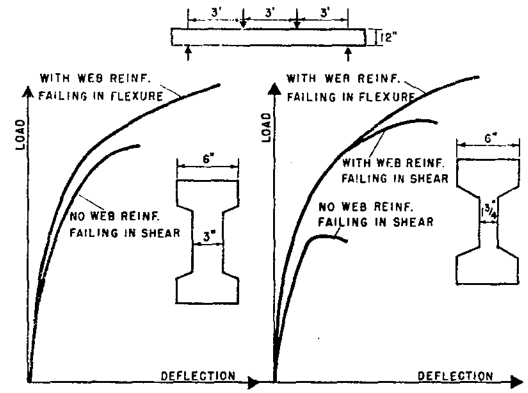


FIG. 5.—DUCTILITY OF BEAMS WITH AND WITHOUT WEB REINFORCEMENT

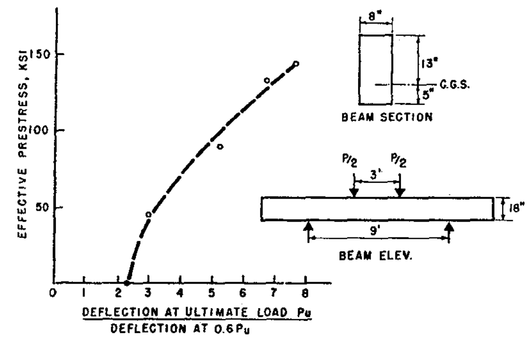


FIG. 6.—DUCTILITY OF BEAMS UNDER COMBINED BENDING AND SHEAR

V. EARTHQUAKE RESISTING COMPONENTS IN PRESTRESSED CONCRETE BUILDINGS

While the elements in prestressed concrete buildings are basically the same as those in reinforced concrete buildings, their layouts and arrangements for earthquake resistance can be radically different. Fig. 8 shows diagramatically the earthquake resisting elements in a concrete building. The vertical elements are the shear walls, the elevator shafts, and the rigid frames formed by girders and columns. The horizontal diaphragm is supplied by the floor and roof slabs, together with their supporting beams and girders.

For a conventionally reinforced concrete building, walls are economically designed to resist earthquake forces. When walls are absent or insufficient, the elevator shafts are reinforced to carry the lateral loads. In either case, the walls or the shafts serve as so-called "shear walls," acting more in shear than in bending. In a prestressed building, these elements can be vertically prestressed to serve as vertical cantilever beams fixed at the foundation and designed to resist earthquake loads in flexure rather than in shear. An outstanding example was the prestressing of the pylon to carry the earthquake forces in a 10-story garage [6]. These shafts, when vertically prestressed, possess high rigidity within the elastic range, and resist moderately heavy earthquakes with no cracks and small deflections and consequently little damage. In case of a heavy earthquake, their rigidity decreases as cracks develop so that they are able to deflect considerably, thus supplying the necessary energy absorption.

When the shafts can be prestressed to carry the major part of the lateral loads, the walls and the frames are relieved of their share and may sometimes be entirely omitted in the building. For other buildings, where the walls or frames can be prestressed or reinforced to carry the lateral loads, the shafts may not have to be prestressed.

The high ductility of a prestressed shaft is indicated by a moment-curvature relationship diagram computed for a 20 ft x 12 ft shaft section, (Fig. 9). For this particular section, the curvature at the start of cracking is 2.4×10^{-6} radians per inch of height, while the curvature at yielding of steel is 14×10^{-6}. The curvature at ultimate load is 100×10^{-6}, over 40 times the curvature at cracking. Although these high curvatures are obtainable only when shear failure is prevented by sufficient stirrups in the webs, they do indicate that these shafts can be designed to withstand heavy earthquakes. When the aspect ratio (height-to-width ratio), of a shaft is very low, e.g., under 2, the shear deflection of the shaft predominates and the ductility can be low; but when the aspect ratio exceeds 4, moment deflection will be more than 5 times the shear deflection, and the shaft is no longer a shear wall but essentially a cantilever beam.

For a floor slab post-tensioned in place, an excellent horizontal diaphragm is provided that ties together the various components of the building. If the slab is post-tensioned in two directions, the concrete, being under compression, will be able to carry high shear and moment in the horizontal plane and generally does so without cracking.

For precast elements, on the other hand, careful connections will have to be designed to tie them together, with a view toward rigidity as well as energy absorption capacities.

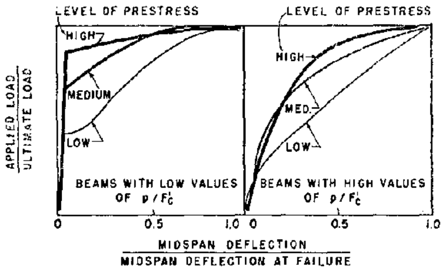


FIG. 7.—LOAD DEFLECTION CURVES FOR BEAMS FAILING IN FLEXURE

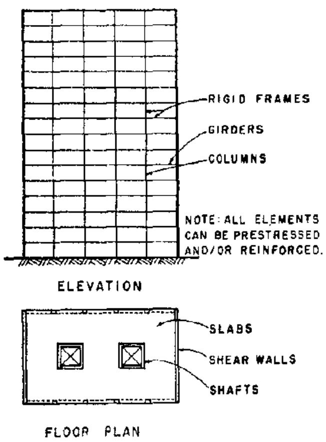


FIG. 8.—EARTHQUAKE RESISTING ELEMENTS IN A CONCRETE BUILDING

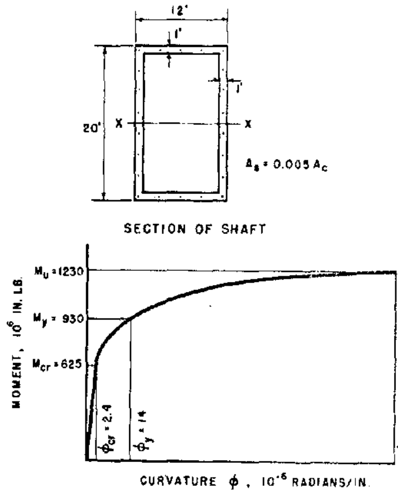


FIG. 9.—M-Φ RELATIONSHIP FOR A PRESTRESSED SHAFT SECTION

VI. SEISMIC DESIGN OF PRESTRESSED CONCRETE MULTI-STORY BUILDINGS

The seismic design of multistory buildings of prestressed concrete is essentially similar to those of conventional reinforced concrete or steel. The main difference lies in the connections for precast members or in the post-tensioning of the vertical and the horizontal load carrying members. Furthermore, because of the frequent absence of frame action in prestressed concrete multistory buildings, their seismic resistance is more dependent on the full use of the elevator and stairway shafts. Because the nature of their resistance is sometimes different from a steel or a reinforced concrete building, careful analysis is desired to assure proper behavior and safety.

Two problems are paramount: the distribution of earthquake shear forces among the various vertical resisting elements of a building, and the action of these elements when subjected to dynamic lateral forces. The shear force distribution is dependent on the elastic properties of the various resisting elements and is usually analyzed assuming rigid horizontal diaphragms.

Present knowledge of prestressed concrete enables us to design both single-story and multistory buildings for earthquake resistance in a safe and economical manner. Such designs can be made to satisfy both the requirements of rigidity under moderate earthquakes and the requirements of energy absorption for the resistance to extra-heavy earthquakes. Other significant points may be summarized as follows:

1. Post-tensioned floors can serve as excellent horizontal diaphragms to tie the building together.

2. In order to increase the resistance of prestressed structures to carry reversal of moments, nonprestressed reinforcing bars may be added at critical locations.

3. For tall buildings, elevator and stairway shafts, especially if they are prestressed, can be designed to serve as the main vertical resisting elements.

4. Dynamic analysis should be made of the effect of earthquakes on tall buildings, especially those with shear walls and shafts. It may reveal substantially different shears and moments in certain floors when compared to the static values obtained from building codes.

Additional research and development is desirable in the design of joints for precast members for earthquake resistance. Data are also lacking concerning the energy absorption capacity of columns under combined effects of high moment and shear.

VII. UNBONDED TENDONS

In the authors' opinion, seismic safety can be equally obtained by either bonded or unbonded construction. However, this has been a controversial issue. Advocates of bonded construction has pointed to the collapse of the Four Seasons Apartment Building during the Alaska 1964 earthquake. This five-story building was almost structurally completed while the earthquake took place. As is well known, this building collapsed on the ground as a result of the elevator shaft which failed at the base. It is also well known that the earthquake produced a lateral force four to five times the code value to which the overlapping of reinforcing bars at the base of the shaft could not resist. Since all the bars were spliced at the base, they formed a weak plane which did not possess sufficient ductility or strength to resist the imposed seismic load. As a result, the shaft was overturned. During the course of overturning, when the shaft tilted some 30° from the vertical, the end anchorages of some unbonded tendons were broken loose while the concrete around the anchorages was twisted and crushed. These tendons, shaken loose from their anchorages, short out 100' or 200' across the street. This spectacular event caused a furor among some engineers raising objections to the use of unbonded tendons. On the other hand, unbonded tendons remain elastic even after a severe earthquake and their structures will cost less to repair and they are thus preferred for shear wall construction.

A lesson learned therefrom has resulted in the development of new bearing-type anchors for unbonded tendons as vs. the old coil-type anchors. The coil anchors need concrete around them and if the concrete fell off around the coils, the strands could be released resulting in this shooting phenomena. However, the new anchors of the bearing type are much better in this respect.

Under seismic loads, unbonded tendons are subjected to a rather small stress range and remain elastic at all times. This is due to the fact that any local strain will be spread out throughout the length of the tendon. From that point of view, the seismic safety of unbonded tendons appears better than bonded tendons, although this is not an important consideration. It can also be shown that the stress range at anchorage is quite low so that fatigue is hardly a problem.

Since this matter of unbonded tendons for seismic regions is a controversial issue, an Appendix (A) is attached herewith, entitled "Report on Prestressed Concrete Members with Unbonded Tendons" by FIP Commission on Seismic Structures.

VIII. CYCLIC LOADING ON PRESTRESSED BEAM COLUMN JOINTS

Only a limited amount of research and construction has been done using prestressed concrete frames. Most building frames with prestressed precast members rely their seismic resistance either on shear walls or on conventionally reinforced concrete frames. An excellent research report titled "Cyclic load tests on prestressed and reinforced concrete beam column joints," by K. J. Thompson and R. Park, June, 1976, University of Canterbury, Christchurch, New Zealand, gives the following conclusions.

1. Beams

The prestressed beams of the test units showed a reduction in strength and stiffness once crushing of the compressed concrete occurred during the inelastic loading cycles, due to the loss of the cover concrete and penetration of curshing into the core causing a reduction in the area of the concrete section in the plastic hinge region. Transverse steel in the form of closed stirrups with minimum concrete cover thickness should be used in such zones to prevent excessive loss of concrete section. Transverse steel will not prevent the loss of the cover concrete but closely spaced closed stirrups will minimize the penetration of crushing into the concrete core between the stirrups. It appears that the stirrup spacing should not exceed 4 in (102 mm) or one quarter of the effective depth of the member.

The flexural capacity at large deformations of the partially prestressed and reinforced concrete beams were not so influenced by concrete crushing, mainly because of the presence of compression reinforcement. Such reinforcement can carry much of the compressive force that was carried by the crushed concrete. However, compression reinforcement may slip through the joint core due to breakdown of bond, particularly if the column section is small and the bar diameter is large, thus reducing its effectiveness. A limiting bar size as a function of column size is evidently required in frames subject to intense seismic load reversals if stiffness and strength degradation due to bar slip is to be prevented.

The flexural strength of the beams in the first inelastic loading runs was up to 16% higher than the theoretical flexural strength, due mainly to the maximum moment being reached at an extreme fibre compressive concrete strain of greater than 0.003 and due to confinement of the compressed beam concrete immediately adjacent to the column face by the column.

2. Columns

In these tests the columns were stronger than the beams and hence were not critical elements, apart from in the joint core regions.

3. Joint Cores

The shear reinforcement of the beam-column joint cores had been designed according to the method of Appendix A of ACI 318-71. In this method the horizontal shear induced in the joint core by the beam forces and the column shear is assumed to be carried by a mechanism involving the concrete (aggregate interlock, etc.) and the hoops (truss action), in a similar fashion to shear in structural members. The beams of the test units all reached at least 95% of their theoretical flexural strength in the first inelastic loading run in each direction, accompanied by yielding of joint core hoops in some units. For those units in which the hoops yielded further load cycles resulted in a degradation of the joint core shear strength. For those units the strength of the joint core then governed the strength of the unit and the greater part of the inelastic deformation of the unit then occurred in the joint core. Thus although the ACI 318-71 approach for joint core shear design allowed the attainment of the design shear strength satisfactorily in the first inelastic loading cycle, degradation of the joint core shear strength occurred in some units in subsequent inelastic load cycles because of large alternating diagonal tension cracks in the joint core due to yielding of hoops leading to a breakdown in the concrete shear resisting mechanism. In the units with nonprestressed longitudinal steel in the beams, joint core shear failure was also encouraged in the subsequent inelastic load cycles by the introduction of beam compressive forces into the joint core mainly by bond. Those units with no central tendon passing through the joint core failed in the joint core even though the shear reinforcement satisfied the requirements of ACI 318-71. Thus a reasonable level of prestress and a central tendon is desirable.

The critical diagonal tension crack was observed to run from corner to corner in the joint core and on this basis it would appear reasonable to provide sufficient horizontal shear reinforcement in the joint core to carry the design shear force across this crack. In seismic design the contribution of the concrete to the shear strength should be neglected due to the degradation of its contribution during reversed loading.

It has also been reported previously that vertical reinforcement spaced around the perimeter of the column section in the joint core acts as shear reinforcement and contributes to the truss action in the joint core, thus improving the joint core shear behavior. A practical method of providing this vertical shear reinforcement is to ensure that the longitudinal reinforcement in the column is spread around the perimeter of the column, and is not just placed at the corners of the section. It may be that neglecting the contribution of the concrete (assuming $v_c = 0$) is conservative when either the column contains bars around the perimeter of the section, or when the column axial load is high, or when the joint core is crossed by several prestressing tendons down its depth, or when the beams enter the column on all four faces. Further tests are required to establish the influence of these variables.

4. Energy Dissipation

All units showed considerable energy dissipation once the maximum moment capacities had been reached. As expected, even after extensive inelastic deformations had been enforced, the prestressed concrete beam showed considerable ability to recover. The ordinary reinforced member showed greater energy dissipation than the partially prestressed member. However comparisons between these specimens are difficult because the inelastic deformations from some units came mainly from the beam plastic hinges, and in others from the shear deformations of the joint cores.

5. Repair

Repairs made to a prestressed beam by replacing the damaged beam concrete showed that it is possible to repair damaged members. Reparis to untis with extensive damage to joint cores would have been much more difficult if not impossible to carry out, however.

IX. JOINTS FOR PRECAST, PRESTRESSED CONCRETE STRUCTURES IN SEISMIC REGIONS

The FIP Committee on Seismic Structures under the direction of Dr. Shunji Inomata has a report drafted and almost ready for publication on the subject of "Examples of Joints for Precast, Prestressed Concrete Structures in Seismic Regions." In this booklet, details of joints, which are being used for actual design in seismic regions, are complied. These joint details have their own merits and demerits from the viewpoint of earthquake resistance and construction work.

These examples of joints are arranged in the following categories:

1. Foundation - Column.
2. Column - Girder.
3. Column - Column.
4. Girder - Girder.
5. Girder - Beam.

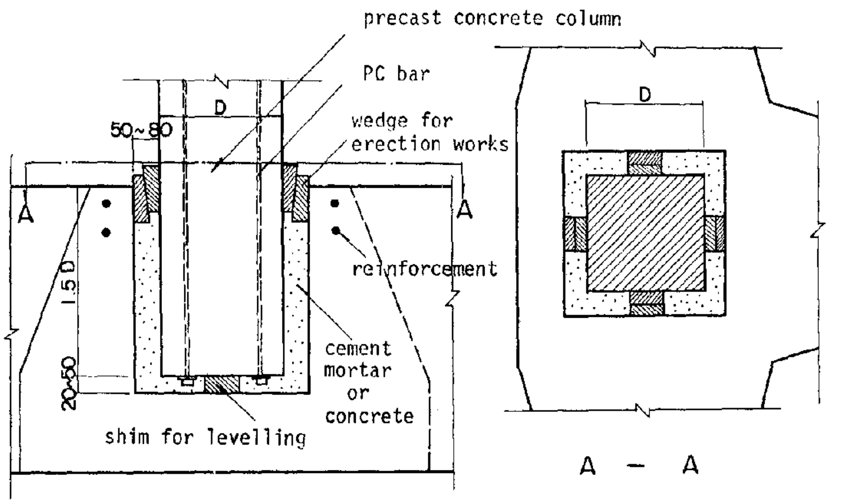
6. Beam - Wall.
7. Beam - Slab.
8. Girder - Slab.
9. Slab - Slab.

In addition, explanatory notes of each detail are itemized as follows:

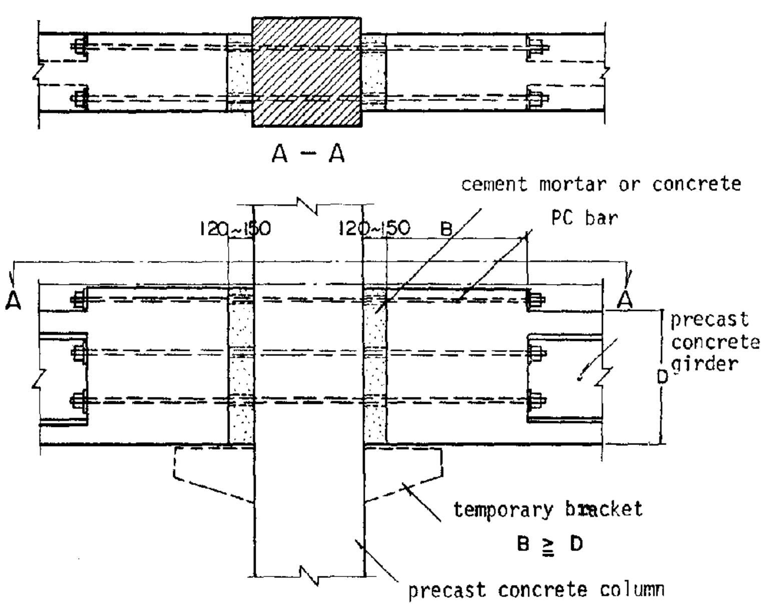
1. Construction method.
2. Special features of the detail.
3. Matters that demand special attention.
4. Mechanical properties of the joint.
5. References, of which 63 are listed.

Since the examples are too many to reproduce, only a few are included as shown in the following.

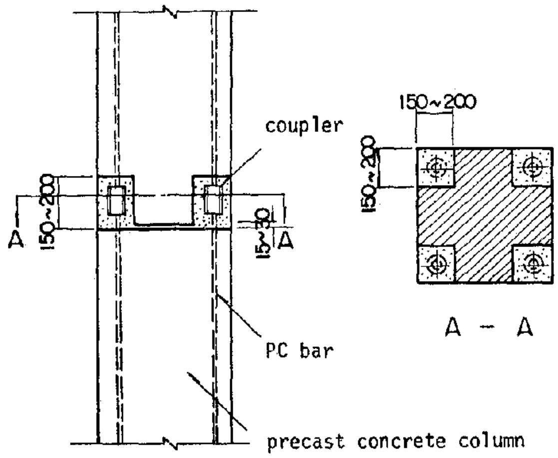
Foundation — Column



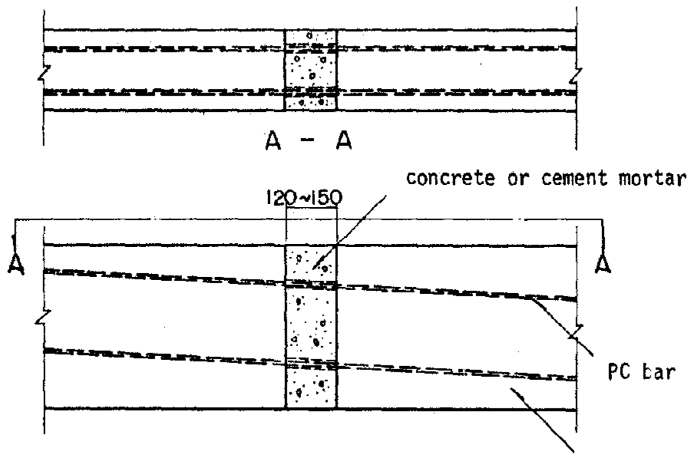
Column — Girder



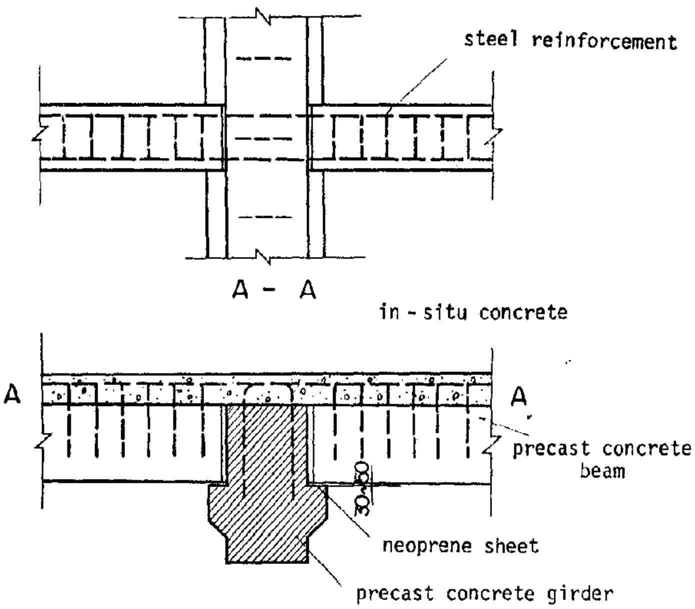
Column – Column



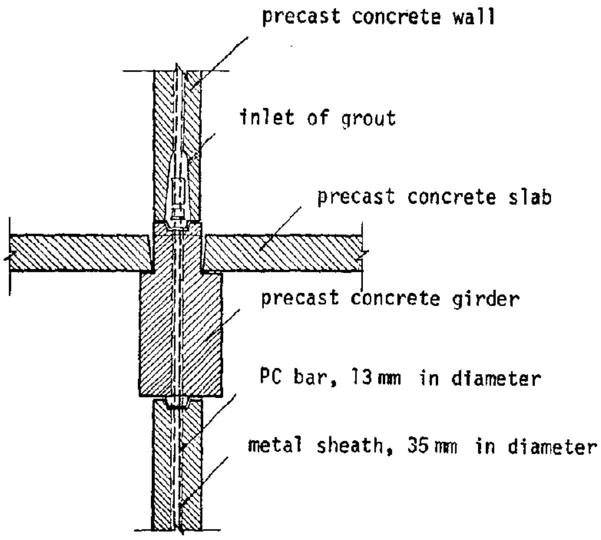
Girder – Girder



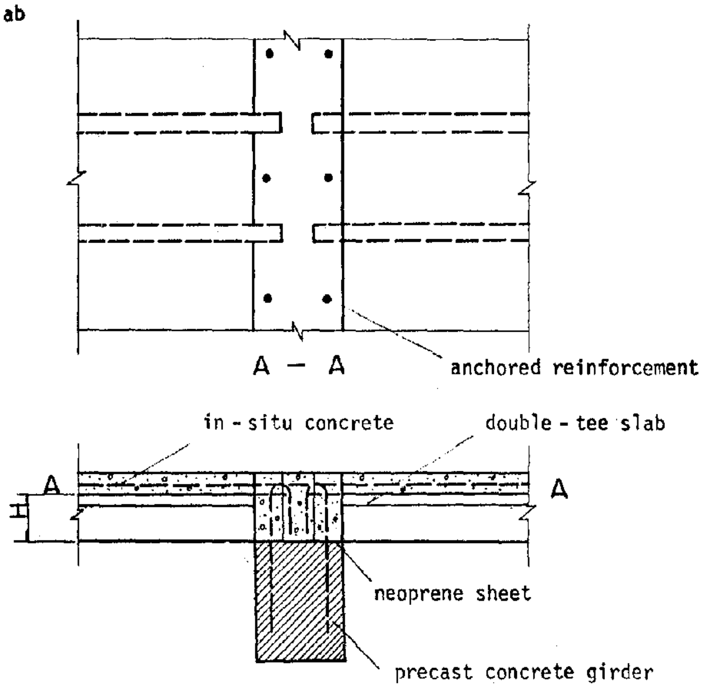
Girder — Beam



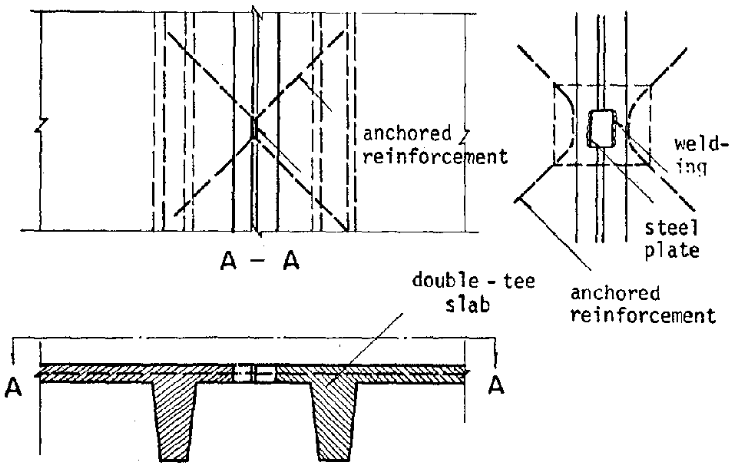
Beam — Wall



Girder — Slab



Slab — Slab



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APPENDIX (A)

REPORT ON PRESTRESSED CONCRETE MEMBERS WITH UNBONDED TENDONS

INTRODUCTION

Failure of the Four Season Apartment building during the 1964 Anchorage earthquake, raised a serious difference of opinion among engineers whether or not bonded tendons are required for satisfactory seismic behavior. Doubt about the safety of unbonded tendons still exists among many engineers and the unbonded tendons are prohibited in some countries to be used for prestressed concrete structures. The main reason will be the doubt about the safety against fracture of the tendon-anchorage assembly under the action of cyclic loading. The fracture of the tendon-anchorage assembly results in a brittle failure of unbonded post-tensioned members, and in some cases the complete collapse of the structure. The unbonded prestressed concrete members would be dangerous, if due attention is not paid to the design and construction.

The FIP Seismic Commission, at the beginning, had a preference for the bonded tendons for aseismic structures, as stated in the reports submitted to the FIP fifth and sixth Congress. Although discussions concerning seismic behavior of the unbonded prestressed concrete structures at the Commission meeting held in New York during the seventh FIP Congress, 1974, have not provided a consensus, it was pointed out that many unbonded prestressed concrete flat plates have been constructed in U.S. and Canada, because of economic and construction considerations. These considerations include, (1) friction losses during stressing, (2) protection of tendons against corrosion during construction and (3) problems associated with grouting procedures for large numbers of tendons in small diameter ducts.

The FIP Seismic Commission had decided for reconsidering the problems of the unbonded tendons. It was thought opportune to begin by preparing a report which is a supplement to the FIP Recommendations for Design of Aseismic Prestressed Concrete Structures, and could provide the engineers with a useful guidance and information on the unbonded tendons.

CORROSION PROTECTION

General

For bonded tendons, the prestressing steel is normally protected by grout injected into the ducts after completion of prestressing operation. For unbonded tendons, the prestressing steel should be coated over the entire length, with a special material, which ensures a permanent corrosion protection. If necessary, the tendons coated against corrosion are sheathed or wrapped with adequate materials such as plastic tube or watertight paper strip for slippage and protection of the coating during construction.

Anchorages should be adequately and permanently protected from corrosion. The anchorages can be encased in concrete or be completely coated with a corrosion-resistant material. The prospective measures shall in every case prevent the entrance of water or aggressive agents.

The tendons coated with protection materials, sheathed for slippage and protection of the coating, are generally cast in concrete. Therefore the unbonded coated tendons remain free from corrosion, because of an excellent protective quality of the concrete. However, as emphasized by R. Park, severe doubt about the effectiveness of the coating with time will remain, if the environment is particularly aggressive. The confident use of the unbonded tendons coated with protection materials would depend on the evidence being available to ensure that the coating remains effective for the life of the structure, and that the tendons are effectively protected at the anchorages.

U.S. and Canada have successful experiences for ten or more years, on the use of coated unbonded tendons, in flat plate constructions. In Japan several structures such as crane girders, piles and retaining walls were prestressed with the unbonded tendons, and satisfactory performance of the protective coating materials has been proved to be effective for several years.

H. Muguruma carried out several tests on the prestressed concrete beams exposed to sea-water. The concrete beams, 10 x 20 cm in cross-section and 100 cm in length were post-tensioned with unbonded prestressing bars of 9.2 mm in diameter. The initial prestress given to the bar was 94 kg/mm^2, which corresponds to 65% of specified ultimate strength. The prestressing bars were coated with bitumastics in three different thickness, that is, 0 (no coating), 0.1 mm and 1.0 mm. An opening of 5 x 10 cm was made at the center of the beam, in order to expose the tendons directly to given environments. Five year test results showed that no corrosive rust was found on all the coated prestressing bars, even in the tidal sea water, while the prestressing bars not coated were fractured after 3 month exposure to the same condition.

Further informations and long-term tests should be necessary to ensure the effectiveness of coating materials.

Coating Materials

The FIP Guides to Good Practice and the ACI-ASCE Committee 423 report on the unbonded tendons require the following properties of the coating materials;

1) Remain free from cracks and not become brittle or fluid over the entire anticipated range of temperatures. In the absence of specific requirements, this is usually taken as -20°C to 70°C.

2) Chemically stable for the life of the structures.

3) Non-reactive with the surrounding materials such as concrete, tendons, wrapping or ducts.

4) Non-corrosive or corrosion-inhibiting.

5) Impervious to moisture.

The coating materials may be bitumastics, asphaltic mastics, greases, wax, epoxies or plastics. The minimum coating thickness will depend on the particular coating material selected but it should be adequate to ensure full continuity and effectivenss with a sufficient allowance for variations in application.

Tests on Properties of Coating Materials

The following tests should be carried out to know the properties of coating materials;

1) Viscosity at a normal temperature

2) Flash point and softening point at a high temperature

3) Flow at an anticipated high temperature (Say 80°C)

4) Cracking of coating at a low temperature (Say -30°C)

5) Corrosion protection capability

6) Anti-acid and anti-alkalin capability

The methods of testing should be specified in each country. The viscosity of coating materials is usually measured by the standard needle penetration testing and the most adequate value seems to be 80 to 120 mm. In order to test the corrosion protection capability, an accelerated testing method could be applied by spraying five percent NaCl solution on the coated surface of a steel plate at a temperature of 35°C. The tests performed in Japan showed that the coating material, now widely used for the most part, could remain without showing any trace of rust for 20 days or more. These test results are thought to be satisfactory to prove the long-term performance of coating materials. However, it should be remarked that these tests could be used to make a comparison between the qualities of two coating materials and could not be thought to be always appropriate for predicting the long-term performance.

TENDONS

Static Tests of Tendon-anchorage Assembly

When an assembly consisting of the tendon and anchorage is statically loaded, the test assembly shall be capable of developing without failure 95% of specified minimum ultimate tensile strength of the corresponding prestressing steel, or calculated tensile stress in the bonded tendons at the critical section under the flexural failure moment, whichever is greater. The total elongation of assembly should not be less than 2% measured over not less than a 3 m gage length. The tendon should elongate appreciably to avoid the possibility of a brittle failure.

It is required to verify the static strength and the ductile failure of the tendon-anchorage assembly, by measuring the total elongation under the static loading.

The FIP Guides to Good Practice requires that the ratio between the maximum force the tendon-anchorage assembly is capable of sustaining, and the actual ultimate tensile strength of the tendon itself shall be at least 0.95. Premature failure of the anchorage and anchorage components must be precluded with certainty. The total strain of the prestressing steel before failure of the anchored tendon must be at least 2%.

When a bonded prestressed concrete beam is loaded, the change in strain in the tendon at the section of maximum moment is equal to the change in strain in the adjacent concrete. The increase in stress in a bonded tendon therefore depends on the local deformations at the section of maximum moment. In the case of an unbonded prestressed concrete beam, the change in strain in the tendon is equal to the average change in strain in the concrete adjacent to the tendon over the whole length of the tendon. As a result, the strain in an unbonded tendon, when the concrete develops its ultimate strain at the compression face, is less than would be the case if the tendon were bonded to the concrete. Therefore the tensile stress in an unbonded tendon cannot exceed that in a bonded tendon at the section of maximum flexural failure moment.

Warwaruk, et al. proposed the following equation for the stress in the unbonded tendon at ultimate state.

$$f_{su} = f_{se} + (2110 - 49.4 \times 10^6 p/f_c)(kg/cm^2)$$

where f_{se} = effective prestressing stress

f_c = cylinder compressive strength of concrete

p = ratio of prestressing steel.

ACI 318-63 gives the following equation for f_{su}.

$$f_{su} = f_{se} + 1055\,(kg/cm^2)$$

Mattock, et al. proposed the following empirical equation for estimating the steel stress in the unbonded tendon at failure state.

$$f_{su} = f_{se} + \frac{1.4 f_c'}{100p} + 700 \ (kg/cm^2)$$

but not greater than the yield strength of the tendon.

Pannell proposed the following design method.

$$Mu/ubd^2 = q_u(i - q_u)$$

$$q_u = \frac{q_e + \lambda}{1 + 2\lambda}$$

where M_u = calculated ultimate moment of resistance

u = crushing strength of 150 mm Cube (N/mm^2)

$q_u = \dfrac{p \cdot f_{su}}{u}$

$q_e = \dfrac{p \cdot f_{se}}{u}$

$\lambda = \dfrac{pdE_s \cdot \varepsilon_{cu}}{uL} \cong 7000pd/uL$

ε_{cu} = limiting strain at which concrete crushes

L = length of tendon from anchorage to anchorage

f_{su} < 0.85 x ultimate strength of prestressing steel

Miyamoto, et al. estimated the compatibility factor, the ratio of increase in strain in steel to the increase in strain in concrete at the level of steel at the region of maximum moment, from the static loading tests on 24 unbonded prestressed concrete beams. The value of the compatibility factor, f, depends on the surface condition of the prestressing bar, as follows;

f = 0.4 ~ 0.6	for the prestressing bars coated with epoxies or bitumastics,
f = 0.2	for the prestressing bars placed in metalic sheath without coating,
f = 1.0	for the bonded prestressing bars.

According to Mattock's equation, the ultimate steel stress change in the unbonded tendon can be 10.5 kg/mm^2 for heavily reinforced member and 56 kg/mm^2 for lightly reinforced member. These values correspond to the tensile stress from 125 kg/mm^2 to 168 kg/mm^2 for a common post-tensioning strand with an effective stress of 90 kg/mm^2, and this range represents 67 and 90% of the ultimate strength of the strand. In the light of this fact, Bondy stated in his report that it seems reasonable to require the tendon-anchorage assembly to develop 95% of the ultimate strength of the prestressing steel. Bondy discussed also on the maximum elongation, refering to his numerical calculation, and stated that 1.5% elongation at the design ultimate moment would be sufficient, except for the particular case of extremely lightly reinforced members, as given in Mattock's report in which 1.9% elongation was expected.

Considering the above mentioned facts, the total ultimate elongation was tentatively specified as 2%, however further investigations would be desirable to specify more reliable value.

Dynamic Loading Tests with Tendon-anchorage Assembly

The anchorage of a tendon in a prestressed concrete member can be subjected to small cyclic load variations. These can be of importance in certain cases, e.q. in unbonded tendons or in bonded tendons if the anchorage is located in a zone where externally applied loads create changes of stress. Thus, the tendon-anchorage assembly must be capable of resisting cycles of repeated load variations.

The FIP Guides to Good Practice requires that the tendon-anchorage assembly must be able to withstand 2 million cycles of repeated loading with an amplitude of 80 N/mm^2 to an upper stress limit of 65% of the characteristic strength of the prestressing steel. The tests shall be performed in a tensile testing machine with the pulsator at a constant load frequency of not more than 500 load cycles per minute and with a constant upper load of 65% of the characteristic strength of the prestressing steel. Where the capacity of the available testing machine is exceeded, the number of wires, strands or bars in the tendon-anchorage assembly to be tested comprising all prestressing steel in the tendon.

In addition to the above minimum requirement, the anchorage must also be able to sustain, to a certain extent, exceptional effects, such as an increase of temperature in case of fire, repetitive and significant change of tendon force due to earthquake, etc. However, no testing methods are given in the FIP Guides.

ACI-ASCE Committee 423 report on unbonded tendons recommends the following requirements. The test assembly shall withstand without failure 500,000 cycles from 60 to 66 percent of its specified ultimate strength and the following additional requirements should be satisfied for unbonded tendons used for structures subject to earthquake loading. The test assembly shall withstand without failure a minimum of 50 cycles of loading corresponding to the following percentage of the minimum specified ultimate strength:

$$\frac{\Delta\sigma_s}{f_{su}} \times 100 = 60 \pm \frac{600}{l + 30}$$

where $\Delta\sigma_s$ = stress amplitude in steel

f_{su} = specified ultimate strength of prestressing steel

l = length in meters of the tendon to be used in the structure.

Miyamoto, et al. carried out fatigue tests on the unbonded prestressed concrete beams which were 20 x 38 cm in cross-section and 450 cm in length. The prestressing bars were coated with bitumastics. The steel strains were measured at the regions of the maximum moment and the both ends of the tendons. The test results showed that the change in strain decreases at the region of the maximum moment, while the change in strain increases at the both ends of the tendons, so long as the number of cyclic loading is less than 1/3 of that corresponds to the fatigure failure of the beam. After that number of cycles, the strains in the tendon increased at the region of the maximum moment and the both ends of the tendon. The difference of the strains in tendon between that at the region of the maximum moment and that at the both ends, was given as: (0.5 ~ 0.7) x (Change in strain at the region of the maximum moment) and was nearly equal to the compatibility factor for the beam subjected to repeated loading up to the decompression state.

Muguruma carried out the tests on 44 unbonded and bonded prestressed concrete beams subjected to repeated loading up to 60 to 90% of static ultimate moment. The changes in steel stress at the region of the maximum moment were from 4.4 to 17.5 kg/mm^2 for the unbonded beams and were from 8.8 to 26 kg/mm^2 for the bonded beams. The test results showed that almost all the beams failed due to the crushing of compression fiber of the concrete, and no specific difference between the fatigue strengths of the unbonded and the bonded beams was not recognized. The test results also showed that the beams whether they are bonded or unbonded could withstand, without failure, 150 cycles of maximum load of 90% of static failure load and the beams could resist 2 million cycles of repeated loading of 65% of static failure load.

Brondum-Nielsen found that the unbonded beams could withstand about 2 million cycles of repeated loading with 5.5 kg/mm^2 stress amplitude in the steel, while the number of cycles of repeated loading until the beams failed became smaller when the stress amplitude in the steel was increased up to 7.6 kg/mm^2. This failure of the beam was attributed to the low fatigue strength of the anchorage.

Chung drew a conclusion from the tests results carried out on 23 unbonded beams subjected to 2 million cycles of repeated loading, that most existent anchorage systems can withstand without failure more than 2 million cycles of repeated loading with a stress amplitude which is usually expected under serivce load.

The requirements of the FIP Guides to Good Practice and of the ACI-ASCE Committee 423 report are not always realistic about the real change in stress in unbonded prestressing steel under the service load. The change in stress in unbonded prestressing steel is always smaller than that in bonded prestressing steel. The 500,000 cycle fatigue test specified in the ACI-ASCE Committee 423 report and 2 million cycle fatigue test specified in the FIP Guides to Good Practice, would be unrealistic for the unbonded tendons, however sufficient data do not yet exist which can establish a lower bound for this test. Until such data are developed, it is recommended that the present specification of the FIP Guides to Good Practice be used, as far as the dynamic tests with tendon-anchorage assembly are concerned, in order to avoid any fatigue failure of the unbonded prestressed concrete structures.

The ACI-ASCE specification on the 50 cycle dynamic test for tendons in seismic zone, corresponds to the stress range between 40 and 80 percent of the ultimate steel strength for the extreme case ($l = 0$). This means the average load is 60% and the amplitude is 40% of the ultimate strength of the tendon. When an unbonded beam is subjected to static loading until the flexural failure, the maximum possible increase in unbonded tendon force will be less than 23% of the ultimate strength of the tendon, as stated in 3.1. Therefore, it could be thought that if alternate equal positive and negative moment acts on an unbonded member during an earthquake, the maximum possible change in the tendon force will be less than two times the above change expected under static loading. Therefore this specification covers almost the entire possible tendon stress range, and also 50 cycles between 40 and 80 percent approximates a reasonable number of seismic cycles.

However, it is necessary to establish more reasonable dynamic tests which can be representative for the actual behavior of unbonded tendon-anchorage assembly during the earthquake motions.

DESIGN CONSIDERATIONS

Minimum Average Prestress

Unbonded prestressed slabs and rectangular beams having a smaller average prestress than the modulus of rupture become flexurally unstable at the cracking load. Such a low value of average prestress is not likely to be specified beams, but can occur relatively often in slabs and flat plates. To avoid a sudden collapse of unbonded prestressed structures at excessive live loads and especially earthquake loads, the minimum value of average prestress in rectangular sections must always be substantially higher than the modulus of rupture which is the maximum calculated tensile stress at carcking in plain concrete beams subjected to flexure. The calcualtion of the tensile stress at cracking is based on the assumption that the stress distribtuion is linear. Although this assumption is not correct and introduces an error, the cracking moment capacity of a prestressed beam can be readily calculated by making the maximum tensile stress in the beam equal to the modulus of rupture.

In the case of bonded beams, cracking of the concrete causes only a gradual decrease in the slope of the load-deflection curve. Some unbonded beams, however, exhibit a sudden decrease of the load capacity at cracking, and the load capacity does not recover even after a significant increase in deflection. In order to avoid such a sudden decrease of the load capacity, the minimum average concrete prestress shall be specified or the minimum amount of the additional bonded steel shall be specified.

Rozvany, et al. demonstrated both theoretically and experimentally the unbonded prestressed concrete slabs and rectangular beams having a smaller average prestress than the modulus of rupture became flexurally unstable at the cracking moment.

Amount of Bonded Reinforcement

In most beams in which the unbonded tendons were the only flexural reinforcement in the beams, if the tendons do not come into contact with the beam between the anchorages, a single wide crack will form at the section of maximum moment. After cracking, the beam behaves as a shallow tied arch, rather than as a flexural member. The crack increases rapidly in width and depth as the load increases. Such cracks usually fork at their upper ends. The deflection increases rapidly after cracking. If the tendons come into contact with the beam between the anchorages, then additional cracks will usually form. However, these cracks are normally few in number and still open much more rapidly than do cracks in bonded members, and sometimes a premature brittle failure of compressive concrete is observed in the unbonded member.

The undesirability of such behavior could be avoided by placing additional bonded unprestressed reinforcement to control cracking. It was founded that quite a moderate amount of additional bonded reinforcement resulted in crack widths and spacings similar to those found in bonded prestressed concrete beams. In addition, the stiffness of the beam after cracking and the ultimate strength were also increased.

ACI 318-71 requires some bonded reinforcement to provide in the recompressed tension zone of flexural members where the prestressing steel is unbonded. For beams and one-way slabs ACI 318-71 requires a bonded steel area of either $Nc/0.5f_y$ or $0.004A$, whichever is larger, where N_c = tensile force in the concrete under load D + 1.2L, D = service load, L = service live load, f_y = yield strength of the nonprestressed reinforcement but not greater than 420 N/mm^2, A = area of that part of the cross-section between the tension face and the center of gravity of the gross section. For two-way slabs, ACI 318-71 requires the bonded steel content to be as for one-way slabs, except that a lesser amount may be placed when there is not tension in precompressed tension zone at the service load.

The ACI-ASCE Committee 423 report on flat plates, which is intended as a supplement to the part of ACI 318-71 dealing with flat plates does not require non-prestressed bonded reinforcement in both positive and negative regions when the concrete tension does not exceed a specified value.

Mattock, et al. carried out comparative tests on bonded and unbonded beams and showed that if the minimum amount, 0.4%, of additional bonded reinforcement is provided, then an unbonded post-tensioned member will behave as a flexural member subject to a combination of transverse and axial forces and also the distribution of the cracks was as good as, or better than, the distribution of the cracks in the bonded member.

A prestressed concrete member with bonded tendons has a greater flexural strength than the equivalent member with unbonded tendons. Typically, the difference in flexural capacity between otherwise identical members may be 10 ~ 30%. The difference is because unbonded tendons can move relative to the concrete between the anchorages, and hence local concentrations of strain cannot build up in the steel at critical sections. The increase in steel strain will tend to occur uniformly over the length of the tendon and result in a relatively small increase in steel stress, as stated in 3.1. When necessary the flexural capacity of a member with unbonded tendons can be increased by the addition of non-prestressed bonded reinforcement to the section.

Muguruma pointed out from the test results on unbonded beams that the difference in the flexural capacity between the bonded and unbonded beams remains approximately constant, if the ratio, h/l, is greater than 1/12, where h = height of beam, l = span length, while the difference diminishs in proportion to increasing h/l, when the ratio, h/l, is smaller than 1/12. Therefore, for slabs or flat plates the ultimate flexural strength can be easily increased by the addition of relatively small amount of non-prestressed bonded reinforcement. The unbonded beam having no additional bonded reinforcements showed larger deflection than does the bonded beam, after the ultimate moment was reached.

Ductility

The unbonded beam reaches the flexural strength at a greater deflection owing to its smaller neutral depth, when compared with the equivalent beam with bonded tendons. This smaller neutral depth results in greater curvature when the concrete commences to crush. It should be noted that had more prestressing steel been present in the unbonded beam at the same effective depth, so that the flexural strengths of the bonded and unbonded beams were identical, the ultimate curvatures at the critical sections of the bonded and unbonded beams would be similar due to the similar neutral axis depths. Instead of increasing the amount of the unbonded tendons, the addition of non-prestressed bonded reinforcement to the section can give the same results. Therefore, it is advisable to design the unbonded section in which the following condition is satisfied.

$$\frac{A_p \cdot f_{pu}}{bd \cdot f_{ck}} + \frac{A_s \cdot f_{yk}}{bd \cdot f_{ck}} < 0.20$$

where f_{pu} = steel stress in unbonded tendon at failure of section.

Mattock, et al. stated that the ductile behavior of the unbonded beam was to be expected, unless the amount of bonded reinforcement is not excessive.

Nawy, et al. investigated moment-rotation relationships of non-bonded post-tensioned I and T Beams. This study investigated the possibility of increasing the rotational capacity of members by introducing continuous rectangular spiral binders at the critical sections of the beams. There was an increase in elastic rotation capacity with increase in the percentage of spiral binders, and an optimum percentage not too far in excess of 2% seemed to exist. The curvature distribution was measured to evaluate the equivalent plastic hinge length and because of high value of localized strain and curvature near the plastic hinge, the equivalent plastic hinge length was small.

More investigations should be carried out on the estimation of the equivalent plastic hinge length for unbonded prestressed concrete members.

Other Factors

In a structure with unbonded tendons, continuous over several spans, the failure of one span may result in the release of the prestressing force along the whole length of the tendons. Such an event could lead to the collapse of the whole structure. Consideration should be given to the consequence of such failure in any specific span to the overall stability of the structural system. One consideration would be to use reduced tendon lengths between anchorages or tendon couplers capable of acting as intermediate anchorages.

ACKNOWLEDGEMENTS

This report was written by the FIP Commission on Seismic Structures chaired by Dr. S. Inomata. The report was initially drafted by Japanese subcommittee and was edited and correlated by Professor Muguruma. The FIP Commission wishes to thank Professor Muguruma for his help during the preparation of this report.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

DESIGN OF PRESTRESSED CONCRETE STRUCTURES

by

R. Park
Professor of Civil Engineering
University of Canterbury

INTRODUCTION

The use of prestressed concrete components has been widely accepted for many years for structures carrying gravity loading. The applications of prestressed concrete to such structures has increased rapidly, encouraged by such advantages as the possibility of pleasing architectural forms, and the suitability of prestressed concrete to prefabricated construction. However the use of prestressed concrete in primary seismic resistant elements such as shear walls and frames has not met with the same acceptance. Part of this caution in the use of prestressed concrete has been due to the paucity of experimental and theoretical studies of prestressed concrete structures subjected to seismic type loading. However in recent years more research studies have been reported in this area. A survey of the research which has been conducted on the resistance of prestressed concrete was published in 1970 [1]. More recent research information has been presented at the FIP Symposium on Seismic Structures [2] which was held in Tbilisi, USSR, in 1972. A summary of the papers of the Tbilisi symposium is contained in a report of the FIP Commission on Seismic Structures which was presented to the 7th Congress of the FIP in New York in 1974 [3].

In the past there has also been a lack of detailed building code provisions for the seismic design of prestressed concrete. For example, the building code of the American Concrete Institute, ACI 318-71 [4] contains special provisions for the seismic design of reinforced concrete structures but does not have corresponding provisions for prestressed concrete structures. This is also true of the Uniform Building Code [5], the SEAOC Code [6], and the first draft of the seismic design provisions of the Applied Technology Council [7]. However updated seismic design recommendations based on available research information are currently being prepared by the Commission on Seismic Structures of the FIP [8]. Also, the Seismic Committee of the New Zealand Prestressed Concrete Institute has recently prepared a set of design recommendations [9] and the New Zealand Standards Association is drafting seismic design recommendations for prestressed concrete.

This paper briefly reviews background studies of seismic design procedures for prestressed concrete structures and comments on possible building code recommendations.

EARLY USES AND STUDIES OF PRESTRESSED CONCRETE IN PRIMARY SEISMIC RESISTANT ELEMENTS

In 1955 Ellison and Lin [10] and Engineering News Record [11] reported the construction of a 9-storey car parking building in San Francisco, in which the key element in resisting seismic forces was a prestressed concrete shear walls near one corner. The bottom 40 ft (12.1 m) of the shear wall was prestressed, the amount of prestress being determined by the condition that no tensile stress should be created in the shear wall under seismic loading, determined from the equivalent lateral loading found on the basis of a 6% seismic coefficient. The tendons were not bonded to the concrete. At this stage of development the availability of post-elastic deformations, which subsequent research demonstrated was important, was not considered.

In a paper presented to the World Conference on Prestressed Concrete in 1957, Ban [12] commented that in Europe studies to obtain a reasonable building frame appropriate for prestressed concrete structures had been undertaken, and several multistorey office buildings had been completed there. The frames were composed of reinforced concrete columns and prestressed concrete beams. Ban went on to say that the prestressed concrete structures encountered were mainly built up with precast members, the joints of which were considered to be simply supported or hinged. However he advocated rigid connection between the beams and the columns in order to resist lateral load. One method of framing multistorey buildings was to assemble prestressed concrete beams and reinforced concrete columns with a reinforced concrete joint. Ban introduced his own proposals for extending the prestressing tendon through to the column and hence placing the whole joint in compression. His scheme was the forerunner of modern precast prestressed beam-column connection details.

In the early 1960's a philosophy of seismic design of concrete structures came to be accepted. Design requirements were that under a moderate earthquake such as may be expected several times during a structure's life, it should survive without damage, and that it should survive without major structural damage in the most severe probable earthquake expected during its life. A further condition is that the structure should not collapse even under earthquake loading of an abnormal intensity. However authorities differed as to the ability of prestressed concrete structures to fulfil these requirements. While Lin [13] was enthusiastic about its suitability, Glogau [14] and others advocated caution in its use.

The publication of Lin's 1964 paper [15] was a stimulus to the debate. He discussed several important aspects of seismic design of prestressed concrete structures. He discussed the special case of allowable stresses and load factors for prestressed concrete. He showed that for a working stress design method it was appropriate to take three quarters of all moments, loads, and shears from seismic loading, and then use normal working stresses. This approach was equivalent to the normal $33\frac{1}{3}$% allowable stress increase for seismic loading. He also considered the ultimate strength method and showed that load factors stated in codes were consistent with the one-third decrease allowed in static load values under dynamic conditions.

The results of some tests studying the energy absorption capacity of prestressed concrete members were presented. Lin noted that this energy absorption capacity was important if structures were to withstand severe earthquakes. Moment-rotation curves were plotted for tests to failure of prestressed concrete beams in bending. The large area beneath the curves was taken to be indicative of high available energy absorption. Of particular interest in the paper was the result of a dynamic computer analysis of a 19-storey prestressed concrete apartment building. The building was first designed using the static equivalent earthquake forces specified by the 1961 Uniform Building Code. The dynamic response of the structure to the North-South component of the 1940 El Centro earthquake was calculated by a digital computer assuming linear-elastic behaviour. The fact that the El Centro earthquake produced forces and displacements about five times the Code values emphasized the need for prestressed structures to be capable of developing large post-elastic deformations if they were to be able to survive major earthquakes.

In discussing Lin's paper Rosenblueth [16] warned against establishing conclusions based only on the curve for first loading. He illustrated the significance of the shape of the unloading force-deformation curves using the two diagrams shown in Fig. 1. Respectively, they represent a nonlinear elastic system and a classical elasto-plastic one, both having the same force-deformation curve for critical loading but with the prestressed concrete load-deformation loop showing zero energy dissipation (i.e. zero hysteretic damping). He demonstrated that the two types of structure considered although characterised by the same force-deformation curve on first loading, could undergo, on the average, maximum loads for a given deformation or maximum deformations for a given load, that were significantly greater for type A than for type B. Rosenblueth did not contend that prestressed concrete structures could be exactly idealized as of type A, but felt that the shape of their force deformation curves relative to their reinforced concrete counterparts and their lower damping characteristics, meant that for a comparable mass and stiffness a prestressed structure was likely to give greater deformations or be called to resist higher forces,

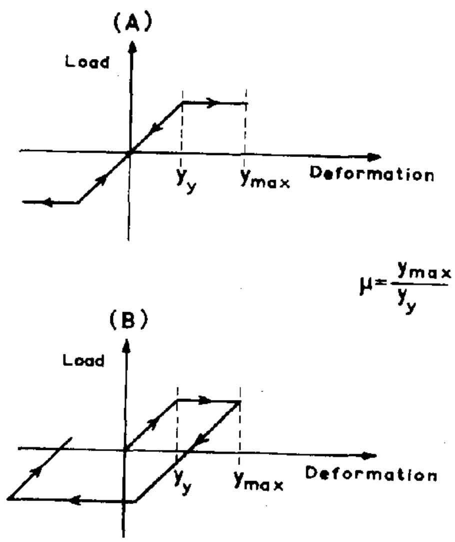


Fig. 1 Idealized Load-Deformation Relationship, for
(A) Prestressed Concrete and
(B) Reinforced Concrete, [16]

than a reinforced concrete structure, under most types of strong earthquakes. He also noted that in fact to resist a given set of forces, a prestressed structure would normally be more flexible than its reinforced counterpart. This flexibility would partly counteract the effect of smaller energy dissipation capacity under cyclic loading.

One of the most important topics discussed during the papers on prestressed concrete at the Third World Conference on Earthquake Engineering in 1965 was that of energy absorption. Guyon [17] stated that "neither theoretically nor experimentally is the energy absorption at failure for the same cross section, the same concrete and the same ultimate moment, smaller for prestressed concrete than for reinforced concrete." Despeyroux [18] emphasized this same point. The energy stored per unit of length can be written as the product of moment and curvature and hence the area under the moment-curvature curves for prestressed concrete and reinforced concrete give an indication of their energy absorption capacities. Despeyroux concluded that there was no reason for the area under the curve for prestressed concrete to be systematically smaller than for reinforced concrete and could in fact be the contrary. However the critical factor affecting the response of a structure under earthquake acceleration cycles is not the energy absorption capacity of its members but rather their energy dissipation characteristics, as was emphasized by Rosenblueth [16]. In the discussion of Despeyroux's paper Candy illustrated this point with Fig. 2. Cyclic loading tests have given curves for the two materials approximately as drawn. The shaded area represented the energy dissipation; the area up to the dashed line represented the energy absorbed. Thus although the energy absorbed by a prestressed member and a reinforced concrete member may be the same, very much more energy would be dissipated in the latter member and thus the response of the latter to an earthquake must be smaller. He concluded that the reduction in response caused by plastic strain is much smaller in prestressed concrete.

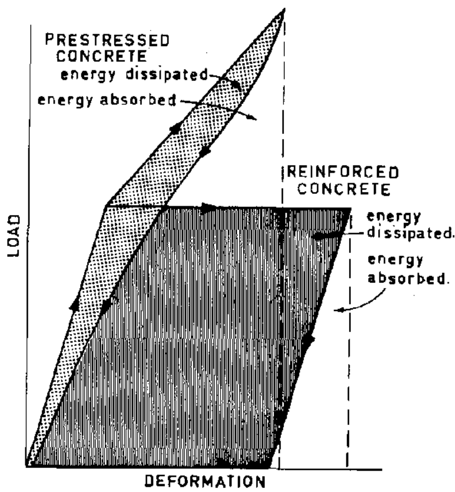


Fig. 2 Dissipation of Energy in Prestressed and Reinforced Concrete Members, [18]

Sutherland [19], Nakano [20] and Zavriev [21] also presented papers on the seismic resistance of prestressed concrete to the Third World Conference on Earthquake Engineering. Zavriev's paper reported some large scale investigation of seismic behaviour of prestressed concrete bridges in the Soviet Union. He felt that the use of prestressed concrete in seismic resistant structures was expedient. Under moderate earthquakes the resistance to plastic deformations would result in little damage to structures and he felt that under heavy earthquakes the behaviour of prestressed concrete sections would be similar to that of reinforced sections, thus eliminating the characteristics of prestressed concrete which increase dynamic loads.

Since 1965 there have been many studies published on the seismic resistance of prestressed concrete, notable among these being those presented at the symposium of the Federation Internationale de la Precontrainte (FIP) on Seismic Structures held at Tbilisi, USSR, in 1972 [2]. The report of the FIP Commission on Seismic Structures to the Seventh Congress of the FIP in New York in 1974 [3] has summarized recent research into the behaviour of prestressed concrete structures subjected to earthquake loading.

FIP RECOMMENDATIONS FOR THE DESIGN OF SEISMIC RESISTANT STRUCTURES

The FIP Commission on Seismic Structures has presented recommendations for seismic design to FIP Congresses since 1966. The recommendations have recently been updated by the Commission and the third draft circulated among Commission members in 1976 [8] is close to the final version which will be available for the 1978 Congress in London. The form of these recommendations has recently been discussed by the chairman of the Commission, S. Inomata [22]. The main points of the recommendations are summarized below.

Seismic Analysis

Equivalent static loads or dynamic analyses may be used to determine the design actions. The seismic loading may be considered separately along the two principal axes of the structure, except that columns, beam-column joint cores, walls and foundations which are part of a structure with a two-way beam system should be designed to withstand the actions induced by the most unfavourable combinations of load effects in both directions, considering the fact that the earthquake may act in any direction.

Limit States for Seismic Loading

Generally limit states at two earthquake load levels should be considered: a moderate earthquake limit state and a severe earthquake limit state. For particularly important structures it may also be desirable to consider an excessive earthquake limit state. The earthquakes can be stated in terms of expected frequency of occurrence (i.e. the return period) as follows: moderate, 10 or 20 years; severe, 50 or 100 years; and excessive, 200 or 500 years.

At the moderate earthquake limit state the concrete may crack but the strain in the tendons should not exceed the initial tensile steel strain at that section at the time of prestressing or the limit of proportionality of the steel, whichever is greater.

At the severe earthquake limit state the structure should be safe from collapse. In general the structural elements of frames should be capable of dissipating seismic energy by the formation of a significant number of plastic hinges in the structure having adequate ductility. All forms of brittle failure should be prevented.

The excessive earthquake limit state should be applied to particularly important structures.

Flexural Ductility

Suitable precautions should be taken to ensure suitable plastic hinge positions and adequate plastic hinge rotation capacity under severe or excessive earthquake loading. The following factors affecting ductility should be taken into account at potential plastic hinge sections: at the design moment the neutral axis depth should not exceed 0.25 of the overall depth of the section, at positions of moment reversal tendons should be placed near both extreme fibres rather than axially only, confinement of concrete should be provided, and the design moment of the section should be at least 1.3 times the cracking moment. Note should be taken that the presence of axial compressive load decreases the ductility of flexural members.

Shear Capacity

In calculating the design shear force the plastic hinge moments should be determined considering the possible overstrengths of the materials because enhanced levels of flexural strength will be accompanied by enhanced shear forces. The enhanced plastic hinge moments could be taken as 1.15 times the flexural capacities calculated on the basis of the characteristic strengths of materials. Cyclic loading causes a reduction in the effectiveness of the concrete shear resisting mechanism and hence in plastic hinge zones all the design shear force should be carried by the web reinforcement.

Bonded and Unbonded Tendons

Prestressing ducts in flexural members of a ductile structural frame should preferably be grouted. In roof or floor systems not contributing to the bending strength of the frame the prestressing ducts need not be grouted provided the unbonded tendons are suitably protected against corrosion and fire and that the anchorages are capable of withstanding fluctuating stresses.

Anchorages

Anchorages for post-tensioned tendons should be positioned away from highly stressed zones such as plastic hinge regions.

Beam-Column Connections

Beam-column connections should be designed to ensure that shear failure does not occur within the joint core. Consideration should be given to possible reduction in shear strength during seismic load reversals.

NEW ZEALAND RECOMMENDATIONS FOR THE DESIGN OF SEISMIC RESISTANT STRUCTURES

The Seismic Committee of the New Zealand Prestressed Concrete Institute has recently published recommendations for the design and detailing of ductile prestressed concrete frames for seismic loading [9]. At present a new concrete design code is being written in New Zealand which will largely be based on ACI 318-71 [4] and its recent amendments, together with additional provisions for seismic design. A draft of Chapter 22: "Prestressed Concrete Members - Additional Seismic Requirements" proposed for the code is given in Appendix II with supplementary sections in Appendix III and some commentary in Appendix IV. This material has been drafted by the New Zealand concrete design code committee but has not yet been circulated for comment in New Zealand. It can therefore only be regarded as a working draft. The draft material is based on the NZPCI recommendations [9].

In the New Zealand loadings code [23] the static horizontal seismic load is found by multiplying together a number of coefficients. One of the coefficients is a structural material factor M which has a value of 1.0 for reinforced concrete structures and a value of 1.2 for prestressed concrete structures. Thus the New Zealand loadings code requires a 20% increase in the seismic load part of the design loads for prestressed concrete structures over that for reinforced concrete structures. The original draft of the New Zealand loadings code sent out for comment in 1973 did not include reference to prestressed concrete, and the commentary with the draft stated that the material factor "for prestressed concrete will be given when more work has been done on response and detailing and when viable earthquake performance records have been obtained". However the work conducted by the Seismic Committee of the New Zealand Prestressed Concrete Institute [9], based mainly on the studies conducted at the University of Canterbury, did result in recommendations for seismic loading for prestressed concrete being made in the final version of the New Zealand loadings code. Nevertheless the New Zealand loadings code [23] does include the following note of caution regarding prestressed concrete.

> "The value of M = 1.2 when used in prestressed concrete ductile frames should at this stage be regarded as tentative and subject to review when sufficient response analyses of multistorey structures subjected to a range of earthquake motions have been made; the increase of 20% above the value for reinforced concrete is intended to allow for the increased response of prestressed concrete structures.
>
> At the present (1975) state of knowledge some authorities are by no means agreed that prestressed concrete is an entirely satisfactory material for use in ductile frames and shear walls. For instance SEAOC 1973 comments as follows:
>
>> "The use of prestressing to develop ductile moment capacity will require testing and is a subject for further study. Other members within the building, not part of the space

> frame, may be precast, prestressed, composite, or any other appropriate system if adequate diaphragms and connections are developed so the building will respond to seismic input as a unit".

Other authorities are concerned at the extent to which concrete crushing and joint deformations are required to dissipate seismic energy.

Designers are urged to adopt a conservative approach until more evidence on response and performance are available. For design and detailing requirements designers are referred to the 1976 recommendations of the N.Z. Prestressed Concrete Institute."

The above statement from the New Zealand loadings code indicates that the code committee had some reservations concerning the performance of prestressed concrete subject to severe seismic shaking and obviously felt that further research was still required in a number of areas.

SOME BACKGROUND TO NEW ZEALAND RECOMMENDATIONS FOR THE DESIGN OF SEISMIC RESISTANT STRUCTURES

Nonlinear Dynamic Response of Simple Prestressed Concrete Systems

Two recent studies of the inelastic response of simple prestressed concrete systems to severe seismic shaking have been conducted at the University of Canterbury [24,25]. These studies were conducted on single-degree-of-freedom systems with idealized load-deformation characteristics and also involved comparison of the response of prestressed concrete systems with idealized partially prestressed and reinforced concrete systems. The displacement response of the systems to earthquake ground motions was calculated using a step by step numerical integration method. The prestressed, partially prestressed and reinforced concrete systems were designed for the same strength using the load factors and loadings recommended for reinforced concrete structures in New Zealand just prior to the 1976 loadings code.

The study by Blakeley [24] concluded that a prestressed concrete system designed to code loading when responding to the north-south component of the 1940 El Centro earthquake will generally have a maximum displacement of about 1.4 times that of a reinforced concrete system with the same strength, initial stiffness and percentage critical viscous damping. For the prestressed concrete system the load-displacement idealization shown in Fig. 3 was used with the empirical factors defining the loop shape determined from experimental results. For the reinforced concrete system, both an elasto-plastic idealization and a degrading stiffness idealization (due to Clough) were used.

