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US/JAPAN COOPERATIVE EARTHQUAKE ENGINEERING RESEARCH PROGRAM SPONSORED BY THE NATIONAL SCIENCE FOUNDATION THROUGH GRANT NUMBER CEE-7816730

DEPARTMENT OF CIVIL ENGINEERING
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ANN ARBOR MICHIGAN 48109

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15. Supplementary Notes

16. Abstract (Limit: 200 words)

Papers delivered at the seminar are presented. Among the topics addressed are anchorage and diaphragm systems for tilt-up-wall buildings; epoxy repaired concrete beams; ways to estimate damages and repairs for buildings affected by the Miyagi-Ken-Oki earthquake; the development of post-earthquake measures for buildings and structures damaged by earthquakes; and an overview of the state-of-the-art in seismic strengthening of existing reinforced concrete buildings in Japan. It is noted that increased activity in repairing and retrofitting structures is taking place in both the United States and Japan. It is recommended that the practice of including presentations by repair and retrofit designers prior to field visits to those specific structures be continued. A seminar agenda and list of participants if provided.

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Meetings	Dynamic structural analysis	Maintenance
Earthquakes	Reinforced concrete	Buildings
Earthquake resistant structures	Beams	Walls

b. Identifiers/Open-Ended Terms

Ground motion	R.D. Hanson, /PI
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PREFACE

Research, design and construction activities in the repair and retrofit of structures for earthquake resistance both in Japan and the United States have been increasing rapidly over the last decade. One way to maximize the benefits of research and experiences of others is to share them at an early stage of development and discuss alternative approaches and techniques. This was the purpose of the US/Japan Cooperative Research Program in Earthquake Engineering on Repair and Retrofit of Structures sponsored by the National Science Foundation through grant number CEE–7816730 to The University of Michigan.

A series of three seminars (May, 1980 in Los Angeles; May, 1981 in Sendai and Tsukuba, Japan; and May, 1982 in San Francisco) were held to share and discuss research results and field experiences. The Proceedings of these three seminars have been published in three volumes. A fourth volume contains an English translation of several Japanese reports on evaluation of earthquake resistance of existing buildings prepared for Shizuoka Prefecture as part of their Earthquake Hazard Reduction Program.

The financial support of the National Science Foundation, and the personal efforts by Dr. John B. Scalzi, NSF Program Manager, in establishing this program; the contributions of Mihran S. Agbabian and James Warner in organizing the Los Angeles meeting and field trip; and the contributions of Loring A. Wyllie, Jr. and Oris H. Degenkolb in organizing the San Francisco meeting and field trip are sincerely appreciated. The meeting and field trip in Japan was organized by Dr. Makoto Watabe and by Dr. Masaya Hirosawa who receive the sincere thanks and appreciation of all US participants.

The opinions, findings, conclusions and recommendations expressed in these volumes are those of the individual contributors and do not necessarily reflect the views of the NSF or other private or governmental organizations.

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Ann Arbor, Michigan

US/Japan Cooperative Research Program in Earthquake Engineering
on Repair and Retrofit of Structures

Proceedings of the Third Seminar—May 1982

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INTRODUCTION

The third joint meeting of the US/JAPAN Cooperative Research Program in Earthquake Engineering on Repair and Retrofit of Structures was held at the Beverly Plaza Hotel in San Francisco from May 13 through May 15, 1982. From the experiences of the earlier two meetings this meeting scheduled a day and a half of technical presentations and discussion separated by a day field study tour of strengthened and retrofitted bridges and buildings in the San Francisco area.

Eight representatives from Japan and sixteen representatives from the United States participated in the technical sessions, discussions and field study.

From Japan:

T. Endo	-	Tokyo Metropolitan University
M. Hirosawa	-	Building Research Institute
T. Iwasaki	-	Public Works Research Institute
T. Kaminosono	-	Building Research Institute
S. Nakata	-	Building Research Institute
S. Noda	-	Port and Harbour Research Institute
S. Okamoto	-	Building Research Institute
T. Okubo	-	Public Works Research Institute

From the United States:

M. S. Agbabian	-	Agbabian Associates
V. V. Bertero	-	University of California at Berkeley
O. H. Degenkolb	-	Consultant
R. D. Ewing	-	Agbabian Associates
N. F. Forell	-	Forell/Elsesser Engineers
G. Greenwood	-	H. J. Degenkolb & Associates
R. D. Hanson	-	University of Michigan
W. T. Holmes	-	Rutherford and Chekene
H. S. Lew	-	National Bureau of Standards
L. Lund	-	Los Angeles Dept. of Water and Power
J. L. Noland	-	Atkinson-Noland and Associates
J. M. Plecnik	-	North Carolina State University
C. F. Scheffey	-	Federal Highway Administration
R. E. Scholl	-	URS/J.A. Blume & Associates
J. Warner	-	Consultant
L. A. Wyllie, Jr.	-	H. J. Degenkolb & Associates

PROGRAM

Session I. 9:00 a.m.-12:30 p.m., May 13, 1982

Anchorage and Diaphragm Systems for Tilt-Up Wall Buildings by R. D. Ewing

Retrofitted Structures Against Earthquake and Research on Them--Activity of Japan Concrete Institute by T. Endo

Experimental Test Results on Epoxy Repaired Beams Subjected to Seismic Loads and/or Fires by J. M. Plecnik

A Repair Effect of Reinforced Concrete Joint Assemblies Subjected to Seismic Loads by S. Nakata

Preliminary Results from Destructive Testing of an Epoxy Repaired Four-Story Reinforced Concrete Frame by R. E. Scholl

A Repairing Test of Full Scale Seven Story Building Subjected to Seismic Load by T. Kaminosono

Seismic Retrofit of Small Pumping and Chlorinating Station Buildings by L. Lund

Session II. 2:00 p.m.-6:00 p.m., May 13, 1982

Campton Hotel Rehabilitation by G. Greenwood

Outline of Research on Estimation of Damages and Repairs of the Buildings Damaged by the Miyagiken-Oki Earthquake, June, 1978 by M. Hirosawa

Repair and Retrofit of Stanford University Buildings by W. T. Holmes and H. Davis

On the Development of Post-Earthquake Measures for Building and Structures Damaged by Earthquakes by M. Hirosawa

Earthquake Bracing Program: United States Survey, Menlo Park, California by N. F. Forell

Examples of Repair and Retrofit Work on Road Bridge Substructures by T. Iwasaki

Bridge Retrofitting Details by O. H. Degenkolb

Group Dinner at New Pisa Restaurant

Session III. Field Trip 8:30 a.m.-10:00 p.m., May 14, 1982

9:00-10:00 a.m. Campton Hotel

10:20-11:00 a.m. Bridge Retrofit (China Basin)

Lunch at Stanford University Faculty Club

1:00-2:00 p.m. Stanford Quadrangle

2:00-3:30 p.m. USGS Building Strengthening

3:30-5:30 p.m. USGS Technical Tour

6:00-10:00 p.m. Dinner at L.A. Wyllie's home

Session IV. 9:00 a.m.-1:00 p.m., May 15, 1982

An Investigation into Methods of Nondestructive Evaluation of Masonry Structures by J. L. Noland

Effects on Behaviors of Reinforced Concrete Frames by Adding Shear Walls by T. Endo

Studies Regarding Repair and Retrofitting of the Imperial County Services Building, El Centro, California by V. V. Bertero

On the Developments of Post-Earthquake Measures for Civil Engineering Structures Damaged by Earthquakes by T. Iwasaki

Reinforcing Steel Considerations Unique to Repair and Retrofit by J. Warner

Damage to Buildings from Urakawa-Oki, Japan Earthquake of March 21, 1982 by S. Okamoto

Discussed but not presented:

An Overview of the State-of-the-Art in Seismic Strengthening of Existing Reinforced Concrete Buildings in Japan

Session V. 1:00 p.m.-1:30 p.m.

Closing Session - Resolutions and Agreements

Free Afternoon and Evening

SUMMARY AND RESOLUTIONS

It was noted that increased activity in repair and retrofitting of structures is taking place in both Japan and the United States. The discussions of the presented papers and associated topics was lively and beneficial to all participants. All participants commented on the friendly, open exchange of ideas experienced during this meeting. It was a common thought that this session was the best of the three held to date. The practice of including presentations by repair and retrofit designers prior to field visits to those specific structures should be continued in the future. Specific resolutions adopted by the participants were:

1. This meeting provided an extremely valuable exchange of technical information such as follows:

   a. Research Program for Assessment Method on inspection and repair/retrofitting of structures damaged by earthquakes.

   b. Examples of repair and strengthening for existing important structures.

   c. Various basic experiments and analysis on repair and retrofitting techniques.

2. However, it was recognized there remained many problems especially on establishment of objective seismic safety. There were few reports which show the details of repair/retrofitting techniques and methods to evaluate their effect.

3. Accordingly, it is expected that related research and development will be actively continued in both countries. Technical exchange should continue through the subcommittee of the UJNR Panel on Wind and Seismic Effects. It is recommended that a full cooperative exchange be held after 1984. The following information could be exchanged:

   a. State of Art report on

      (i) actual conditions on repair/retrofitting of existing structures

      (ii) actual conditions on repair/retrofitting structures damaged by earthquakes

      (iii) actual conditions on inspection of damaged structures.

   b. Data on rehabilitation techniques including their details and methods to evaluate their effectiveness.

   c. Proposals for Assessment Method on rehabilitation.

4. The reports presented in these three workshops will be published during 1982 and distributed to seminar participants and members of the UJNR Panel on Wind and Seismic Effects.

ANCHORAGE AND DIAPHRAGM SYSTEMS FOR TILT-UP-WALL BUILDINGS

by

M.S. Agbabian* and R.D. Ewing*

SUMMARY

The demand for tilt-up-wall (TUW) buildings has progressively increased during the last three decades, due to the speed, ease, and economy offered by TUW construction. Practical and reliable analysis methods and design guidelines are needed for the mitigation of seismic hazards in these types of buildings, as noted in a recent report by the Structural Engineers Association of Southern California. Accordingly, the National Science Foundation has sponsored a two-year research program for TUW structures, entitled "Guidelines for Mitigation of Seismic Hazards in Tilt-Up-Wall Structures." This paper briefly summarizes the current progress of the ongoing research program.

A categorization study identifies the current inventory of TUW buildings and identifies their materials and methods of construction. This study provides data to assist in developing mathematical models for the analysis and testing of typical TUW buildings. Preliminary analyses clearly indicate that typical TUW buildings can be quite adequately analyzed using lumped parameter models. Moreover, the analysis model can account for the interaction among the structural components and can be used to define the kinematic environment for the component testing of full-scale TUW panels subjected to out-of-plane motions. In addition, a forthcoming experimental program for the dynamic, out-of-plane testing of typical TUW panels is outlined.

INTRODUCTION

Tilt-up-wall (TUW) construction is a form of precast concrete construction used primarily for one- or two-story buildings, and in a few cases for multistory buildings. The principal feature of the construction method is the manner in which walls of the building are fabricated and placed. Wall panels are cast in a horizontal position at the site and after curing for as little as two days can be tilted up and moved into place. The panels are fabricated in full height sections and several panels are required to complete the side and end walls of a building. Another advantage of TUW buildings is that they offer some economy with respect to other traditional types of construction.

*Agbabian Associates, El Segundo, California

The demand for economical building systems has progressively increased during the last three decades, and TUW construction has grown very rapidly throughout the United States, including seismically active areas.

The structural integrity of tilt-up buildings during seismic loading has been observed only to a limited degree. Damage to TUW buildings was reported in the great Alaskan earthquake of 1964 and in the San Fernando earthquake of 1971. The reported damage has been attributed mainly to failure of the connections between the TUW panels and roof diaphragms. However, TUW structures built with earthquake resistant panel-to-diaphragm connections have not yet been tested by damaging earthquakes to evaluate their performance. The engineering profession has recognized the need for design guidelines for TUW construction. Several areas of consideration such as panel height-to-thickness ratios, panel anchorage, joinery between wall panels, and panel reinforcement have been addressed (SEAOSC, 1979). Moreover, the profession recognizes that additional work needs to be performed to provide more complete guidelines.

The National Science Foundation (NSF) has approved a two-year research program for TUW structures, entitled "Guidelines for Mitigation of Seismic Hazards in Tilt-Up-Wall Structures" and funding has been provided for the first year.

The scope of the research program includes: (1) categorization of existing TUW construction in the United States; (2) analyses of typical TUW structures with variations in diaphragm systems, panel height-to-thickness ratios, and building aspect ratios; (3) full-scale, component tests of representative TUW panels subjected to dynamic, out-of-plane motions resulting from an ensemble of earthquake motions representing highly seismic zones of the United States; (4) verification and calibration of mathematical models for the analysis of TUW structures; (5) evaluation of the effect of various diaphragm systems on panel response; (6) evaluation of anchorage requirements; and (7) development of guidelines for mitigation of seismic hazards in TUW buildings. The progress of this research project is reported in this paper.

CATEGORIZATION OF TUW CONSTRUCTION

A survey of existing TUW construction indicated that panels as high as 65 ft (19.8 m) with height-to-thickness ratios in excess of 60 have been built in California. Some important structural features of components of TUW structures in different parts of the United States are categorized in Table 1, and some typical wall design parameters are given in Table 2. Typical details of a wood ledger and panel footing are shown in Figures 1 and 2, respectively.

TABLE 1. SURVEY OF TILT-UP CONSTRUCTION SYSTEMS IN THE UNITED STATES

Region	Roof Diaphragms	Pilasters	Foundation	Ledgers	Wall Connected to Floor	Code H/t Limit	Remarks
California	Wood	No	Continuous and isolated	60% Wood ‡ 40% Steel sections	Yes	25 using ACI 318-77 25-42 w/slenderness analysis 42-50 in special cases	Diaphragm designed to resist lateral load
Washington and Oregon	Wood	No	Continuous	Wood	Yes	see Calif.	Diaphragm designed to resist lateral loads
Arizona and Utah	Wood	No	Continuous	Wood	Yes	No limit	Diaphragm designed to resist lateral loads
Texas and Southeast Georgia, Florida, Tennessee, Carolinas	Steel	No	Continuous and isolated	Steel sections act as diaphragm chord	Yes	No limit	Diaphragm action depends on proper location of chords
Ohio and North Central States	Steel	No	Continuous and isolated	Steel sections or pockets in wall with bearing plates *	Yes	No limit (Generally ≤ 50)	Diaphragm action depends on proper location of chords and bracing
Colorado	Prestressed concrete	Yes	Continuous	Pilasters	Yes	No limit	
New Jersey and Mid-Atlantic Coast States	Steel	Steel cols. or cast in place or no pilasters	Isolated	Steel section	Yes	No limit	
Northeast	TUW is gradually getting popular in some areas						

Note: Information shown in this table indicates the system that is mostly used in a particular region

Vertical ties between panels are almost eleminated in all states

‡ Approximate Ratios

* Growing tendency to do away with parapet part of TUW and load roof joists directly on the top of TUW.

TABLE 2. TYPICAL WALL PANEL DESIGN PARAMETERS (Kripanarayanan, 1980)

Parameter	Value
Unit weight of concrete (w), pcf	150
Compressive strength of concrete (f'_c), psi	≤4,000
Yield strength of reinforcement (f_y), ksi	60
Capacity reduction factor (ϕ)	1.0
Panel thickness (h), in.	5½, 6½, 7½
Reinforcement ratios (ρ), percent	0.15, 0.25, 0.50, 0.75
Transverse loads (q_u), psf	0, 15, 30, 45
End eccentricities (e), in.	1.0, h/2; h/2 + 3½

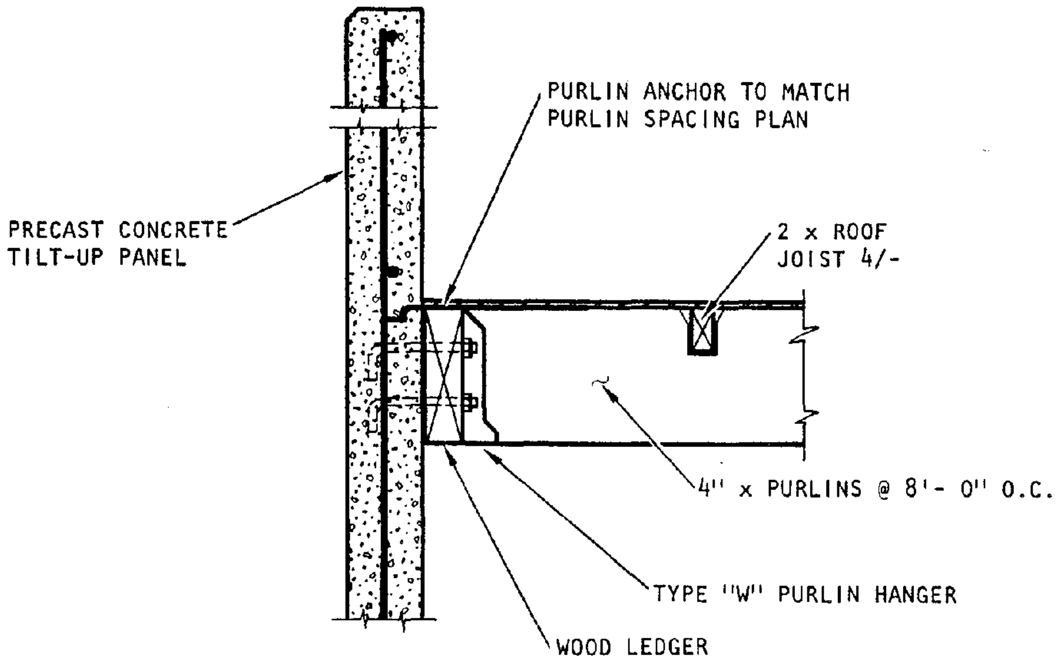


FIGURE 1. WOOD LEDGER AND WOOD PURLIN

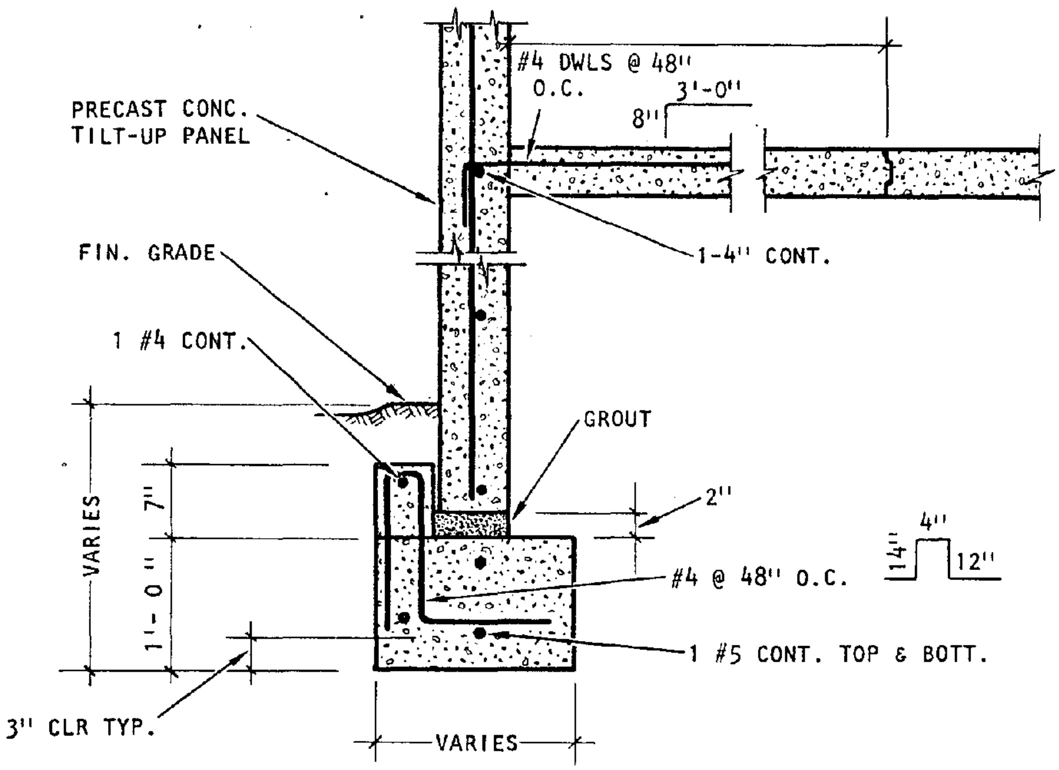


FIGURE 2. PANEL FOOTING

EXAMPLE ANALYSIS OF A TUW STRUCTURE

The typical one-story warehouse-type building shown in Figure 3 was selected for the analysis. The building consists of a wood roof diaphragm 300 ft (91.4 m) long by 150 ft (45.7 m) wide supported on four sides by 20 ft (6.1 m) high concrete tilt-up walls. The critical orientation of the earthquake motion is perpendicular to the long dimension of the structure as shown in Figure 3.

For the aspect ratio of the end wall considered in this example, the earthquake ground motion will be transmitted from the foundation level to the top of the end shear walls with very little modification (Adham and Ewing, 1978). Therefore, the earthquake ground motion is assumed to be applied directly at the ends of the roof diaphragm, as well as at the bottom of the TUWs.

For the critical orientation of the earthquake motion, the side TUWs will undergo primarily bending deformations while the diaphragm will undergo primarily shear deformations. Accordingly, the roof diaphragm is modeled as a deep shear beam and the TUW panels are modeled as flexural beams. Due to the assumed symmetry in both geometry and loading, only half of the building about the centerline of the long dimension is considered in the model. A schematic of such a model is shown in Figure 4. A further simplification is made by assuming the response of the two side walls to be identical, where the corresponding upstream and the downstream tilt-up side wall segments can be lumped into one single beam.

The tilt-up side wall panels are represented by linear elastic uniform beam elements in this preliminary analysis. However, nonlinear beam elements will be used in the final models, since yielding of the panels is anticipated. Moreover, nonlinearities will be needed if the P-Δ effects are important. The properties of these beam elements are prescribed by the length of the beam, elastic modulus, density, shear area, and principal moment of inertia associated with bending (Table 3). The TUW panels have a height-to-thickness ratio, H/t, of approximately 44, and a 5% damping ratio for the beams is used in this analysis.

TABLE 3. LINEAR BEAM ELEMENT PROPERTIES

Density, lb/ft^3	Young's Modulus E, psi	Shear Modulus G, psi	Thickness, in.	Width, ft	Shape Factor, K
145	3.1×10^6	1.3×10^6	5.5	37.5	0.85

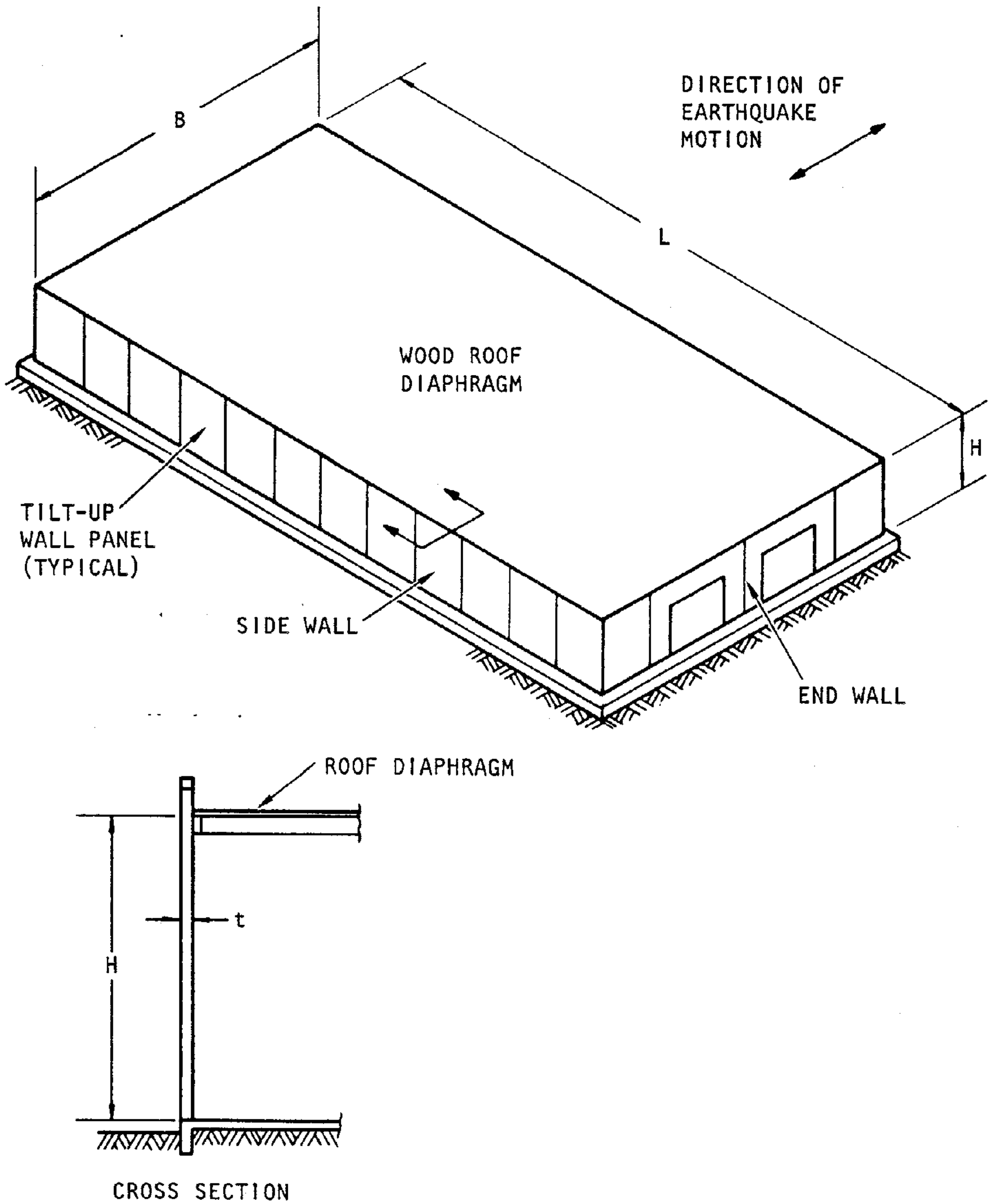


FIGURE 3. ONE-STORY TILT-UP-WALL BUILDING WITH A WOOD ROOF DIAPHRAGM

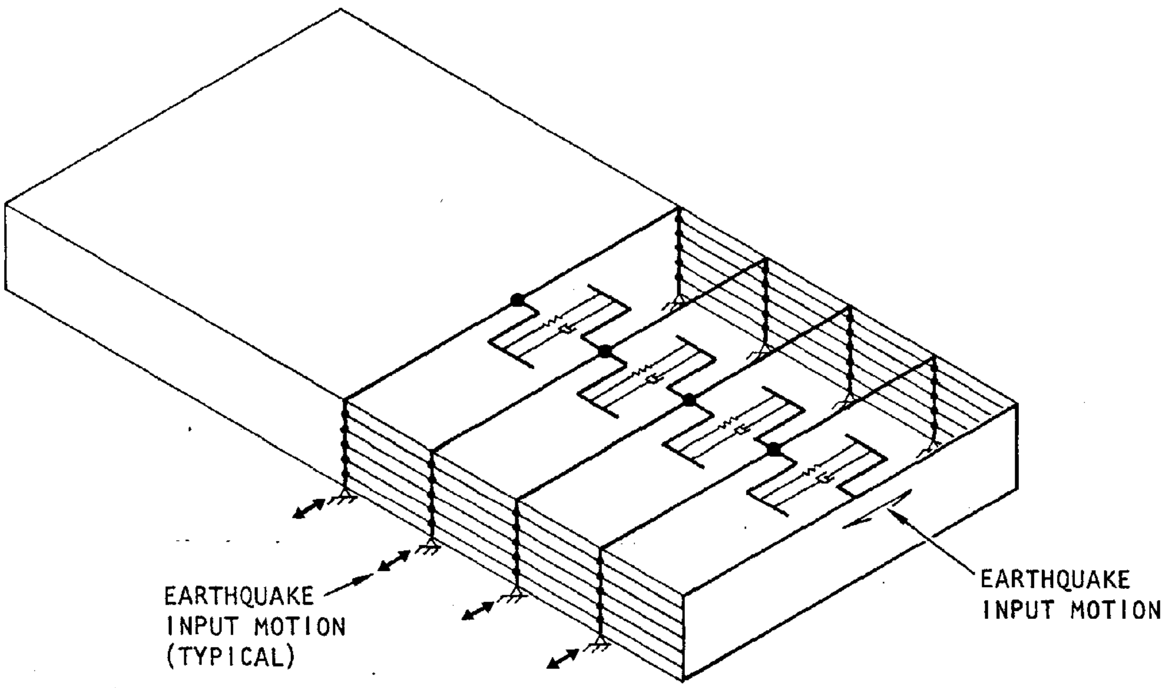


FIGURE 4. ANALYTICAL MODEL FOR TUW BUILDING

The roof diaphragm segments are represented by nonlinear, inelastic, hysteretic shear springs and viscous dampers. The force-deformation relationship of these spring elements are modeled based on cyclic in-plane static loading tests that have been conducted on 20 ft (6.1 m) by 60 ft (18.3 m) plywood diaphragms as shown in Figures 5a and 5b (ABK, 1981a). In this model the total deformation mechanism of the diaphragm between the two points of loading is smeared into a nonlinear, hysteretic shear spring element (Fig. 5c). The nonlinear, hysteretic behavior of plywood diaphragms is typified by the cyclic test results shown in Figure 6a. An analytical representation for this type of diaphragm is shown in Figures 6b and 6c, where Figure 6b shows the overall force-deflection envelope and Figure 6c shows a typical cyclic load path. A second-order curve has been selected to represent the force-deflection envelope for the diaphragm as given below:

$$F(e) = \frac{F_u e}{\frac{F_u}{K_i} + |e|}$$

where

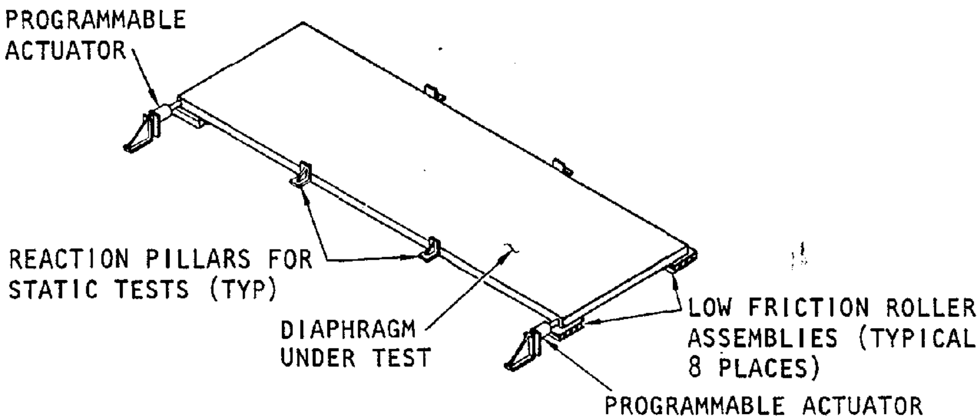
$F(e)$ = Spring force

e = Spring deformation in terms of the relative displacement of its ends

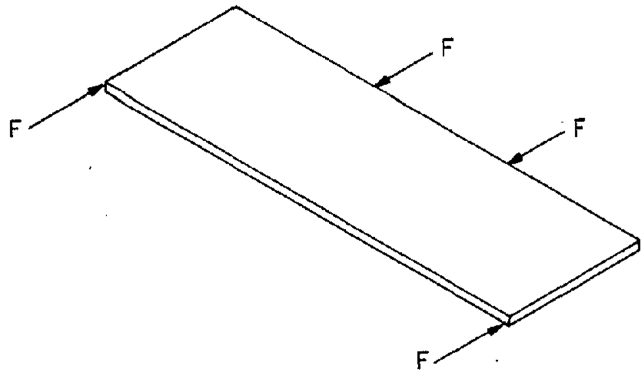
F_u = Ultimate capacity of spring at large values of e

K_i = Initial stiffness of spring

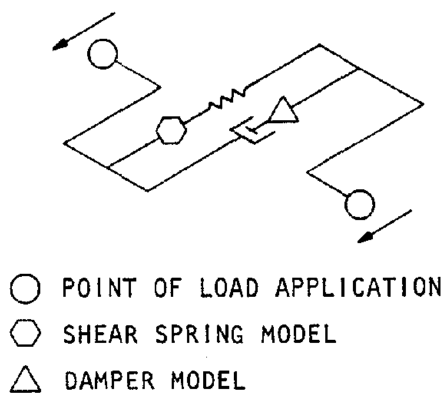
The unloading and reloading portions of the force-deflection curve are idealized by piecewise linear segments (Fig. 6c) that are controlled by the unloading slope K_2, and the residual force or pinch force value, F_1. The values of the constants defined above are based on the data obtained from tests (ABK, 1981a). Due to differences in dimensions between the TUW model and test specimen, scaling laws have been used to calculate the values of diaphragm properties for the model. In the scaling, stiffness is inversely proportional to segment length, ℓ, and directly proportional to depth, d, ($K\,\ell/d$ = constant) and force is directly proportional to depth (F/d = constant). These values are shown in Table 4.



(a) Test setup for quasi-static testing of diaphragms

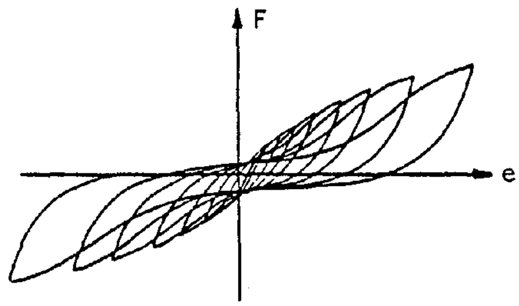


(b) Idealized loading configuration

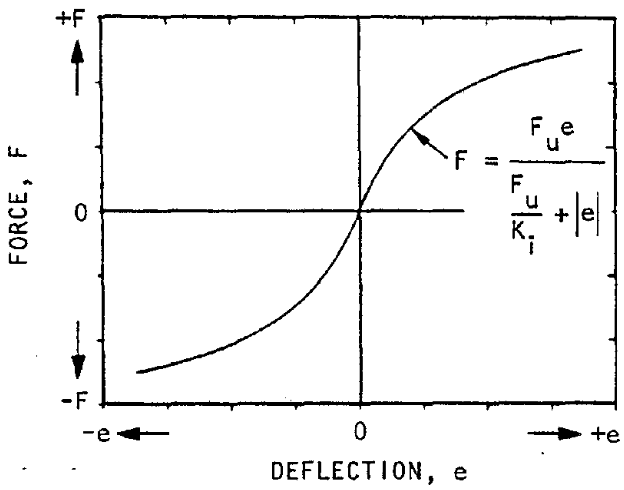


(c) Nonlinear spring and dashpot model
representing diaphragm segment

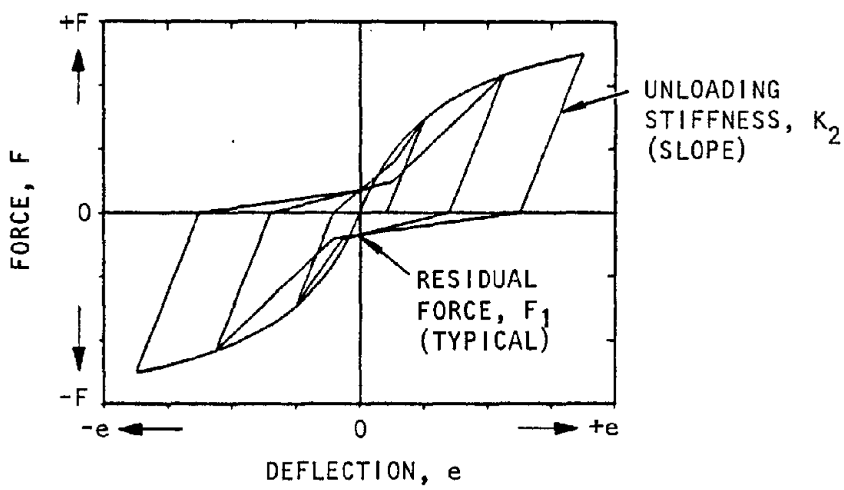
FIGURE 5. TEST CONFIGURATION FOR OBTAINING PROPERTIES OF DIAPHRAGMS



(a) Typical cyclic load deflection diagram for plywood diaphragms



(b) Force-deflection envelope of model



(c) Typical cyclic load-deflection diagram for model

FIGURE 6. LOAD DEFLECTION MODEL FOR WOOD DIAPHRAGMS

TABLE 4. PARAMETERS FOR THE ROOF DIAPHRAGM MODEL

Unit Weight, lb/ft^2	K_i, kip/ft	K_2, kip/ft	F_1, kip	F_u, kip
20	1300	1300	24	240

The N69W component of the 1971 Castaic acceleration record was selected as the basis for the input ground motion to the TUW building analysis. The reason for selecting this record is because of its high-frequency content and the relatively early arrival of the peak acceleration (-104.5 in./sec^2 at 1.9 sec); moreover, it represents a typical, strong nearby event for the California Pacific Coast region.

The intensity of ground shaking used in this study represents the level of shaking expected in a highly seismic area such as Los Angeles. The Effective Peak Acceleration (EPA) of such an area is 0.40 g (ATC, 1978). The acceleration input was, therefore, scaled to the 0.40 g EPA level by multiplying by a uniform scaling factor of 1.8. The unscaled response spectra are shown in Figure 7, and the scaled displacement, velocity, and acceleration time-histories are shown in Figures 8a, 8b, and 8c, respectively.

RESULTS OF THE ANALYSIS

The analysis was performed using the STARS/III computer program (AA, 1981). The input to the analysis consists of the first six seconds of the scaled N69W component of the 1971 Castaic record. Figures 8b and 8c indicate that the major peaks of the velocity and acceleration occur within the first two seconds of the time history. Therefore, the first six seconds of the input motion should induce significant structural response.

The results of the analysis indicate that the most critical response of the panels (for H/t = 44) occurs at the midspan between the two end walls. The variations of the absolute values of the maximum bending moment and acceleration along the height of the middle panel are given in Figure 9. The results of the analysis are compared to those obtained by using current design methods as shown in Table 5. This comparison indicates that the dynamic seismic, elastic moments are approximately twice the equivalent static seismic moments calculated by current design methods. The maximum input peak acceleration of 0.48 g was amplified to 0.60 g at the roof level. This represents a 25% increase in the force at the connection of the panel to the roof diaphragm.

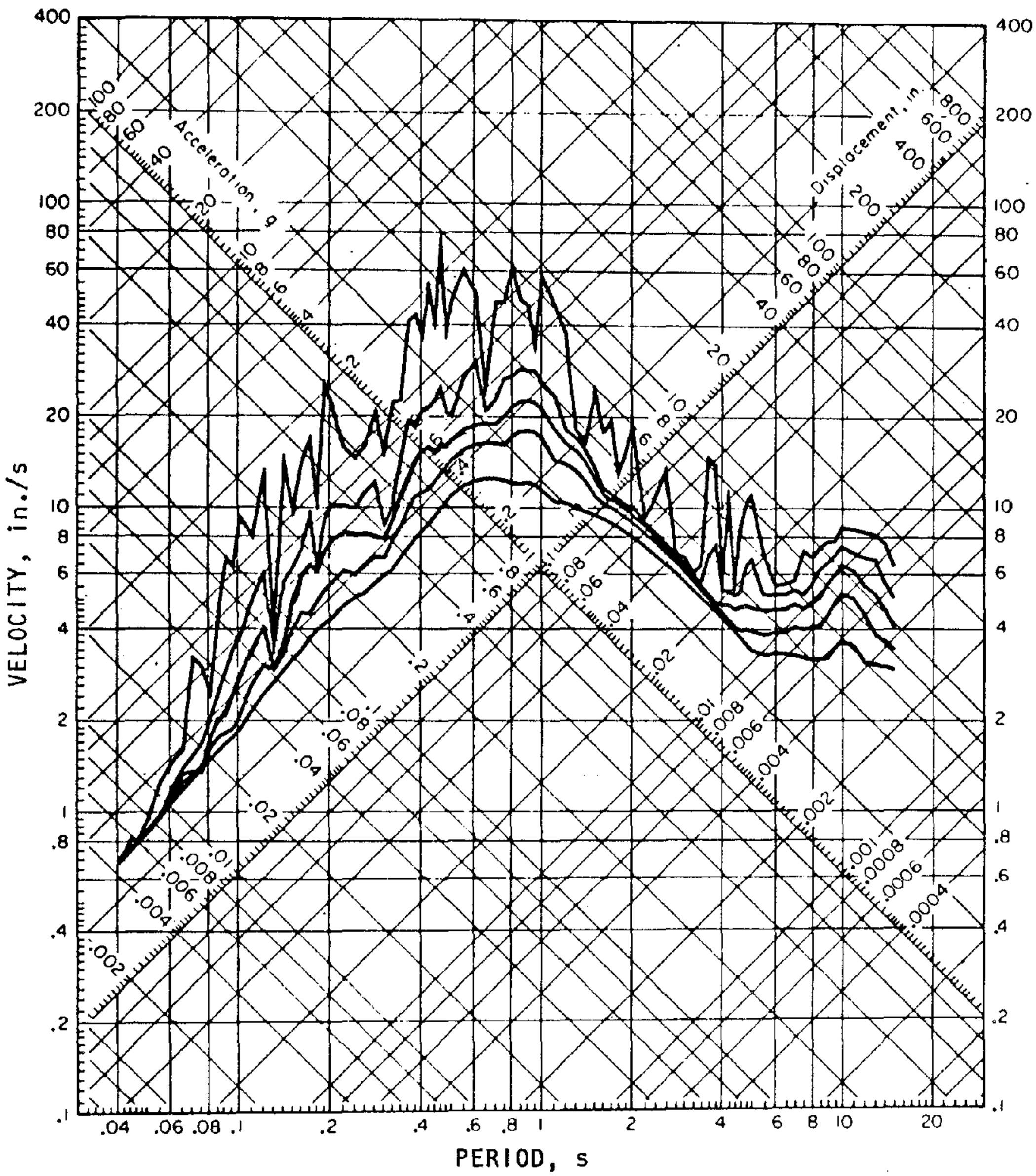
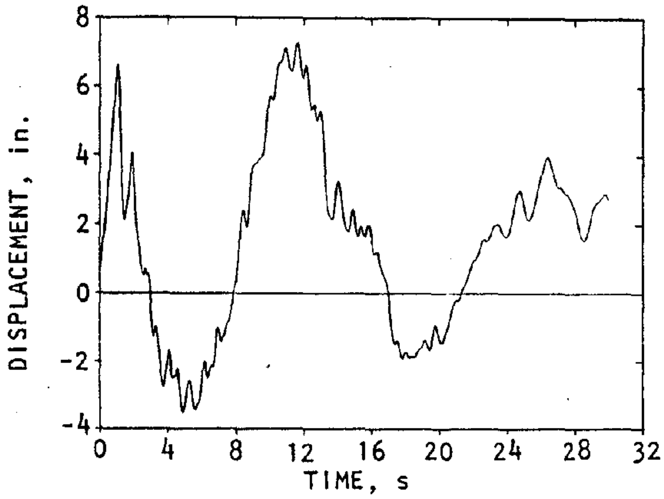
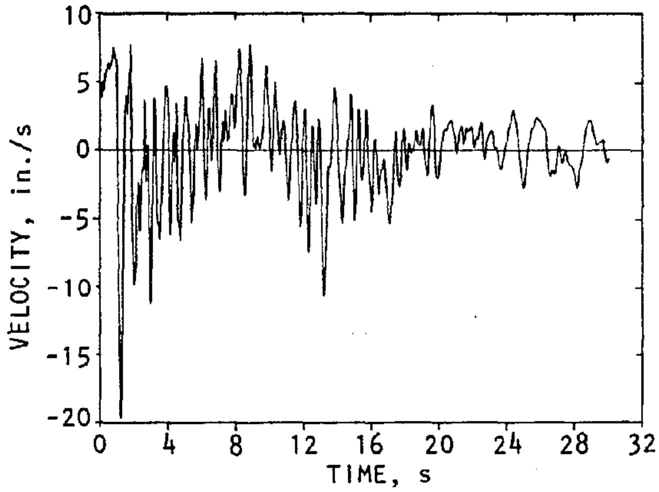


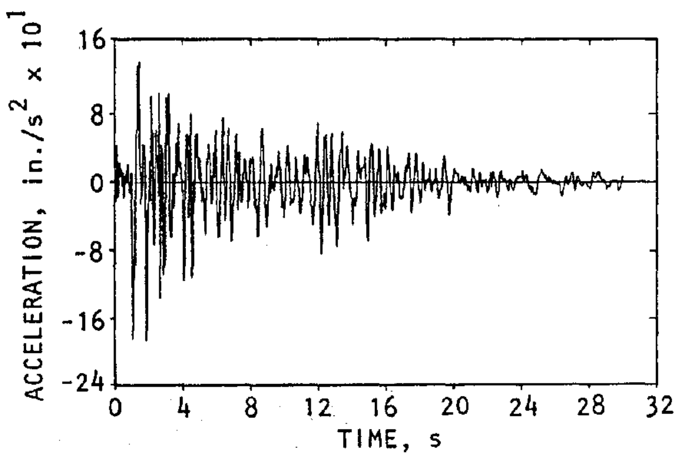
FIGURE 7. RESPONSE SPECTRUM OF SAN FERNANDO EARTHQUAKE, 9 FEBRUARY 1971: CASTAIC OLD RIDGE ROUTE, COMP N69W (Damping values are 0, 2, 5, 10, and 20 percent of critical–unscaled)



(a) Input displacement

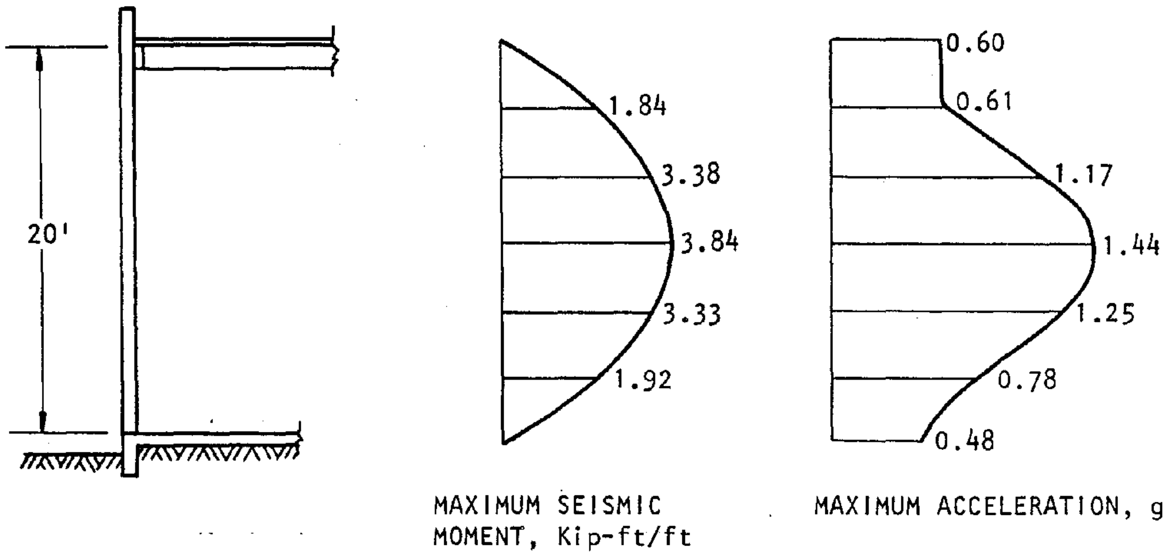


(b) Input velocity



(c) Input acceleration

FIGURE 8. EARTHQUAKE INPUT MOTION, CASTAIC SCALED BY 1.80 (1 in. = 2.54 cm)



(a) TUW section

(b) Absolute maximum moment at midspan TUW

(c) Absolute maximum acceleration at midspan TUW

FIGURE 9. MAXIMUM MOMENTS AND ACCELERATIONS IN THE MIDSPAN TUW

TABLE 5. COMPARISON OF RESULTS WITH THOSE OBTAINED BY STATICALLY EQUIVALENT METHOD* AT MIDHEIGHT OF MIDSPAN PANEL

Dynamic Seismic Moment, kip-ft/ft	Ratio of $\frac{\text{Dynamic Seismic Moment}}{\text{Static Seismic Moment}}$	Total Dynamic Moment, kip-ft/ft	Ratio of $\frac{\text{Total Dynamic Moment}}{\text{Total Static Moment}}$	Ratio of $\frac{\text{Total Dynamic Moment}}{\text{Design Moment Strength}}$
3.84	2.06	4.28	1.85	1.51

*Wyatt (1980)

Definition of Terms:

Dynamic Seismic Moment: Maximum bending moment (absolute value) computed by STARS code

Static Seismic Moment: Portion of moment due to statically equivalent inertia load as computed by statically equivalent method

Total Static Moment: Static seismic moment plus static moments due to eccentric loading and P-δ effect

Total Dynamic Moment: Dynamic seismic moment plus static moments due to eccentric loading and P-δ effect

Design Moment Strength: Ultimate moment the TUW section can carry

The results of the analysis indicate that the connections at the panel-to-roof diaphragm level can be subjected to forces higher than those obtained from equivalent static design methods. Also, the dynamic seismic moments at the mid-height of the panel can be considerably higher than the design moments calculated by the current design methods. The analysis also indicates that the roof diaphragm behaves like a linear shear beam for the middle sections. However, the portions of the diaphragm adjacent to the end walls have a pronounced nonlinear, hysteretic shear beam response.

TUW WALL PANEL TEST DESCRIPTION

Experimental Philosophy

There are several structural response considerations that must be addressed in the development of a methodology for the mitigation of seismic hazards in TUW buildings. The actual response of these structures is nonlinear and involves interaction among many of the structural elements, such as the end walls, diaphragm, side walls, and wall/diaphragm anchorage. The most ideal method of hazard assessment would combine nonlinear dynamic analyses of complete structures and the dynamic, full-scale testing of the same structures, where the testing would be used to verify and/or calibrate the analyses. However, facilities are not currently available in the United States for the testing of complete buildings at full scale. Moreover, if such facilties did exist they would be very expensive when parametric variations are required.

The experimental philosophy used in this study is based on the development of full-scale, dynamic, component tests for TUW panels that use kinematic motions obtained from analyses that account for the interaction among the structural elements. Although the accountability cannot be perfect, acceptable levels can be obtained.

Component Test Design

A substantial amount of the testing of reinforced concrete has been directed toward in-plane loadings. In TUW construction, the most important response is in the out-of-plane direction. Accordingly, the component testing of TUW panels subjected to out-of-plane motions has been identified as essential in the development of guidelines for seismic hazard mitigation.

To provide a reasonable simulation of the in-situ condition, the interaction among the structural elements of the building should be included. As stated in the experimental philosophy, this interaction will be included in the component tests by using kinematic motions obtained from an analytical model that includes the structural system interaction.

For the component tests on TUW panels, the input to the base and top of the specimens will be taken from a series of analyses using a model similar to the one described earlier. For a typical ground level element of a one-story building, the kinematic input to the component test will be a ground motion at the base and a compatible roof diaphragm motion from the analyses at the top, where variations in the diaphragm stiffness characteristics will be included.

Brief Description of Specimens

A study of existing TUW buildings in the United States (Adham, 1981) was conducted to identify and categorize the current class of buildings and to identify their materials and methods of construction. The specimens to be tested will be representative of typical TUW elements found in the study.

The TUW specimens will be 4 in. (102 mm) thick, 3 ft 6 in. (1.07 m) wide, and 12 ft to 20 ft (3.7 m to 6.1 m) high, and will be fabricated using typical materials. The wall panel parameters are given in Table 6. A total of 10 specimens will be fabricated at H/t ratios ranging between 36 and 60. The H/t ratio of 36 has been established by the SEAOSC Yellow Book (SEAOSC, 1979), as the standard to be used, while the ratio of 60 represents the upper limit at the present time.

At least two different reinforcing steel ratios will be tested. These ratios are representative of the current practice in TUW construction. Three number 3 bars and five number 3 bars will provide ratios of 0.2% and 0.33%, respectively.

Tests by ACI-SEAOSC Slender Wall Committee (ACI-SEAOSC, 1980) have indicated that changing the ledger weight did not cause significant changes in the static response of the walls. Therefore, one representative level of 500 lb/ft overburden mass is used to simulate ledger load.

Since the experimental program is limited to ten wall specimens, a sequential design process will be adopted for the test specimens; where the specimens will be designed, fabricated, tested, and analyzed in groups. The first, second, and third groups will consist of two specimens each, while the fourth group will consist of four specimens. The use of a sequential design process should maximize the usefulness of the available specimens. The first group will consist of the highest H/t ratio of 60 with a maximum and minimum of reinforcing steel. These specimens represent the most likely condition for failure and will help guide the design of the subsequent specimens.

TABLE 6. WALL PANEL PARAMETERS

Parameter	Value
Unit weight of concrete (w), pfc	150
Compressive strength of concrete (f'_c), psi	$3000 \leq f'_c \leq 4000$
Grade of reinforcing steel	60
Capacity reduction factor (ϕ)	1.0
Panel thickness (t), in.	4
Reinforcement ratios (ρ), percent	0.20, 0.33
Earthquake ground motion input	Based on ATC Effective Peak Acceleration of 0.1 through 0.4
End eccentricities (e), in.	$t/2 + 3\frac{1}{2}$
Ledger load	500 lb/ft
Height to thickness ratios, H/t	36 through 60

Test Set-Up and Instrumentation

The TUW panels will be installed in a test fixture that allows the base and top of the wall to be moved independently in the out-of-plane direction by servocontrolled hydraulic actuators. The tests will include both out-of-plane, quasi-static and dynamic testing. The test setups for the dynamic, out-of-plane shaking and the quasi-static, out-of-plane displacement of the TUW panels are shown schematically in Figures 10 and 11. The wall specimens will be mounted on a base fixture with an attachment lug for the hydraulic actuator. When the specimen is installed in the test apparatus, the base fixture will rest on a low friction support (shown as rollered in the figure) that will allow the base of the wall to be displaced by the hydraulic actuator. The base fixture will be designed to provide a realistic boundary condition at the base. A mechanical header/ledger with an attachment lug for the hydraulic actuator will be attached to the top of the wall specimen and will allow the top of the specimen to be displaced, but will not restrict rotation of the top of the specimen. The vertical overburden loads will be applied to the ledger through attachment rods that will maintain a precise relationship between the vertical load and the center of the specimen.

The basic instrumentation for the measurement of the dynamic and static responses and forcing functions, consist of load cells, velocity transducers, and displacement sensors (string potentiometers) as shown in Figure 12. Except for the velocity transducer at the top of the specimen (V1A), all instruments will measure out-of-plane responses and forcing functions. The out-of-plane response of the specimen is expected to induce vertical in-plane motions that can be measured by the velocity transducer V1A. The velocity and displacement sensors will be mounted to a stable reference frame that is independent of the frame for the forcing system. This type of instrument mounting will eliminate the need to account for the flexibility of the actuator reaction structure. Previous testing (ABK, 1981b) showed that the use of accelerometers to monitor the dynamic response of unreinforced masonry wall panels proved to be unsatisfactory. These instruments can be very sensitive to noise induced by the test apparatus and can result in acceleration data that is highly spiked and contaminated. However, the effectiveness of the velocity transducers will be evaluated before the instrumentation is finalized.