The study by Thompson [25] extended the work by comparing the displacement response of a range of prestressed, partially prestressed and reinforced concrete systems to a number of earthquake ground motions. Prestressed concrete was idealized using the empirical loops determined by Blakeley [24] and shown in Fig. 4(b). Reinforced concrete was idealized using the

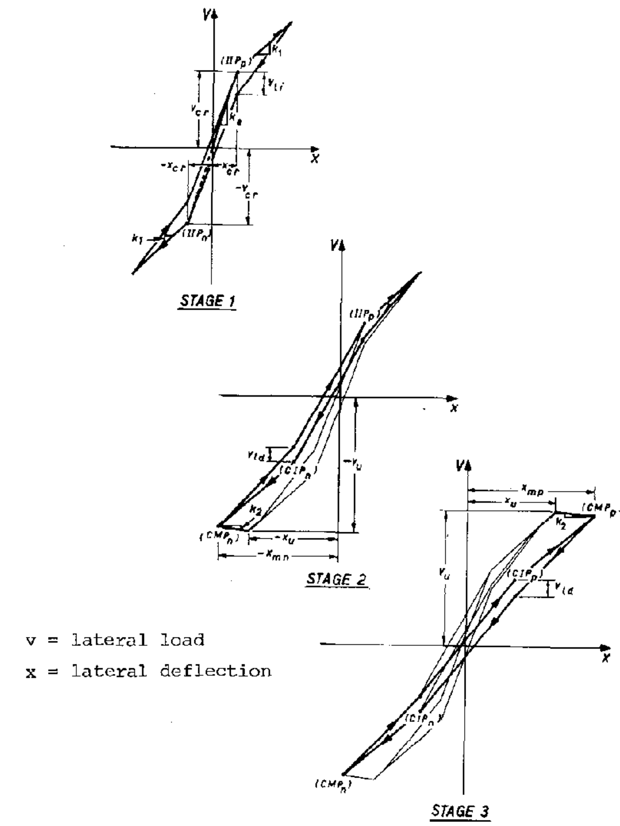
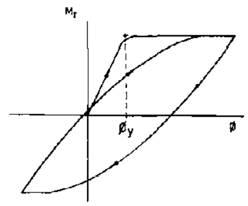


Fig. 3 Idealized Load-Deformation Relationships for Prestressed Concrete, [24]

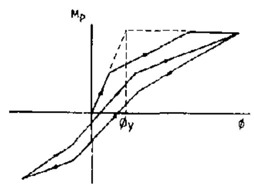
Ramberg-Osgood function illustrated in Fig. 4(a). Partially prestressed concrete was idealized by combining the idealized responses for prestressed concrete $M_p(\phi)$ with that for reinforced $M_r(\phi)$ such that at a curvature ϕ the total moment sustained by the partially prestressed system is

$$M(\phi) \quad = \quad \alpha M_r(\phi) + \beta M_p(\phi) \tag{1}$$

where the coefficient β is the ratio of the flexural strength contribution from the prestressing steel to the total flexural strength of the section and the coefficient α is the ratio of the flexural strength contribution from the nonprestressed steel to the total flexural strength of the section. The idealized non-linear cyclic load-displacement curves were of similar shape to the idealized cyclic $M - \phi$ curves of Fig. 4. The first 15 seconds of three earthquake records were used: El Centro 1940 N-S component, and the artificial earthquakes A-2 and B-2 due to Jennings, Housner and Tsai [26].

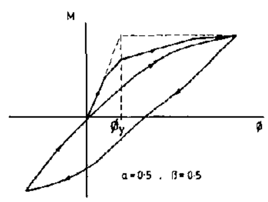


(a) Ramberg - Osgood System.



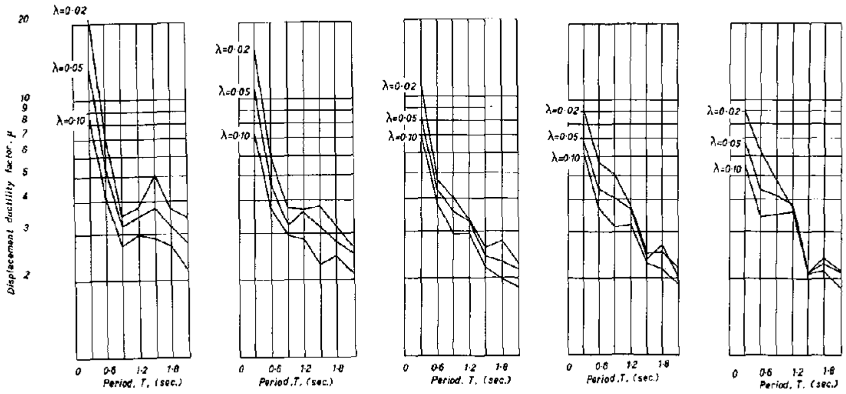
(b) Prestressed Concrete System.

Fig. 4 Moment-Curvature Idealizations for Reinforced, Prestressed and Partially Prestressed Concrete,[25]



(c) Partially Prestressed Concrete System.

The displacement ductility factor μ is defined in this study as the displacement divided by the "first yield" displacement, where the "first yield" displacement is defined as the displacement at the point of intersection of the elastic slope of the load-deflection curve and the horizontal line at ultimate load. This definition gives an identical "first yield displacement" for all loop shapes with the same initial stiffness and strength and thus is a good basis for comparison of prestressed, partially prestressed and reinforced concrete systems. Fig. 5 shows the variation of the maximum displacement ductility factor demand with period T and critical viscous damping ratio λ for prestressed, partially prestressed and reinforced concrete systems of the same strength (from code loading) responding to the El Centro 1940 N-S earthquake. The displacement ductility demand is shown to decrease with increasing T and increasing λ. Note the high displacement ductility demand for small period structures. Fig. 6 shows the variation in displacement demand for systems with various α and β values and with λ = 2% and T = 0.6 seconds. There was a general trend of



(a) Prestressed $\alpha=0$ $\beta=1$ (b) Partially prestressed $\alpha=\frac{1}{4}$ $\beta=\frac{3}{4}$ (a) Partially prestressed $\alpha=\frac{1}{2}$ $\beta=\frac{1}{2}$ (d) Partially prestressed $\alpha=\frac{3}{4}$ $\beta=\frac{1}{4}$ (e) Reinforced $\alpha=1$ $\beta=0$

Fig. 5 Maximum Displacement Ductility Factor Demand for Prestressed, Partially Prestressed and Reinforced Concrete Systems for Zone A Non Public Buildings Responding to El Centro 1940 N-S Earthquake, [25]

decreasing displacement response with increasing value of α, except for the small period (0.3 second) system. In this study the maximum displacement of the prestressed concrete system was on average found to be 1.3 times that of the reinforced concrete system with the same strength, initial stiffness and viscous damping. However, this ratio showed wide variation from the average value of 1.3, ranging between 0.7 and 2.4 for the range of cases studied of λ = 0.02 to 0.10 and T = 0.3 to 2.1 seconds. The ratios of the average displacement ductility demand of the fully prestressed system for the A-2 and B-2 earthquakes to that for the El Centro earthquake was 2.14 and 0.6, respectively. The same two ratios for the reinforced concrete system were 2.11 and 0.90.

In practice a prestressed concrete structure is likely to have a lower critical viscous damping ratio than its reinforced concrete counterpart, a factor which will tend to increase the ratio of maximum displacement response of the prestressed concrete system to the maximum displacement response of the reinforced concrete system. However this effect will be counteracted by the greater flexibility of the prestressed concrete frame (because of smaller member sizes), which results in reduced ductility demand even if the design strength decreases with increasing period of vibration in accordance with code requirements.

The above studies [24,25] indicated that prestressed concrete portal frame structures designed using the seismic loading recommended for reinforced concrete structures can be made sufficiently ductile to achieve the required displacement response. This conclusion was reached by examining the modes of inelastic displacement of portal frame structures

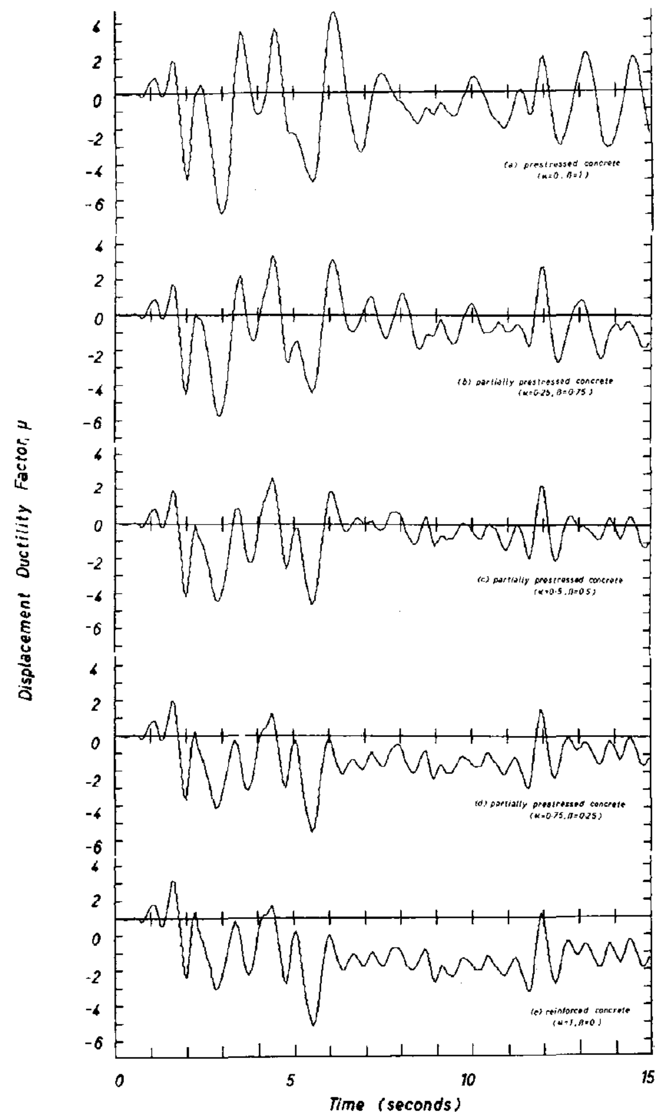


Fig. 6 Displacement Ductility Response for Zone A Non Public Buildings with λ = 2% and T = 0.6 seconds Subjected to El Centro 1940 N-S Earthquake, [25]

(e.g. sidesway mechanisms with plastic hinges in beams, columns or both) and relating the imposed displacement ductility factor to the curvature ductility factor demand of the plastic hinge sections. This approximate analysis showed that the curvature ductility requirements of the sections could be met providing the sections were properly detailed. However the higher displacement response than reinforced concrete means that a greater deformation capacity is necessary for prestressed concrete structures. Also, the higher displacement response means that a greater level of non-structural damage could be expected in a prestressed concrete building structure than in a comparable reinforced concrete structure of similar strength. If an equivalent displacement concept is adopted, and if damage is related to displacement, there is a case for the use of higher load factors for the seismic design of prestressed concrete structures than for reinforced concrete structures. The 20% increase in seismic load for prestressed concrete structures over that used for reinforced concrete structures adopted by the 1976 New Zealand loadings code [23] is an attempt to take these factors into account.

It is evident that a need exists for more nonlinear dynamic analyses of prestressed concrete buildings to establish more precisely the displacement ductility demand, and the curvature ductility required of plastic hinge sections, during severe earthquakes. Spencer, for example [28], has conducted some analyses of multistorey frames and found that the displacement ductility demand for prestressed concrete structures exceeded that for reinforced concrete structures by the same order as quoted here for single-degree-of-freedom systems. More dynamic studies on a range of building types are required.

Theoretical Studies of the Curvature Ductility of Flexural Members

The available ductility of a section is illustrated by the shape of the moment-curvature ($M - \phi$) curve. The M-ϕ curve obtained for monotonic loading gives a good approximation for the envelope curve for cyclic loading providing strength degradation due to concrete damage is not significant. Theoretical M-ϕ curves can be obtained using idealized stress-strain relationships for concrete and steel by satisfying the requirements of strain compatibility and equilibrium for the section while incrementing the extreme fibre strain. The idealized stress-strain relationships for prestressing steel and nonprestressed mild steel used in the studies by Blakeley [24] and Thompson [25] were obtained by fitting equations to measured experimental curves. The stress-strain relationship used for the confined core concrete was that proposed by Kent and Park [27] which allows for the effect of transverse steel content on the ductility of the concrete. For the cover concrete (outside the transverse steel) a stress-strain curve was used which was closer to that for unconfined concrete. Good agreement was obtained between the theoretical M-ϕ curves when checked against measured experimental curves [24,25].

The theoretical approach was used to study the effect of several beam variables. The rectangular section studied was 18 in (457 mm) deep by 9 in (229 mm) wide containing No. 3 (9.5 mm dia.) stirrups at $3\frac{1}{2}$ in (89 mm) centres, with $1\frac{1}{2}$ in (38 mm) cover to the stirrups [25]. The stress-strain curves for the concrete and high tensile steel assumed for the beam section

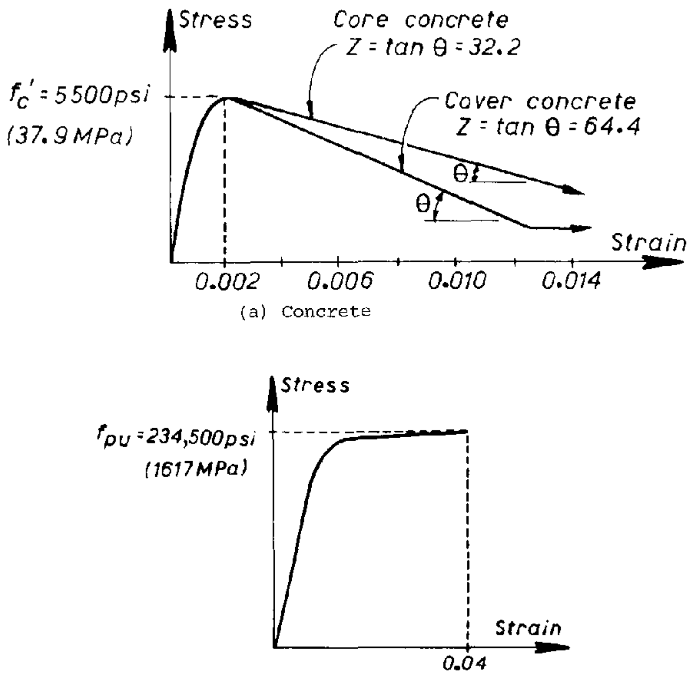


Fig. 7 Idealized Stress-Strain Curves for Concrete and High Tensile Steel, [25]

studied are shown in Fig. 7. The results of the moment-curvature analyses are illustrated in Figs. 8 to 11 and are discussed below.

Fig. 8 shows the theoretical M-ϕ curves obtained for the section eccentrically prestressed by a tendon near the extreme tension fibre. The reduction of the available curvature ductility with increasing content of prestressing steel has led the author, Blakeley [24] and Thompson [25] to recommend that for seismic design the following requirement should be observed

$$\rho_p f_{ps} / f_c' \leqslant 0.2 \qquad (2)$$

where $\rho_p = A_p/bd$, A_p = area of prestressing steel, b = width of section, d = effective depth of section, f_{ps} = stress in prestressing steel at the flexural strength, and f_c' = compressive cylinder strength of concrete. For the section shown in Fig. 8, Eq. 2 requires the gross area steel ratio

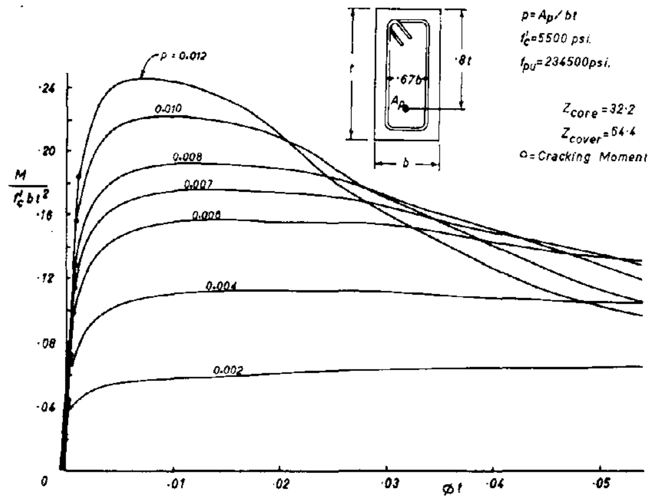


Fig. 8 Moment-Curvature Relationships for Section With Various Contents of Eccentrically Placed Prestressing Steel,[25]

$p = A_p/bt$, where t = section overall depth, to be $p \leqslant 0.0046$ and it is evident from the curves of that figure that this requirement will ensure reasonable curvature ductility. Chapter 18 of ACI 318-71 [4] allows 50% more prestressing steel than Eq. 2 (i.e. $p \leqslant 0.0069$ for the section of Fig. 8), but it is evident from Fig. 8 that the requirement of Eq. 2 is more reasonable than the less severe ACI requirement if the moment is to remain near maximum over a large range of curvature

Fig. 9 shows that with prestressing steel present in the compression zone the theoretical curvature ductility of the section is not reduced by increase in prestressing steel content. This is a consequence of the prestressing steel acting as compression steel at large curvatures, providing that steel is restrained against buckling by the surrounding concrete and transverse steel. Successive cycles of reversed flexure may cause concrete damage leading to buckling. To avoid excessive concrete damage it would appear reasonable to require that all beam sections be capable of reaching a specified curvature at a given extreme fibre concrete strain. This in general requires a limitation on the maximum allowable neutral axis depth since for sections with tendons at various levels down the depth it is difficult to set a limiting value for $\rho_p f_{ps}/f'_c$ because tendons at different levels lead to different M-ϕ curves. It is to be noted that Eq. 2 means that $A_p f_{ps} \leqslant 0.2f'_c bd$ and hence that the maximum possible tensile force in the tendons at the flexural strength is $0.2f'_c bd$. On this basis the maximum possible depth of the concrete compressive

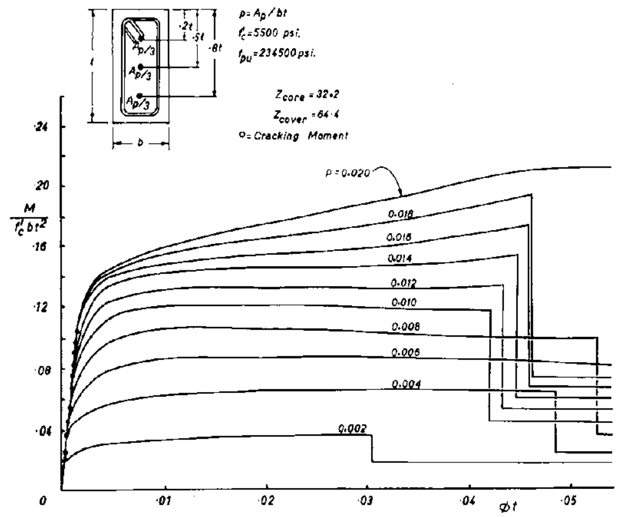


Fig. 9 Moment-Curvature Relationships for Section With Three Symmetrically Placed Prestressing Tendons, [25]

rectangular stress block is

$$a = \frac{0.2f'_c bd}{0.85f'_c b} = 0.235d$$

If d = 0.85t this requirement may be written as

$$a \leqslant 0.2t \qquad (3)$$

where t is the overall depth of the section. Hence for sections with tendons placed at various levels down the depth it could be required that the ratio of the depth of the concrete compressive rectangular stress block to the overall member depth is not greater than 0.2. This will result in the curvature of a member with general tendon positions for a given ultimate concrete strain always being at least equal to that for a member with all tendons placed near the extreme tension fibre. The requirement of Eq. 3 has been adopted by the Seismic Committee of the New Zealand Prestressed Concrete Institute [9] and is part of the proposed New Zealand provisions given in Appendix II.

It is of interest to compare M-ϕ curves for sections with different arrangements of prestressing steel. In seismic design moment reversals will require many sections to have both negative and positive moment strength and hence tendons will often exist near both extreme fibres of the section and near mid-depth. Fig. 10 shows theoretical M-ϕ curves for the section with up to five tendons symmetrically distributed down the depth. The total prestressing steel content is the same for each of the five cases, being 0.00696 of the gross concrete section. For the case of all steel concentrated in a single central tendon, N = 1, the moment capacity is more sensitive to a deterioration of the compressed concrete and a significant reduction of moment capacity occurs at high curvatures. However there is little difference in the moment capacity for two or more tendons and such sections are able to maintain near maximum moment capacity at high curvatures. Therefore two or more tendons are to be preferred.

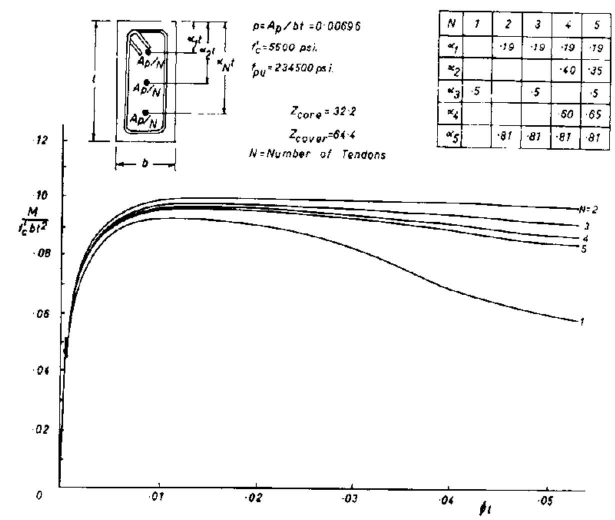


Fig. 10 Moment-Curvature Relationships for Section With Various Numbers of Symmetrically Placed Prestressing Tendons, [25]

The effect of increasing prestressing steel content on the curvature ductility of partially prestressed sections is also of interest. Fig. 11 shows the M-ϕ curves for the section with symmetrically placed non-prestressed steel top and bottom having a total area of 0.0124 of the gross concrete section, and various contents of prestressing steel. The non-prestressed steel had a yield strength of 40,000 psi (276 MPa) and the stress-strain curve adopted for it was as measured in typical samples, including strain hardening which commences at 16 times the yield strain. The figure shows that in this case increase in the content of prestressing steel results in an increase in flexural strength without significant reduction in ductility.

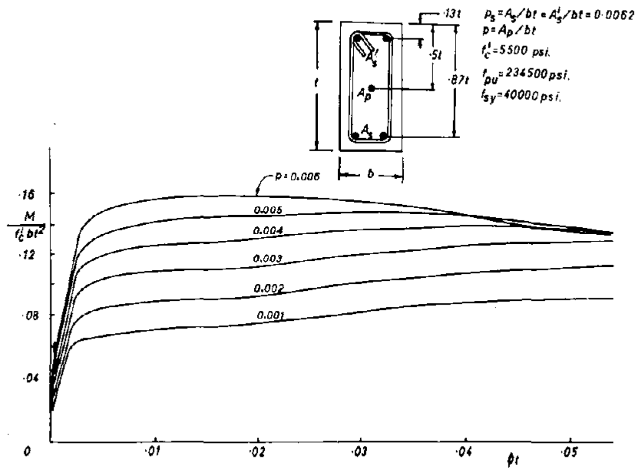


Fig. 11 Moment-Curvature Relationships for Partially Prestressed Section With Various Contents of Nonprestressed Steel, [25]

Other M-ϕ analyses also have shown that for the section studied a 3 to 4 in (76 to 102 mm) spacing of closed stirrups gave reasonable curvature ductility and that greater spacing was undesirable, and that the concrete cover thickness to the stirrups should be made as small as possible to avoid a significant reduction in moment capacity when the cover concrete crushes.

The ductility of prestressed concrete columns has been considered elsewehere [24]. As expected, the available curvature ductility of a prestressed concrete column reduces with axial load level, and as for reinforced concrete columns special transverse confining steel is necessary in prestressed concrete columns once the axial load exceeds some nominal value such as $0.1P_o$, where P_o = strength of column when load is applied with zero eccentricity.

The experimental results of tests [24,25] have given good confirmation of the results of the theoretical moment-curvature analyses.

The requirements for flexural steel in members, and for transverse steel, are presented in a form suitable for inclusion in a code in Appendix II and III and are discussed briefly in Appendix IV.

Tests on Beam-Column Joints

A critical aspect of the experimental results from prestressed and partially prestressed concrete beam-column joint specimens was the joint core behaviour [25]. Shear reinforcement in the joint core of the specimens had been designed using the method recommended for reinforced concrete in Appendix A of ACI 318-71 [4]. In this method the horizontal shear induced in the joint core by the beam internal forces and column shear is assumed to be carried by a mechanism involving the concrete (from aggregate interlock, etc) plus a mechanism involving the hoops assuming 45° diagonal tension cracking and concrete struts, as in structural members. The columns had been designed to be stronger than the beams and the test specimens were subjected to static cyclic loading simulating seismic loading. In all the units tested the beams reached at least 95% of their theoretical flexural strength in the first inelastic loading cycle accompanied by yielding of the joint core hoops in some units. For those units in which the joint core hoops yielded, further inelastic loading cycles resulted in a degradation of the joint core shear strength due to repeated opening and closing of diagonal tension cracks in alternating directions. In these units the shear strength of the joint core governed the strength of the unit and the inelastic deformation of the unit occurred mainly in the joint core. Thus although the ACI 318-71 Appendix A approach allowed the attainment of the required joint core shear strength satisfactorily in the first inelastic load cycle, in some units degradation of joint core shear capacity occurred in subsequent inelastic loading cycles. Thus the ACI 318-71 Appendix A approach for joint core shear strength cannot be regarded as being adequate for plane frames subjected to intense cycles of seismic loading. In those units without a prestressing tendon at middepth joint core shear failure always occurred, illustrating the benefit to joint core behaviour to be gained from the presence of a central tendon.

It appears that the shear carried by the concrete should be neglected except where either the vertical compressive stress on the joint core is significant or when the beams are detailed so that yielding of the flexural steel in the beams cannot occur adjacent to the column faces. Where prestresssing tendons exist near the beam mid-depth they can be taken as carrying some horizontal shear force. Vertical shear force can be provided for by ensuring that intermediate longitudinal column bars exist between the corner bars at the side faces of the column.

In the tests [25] the critical diagonal tension crack was observed to run from corner to corner of the joint core. It is recommended that the shear force to be carried by the horizontal shear reinforcement in the joint core should be determined from the force in the bars which cross the corner to corner crack.

Detailed requirements for joint shear reinforcement are discussed elsewhere (see Appendix II and III).

CONCLUSIONS

Background studies have indicated that earthquake resistant prestressed concrete structures can be designed. The greater deflection response of a prestressed concrete structure to a severe earthquake means that consideration should be given to the use of greater seismic design loads for prestressed concrete structures than for reinforced concrete structures. Care should be taken to ensure suitable positions of plastic hinges and adequate plastic hinge rotation under severe earthquake loading in the members of the structure. Design should be such that when loaded into the inelastic range the structure deforms in a ductile manner due mainly to plastic hinge rotation of flexural members and that all brittle forms of failure are avoided. Beam-column connections need careful detailing if strength degradation due to seismic load reversals is to be avoided.

On the whole it appears that sufficient evidence is now available to enable comprehensive seismic design provisions for prestressed concrete to be produced. The proposed New Zealand recommendations provide an example of a suitable set of recommendations.

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APPENDIX II - PROPOSED PROVISIONS FOR NEW ZEALAND CONCRETE DESIGN CODE - "CHAPTER 22: PRESTRESSED CONCRETE MEMBERS - ADDITIONAL SEISMIC REQUIREMENTS"

22.0 Notation

a = depth of equivalent rectangular stress block
D = dead loads as defined by NZS 4203:1976[1]
E = earthquake loads as defined by NZS 4203:1976[1]
f'_c = specified compressive strength of concrete
h = overall thickness of member
L_R = reduced live loads as defined by NZS 4203:1976[1]
N_u = design axial compressive load acting normal to the cross section
P_o = axial load strength of member when the external load is applied without eccentricity
U = required ultimate load capacity.

22.1 Scope

This section covers the design of prestressed and partially prestressed concrete members of ductile moment resisting space frames and joints between such members.

1. "Code of Practice for General Structural Design and Design Loadings for Buildings", NZS 4203:1976, Standards Association of New Zealand.

22.2 Materials

22.2.1 - Wires and strands for tendons in prestressed concrete shall conform with the provisions of NZS 1417[2] or BS 3617[3] respectively, or shall be of equivalent quality.

22.2.2 - f'_c shall not exceed 8000 psi (55 MPa) unless special transverse reinforcement is provided.

22.2.3 - Post-tensioned tendons in moment resisting frame members shall be grouted, except as allowed by Section 22.6.3.

22.3 Design of Flexural Members

22.3.1 - Dimensions of prestressed flexural members shall be in accordance with the provisions of Section 16.2*.

22.3.2 - Provided the limits to flexural steel are in accordance with Section 22.3.3, elastically derived bending moments may be redistributed in accordance with the provisions of Section 16.3*.

22.3.3 - The content of prestressed plus non-prestressed flexural steel shall be such that (a) at the flexural capacity of sections in potential plastic hinge zones

$$a/h \leqslant 0.2 \qquad (22.1)$$

unless special transverse reinforcement is provided in accordance with that recommended for potential plastic hinge regions in reinforced columns* in which case

$$a/h \leqslant 0.3 \qquad (22.2)$$

and (b) the flexural strength of the section exceeds the cracking strength, when allowance is made for likely variations in prestress and the strength of materials. In the absence of special studies it shall be assumed that the maximum concrete tensile strength prior to cracking is $12\sqrt{f'_c}$ psi ($\sqrt{f'_c}$ MPa), and allowance shall be made for a variation of 10% in the calculated level of prestress at the section under consideration.

22.3.4 - The effective reinforcement near column faces in T and L beams built integrally, with slabs shall be subject to the provisions of Sections 16.4.4(a) to 16.4.4(d)*. In all cases at least 75% of the tensile force capacity in each face, providing the required flexural capacity, shall be provided by reinforcement passing through, or anchored in, the column core. When longitudinal reinforcement is governed by the load combination $U = 1.4D + 1.7L_R$, then only 75% of the tensile force capacity required for the load combination $U = D + 1.3L_R + E$ is required to be provided by steel passing through or anchored in the column core.

2. "Steel Wire for Prestressed Concrete", NZS 1417:1971, Standards Association of New Zealand.
3. "Seven Wire Steel Strand for Prestressed Concrete", BS 3617:1971, British Standards Institution, London.

* See additional sections in Appendix III.

22.3.5 - Stirrups ties, not less than 6 mm diameter, shall be provided to restrain buckling of compression bars, in accordance with the provisions of Section 16.5*.

22.4 Design of Columns

22.4.1 - Design of prestressed concrete columns in moment resisting frames shall be subject to the provisions for reinforced concrete columns, and to the additional requirements of Sections 22.4.2 and 22.4.3.

22.4.2 - The flexural strength of a column section shall be greater than the maximum likely column cracking moment, as calculated in accordance with Section 22.3.3(b), including the effect of axial load and prestress.

22.4.3 - Special transverse reinforcement in accordance with that recommended for potential plastic hinge regions in reinforced concrete columns* shall be provided in the end regions of columns if either:

(a) the design load on the column exceeds $0.1P_o$, where P_o includes the effect of prestress,

or (b) if a/h is greater than 0.2 at the flexural capacity of the section,

or (c) ductility of one or two storey frames or bridge piers is provided by column sidesway mechanisms involving energy dissipation by deliberate plastic hinging in columns.

22.5 Shear Strength Requirements

Shear strength requirements shall be in accordance with the provisions of Chapter 19.* At sections of beams and columns designed to provide ductility by plastic hinging, the value of N_u in Eq. 19.1* shall include the prestress force, after losses, of only those tendons situated within the central third of the section depth. At sections away from potential plastic hinge regions, N_u may include the prestress force, after losses, of all tendons at the critical section.

22.6 Joints in Prestressed Frames

22.6.1 - Anchorages for post-tensioned tendons shall not be placed within beam-column joint cores.

22.6.2 - Except as provided by Section 22.6.3, the beam prestressing tendons which pass through joint cores shall be placed at the face of the columns so that at least one tendon is located at not more than 6 in (150 mm) from the beam top and at least one at not more than 6 in (150 mm) from the beam bottom.

22.6.3 - When partially prestressed beams are designed with mild steel reinforcement providing at least 80% of the seismic resistance, prestress may be provided by one or more tendons passing through the joint core and located within the middle third of the beam depth, at the face of the column. In such cases post-tensioned tendons may be ungrouted, provided anchorages are detailed to ensure that anchorage failure, or tendon detensioning, cannot occur under seismic loads.

* See additional sections in Appendix III

22.6.4 - Ducts for post-tensioned grouted tendons through beam-column joints shall be corrugated, or provide equivalent bond characteristics.

22.6.5 - Connections between precast members at beam-column joints shall be acceptable provided that the jointing material has sufficient strength to withstand the compressive and transverse forces to which it may be subjected. The interfaces shall be roughened or keyed to ensure good shear transfer and the retention of the jointing material after cracking.

22.6.6 - Design of joint reinforcement shall be in accordance with the provisions of Chapter 21*.

APPENDIX III - PROPOSED REVISIONS FOR NEW ZEALAND CONCRETE DESIGN CODE - DESCRIPTION OF SECTIONS REQUIRED IN ADDITION TO CHAPTER 22

The main parts of the additional sections which affect prestressed concrete are summarized below:

Dimensions

Section 16.2.1 requires that the depth and width of rectangular continuous flexural members and columns with moments of opposite signs at each end shall be such that

$$l_n/b_w < 16 \quad \text{and} \quad l_n h/b_w^2 < 65$$

when l_n = clear span, h = overall depth of section and b_w = web width.

Section 16.2.2 requires that the depth and width of rectangular cantilever members shall be such that

$$l_n/b_w < 10 \quad \text{and} \quad l_n h/b_w^2 < 38$$

Section 16.2.3 requires that for T and L beams in which the flange is integrally built with the web, the width of web and depth of section shall be such that the limiting l_n/b_w values given above are not exceeded by more than 50%.

Moment Redistribution

Section 16.3.1 requires that the amount of moment redistribution used in design for any span of a beam forming part of a continuous structure shall not exceed 30% of the maximum moment derived for that span from elastic analysis for any combination of design earthquake and gravity loading.

Section 16.3.2 requires that the redistribution of shear forces between columns shall not exceed 15% of the smaller shear force acting on any of the columns involved.

Section 16.3.3 requires that at all times the requirements of static equilibrium shall be satisfied.

* See additional sections in Appendix III

Longitudinal Reinforcement in Flexural Members

Section 16.4.4 requires that in T beams built integrally with slabs the reinforcement in the following widths of slab each side of the column shall be considered to be effective in adding to the moment strength of the member at the faces of the column:

(a) 4 times the slab thickness from the sides of interior columns where a transverse beam of similar dimensions frames into the column.

(b) $2\frac{1}{2}$ times the slab thickness from the sides of an interior column where no transverse beam exists.

(c) 2 times the slab thickness from the sides of an exterior column where a transverse beam (along the edge of the floor) of similar dimensions frames into the column into which the reinforcement is anchored.

(d) Zero times the slab thickness from the sides of an exterior column where no transverse beam (along the edge of the floor) exists.

Transverse Reinforcement in Flexural Members

Section 16.5.1 requires that stirrup ties, not less than $\frac{1}{4}$ in (6 mm) diameter, shall be provided in accordance with Sections 16.5.2 to 16.5.6 in potential plastic hinge regions defined as follows:

(a) Over a length equal to twice the member depth, measured from the critical section at the face of the supporting column, wall or beam toward midspan, at both ends of the flexural member.

(b) Where the member strength is such that the critical section of the plastic hinge is located at a distance of not less than the depth of the member away from a column or wall face over a length that commences between the column or wall face at least 0.5h from the critical section and extends at least 1.5h past the critical section toward midspan, where h = member depth.

(c) Over lengths equal to the flexural member depth on both sides of a section, away from the support, where flexural yielding may occur in one face of the member only as a result of inelastic displacements of the frame.

Section 16.5.2 requires that stirrup ties shall be arranged so that each upper or lower face beam bar or bundle of bars is restrained against buckling in any direction by a 90° bend of a stirrup tie, except that when two or more bars, at not more than 8 in (200 mm) centres apart, are so restrained by the same stirrup tie any bars between them are exempt from this requirement.

Section 16.5.3 requires that the area of one leg of a stirrup tie, A_{te}, in the direction of potential buckling of the main longitudinal bar, shall be computed from

$$A_{te} = \frac{\Sigma A_b f_y}{16\, f_{yt}} \cdot \frac{s}{4} \qquad (16\text{-}8)$$

where ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie including the tributory area of any bars exempt from being tied in accordance with Section 16.5.2, f_y = yield strength of longitudinal steel, f_{yt} = yield strength of ties and s = tie spacing in inches. Longitudinal bars centered more than 3 in (75 mm) from the inner face of stirrup ties need not be considered in determining the value of ΣA_b.

Section 16.5.4 requires that the longitudinal bars placed in second or third layers from the top or the bottom face of a beam should be tied laterally as required in Section 16.5.3 if they are centered further than 4 in (100 mm) from the first layer.

Section 16.5.5 requires that except as permitted in Section 16.5.6 the spacing of stirrup ties shall not exceed d/4, six times the diameter of the longitudinal bar to be restrained in the outer layers, or 4 in (100 mm), whichever is least. The first stirrup tie in a beam shall be as close as practicable to the column ties but not further than 2 in (50 mm) from the column tie.

Section 16.5.6 requires that in potential positive hinge regions defined in Section 16.5.1 (c) the spacing of stirrup ties shall not exceed d/3, 12 times the diameter of the longitudinal compression bar to be restrained,or 4 in (200 mm), whichever is least.

Section 16.5.7 allows stirrup ties to fully contribute to the shear strength of flexural members.

Shear Strength Requirements of Members

Section 19.1.1 requires that in flexural members the design shear force shall be determined for when the flexural overcapacity is reached at the most probable plastic hinge locations within the span and the factored gravity load is present.

Section 19.2.1 requires that in flexural members in regions where stirrup ties are required according to Section 16.5.1(a) and (b) the nominal shear stress carried by the concrete v_c shall be assumed to be zero for any seismic load combination.

Section 19.2.2 requires that in end regions of columns of not less than the depth of the member in the direction of the shear force, one-sixth of the clear height of member, or 18 in (450 mm), the nominal shear stress carried by the concrete v_c shall be assumed to be zero unless the design axial compressive force produces an average stress in excess of $0.1f'_c$ when v_c shall be computed from

$$v_c = 3\left(1 + \frac{f'_c}{3630}\right)\sqrt{\frac{N_u}{A_g} - \frac{f'_c}{10}} \text{ psi} \qquad (19\text{-}1)$$

$$\left[v_c = 0.25\left(1 + \frac{f'_c}{25}\right)\sqrt{\frac{N_u}{A_g} - \frac{f'_c}{10}} \text{ MPa} \right]$$

where f'_c = specified compressive strength of concrete, N_u = axial compressive load acting on cross section, and A_g = gross area of cross section.

In regions other than those defined in Sections 19.2.1 and 19.2.2 shear is carried by the concrete in accordance with the equations of ACI 318-71. In all regions shear reinforcement is provided to carry that shear not allocated to the concrete.

Shear Strength Requirements of Beam-Column Joints

Section 21.2 requires that the design horizontal shear force in the joint, V_j shall be calculated from the concentrated internal concrete and steel beam forces, when the overstrength of the weaker members is developed, and the corresponding shear force in the column.

Section 21.4 requires that the horizontal shear force carried by the concrete V_c shall be assumed to be zero except when the minimum average compressive stress on the gross area of the column above the joint, including prestress where applicable, exceeds $0.1f'_c$ then V_c shall be taken as v_c from Eq. (19.1) times A_{gj}, where A_{gj} = gross column area or that part of the column area within lines which are 3 in (75 mm) each side of the beam under consideration, which ever is smaller. When beams are prestressed through the joint the horizontal shear force carried by the prestressing steel shall be taken as $0.7P_{cs}$, where P_{cs} is the force in the prestressing steel located within the middle one-third of beam depth. When beams are detailed so that plastic hinges cannot occur within a distance of one-half a beam depth from the column face a greater value for v_c than above can be used. The horizontal shear reinforcement provided to carry the horizontal shear force not carried by the concrete or the prestressing steel V_s shall be calculated from

$$A_v = V_s/nf_y\phi \qquad (21.7)$$

where A_v = area of each set of shear reinforcement, n = number of layers of shear reinforcement in joint core, f_y = yield strength of shear reinforcement, and ϕ is the capacity reduction factor = 0.85.

Section 21.5 requires that vertical shear reinforcement in the joint shall exist in the form of either intermediate column bars placed between corner bars or special vertical ties or bars in the joint region. Where intermediate column bars are utilised the spacing of column bars in the side faces shall not exceed 6 in (150 mm) and there should be at least one intermediate column bar between the corner bars at each side face of the column.

Section 21.6 requires that the amount of horizontal confining steel in the joint shall not be less than the special transverse steel required in the adjacent column ends except that this amount may be halved when beams frame in on four faces of the column.

Special Transverse Reinforcement of Reinforced Concrete Columns

Chapter 17 requires that special transverse reinforcement in the form of spiral or hoop reinforcement, with or without supplementary cross ties, shall be placed over the end regions of the columns. The end region is the greater of the maximum column side dimension or diameter, or one-sixth of the clear height of the column, or 18 in (450 mm). The quantity of special transverse steel is based on the 1973 SEAOC Code amount, modified to take level of maximum design compressive load P_e into account. For circular spirals the recommended quantity is 50% of the SEAOC amount at $P_e/f'_cA_g = 0.1$ increasing linearly to 125% of the SEAOC amount at $P_e/f'_cA_g = 0.7$. For rectangular hoops and supplementary cross ties the recommended quantity is 50% of the SEAOC amount at $P_e/f'_cA_g = 0.1$ increasing linearly to 133% of the SEAOC amount at $P_e/f'_cA_g \geqq 0.6$. Special columns with $P_e > 0.7f'_cA_g$, and hoop columns with $P_e > 0.6f'_cA_g$, should not be used.

APPENDIX IV - SOME COMMENTARY ON PROPOSED PROVISIONS FOR NEW ZEALAND CONCRETE DESIGN CODE - "CHAPTER 22: PRESTRESSED CONCRETE MEMBERS - ADDITIONAL SEISMIC REQUIREMENTS"

22.1 Scope

The design requirements for prestressed and partially prestressed members are similar in principle to those for non-prestressed members and many provisions in previous chapters of the code apply to prestressed as well as to reinforced members. Chapter 22 defines provisions peculiar to members with prestressing, and indicates which sections of other chapters shall be adopted in design.

22.2 Materials

22.2.1 - It is of particular importance that the prestressing steel complies with the specified requirements for percentage elongation at rupture, to ensure adequate ductility.

22.2.2 - The slope of the falling branch of the concrete stress-strain curve increases, and the ultimate compressive strain reduces, with increasing concrete strength. Consequently, unless special transverse reinforcement in accordance with that recommended for the potential plastic hinge regions of reinforced concrete columns is provided to increase the ultimate compressive strain, very high strength concrete should not be used in plastic hinge regions.

22.2.3 - The use of ungrouted post-tensioned tendons in moment resisting frames is undesirable for the following reasons:

(a) The tendons remain in the elastic range and therefore total reliance is placed on the concrete for energy dissipation and compressive strength.

(b) Ductility is likely to be provided by inelastic flexural strains associated with a single wide crack at the critical section. The reduced equivalent plastic hinge length, in comparison with that for a bonded tendon, may significantly reduce the available ductility.

(c) It is difficult to accurately predict the ultimate moment capacity of ungrouted sections under reversed loading. Consequently column moments induced by beam overstrength are equally difficult to predict.

(d) Fluctuation of tendon forces could cause failure of the anchorage with the catastrophic result of release of prestressing force.

These arguments are less valid when the prestress is used mainly to balance gravity loads, with non-prestressed steel reinforcement providing the bulk of the seismic resistance. Under these circumstances, ungrouted tendons are permitted, provided that the provisions of Section 22.6.3 are satisfied.

22.3 Design of Flexural Members

22.3.1 - The limiting dimensions are based on considerations similar to that used in the British Code of Practice CP110:1972 to prevent lateral instability. However to take into account stiffness degradation due to cyclic loading only one-third of the limiting slenderness ratios of the British Code have been allowed.

22.3.2 - Design for ductile behaviour implies substantial capacity for moment redistribution. Moments found from elastic analysis can be redistributed to gain a more advantageous design moment envelope and thus a more efficient design.

22.3.3(a) - The object is to ensure ductile behaviour in plastic hinge zones. The background to Eq. (22.1) has been discussed previously (see pages 14 to 16). If greater flexural steel contents than that implied by Eq. (22.1) are necessary then special transverse confining steel should be provided to ensure adequate curvature ductility.

22.3.3(b) - The section should crack before the flexural strength is reached, otherwise a brittle failure may result. Allowance must be made for the case of a high modulus of rupture [for example $10\sqrt{f'_c}$ psi ($0.8\sqrt{f'_c}$ MPa) or higher is occasionally measured in tests], and the concrete cylinder strength f'_c being higher than specified. The value of maximum concrete tensile strength prior to cracking of $12\sqrt{f'_c}$ psi ($\sqrt{f'_c}$ MPa) required in the absence of special studies corresponds to $10\sqrt{1.5f'_c}$ psi ($0.8\sqrt{1.5f'_c}$ MPa), that is, allowance is made for concrete strength exceeding the specified value by 50%. Specifying a minimum ratio between ultimate and cracking moment is non-conservative for lightly prestressed members, and overly conservative for members with heavy prestress and axial load.

22.3.4 - The effect of slab steel in contributing to the ultimate beam moments may be considered in assessing beam overstrength, and must be considered when calculating the moments induced in columns when plastic hinges form in beams. The slab widths considered effective are approximations which attempt to take into account the torsional stiffness of the slab regions each side of the column in the various cases.

22.3.5 - Stirrup ties are necessary to confine the concrete and to prevent buckling of longitudinal steel in potential plastic hinge regions. The specified maximum spacing of 4 in (100 mm) is to ensure reasonable confinement of concrete, and of six longitudinal bar diameters is to ensure that buckling of nonprestressed bars will not occur when cycles of reversed loading cause a reduction in the tangent modulus of the steel due to the Bauschinger effect. When yielding at a section can only occur due to moment in one direction the requirements necessary to prevent bar buckling can be relaxed. Eq. (16.8) is based on the assumption that the tie force should not be less than one-sixteenth of the longitudinal force in the bar or bars it is to restrain except that when the tie spacing is less than 4 in (100 mm) this requirement can be reduced.

22.4 Design of Columns

22.4.3 - The special transverse steel recommended when the design load exceeds $0.1P_o$ is as for reinforced concrete columns. The amount of special transverse steel recommended is based on the 1973 SEAOC Code amounts modified to take account of the level of the axial compressive load on the column, since analytical studies of reinforced concrete columns have shown the SEAOC amounts to be conservative at low axial load levels and unsafe at very high axial load levels.

The a/h limit specified for beams in Section 22.3.3 is also used as a limit for columns because deeper compression stress blocks may require special transverse steel to adequately confine the concrete at ultimate capacity. Special care needs to be taken for columns deliberately designed to dissipate energy by the formation of column hinges, particularly if high axial load levels can occur. The amount of special transverse steel should be as required for the potential plastic hinge regions of reinforced concrete columns.

22.5 Shear Strength Requirements

In flexural members, v_c is assumed to be zero in potential plastic hinge regions due to the degradation of the shear carried by the concretecaused by reversed loading. Eq. (19.1) for the end regions of columns gives a gradual increase in v_c for compressive stresses above $0.1f'_c$ ($v_c = 0$ at smaller compressive stresses). In end regions of columns the axial compressive force N_u on the section in Eq. (19.1) should include the force from these tendons close to mid-depth of the section but the tendons near the extreme fibres should not be included since they may not be fully effective after several cycles of inelastic plastic hinge rotation.

22.6 Joints in Prestressed Frames

22.6.1 - Anchorages must be kept out of beam-column joint cores in order to avoid tensile bursting stresses in a region already subjected to severe diagonal tension from beam and column forces. At exterior joints, anchorages can be placed in stubs outside the joint core region.

22.6.2 - Such an arrangement of tendons results in more ductile plastic hinge behaviour of beams under inelastic cyclic loading than when the tendons are all concentrated at mid-depth in the beam.

22.6.3 - A possible design technique to satisfy this section would involve prestressing steel designed only to balance gravity loads (for example, $D + 1.3L_R$), with the additional required seismic capacity and ductility provided by top and bottom layers of non-prestressed steel reinforcement. Under these circumstances the beam prestressing tendon or tendons at the column faces could be located in the central third of the beam depth to avoid loss of effective prestress force under reversed inelastic cycling, and to improve the shear resistance of the joint core.

22.6.4 - Corrugated ducts provide the best bond transfer between tendon and concrete and are thus preferred in regions of high bond stress, such as joint cores.

22.6.5 - Limited testing has indicated that precast joints at the faces of columns can function effectively with no other connection through the jointing material than the grouted tendons. Some form of mechanical interlock is required to hold the jointing material in place. Where possible, the plastic hinge zones should be forced to form away from the jointing faces, by the use of suitable reinforcing details, haunches, cruciform columns, or other means.

22.6.6 - Beam-column joints have been discussed previously (see page 19).

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

SEISMIC DESIGN OF PRECAST CONCRETE PANEL BUILDINGS

by

James M. Becker
Assistant Professor of
Civil Engineering
M.I.T.

and

Carlos Llorente
Research Assistant
Dept. of Civil Engineering
M.I.T.

INTRODUCTION

The continually expanding use of precast concrete buildings in seismic regions is presenting new challenges in the area of seismic design. One form of such construction, precast concrete panel buildings, has found wide acceptance throughout the world and has begun to find its place within the building industry of the United States. Panelized building systems are constructed of large precast concrete panels used as both vertical and horizontal structural components. Vertical elements, referred to as panels, are combined to create both load bearing and non-load bearing shear walls. Horizontal elements act as both floor and roof systems for gravity loads and as shear diaphragms for resisting lateral loads. Precast concrete panel construction has found its widest use in residential construction where the panels may serve as multi-functional building elements.

The extensive development of panelized construction was a direct response to housing needs throughout Europe immediately after World War II. As with all new structural forms, panelized systems have undergone the pains of growth and acceptance. Because of their viability as a design solution to housing problems in Europe, they were well on their way to wide usage before their sensitivity to progressive collapse was highlighted by the Ronan Point failure of 1968 [31].* Ronan Point and subsequent examinations into abnormal loads and progressive collapse [13,26,39,43,84,89,105] illustrated one of the fundamental differences between cast-in-place and precast construction - the necessity of developing the overall structural integrity of the building by establishing continuity in connection regions.

Precast concrete panel buildings were initially developed for use in regions that were essentially nonseismic in character. However, their use soon spread to the seismic regions of Europe, Japan and North and South America. In many seismic regions, initial use was limited to low rise (five stories or less) structures [76,100]. This initial limitation never existed in some regions and was eventually relaxed in the others. It is common today to find panelized buildings ranging from 10 to 20 stories in seismic regions throughout the world.

Precast concrete elements find many uses in today's construction industry. This paper will address only those structures for which panel type elements constitute the main load carrying members. Some of the questions of overall system behavior are similar to those of cast-in-place shear wall construction and reinforced masonry construction. Joinery problems associated

* Numbers in brackets refer to references listed in Appendix 1.

with floor framing are also shared with some reinforced masonry construction. In general, however, large panel precast concrete buildings present a unique challenge in seismic design, a challenge that contains both opportunities and problems.

The body of this paper is divided into six additional sections. The first section acquaints the reader with the basic configuration of panelized buildings and the importance of their connection regions. The second section identifies major seismic design considerations for panelized systems. The third section examines the structural behavior of the basic components of the system: panels, connections and floor systems. The fourth section discusses the seismic response of panelized buildings based upon observation and mathematical analysis. The fifth section brings the previous sections into focus by exploring their seismic design implications. The sixth section summarizes the paper and identifies research that will be needed to answer many of the questions raised in the paper.

PRECAST CONCRETE PANEL BUILDINGS

Precast concrete panel buildings are constructed of large planar concrete elements. These large panel prefabricates are assembled into three basic structural configurations [44]:

1) *cross-wall.* The cross wall panel structure is one in which the bearing walls are perpendicular to the building axis. One-way floor and roof slabs span between the bearing walls (see Figure 1). Non-bearing wall panels parallel to the building axis provide lateral load resistance in the longitudinal direction.

2) *long-wall.* In long wall, or spine systems, the bearing walls are parallel to the building axis, and again, one-way slabs span between bearing walls (see Figure 2). Non-bearing walls perpendicular to the building axis provide bracing in that direction.

3) *two-way system.* The third type of large panel system, the two-way or ring type, consists of bearing walls in both directions, carrying two-way slabs (see Figure 3). These slabs must be bay-sized, unlike the much narrower planks common in one-way systems.

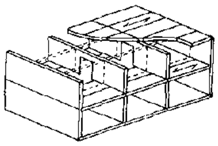
Figure 1: Cross-Wall System

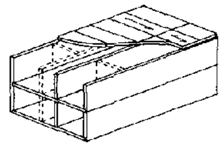
Figure 2: Long Wall System

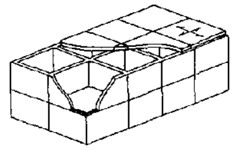
Figure 3: Two-Way System

A fourth categorization, called a mixed system, is often used, in which one-way floor systems are supported by cross walls and long (or spine) walls in different portions of the structure.

Connections in large panel construction serve several basic functions. Zeck[107], in her report entitled "Joints in Large Panel Precast Concrete Structures" has surveyed the wide range of connections used to serve these functions. Figure 4 presents a classification system developed by Zeck for connections based on their type of construction and on their location in the structure.

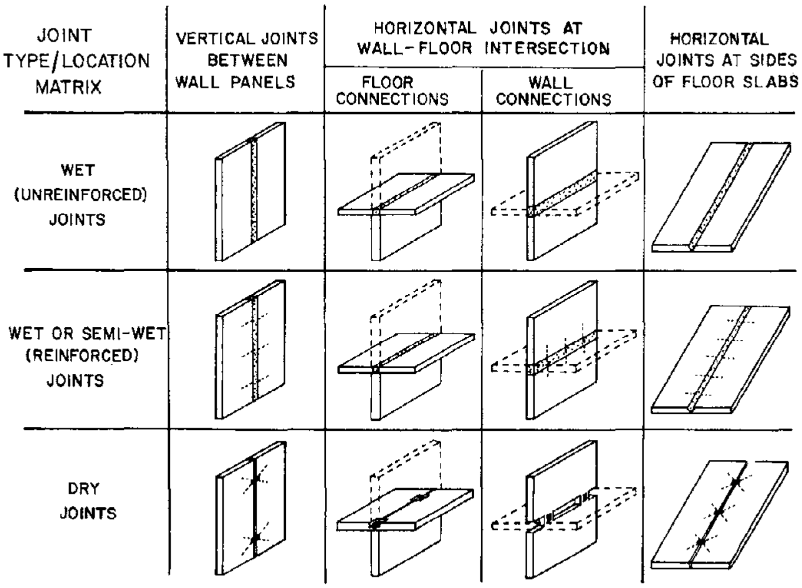

Figure 4 Connection Classification Matrix [107]

Large panel systems give the overall appearance of cast-in-place concrete shear wall systems and lateral force analysis is often carried out on the basis of this assumption [17]. While this assumption may serve a legitimate design function, it fails to recognize a fundamental difference between cast-in-place and precast construction. To quote Despeyroux:

> The connections constitute a large number of special points in a structure, and it has to be considered, on the one hand, under what conditions their strength is ensured and, on the other, what deformations are produced when this strength is utilized. Obviously, the inner restraints to which the structural members are subjected depend on the behavior of the connections, and the slight deformations which the latter may allow are liable to produce considerable redistribution of force and moments in all other parts of the structure. Thus, the behavior of the structure as a whole may turn out to be quite different from that of a monolithic system.[21]

This focus on the connection areas and their overall influence on structural behavior is the critical difference in panelized construction. The existence of these connections throughout the structure creates a system threaded by an interconnected system of discontinuities. As suggested by Despeyroux, these are discontinuities both of strength and stiffness. Seldom does the complexity of a connection allow for the development of strength comparable to that of the surrounding panel. By the very nature of the construction process, the connection introduces natural planes of low stiffness and weakness in which large deformations (e.g., slippage) may be required for the development of ultimate strength. Thus, connections are not just additional elements to be designed based upon an overall analysis procedure, rather they may in themselves provide a fundamental mechanism for altering structural behavior.

For a more in-depth review of panelized construction, several books and reports are available. For general design considerations that are non-seismic, the books published by Lewicki [41], Sebestyen [93], and Koncz [38] provide excellent material. A recent publication by the Prestressed Concrete Institute entitled Design Considerations for a Precast Prestressed Apartment Building [20], presents a compendium of papers covering the step-by-step design of a panelized structure. The only two works that directly address seismic design in panelized construction are by Polyakov [76] and Suenaga [100]. Polyakov's book is a general text, Design of Earthquake Resistant Structures, while Suenaga's report is a state-of-the-art compendium for Japan.

SEISMIC DESIGN CONSIDERATIONS

From a seismic perspective, limited experience has indicated that panelized construction will generally behave in a favorable manner in terms of serviceability considerations in moderate earthquakes. The major design issue is the behavior of such structures in earthquakes relative to ultimate limit states; that is, their inelastic behavior under severe earthquakes, where freedom from collapse and damage limitations are the principal criteria. Failure, in terms of ultimate limit states, can be defined in terms of component failure, excessive deformation or loss of overall stability.

Component failure can be defined as the inability of a given component to carry or transfer the required load. Excessive deformations would not initially appear to be a problem in such structures; however, the allowance for slippage in connections and coupling effects of lintels may potentially lead to excessive deformation. Instability could result from an overall softening of connection and loss of continuity if there is excessive degradation of connection regions during an earthquake. In light of the above considerations, the following are identified as areas of concern: 1) design force levels; 2) design of panels; 3) design of connections; 4) role of floor diaphragm; and 5) overall structural integrity.

In commenting on appropriate force levels for the design of panelized construction, reference is made to the basic design practice of using statically equivalent lateral forces and either using an allowable stress design or a load factor ultimate strength design approach. The important issue in terms of these design practices is the concept of ductility implied by current codifications. Quoting from Speyer's report [98] for the PCI on Precast

Concrete Bearing Wall Buildings:

> Large panel structures constructed in accordance with this report should be considered non-ductile, unless they are continuously reinforced for their full height (in accordance with Reference 1) or are post-tensioned to insure ductile behavior.

Because shear walls are often load bearing walls, the need for continuous tensile reinforcement is often minimum and the development of true flexural ductility is questionable. The question of ductility, however, is not so easily dismissed, for the real issue is one of energy dissipation.

Current seismic design procedures used in the United States are based on the premise that the structures are built with sufficient 'ductility' to allow the structure to survive a major earthquake. The 'ductility' or energy dissipating mechanism most widely accepted is associated with flexural inelasticity. Flexural inelasticity can be easily developed in frame type structures, and these concepts have been extended to cast-in-place shear walls [6, 62]. The development of such flexural ductility in panelized structures may not be feasible. While ductility for framed structures is generally assumed to be in excess of 4, Borges has suggested that panel structures may have comparable ductility factors of less than 2 [8]. The question, then, is how do precast concrete panel building dissipate energy when subjected to major seismic excitation. The corollary question becomes, what are appropriate design forces for such structures.

The design of the actual precast panels is potentially the least difficult aspect of panelized construction. Connections are generally the weak point in terms of overall bending, axial and shear behavior for a panelized shear wall. The design of the panels must, however, consider questions of out-of-plane stability [39], eccentricity of load, reinforcement around penetrations and interfacing with the connection zone. The need to detail the panel for connection is considered a part of the connection design, not the panel design.

The design of connection details has generally involved balancing the goals of economics and ease of construction on the one hand, with the conflicting needs of load transfer and continuity on the other. These conflicting goals make it difficult in many panelized systems to economically achieve a positive continuity in reinforcement. In non-seismic regions, the tendency has been to design vertical joinery capable of developing coupled behavior between adjacent vertical elements. This stems from a desire to minimize deformations due to wind by developing the full bending stiffness of a given wall system. In such design situations, the horizontal connections have been seen in terms of their ability to transmit vertical loads. In general, the connection would stay in compression due to the prestressing effect of gravity loads.

In seismic situations, the questions of joint behavior may be viewed from a different perspective. Some movement in vertical connections may be fundamental to the energy dissipation mechanism and may not adversely effect resistance against overturning. Secondly, tension across horizontal connections must be considered a possibility along with the concurrent redistribution of shear stresses in the horizontal plane. Along with the possibility of tensile strains comes the increased demand for compressive load transfer and a concern for the nature of the ultimate and cyclic behavior of horizontal connections in terms of gravity load transfer.

It is common in lateral load design to consider that the floor acts as a rigid shear diaphragm distributing lateral forces in a given level of a building. The relative stiffness of floor systems to shear walls in some panelized structures may call this assumption into question. If floor diaphragms behave in a relatively flexible manner, two issues are to be raised. First, how does such flexible shear diaphragm behavior effect the overall response of the structure; and second, to what forces and displacements will such a flexible floor be subjected.

The potential degradation of the connection areas during a severe earthquake may bring the overall stability of the system into question. This concern is identical to that expressed over progressive collapse due to an abnormal load. The assurance of a general structural integrity [26] is paramount in providing an aseismic design. The development of such integrity requires continuous reinforced ties (see Figure 5) throughout the structure [26, 98]. These tie requirements are not intended to supplement traditional strength requirements, rather they are intended to hold the structure together in order to provide stability in a damaged state.

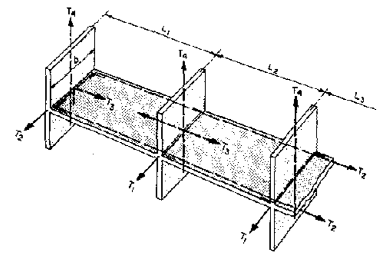

Figure 5 Tie Reinforcement Recommended by PCI [98]

BEHAVIOR OF COMPONENTS

This section of the paper discusses the component parts of panelized construction. In order to understand the behavior of the entire system, it is first necessary to understand the behavior of its component parts. The material covered will emphasize those aspects of behavior most relevant to the seismic response of the system. Three components will be covered: panels, connections and floor systems.

Panels

Under seismic conditions, a panelized wall will behave in a manner similar to an axially loaded (prestressed) cantilever beam. The panels must be capable of carrying in-plane axial, bending and shear forces. In addition, a panel may be subjected to local out-of-plane bending due to lateral acceleration or wind forces. During earthquakes, the panelized wall, and thus the panel, must resist the overturning moments and shears. The overturning moments give rise to axial forces within the panel which are additive to the compressive force associated with gravity load. Thus, the primary consideration for panel design, in terms of seismic safety, is their ultimate behavior under shear and axial loads. The effect of load reversal on this behavior must be considered along with the panel's potential capability to dissipate energy.

There is a wide range in the size of wall panels used throughout the world [22,33]. Panels are generally a single story in height, though a significant exception is the use of full building height panels (2 to 4 stories in height) for lowrise structures [83]. The width of panels ranges from 2 meters (6.6 feet) to 13.7 meters (45 feet), though the narrower panels are not common as major building elements in the United States. The multifunctional use of panelized walls gives rise to various cross sections: solid, solid ribbed, composite sandwich, non-composite sandwich and hollow core [39]. Walls may also be solid in elevation, or have significant penetrations in the form of windows or doors.

The remainder of this discussion will be addressed to panels whose cross-section is solid, though the comments hold in general for all wall panels. While this discussion is directed at panel behavior, it must be remembered that panel strength can be adversely effected by edge conditions. These edge condition effects will be discussed in the next section.

A recent report by Kripanarayanan and Fintel [39], entitled "Wall Panels: Analysis and Design Criteria," presents an excellent summary of wall panel design methods for normal loadings and recommended design considerations to include the effects of abnormal load conditions (but not earthquake loads). The emphasis of this report is on the behavior of the wall under combined axial loads and out-of-plane bending, where the out-of-plane bending is due to normal forces on the wall and eccentricities due to joint configuration and fabrication and erection procedures. Overturning moments due to lateral loads are considered to induce additional axial forces in a "critical strip." The report concludes by suggesting two wall designs: a peripherally reinforced panel when strength and serviceability requirements do not necessitate vertical steel, and a uniformly reinforced panel when vertical steel is required. Another form of reinforcing has been suggested for panels. This is a ladder form in which paired vertical bars are placed at intervals within the panel [42,76].

Under normal loading conditions, it is possible to design panelized walls in which no tension occurs due to lateral loads. This zero tension condition may even occur when code earthquake design forces are considered. However, Frank [29] has shown that tension does occur in even moderate earthquakes (10% peak acceleration).

To achieve ductile behavior in a wall in which tension exists, Appendix A of the ACI Code [2] requires that a minimum amount of steel be placed at the extreme ends of the wall. For a wall panel that is 7.3 meters (24 feet) long and 20.3 cm (8 inches) wide, this would mean approximately 45 cm^2 (7 in^2) of Grade 60 steel at each end, or 27.2 cm^2 (4.22 in^2), suggested by Allen, et al [1]. The use of this much steel would place severe economic and construction constraints on the use of panelized construction. Even if this level of reinforcement were placed in the panel, it would be difficult to make it continuous through the connection. The suggested vertical tie steel recommended by the PCI for abnormal loads [98] is only a total of 22.6 cm^2

(3.5 in^2) for both ends of the panel. It is reasonable to conclude that in terms of both ACI Code [2] and other recommended practices [1], for achieving flexural ductility, that panel structures used in the United States are generally non-ductile structures. This conclusion is specifically addressed to flexural ductility and not the structure's overall energy dissipating capabilities.