The data from each instrument will be recorded on magnetic tape in digital form for subsequent processing and archiving. Additional data recording will be obtained in the form of still photographs, motion pictures, and observer notes or test logs.

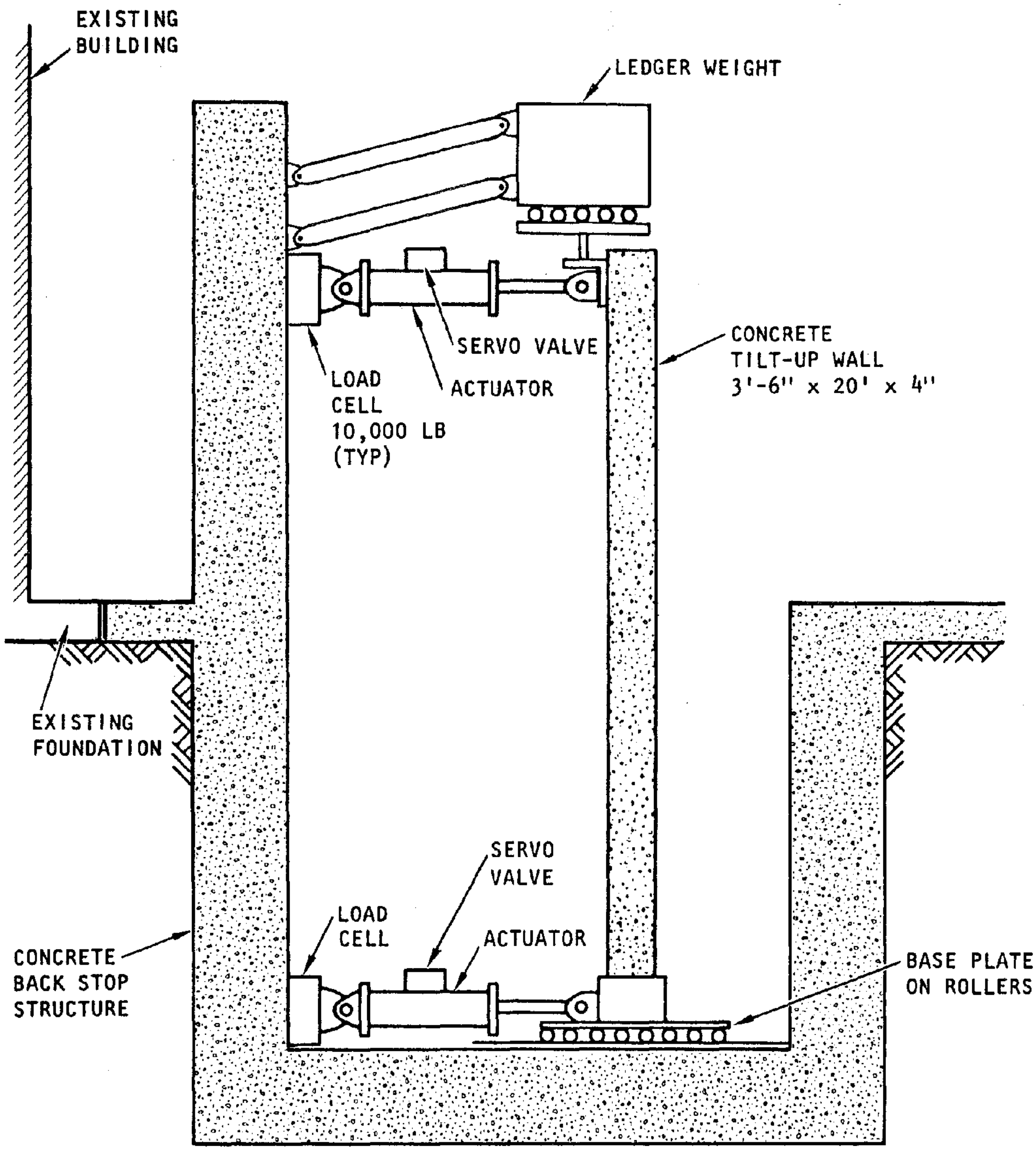


FIGURE 10. TEST SETUP FOR DYNAMIC TESTING OF WALLS

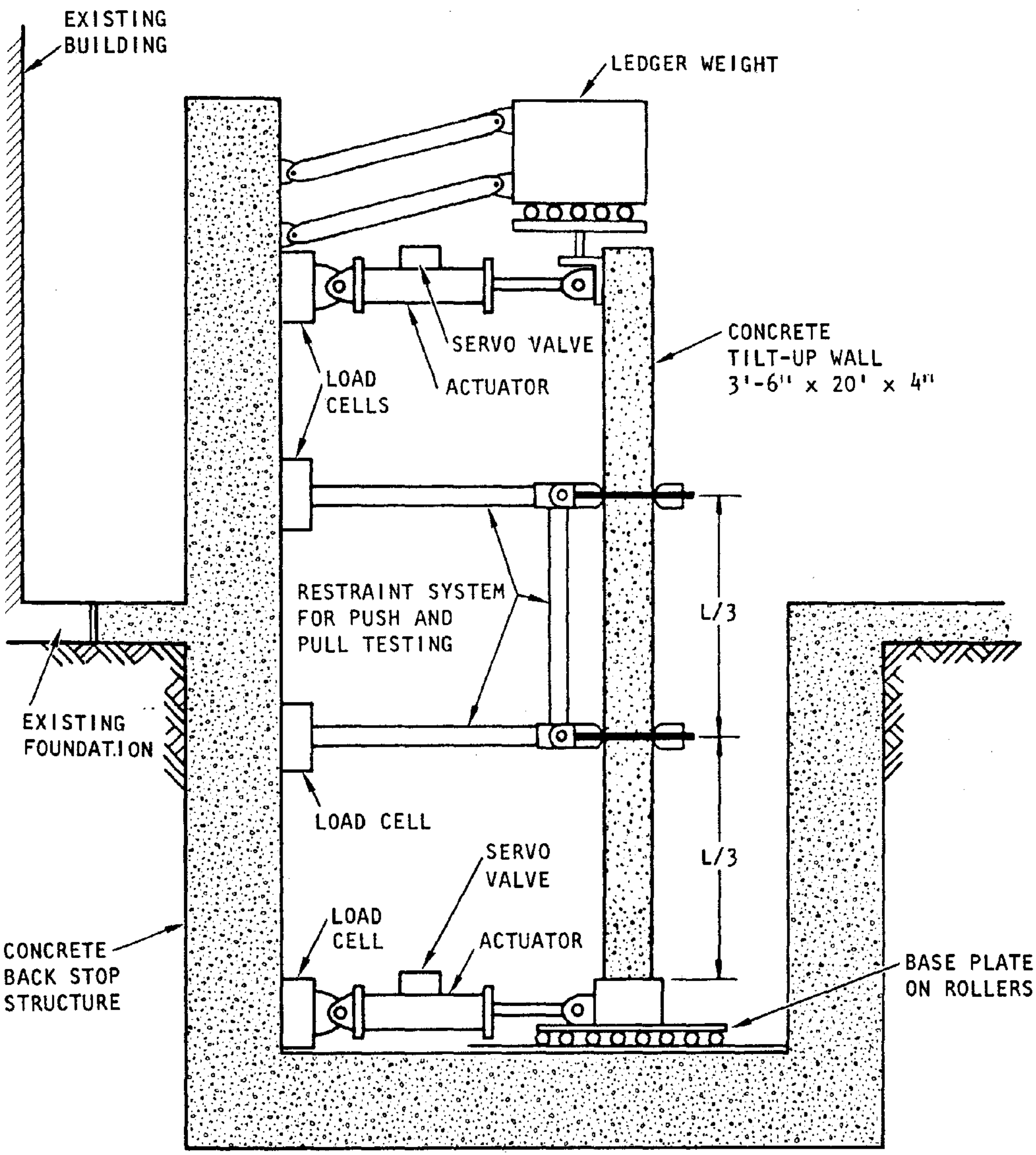


FIGURE 11. TEST SETUP FOR QUASI-STATIC TESTING OF WALLS IN THE DYNAMIC TEST FACILITY

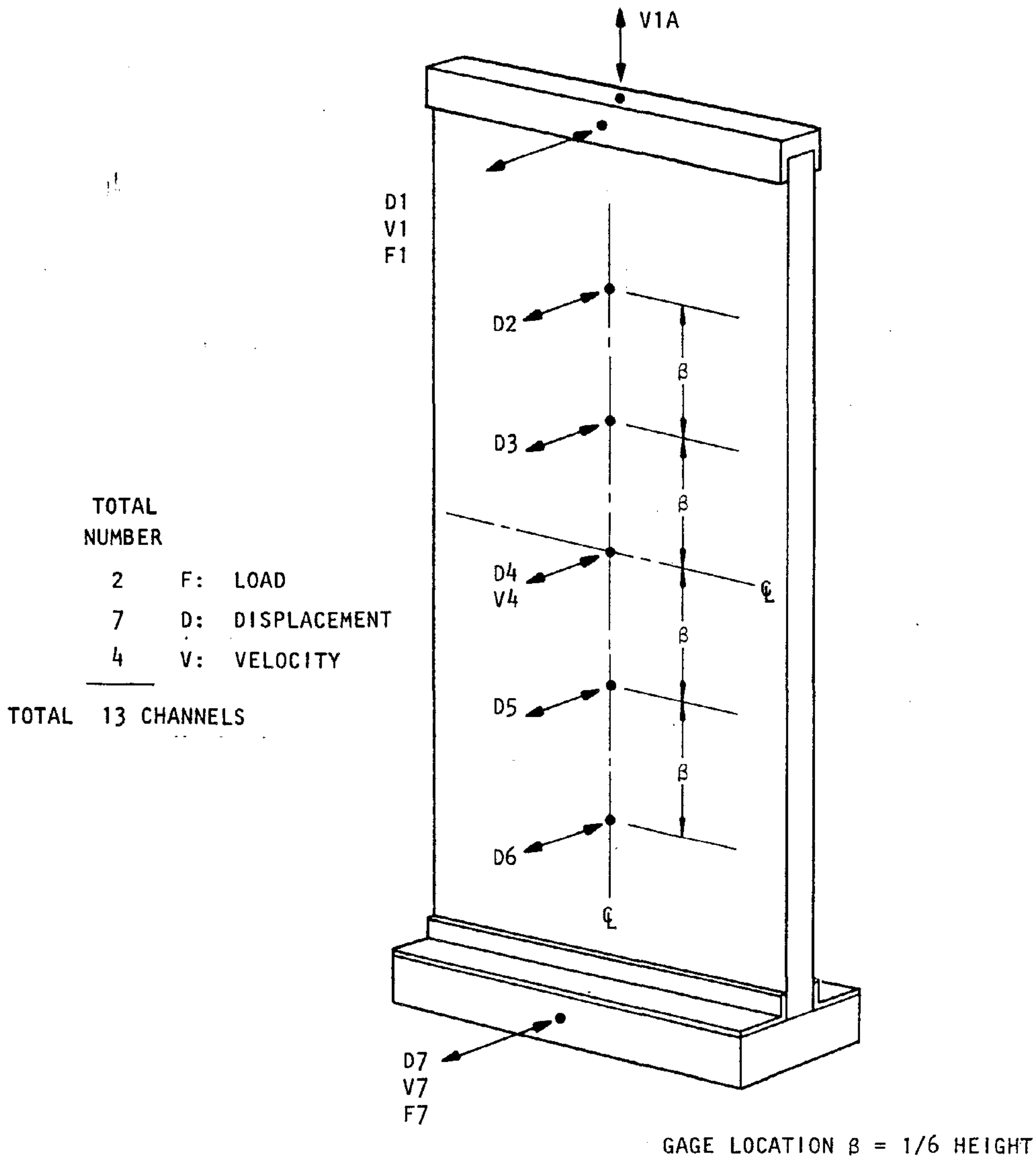


FIGURE 12. WALL INSTRUMENTATION

Test Modes and Sequences

Each wall panel will be subjected to a sequence of dynamic input motion pairs, or motion sets (MS), that consist of a compatible pair of kinematic motions, one for the base and one for the top of the wall. The kinematic motions will be obtained from the analysis model described previously and will be based on actual earthquake ground motions records that represent different magnitude and frequency content. The ground motions will be scaled so as to cover the range of seismicity from an Effective Peak Acceleration (EPA) of 0.20 g to 0.40 g for moderate to highly seismic areas (ATC, 1978). These ground motions will be used as input to the nonlinear, dynamic analysis model of typical TUW buildings that accounts for the nonlinear response of the diaphragm, including both stiff and flexible diaphragms and the dynamic inertial effect of the tilt-up walls. The motion of the diaphragm obtained from the analytical model will be used to simulate the kinematic input at the top of the wall panel, while the earthquake ground motion will be used to simulate the kinematic input at the base of the wall panel. All of the kinematic motions to be used in the test program will assume nonyielding anchorages between the tilt-up wall and the roof diaphragms.

Tha actual testing sequence has not been finalized. In general, the dynamic tests will be preceded by very low-level cyclic, quasi-static tests (motions that produce less than cracking moments) to provide elastic characteristics of the specimens. The dynamic testing will start with motion sets of the lowest intensity and progressively proceed to motion sets with higher intensity levels of motion until the specimen collapses or sustains excessive failure modes. Depending on the condition of the specimens after dynamic testing, some cyclic, quasi-static tests will be performed to obtain the nonlinear, hysteretic characteristics of the panels.

Displacement Controlled Hydraulic Actuation System

The kinematic input to the base and top of the wall specimens will be delivered by a high-pressure hydraulic actuation system that is controlled by displacement. This method of control provides the most reliable system for command and measurement. A schematic of the displacement controlled hydraulic actuation system is shown in Figure 13. The earthquake ground motion and diaphragm/TUW wall system response records will be obtained in digitized form and written on tape. This tape will be introduced into a command and control computer, which is programmed to selectively convert the digital input data into anlog form. Analog position commands will be transmitted to a multichannel, servocontrol amplifier system that will send control signals to

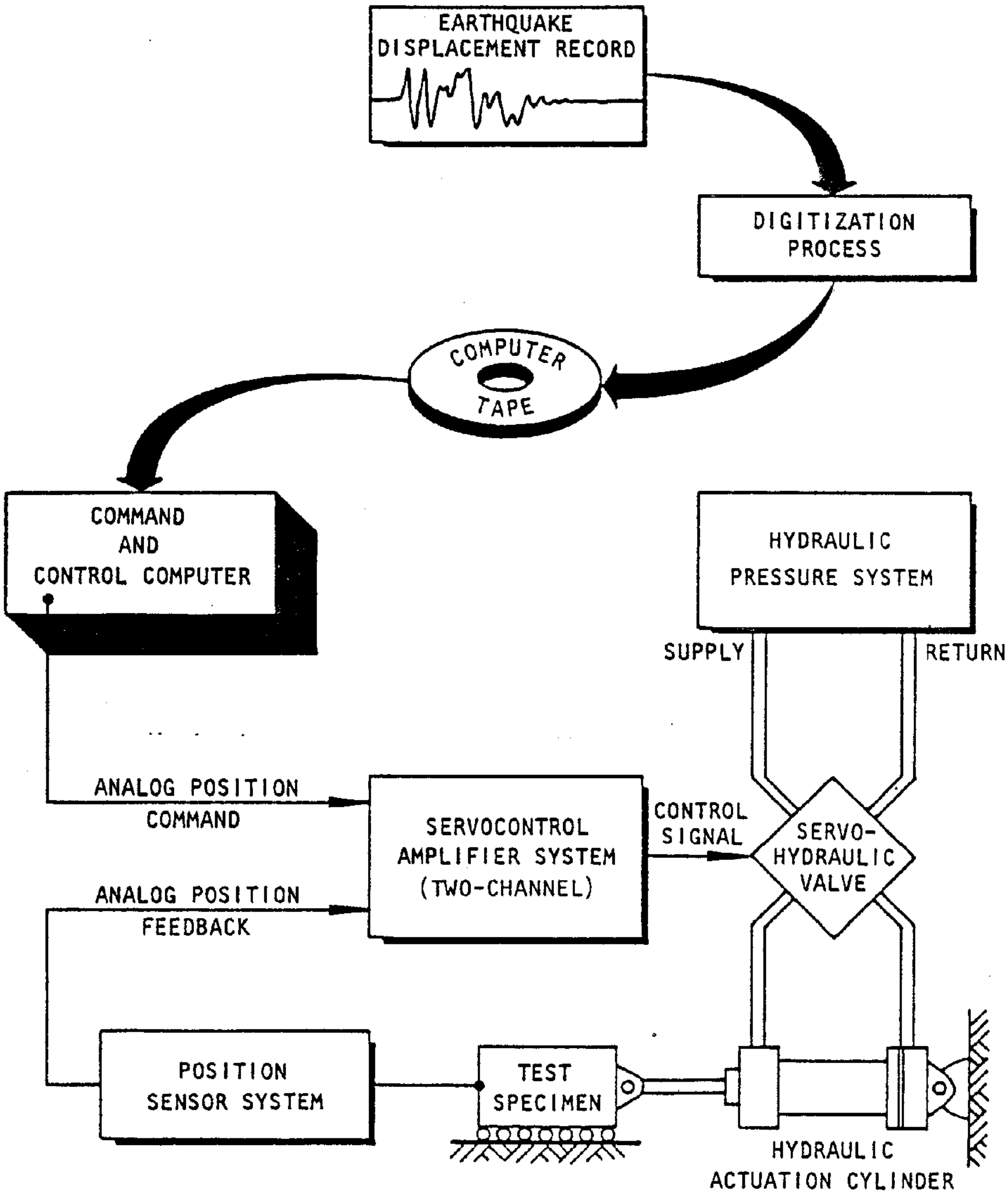


FIGURE 13. DISPLACEMENT-CONTROLLED HYDRAULIC ACTUATION SYSTEM

the servohydraulic valves mounted on the hydraulic actuation cylinders. Hydraulic pressure will then be delivered to the actuators which will drive the base and top of the test specimen. String potentiometer position sensors will monitor the attained displacements, which will be returned to the servocontrol amplifier system and compared with the command displacement. Differences between the feedback and command signals will be monitored and corrections in the command signals, if required, can be made to maintain a maximum permissible error of 10%.

FURTHER STUDIES AND CONCLUSIONS

The preliminary analyses conducted so far clearly indicate that typical TUW buildings can be quite adequately analyzed using lumped parameter models. The analyses show that the TUW building response is highly nonlinear for moderate to highly seismic areas and elastic analyses are not valid for these regions. Moreover, the analysis model can account for the interaction among the structural components and can be used to define the kinematic environment for the component testing of full-scale TUW panels subjected to out-of-plane motions.

In the continuing analysis phase, the model will be used to assess the effect of variations in diaphragm systems, TUW panel H/t ratios, and building aspect ratios on the TUW panel response and anchorage requirements. In addition, the model will be extended to include nonlinear TUW panels elements and the P-Δ effects.

During the forthcoming test program, the response of TUW panels will be studied and failure criteria will be established and related to performance criteria. In addition, the analytical model will be refined, calibrated, and verified for use in predicting the dynamic response of TUW buildings. The final goal will be a reliable set of guidelines for the mitigation of seismic hazards in TUW buildings.

ACKNOWLEDGMENTS

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RETROFITTED STRUCTURES AGAINST EARTHQUAKE AND RESEARCH ON THEM

- Activity of Japan Concrete Institute -

Toneo Endo*

INTRODUCTION

Several reinforced concrete buildings, damaged by the Tokachi-oki Earthquake, 1968, were repaired and strengthened.[1] Thereafter some of existing reinforced concrete buildings in Japan were estimated as not secure against severe earthquake. In order to strengthened existing buildings, experimental studies on strengthening of columns,[2),3)] of wing walls[3)] and of infilled walls[4)5)] were published in Japan, while a study by Kahn[6)] on infilled walles published in U.S.A.

Based on the above-mentioned experimental studies,"Criterion on the evaluation of seismic safety of existing reinforced concrete buildings"[7)] and "Design Guidelines for aseismic retrofitting of existing reinforced concrete buildings"[8)] were published in 1977. Many reinforced concrete buildings were strengthened refering to the above criterion and guidelines. The Miyagiken-oki Earthquake, 1978, revision of Enforcement order by building standard law in 1981, and the prediction of Tokai Earthquake pushed forward the strengthening of building. In addition to the buildings, many bridges were repaired and retrofitting due to the damage by the Miyagiken-oki Earthquake.

Recently a number of strengthend members were tested while several special problems, e.g. effect of wedge anchors,[9)] were discussed.

"Seismic Strengthening Subcommittee" was organized under "Research Committee" in Japan Concrete Institute. The purpose of the subcommittee is to make public the techniques of retrofitting structures, after collecting data on retrofitted structures in practice and also collecting reports of experimental and analytical studies about retrofitting.

*Associate Prof. of Tokyo Metropolitan Univ.

ORGANIZATION OF THE SUBCOMMITTEE

"Seismic Strengthening Subcommittee" started September 1981. Engineers and researchers working in charge of retrofitting structures in various fields became members.

The present members are listed as follows.

Toneo Endo	Chairman	
Akira Okifuji	Secretary	Taisei Corporation
Shunsuke Sugano	Secretary	Takenaka Koumuten Co.
Koji Ishii		Kajima Corporation
Yuzo Yoneyama		Tokyo Metropolitan Governmental Office
Kazuyoshi Ohshima		Ministry of Construction
Tadayoshi Ishibashi		Japan National Railway
Hiroshi Imai		Tsukuba University
Yasuyuki Kamiya		Shizuoka Prefecture Governmental Office
Shigenao Sato		Metropolitan Expressway Public Corporation
Noboru Sakaguchi		Shimizu Construction Co.
Yukio Sawabe		Nippon Telegraph and Telephone Public Corporation
Yasushi Shimizu		Tokyo Metropolitan Univ.
Toshimasa Tada		Ohbayashi-Gumi Co.
Kaname Takahara		Oka Architects and Engineers Office
Shinsuke Nakata		Building Research Institute
Takaya Nakamura		Yokohama Municipal Office
Toshio Hayashi		Taisei Corporation
Masaru Fujimura		Takenaka Komuten Co.
Hideaki Yamaoka		Japan National Railway

Examples

At present 115 examples are collected by the subcommittee. Some remarkable points about items of the data are described as follows.

(1) Many school buildings and governmental offices were retrofitted for there importance. Number of these types of retrofitted buildings exceeds 30% of the retrofittes structure.

(2) More than 40% of the retrofitted buildings were completed between 1966 to 1970.

(3) About 80% of the buildings were strengthened by means of additional shear wall. Besides the harf of the 80% were retrofitted by other method simultaneously .

Most buildings of the data were retrofitted reffering to the above-mentioned "Criterion and Guide-lines". therefore Is indeces were calculated in order to know seismic performance of the buildings. In case of the most effectively retrofitted building, Is index changed from 0.28 before retrofitting to 1.43 after retrofitting by means of infilled walls, wing walls and shear reinforcement of columns together.

Reserches

34 research papers were discussed by the subcommittee. Especially papers on anchors between old and new concretes were discussed in detail, because both building engineers and bridge engineers were interested in them. The subcommittee is trying to make design proposal about anchors.

Panel Discussion

The panel discussion on "Seismic strengthening of existing reinforced concrete structure" is scheduled January 1983. All data collected by the subcommittee and their analysis will be published there.

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7) "Criterion on the Evaluation of Seismic Safety of Existing Reinforced Concrete Buildings" The Japan Building Disaster Prevention Association, March, 1977, Presented at 2nd Joint Meeting.

8) "Design Guidelines for Aseismic Retrofitting of Existing Reinforced Concrete Buildings", The Japan Building Disaster Prevention Association, March, 1977, Presented at 2nd Joint Meeting.

9) The Shear strength test on connection between reinforced concrete beam and wall by wegde anchor.

PRELIMINARY REPORT
on
EXPERIMENTAL TESTING OF EPOXY REPAIRED CONCRETE BEAMS

Prepared for

3rd US/JAPAN SEMINAR ON REPAIR AND RETROFIT OF STRUCTURES
in San Francisco on May 13-15, 1982

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1. INTRODUCTION

This paper presents a brief description of an experimental research program concerned with the strength and behavior of epoxy repaired concrete beams subjected to elevated uniform temperatures and standard fire exposures. This research project is to be completed in March, 1983, hence, the test results presented herein are not complete. Future reports and articles will contain a complete summary of the research program and the test results.

2. OBJECTIVES OF THE RESEARCH PROGRAM

This research program has been designed to investigate the strength and behavior of epoxy repaired concrete beams. Several unique objectives of the research program are described below along with a discussion on the significance of this research program.

2.1 OBJECTIVE ONE: EPOXY BURNOUT AND SURFACE COATINGS

Fire research studies on cracked and subsequently epoxy repaired shear walls subjected to fire exposure showed that epoxy adhesives vaporized or burned out to a depth of 3 to 4 inches inside the crack as indicated in Figs. 1 and 2. To prevent such burnout, several fire protective surface coatings were applied and 1 inch thick plaster proved to be most effective. In OBJECTIVE ONE, the depth of epoxy burnout and the effectiveness of surface coatings have similarly been investigated for the epoxy repaired concrete beams. Surface coatings that were considered included vermiculite, several thicknesses of gypsum plaster, and intumescent coatings.

2.2 OBJECTIVE TWO: EPOXY RE-BONDING OF STEEL REINFORCEMENT TO CONCRETE

Inspections have been made of about 10 buildings where concrete beams were damaged by the 1971 San Fernando Earthquake and subsequently repaired with epoxy adhesives. From these inspections, the most common type of beam cracks repaired with epoxy adhesives were the shear-flexural cracks near the beam-column joint and the pure flexural cracks away from any type of joints (Ref. 1). Several beam-column joints were also injected with epoxy adhesives but are not considered herein.

In OBJECTIVE TWO, an attempt has been made to determine (1) how and if reinforcement away from the beam-column joints can be effectively re-bonded to the concrete, and (2) what are the effects of temperature on such a re-bonding procedure. Four commonly used epoxy adhesives and a new low-viscosity extremely long pot life epoxy adhesive have been considered in the investigations dealing with this objective.

2.3 OBJECTIVE THREE: STUDY OF SMALL AND INTERMEDIATE SCALE-EPOXY REPAIRED CONCRETE BEAMS

The previous two objectives investigated separately the extent of epoxy burnout under fire exposure and the feasibility of re-bonding steel reinforcement to concrete. The experiments in OBJECTIVE THREE will attempt to investigate collectively these same two parameters through the testing of beams at uniform temperatures and fire exposure with beams scaled to dimensions which can be tested in CSULB furnaces and ovens.

Thermal gradients in most beams are a two-dimensional problem. Hence, the depth and thickness, but not the length, of the experimental beam specimens must be representative of the actual epoxy repaired beams. Figs. 3 and 4 provide the geometry and span lengths of both the small and intermediate-scale beam specimens. The results from these beam tests were also used to design geometry and test procedure for the full-scale E-119 beam tests proposed in the next sub-section.

2.4 OBJECTIVE FOUR: FULL-SCALE E-119 FIRE EXPOSURE TESTS ON CONCRETE BEAMS

The ASTM E-119 specification provides the standard test procedure for fire testing of structural components. Full-scale E-119 tests are extremely expensive; hence, only two full-scale fire exposure tests on beams have been performed in this experimental program. In order to obtain the most information possible from the full-scale E-119 tests, a group of eleven intermediate size fire exposure tests on epoxy repaired beams have been performed prior to the full-scale tests as noted in OBJECTIVE THREE.

3. SIGNIFICANCE OF THIS RESEARCH PROGRAM

The first and foremost reason for research in the area of epoxy repaired structures is the need for public safety. Adhesive Engineering Company of San Carlos, California, alone has provided epoxy materials for repair of about 1500 building structures throughout the world and nearly 1000 in California, and additional epoxy repairs are planned or already in progress. Furthermore, structural damage from future earthquakes will probably require additional epoxy repair. Although the possibility of a fire in an epoxy repaired structure is not insignificant, the fire rating of epoxy repaired structures is of greater significance. Each year, 12,000 lives are lost in the U.S.A. due to fire. The present lack of knowledge concerning the behavior of epoxy repaired structural components such as beams during fire does present a danger to public safety.

4. REASONS FOR REPAIR OF DAMAGED STRUCTURES

Regardless of the cause of crack formations, once a crack is formed, the ability of the cracked structural element to withstand future seismic or other types of loads is necessarily reduced. As noted by N. M. Newmark in Ref. 3, a structure may often be damaged but nevertheless survive the initial severe earthquake, yet collapse under relatively mild after-shocks. Therefore, structures with extensive crack formations must either be demolished or repaired as soon as possible. If restoration is feasible, the cracked structural elements should be repaired due to the following reasons:

1. Loss of strength due to inability of the cracked elements to resist and transmit the design loads.

2. Water combined with freeze-thaw cycling results in spalling.

3. Exposure of reinforcing steel in the area of cracks results in corrosion.

4. Leakage of fluids through crack formation.

5. Visible cracks are unsightly and give a sense of insecurity to the public.

5. BRIEF REVIEW OF EPOXY REPAIR PROCESS AND EXISTING FIRE CODES

Both the 1964 Anchorage Earthquake and the 1971 San Fernando Earthquake created many damaged structures with extensive crack formations. As a result, many companies were organized to repair concrete or masonry structures with various adhesives. As the repair process progressed, the epoxy type adhesives proved to be the most effective. However, little or no experimental information was available as to the behavior of epoxy repaired structures. Many companies were also formulating proprietary epoxy adhesives of questionable value, thus creating more unknown factors for the building officials involved in granting repair permits.

In retrospect, many permits were apparently granted on the basis of the strength properties of the proposed epoxy adhesives and the assurance of full penetration by means of standard core samples. The City of Los Angeles developed a "Procedure for Processing Requests for the Repair of Earthquake Damaged Buildings by Using Epoxy". Apparently realizing that fire exposure in epoxy repaired structures may reduce the fire ratings and thus create additional public safety hazards, this document stated that a 3/4 inch plaster fire protection shall be provided for any members in which the exposed surface of epoxy used for repair exceeds 1/4 inch width. However, the overwhelming majority of epoxy repaired cracks are <u>less</u> than 1/4 inch wide, especially in concrete beams. With the exception of this plaster protection as recommended by the City of Los Angeles, no other specification exists on the need for fire protection for epoxy repaired concrete components. A final draft of ACI Committee 502 discusses some aspects of fire exposure on epoxy repaired concrete.

6. DESCRIPTION OF ASTM AND SDHI FIRE EXPOSURES

The epoxy repaired concrete beam specimens described in this report were subjected to "pseudo-fire exposures" designed to simulate two different types of building fires. The two-hour duration ASTM E119 fire exposure attempts to model a long duration fire with constantly increasing temperature, so that the cool down behavior is not represented. A short duration high intensity (SDHI) fire which peaks at about 0.2 hours, has a rapid temperature drop for a period of 0.4 hours and is followed by a slow cooling to room temperature. This SDHI time-temperature curve has been proposed by many including Professor Bresler of U. C. Berkeley. Both the ASTM and the SDHI time-temperature curves are provided in Fig. 5. As indicated by the results in subsequent sections, the ASTM E119 type fire exposure is far more severe than the SDHI type on the fire rating of epoxy repaired structures.

7.0 FIRE SURFACE COATINGS

Relatively low ultimate strength results for epoxy repaired wall and beam specimens subjected to ASTM E119 fire exposure prompted a search for effective fire surface coatings which would decrease the depth of epoxy burnout and increase both the hot and residual strengths. Therefore, a series of surface coatings were applied to the fire surface for the purpose of investigating fire protection. These surface coatings were grouped into three categories including (1) gypsum plaster, (2) thin inorganic surface coatings, and (3) thin organic surface coatings.

Gypsum plaster was mixed and applied to the fire exposed surfaces according to the appropriate specifications in the 1976 UBC. Total plaster thickness of 1 in. (7/8 in. thick base coat with sand aggregates and 1/8 in. thick finish coat) or 1/2 in. (3/8 in. thick base coat and 1/8 in. thick finish coat) were applied to the fire exposed surfaces of the intermediate and full scale beams. The plaster was allowed to cure for at least 60 days prior to fire testing under laboratory conditions. Moisture content in the plaster at the time of fire testing was approximately 1.6%.

Thin inorganic surface coatings were also applied to the fire exposed surface in thicknesses of 0.050 in. and 0.100 in. These inorganic coatings consisted of a one to one mixture on volume basis of sodium silicate and Type I Portland Cement. This inorganic coating was applied to the beam fire surface with a trowel and cured a minimum of seven days prior to fire exposure. The fire test results showed that this type of thin inorganic surface coatings are totally ineffective. Thin organic surface coatings were also applied to the fire surfaces in the form of "fire resistant epoxy foams" and fire retardant intumescent paints. The thickness of these coatings included 0.050 in. and 0.100 in. and were applied to the fire surface with a trowel. These inorganic surface coatings were cured for a minimum of seven days prior to fire testing. The test results for these organic surface coatings were also not favorable.

8. DESCRIPTION OF MATERIALS AND SPECIMEN PREPARATION

8.1 EPOXY ADHESIVES AND STRUCTURAL EPOXY ADHESIVES

Epoxy adhesives represent a wide range of chemical polymers which, if mixed with various additives, result in materials with extremely diverse physical properties. Although discovered about 1907, epoxy resins did not become commercially significant until immediately after World War II when their usage was expanded into many industries including aircraft, electrical, building, medical, dental, et cetera. The use of epoxy adhesives for structural repair of cracks was apparently first attempted in California in 1954.

Most epoxy adhesives are thermosetting resins derived from the oil refining intermediate products; epichlorohydrin and bisphenol A. The ratio of these two products and the addition of other chemicals and hardeners largely affect the resulting physical properties of the epoxy adhesives. The addition of fillers such as lime, sand, cements, etc., also result in additional changes in physical properties. Literally millions of filled or unfilled proprietary epoxy adhesives are presently on the market. However, the number and type of unfilled epoxy adhesives used for injection into cracked structures is rather limited.

Five different structural epoxy adhesives were considered in this research program on epoxy repaired beams. Fillers, such as lime, cement or sand were not added to the epoxy adhesives either before or during the injection. These adhesives were chosen because they were representative of most low viscosity epoxy adhesives that have been or are being used for the repair of damaged concrete beam and wall structural components in California. These low viscosity epoxy adhesives were obtained from four manufacturers including: Delta Plastics Company of Visalia, California; Hunt Process Company of Santa Fe Springs, California; IPA Systems of Philadelphia; and Adhesive Engineering of San Carlos, California. The range of mechanical properties at room temperature for low viscosity epoxy adhesives supplied by these four manufacturers are provided in Table 1. Note that the physical properties of the extremely long pot life epoxy have been provided separately. Most of the above data has been provided by the manufacturers. Pot life is defined as the time available for use after mixing the hardener and epoxy resin but before the adhesives begin to gel. Since the heat distortion and strength transition temperature were similar for all five low viscosity exposies, slight variation in the strength properties of epoxy adhesives at room temperature did not affect fire test results.

8.2 SPECIMEN PREPARATION

Three unique specimen sizes were utilized in this beam testing program including small, intermediate and full scale beams. About 100 small-scale specimens were constructed according to the dimensions and steel placement as shown in Fig. 3. These small-scale specimens were used primarily in the study of epoxy repaired beams at uniform elevated temperatures in the range of 75^{o}F to 400^{o}F. All small-scale specimens were of rectangular cross-section with tensile reinforcement consisting of one #5 bar, Grade 60 steel. Eleven intermediate size specimens were fabricated as shown in Fig. 4. Both rectangular and T cross-sectional shapes were considered. Except for the length and percentage of reinforcement of the intermediate size specimens, the cross-sectional dimensions are representative of typical beams damaged and repaired with epoxy adhesives after the 1971 San Fernando earthquake. These intermediate-scale specimens were designed to investigate the thermal gradients and the effects of various types of fire protection coatings on the epoxy repaired concrete beams. As for the full-scale specimens, the intermediate size specimens were subjected to standard fire exposure time-temperature curves as described earlier. The experimental results obtained for the intermediate size specimens were also used to design the steel reinforcement and testing procedure for the full-scale specimens. Four of the eleven intermediate size specimens were tested at the Ohio State University, Fire Research Lab, and the rest at CSULB. The full-scale specimens will be tested at Ohio State University, Fire Research Lab.

9. TESTING PROCEDURE

The testing procedure is outlined below for the small and intermediate scale specimens. The testing program for the full-scale E-119 tests has not as yet been fully determined and as a result is not provided herein. All beams were subjected to temperature or fire exposure at least six months after the concrete had been cast.

9.1 DESCRIPTION OF TEST PROCEDURE FOR SMALL-SCALE SPECIMENS

All uniform elevated temperature tests for small-scale beams were conducted at California State University, Long Beach, in an electrically heated oven. The specimens were first broken as a simply supported beam with center load to form either flexural or shear cracks, injected with epoxy adhesives and cured at least 7 days under laboratory conditions prior to any temperature exposure. After the foregoing processes were completed, the beam specimens were placed in an electrically heated oven for a period of 2 hours at a specified temperature. Three to five specimens were tested at each specified temperature, ranging from 70°F to 400°F. For the "hot strength" tests, after 2-hour temperature exposure, the specimens were subjected to single concentrated failure load at the center of the beam span at 10 minutes after the end of the temperature exposure. For the "residual strength" tests, after the two-hour uniform temperature exposure, the specimens were cooled down to room temperature for a period of 7 days prior to testing as a simply supported beam with center load. Deflections and corresponding loads were recorded in order to graph the load vs. displacement curve. Failure modes were also recorded along with the corresponding crack pattern.

9.2 DESCRIPTION OF TEST PROCEDURE FOR INTERMEDIATE-SCALE SPECIMENS

The cross-sectional dimensions of the intermediate-scale specimens (see Fig. 4) correspond to several typical concrete beams repaired with epoxy adhesives as the result of damage suffered during the 1971 San Fernando Earthquake. Since the thermal gradients in concrete beams exposed to fire can be modelled as a two-dimensional problem, the length of the beam specimen need not be similar to actual beam lengths in buildings. Hence, the length of the intermediate-scale specimens was chosen as 8 ft. in order to allow for fire testing at the CSULB furnaces. To simulate realistic shear forces and bending moments, a loading system as given in Fig. 4 was devised. The dual hydraulic jacks at beam ends were used to create pure bending moment in the beam. The vertical load on the top or compression face of the beam was used to generate additional bending moments and simulate realistic shear forces present in concrete beams. The entire loading procedure both for the original unrepaired and the epoxy repaired intermediate-scale specimens was as follows (Reinforcement consisted of two #5 bars):

1. Apply hydraulic jacking force of 10,000 lbs. per jack or 5,000 lbs. per R-bar
2. Load vertically the specimen up to 4,000 lbs.
3. Apply hydraulic jacking force to 30,000 lbs. per jack or 15,000 lbs. per R-bar and hold at this load. This procedure resulted in bar stresses at 0.8 Fy.
4. Increase vertical load to attain about 90% of the ultimate design flexural capacity of the beam.
5. Reduce to zero the vertical load and reduce the hydraulic jack force to 10,000 lbs.
6. Repeat steps 1 through 5 for five more load cycles.
7. After the sixth load cycle, load the specimen to its ultimate strength by increasing the vertical load.

All intermediate-scale specimens were subjected to either the 2-hour ASTM E-119 fire exposure curve or the 1-hour SDHI time temperature curve.

10. PRESENTATION OF TEST RESULTS FOR SMALL-SCALE SPECIMENS

This section provides a summary of test results for the epoxy-repaired concrete beams subjected to elevated temperatures. Test results are grouped into separate categories depending on the temperatures, testing conditions, and types of failure.

Four types of failure in the test program were observed for all small-scale specimens: (1) flexural failure in the concrete; (2) flexural failure in the epoxy; (3) shear failure in the concrete; and (4) shear failure in the epoxy. Figure 6 provides a graphical convention to illustrate a general specimen failure in the elevated temperature test program for the small-scale beams.

Flexural type failure in the concrete is best demonstrated in Figure 7. The major cracks that caused failure in most specimens propagated completely through the concrete and usually several inches away from the epoxy repaired crack. In some cases, minor vertical cracks were also presented in the specimens. This type of failure appeared only in the residual strength tests and for temperature exposures under 200^{o}F.

The flexural failure in the epoxy is illustrated in Figure 8. The major cracks which contributed to this type of failure propagated through the epoxy layer. Observations showed that there was occasional debonding failure at the interface between steel bars and concrete, followed by widening and extension of cracks. This type of failure mostly appeared in hot strength tests.

Shear cracks in the concrete represented was another type of failure that generally occurred for the residual strength tests for the 45-inch long beam specimens. In this failure mode, diagonal tension cracks near mid-span first appeared along an angle of 30 to 40 degrees from horizontal, then combined with original minor flexural cracks, followed by sudded debonding of steel bars to the beam end as shown in Figure 9.

The last type of shear failure occurred most frequently in the hot strength tests at temperatures above 200^{o}F for 45-inch long specimens. Cracks were induced along old epoxy-repaired sections and finally failed as shear cracks. Those specimens which have initial shear cracks near supports were grouped into this category, as demonstrated in Figure 10.

The test results for small-scale specimens are summarized in the graphs provided in Figs. 11 to 18. These graphs illustrate the effects of temperature on stiffness of the beams. Stiffness was defined as follows for simply supported beams with center concentrated loads:

δ = displacement at center of beam

$$\delta = \frac{P\ell^3}{48EI}$$

ℓ = beam span length

P = center concentrated loads

EI = beam stiffness

$$EI = \left(\frac{\ell^3}{48}\right)\left(\frac{P}{\delta}\right)$$

With ℓ, P and δ known or measured, the beam stiffness, EI, can be determined and these values are provided in Figs. 11 to 18 for the various test conditions and types of cracks. P_u refers to the ultimate beam load.

TABLE 1: PROPERTIES OF EPOXY ADHESIVES

PROPERTIES OF LOW VISCOSITY EPOXY ADHESIVES (SHORT POT LIFE)

Viscosity (cps)	300 - 800
Compressive Strength at 70°F(psi)	12,000 - 17,000
Tensile Strength at 70°F(psi)	7,000 - 12,000
Pot Life (minutes)	20 - 120
Heat Distortion Temperature ($^{\circ}$F)	120 - 145
Strength Transition Temperature ($^{\circ}$F)	220 - 240

PROPERTIES FOR EXTREMELY LONG POT LIFE
EPOXY (ADHESIVE ENGINEERING, 1077-II)

Viscosity (cps)	750
Compressive Strength at 70°F(psi)	14,500
Tensile Strength at 70°F(psi)	8,800
Pot Life in Crack (minutes)	>360
Heat Distortion Temperature ($^{\circ}$F)	120

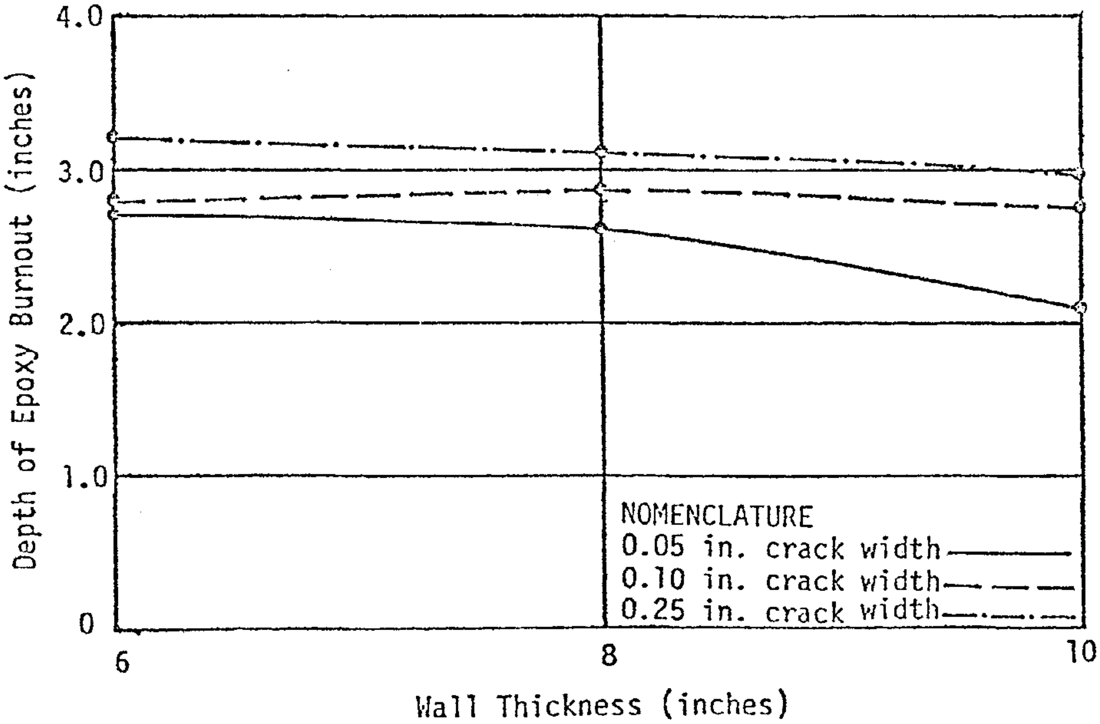


FIG. 1: AVERAGE DEPTH OF EPOXY BURNOUT AS A FUNCTION OF WALL THICKNESS FOR 2-HOUR ASTM E-119 FIRE EXPOSURE

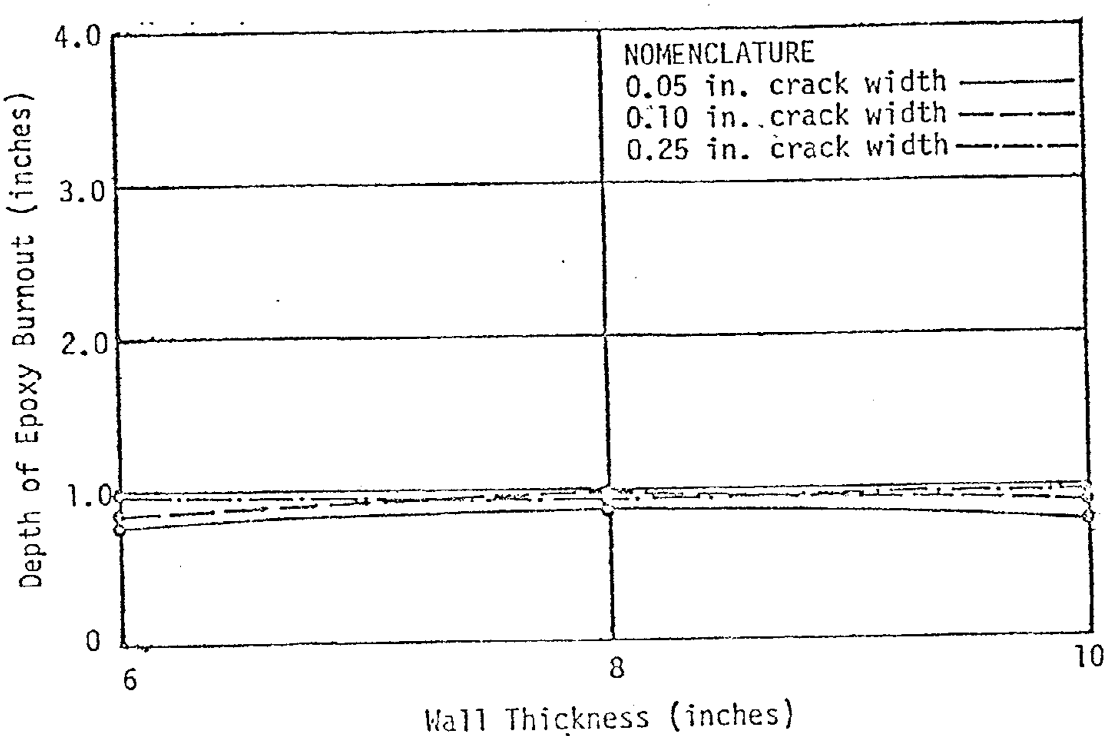


FIG. 2: AVERAGE DEPTH OF EPOXY BURNOUT AS A FUNCTION OF WALL THICKNESS FOR A 1-HOUR SDHI FIRE EXPOSURE

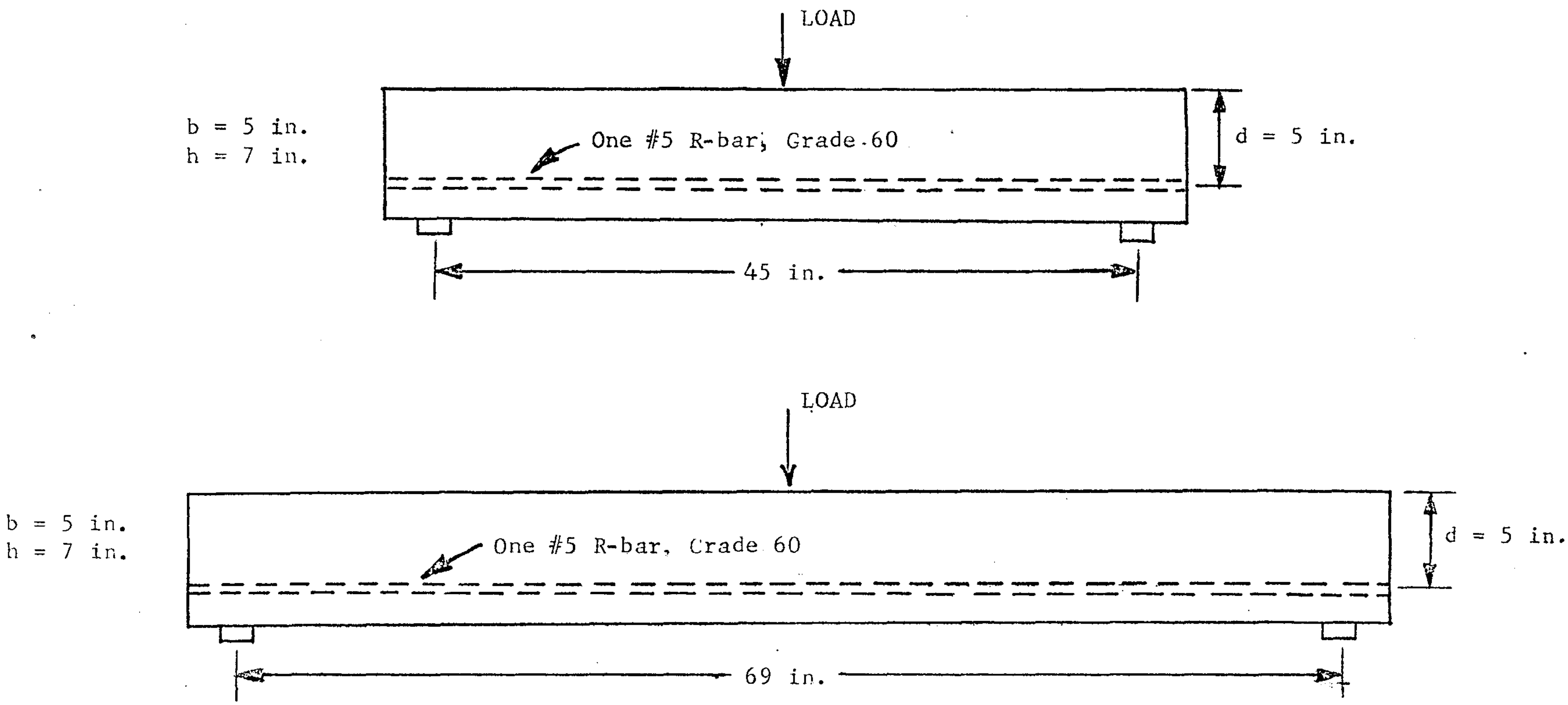


FIG. 3: GEOMETRY, REINFORCEMENT, LOAD AND SUPPORT CONDITIONS FOR SMALL-SCALE SPECIMENS

CENTRAL BEAM DIMENSIONS: b = 8 in.; h = 18 in.; d = 16 in.; 2-#5 R-bars, Grade 60.
No shear reinforcement.