The shear capacity requirements for panels under normal loading conditions would seldom require reinforcing. However, Appendix A of the ACI Code [2] requires a minimum amount of uniformly spaced vertical and horizontal steel in a shear wall (0.25%). In panelized construction, cracking due to shear is generally not expected because of the weakness in shear of the horizontal connection. However, the need to transmit shear forces through a panel can lead to significant inelastic behavior in a penetrated panel [13,100]. This inelastic behavior may indeed provide flexural ductility by bending in posts and lintels and can be the basis of energy dissipation. This form of mechanism may be possible in solid walls through the use of such details as the slitting introduced by Muto in shear panels for framed structures [54].

Observations of earthquake damage in panelized construction have always indicated cracking in connection areas and seldom any signs of distress in the panels [75,76]. While designers may attempt to harden connections to induce failure in panels, both current evidence and practical consideration seem to indicate that panels will not be the main source of inelastic response. This can obviously be changed by the purposeful design of panels to dissipate energy through controlled shear distortions. It seems, however, that normal flexural ductility of the wall cannot be counted upon as an energy dissipating device and that other mechanisms, though potentially less efficient than flexural ductility, must be considered.

Connections

Connections are the critical component in panelized construction with respect to acceptable seismic response. Connections are normally thought of as the mechanism by which the panels are joined and the load transferred. However, they may also serve as regions of energy dissipation. Indeed, if the panelized walls are not appropriately reinforced for overall flexural ductility, or panels specifically designed for energy dissipation in shear distortion, the connection regions may provide the primary means of energy dissipation.

The most complex connection, in terms of load transmission, is that of an interior horizontal connection. Figure 6 illustrates the load transmission function of a typical horizontal connection. The main load transfer characteristics of the connection are its ability to transfer shear forces and axial panel forces. In vertical connections only shear transfer capability is normally considered. The remainder of this section is divided into three

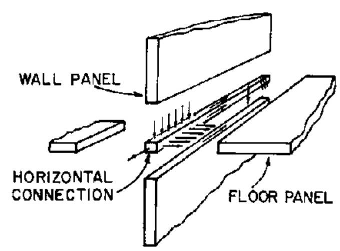

Figure 6. Force Transmission in Horizontal Connection [26]

portions. The first two discuss the transfer of axial panel forces, that is, compressive and tensile forces, transverse to the connection. The third examines the transfer of shear forces under monotonic and cyclic loading.

Transfer of Compressive Forces -- The load bearing capacity of a panelized wall is usually limited by the strength of its horizontal connections. Within the connection region there can be as many as five different materials: the wall panels, the floor planks, bearing material for the floor planks, grout and dry pack concrete. Several factors contribute to an extremely complicated load response behavior: the potential variations in material properties, the range of connection reinforcement, the complex load conditions and the sensitivity of the connection to creep and shrinkage effects. In addition, if the panels place any rotational restraint on the floor planks, negative moments will be induced in the plank under normal loadings; in turn, these moments create additional stresses in the connection region.

The behavioral complexity of the connection is evidenced by the considerable variations in joint design procedure found throughout the world [1,34, 41,45]. The main concerns, in terms of seismic response, are the failure mechanisms for these connections, the ultimate response (load-deformation behavior) and the effect of cyclic loading. Of particular concern is the possibility of brittle behavior within the connection region because of the lack of confining reinforcement. Unfortunately, almost all experimental data currently available is in terms of the connections' ultimate strength, with little, if any, data on load deformation behavior or the effects of cyclic loading.

Figure 7 shows the cross section of a typical horizontal connection in the United States. Of particular note, relative to European connections, is the hollow core prestressed planking and the use of a bearing material. The experimental work carried out on this type of connection has in general been done on a proprietary basis for industrial groups [11]. However, HUD sponsored work on these connections is now being carried on by the Portland Cement Association Laboratory [27]. Both the proprietary and PCA experimental work indicate that these platform connections can have significantly lower capacity than the gross capacity of the adjacent wall, and that the behavior can be somewhat brittle.

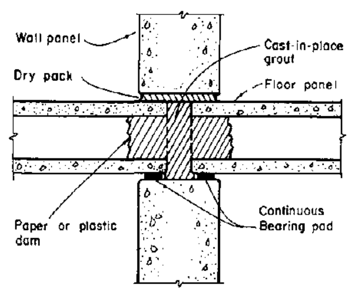

Figure 7: Typical American Platform Connection [26]

Experimental work currently available from outside of the United States [4,41,45,46] confirms the above comments about joint strength. Figure 8 shows the failure patterns observed by Lugez. The cross-sections shown in Figure 8 show the two most common horizontal connection types: platform, or closed connections, and wedge connections. In another series of experiments, Lugez and Zarzycki [46] studied a wide range of joint parameters with results, expressed as the ratio of connection strength to gross panel strength, ranging from a high of 0.88 to a low of 0.19. In a comparable finite element, analytical study, Backler et al [4] found that the joint strength ranged from a high of 0.51 to a low of 0.17 of the wall strength. Similar analytical studies have been carried out by Iqbal and Fintel [34] to examine the effect of various connection parameters. Figure 9 shows results in which the connection grout is 0.36 as stiff as the floor plank.

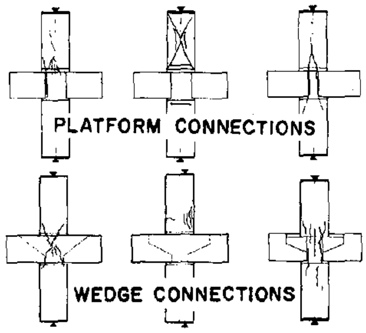

Figure 8: Typical Compression Failures in Horizontal Connections [45]

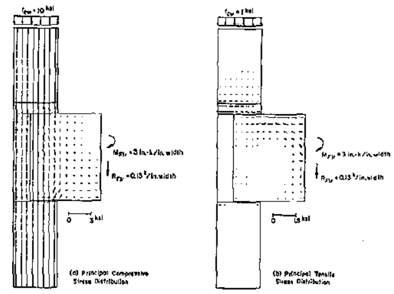

Figure 9: Principal Stresses in Horizontal Connection [34]

The experimental and analytical results cited above are not necessarily typical of the range of expected connection behavior. It must be remembered the lower bound of these studies is generally for connections purposefully designed to fail at low load levels. In addition, the strength comparisons are against the crushing strength of the wall with no consideration of stability effects. Regardless of these qualifications, the horizontal connection region must be looked upon as a potential location for failure during an earthquake, where the main failure mechanisms, tensile splitting or crushing, are generally of a brittle nature.

Several modifications of panel walls are possible to help avoid this splitting type failure. The simplest is to carefully control the material properties of the grout and dry pack concrete to reduce the possibility of high tensile stresses [34]. This approach might still leave the connection with a reasonably brittle characteristic. The use of transverse reinforcing near the edge of the panel would give the panel a post splitting strength [39,42,83] and potentially provide a more ductile failure. Polyakov [76] cites Russian practice as requiring the use of corbels at the top panels to support the floor system for long span lengths. This has also been suggested for American practice by Buettner [11].

<u>Transfer of Tensile Forces</u> -- While it is possible to have a no-tensile strain design situation in load bearing panelized walls, non-load bearing walls and the demands for ductility in load bearing walls require the development of some reinforced flexural strength. The development of continuity in the necessary tensile reinforcement presents one of the more difficult detailing

problems in panelized construction. The ACI minimum reinforcing requirements and the PCI recommended tie requirements both require the development of continuous reinforcement in excess of 0.15%.

While some systems use welding as a means of developing reinforcing continuity across the connection[76,85,100], this is generally considered uneconomic for American practice and also questionable from a quality control perspective. The potential lack of adhesion between the precast panels and the in situ connection concrete may have the effect of localizing the yielding of tensile reinforcement in the connection region. One must also keep in mind that this steel will, in all likelihood, also serve as part of a shear friction mechanism. Another potential solution is the embedment of reinforcing bars or loops into the grouted connection; however, this solution is questionable because of the potential loss of continuity with connection degradation.

The Japanese have developed an effective sleeve splice, which has undergone extensive testing[101] and has seen use in major structures in earthquake regions [55]. Suenaga has done experimental work on panel subassemblies comparing the behavior of this type of connection with that of a more traditional welded connection[100] indicating excellent behavioral characteristics for the sleeve splice. Another approach for the development of normal reinforcing patterns is to grout the reinforcing bar into the panel with adequate development length, thus each bar passes continuously through the connection region [82]. This type of connection may be subjected to a cyclic degradation of bond strength as indicated in the work of Bresler and Bertero [10].

Special welded details have been developed in which the main reinforcing in the panels is welded to connection plates and in turn these plates can be welded together in the field [85]. These welded connections can provide required vertical continuity, but to avoid brittle behavior, the failure mechanism must be yielding of steel within the panel. In addition to these welded connections, bolted connections have also been used for vertical continuity [96].

Two additional techniques have been developed for providing vertical continuity in the panelized wall. A system currently in use in Venezuela [86] places reinforcement in the vertical connections. The amount of reinforcement required for earthquake protection might effectively make this a cast-in-place column. A method in use in the United States developes vertical continuity by the use of ungrouted post-tensioning bars [28,53]. It is felt by designers using this method, that by having a low initial prestress, the bars would remain in the elastic range during seismic excitation.

As can be seen from the above discussion, there is a wide range of solutions for achieving vertical continuity. Few of these methods offer the opportunity to economically develop the required minimum reinforcement for flexural ductility. Moreover, very little experimental or theoretical data is available to justify design assumptions required by many of these methods. As was mentioned earlier, the development of flexural ductility, requiring vertical continuity of minimal reinforcement is difficult in panelized construction. However, the potentially more important issue is the maintenance of stability in an earthquake damaged structure. This definitely requires the development of vertically continuous reinforcement.

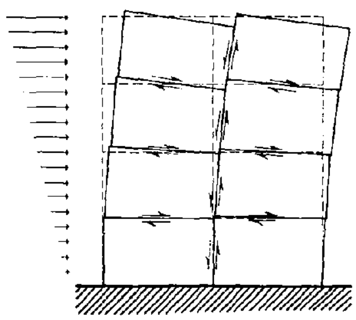

Figure 10: Shear Transfer in Panelized Wall

Transfer of Shear Forces -- In order for a panelized shear wall to effectively resist lateral forces, both vertical and horizontal joints must be capable of transferring shear (see Figure 10). When serviceability considerations dominate design, a strong emphasis has been placed on developing the shear transfer mechanism for vertical connections. This concern reflects the desire to provide a stiff structure in order to minimize lateral drift. Seismic design must consider the ultimate behavior of the structure. Under these circumstances it may be reasonable to allow some movement to occur within the vertical joint.

Movement or slippage may well have a positive impact upon overall seismic resistance by decreasing the stiffness of the structure and providing a potential mechanism for energy dissipation. The tensile and compressive forces in the wall may increase if the seismic forces are not decreased by the slippage in the vertical connection. With the potential of increased axial forces and the need to transfer shear, the horizontal connection becomes a critical design consideration for the panelized structure. As with the vertical connection, the horizontal connection can provide a source of energy dissipation. However, it is imperative that this connection maintain its overall integrity because of its role in the stability of the structure. If tension is allowed to develop across the horizontal connection, the redistribution of shear forces will place a significant demand on localized portions of the connection and adjacent panels.

There are basically two types of connections for developing shear transfer capabilities: wet connections (using reinforced or unreinforced cast-in-place concrete) or dry connections (using bolted or welded metal details). Wet connections provide for a uniform shear transfer across the length of the connection, assuming there is no tensile strain across the opening. Dry connections aggregate the forces to be transferred to a prepared detail. Assuming that the design of the bolting or welding is such as to avoid brittle failure, the behavior of dry connections will depend upon how the connection detail is anchored to the panel. This anchorage is usually provided by embedded reinforcing bars or shear studs; however, details can be directly attached to the main reinforcement of the panel [76,85,100].

Behavior of Wet Connections in Shear -- The shear behavior of wet connections is dependent upon four basic parameters:

- Concrete Strength - of both the precast panels and for the in-situ connection concrete.

- Surface Preparation of Panels - panel edges may be left plain or they can be grooved or castellated (see Figure 11).

- Connection Reinforcement - this includes type of steel and location of steel within the connection. Steel is often used longitudinally within the connection as well as across the connection.

- Force Transverse to Connection - this force can be from gravity loads or post-tensioning.

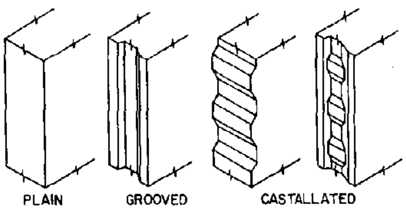

Figure 11: Typical Panel Edge Preparation

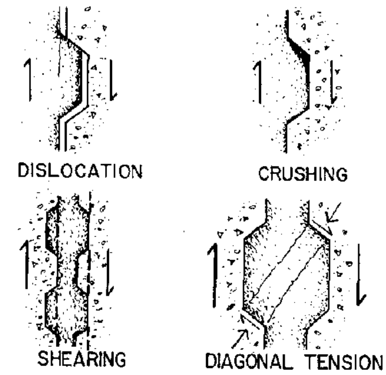

Figure 12: Failure Modes in Castellated Connections

There are two shear transfer considerations in wet connections: slippage on the panel connection interface and shear distortion and degradation of the in situ connection concrete. In the case of castellated connections, it is possible to have shear related failures (i.e., cracking or crushing of concrete) in either the panel or in situ connection concrete (see Figure 12). The transfer of shear across the interface can occur through one of three mechanisms: cohesion, friction (and dowel action), and direct bearing (in castellated connections only).

Under constantly increasing shear, two basic types of load-deformation responses have been observed: Figure 13 shows a typical load-deformation relationship for a plain or grooved connection. This type of curve represents a shear friction mechanism, with the characteristic strength often taken at 1 mm (0.4 in). Laboratory experiments are generally run on precracked or oiled connections, thus avoiding the possible influence of bonding (i.e., adhesion) between in situ connection concrete and the precast panels. The nature of the construction process and the effects of creep and shrinkage are normally considered to destroy bonding in the connections.

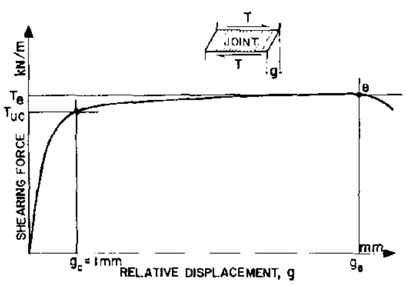

Figure 13: Behavior of Plain Connection under Constantly Increasing Load [40]

When bond is present, the load-deformation behavior of the connection may be significantly altered. Figure 14 indicates the two possible effects of bonding: weak bonding and strong bonding. The strong bonding case can have strengths as high as 3 or 4 times that associated with shear friction mechanisms [40]. If such strong bonding does occur in portions of a panelized structure, higher seismic forces and less ductile behavior may result [36]. In effect, strong bonding can lead to more monolithic behavior, which, for panelized structures, may be undesirable.

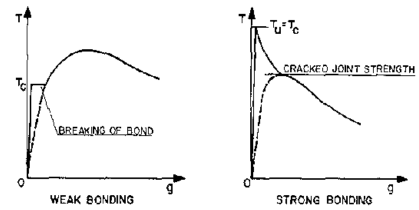

Figure 14: Effect of Bonding on Connection Behavior [40]

The ultimate shear strength of castellated or keyed connections is dependent upon shear transfer through direct bearing of the keys and in situ connection concrete, along with shear friction. Figure 15 shows a typical load-deformation relationship of a castellated or keyed connection. The significant difference between the castellated and plain connection is that the connections' ultimate strength drops off with increasing deformation. The decrease of strength is normally associated with the degradation of the direct bearing transfer (see Figure 12). The residual strength is associated with a shear friction mechanism. A significant difference between the ultimate strength of the connection and its residual strength may, as in the case of strong bonding, lead to less than desirable seismic response. The ultimate strength of keyed connections is very dependent upon the characteristics of the keys, as can be seen in Figure 16. Figure 16 is from a report by Hansen et al [32] in which theoretical strength contours are plotted as a function of the ratio, B/A, between the cross sectional area of the keys, divided by the

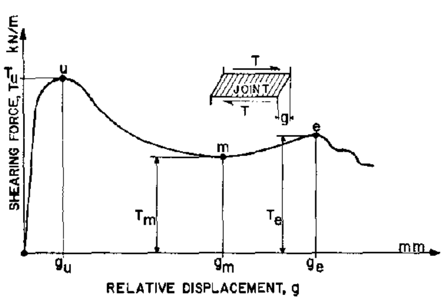

Figure 15: Behavior of Castellated Connection Under Constantly Increasing Load [40]

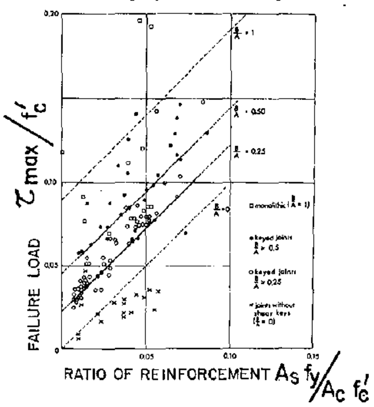

Figure 16: Influence of Keyes and Reinforcement on Ultimate Shear Strength of Connection

area of the overall connection. The residual strength of the connection, however, as seen in Figure 15, is far less dependent upon the geometry of the castellations [40].

Both the ultimate strength of plain connections and the residual strength of castellated connections are dependent upon shear friction mechanisms. The clamping forces required for this mechanism can come from either transverse steel and/or gravity loads. The majority of researchers indicate very little influence from dowel action. Figure 17 and Figure 18 present envelopes for experimental data on the shear deformation behavior of plain and castellated connections respectively. Both of these Figures include experimental results for both horizontal [5, 24, 80] and vertical connections [77, 78 79, 80]. The load axis has been normalized by dividing the shear force by the clamping force, $N + A_s f_y$, where N is the transverse load, A_s the transverse reinforcing steel, and f_y the yield strength of the reinforcing steel. Thus, the vertical load axis is effectively the coefficient of friction, μ. For the plain connections, μ ranges from 0.65 to 1.00. This upper bound is the value recommended by section 11.15 of the ACI Code [2], while the lower bound is closer to the 0.7 value recommended by several researchers [24], and the 0.6 value recommended by Mattock [52] for smooth concrete. Results for specimens with strong bonding (see Figure 17) indicate the potential drop in strength with bond failure. While all of these results are for situations with either a compressive transverse load or no transverse load, other reported experimental results seem to indicate the same basic effect for transverse tensile forces [75], until N exceeds $A_s f_y$. However, these values are based on monotonically increasing loads and do not account for the possible degradation of the friction mechanism due to seismic reversals.

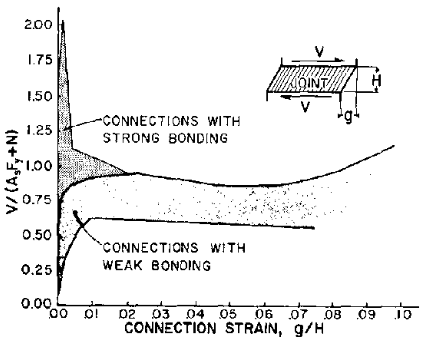

Figure 17: Bounds of Experimental Behavior for Plain Connections [5,24,77,80]

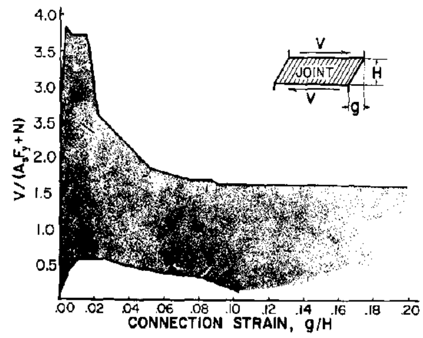

Figure 18: Bounds of Experimental Behavior for Castellated Connections [5,24,77,78,79,80]

The experimental results for castellated connections have been plotted (see Figure 18) with the same vertical axis as the plain connections. This creates a greater dispersion in ultimate loads than if the direct bearing effect of the keyes had been included; however, it does allow for the examination of the residual strength. The range of μ values associated with this residual strength is far greater than for plain connections, ranging from 0.5 to 1.5. This dispersion of results is probably due to the damaged state of the connection after the failure of the direct bearing mechanism.

The above discussion has been based upon experimental work for constantly increasing loads. For seismic design, it is necessary to consider the response of connections to load reversals. The experimental results available for such loadings are quite limited; however, it is possible to ascertain several pertinent behavioral characteristics from the data available. The decrease in ultimate load capacity for castellated connections due to cyclic loads has been examined by Pommeret [80]. In these experiments, the number of cycles to failure was determined for repetitious peak loads at a specific fraction of the connections' ultimate capacity (see Figure 19). Typical results are: 6 cycles to failure for $\pm$ 0.91T_u; 16 cycles to failure for $\pm$ 0.82T_u; and 979 cycles to failure for $\pm$ 0.73T_u [40]. The definition of failure in these tests is simply the inability to achieve the prescribed load level. It was observed that while the cyclic loads could have a significant effect on the peak load values, they did not appear to effect the residual strength of the connection. Controlled deformation tests were also carried out [40]; the magnitude of the limiting deformations ($\pm$ 0.1 mm) carried the connection to only 70% of its ultimate strength. These experiments did not test the connection to the extent needed for predicting ultimate seismic response; rather, they showed a distinct stability limit at about 70% of the ultimate capacity of the connection. Some cyclic tests for predicting earthquake response, recently reported, indicate a significant degradation of connection stiffness, in the range of 65 to 70% [88].

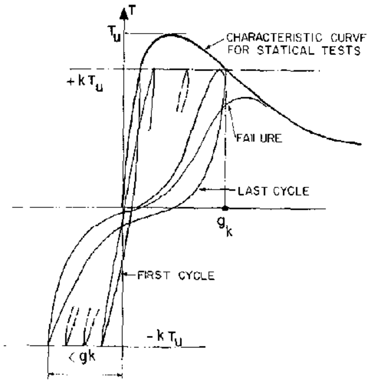

Figure 19: Schematic of Load Controlled Cyclic Tests [40]

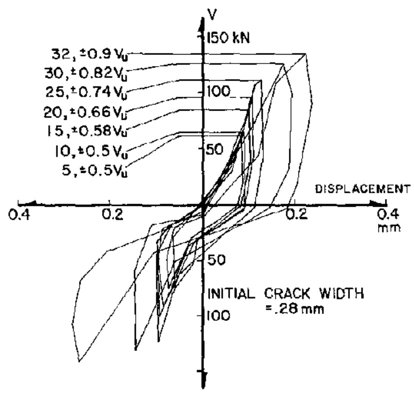

Figure 20: Shear Friction Transfer for Cyclically Reversing Loads [51]

Once shear forces exceed the ultimate strength in castellated connections, their behavior, along with that of plain connections, should be similar to that associated with shear friction. A difference in this behavior might occur in the presence of a transverse load which would not require slip for the clamping force to be generated. The work on the cyclic characteristics of shear friction mechanisms in particular [51,61] and shear failures in general [47, 81] provides a basis for predicting the hysteretic behavior of connections. Figure 20 presents selected hysteresis loops of a cyclic shear friction test reported by Mattock [51]. These tests were load controlled, with the load being increased after each 5 cycle interval. The pinched hysteresis loop, along with the increasing stiffness near the end of the cycle, seen in Figure

20, may be typical of cyclic shear behavior to be expected in the connections of panelized construction. On the basis of these findings, Mattock suggests that a dynamic shear friction coefficient that is 80% of the static shear friction coefficient be used in seismic design. If this recommendation is coupled with the recommended static friction coefficient of 0.7, mentioned earlier, a coefficient of approximately 0.56 should be used for seismic design of panelized connection. The degraded friction coefficient is still somewhat high when compared to that used by Brankov and Sachanski [9], $\mu = 0.4$, for nonlinear analysis of panelized construction.

Transverse reinforcement in vertical connections may be placed uniformly throughout the connection or concentrated in the horizontal connection. This latter approach was suggested by Backler, et al [4] as providing a ductile connection.

Experimental work on a similar type of connection, referred to as a vertical lock-joint, has been carried on by Pollner [72,73]. Figure 21 shows both the testing apparatus used and typical results for a static alternating load. The resulting hysteresis loops appear to be fuller than Mattock's shear friction results and closer to shear dominated beam behavior [47] or Pauley, et al's [61] results for construction joints in shear walls. The connection in Pollner's experiment is under some axial load, which may account for the fuller loop. The existence of this fuller hysteresis loop, also observed in the work of Suenaga [100], indicates that panelized connections may have energy dissipating characteristics superior to that associated with shear-friction achieved with only transverse reinforcement. It is the existence of this potential source of energy dissipation that has led some researchers to conclude that panelized construction may have reasonable damping characteristics when subjected to seismic excitation [5,21,100]. If connections are to be regions of energy dissipation, then care must be taken to provide connections that have reasonable hysteretic behavior. If this is to be the case, more work is required to determine appropriate connection configurations.

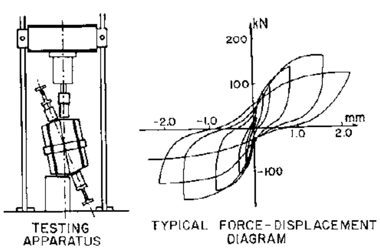

Figure 21: Cyclic Testing of Vertical Lock Joint [72]

Behavior of Dry Connections in Shear -- Dry connections are often used for the vertical connection of panels in systems found in the United States. However, these connections are by no means limited to this use. When panels are joined by dry connections, the space between the panels is often grouted. In these cases, the shear transfer mechanism has been found to be shear friction [100], rather than the explicitly developed dry connection. It is important for the designer to realize that if grout is used in the connection, the dry connection details may be subjected to forces associated with clamping rather than shearing action.

Dry connections can fail either in the connectors, that is the failure of a weld, bolt, or connection detail, or the attachment of the connectors to the panel. While it may be possible to develop details that in themselves may exhibit ductile behavior [96], the more probable inelasticity will come from the means of attachment to the panel. While large weldments may be used [85], the usual dry connection found in United States construction more frequently uses small scale welded [83] or bolted details [96]. Spencer and Neille [97] have conducted a series of cyclic load tests on a typical embedded connection. The results from two different experiments are presented in Figure 22 along with the testing apparatus. One set of results (a) is for an experiment in which loadings were displacement controlled after initial yield. The second set of results (b) is for a connection that was continually overloaded. Spencer and Neille concluded that these connections, if properly designed, would allow for significant inelastic deformation and could be used in earthquake resistant design. The actual energy dissipating characteristics look similar to that seen in wet connections. However, appropriate design must include embeddment into, if not attachment to, the main panel reinforcement, in order to avoid a complete loss of strength.

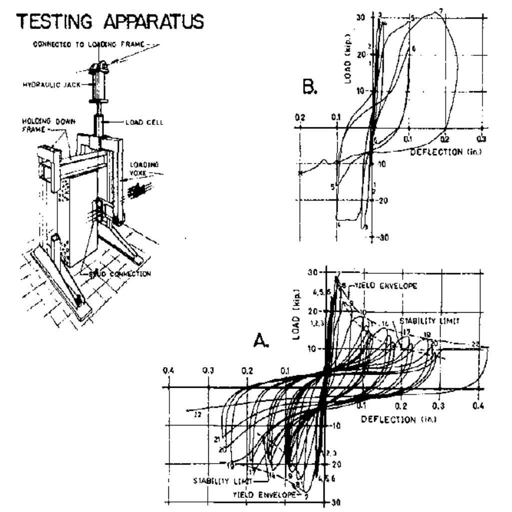

Figure 22: Cyclic Testing of Dry Connections [97]

Related experimental work has been carried on at the University of Michigan [35]. This research examined the seismic upgrading of frame structures by the use of cast-in-place or precast infill panels. One option was the use of small infill panels welded together with embedded connections. It was concluded that this small panel option had significant ductile behavior. While all of this ductility was not concentrated in the connections, the connections did contribute to the overall ductility of the response.

The limited data available on dry connections suggests that they can be used effectively in seismic design. However, the actual data base is far too small to reach any substantial conclusions. It seems apparent that for dry connections to be effective, inelastic behavior in the panel, at least in a local region, must be expected and accounted for in design.

Floor Systems

The floor systems in panelized construction, as with any building system, act as shear diaphragms during an earthquake. This action as a shear diaphragm or shear beam, requires that the floor system be tied into the vertical elements of the structure and that the precast elements of the floor system be tied together in order to act as a shear beam. To achieve shear beam action from the precast floor elements, the general design approach is to develop shear transfer capabilities between the floor panels and use perimeter reinforcement to hold the system together and provide tensile reinforcement for flexure. Work reported by both Polyakov [76] and Lewicki [42] indicate a sigicant decrease in shear diaphragm rigidity in panelized systems when compared to cast-in-place systems. Normal analysis and design procedures assume floor diaphragms to be rigid relative to lateral load resisting systems. This assumption is reasonable for framed structures when contrasting the stiff monolithic floor with the flexible frame. In panel structures, however, this assumption becomes questionable when contrasting the more flexible precast floor with the stiff walls. This decrease in relative rigidity may well lead to an active participation by the floor in the dynamic response of the system.

Floor systems often found in the United States are constructed out of hollow core precast planks or precast tee or double tee sections. Design practice has often been to cast a topping layer on these systems reinforced by welded wire fabric [83]. The use of such a cast-in-place layer greatly stiffens and strengthens the floor. However, precast planks and tees are also used without toppings, which then requires the development of shear transfer capabilities between floor elements. Figure 23 illustrates the response normally expected of precast floor systems. In both cases shown in this figure, it is necessary to place tie steel across the ends of the floor elements [83]; in Figure 23a the steel acts to clamp the elements together and in Figure 23b the steel acts as a tensile element in flexure.

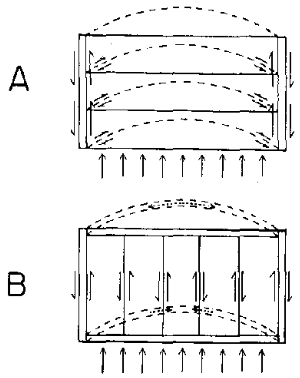

Figure 23: Potential Diaphragm Action

Shear transfer in untopped floor systems is usually achieved by grouting floor planks together (see Figure 24) or, in the case of tees, through the welding or bolting of embedded dry connections. Since these dry connections are similar to the ones discussed in the previous section, the remainder of this disucssion is directed toward grouted precast floor planks.

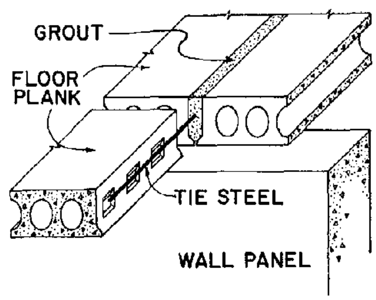

Figure 24: Grouting of Floor Planks

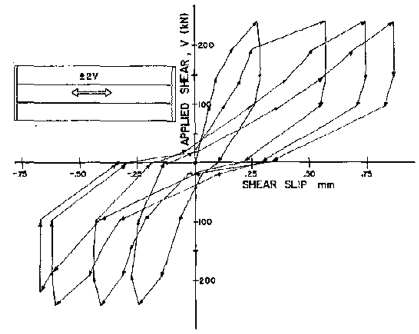

Figure 25: Cyclic Testing of Hollow Core Prestressed Planks [18]

Like the data available for the compressive strength of horizontal connections using pretensioned hollow core planks, research on their shear transfer characteristics has been conducted on a proprietary basis. These tests have indicated a properly designed floor system can develop in-plane shear friction coefficients in excess of 1.0 when shear keyes or indentations are placed on the plank's edge. The only cyclic test data available on these floor systems has been carried out by Concrete Technology Associates [18] (see Figure 25). The ultimate shear capacity between planks was predicted to be approximately 356 kN (80 kips) using a shear friction coefficient of 1.0. The observed behavior is that to be expected of a shear transfer through friction, that is the deformational ductility is excellent, but the energy dissipation capacity is limited.

The connection of the floor to the vertical elements of the system is generally achieved through the embeddment of reinforcement from the floor system into the load bearing horizontal connection or the use of bolted or welded connection details to non-load bearing walls. The development of a full tie beam is of great importance in achieving favorable behavior of the floor system. This requirement is identical to the tie requirements for general structural integrity suggested by Fintel [26] and Speyer [98].

SEISMIC RESPONSE OF PANELIZED STRUCTURES

The ability to design aseismic panelized structures is dependent upon the designer's thorough grasp of their potential seismic response. At their most basic level, panelized buildings are a collection of vertical cantilever beams. For low amplitude motion, panelized structures will behave in a manner similar to any bearing wall structure of like geometry. However, as soon as the response enters the nonlinear, inelastic range, panelized structures will begin to behave in a distinctive fashion. This uniqueness comes from the influence of the connections, in terms of both their stiffness and strength. In addition to the effect of the connections, the seismic response of panelized structures is also strongly influenced by soil-structure interaction and various types of coupling between wall elements.

Because of the limited experience with panelized construction in major earthquakes, the data available on their dynamic characteristics is based on both field work, that is, low amplitude shaking studies, and experimental work. The experimental work covers a wide range from subassembly static equivalent tests [37,86], to shaking tests [8,59,71,76], to ambient (wind microtremors, quick release) vibration tests [59,76,99] and explosively simulated earthquakes [76]. Table 1 attempts to compile a cross-section of this data in terms of primary periods and estimated damping coefficients. The

Reference	Number of Stories	Height (m)	Test Method	TRANSVERSE DIRECTION				LONGITUDINAL DIRECTION		
				Width (m)	Av. Bay Spacing (m)	Period (sec)	Damping (% Critical)	Length (m)	Period (sec)	Damping (% Critical)
Staably [99]	16	44.5	Wind	12.0	3.6	0.63	-	18.6	-	-
Petrovski, et al [71]	9	27.6	Forced	12.5	3.5	0.38	1.9	45.3	0.33	1.7
Borges, et al [8,86]	15	39.3	Forced	11.0	7.8	0.53	2-5	37.0	0.32	2-5
Osawa, et al [59]	7	19.1	Forced	13.2	8.0	0.19	-	55.8	0.24	-
Polyakov, et al [75,76]	9	27.0	Micro-Tremors	12.0	-	0.4	-	86.2	0.32	-
	12	39.0	Micro-Tremors	12.0	-	0.52	-	67.2	0.36	-

Table 1: Field Data on Dynamic Characteristics of Panelized Construction

results presented in Table 1 are for low amplitude motion and indicate very stiff buildings with reasonably low damping coefficients.

The structures from which the information in Table 1 is derived are generally not similar to American panelized structures. American systems use fewer walls and would thus tend to behave in a somewhat more flexible fashion. In addition, the damping coefficients have been estimated for low amplitude vibration and may well be higher for larger amplitude vibrations. Suenaga[101] has estimated damping coefficients of from 10 to 15% for large amplitude motion. Suenaga's tests, as well as Borges' subassembly work [8,35, 100], were both for penetrated panels. The energy dissipating capacity of such panels may well be in the flexural inelasticity of the lintels and piers. If this is the case, the damping characteristics of a panelized structure may be directionally dependent, that is, dependent upon whether the panels are solid or penetrated.

Panelized structures are in general quite stiff, and thus their seismic response may be very dependent upon the effects of soil-structure interaction. The shaking tests by both Petrovski [71] and Borges [8] have indicated significant modification of the primary mode of vibration. Figure 26 is typical of such results and clearly shows both a rocking and swaying motion in the primary response. Russian work has also indicated that for shallow foundations, the primary response may well be a simple rocking motion [95]. Assuming a stable response, this modification due to soil-structure interaction may have a beneficial effect, by both increasing the primary period and adding significant damping.

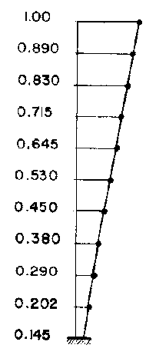

Figure 26: Primary Longitudinal Mode Shape for Forced Vibration Test, T=0.33 Sec. [71]

Since actual earthquake experience and experimental information is limited, it is necessary for both the researcher and designer to turn to analytical procedure to develop a generalized sense of seismic behavior. The next portion of this section will briefly discuss the techniques available for analyzing panelized structures. The three portions after that will discuss specific behavioral and modeling questions: coupling effects, connection modeling and nonlinear panel behavior. Finally, the results of several analytical studies pertaining to American panelized systems will be presented.

Methods of Analysis

While there are several similarities between analytical modeling of shear wall systems and panelized systems, very little has been done that directly addresses the dynamic modeling of panelized systems, particularly in terms of nonlinear, inelastic seismic response. Accepting the analogy between shear wall type structures and panelized systems, there are four basic modeling techniques available: beam models, continuous medium methods, frame analogy methods and finite element methods. These techniques represent varying degrees of refinement. The beam and continuous medium models are the simplest, and are commonly used for design. They are, however, only directly applicable to a small range of problems. The frame analogy and finite element methods, on the other hand, are applicable to a wider range of problems.

Several summary articles are available that describe these analysis procedures [26,29,48,66]. The following is a brief summary of the four basic models.

Beam Models -- Beam models provide the simplest means of analyzing panelized systems in which lateral rigidity is fairly uniform [60,66]. Assuming the beam to be cantilevered from its foundation and mass uniformly spread over its height, simple solutions can be obtained for basic dynamic behavior. Polyakov has modified this model in developing an approach for considering the effect of floor diaphragm flexibility [76]. This basic beam model can also be modified to carry out simple nonlinear inelastic analysis [8].

It is also possible to use basic beam elements of one story high in discretizing panel systems for computer coding. In this case, reasonably accurate primary mode responses can be obtained for structures greater than five stories when considering only flexural deformations [29]. At the five story height, an error of approximately 15% can be expected in the primary period if shear deformations are not included. In taller structures it may be necessary to include a rotational degree of freedom to accurately model higher mode responses.

Continuous Medium Methods -- The continuous medium method is normally employed as an analytical tool in the design of shear walls where irregularities make it impossible to use the simpler beam methods. In this method door and window lintels, floor slabs, and vertical wall joints are assumed to be replaced by continuous bands of beam laminae or shear media [15,87]. These bands connect the continuous vertical elements to allow the system to act in a coupled fashion. Under loading, the walls are deformed and shear forces are induced in the beam laminae or shear media. Generally, it is assumed that properties remain constant over the height of the building, but variations can be handled.

This method has found wide use in analyzing panelized structures [41, 70,94]. These uses have been directly analogous to those for shear wall structures of like geometry. In addition to simple variations in elastic shear stiffness, continuous medium models can also be used when the lintels are considered to be elasto-plastic [94]. Petersson [70] has made use of a modified continuous media method, a stringer method, in the analysis of panelized structures for progressive collapse. Dynamic analyses, although not commonly done, can be performed using continuous media methods by discretizing the system in a manner similar to that in the beam method.

Frame Analogy Method -- In the frame analogy method, the structure being analyzed is broken down into an equivalent frame system [3,41,48,49]. This system allows for greater flexibility in modeling diverse system geometries, aspects of nonlinear behavior and certain connection details. While there are several variations of this method, the most commonly used is built around a line element with rigid ends and a flexible midsection which can be deformed in response to axial, bending and shear forces. The frame presents an excellent modeling tool for systems with penetrations which create a system of coupled vertical elements.

Extensive modeling of panelized structures has been done using the frame analogy [3,41,76]. The method lends itself to efficient large scale programs for three dimensional analysis of shear wall type structures, for example, the Shear Wall portion of GENESYS [30]. Of particular interest are the efforts to include the effect of connection regions in this procedure. MacLeod has included the effects of vertical connections using spring elements [50] and Polyakov has suggested approaches for horizontal connections [76].

While the frame analogy lends itself to elastic analysis and can be easily adapted for dynamic analysis, its use in the inelastic range is limited. Aoyama [3] has successfully used the frame analogy in analyzing panelized systems in the inelastic range when large penetrations allowed flexural failure in the mulluions and assured a minimal distress in the connection region.

Finite Element Method -- The finite element method is the most versatile of the modeling techniques used in analyzing the behavior of structural systems. The method is basically a discretization process in which the behavior of the actual structure is expressed by the response of a specified series of nodal coordinates. This method has been extensively used in modeling the behavior of shear walls [69,106,108] and panelized systems [9,23 29,44]. Specialized finite element programs have been developed for analyzing panelized structures by Petersson [67] and Schwing and Mehlhorn [92].

In the linear elastic analysis of large structural systems, the size of the problem can be reduced by employing substructuring [29,44,68]. In this approach, the structure is divided into a series of smaller substructures in which internal degrees of freedom may be eliminated through static condensation. In the case of panelized structures, the partitioning of the structure will generally correspond to the panels. The static condensation procedure thus allows the creation of 'super' elements that correspond to actual panels and can be assembled in a manner directly analogous to the actual structure. This procedure has been used by Petersson [65,67], Frank [29] and Llorente [44] in modeling the dynamic response of panelized structures. Petersson's work was specifically directed toward progressive collapse, while both Frank and Llorente were concerned with seismic response. Another approach for developing full panel finite elements has been the experimental determination of the appropriate stiffness terms [23].

The successful use of 'super' elements, in terms of their numeric convenience, depends upon the panel responding in the elastic range. All of the researchers above have made this assumption in order to concentrate their modeling effort in the inelastic response of the connection regions. The problem of connection modeling will be handled separately later in this section.

Coupling Behavior of Panelized Shear Walls

The response of panelized systems can be extremely sensitive to interaction between various wall elements. This interaction is often called coupling, though, in the context of panelized construction, the meaning is quite broad. There are basically four types of coupling that must be considered: in-plane coupling through window and door lintels, in-plane coupling through vertical connections, coupling to a transverse wall through a vertical connection, and out-of-plane coupling of parallel walls through floor diaphragms.

In-Plane Coupling, Lintels -- In-plane coupling can have a significant impact on the response of coplanar shear walls [12,62,74]. The effect of such coupling is twofold: the modification of wall stiffness and the potential energy dissipation in the inelastic response of the coupling mechanism. Figure 27 indicates the effect of floor slab coupling on the primary transverse period for typical cross wall structures of 5, 10 and 15 stories (the story height is approximately 2.7 m or 9 ft). The curves in these figures are normalized to the period of an uncoupled wall, T*, and a stiffness based on full (mid-bay to mid-bay) participation of an uncracked, homogeneous floor slab, K*. Suggested coupling stiffnesses by Schwaighofer and Collins [90] and Tso and Mahmoud [103] are also indicated on the figure. This figure indicates the extreme sensitivity of coplanar shear walls to any form of coupling.

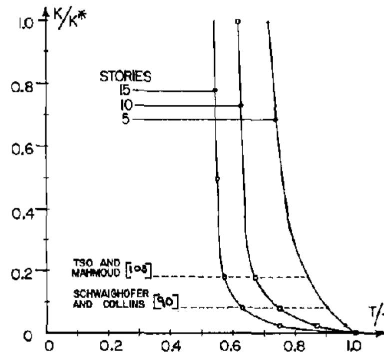

Figure 27: Effect of Floor Slab Coupling on Primary Period of Coplanar Walls

In panelized construction, these coupling lintels can be created within the precast concrete panels during the fabrication process, or by the precast floor systems connecting the panels. In the case of the precast lintels, it is possible to put in adequate reinforcement to create a truly ductile coupling member. When the coupling is due to precast floor elements, particularly hollow core, prestressed planks, there probably will be a minimal coupling stiffness; however, even this minimal stiffness may have a significant impact in taller structures.

The effect of the inelastic response of the floor coupling mechanism has been examined by several researchers [12,16,74]. Figure 28 shows the results of a study by Burnett and Rajendra [12] which are reasonably typical of such studies. Pauley [60,62] has suggested that the mechanism for in-plane coupling, such as a lintle, can be developed as an energy dissipator. While it is highly questionable if precast floor elements can serve this purpose, it may be possible to use the lintels within penetrated panels as energy dissipating mechanisms.

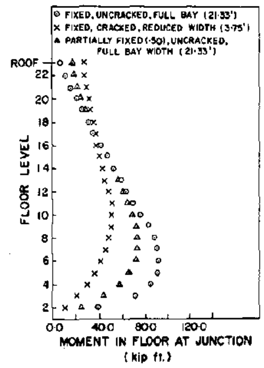

Figure 28: Equivalent Bending Moment in Floor Slab at Wall-Floor Connection [12]

In-plane Coupling, Vertical Connections -- In systems in which panels are reasonably narrow (that is, a shear wall is more than one panel wide), the response of the system is dependent upon the effectiveness of the vertical connections. This coupling is achieved through either wet or dry connections, or in one reported case, horizontal post-tensioning [99]. In nonseismic regions, the effectiveness of the vertical joint has been of prime importance because of the emphasis on response to wind loads, that is, the development of stiff structures.

Many studies have been carried out to assess the effect of vertical connections [7,64,94]. Studies have been done considering the vertical connection as an elasto-plastic medium [12,94] and as a nonlinear, inelastic medium [74]. In general, these studies produce results similar to those on coupling through a lintel. The vertical connection, like the lintel, may present opportunities for energy dissipation. Pollner [72,73] has attempted to develop such an approach with his work with lock-joints. The large panel sizes used in American systems often do not require coplanar vertical connections and thus lose this opportunity to develop an energy dissipating mechanism.

Coupling of Perpendicular Walls -- The coupling, or connection, of perpendicular walls is often an important means of creating additional stiffness and strength in panelized construction. This coupling can be used to create T or I sections, or also to develop box type sections around elevator cores and stairwells. In many systems used in the United States, this type of detail connecting perpendicular walls is often created by the use of welded or bolted connections. The modeling of such connections requires a point transfer of forces. As was mentioned earlier, MacLeod [50] has suggested the use of a spring or point stiffness coupling element in such cases. Work is currently underway at MIT to model this form of perpendicular coupling and to assess the effect of a degrading stiffness in the mechanical connector on the overall seismic response of the system.

Out-of-Plane Coupling, Floor Diaphragms -- Both shaking tests of actual structures [71,75] and theoretical studies [76] have indicated that the relative flexibility of precast floor systems to panelized shear walls can have a significant effect on the dynamic characteristics of a panelized structure. This out-of-plane coupling by the floor diaphragm can effect both the overall response of the structure and the distribution of seismic forces within the structure. Polyakov [75] has recommended that this coupling effect always be considered in the design of taller panelized structures.

Figure 29 shows the first four modes of vibration of a combination shear wall and frame structure presented in Polyakov's book [76]. Of particular interest are the first and fourth modes, which indicate a force distribution

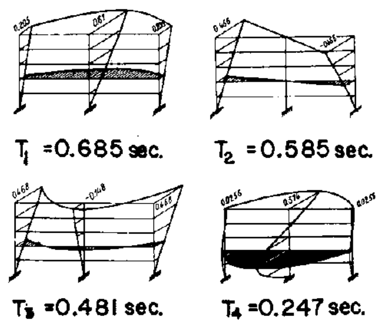

Figure 29: Transverse Modes of Vibration [76]

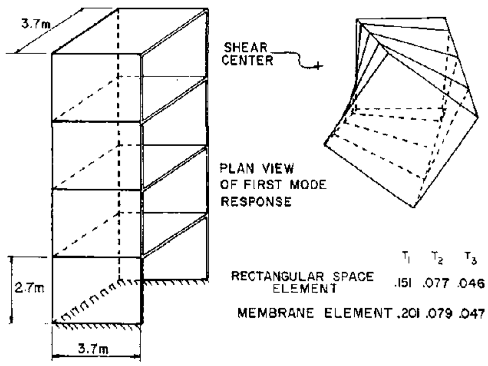

Figure 30: Three Dimensional Response of Channel

significantly different than that obtained with the normal assumption of a rigid floor diaphragm. The anti-symmetric second and third modes would not, in theory, be excited during an earthquake. However, any nonsymmetric property of the structure, such as foundation conditions, might initiate their participation in the seismic response.

The modeling of such three dimensional effects using finite element procedures has been examined by Zienkiewicz and Paukeret[108]. Their findings indicate that shear wall type structures can be successfully modeled by finite elements, considering membrane stresses and ignoring out-of-plane bending. This work was carried out for static load cases, with a uniform horizontal load. Petersson's program [67] is capable of such three dimensional modeling, as illustrated by his consideration of the torsional restraint as a mechanism for avoiding progressive collapse [66]. Recent analytical studies carried out at MIT indicate that neglecting out-of-plane bending may, when significant torsional effects are present, lead to significant error. A channel section typical of stairwell bracing showed a 33% increase in its primary period when out-of-plane stiffness was not considered (see Figure 30). These preliminary results seem to indicate that the appropriateness of using only membrane stress elements is subject to some qualification.

Modeling of Connection Behavior

Nonlinear modeling of the connection areas under biaxial stress states present special problems. Two basic approaches have been developed to handle these. In the first approach, the connections are modeled as a series of orthogonal springs. This procedure was originally used by Ngo and Scordelis [58], who used these elements to transfer forces between concrete and reinforcement in analyzing reinforced concrete beams. The model has since been used by Franklin [25] to transfer load between reinforced concrete frames and shear panels, by Yuzugulla and Schnobrich[106] in analyzing the behavior of shear wall frame systems, and by Schwing and Mehlhorn [91,92] in calculating the stiffness of the linkage elements based upon load deformation curves obtained from experimental tests. The stiffness of the shear spring is assumed to depend upon the load simultaneously present in the normal spring: compressive stresses cause an increase in the shear capacity, while no shear transfer takes place under tensile stresses.

In the second approach, the connections are modeled as anisotropic finite elements. This is the approach used by Kärrholm and Petersson [58, 63,66] in their work on the progressive collapse of panelized buildings. They have defined stress-strain relationships for the various types of connections (horizontal, vertical, and intersection) which may exist in panelized systems. The response of the structure to the load, which may be applied suddenly or in increments, is calculated for small increments of time according to these stress-strain relationships. Shear slip has also been incorporated in these models, and has been assumed to occur when the shear stress reaches 70% of capacity.

The nonlinear analysis of panelized structures presents an extremely complex problem. Yet the above analyses were only for constantly increasing loads. The complexity of the problem increases dramatically for reversal type loadings (e.g., seismic response). Nonlinear behavior of the connection under cyclic loadings may take several forms. First, there is the nonlinear behavior (cracking and softening) of the in situ connection concrete. Secondly, there is the possibility of the connection reinforcement yielding, and thirdly, slippage may occur along the connection-to-panel interface.

Of these, slippage and degradation of the connection appear to be the largest contributors to the nonlinear behavior of the connections. A simple method of modeling these nonlinearities is the definition of a pseudo-shearing modulus. This was the approach followed by Frank in his studies on the dynamic response of panelized systems [29]. Like Petersson, Frank also used anisotropic finite elements to model the connections. This allowed the bending and shearing modulii to be varied independently so a wide range of potential behavior could be analyzed.

One of the potential problems that would be unique to panelized construction is a type of rocking motion that can occur in the plane of the horizontal connection and in the foundation [44,56,95]. While this may not be a pure rocking motion, the inclusion of minimal reinforcing requirements has a small effect at best. The recent work of Llorente [44] has attempted to study this phenomena in the dynamic range. To carry out these studies, an anisotropic model was used with a constituative relationship that allows the transverse stiffness to go to zero under tensile strains. The analysis was also extended to examine the behavior under a minimal amount of tensile stiffness or the inclusion of ungrouted post-tensioning bars over the height of the structure. Results of this study will be included later in the paper.

In order to appropriately model the effects of the connection region in terms of seismic response, it is necessary to develop models that will allow for cyclic loading and stiffness degradation. The recent work of Brankov and Sachanski [9] has suggested just such a model. This model includes a degrading stiffness based on the cumulative displacement occuring during the seismic response; however, it does not include a degrading strength.

The model starts with a degraded initial state, that is, a shear friction coefficient of 40%. Results of this study are presented in Figure 31. The solid line indicates the shears and deformations associated when no movement is allowed in the connections. The dashed line is for an analysis achieved allowing for deformations in the connection areas and the dotted line is for deformations allowed in the connection area without any friction considerations in the connection areas.

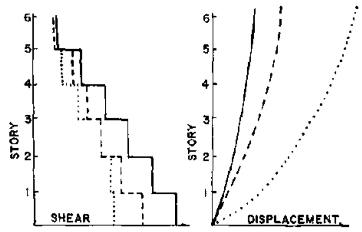

Figure 31: Seismic Response Considering Connection Behavior [9]

Modeling of Nonlinear Behavior in Panels

All of the above analyses assumed that the panels remained elastic. Nonlinear panel behavior, however, could play an important role in the seismic behavior of panelized systems, especially when dry connections are used. Most of the research in the area of nonlinear panel behavior has centered on modeling the response of single panel units. This research has been basically concerned with their use as infill shear walls.

Of the models available, the models developed by Cervenka and Gerstle [14] and Darwin and Pecknold [19] appear to be the most advanced. In these models the basic nonlinearities considered are the cracking of the concrete and the yielding of the reinforcement. The basic difference in the two approaches is the assumed stress-strain relationship of the concrete and the number of open cracks allowed. Cervenka and Gerstle, in their model, assume an elasto-plastic stress-strain relationship and allow only one open crack to form. Darwin and Pecknold, on the other hand, assume a stress-strain relationship with a descending second branch and allow for multiple open cracks. Although both models performed equally well under constantly increasing loadings, Darwin and Pecknold's model performed better under cyclic loadings.

These models serve well to predict the nonlinear cyclic behavior of single reinforced concrete panels. However, the incorporation of such highly complex models as these into a large-scale structural analysis would be prohibitive in terms of the required computation time and may be of questionable accuracy. On the other hand, they can be very helpful in gathering information on the ductility and energy dissipation characteristics of panel elements, and in developing simplified models for the nonlinear analysis of panelized systems.

Seismic Response of American Cross-Wall System

The majority of information available on the seismic response of panelized buildings has been compiled for systems that are not common in the United States. This section thus reports on analytical studies being carried out at the Massachusetts Institute of Technology [29,44] on the seismic response of typical American cross-wall structures. A plan view of the cross-wall building is shown in Figure 32. An interior load bearing wall was isolated (see Figure 33) as the basis for these studies. The cross-wall was considered uncoupled from the adjacent coplanar wall because of the lack of stiffness in the corridor lintel. In addition, the floors were assumed to be rigid and the foundation was also assumed to be rigid. While it is realized that these assumptions are contrary to many of the points discussed earlier, it was felt that this simplification was necessary to isolate the problem in the preliminary phase of these studies.

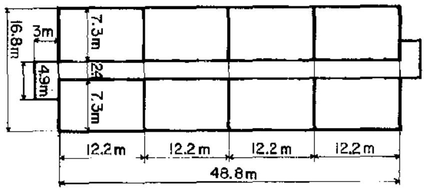

Figure 32: Plan View of American Cross-Wall Building

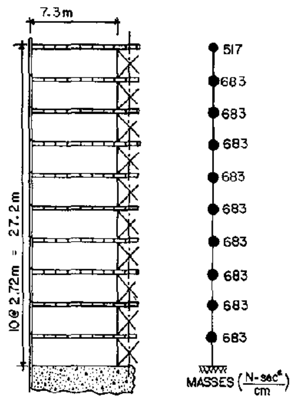

Figure 33: Isolated Interior Cross-Wall

The initial portion of this study was carried out by Frank [29]. In studying the structure, it was assumed that the horizontal connection region was a plane of weakness in terms of both strength and stiffness. In addition, the assumption was made that the cross-wall would be uncoupled from the transverse facade wall. This was a lower bound assumption based only in part on the relatively weak bolted connection.

The panels were modeled as a 'super' finite element. A pseudo-shear stiffness, accounting for both shear deformation and slippage, was used in the anisotropic connection element. A parametric study was carried out for structures of various heights, with various width cross-walls. In addition, the connection stiffness was varied relative to the panel stiffness.

Initially, basic dynamic characteristics were determined for this isolated cross-wall. It was found that decreasing both the axial and shear stiffness of the connection area had little effect on the primary period, but did influence the higher modes (this can be seen in Figure 34). The connection relation in Figure 34 refers to connection stiffness parameters relative to panel stiffness parameters (E_c/E_p:G_c/G_p). In obtaining the mode shapes, the symmetrical vertical modes kept appearing with periods close to the initial lateral modes. These significant vertical modes, along with a connection dependency on vertical loads, seems to support a concern over vertical accelerations expressed by other researchers [9].

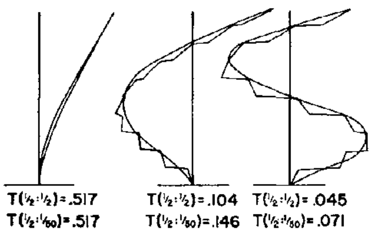

Figure 34: Influence of Connection Stiffness on Mode Shapes and Periods [29]

Number of Stories	Base Shear/Weight of Structure		
	Percent Damping		
	2%	5%	10%
5	0.49	0.39	0.35
10	0.40	0.32	0.18
15	0.26	0.21	0.15

Table 2: Variation of Base Shear with Height and Damping [29] (peak accel., 0.3g; conn., 1/2:1/2; panel, 7.3mx2.4m)

On the basis of the dynamic characteristics determined in the initial part of the study, a series of modal analyses were performed using a mean response spectra developed by Vanmarke and Biggs [104]. Table 2 presents a portion of these results in terms of equivalent base shear acceleration coefficients, where the response spectra is scaled to a peak acceleration of 30% of gravity. In order to obtain a sense of potential damping characteristics for this structure, the connection area was softened as the damping was increased. Table 3 presents typical results for a 10 story structure. It can be seen that the increased damping has a significant effect relative to the connection softening. The mean response spectra was scaled to 0.05g peak acceleration for comparison to several forms of code calculations. In general, it was found the codes using static equivalent lateral force methodologies tended to underestimate both base shear and overturning moments.

In general, it was concluded that as long as the structure remained linear elastic, its seismic response would be similar to that of an equivalent shear wall structure, with the possible exception of horizontal displacement. However, Frank noted the possibility of the horizontal connections being potential zones of energy dissipation. In checking the analysis results against the overturning strength, it was noted that the probable failure mode would be due to overturning. Tensile strains were expected to occur in the wall at relatively low peak accelerations (0.05 to 0.10g). This has also been suggested as a failure mode typical in panelized construction by Nassonova and Fraint [56]. It must be assumed that horizontal connections in panelized construction have no inherent tensile strength, thus any tensile strains in the wall will be aggregated in the formation of a crack in the connection.

Connection	Force at Base	Percent Damping		
		2%	5%	7%
1/2:1/2	Shear Moment	2616 (588) 51692 (38111)	2092 (470) 41224 (30398)	
1/2:1/10	Shear Moment	2577 (579) 50588 (37297)	2065 (464) 40406 (29790)	
1/2:1/25	Shear Moment	2577 (579) 49639 (36597)	2056 (462) 3953 (29142)	1860 (418) 35723 (26337)
1/2:1/50	Shear Moment		2051 (461) 34224 (25232)	1860 (418) 34643 (25541)
1/4:1/50	Shear Moment			1789 (402) 32998 (24328)

Table 3: Probable Range of Response - 10 Story Cross-Wall [29]
Units: Shear, kN (kips); Moment, kN-m (kips-ft)
(peak accel., 0.3g; panel, 7.3mx2.4m)

Llorente [44] has developed a program that models the nonlinear response created by this cracking phenomena. This modeling is done by allowing for different stiffnesses in tension and compression during the integration of the connection element stiffness matrix. The nonlinear equilibrium equations are integrated using the central difference method. Using this approach, a series of parametric studies has been completed on the same isolated shear wall as Frank. The basis for these studies was an artificial ground motion generated from the Newmark--Blume--Kapur response spectra [57] for 2% damping. A fifteen second record was used (with a 2 second rise time, 10 seconds of high amplitude vibration and 3 second decline time). Longer records are required, since the peak response of the structure often occurs after early acceleration peaks have initiated a rocking motion.

The seismic response of three different walls were studied. The first wall assumed the material to have the same elastic properties in tension and compression (tension material); a second wall was assumed to have ACI minimum reinforcement (0.25%) and therefore a tensile stiffness that was 4% of the compressive stiffness (reduced tension material); and a third wall was assumed to have no tensile strength, but was prestressed by ungrouted tendons up to an additional concrete stress of 200 psi (P/T material). The post-tensioning tendons were assumed anchored at the foundation and roof level only and were assumed, along with the regular reinforcing in the reduced tension wall, to be infinitely elastic. Results typical of this study, for a 10 story structure with 5% of critical damping, subjected to a peak acceleration of 0.15g, are given in Figure 35. Of particular interest is the length of crack opening and the high shear stresses around the tip of the crack. These peak shear stresses indicate the need to consider an inelastic material model to account for the obvious need for shear redistribution through slippage.

Table 4 presents results for a series of such runs. It is not uncommon for the peak crack width to approach 50% of the cross-sectional width in both the partial tension and post-tension cases.

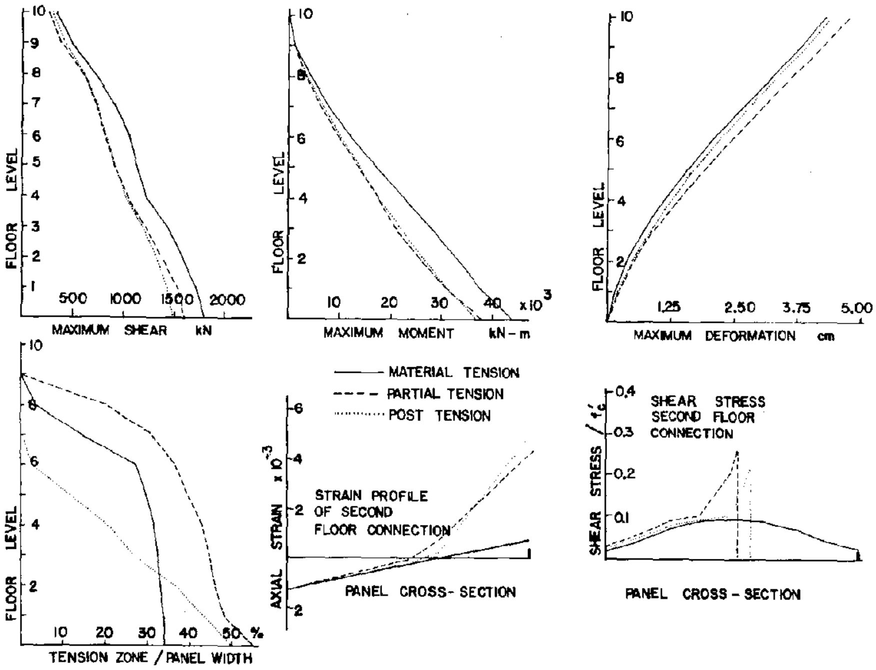

Figure 35: Seismic Response of an Isolated 10 Story Cross-Wall [44]
(peak accel., 0.15g; 5% critical damping; panel, 7.3m x 2.4m)

NUMBER OF STORIES	MATERIAL TYPE	MAXIMUM BASE SHEAR kN			MAXIMUM BASE MOMENT kN-m			MAXIMUM ROOF DEFLECTION (cm)			MAXIMUM TENSION ZONE / WIDTH (%)		
		0.10G	0.15G	0.20G	0.10G	0.15G	0.20G	0.10G	0.15G	0.20G	0.10G	0.15G	0.20G
TEN STORIES	TENSION	1177	1766	2354	29008	43517	58016	2.82	4.23	5.63	24.3	34.0	38.0
	NO TENSION W/PT BARS	1118	1462	1707	28714	36601	43762	2.97	4.34	6.83	29.0	49.7	64.2
	PARTIAL TENSION	1059	1579	1903	26153	37592	47039	2.89	4.69	6.75	35.8	54.5	59.6
FIFTEEN STORIES	TENSION	1472	2207	-	36670	55005	-	7.16	10.74	-	19.1	30.0	-
	NO TENSION W/PT BARS	1403	1982	-	36434	47304	-	7.16	10.42	-	21.8	39.2	-
	PARTIAL TENSION	1413	2521	-	34639	51257	-	7.08	10.22	-	28.8	49.0	-

Table 4: Seismic Response of Isolated Cross-Wall, Including Rocking Motion [44]
(5% critical damping; panel, 7.3m x 2.4m)

It can be seen from the data in Table 4 that rocking motion will not necessarily lead to increases in force level; indeed, it may contribute to a lessening of forces. However, the rocking does contribute to a rapid increase in deformation, indicating a limit for a stable response. As suggested by Polyakov [76], the cracked cross-section may cause an extremely high load to be placed on the corner of the panel as the gravity load of the structure, the compressive block due to overturning, and lateral shears are all concentrated within a narrowing portion of the panel. Figure 36 illustrates the extreme deformed shape of the isolated wall during rocking motion. The concentration of forces associated with this motion may well cause failure in either the connection or the panel.

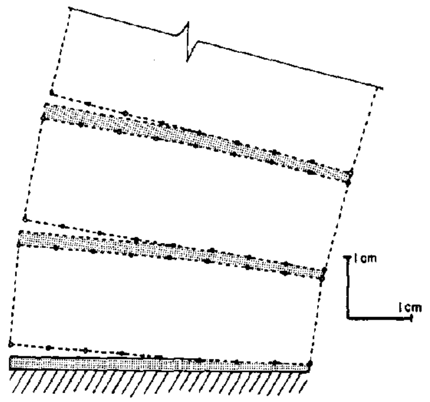

Figure 36: Deformed Shape During Rocking Motion - Lower Three Panels

The results reported in this section are from the first half of a study currently underway. Further work in this study will include a nonlinear material model for the horizontal connection, the inclusion of a vertical connection and transverse wall and soil-structure interaction. While the previous results are for idealized models, they illustrate both the similarities and dissimilarities between panelized construction and cast-in-place construction. The behavior and design of panelized structures must consider the planes of weakness that are created by the connections. The designer must consider the sensitivity of the structure to the behavior of the connections, but also the demands that they in turn place on the panels. These preliminary results indicate the extremely important role the compressive behavior of the horizontal connection will play in the overall safety of the structure and the need of the panel to be able to handle reasonably concentrated loads in order to safely develop the structure's ultimate capacities.

DESIGN IMPLICATIONS OF RESEARCH

The material presented in this paper has been intended to illustrate the uniqueness of panelized construction, particularly as it relates to seismic design. While experience with these structures in actual earthquakes is extremely limited, their use throughout the world is increasing. An attempt has been made to highlight the significant differences between panelized structures in the United States and those used elsewhere. Also, the discussion has been directed toward response of panelized structures to earthquakes as might be associated with major areas of seismicity. The author acknowledges that design for areas of less intense seismicity may not place as great a demand on these structures as has been indicated herein; however, it is only through an understanding of the actual response of structures that we can begin to base rational design procedures. The following discussion attempts to summarize the previous material into several major design issues that must be addressed in developing aseismic panelized construction.

Seismic Design Forces

Current seismic design codes used in the United States were not developed for panelized construction. The design forces and their distribution are based upon assumptions of flexural ductility (energy dissipation) and experience not directly applicable to panelized construction. It is not necessarily a simple question of using higher or lower design forces, but rather a question of developing a fundamental understanding of how panelized structures behave and how they should be designed. Until such an understanding is developed, the designer is called upon to use his best judgement. The studies by Llorente [44] indicate reasonably favorable behavior, up to some stability limit, if necessary details are properly designed. Beyond that limit, it would appear that there may be some form of dynamic instability. It is not clear at this time what that limit might be, or if analytical studies based on isolated shear walls reasonably predict the response of the overall structure; however, there is reason for caution.

Distribution of Seismic Forces

Several times in the course of this paper the effects of floor stiffness in the dynamic response of the structure have been cited. These effects may lead to a horizontal distribution of seismic forces quite different than normally achieved using a rigid floor assumption. While Polyakov [76] has given suggested design procedures for this situation, no such standard procedures are in use in the United States. Short of an expensive computer analysis, the designer is again forced to use his best judgement. If, indeed, there is a distribution dependent upon the flexibility of the floor system, then the floor must also be designed to handle the associated deformations and related forces.

Energy Dissipation

The question of ductility raised above is really one of energy dissipation; that is, what is the mechanism of survival for the structure subjected to a major earthquake. The following list presents five possible options for the designer; obviously they can also be used in some combination.

- *'Ductile' Shear Wall* -- Design the shear walls of the structure so that they can develop flexural ductility. This requires the development of an economical means of insuring adequate vertical reinforcement, and design the horizontal connection so it does not function as a plane of weakness in shear.

- *Energy Dissipating Connection* -- Take advantage of vertical and horizontal connections as planes of weakness and specifically design them to maximize their energy dissipating qualities. By allowing motion in the connection, one must be prepared to accept potential damage, both structural and non-structural. If connections are used as energy dissipators, extreme caution must be taken to insure their integrity, even as they degrade. This is particularly true for horizontal connections, upon which the ultimate stability of the structure is dependent.

- *Energy Dissipating Panels* -- Solid panels are generally too strong, relative to connections, to be forced into overall inelastic behavior. It has already been mentioned that panels with penetrations, that is, windows and doors, can provide an energy dissipating capacity. This dissipation is due to the flexural inelasticity of piers and lintels due to shear distortion. Purposeful mechanisms, such as slits [54], could be built into solid panels to give them energy dissipating qualities under shear distortion.

- *Soft Story* -- Several researchers [8,75,76] have suggested that soft stories may prove to be excellent design solutions. Recent experience has called the concept into question due to overall stability considerations. However, it may well be possible by using additional bracing or incorporating the soft story concept into the foundation to take advantage of the concept while providing the necessary stability.

- *Elastic/Brittle Design* -- For lowrise structures (less than five stories) and for areas of modest seismicity, it may be reasonable to design panelized structures to remain elastic during earthquakes. If this approach is taken, then the connections and panels can be designed on the sole basis of strength, with no consideration for inelastic response.

General Structural Integrity

In any major earthquake, a panelized structure, as well as all structures, will undergo some damage. It is necessary to guarantee that such a damaged structure retain its overall stability. The maintenance of this stability in a damaged state is directly analogous to the maintenance of stability after damage caused by an abnormal (non-seismic) loading. This has been the basis of the concern about progressive collapse and is also a major consideration in seismic design. The strength of the structure must not be so degraded, through seismic reversals, that the structure is no longer capable of supporting gravity loads. This potential for degradation may be a critical parameter in the design of horizontal connections. In addition, the overall integrity of the structure must be guaranteed by tieing the components together. This has been the basis of the alternate load path philosophy advocated for progressive collapse [26,98] and is equally valid in seismic design.

CONCLUSIONS

Over the last 20 years, many researchers throughout the world have been involved in developing panelized structures for use in seismic regions. This paper has attempted to present some of the material related to these activities. There is a large body of relevant material; unfortunately it is scattered throughout the world and often relevant only to very specific systems. For this reason, the authors wish to alert the reader to the fact that no portion of this paper can really be considered complete or all-inclusive.

Panelized systems provide an unusual challenge for both the design and research engineer. Here is a construction system that has potential economic benefits in a period of ever increasing costs. It is, however, a system with a limited experiential base. In order to retain these economic benefits and yet provide a seismically safe structure, the designer must look to new approaches in earthquake engineering. The designer cannot move forward alone, he requires the complimentary work of research efforts to bring forward needed information to confirm or deny new approaches.

The seismic behavior of panelized systems will be dependent upon connection characteristics. It is in understanding the role of connections in the overall response of the structure that the challenge lies. Two distinct options lie before the design engineer: provide for energy dissipation in the connections, or provide for energy dissipation in the panels. Regardless of the approach, the designer must insure the integrity of the system when potential degradation occurs in the process of energy dissipation. Of particular concern is the maintenance of a horizontal connection capable of continual transfer of gravity loads.

To move forward in the seismic design of panelized structures, four distinct and closely interrelated activities must take place. First, a continued effort must be made to bring together available information from throughout the world. This requires the development and maintenance of contacts with many research organizations, particularly in Eastern Europe, Russia and Japan.

Experimental work is required to develop an understanding of the component behavior of typical American systems. Work is needed on the transverse load and shear load characteristics of horizontal connections subjected to cyclic loading. The cyclic behavior of dry type embedded connections must also be studied. Experimental procedures must also be developed to explore the role of the floor diaphragm and potential rocking motion.

Concurrent with experimental efforts, analytical studies should be carried out to place experimental information in a proper context and to explore parametric sensitivities. Experimental results can also be used to calibrate computer models. In situ shaking studies should be carried out on panelized structures that now exist and those that are under construction. This data can also help calibrate programs and be used to examine the importance of soil-structure interaction and three dimensional behavior.

It is through such an iterative procedure of both analytical and experimental studies that a fundamental understanding of seismic response can be achieved. Unfortunately, the best experiment is an actual earthquake. It would seem appropriate to be prepared to examine such structures when and if they are so tested. In addition, the instrumentation of selected panellized structures in active seismic regions may provide valuable information.

Panelized structures present both opportunities and problems in seismic design. Their unique characteristics render traditional approaches to seismic design inapplicable, and call instead for the development of new design techniques. It is toward this goal that this paper has been addressed.

ACKNOWLEDGMENTS

The research being conducted at M.I.T. on the Seismic Resistance of Precast Panel Buildings is sponsored under Grant ENV 75-03778 from the National Science Foundation. The assistance on the project of Una Zeck, Robert Frank and Alan Unemori is gratefully acknowledged. The authors would also like to take this opportunity to thank the many precasters, designers and researchers who have willingly shared their experience and data on panelized construction. The graphical work for this paper was prepared by Elizabeth Whitbeck and Paul Fallon. Finally, a much deserved thank you to an excellent typist, Denali Delmar.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

AN EVALUATION OF THE STATE OF THE ART IN THE DESIGN AND CONSTRUCTION OF PREFABRICATED BUILDINGS IN SEISMICALLY ACTIVE AREAS OF THE UNITED STATES

by

Robert E. Englekirk
Vice-President, Ruthroff & Englekirk, Inc.
Adjunct Associate Professor, UCLA

INTRODUCTION

In order to be able to discuss the engineering problems associated with prefabricated construction, one must be aware of the general application of prefabricated construction in the United States in seismically active areas. The use of prefabricated construction may be subdivided into four general groupings.

First and fullest application of prefabricated structures is what I will refer to as FULL MODULE PREFABRICATION. Full module prefabrication involves the prefabrication of an entire module (in this case, a hotel room) in a factory. The unit is transported to the job site, and erected by crane. This type of prefabrication has, to my knowledge, only been used once in the United States in the construction of the San Antonio Hilton. Its use in this particular instance was primarily dictated by site availability, and the date at which beneficial occupancy was required.

The second application, PARTIAL PREFABRICATION, consists of prefabricating walls, floor slabs, kitchen units, and bathroom units in an assembly plant, transporting them to the field, and erecting the structure one floor at a time. The applications here are primarily hotels and apartment houses. The development of this program was significantly advanced by a program sponsored by HUD called "Operation Breakthrough". Unfortunately, only one manufacturer has been able to successfully develop projects under this program. Forest City Dillon Inc. has built approximately 100 projects throughout the United States, including sixteen in seismic zone three. These projects represent some 18,000 residential units.

The third category, and certainly the largest in dollar volume is PREFABRICATED COMPONENTS OF CONSTRUCTION. Here, walls and floors are constructed in the plant, brought to the field, and erected. The walls, are usually precast concrete, solid or with voids, though recently prefabricated concrete block panels have been introduced. Floor system components are, of course, the wide variety of precast prestressed units, including double and single tees, extruded planks, solid planks, etc.

Fourth, and probably the oldest, is SITE PREFABRICATION. The most common application is the site precasting of wall panels, which are then erected and combined with other structural elements, much as are the prefabricated components of category three. I have listed it as a separate category, as the constraints on component development are distinct. Early application of this type of construction was primarily limited to industrial "tilt-up" or "tilt-wall" construction. Another common application is in high rise construction where either floor or wall panels are stack cast on the site.

One cannot fully appreciate the magnitude of the problems facing the prefabricated construction industry unless one understands the construction industry, and how it develops its product, the completed building.

The traditional approach to developing a project has always been for a developer or owner to select an architect and consulting team. This team would then work with the owner in developing a set of plans, which would represent the type of development proposed for a particular location, the construction budget being consistent with a financial proforma. After carefully preparing these plans and obtaining the required governmental approvals, the plans would be submitted to several general contractors who would make proposals to the owner as to the total cost and time involved in completing the work indicated on the plan. The owner would then select a contractor who most closely fit his needs, and proceed with the construction of the project.

The late sixties and early seventies, with its tight money, rapid inflation and general material shortages made this approach to the development of a project extremely risky. Almost universally, owners would find that the cost of constructing a project as designed was not consistent with the financing available within a reasonable time frame. This usually resulted in an entire redesign of the project, an expensive option, both in time and money, or the abandonment of a project at a loss of around 10 percent -- also quite expensive.

From this rather universal dilemma developed what is commonly referred to as the 'fast track' approach to building design. Normally, in the fast track approach, a contractor is made a part of the development team during the schematic phase of design, and his cost and availability input is introduced into the project at the earliest stage of development. The introduction of the contractor into the development team does not solve the entire problem, as material availability and price change at a relatively rapid rate. Sequentially selecting, purchasing and constructing components before the final plans are developed or approved by governing agencies has become the only way to effectively control a project.

On a recent fast track project, a steel frame was selected for the basic system for an eight-story structure. The budget, guaranteeing cost of construction and schedule, was developed by the general contractor, based upon schematic plans and specifications. The steel fabricator was selected. His proposal was based on a set of schematic structural drawings. Mill orders were placed for basic structural shapes. Final structural plans for the steel frame were then prepared over the next four weeks. These plans were then submitted to the steel fabricator, as well as to the building department. Over the next eight week period, shop drawings were prepared, and plan checks completed by the building department. By the time the permit was obtained, the steel was ready to be fabricated, and foundations built. This resulted in a savings of approximately three months on this portion of the construction schedule. Obviously, if plans were developed in a normal process, i.e. full architectural and structural plans developed and bid, the total construction schedule would be extended some six to eight months.

What is the relevance of this example...especially as it applies to prefabricated buildings? Obviously, there is an inherent risk in proceeding in this fashion. Should the building official not agree with any of the design assumptions, an extremely embarrassing situation could develop. Mill ordered steel would probably have to be reworked in the shop or reordered. Several years could be spent in court, attempting to straighten out whose responsibility it was that the system was not built as originally envisioned. Since design concepts, criteria and construction techniques of steel structures are quite well established, as it generally is, in poured in place concrete construction, the exposure is usually limited to the addition of a stiffener plate here or there, another doubler plate, or problems of this order of magnitude, none of which are significant in terms of the overall cost of construction.

Can the prefabricated construction industry be competitive in this market? The principal advantage to prefabricated construction is the speed in which it can be erected. Thus, it seems to have answered the most basic need of the construction industry. If, however, the prefabricated construction industry cannot take advantage of the fast track approach, it would be at a disadvantage.

Of more importance, even than speed of construction to the effective use of the fast track approach, is SYSTEM RELIABILITY. Recall that in the fast track approach, the construction costs are determined during the schematic design phase, and hence to a large extent the subsequent development of the design may evolve about the system and elements selected.

Our firm was involved in the design of a partially prefabricated building several years ago. The proposed approach involved site precasting of wall panels sixty feet high, twenty-eight feet deep, and five-and-a-half inches thick as the basic, vertical and lateral load carrying system for a six-story hotel. Since the proposal was a cost saving redesign proposed by a contractor in order to save an otherwise doomed project, we were forced into a fast track approach. Panel forming and casting was started, even before plans were submitted to the building department. A special permit was obtained to construct the 'foundation only', and construction was started. In other words, we "let it all hang out". During the next month, I had numerous nightmares, often recalling verbal abuses leveled at me by many building officials regarding the inability of prefabricated buildings to withstand earthquakes. Probably most abusive was a Los Angeles County building official who, after arguing for some four months, gave us permission to build a two-story prefabricated motel building in Valencia (some ten miles from the epicenter of the San Fernando Earthquake) only after informing me that he would visit the job before permitting occupancy, "kick the corner, and watch it all fall down". He never did, of course, and the San Fernando Earthquake had no effect on it either.

Fortunately, the building official for the six-story precast hotel was not a skeptic, and a permit was issued in a reasonable time frame. Had he been a skeptic, he could have pointed to various ambiguous portions of the building code and required a major modification to the plans, or a considerable delay via normal appelate procedures in obtaining a permit, thus defeating the entire fast track approach.

This skeptical attitude taken by engineers, plan checkers and building officials is well taken, as little or no information, design procedures or research is available on the critical aspects of prefabricated construction.

STATE OF THE ART

Categorization into basic design groupings is helpful in the evaluation of the lack of knowledge which exists in the field of prefabricated buildings. Four general areas have been selected:

--State of the Art in Design and Analysis of Buildings
--State of the Practice of Building Design
--Evaluation of Component Reliability
--Analysis of the Reliability and Design Criteria Associated with Component Connections

State of the Art in Design and Analysis of Buildings

It is probably not correct to refer to dynamic analysis as a state of the art approach to designing or analyzing a building. Many engineers are currently using it in the design of all significant buildings. Our firm almost always performs a dynamic analysis of a project to some level of sophistication, to satisfy ourselves that our code, or modified code design approach, is realistic. The building departments in many parts of the state of California require that a dynamic analysis be performed on any unusual type structure, and almost all codes require a dynamic analysis on buildings over 160 feet in height. It must, however, be classified as a state of the art approach in the analysis of prefabricated buildings, as so little information is available upon which to base many decisions.

Response spectra developed by Geotechnical Consultants usually postulate ground acceleration at the surface in the range of .3 to .5g. The amplified portion of response spectra for most buildings ranges from .5 to 1g, depending on the type of soil conditions and damping values prescribed. Prefabricated structures are of two basic types: intermediate height high rise residential buildings of six to twelve stories braced by prefabricated shear walls at 28 foot centers, and large one and two-story department stores laterally braced at the perimeter by short long shear walls. Both of these building classifications usually fall analytically in period ranges at or below .3 seconds. Since this places them within the range of design criteria associated with the ground motion as opposed to amplified ground motion, the total base shear is not much different than that prescribed by the latest building codes. The major area of concern of all engineers associated with this type of construction is whether or not this is an appropriate force level. Should higher force levels based on longer true periods be used for prefabricated buildings in consideration for the longer periods which might occur at high force levels as a result of joint slippage? If the true period of a building is longer than the associated analytical period, what then would be appropriate levels of damping? The lengthening of building periods in this region is obviously of much more concern than it is in frame structures. Assuming that it does not occur leads to a non-conservative approach for prefabricated buildings, whereas it is a conservative approach for frame structures. The design implications, however, are quite substantial.

If higher force levels are indicated, do these force levels apply to overturning forces as well as to shear forces, especially in lower portions of buildings and in the design of foundations? How do diaphragm forces affect the design requirements for shear walls of low rise buildings? Is it appropriate to consider the fundamental period of a structure as being .2 sec. even though this is indicated by the stiffness of the restraining system, or should a higher natural building period be used to account for the dynamic reaction of large diaphragms on these short shear walls?

Since the response of prefabricated buildings to either ambient or forced vibration tests, or the evaluation of the response of existing buildings subjected to earthquakes has not been made, the engineer must make many decisions based on his intuition.

State of the Practice of Building Design

The state of the practice of building design is best summarized by the building codes enforced in the state of California. The most universally adopted procedure is that contained in the Uniform Building Code [3]. The seismic design portions of this code were developed to a large extent by the seismology committee of the Structural Engineers Association of California. The salient points applicable to the design of prefabricated structures are the requirement to design shear walls, and shear resisting elements of buildings, for an ultimate base shear of .44g. The .44g is determined by using a value of ZKCS of .186, a load factor of 2, and a safety factor of .85. The associated design overturning moment is .28, here using a value of ZKCS of .186, a load factor of 1.4, and a safety factor of .85. This is not inconsistent with the force levels indicated by most geological reports for maximum anticipated ground acceleration.

These force levels are then used to design the various elements of the structure using the stipulated ultimate for the various materials. Unfortunately, many of the ultimate values listed in the building code often include rather substantial factors of safety. The Uniform Building Code requirements for wall stresses, when compared with those design procedures developed by Kripanarayanan and Fintel [1] are quite conservative.

This condition is further agravated by the requirement in the UBC that concrete bearing walls must have a minimum thickness of six inches for the upper 15 most feet of height, and increased in thickness by one inch per 25 feet unless, of course, "structural analysis shows adequate strength and stability." The interpretation of arbitrary provisions in building codes is a tremendous source of frustration to the practicing engineer. For example, this portion of the code was cited on one of my projects by the building office. The building was a twelve-story prefabricated structure. Had this provision been enforced, the thicknesses of the walls would have had to be nine inches at the base, as opposed to the five-and-a-half inch thick walls which we proposed. The building official refused to consider the fact that the walls were spaced at 12 feet on center, and had a maximum clear height of eight feet, and thus were capable of carrying quite safely any loads indicated by any design criteria. Fortunately, he finally agreed to consider the wall panels as precast wall panels if we could establish that the entire vertical and lateral load could be supported by the ends of the panel. It goes without saying that interpretation of this type results in a considerable factor of safety in the overall design.

If this same conservatism were used with a dynamic analysis on a building of this type, the true overturning capacity would be on the order of 1.4g. Overconservatism in this area is an extremely expensive item insofar as prefabricated construction is concerned, as precast assemblies must have uniform thickness, not only for the length of the panel, but also for the height of the building. Thus, if a ten inch panel is indicated at the lowest level, when in fact an eight inch panel would be safe, a considerable amount of extra concrete and weight would be added to the overall structure.

Evaluation of Component Reliability

Component reliability is another extremely important area to the design engineer. Probably the best and most controversial example of component reliability is the floor diaphragms of large structures utilizing prefabricated floor construction with perimeter shear walls. Here, the designer is usually faced with heavy prefabricated beams and slabs or other precast prestressed assemblages tied together with a relatively thin poured in place topping slab of two-and-a-half to three-and-a-half inches average thickness. The design forces which this diaphragm must sustain during a major earthquake, and the load deflection curves associated with its response to this ground motion, are of paramount importance to the overall stability of the building.

Countless tests have been performed on metal decks supporting concrete fill or vermiculite concrete. However, as yet no program has been developed for testing diaphragms consisting of prefabricated elements tied together with a topping slab. The areas of concern to the designer that need considerable investigation, have primarily to do with the stiffness and strength of the overall composite system. For instance, is it reasonable to assume that the precast elements contribute to the overall stiffness of the diaphragm? This would seem like a reasonable assumption, since prefabricated units are placed adjacent to each other, and the joint is usually some type of grouted shear key. A reinforced topping slab connects the units, developing a composite system for resisting vertical and probably lateral loads. The contribution of the prefabricated elements to the overall diaphragm stiffness is obviously important when attempting to evaluate the force or stress level to which the system might be subjected during an earthquake. This also effects the amount of distortion which might be anticipated at the exterior wall perpendicular to the direction of motion. The common assumption is that only the topping slab contributes to diaphragm stiffness.

The shear strength of thin diaphragms is usually based on diagonal tension stresses, reinforcing requirements being determined in accordance with diagonal tension theory. An area of concern to many engineers is the effect of shrinkage cracks on the strengths and reinforcing requirements of a diaphragm.

Reliability of Component Connectors

The design philosophies associated with the development of these details is considered the most important part of any building theory by many engineers. Though each detail is tailor made for a particular construction problem, they may be subdivided into three general categories: shear, tension, and compression transfer. Since virtually no guidance can be found in the literature, the only criterion which exists is contained in building codes.

The transfer of shear along a horizontal joint, as treated in the building code, refers to shear stress as being transferred as "though in masonry through a mortar bed, by shear key, or by reinforcing bars used as ties with shear values similar to those associated with bolts set in either concrete or masonry". Shear keys or mortar bed analogy is not very popular with the design engineer because joints of this type usually open up, as they are natural places for shrinkage cracks to occur. Thus, confidence in this type of detail and analysis is not high among the profession even though no code provisions prohibit it. The installation of mortar beds and shear keys are usually difficult and extremely time consuming. The use of dowel theory is virtually impossible as the force levels prescribed by the building codes cannot be resisted by the ultimate capacities of about 3,000 pounds per 5/8 inch diameter bar. Shear friction theory, though not specifically developed to handle these cases, has become the most widely used approach for shear transfer from element to element. The reliability of this method, when applied in details of precast prefabricated construction must certainly be established if it is to be heavily relied upon in the industry.

Tension ties in wall panels are always a problem. The reasoning again stems from the type of construction involved, primarily the fact that the thickness of the panel is constant throughout its height and width. This does not permit boundary elements on the end of the panel unless these boundary elements are cast in poured in place concrete, a very arduous and time consuming task which, in addition to its cost, also introduces one more joint in the structure. The problem lies essentially in the reliability of stress transfer in heavily loaded members joined in thin concrete elements, with minimal confinement.

Compression transfer problems lie in the fact that many walls, as in the prefabricated buildings currently being erected by Forest City Dillon, have regular voids spaced along the entire length of the panel. These voids are filled once the wall panel is in place, and the floor slab and its topping are being installed. The problem here is in the incompatibility of the elements. The precast wall panel is constructed in the shop. The plant fabricated floor slabs are placed on the edge of these panels, and the voids in the wall panels are then filled with concrete. Many people in the industry are concerned about the vertical shrinkage of the concrete after it is placed, and its ability to transfer compression loads adequately without some type of expansive or positive grouting between elements (see Figure 2).

WHAT MUST BE DONE TO DEVELOP SAFE, ECONOMICAL, PREFABRICATED BUILDING IN ACTIVE SEISMIC AREAS

Some of the major problem areas associated with prefabricated construction have been alluded to. Most engineers are conservative by nature and unfortunately skeptics when it comes to accepting new concepts. This is also true of the building industry as a whole in the United States. We have been years behind Europe in the development of new construction techniques, and prefabricated buildings is certainly no exception. Prefabricated structures are undertaken only when there is a clear indication of substantial savings in the cost of construction.

It is only possible to gain product acceptance by resolving the many uncertainties which currently exist. Extensive test programs must be developed. Thorough analytical evaluations of building systems must be undertaken. The results of these efforts must be made readily available to the design professional.

The builders of prefabricated structures and/or components have no association to promote their basic concept or product. Each system is tailor made to the particular application and usually proprietary. Having no association similar to the AISC, ACI, PCI, no funding of an intelligent comprehensive test program is available. The development of meaningful criteria is further hampered by the complexities of prefabricated construction. Researchers are not familiar with system constraints, typical procedures, or details. The wide variety of products and applications further hampers effective research activity.

Problem areas have been ordered such that priority has been given to those design features which most readily affect the overall stability of a building system, as well as the overall economics of prefabricated structures.

Overturning

The first and most important problem facing not only the precast industry, but shear wall construction in general, is a realistic evaluation of overturning forces. It is presented first, as it has considerably more effect on prefabricated structures than poured in place shear walls for reasons previously mentioned.

Probably the best way to visualize the problem faced by the precast industry is to examine a typical building. The partial plan of the building shown of Figure 1 is what one might expect in a twelve-story hotel or apartment type structure. If we assume clear story heights of eight feet, the overall height of the building would be on the order of 104 feet, allowing for eight inches of floor construction. An examination of a typical transverse shear wall would indicate the following. The code prescribed period would be .65 seconds. The ultimate base shear in the wall would be 520 kips.

$$V = KCSUW = 1.33 \times .083 \times 1.5 \times 2.0 \times 1560 = 520 \text{ kips}$$

The ultimate code overturning moment would be 25,360 ft-kips. A spectral analysis utilizing a spectral acceleration of .5g would generate a base shear of 400 kips, as opposed to the code required ultimate shear of 520. The overturning moment indicated by this spectral analysis would be 28,000 ft-kips.

If we were to assume, as the code generally requires for structures for this type, and incidentally, as is common practice, that the wall behaves as a homogeneous isotropic material, we would find that the maximum compressive stress resulting from a combination of a vertical load of 1,200 kips, and an overturning moment of 25,360 ft-kips at the base of the wall would be on the order of 2.17 ksi. The allowable ultimate stress permitted by the ACI Code [2] or Uniform Building Code [3] is developed in equation 14-1. This allowable ultimate stress for the wall in question is 1.4 ksi. Obviously, the wall does not comply. If ultimate strength concepts were used to analyze the wall, some 60 or 70 inches of that wall would be under an average compressive stress of 3.06 ksi. This stress is considerably below the buckling load of the wall. We would also not find any problems with permitting a stem of a T-beam to attain these stresses. Why, then, should the ultimate strength in walls of this type be limited to 1.4 ksi?

An associated area of controversy lies in the rationalization of the tension stresses induced by this bending moment. One approach is to provide reinforcing required or indicated by the assumed tension block, much as is done in providing supplementary bonded reinforcing to control cracking in post tension beams or slabs. Using this approach, the indicated ultimate tensile force is 740 kips. Using grade 40 reinforcing, this amounts to some 18.5 s.i. of steel, or, using ultimate design theory, some 430 kips. The transfer of tension forces of this magnitude across joints creates a tremendous hardship on precast, prefabricated construction. Weld details are typically frowned upon by reviewing engineers under the assumption that the reinforcing in the ends of the wall must yield, and therefore weld induced points of brittleness in the reinforcing will lead to premature failure of the detail. Splicing reinforcing using development lengths provided in the code requires almost the full height of the panel. The reliability of an unconfined lap splice in thin panels is also a concern. The desired objective should be to conservatively estimate tensile forces and provide a practical elastic joint.

Shear Transfer

The transfer of shear stresses between precast elements is probably even more controversial than axial loads on precast walls. Figure [2] is representative of a typical joint between adjacent wall panels and adjacent floor slabs. There are many variations of this basic detail, all developed to fit the variety of precast elements and construction techniques available on today's market.

The shear transfer at A is always the most critical item. The grout, or concrete, above the shear transfer line is almost always wet concrete placed over a previously dried solid concrete panel. Since the wall serves as a resting place for the adjoining precast floor panels, the throat of the detail is often reduced to as little as two-and-a-half inches. In the example problem, the entire ultimate shear of 520 kips must be transferred through this two-and-a-half inch slot. The resulting shear stress is 580 psi, greater than the maximum allowable shear stress permitted as a measure of diagonal tension. The common approach in the practice today is to assume that shear friction theory applies to these conditions, and that an allowable stress of 800 psi is acceptable. This assumption, however, is not documented in the literature, nor indicated in design examples or procedural applications. Very often, reviewers are highly critical of this type of a detail, and insist on rather large factors of safety, even though design shears are representative of spectral accelerations in excess of .5g.

Shear transfer at B is also critical. Here, the problems are somewhat different. There are two basic conditions which usually occur at this point. Case 1 involves a condition in which the floor slab is poured, and a hollow core wall is placed on top of the previously poured floor. This requires dropping concrete eight feet through the voids, and counting on its ability to bond, or transfer stress at the floor line. Cleanouts or voids are provided at the lower part of the panel to insure that grout has reached the lowest surface under enough pressure to create a good bond.

Case 2 involves the placement of a prefabricated solid panel on a previously poured floor slab. In these instances, vertical reinforcing dowels are usually embedded in partial voids cast in the panel, which are then grouted from the side. Usually, an attempt is made in developing a shear key at the base of the wall, grouting that shear key so as to transfer the shear.

A dry joint of some variety with an associated allowable shear transfer mechanism would be invaluable to the prefabricated concrete building industry. The need for an extensive test program on joints of this type is essential to the effective economical use of prefabricated products.

The shear transfer problem in large buildings with perimeter shear walls is also of major concern to the structural engineer. A typical condition is detailed on Figure 3. Very high shear stresses must be transferred from a large diaphragm to a previously constructed masonry wall. Shear transfer techniques of all varieties are often employed. Precast elements are often held back from the wall to permit pour strips of varying widths adjacent to the wall. Shear keys are installed in the walls. Dowels are cast in the wall, and extended into the topping slab diagonally. Reliable procedures for transferring stresses on the order of five and six kips per lineal foot must be developed if any reliance is to be placed on this type of construction.

Test programs must be developed to determine the effectiveness of shear transfer across joints utilizing dowels embedded in a three inch topping slab. Test results should be compared with shear friction theory since it is the most widely used concept for evaluating the strength of this type of joint. A variety of details including shear keys, pour strips, combined with various wall surfaces should be investigated.

Diaphragm Stiffness and Reliability

An unusually large diaphragm, though one which would be allowed by building codes, is shown on Figure 4. The current code [3] would require a minimum shear capacity in this diaphragm equivalent to a uniform acceleration of .25g. This would generate a maximum shear force along line 1 of 5.25 kips per foot with an associated shear stress in the topping slab of 146 psi. The calculated deflection of the diaphragm based on the moment of inertia and area of the topping slab would be less than .25 inches. Chord tension forces occurring at 2-A and 2-B is on the order of 400 kips. The calculated period for diaphragms of this type is on the order of .2 sec or less.

Is the calculated stiffness and hence period and deflection determination accurate? Does the theory account for the cracking which occurs in the topping slab? Does the amount of reinforcing crossing cracks affect the overall stiffness of the system? Forced or ambient vibration tests might be of some assistance in evaluating diaphragm stiffness. Destructive testing of bays or models could be very informative.

Are anticipated diaphragm forces considerably higher than those suggested by the code? Many engineers feel that they are. Analytic work on assemblies such as floor diaphragms is required. What are the dynamic loads imposed on the shear walls?

Is the strength of the diaphragm severely effected by cracking on the order of 1/32 of an inch? Whereas there has been quite a bit of work done on shear transfer across cracks, little or none of it seems to apply to thin reinforced diaphragms. Mesh is the ideal reinforcing for diaphragms of this type. Is it effective in terms of shear transfer across random cracks? Tests should be performed on thin stiffened topping slabs reinforced with mesh (6 X 6 - 6/6, 6 X 6, 10/10) and various reinforcing patterns (#3's @ 18 inches on center, #3 @ 12 inches on center).

Indicated chord forces are quite high. Are these forces realistic? Destructive model testing might shed some light on this subject.

The list is endless. Damping, construction joint strength, and many other subjects could be added. Many years of research are required to develop a reliable design criterion.

Dynamic Characteristics of Prefabricated Buildings

Many prefabricated buildings in the three to eight story range designed dynamically using spectral techniques use spectral intensities which are not in the amplified portion of the spectrum. The use of the ground motion spectrum is based upon the unusually stiff building which results with the normal application of prefabricated products. The twelve story building previously discussed (Figure 1) has a computed fundamental transverse period of about .45 sec. Forced vibration tests coupled with the installation of a permanent monitoring program in intermediate height high rise buildings should produce valuable insight into the dynamic characteristics of these buildings.

CONCLUSION

It should be fairly obvious that very little reliable information is available to the engineer who wishes to undertake the design of a prefabricated building in a seismically active area. The alternatives available are:

--to design all structures using proven systems and deny society of the economies afforded by prefabricated buildings

--expose society to the higher level of risk associated with extending existing research to pseudo-analogous conditions

--apply unrealistic safety factors to all prefabricated construction to account for the lack of information available, thus not taking full advantage of available economies.

None of these alternatives are attractive to the engineering profession. The only true engineering approach is to develop those tools necessary to provide society with the most economical solution consistent with the natural and functional constraints of a project.

1. Kripanarayanan, K. M. and Fintel, M., "Analysis and Design of Tilt-up Reinforced Concrete Wall Panels", Journal of the American Concrete Institute, January, 1974.

2. Building Code Requirements for Reinforced Concrete (ACI 318-71), November, 1971.

3. Uniform Building Code, 1976 edition, International Conference of Building Officials, Whittier, California.

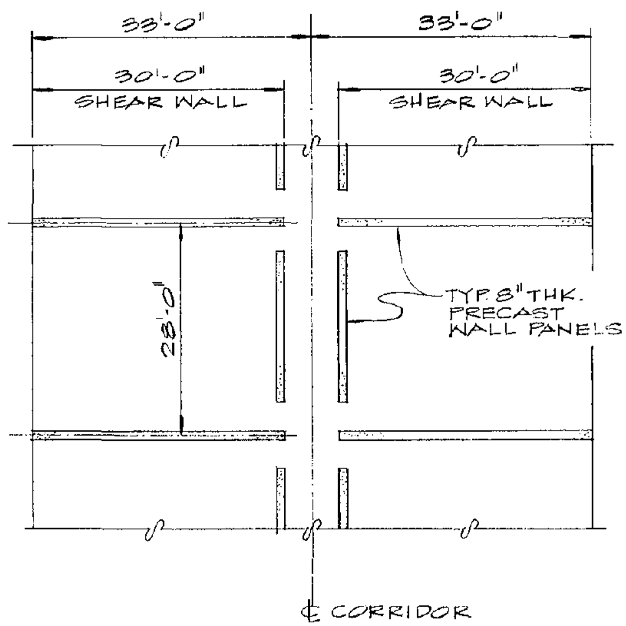


FIG. 1 TYPICAL RESIDENTIAL FLOOR PLAN

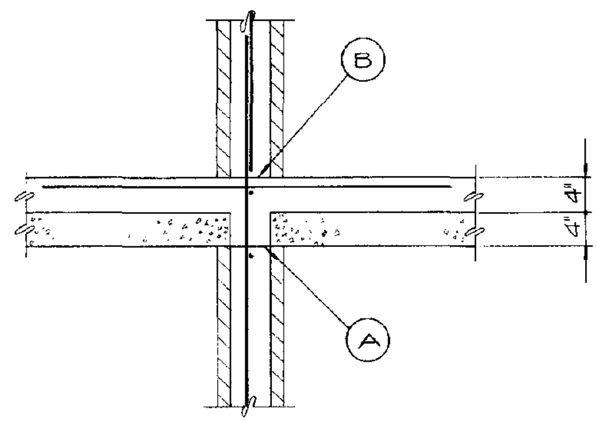


FIG. 2 TYPICAL JOINT DETAIL

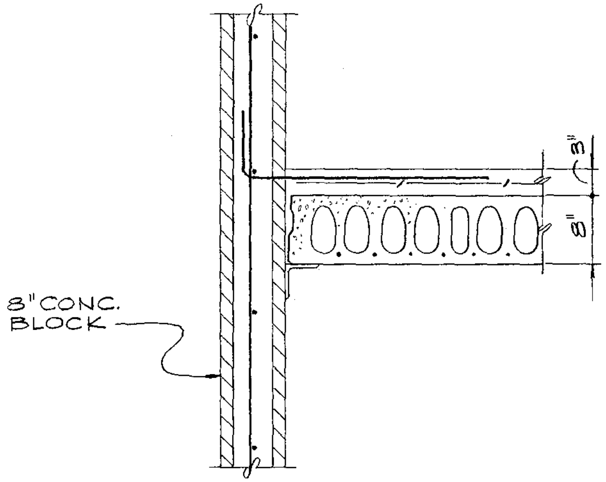


FIG. 3 TYPICAL JOINT DETAIL

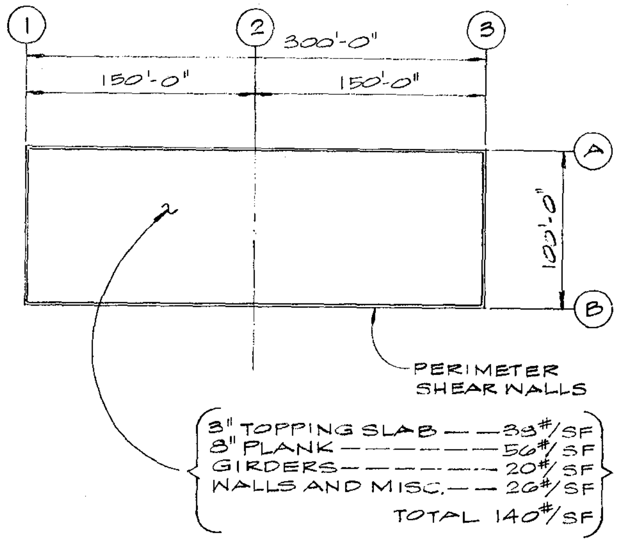


FIG. 4 FLOOR PLAN

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

SOME ASPECTS OF APPLICATION AND BEHAVIOUR OF LARGE PANEL SYSTEMS IN SEISMIC REGIONS OF EUROPE

by

M. Velkov
Professor of Earthquake Engineering
Institute of Earthquake Engineering
and Earthquake Seismology
University of "Kiril and Metodij"
Skopje, Yugoslavia

D. Jurukovski*
Associate Professor
Institute of Earthquake Engineering
and Earthquake Seismology
University of "Kiril and Metodij"
Skopje, Yugoslavia

1. INTRODUCTION

The problems of referring to application of large panel precast systems in seismic regions are extremely important and are closely related to a wide complex of conditions. Due to its appearance, this system is attractive and is widely used in housing construction all over Europe. During recent years many countries have shown an increasing interest in the application of large panel systems, so at present even developed countries like the U.S. and Japan are interested in the application of this system. The large panel buildings constructed in precast systems can satisfy the increasing demand for mass housing construction.

The construction of such systems in seismic regions is mainly based on investigations both theoretical and experimental with special attention to the structural connections. At the same time, efforts are made to stick to the basic design concept, which, in the case of large panel construction, means achievement of the closest possible similarity with the monolith structural systems.

Until the earthquake of Tashkent, USSR, in 1967 and the disastrous earthquake of Romania of March 4, 1977, there were no available data for the behaviour of these systems during strong earthquakes.

From the experience with the development and application of this structural system, the following can be concluded: The development and application of large panel systems has been accompanied by experimental and theoretical research in this field. The results of some investigation programmes enable verification of the stability of the investigated systems and provide a basis for design of new structures in large panel systems.

The adopted concept that the design of precast large panel systems should be similar or identical to the monolith system cannot be readily applied since all problems related to the stability of earthquake resistant monolith structures have not been solved yet. Therefore, the advanced development of housing construction all over the world, as well as the development and wide application of this system should initiate experimental and theoretical investigations aiming towards a definition of the seismic stability of large panel systems.

*Author of Section 3.4

2. Application of Large Panel Systems in Europe

After the Second World War, in order to satisfy the extensive housing programme and to meet quickly the housing requirement, many European countries, such as the USSR, France, Denmark, England, West Germany, East Germany, Hungary, Poland, Romania, Bulgaria, Yugoslavia, etc., adopted the precast large panel system as favourable structural and technological system.

This system was adopted in the mass construction both in inactive and active seismic regions. In the earthquake endangered Eastern European countries the basic design concept was to construct large panel structures with appropriate connections which would make them similar to monolithic cast in situ systems (monolithic cast in situ shear walls). The concept was proved satisfactory in the behaviour of these structural systems during the Romanian earthquake. However, in seismically safe countries, the level and extent of cast-in-place procedure is rather decreased in order to simplify and speed up the mounting technology. The technological development brought to light the instability problem of this type of structure which was emphasized after the collapse of such a building in Ronan Point, London, England.

Most of the Eastern European countries have their national codes comprising general regulations for design and calculation of this type of structure without going deeper into the complexity of their performance in the nonlinear range (e.g. nonlinear performance of connections).

"Les Code International" are regulations for large panel structures which were adopted in 1969 by the C.E.B., CIB and UEA and are rather detailed recommendations which refer to the design of these structures for dead load plus wind load. However, due to the lack of both theoretical and experimental investigation results they do not include any criteria or regulations for aseismic design of these structures. Nevertheless, these structures are widely used, as we mentioned before, even in seismic regions all over Europe, mainly up to five storeys in height. The progress of experimental and theoretical investigations enables construction of multistorey large panel buildings of eight storeys in the USSR, ten storeys in Romania and recently construction of buildings taller than 10 storeys. In Yugoslavia, after the Skopje earthquake of July 26, 1963, large panel buildings of 4 storey height were introduced, while today buildings as tall as 21 storeys are constructed, such as the apartment block buildings in Zemun, Belgrade.

3. Seismic Design Considerations

3.1 Structural Components and Configurations

When considering a structural system of precast large panels, the designers almost always think of a monolith reinforced concrete structure. In order to realize such a system, the structure and the structural members should be designed to include structural walls able to withstand both vertical and horizontal loads. This means that monolithic structures of panels acting as structural walls could also be realized as large panel systems. This gives a possibility for realization of the system according to alternative technological, prefabrication and architectural programmes.

The basic concept for design of a precast system, including the transformation and design of assemblages of a monolithic structure into a large panel system, is developing according to the following criteria: technological prefabrication procedure, transportation, capacity of transportation facilities, and the connection concept. According to European practice, the connecting point between panels are the intersecting edges between the walls and the intersection between the horizontal panels and walls.

Seismic considerations are given in correlation with the position and the general conformation concept of connections, as follows:

(a) In order to provide sufficient strength of the system for earthquake effects, it is necessary to incorporate earthquake resistant structural members in both directions of the structure. Currently, in all seismic regions of Europe especially in Eastern European, a two-way system which mobilizes all the walls to withstand the seismic effects is adopted. This arrangement of vertical structural panels is rather inefficient for small apartments. In future, thought should be given to the concept that only a part of the existing walls should act as structural walls during earthquakes. In this way, the external facade walls and part of the internal walls would act as non-bearing walls. In this way, the total length of connections would be decreased and a possibility given for application of light weight material for the nonstructural members in the system. The system so designed would have a much simpler configuration and at the same time more clearly defined response to earthquake effects.

(b) In general, the walls may have various sections, rectangular and flanged. Even the possibility for barbell section should not be excluded, although panel walls of such section have not been used so far. Concerning reinforcement, the reinforcing bars may be longitudinal, transverse, and additional "confinement" reinforcement. Their combination can improve the system as a whole, resulting in better performance during earthquakes.

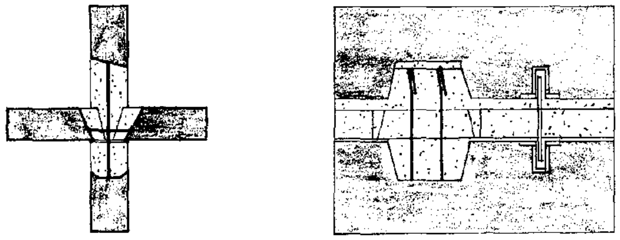
(c) It is desirable to define in advance the zones where development of nonlinear deformations is allowed, avoiding the performance in plastic range particularly if it is necessary that the nonlinear deformations of the structural panels are realized through reinforcement yielding due to overall bending caused by overturning moment.

(d) In some cases the possibility should be considered to design a structure with one part monolith, where nonlinear deformations are expected.

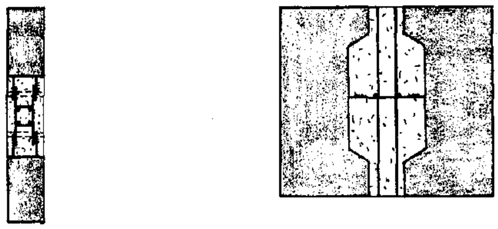
In the above paragraphs the experience with monolithic structures was summarized in order to enable its application in precast large panel systems.

3.2 Connections

Today, many solutions for connections in large panel systems are in use. Depending upon the construction method the connections can be dry, wet and semi-wet. Their solution should provide sufficient strength to the structure. The connections used in Europe are: reinforced concrete wet connections with welded anchor reinforcement, dry connections with welded anchor reinforcement, and prestressed combined (semi-wet) connections. The most frequent type of connection used in the seismic zones of Europe is the wet reinforced concrete connection with welded reinforcement. Figures 1, 2, 3 and 4 show several types of connections used in the seismic zones of Yugoslavia.



Horizontal joints



Vertical joints

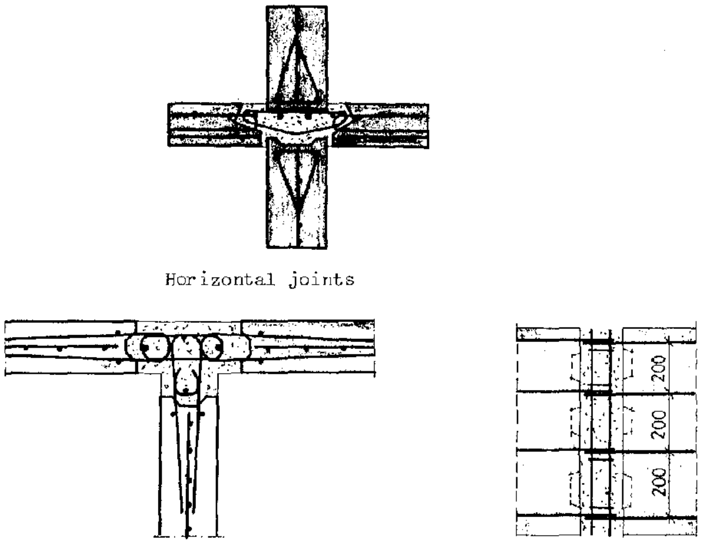
Fig. 1 Joint system used for "Hidrogadnja" 21 storey building constructed in Zemun - Belgrade

In order to provide a force transfer between two panels, which force can be due to compression, tension or shear, acting in the direction of or transverse to the connection, the following effect combinations should be analysed: configuration of panel edges, additional concrete for casting in place, reinforcement or other steel devices, transmission of stresses in reinforcement (welding, slip and stirrups).

From the aseismic design viewpoint it is very important that, through various combinations of the above stated parameters for connections, the influence of not only the strength capacity but the elastic and postelastic deformability characteristics, the energy absorption and dissipation capacity, as well as the ductility should be defined. Based on these characteristics, the connections can be elastic brittle or having sufficient postelastic deformable capacity, the latter being recommended for use.

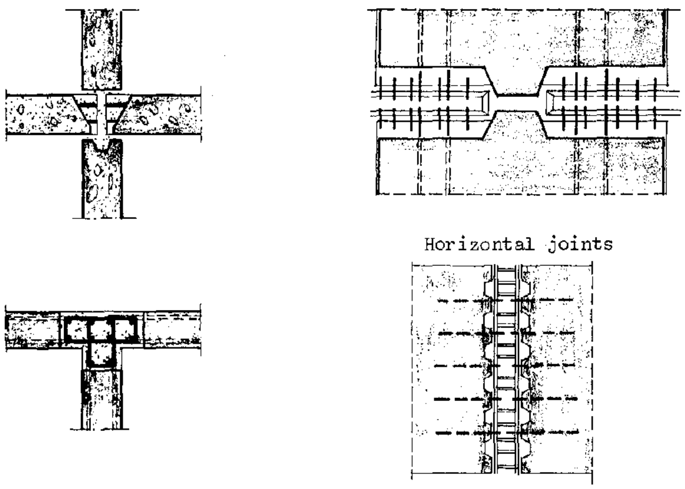
Taking into account these criteria, which at the same time are the basic concept of earthquake resistant design of structures, the designer should use only such types of connections which have performed sufficiently in the postelastic range, without strength deterioration under cyclic loading, during experimental tests.

A typical example of elasto plastic connections are the vertical connections tested in Saint Remy-les Chevereuse, France, by Pommeret (Figs. 5 and 6).



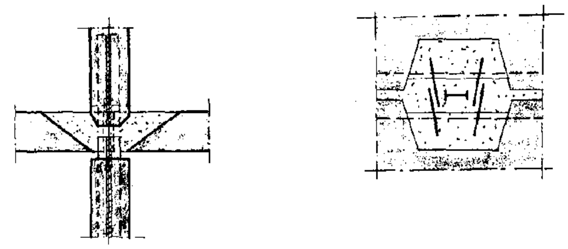
Vertical joints

Fig. 2 Modified "Balancy" system for tall buildings in Belgrade zone

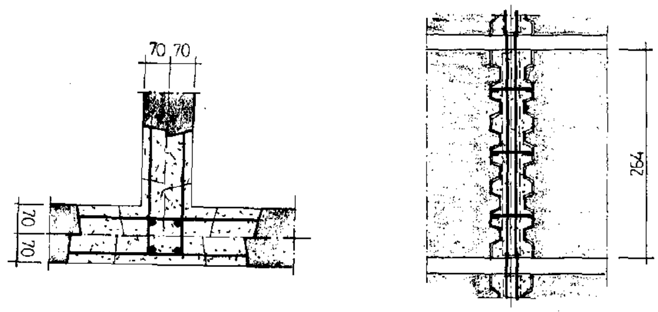


Vertical joints

Fig. 3 Joint system "Spuz" in Titograd



Horizontal joints



Vertical joints

Fig. 4 Joint system "Karpos" in Skopje for 9 storey height

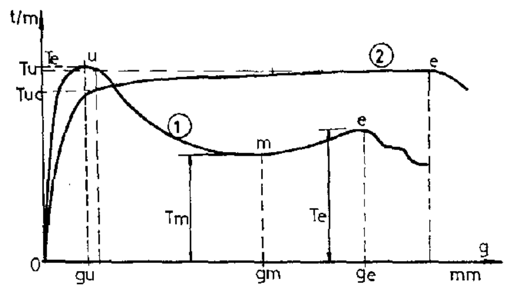


Fig. 5 Typical curve for castellated panel edge configurations (1)

Typical curve for plan panel edge configurations (2)

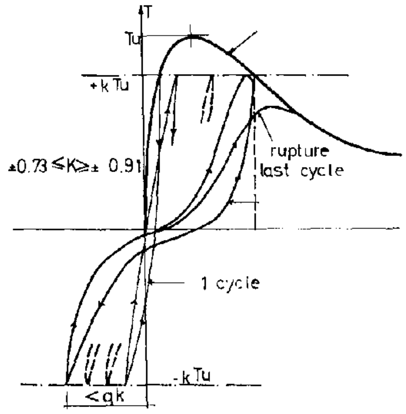


Fig. 6

Shear tests were carried out on many members with static one-way load and different types of joints (1) and (2). For correlation purposes experiments applying cyclic loads were carried out as shown on the above figure. The purpose of Pommeret's investigations was to define the influence of each parameter and give recommendations for elaboration of design criteria.

Interesting to be mentioned here is the example of the experimental research on deformational and strength characteristics of connections of a particular system constructed in Belgrade, Yugoslavia being 21 storeys high, as shown in Fig. 7.

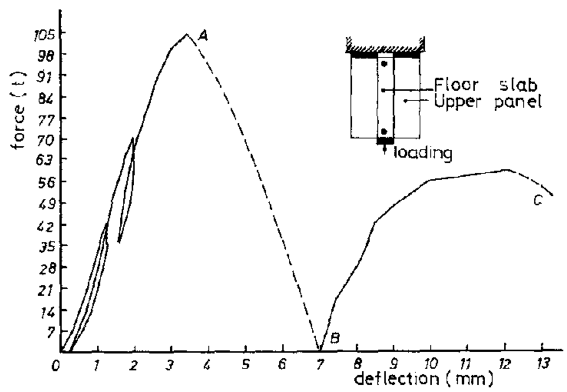


Fig. 7 Stress-strain relationship

3.3 Analysis of the Total Structure and the Structural Members

Having in mind that this type of structure is constructed in series, a detailed analysis of structural members and the structure as a whole in combination with experimental investigation is necessary.

Seismic analysis of structural members is rather difficult because of the complex character of the precast large panel systems. The structure works in space and the structural members are interconnected in both directions.

During the mathematical model formulation of the structure some simplifications are necessary, even in case of complex three-dimensional analysis of models (application of TABS programme, for example) which consists of:

- Definition of structural members in the two orthogonal direction as rectangular or flanged sections

- To assume that deformations of joints should be the same as those of monolith structures

- Damping is usually considered as a factor of constant value, however the damping effect is much more difficult to be determined as it has significant influence upon the dynamic response of structural systems

- Influence of soil-structure interaction, which is extremely important for this type of structure, particularly for lower structures, it is the factor which affects the mathematical model formulation considerably.

The above-mentioned aspects refer to the linear treatment of the structural model response under earthquake effects; however, modeling of the nonlinear performance of the system is a rather more complex and cumbersome problem.

In European practice the analysis of large panel systems is a rather simple one according to the existing Technical Provisions for aseismic design. It should also be mentioned here that the three-dimensional response analysis of a 21 storey large panel building carried out in Yugoslavia is a combination of field and laboratory investigations.

Stress analysis--The stress analysis of both structural members and connections (horizontal and vertical) is of significant importance. The European practice usually applies empirical relations which are partially based on experimental research. Important to be mentioned here are the studies of Pommeret, where the author derived empirical formulae for ultimate state calculation of joints. Special attention is given to vertical joints depending upon the finishing of the contact surface of the panel.

In defining the stress state of joints, i.e. their ultimate strength and deformability, the following are of special importance:

- The mechanism of linear and nonlinear deformations within the system (opening of joints)

- Type of joints, plain, castellated, with or without shear keys

- Amount and distribution of longitudinal and transverse reinforcement, its quality and extension

- Cast-in-place procedure and friction in joints

3.4 Experimental Full-Scale Tests of Joints and Structural Assemblages

The increased application of industrialized methods of construction all over the world requires both efforts and responsibilities of structural engineers in order to satisfy the necessary stability criteria. The problem is even more complex since there are many precast systems which are equally constructed in seismically active and inactive regions. However, due to the relatively short period of application of this type of structure in practice, there is a lack of experimental data from observations of damage to these structures during actual earthquake effects. A compensation for this lack can be provided by programmed laboratory or field full-scale tests of structures.

The purpose of these experimental investigations is to:

(1) Define mechanical properties of horizontal and vertical joints of large panel precast systems.

(2) Define the behaviour of multi-storey panels in given scale, in one plane, with simulation of end conditions.

(3) Full-scale forced vibration studies and definition of the dynamic properties of structures.

The investigations under (1) and (2) are performed under quasi-static conditions. Beside the strength characteristics of connections, very important information is the behaviour of connections under reverse loads, i.e. their capacity to withstand a large number of cyclic loads in postelastic range. For realization of such investigations, specific equipment consisting of multi-component excitation system is necessary with precise force and deformation control and data acquisition system. With such investigations data concerning the ultimate strength of connections, ductility and reverse load capacity can be obtained for loads due to pure shear, shear and moment with or without gravity loads, etc.

Investigations under (3) are carried out by full-scale force vibration tests of structures. The purpose of these studies is to define the dynamic properties of the structural system for the first several translational modes of vibration in the two orthogonal directions and torsion. Very important information for the defined modes of vibration is the viscous damping capacity. It should be pointed out that the above-mentioned investigations are performed for linear behaviour of structures. The Institute of Earthquake Engineering and Engineering Seismology (IZIIS), University of "Kiril and Metodij," Skopje, has conducted such investigations on many structures in Yugoslavia, constructed in different precast systems and different heights, from 5 to 21 storeys. In some cases, valuable data concerning the soil-structure interaction are obtained by these investigations. The results obtained are published in separate reports of the Institute. General conclusions cannot be made since all structures behave differently under dynamic harmonic excitations. For some of the investigated structures, linear mathematical models have been formulated. Beside these investigations IZIIS has carried out investigations of connections under quasi-static loads as mentioned under (1). Some results of the investigations being carried out on two typical systems adopted in Yugoslavia will be presented herein.

The first system which will be presented here consists of large panel members connected by shear keys with vertical and horizontal joints. Beside the shear keys, the connection between two panels of this system is provided by cast-in-place reinforced concrete columns. The extension of reinforcement within shear keys is done by welded joints. All types of joints for this system were tested, namely three elements of each joint. Figure 8 shows the configuration of one element under testing conditions. The loading was maintained by one direction force. Figure 9 shows the sequence of cracks for different levels of exciting force. It is obvious that during this test the shear capacity of the upper and lower panels was searched for the contact plain with the floor slabs. Figure 7 shows the stress-strain relationship for the joint represented in Fig. 8. From the beginning, up to the maximum force value, the stiffness changes almost linearly. Under a force of 105, ot (point A), the section is subjected to rapid yielding, while the member is unloaded to zero (point B). In this unloading a permanent deformation of 7,0 mm is achieved. From position B the member is loaded again up to the stage of complete failure (point C), when complete separation of members takes place. The second loading stage (B-C) shows considerably larger ductility.

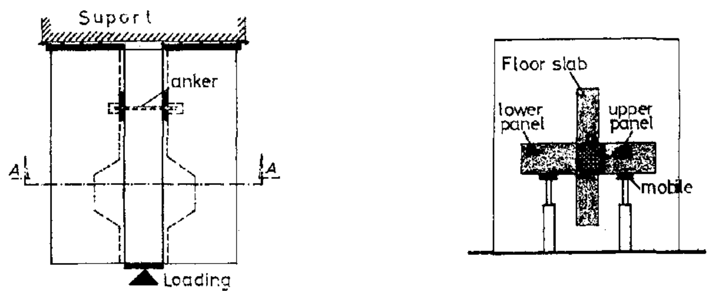


Fig. 8 Position and configuration of tested panels

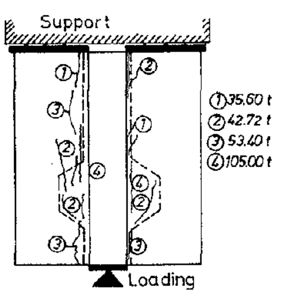


Fig. 9 Crack lines

The second system, the connections of which were experimentally tested, consists of large panel members framed by cast-in-place reinforced concrete columns and belt courses, Fig. 3. From many experiments carried out on this system, Fig. 10 shows the stress-strain relationship. The gravity load effect has been included also. In order to provide better connection between the vertical and horizontal panels, the contact surface was covered by an epoxy layer.

Based on the laboratory tests carried out in IZIIS as well as on the analysis made, it should be pointed out that the experimental tests of connections mentioned under (1) are the minimum required experimental investigations for definition of the

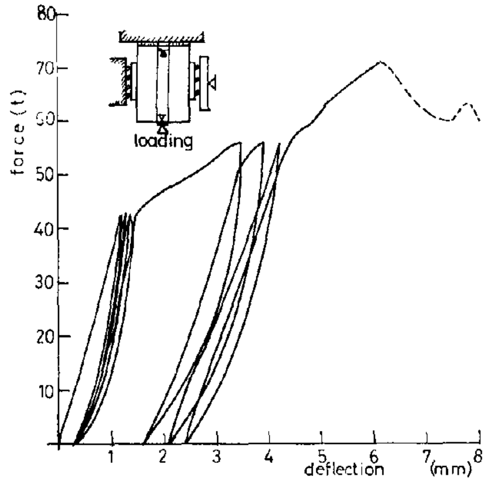


Fig. 10 Stress-strain relationship

stability of a structure constructed on the given system. For definition of the suitability for construction in seismic zones of a precast system, tests of multi-storey assemblages of connections and assemblages under quasi-static reverse loads are necessary.

3.5 Design Procedure

Figure 11 gives a block diagram of experimental and theoretical investigations necessary for defining the seismic stability of large panel systems.

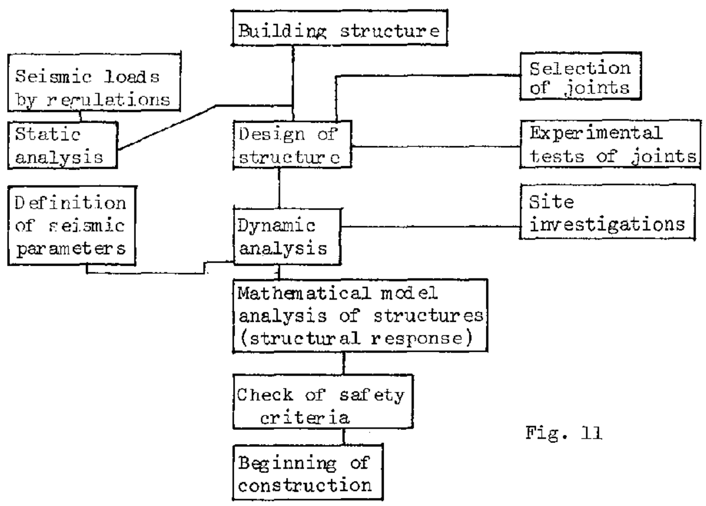


Fig. 11

4. Behaviour of Large Panel Buildings during the Romanian Earthquake of March 4, 1977

Our experience gathered from the failure of several prefabricated buildings caused by the Agadir earthquake of 1960 could give only little data concerning the behaviour of precast structures during earthquakes. However, a general conclusion is made that precast structures suffer more damage than monolithic structures.

We could say that up to the Romanian earthquake there were almost no data about the behaviour of precast structures during strong earthquakes. During this event the behaviour of the entire precast system in a wider range could be verified for the first time, this specially referring to large panel systems which in Romania have been applied as mass construction during the last twenty years. The large panel systems constructed on the territory of the whole country amounting to 120.000 - 150.000 apartments (out of which Bucharest has 75.000, Ploeshti 12.000 and Kraiova 6000 - 8000) give a good possibility for analysis of their behaviour, including parameters such as earthquake intensity, frequency content, soil conditions, height of the buildings, types of members, connections, and so forth.

The earthquake epicenter of the March 4, 1977 earthquake was on the slope of the Karpathain chain, at a depth of about 100 km and had a magnitude of 7.2 according to the Richter scale.

The failures and damage due to the earthquake effect were experienced in an area of 80.000 km^2, which is 1/3 of the whole Romanian territory. Also, some destruction is evident in Bulgaria along the river Danube. The earthquake was felt in Yugoslavia, too, with different intensity (Fig. 12).

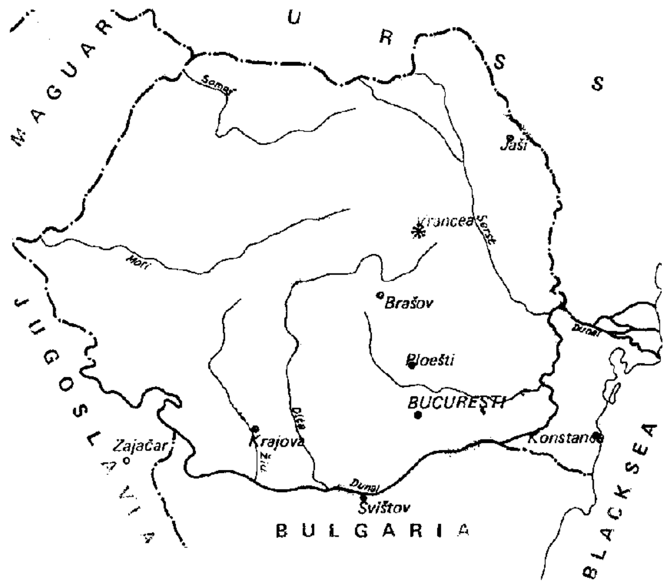


Fig. 12 Map of Romania showing the region affected by March 4, 1977 earthquake

The geological, geophysical and geomechanical characteristics of the territory, as well as the large energy released in the epicenter, clarify the destruction and damage of such a vast area.

In Bucharest, a ground surface acceleration of 0.20 g (component N-S) was recorded by instrument SMAC, while at a distance of about 700 km in Nis, Yugoslavis, ground acceleration of 0.04 g E-W component was recorded by SMA-1 instrument. Intersting to be mentioned here is the frequency content of this earthquake if it is compared to some other earthquakes (Fig. 13).

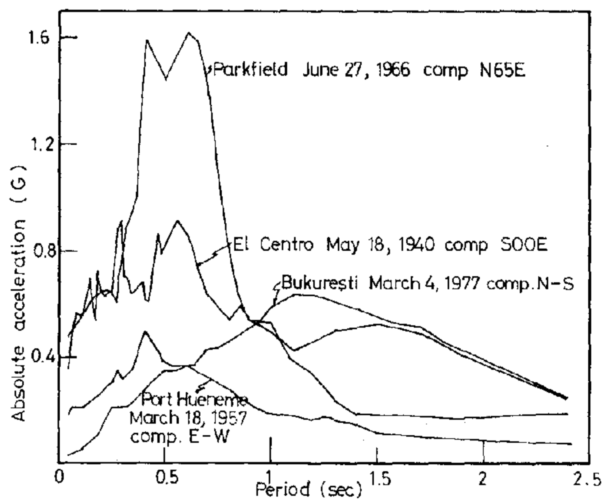


Fig. 13 Absolute acceleration response spectra for damping 5% of critical

The behaviour of different structural systems during the earthquake could be summarized in general as:

- Slender reinforced concrete frame structures with brick masonry infilled walls, constructed between 1930 and 1940, without earthquake resistant design requirements, with low quality characteristics of concrete, insufficient percentage of reinforcement and unfavourable structural composition. About 30 structures of this type failed while many of them were badly damaged.