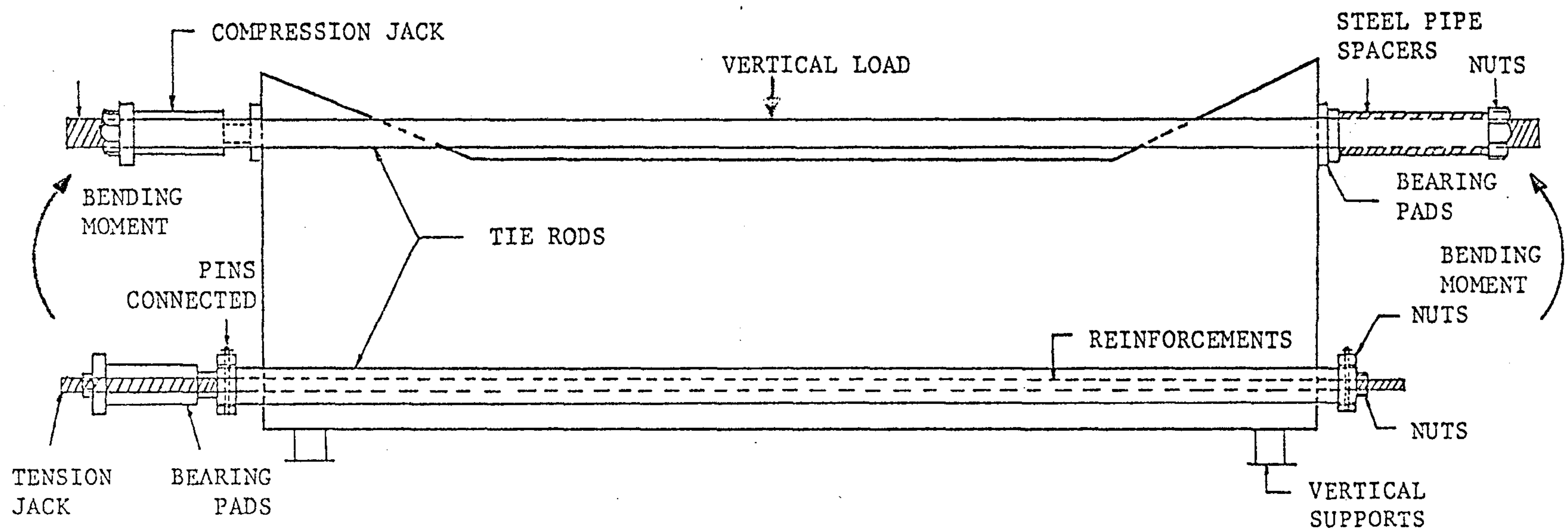


FIG. 4: GEOMETRY, REINFORCEMENT, AND LOADING PROCEDURE FOR INTERMEDIATE-SCALE SPECIMENS

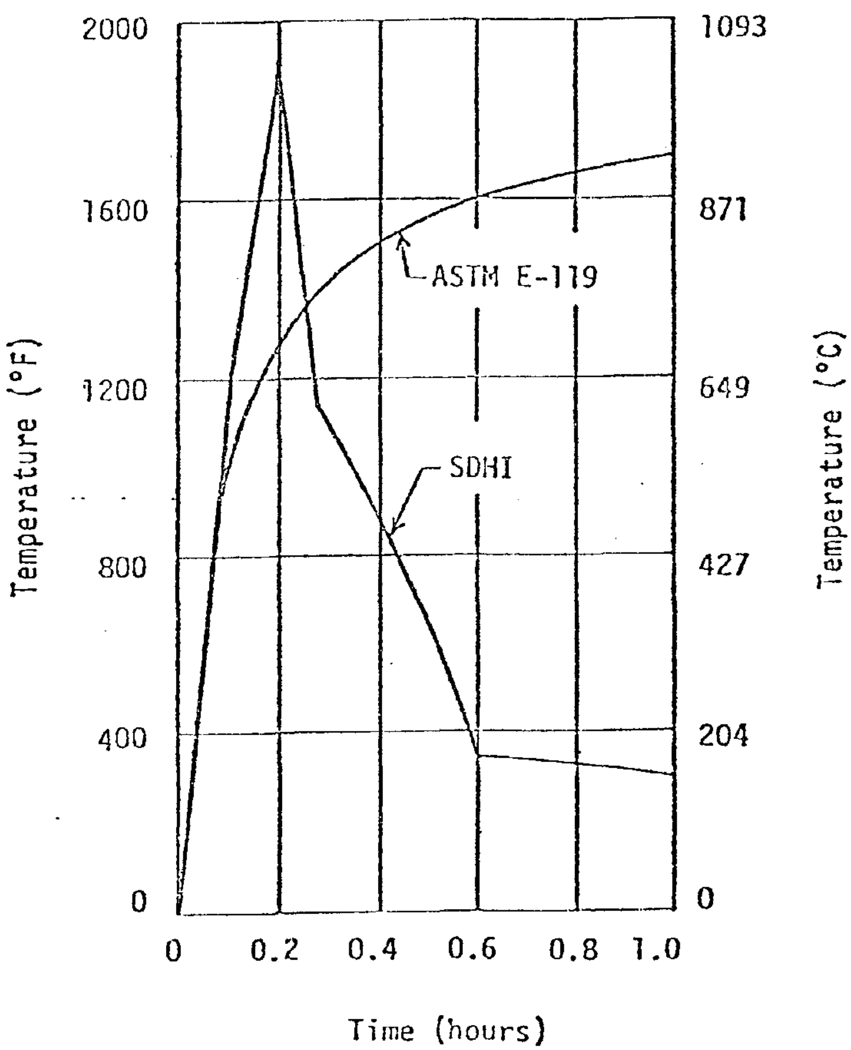
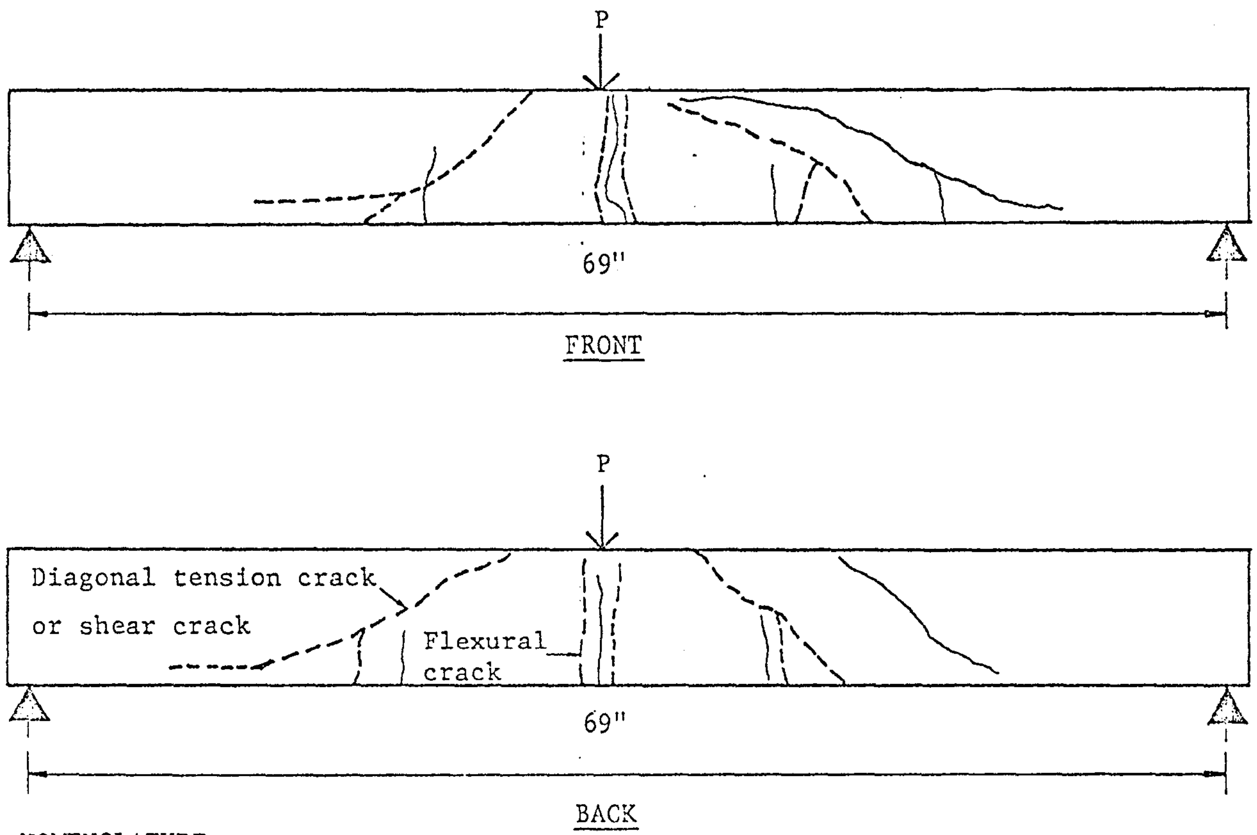


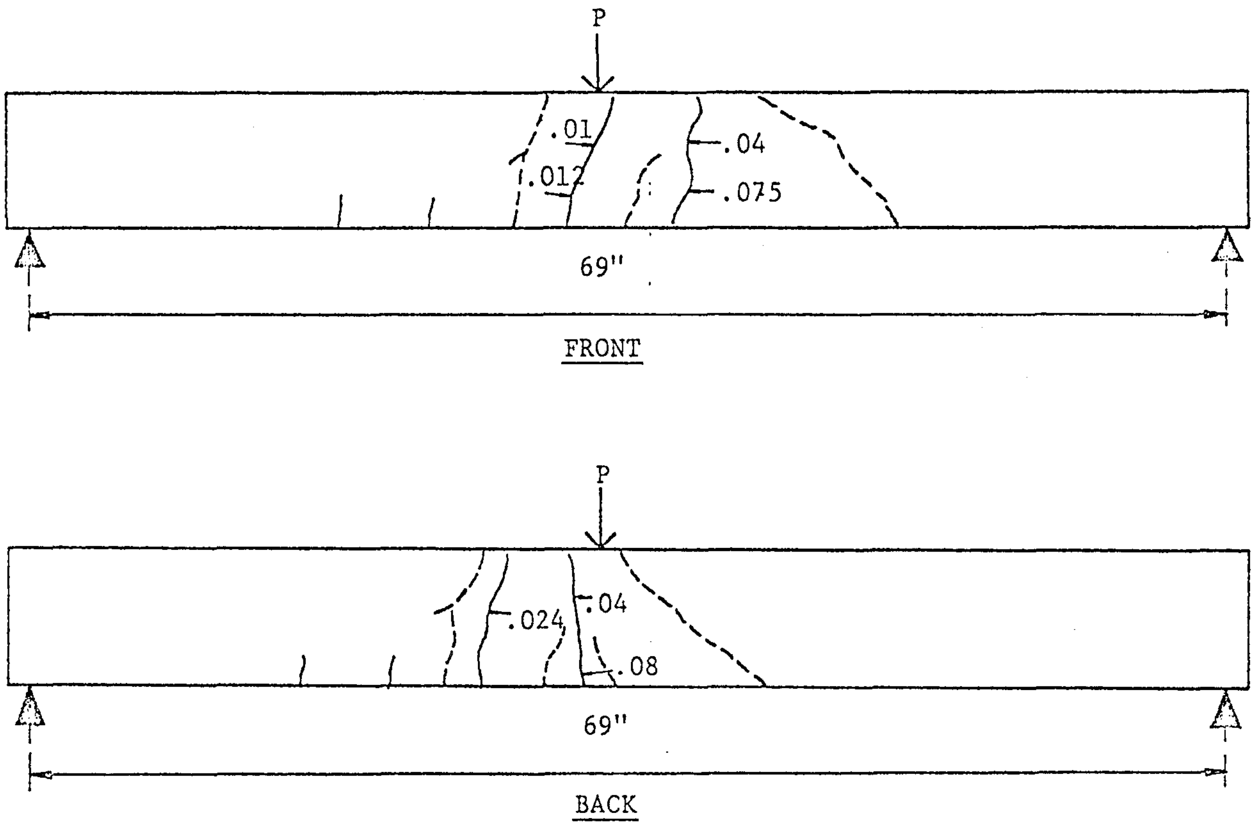
FIG. 5: ASTM E-119 AND SDHI TIME TEMPERATURE CURVES



NOMENCLATURE:

--- New cracks after temperature exposure
—— Old cracks prior to epoxy repair

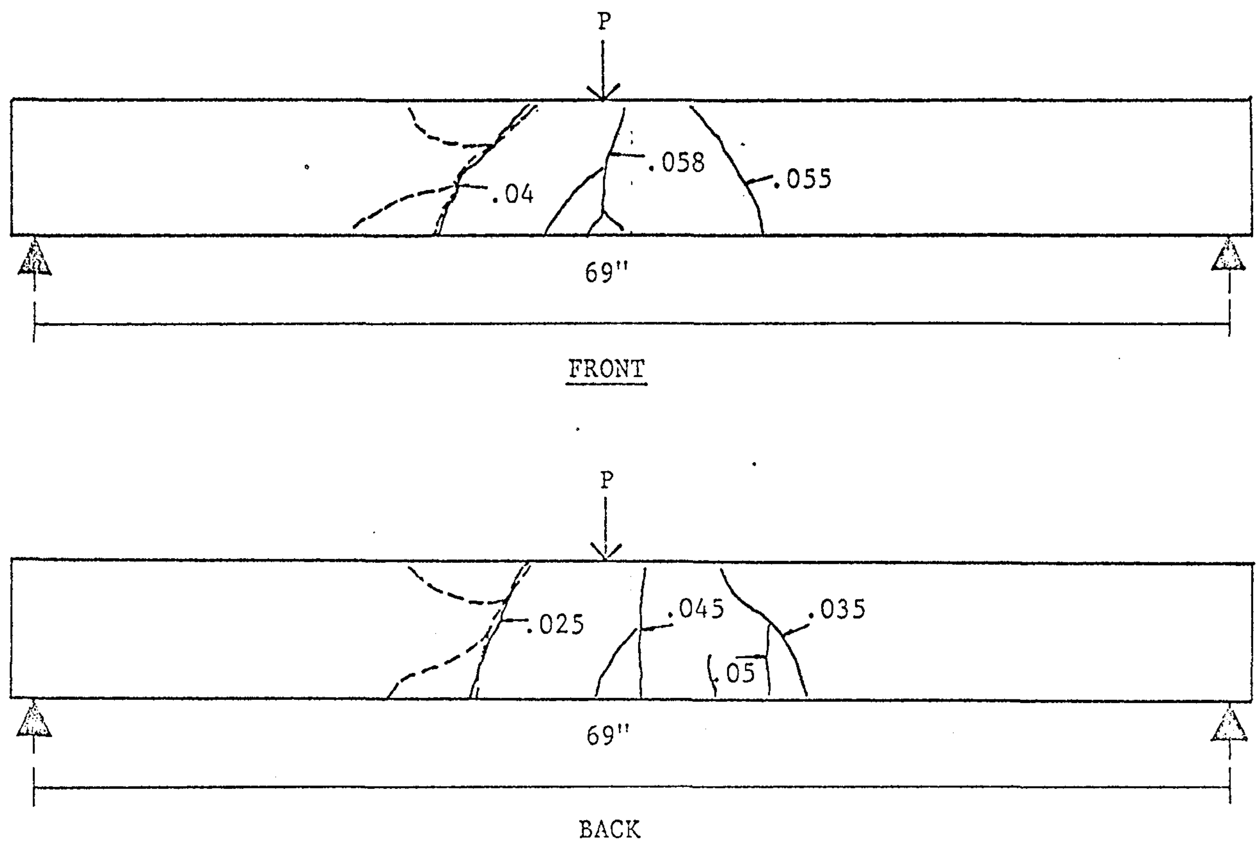
FIG. 6: TYPICAL TYPES OF FAILURE MODES IN THE SMALL-SCALE SPECIMENS



NOMENCLATURE:

--- New Cracks after temperature exposure
—— Old cracks prior to epoxy repair
The numbers represent crack widths in inches

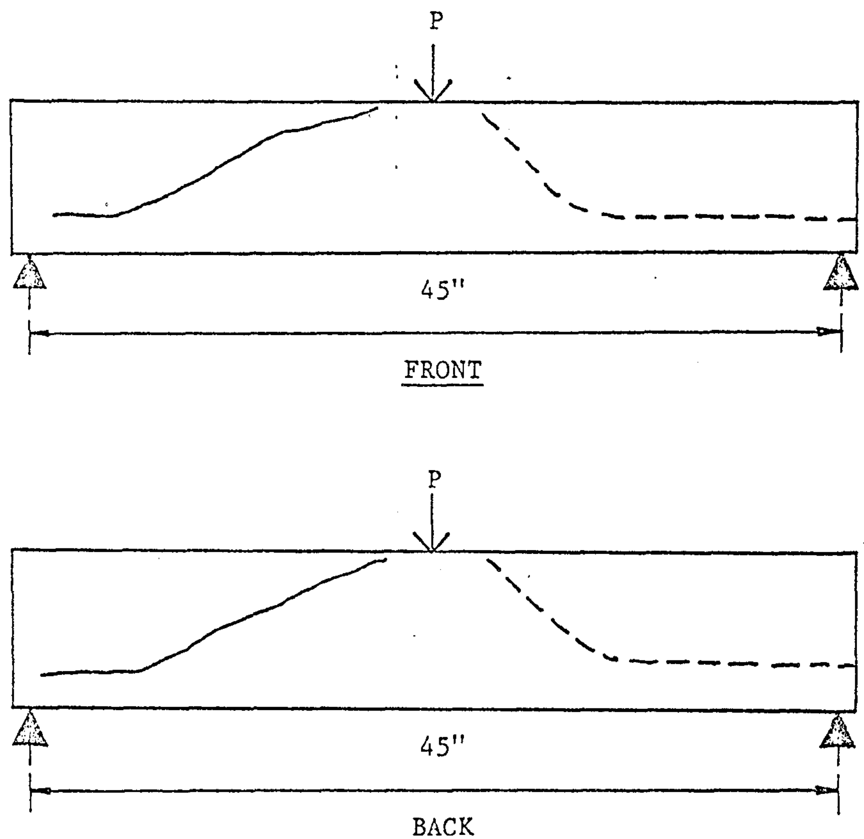
FIG. 7: TYPICAL FLEXURAL FAILURE IN CONCRETE



NOMENCLATURE:

--- New cracks after temperature exposure
—— Old cracks prior to epoxy repair
The numbers represent crack widths in inches

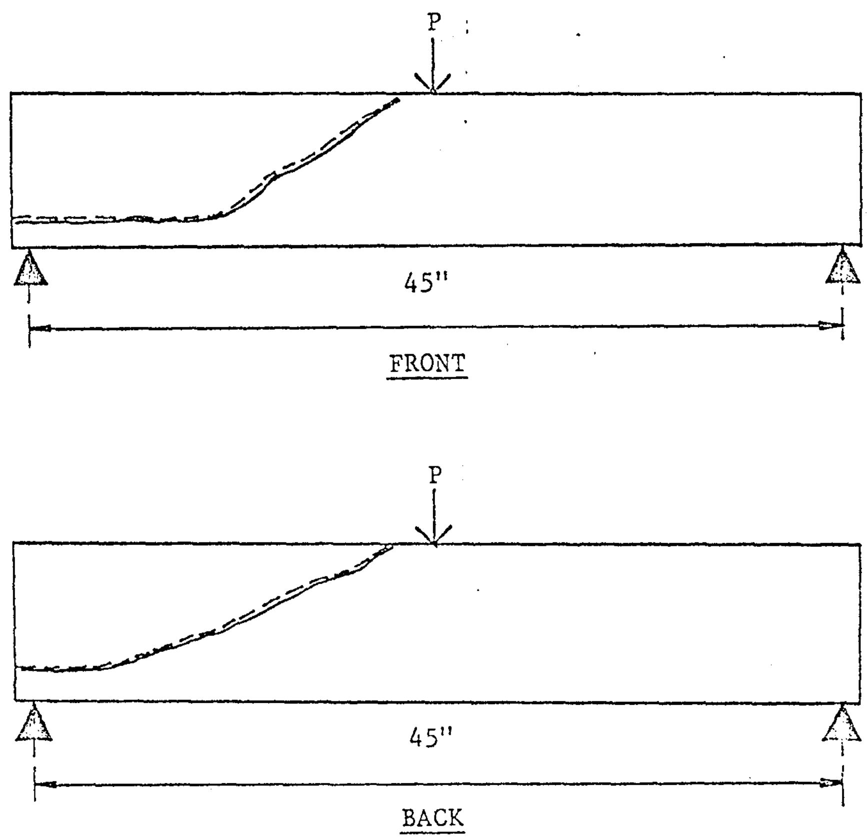
FIG. 8: TYPICAL FLEXURAL FAILURE IN THE EPOXY



NOMENCLATURE:

--- New Cracks after temperature exposure
—— Old cracks prior to epoxy repair

FIG. 9: TYPICAL SHEAR FAILURE IN THE CONCRETE



NOMENCLATURE:

--- New cracks after temperature exposure
—— Old cracks prior to epoxy repair

FIG. 10: TYPICAL SHEAR FAILURE IN THE EPOXY

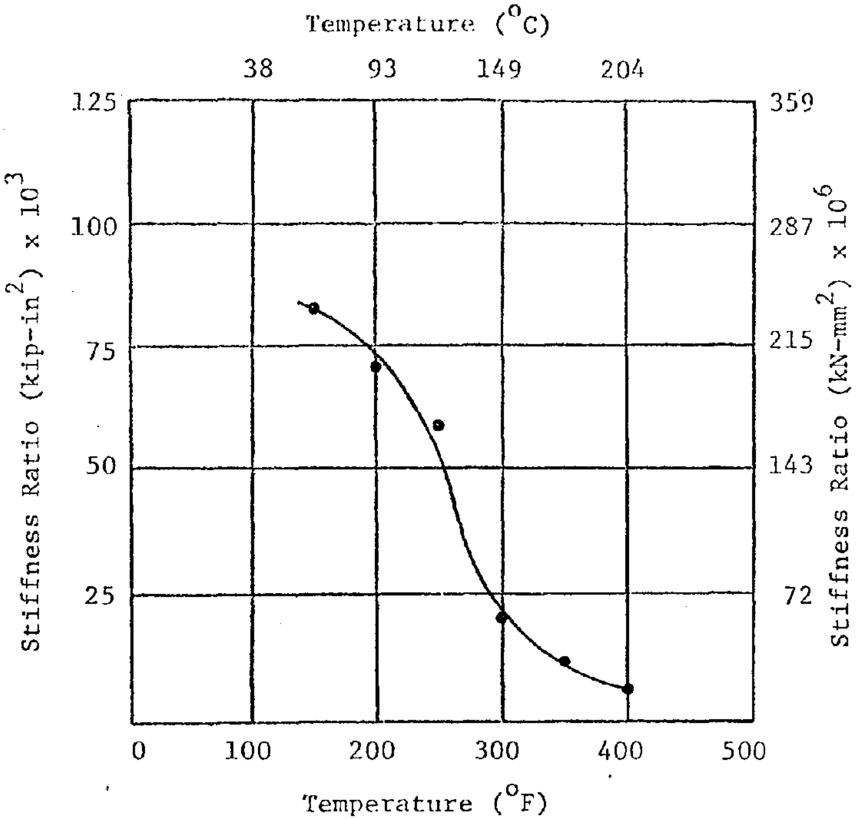


Figure 11: Effect of Temperature on Stiffness at P = 0 for Shear Cracks (Hot Strength)

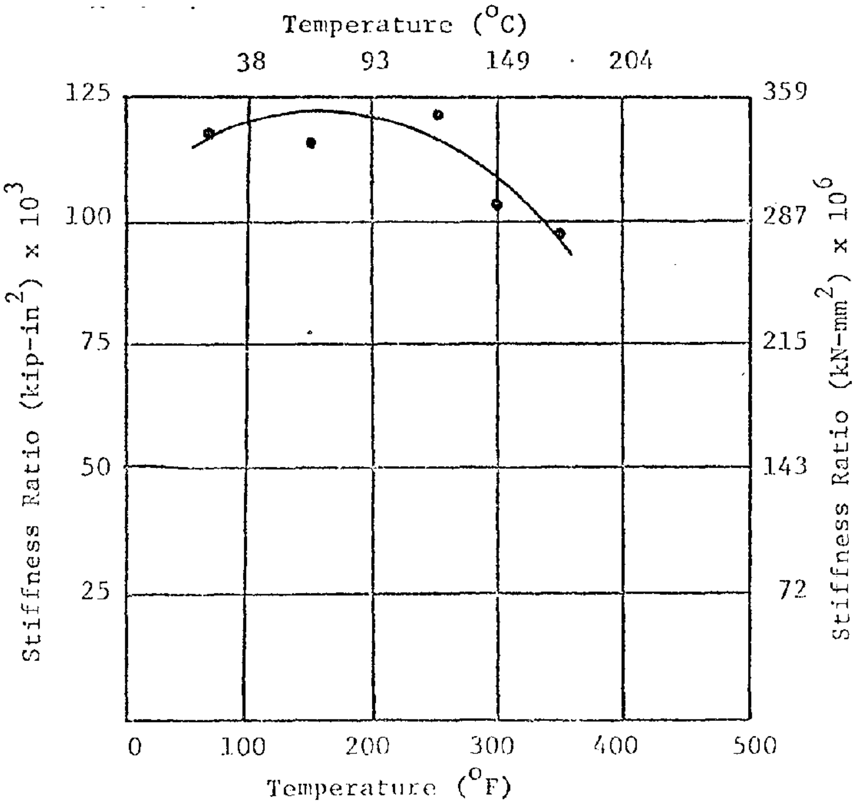


Figure 12: Effect of Temperature on Stiffness at P = 0 for Shear Cracks (Residual Strength)

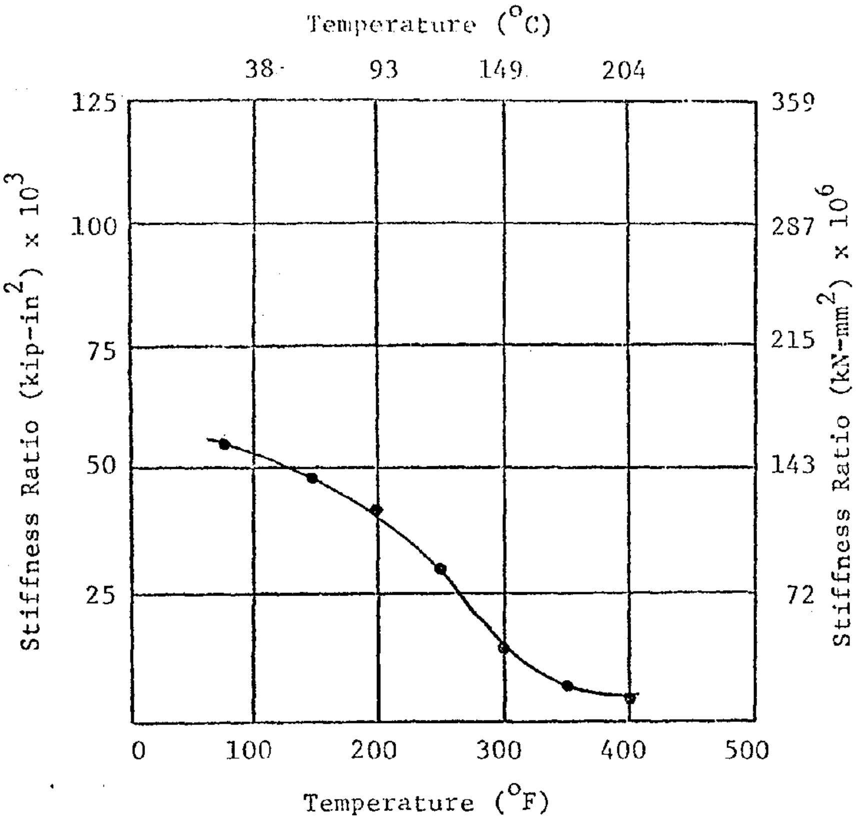


Figure 13: Effect of Temperature on Stiffness at $P = 0.5\ P_u$ for Shear Cracks (Hot Strength)

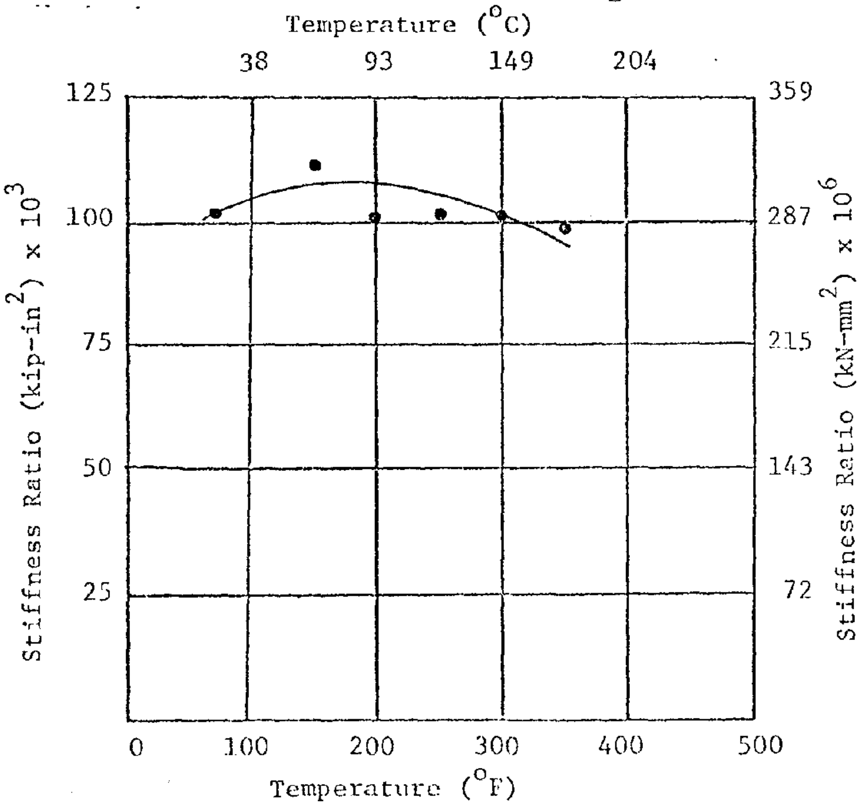


Figure 14: Effect of Temperature on Stiffness at $P = 0.5\ P_u$ for Shear Cracks Residual Strength)

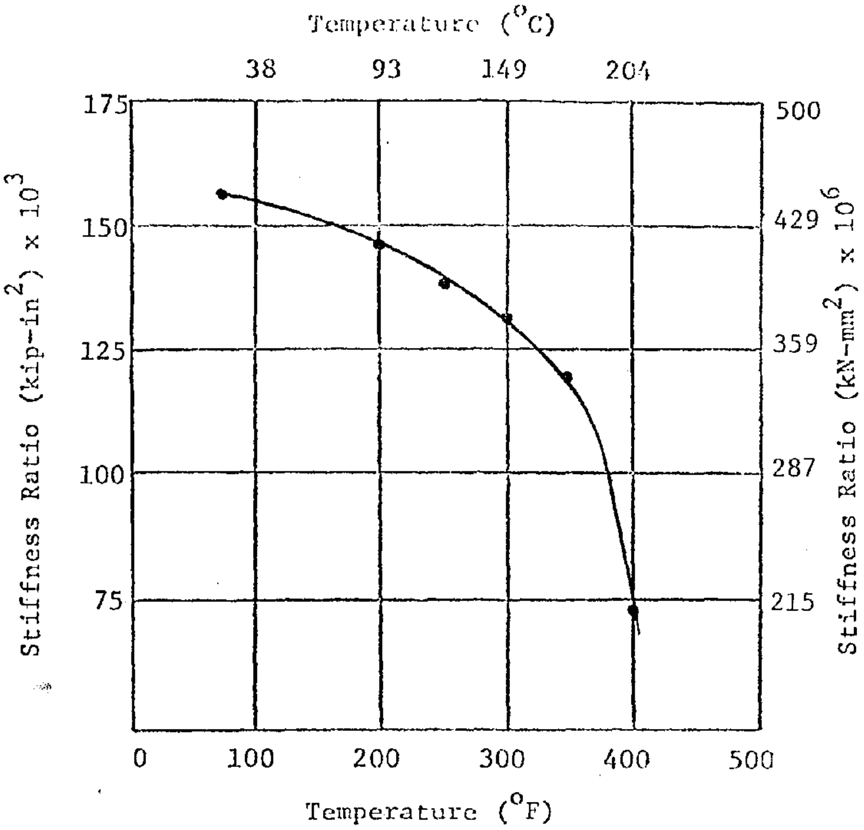


Figure 15: Effect of Temperature on Stiffness at P = 0 for Flexural Cracks (Hot Strength)

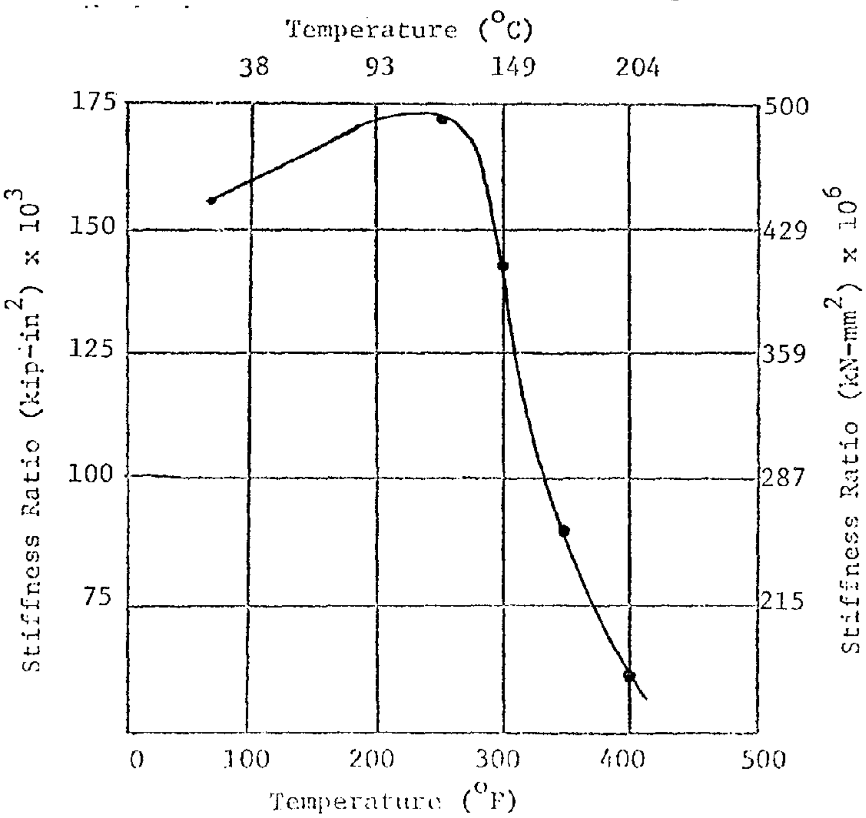


Figure 16: Effect of Temperature on Stiffness at P = 0 for Flexural Cracks (Residual Strength)

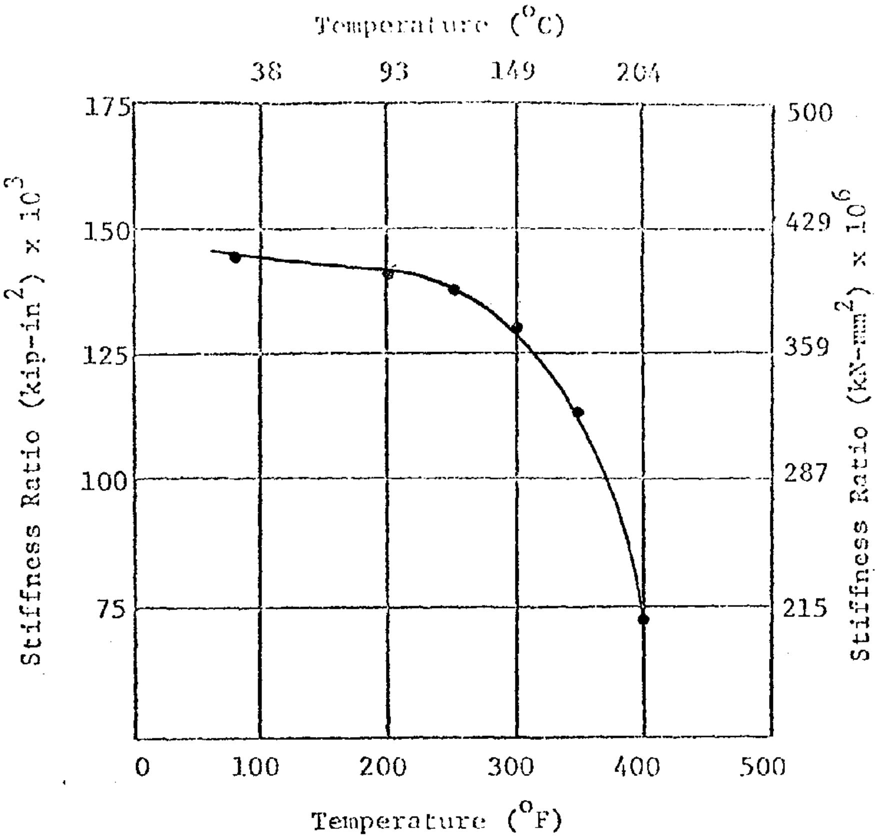


Figure 17: Effect of Temperature on Stiffness at $P = 0.5\ P_u$ for Flexural Cracks (Hot Strength)

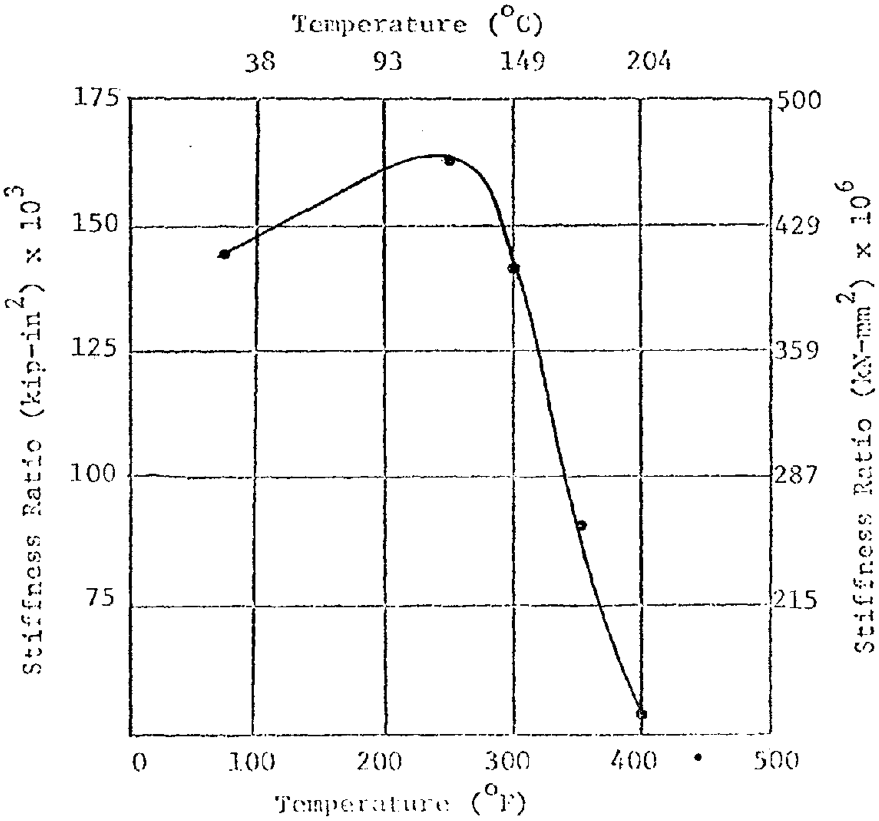


Figure 18: Effect of Temperature on Stiffness at $P = 0.5\ P_u$ for Flexural Cracks (Residual Strength)

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A REPAIR EFFECT OF REINFORCED CONCRETE JOINT ASSEMBLIES SUBJECTED TO SEISMIC LOADS

by

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ABSTRACT

One-half scale reinforced concrete beam-column assemblies were tested to study the effect of repair after being damaged by earthquake loads. The specimens represented second-floor interior and exterior beam-column assemblies of a seven-story full-scale test structure.

The placement of beam and column longitudinal reinforcement was the same in all specimens. The amount and arrangement of lateral reinforcement was varied in specimens following the U.S. and Japanese design requirements. More web reinforcement in beams and columns was required by the U.S. design code. Some of the test specimens were repaired only at the hinge zone of their beam end, and others were done at all parts of the specimen. Epoxy resin was used for the repair works into those test specimens.

Through this test results, it was clarified that the epoxy resin was an effective material for the damaged members to recover their stiffness and strength.

1. INTRODUCTION

A U.S.-Japan Cooperative Research Program Utilizing Large Scale Testing Facilities was initiated in the summer of 1977. The planning group of the program prepared recommendations to improve seismic safety practices through studies and to determine the relationship among full-scale tests, component tests, and analytical studies.

It was recommended that a full-scale seven-story reinforced concrete building structure representing good current practice by tested, and that a series of coordinated experiments associated with the full-scale tests be conducted in Japan and the United States on reinforced concrete joint assemblies, walls and frames. The results of all associated tests in both countries should be fully correlated with each other and with the results of the tests made in Japan on the full-scale seven-story structure.

It was not intended to test the subassemblages beyond a deformation range expected during the full-scale test. Results from the tests will be used to model the stiffness properties of structural members in a nonlinear dynamic analysis, which is used to determine a probable maximum deformation range of the full-scale structure during an earthquake under consideration. In the course of this cooperation project, it was decided that the repair tests of a full scale structure should have been supplemented. However, the repair works for the full scale structure were partially done due to the limited budget. Then the corresponding joint assemblies which were partially and completely repaired were planned to be tested. From the test results, it was confirmed that the repair works being injected to the whole cracks by epoxy resin operated the reasonable recover for the stiffness and strength of the damaged structure.

2. TEST SPECIMENS

The lateral load resistance of the prototype structure is provided by interacting structural wall and frames; two moment resisting frames and one structural wall-frame as shown in Fig. 1. The structure has three spans in the direction of loading. The structure is symmetric about the center.

Spans are 6m, 5m and 6m, in the direction of loading, and 6m and 6m in the transverse direction. Inter-story heights are 3.75m in the first story and 3.0m from the second to the seventh stories. The column dimensions (0.50m x 0.50m) are the same throughout the structure. The beam dimensions (0.30m x 0.50m) are common in the longitudinal direction. The transverse beams have dimensions of 0.30m x 0.45m.

The amount of longitudinal reinforcement is the same in each story and in each span. All columns are reinforced with 8-D22 (No.7) deformed bars (Fig. 2). Gross reinforcement ratio P_g is 1.24%. The amount of longitudinal reinforcement is different at beam end and center (Fig.2). The beam end is reinforced by 3-D19 (No.6) deformed bars (reinforcement ratio of 0.65%) at the top, and by 2-D19 (No. 6) deformed bars (reinforcement ratio of 0.43%) at the bottom. The slab reinforcement is the same in each floor (Fig.3). The slab is reinforced by D10 (No.3) deformed bars in double layers. Additional negative reinforcement (D13, No.4) is provided perpendicular to beams.

The behavior of the entire structure will be effected by the amount of lateral reinforcement and the method of reinforcement placement. The United States and Japan require different amounts of lateral reinforcement and reinforcement detailing. Therefore, it was desired to study the effect of different reinforcement practices in the first phase of this test series.

Two types of beam-column subassemblies were chosen as test specimens for additional repair tests.

(a) interior beam-column assemblies (I-series)

(b) exterior beam-column assemblies (E-series)

The inflection points were assumed to be located at the mid-span of beams and at mid-height of columns. The specimen was taken as the portion bounded by the inflection points.

The dimensions of each test specimen were one-half those of the full-scale structure. However, the amount of reinforcement was not precisely scaled due to the limitation in available bar sizes. Slab reinforcement in the half-scale specimens was placed in a single layer.

The column and beam sections with longitudinal reinforcement are shown in Fig. 4. Although the amount of longitudinal reinforcement at the end and at the center of a beam was not the same in the full-scale structure, the test specimens had uniform longitudinal reinforcement along the beams. Three D10 (No.3) bars were used at the top (tensile reinforcement ratio of 0.65%), and two D10 (No.3) bars were placed at the bottom (tensile reinforcement ratio of 0.43%) of a beam section. Four D13 (No.4) bars were used in the corners and four D10 (No.3) bars were placed at the middle face of a column section. The gross reinforcement ratio was 1.27 percent.

Three specimens in each series were desingned using (a) Japanese design practice with slab, (b) U.S. practice with slab, and (c) Japanese practice without slab. The slab width was chosen to be 75cm on the basis of the effective width of T-beams. The effective width requirements are comparable in the two countries. Dimensions and reinforcement for test specimens are summarized in Table 1 and their details are shown in Fig. 5.

The major difference in U.S. and Japanese design requirements are the amount and arrangement of lateral reinforcement, and the method of anchoring beam longitudinal reinforcement in the beam-column connection. The amount of beam lateral reinforcement in the specimens was determined by the minimum requirements in both countries because the expected shear forces in the members were small. It is a Japanese construction practice in the exterior beam-column connection to bend down the beam bottom longitudinal reinforcement and to anchor the reinforcement in the lower column because the concrete is normally cast from a slab face to the next slab face above at a time.

It was decided to use deformed bars for lateral reinforcement. The smallest deformed bar available was D6 (No.2), and was used in the specimens.

Architectural Institute of Japan Standard (AIJ Standard) for Structural Calculation of Reinforced Concrete Structures-1979-(2) requires the amount of beam lateral reinforcement as follows;

(a) the web reinforcement shall be at least 9mm diameter plain bars or at least 10mm diameter deformed bars (D10),

(b) the web reinforcing bar shall be spaced at not more than one-half the overall beam depth and at not more than 25cm,

(c) the minimum web reinforcement ratio shall be not less than 0.2 percent.

Therefore, the spacing of beam web reinforcement in Japanese specimens was determined to be 12.5cm, one half the overall depth of the beam section.

The spacing of beam web reinforcement in U.S. specimens was determined as one-fourth of the effective depth of the beam section as specified in ACI standard 318-77.

High-early strength concrete was used in the specimens. All specimens were cast outdoors, from the same batch of concrete, in the up-right position. The specimens were left outdoors during the curing period in February, but the temperature never got down below 4°C (39°F).

The average compressive strength of concrete was 340kg/cm^2 (4,900psi or 33PMa), which was 20 percent higher than the specified strength of 270kg/cm^2 (4,000psi or 26.5PMa).

The beam-column subassemblies were tested at 17 to 55 days from concrete casting. The increase in concrete strength was observed to be small during this period.

Reinforcing bars (D6, D10 and D13) were tested in accordance with Japan Industrial Standard, similar to ASTM in U.S. The stress-strain curves of three kinds of reinforcing bars are shown in Fig. 6. The yield stresses of D6 (No.2), D10 (No.3) and D13 (No.4) were 3.77ton/cm^2 (54.7ksi or 370MPa), 3.75ton/cm^2 (54.4ksi or 368MPa), and 4.07ton/cm^2(59.1ksi or 399MPa), respectively. These values were approximately by 5 to 10 percent higher than the nominal strength of 3.5ton/cm^2 (50.8ksi or 343MPa).

D10 and D13 reinforcing bars showed clear yield plateaus, and D6 bars yielded gradually.

3. OUTLINE OF LOADING TESTS

Varying forced displacements were applied at the free ends of the beams. The loading apparatus for series I (interior) and E (exterior) beam-column subassembly specimens is shown in Fig.7.

Constant axial load was applied to the column of series I and E specimens,simulating the gravity load corresponding to the working load condition in the full-scale structure. The level of axial stress was 38.4kg/cm^2 (557psi or 3.77MPa) in series I specimens, and 36.5kg/cm^2 (527psi or 3.65 MPa) in series E specimens.

A 100-ton capacity actuator with ±30cm stroke was used to apply the constant axial load in the column. The load was measured by a 100-ton load cell. An electrical pump with automatic control was used to apply hydraulic pressure in the 100-ton actuator.

Two 30-ton capacity actuators (±30cm stroke) were used to displace the beam free ends of series I specimen. The same actuator was used in a series E specimen. Two 30-ton capacity load cells were attached to these actuators.

The existing heavy steel reaction frame was used for the test. The reaction frame is made of 900 x 300 x 16mm H sections for columns, and 600 x 300 x 12mm H section for beams. Some extension pipe members were added to the actuator load cell assembly to fill the space between the top and bottom beams of the reaction frame.

The top end of the series I and E columns was supported by a vertical roller. The other ends of all specimens were either supported through a mechanical hinge, or connected to a loading device through a mechanical hinge. The location of mechanical hinges and roller in the specimens corresponded to the assumed location of inflection points in the full-scale structure.

The loading program is schematically shown in Fig. 8. In a series I specimen, the displacements at both beam ends were moved by the same amplitude, but in the opposite directions.

Although the lateral movement at the column tips was prevented during the experiment, the story displacement of the prototype structure could be determined from the displacement measurements of the specimen. In the prototype structure, the inflection points in beams will not appreciably move in the vertical direction. Therefore, if the beam tips were assumed to have stayed horizontally, the story lateral displacement between the two column inflection points can be determined as shown in Fig. 9.

γ_p : shear distortion angle of beam-column joint panel

h_c : one-half of column clear height

ℓ_b : one-half of beam clear span

Sometimes it is convenient to know a story deformation angle, R, rather than a relative story displacement. The story deformation angle is defined as the relative story displacement, σ_{story}, divided by the inter-story height h.

$$R = \sigma_{story} / h$$

The damage of nonstructural elements in a structure due to earthquake motion is often related to this able.

Story shear, Q, was approximately obtained from the moment equilibrium of the subassemblage as given below.

$$Q = \frac{P_1 + P_2}{2} \times \frac{\ell}{h}$$

where

p_1, p_2: lateral loads applied at beam tips
ℓ: span length

4. OUTLINE OF TEST RESULTS BEFORE REPAIR

4.1 I-Series

I-Series Specimens (I-1, -2 and -3) were cross-shaped, representing a second-floor interior beam-column subassemblage of the prototype building. Specimens I-1 and I-2 had a slab, but Specimen I-3 did not have it.

General behavior of Specimens I-1 and I-2 was similar, but different from that of Specimen I-3.

Figure 10 shows cracking patterns of these three specimens after the test. Tensile yielding of beam reinforcement was observed in the first cycle in all specimens.

Specimens I-1 and I-2 showed almost identical crack patterns, indicating little effect of the difference in the amount of web reinforcement on the behavior in this range of deformation.

Flexural cracks were first observed in the beams or slab. Slab cracks developed perpendicular to the beam axis extending over the entire slab width. Diagonal shear cracks in the beam beneath the slab were observed at approximately $R = 1/200$, starting from the slab side only. The slab contributed to the flexural capacity when the beam top was in tension, hence the larger shear force was developed to cause shear cracks. However, when the beam bottom was in tension, the slab contribution to the flexural capacity was not large enough to cause shear cracks.

When the diformation increased to approximately $R = 1/50$, the cracks developed along the beam-slab boundary line. Wide flexural cracks in the beam lower part concentrated at the column faces. No compressive crushing was observed in these two specimens. Minor flexural cracks were observed in the column at the beam faces.

In the case of Specimen I-3, the shear force in the beams was limited by a relatively low flexural yielding capacity. Therefore, the cracks were all due to bending without compressive crushing. The beams did not develop diagonal shear cracks. No flexural crack was observed in the columns, nor in the beam-column joint panel.

4.2 E-Series

E-Series Specimens (E-1, -2 and -3) were of T-shape, representing a second-floor exterior beam-column subassemblage of the prototype full-scale building. Specimens E-1 and E-2 were constructed with a slab, and Specimen E-3 without a slab.

Figure 11 shows crack patterns of three specimens after repair.

Flexural cracks were observed first in all specimens. No flexural crack was observed in columns, but small vertical cracks were observed above the slab near the beam-column joint in Specimens E-1 and 2. These cracks were not significant.

The behavior of the beams was dominated by flexure. Wider cracks were observed on the slab surface of Specimen E-1 than that of Specimen E-2.

In Specimen E-2, narrow cracks appeared on the entire slab face in the first cycle. In Specimen E-1, wide cracks developed near the beam-column joint, and cracks spreaded toward loading point in later loading cycles.

Diagonal shear cracks were observed in the web of beams in Specimens E-1 and E-2 only in one direction, starting from the slab side. These cracks were developed when the slab was in tension, causing the slab longitudinal reinforcement to yield. No diagonal crack was observed in the beam of Specimen E-3 because the flexural capacity of the beam was low.

Cracks observed on the slab surface of Specimens E-1 and E-2 near the joint were perpendicular to the beam longitudinal axis, whereas the cracks near the loading point were in a fanshape pattern with a pivot at the loading point.

A wide flexural crack was observed in the beam lower portion along the column face, in Specimens E-1 and E-2 when the Specimens were displaced by R = 1/100. The second wide crack appeared approximately 10cm from the first crack in the two specimens at R = 1/50.

Spalling of the cover concrete occurred at the bottom face of the beam adjacent to the beam-column joint at the final stage of loading in Specimen E-1. No reinforcement bucking was observed.

In Specimens I-1 and I-2, measured beam top yield moments are lower than the calculated values. This is because yielding of all the slab reinforcement occurred slightly after the yielding of the beam reinforcement. In the case of beam bottom tensile yielding, the values of measured yield moments in Specimens I-1 and I-2 were about 30 percent larger than that in Specimen I-3.

5. REPAIR WORKS

The location of the cracks was concentrared in the hinge zone of the beams. For the crack repairng, epoxy resin was used. In Figs.10 and 11, the hatched portions of the cracks were injected by epoxy resin. In Specimens I-1 and E-1, only hinge zone of beams were repaired and other specimens were repaired in their full cracks.

The physical properties of the epoxy resin is shown in Table 1. Young's modulus of epoxy resin is far smaller than that of concrete. However the tensile strength of epoxy resin is about ten times higher than that of concrete. Compression failure and exfoliation of concrete at the beams were observed. These exfoliated portions were pasted by epoxy mortar after being cleaned up. The physical properties of this epoxy mortar is shown in Table 2.

6. TEST RESULTS AFTER REPAIR WORKS

6.1 I Series

The vertical displacements were applied at beam ends of the subassemblage to simulate the lateral load induced in the prototype structure. The inflection point of the beam in the prototype building were assumed at the mid-span. In Figs. 10 and 11, the right hand side shows the final crack pattern after repair.

Figures 12, 13 and 14 show story shear interstory displacement relationship of Specimens I-1, I-2 and I-3, respectively. In these figures, the solid and broken lines are the hysteresis before and after repairing respectively.

Specimens I-1 and I-2 showed generally identical behavior up to final stage of the tests before and after repair. The two specimens showed conspicuous slip behavior in the fifth cycle (R = 1/50) of loading. Such a behaviour was observed also after repair works. However, Specimen I-1 which was repaired partially showed smaller stiffness in the first cycle.

This slip behavior must be closely related to the wide flexural cracks concentrated in the beam lower portion at the column faces.

Hysteresis shape of Specimen I-3 was different from those of Specimens I-1 and I-2. Hysteresis loops of Specimen I-3 were fat and stable, before and after repair, and strength decay was not observed. The effect of slab on the hysteresis behavior was significant. Through these tests, the overall hysteresis loops after repair showed larger strength than those before repairs. In order to clarify such a discrepancy, it is considered that more basic tests on epoxy materials should be planned.

The overall deflection of the subassemblage was composed of the beam, column and beam-column joint deformations. The beam deformation shared a large portion of the subassemblage deformation, particularly after yielding throughout before and after repair tests.

6.2 E-Series

Force-deflection relationship of the specimens is discussed in terms of story shear and inter-story displacement. Figures 15, 16 and 17 show the relations for Specimens E-1, 2 and 3, respectively. The inter-story displacement amplitudes of corresponding loading cycles were not identical in Specimens E-1 and E-2 before repair because the loading control by the beam end displacement was not successful during the two tests.

The amplitudes of story shear in the positive and negative directions were different in Specimens E-1 and E-2, directly reflecting the strength contribution from the slab. When the slab was in tension, the sign of story shear was defined positive. The maximum positive shear was more than 2.5 times the maximum negative shear in both cases before and after repair. The difference was also attributable to the more negative reinforcement in the beam.

The load-deflection relationship of Specimen E-1 before repair (solid line) was almost identical to that of Specimen E-2 up to a displacement amplitude of approximately 40mm (story displacement angle of R=1/50). In the second cycle at R=1/50 before repair, Specimen E-1 showed a larger reduction in resistance at the peak displacement than Specimen E-2. The resistance of Specimen E-1 at story displacement angle R = 1/50 was 90 per cent of the maximum resistance, whereas the resistance of Specimen E-2 was 96 percent before repair. The resistance of Specimen E-1 deteriorated further, and became lower than that of Specimen E-2 beyond 40mm displacement. Larger deterioration of resistance must be attributable to the smaller amount of web reinforcement in the beam and beam-column connection and the method of anchoring the beam longitudinal reinforcement in Specimen E-1. However, the deterioration appeared in a displacement range much larger than the expected maximum earthquake response displacement. After repair works, the resistance of Specimen E-1 also deteriorated suddenly at story displacement angle R = 1/50 because of the exfoliation of slab from the beam web , whose portion was not repaired by epoxy resin. However, Specimen E-2 which was fully repaired did not cause such a deterioration.

Both Specimens E-1 and E-2, both before and after repair, showed a slip-type hysteresis shape in the positive direction (slab in tension) at the story displacement angle greater than R = 1/100. Up to this displacement, the hysteresis loops were stable.

Specimen E-3 showed a hysteresis loop shape different from Specimens E-1 and E-2; i.e., the hysteresis loop was fat and stable, dominated by flexural behavior before and after repair. This desirable behavior was obtained because beam shear was limited by low flexural capacity of the beam without the slab contribution. In this test specimen, the broken line (after repair) also showed higher resistance than the original test result (solid line).