- Structures constructed during the last twenty years in monolithic reinforced concrete systems: bearing walls, infilled frame systems and composite systems consisting of frames, shear walls and bearing walls. These systems give relatively good performance, showing different types of damage which are mainly cracks which corresond to their postelastic behaviour. It should be mentioned here that the first codes of Romania were enforced after the Romanian earthquake of November 10, 1940, while the contemporary regulations based on spectral analysis were brought in in 1963.

- The experienced behaviour of the precast structural systems was satisfactory, above all expectations, in spite of the different qualities of construction of different types of structures.

The large panel structures were introduced in Romania about twenty years ago. A priority was given to these systems during the last ten years, so today about 75.000 apartments have been constructed in Bucharest in this system of 5 to 11 storey height. In Ploeshti, which, according to the seismic zoning map, is included in the zone of higher seismic intensity, the number of stories is limited to 5 storeys.

According to European practice, Romania has adopted the "two-way" system. Usually, all the panel walls both internal and facade are bearing walls. In Bucharest, there is a panel system constructed 15 years ago, of eight storeys, the external walls of which are not bearing walls. The foundation structure, as well as the basement, is monolithic. An exception to this is a 10 storey building in Bucharest which has as precast basement on precast foundations. The first slab above the basement was constructed differently, both monolithic and precast; however, in Ploeshti it is almost always a monolithic structure.

The connections of panels both horizontal and vertical are usually wet connections placed in concrete in situ with welded anchor reinforcement, which is characteristic of European systems.

Structural systems are mainly designed and analysed according to the existing aseismic regulations, applying static methods for definition of the static values while the stresses are defined for ultimate stress states.

The principal structural characteristics of the systems used are as follows:

1. Two-way system of 8 storeys and non-bearing facade panels. These structures have no basement and the prefabricated system is placed on monolith foundations. It was constructed in series some 15 years ago in Bucharest. It is solved with monolithic horizontal and vertical joints, welded reinforcement of vertical panels and monolith panels above the last precast structure. Figure 14 shows some details of this system.

2. Two-way system of 10 storeys with basement. It is a precast basement structure on monolithic foundations. The system is constructed in series in Bucharest and its use will continue in the future, despite the recent earthquake event. Figure 15 shows some details of this system.

3. Many structures in Bucharest have 5 storeys and monolithic basement. Figure 16 gives details of some members and connections of this system. It should be mentioned here that all panel systems in Bucharest have a shear base coefficient of 7-9%, up to ultimate state.

4. In Ploeshti, which is closer to the epicentral region, construction of large panel structures is limited to 5 storeys. They always have monolith basements with monolith floor slab above it. The walls are in two-way system. Figure 17 gives details of the most frequent type of system used. It should be noticed here the enlarged section of the monolithic column in order to increase its shear strength. This system was previously constructed without this enlargement for shears. The base shear coefficient is 15%.

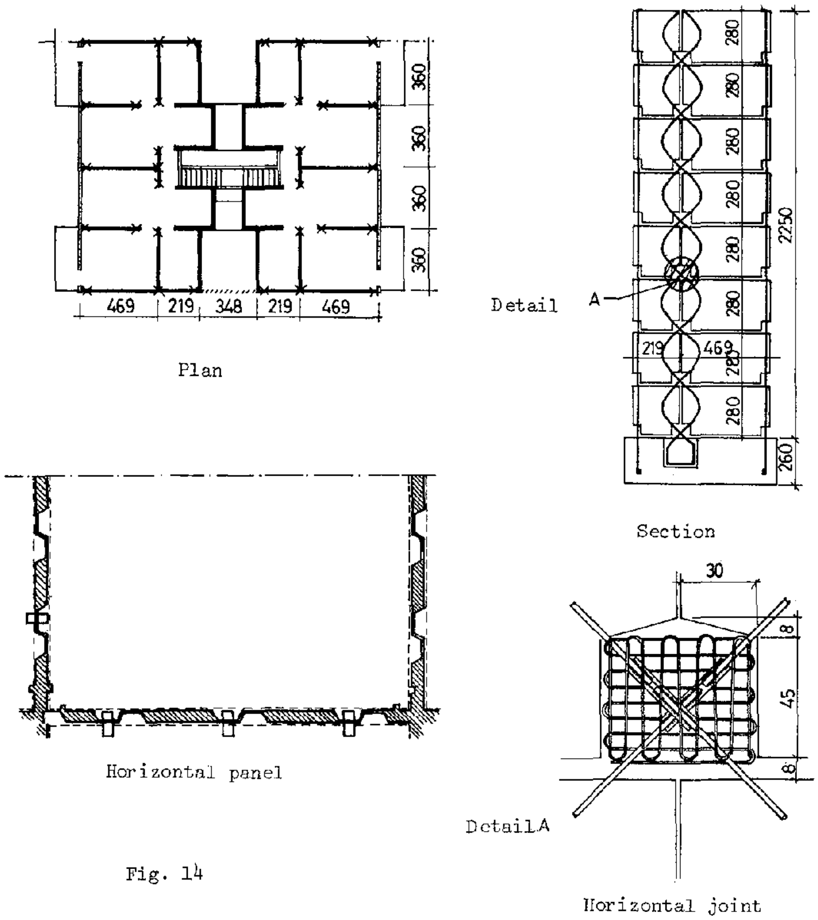


Fig. 14

In Kraiova, buildings with the same design, same number of storeys and similar solutions are constructed. The plan and reinforcement distribution are given in Fig. 18. These structures have a shear base coefficient of 10-12% up to the yield point in bending of the reinforcement.

5. In Kraiova a large panel system is both prefabricated and constructed. It is a 5 storey system of box type, i.e. complete room. Each apartment consists of 3,5 boxes monolithically connected in situ along the edges by welding and placed concrete. Each corner is then prestressed by vertical cables of 12 t (wire of 7Ø4 mm) along the height of the building, The prestressed cables

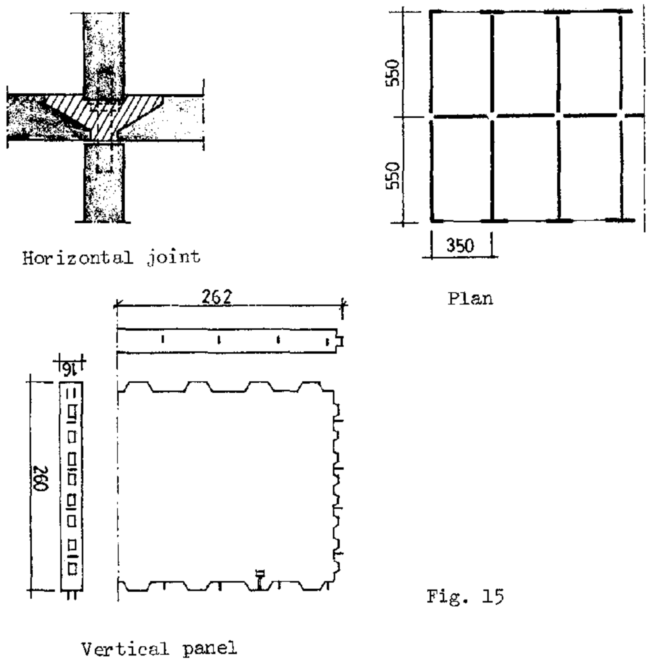


Fig. 15

are then anchored to the monolithic basement walls which were constructed in situ together with the foundation. Figure 19 shows details of this system.

In all Romanian towns all the large panel structures behaved very well and generally speaking, they did not experience any significant structural damage.

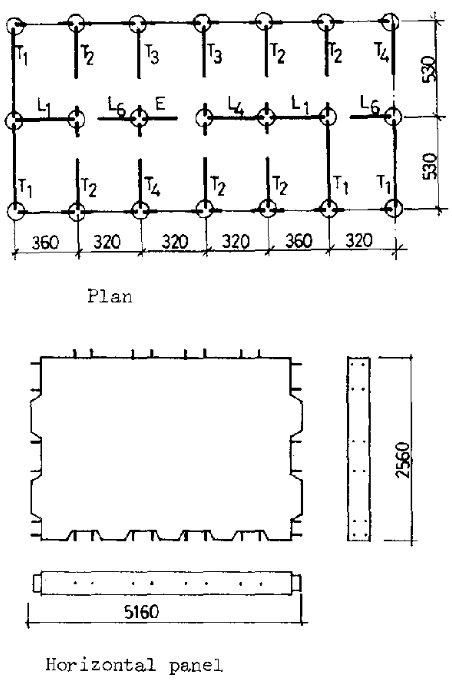
The overall performance of members within the structural systems was qualified as:

- Damage of the foundation structure has not been observed.

- Horizontal panels performed as horizontal rigid diaphragms, without damage

- In vertical panels there are no observable cracks. An exception is a building in Kraiova where several longitudinal internal panel walls (without openings) developed fine vertical cracks.

- Also, in some structures on the first and second floors there are shrinkage cracks in joints in the contacts between the concrete placed in situ and the panels which specially refer to the vertical joints,



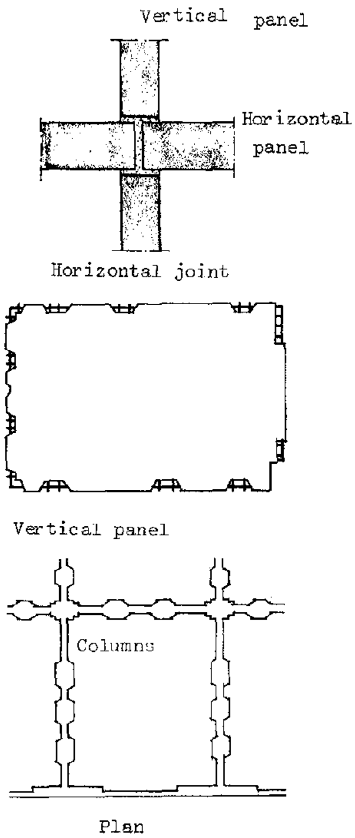


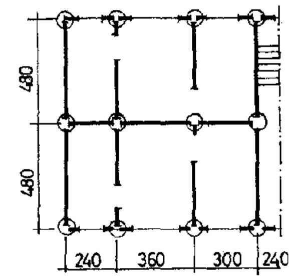
Fig. 16

especially in the flanged joints. The order of these cracks is from 0.1 to 0.3 mm, rarely bigger than that. The cracks are mainly concentrated on the first and less on the second floor and as a rule in the intermediate infilled panel walls.

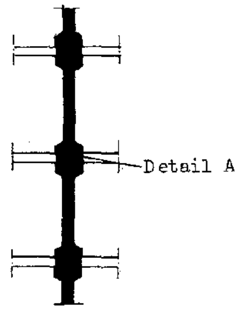
- The horizontal joints occasionally develop cracks close to the place where vertical cracks appear and stretch 1 to 2,0 m from the contact edges towards the middle of the room.

It should be mentioned here that such cracks in the vertical joints close to horizontal cracks are observed on on much smaller number of structures, regardless of the system and location (Bucharest, Ploeshti, and Kraiova).

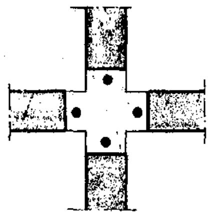
- Sometimes, very fine cracks appear in the connection with prefabricated staircase.



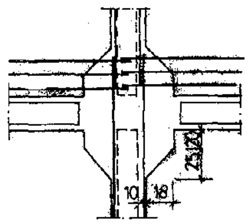
Plan



Vertical section of column

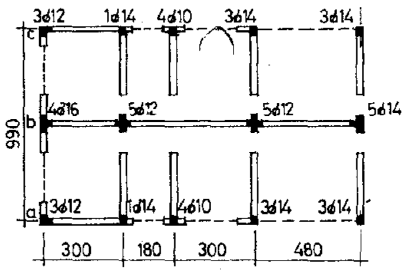


Horizontal section of column

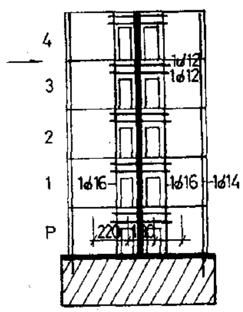


Detail A

Fig. 17



Plan



Section

Fig. 18

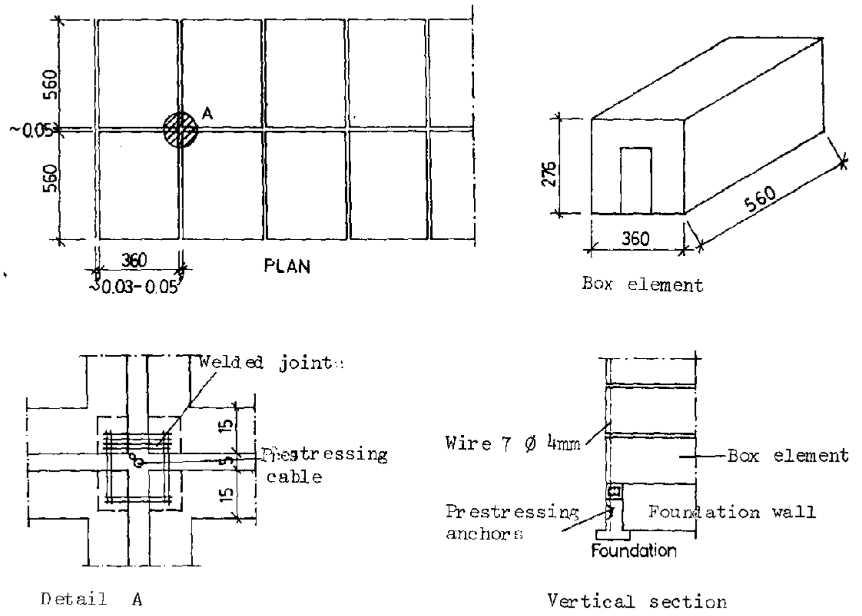


Fig. 19

- An interesting case observed in Kraiova should be mentioned here, namely in some structures the reinforcement is anchored to the belt course at the level of the floor slab above the basement. In such a structure, there was a case of a joint opening under the first slab in the place where the column reinforcement was anchored.

The satisfactory performance of the system, as compared to other systems, during the March 4, 1977 earthquake can be explained by:

- High base shear coefficient as compared to the predominant natural dynamic characteristics of structures, soil conditions and the type and intensity of the earthquake motion - frequency content.

- Sufficient number and favourable distribution of the panels in the two-way system.

- High level of the cast-in-place of connections, the required length, which provides sufficient monolithic effect regardless of the poor quality of construction.

- The whole building worked as a box system with a capacity for energy dissipation in the ground at the soil-foundation level.

- Possibilities for bigger damping of the whole system due to joints.

- Energy dissipation in the fine cracks on the contact in vertical and partly in horizontal joints in the zones of shrinkage cracks.

- The quality of concrete is much better than in the case of monolithic structures even if there are some faults in the cast-in-place joints and welding of reinforcement.

CONCLUSIONS AND RECOMMENDATIONS

The large panel structures are usually constructed in large series according to the adopted technological solution and similar design. Therefore, for definition of the seismic stability of the system,complete experimental and theoretical investigations should necessarily be carried out.

The mathematical model formulation needs much more study both for linear and nonlinear range. Simplification is usually introduced by secant stiffness. The deformable capacity and ductility are closely related with the connection system applied. As it is well known, some experimental studies have already been carried out; however, there are no sufficient data which would explain the behaviour of all types of connections. The performance of joint types: welded bolted, post-tensioned are less interesting than the cast-in-place wet joints. For complete understanding of the performance of a structure, the behaviour of joints should also be defined. Therefore, at least minimum experimental investigations of joint characteristics are necessary.

The opening effect of joints, i.e. development of cracks in joints, is very important as well as the degree of participation of the flange walls in resisting lateral loads. The local damage of a structure due to earthquakes could endanger the whole stability of the system. In that case, joints become critical points for development of progressive collapse. This problem should be treated in the design of structures. The soil-structure interaction is very important in the aseismic design. Foundations should, through their proportions and solution, make use of the shear base capacity of the structure.

In the aseismic design standards, these structures should not be regarded as identical with monolithic ones. If equivalent earthquake loads are in question, they should be larger than those for ordinary monolithic structures.

Mathematical models should be formulated according to extensive experimental research. Theoretical structural analyses should be as detailed as possible. Advantage has the three-dimensional space analysis for both static and dynamic effects.

Nowadays, the finite element method is the most frequently used for the formulation of mathematical models.

The large panel system is extensively used all over Europe today. The satisfactory performance of these structures during the Romanian earthquake would only contribute to wider application of this system, even for taller buildings in seismic zones.

However, these conclusions should not be generalized since real behaviour of structures during earthquakes depends upon the earthquake intensity and frequency content, the soil conditions and the structural parameters.

Having in mind that, for the first time, such a big zone covered by large panel systems with over 150.000 apartments was affected by a strong earthquake, an international research project which would investigate the behaviour of these structures during strong earthquakes is necessary, and would enable elaboration of recommendations and instructions for aseismic design of large panel systems in the future.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

EARTHQUAKE RESISTANT DESIGN OF PRECAST CONCRETE BEARING WALL TYPE STRUCTURES - A DESIGNER'S DILEMMA

by

VILAS S. MUJUMDAR
Principal Structural Engineer

ECODYNE CORP/ECODYNE COOLING PRODUCTS DIV
Santa Rosa, California

INTRODUCTION

The material presented herein is limited to precast concrete bearing wall type buildings. Furthermore, it is limited to the construction methods and the types of details most commonly employed in the United States and Canada. It is felt that the layout of the buildings, construction methods, economics of construction, and methods of joining precast concrete components in the United States and Canada are significantly different from European practices, and, therefore, the emphasis is on North American practice.

The majority of the precast concrete bearing wall buildings in this country are constructed for well defined compartmentalized use, such as apartment complexes, housing for elderly, hotels, motels, etc. By the necessity of space planning, these buildings tend to be what is commonly known as "double loaded corridor type," which means the plan layout is rectangular in general with length to width ratio varying from 2 to 5 (Figure 1). High rise buildings in congested metropolitan areas have also used a "pinwheel" type design (Figure 2).

Some isolated projects such as Habitat '67 in Montreal, Canada and the San Antonio Hilton in the U.S.A. have used volumetric concrete units with most of the finishes, including interior decorations, completed before erection and placement of the modules in their final location. The author was actively involved in Habitat '67 project.

Some other systems utilize combinations of walls, volumetric utility modules, and other specially designed components. Recently active interest has been generated in tilt-up wall panel construction. These wall panels are site fabricated and tilted-up to their final location. The advantage being that the large surfaces can be cast since no transportation and erection restrictions apply. Usually this type of construction is limited to exterior wall panels and has not found wide applications in structures other than warehouses, low rise office buildings, and buildings of similar nature.

However, the most commonly designed and constructed buildings are "double loaded corridor type." These buildings could be further divided into two main categories, namely: 1) cross wall type (Figure 1), and 2) long wall type (Figure 3). These are briefly described below.

1. Cross Wall Type - Bearing walls are normally spaced from 22' to 36' on centers along the long direction of the building. The walls across the corridor usually are not connected for any designed lateral force transfer. In that regard, they are not coupled shear walls although some coupling action exists. The prestressed concrete floors of the extruded type span the walls and are supported directly on them with a plastic bearing strip to prevent the chipping of edges and to allow small rotational deformations. The lateral force resisting system in the long direction is usually provided by a combination of staircore units, elevator shafts, and possibly some shear walls if required. The external skin of the building is usually architectural in nature and non-load bearing. Bricks and metal skins are used frequently except where precast concrete parapets and balconies are provided. Sometimes architecturally treated concrete wall panels are also used.

2. Long Wall Type - The plan layout of this building is essentially the same as described under the cross wall type. However, the structural components are arranged differently. Usually, a wall along one side of the corridor runs the length of the building. This wall, of course, is in several room size sections and contains openings for doors as well. The framing at the exterior long edges of the building consists of either beams and columns, other precast walls, or special units. The floor spans between the corridor wall and exterior framing and may cantilever out for balconies, if required.

Although most of us are familiar with all of the above descriptions and are available elsewhere, these are made a part of this paper to re-emphasis the layout of these most commonly designed and constructed buildings in North America and to try to understand their behavior.

SOME IMPORTANT CONSIDERATIONS

Slenderness, Seismic Zone, and Codes

Before one can establish a design criteria, it is necessary to take into consideration the height of the building, seismic zone, and regulatory code.

The height of the building is important because it influences the behavior of shear wall. Let us consider a typical 24' long wall in an apartment building. The height/length ratios of this wall are as follows and consequently the mode of behavior of this wall under lateral loads is different.

	$(H)^2$ ft.	L (ft)	H/L^1
3 Story Building	26.01	24	1.08
6 Story Building	52.02	24	2.16
12 Story Building	104.04	24	4.34
18 Story Building	156.06	24	6.50

It can be observed that as the height of the building increases, the flexural behavior of the wall influences the design. Whereas in the low rise buildings, it is the shear behavior which predominates. Perhaps it is more appropriate to concentrate on the slenderness ratio of the lateral load resisting system. While these concepts hold true for a monolithic wall, what concepts of behavior should one apply to precast concrete bearing wall buildings with the full knowledge that joint-slip occurs and may be particularly important where H/L ratios are low as compared to more slender walls for the same given lateral load at a joint? Do we need to provide vertical connections between walls uniformly along the length to reduce slip and provide adequate shear friction reinforcement rather than connections only at ends?

When one talks about seismic loads, immediately a designer's mind is focused to high-seismicity areas. One must, however, recognize that in zones of medium to low seismicity, the design provisions could be modified in terms of ductility and perhaps several other requirements.

The code of practice most widely used is either the Uniform Building Code or variations of it. This code does not address itself to precast concrete buildings. In fact, very little guidance is available to the designer and at present all the designs are accomplished assuming the walls to be similar to cast-in-place concrete walls.

Wall Behavior

Since the basic criteria behind the UBC code is high seismicity regions, the requirements call for ductility. However, very little about the wall ductility is known and it depends upon which mode of shear wall behavior predominates, i.e. if the shear deformation is significant (some authors suggest 10% of the total deformation as significant (2), then the shear wall is a non-ductile element. On the other hand, if the flexural deformation is the primary mode while storing flexural strain energy, then the wall behaves like a flexural beam and could be made a ductile element if properly reinforced.

How does one provide ductility in precast concrete shear walls? If the ductility is defined as the ratio of total energy capacity of a member to its elastic energy capacity, can one achieve ductility through shear strain energy capacity rather than flexural strain energy if that is the primary mode of behavior? Ductility depends on several factors and can be conviently written as:

$$u = F\ (ka.\ p.\ Ks.\ Kc.\ \rho p)$$

where $Ka = \frac{Pe}{Pb}$ ratio of design axial load to axial load capacity at balanced condition.

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1. The slight error in these figures is due to rounding off to the second decimal.

2. Typical floor to floor height 8' 8".

p = ratio of non-prestressed steel.

Ks = shape factor for wall.

Kc = coefficient based upon design concrete strength.

ρ_p = ratio of prestressed reinforcement.

u = ductility.

F = Function

While the effect of each of the above factors could be further expounded upon and explained in detail, it is not the objective of this paper. They are listed here merely to draw attention to the numbers of variables involved in determining the ductility of a given shear wall.

At least two other variables have to be added to the precast concrete walls. 1) Effect of joint and joint material, and 2) effect of discrete connections. Then, for precast concrete walls,

$$u_p = F\ (Ka.\ p.\ Ks.\ Kc.\ \rho p.\ Jc.\ Jh.)$$

where

u_p = ductility for precast concrete walls.

Jc = coefficient dependent upon joint material and joint thickness.

Jh = coefficient dependent upon the hardware material, number of connections, and anchorage details.

F = Function.

Rest of the factors are same as in above.

Design Criteria

Finally, when a design engineer faces the problem of designing an actual building, he has to use some code of practice as dictated by a particular regulatory authority. Since most codes for seismic design are based on UBC, let us consider the UBC as our basic code of practice. Some of the questions that arise in a designer's mind are listed and briefly discussed below.

Wind or Earthquake

While this might sound like a relatively simple decision to make, let us ponder on it for a while. If a building is long and narrow like the typical building in Figure 1, consider two heights: 1) 8 story building, and 2) 18 story building. Buildings to be designed for zone '1' earthquake and wind pressure corresponding to zone '25'.

In an 8 story building you will find that the earthquake forces govern in both directions. Whereas in an 18 story building, you will note that earthquake forces govern in one direction but the wind forces govern in another direction. Can we design the building on this basis? We know that in a real earthquake the intensity of loads will be several times higher than code specified forces.

If, as a prudent design, the building is designed for earthquake loads in both directions, can this be economically competitive with the design of others who strictly follow the code? While the wind force is a function of shape and geometry of the building for a given location, the seismic loads are essentially "inertial" in nature and depend on the mass of the structure for a given location and framing system. Although a lateral load resisting system is symmetrical, due considerations must be given to forces generated due to torsion in seismic loads.

Base Shear

One of the basic quantities a designer establishes is the base shear, assuming of course that the mass is calculated. Base shear is defined by the code as:

$$V = ZKICSW$$

'Z' and 'K' are well defined for a given seismic region and framing system. Assuming ours to be an apartment building, $I = 1.0$ and let us say $S = 1.5$ for simplicity. 'W' can be accurately calculated. That leaves us with determination of 'C'. According to UBC 1976, 'C' is as below:

$$C = \frac{I}{15\sqrt{T}}$$

and

$$T = \frac{0.05hn}{\sqrt{D}}$$

This is where the problem starts. Due to the presence of joints, is the formula applicable to precast shear walls? What influence, if any, do the joints and connection hardware have? The connecting hardware, joint material, and wall material all have different properties. How does this affect the fundamental elastic vibration period? All of us assume the wall to be a continuously connected homogeneous member for determing 'T'. We don't know whether we are on the conservative or non-conservative side!

Distribution and Transfer of Forces

Having determined the total base shear based on the assumption of monolithic construction, one is faced with distributing and transferring these forces to various elements and ultimately to foundation systems.

According to the current code practice, the distribution of forces over the height of the building results in a triangular shaped force diagram (see Figure 4) with a possible "whiplash" force at the top of the building if 'T' is greater than 0.7 secs. (1976 UBC Code). This "whiplash" force was related to the slenderness of the lateral force resisting system in earlier codes. In the 1976 code, however, it is related to 'T' which is a function of plan dimension of the total building rather than the lateral force resisting system. The additional force at the top is based on higher deflections and therefore higher accelerations at the top.

Are the assumptions of the linear distribution and additional forces only at the top correct for precast concrete bearing wall buildings? Does the period and load distribution change with the rapid joint degradation and as the higher modes of vibration become significantly important? If this is true, what is the additional force at several other levels and what redistribution takes place? Does uniform shear wall, in effect, become a tapering cantilever with progressive opening of joints? What effect does the joint-slip have on redistribution of forces? These questions at present do not have answers.

Joints, Interconnections, and Cyclic Loading

Joints are least understood and mostly ignored by the design engineers, and yet it is the most important aspect of precast concrete buildings. Two specific joints I would like to concentrate upon are: 1) bearing joint, and 2) horizontal joint between floor units. There are, of course, several other joints of importance, but let us consider these two joints only at the present time.

Bearing Joint - This most commonly used joint is shown in Figure 5. At one particular section of this joint as many as 5 - 6 different materials may be interacting. At the very least, the load transfer mechanism is complex. Research work is being carried out at PCA to study the behavior of such a joint under lateral loads with various levels of precompression. Some work, theoretical and physical testing, has been accomplished to determine the vertical load carrying capacity of such a joint.

Horizontal Joint Between Floors - The horizontal joint between precast prestressed concrete floor members (Figure 6) is of prime importance, particularl if no topping concrete is used. In many buildings the topping is ommitted for economic reasons. This joint has interface forces due to diaphragm action which need to be transferred to the shear walls. Almost always, this joint, in the form of a key, is grouted along with a rebar in it. The rebar spans over the wall to form a tie between adjacent span floor members. Since the grout is fluid with a very high water/cement ratio and the floor planks are virtually dry, they absorb water from the grout and create a definite crack along the joint. The bond characteristics of the rebar in the narrow key, at best, are intutive in nature.

The differential camber between adjacent units, particularly around the stairs, elevators, and at the roof level, further aggrevates the problem. What design forces can one assume for this joint? In other words, what is the capacity of this joint?

Connections

Primary connections are used for shear transfer from floor systems to walls and for vertical continuity between walls. Shear from the floor diaphragm is transferred to walls by gravity friction, shear-friction mechanism, clamping actions, and sometimes by direct bearing on concrete. Due to the opening of the bearing joint, particularly at lower levels, the shear force cannot be transferred uniformly along the length of the wall. Also, due to rocking motion (5), some crushing of concrete and joint material is expected. How much length of wall is really effective in taking shear forces and finally transferring it to the foundation system?

Vertical continuity is normally achieved in two ways: 1) by bolted connections, usually only two per wall, located near the end of the walls, and 2) by post-tensioning bars, also usually only two, near the end of the walls.

In case of bolted connections, the yielding of the bolt and possible failure in tension or compression has to be considered. But remember, in its entire length the wall is usually connected only at two locations to the wall above and below. Is this correct? Do we need additional connections in the middle region of the wall or uniformly spaced connections over the entire length? How does the shear friction mechanism work with only two bolts, particularly where shear deformations are important and joint slip has to be taken into account?

Where continuity is achieved through post-tensioning, two methods prevail: 1) ungrouted bars, and 2) grouted bars. The primary reason to use post-tensioning is to at least nullify the tensile stresses under the code specified loads. The bars are typically stressed to 70% of their ultimate value. Considering the intensities of forces under a real earthquake, does this level of prestressing force make any sense? Should we be stressing the bars to only, say 40% -50% of the ultimate capacity and use a larger number of bars? Is the post-tensioned bar fully or partially effective as shear friction reinforcement?

Since the failure of anchorages (pulling out) during the 1964 Alaskan earthquake, the unbonded post-tensioning system has been heavily criticized. However, today most of these anchorages are direct bearing type and do not depend upon the strength of surrounding concrete for anchorage strength as in the case of conical spiral anchorage. The anchorages are located, however, in the very region where reverse load cycling is most intense, i.e. within the joints. Maybe we should work towards post-tensioning only every three floors and stagger the anchorages (Figure 7). How does an unbonded bar behave after its low level of prestress is nullified? In case of bonded construction, what is the level of force at which the bond becomes ineffective? The bar then behaves as in unbonded construction. It is extremely difficult to achieve any confining reinforcement around the post-tensioning bar since it interfers with the wall reinforcement during production process and the labor costs are much higher.

Horizontal Shear Diaphragm

For distribution of lateral loads, it is assumed that the floor diaphragm is rigid. However, several questions must be raised. In case of concrete topping, normally 2" thick with only temperature reinforcement, many designers assume that the topping thickness is the depth of the diaphragm and that it may not be rigid. In untopped as well as in topped construction, the floor system is full of air voids without any reinforcement in the top face of the extruded floor plank. This is the most widely used floor member. What is the strength of such a diaphragm and is the assumption of rigid diaphragm true? Do we need chord reinforcement in each key joint or only at the edges of the total depth of the diaphragm in case of untopped construction?

What forces in addition to shear forces are developed at the diaphragm-wall interface due to rocking, opening of joints, and yielding of post-tensioning or other connecting hardware? What is the effect of all of these forces on the bearing ends of precast concrete planks? Will they eventually become unstable and fall down due to narrow bearing widths? How can we increase the bearing width if necessary?

Energy Dissipation

Since not much information is available on this subject, designers don't understand how this is achieved and hence they tend to ignore it. The important consideration should be that "energy is dissipated" and not "absorbed" and thrown back into the system. Some work on understanding this mechanism is being conducted at MIT (5).

OTHER DESIGN CONSIDERATIONS

Foundations

Many bearing walls are supported on individual strip footing with ties between the footing and first story walls. Since the walls are long and rigid, the concept of a foundation trying to hold the wall down may no longer be true. In fact, the footing wall may become part of the shear wall and the foundation rotations higher than expected may result thereby aggrevating the seismic loads.

Other Support Systems

In several designs, shear walls are discontinued at the first story level to create large open areas and are supported on column-beam frame. While this may provide some "soft story" advantages, my concern is about the load transfer mechanism between the frame and the wall system above. There is arching action (Figure 8) and the beam ends have higher bearing stresses (Figure 9). In addition to this, the stresses due to seismic loads must be added, particularly the high impact compressive forces as a result of rocking motion. This support mechanism must be well understood and properly designed.

Torsion

The earthquake forces are supposed to be acting in any direction and it is only for the convenience of the design that the force components are resolved in two perpendicular directions. A designer must always keep that in mind. All elements in the plan layout of the building participate in resisting torsional forces. Due to this, some elements could develop "out of plane" instability. All elements must be well tied together. These forces should also be considered while developing connections between floor diaphragm and walls.

Vertical Acceleration

This is never considered by the designer since no guidelines are available. However, vertical accelerations in excess of 1.0 g have been observed in some European earthquakes. Due to the componetized structure of the bearing wall building, this consideration is very important.

Ductility/Flexibility

Many of us get confused by the two terms and think that they mean the same thing. In fact, the meanings are quite different. Ductility is the ability to undergo large strain deformations without breaking, e.g. gold. Whereas flexibility is the ability to bend without breaking up to a critical load limit. Once that limit is exceeded, a sudden brittle failure occurs, e.g. fiberglass.

Whenever in Doubt - - - -

As is most common in the everyday design practice, an engineer will design conservatively whenever the forces are not properly understood. While this may turn out to be a good practice in several designs, in some areas of seismic resistant design it is not advisable, e.g. connections of exterior precast non-load sharing members could be over designed, thereby providing extra strength but little ductility and thereby generating discrete areas for brittle failure.

Economic Considerations

Finally as a fact of life in everyday design, precast concrete bearing wall structures have to compete with other materials and particularly with reinforced hollow block masonry. Not many questions arise in designing the hollow masonry walls since they could be effectively tied vertically and with the floor members. These could be designed like other continuous shear wall structures.

The author has designed over 4000 housing units utilizing precast concrete bearing walls, volumetric units, combinations of shear walls, and special units. A majority of these buildings are in seismic regions. We had to justify the use of precast concrete bearing walls as compared to reinforced masonry based on economics. If the precast concrete bearing wall buildings are going to be economically competitive with other materials, we as designers must understand their behavior to exercise best engineering judgement in producing economic design. The precast concrete industry must be willing to work with designers and researchers in developing and using new products, sound concepts, and different connection methods.

CONCLUSIONS

Understanding the behavior of precast concrete bearing wall buildings is very complex and much more research must be done before designers can fully comprehend the problems involved. Vertical continuity and tieing of all the components is extremely important. Joint behavior and its overall effects needs to be studied carefully. Structural layout considerations Figure 10 should play a more important role in the overall building design. New code specified load levels may need to be generated for these buildings. An understanding that the seismic forces are dynamic in nature and only "static" in codes for simplicity has more significance in multi-jointed precast concrete bearing wall type structures as compared to monolithic concrete structures. Until a final recommendation is available, the designers need to be extra cautious while undertaking design of these buildings. Finally, with the evidence of excellent behavior of shear wall building in recent earthquakes, the precast concrete bearing wall buildings may also find wide application in seismically active regions, if properly designed.

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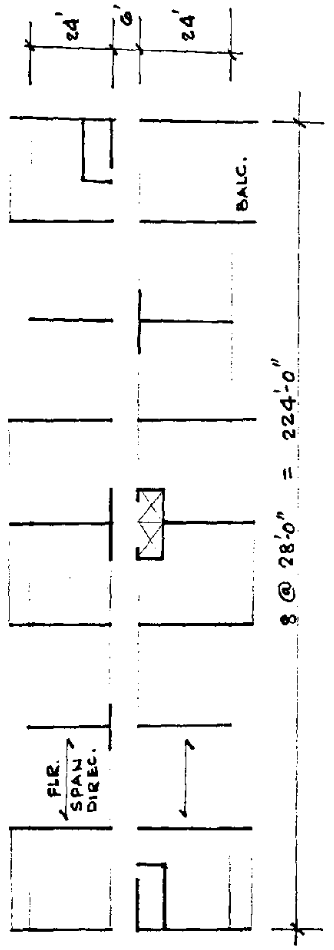

FIG. 1

TYP. CROSS-WALL BUILDING

(ONLY STRUCT. SHEAR WALLS SHOWN).

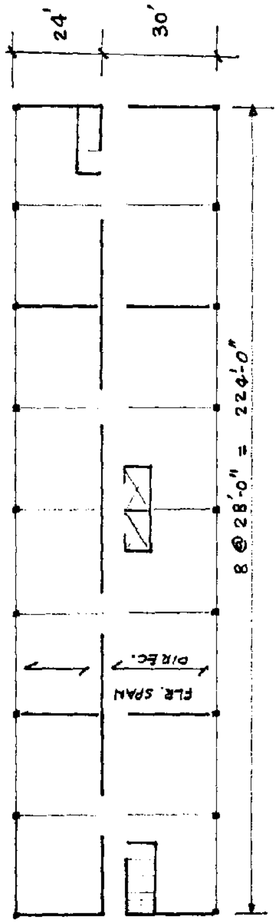

FIG. 2

TYP. LONG-WALL BUILDING

(ONLY STRUCT. SHEAR WALLS SHOWN).

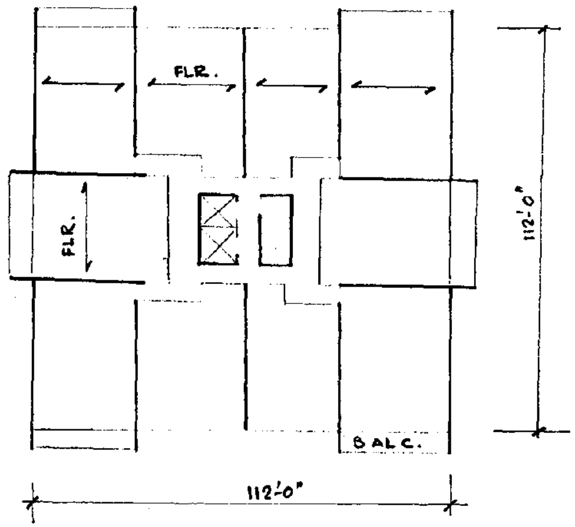

FIG. 2.

TYP. PIN-WHEEL TYPE.
(STR. SHEAR WALLS SHOWN DARK).

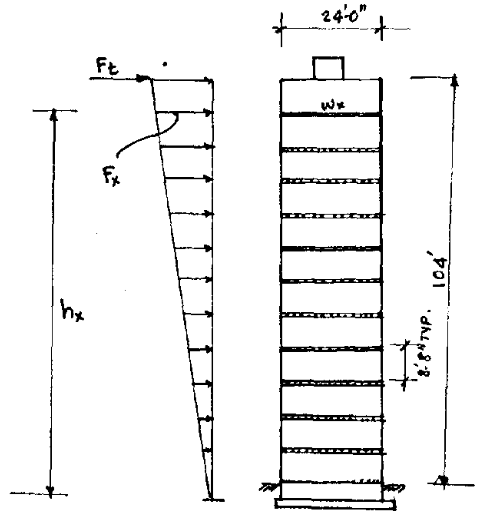

FIG. 4.

TYP. SHEAR WALL
E.Q. LOAD.

BASE SHEAR $V = ZKCSIW$.

$F_t = 0.07TV$ (0.25V max.)

$$F_x = \frac{(V - F_t)\, w_x h_x}{\sum^n w_i h_i}$$

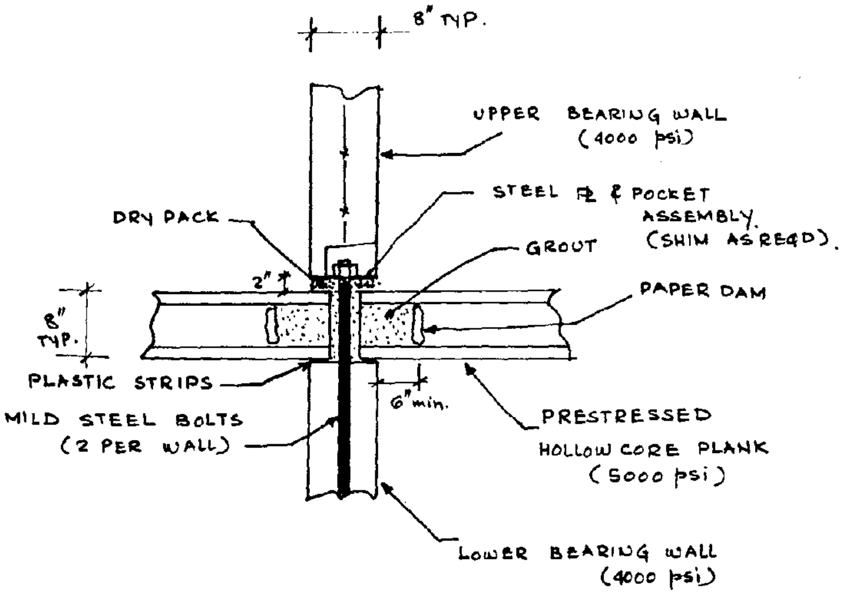

FIG. 5.

TYP. BEARING JOINT (NO TOPPING).

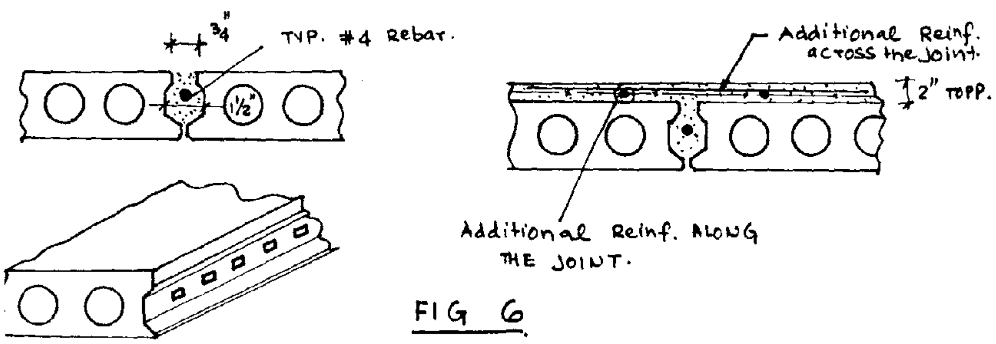

FIG 6

TYP. HORIZONTAL JOINT WITH & WITHOUT TOPPING.

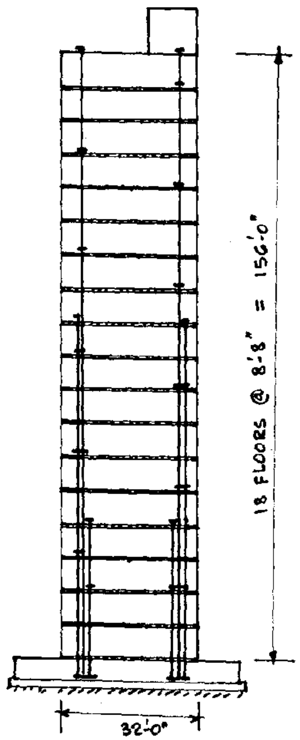

FIG. 7.
STAGGERED POST-TENSIONING ANCHORAGES

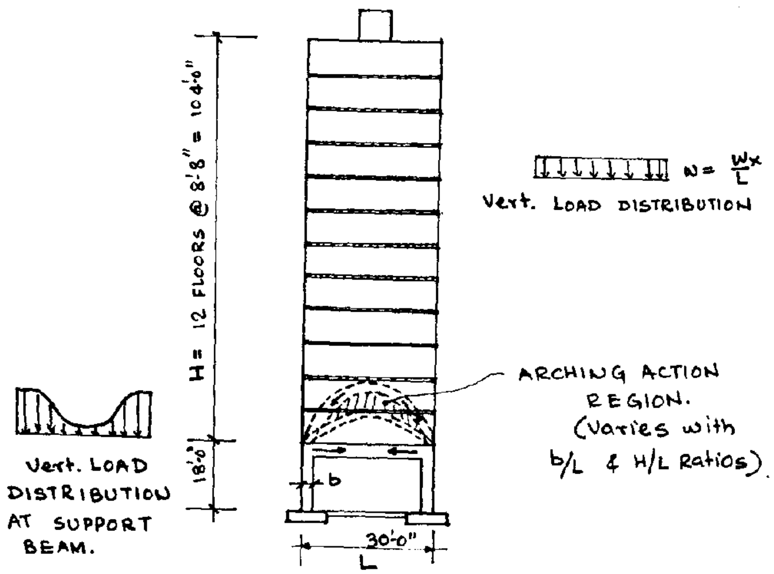

FIG. 8
ARCHING ACTION

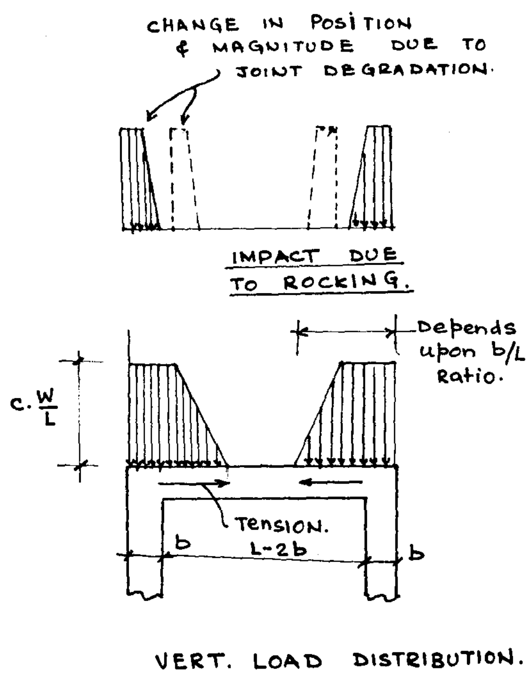

FIG. 9.

BEARING STRESSES AT BEAM ENDS, ALONG WITH SHEAR & TENSION.

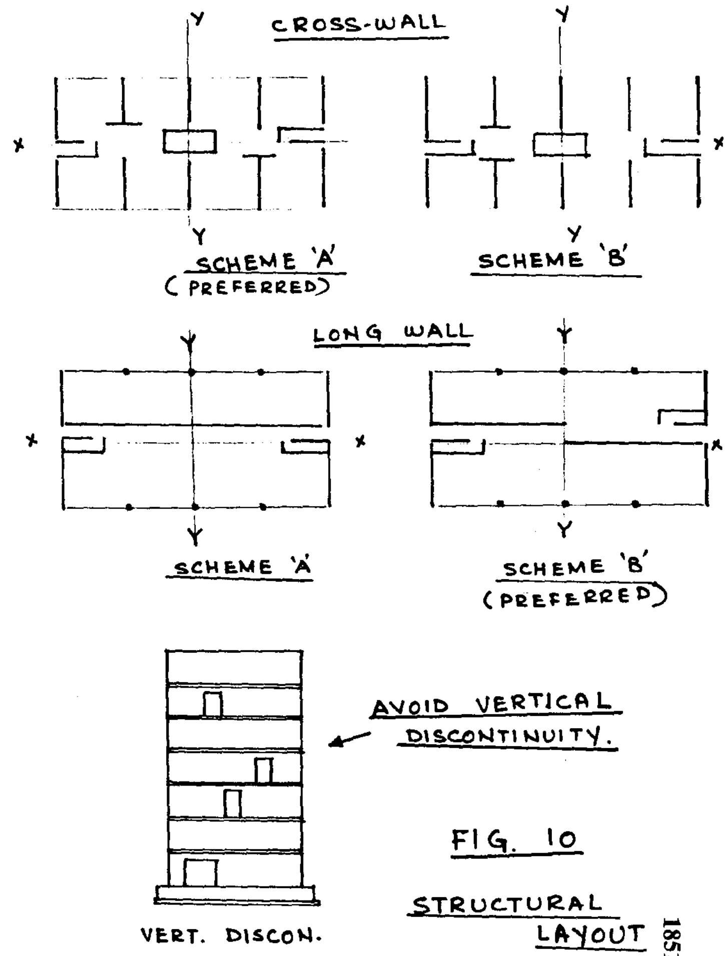

FIG. 10

STRUCTURAL LAYOUT

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

SEISMIC RESISTANCE vs. PROGRESSIVE COLLAPSE OF PRECAST CONCRETE PANEL BUILDINGS

by

G. Robert Fuller
Structural Engineer
Office of Housing, U.S. Department of HUD

INTRODUCTION

There are certain design compatabilities which exist between seismic considerations and the concept of "general structural integrity" or resistance of buildings to progressive collapse. In addition, multistory precast concrete bearing wall buildings are subject to weaknesses with regard to seismic resistance as well as progressive collapse. However, there is very little research on the general structural integrity of precast concrete panel buildings as evidenced by the draft preprints of this Research Workshop on Earthquake Resistant Reinforced Concrete Building Construction. The state-of-knowledge of the behavior of precast concrete bearing wall buildings to seismic loads is also very limited. Correlation of general design provisions for seismic and progressive collapse will be presented. Voids in the state-of-knowledge and research needs will also be discussed.

STRUCTURAL BEHAVIOR

Seismic Response

There is a fundamental difference between the behavior of cast-in-place concrete buildings and precast panel buildings when subjected to dynamic lateral forces. Research at MIT (1) has shown that there is an extremely complex mechanism that occurs when a panelized structure is subjected to dynamic oscillatory forces. Their analytical studies seem to show that connections behave nonlinearly in several different ways when subjected to cyclic loading. There is a possibility of cracking and softening of the insitu concrete joint, yielding of the connection reinforcement, and slippage along the panel-joint interface.

A rocking motion may be induced which is peculiar to panelized construction. This rocking motion plus stress concentrations caused by overturning and lateral shears could cause failure in either the connection or in the panel at the corners.

Design forces and force distributions in seismic design codes are based on flexural ductility which may not be directly applicable to concrete panel buildings. Medium-rise structures (under six stories) of precast concrete panels probably have sufficient strength and stiffness to remain in the elastic range during earthquakes. Therefore, buildings of a limited height could probably be designed for strength only. Their energy dissipating properties would thus preclude behavior in the inelastic range.

Highrise precast panel structures, however, should be designed with adequate vertical reinforcement and horizontal shear-resistant joints, and with ductile vertical and horizontal connections between all precast elements. In addition, precautions must be taken to prevent degradation or softening of the horizontal insitu concrete joint at the edges of the wall panels.

Another area of dissimilarity in response of precast vs. insitu concrete buildings to dynamic cyclic loading, is floor diaphragms. The relative flexibilities of the two systems are quite different. The overall response of the structure and distribution of seismic forces within the structure is affected by the varied diaphragm action. Any nonsymmetry or eccentricities in the building could further compound the problem. The need for peripheral and horizontal transverse and longitudinal reinforcement to maintain diaphragm action in precast concrete floor systems is apparent. Much more dynamic testing must be conducted to verify the analytical results reported by Becker and Llorente, MIT (1).

Progressive Collapse

Most structures will remain stable under normal gravity loads and also are usually designed to withstand normal lateral loads such as wind and earthquakes. HUD's major concern is how resistant are buildings, constructed in areas of low seismicity and low probable wind forces, to abnormal loading conditions such as explosions, vehicular collisions, tornados, faulty practice, excessive eccentricities, etc? Designers generally consider these abnormal loadings to have a low probability of occurrence and therefore neglect them (5).

Generally, also neglected by designers is the overall three-dimensional stability of statically indeterminate multistory structures. Engineers in many areas of the country are not familiar with nor concerned with dynamic response of buildings to lateral wind and earthquake forces. Bearing walls and load-carrying elements are generally designed for only gravity loads, therefore continuity and ductility are not considered of major import. This demonstrates the need for development of a "general structural integrity" of all buildings.

Research sponsored by HUD at the Portland Cement Association (PCA) has consisted of analytical and experimental studies of the behavior of structures subjected to abnormal loads (10). By considering various load-carrying elements ineffective to support load, remaining structures were tested. Conclusions from this research are that to retain general structural integrity, all connections between elements in a precast concrete panel building need tensile continuity and ductility.

Continuity is essential to develop bridging capabilities needed to transmit and redistribute loads around an ineffective or collapsed member. Ductility is necessary not only to sustain deformations that may be associated with conditions in the damaged state, but also to establish some measure of energy absorption under dynamic effects of either normal or abnormal loads.

Assuming that local damage occurs, ties should be provided to develop alternate structural actions to bridge the damaged area. The following ties, with applicable rationale developed by PCA, are required to prevent progressive collapse:

a) Transverse: Permit cantilever action and beam action in wall panels and assist floor diaphragm action;

b) Vertical: Provide a tie-down for wall panels to prevent overturning, suspend wall structure from cantilevered portion above, and ensure adequate shear capacity in horizontal connections;

c) Longitudinal: Permit a suspension system (partial catenary or membrane) in floor elements; and

d) Peripheral: Ensure floor diaphragm action and provide beam action at exterior walls.

But what really occurs when a concrete panelized building is subjected to seismic forces or abnormal loadings? A great deal of investigation needs to be conducted in this area. One effort along these lines is research now being supported by HUD at Drexel University: "The Nature and Mechanism of Progressive Collapse in Industrialized Buildings Utilizing Established Small Scale Direct Model Techniques." Three-dimensional small scale physical models of precast concrete bearing wall buildings will be tested and studied analytically. Attention will be focused on various types of connections and their contributions to the mechanism to prevent progressive collapse. Both elastic and inelastic models will be used. Assemblies will be tested to duplicate prototype behavior of tests conducted at PCA. Small scale models of components and joints will be tested in order to verify their validity and also to verify the tests conducted at PCA. Hopefully, more modeling of entire structures with the addition of testing to evaluate their response to dynamic seismic forces will be initiated.

Changes also need to be made in the structural analysis and design courses in many of our universities. Most undergraduate schools do not instruct students to design for earthquakes. Since the probability of major seismic activity is considered low or nonexistent by most building officials, the academic community apparently does not consider the subject to be of sufficient importance.

Another problem which arises in structural design is that students are instructed to design elements of a building without viewing the completed structure as a unit. In monolithic concrete structures and those of steel with continuity provided at all connections, general structural integrity is usually provided. Precast concrete buildings, on the other hand, need to have special design considerations for ductility and continuity. This applies not only to buildings in seismic zones and high wind areas but also to those in other areas of the country as well. Providing for resistance to earthquake forces will automatically increase the resistance of buildings to a progressive collapse type of failure.

DESIGN RECOMMENDATIONS

In traditional structural design, the designer is able to focus on known load conditions. Building codes provide exact advice on dead, live, wind and seismic loads. The designer generally accepts these loads and designs his structure to resist them in as efficient a means as possible. With the advent of the design concept to account for abnormal loadings to avoid progressive collapse, the designer is required to consider unknown or unforeseen forces (5).

With respect to progressive collapse, a successful design is one resulting in a structure that both limits the extent of failure and bridges over the failed area if an abnormal loading occurs. The capacity to limit the failed area is generally proportional to the structure's continuity and ductility or "general structural integrity." The challenge is then to provide a reasonable amount of continuity and ductility in all structures as economically as possible.

This continuity and ductility is also the necessary ingredient for aseismic design of buildings. The degree of ductility and continuity built into many forms of construction usually depends on the design requirements for wind and seismic forces. Improving the general structural integrity of buildings by providing adequate ties and connections should improve the resistance to seismic forces by increasing the floor diaphragm action, wall cantilever action and joint continuity.

However, some structures might satisfy quasi-static seismic code design forces for Zone 3, but might not prevent progressive collapse. This could be true for buildings under construction, buildings subjected to abnormal vertical loads, or under reinforced structures with low overturning moments. Designing according to the code (static design) might also be insufficient to resist even moderate earthquakes. The structure may have adequate lateral shear capacity and yet very little ductility needed to resist dynamic cyclic loading.

Graham Powell in his paper (9), states "----, the designer of an earthquake-resistant structure must pay much greater attention to the distribution of forces and deformations in the structure than the designer of a structure in a non-seismic area, and therefore has a much greater need for sophisticated analyses." I agree with the need for more sophisticated analysis techniques for seismic design, but the designer of buildings in non-seismic areas should not be too complacent and ignore provisions for continuity and ductility. This has been the problem in the past and demonstrates the need for a re-education of many structural engineers.

The most important means to minimize the risk of progressive collapse in panel bearing wall structures is to provide adequate horizontal, vertical and peripheral ties between all structural elements. At a research workshop held in Austin, Texas in November 1975, emphasis on ductility and continuity similar to that used for seismic and wind design was considered a very important method to avoid progressive collapse (7). The concensus was that most structures designed and detailed to resist seismic loads in Seismic Zones 2 and 3 (6) in the U.S. would have a low susceptibility to progressive collapse.

In structures designed to resist substantial lateral forces, designers are familiar with the concept of developing diaphragm action in floor and wall elements. These diaphragms provide horizontal and vertical flow paths for the lateral forces. However, in some areas of the country, designers do not need to consider diaphragm action for just gravity loads. Therefore, little attention is paid to the need for tying all elements together. Precast panel buildings and masonry bearing wall buildings with generally inadequate connections are common in areas of low seismicity and low wind forces and are susceptible to progressive collapse (7).

Proper floor diaphragm action is required in designs for resistance to both seismic forces and abnormal loads. The diaphragm action necessary to distribute lateral loads to vertical bracing elements is discussed in Irwin Speyer's paper on "Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads," PCI Journal, March-April 1976 (11). He notes: "Special consideration should be given to the interconnection of precast floor elements in Seismic Zones 2 and 3." Design recommendations are made with regard to minimum ties required to prevent progressive collapse (develop general structural integrity). Reference is also made to the PCI Design Handbook for Precast and Prestressed Concrete which has a design methodology for diaphragm action to resist lateral loads.

The potential degradation of connection areas during a severe earthquake may bring the overall stability of the system into question, according to the MIT analytical studies (1). This concern is identical to that expressed over progressive collapse due to abnormal loading. The assurance of general structural integrity is paramount in providing an aseismic design. The development of such integrity requires continuous reinforced ties throughout the structure. These tie requirements are not intended to supplement traditional strength requirements, rather they are intended to hold the structure together in order to provide stability in a damaged state.

In any major earthquake a panelized structure, as well as all other structures, will undergo some damage. It is necessary to guarantee that such a damaged structure retain its overall stability. The maintenance of this stability in a damaged state is directly analogous to the maintenance of stability after damage caused by an abnormal (non-seismic) loading. The strength of the structure must not be so degraded, through seismic reversals, that the structure is no longer capable of supporting gravity loads. This potential for degradation may be a critical parameter in the design of horizontal joints. In addition, the overall integrity of the structure must be guaranteed by tying the components together. This is the basis of alternate load path philosophy advocated for progressive collapse and is equally valid for seismic design (1).

Recommended peripheral, vertical, transverse and longitudinal ties with applicable rationale may be found in PCA Draft Report No. 4 (10) and the PCI Journal article (11).

CODE PROVISIONS

Most engineers familiar with aseismic analysis and design procedures realize the shortcomings of code requirements and also recognize the difficulty with which design recommendations are incorporated into building codes. A uniform policy for the design and construction of residential buildings to resist earthquakes and abnormal loadings is needed.

Building codes generally have provisions for aseismic construction (though somewhat difficult to enforce outside of the West Coast). However, building code provisions do not exist for resistance to progressive collapse. If we can adopt a policy to provide general structural integrity in all buildings, perhaps we can solve the problem of resistance to abnormal loads as well as seismic forces.

Improvements will also be needed in building code provisions for design to resist earthquake forces. "What is the current ability to design buildings to prevent collapse during earthquakes?" This was a question discussed by Henry Degenkolb at a 1972 Conference on Seismic Risk Assessment for Building Standards (2). "We can design to prevent collapse, but the mere adoption of a code will not do it. It requires competent design and suitable materials, backed up by adequate checking and thorough inspection." He also put in perspective the main theme of this paper: "The most important item in earthquake-resistant design is not even mentioned or specified in the Code - the structures must be tied together to act as a unit." Many construction materials and systems have this inherent ability but others such as precast panelized structures do not.

There have been many examples from recent earthquakes such as Anchorage and San Fernando that demonstrate that precast concrete buildings may comply with building code standards for Zone 3 (as they are usually enforced), but brittle connections with no ductility failed when they were overloaded. With ductility, buildings survived; without it they have collapsed. It is not so much the magnitude of the lateral force but the way the designer frames his building to resist dynamic forces. In most parts of the country where forces are known, only strength and force levels need be specified in building codes and specifications. If a structure is strong enough, it is sufficient. In earthquake country, however, we need not only strength but ductility. The building must remain stable and act as a unit even when design forces are exceeded. The structure's deformation capability is at least as important as its strength (2).

Donovan has drawn similar conclusions as stated in his draft paper for this workshop (3): "Those structures which suffered the most distress in the San Fernando Earthquake were frequently found to satisfy code requirements while having deficiencies in continuity, etc., which cannot be remedied by code modifications alone, especially the simple expedient of requiring higher forces. Higher force requirements will result in stiffer structures. As many of the major problems in earthquakes are produced by displacements, designs which produce stiffer structures may be self-defeating, especially for relatively brittle structural materials such as reinforced concrete."

Uzemeri et al, present a code situation that should be alarming to all designers of precast concrete structures in areas of high seismicity (12). The National Building Code of Canada (NBC) requires that buildings of more than three stories in height and located in Seismic Zones 2 and 3 have structural systems described in Table II, Cases 1 thru 6.

The NBC Commentary explains that "continuously reinforced concrete" as used in the definition of Case 6, refers to reinforced concrete conforming to CSA A23.3 - 1973. The Commentary also states that precast concrete construction may be used in Case 6 if reinforcing is made continuous by means of lapped or welded splices in accordance with CSA A23.3. Splices also need to be encased in cast-in-place concrete. The designer can detail a building as he would for a non-seismic region and use a "K" factor of 1.3. Because the NBC Commentary specifically permits precast concrete structures under Case 6, numerous precast buildings have been erected in high seismic risk regions in Canada. These buildings may or may not contain adequate provisions for continuity and ductility required to resist dynamic oscillatory forces. The writers of the paper feel that because of the difficulty in accurately predicting the actual magnitude of possible earthquakes, concrete structures with only "nominal ductility" should not be constructed in Zone 3 (12).

RESEARCH NEEDS

As should be apparent from this report and other papers presented at this workshop, little is known about the inelastic behavior of precast concrete bearing wall buildings subjected to severe earthquakes. We intuitively know that by providing ductile connections and continuity in reinforcement we can provide general structural integrity to avoid progressive collapse and resist seismic effects.

However, little research data is available to verify our assumptions or to give structural engineers the assurance needed to design precast concrete bearing wall buildings with the necessary conservatism. Many research and development needs, noted at the 1975 Research Workshop on Progressive Collapse in Austin, Texas (7), pertain to this seismic-resistance workshop. Specific values of tie forces for many types of construction need to be developed. Also needed are improved detailing procedures to ensure that ties function properly under extremely large deflections.

The paper by Becker and Llorente (1) and research at the Portland Cement Association (10) also point out many deficient areas in the state-of-knowledge.

Bruce Olsen (8) recommends further research in areas such as evaluation of structural walls as independent elements and as parts of box elements, evaluation of horizontal floor diaphragms, and investigation of precast articulated structures. Englekirk (4) states that, "It should be fairly obvious that very little reliable information is available to the engineer who wishes to undertake the design of a prefabricated building in a seismically active area." He recommends four areas of research:
1) Determine dynamic characteristics of prefab highrise buildings,
2) Evaluate overturning forces on prefab shear walls, 3) Thoroughly study connection details typically used by prefab building industry, and
4) Determine dynamic characteristics and strength of diaphragms composed of prefab components and thin topping slab.

Hopefully a greater concerted effort will be promoted by industry, academia, and governmental agencies to study the performance and behavior of precast concrete bearing wall buildings subject to earthquakes and abnormal loads.

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PRODUCTION AND REPAIR ASPECTS OF INDUSTRIALIZED BUILDINGS FOR ERCBC

by

Weston T. Hester
Assistant Professor
Construction Engineering and Management
Department of Civil Engineering
University of California
Berkeley

1. INTRODUCTION

Precast concrete manufacturers are seeking a greater proportion of the construction market throughout the United States, including seismic regions. This marketing effort is being made by the Prestressed Concrete Institute (PCI) on behalf of the industry as a whole, and by individual PCI members. It is important to recognize both the national and local effort when considering the precast industry. PCI, manufacturers of proprietary systems, and individual PCI-member companies have worked co-operatively to develop national standards and practices for precast concrete design and production. At the same time, individual precasters working out of one or more metropolitan regions retain a great deal of autonomy and individuality in the units they manufacture, and production practices used. This brief discussion will focus on both national standards and local production practices associated with precast concrete, and their relationship to structural seismic performance.

Actual precast production practices may be limited by available materials, industry standards, and project specifications, but the precaster retains substantial flexibility in his operations. Each precaster establishes a particular product line (e.g., double tees, hollow core slabs, flat panels), and selects materials to reflect his marketing conditions and priorities. Two aspects of precast production practices are particularly significant for structural seismic performance. First, production materials, steel and concrete, are selected not only on the basis of the structure's service performance, but also in view of their economy, availability, and contribution to high-speed production. In the following discussion production considerations associated with the seismic performance of concrete and mild steel reinforcement are introduced. Second, the characteristics and significance of design modifications during production are identified and discussed.