6.3 Effects of Repair Works

Fig. 18 shows the envelope curves of interstory displacement in each test specimen before and after repair works. The rules of loaidng reversals between before and after repairs were not equal. However, the difference of the envelope curves was clearly observed between before and after repair works. Specimens I-1 and E-1 were repaired by epoxy resin at only hinge zones of their beam assemblies. In the small amplitude (R = 1/400) the shear resistances of these partially repaired specimens were smaller than those of original ones. Such a phenomena was not clearly observed in other four test specimens whose cracks were all repaired. Fig. 19 shows the ratio of shear resistance of the specimen after repair based upon this shear resistance before repair in every amplitude. In the large amplitude, the shear resistance ratio becomes about 1.2 times of the original one in both I E series.

It is considered that this rising up probably depend upon the bond improvement between main bars and concrete by epoxy resin injecting and the strain hardening of longitudinal reinforcements. However, the course of this rise of resistance could not precisely be defined. In order to testify this course, the authors are now discussing the plans of the basic and material tests on the bond characteristics of the test piece with epoxy resin and the plastic behaviour of reinforcement under a long term period.

7. CONCLUDING REMARKS

It was testified that epoxy resin and epoxy mortal were effective materials for the repair of reinforced concrete members damaged by earthquakes. At the same time these materials caused the unexpected strengthening of the members. It is considered that the design criteria of this strengthening should be clear experimentally.

Table 1 Summary of Specimens

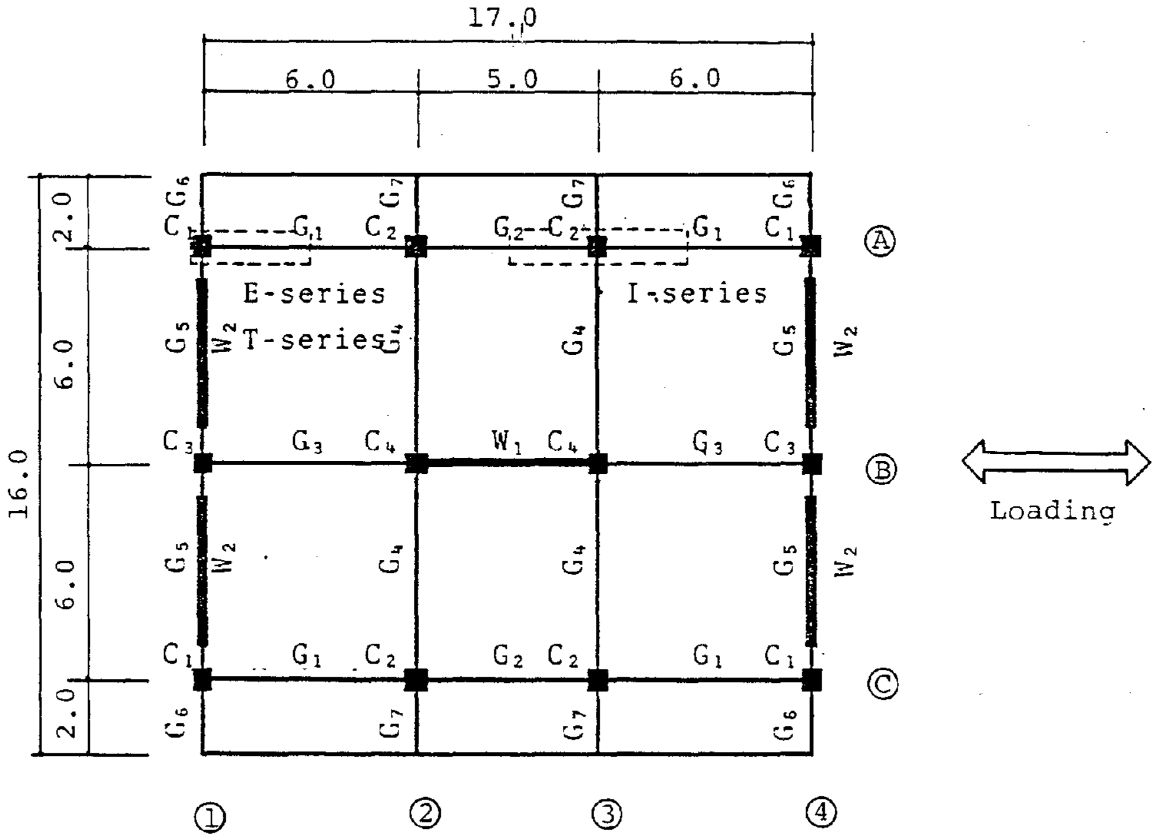
Specimen	Slab	Design Code	Lateral Reinforcement Ratio	
			Column(%)	beam (%)
I-1	O	Japan	0.290	0.238
I-2	O	U.S.A	1.310	0.430
I-3	X	Japan	0.290	0.238
E-1	O	Japan	0.290	0.238
E-2	O	U.S.A	1.310	0.430
E-3	X	Japan	0.290	0.238
T-1	O	Japan	0.290	0.238
T-2	O	U.S.A	1.310	0.430

Table 2 Physical Properties of Epoxy Resin (Beam-Column Assemblages Tests)

Items	Unit	Test Values
Specific Gravity	——	1.14
Viscosity	cp	320
Compressive Strength	kg/cm^2	772
Young's Modulus (Compression)	kg/cm^2	24500
Tensile Shear Strength	kg/cm^2	121

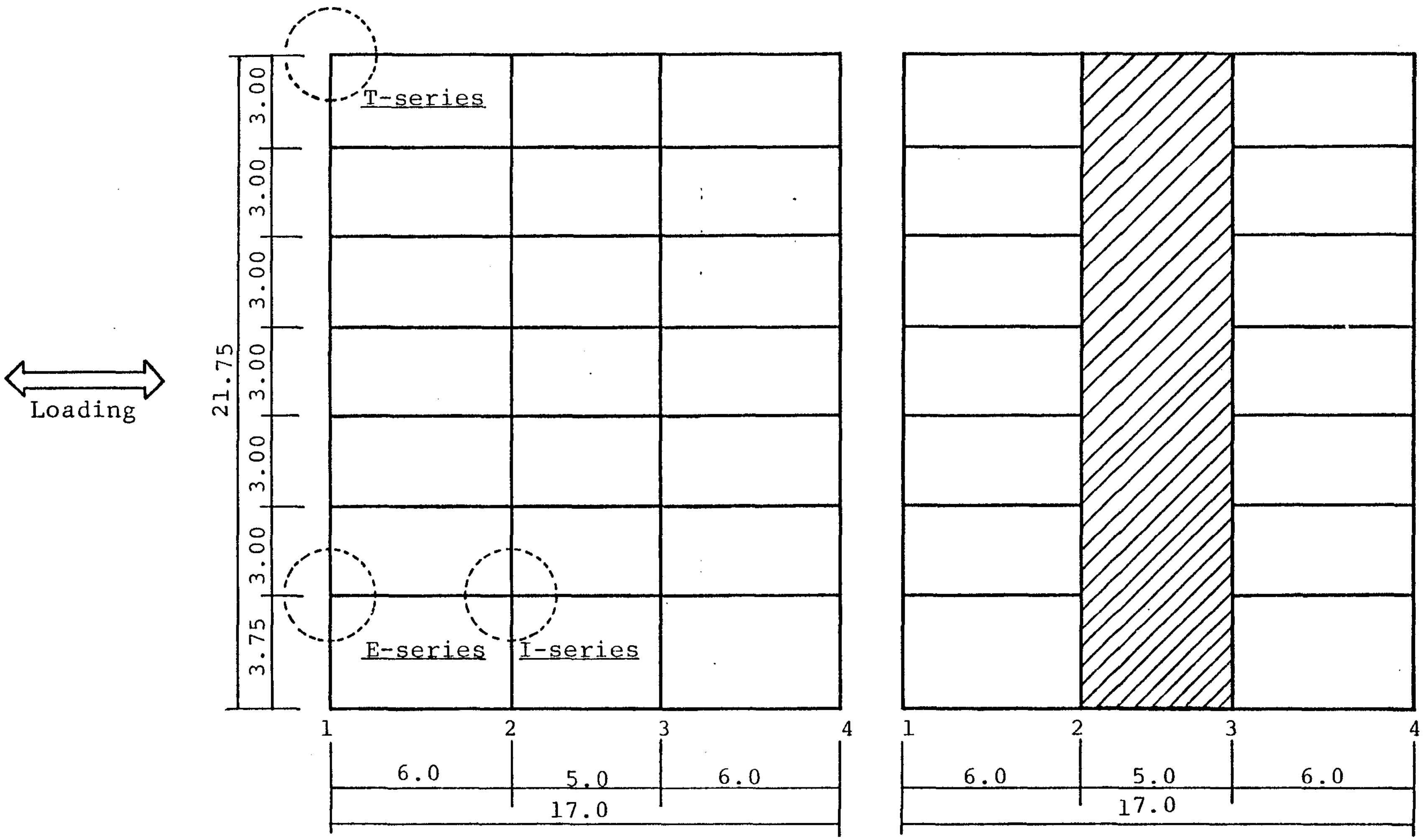
Table 3 Physical Properties of Epoxy Mortar (Beam-Column Assemblages Tests)

Items	Test	Unit	Test Values
Specific Gravity	JIS K 7112	——	1.70
Flexural Strength	JIS K 7203	kg/cm^2	510
Compressive Strength	JIS K 7208	kg/cm^2	841
Young's Modulus	JIS K 7208	kg/cm^2	69200
Tensile Strength	JIS K 7113	kg/cm^2	285
Impact Strength	JIS K 7111	$kg*cm/cm^2$	1.8
Hardness	JIS K 7215	H_dD	89
Tens. Shear Strength	JIS K 6850	kg/cm^2	130



(a) Plan (unit:m)

Fig. 1 : Full-Scale Structure



(b) Elevation : A,C Frame (unit:m)

(c) Elevation : B Frame (unit:m)

Fig. 1 (Continued) Full-Scale Structure

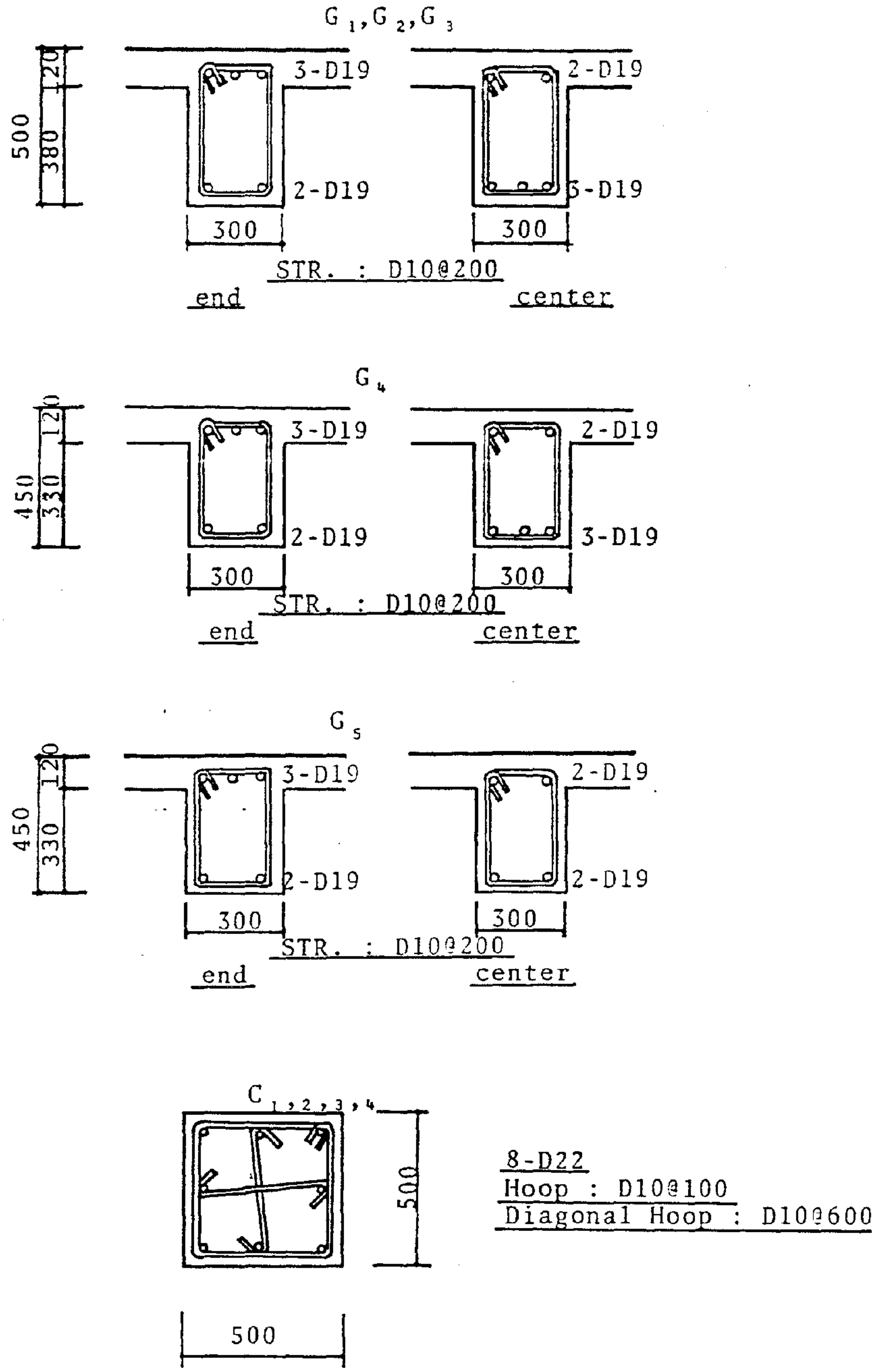
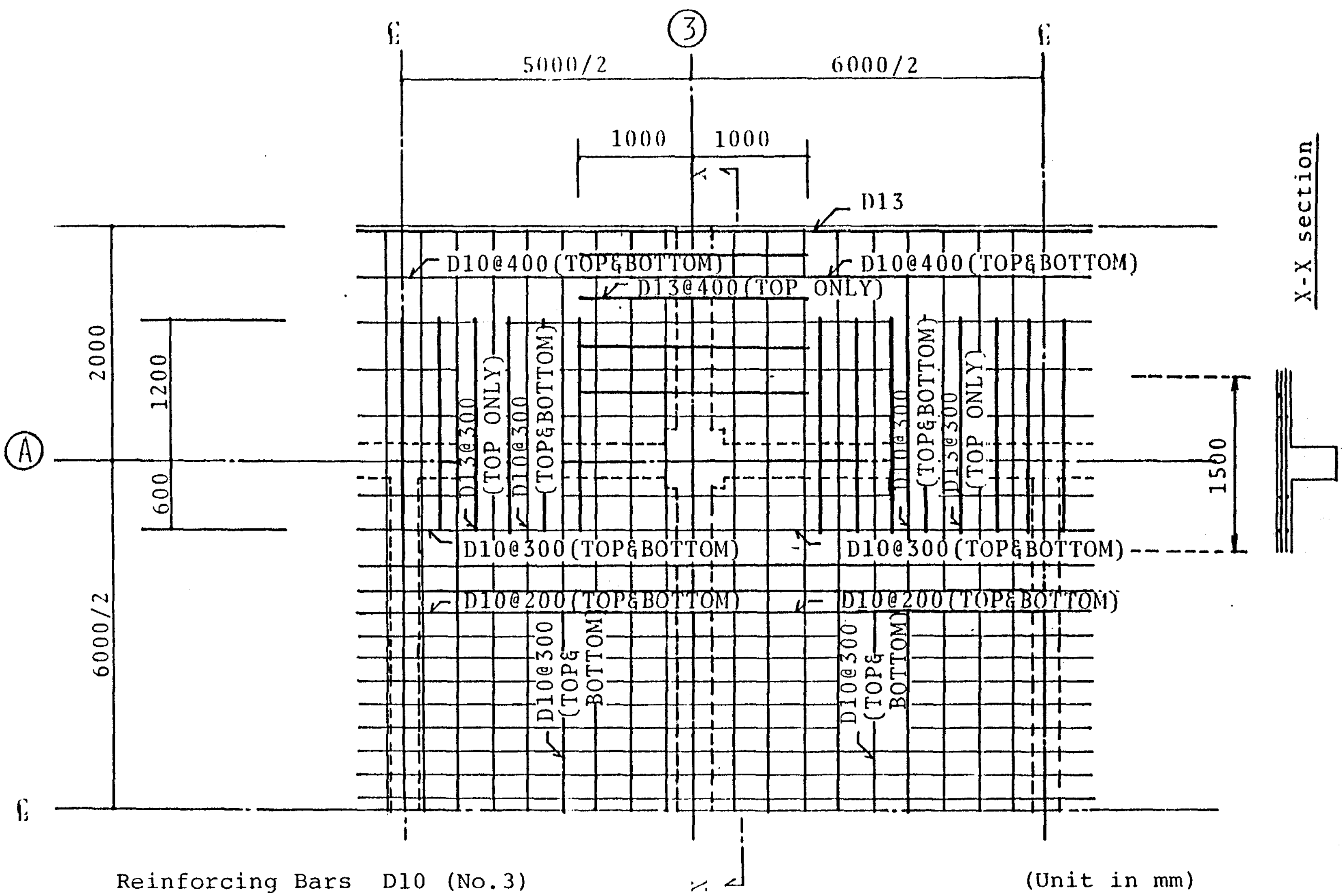


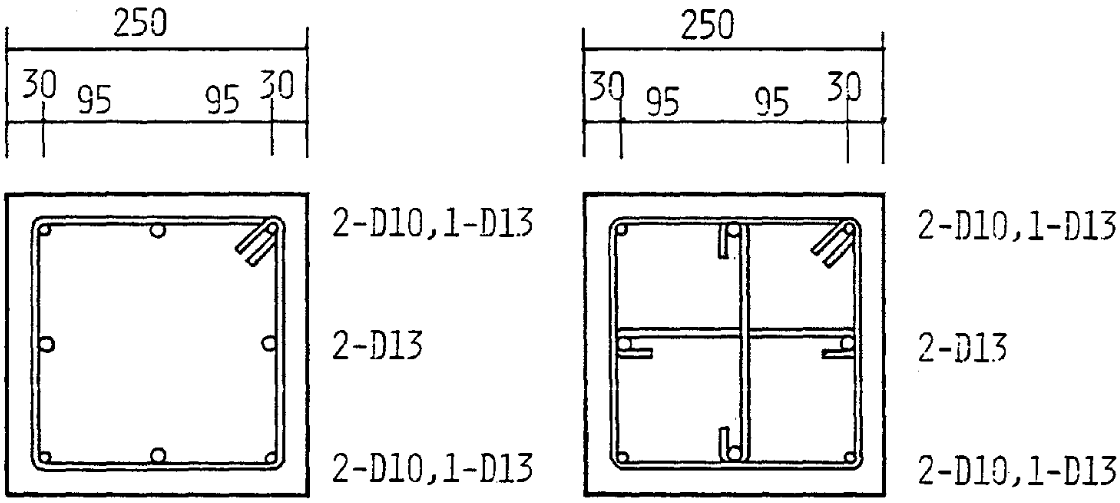
Fig. 2 : Longitudinal Reinforcement in Full-Scale Structure



Reinforcing Bars D10 (No.3)
D13 (No.4)

(Unit in mm)

Fig. 3 : Details of Slab Reinforcement

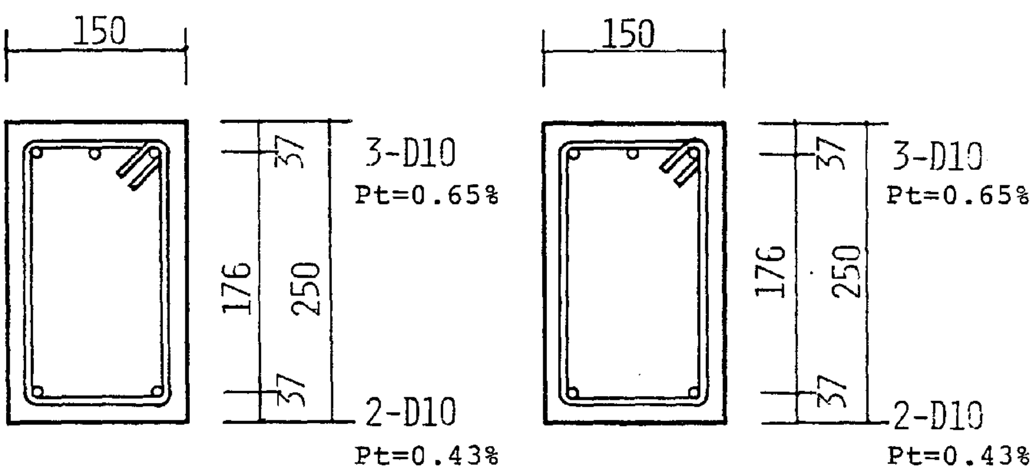


Pt=0.43%
Pg=1.27%
Japanese Specimen

Pt=0.43%
Pg=1.27%
U.S. Specimen

(a) Column

Pt : Tensile Reinforcement Ratio
Pg : Gross Reinfocement Ratio

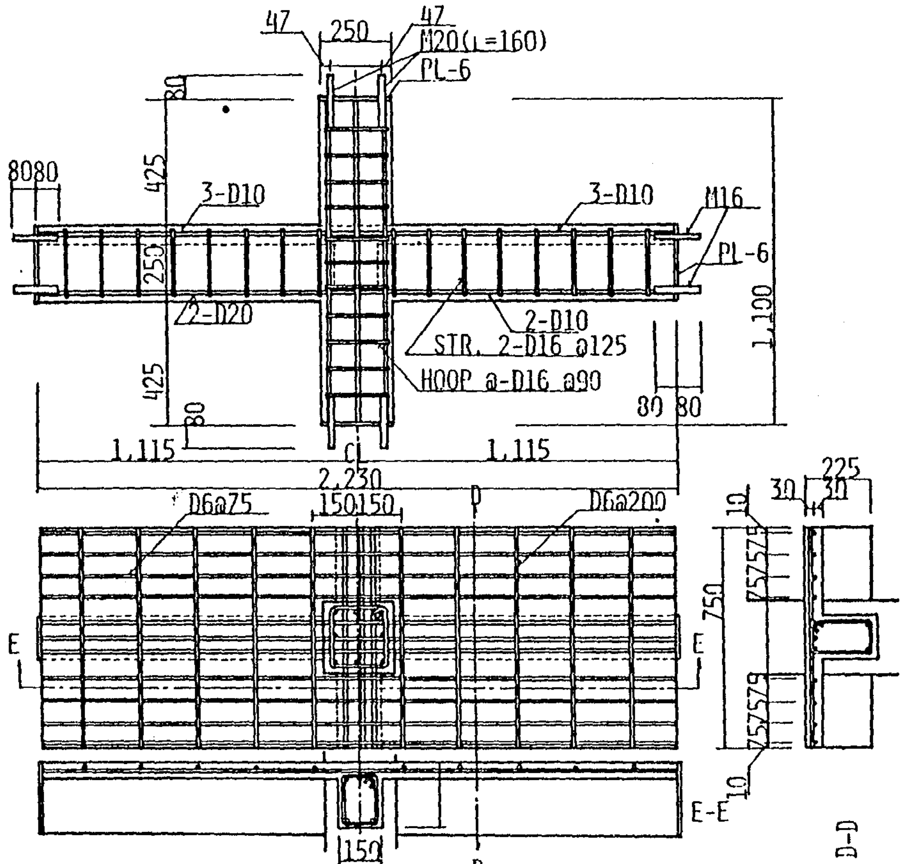


Japanese Specimen

U.S. Specimen

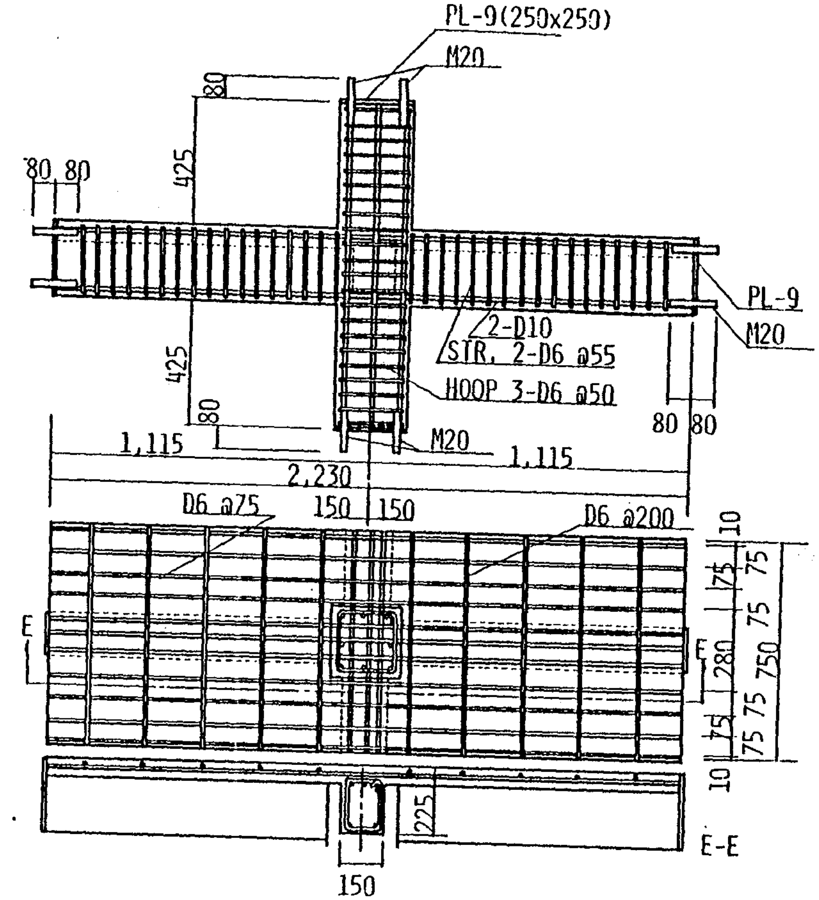
(b) Beam

Fig. 4 : Longitudinal Reinforcement in Beam-Column Subassembly Test



(Unit in mm)

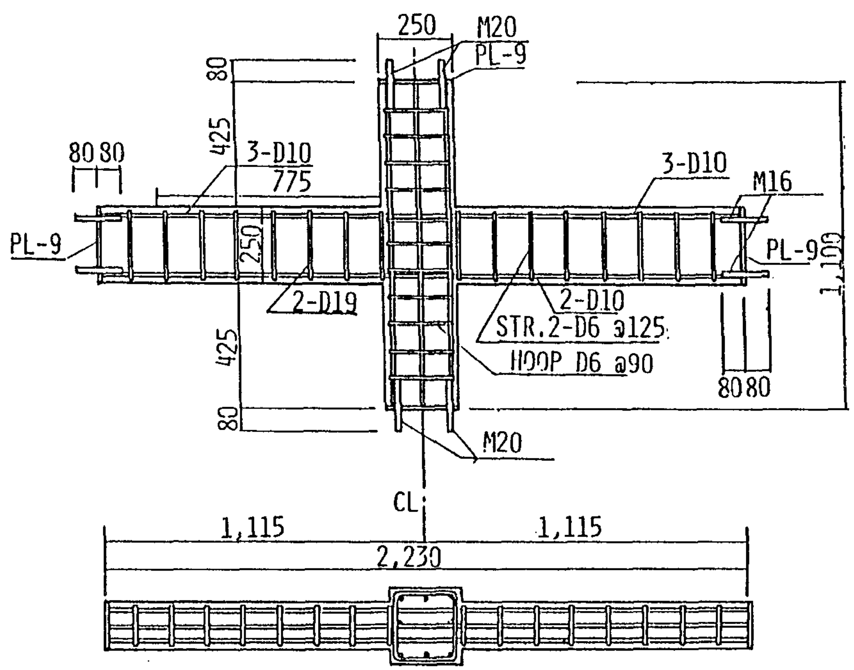
(a) Specimen I-1



(Unit in mm)

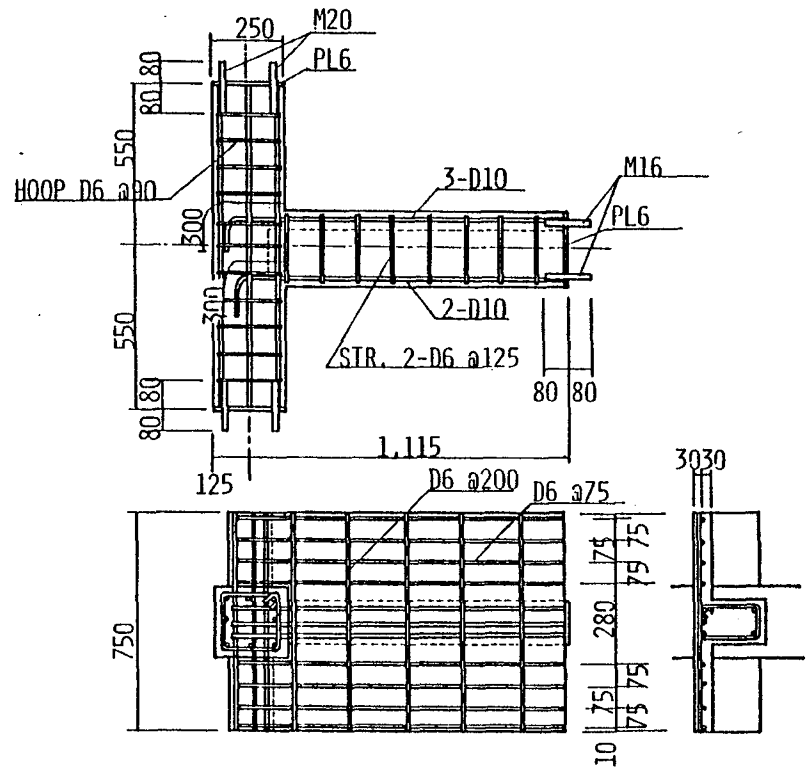
(b) Specimen I-2

Fig. 5 : Details of Test Specimens



(Unit in mm)

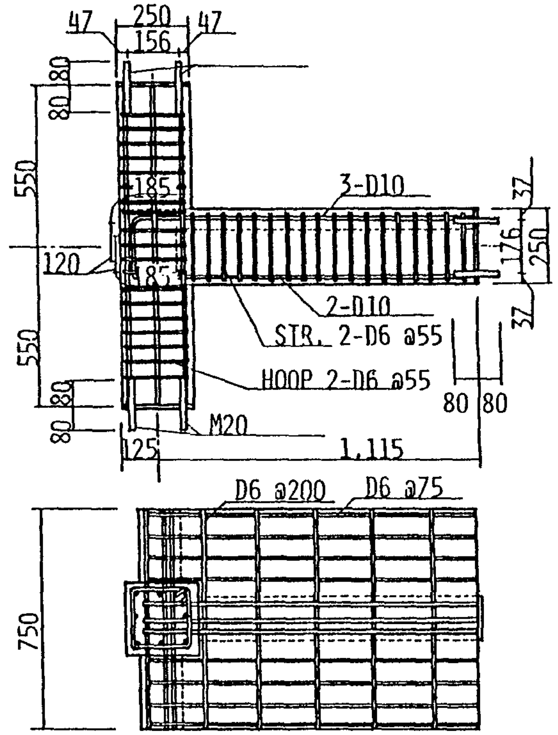
(c) Specimen I-3



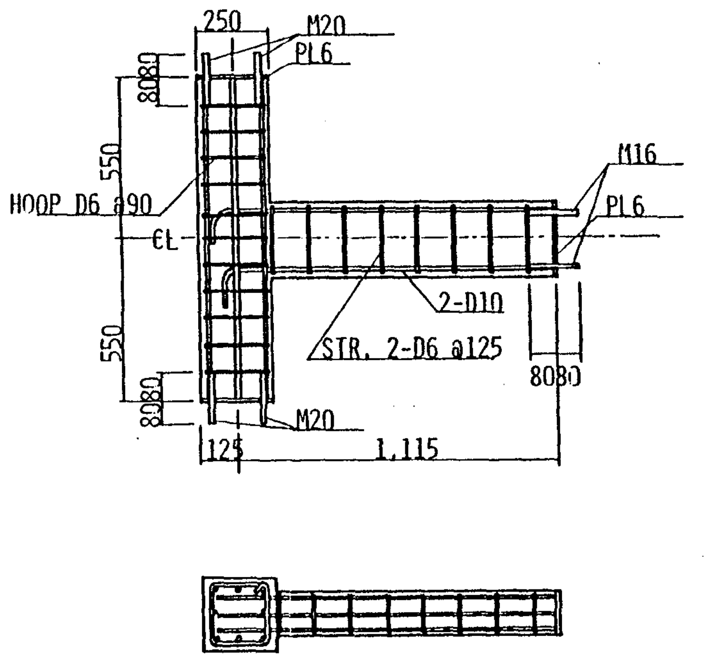
(Unit in mm)

(d) Specimen E-1

Fig. 5 : Details of Test Specimens (Continued)

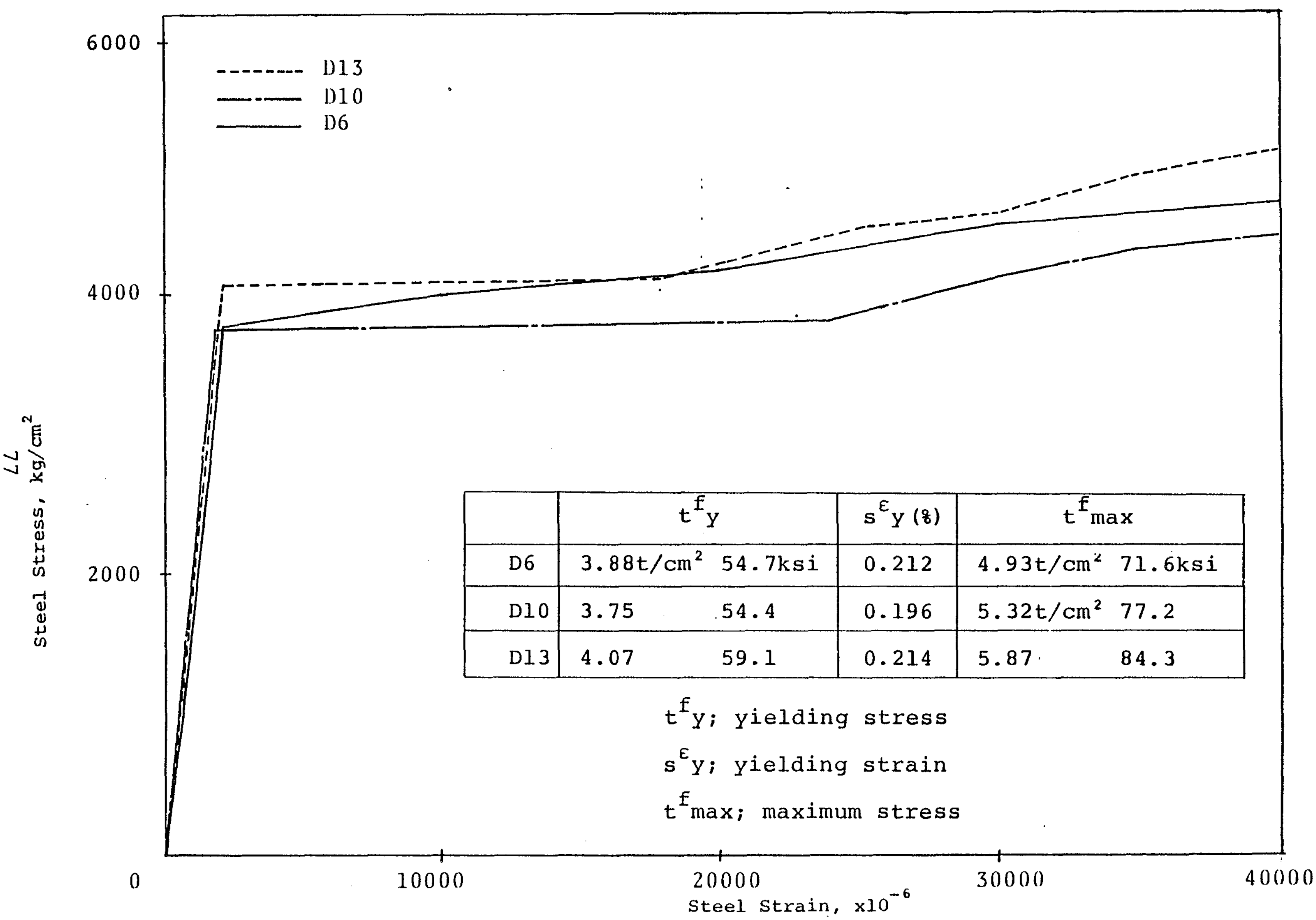


(e) Specimen E-2



(f) Specimen E-3

Fig. 5 : Details of Test Specimens (Continued)



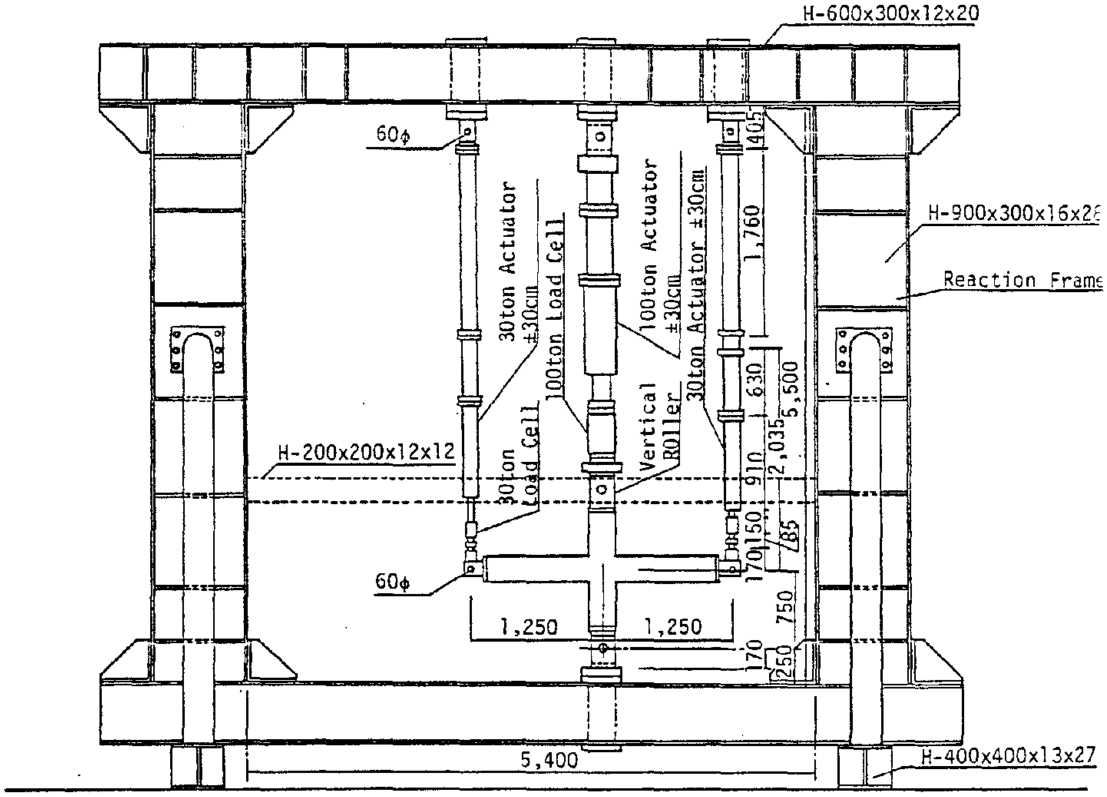
	$_tf_y$	$_s\varepsilon_y$ (%)	$_tf_{max}$
D6	3.88t/cm² 54.7ksi	0.212	4.93t/cm² 71.6ksi
D10	3.75 54.4	0.196	5.32t/cm² 77.2
D13	4.07 59.1	0.214	5.87 84.3

$_tf_y$; yielding stress

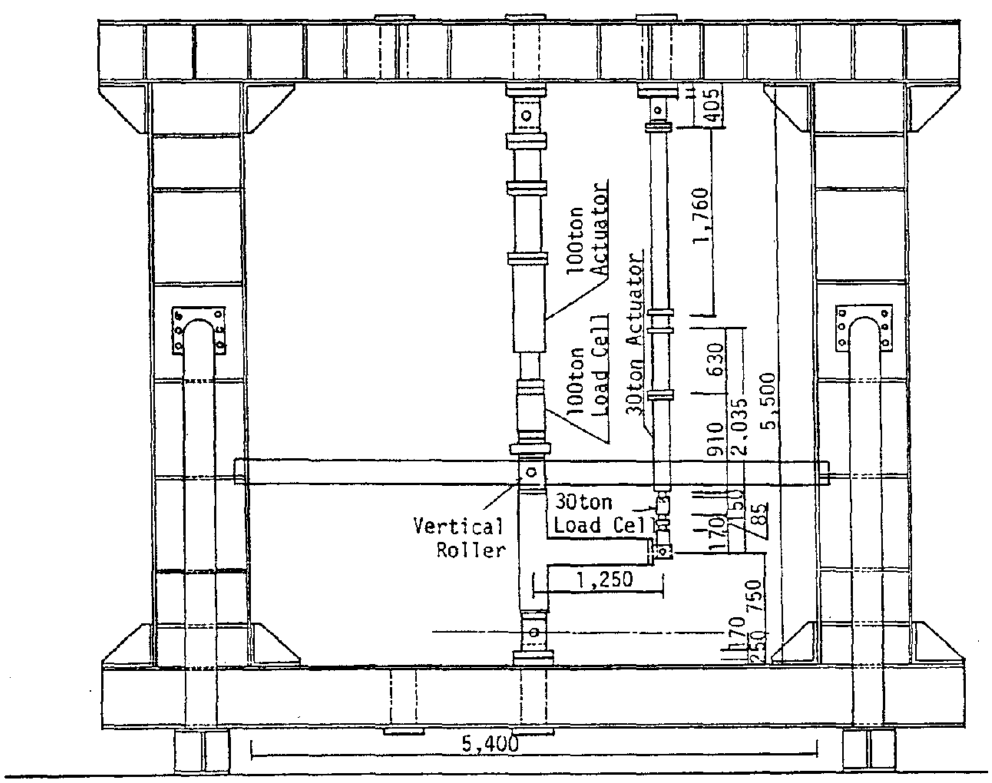
$_s\varepsilon_y$; yielding strain

$_tf_{max}$; maximum stress

Fig. 6 : Stress-Strain Relationship of Reinforcing Bars



(a) I-Series Test



(b) E and T Series Test

Fig. 7 : Loading Apparatus

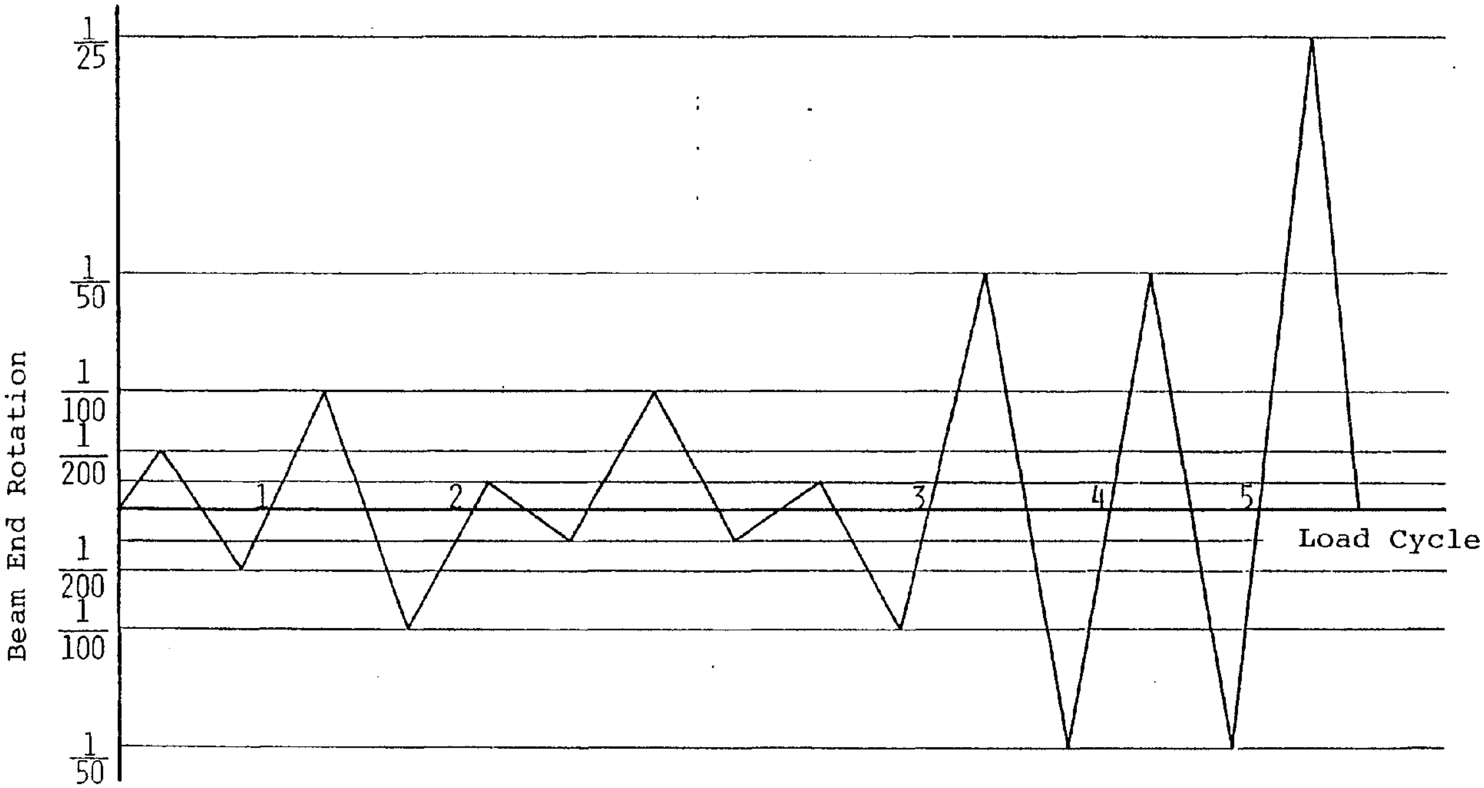


Fig. 8 : Loading Program

$$\delta\ \text{Story} = \{\delta_3 + \delta_4 + \gamma_p(\ell_b + b)\} \times \frac{h_c + a}{\ell_b + b} + \delta_1 + \delta_2 + \gamma_p\,(h_c + a)$$

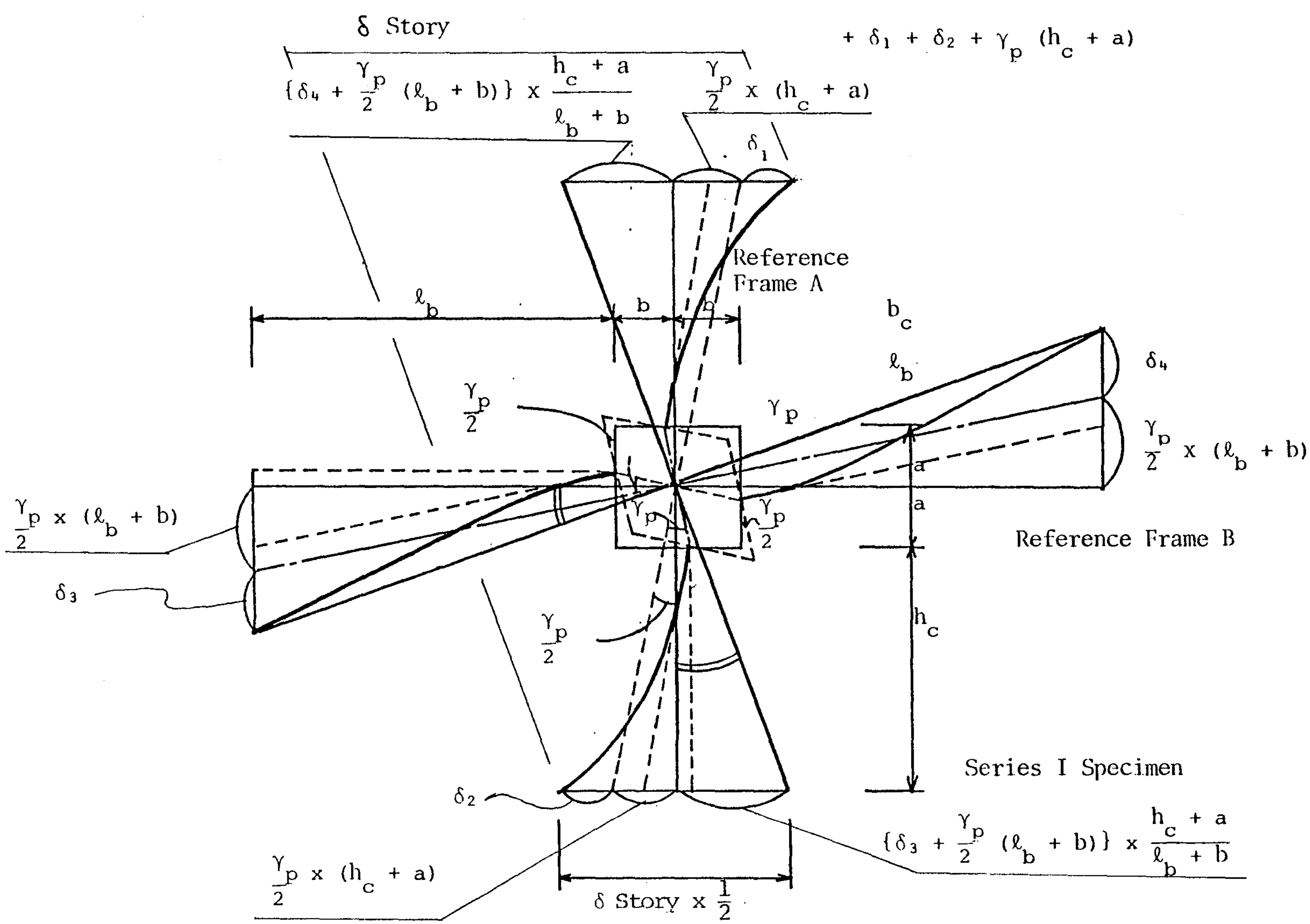
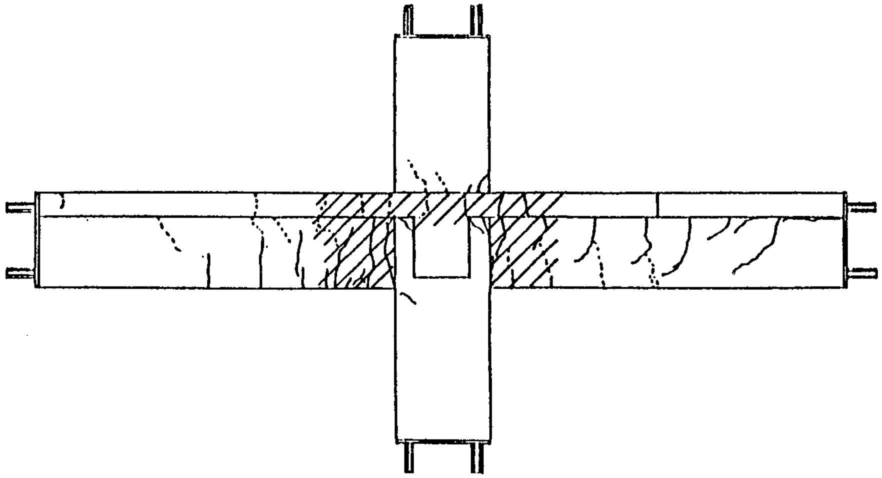
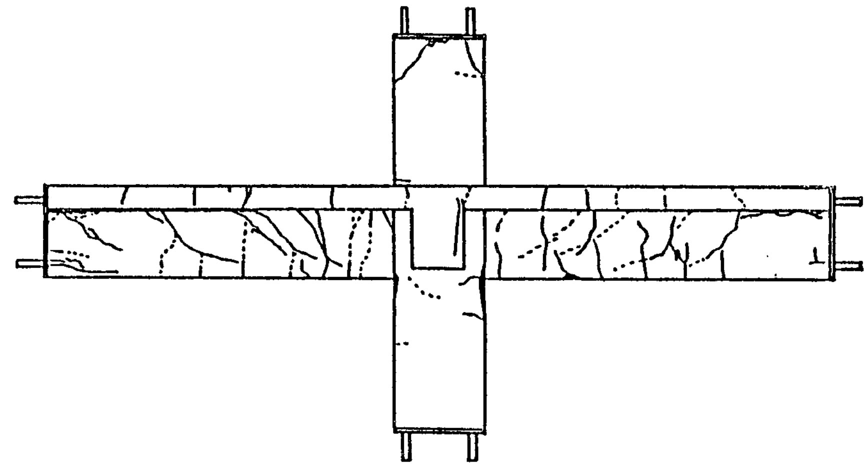


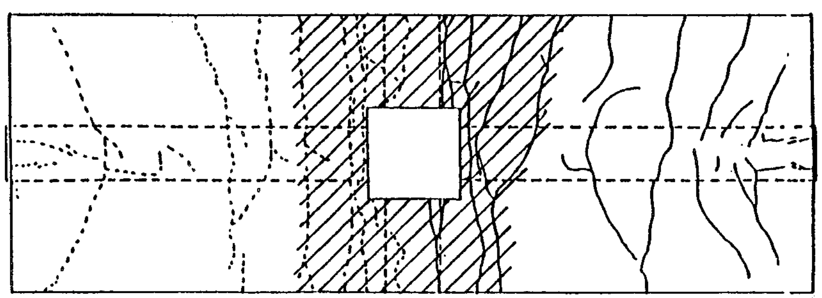
Fig. 9 : Determination of Story Displacement



1) Side View

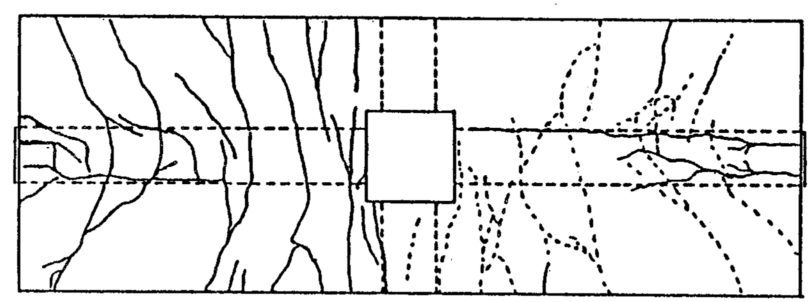


1) Side View



2) Plan View

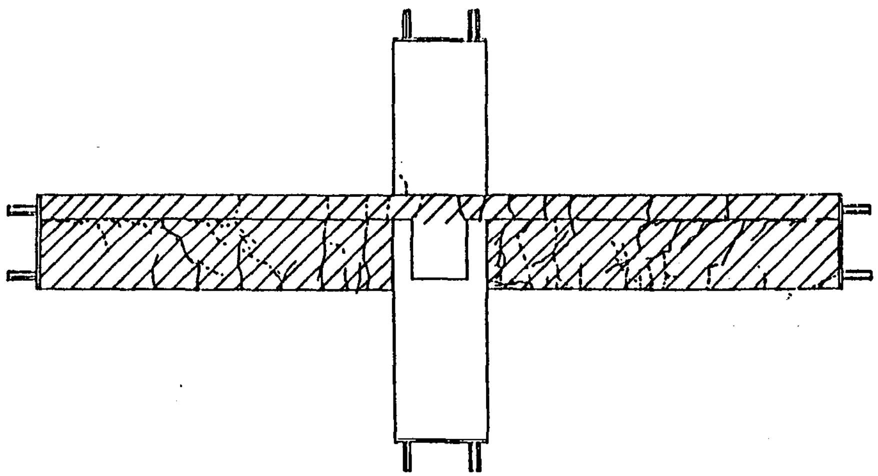
a) Original



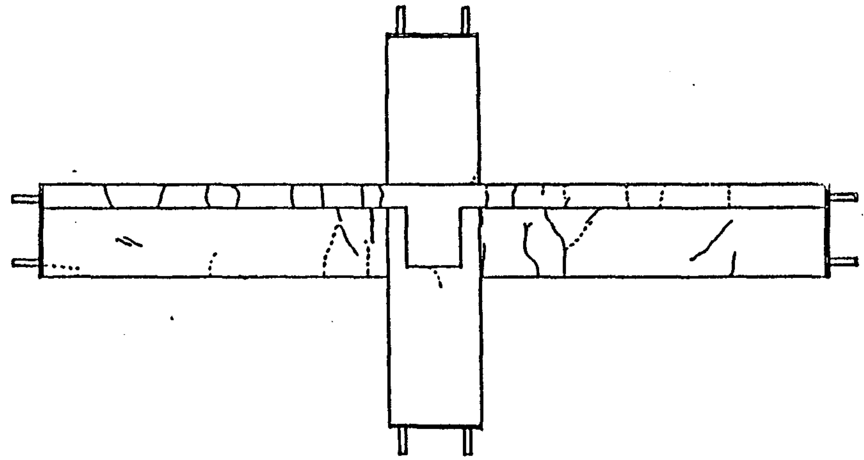
2) Plan View

b) Repaired

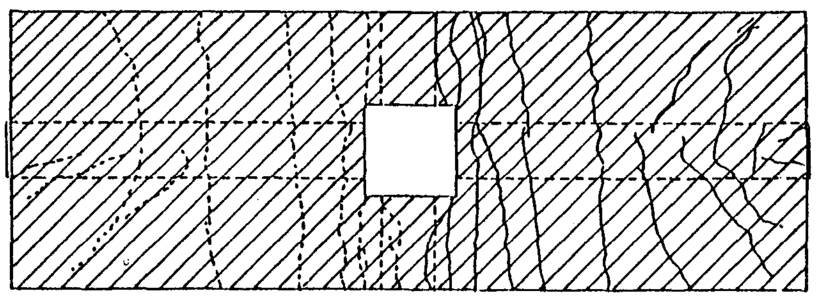
Fig. 10 Crack Pattern of Specimen I-1



1) Side View

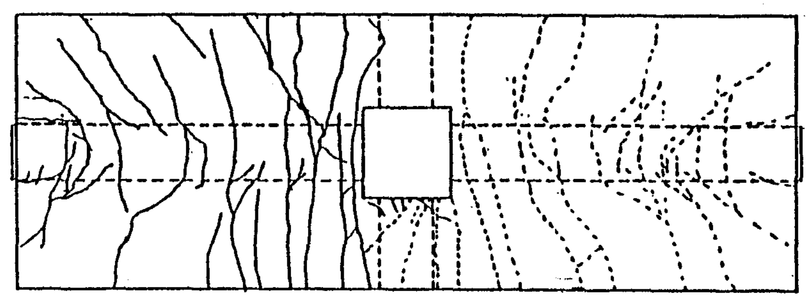


1) Side View



2) Plan View

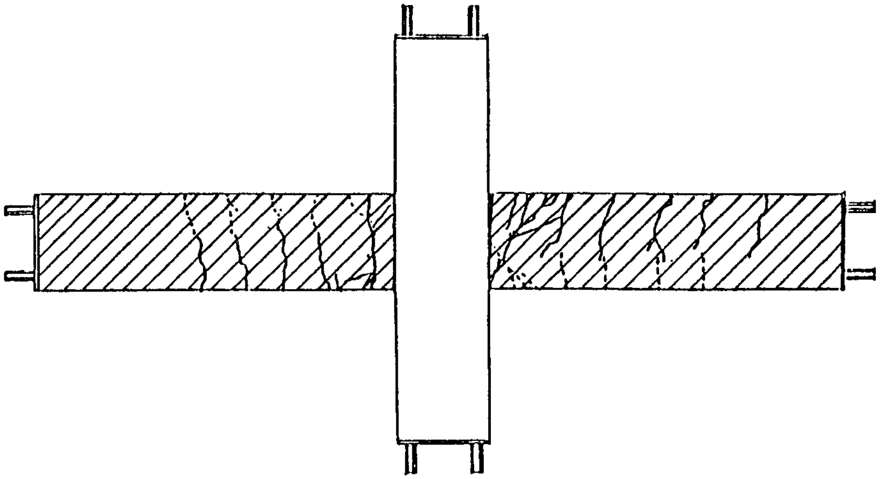
a) Original



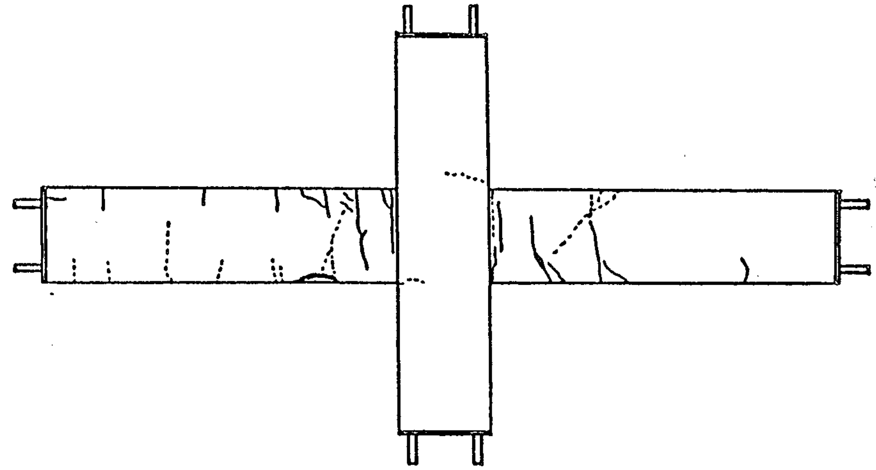
2) Plan View

b) Repaired

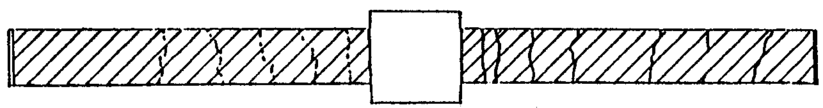
Fig. 10 Crack Pattern of Specimen I-2 (Continued)



1) Side View



1) Side View



2) Plan View

a) Original

Fig. 10 Crack Pattern of Specimen I-3 (Continued)

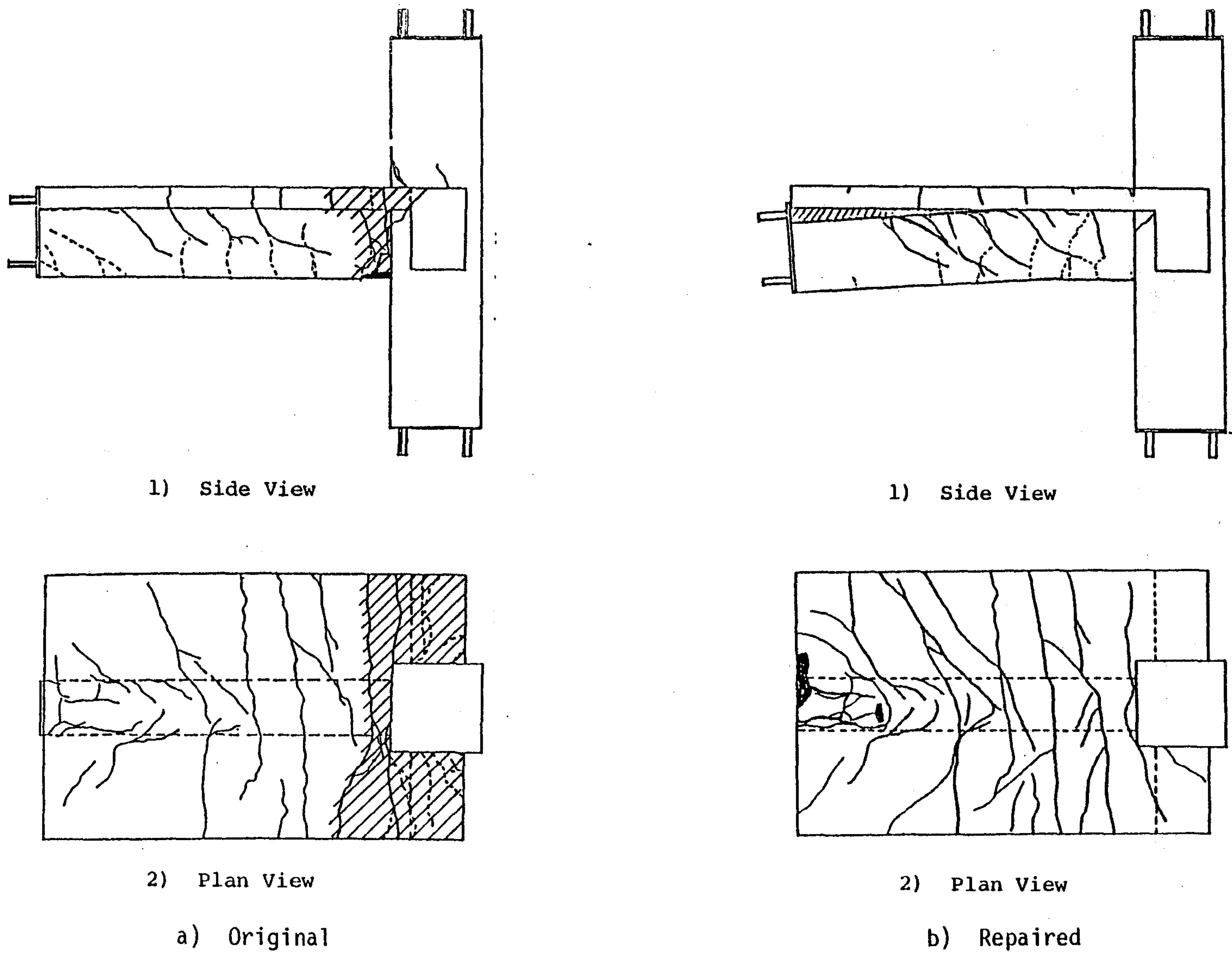
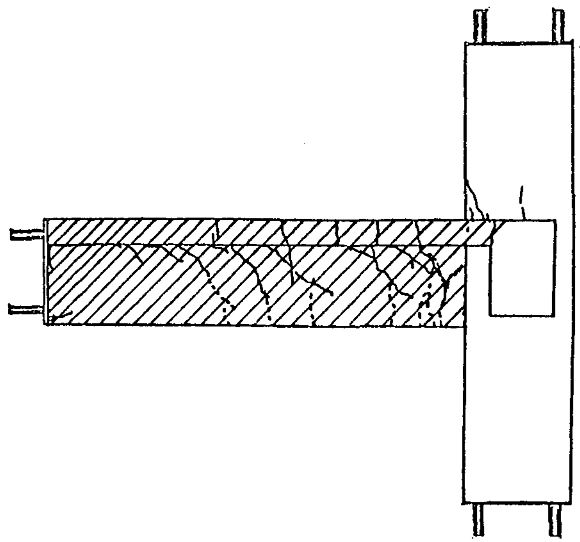
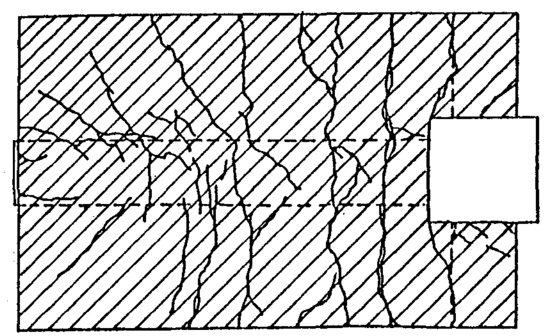


Fig. 11 Crack Pattern of Specimen E-1

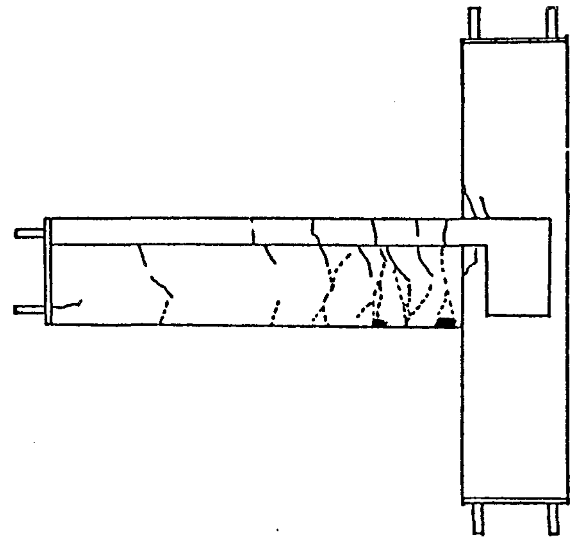


1) Side view

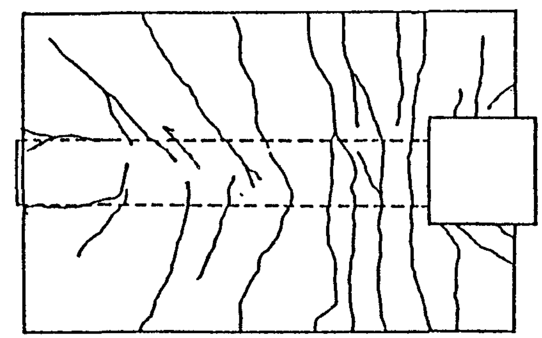


2) Plan View

a) Original



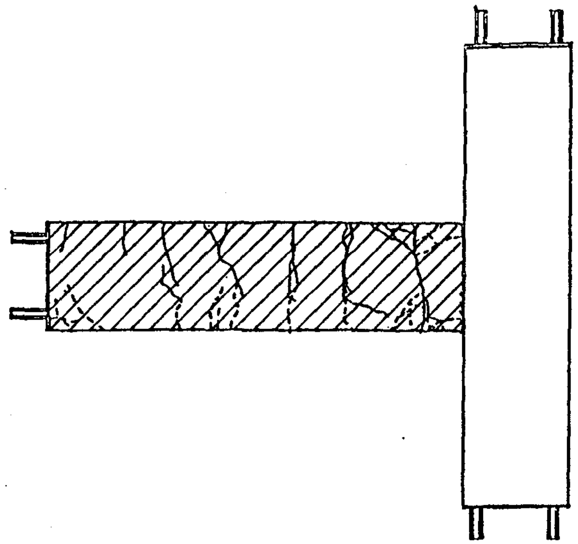
1) Side View



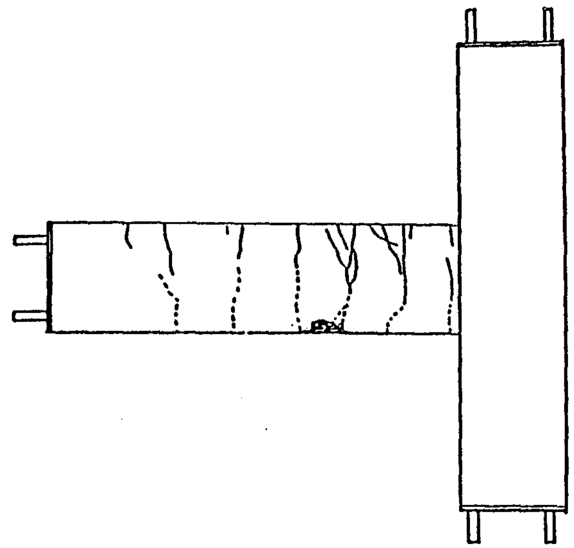
2) Plan View

b) Repaired

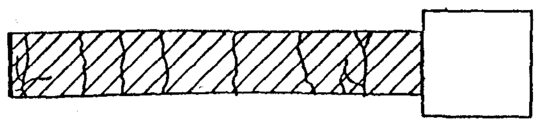
Fig. 11 Crack Pattern of Specimen E-2 (Continued)



1) Side View



1) Side View



2) Plan View

a) Original

Fig. 11 Crack Pattern of Specimen E-3 (Continued)

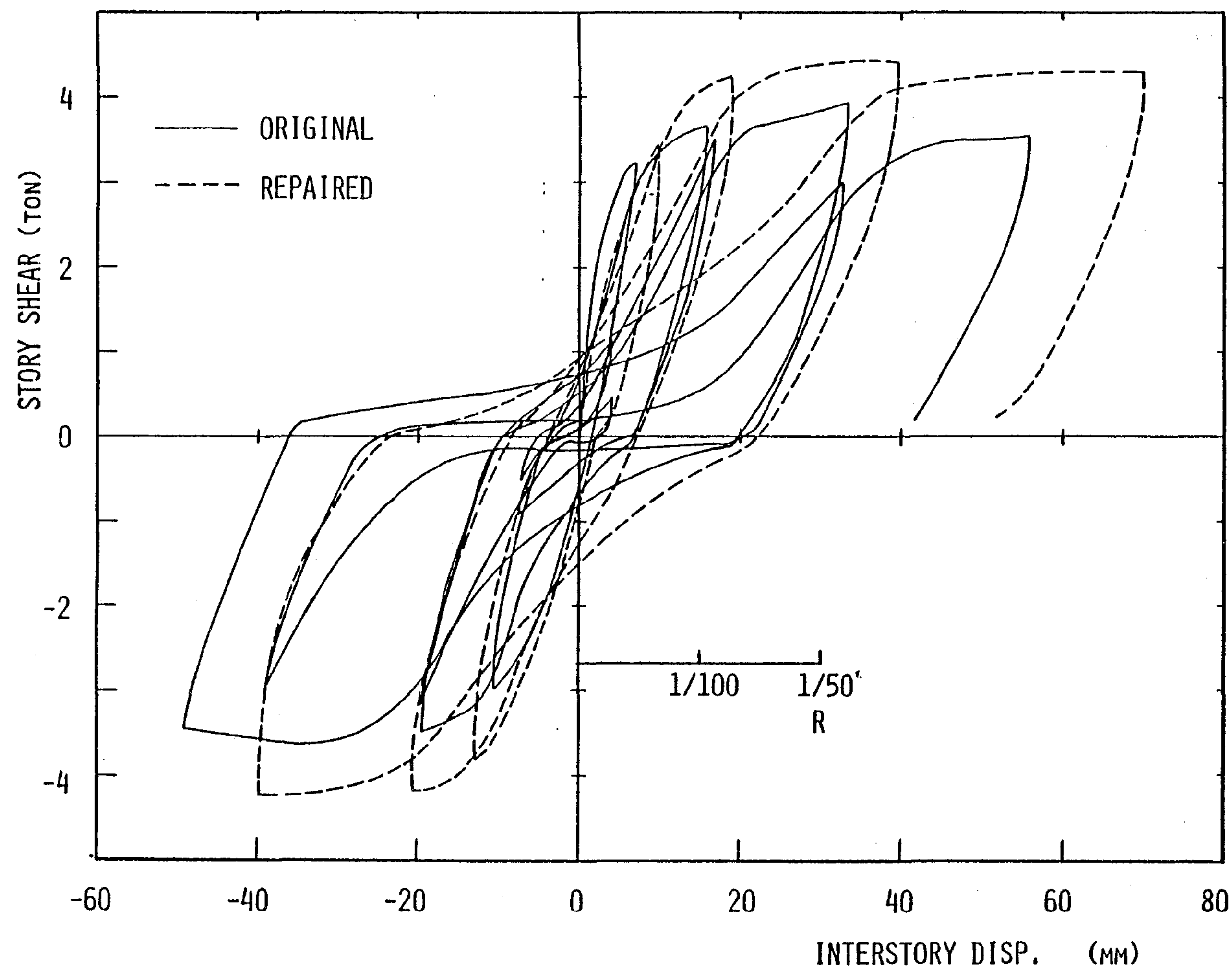


FIG. 12 STORY SHEAR - INTERSTORY DISP. RELATIONSHIP SPECIMEN I-1

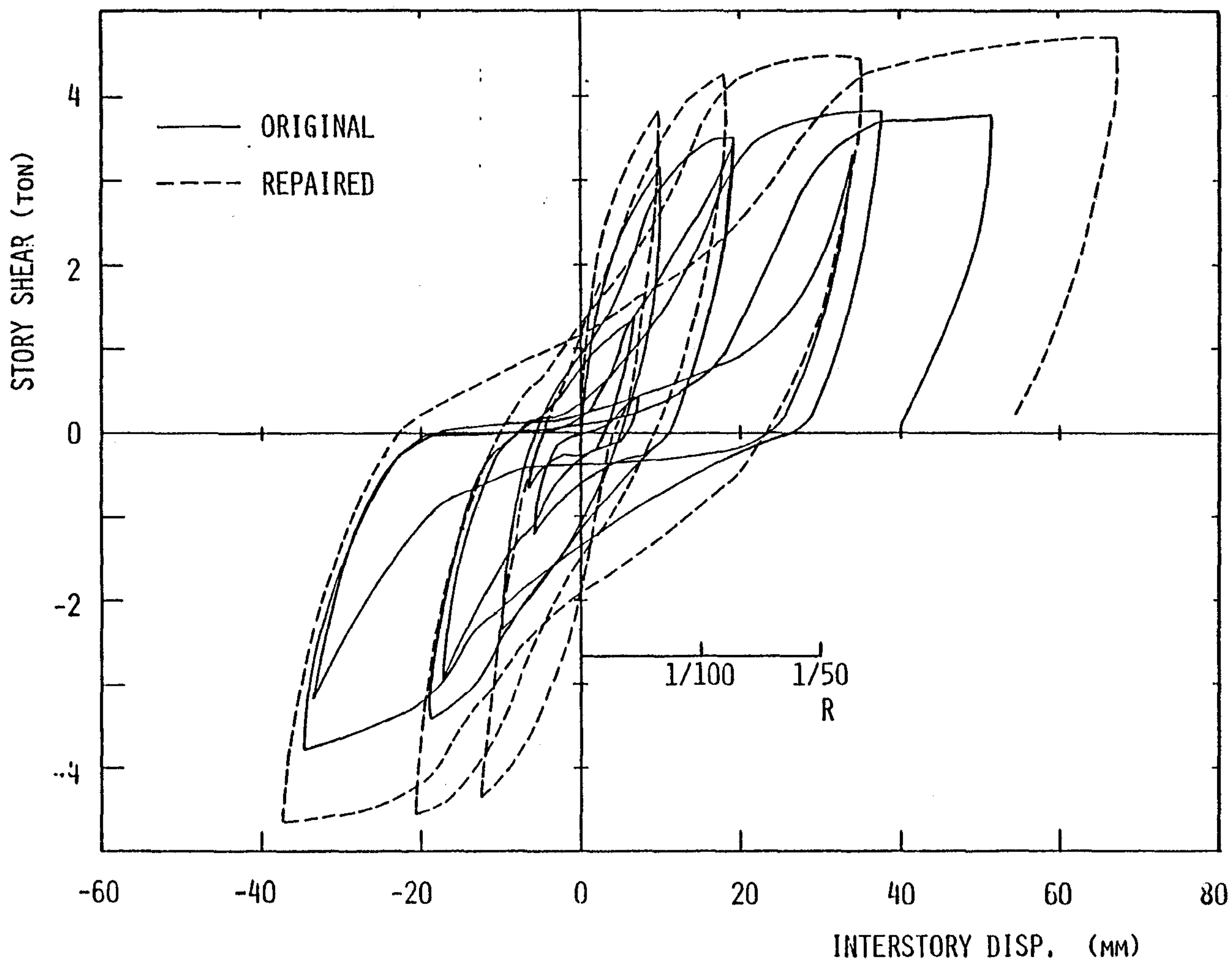


FIG. 13 STORY SHEAR - INTERSTORY DISP. RELATIONSHIP SPECIMEN I-2

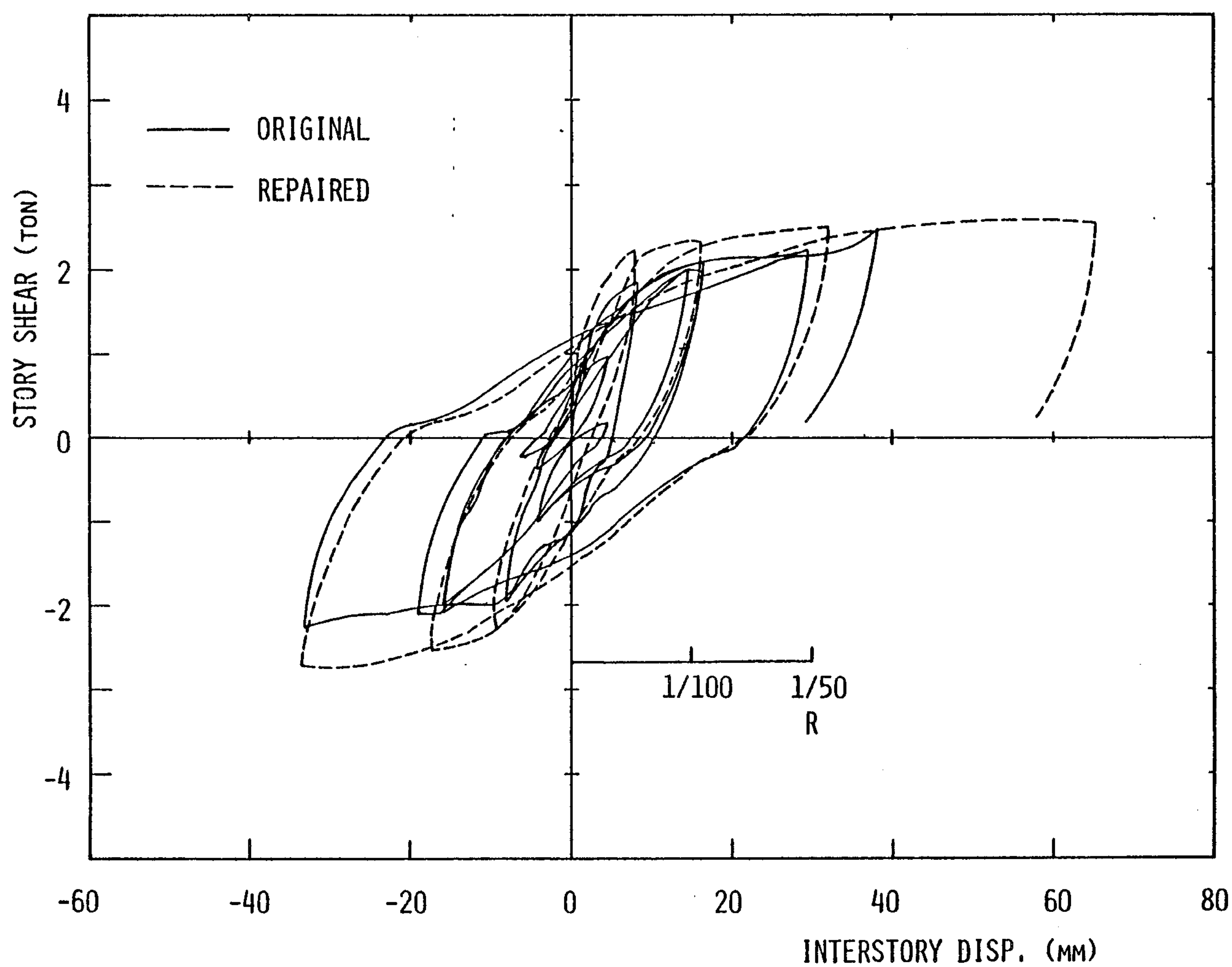


FIG. 14 STORY SHEAR - INTERSTORY DISP. RELATIONSHIP SPECIMEN I-3

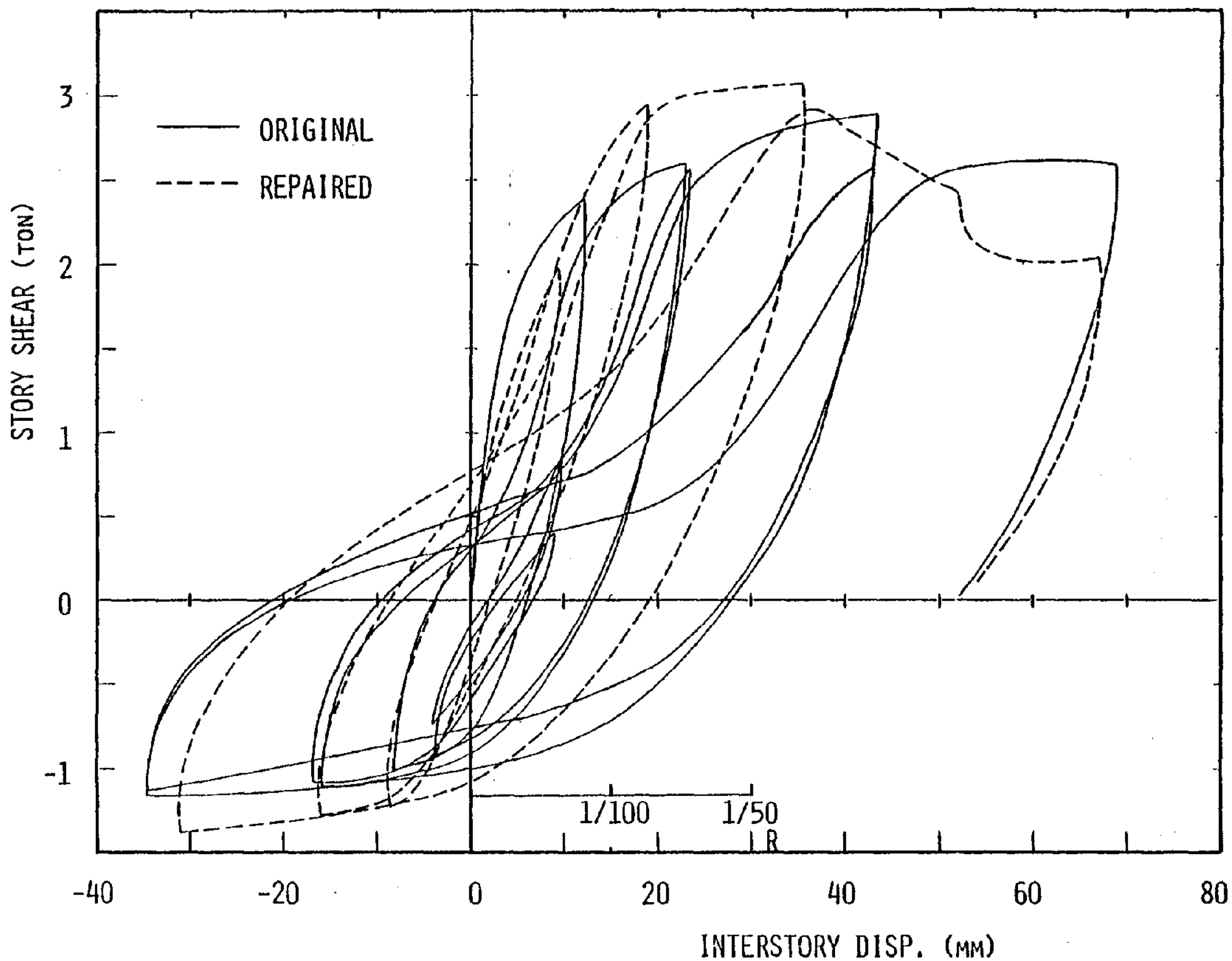


FIG. 15 STORY SHEAR - INTERSTORY DISP. RELATIONSHIP SPECIMEN E-1

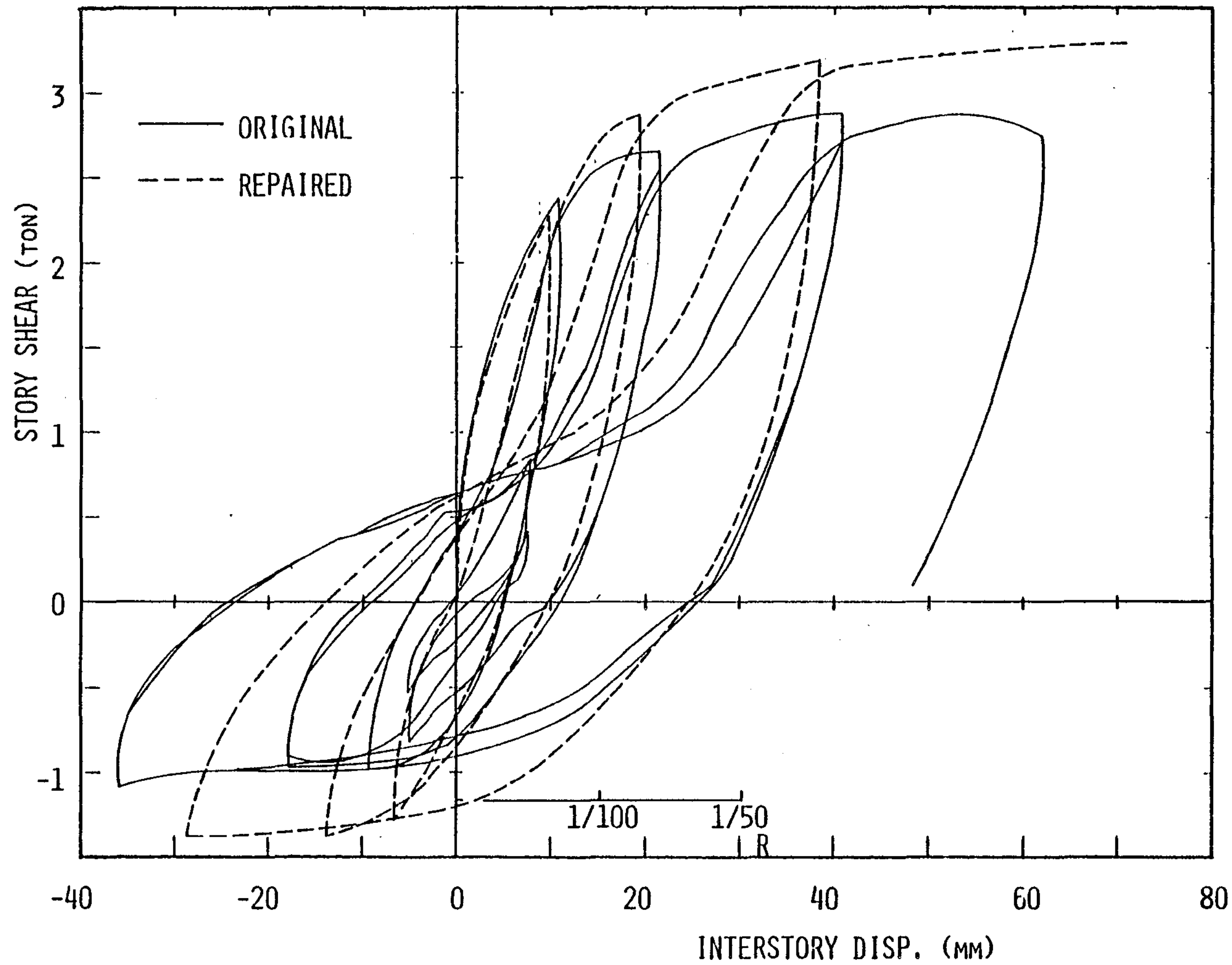


FIG. 16 STORY SHEAR - INTERSTORY DISP. RELATIONSHIP SPECIMEN E-2

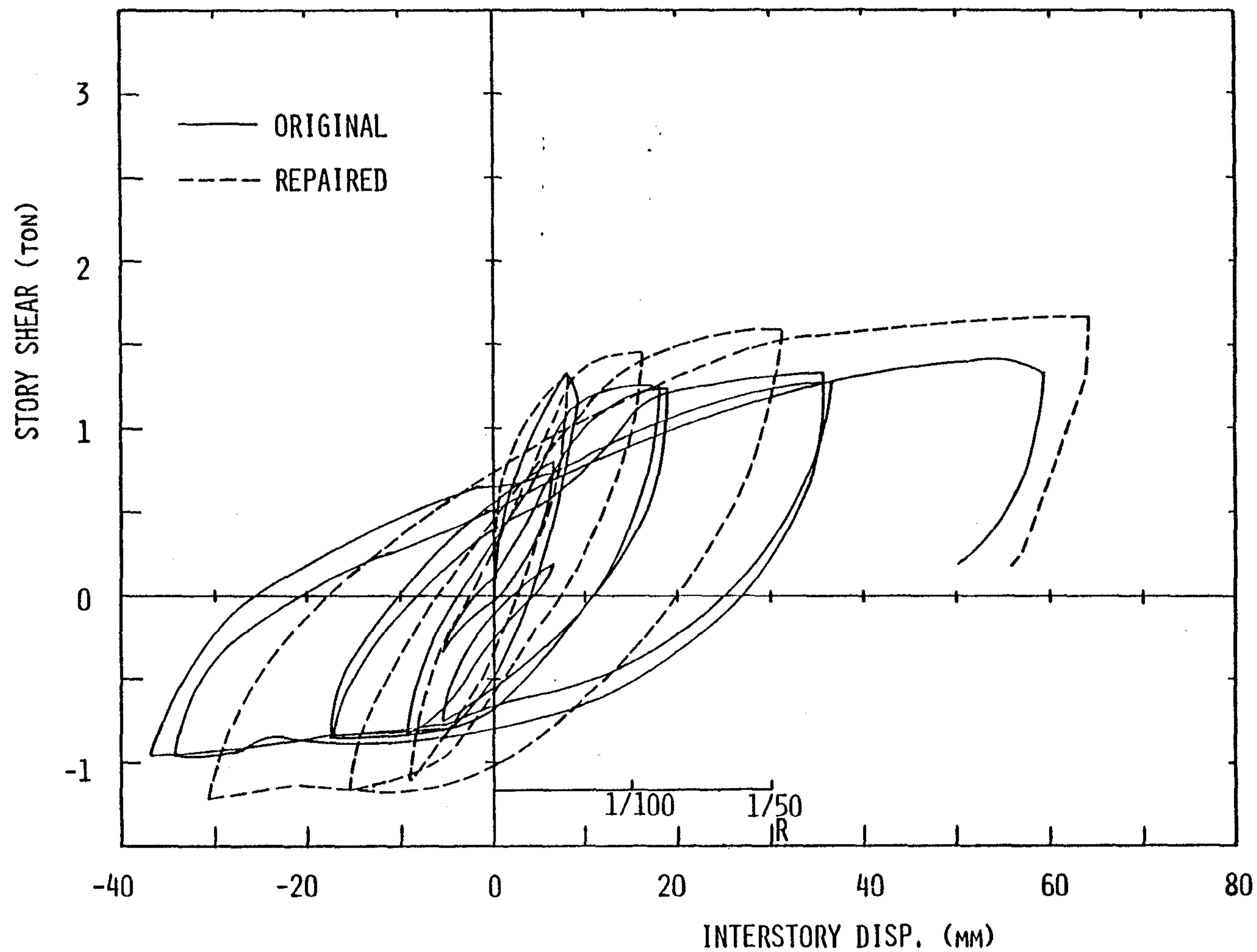


FIG. 17 STORY SHEAR - INTERSTORY DISP. RELATIONSHIP SPECIMEN E-3

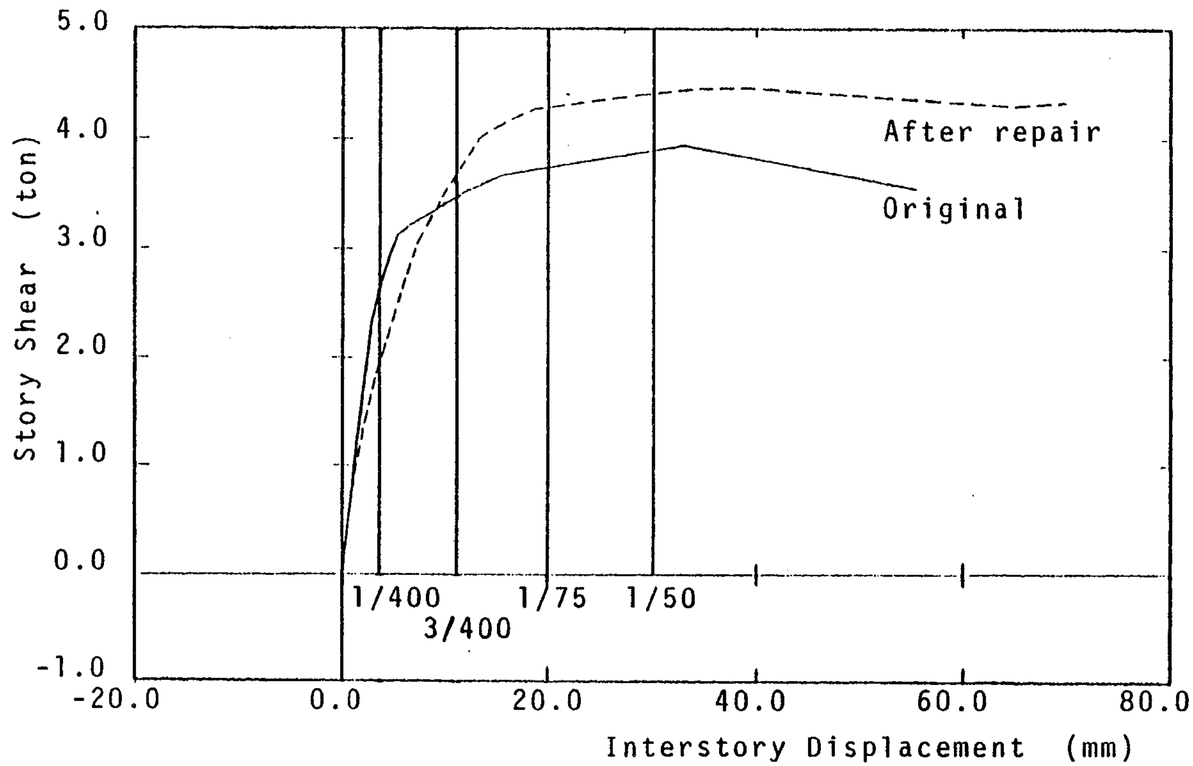


Fig. 18 Envelope Curve of Specimen I-1

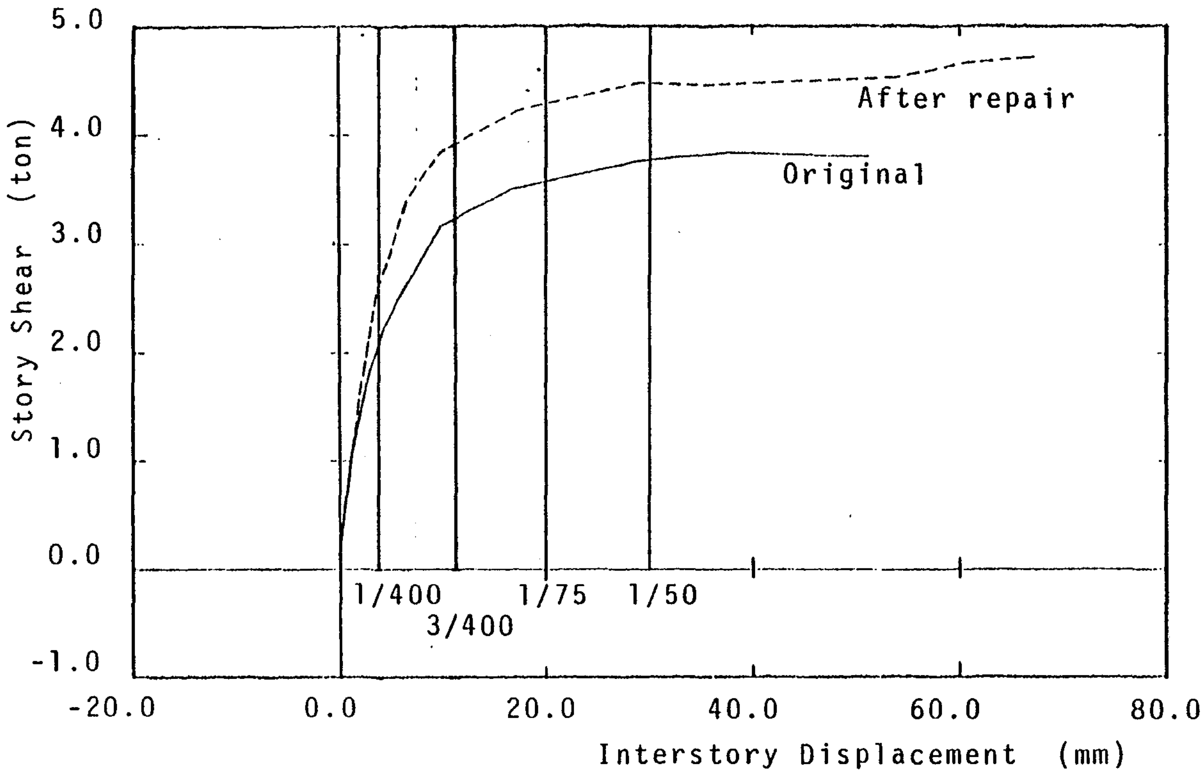


Fig. 18 Envelope Curve of Specimen I-2 (Continued)

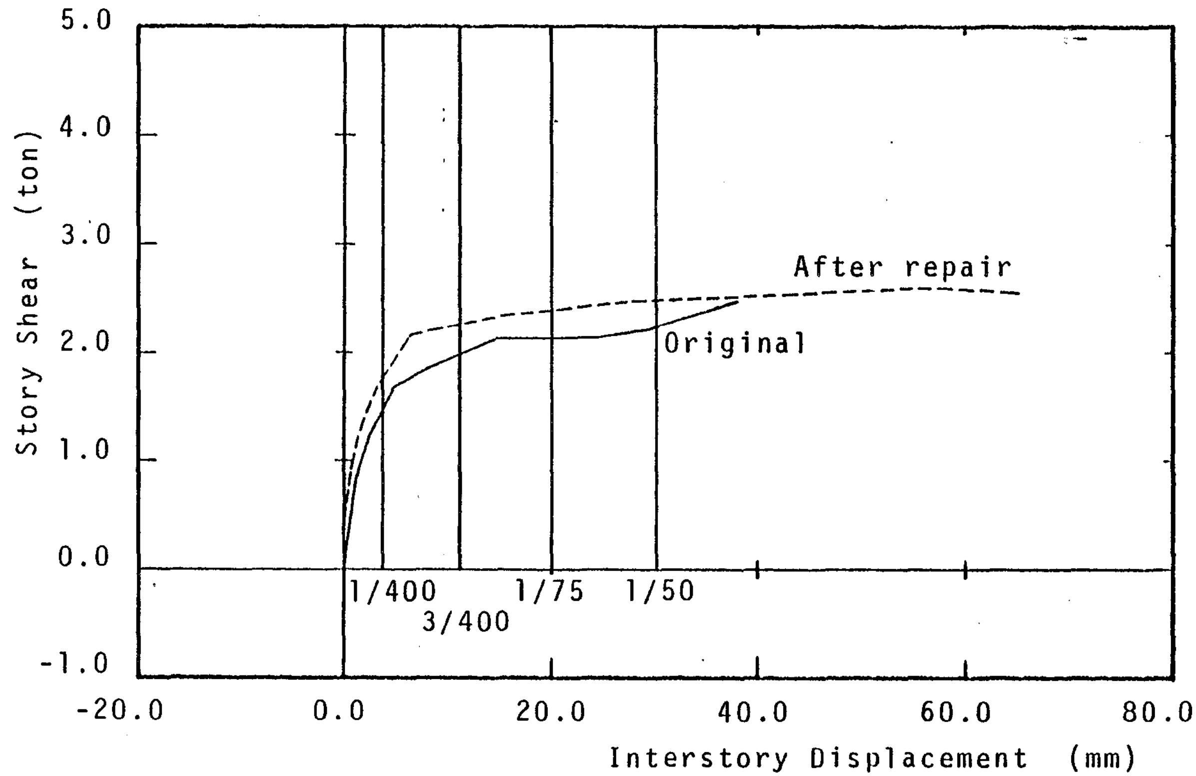


Fig. 18 Envelope Curve of Specimen I-3 (Continued)

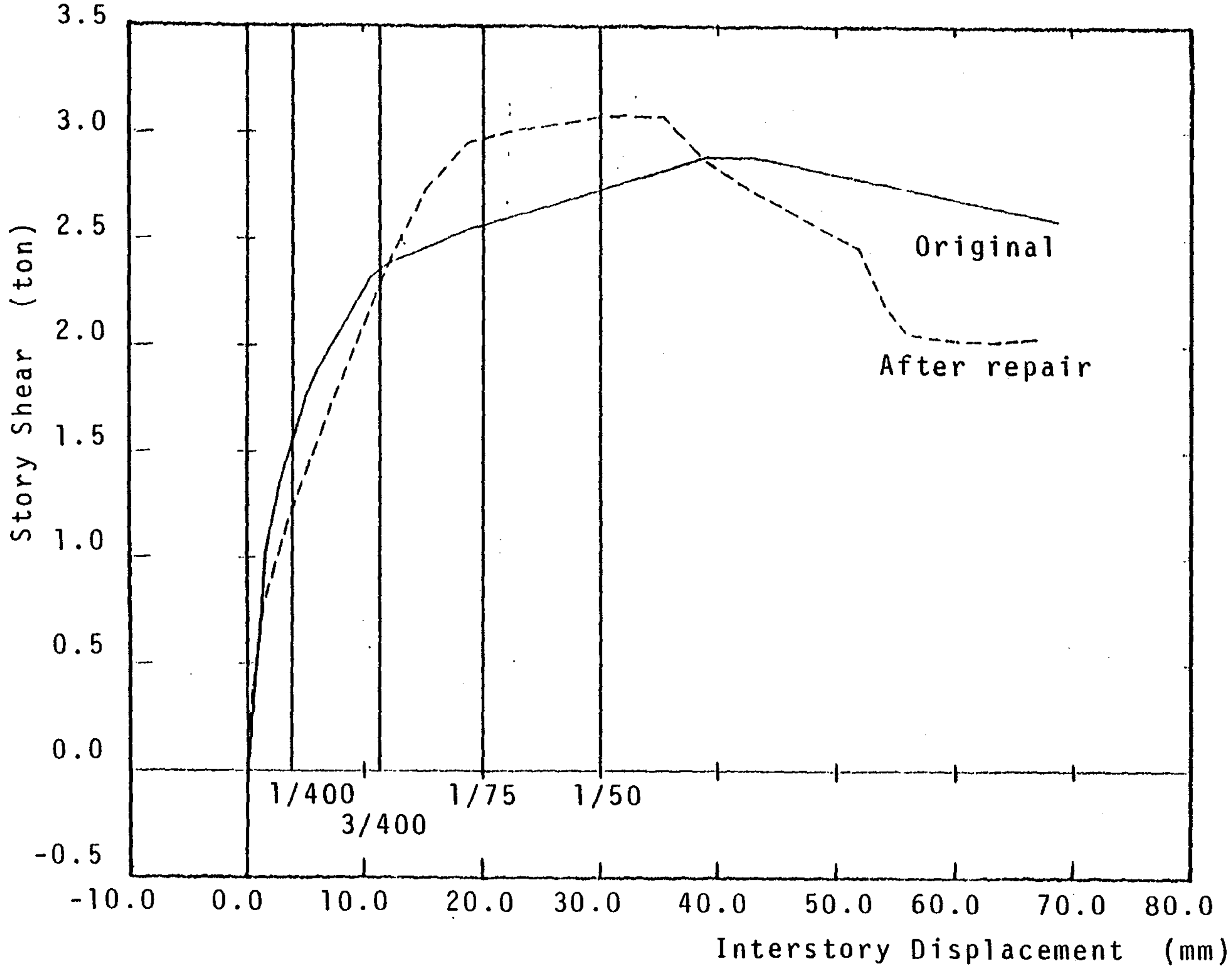


Fig. 18 Envelope Curve of Specimen E-1 (Continued)

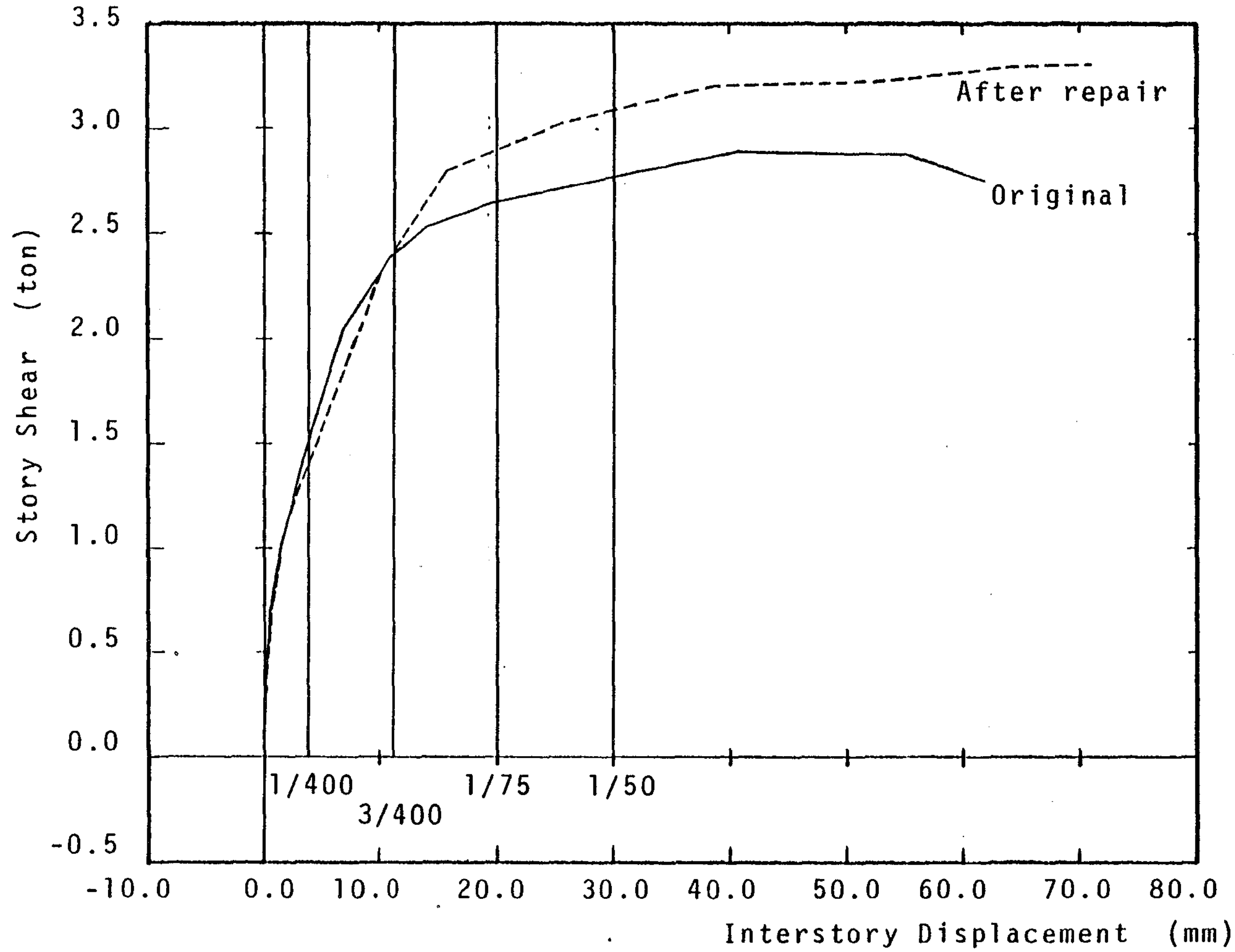


Fig. 18 Envelope Curve of Specimen E-2 (Continued)