2. CONCRETE

2.1 Existing Practice

The provisions of ACI 318-71, Building Code Requirements for Reinforced Concrete, are the most widely used means to specify and control concrete mechanical properties. The ACI-318 requirements apply to minimum compressive strength, but do not consider concrete stress-strain characteristics, values for failure strain, or mechanical behavior under cyclical loading. Reflecting this emphasis, few precasters seriously consider and incorporate the general mechanical properties for their locally produced concrete. Furthermore, for simplicity in production, most of the precast industry has effectively standardized on a minimum 28-day design strength of 5,000 psi (34 Mpa), even though this may exceed the strength actually required. In some seismically active areas, such as Hawaii and the Northwestern United States, excellent aggregate facilitate routinely obtaining 28-day strengths in excess of 6,000 to 8,000 psi (41 to 55 Mpa), and a higher level of strength is incorporated in design. Where high compressive strength, over 7,000 psi (48.3 Mpa), concrete in intentionally incorporated in the design, some engineers require that compressive stress-strain characteristics be determined, but this requirement is infrequently encountered.

The relationship between specified design strengths and those actually achieved varies widely. The level of strength readily attainable with available materials is the most important factor, but others include economizing on cement consumption (with consequent low strengths), mix proportioning primarily for early (1-day) stripping strengths, and utilizing very high concrete strengths to facilitate product handling. It has been the author's experience that 28-day compressive strengths tend to be about 20% higher than the level actually specified, primarily due to the latter two factors. However, the contrary case has been encountered where economy-minded producers strictly adhered to the minimum design strength, regardless of benefits arising from its modification.

To minimize distress arising through product stripping and handling the producer, independently of the designer, will frequently increase minimum 28-day strengths up to 3,000 psi (21 Mpa) over the level specified. This action is increasingly common with the growing trend towards precasting of delicately shaped members, higher stripping stresses for prestressed members, and the expanded availability of super water-reducing admixtures.

Example 1.

A designer established a minimum 28-day strength of 6,000 psi (41 Mpa), approximately the level normally attainable, and would not allow the precast units to be shipped until they had attained at least 5,500 psi (38 Mpa). A particular unit, not yet produced, was critical for the building sequence, so the producer modified the basic mix to provide 5,500 psi (38 Mpa) in 1 day, with a consequent 10,000 psi (68 Mpa) strength at 28 days.

Example 2.

The design engineer for one structure specified a 5,000 psi (34 Mpa), minimum 28-day strength for all prestressed/precast members. In place, the precast members have a 3-inch (7.6 cm) structural concrete topping, and 5,000 psi (34 Mpa) is an appropriate design strength. However, because of the large quantity of prestressing within the member, the producer established that a stripping (12 hours) strength of 6,500 psi (45 Mpa) was required.

2.2 Concrete Mechanical Properties

All concrete mechanical properties are sensitive to the aggregate and mix proportions employed, but some tests show that stress-strain characteristics are particularly aggregate dependent. [3] [4] [5] [7] [8] [9] Occasionally the producer selects materials and mix proportions which satisfy strength requirements but also exhibit marginal or poor mechanical properties. Specifically, this frequently occurs with some lightweight aggregates and poor quality normal-weight aggregates.

High strength, over 5,000 psi (34 Mpa), lightweight concrete (110 lb/ft^3) (760 kg/m^3) is utilized to minimize the weight of long-span prestressed members, to reduce the collapsing and sloughing of extruded-concrete members, and as a means of structural insulation.

Example 3.

One hollow-core precast panel system is most readily produced with a finely graded lightweight aggregate material. Reduced concrete weight prevents collapsing and sloughing of the core voids, and the aggregate surface properties facilitate very high levels of strength development.

Mechanical properties of lightweight concrete are highly aggregate dependent. High quality aggregates can readily provide high concrete compressive strengths and contribute to other desired mechanical properties. These aggregates usually have a coarsely textured, high strength ceramic shell which can develop an extraordinary paste-aggregate bond, and produce high concrete compressive strengths. Unfortunately, many lightweight aggregates have limiting properties which contribute to a concrete which is susceptible to large creep and shrinkage strains and abrupt compressive failures. If only poor or marginal aggregates are available, compressive strength is maintained (at l-ast partially) by utilizing normal-weight coarse aggregate as part of the mix. However, other desirable mechanical properties may not be maintained in this manner.

In some local areas, or entire geographic regions, normal-weight aggregate supplies are limited to materials with poor or marginal gradations, shapes, or surface or mineralogic properties. Common examples of undesirable characteristics include excessive proportions of contaminant clays or crusher fines, exceptionally smooth aggregate surfaces, and highly elongated shapes. Varying coarse and fine aggregate proportions, cement content, and carefully blending aggregate sources, the producer may attain the desired compressive strengths and other mechanical properties. In some cases, even though the desired strength is obtained, other mechanical properties remain at a reduced level.

Example 4.

A coarse aggregate elongated in shape, poorly graded, and with a smooth surface, was capable of economically attaining quite high compressive strengths when properly proportioned with a suitable fine aggregate and high cement contents. Unfortunately, the coarse aggregate properties also contributed to a low elastic modulus and failure strain.

2.3 Conclusion

The precast industry is predicated upon the ability to develop very high-early and long term strengths. Because actual concrete strengths frequently exceed those specified by the design engineer it is desirable to identify the mechanical properties of concrete actually utilized, and assess the significance of these properties for current design practices.

It is also evident that some lightweight and normal-weight concretes are capable of achieving desired compressive strengths, while demonstrating less-than-desirable levels for other mechanical properties. Representative samples of these concretes should be investigated, and a suitable response to their current use be developed.

3. REINFORCEMENT

3.1 Existing Reinforcement Practices

Precasting represents one specialized segment of the larger reinforced concrete market. McDermott [6] and Black [2] considered the steel reinforcing industry as a whole, and it is appropriate to extend their discussion to precasting pratices. The following discussion identifies current reinforcement practices within the precast industry with particular emphasis on the use of welded wire fabric (WWF), mild steel bars, and welding.

A concerted effort is being made by the precast industry to effectively standardize selected reinforcement designs and materials. Technical committees within PCI have developed recommended connection designs and reinforcement patterns. Members of PCI and the Welded Wire Fabric Institute (WWFI) have jointly developed welded wire fabric (WWF) designs for use within the precast industry. The American Society for Testing and Materials (ASTM) has established specifications for chemical andmechanical characteristics of reinforcing bars, welded wire fabric, and prestressing steels.

3.2 Welded Wire Fabric (WWF)

Both mild steel bars and WWF are widely used in precast construction. WWF is used extensively as a shear and flexure reinforcement in panelized precast products and in standardized products such as double and single tees. Almost all WWF used in precasting is undeformed, except where it is specifically designed as shear reinforcement. Design and specification of WWF patterns and applications is effectively controlled through provisions of ACI-318, and mechanical properties are covered in several ASTM specifications. It has been the author's experience, based on mechanical properties reported in accordance with ASTM requirements, that welded wire fabric is the most consistent and high quality of the mild steel materials. However, although manufacturers "certified" that

ASTM elongation and area reduction requirements are met or exceeded, none presented stress-strain curves for samples tested.

3.3 Bar Reinforcement

Bar reinforcement is extensively used, of course, and a general effort is made to use smaller bar sizes, preferably smaller than No. 8's. Virtually all bar stock is specified according to the long-standing ASTM standards A-617, A-615, or A-616 and, as McDermott [6] points out, use of ASTM A-706 is prohibitive economically. Although the ASTM standards serve an essential role in controlling steel chemical and mechanical properties, the characteristics of steel available to the precaster are heavily influenced by the level of United States demand for steel, and the corporate marketing policy of specific bar manufacturers.

During national steel shortages the author has observed, albeit qualitatively, a general decline in steel quality as measured by carbon content, yield stress, and ductility in handling. This development is exacerbated by the importation of foreign steels. While many foreign steels maintain a consistently high quality, others have proven to be highly erratic in chemical characteristics and mechanical properties. Little can be done to compensate for variation in steel properties as a function of market conditions, but the designer should be aware that this variation exists.

Manufacturers of reinforcing steels are individually subject to pressures of economy, raw material quality, corporate objectives, and market demands. Accordingly, steel products available from different manufacturers vary significantly in quality and performance. Since most mechanical characteristics are restricted by ASTM specification limits, the greatest variation occurs in steel chemical composition and overall stress-strain characteristics. (Note: Actual stress-strain curves are not normally provided). Although implicitly obvious, these variations are seldom recognized in purchasing or specification decisions by the precaster or designer.

Because precast operations are by nature fixed in one geographic location, and are consumers of a reasonably static mix of bar sizes and grades, an effort is usually made to establish a regular purchasing arrangement with one or more principal steel suppliers. Therefore, the characteristics of steel utilized by the precaster are consistent. Unfortunately, at least one of these suppliers may provide materials with marginal characteristics, with a consequently deleterious effect on seismic capacity of precast units produced.

3.4 Structural Welding

Structural welding of reinforcing steels has limited application with cast-in-place (CIP) construction, but is widely used in precasting. Naturally, the ultimate strength and ductility of the weld and associated reinforcement has a determinant influence on the performance of the completed structure.

PCI has issued standard connection designs incorporating structural welding of reinforcing steels, and AWS has issued selected specifications for the welding process. PCI and AWS guidelines emphasize proper weld placement, dimensioning, and welding procedures. Actual weld placements and dimensions vary with the welder and practice of the firm, but it has been the author's experience that weld dimensions are in excess of those required. Weld ductility is inevitably more variable and difficult to measure and control than simple dimensions. Weld ductility is influenced by the steel chemical composition, and welding procedures used.

Ductility of welded reinforcing steel connections is strongly influenced by the chemical composition of the base steels, i.e. proportions of carbon, manganese, and other alloying elements. Mill test certificates provided by the manufacturer list chemical and mechanical characteristics for each heat of steel received. Using the reported chemical composition and AWS guidelines, proper welding practices for that specific heat may be established. In general, for in-plant control this procedure has worked so that non-welded bars and those requiring special handling have been identified, and excessively brittle welds avoided. However, it is rarely practical to sort and control the chemical composition of miscellaneous reinforcing steels used in the field.

Some cases have been encountered where the reported chemical compositions did not accurately reflect actual conditions, but this has not been a major problem. Some designers and precasters erroneously use bar Grade as a measure of weldability. Since Grade 60 steels typically have high carbon and alloying metal contents, and Grade 40 steels a much lower level, welding is sometimes arbitrarily limited to Grade 40 stock. This practice is seldom realistic in light of actual conditions. Numerous instances have been encountered where bars were designated Grade 40, but had excessive carbon and manganese contents, and high yield stresses. Likewise, Grade 60 steels occasionally have low carbon and manganese contents, and are quite satisfactorily weldable.

Good welding practice includes use of sound welding procedures and properly selected and maintained equipment. It is likely that the single largest source of confusion and improper welding practices within the precast industry is the matter of preheating high-carbon reinforcing steels. If high carbon and manganese reinforcing steels are heated prior to welding, i.e., "preheated", and maintained at this temperature throughout the welding process, the final weld may retain a substantial amount of ductility. The minimum "preheat" temperature required varies with steel chemical composition, and may be calculated on the basis of AWS [1] guidelines. Proper preheating requires that the precaster sort bars according to their preheating requirements and establish a regular routine for heating the base metals to be joined. The extra effort associated with proper preheating practices causes some less sophisticated precasters to ignore the requirements entirely, and others to establish practices even more rigorous than those required. Where preheating is required, many precasters use gas torches to heat the entire weldment to temperatures substantially in excess of those required, and thereby avoid reheating during the welding process. This "over-response" to preheating requirements appears to be satisfactory. In other instances, particularly in field welding practices, facilities may not exist for preheating, and it will be omitted altogether.

The application of quality control programs to welding practices is a difficult and complex issue. A large volume of variable-quality materials are used in precastor and under the full range of environmental conditions. Accordingly, even a cursory effort to assess and control material quality and application requires a well-organized, full-time program. The control of the welding process itself is substantially more difficult. Ideally, each weld would be "inspected" as it is made, including evaluation of preheating conditions, welding machine settings, welder speed and method, etc.. Inspection of completed welds is frequently impractical if not impossible because of the lack of usable test methods, and complex shapes and constructions involved. As a result of these and other factors, principally the excellent service record of welds used in precast construction, few precasters have an active welding quality control program.

3.5 Conclusions

The widespread use of welded wire fabric (WWF) and structural welding of reinforcing steels deserves careful evaluation in light of seismic considerations. Since WWF applications and structural welding practices are specifically developed for use in precast construction a careful review of the state-of-art in practice must precede any testing program.

4. STRUCTURAL MODIFICATIONS DURING PRODUCTION

4.1 Introduction

There are at least three basic types of structural modifications which occur during production, including (1) the structural strengthening of specific products, (2) changes in the original design to accommodate ease of fabrication and/or correction of errors in the original design, and (3) the repair of errors which are identified after fabrication of the product.

4.2 The Producer's Responsibility

The designer assumes responsibility for seismic capacity of the structure as a whole, and the individual constitutive elements. However, virtually all designers provide the precaster with the caveat that

> "The precaster shall be responsible for ensuring adequate concrete strength and providing all additional reinforcement necessary to resist stresses associated with product stripping and handling."

Accordingly, the precaster designs each unit in light of loadings associated with final service conditions, and the production process. For most precast products the reinforcement provided in the original design is adequate. However, panels which are unusually flexible, have large openings, or may tend to buckle during stripping and handling, require extra reinforcement. Typically this consists of small diameter (#4 to #6) mild steel bars around openings, and additional WWF and bars in flexural members. This additional reinforcement is significant in that very few designers reconsider their original design in light of the member's modified stiffness and seismic ductility.

4.3 Production Motivated Modifications

As discussed earlier, to facilitate fabrication and design the precaster attempts to standardize materials and design details. If the original design deviates from the precaster's practice, and particularly if the design will be difficult or impossible to implement, the precaster may request a variance or basic design change. Typically, variances or changes requested include minor variations in panel shapes, change of panel thicknesses to standard dimensions, and simplification of reinforcement and connection designs. The issue of subsequent design changes is a delicate one for both the designer and precaster. Design modifications, either correcting errors made in the original design or responding to the exigencies of fabrication and construction, may alter the structure's seismic performance. A comprehensive denial of design changes during construction is unrealistic (although sometimes done), and a complete re-analysis of the structure for small changes in the design of individual elements is not cost effective. It is recommended, therefore, that characteristic types of changes in the original design which are significant be identified, and that only appropriate re-analysis of the structure be considered.

4.4 Repair of Distressed Products

Good practice and most contract specifications require that prestressed and precast products be designed to not distress (i.e., crack, spall or crush) as they are removed from casting molds and handled. Proper design, however, cannot be expected to compensate for all of the potentially damaging conditions encountered from production through erection. Repairs are endemic to all construction. The principal concern is that they be completed economically and without compromising cosmetic appearance or structural effectiveness. In light of seismic considerations, two classes of structural repair are important. Restorative repairs are carried on products distressed during stripping, handling, or erection. Modification-repairs are made on products which through improper design or fabrication have embedded connection plates or details missing or misplaced.

The assessment and restorative repair of distressed products is both an art and a science. A knowledge of engineering principles, repair methods and materials, their performance and limitation, is important for sound evaluation of the distressed product and determination of the best repair. Subsequently, using more art than science, an experienced concrete finisher can quickly and expertly repair even seriously distressed conditions, restoring both structural capacity and cosmetic appearance. In principle, only the design engineer should determine the significance of a distressed section but in most cases an engineer's review of each crack or spall is not required, and a standard repair may be made (if a questionable situation does arise the engineer should be informed). Common repair methods include recasting with ordinary concrete, utilizing proprietary cement mortar blends or grouts to reconstitute the section, and the use of epoxies for recasting and injection repairs. As in cast-in-place construction, additional studies of these repair methods and their relative efficacy should be made.

Where embedded connection details are omitted or misplaced the existing condition may be approved on the basis of an engineering analysis, or structural modification may be required. To design and install connection details in hardened concrete to perform at the same level as those originally specified requires well developed experience and skill. The most common forms of repair include bolting or welding of substitute connection plates to expansive bolts (e.g., "Read-Heads")embedded within the concrete. The use of expansion bolts is the most widespread but in some cases threaded bolts or reinforcing rods are fixed to the concrete with epoxies, grouts or proprietary cements. Although these practices have performed with varying success to date an evaluation of existing practices and alternate solutions is recommended.

4.5 Conclusions

Designer-approved structural modification of the original design for precast products regularly occurs as part of the production process. The extent of modification varies from small structural changes to facilitate fabrication, to major alterations carried out as restorative repairs.

The seismic significance of these modifications is often under-appreciated and/ or misunderstood. A survey should be made of structural modifications currently being made, and their significance established.

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ANALYTICAL AND EXPERIMENTAL STUDIES OF PRESTRESSED AND PRECAST CONCRETE ELEMENTS

by

Neil M. Hawkins
Professor of Civil Engineering
University of Washington

INTRODUCTION

The purposes of this report are: (1) to review and summarize the available analytical and experimental studies concerned with the seismic resistance of prestressed and precast concrete elements and their subassemblages; and (2) to assess the significance of those studies and future research needs. Separate reports are presented for prestressed concrete and precast concrete.

PRESTRESSED CONCRETE

Background

There have been numerous previous reviews of the state-of-knowledge on the seismic response characteristics of prestressed concrete. Almost as much effort has been devoted to reviewing the work of others as has been spent advancing the state-of-knowledge. The state-of-knowledge a decade ago and the contemporary questions of prime concern to designers are clearly outlined by Lin [1][2] and Despeyroux [30]. The most frequently asked questions concerned: (a) possible detrimental effects of the eccentricity of the tendon on reversed loading capabilities; (b) the abruptness of the failure and the degree of energy absorption for loading reversals; (c) the design of joints; (d) appropriate earthquake input loading criteria and drift constraints; and (e) appropriate structural configurations for seismic zones. Lin's, Despeyroux's and Guyon's [18] articles provide the best answers available at the time to each of those questions. In 1968 the Cement and Concrete Association [32], published a bibliography of articles on the design of earthquake resistant prestressed concrete structures and experience with the effects of earthquakes on such structures, and Newmark and Hall [27] discussed the design of reinforced and prestressed concrete structures with particular emphasis on their dynamic response characteristics. In 1970 [66] and 1971 [23] Blakely and Park provided excellent, comprehensive, historical reviews of the performance of prestressed structures in earthquakes and of simulated seismic loading tests on prestressed concrete elements. Four reviews since that time have simultaneously examined results for structural steel, reinforced concrete and prestressed concrete and have dealt with experimental studies [28], hysteresis studies [13, 24] and theoretical predictions of the response of elements [14]. The paucity of information and understanding on the behavior of prestressed concrete elements, as opposed to structural steel and reinforced concrete elements, is readily apparent from those reviews.

The response spectrum approach allows the designer two choices [27]. A high resistance can be provided and the structure kept virtually elastic or a high energy absorption can be provided and the structure permitted to deform inelastically. To a large extent the K factor of the UBC Code also recognizes those extremes. Highly prestressed structures fit more closely the first than the second category. However, while providing a high resistance they develop large deformations approaching those associated with the second category. Current building code provisions have been built around the response characteristics of structural steel and reinforced concrete frames and the characteristics of highly prestressed frames do not mesh neatly with those concepts. The difficulties have been succinctly noted by Rosenblueth [67] and more comprehensively explained by Nakano [19]. Consider the three diagrams shown in Fig. 1 that represent respectively a nonlinear elastic system (Fig. 1A), an elasto-plastic system (Fig. 1B) and steady state vibration curves for cracked prestressed beams obtained by Spencer [22], (Fig. 1C). System A is a more realistic idealization of the response of Fig. 1C than system B. System A will develop, on the average, a peak deformation averaging about $y\sqrt{\mu}$, when the average acceleration spectrum is hyperbolic for the range T to $T\sqrt{\mu}$. The quantities y, T and μ are defined on Fig. 1 and are for an "equivalent" linear system (a system that has the same natural period and damping coefficient as the small-oscillation values for the nonlinear system under consideration). In contrast, for system B, peak y values will be near those of the equivalent linear system provided T lies within the hyperbolic range of the acceleration spectrum. Thus, although the virgin loading characteristics of the systems are the same, System A will be subjected to maximum forces and deformations approximately $\sqrt{\mu}$ greater than those for system B. Further, if the ground is soft and a prevailing ground motion period exists in the T to $T\sqrt{\mu}$ range, structures of type A will have an even less favorable response. In contrast, if the spectral acceleration decreases faster than 1/T for the same range they will have a more favorable response.

Where prestressed concrete members are used as primary lateral load resisting elements they are likely to be post-tensioned. General design concepts for post-tensioned structures are now almost the complete reverse of those of two decades ago. In contrast, conceptual changes have been considerably less marked for pretensioned concrete so that now design philosophies for pre- and post-tensioned concrete differ markedly. In the early days of post-tensioning flexural tensile stresses were minimized with the tendons being draped to balance the dead load plus some fraction of the live load. The resulting prestress was relatively high, stresses seldom controlled, and there was little additional deformed bar reinforcement. Now the post-tensioning force is typically determined from allowable flexural tensile stress limitations or even deflection criteria. Members are often on the point of cracking or are cracked under gravity loads. Deformed bar reinforcement is used to control cracking and provide the necessary ultimate strength. This combination of prestressing tendons and deformed bar reinforcement differs markedly from that used in most experimental work to date and results in hysteretic response characteristics more like those of Fig. 1B than those of Fig. 1C.

Many of the difficulties associated with prestressed concrete in seismic zones are conceptual. The difficulties lie not in analyses but in prestressing practices and ingenuity in engineering is needed to overcome those difficulties [16].

Experiments

Damping--Internal damping can be divided into (a) elastic damping of the equivalent viscous type and (b) inelastic hysteretic damping. This section concerns elastic damping. For small oscillations critical damping is greater than that in structural steel and substantially less than in reinforced concrete. Typical elastic damping ratios are 1% when the prestressing is high enough that the section is uncracked, and 2% when there is cracking but no additional deformed bar reinforcement. Thus, the cladding has a substantial effect and typically increases damping by about 3% [13] for prestressed structures.

In an early investigation of damping, Penzien [3] conducted both steady state and free vibration tests on small scale prestressed concrete beams. Variables were the concrete compressive strength, the grade of the prestressing bars and the eccentricity of those bars. The steady state tests showed that the level of prestress and the concrete strength affected damping only in so far as they affected cracking. In the free vibration tests the degree of damping depended largely on whether the previous loading history for the member or the amplitude of the displacement had caused cracking. In a later study Spencer [22] also conducted steady state vibration tests on small scale beams. Variables were the level of prestress, post-tensioning or pretensioning, and the application of end rotations producing either uniform moment or uniform shear along the length of the member. All members were centrally prestressed and contained no deformed bar reinforcement. Damping ratios were not frequency dependent and steady state conditions were readily achieved. Values increased with increasing end rotations and with increasing prestress and were higher for shear than for moment loading. In contrast to Spencer's results, Brondum-Nielsen [29] recently conducted free vibration tests on centrally prestressed beams of a size more likely in practice and observed that the damping decreased as the prestress level increased or the stress amplitude decreased. In a series of tests on 1/3 scale four story prestressed frames containing both prestressing tendons and ordinary reinforcement, Nakano [19] observed about 1% damping prior to cracking, about 3% at cracking and values approaching 7% shortly before inelastic action developed. Further, because the dynamic forces varied over the height of the structure the degree of damping also varied.

Since the amount of damping that occurs before cracking is small, the response of structures in the field will depend strongly on external damping effects. The influence of non-structural members and interactions with the soil and supporting members must be carefully evaluated [50]. Experiments are needed that define the mechanism of damping after cracking. Spencer has hypothesized that the higher damping ratios for shear as opposed to moment loading are because interface shear transfer and bond slip effects determine the damping value and those effects are higher for shear loading. If that hypothesis is correct damping ratios should be less for lightweight concrete since interface shear transfer effects are smaller, less for strand than for wires because bond slip effects are less for strand and greater for shorter than longer span members because shear and bond slip effects increase as the span decreases.

Characteristics of prestressing steels--Even though the response of elements in which flexural effects govern is known to strongly reflect the stress-strain cyclic load characteristics for the prestressing tendons, little experimental work has been done on defining those characteristics. Both References [20] and [17] report limited tensile cycle only data. The result reported in Reference [17] and its idealization for moment-curvature analyses are shown in Fig. 2. That data show that, as typical for high tensile steels, there is a limited amount of hysteresis with unloading and reloading, that the amount increases with increasing maximum strains and that there is a slow reduction in the stiffness with increasing maximum strains. Larger variations in performance and behavior peculiar to the metallurgical, finishing and prior load history for the steel is to be expected for loading reversals. Research is needed to define the effects of those variables and of loading rate on the stress-strain characteristics for the steel.

Behavior of beam elements critical in flexure--Many researchers have investigated experimentally the high intensity reversed cyclic loading behavior of beam elements critical in flexure [11, 15, 20, 22, 36, 46]. Further information on the performance of such elements can also be gained from beam-column subassemblage [23, 61] and frame tests [19]. Most of those elements have been of extremely small size and outside the regime of practical proportions. Many have contained unreasonable amounts of longitudinal reinforcement and prestress levels, or there has been no stirrups, the tendons have not been eccentric or draped or no additional bonded longitudinal deformed bars have been used to ensure net ρf_y values similar to those necessary for satisfactory performance of reinforced concrete elements [27].

In the first study of flexural members, Mugurama [46] reported uni-directional loading tests on small scale, rectangular, eccentrically prestressed beams. Tendons were both grouted and ungrouted. Variables were the loading, monotonically increasing, repeated between a minimum and a constant extreme compression fiber strain, and repeated between a minimum and a constant maximum load that initially caused extreme compression fiber strains varying between 0.0013 and 0.003. Spencer's tests [22], described previously, were also on small scale rectangular beams, but in his case the tendons were central and the loading reversed. Paranagama and Edwards [20] conducted uni-directional loading tests on eight small scale, rectangular, eccentrically pretensioned beams representing under, balanced and over-reinforced designs. Inomato [11] compared the reversed cyclic loading behavior of 12 half scale, rectangular,reinforced, partially prestressed and fully prestressed beams designed for either the same ultimate strength or the same working load. All beams had a stub at midspan to simulate a beam-column joint and for two specimens there was a mortar joint between the beam and the stub. In 1973, Mugurama [15] reported two series of tests on small scale rectangular lightweight concrete beams. In one series of uni-directional loading tests both stirrups and deformed bar longitudinal reinforcement were used in addition to the straight eccentric tendon. Variables were the loading intensity with unloading-reloading at extreme compression fiber strains of 0.0018, 0.002, 0.0022 etc. and the number of load cycles, 1, 3, 5, 9 or 15. In the second series of reversed cyclic loading tests on specimens with similar cross-sections, variables were the presence or absence of stirrups and/or deformed bar longitudinal steel and the eccentricity of the tendon. In 1977, Kvitsaridze [36] summarized the results of monotonic and uni-directional repeated loading tests on small scale rectangular pretensioned beams for which the variables were the level of prestress and the use of static or seismic loading rates.

In 1971 Blakeley and Park [23] reported reversed cyclic loading tests on four rectangular full scale exterior beam-column subassemblages. The columns were pretensioned and the beams post-tensioned and grouted. Additional longitudinal deformed bar steel and ties were provided in the columns and beams with the stirrup steel in two units conforming to ACI 318-71 requirements and being considerably greater than that amount in another two units. A mortar joint was provided between the beam and the column with only the tendons continuous across that joint. Two specimens were proportioned to form plastic hinges in the beam and two to form plastic hinges in the columns. In 1977 Thompson and Park [61] reported reversed cyclic loading tests on 10 rectangular full scale, fully prestressed, partially prestressed or reinforced concrete beam-interior column subassemblages cast as a single unit. Tendons were axially post-tensioned and grouted and the column made considerably stronger than the beam for all specimens. Additional variables were the shear resistance provided in the beams and the joints. One prestressed unit was loaded to failure, repaired and then retested.

Nakano [19] has reported the only test on a full frame. His structure was a three dimensional one third scale model of a four story structure with prestressed columns and beams. Each floor was laterally loaded. Four different joint details and four different slab details were used. A mortar joint with only prestressing across it was provided between the beams and the columns.

From the results of the preceding experiments and the performance of prestressed concrete elements in real earthquakes [4, 8, 12, 31, 33, 50, 51, 68, 70] the following conclusions can be drawn:

(1) Most prestressed concrete elements, when designed for loading reversals, perform well in earthquakes. The failures that have occurred have been due mainly to failures of the supporting structures or connections. Many elements have held together with little damage even though they have dropped to the ground with considerable force. Major consideration must be given to the strength of connections and supporting structures.

(2) Elements should be designed to withstand reversals of moment even when such reversals are not indicated by analysis. This consideration is extremely important for the corners of rigid frames [50][70] or the ends of members where unintentional frame action [4] may occur. Generally, deformed bar reinforcement and confinement by stirrups are necessary to provide adequate strength under moment reversals [10][50].

(3) Unless the first damaging load exceeds about 80% of the collapse load the capacity in the reverse direction is unaffected [11, 15]. Failure under high intensity loading reversals will initiate once the extreme compression fiber strain exceeds about 0.002 [46]. For cycling to greater strains there is a loss in strength and stiffness due to spalling of the compressed concrete and penetration of crushing into the core of the member [61]. That degradation can be

slowed and the ductility and energy absorption increased by the addition of either bonded compression reinforcement or confinement but preferably both. Unless confinement is provided there is a marked degradation in the flexural capacity for beams reversed cyclically loaded to in excess of 90% of their flexural capacities [15]. Confinement should be by closed stirrups with a spacing not exceeding d/4 [50, 61]. Bonded compression reinforcement takes up some of the loss in compressive force caused by concrete spalling permitting a small increase in the ductility at the maximum load and some slowing in the subsequent rate of strength degradation for increasing displacements. Care must be taken to avoid bond failures of the compression steel or tendons acting in compression. With stirrups and compression reinforcement there can be a gain in ultimate strength as large as 16% since ultimate concrete compressive strains develop well in excess of 0.003.

(4) Seismic loading rates can result in strength increases of four to seven percent and ductility increases of 10 to 15% [36]. Since seismic loadings can result in rates of straining in critical regions varying from as much as 2.5 in./in./sec. in rigid structures to as little as 0.025 in./in./sec. in tall, slender structures, [28], it is generally appropriate for design computations to be based on static loading strengths only [50].

(5) Prior to crushing of the concrete or marked inelasticity of the prestressing steel, loading-unloading curves are bi-linear with ranges corresponding to crack open and crack closed conditions. The loading and unloading curves closely parallel one another [22, 24] as shown in Fig. 1C. The width between the curves decreases with cycling to a constant minimum value once cracking stabilizes after two to three cycles to a new peak value. There is no sharp change in stiffness with cracking and only a slow gradual decrease determined mainly by the magnitude of the previous maximum loading [19, 20]. Crack reopening moments do not change significantly with cycling or with increasing previous maximum loadings [15, 22]. For a central tendon the closed crack section stiffness degenerates with cycling to about half the uncracked section stiffness. The ratio of the crack open to crack closed stiffness also falls until it stabilizes at about 0.4 for high prestress levels and 0.2 for low prestress levels [22, 23]. In design, cross-sections may need to be modified to recognize such effects [30][44]. The addition of moderate amounts of longitudinal deformed bar reinforcement or confinement does not markedly alter these stiffness characteristics [15].

(6) Prestressed elements show marked elastic recoveries even after considerable inelastic deformations [61]. Contrasted in Fig. 3 are beam moment-end deflection relationships for three beam-column specimens with similar theoretical flexural strengths and with prestress levels of 1,160, 386 and 0 psi respectively. Because of elastic recovery effects, residual damage and permanent deformations for a structure surviving a major earthquake are less for prestressed than reinforced elements [19, 31, 44].

(7) The dynamic loading response for small vibrations is not the same as that for large vibrations. Strain levels in the extreme compression fiber must exceed about 0.0006 before large vibration response is obtained.

(8) Energy dissipation for prestressed concrete elements is less than for reinforced concrete elements because of elastic recovery effects. In general,

the residual tensile force in the prestressing steel is adequate to close previously open cracks. Thus, significant energy dissipation does not develop until the deformed bar reinforcement yields, the prestressing steel yields, or the concrete crushes [17, 18, 19, 23, 28].

(9) Prestressed members damaged by inelastic action can be readily repaired and most of their resistance restored [61].

(10) Recognition of the two level earthquake loading concept (moderate and severe earthquakes) is desirable [50]. Cracking and cyclic effects of the first earthquake may significantly change the response of the frame for the second earthquake.

(11) Mortar joints between members can perform well and remain essentially rigid provided the tendon maintains compression across them [11, 19, 23].

(12) Plastic hinge lengths typically equal half the beam depth or half the column depth regardless of the presence of a mortar joint within that length. There is no significant change in the plastic hinge length with cycling or increasing inelastic rotations [23, 61].

(13) For sections similar in all respects except for the eccentricity of the tendon, there are no marked changes in strength, stiffness, energy absorption or energy dissipation characteristics with changing tendon eccentricities [15].

There are valid economic reasons for exploring the use of prestressed concrete framing as a primary lateral load resisting system in seismic zones [50]. One approach might be to proportion tendon quantities from vertical service load stresses and deflection criteria and to provide bonded reinforcing bar based on ultimate vertical load and earthquake load criteria. Such procedures are likely to lead to greater reinforcing bar areas than in the tests conducted to date and provide hysteresis loops less anemic than those obtained to date. Experimental research should be conducted based on this model and recommendations developed for limitations on the amounts and distributions of prestressed and non-prestressed reinforcement for seismic loading, on confinement requirements for prestressed members, and on desirable minimum values for the ratio of the ultimate moment to the cracking moment. After such tests have been carried to completion typical units should be repaired and retested so that the difficulties in making such repairs and their effectiveness is determined.

Behavior of beam elements critical in shear and bond--There is little information available on the behavior of elements critical in shear or bond and subject to high intensity cyclic loadings [24, 27]. In Blakeley and Park's tests [23] large inelastic deformations were obtained even when the shear reinforcement only satisfied ACI 318-71's requirements for prestressed concrete. In Thompson and Park's tests [61] the shear stresses in the beams varied between $1.52\sqrt{f'_c}$ and $1.8\sqrt{f'_c}$ so that either the concrete or the steel was capable of carrying the entire shear. Again no adverse shear effects were observed. In contrast, from the performance of prestressed structures in earthquake [4] it is apparent that shear can be a problem when proper provision is not made for moment reversals or frame action. There is general agreement that members must be proportioned so that their shear strengths exceed their flexural strengths [50, 70]. The tests by Kaar and Hanson [5] have provided some relevant information on uni-directional repeated loading effects for members with a crack in or near the transfer length. The bond transfer lengths and the performance under cyclic loading were very sensitive to the surface condition and method of release for the strand. For their 3/8 in. diameter 7 wire strands lengths varied from 40 diameters for lightly rusted strands released gently to 80 diameters for smooth strands released suddenly. Lightly rusted strands could take $3x10^6$ cycles of a loading severe enough to open a crack more than 0.001 in. without requiring a distance from the support to the load greater than the transfer length. In contrast, that length had to be 50% more than the transfer length for only 1,000 cycles of a similar loading and a smooth strand.

Systematic examinations should be made, using inelastic and reversed loadings of: (a) the behavior of prestressed beam elements subject to high shear stresses, (b) combined transfer and anchorage length requirements for pretensioning strands, and (c) the effectiveness of grouting, and anchorage requirements for post-tensioning tendons.

Grouting and anchorages--There is a wide divergence of opinion on whether ducts should be grouted [51]. Kvitsaridze [36] found that the energy absorption for grouted bars was 30 to 40% greater than that for bonded bars but only 10 to 20% greater than that for unbonded bars. The difference was mainly due to varying plastic hinge lengths. Muguruma [46] found that while the response of bonded beams was better in the first and perhaps the second and third cycles to a new peak, any difference dissipated rapidly with further cycling as a result of the grouted tendons loosing bond. Under cyclic loading the fatigue life of a tendon assembly is likely to be cut by a factor of 10 or its endurance limit by a factor of 2 if tendons are unbonded as opposed to bonded. With bonded tendons the fatigue characteristics of the tendon control while with unbonded tendons those of the anchorage control [48, 71]. The fatigue characteristics of the anchorage are sensitive to the number of tendons anchored, the method of gripping the tendons, the hardness of the seating material and any local bending effects at the anchorage [71]. All these findings are for uni-directional loading. Park's tests [23, 61] show that debonding is aggravated by loading reversals on the tendon. One of the main reasons for grouting tendons is to prevent corrosion [70]. However, if the earthquake causes longitudinal cracking through the grout, capillary action could accelerate corrosion. Because of the rapid loss in bond with cyclic loading and the increased possibility of corrosion if cracking develops, why bother with grouting?

Careful consideration must be given to the location of tendon anchorages [50]. They should not be placed in regions of high bending or rotation since their capacity can be adversely affected [70]. Consideration must also be given to the flow of forces from the anchorage. For example, if they are anchored at the outer face of an exterior column, does the flow of forces adversely affect the behavior of the joint? In Blakeley and Park's tests [23] a short beam stub protruding beyond the column was used to anchor the tendons. In Hawkins and Trongtham's slab-exterior column test [69] the tendons were anchored at the exterior face of the column. They were ineffective for controlling torsional cracking at the junction of the discontinuous edge of the slab and the column. There is some indication that the flow of forces from the anchorages caused that cracking earlier than expected.

Beam-column connections--In Thompson and Park's slab-interior column subassemblage tests [61] the ratio of the maximum horizontal shear expected to act on the connection to the theoretical strength for the core predicted from Committee 352's recommendations ranged from 0.71 to 1.13. Considerable degradation of the joint occurred for those joints in which the hoop steel yielded during the first inelastic load cycle to 95% of the theoretical flexural strength of the beam. The strength of the joint then controlled the strength of the subassemblage and a large portion of the subsequent inelastic deformations occurred in the joint along diagonal tension cracks. The units with tendons passing through the central protion of the joint performed better than those without tendons. Thompson and Park recommended that the hoop steel should be capable of carrying all the joint shear and that the contribution of the concrete to the shear strength should be ignored. In Nakano's frame [19] prestressing tendons from both the column and the beam passed through the joint. The characteristics of the joints affected cracking in the surrounding slabs and frame. Particular care is needed in the detailing of portal frame corners where both beam and column tendons may be anchored at adjacent edges [70]. As for reinforced concrete structures it is highly desirable that joints should be ductile and stronger than the members joined for the maximum loads and deformations expected as a result of seismic loadings.

Research is needed to define the contributions of concrete, tendon forces and hoop steel to the strength and deformation characteristics of reversed cyclically loaded prestressed concrete beam-column joints. In such tests particular attention should be given to requirements for bonding of the tendons through the joints and location of anchorages at the external faces of joints.

Slab-column connections--Results of moment transfer tests on six reinforced concrete column-prestressed concrete slab connections have been reported by Hawkins and Trongtham [59, 60]. One specimen simulated a lift slab-interior column connection, another simulated a cast-in-place slab-exterior column connection and four specimens simulated cast-in-place slab-interior column connections. In the latter tests variables were the loading history and the distribution of the tendons. In one specimen tendons in the direction of moment transfer were bundled through the column, in a second they were bundled in the transverse direction and in the other two specimens the tendons were distributed in both directions. The proportions and loading for these full scale specimens were chosen so that the stress and deformation conditions on the connections and the reinforcement in the connection region closely simulated those likely in a prototype structure. Thus, to provide the ultimate capacity required for the prototype and to better distribute cracking in the column region, bonded deformed bar reinforcement was provided in accordance with ACI 318's requirements. Details of the dimensions and reinforcement for a specimen with tendons bundled in the transverse directions are shown in Fig. 4a. Three of the connections were loaded to a shear equal to the design dead load plus two live loads and then subjected to three reversed cycles of moment transfer loading to between 30 and 65 percent of the ultimate moment measured in a subsequent monotonic moment transfer test to failure. The other connection was loaded to a shear equal to the design dead load and then subjected to three reversed cycles of moment transfer to 40 percent of the ultimate moment measured in a subsequent monotonic moment transfer test. Lateral load edge deflection results for the first three specimens are shown in Fig. 4b and for the fourth specimen in Fig. 5. For the first three specimens it is apparent that the best hysteretic performance was obtained when the amount of prestressing tendons through the column region was a minimum. For all three specimens the width of the hysteresis loops decreased rapidly with cycling showing the dominating elastic recovery effects caused by the prestressing tendons in spite of yielding of the bonded bars passing through the column. In Fig. 5 the response for the fourth slab is compared with that for a similarly loaded reinforced concrete slab with integral beam stirrups. Again it is apparent that in spite of the distributed tendons in the prestressed slab its hysteretic behavior is not as good as that of the reinforced slab. Elastic recovery effects again dominate. However, the behavior at the ultimate capacity indicates that its energy absorption for cyclic loading would probably be as good as that for the reinforced slab. From these tests it was concluded that the ultimate shear strength could be calculated from Eq. (11-12) of ACI 318-71 with f_{pc} taken as the axial prestress in the direction of moment transfer and V_p/b_wd taken as the sum of the vertical components of all prestressing tendons crossing the critical section divided by the area of the critical section.

Flat plate construction is frequently used as part of the gravity load carrying framework in seismic zones. Then slab-column connections must be capable of transferring all required loads at the deformations likely in a severe earthquake. If a designer has any doubts about the safety of those connections, he is likely to consider strengthening them by prestressing the slab before he resorts to re-designing the lateral load resisting elements. Thus, even more than for the prestressed frame there are valid economic and practical reasons for exploring the likely behavior of prestressed slab-column connections subjected to simulated seismic loading. That research should build on the type of structural design concept utilized by Hawkins and Trongtham. From the results of high intensity reversed cyclic loading tests recommendations should be developed for limitations on the amount and distribution of prestressed and bonded reinforcement in the column area and for predicting the stiffness of such connections for lateral loading. Particular attention should be paid to slab-exterior column connections for which specific recommendations should be made for location of tendon anchorages and for assessment of the effective prestress forces acting on the critical section for shear.

Frames--As discussed in the section on beam elements critical in flexure Nakano [19] has reported the only test on a full frame. The measured and predicted values for the first mode of vibration and the natural period agreed closely. Damping prior to cracking was negligible and only seven percent after cracking. Torsional cracking of the edge beams transverse to the direction of loading occurred earlier than expected and reduced the rigidity of the floor system. There was no sharp change in stiffness with cracking and only a slow continuous decrease in the post-cracking range. The natural period increased slowly with increasing post-cracking loads and shortly prior to the formation of the first plastic hinge in the frame the period was only 30% greater than the initial value. Nakano concluded that prestressed concrete frames are likely to have seismic response characteristics very different to those of reinforced concrete frames. Under a seismic loading that would stress the latter inelastically, the prestressed frame will probably still respond elastically. His structure did not go inelastic until deflections of the order of twice the maximum deflection ordinarily permitted for steel frames in Japan. Details of the frame in the direction of loading, together with the lateral laod-deflection diagram for each floor are shown in Fig. 6. In the subsequent discussion Nakano used the lateral load-deflection test result for a one story frame shown in Fig. 7 as evidence that adequate ductility and energy absorption can be provided by appropriate design of prestressed frames.

Modeling of Load-Deformation Results

The two existing methods [17, 20] for theoretically predicting measured load-deformation results have been succinctly reviewed by Park [14]. In their 1969 investigation Paranagama and Edwards [20] utilized a variable strain compatability factor F to obtain agreement between the measured and predicted results for their pretensioned beams. The factor F related the steel and concrete strains at the level of the prestressing steel. Other assumptions were a linear distribution of concrete strains over the depth of the member, and known loading-unloading stress-strain relationships, respectively. The value of F required to achieve agreement with the measured moment-curvature envelopes was high initially and only tended to unity at ultimate load. Use of the same F factors did not yield good agreement with the measured loading-unloading curves. Agreement for that case required use of another set of F factors. In retrospect it is apparent that the poor agreement between measured and predicted results was due primarily to inadequate information on appropriate stress-strain relationships for the concrete and steel. Blakeley and Park [17] developed more accurate models of measured loading-unloading stress-strain relationships for concrete and prestressing steel and utilized them, without having to resort to any compatability factor, F, to predict the moment-curvature relationships they measured in their tests [23]. The good agreement they obtained is apparent from Fig. 8 which shows in Figs. 8(a) and 8(b) theoretical and experimental curves for sub-assumblages with hinges in the beam and column respectively. Because of that agreement they concluded that neglect of bond slip effects was reasonable. However, they noted that to obtain reasonable agreement it was essential to include hysteresis effects in the idealized stress-strain curves for the steel. Even then the measured hysteresis loops in the elastic range were still slightly greater than the theoretical. They attributed that effect to incomplete closure of existing cracks. Based on their test results and their modeling work, Blakely and Park proposed a three stage idealized moment-curvature model with stage representing behavior before crushing, behavior after crushing in one direction and behavior after crushing in both directions. Their modeling work predicts that until the concrete crushes elastic recovery effects dominate so that hysteresis loops have the shape shown in Fig. 1C. Significant energy dissipation and therefore alteration in the shape of the hysteresis loops does not develop until after the concrete crushes.

While reasonable models have been developed for elements critical in flexure, prediction of the complete response of frames will undoubtedly need models for elements on which high shear and bond forces act in addition to flexure, models for elements with unbonded tendons and partially prestressed sections, models for beam-column connections, models for slab-column connections and models for torsionally distressed elements. The development of these models should be deferred until adequate experimental data are available.

Analytical Studies

Prestressed concrete has not won rapid acceptance as a lateral load resisting material in seismic zones because of legitimate fears about the stiffness and energy dissipation of structures built from it. Despeyroux [30] has noted that flexibility is not a defect since the cross-section can be modified to compensate for it. The defect is not knowing appropriate values for the stiffness of a wide variety of sections nor the rate at which stiffness degradation is likely for those sections. As discussed previously, Rosenblueth [67] has noted that prestressed concrete structures may well have to be designed for load factors higher than those for structural steel and reinforced concrete. The more detailed work of Blakeley and Park [55] supports that contention as does the Japanese approach of using a higher load factor for prestressed than for reinforced concrete [70].

The earliest theoretical study was that of Spencer [21] who used a step by step integration technique to examine the non-linear dynamic responses of two reinforced and six prestressed concrete versions of a twenty story frame structure subjected to the first eight seconds of the N-S component of the El-Centro earthquake. The structure analyzed was that discussed by Clough and Benuska [73]. For the prestressed concrete frames the end momnet-end rotation hysteresis loops for lateral loading were idealized as shown in Fig. 1C. Cracking moments were taken as twice and six times the design moments for girders and columns, respectively. For the reinforced concrete frames hysteresis loops were idealized as shown in Fig. 1B with a strain hardening stiffness in the inelastic range. A special model beam was used for each frame member. That beam could have moment-rotation loops like those in either Fig. 1B or 1C. For prestressed concrete members ductility factors were expressed in terms of the curvature at cracking. The flexural rigidities and the cracking moments for the prestressed concrete members were taken as the same as the flexural rigidities and initial yield moments for the reinforced concrete members. For structures with the same mass proportional viscous damping the lateral displacements were up to 50 percent greater and the interstory drifts up to 70 percent greater for the prestressed concrete frame. However, maximum ductility demands for members were 40 percent less and for rotations 70 percent less for the prestressed structure. Other variables examined for the prestressed structures were: (1) the use of interfloor viscous damping rather than mass proportional viscous damping; (2) the simultaneous use of both types of damping; (3) a doubling of the width of the hysteresis loops to allow for energy dissipation by walls, floors, partitions etc.; (4) a doubling of the post-cracking stiffness of the prestressed members; and (5) a reduction in the column cracking moments to only twice the design moment. All changes except that of increasing the hysteretic damping resulted in little improvement in the behavior of the prestressed frame. Spencer concluded that while prestressed concrete structures could be designed to withstand large earthquakes without significant structural damage, large amounts of non structural damage would be likely unless cladding, partition and fixture requirements were adjusted appropriately. Further allowance would have to be made for the large compressive forces likely in the exterior columns.

In a second paper [52] Spencer analyzed the effect of assuming that the non-structural elements of his twenty story prestressed frame had varying stiffnesses and frequency-independent force deflection moments. The non-structural elements were assumed to exert equal and opposite horizontal forces on adjacent floors and their hysteresis effects defined by Ramberg-Osgood functions that either provided significant energy dissipation or little energy dissipation. Two specific yield stresses (high and low) were used with those functions but no specific provision made for the sudden loss in stiffness and strength that would occur with brittle non-structural damping. Spencer concluded there could be severe consequences if the stiffness of the non-structural elements were reduced by failures part way through the dynamic response. Otherwise non-structural elements were very effective in reducing inter-story drifts. The more effective elements were those which were relatively stiff and non-yielding. They reduced interstory drifts to one half to one third those of the same frame without non-structural elements. The effectiveness of the non-structural elements decreased as their stiffness and yield strength decreased so that more energy was dissipated by hysteretic effects. The inclusion of non-structural elements or variation in the properties of those elements had little effect on column shears. Omitting elements for every third floor over the height of the structure resulted in almost complete loss of drift control by those elements. However, omission of those elements in the central four floors of the structure had no serious consequences.

In a 1972 paper, Parme [44] outlined the concepts underlying American practice in seismic design and indicated how those concepts were applicable to prestressed concrete construction. He pointed out that for the same np value (modular ratio times reinforcement ratio) the ratio of the rotation induced in a cracked section to that induced in an uncracked section is much higher for stressed concrete than for reinforced concrete. The marked decrease in stiffness with increasing moments above that for cracking means that for prestressed concrete rotational ductility demands are best related to conditions at cracking. For a reinforced concrete structure if an overall translational ductility of about four is required for the frame then a rotational ductility of about 16 is needed for the girder. In contrast, in a prestressed concrete frame ductility demands are nearly all likely to occur within the elastic cracked range of the section. Translational demands are typically halved and rotational demands lowered accordingly. For prestressed concrete, if the building is made stiff enough for earthquakes, it is difficult to also make it flexible enough to accommodate dimensional changes due to temperature, creep and shrinkage. In their 1974 paper Paz and Cassaro [69] indicated how restrained dimensional changes may be taken into account analytically in prestressed concrete seismic response predictions.

In the 1970 AIJ report [70] the problem of the compatability of prestressed concrete framing with finishes and lateral load resisting elements of other materials is highlighted. The longer natural period and lower damping characteristics of prestressed concrete may make the use of a prestressed concrete frame and reinforced concrete shear walls, or prestressed shear walls and a reinforced concrete frame, incompatible. Fractures develop at the junction between the two materials due to their different vibration characteristics.

In one of the more illuminating investigations to date, Blakeley and Park [55] compared the predicted dynamic responses of a single degree-of-freedom portal frame structure when that structure was built from: (a) an elastic material, (b) prestressed concrete; (c) an elasto-plastic material and (d) a degrading stiffness material conforming to Clough's model. The structures were subjected to the N-S component of the El-Centro earthquake and were designed for a lateral force of 1.4 times the seismic load specified in the New Zealand Code. The prestressed concrete model was assumed to have the three stage idealized moment-curvature relationship discussed in the Section on modeling. Variations were made in the percent damping and the period of vibration for the building. Shown in Fig. 9a are displacement response curves and in Fig. 9b load-displacement curves for the structure with 2% damping and a 0.9 second period. As typified by these results the prestressed concrete system usually had a greater effective period of vibration and was subjected to greater amplitudes of vibration than the other two displacement systems. The curves in Fig. 9b clearly show the number of excursions into the inelastic range for each system. The prestressed concrete structure is obviously less heavily shaken than the other two non-linear systems. Shown in Fig. 9c are the maximum displacement ductility factors required for each system for 2% and 10% damping. Ductility factors for prestressed concrete are defined with respect to conditions at first cracking whereas those for the other two non-linear systems are defined with respect to conditions at first yield. The decrease in the displacement ductility demand with increasing period is an important effect. It favors the more flexible construction usually found with prestressed concrete. Also studied were the hysteretic energy dissipation of the prestressed concrete system and section curvature requirements for prestressed concrete. The hysteretic energy dissipation of the prestressed concrete system was still significant, relative to the degrading stiffness system in spite of the small area of the hysteresis loops, because of the average amplitude of the vibration cycles was greater for the prestressed case. Section curvature requirements could be readily satisfied when the steel areas and axial load levels were kept small. It is concluded that under seismic loadings prestressed concrete frames will generally have maximum displacements about 1.4 times those of reinforced concrete frames with the same strength, initial stiffness and percentage damping. While such prestressed structures can be made adequately ductile by keeping steel areas and axial load levels low, it may be wise to increase the design earthquake loads for prestressed concrete by about 20% in order to reduce damage to non-structural components. Alternatively the load factors could be left the same and drift requirements made more stringent than for reinforced concrete frames. That approach is, however, less reasonable since the advantages of the prestressed concrete structure are then largely lost.

Future analytical studies should concentrate on determining the likely response of a wide range of prestressed and partially prestressed concrete structures subjected to a variety of earthquake motions in order to better define likely ductility demands and possible problem areas in satisfying section-curvature requirements. A thorough investigation is needed of the appropriateness of making the responses of prestressed concrete frames more compatable with those of structural steel and reinforced concrete by increasing the load factor of prestressed concrete for earthquake loading or tightening its drift limitations.

PRECAST CONCRETE

Background

Precast concrete framing is not widely used as a primary lateral load resisting system in seismic zones within the United States [54] although prior to the publication of the 1976 Uniform Building Code it was sometimes used under the K equal to unity provisions [74] for non-ductile frames of buildings less than 160 ft. in height. Precast panel construction, particularly of the tilt-up type is frequently used for low rise structures but not for multistory structures. In contrast such construction is widely used in earthquake regions in Japan [64, 70], Cuba, and Eastern Europe [34, 41, 51]. Even where advantage might be taken of precast wall panels as stiffening elements, U.S. building code provisions usually work against the designer [47]. Part of the problem in precast panel construction in the U.S.A. is one of economics. In order to minimize job site operations and meet consumer demands the optimum size for U.S. units is typically larger than in other countries. With fewer connections assurance of the integrity of every connection for seismic loading becomes more critical. The lack of knowledge on the likely seismic behavior of typical large panel connections has made designers reluctant to use such construction in U.S. seismic zones.

Many of the problems associated with precast panel construction have been pointed out in References [6] and [70]. While it is preferable that the gravity loads of floor units counteract lateral loading uplift forces such construction creates camber problems between adjacent slabs spanning in different directions. Expansion joints are also a problem and must be arranged so that hammering is prevented. Probably the main problem is the interconnecting of precast floor units to achieve diaphragm action. Mechanical ties between elements in grouted keyways are usually necessary unless a properly reinforced cast-in-place topping is used. Where the floor slab area is large and the out-of-plane rigidity of the surrounding frame is inadequate, the floor system tends to deform as indicated by the broken line in Fig. 10 and peripheral ties are necessary to minimize that action. For vertical elements continuous ties are necessary from floor to roof. Their size must be not less than some minimum (generally 3,000 lb/in-ft of wall in the U.S.A.), there must be at least 2 ties per panel and their size must always be adequate to resist any uplift. Post-tensioning is often also used to resist uplift. Joints between vertical elements generally require grouting, mechanical ties and or keys.

The weak point in precast construction is the joints. While it is desirable that connections be ductile and stronger than the units joined, such conditions are extremely difficult to satisfy [35]. Unless cast-in-situ connections, grouting or post-tensioning are used, connecting elements must themselves be anchored in the precast panels and then those anchors frequently become the weak element [45]. The possibility of horizontal distortions due to torsional actions must be considered. Good practice calls for at least two lines of resistance on any plane and constraint against motion in any direction. At the same time connections must be flexible enough to accomodate dimensional changes due to temperature, creep and shrinkage and rotational effects due to drift. The simpler the design concept for the connection the simpler the connection and the greater possibility that the actual performance will mirror the predicted performance.

Joints

Precast connections can generally be divided into two broad groups, "load supporting connections" and "lateral connections." The former transfer the gravity loads of a unit while the latter restrain in-plane rotations of the unit. While many types of connections have been developed [56][75] including those utilizing embedded structural steel shapes, steel shapes anchored by studs, welded reinforcing bars or bolts, and connections made by post-tensioning or by grouting reinforcing bars, little simulated seismic testing has been conducted on such connections.

In 1976 Spencer and Neille [9] reported tests on six welded headed stud connections. Details of their specimens are shown in Fig. 11a. Reversed cyclic shear forces were applied in the longitudinal direction of the angle iron, as shown in Fig. 11a, at a distance 7/8 in. from the face of the angle. The loading set-up allowed both pullout of the studs and rotation of the connection about a horizontal axis. Connection A1 was loaded monotonically to failure while the other five connections were cyclically loaded at frequencies in the range of 0.01 to 0.02Hz. Shown in Fig. 11b by solid lines are the hysteresis loops for the typical cyclically loaded specimen A3. The broken line, appropriately labelled, is the response for the monotonically loaded specimen A_1. The response for A_1 can be realistically idealized as elasto-plastic. Failure occurred by splitting along a plane outside the stud heads at a displacement of 0.5 in. For the cyclically load specimens hysteretic damping and strength degeneration with cycling was effectively negligible until the applied load exceeded about 80 to 85% of the static strength. Then the width of the hysteresis loops increased rapidly, taking on a characteristic S-shape. With further increases in displacement the stiffness decreased and with cycling between constant peak deflections the capacity and the stiffness decreased from what is termed the yield envelope in Fig. 11b to the stability limit. The latter was the minimum capacity for a large number of cycles. Failure finally occurred at about one third of the displacement and one third of the capacity of the monotnically loaded specimen. Actual values for failure of individual cyclically loaded specimens varied widely dependent on their loaded histories. One of the cyclically loaded specimens failed by splitting of the concrete outside the stud heads in the same manner as specimen A1. Although that cracking occurred in all specimens, three of the other four specimens failed by shearing at the fusion welds while the fourth failed where a panel reinforcing bar was tack welded to the stud shank.

The reversed cyclic load response for load and displacement levels considerably less than those for the maximum capacity has been studied by Yamada [78] and Uchida [79] with similar findings. Results for Uchida's tests on 3/4 in. diameter studs embedded in 3,000 psi lightweight concrete are shown in Fig. 11c. Ten cycles were applied to three specimens, A, B and C at load levels equal to about 50, 75 and 25%, respectively, of the monotonic loading capacity. For clarity, only the loops of the first and tenth cycle are shown on Fig. 11c. For all levels of loading the hysteresis loops were spindle shaped for the first half cycle only and thereafter became progressively more S-shaped with cycling. Although slips increased with cycling, the loops for all three load levels became stable and converged on a certain limiting position. Thus, the stiffness decreased with cycling. The average decrease in stiffness between the first and tenth cycle was 30% for load levels A and C and 50% for load level B.

Additional insight into the response of Spencer and Neille's specimens is provided by recent testing at the University of Washington [76]. Specimens of the type shown in Fig. 12 have been loaded to failure monotonically and reversed cyclically. It hasbeen found that provided the M/Vd ratio at the face of the concrete is less than 0.5, failure under monotonic loading was by shearing of the studs. The quantity d is the distance in the plane of loading between the outer layers of studs, M the moment at the face of the connection and V, the shear on the connection. Where M/Vd was greater than 0.5, cracking developed, as shown in Fig. 12, beyond the head of the studs and within the body of the concrete. Failure due to pull-out of the top studs followed quickly. That failure was due to a prying action since the embedment depth for the stud was greater than that necessary to develop their tensile strength. While the provision of additional cross-ties, as indicated by broken lines in Fig. 12, did increase the strength for M/Vd ratios greater than 0.5, the stud embedment depth had to be made almost double that for development of the tensile capacity before the studs rather than the concrete failed. For reversed cyclic loading between constant peak loads and for studs with embedment depths double that required from tensile considerations, failure occurred suddenly at the fusion welds after about five cycles to 80 percent of the monotonic capacity. The hysteresis loops prior to failure developed the same characteristic S-shape as that reported by Spencer and Neille. A similar behavior has been observed at the University of Washington in reversed cyclic loadings of metal deck composite steel and concrete push-off specimens [77]. Undoubtedly the development of concrete cracking is a major factor affecting the mode of failure of any type of embedded stud or anchor. The maximum strength and the degeneration in that strength with increasing displacements depends primarily on concrete crack growth rates. Increasing the reinforcement in the cracking zone will restrain that growth but not alter the basic stiffness and strength degeneration characteristics. A significant number of headed stud and bolt anchors have imperfections and fail in a brittle manner at the fusion weld or in the bolt thread after only a limited number of reversed cyclic loadings to about half the stud or anchors capacity. A common failure rate is 10% of the test specimens in a batch. Thus a redundancy rule must be applied to the design of such connections. Fifty percent of them should be capable of carrying the maximum credible load on the unit.

A second means of transferring shear and low level axial or bending stresses between units is by interface shear transfer effects. If the plane under consideration is an existing crack or an interface, failure in this mode involves slippage along the crack or interface. If the plane is located in monolithic concrete a number of diagonal cracks develop across the interface and failure involves a truss action along the plane. If there are high intensity reversals of shear at loads only slightly less than those for failure, a pattern of crossing diagonal cracks occurs. For either monolithic or precracked concrete satisfactory shear transfer behavior requires some clamping action across the shearing plane. One means of providing that clamping force is by transverse steel. As movement develops along the plane and cracking develops, the two side of the crack along the plane ride up on each other. That action tries to open the crack and stresses the transverse steel. The shear for failure is proportional to the clamping force across the crack. With deformed bar reinforcement and no direct stresses across the crack, the clamping force equals ρf_y (transverse reinforcement ratio multiplied by its yield strength).

While many researchers have contributed to the current understanding of interface shear transfer behavior [43, 81] only the comprehensive investigations of Mattock and his co-workers are discussed here since their work represents the main body of existing knowledge for shear transfer with bonded reinforcement crossing the shear plane [81, 82, 83, 53, 57, 58]. For monotonic loading to failure and a pre-existing crack less than 0.025 in. wide along the shear plane, the shear transfer strengths, v_u, are conservatively predicted by expression $[400 + 0.8(\rho f_y + \sigma_{Nx})]$ psi but not greater than $0.3f'_c$ for sand and gravel concrete, by $[250 + 0.8(\rho f_y + \sigma_{Nx})]$ psi but not greater than $0.2f'_c$ or 1,000 psi for sanded lightweight concrete weighing not less than 105 lb/cu.ft., and by $[200 + 0.8(\rho f_y + \sigma_{Nx})]$ psi but not greater than $0.2f'_c$ or 800 psi for all lightweight concrete weighing not less than 92 lb/cu.ft. The quantity σ_{Nx} is the normal stress acting across the shear plane and is positive when compression and negative when tension. Where the ρf_y value is less than 200 psi, crack widths readily develop in excess of 0.025 in. and the capacities quoted above should be reduced linearly to zero for lesser ρf_y values. The use of f_y values greater than 60,000 psi is also not recommended. Where moment equal to or less than the flexural ultimate strength of the section has acted simultaneously across the crack the shear that can be transferred has not been below that discussed above. However, in order for the shear transfer reinforcement to be fully effective at least two thirds of it should be located in the flexural tension zone. Where lightweight concrete has been used the type of lightweight aggregate, coated or crushed, has not significantly affected results. Tests on specimens with the reinforcement crossing the shear plane at an angle have shown that when the component of the shear force parallel to the reinforcement tends to produce tension in that reinforcement then the shear capacity can be taken as the sum of $\rho f_y \cos \gamma$ plus $\sin \gamma$ times the strengths quoted previously where γ is the angle between the inclined bars and the direction of the shearing action and has values between zero and ninety degrees. Changes in strength, size and spacing of reinforcement affect the shear strength only insofar as they affect the reinforcement parameter ρf_y. When the interface lies between precast and cast-in-place concrete the strength is very sensitive to the roughness of the interface. Ninety to ninety-five percent of the ultimate shear strength of a cracked non-composite member can be attained provided the interface is deliberately roughened to a depth of at least one quarter of an inch and all laitance removed. In that case the use of a bond breaker does not affect the capacity although the maximum load is reached at a slightly greater slip than the same specimen made non-compositely. If the interface is smooth, 50 percent of the ultimate strength is lost and the shear capacity drops to the shearing yield strength of the reinforcement crossing the interface.

Mattock and his co-worker's interface shear transfer reversed cyclic loading tests [53, 57, 58] have been made using the set-up shown in Fig. 13a to test specimens of the type shown in Fig. 13b. Both the slip along the central shearing plane and the separation across that plane were measured continuously using linear differential transducers attached to the center of the specimen. Varia-

variables considered to date have included: (a) the load history, monotonically increasing, cyclically reversing, and monotonically increasing following cyclic reversals; (b) the width of the initially induced crack, 0.010, 0.015 and 0.025 in.; (c) the ρf_y value for the reinforcement crossing the shear plane, 360 to 660 psi; (d) the type of aggregate, sand and gravel, and lightweight; (e) composite specimens with the same or different strength concretes on opposite sides of the shear plane, 3,000 and 6,000 psi; and (f) the effect of bond on the behavior of composite specimens.

The response of a typical shear transfer specimen MC-1 is shown in Fig. 14. Diagrams (a) through (d) show representative shear-slip hysteresis loops and diagrams (e) through (h) show the corresponding slip-separation curves. Three factors contribute to the shear resistance; (1) direct bearing of small asperities interlocking along the crack, (2) friction between the adjacent crack faces, and (3) dowel action of the reinforcement crossing the crack. In Reference [53] the manner in which the known response characteristics for each of those three factors can be combined to give the observed shape of the shear-slip curves is documented. Initially and in early loadings to an increased maximum most of the shear resistance is provided by the first factor. However, with cycling and increasing load those asperities are sheared off. The stiffness for low shears becomes very small and the resistance at high shears is provided primarily by the second and third factors. The shear-slip curves become almost Z-shaped. As the maximum load in the cycle increased both the slip and the separation at the maximum shear increased. However, until the last few cycles before failure the separation at zero shear remained almost constant so that the slip-separation curves took on a characteristic quarter moon shape. In the last few cycles the separation at zero shear increased with cycling, the shear stiffness began to decrease approaching the maximum load, and slips and separations at maximum load increased significantly with each cycle.

As apparent from Fig. 15 for cyclic loading the slips were greater and the separations less than at the same magnitude of shear in companion monotonic loading tests. Further, there was a considerable decrease in the maximum strength as a result of cycling. The decrease increased with increasing initial crack width above 0.015 in. and was more for lightweight concrete than sand and gravel concrete. The typical reduction in strength was 20% for 0.015 in. and less crack widths. As the width of the initial crack increased the shear stiffness decreased and the slip increased for all levels of loading. The cyclic load response of lightweight concrete specimens was considerably poorer than that of sand and gravel concrete specimens. For all load levels the separation at a given slip was less for lightweight concrete reflecting the greater smoothness of its cracked interface compared to that of sand and gravel concrete. As a result the lightweight concrete specimens were never able to develop stable shear-slip curves for cycling between constant peak loads. When previous cycles of loading had not caused major damage so that the separation at zero shear had not started to increase, the shear strength for a subsequent monotonic loading to failure was unaltered. The composite interface shear transfer results have highlighted the importance of bond along an interface even when there is adequate surface roughness. When a bond breaker was used [57] dowelling and bearing effects were lost so that the slip was greater and the shear stiffness less at all load levels than for an interface without a bond breaker. Effectively the shear resistance dropped to ρf_y for a roughened surface and $0.6\rho f_y$ for an unroughened surface. However, without a bond breaker and with a surface roughened to a depth of at least ¼ in., the capacity and shear-slip relationships for cyclic loading are comparable to those for cracked non-composite specimens [58]. The abruptness of the failure for both monotonic and cyclic loading decreased as the concrete strength, the degree of bond and the surface roughness decreased.

The available results indicate conclusively that interface shear transfer is not a good absorber or dissipator of energy for cyclic loading even though high shear stresses can be developed across an interface with appropriately detailed reinforcement. This fundamental research provides considerable insight into the mechanisms of shear transfer as well as providing information directly applicable to joints in both precast and cast-in-situ concrete. Research is needed on cyclic loading interface shear transfer effects due to interface size and configurations, bar size, direct stresses normal and parallel to the interface, reinforcement paralleling the interface and high strength reinforcement crossing the interface. Most of the tests conducted to date have used load control devices so that the post-maximum cyclic response characteristics have been unobtainable. Future tests should define those characteristics for the variables examined to date as well as those listed above.

An excellent summary of information on many other types of joints is provided by Reference [54]. The types of connections in use vary widely. Most are designed empirically and have not been studied experimentally or theoretically. Their ultimate strength and likely seismic loading behavior are unknown. Further, where testing has been conducted the characteristics of supposedly identical specimens often vary widely so that the establishment of consistent as well as predictable behavior is a major problem. In view of current U.S. practice research is particularly desirable on the characteristics of post-tensioned joints and joints utilizing embedded steel sections [75]. Some understanding of likely behavior for the latter case can be obtained from Reference [80]. Different types of precast joints in common use in Japan are detailed in Reference [70] while those recommended for low rise precast wall panel apartment buildings are detailed in Reference [65].

Experiments

Components--There have been few tests on prefabricated sub-assemblages. In Reference [41] results are described of analytical studies and about one third scale tests of a thin-walled spatial box unit system. The construction consisted of bearing wall box-cap units with an attached structural concrete floor panel and in fill suspended end wall panels. Units were connected by monolithic vertical reinforced concrete joint tongues and supported at the corners only by those joints. Coupling between box units was by welded or cast-in-place joints. The cyclic load tests showed that from early on the units worked non elastically and that there was considerable redistribution of stresses between units. To correctly assess the stiffness of the models account had to be taken of the stiffness of the suspended wall panels. The main problems were control of the skewness of the boxes and inadequate performance of the box unit to wall connections.

Reversed cyclic load tests on half and full scale lightweight concrete wall panels (f'_c = 600 psi) are described in Reference [62]. The single and multiple panels were connected at their ends by transverse members. The response depended strongly on the fixation conditions for the top of the panels. As the number of panels in the direction of loading increased and as the degree of fixation at the top of the panels increased, the unit shear stress for failure increased and the mode of failure changed from flexure to shear. It was found that while cast-in-place collar beams could be replaced by precast beams and satisfactory performance still obtained provided joint details were carefully thought out, panels contained within or connected to a steel frame did not perform satisfactorily due to slip inducing local cracking at the ends of the panels.

Multistory Structures--As part of a program to develop 15 story precast concrete apartments for Japan, lateral load tests were made on a full scale model representing a one bay by one bay area of the lower three floors of a prototype structure [63]. The precast vertical elements in the direction of the reversing loads were connected by cast-in-place concrete girders and floors so that they formed portal frames. Wall panels were provided in the transverse direction. Axial loads on the columns simulated the missing upper portions of the structure. The order in which bending and shear cracks developed, and the initial elastic stiffness was in agreement with that predicted by elastic analysis. The stiffness between cracking and the plastic capacity was about one third the elastic stiffness and predictable by proper consideration of cracking and yielding deformations. At failure plastic hinges formed at the bottom of the first story columns and at the ends of the beams. All horizontal and vertical joints performed well and even after the structure was loaded well into the inelastic range there was considerable elastic recovery on removal on the lateral loads.

In another Japanese study [49] the lateral load results of Fig. 16 were obtained for six full scale multi-story frames representing parts of apartment buildings assembled from precast elements. The first structure was a five story precast large panel building, four panels wide [25]. The horizontal joints were made by welding together steel plates anchored in the panels. The cast-in-situ vertical joints contained reinforcing bars which were welded to other bars protruding from the panels. The structure had a capacity 7.6 times the design load and initially damping was 6.3%. After the structure reached its ultimate load and deformations were increased there was a considerable drop in capacity due to shear failure of the wall panels. It was concluded that the stiffness of this structure should be assessed assuming floor slabs to be rigid, panels connected by wet joints to behave monolithically and panels connected by dry joints to behave independently. The second structure was also five stories high but of large panel construction being only two panels wide. The structure had a capacity 4.8 times the design load and the coupling beams failed badly in shear before the maximum load was reached. The third and fourth structures were four stories high and two panels wide [26]. Vertical connections between columns and girders were made by post-tensioning and joints between floor slabs and between floor slabs and beams by welding together steel plates anchored in each element. In the longitudinal direction there was a precast framework of girders and columns. For one structure wall panels had a length about twice their height. In the other structure the wall panels were square. For the third structure horizontal slippage began at 1.2 times the design load so that for higher loads displacements were much greater than the theoretical. The maximum capacity was 3.9 times the design load with the failure mechanism being that of shear in the second and third floor beams. For the fourth structure vertical slippage between wall columns and wall beams began at twice the design load and the stiffness decreased rapidly. The maximum capacity was four times the design load and that capacity decreased rapidly with cycling due to shear crack development in wall columns and beams. The fifth and sixth structures were made with precast rigid frames in the longitudinal direction and precast shear walls in the transverse directions. Connections were made by bolting and grouting. For the fifth structure the ultimate load was 1.8 times the design and the behavior was very ductile with little reduction in capacity with reversed cycling. For the sixth structure the ultimate load was 2.1 times the design and there was a significant decrease in the capacity with cycling due to slippage of the longitudinal reinforcement in the first story column to column connection.

Other recent investigations include shaking table tests on one third scale multi-story structures [37] and forced vibrations of a 17 story building [40]. From all the testing conducted to date it is apparent that while the joints of the system are usually the weak point, the response also depends on the rigidity and strength of the members joined and those characteristics in turn depend on the method of jointing. While precast structures that behave in a ductile manner under cyclic loading can be developed by careful attention to details it is not easy to predict the strength and response of such structures. Not only is the analysis difficult, laboratory and field experience shows that the strength and the response are very dependent on the quality and consistency of the construction work. If precast concrete construction is to be widely used in seismic zones in the U.S.A. the precast industry must lead the way and develop recommended design concepts that are structurally sound and economically viable. The industry must then work with research organizations to prove the validity of those concepts for seismic loadings, to refine and simplify details and develop guidelines for seismic analysis. Until such steps are taken experiments on subassemblages and frames are not warranted.

Analytical Investigations

Analytical work oriented towards particular building systems has been conducted in Eastern Europe and Japan. Bukharbaev [34] has used a finite strip method to determine the strained three-dimensional behavior of box units and found predicted responses agreeing closely with field data. This method results in 20 to 40% increases in predicted rigidity for the longitudinal direction and up to 80% for the lateral directions as compared to plane stress or strain predictions. Okamoto and Yokoyama [39] have described the development of a building consisting of shear walls positioned three dimensionally in a checkered pattern and reinforced around their peripheries by frames. The viability of their design concept was proven experimentally. For analysis the frame is considered as composed of unit walls with each wall as a finite element and stiffness matrices are developed for the entire frame for a finite number of deformation stages. Kanoh [65] has provided a commentary on the AIJ Standard for structural design of low rise apartments and described how those design rules were evolved from full size tests on buildings.

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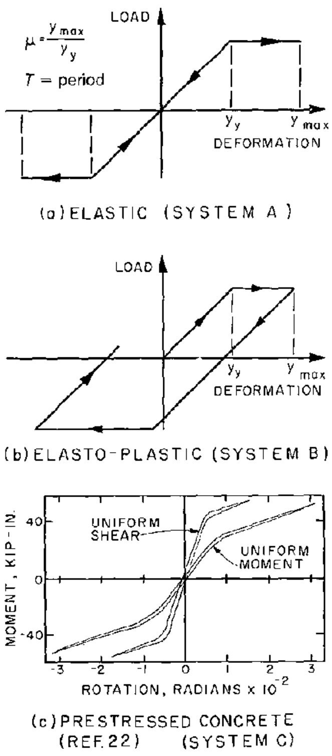


Fig. 1. Typical force-deformation response.

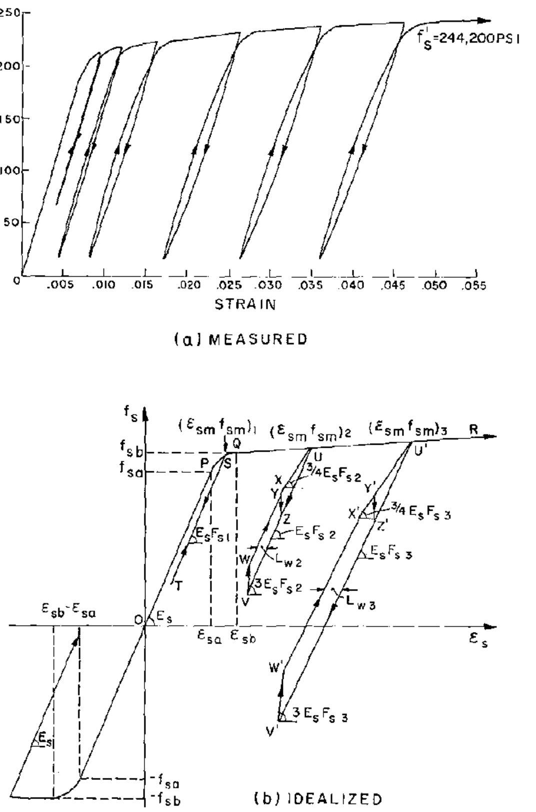


Fig. 2. Measured and idealized stress-strain relationships for prestressing steels (Reference 14).

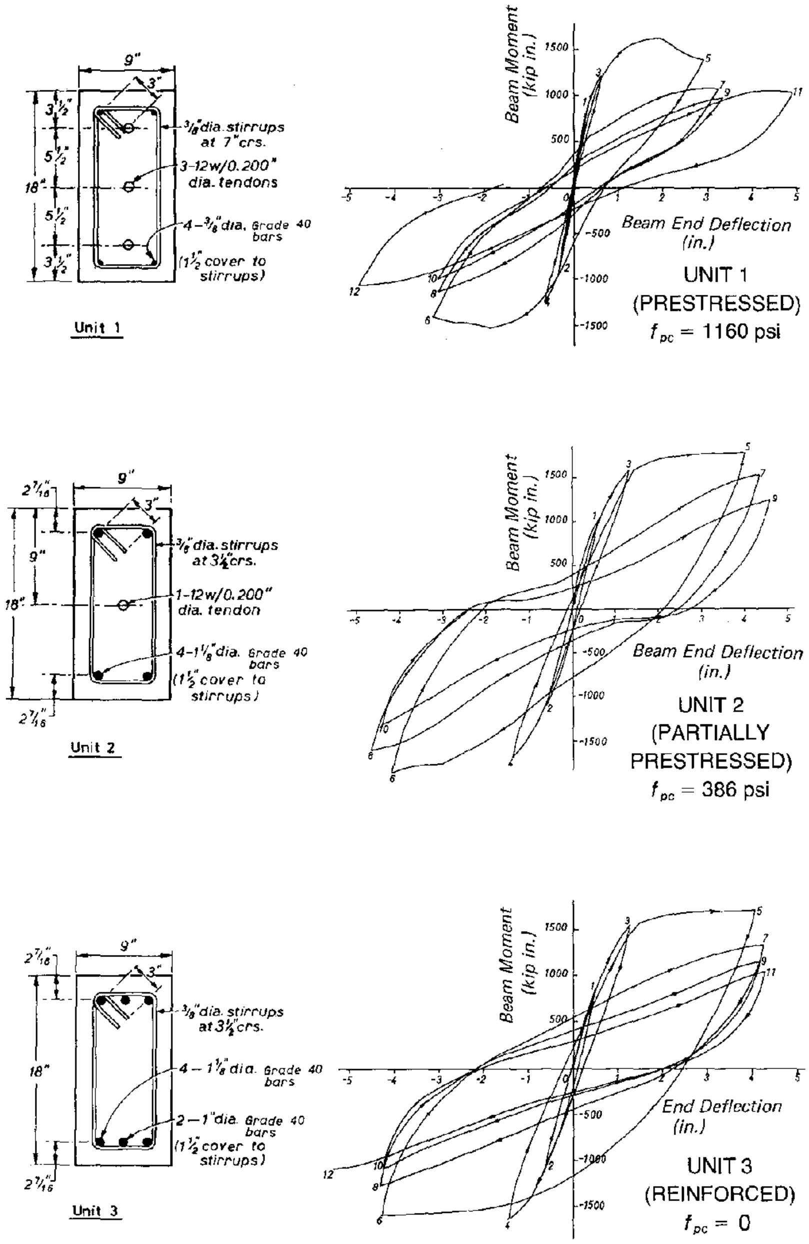


Fig. 3. Moment-end deflection relationships for beam-column subassemblages (Reference 61).

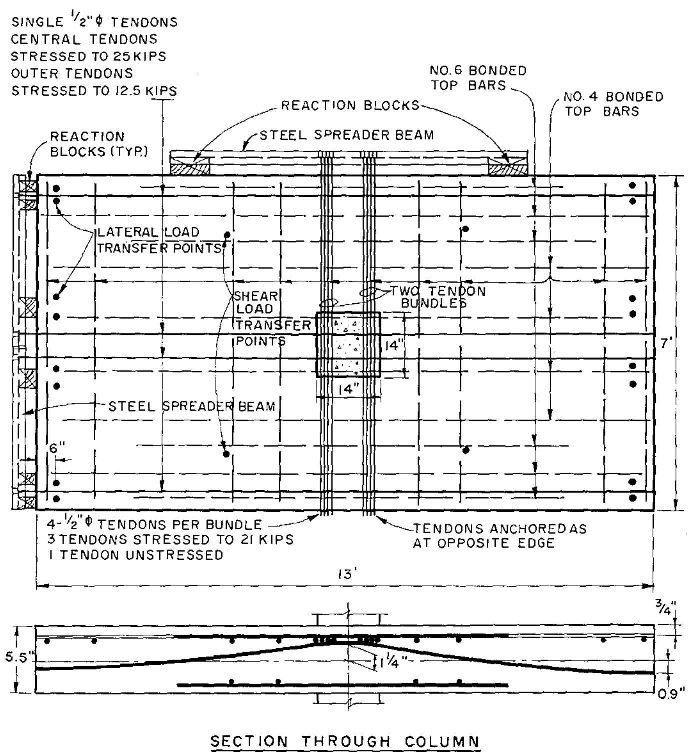


Fig. 4a. Dimensions and reinforcement for Specimen 1 (Reference 59).

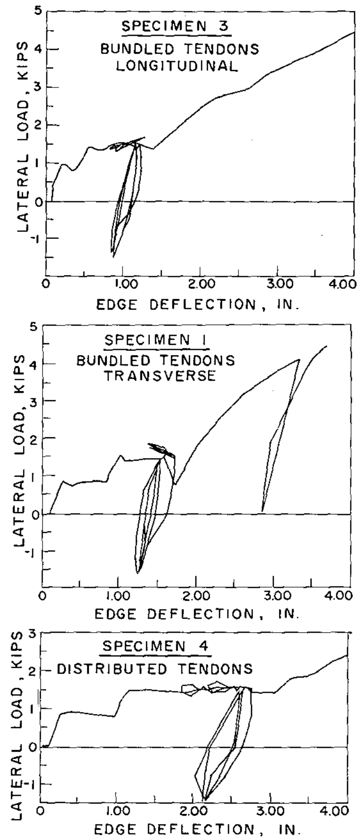


Fig. 4b. Lateral load-edge deflection relationships for slab-column subassemblages (Reference 59).

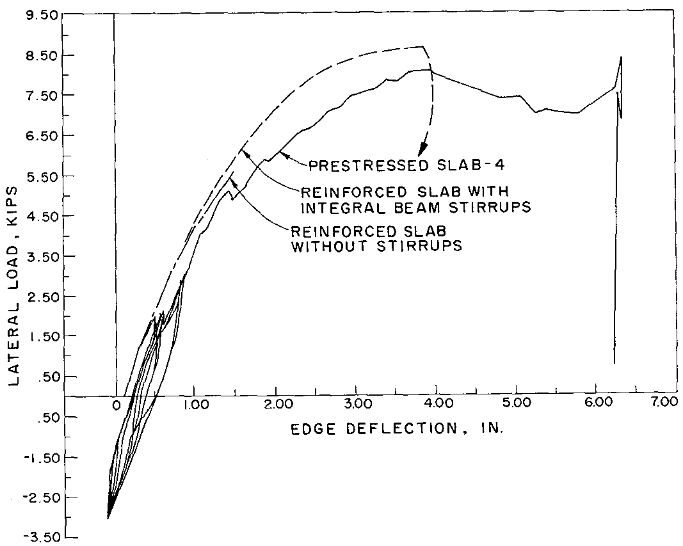


Fig. 5. Comparison of lateral load-edge deflection relationships for reinforced and prestressed concrete slab-column subassemblages (Reference 59).

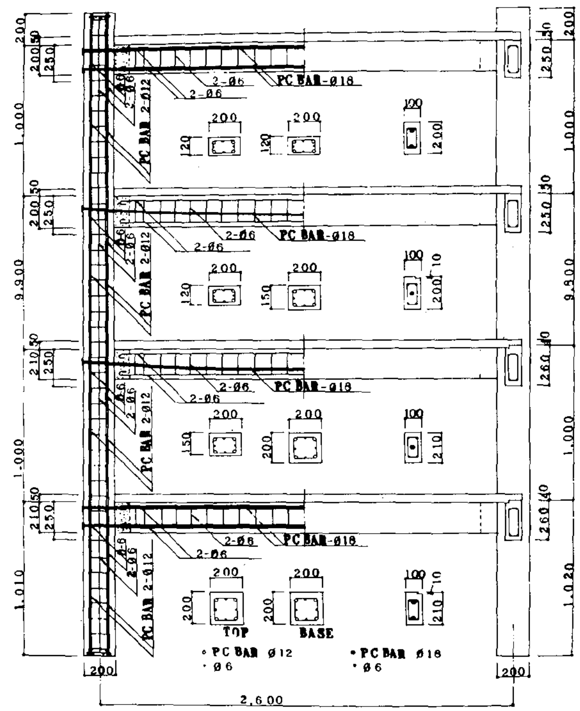


Fig. 6a. Dimensional and reinforcement details of four-story prestressed concrete model frame in direction of loading (Reference 19). For load response see Fig. 6b on next page. Note that main dimensions are in meters and cross-sectional dimensions are in millimeters.

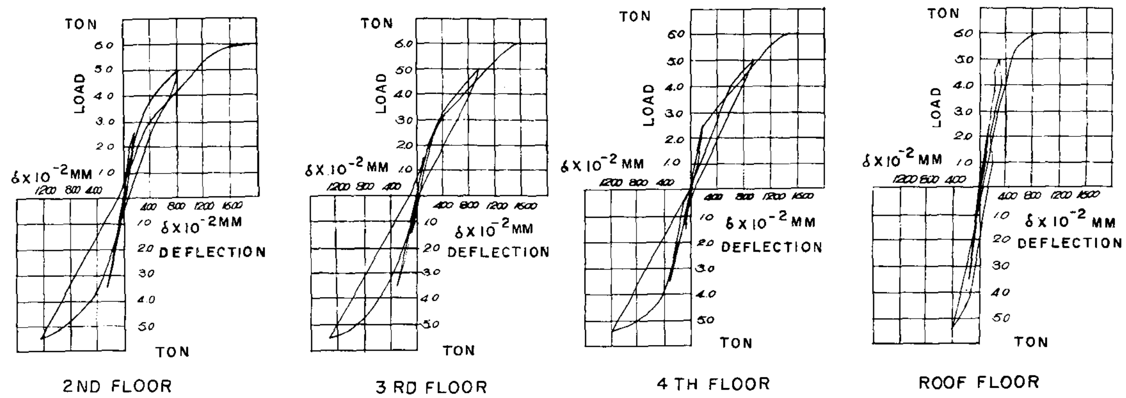


Fig. 6b. Lateral load response of four-story prestressed concrete frame (Reference 19). For details of frame see Fig. 6a on previous page.

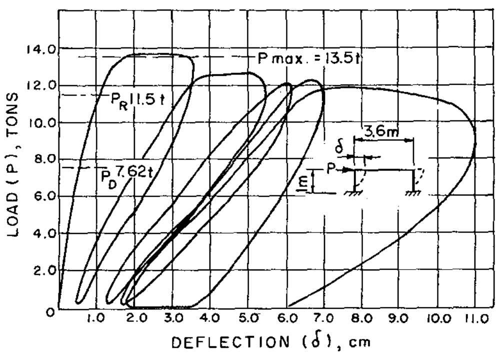


Fig. 7. Lateral load-deflection relationship for one-story prestressed concrete portal frame (Reference 70).

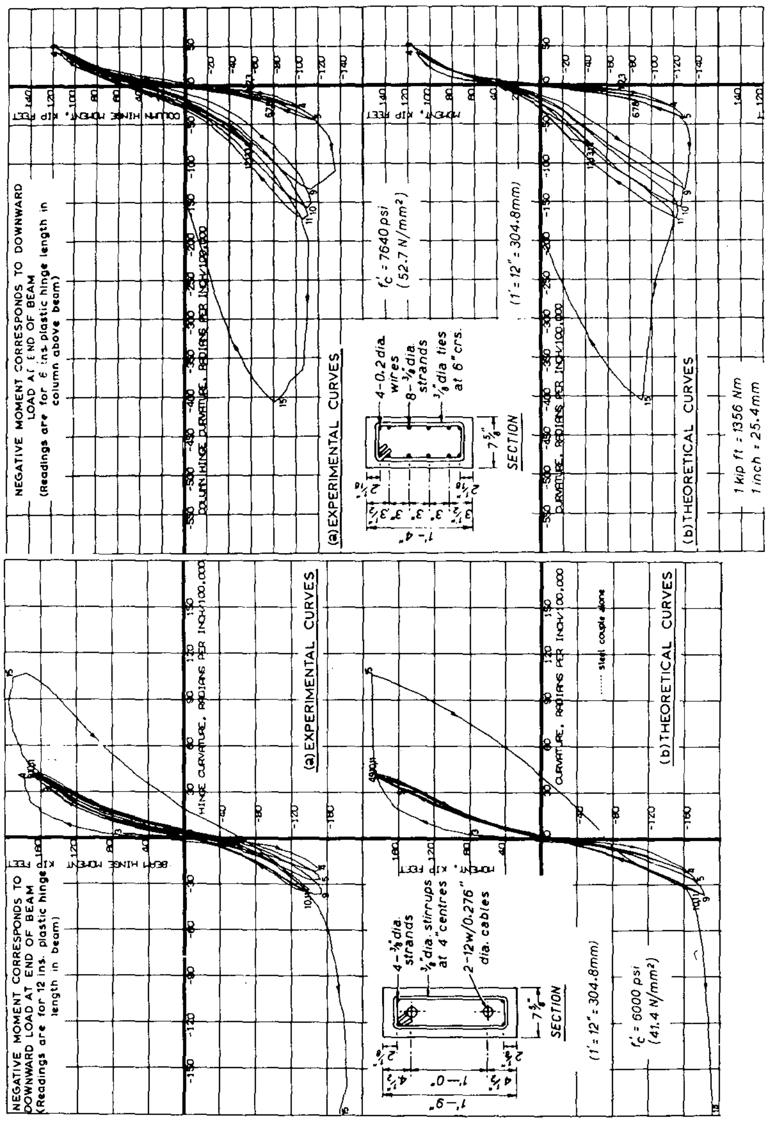
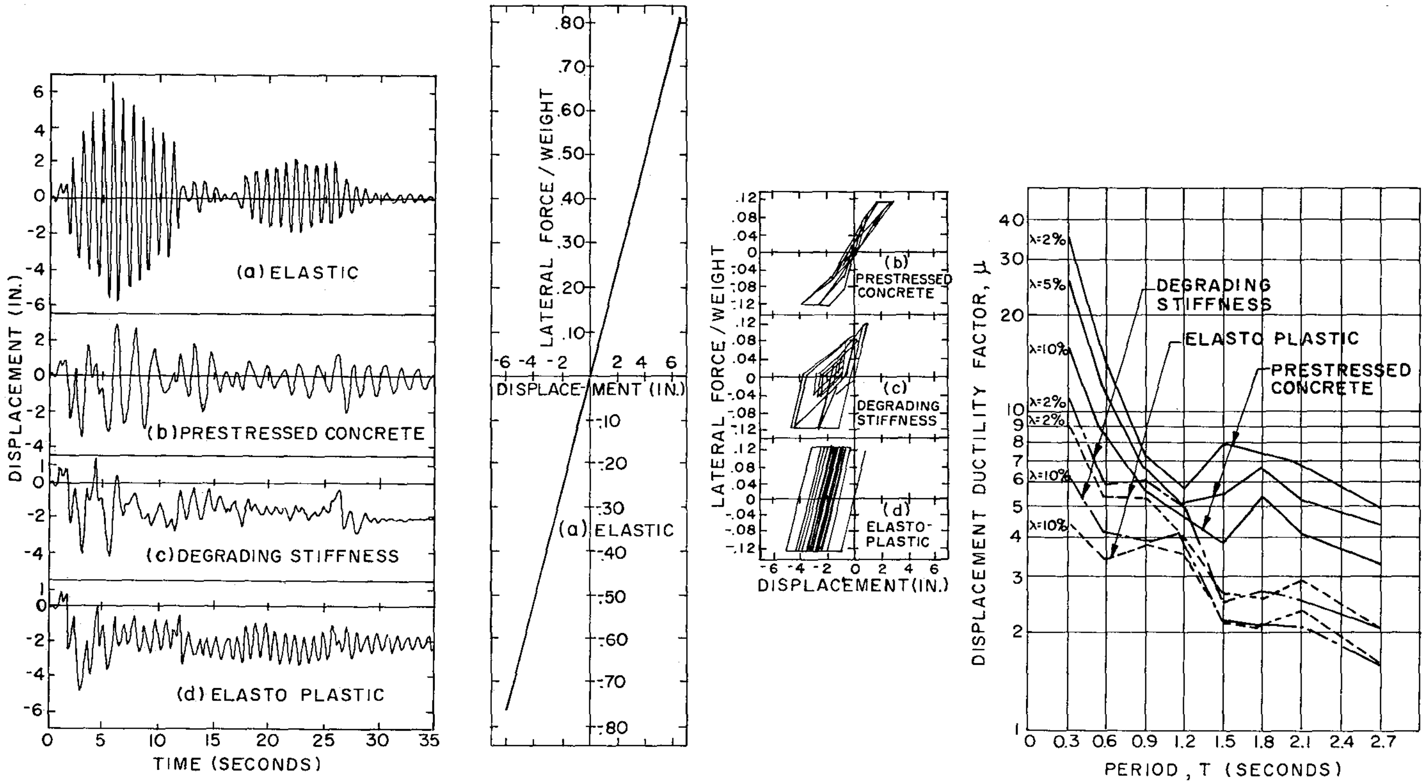


Fig. 8. Measured and predicted moment-curvature curves for beam and column regions of beam-column subassemblages (Reference 17).



(a) DISPLACEMENT-TIME RESPONSES

(b) LOAD-DISPLACEMENT RELATIONSHIPS

(c) MAXIMUM DISPLACEMENT DUCTILITY FACTORS

Fig. 9. Comparative responses of one story one degree of freedom portal frames with differing stiffness models (Reference 55).

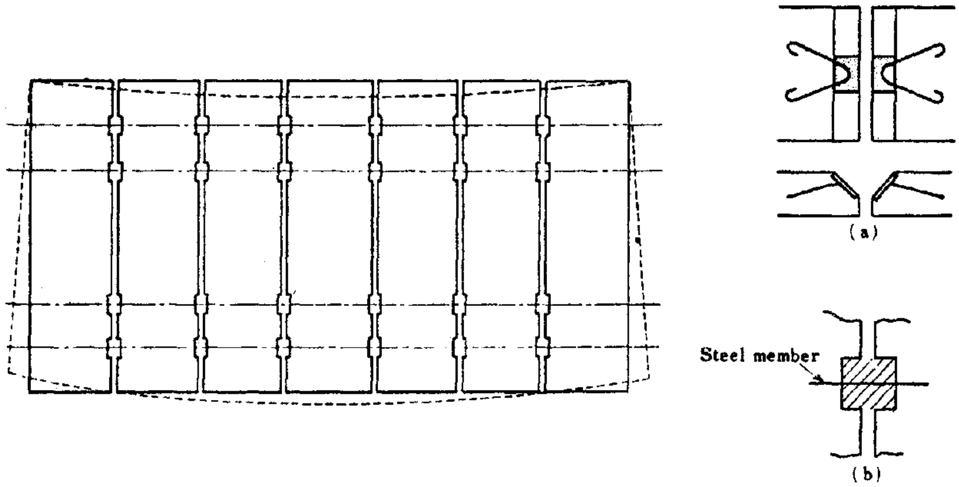


Fig. 10 DEFORMATION OF PRECAST FLOOR SYSTEM

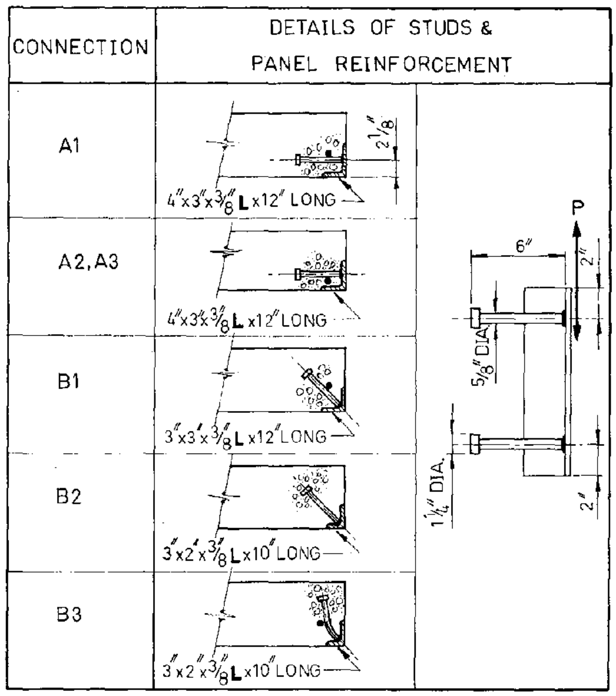


Fig. 11a DETAILS OF REFERENCE [9] SPECIMENS

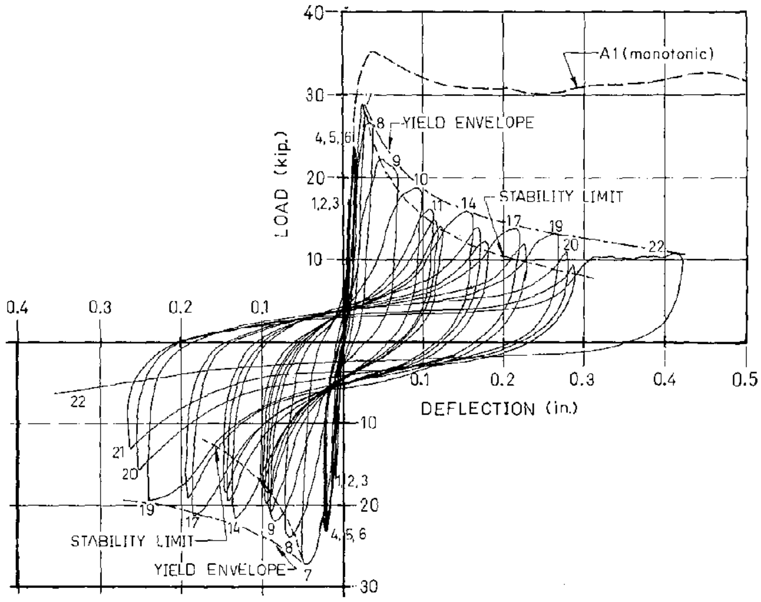


Fig. 11b RESULTS FOR SPECIMENS A1 AND A3 [9]

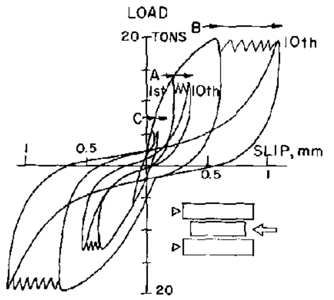


Fig. 11c UCHIDA'S TEST RESULTS [79]

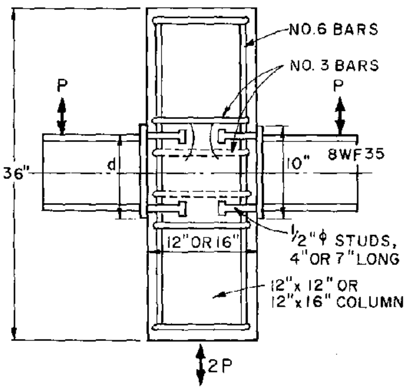


Fig. 12 HEADED STUD SHEAR TRANSFER TESTS [76]

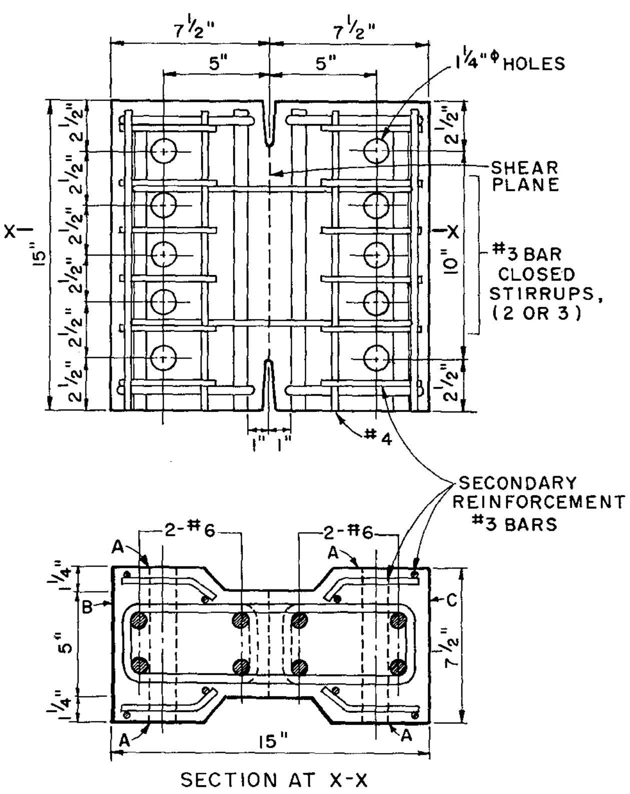


Fig. 13(a) TEST FRAME

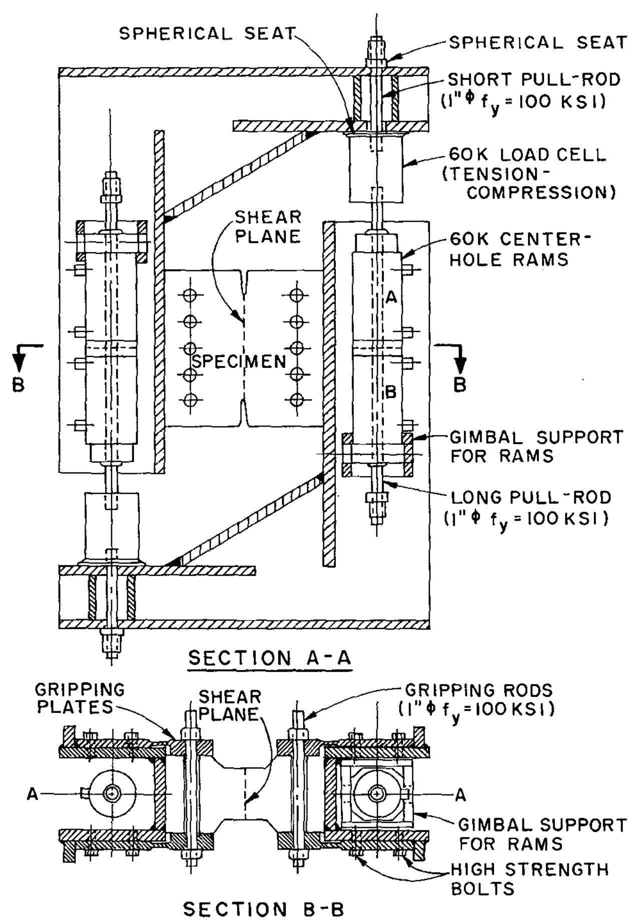
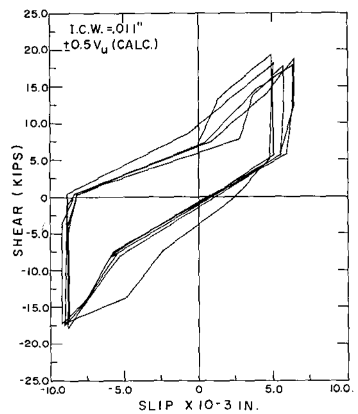
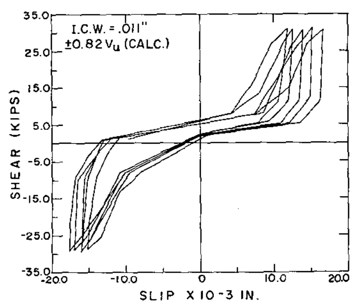


Fig. 13 SHEAR TRANSFER TESTS [53]

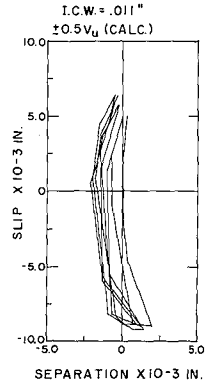
Fig. 13(b) TEST SPECIMEN



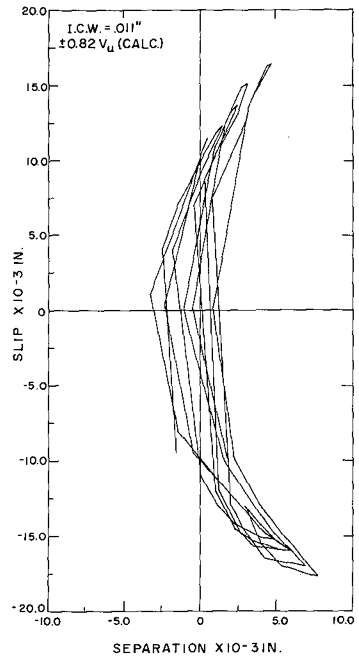
(a) SHEAR-SLIP CURVES, CYCLES 1-5



(b) SHEAR-SLIP CURVES, CYCLES 25-30

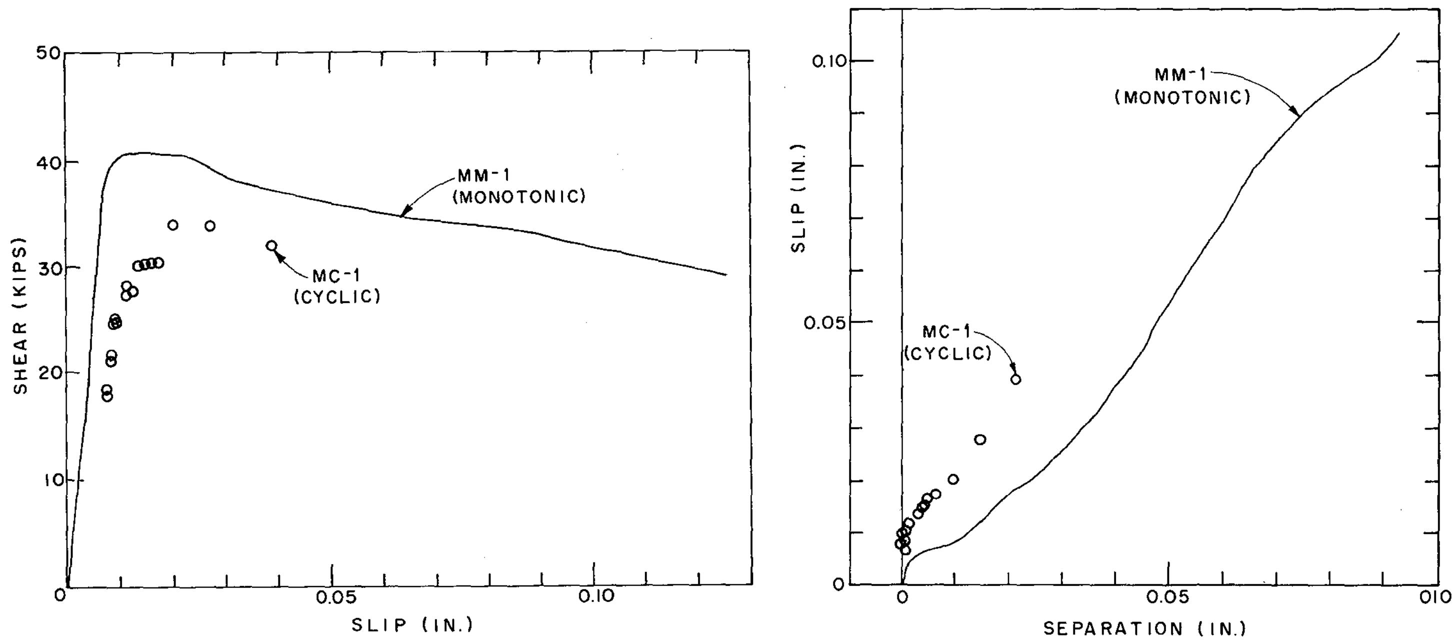


(c) SLIP SEPARATION CURVES, CYCLES 1-5



(d) SLIP SEPARATION CURVES, CYCLES 25-30

Fig. 14 RESPONSE OF SHEAR TRANSFER SPECIMEN MC-1 [53]



(a) SHEAR-SLIP RELATIONSHIPS

(b) SLIP-SEPARATION RELATIONSHIPS

Fig. 15 COMPARISON OF MONOTONIC AND CYCLIC SHEAR TRANSFER RESULTS [53]

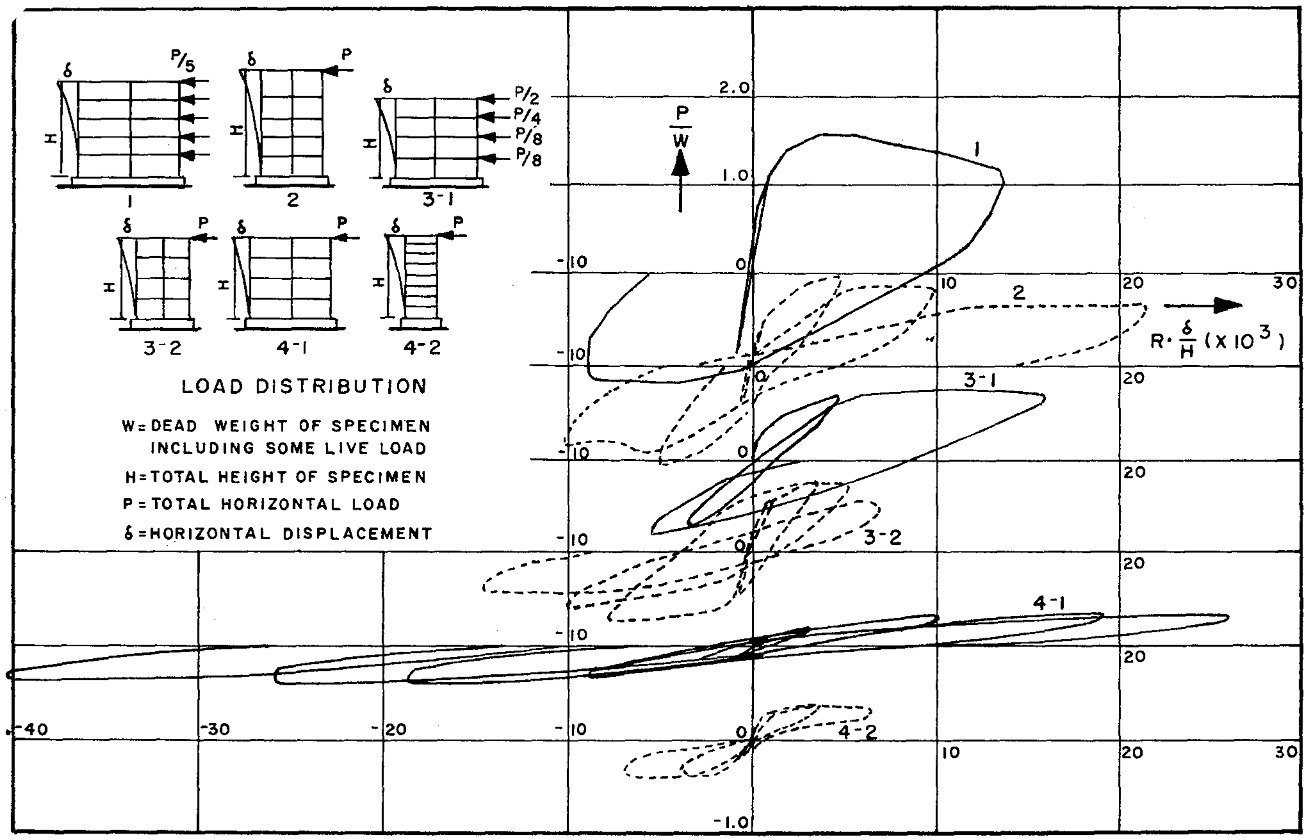


Fig. 16 LATERAL RESPONSE RESULTS FOR FRAME TESTS [49]

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

EXPERIMENTAL INVESTIGATIONS OF SUBASSEMBLAGES OF PARTIALLY PRESTRESSED AND PRESTRESSED CONCRETE FRAMED STRUCTURES

by

R. Park and K.J. Thompson

R. Park
Professor of Civil Engineering
University of Canterbury

K.J. Thompson
Assistant Engineer
New Zealand Ministry of Works and Development

INTRODUCTION

Many countries have not developed detailed building code provisions for the seismic design of prestressed concrete because of the lack of comprehensive experimental and theoretical studies of prestressed concrete structures subjected to seismic type loading.

This paper presents the results of some recent experimental work conducted on partially prestressed and prestressed concrete beam-column assemblies in New Zealand and makes design recommendations based on the analysis of these and other test results.

REVIEW OF PREVIOUS EXPERIMENTAL RESEARCH INTO THE SEISMIC RESISTANCE OF PRESTRESSED CONCRETE FRAMES

A detailed review of research into the seismic behaviour of prestressed concrete structures and structural elements has been reported by Blakeley, Park and Shepherd [1] covering known existing studies up to 1970. This previous research included cyclic loading tests by Nakano [2,3] on model single storey and four storey frames, by Spencer [4] on prestressed concrete members, and by Inomata [5] on both prestressed and reinforced concrete members. This previous research pointed to the smaller energy dissipation of prestressed concrete under cyclic loading (i.e. smaller hysteretic damping), and hence the possible larger deflection response to a severe earthquake, than a comparable reinforced concrete structure, but also indicated that adequate ductility of prestressed concrete sections could be provided. At the FIP Symposium on Seismic Structures in Tbilisi, USSR, in 1972, many papers containing research into the seismic resistance of prestressed concrete structures and their actual performance in earthquakes were presented. For example, Ban [6] reported on research conducted in Japan and concluded that prestressed concrete members designed in accordance with the 1966 FIP-CEB recommendations [7] behave as well as comparable reinforced concrete members when subjected to reversed inelastic loading. Blakeley and Park [8] summarized research in New Zealand involving tests on members and beam-column joints and analytical studies.

The FIP Commission on Seismic Structures reported to the 7th Congress of the FIP in New York in 1974. The report [9] presented general design information and summarized the papers presented at the 1972 Tbilisi FIP Symposium on Seismic Structures. The report concluded that the Tbilisi Symposium had quite definitely confirmed the possibility of the wide application of prestressed concrete in seismic resistant construction and its effectiveness under seismic loading for the structure of buildings and facilities. However it was also concluded that for more rational design the development of regulations is necessary, and that it was also necessary to widen the scope of research work and to accumulate more experimental data.

Thus there is still a need for more information on the behaviour of prestressed concrete structures under inelastic cyclic loading to establish more accurately such properties as plastic rotation capacity, stiffness degradation, and hysteretic damping capacity, so as: (a) to enable detailed design provisions to be drafted for the seismic design of prestressed concrete members, and (b) to enable more accurate studies to be conducted of the nonlinear dynamic response of multistorey prestressed concrete structures to severe earthquakes to establish the likely deflection response and ductility demands.

PREVIOUS EXPERIMENTAL STUDY OF THE SEISMIC RESISTANCE OF PRESTRESSED CONCRETE SUBASSEMBLAGES AT UNIVERSITY OF CANTERBURY

The beam-exterior column tests reported on at the 1972 FIP Tbilisi Symposium by Blakeley and Park [8], and reported in more detail elsewhere [10,11], involved four full-size precast concrete assemblies. Each assembly consisted of a column intersected by a beam on one side, representing that part of a frame in the region of a beam-exterior column joint between the points of contraflexure of the members. Each column was pretensioned. Each beam was post-tensioned by grouted tendons which passed through the column and were anchored in a beam stub. Mortar joints existed between the precast concrete elements at the column faces. Two of the assemblies were designed to form plastic hinges in the beams adjacent to the columns and two were designed to form plastic hinges in the columns adjacent to the beams when subjected to static cyclic loading simulating a severe earthquake. The axial load on the columns was less than 20% of the axial load capacity. Fig. 1 shows an assembly during test with plastic hinges forming in the columns.

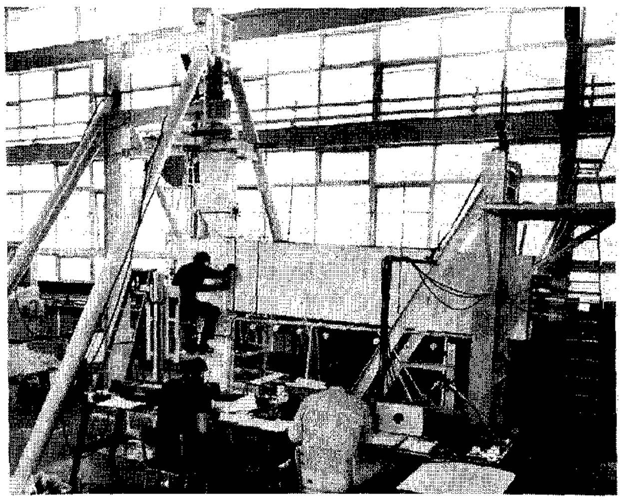


Fig. 1 Beam-Exterior Column Assembly Under Test [8,10,11]

The following conclusions were reached from the test results: (a) energy dissipation is relatively small prior to the commencement of crushing of the concrete, but after crushing has commenced it may be substantial, (b) large ductility can be available in prestressed concrete members, even when the transverse reinforcement satisfies only normal prestressed concrete code requirements for shear, (c) it is recommended that corrugated metal ducts be used for post-tensioned tendons through columns for exterior and interior beam-column assemblies so as to minimize the possibility of a bond failure between the ducts and the columns at large deformations, (d) mortar joints at critical sections between the precast post-tensioned frame members can behave well under the load reversals.

It was found possible to predict with good accuracy the shape of the measured moment-curvature hysteresis loops at the plastic hinge sections using an analysis based on idealized stress-strain relationships for concrete and steel subjected to reversed loading [12,11]. These analytical moment-curvature loops involved considerable computer time in their derivation and an empirical moment-curvature loop shape for prestressed concrete was proposed which took the degree of stiffness degradation and energy dissipation into account. This empirical moment-curvature loop has been used in nonlinear dynamic analyses of single degree of freedom prestressed concrete structures to predict their response to a severe earthquake [13,11].

RECENT EXPERIMENTAL STUDY OF THE SEISMIC RESISTANCE OF PRESTRESSED CONCRETE SUBASSEMBLAGES AT UNIVERSITY OF CANTERBURY

A further series of tests involving ten beam-column assemblies has recently been completed at the University of Canterbury [14,15]. The beams in this recent test series contained a range of proportions of prestressing steel and nonprestressed reinforcing steel to allow a comparison of prestressed and reinforced concrete frames and to establish the possible advantages of combining both systems. Also, the tests involved interior columns, rather than the exterior columns tested previously. The tests were aimed at determining the deformation capacity and degree of damage of such frames when responding to severe seismic load reversals and were to establish further basic information for the design of framed structures for earthquake resistance. The results obtained from the ten beam-interior column assemblies tested are summarized in the following sections. The results may be seen reported in more detail elsewhere [14,15].

TEST SPECIMENS

General Description

The beam-interior column test assemblies represent the part of the multi-storey plane frame shown encircled in Fig. 2. The assembly can be regarded as being the part of the frame between the points of contraflexure at a typical beam-interior column joint. The assembly was loaded as shown in Fig. 3a by an axial load P on the column representing load due to the weight of the building, floor loads and structure overturning moment, and by vertical loads V on the ends of the beams representing shear induced by earthquake loading. The applied beam loads induce reactive shears H at the ends of the column. By reversing the direction of the vertical loads the effects of earthquake loading were simulated.

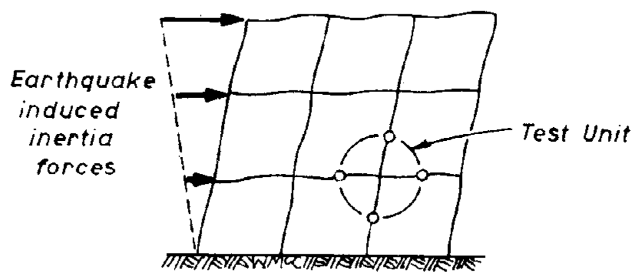


Fig. 2 Building Frame Subjected to Earthquake Motions

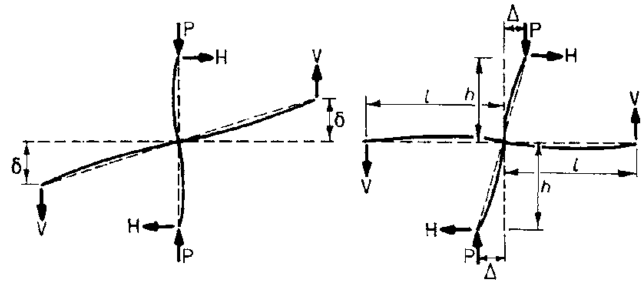


Fig. 3(a) Beam-Column Test Assembly (b) Real Beam-Column Situation

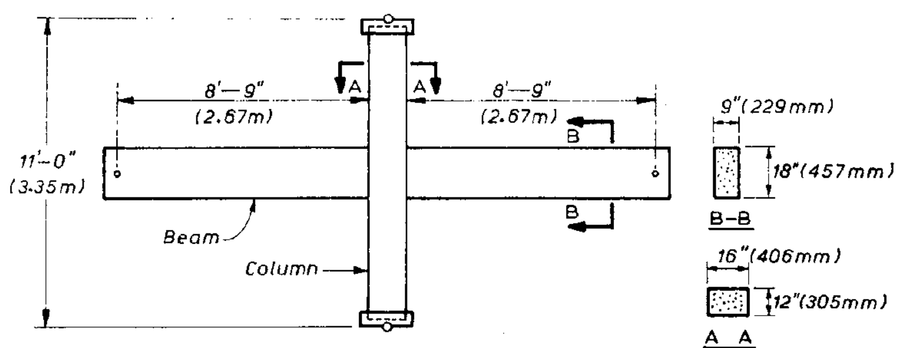


Fig. 4 Dimensions of Beam-Interior Column Test Assembly

It is to be noted that in the actual case of a frame under earthquake loading the ends of the beams of the test units would remain approximately on the same horizontal line and the column ends would be displaced horizontally, as illustrated in Fig. 3b. Imposing vertical deflections on the beam ends instead of imposing horizontal deflections on the column ends results only in a difference in the horizontal shears H acting on the columns; no change results in the beam or column moments for a given V. In the real frame situation, column top horizontal displacement Δ may be related to the beam end vertical displacement δ by observing the rigid body rotation of the specimen required to obtain the loading in Fig. 3b from the loading in Fig. 3a. The relationship given by such rigid body rotations is $\delta/l = \Delta/h$. This means that the column moment at the beam due to Δ is $P\Delta = P\delta h/l$, which may be considerable if P or δ are significant. The $P\Delta$ moment should be taken into account when assessing the seismic shears capable of being carried at large deflections. Equilibrium of the specimen in Fig. 3b requires that $H = (Vl - P\Delta)/h$, where H may be regarded as the seismic shear capacity of the beam-column assembly if Vl is the ultimate moment capacity of the beam (or column) section.

Dimensions and Details of Concrete and Steel of Beam-Column Assemblies

Fig. 4 shows the overall dimensions of the test assemblies. The beams and columns were cast monolithically. The flexural steel content was such that the column section was stronger than the beam section so that under severe seismic loading plastic hinging was enforced in the beam rather than in the column. Thus the critical sections were in the beam adjacent to the column faces. Ten assemblies were tested. The beam and column cross sections giving the steel details are shown in Figs. 5 and 6.

The beams contained various quantities of prestressed and nonprestressed steel and were designed to have approximately the same flexural strength. The prestressing tendons were post-tensioned to approximately 70% of their ultimate strength at transfer and grouted. The ultimate tensile strength of the prestressing tendons varied between 229 and 247 ksi (1,580 and 1,700 MPa). The prestress in the concrete of the beams of each unit at transfer, as measured by load cells at the ends of the tendons, is shown in Table 1. The nonprestressed longitudinal steel in the beams was from deformed bar with yield strength varying between 40.4 and 49.2 ksi (279 and 330 MPa). The columns contained only nonprestressed steel. The longitudinal steel in the columns was from deformed bar with yield strength varying between 56.7 and 60.2 ksi (391 and 415 MPa). The transverse reinforcement in all members was from plain round bar with yield strength varying between 41.9 and 48.8 ksi (289 and 337 MPa). The concrete compressive cylinder strength at the time of testing the units varied between 4,530 and 5,840 psi (31.2 and 40.3 MPa).

Theoretical Flexural Strength of Members

Fig. 7 shows a beam section at the flexural strength with the usual assumptions of ACI 318-71 [16] made for concrete strain and equivalent concrete stress distribution. The flexural strength of the members was calculated using the actual stress-strain curves for the prestressed and nonprestressed steel and the assumed ACI concrete compressive stress block, by satisfying the requirements of equilibrium of internal forces and strain

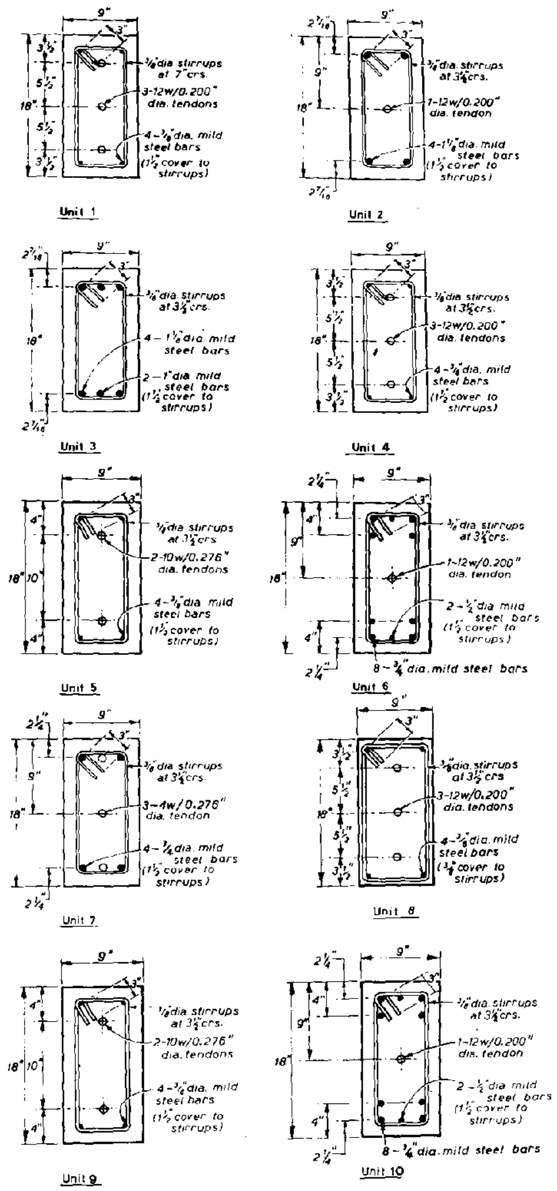
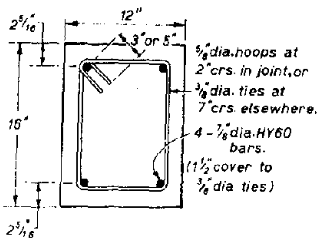
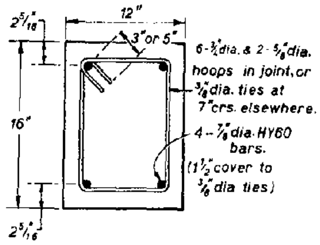


Fig. 5 Beam Sections of Units (1" = 25.4 mm)



Units 1 to 8



Units 9 and 10

Fig. 6 Column Sections of Units (1" = 25.4 mm)

Table 1 : Prestress and Theoretical Strength of Members According to ACI 318-71 [16]

Unit	Concrete Prestress in Beam at Transfer, psi	Theoretical Flexural Strength*, kip in		Theoretical Shear Strength+, kips		
		Beam	Column^x	Beam	Column	Joint Core
1	1160	1495	2047	36.6	53.4	217
2	386	1730	2134	57.1	56.6	221
3	-	1672	2137	61.6	56.2	220
4	1123	1588	2077	56.3	55.0	219
5	1198	1734	2107	61.6	58.0	220
6	371	1708	2121	67.6	58.4	220
7	675	1601	2123	67.5	58.3	220
8	1102	1571	2062	59.4	56.8	218
9	1228	1629	2174	58.8	55.1	288
10	362	1657	2131	69.9	60.0	293

* Using for concrete the ACI stress block and an extreme fibre compressive strain of 0.003

+ Using ACI 318-71 approach carrying shear on concrete and transverse steel, Eqs. 1, 2 and 3

x With an axial load of 100 tons (996 kN) present.

Note: 1 psi = 0.00689 MPa, 1 kip in = 113 Nm, 1 kip = 4.45 kN.

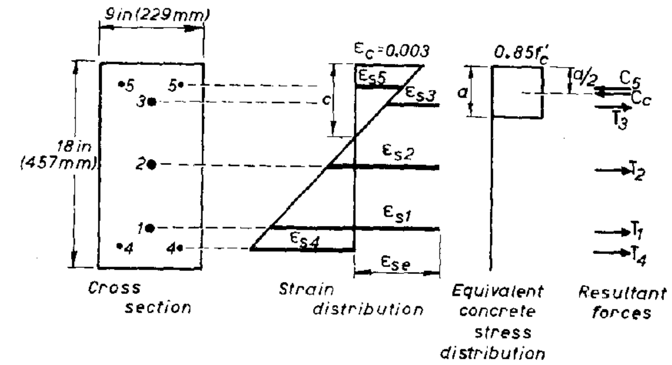


Fig. 7 Beam Section at the Flexural Strength

compatibility, with an extreme fibre concrete compressive strain of 0.003. Perfect bond between steel and concrete was assumed. The neutral axis position and the internal forces in the beams so found are shown in Table 2, using the notion of Fig. 7 where T_1, T_2 and T_3 refer to the forces in the prestressed steel and T_4 and C_5 refer to the forces in the nonprestressed steel. The theoretical flexural strength for the beams was obtained by taking the moments of these internal forces about a convenient axis and is given in Table 1.

Table 2 : Theoretical Neutral Axis Depths and Internal Forces in Beams at Flexural Strength

Unit	c in	a in	T_1 kips	T_2 kips	T_3 kips	T_4 kips	C_5 kips	C_c kips
1	6.43	5.27	75.3	69.2	41.9	9.0	9.0	186.4
2	3.21	2.49	-	78.8	-	82.9	56.9	104.8
3	2.78	2.17	-	-	-	118.2	29.0	89.2
4	6.36	5.07	77.5	72.9	45.1	10.5	10.2	195.9
5	6.47	5.09	129.3	-	75.1	10.4	10.4	204.3
6	3.71	2.90	-	79.9	-	84.2	45.3	118.8
7	4.00	3.13	51.7	48.7	27.8	36.1	36.6	127.7
8	6.32	5.12	76.1	69.8	42.4	10.7	10.6	188.4
9	6.90	5.69	124.8	-	71.6	11.0	10.6	196.8
10	3.63	2.76	-	75.4	-	81.0	33.7	122.9

Note: 1 in = 25.4 mm, 1 kip = 4.45 kN.