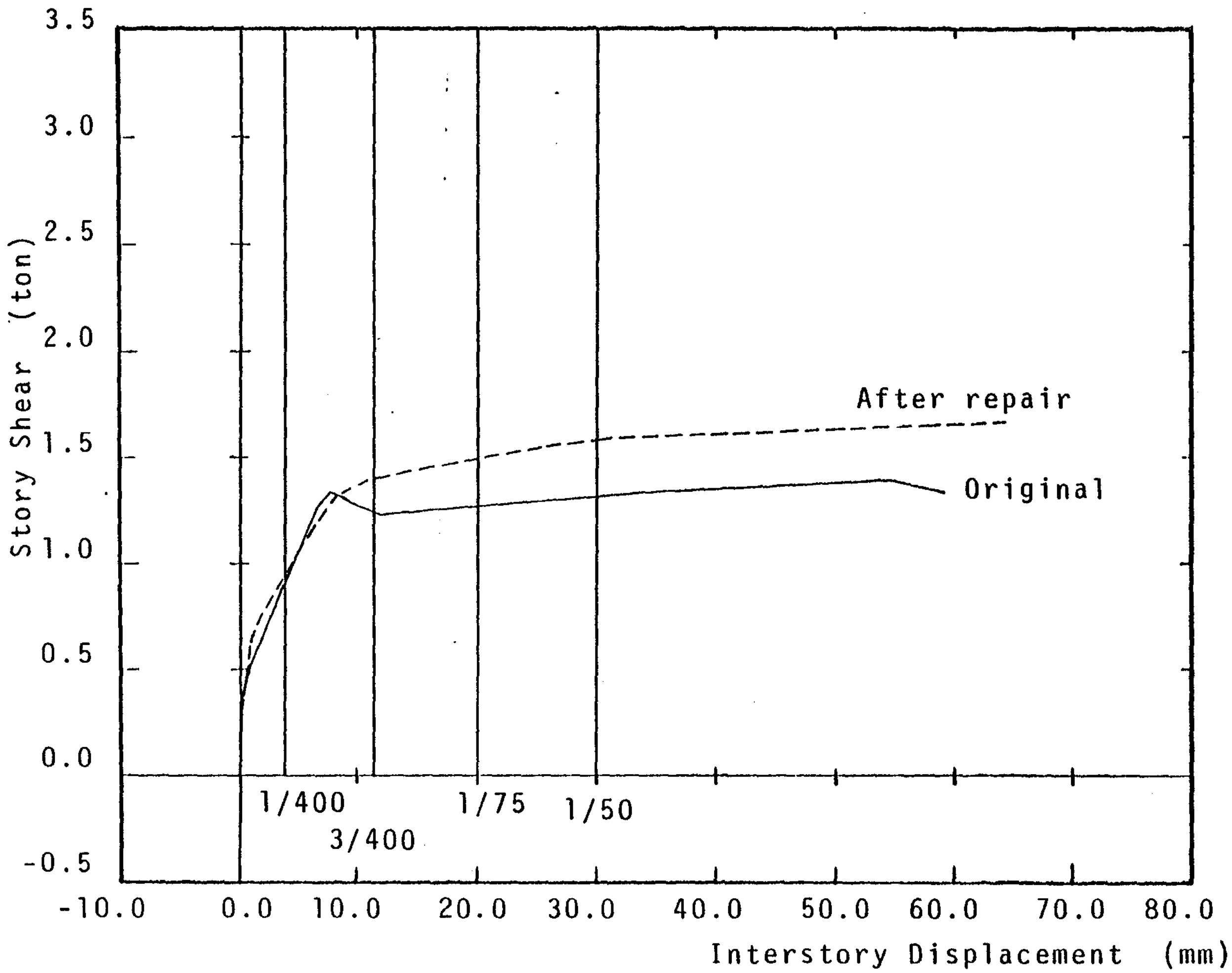


Fig. 18 Envelope Curve of Specimen E-3 (Continued)

I-Series Specimen

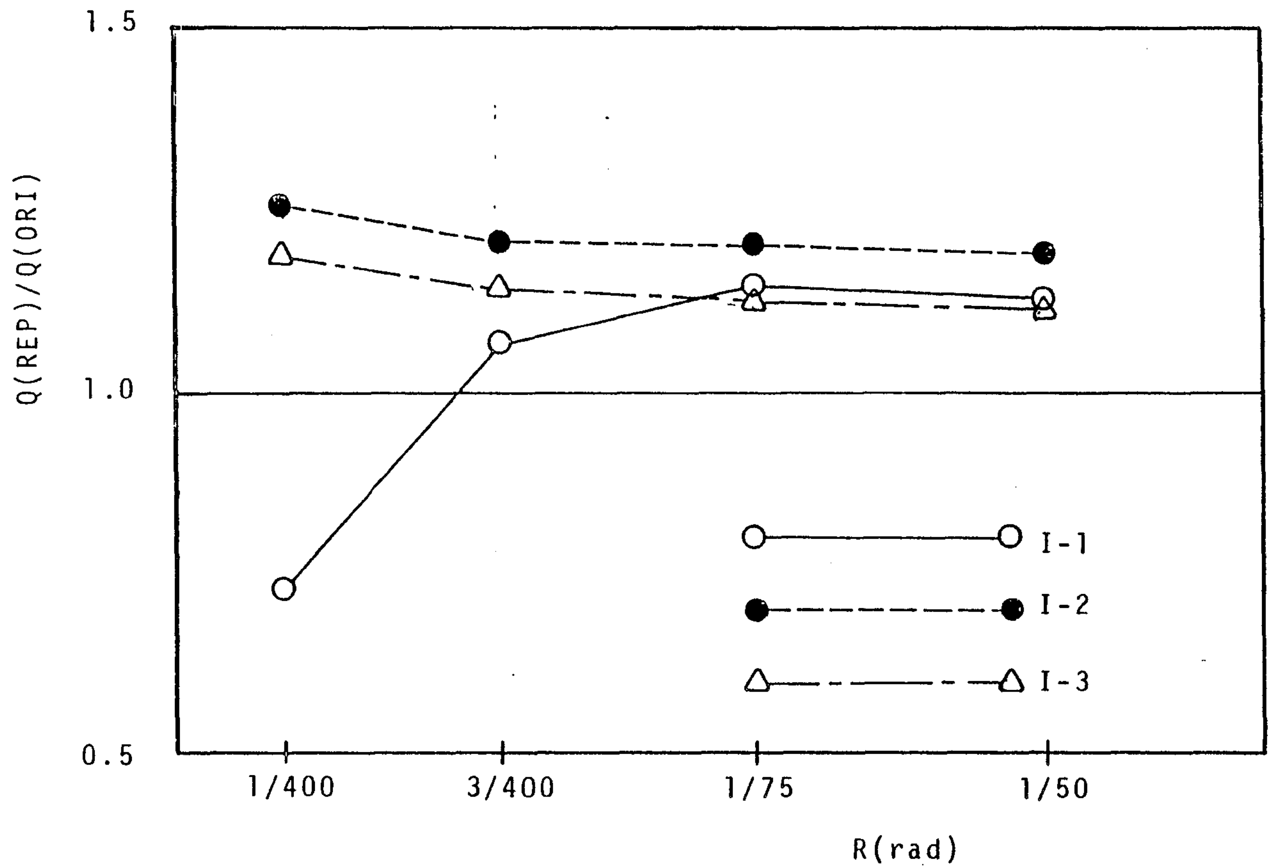


Fig. 19 Ratio of Q(REP)/Q(ORI)

E-Series Specimen

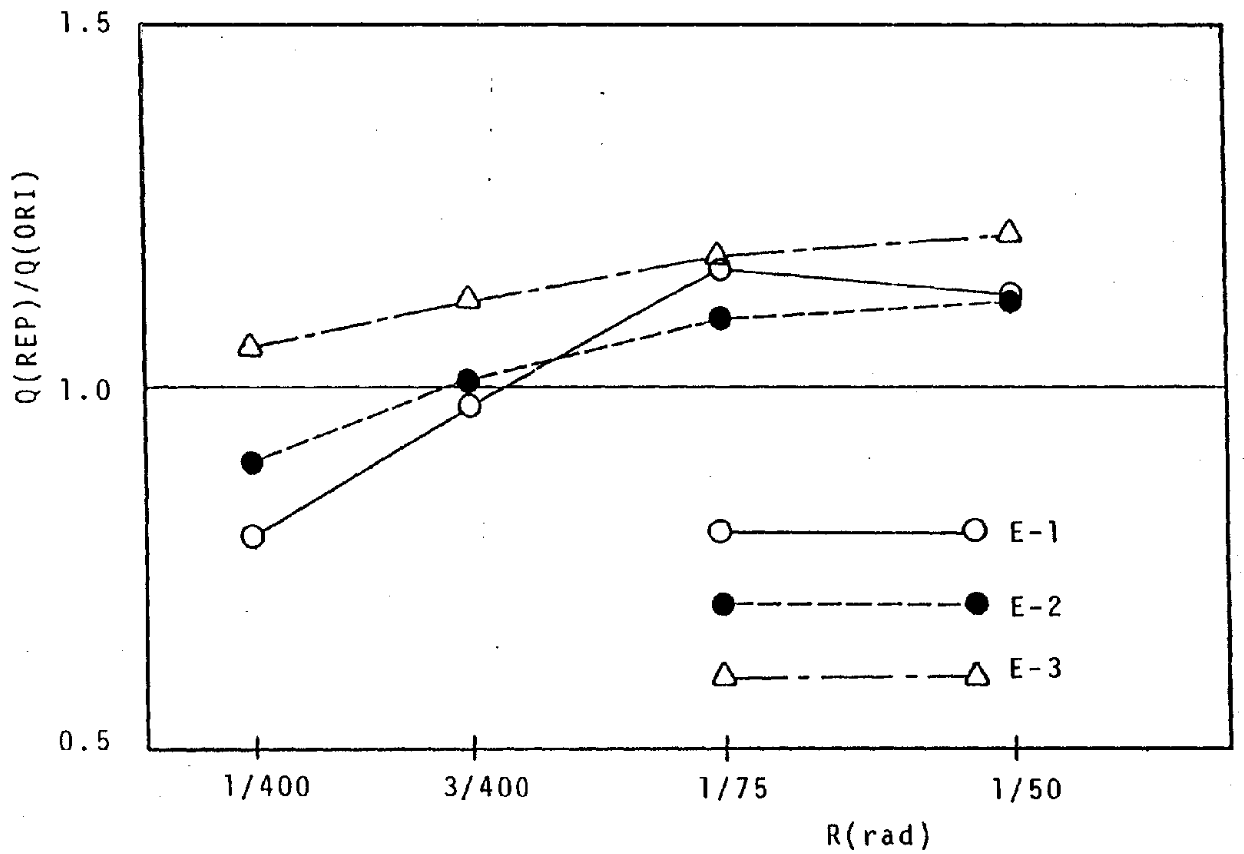


Fig. 19 Ratio of Q(REP)/Q(ORI) (Continued)

A REPAIRING TEST OF FULL SCALE SEVEN STORY REINFORCED CONCRETE BUILDING SUBJECTED TO SEISMIC LOAD

by

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SUMMARY

After the pseudo dynamic test of the full scale reinforced concrete structure, repair works and the installation of non structural elements to the damaged structure were carried out. The hinge zone of the beams and the shear wall of the structure were repaired by epoxy resin. The repair works proved to be economically effective for the reuse of the damaged structure after earthquake damage from the viewpoint of recovering in the stiffness and the strength of the structure. The aseismic arrangements of non structural elements such as partition walls, spandrel walls, window glasses, etc. were testified through a series of pseudo dynamic tests.

1. TEST SPECIMEN

Figure 1 illustrates the test model set on the test floor of the BRI Large Structure Laboratory. The model is a seven story reinforced concrete building which is 21.75m in total height and 272m^2 in floor area. The story height is 3.75m in the first story and 3.0m in the second through seventh storied. The coss section of the columns and beams is 500mm x 500mm and 300mm x 500mm respectively. The load was applied in the x direction (Fig. 1). The model has a shear wall of 200mm in thickness in the middle plane parallel to the loading (X) direction (Plane B in Fig. 1). The wall was considered to be the primary lateral load resisting element. Shear walls of 150mm in thickness were also arrayed in the exterior planes perpendicular to the loading (X) direction (Planes 1 and 4 in Fig. 1). The walls, isolated from the surrounding columns, were expected to restrain out-of-plane deformation of the model during loading.

Members of the model were designed based upon the present building specifications of both USA and Japan as well as preliminary response analysis. Since the study intended to achieve an economical design, the sections were considerably less reinforced than those conforming with US and Japanese practices. The details of reinforcement are shown in Fig. 2. In accordance with the US practice, Boundary columns attached to the shear wall were heavily reinforced in the first and second stories in order to ensure sufficient ductility of the wall. Each of closed hoops and cross ties, therefore, were arranged with a pitch of 100mm. The shear wall did not have any beam in its own plane. To validate the design, a numerical frame analysis was made by using the Degrading Trilinear model. The Miyagiken-oki earthquake in 1978 was employed for the analysis, and the resultant story rotation was 1/50 and 1/100 for the maximum acceleration of 500 and 300 gal respectively.

Concrete was mixed so that the compressive strength would arrive at $270 kg/cm^2$ after twenty eight days. Reinforcing bars of SD35, equivalent to Grade 50, were used. The material properties of the concrete and steel are tabulated in Table 1.

2. OUTLINE OF TEST BEFORE REPAIR

Table 2 shows the sequence of the entire test program. Vibration and static tests were executed twice: before and after the pseudo-dynamic test. Vibrational and stiffness characteristics of the specimen were investigated in both the intact and damage stages of the specimen.

For the pseudo-dynamic test, a total of 541 wire strain gages were glued at strategic locations on the concrete and reinforcing bars. In addition, vertical and horizontal displacements of the specimen, rotation of beam-column connections, and shear deformation of the shear wall were measured by Linear Variable Differential Transformers (LVDT's).

The specimen was fortuitously subjected to a natural earthquake with a magnitude of5.0 on January 28, 1981.

The translational natural period and damping coefficient of the specimen were obtained from the earthquake record, free and forced vibration tests, a static test, and frame analysis. Correlation among the vibration tests and earthquake record is excellent with respect to the natural period. The damping coefficient derived from the earthquake record is greater by 30 percent than those obtained from the vibration tests. It should be noted, however, that formworks for construction were clamped to the specimen when the earthquake hit the specimen. According to the forced vibration test, the natural periods of the first and second modes are 0.43 sec. and 0.11 sec. respectively, while the damping coefficient corresponding to the first mode is about 2 percent.

Before the pseudo-dynamic tests, single load application test and inverted triangular load test were carried out in order to check the dynamic properties of the structure in a elastic stage. Test 1(PSD1) was programmed in order to evaluate the accuaracy of the pseudo-dynamic testing of equivalent single degree of freedom. No cracks were observed after Test 1.

In Test 2(PSD2), the ground motion used in PSD1 was input at second time with the maximum ground acceleration of 105 gal.

The fundamental natural period was found to be 0.55 sec., which is 1.28 times longer than the elastic natural period (0.43 sec.). In the test cracks developed in the lower part of the shear wall, boundary beams, and slabs.

In the Test 3 (PSD3), the maximum displacement of 240mm at the roof level (an angle of rotation of 1/91) was attained.

The natural period was stretched to 1.16 sec. in this stage.

During the test, many shear and flexural cracks developed in the lower part of the shear wall. Concrete fragments were chipped off at shear wall-to-boundary beam junctions, and concrete started crushing at the base of boundary columns.

No new cracks were observed during the Test 4 (PSD4), but crack width had developed wider substantially (Fig.3). At the maximum displacement level, a flexural crack of 4mm in width developed at the base of boundary column, where shear cracks extended in the shear wall were more than 1mm wide. Boundary beams sustained severe damage in the test. Concrete pieces fell off at their junctions with the shear wall, and one of the reinforcing bars in the beam of the sixth story buckled severely.

During the pseudo-dynamic test in a free vibration mode, which was executed in the last stage of the test, the natural period was 1.48 sec. This natural period was three times as long as the natural period in the intact stage.

3. REPAIR OF SHEAR WALL AND HINGE ZONES

The location of the cracks in the beams were concentrated in the hinge zone.

In the shear wall, cracks were observed in the lower three stories, and crushing of concrete did not occur. For the crack repairing, epoxy resin was used. In Fig.3,a),b) and c), only the hatched portions of the cracks were injected by epoxy resin because of our limited budget. Only the column assemblies of the top of the 7th floor and the bottom of the first floor were repaired.

The physical properties of the epoxy resin is shown in Table 3. Young's modulus of epoxy resin is far smaller than that of concrete. However, the tensile strength of epoxy resin is about ten times higher than that of concrete. Table 4 shows the damage of beam ends connecting to the shear wall. Compression failure and exfoliation of concrete at almost all of the beam were observed. These exfoliated portions were pasted by epoxy mortar after being cleaned up. The physical properties of this epoxy mortar is shown in Table 5. At the four places of the total fourteen beam ends, adjacent to the shear wall, the bottom longitudinal reinforcing bars buckled. In order to reinstate these bars to original slab, three kinds of repair procedures were applied as shown in Fig. 4 according to the grade of buckling.

One of these repair procedures was to repair the buckled bars by special steel plate (6 mm thickness and 50 mm width) which was fixed by an inserted anchor bolt (Fig. 4b). At the place which was most severed buckled on the top floor level, a U-shaped stirrup bar was installed, and welded to a steel plate at the floor level for its anchor, by removing a part of the concrete floor slab. At the buckled bars, additional bars of the same size were installed by welding them to these buckled bars (Fig. 4c).

4. WORKS OF NON STRUCTURAL MEMBERS

4.1 Spandrel Wall Works

In parallel with the repair work, reinforced concrete spandrel walls were set at one span of A and C frames from the second floor to the top floor level as shown in Fig. 5. On the second and third floors, the reinforcing bars of the walls were anchored to the columns and floor slab by inserting anchor bolts. At other stories, those anchors had been set in advance. As shown in the lower left hand side in Fig. 5, connecting parts of the spandrel wall with the column of frame B is different from those parts of frame A which have a narrow width (sixty milimeters). The brick masonry spandrel walls are installed in the left hand span of frames A and C of the fifth floor level as shown in the same figure. Frame A has one centimeter clearance between the column and the brick masonry spandrel.

4.2. Partition Walls

Details of partition wall settings are shown in Table 6 and Fig.6. These walls were installed only in the third and fifth floors. The wall types used were gypsum board framed by light gage steels (GBM-1,2), gypsum lath mortar or plaster board framed by timber, cement mortar and artificial light-weight concrete board framed by light gage steel, and concrete brick masonry.

Glass windows and their frames were set on the spandrel wall in the third and fifth floors as shown in Fig. 5. These frames are composed of three glass windows, of which the central one can slide and the others are fixed. The glass is five milimeters thick, and some windows were covered with polyester films or installed wire fabric in order to prevent glass scattering caused by story drift. Two kinds of putty to fix the glass frames were used. One is hardening putty and the other is flexible, which is considered as aseismicly effective. Pipe lines for the water supply were also installed through all stories. However the test results are not yet available.

5. TEST RESULTS AFTER REPAIRING

5.1 Behaviour of the Structure

The sequence of pseudo dynamic tests for the full scale test building after repairing was planned to be quite the same as those before repairing. They included four steps: modified Miyagi Oki earthquake N-S direction 1978 maximum 23.5 gal and 105.0 gal, modified Taft E-W direction 1952, max. 320 gal and Hachinohe E-W direction max. 350 gal. Each test series is respectively named as Test 5,6,7 and 8. However, in the actual test series, Test 8 was changed to a static loading test of uniform loading distribution because of the problem of the actuator capacity. Figs. 7 and 8 show response time histories of horizontal displacement at the top story in Tests 5 and 6.

The solid and broken lines are the response before and after reparing respectively. The response after repairing is larger than the one before repairing. After 0.75 seconds of this test procedure, the free vibration test under the pseudo dynamic test system was continuous. The natural period of this structure was 0.57 second, i.e. 1.27 times larger than the natural period of 0.45 second in the intact situation which obtained just before the first test. The relationships between total shear force versus the top story displacement are shown in Figs.9 and 10. After repair, the initial stiffness was smaller than that before repair. The top floor displacement at two hundred ton total shear force in the first test series was about thirty milimeters. However, it increased to about fifty milimeters after repairing.

Figs. 11, 12 and 13 show the damages of the pure frame (Frame A), the wall fram (Frame B) and of floor slab of the full scale structure respectively in Tests 2 and 6. The damage grade of the shear wall after repair (Test 6) was more severe than these before repair (Test 2). In Fig. 11, many cracks are observed in the pure frame after repair because the crack repair works by epoxy resin were done only at the beam end portions after Test 4. As shown in Fig. 10, the maximum response at the top floor level after repair was larger than these before repair. It was considered that this difference depended upon the discrepancy between the stiffness of original structure and these of repaired structure. The peak-to-peak-stiffness of the hysteresis loop before repair is more than two times larger than after repair in Fig. 10.

Response time history and restoring force characteristics in Test 7 (320 gal input) are shown in Figs. 14 and 15 respectively. In this test series, maximum response is about twenty centimeters, which is almost same as the response before repairing for the same input earthquake in spite of the degradation of structure stiffness. The maximum shear capacity of this structure is about 450 ton, which can be covered by the calculation based on the plastic behaviour of a full scale structure.

The crack patterns of the structure in Test 7 are shown in Figs. 16, 17 and 18. The cracks in the shear wall after repair were observed as same as those before repair.

The concrete is exfoliated by compressive failure at the beam lower ends connecting to shear wall before repair. However, after repair works as shown in Fig. 4 the exfoliation of concrete was not observed in Test 7 as in the same earthquake input as in Test 3. The number of cracks at floor slab after repair did not increase compared with those before repair as shown in Fig. 18.

Thest 8 was conducted by static loading distributed over each story uniformly. Alternative loading reversals are shown in Fig. 19. This test is controlled by the top floor level displacement angle (R), that is the horizontal displacement at the roof divided by the total height of the test structure.

During the load cycle R; 1/75, the shear wall of the first story was extremely damaged, then this final loading test terminated.

At the final stage of this test, the main reinforcements were broken out, and the concrete is exfoliated along the full span of this first story shear wall (Figs. 20, 21 and 22). Fig. 23 shows the hysteresis loops of horizontal displacement to the floor. Maximum shear capacity became almost six hundred tons under uniform loading for each story.

5.2 Behaviour of Non Structural Elements

Test results of non structural elements are now being compiled. Here only the general damage to partition walls and window glasses are reported. Fig. 24 shows the damage of partition walls. This damage is concentrated around door openings, and appeared remarkably at the stage of 1/200 story drift angle. And the trouble of doors opening occurred in a earlier stage of the earthquake input load.

Fig. 25 shows one example of progressive injury in window glasses and spandrel walls. The damage of window glasses depends upon the degree of fixing to the window sash. The glasses whose frame was fixed by hardening putty were broken in an earlier stage of the earthquake input level. On the other hand in the case of frames fixed by flexible putty, the destruction of glasses occurred in a later stage (story drift angle: 1/100).

6. RESULTS OF VIBRATION TEST

The vibration tests were carried out in the same way as those before repair. The observed date of the natural period of the test structure through these tests are shown in Fig. 26. The values of the natural period from the free vibration test and the forced vibration test are almost same. The natural period reduced to about 0.6 sec. after repair works. This shows that the stiffness of the structure was recovered by repair works. And the natural period became 0.5 sec. by the installation of non structural elements. The natural period got from the pseudo dynamic free vibration test was larger than that from other vibration tests. This indicates that the natural period of the structure depends upon the displacement smplitude in vibration.

7. CONCLUDING REMARKS

This paper reports the test of a full-scale reinforced concrete building conducted at the Building Research Institute, Ministry of Construction. Various test programs were carried out in order to investigate the seismic characteristics of the building. These programs included vibration test, static test and pseudo-dynamic test. Major findings and areas of further research are summarized as follows:

1. The fundamental natural period of the specimen - 0.43 sec. in the intact stage - was lengthened in accordance with the level of damage that the specimen sustained. At the end of Test PSD4, by which time the specimen had undergone severe damage, the natural period was 1.48 sec., more than three times as long as the initial natural period. After the repair works the natural period covered to 0.50 second.

2. The frame analysis was found to adequately simulate the static behavior of the specimen. According to the test and its analysis, the shear wall resisted in its elestic range about 55 percent of the total shear force applied to the fifth floor.
3. The maximum base shear carried by the specimen was 440 ton. On the other hand, the maximum base shear computed by means of the plastic hinge method was 429 ton, which is 98 percent of the experimental maximum base shear. After repair works the test structure kept the shear capacity as same as these before repair.
4. Dynamic analysis of a one-mass system succeeded in sumulating the behavior of the specimen under the pseudo-dynamic test. We are continuing the effort to correlate the response of the equivalent SDOF system with the true response of the specimen.
5. After repair works and the installation of non structural elements for the test specimen, the repair works in reasonable expense for earthquake damage were testified to be economically effective. The test resutls of non structural elements gave the precious materials for better asseismicity of non structural elements in future.

Table 1 Material Properties of Concrete and Steel

(a) Mechanical properties of concrete

Before Test (PSD 1)

	Fc (kg/cm^2)	cε_B (%)	E_0 ($\times 10^5$ kg/cm^2)	$E_{1/3}$ ($\times 10^5$ kg/cm^2)	Fsp (kg/cm^2)
1st STORY	289.4	0.218	2.72	2.37	24.2
5th STORY	294.5	0.210	3.08	2.54	23.6

After Test (PSD 4)

	Fc (kg/cm^2)	cε_B (%)	E_0 ($\times 10^5$ kg/cm^2)	$E_{1/3}$ ($\times 10^5$ kg/cm^2)	Fsp (kg/cm^2)
1st STORY	283.8	0.222	2.22	2.13	23.8
5th STORY	291.5	0.219	2.29	2.14	24.6

Fc : Compressive strength of field cured $10^{\phi}\times 20$ cylinder
cε_B: Strain at compressive strength
E_0 : Initial tangent modulus
$E_{1/3}$: Secant modulus at one-third of compressive strength
Fsp: Splitting strength

(b) Mechanical properties of reinforcing bars

	σ_y (t/cm^2)	σ_u (t/cm^2)	ε_y (%)	ε_{st} (%)	ε_u (%)	Es ($\times 10^6$ kg/cm^2)
D 10	3.87	5.67	0.210	1.80	16.55	1.84
D 13	3.93	5.65	0.211	——	19.31	1.86
D 16	3.85	5.72	0.221	1.94	17.46	1.74
D 19	3.65	5.73	0.214	1.68	19.84	1.71
D 22	3.53	5.75	0.191	1.23	21.42	1.85
D 25	3.78	5.66	0.201	2.18	19.70	1.88

σ_y : Yield stress
σ_u : Maximum stress
ε_y : Yield strain
ε_{st} : Strain hardening strain
ε_u : Elongation
Es : Elastic modulus

Table 2 Test Sequence & Description of Programs

Test No.		Contents
VT 1	Free & forced vibration tests	
FLL 1	Single load application tests	
SL 1	Static tests under inverted triangular load distribution	
PSD 1		$\delta max=\pm 3mm (Rmax=1/7000)$ MIYAGIKENOKI TOHOKU U. NS Gmax=23.5gal
PSD 2		$\delta max=\pm 55mm (Rmax=1/400)$ MIYAGIKENOKI TOHOKU U. NS Gmax=105gal
PSD 3		$\delta max=\pm 163mm (Rmax=3/400)$ 1952 TAFT EW Gmax=320gal
PSD 4		$\delta max=\pm 290mm (Rmax=1/75)$ TOKACHIOKI HACHINOHE EW Gmax=350gal
FLL 2	Single load application tests	
VT 2	Free & forced vibration tests	
Repairs by epoxy injection		
VT 3	Free & forced vibration tests	
Arrangement of non-structural elements		
VT 3 FLL 3 SL 2	Vibration tests Single load application tests Static tests under inverted triangular load distribution	
PSD 5-7	Pseudo-dynamic tests as SDOF system (1/7000 - 3/400)	
SL 3	Static tests under uniform load distribution (Rmax=1/50)	

δmax : The target maximum displacement at roof floor

Rmax : The target maximum angle of rotation δmax/H
(H : Total heigt of the speciwen)

Gmax : The maximum acceleration of input ground motion

Table 3 Physical Properties of Epoxy Resin

Items	unit	Test Value	
Specific Gravity		118	JIS K 5911
Viscosity	c.p.s	340	BH (20°C)
Pot Life	minute	19	(20°C-500g)
Hardness	shore D	D-87	ASTM D 2240 (20°C-7)
Tensile Strength	kg/cm	527	JIS K 6911 (20°C-7)
Compressive Strength	kg/cm	922	JIS K 6911 (20°C-7)
Tensile Shear Strenght	kg/cm	144	JIS K 6850 (20°C-7)
Impact Strength	kg.cm/cm	107	JIS K 6911 (20°C-7)
Bond Strength for Cement Mortar	kg/cm	36	(20°C-7)

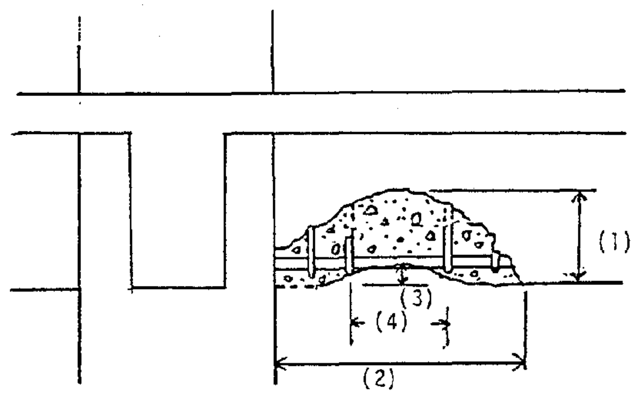
Base Resin ; Epoxy Resin

Curing Agent ; Poly-amide Amin

Table 5 Physical Properties of Epoxy Mortar

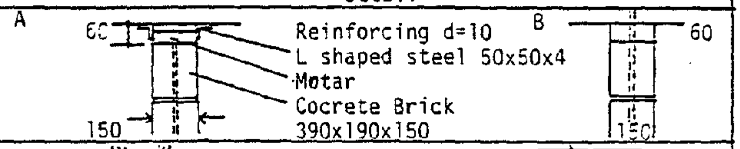
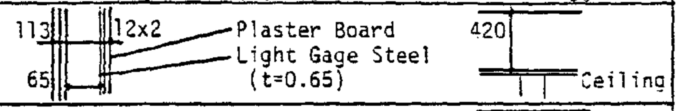
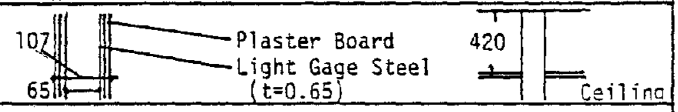
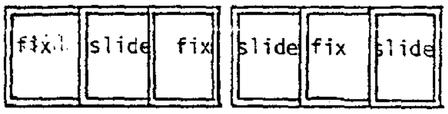
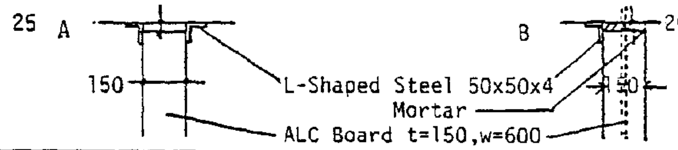
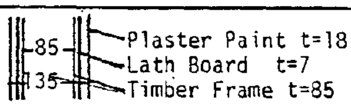
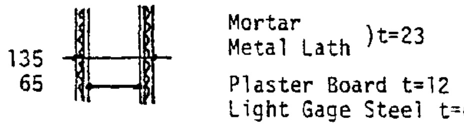
Items	Test	unit	101	
Specific Gravity	JIS K 7112	-	1.70$\pm$0.10	(1.72)
Flexural Strength	JIS K 7203	kg/cm	400	(425)
Compressive Strength	JIS K 7208	kg/cm	600	(837)
Youngs Modulus	JIS K 7208	kg/cm	(4.0-8.0)x10	(6.73)
Tensile Strength	JIS K 7113	kg/cm	200	(257)
Impact Strength	JIS K 7111	kg.cm/cm	1.5	(1.88)
Hardness	JIS K 7215	HoD	85	
Tens. Shear Strength	JIS K 6850	kg/cm	110	(145)

Table 4 Detailed Damage of Beam Ends



Location		Spalled Region			(4) Max. Stirrup Spacing	Buckling	repair Procedure
Story	N,S	(1) Max Height	(2) Max Length	(3) Max Depth			
7TH	N	17 (cm)	45	4	19	Slight	2
7TH	S	12	30	3	?	No	1
6TH	N	20	50	7	25	Severe	3
6TH	S	22	50	3	12	No	1
5TH	N	14	50	3	8	No	1
5TH	S	13	40	3	12	No	1
4TH	N	17	50	3	10	No	1
4TH	S	13	40	3	15	No	1
3RD	N	15	60	4	21	Slight	2
3RD	S	16	40	3	13	No	1
2ND	N	17	40	3	14	No	1
2ND	S	17	40	3	15	Slight	2
1ST	N	5	15	3	?	No	1
1ST	S	10	20	3	12	No	1

Table 6 Details of Non Structural Elements

Material	Symbol	Detail
Concrete Brick Masonry	CB	
Plaster Board Framed by Light Steel (Floor Slab ≐ Ceiling)	GBM-1	
Plaster Board Framed by Light Steel (Slab ≐ Slab)	GBM-2	
Window Sash	AL-1 AL-2	 Flexible Putty Hardening Putty AL-1; Float Glass t=5mm Vinyl Tape Stick AL-2; Float Glass t=5mm Polyester Film Stick
Artificial Light Weight Concrete Board	ALC	
Lath Board Plaster Finishing Framed Timber	GBW	
Metal Lath Mortar Finishing Framed Light Gage Steel	MSM	
Window Sash	AL-3 AL-4	 Flexible Putty Haddening Putty AL-3; Float Glass t=5mm Vinyl Tape Stick AL-4; Wire Mesh Glass t=6.3mm

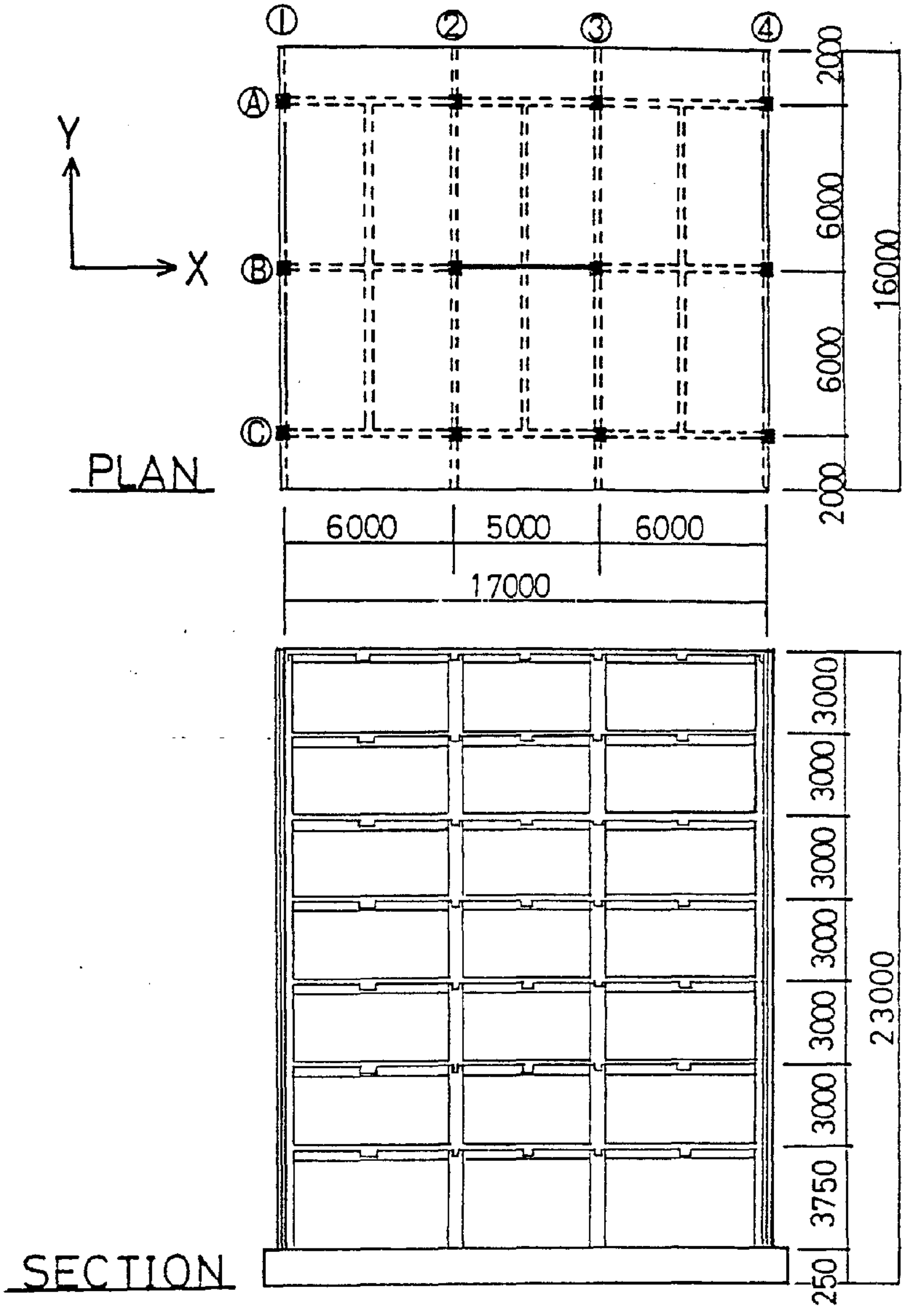


Fig. 1 Plan,Elevation,and Dimension of the Specimen

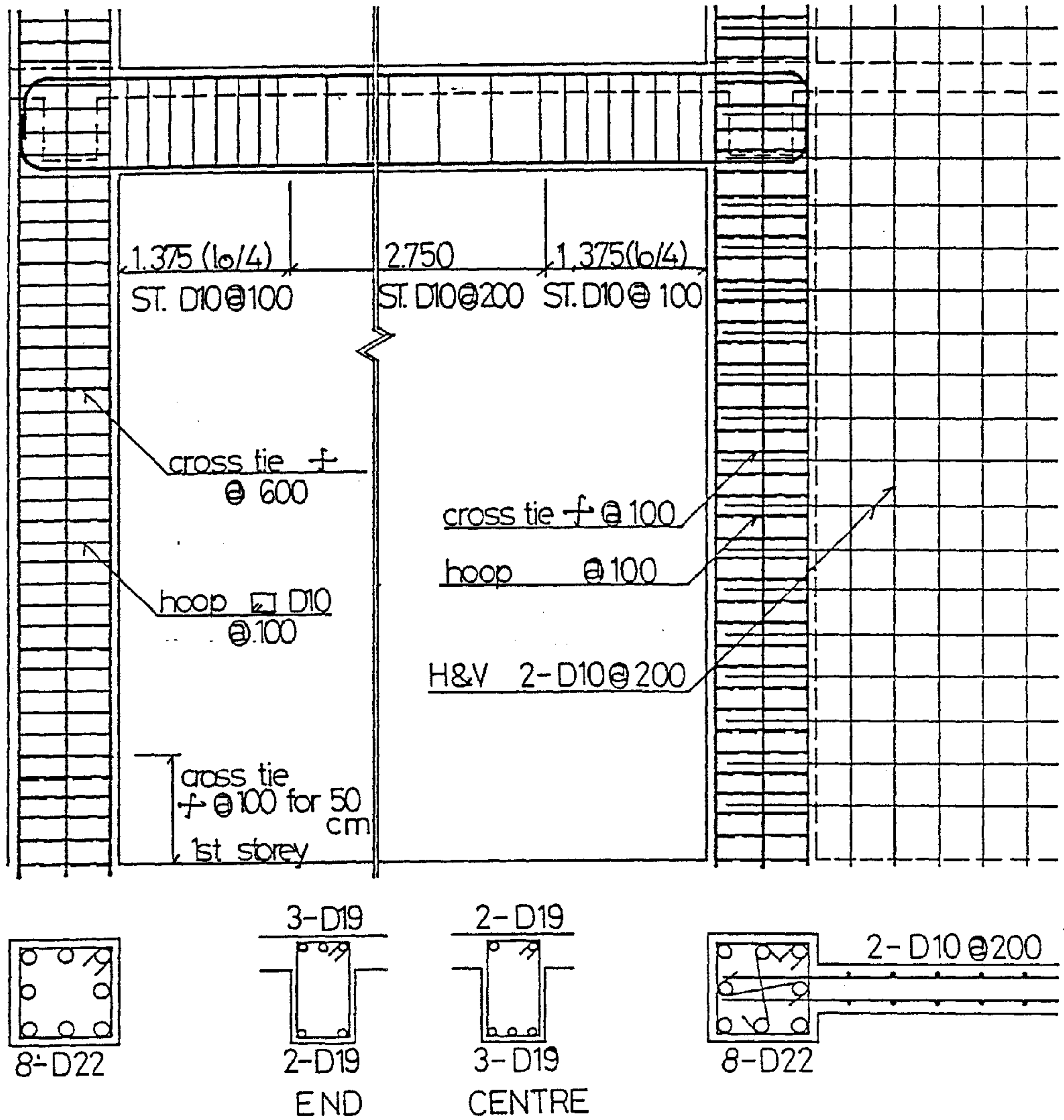
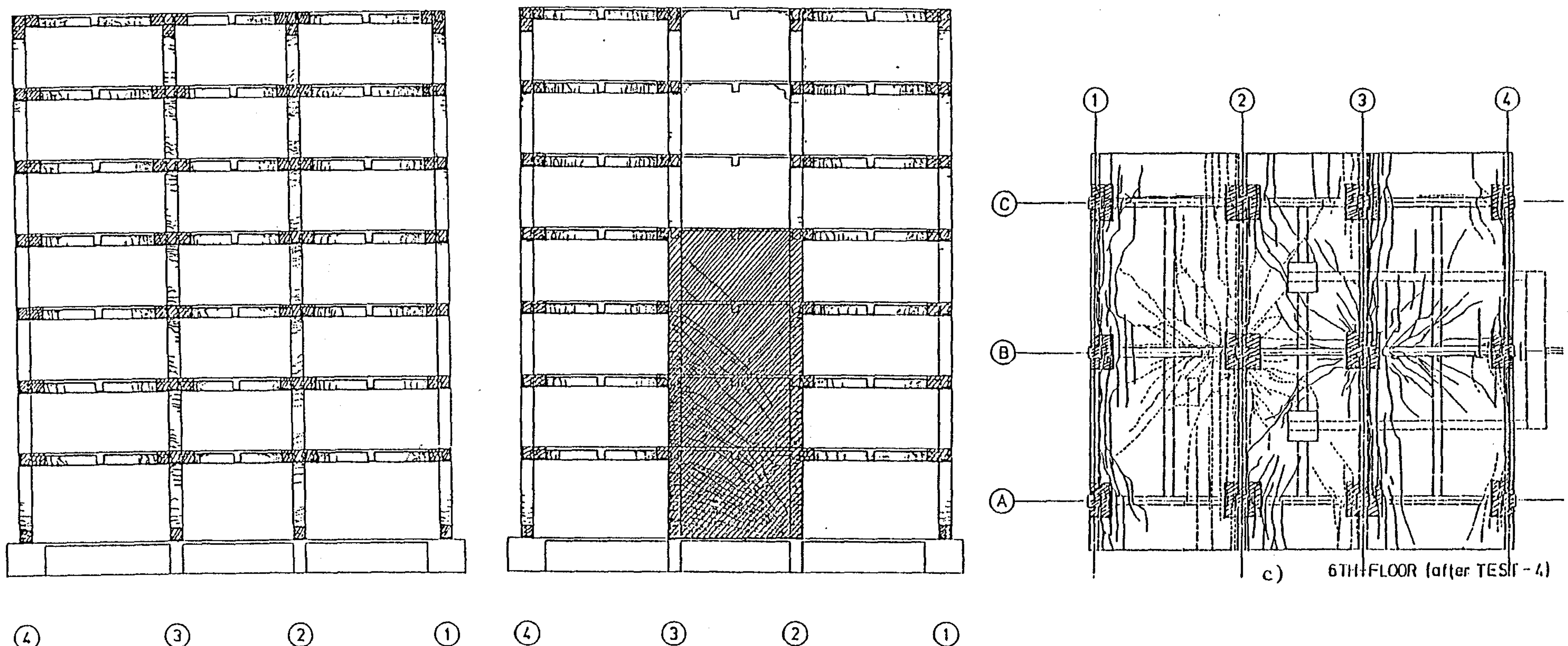


Fig. 2 Details of Reinforcing First Floor



a) FRAME-A (after TEST-4)   b) FRAME-B (after TEST-4)   c) 6TH FLOOR (after TEST-4)

Fig. 3   Repaired Portions by Epoxy Resin

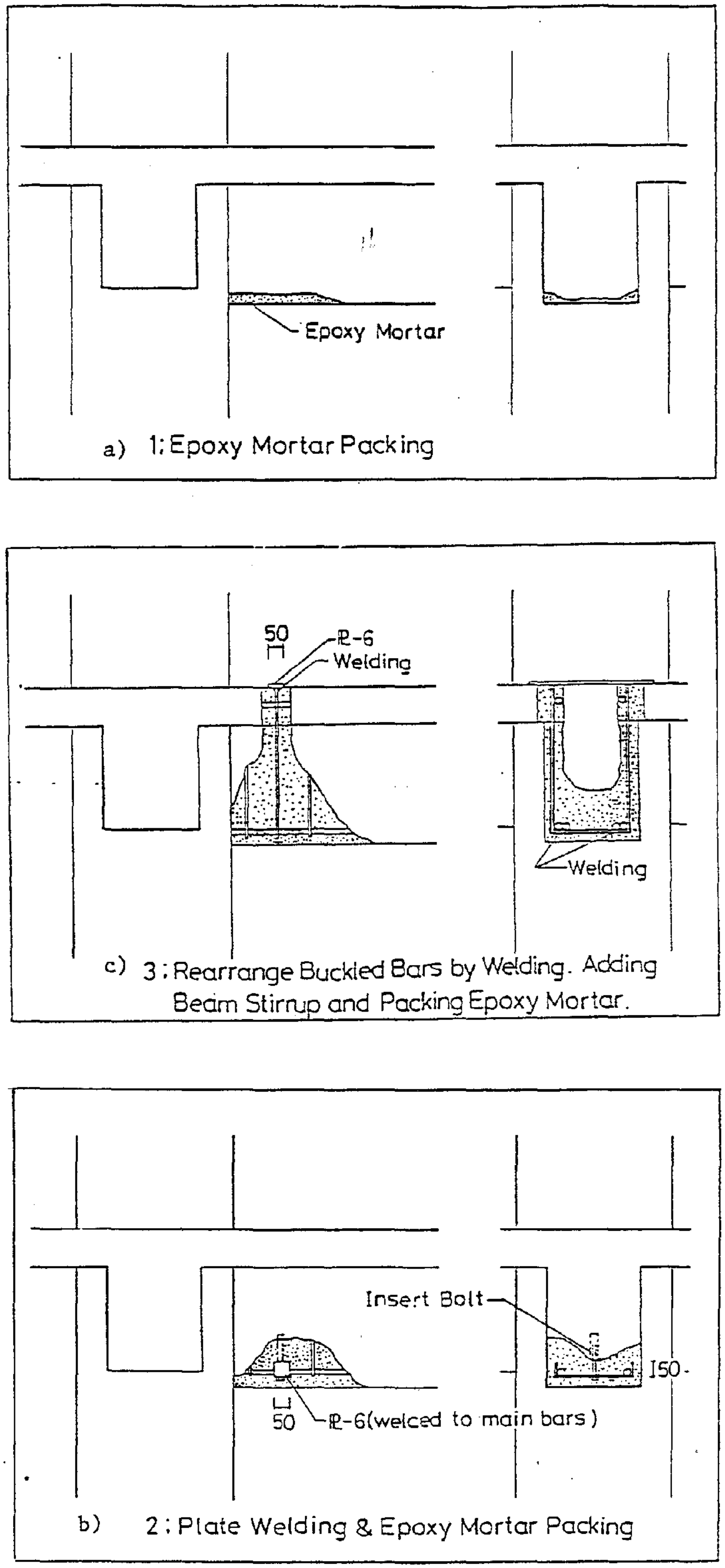


Fig. 4 Details of Repair Works at Beam End

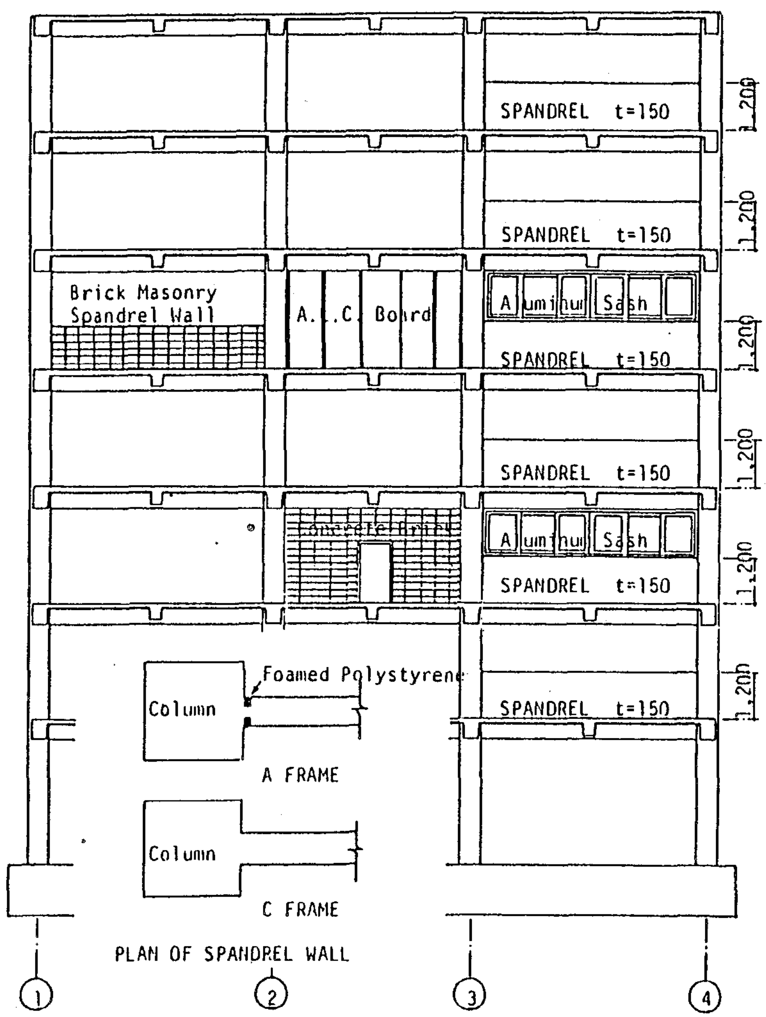
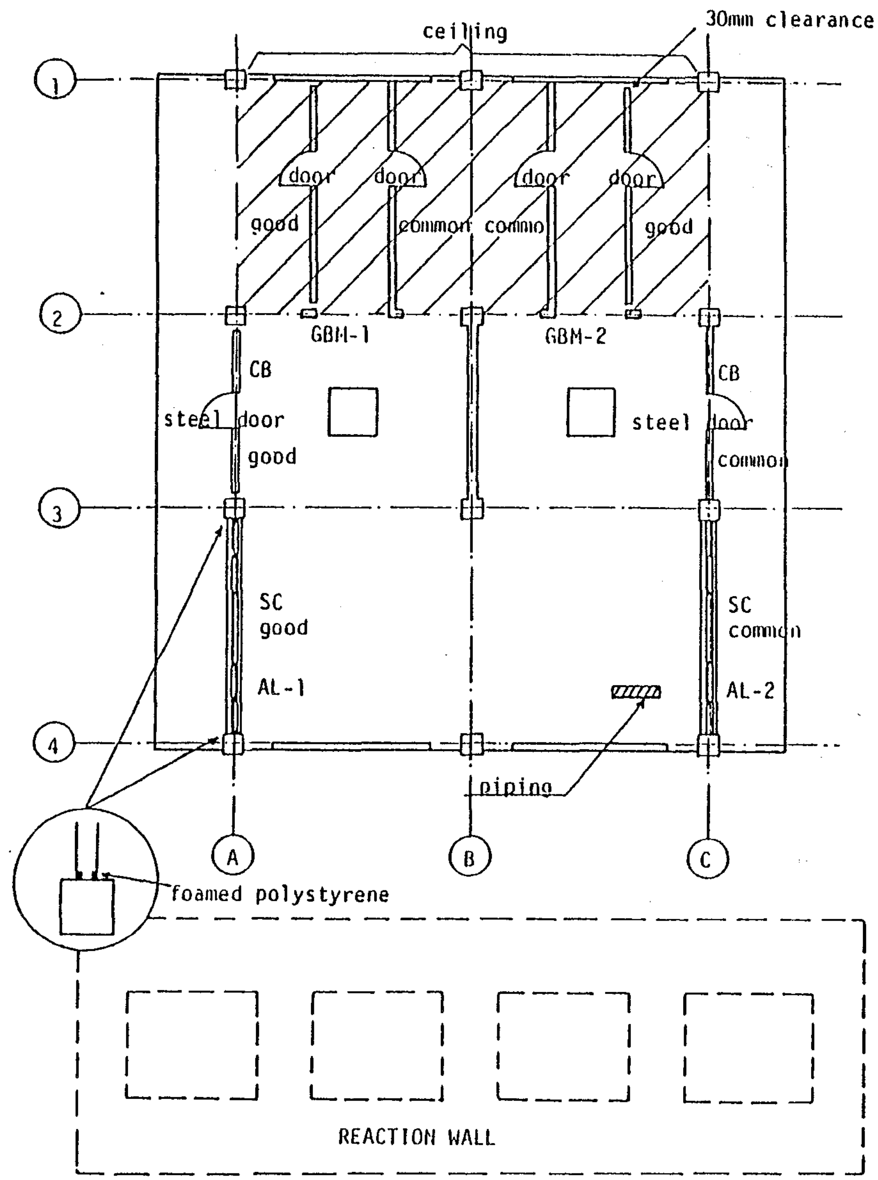


Fig. 5 Installation of Spandrel Walls



Nonstructural Elements (at 3 story)

Fig. 6 Installation of Partition Walls

TEST 5

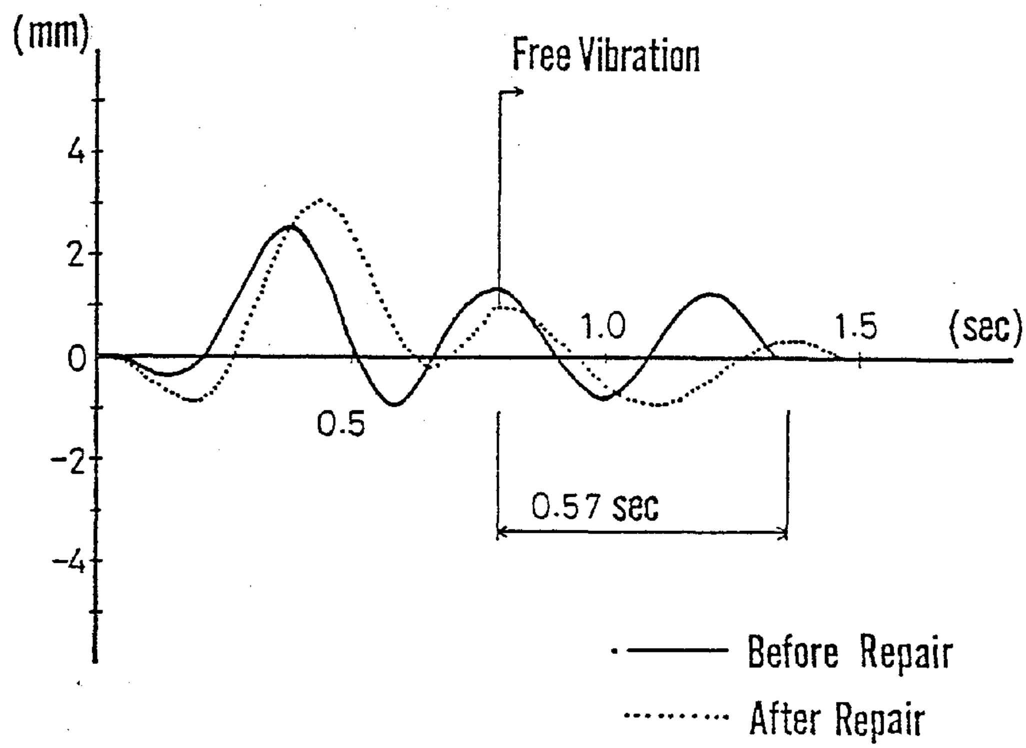


Fig. 7 Response Time Hystory

TEST 6

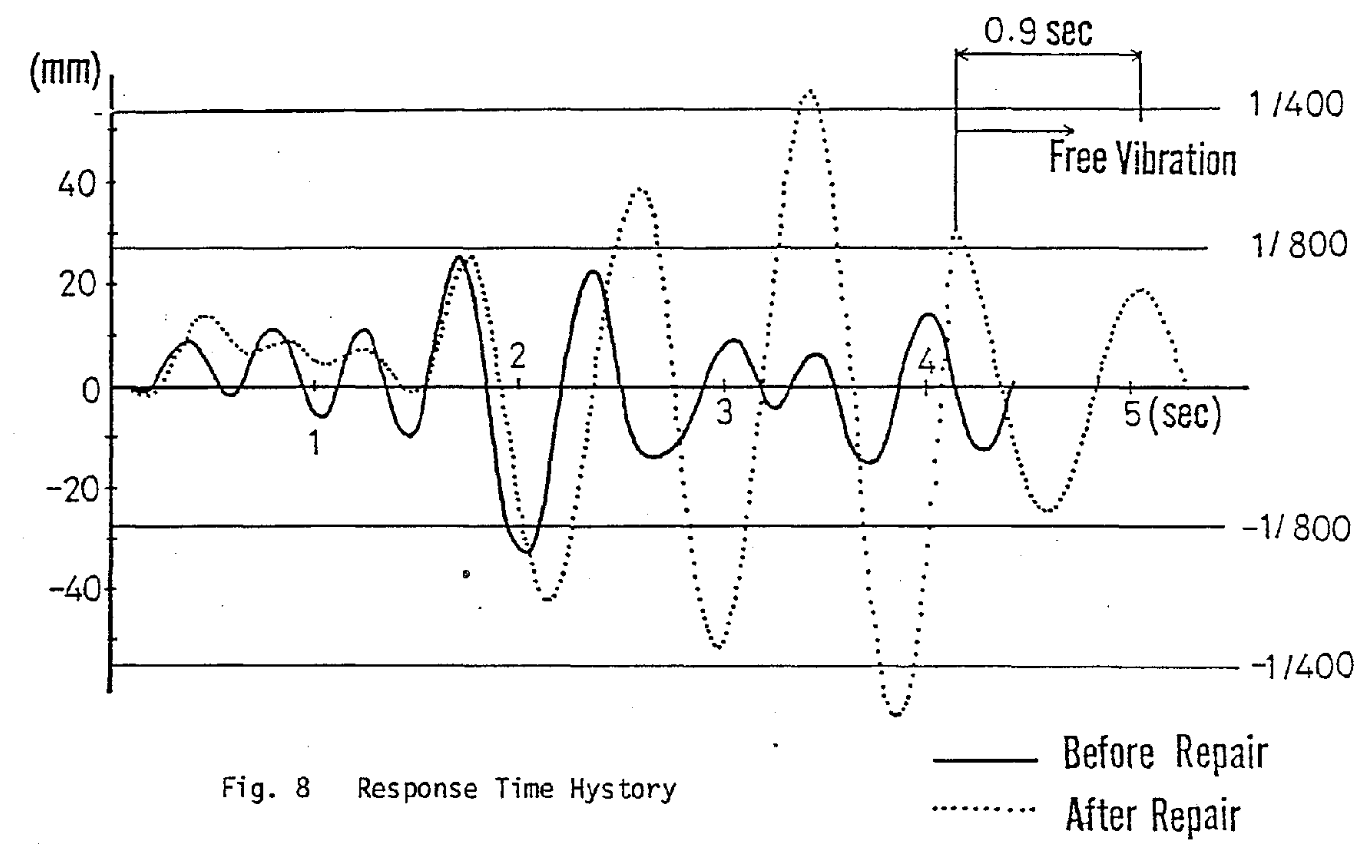


Fig. 8 Response Time Hystory

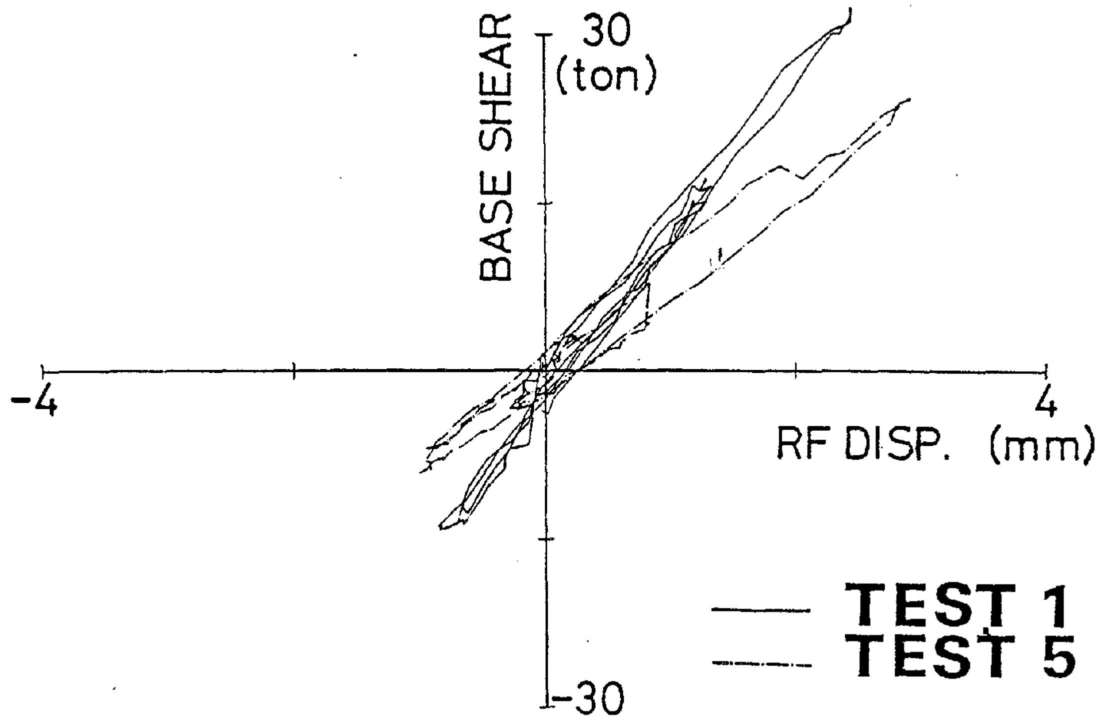


Fig. 9 Total Shear Force vs. RF Displacement Relationship

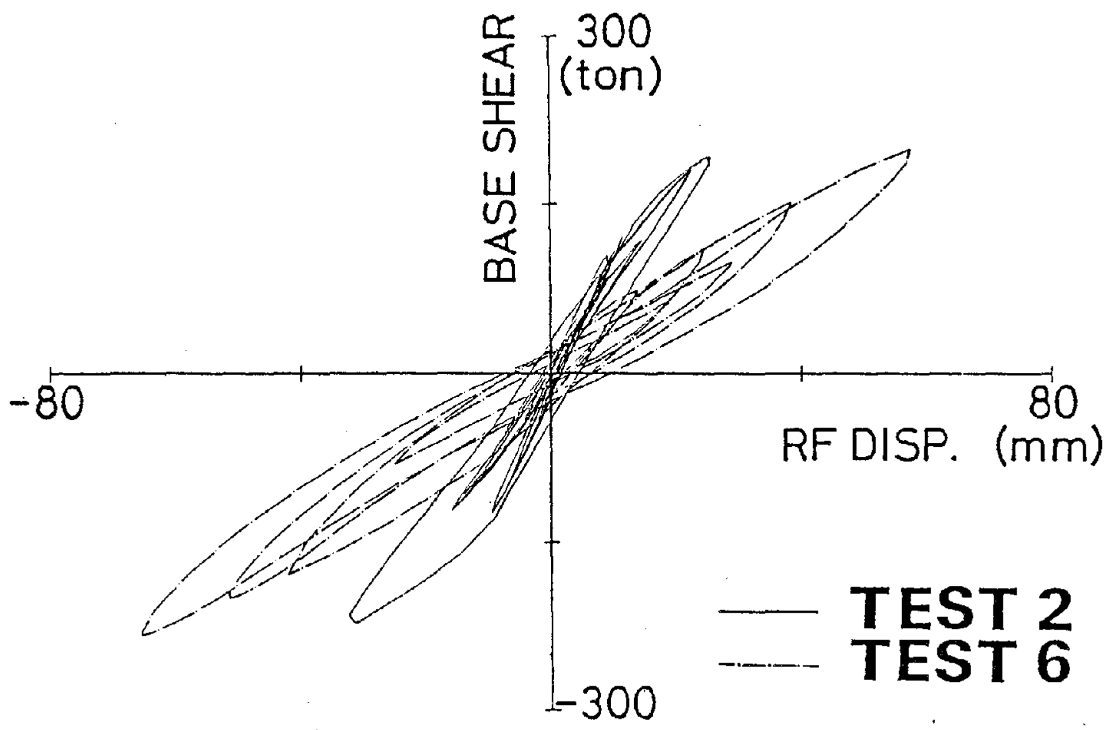


Fig. 10 Total Shear Force vs. RF Displacement Relationship

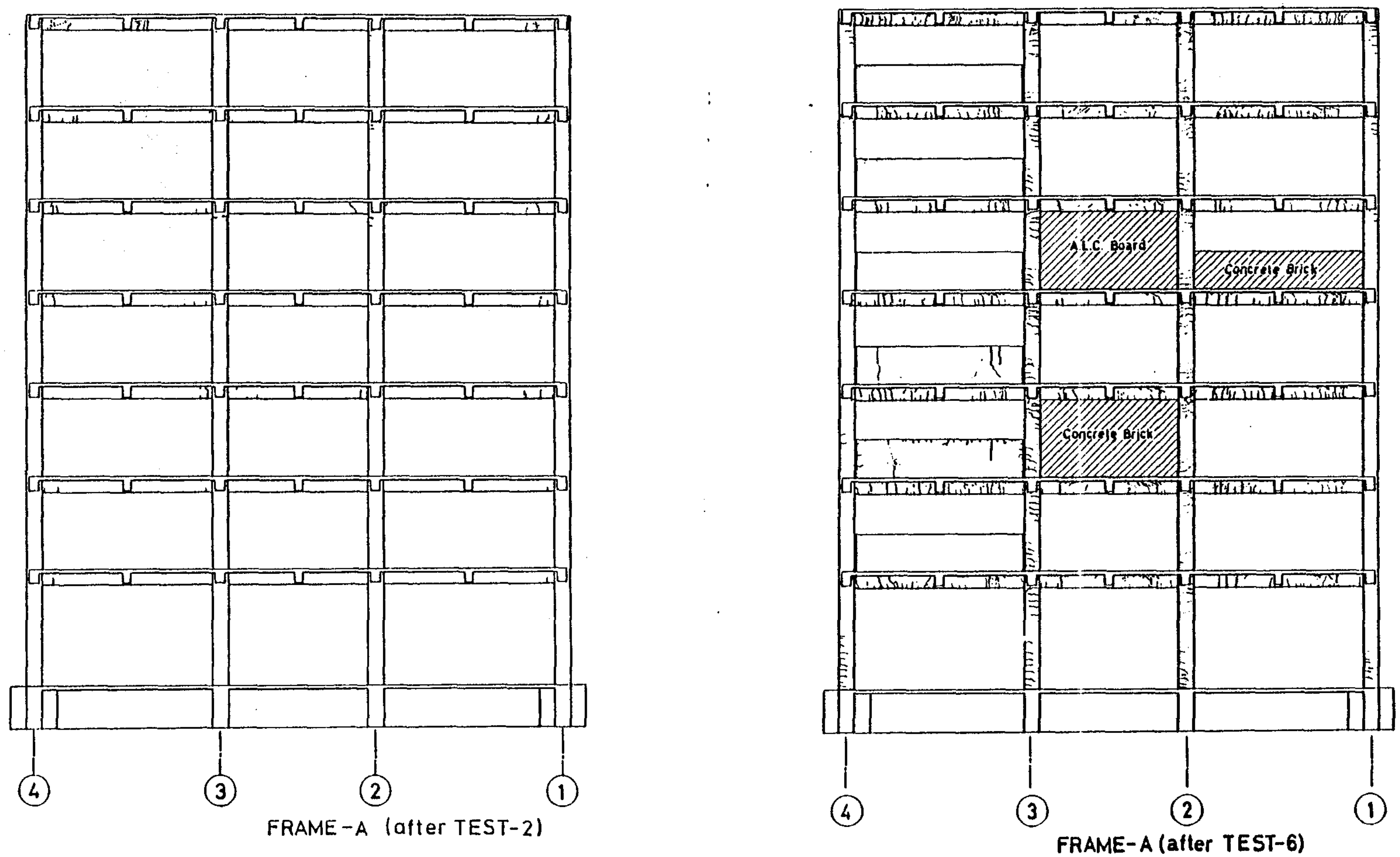


Fig. 11 Crack Pattern of Frame-A (Test 2&6)

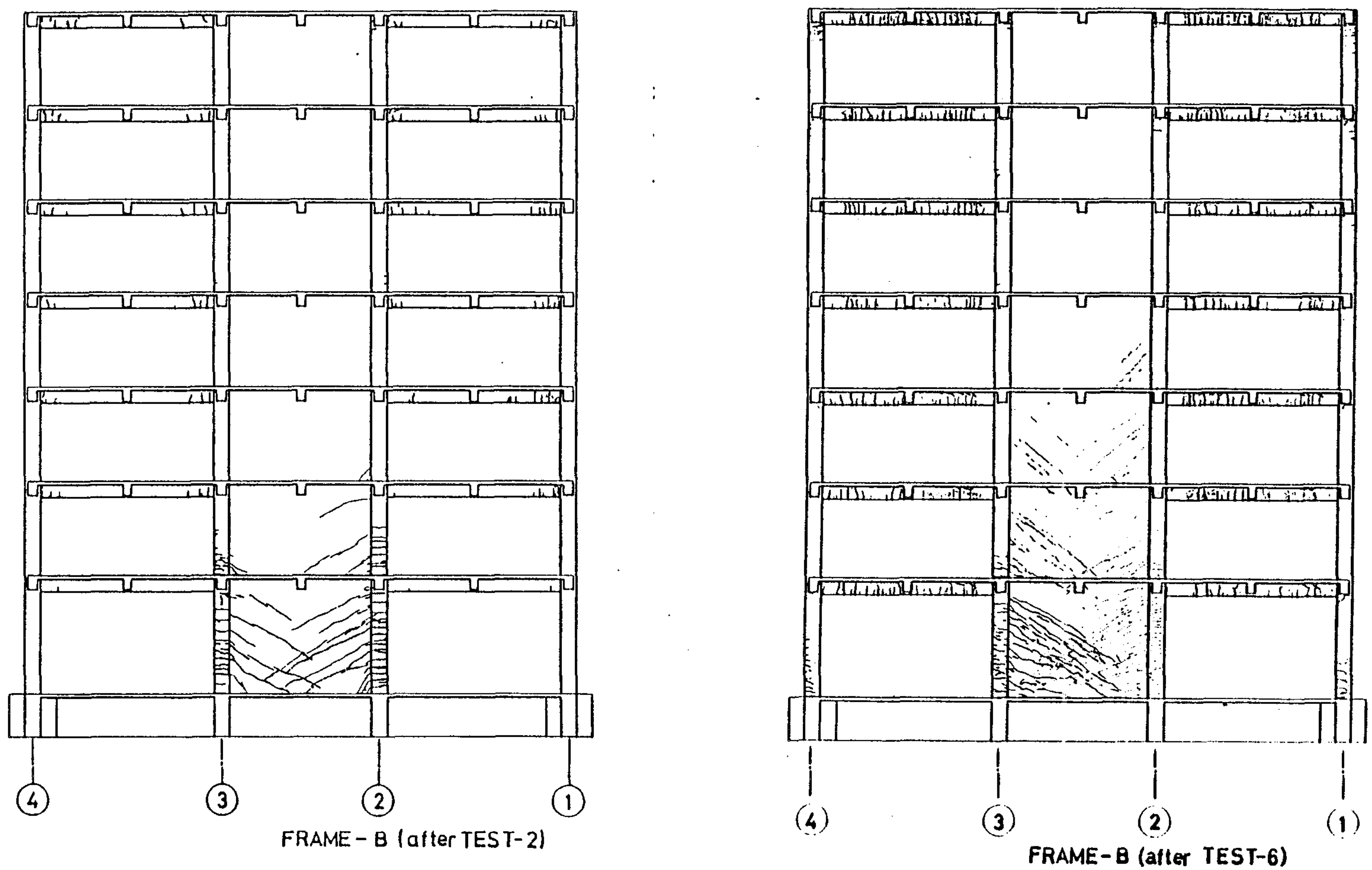
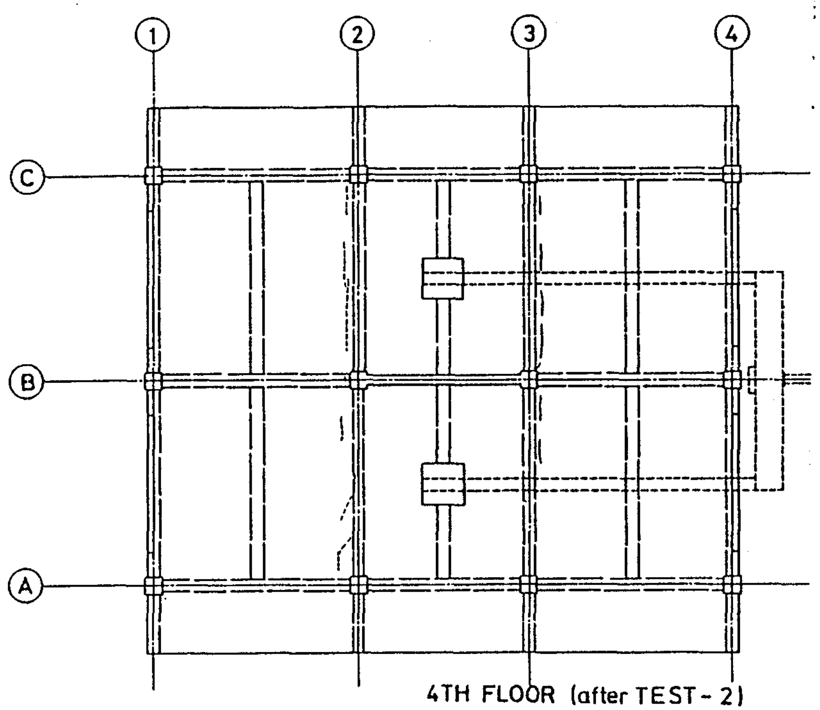


Fig. 12 Crack Pattern of Frame-B (Test 2&6)



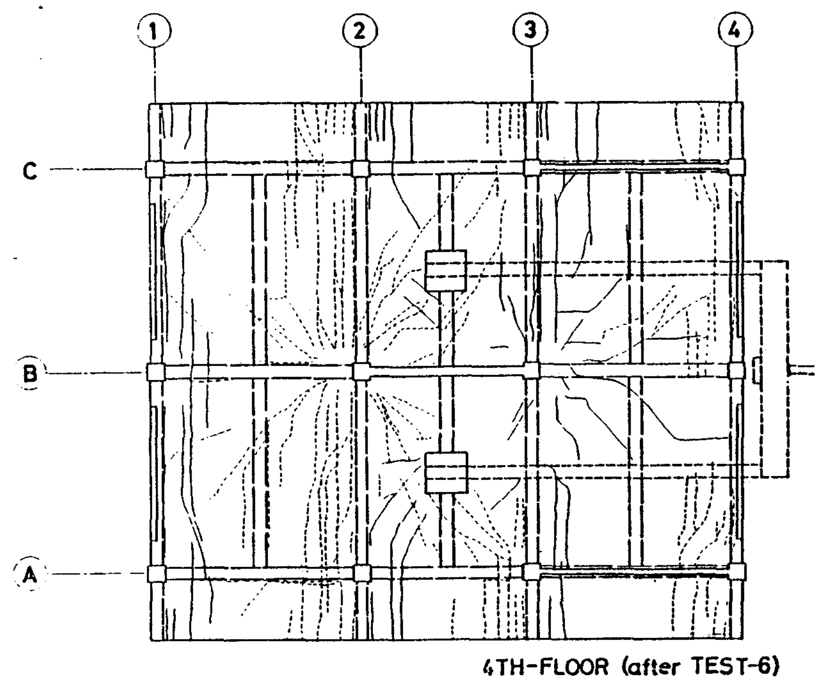


Fig. 13 Crack Pattern of 4th Floor (Test 2&6)

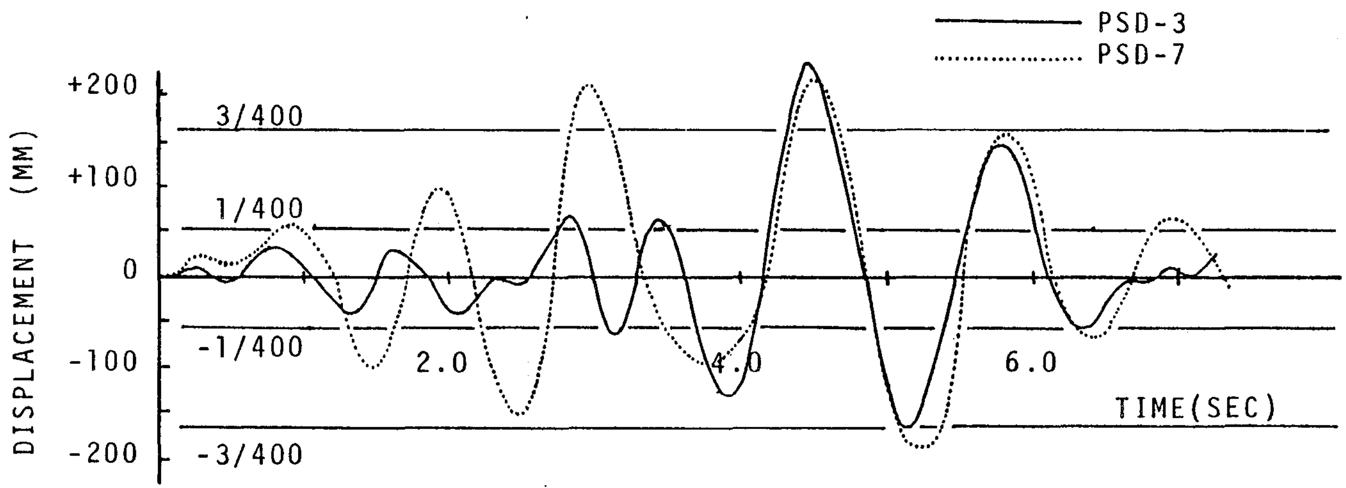


Fig. 14 Response Time Hystory

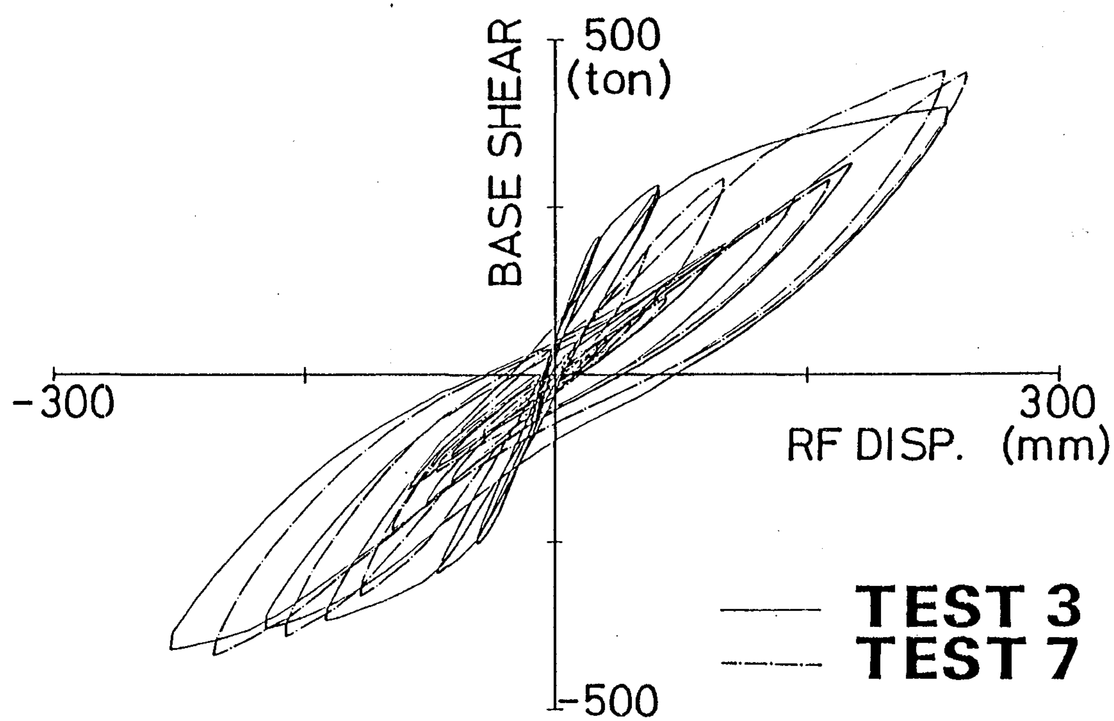


Fig. 15 Total Shear Force vs. RF Displacement Relationship

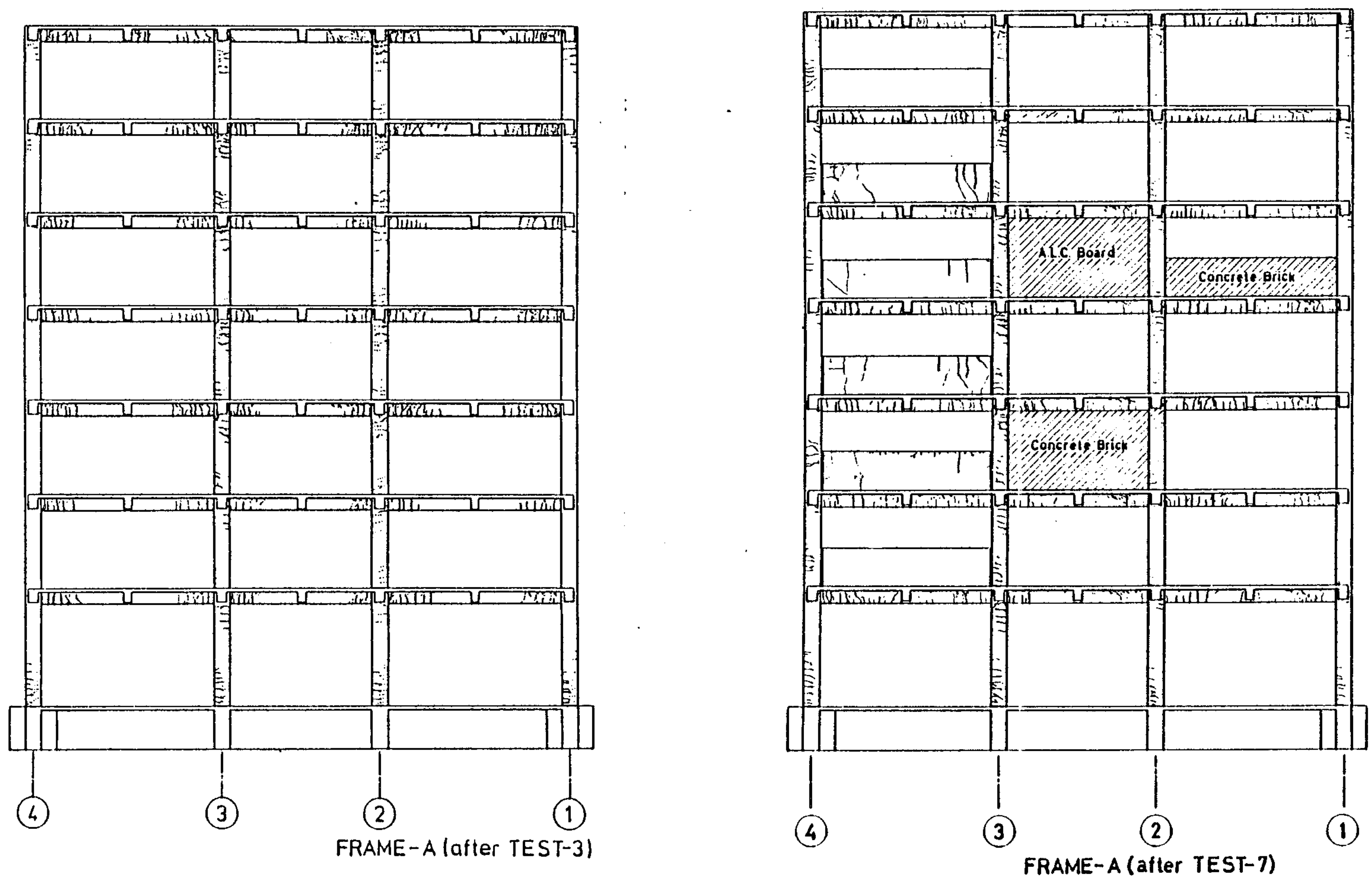
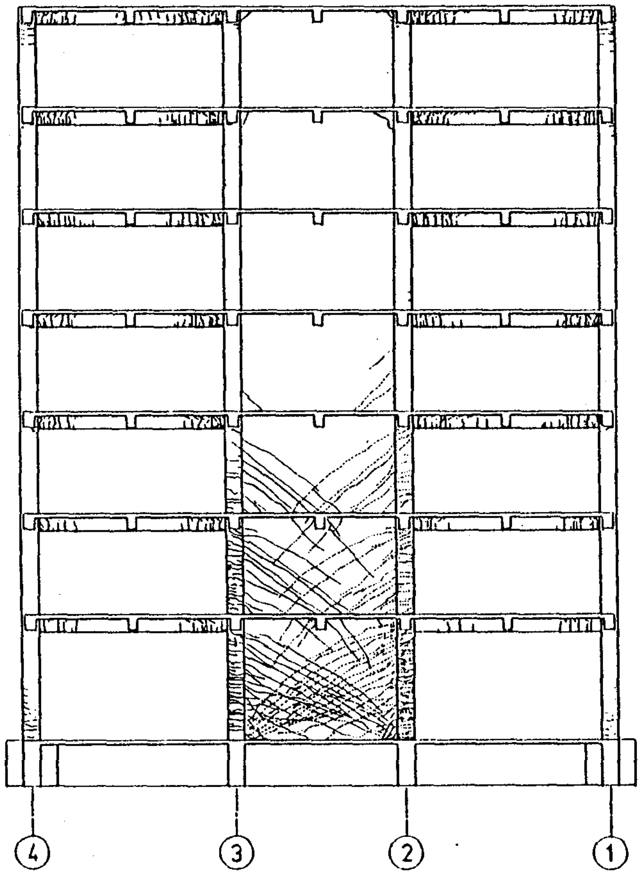
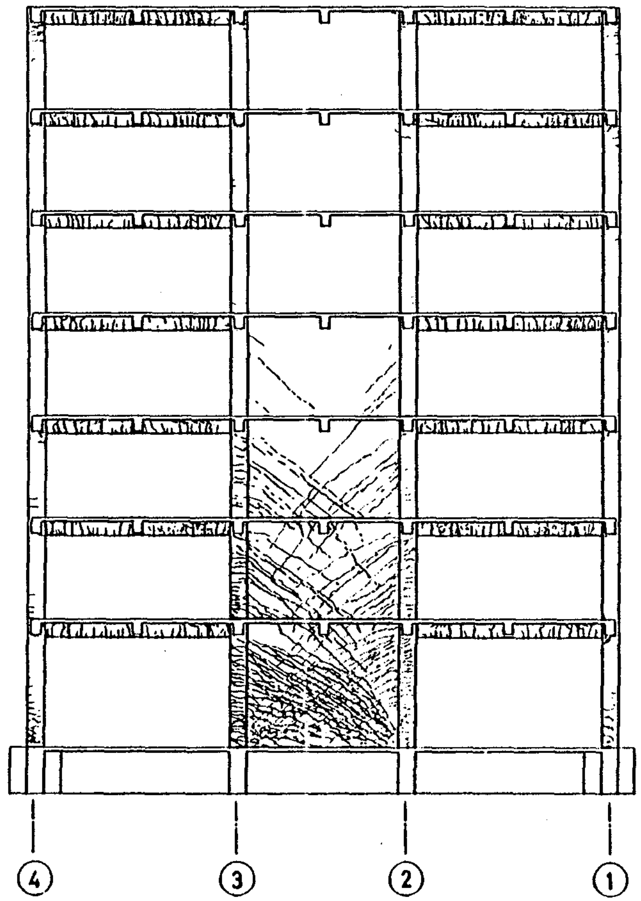


Fig. 16 Crack Pattern of Frame-A (Test 3&7)



FRAME-B (after TEST-3)



FRAME-B (after TEST-7)

Fig. 17 Crack Pattern of Frame-B (Test 3&7)

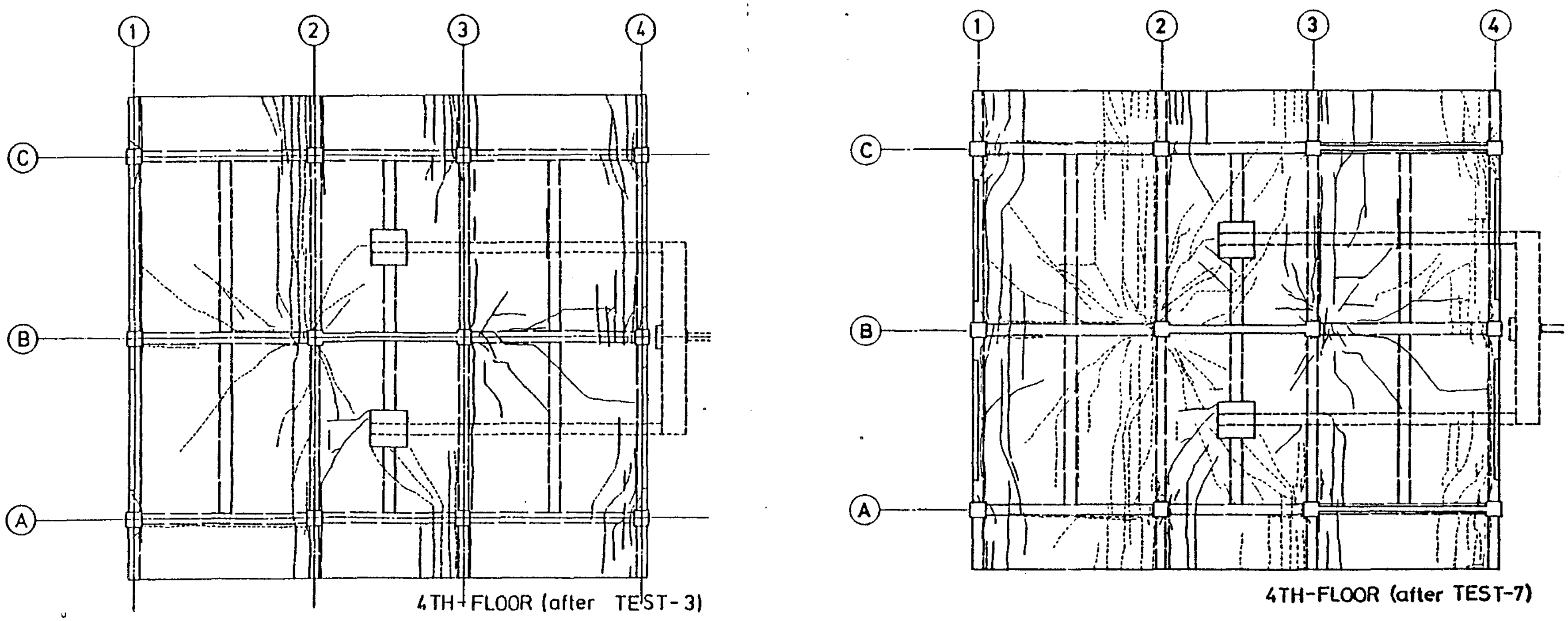


Fig. 18 Crack Pattern of 4th Floor (Test 3&7)

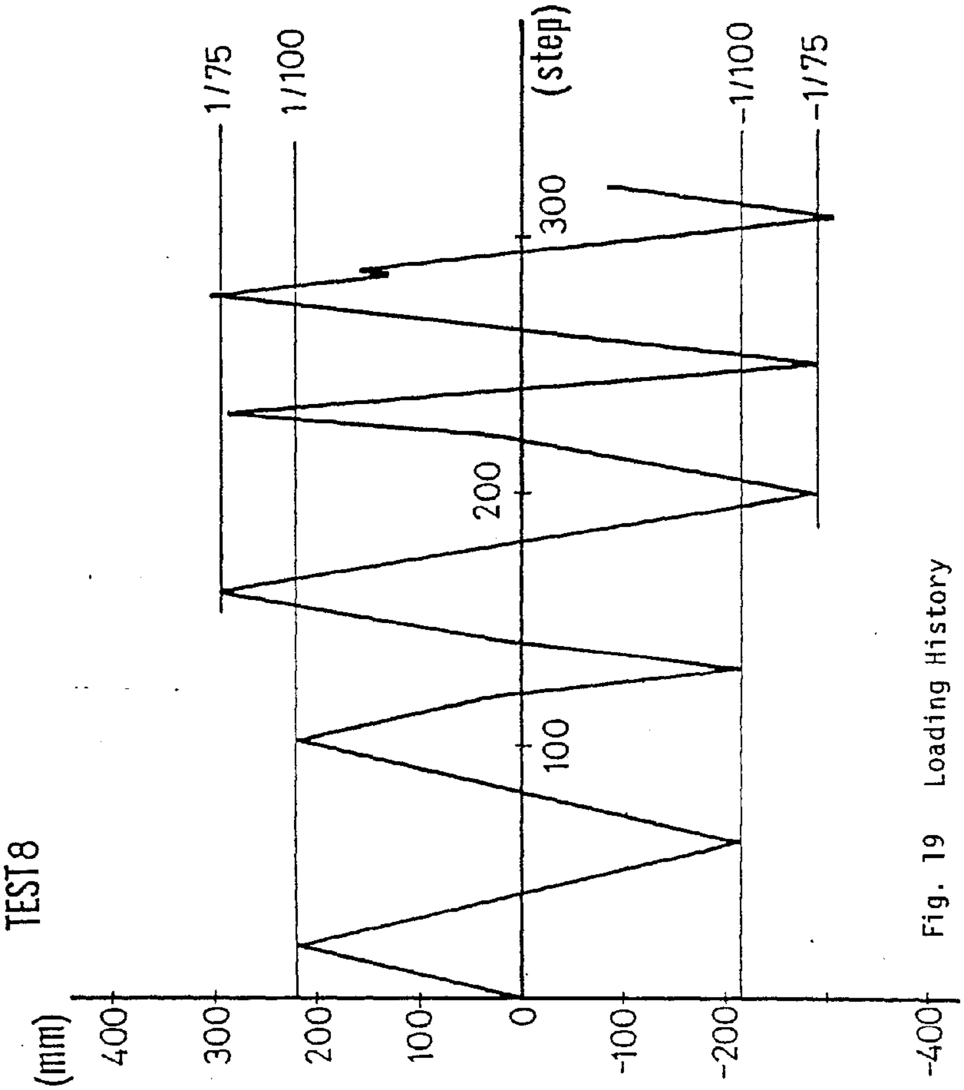
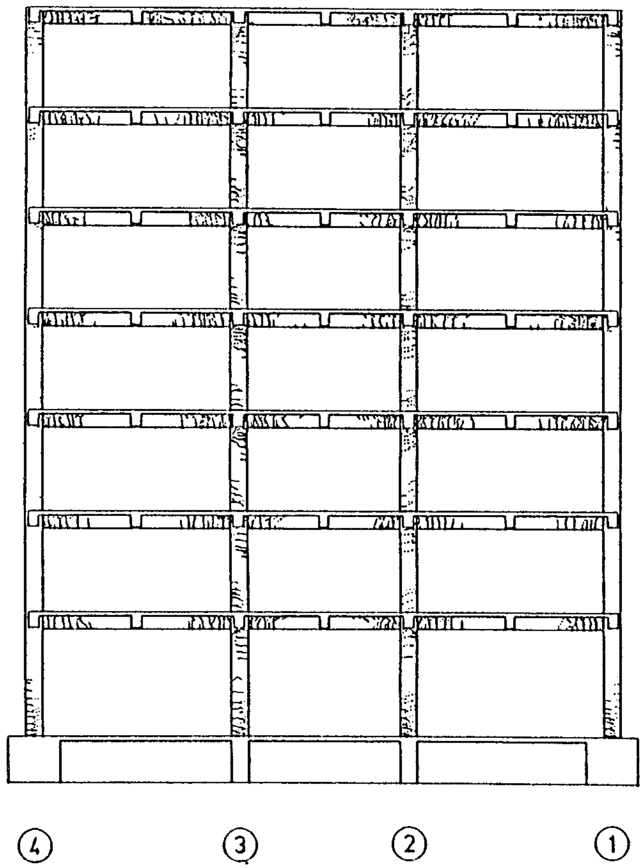


Fig. 19 Loading History



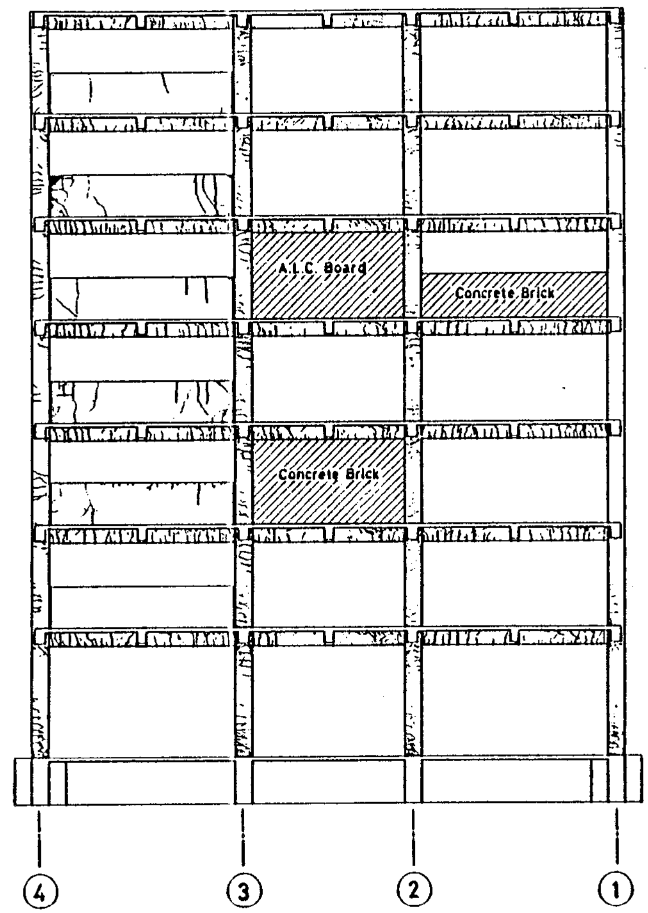
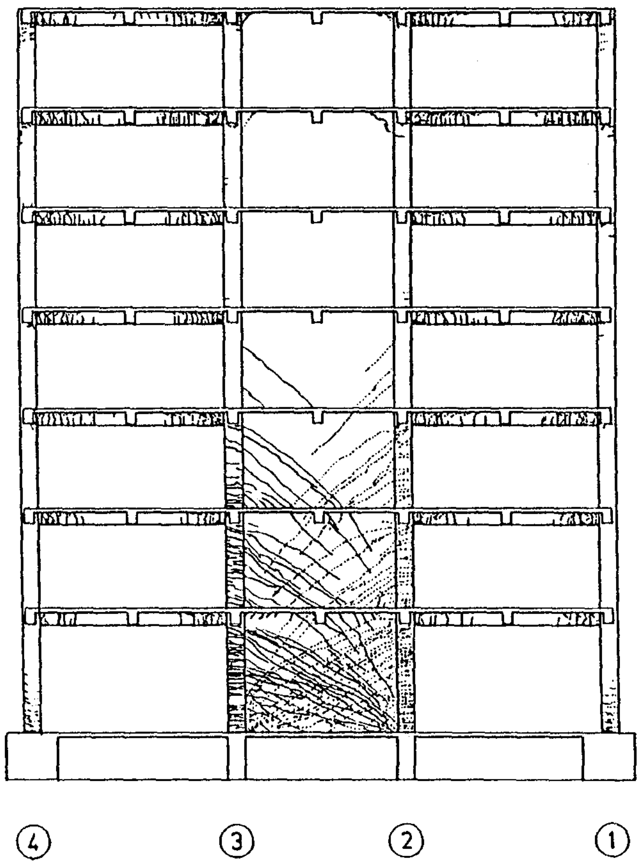


Fig. 20 Crack Pattern of Frame-A (Test 4&8)



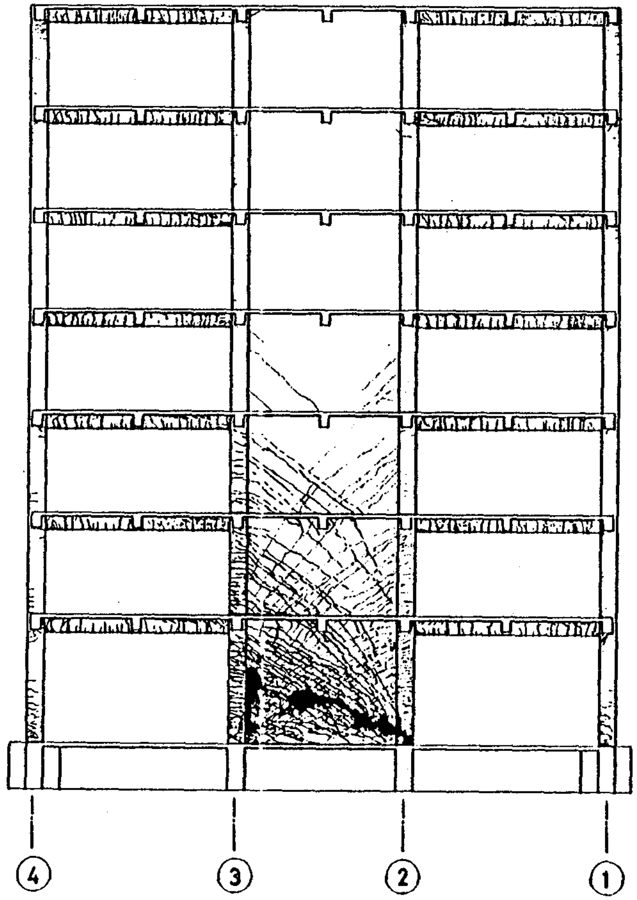


Fig. 21 Crack Pattern of Frame-B (Test 4&8)

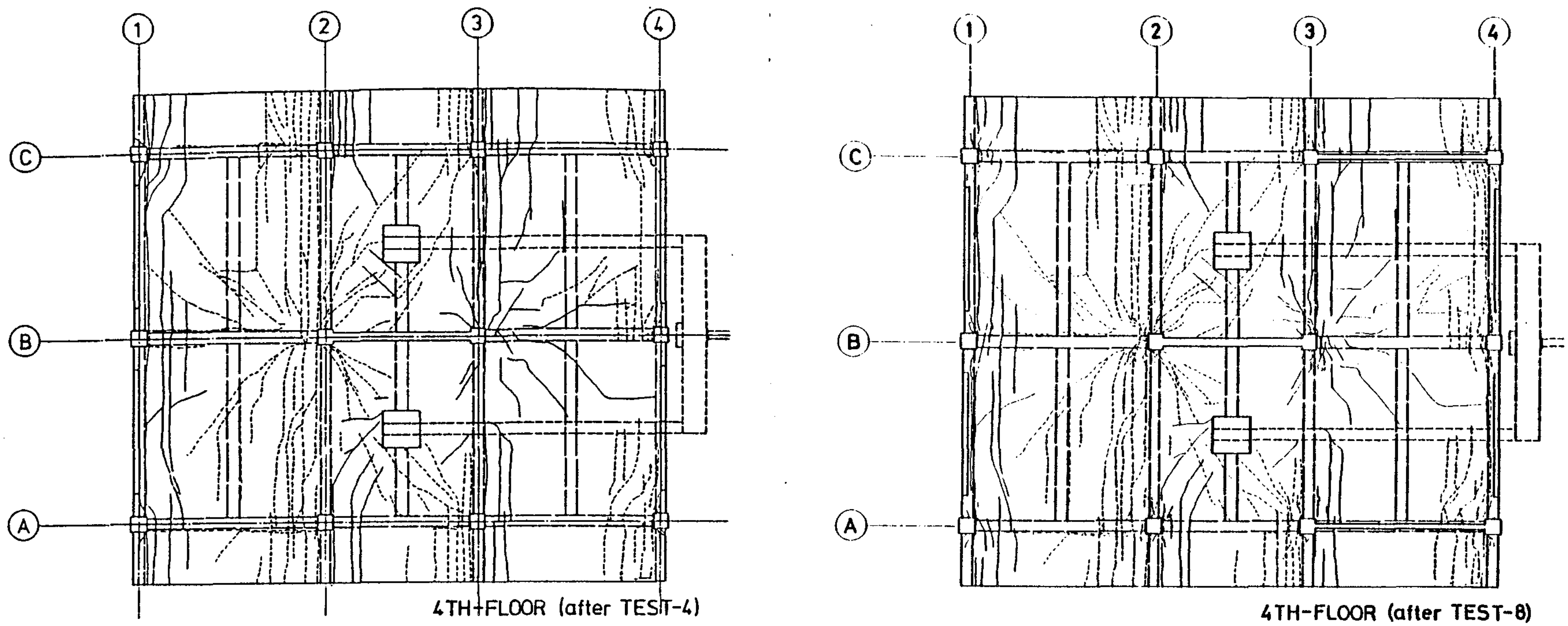


Fig. 22   Crack Pattern of 4th Floor (Test 4&8)