Table 1 also shows the theoretical flexural strength of the column sections calculated by the same procedure when a 100 ton (996 kN) axial column load was present as in the tests. Note that the column flexural strengths were greater than the beam flexural strengths.

The capacity reduction factor ϕ was assumed to be unity in all flexural strength calculations.

Theoretical Shear Strength of Members

The theoretical horizontal shear strength of the members was calculated using the ACI 318-71 [16] procedure for members with shear reinforcement perpendicular to the flexural reinforcement, namely

$$V_u = V_c + V_s \tag{1}$$

where $$V_c = v_c bd \tag{2}$$

and $$V_s = A_v f_y d/s \tag{3}$$

in which v_c = nominal shear stress carried by the concrete, b = width of section, d = distance from extreme compression fibre to centroid of tension steel but in the case of a prestressed concrete member not less than 0.8 of the overall depth of the section, A_v = area of transverse shear reinforcement within distance s, f_y = yield strength of shear reinforcement, and s = spacing of shear reinforcement. The capacity reduction factor ϕ was taken as unity.

For the beams the shear stress carried by the concrete v_c was conservatively assumed to be $2\sqrt{f'_c}$ psi ($0.167\sqrt{f'_c}$ MPa) in Eq. 2. The theoretical shear strengths calculated for the beams on this basis from Eqs. 1 to 3 are shown in Table 1. The maximum theoretical shear force which can be applied to the beam in the tests is the beam ultimate moment capacity divided by the shear span of the beam. This ignores the small contribution to the shear carried by the beam dead weight. Using the theoretical beam flexural strengths of Table 1 and the beam shear span of 8 ft 9 in (2.67 m), the maximum theoretical applied beam shear force varied between 0.23 and 0.39 of the beam theoretical shear strength. The maximum nominal shear stress caused by this applied shear force in the beams varied between $1.52\sqrt{f'_c}$ and $1.78\sqrt{f'_c}$ psi ($0.126\sqrt{f'_c}$ and $0.148\sqrt{f'_c}$ MPa). Thus beam shear should not be critical as either the concrete or stirrups was capable of carrying the entire maximum applied shear force alone.

For the columns the nominal shear stress carried by the concrete was calculated using the ACI 318-71 [16] expression

$$v_c = 2\ (1 + 0.0005N_u/A_g)\sqrt{f'_c}\ \text{psi} \tag{4a}$$

$$\text{or} \quad v_c = 0.166\ (1 + 0.0725N_u/A_g)\sqrt{f'_c}\ \text{MPa} \tag{4b}$$

where N_u = column axial compressive load, A_g = gross area of column, and the units are lb and in in Eq. 4a and N and mm in Eq. 4b. For the axial column load of 100 tons (996 kN) as in these tests Eq. 4 gives v_c as $3.17\sqrt{f'_c}$ psi ($0.263\sqrt{f'_c}$ MPa). The theoretical shear strengths of the columns calculated from Eqs. 1 to 3 using this v_c value and for the quantity of column ties in the columns are shown in Table 1. The maximum theoretical shear force which can be applied to the column if the column reaches flexural strength is the column ultimate moment capacity divided by the column shear span of 4 ft 9 in (1.45 m). The maximum theoretical column shear force so calculated varied between 0.62 and 0.69 of the theoretical column shear strength. Thus column shear should not be critical during the tests.

Theoretical Horizontal Shear Strength of Joint Cores

The theoretical horizontal shear strength of a reinforced concrete beam-column joint core according to Appendix A of ACI 318-71 [16] may be found from Eqs. 1 to 3 applied to the column in the joint core region. For the test units, v_c from Eq. 4 is $3.17\sqrt{f'_c}$ psi ($0.263\sqrt{f'_c}$ MPa) as for the column, b and d are as for the column section, and $A_v f_y d/s$ applies to the column hoops in the joint core. The theoretical shear strength of the joint cores so calculated from Eqs. 1 to 3 for all units are shown in Table 1.

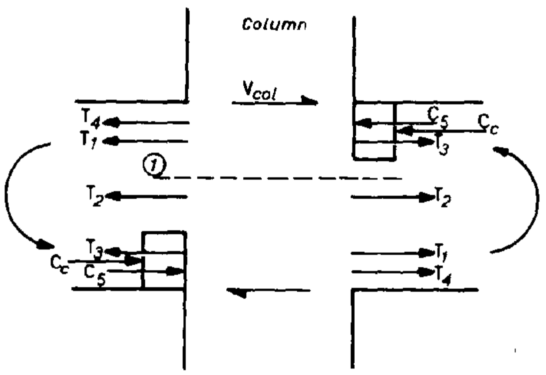


Fig. 8 Beam Internal Forces and Column Shear Acting on Joint Core

Fig. 8 shows the beam internal forces and the column shear force acting on the joint core at the theoretical ultimate moment of the beam. The beam internal forces are given in Table 2 and V_{col} is M_{col} divided by the column shear span of 4 ft 9 in (1.45 m), where M_{col} is the moment in the column at the face of the joint core when the beam reaches its flexural strength. Consideration of the total forces acting above or below horizontal planes across the joint (for example plane 1 of Fig. 8) shows that the maximum horizontal shear force occurs in the middle region between the neutral axis positions of the beam sections. Therefore the maximum theoretical horizontal shear force in the joint core is

$$V_{max} = T_4 + T_1 + C_5 + C_c - T_3 - V_{col} \tag{5}$$

The ratio of maximum theoretical horizontal shear force in the joint core (calculated from Eq. 5 using the theoretical internal forces in the beams given in Table 2 and the calculated column shear) to the theoretical joint core horizontal shear strength (calculated using the ACI 318-71 approach) for the units ranged between 0.71 and 1.12 with a value greater than 1.01 only in the case of Unit 5. The horizontal shear reinforcement in the joint core was theoretically capable of carrying without the assistance of v_c at least 75% of the maximum horizontal shear force there. It is evident that according to the ACI procedure, with the exception of Unit 5, the joint cores were adequately reinforced for shear.

Confinement Steel in the Columns and Joint Cores

When the axial load acting on the column exceeds $0.4P_b$, where P_b is the column load at balanced failure, Appendix A of ACI 318-71 [16] requires rectangular hoops in the joint region with the area of one leg at least equal to

$$A_{sh} = l_h \rho_s s_h / 2 \tag{6}$$

where $$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_y} \geqslant 0.12\,\frac{f'_c}{f_y} \tag{7}$$

and l_h = maximum unsupported length of hoop side, s_h = spacing of hoops, A_g = gross area of column section, A_{ch} = area of rectangular core of column measured to outside of hoops, f'_c = concrete compressive cylinder strength and f_y = steel hoop yield strength.

For columns of test units the applied load exceeded $0.4P_b$. The required spacing of No. 5 (15.9 mm dia.) hoops given by Eqs. 6 and 7 for Units 1 to 8 varied between 1.7 and 2.1 in (43 and 53 mm). The No. 5 (15.9 mm dia.) hoops present in the joint cores of Units 1 to 8 were at 2 in (51 mm) centres and hence the ACI requirement was almost met there. In Units 9 and 10 the joint core hoops were mainly from No. 6 (19.1 mm dia.) bar at 2 in (51 mm) centres and hence the ACI requirement was easily met there for these two units. Elsewhere in the columns of all test units no attempt was made to meet this requirement because plastic hinges were not expected to form in the columns.

EXPERIMENTAL PROCEDURE

The test units were subjected to a series of static reversed loading cycles which attempted to simulate a very severe earthquake. Throughout the application of load the deflections imposed on both beam ends were equal. The loading sequence consisted of two initial loading cycles to beyond the service load but not to ultimate load, and then generally four loading cycles to well into the inelastic range. The maximum beam end deflections imposed in the inelastic range were between 3 in (76 mm) and 5 in (127 mm). It was calculated theoretically that when yielding of the steel of the reinforced concrete beam of Unit 3 commenced, the beam end deflection would be 1.1 in (28 mm), assuming fully cracked beams and columns. Thus theoretically a beam end deflection of 4.4 in (112 mm) of Unit 3 was required to enforce a displacement ductility factor of 4 if all the members were fully cracked. The prestressed and partially prestressed beams were subjected to deflections of approximately the same magnitude as for the reinforced concrete beam in order to compare their behaviour.

The cyclic loading on the test units was applied statically over a time period of days rather than dynamically over a time period of seconds as would occur in an earthquake. Nevertheless it is considered that static reversed load tests should give a good estimate of the actual structural behaviour under rapid earthquake load reversals.

The test rig with a beam-column unit in place is shown in Fig. 9. During the tests a constant axial load of 100 tons (996 kN) was applied to the column through a hydraulic jack. This column load was $0.22P_o$ and $0.63P_b$, where P_o is the axial ultimate load capacity and P_b is the balanced failure load of the column for the column material design strengths of f_y = 60 ksi (414 MPa) and f'_c = 5500 psi (38 MPa). The beam ends were loaded by hydraulic jacks operated by hand pumps and the imposed deflection of the beam ends could be controlled so that both the rising and falling branches of the

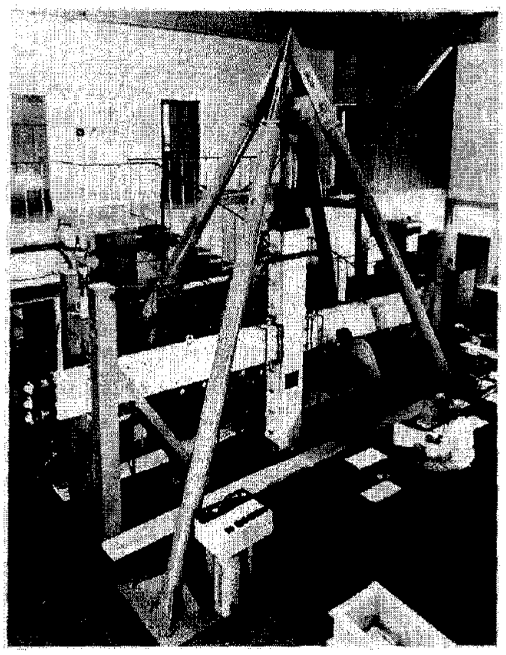


Fig. 9 A Unit in Test Frame During Testing

load-displacement curves could be followed. Strains were measured using mechanical or electrical resistance strain gauges. Rotations in plastic hinge regions in the beams were also measured by dial gauges attached to frameworks which in turn were attached to pins in the concrete.

The damaged concrete of one unit (Unit 1) was repaired after testing to check whether satisfactory structural behaviour could be achieved after repair. The repair was carried out by jacking the damaged beams back to the horizontal position (straightening the structure is probably the most difficult aspect in a real repair job), chipping away the damaged concrete in the plastic hinge regions of the beams, placing two No.3 (9.5 mm dia.) stirrups around the remaining core of the beams in each damaged region, and replacing the removed concrete by new concrete to give the original cross-sectional dimensions. Fig.10 shows Unit 1 during repair. After a time interval for curing the gaining strength the repaired test unit was subjected to cyclic loading. Only Unit 1 was repaired and retested.

TEST RESULTS

Behaviour of Units With Fully Prestressed Beams (Units 1,4,5,8 and 9)

These five units were able to be loaded to well beyond the service load and then unloaded with almost complete deflection recovery and negligible visual residual damage. In the subsequent loading cycles, after the beam moment capacities had been reached and crushing of the concrete had commenced, the energy dissipation capacity of the members increased and significant degradation of stiffness and strength occurred. Figs. 11 to 16 show the measured beam moment at column face-beam end deflection curves and illustrate the damage visible at the end of the first loading cycle to the ultimate beam moment capacity. With subsequent loading cycles into the inelastic range the inelastic deformations concentrated in the beam plastic hinge regions of Units 1 and 4 and mainly in the joint cores of Units 5, 8 and 9.

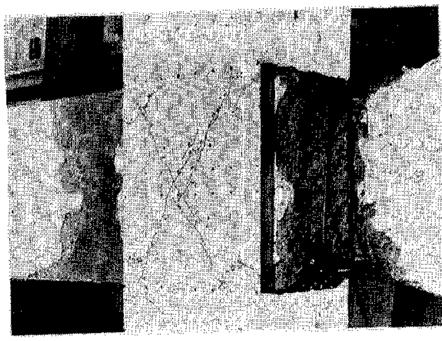
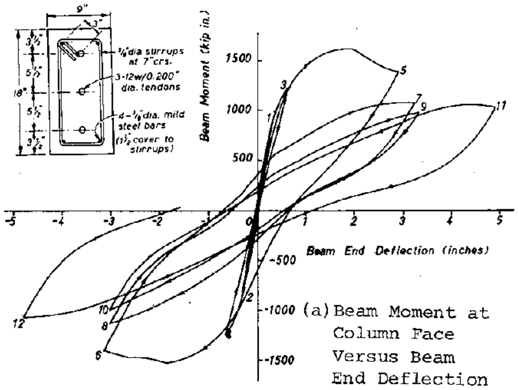
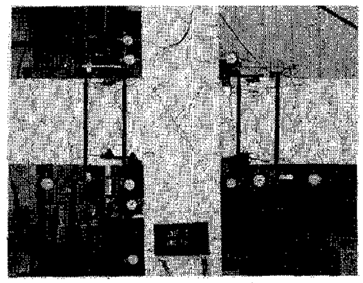


Fig. 10 Repair of Unit 1: New Concrete Placed at One Section, Other Section About to be Boxed Ready for Placing Concrete

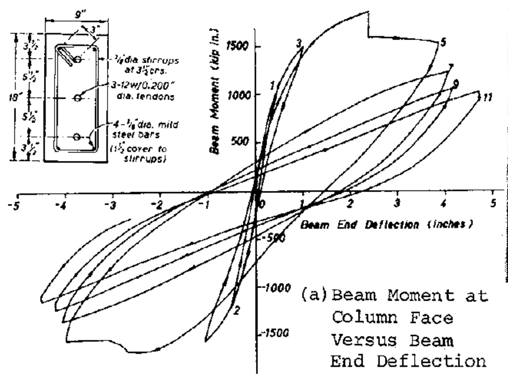


(a) Beam Moment at Column Face Versus Beam End Deflection

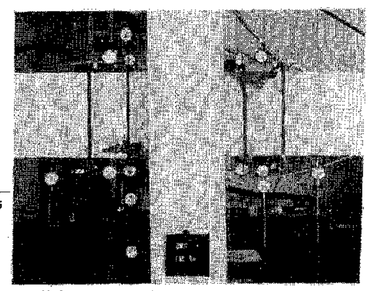


(b) At Maximum Moment (End of Loading Run 6)

Fig. 11 Unit 1 (1 kip in = 113 Nm, 1 in = 25.4 mm)

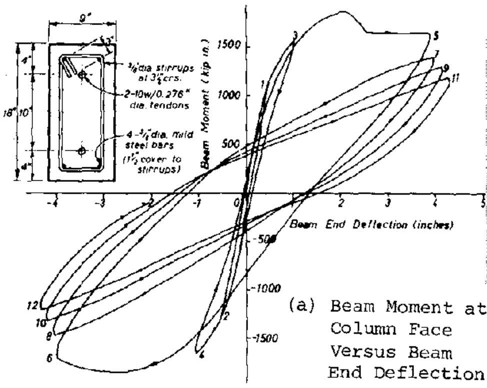


(a) Beam Moment at Column Face Versus Beam End Deflection

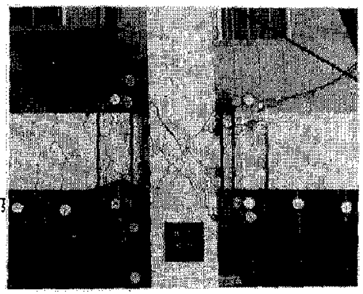


(b) At Maximum Moment (End of Loading Run 6)

Fig. 12 Unit 4 (1 kip in = 113 Nm, 1 in = 25.4 mm)

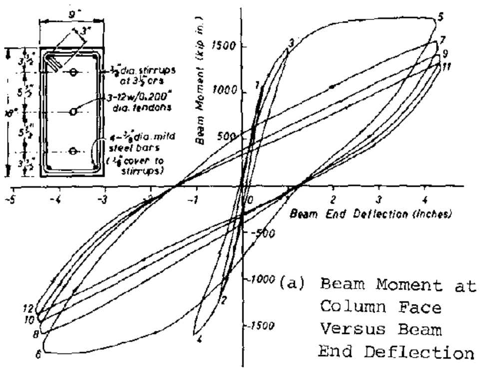


(a) Beam Moment at Column Face Versus Beam End Deflection

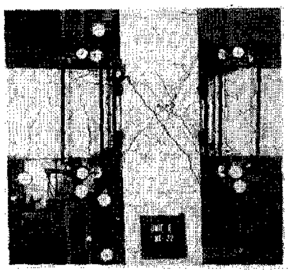


(b) At Maximum Moment (End of Loading Run 6)

Fig. 13 Unit 5 (1 kip in = 113 Nm, 1 in = 25.4 mm)

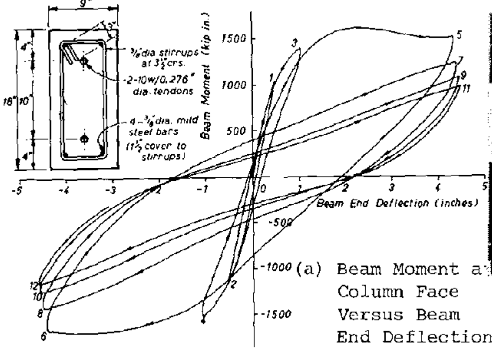


(a) Beam Moment at Column Face Versus Beam End Deflection

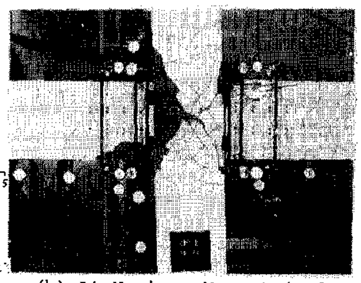


(b) At Maximum Moment (End of Loading Run 6)

Fig. 14 Unit 8 (1 kip in = 113 Nm, 1 in = 25.4 mm)

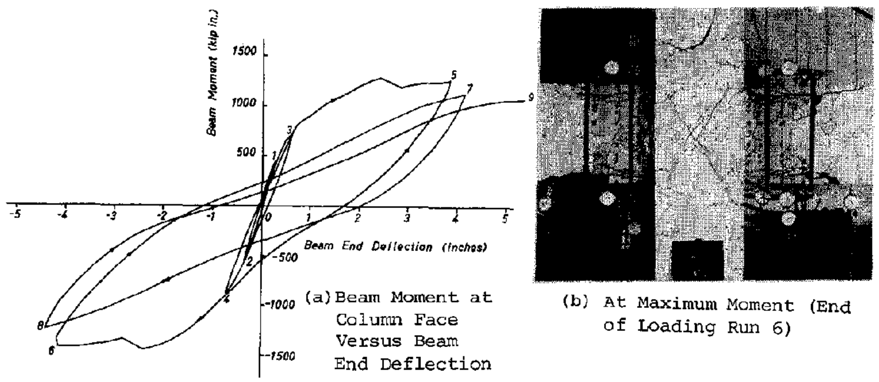


(a) Beam Moment at Column Face Versus Beam End Deflection



(b) At Maximum Moment (End of Loading Run 6)

Fig. 15 Unit 9 (1 kip in = 113 Nm, 1 in = 25.4 mm)

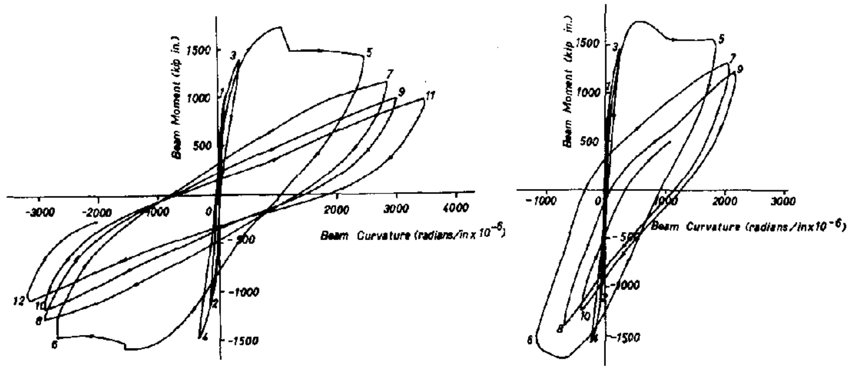


(a) Beam Moment at Column Face Versus Beam End Deflection

(b) At Maximum Moment (End of Loading Run 6)

Fig. 16 Unit 1 Repaired (1 kip in = 113 Nm, 1 in = 25.4 mm)

The extent of inelastic deformation occurring in the beams may be illustrated by examining the moment-curvature curves measured in the plastic hinge regions. Fig. 17a and b shows the average moment-curvature curves measured over a 12 in (305 mm) gauge length in the beams adjacent to the column faces of Units 4 and 5. The similarity of shape of Fig. 17a with the moment-deflection curve of Fig. 12a indicates that most of the inelastic deformation occurred in the plastic hinge region of Unit 4 during all the loading cycles. However, the difference in shape of Fig. 17b when compared with the moment-deflection curve of Fig. 13a (note the decrease in loop area of Fig. 17b as the load cycles proceed) indicates that the inelastic deformation occurred in the plastic hinge region of Unit 5 in the first load run but with subsequent loading cycles the inelastic deformation occurred increasingly elsewhere in the unit, namely in the joint core.



(a) Unit 4

(b) Unit 5

Fig. 17 Average Moment-Curvature Relationships Measured Over a 12 in Gauge Length in Beam Adjacent to Column Face (1 kip in = 113 Nm, 1 in = 25.4 mm)

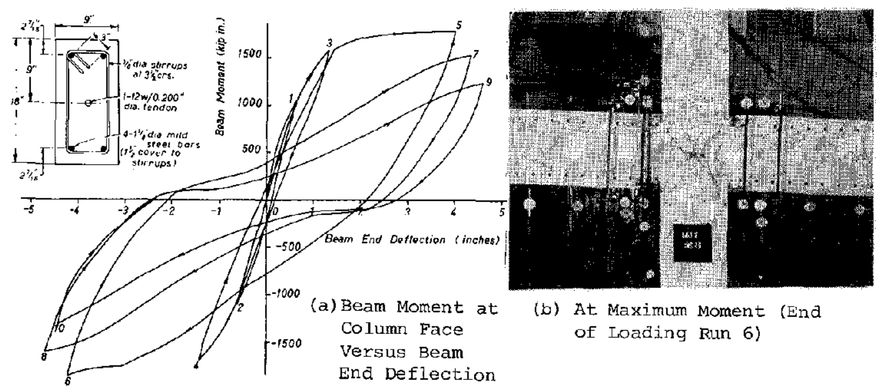
The degradation in the stiffness and strength of Units 1 and 4 was due to the reduction in the sectional area of the beams when the cover concrete crushed and also to the reduction of prestressing force caused by the residual inelastic steel strains. Comparison of the moment-deflection curves for Units 1 and 4 (Figs. 11a and 12a) shows that the closer spacing of stirrup ties in Unit 4 did not have a marked influence on the ductility of that unit. This was because the closer stirrup spacing did not prevent loss of the cover concrete and hence did not prevent loss of stiffness and strength due to crushing of the concrete cover. However close inspection of Figs. 11a and 12a does indicate that the closer stirrup spacing in the beams of Unit 4 enabled the beam moment to be still increasing significantly when each loading run was terminated, whereas for Unit 1 the beam moment change was small at the ends of the loading runs because of the deeper penetration of concrete crushing into the beam core between the stirrups. The $3\frac{1}{2}$ in (89 mm) spacing of stirrup ties in the beams of Unit 4 was approximately d/4, if the effective depth d is taken as 0.8 of the overall depth of the member. Comparison of the moment-deflection curves of Units 4 and 8 (Figs. 12a and 14a) shows the influence of using $1\frac{1}{2}$ in (38.1 mm) and $\frac{3}{8}$ in (9.5 mm) thickness of concrete cover to the stirrups, respectively. It is evident that the reduction in moment capacity when concrete crushing commenced (loading runs 5 and 6) was less significant for the member with the smaller concrete cover (Unit 8) because of the larger area of confined core. Thus to retain the flexural strength of members at a high level after concrete crushing the cover thickness should be kept as small as possible.

The behaviour of the joint cores of Units 1 and 4 was satisfactory. The degradation of the strength and stiffness of Units 5, 8 and 9 was due to the gradual deterioration of the joint cores of those units, which commenced after the first inelastic loading run. This deterioration of the joint cores was the result of repeated opening and closing of diagonal tension cracks in alternating directions and yielding of the hoop reinforcement in the joint cores. Unit 5 contained the same joint core shear reinforcement as Units 1 and 4, but lacked a prestressing tendon at the centre of the beam depth. Unit 9 contained more joint shear reinforcement than Units 1 and 4 but also lacked a central prestressing tendon. Thus the presence of a tendon at mid-depth of the beam passing through the joint core was shown to be beneficial to joint core behaviour. The failure of the joint core of Unit 8, even though it had a central prestressing tendon through the joint core and the same joint reinforcement as Units 1 and 4, occurred partly because of the high level of moment being maintained in the loading cycles in the inelastic range due to the small thickness of concrete cover, thus imposing large horizontal shear forces on the joint core for much of the loading cycles. Also, a reduction in the prestress on the central region of the joint occurred in this unit during the inelastic loading runs because the central prestressing tendon was subjected to a large inelastic strain in loading run 5. Thus for Unit 8, in the loading runs following loading run 5, the central prestressing tendon was not as effective as in Units 1 and 4.

The repair of Unit 1 by straightening the damaged beam, removing the damaged concrete in the plastic hinge region and replacing the concrete there, demonstrated that it is possible to repair prestressed concrete members, provided the initial straightening operation can be achieved in practice. Fig. 16a shows the moment-deflection behaviour of the repaired unit. The two extra stirrups placed within the new concrete at each repaired plastic hinge region helped confine the concrete and the performance of the repaired unit in the subsequent load testing was satisfactory. The new concrete was not prestressed, and the prestressing tendons merely acted as ordinary reinforcement. However the overall performance of the repaired unit was satisfactory although as expected the crack control in the new concrete was not as good as in the original unit. For instance, at 63% of the experimental ultimate moment the measured maximum crack width in the new concrete was approximately 0.015 in (0.38 mm) whereas in the original prestressed beam that magnitude of measured maximum crack width was not reached until 98% of the experimental ultimate moment had been applied.

Behaviour of Units With Partially Prestressed Beams (Units 2, 6, 7 and 10)

These units were able to be loaded to well beyond service load and then unloaded with almost complete deflection recovery and little visible residual damage. In the subsequent load cycles, after the beam moment capacity had been reached and crushing of the concrete had commenced, the energy dissipation capacity of the members was considerable (greater than for the units with fully prestressed beams) although significant degradation of stiffness, and in some cases strength, occurred. Figs. 18 to 21 show the measured beam moment at column face - beam end deflection curves and illustrate the damage visible at the end of the first loading cycle to ultimate beam moment capacity. With subsequent loading cycles into the inelastic range. the inelastic deformations concentrated in the beam plastic hinges of Units 7 and 10 and mainly in the joint cores of Units 2 and 6.

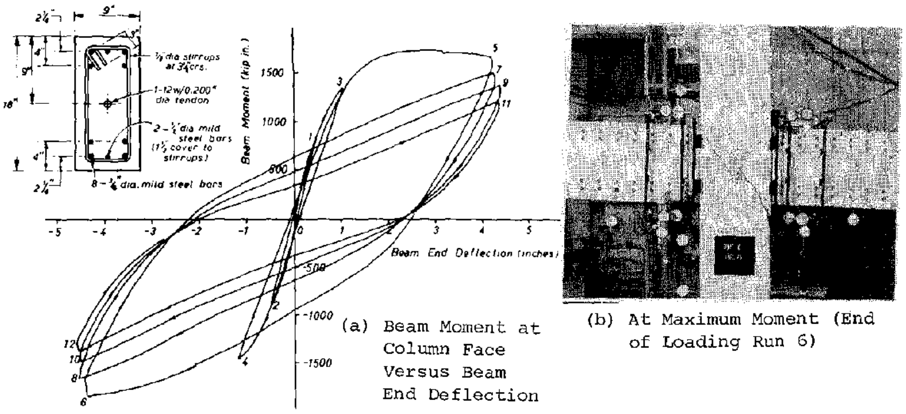


(a) Beam Moment at Column Face Versus Beam End Deflection

(b) At Maximum Moment (End of Loading Run 6)

Fig. 18 Unit 2 (1 kip in = 113 Nm, 1 in = 25.4 mm)

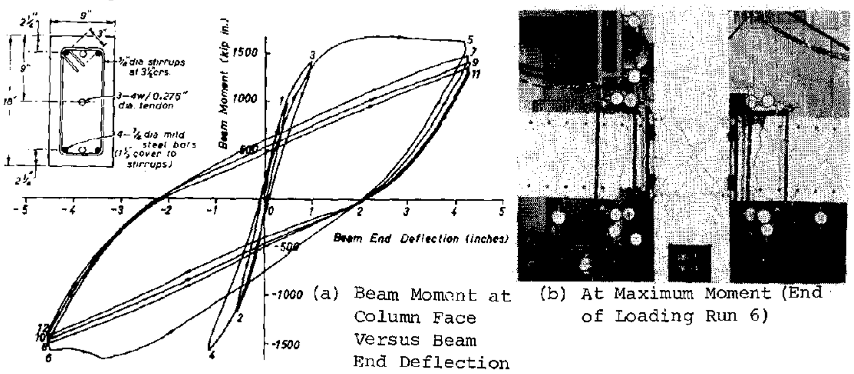
The beneficial effect of the nonprestressed compression steel in the compression zones of the beam plastic hinges of Units 7 and 10 is evident from the stable moment-deflection curves for these two units after the first inelastic load run (see Figs. 20a and 21a). Although the stiffness in the subsequent load cycles is smaller than the initial elastic stiffness the strength degradation was not large, since after crushing of the concrete had commenced the compression steel was able to carry some of the compression previously carried by the cover concrete and therefore helped maintain the internal lever arm. Also there was no significant slip of the No. 6



(a) Beam Moment at Column Face Versus Beam End Deflection

(b) At Maximum Moment (End of Loading Run 6)

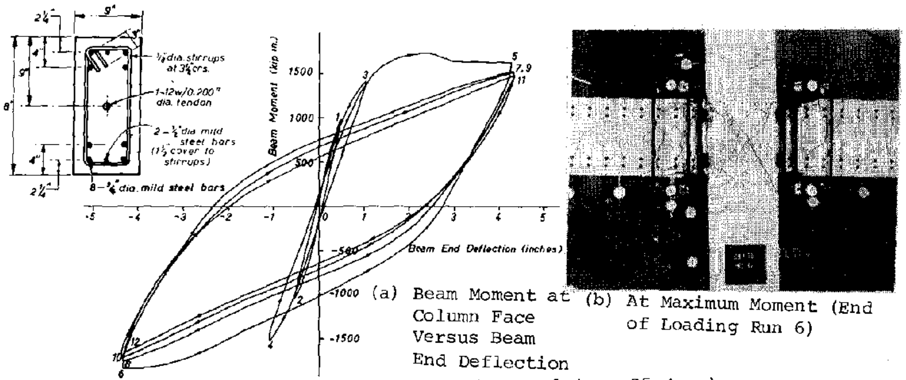
Fig. 19 Unit 6 (1 kip in = 113 Nm, 1 in = 25.4 mm)



(a) Beam Moment at Column Face Versus Beam End Deflection

(b) At Maximum Moment (End of Loading Run 6)

Fig. 20 Unit 7 (1 kip in = 113 Nm, 1 in = 25.4 mm)



(a) Beam Moment at Column Face Versus Beam End Deflection

(b) At Maximum Moment (End of Loading Run 6)

Fig. 21 Unit 10 (1 kip in = 113 Nm, 1 in = 25.4 mm)

(19.1 mm dia.) beam bars through the joint core.

The behaviour of the joint cores of Units 7 and 10 was satisfactory. In the case of Units 2 and 6 the degradation of strength and stiffness was due to deterioration of the joint cores of those units which commenced after the first inelastic loading run. Units 2, 6 and 7 contained the same joint shear reinforcement but Unit 7 did not contain as much nonprestressed longitudinal steel as Units 2 and 6. Evidently the bond forces from the nonprestressed steel in Units 2 and 6 caused the difference. Unit 10 contained more shear reinforcement in the joint core than the other three units but otherwise was identical to Unit 6. All four units contained a prestressing tendon at mid-depth in the beam at the joint, but this was evidently unable to prevent joint core shear degradation in the case of Units 2 and 6. In Unit 2 the No. 9 (28.6 mm dia.) beam bars slipped backwards and forwards through the joint core from loading run 7 onwards but in Unit 6 in which the No. 6 (19.1 mm dia.) beam bars were used there was no significant slip.

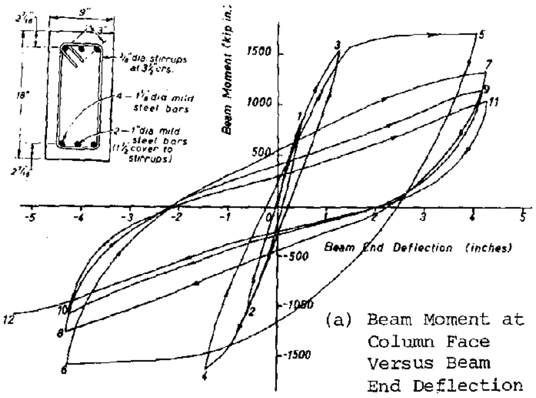
Behaviour of Unit With Ordinary Reinforced Beam (Unit 3)

Fig. 22a and b shows the measured beam moment at the column face - beam end deflection curves and illustrates the damage visible at the end of the first load cycle to ultimate beam moment capacity. After the maximum moment had been reached in each direction there was deterioration of the joint core concrete and the subsequent reduction in stiffness and strength of the unit was due to damage concentrating in the joint core. The Nos. 8 and 9 (25.4 and 28.6 mm dia.) beam bars slipped through the joint core during the load cycles in the inelastic range. Joint core shear failure in this reinforced concrete unit was more comprehensive than in any of the other units. Fig. 22c shows the average moment-curvature curve measured over a 10 in (254 mm) gauge length in the beam near the column face and it is evident that most of the inelastic deformation occurred in the joint core and not in the plastic hinge after the first loading run into the inelastic range.

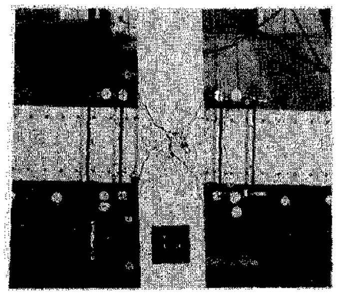
ANALYSIS OF TEST RESULTS

Analysis of Plastic Hinge Behaviour

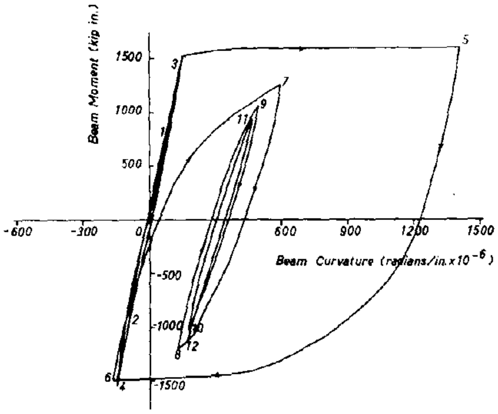
Table 3 shows a comparison of the measured flexural strength of the beams at the critical sections in the first loading runs into the inelastic range in each direction (loading runs 5 and 6) with the theoretical moment capacity calculated using the ACI rectangular concrete compressive stress block, an extreme fibre strain of 0.003 and the actual stress-strain curves for the steel. The agreement is reasonable. The measured values are generally higher than the theoretical values since the maximum moment was reached at an extreme fibre concrete compressive strain of greater than 0.003. During the subsequent loading runs, in the case of those units where inelastic deformations concentrated in the plastic hinge regions of the beams (Units 1, 4, 7 and 10), there was an appreciable degradation of flexural strength in the case of Units 1 and 4 but not in the case of Units 7 and 10. This was due to the different tension forces in the steel and compression steel contents of those members. The theoretical ratio of the depth of rectangular concrete compressive block to overall depth (a/h) of the beams of Units 1 and 4 was 0.29 and 0.28, respectively (see Table 2).



(a) Beam Moment at Column Face Versus Beam End Deflection



(b) At Maximum Moment (End of Loading Run 6)



(c) Average Moment-Curvature Relationship Measured Over a 10 in Gauge Length in Beam Commencing 1 in from Column Face

Fig. 22 Unit 3 (1 kip in = 113 Nm, 1 in = 25.4 mm)

Table 3 : Flexural Strength of Beams at the Column Face in Loading Runs 5 and 6

Unit	Max. Experimental Moment in Loading Run 5 / Theoretical Ultimate Moment	Max. Experimental Moment in Loading Run 6 / Theoretical Ultimate Moment
1	1.09	1.03
Repaired 1	0.86*	0.97*
2	1.04	1.08
3	1.03	0.95
4	1.16	1.05
5	1.07	1.08
6	1.02	1.07
7	1.06	1.03
8	1.15	1.14
9	0.99	1.04
10	1.05	1.11

* Theoretical ultimate moment used is that of the original section.

Had there been a smaller tension force in the steel, or significant compression steel, in the beams of Units 1 and 4, so that the ratio of a/h was smaller, the degradation of flexural strength with subsequent load cycles would not have been so marked. In the case of Units 7 and 10, which had a smaller tension force in the steel and significant compression steel present, the theoretical ratio of a/h was 0.17 and 0.15, respectively (see Table 2), and the degradation of flexural strength with subsequent load cycles was much less marked. The draft New Zealand concrete design code will proposed that a/h should not exceed 0.2 for beams with stirrup ties present as in the test units, and will propose that a/h should only be allowed to approach 0.3 in beams if as much special transverse steel is present as in the potential plastic hinge zones of columns. It was also evident from the tests that the $3\frac{1}{2}$ in (89 mm) spacing of stirrup ties used in the plastic hinge regions of the beams prevented excessive penetration of concrete crushing into the core concrete between the stirrups, and that this spacing prevented buckling of the longitudinal beam steel during the loading cycles. The draft New Zealand concrete design code will propose that the spacing of stirrup ties in the plastic hinge zones of beams should not exceed the smaller of 4 in (100 mm), d/4, or six longitudinal bar diameters, if $a/h \leqslant 0.2$.

Longitudinal strains measured on the members allowed the curvature distribution along the members to be calculated. Fig. 23 shows the curvature distribution measured for Unit 4 at the peaks of loading runs 3, 5 and 11. For this unit plastic hinging developed in the beam and there was no joint core deterioration. The area of the measured curvature diagram gives the rotation which occurs along the member. This area can be divided into a triangular region of elastic curvature along the length of the member plus a region of plastic curvature near the critical section [10,18]. The plastic rotation can be conveniently expressed as

$$\theta_p = (\phi_{max} - \phi_e) l_p \qquad (8)$$

where ϕ_{max} = maximum curvature, ϕ_e = maximum elastic curvature, and l_p = equivalent plastic hinge length. The equivalent plastic hinge length, defined as in Eq. 8, was calculated for a number of beams from the measured curvature distributions at the peaks of the loading cycles. In these calculations the equivalent plastic hinge length was found from the area of plastic curvature measured divided by the difference between the measured maximum curvature and the measured maximum elastic curvature [15]. The mean equivalent plastic hinge length calculated for Units 1 to 4 for the first inelastic deformation application (run 5) was 8.38 in (213 mm). Since the beam was 18 in (457 mm) deep the equivalent plastic hinge length could be taken as approximately one half of the overall depth of the member, which agrees with the findings of the previous test series [10]. A feature of the observed plastic hinge rotation with cyclic loading was that the equivalent plastic hinge length did not decrease with further cycles of inelastic rotations. Hence equivalent plastic hinge lengths measured in the first inelastic cycle should give safe estimates of available plastic rotation capacity in subsequent cycles.

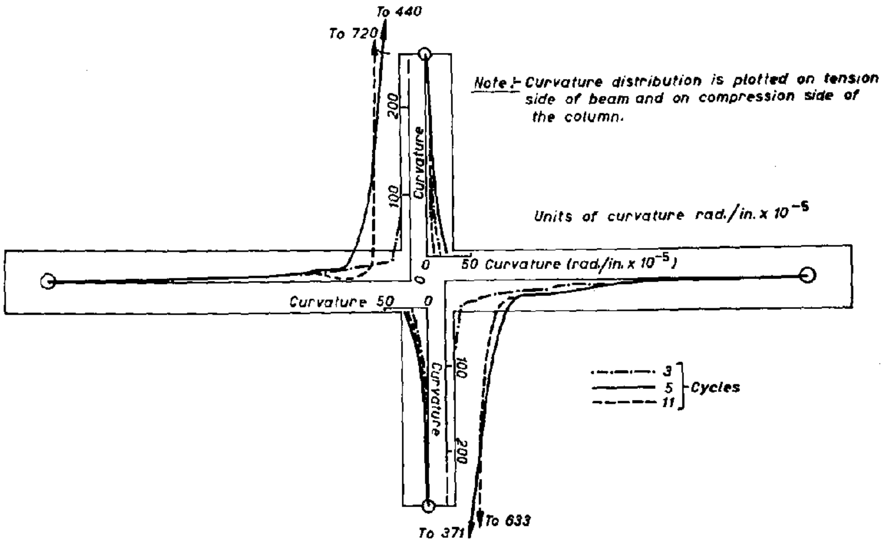
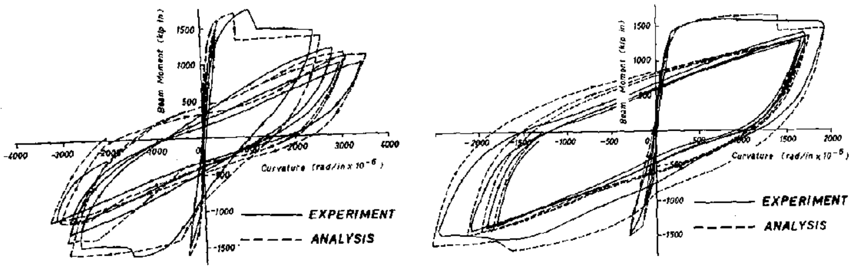


Fig. 23 Measured Curvature Distribution Along Members of Unit 4 at Peaks of Odd Loading Runs (1 in = 25.4 mm)

The measured moment-curvature relationships in the plastic hinge regions of the beams were also compared with theoretically derived moment-curvature relationships. The theoretical moment-curvature relationships were computed assuming that plane sections remain plane, satisfying the requirements of strain compatibility and equilibrium, and using idealised stress-strain relationships for the steel and concrete. The stress-strain relationships for steel used closely followed that obtained experimentally from cyclic load tests on the steel from the beams [15]. The stress-strain relationship for the concrete used was similar to that devised previously [11,12] for cyclic loading. The theoretical approach computed the moment and associated curvature at strain increments between specified curvature limits. An iterative procedure was used to determine the neutral axis depth at each strain increment and the computation is lengthy because of this and because the strain history of the elements into which the section is subdivided needs to be followed to calculate the appropriate stress. Prestressed, partially prestressed and reinforced concrete sections can be analysed using this technique. The theoretical procedure results in good accuracy, as is evident from the comparison of the theoretical moment-curvature curves obtained from this analysis and the measured curves from the experiment for Units 4 and 7 shown in Fig. 24. In the analysis allowance was made for the possible buckling of the prestressing steel at large compressive strains by limiting the level of compressive stress that steel can reach. Note that in the analytical curves the concrete cover is assumed to crush and be ineffective at compressive strains greater than 0.004; that is, only the concrete core and the steel are assumed to be effective at compressive strains greater



(a) Unit 4 (b) Unit 7

Fig. 24 Comparison of Theoretical Moment-Curvature Relationships with Average Moment-Curvature Relationship Measured over a 12 in (305 mm) Gauge Length in Beam Adjacent to Column Face (1 kip in = 113 Nm, 1 in = 25.4 mm)

than 0.004. The experimental curves for Unit 4 (Fig. 24a) show a reduction in moment capacity in the first inelastic cycle at a somewhat higher curvature than that corresponding to a concrete strain of 0.004, indicating a higher crushing strain. In the experimental curves for Unit 7 (Fig. 24b) the reduction in moment capacity due to loss of concrete cover is not so obvious, possibly because of the effect of the compression steel. Analytical curves such as shown in Fig. 24 can be used to derive idealized moment-curvature loops for sections with a range of prestressed and nonprestressed steel contents for use in nonlinear dynamic analyses [15].

Analysis of Joint Core Shear Reinforcement

The critical diagonal tension cracks observed in the joint cores during the tests appeared to run from corner to corner of the joint core (see Figs. 11b to 16b and 18b to 22b) rather than at the 45° angle assumed in ACI 318-71. This observation has been made previously for reinforced concrete beam-column joints [18,17] and is not surprising since the corner to corner crack is parallel to the diagonal compression strut which runs across the joint core between the compression zones of the intersecting members (see Fig. 26a). This observation does suggest that an alternative procedure for calculating the horizontal shear force carried by the shear reinforcement would be to determine the force in the shear reinforcement which crosses the corner to corner crack [19,18,15]. The horizontal shear force carried by the horizontal shear reinforcement on this basis is

$$V_s = n A_v f_y \tag{9}$$

where A_v = area of one layer of shear reinforcement, f_y = yield strength of shear reinforcement and n = number of layers of shear reinforcement crossing the corner to corner crack.

In addition, these tests, and previously conducted tests on reinforced concrete beam-column joints [17,18,19], have demonstrated that the horizontal shear force carried by the concrete shear resisting mechanism in the joint core decreases significantly when intense cycles of seismic type loading are applied to the joint, particularly if the compressive load on the column is small. The degradation of shear carried by the concrete is due to repeated opening and closing of diagonal tension cracks in alternating directions in the joint core. This results in a gradual transfer of shear force from the concrete to the shear reinforcement as the concrete shear resisting mechanism degrades, and was illustrated by the strains measured on the joint core hoops of the units during the tests. The maximum stress in the steel hoops in the joint cores of Units 1, 4, 7 and 10 (which did not soften significantly during the tests) were 0.37, 0.73, 0.84 and 0.97 of the hoop yield strength. Fig. 25a shows the strains measured on the steel hoops in the joint core of Unit 4 to illustrate a typical distribution of hoop strains when yielding did not occur, and the gradual increase in hoops strains as the loading cycles progressed. For the remaining units, the yield strength was reached in the joint core hoops generally in the second inelastic load run (run 6), and with subsequent load runs the hoop strains increased appreciably as the joint core concrete became badly cracked. Fig. 25b illustrates the hoop strains in Unit 5 as an example of the hoop strain distribution in a joint core that did soften significantly. Nevertheless, considering the test results overall, it is felt that when the mean compressive stress on the column above the joint exceeds some nominal value, such as $0.1f'_c$, some horizontal shear can be allocated to the concrete. Comprehensive evidence of the horizontal shear which can be carried by the concrete in the joint core for a wide range of column loads after severe seismic loading is lacking. However the draft New

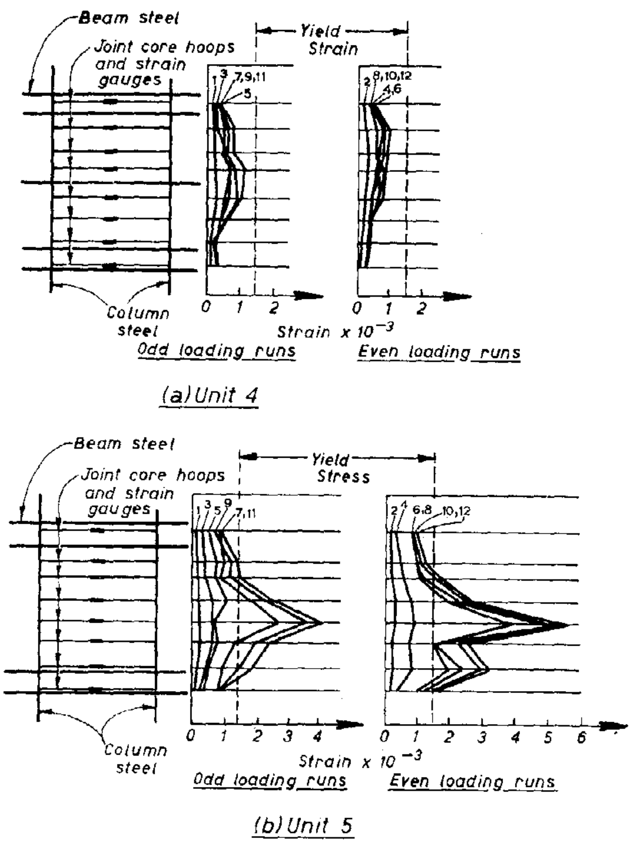


Fig. 25 Hoop Strains Measured in Joint Core of Units 4 and 5 at Peaks of Loading Runs

Zealand concrete design code will propose that the horizontal shear force carried by the concrete shear resisting mechanism when $N_u/A_g \geqslant 0.1f'_c$ is given by

$$V_c = 3\left(1 + \frac{f'_c}{3630}\right)\sqrt{\frac{N_u}{A_g} - \frac{f'_c}{10}}\; h\,b_c \quad \text{lb} \tag{10a}$$

$$\text{or } V_c = 0.25\left(1 + \frac{f'_c}{25}\right)\sqrt{\frac{N_u}{A_g} - \frac{f'_c}{10}}\; h\,b_c \quad \text{N} \tag{10b}$$

where N_u = column axial compressive load, A_g = gross area of column, f'_c = concrete compressive cylinder strength, h = overall depth of column, and b_c = width of column except that where the column width is greater than the beam width the value of b_c taken is not to exceed the width of the beam under consideration plus 3 in (150 mm) each side of the beam. That is, $h b_c = A_g$ unless the column width exceeds the beam width by more than 3 in (150 mm) on one or both sides. The units in Eq. 10a are lb and in and in Eq. 10b are N and mm. Eq. 10 gives $V_c = 0$ when $N_u/A_g = 0.1f'_c$ and increasing values of V_c with higher values of N_u/A_g. The present series of tests showed that in general the units with an effective prestressing tendon across the central region of the joint core throughout the test sequence were able to carry the applied horizontal shear forces successfully, provided sufficient joint core hoop reinforcement was present. However if no prestressing tendon was present in the central region of the joint core, or if its effect was reduced by inelastic strains, shear failure occurred even when a large amount of shear reinforcement was provided, as in the case of Unit 9. The draft New Zealand concrete design code will propose that the horizontal shear force carried by the prestressing steel is

$$V_p = 0.7P_{cs} \tag{11}$$

where P_{cs} = force in prestressing steel located in middle one-third of beam depth, after losses. The 0.7 factor in Eq. 11 is to take into account the possible reduction in P_{cs} due to inelastic strains in the steel.

Thus, as an alternative to the ACI 318-71 method, the horizontal shear strength of the joint core can be written as

$$V_u = V_c + V_p + V_s \tag{12}$$

where V_c, V_p and V_s are given by Eqs. 10, 11, and 9.

Table 4 sets out the theoretical maximum applied horizontal joint shear forces calculated using Eq. 5 with the forces as in Table 2. Table 3 also sets out the theoretical horizontal shear strengths for each unit calculated by: (i) The ACI 318-71 [16] design procedure (Eqs. 1 to 4) with shear carried by the concrete and by the hoops in the joint core across a 45° crack, and (ii) The alternative design procedure (Eqs. 9 to 12) with shear carried by the concrete, by the mid-depth prestressing steel, and by the hoops in the joint core across the corner to corner crack. According to Table 4, the ACI approach indicates that only the joint core of Unit 5 was significantly understrength, but that Units 1, 2, 4, 6 and 8 were within 5% of the theoretical ACI strength. Also, the table indicates that only the joint core of Unit 5 was within 5% of the theoretical strength proposed by the alternative method. In fact, as noted in the table, only in the case of Units 1, 4, 7 and 10 did the joint core hoops not yield and softening of joint core

Table 4 : Joint Core Theoretical Maximum Applied Horizontal Shears and Theoretical Horizontal Shear Strengths

Unit	Theoretical Maximum Applied Horizontal Shear Force, kips[a]	Theoretical Horizontal Shear Strength by Method (i), kips[b]			Theoretical Horizontal Shear Strength by Method (ii), kips[c]			
		V_c	V_s	V_u	V_c	V_p	V_s	V_u
1	212	35	182	217	35	44	212	291
2*	214	39	182	221	36	44	212	292
3*	207	38	182	220	36	0	212	248
4	221	37	182	219	35	42	212	289
5*	249	38	182	220	36	0	212	248
6*	218	38	182	220	36	42	212	290
7	196	38	182	220	36	25	212	273
8*	216	36	182	218	35	42	212	291
9*	243	35	253	288	35	0	295	330
10	209	40	253	293	36	41	295	372

* Yielding of hoops and softening in joint core eventually occurred during the inelastic loading cycles in the tests.

a At theoretical ultimate moment of beams.

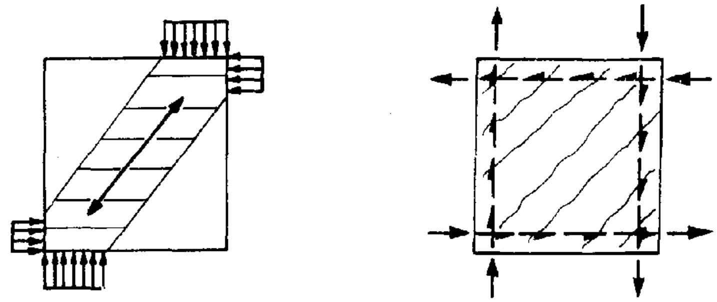
b Using ACI 318-71 Eqs. 1 to 4.

c Using Eqs. 9 to 12.

Note: 1 kip = 4.45 kN.

not occur during the tests. One reason for these inelastic deformations in the joint cores would have been that the actual maximum applied horizontal shear forces were greater than the theoretical values in Table 4, as was evident from the fact that the maximum measured moments exceeded the theoretical ultimate moments by up to 16%.

However it is considered that the main reason why many of the test units eventually failed in joint shear was that there was no vertical shear reinforcement present in the joint cores of any of the units. Shear across a joint core is transferred by two mechanisms [17]: (a) a diagonal compression strut which transfers the concrete compression forces between the compression zones of the intersecting members (Fig. 26a), and (b) diagonal tension forces induced into the joint core by the bond forces from the longitudinal steel in the members (Fig. 26b). Note that the diagonal compression strut is able to transfer that part of the bond forces which can be transferred to the strut within the concrete compression zones at the corners of the joint core. Hence a large axial compressive force on the column which results in a wide diagonal compression strut will allow a greater bond force to be transferred across the joint core by the diagonal compression strut and thus require less shear transfer by the mechanism of Fig. 26b. This is the reason for the beneficial effect of axial compression on the shear strength of the joint core. It is evident that the most favourable situation for the joint core is when as much of the shear as possible is transferred by the mechanism of Fig. 26a, since a diagonal compression strut theoretically does not require any joint core steel to function. Note however that the opening and closing of diagonal tension cracks in alternating directions will eventually weaken the diagonal compression strut unless effectively confined by joint core steel. Also, if the forces are introduced into the joint core mainly by bond (Fig. 26b)



(a) Diagonal compression strut between compression zones of members

(b) Diagonal tension from bond forces in longitudinal bars

Fig. 26 Shear Transfer Across Beam-Column Joint Core

substantial joint core reinforcement will be necessary to carry the diagonal tension forces induced by the vertical and horizontal bond forces, and that reinforcement in the joint core will be required in both the horizontal and vertical directions to carry the horizontal and vertical components of the diagonal tension forces [17]. In the test units with nonprestressed longitudinal steel in the beams, in the first inelastic load run the actions shown in Fig. 26a and b are both present. However, yielding of the longitudinal tension steel in the beams will mean that when the loading direction is reversed an open crack may remain in the beam concrete "compression zone" resulting in the beam compression force being applied to the joint core mainly by the longitudinal compression steel until that steel yields and the crack closes. When a full depth crack exists in the beam at the column face the actions from the beam are only those shown in Fig. 26b. However since the column steel has not yielded the compression forces in concrete from the column can still be transferred by the diagonal compression strut. Nevertheless with an open crack in the beam at the column face the beam actions need to be transferred mainly by diagonal tension unless the column compression is large enough for the compressed column concrete to pick up much of the beam steel bond forces. This reduction in the effectiveness of the diagonal compression strut,as far as beam forces are concerned, caused by full depth cracking, is the main reason for the degradation of the joint shear carried by the concrete and hence is the main reason for the gradual transfer of shear to be resisted to the shear reinforcement. This effect apparently occurred in those units with significant quantities of nonprestressed steel (Units 2, 3, 6, 7 and 10) leading eventually to shear distress in the joint cores of Units 2, 3 and 6. The test units had longitudinal column bars placed at only the four corners of each column. Had the columns of these test units contained a number of column bars distributed around the section perimeter, which could have acted as vertical shear reinforcement, shear failure of the joint cores may not have occurred. Also, Units 3, 5 and 9 had no central prestressing tendon and,in the absence of intermediate column bars between the corner bars,were particularly vulnerable to joint core shear distress. Unit 8 maintained a high moment capacity during the loading cycles and the high joint core shear forces maintained eventually led to joint core shear distress.

The function of vertical shear reinforcement has been previously discussed [17] and recent beam-column joint tests,by Blakeley, Megget and

Priestley [20] and Beckingsale [19], have shown that the presence of longitudinal column reinforcement distributed around the perimeter of the column section does contribute towards the joint core shear resistance. The intermediate column bars in those tests were able to help control the diagonal tension cracks and to contribute to the truss action carrying diagonal tension within the joint core. Also the pretensioned strands present in the central region of the column sections in the beam-column assemblies tested by Blakeley and Park [10] apparently had assisted the joint shear transfer since those units showed no sign of joint core distress. The results of the present series of tests appears to confirm that column bars should be placed around the perimeter of columns to help carry the vertical components of joint core forces; that is, the use of four bar columns should be discouraged. The actual amount of vertical shear reinforcement required in typical joint cores is still in need of detailed clarification but it is considered that at least one intermediate bar should exist between the corners bars on each side of the joint core and that the vertical column bars should not be spaced at more than 6 in (150 mm) centres. Methods for calculating the amount of vertical reinforcement necessary in joint cores for shear transfer are being considered currently for inclusion in the draft New Zealand concrete design code.

Analysis of Bond Strength of Beam Flexural Steel

Slip of nonprestressed longitudinal bars in the beams occurred through the joint core during some of the tests. Reference to Fig. 8 shows that when beam moments of opposite sign occur on opposite sides of a column substantial forces need to be transferred from the steel to concrete in the joint core by bond if the beam forces are to develop each side of the column as calculated. For example, the total force to be transferred by bond by the top and bottom layers of nonprestressed bars in Fig. 8 is $T_4 + C_5$. The bond stresses so generated can be much higher than those allowed in ACI 318-71 for anchorage. Hence it can be expected that slip of these bars may occur if the bar diameter is large and/or the column depth is small. In fact slip did occur in the case of the No. 9 (28.6 mm dia.) bars in Units 2 and 3 and the No. 8 (25.4 mm dia.) bars in Unit 3, but no significant slip occurred in the case of the No. 6 (19.1 mm dia.) bars in Units 6, 7 and 10. All longitudinal nonprestressed steel in the units was from deformed bar. Since the columns were 16 in (406 mm) deep, a simple recommendation which arises from this result is that the ratio of bar diameter to column depth should not exceed 0.75/16 = 1/21. In the light of this result and other tests [19,20] the draft New Zealand loadings code will propose that when plastic hinges can form in the beams adjacent to the column the diameter of nonprestressed beam bars passing through the joint should not exceed 1/25th of the column depth in that direction for deformed bars with a yield strength of 40 ksi (276 MPa). Higher yield strength nonprestressed bars would need a more severe limitation. The prestressing tendons were grouted in corrugated metal ducts and did not show any signs of slip through the joint core during the tests. Analysis of the bond stresses induced by the tendon forces indicated that the bond stresses were only high in the case of the beam sections where there were only two tendons (Units 5 and 9). For tendons embedded deep in a member the bond situation is particularly favourable. The tendon sizes used in these tests resulted in satisfactory bond behaviour.

CONCLUSIONS

The conclusions from the test series are as follows:

Beams

The prestressed beams of the test units showed a reduction in strength and stiffness once crushing of the compressed concrete commenced during the loading cycles in the inelastic range, because the loss of the cover concrete resulted in a reduction in the area of the effective concrete section in the plastic hinge regions. In such zones transverse steel in the form of stirrup ties should be placed with minimum cover to prevent excessive loss of concrete section and at reasonably close spacing to effectively confine the core concrete. It is recommended that the spacing of stirrup ties should not exceed 4 in (102 mm) or one quarter of the effective depth of the member.

The flexural capacity at large deformations of the partially prestressed and reinforced concrete beams was not so influenced by crushing of the cover concrete, mainly because of the smaller neutral axis depth and the presence of compression reinforcement. Compression reinforcement can carry part of the compressive force that was carried by the crushed cover concrete. However stirrup ties as for prestressed beams are still required. In addition, stirrup ties should also be close enough to prevent buckling of the nonprestressed compression steel. It is recommended that the spacing of stirrup ties should not exceed six longitudinal bar diameters in order to laterally support the bars against buckling. Also, nonprestressed reinforcement may slip through the joint core due to breakdown of bond, particularly if the column section is small and the bar diameter is large, thus reducing its effectiveness. A limiting bar size as a function of column size is evidently required in frames subject to intense seismic load reversals when plastic hinges form adjacent to columns if stiffness and strength degradation due to bar slip is to be prevented. It is recommended that the diameter of deformed bars should not exceed 1/25th of the column depth for steel with a yield strength of 40 ksi (276 MPa).

The flexural strength of the beams in the first inelastic loading runs was up to 16% higher than the theoretical flexural strength, due mainly to the maximum moment being reached at an extreme fibre compressive concrete strain of greater than 0.003 and due to confinement of the compressed beam concrete immediately adjacent to the column face by the column.

Columns

In these tests the columns were stronger than the beams and hence were not critical elements, apart from in the joint core regions.

Joint Cores

The shear reinforcement of the beam-column joint cores had been designed according to the method of Appendix A of ACI 318-71 [16]. The beams of the test units all reached at least 95% of their theoretical flexural strength in the first inelastic loading run in each direction, accompanied by yielding of joint core hoops in some units, indicating that the ACI design approach was adequate for the first inelastic load cycle. For those units in which the hoops yielded, further load cycles resulted in a degradation of the joint core shear strength. For those units the strength of the joint core then governed the strength of the unit and the greater part of the inelastic deformation of the unit then occurred in the joint core. Thus although the ACI approach for joint core shear design allowed the attainment of the design shear strength satisfactorily in the first inelastic load cycle, degradation of joint core shear strength occurred in some units in subsequent inelastic load cycles because of large alternating diagonal tension cracks in the joint core due to yielding of hoops leading to a breakdown in the concrete shear resisting mechanism. In the units with nonprestressed longitudinal steel in the beams, it was evident that joint core shear failure was also brought about in the subsequent inelastic load cyles by the introduction of beam compressive forces into the joint core mainly by bond forces from the longitudinal steel. Those units with no central tendon passing through the joint core were particularly vulnerable to shear failure of the joint core.

A more logical procedure for joint core shear design, than the ACI 318-71 approach, can be recommended on the basis of the observed joint core behaviour. It was observed that the shear carried by the concrete shear resisting mechanisms reduced with cyclic loading, that central prestressing tendons in the beam aided joint core shear transfer, and that the critical diagonal tension crack ran from corner to corner of the joint core. Thus the design horizontal shear force can be transferred by the sum of the shear carried by the concrete (only when the mean column compressive stress exceeds $0.1f'_c$, as given by Eq. 10) plus that carried by prestressing tendons in the middle one-third of the beam depth (Eq. 11) plus that carried by the horizontal shear reinforcement crossing the corner to corner crack (Eq. 9).

It has also been reported previously that vertical reinforcement spaced around the perimeter of the column section in the joint core acts also as shear reinforcement, contributing to the truss action in the joint core which is necessary to transfer much of the shear introduced by bond forces, and therefore improves the joint core shear behaviour. A practical method of providing this vertical shear reinforcement is to ensure that the longitudinal reinforcement in the column is spread around the perimeter of the column, and is not just placed at the corners of the section. The test units did not contain intermediate column bars between the corner bars and it is considered that had adequate vertical shear reinforcement been present, joint core shear distress would not have occurred. This conclusion needs further testing to check its validity.

The tests indicated the difficulty of preventing joint core distress during severe seismic loading. However inelastic joint core deformation must be regarded as undesirable because of the difficulty of subsequent repair of joint cores and the possible collapse of the building due to loss of load carrying capacity of the columns in the joint core region. Therefore the formation of plastic hinges in the beams is to be preferred when severe seismic loading is to be sustained.

Energy Dissipation

All units showed considerable energy dissipation once the maximum moment capacities had been reached. As expected, even after extensive inelastic deformations had been enforced, the prestressed concrete beam showed considerable deflection recovery. The ordinary reinforced member showed greater energy dissipation than the partially prestressed member. However comparisons between these specimens are difficult because the inelastic deformations from some units came mainly from the beam plastic hinges, and in others from the shear deformations of the joint cores.

Repair

Repairs made to a prestressed beam by replacing the damaged beam concrete showed that it is possible to repair damaged members. Repairs to units with extensive damage to joint cores would have been much more difficult if not impossible to carry out, however.

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