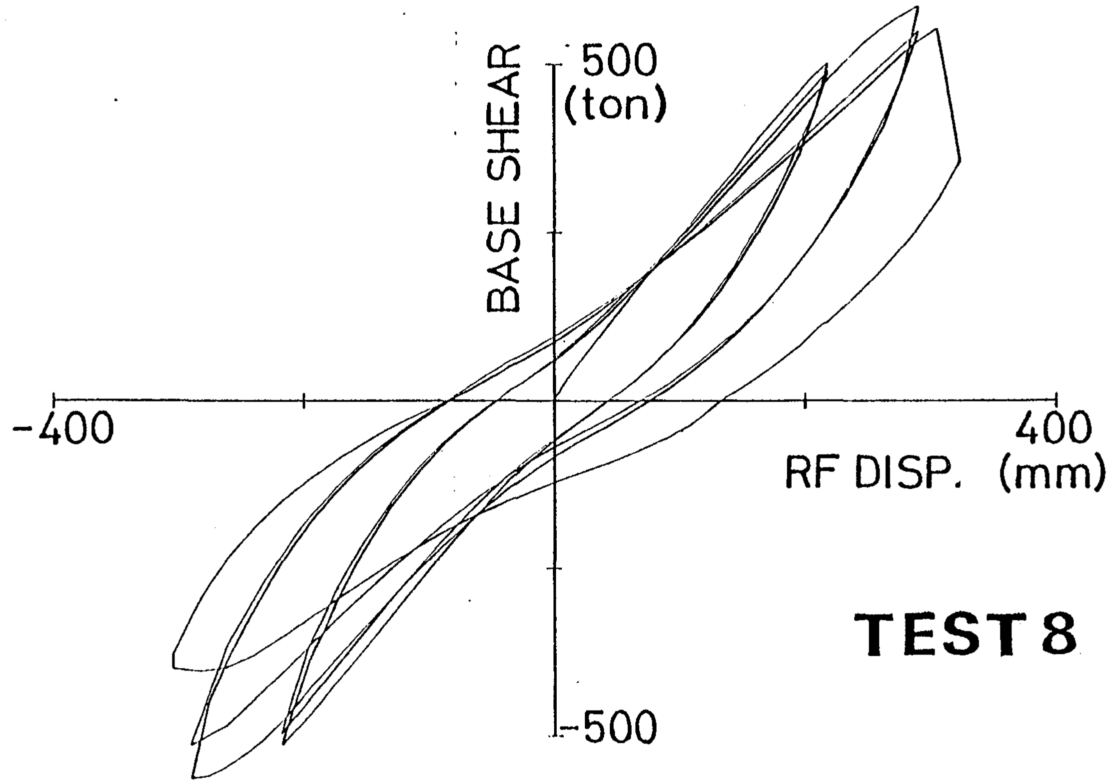
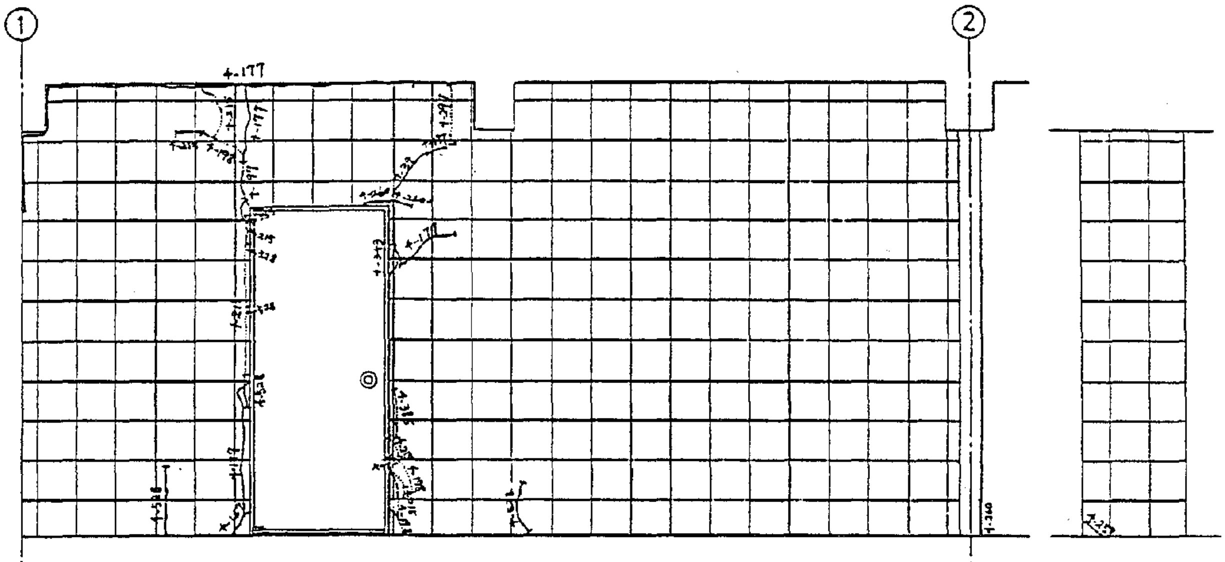
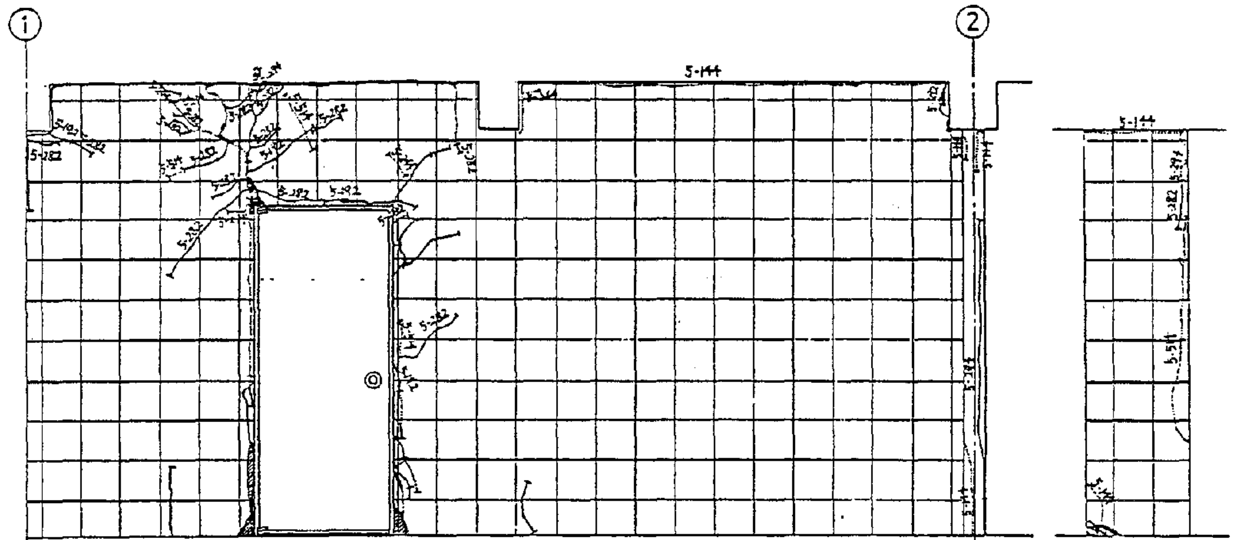


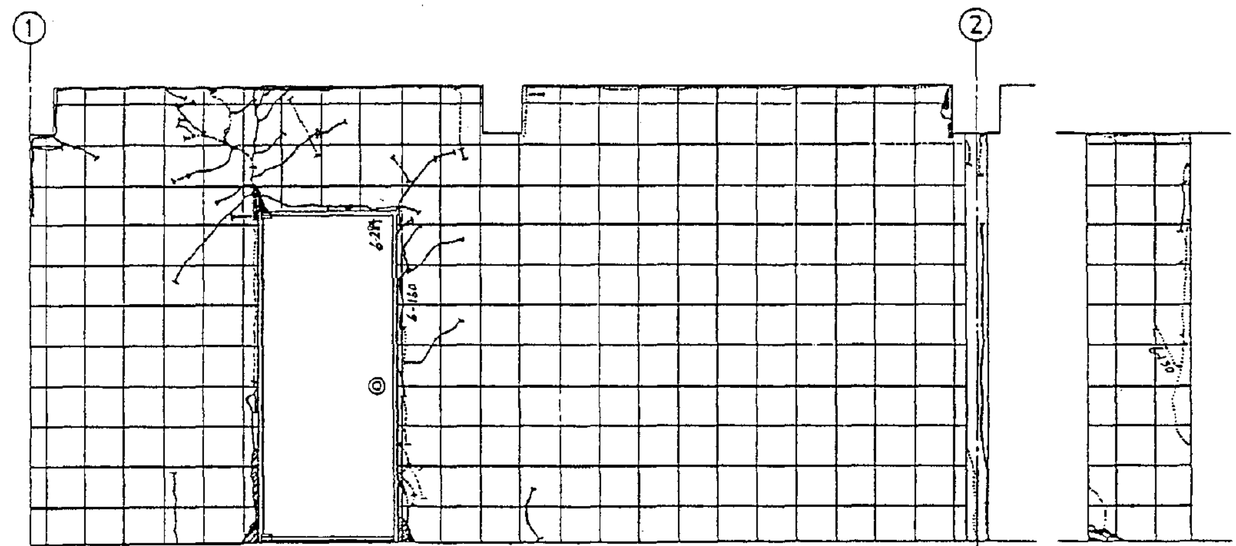
Fig.23 Total Shear Force vs. RF Displacement Relationship



Test 6

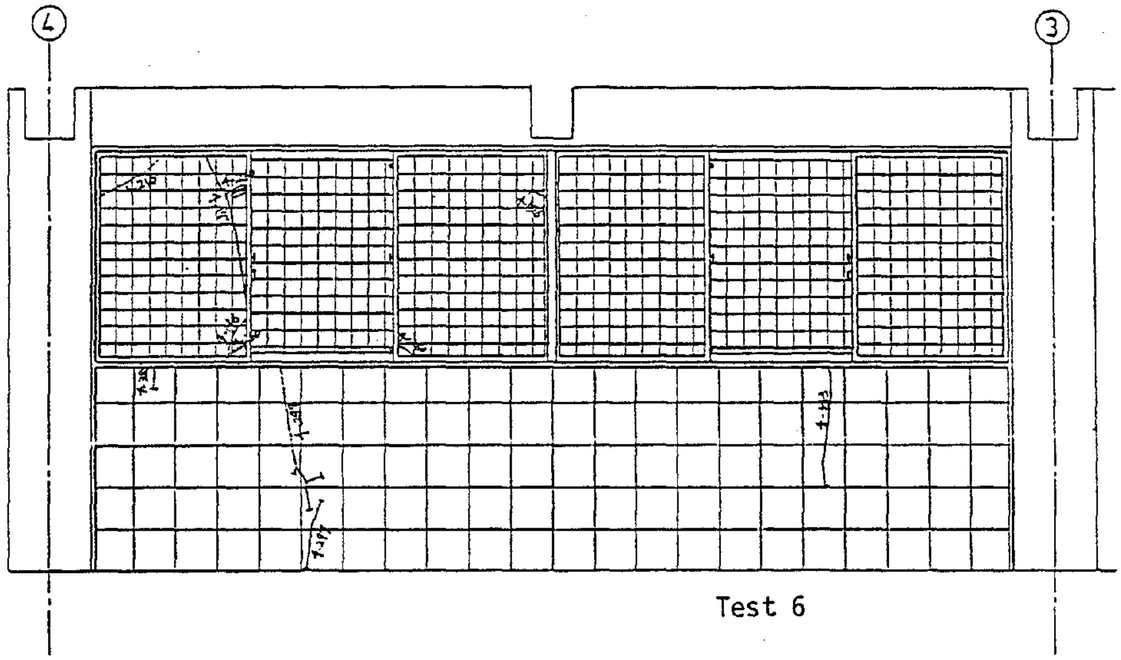


Test 7

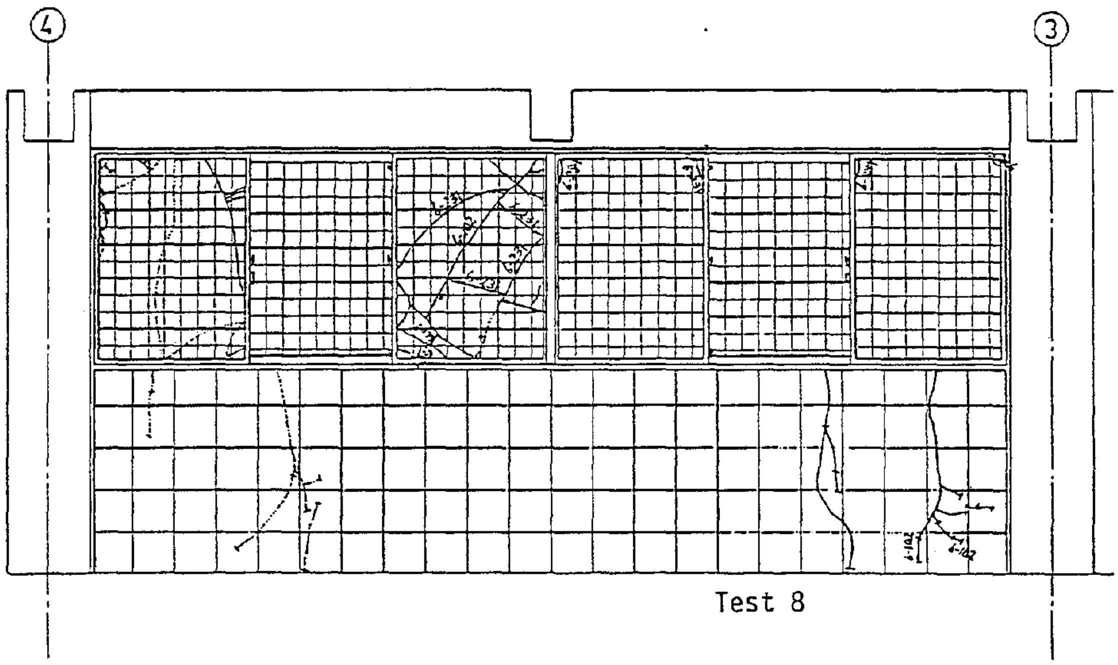


Test 8

Fig. 24 Damage of Partition Wall



Test 6



Test 8

Fig. 25 Damage of Glasses & Spandrel Wall

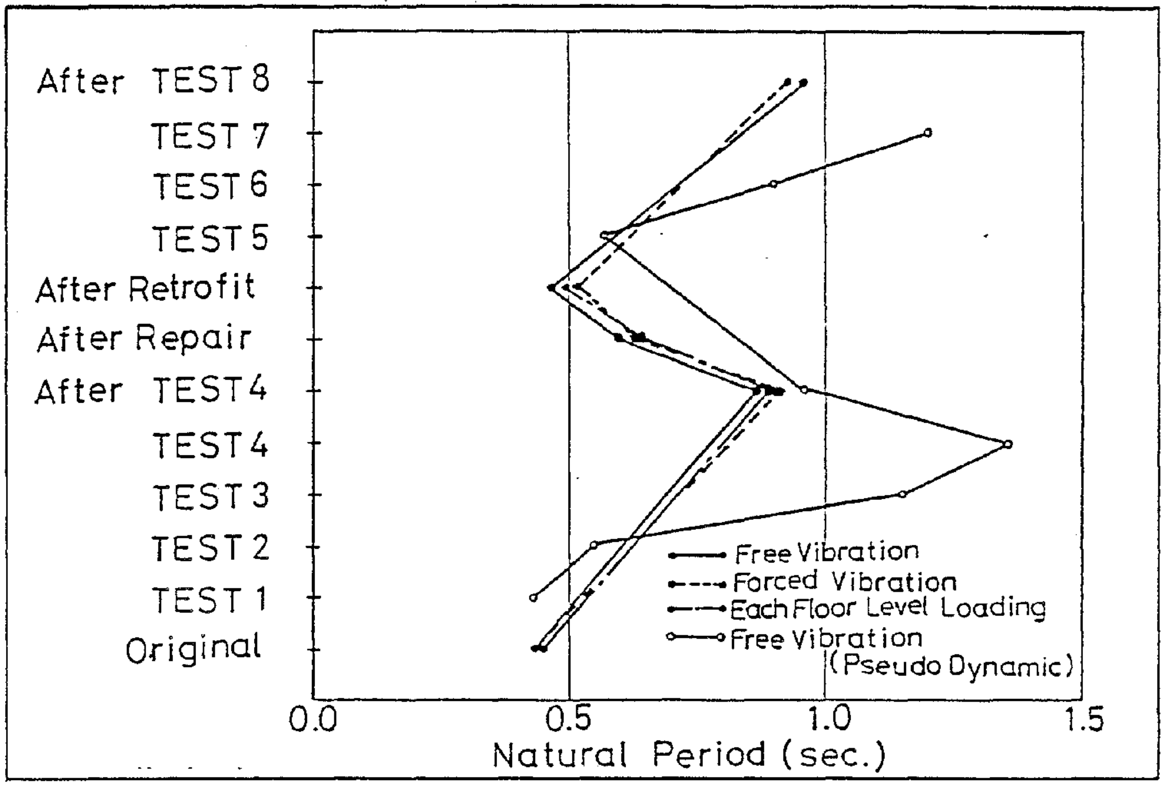


Fig. 26 Transition of Measured Natural Period by Vibration Test

SEISMIC RETROFIT OF SMALL PUMPING AND CHLORINATING STATION BUILDINGS

by

Le Val Lund[I]

SYNOPSIS

After the 1971 San Fernando earthquake, the Los Angeles Department of Water and Power made an earthquake vulnerability assessment survey of its existing buildings containing essential water pumping and chlorinating facilities. The purpose of the survey was to identify buildings which could be damaged by an earthquake and subsequently cause the water facilities to be inoperative. Facilities which could be seismically upgraded at moderate cost were designated to be retrofitted as compared to those facilities which required reconstruction at a higher cost.

INTRODUCTION

The 9 February 1971 earthquake in San Fernando, California registered a Richter magnitude of 6.4. This moderate size earthquake can be expected in Southern California approximately every six years. Southern California consists of a highly urbanized area along the coast, with sparsely developed desert areas inland. Depending on the epicentral location, potential damage to facilities varies. During the San Fernando earthquake, damage occurred to lifeline water pumping and chlorinating facilities consisting of shifting equipment, broken piping, building structural damage, and loss of power supply.

EARTHQUAKE VULNERABILITY ASSESSMENT

After the 1971 earthquake, the Los Angeles Department of Water and Power initiated an earthquake vulnerability assessment of all its lifeline facilities including pumping and chlorination stations. The City topography ranges in elevation from sea level to 750 metres (2440 feet); and therefore, it is necessary to use booster water pumps to lift water to the higher elevations in the City. Since the City receives water from either the underground water basin or a watershed which is generally free of water quality problems, the treatment is presently limited to chlorination when the water is distributed from open storage reservoirs.

[I]Engineer Los Angeles Aqueduct, Department of Water and Power, Los Angeles.

The vulnerability assessment survey was limited to the potential damage from a moderate earthquake to these small stations and was made by field construction inspection personnel. The guidelines for the survey were to identify the following:

1. Safety Hazards - No fire extinguisher or chlorine gas masks, freestanding cabinets, or racks; and inadequate ventilation.

2. Anchorage Deficiencies - Battery rack, pumping and chlorinating equipment, chlorine tanks, transformers, and electrical switchboard.

3. Cracks and/or spalls on concrete or masonry walls.

4. Lack of flexible connections at suction and discharge lines.

5. Recommend further investigation for seismic structural adequacy.

The assessment resulted in a report titled "Report on Water System Vulnerability to Earthquakes" dated March 1974.

CONSTRUCTION

The buildings which were recommended to be investigated for their seismic structural adequacy were of reinforced concrete or reinforced concrete frame with unreinforced masonry filler walls construction. The buildings were constructed prior to the 10 March 1933 Long Beach earthquake, which had a Richter magnitude of 6.3.

SEISMIC PROBLEM

The buildings had the following seismic problems:

1. Roof not designed to act as a diaphram.

2. Side and end walls not anchored to roof.

3. Unreinforced masonry filler walls.

RESOLUTION OF PROBLEM

1. Remove Spanish tile roofing and sheathing.

2. Install wood plate and ledger to anchor roof to side and end walls.

3. Bolt wood plate and wood ledger to walls.

4. Install steel plate straps to anchor roof purlins to end walls.

5. Replace wood sheathing.

6. Install steel plate straps to transfer load from wood plate to sheathing.

7. Install plywood over sheathing to create diaphram.

8. Install steel tie straps to the roof together at ridge.

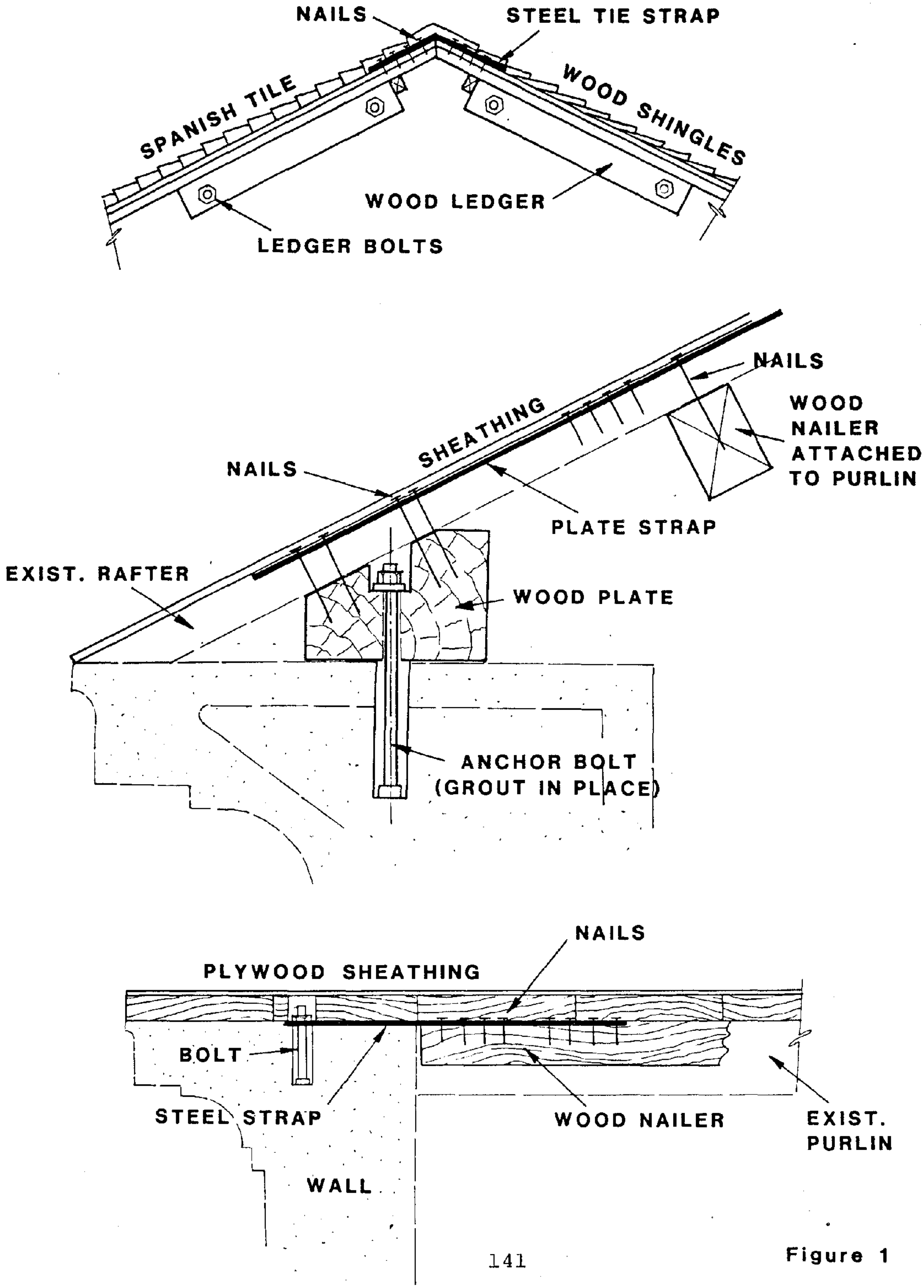
9. Replace roofing with tile or shingles over building paper.

10. Place plywood on interior of unreinforced filler walls with studs and walers to prevent wall falling on operating equipment.

The construction details are shown in Figure 1 and construction costs (1977-79) in Table 1.

CONCLUSION

The retrofit of small pumping and chlorination stations was only a portion of the Los Angeles Department of Water and Power seismic improvement program, which included complete reconstruction of reservoirs, pumping stations, and chlorination stations. The seismic improvement program is continuing and other seismic improvement projects will be initiated in the future.



NAILS
STEEL TIE STRAP
SPANISH TILE
WOOD SHINGLES
WOOD LEDGER
LEDGER BOLTS
NAILS
SHEATHING
NAILS
WOOD NAILER ATTACHED TO PURLIN
PLATE STRAP
EXIST. RAFTER
WOOD PLATE
ANCHOR BOLT (GROUT IN PLACE)
NAILS
PLYWOOD SHEATHING
BOLT
STEEL STRAP
WOOD NAILER
EXIST. PURLIN
WALL

Figure 1

SEISMIC IMPROVEMENTS

PROJECT	COST
GARVANZA PUMPING STATION	$30,114
HARBOR CITY PUMPING STATION	$ 8,546
MACLAY HILINE CHLORINATION STATION	$18,700 (EST.)
MT. WASHINGTON PUMPING STATION	$19,877
RIVERSIDE HYDRAULIC TEST LAB	$31,073
STONE CANYON CHLORINATION STATION	$20,246
99TH STREET PUMPING STATION CONTROL HOUSE	$11,130

Table 1

May 13, 1982

CAMPTON HOTEL REHABILITATION

The Campton Hotel, formerly called the Drake Wiltshire Hotel, is located at 340 Stockton Street near the northeast corner of Union Square in San Francisco.

The property was purchased by Ayala International Co. Ltd., a Philippine Corporation, who in turn hired Bechtel International Corporation headquartered in Gaithersburg, Maryland as Construction Manager. Bechtel has engaged a design team of consultants including an Architect, Structural Engineer, Mechanical Engineer, Electrical Engineer, Kitchen and Laundry Designers, and an Interior Decorator who has the largest influence on the interior (and exterior) appearance of the rehabilitated building.

The Campton Hotel is actually two buildings. The taller building, built about 1912, is 40 feet by 80 feet (12.2m x 24.4m) in plan and has sixteen stories plus a full basement. This building has a steel frame. The girders are connected to the columns with bracketed, knee braced connections. The floors are framed with a patented concrete joist 24 inches (61mm) o.c. system. The visible exterior walls are reinforced concrete. The exterior walls below the ninth floor are hollow terra cotta.

The lower building built about 1915 is 30 feet by 130 feet (9m x 39.5m) in plan. The rear part of the lower building is 45 feet (13.8m) wide and returns around the east end of the taller building. This building has seven stories plus a full basement. The building is constructed of reinforced concrete. Concrete beams about 7 feet (2.1m) on center span the transverse width of the building and support concrete slab floors. The exterior walls of the building are of reinforced concrete 6 inches (150mm) thick. The front (west) and south walls have a brick veneer finish.

All of the architectural and mechanical features of the buildings have been demolished and removed, leaving the structural shell. In a rehabilitation this extensive, the San Francisco Building Code requires the structure be braced to the seismic force level of the present code. Certain other seismic requirements are sometimes waived such as code requirements for a moment resisting steel frame for buildings exceeding 160 feet (48.8m) in height and ductility requirements in certain types of foundation systems.

The new bracing system for the sixteen-story building has been designed to resist seismic forces of about 0.05g resulting in a base shear of approximately 450 kips in each direction. These forces are resisted by a combination of new concrete exterior walls below the ninth floor and steel bracing on three sides of the open atrium. The two new concrete elevator shafts (except the exterior walls) resist no computed lateral forces.

The lower seven-story building has a computed seismic force of about 0.06g in the north-south direction resulting in a base shear of 360 kips and .07g in the east-west direction resulting in a base shear of 425 kips. The north-south forces are resisted by strengthening the existing front wall with gunite concrete, a new transverse cast-in-place shear wall near the front of the building and the new stair tower wall acting with the existing rear wall of the building. The east-west forces are resisted by strengthening parts of the exterior longitudinal walls with gunite concrete and the addition of an interior cast-in-place concrete shear wall near the rear of the building. No shear resistance was assigned to the existing exterior walls with the exception of the rear solid wall which has very low computed shear stresses.

Bechtel International as agent for the Owner has taken separate bids for the Structural, Mechanical, Electrical, Elevator and Architectural work to be done, and will supervise the contracts for the Owner. This rehabilitation project will result in a 140-room luxury hotel. The projected opening date for the hotel is September 1983. Ayala Corporation has engaged a Hotel Manager who will publicize the facility prior to its opening and administer its operation after it has opened.

George E. Greenwood

H. J. DEGENKOLB ASSOCIATES, ENGINEERS

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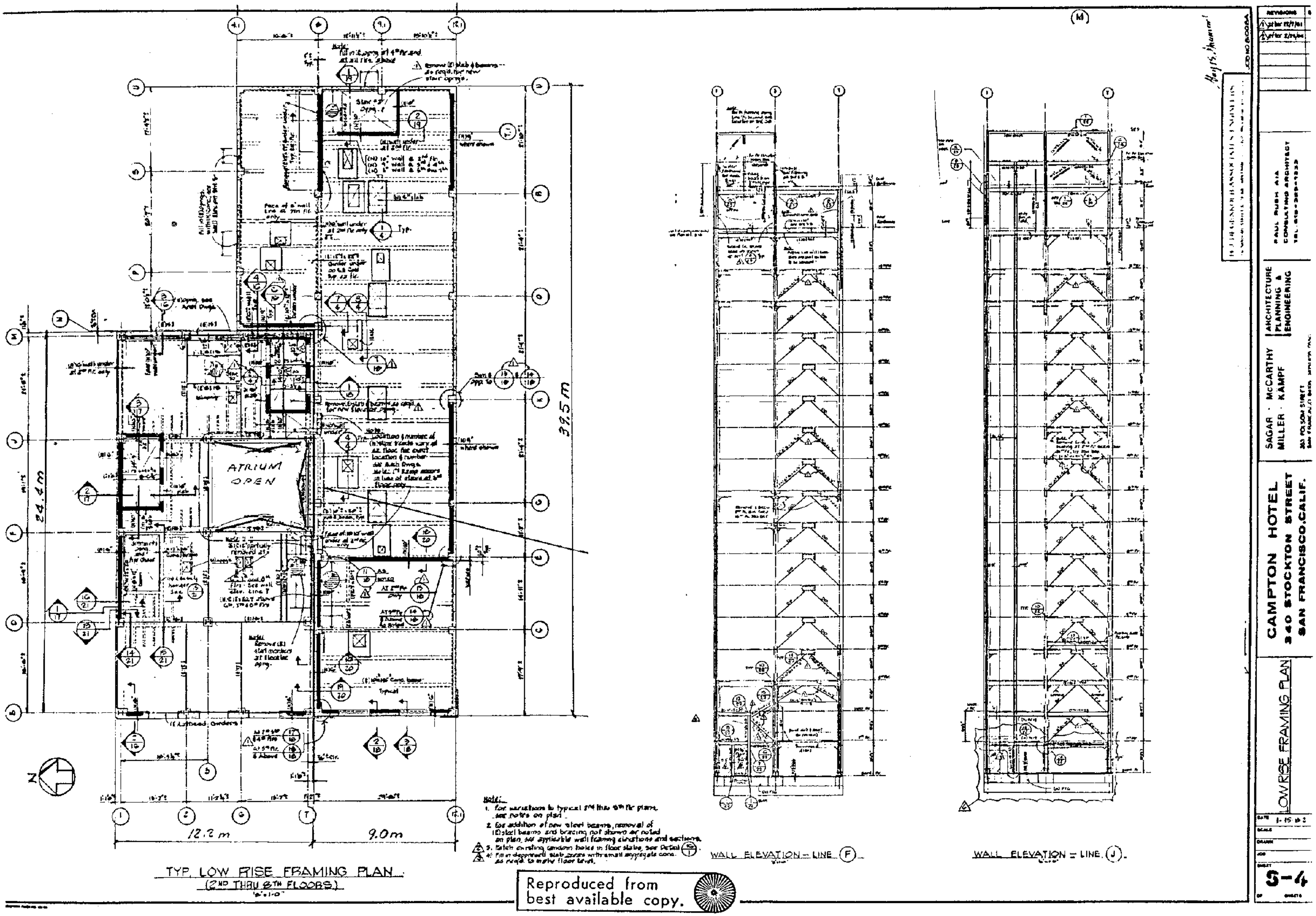
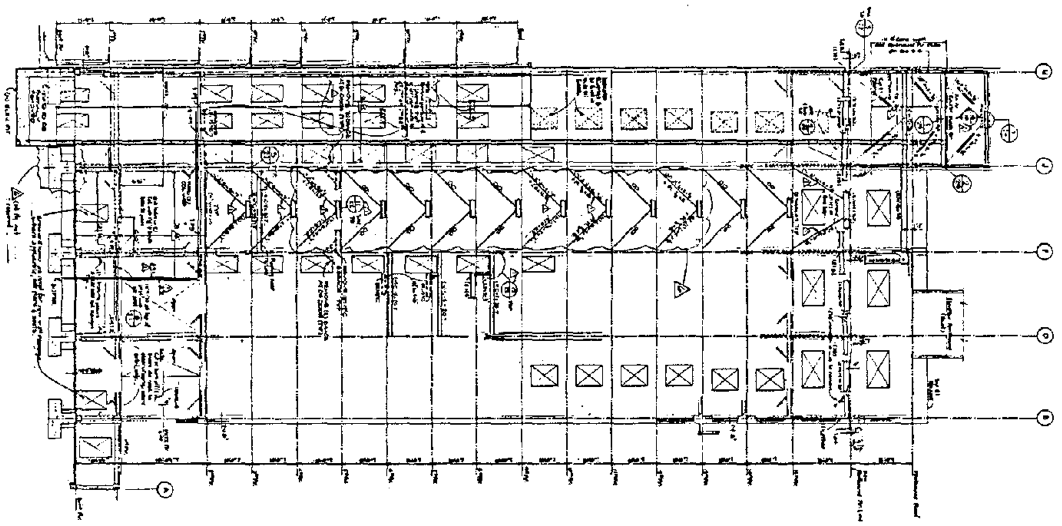


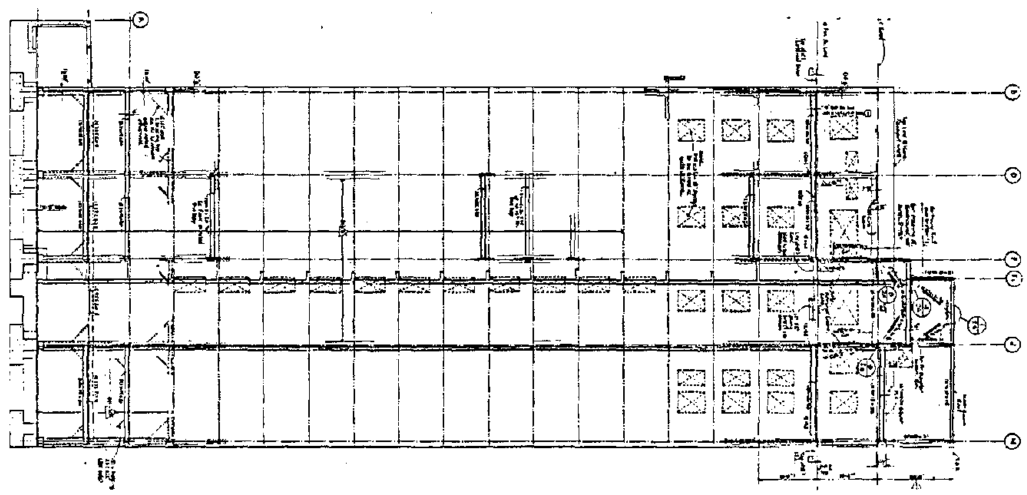
ATRIUM OPEN
39.5 m
24.4 m
12.2 m
9.0 m
TYP. LOW RISE FRAMING PLAN
(2ND THRU 8TH FLOORS)
WALL ELEVATION - LINE (F)
WALL ELEVATION = LINE (J)
CAMPTON HOTEL
340 STOCKTON STREET
SAN FRANCISCO, CALIF.
SAGAR · McCARTHY
MILLER · KAMPF
ARCHITECTURE
PLANNING &
ENGINEERING
PAUL RUSH AIA
CONSULTING ARCHITECT
LOW RISE FRAMING PLAN
S-4

5 1/1

WALL ELEVATION - LINE (7)



WALL ELEVATION - LINE (1)



H.J. DEGENKOLB ASSOCIATES ENGINEERS

JOB NO. 5006A

SHEET			SCALE	DATE					REVISIONS
S-7				1-15-82	WALL ELEVATIONS	CAMPTON HOTEL 340 STOCKTON STREET SAN FRANCISCO, CALIF.	SAGAR · McCARTHY MILLER · KAMPF 353 FOLSOM STREET SAN FRANCISCO 94105	ARCHITECTURE PLANNING & ENGINEERING	PAUL RUSH AIA CONSULTING ARCHITECT

OUTLINE OF RESEARCH ON ESTIMATION OF DAMAGES AND REPAIRS OF THE BUILDINGS DAMAGED BY THE MIYAGI-KEN-OKI EARTHQUAKE, JUNE, 1978

Toshio Shiga
Professor of Tohoku University

Akenori Shibata
Associate Professor of Tohoku University

Masaya Hirosawa
Director, Structural Engineering Department
Building Research Institute, Ministry of Construction

Presented at the Third Joint Meeting of Repair and Retrofit of Existing Structures, UJNR, San Francisco, California, U.S.A., May 13-15, 1982

ABSTRUCT

This report is a summarized translation by M. Hirosawa from the report which was compiled by Prof. T. Shiga and A. Shibata as one of the research products of the big project, "Developement of Post-Earthquake Measures for Buildings and Structures Damaged by Earthquake."

The objective of the research is to make clear actual conditions on buildings suffered from the 1978 Miyagi-ken-oki earthquake.

In the original report, detailes of investigated results on seven R.C. buildings, four steel buildings and one S.R.C. building are described and these results are summarized on the points of damage estimation and repair and strenghtening works executed on them.

In this report, outline of the original report and one example of the detailed investigated results are introduced.

1. RESEARCH OBJECTIVES

1.1 Objective and Background of the Big Project on the Structures Damaged by Earthquake.

Ministry of Construction, Japan started the Big Project in a 5 years' program from 1981. The objective of the project is to establish inspection method for damage dagree and guidelines for repair and strengthening method on buildings and grounds damaged by earthquake.

Concerning existing buildings which do not suffer from earthquake but are considred unsafe from earthquake, estimation methods for their seismic performances and retrofitting recommendations were already compiled and they are already applied to actual existing buildings.

However we don't have any consolidated recommendations on damaged buildings by earthquake similar to the aboves. Accordingly, we decided to carry out adequare experimental researches and related investigations and to establish recommendations necessary to buildings and grounds damaged by earthquake.

1.2 Purpose of the Investigation Research

This investigation research was carried out as one of the related researches to the big project by a research group[1)] and the immediate purpose of the investigation is to clarify actual conditions on damage estimation and retrofitting plans of several buildings damaged by the 1978, Miyagi-ken-oki Earthquake.

2. OUTLINE OF THE INVESTIGATION

2.1 Method to Select Objective Buildings

Numbers of buildings which were suffered from the 1978 Miyagi-ken-oki Earthquake and investigated at that date by members of A.I.J. are counted as follows in the lists of the investigation report by A.I.J..

R.C. buildings : Out of about eighty buildings in total, collapsed buildings are five and heavily damaged are more than ten.

Steel buildings : Out of one hundred and sixty in total, collapsed are six and heavily damaged are twenty.

Twelve collapsed or heavily damaged buildings were selected as objective ones for the investigation out of the above mentioned buildings. This selection were made after discussing about the following items.

a. Suitability as examples
b. Difficulty to get sufficient data
c. Amount of necessary investigation job

The selected buildings are listed up in Table 1 with their use.

Note) Members of the investigation research group :
Tohoki University : T. Shiga, M. Izumi, A. Shibata, M. Yamada, H. Mihashi, J. Shibuya, J. Takahashi.
Tohoku Institute of Technology : S. Kawamata, J. Suzuya, J. Onose, Y. Abe

Table 1 List of the objective Buildings

Kind of Construction	Use of the Buildings	Number of the Buildings
R.C.	School Governmental office	5 2) 7
Steel	Office Store house	2 2) 4
S.R.C.	Public apartment house	1

As shown in the list, reinforced concrete buildings are all public ones. This does not result from special reasons but result from only difficulty to get sufficient data. The fact that selected steel buildings are all private ones depends on that there is no public building suffered from severe damages. Further, as there is no steel reinforced concrete building of which super structure was remarkably damaged, a public apartment house with collapsed fundation piles was selected.

2.2 Measures Taken Soon After the Earthquake

Measures being taken soon after the earthquake and the outlines of their retrofitting works are summarized in Table 2.

Out of the listed twelve buildings, three buildings experienced two earthquakes in Feb. and June, 1978.

Reinforced concrete buildings

Concerning all of the investigated and seriously damaged buildings except two buildings of Izumi-High School, measures of "off limit" were taken from the next day of the earthquake. The actual processes up to the measure could not be made clear by this investigation except that "off limit" was decided by professores of the architectural structural branch in case of Tohoku Institute of Technology.

In cases of the prefectural Izumi-High School and the municipal Tonan-High School, it is not clear whether school authorities had decided "off limit" or prefectural (municipal) authorities had decided.

In cases of Ishigoe Community Center and Saigo Elementary School which suffered from the both earthquakes in Feb. and June, the situation is the same as the above.

Only in case of Tukidate Common Governmental Office, the building was continuously used after the both earthquakes basing on judgement by responsible person of the building.

It was supposed that judgement of "off limit" was done mainly basing on the estimation of the damage degree of columns. Measures of "off limit" were taken in case when allmost columns in the first floor were destroied and considered not to be safe from the vertical load. Such measures as taken in the above mentioned buildings and times for the measures were both considered to be generally adequate.

However, when the investigaters visited the objective buildings about one week after the earthquake, they knew that the class rooms in the second and third floor of the C buildings of Izumi-High School and teacher's room in the first floor of Tonan-High School had been continuously used and they recommended the responsible officer to stop to use.

In case of provite buildings, although they are not listed up in the table, the time to have decided that they were to be "off limit" and the damage dagree being used as standard for judgement are considered not to be different from the investigated cases. It was supporsed that many of these measures were carried out basing on the owner's judgement or the discussed result with the designers or constructores.

Steel buildings

The 26 steel buildings mentioned above which suffered from more damages than moderate were all off limited to enter. Concerning the process that these measures were taken, there are more unclear points than the case of R.C. buildings. It is because these are all private buildings and remaining data are very few.

These measures are considered to be judged by owners' themselves or to be decided by owners after consulting with designers and/or constructors.

It may be said that adopted measures and the times were generally resonable.

Steel framed reinforced concrete buildings

This publec apartment house suffered form the Earthquake at the stage that construction works had finished and finish works just started, and no habitant was there. As "A" building inclined to the south, city officials decided immediately to stop the work and "Off limit".

2.3 Repair and Retrofitting Work

Reinforced concrete buildings

Concerning the buildings of which damage degrees were generally judged serious, one of the counter measures, i.e., reconstruction after demolition, reuse after repair and/or retrofitting was adopted. On the buildings of half damaged, only one building was demolished and the others are now on reuse. In case of the former, they would think much of their defects caused by retrofitting than the possibility of retrofitting.

Technical judgements of demolition or reuse were generally done according to the assessments on the collapse dagree of the first floor column. In the case when the measures of reuse after repair were decided, not only repair but strengthening were adopted in every case.

And it is matter of course, such factors as oldness of buildings, functional damage after retrofitting, repair costs and other economic factors were taken into consideration to decide the measures. These technical judgement were done at earlier stage at the earthquake both in Feb. and June. However, the case of Tukidate Commen Governmental Office is fairly particular and the final decision to demolish and rebuild it was officially done in November of the year. Moreover, all of the above mentioned technical judgements and retrofitting designs were carried out under the counsel of professors councerned.

The costs of repair and retrofitting works of the investigated buildings are shown in Table 1. As shown in the table, the ratioes of the retrofitting costs to the initial construction cost are between 12%

and 37%.

Three buildings which had already suffered from the earlier earthquake in Feb., suffered from another earthquake in June at the different stages of retrofitting work.

Saigo Elementary School was severly damaged by the earlier earthquake and just before retrofitting work at the latter earthquake. However fortunately, the damage didn't progress. On the other hand, Tukidate Common Governmental Office was half damaged by the earlier earthquake and just finished retrofitting works at the latter earthquake. The damage of the building progressed fairly at the next earthquake and the demolition was decided.

Steel buildings

Some of the buildings of which damage degrees were reported serious in the A.I.J. Report were demolished as a whole and on the others only upper stories were domolished. Further, the buildings reported as half damaged were all repaired and/or retrofitted.

The processes resulted in the above mentioned measures were not made clear by this investigation, because all the buildings were private and informations were very few.

In case of steel buildings, the costs for demolition work and for retrofitting work could not be so clear as in the case of R.C. buildings.

Steel framed reinforced concrete building (Damages to A.C. piles)

As there was no resident and no recruitment for residence in this public apartment house at the earthquake, they could select adequare measure.

Although many Autoclaved piles were damaged seriously at their upper portions, it was made clear that a gravel layer of about 8m thick exists below G.L.-6.5m. So, it was judged possible and appropriate to construct newly direct foundations on the gravel layer.

University professors cooperated to make up the retrofitting plans and B.R.I. staffs cooperated to make plan to execute the retrofitting works .

The ratio of the retrofitting cost to the total construction cost reaches to 33%.

3. REPAIRING AND STRENGTHENING OF RC DAMAGED BUILDING (IN THE CASE OF ISHIGOECHO COMMUNITY CENTER)

3.1 Synopsis

This paper is intended to present an example of repairing and strenghtening of the RC building which have suffered from severe damages during earthquake motion. Although the building concerned having received sereve earthquake damages on February 20th, 1978, was shocked again by another earthquake on June 12th in the same year when it was under repairing and strenghtening, the extent of structural damages due to the second earthquake remained limitedly small because of an appropriate restoration program for the first earthquake damages.

3.2 Structural Damages (Due to the First Earthquake)

As is shown in Fig. 1, structural damages due to the first earthquake were concentrated on the first story. Shear walls and columns with spandrel or skirting walls in the first story seriously failed in shear or flexural-shear.

On the contrary, structural damages in the second story seemed to remain small.

3.3 First Phase Inspection for Earthquake Damages

Through the first phase inspection for earthquake damages, such as damage-measurement by the eye, concrete strength prediction by means of Schmidt hammer, microtremor measurement for building and surrounding soil, and observation for overturning of grave stones, the following items were made clear.

a. The quality and strength of concrete and construction work for concrete-casting are questionable.
b. The predominant period of soil in the vicinity of the building is remarkably long, which implies that the soil condition is not desirable.
c. It is likely from observation for overturned grave stones that the maximum ground acceleration have reached about 0.4g.

As a result of variety of inspections for earthquake damages, severe building damages were believed to take plece by the following facts;

a. The building response acceleration was considerably in high level.
b. Shear walls were eccentrically placed in the longitudinal direction, which would result in torsional response.
c. Many columns were those with only a short flexible portion which would lead to force concentration due to their high stiffness.
d. Shear walls would not be expected to carry shear force sufficiently, as the quality of concrete and the workmanship of construction are very poor.

After the first phase inspections for earthquake damages, the building was considered to be capable of sustaining vertical loads except for a few number of columns and walls. It was concluded, however, that an immediate damage-restoration was necessary because during the coming earthquake, building damages would expand to a large extent even if the building collapse would be fortunately aboided.

3.4 Damage-Restoration Program

It was far from easy to conclude on the basis of inspection results whether the building should be demolished for re-building or could be restored for reuse. However, it was finally decided that the building would be reused after the appropriate restoration because for the financial reason the town authorities had a strong intention to use it again.

As for the damage-restoration program, repairing of damaged portion of the building only seemed to be insufficient to ensure the seismic-resistant capacity of the building against the coming earthquake.

Consequently an extensive strengthening of the building was determined to be conducted primarily by means of installing new shear walls.

The reasons why the proposal to newly place shear walls were chosen as a means for enhancing the seismic-resistant capacity are as follows.

a. If the principle of column-strengthening is adopted without placing any new wall, most of the columns should be strengthened, which will result in large increase of construction work and cost, compared with those for wall-placing. In addition, when column sizes are increased without increasing flexible portion of column height, columns are more likely to be of shear-failing type.

b. Strengthening of columns with short flexible portion of height by attaching new side walls leads to shear force concentration on strengthened columns due to increasing of their stiffness, which will possibly result in brittle shear failure.

c. The principle of new-wall-placing has advantages of improving the current condition that walls are eccentrically plased and of increasing shear-force-carrying capacity of walls as well as decreasing shear force demand imposed on columns.

Thus it was concluded that new-wall-placing was the most effective and appropriate strengthening method for the restoration program, and as is shown in Fig. 2, the location of new-wall-placing was determined such that the eccentricity in stiffeness might be reduced as much as possible.

3.5 Effectiveness of Restoration

During the restoration work, the building was hit agin by an earthquake on June 12, 1978, at which time new walls had already been placed, while existing columns and walls had not yet repaired.

The extent of earthquake damages for this time seemed to be restrained in comparison with that for the last time although the restoration work had not completely been finished, which meant that the restoration program was sufficiently effective for increasing the seismic-resistant capacity of the building. This could also be proved by the fact that the building period was shortened due to the stiffness recovery by strenghtening i.e., the precominant periods in the longitudinal direction of the building obtained from the microtremor measurement were found to be 0.31sec. and 0.22sec., respecively immediately after February earthquake and after the restoration work.

4. CONCLUSION

The followings are items recommended for future plan basing on the investigated results.

1) Concerning decision of countermeasures soon after earthquake and judgement of "keep off", estimations mainly on damage degrees of columns in case of R/C buildings and mainly on permanent story drifts in case of steel buildings are often used as the criteria. Accordingly, judgement for countermeasures may be possible to be done by architectural engineers etc. Other than structural engineers except the case in which damage degree is delicate for the decision.

Execution of training to master the essentials for the damage estimation is desirable.

2) Direction and advisory for the countermeasures shall be done under carefull attension because these shall be accompanied with responsibility. Therefore,it is deriable to consolidate an adequate execution system for direction and advisory on countermeasures to be taken soon after earthquake.

3) Concerning the technical judgements on whether a certain damaged building shall be demolished or may be rapaired, the similar estimations to that in the case of decision of the measures soon after arthquake are often used as the criteria. However, these judgements usually depend on specialists in structural engineering being different from the case mentioned above,

4) Final decisions for demolition or repair are usually done depending on not only technical judgement but judgements on such factors as economical restriction, possibility for an alternative building and so on.

It is one of serious cases that technical judgement would be neglected.

5) Classifing damages into several grades shall be conduct by a certain common scale from the both points of prevention of earthquake damage and of advancement of research. It hoped that common scales to every structures would be decided.

ACKNOWLEDGMENTS

The authors wish to thank the members of the researcher's group on this subject for their contribution to carry out many detailed investigations and to compile the investigated results.

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Table 1a A LIST OF INVESTIGATED RESULTS ON R/C BUILDINGS

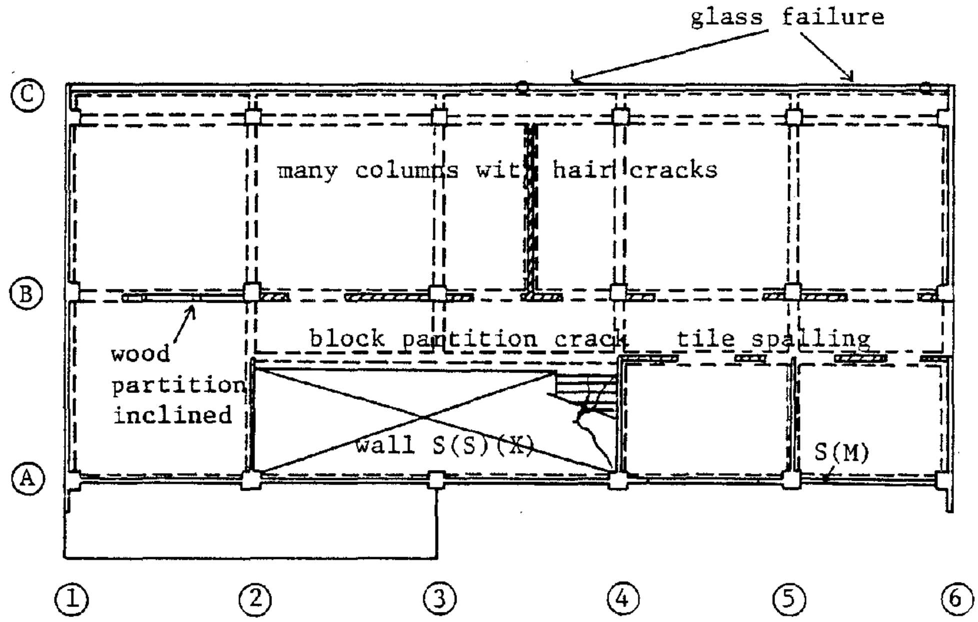
Name of the Building	Number of stories (above/under the Ground)	Occurrence Time of The Earthquake (1978)	Damage Degree	Outline of the Damages	Counter-Measure Soon After the Earthquake	Repair and Strengthening Works: Plan	Repair and Strengthening Works: Term	Repair and Strengthening Works: Cost (Million Yen)	Repair and Strengthening Works: Objective Total Floor Area (m^2)	Repair Cost / Initial Total Cost (Current Price) (%)	Repair Cost / Ob. Total Fl. Area (Tousand yen/m^2)
Ishigoecho Community Center	2/0	Feb.	Serious	Shear failure of the 1st fl. walls S.F. of a part of columns	Off limit	Increase of shear walls Repair of columns & walls	May, 1978- Aug., 1978	1,800	634	21	28.4
		June	Slight	S.F. of the 2nd fl. walls Slight damages in the 1st fl.	Off limit	Reset of conc. to the 2nd fl. walls Retrofitting of the 2nd fl. slabs					
Saigo Elementary School	2/0	Feb.	Serious	S.F. of the south side columns and the north side walls at the 1st fl.	Off Limit (Only No.1 Building)	Demolition reconstruction (No.1 Building) Increase of shear walls (No.2 and 3 Buildings)	Sep., 1978- May, 1979	5,650 (No.2 and 3 Buildings)	1,504	37	37.6
		June	Serious	No progress in the damages	Same as the Above	No change in the plan					
Tukidate Common Governmental Office	3/1	Feb.	Half	Shear cracks in the north side walls Flexure cracks in the south side columns	Keep on Use	Repair by epoxy resin of the cracks of walls and columns	March, 1978- April, 1978				
		June	Half	Progress of the above damages	Same as the Above	Demolition and reconstruction		28,000 (Rough Estimate)	4,368	45 (Abandoned)	64.1
Izumi High School[2]	3/0	June	Serious	S.F. of the north side columns at the 1st fl. of the A, B and C buildings	Off Limit (Only A Building)	Increase of shear walls in the north side, repair and/ or strengthening of columns	Sep., 1978- March, 1979	16,870	7,850	20	21.5
Tohoku Institute of Technology — No.5 Building[3]	5/3	June	Serious	S.F. of the north side columns at the 1st-4th fl.	Off Limit (The 1st-4th Floor)	Construction of steel braces in the ridge direction Increase of shear walls in the span direction	Sep., 1978- Dec., 1979	24,680	12,500	15	19.7
Tohoku Institute of Technology — No.3 Building	4/0	June	Serious	S.F. of the north side columns at the 1st-4th fl.	Off Limit	Demolition and reconstruction	March, 1979- April, 1980 (Reconstruction)	11,600 (Estimate)	3,629	23 (Abandoned)	32.0
Tonan High School	3/0	June	Serious	S.F. fo the outer columns and S. cracks of the inner columns at the 1st fl.	Off Limit	Demolition and reconstruction	March, 1979- Oct., 1980 (Reconstruction)		4,211		

Note : 2), 3) shows Refference No. · Damage Degree ; Serious Damage – Damage state for which strengthening is considered necessary in large scale
Half Damge – Damage state for which partial repair and and strengthening is considered necessary

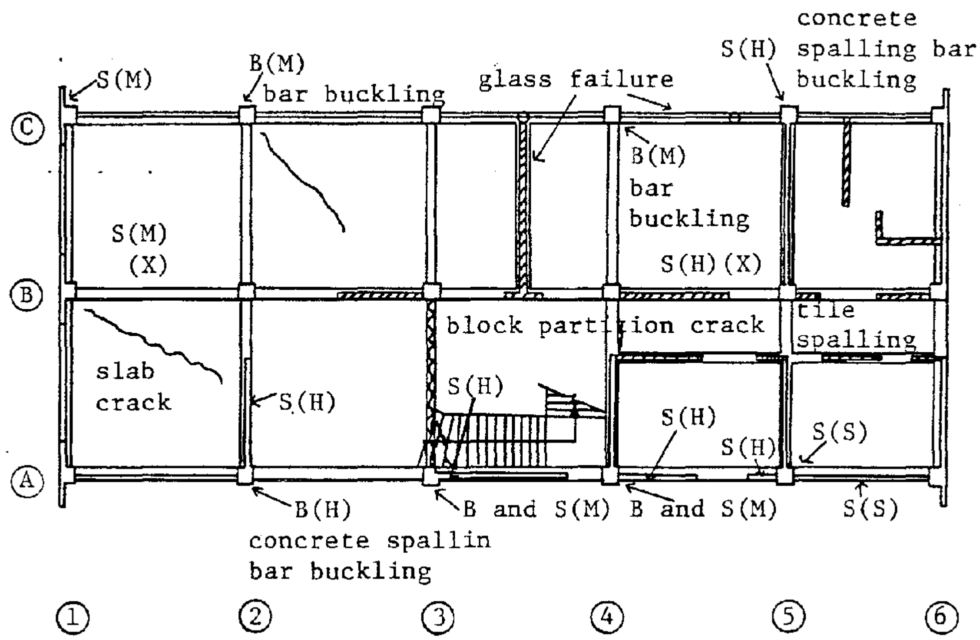
Table 1b A LIST OF INVESTIGATED RESULTS ON STEEL AND S.R.C. BUILDINGS

Name of the Building		Number of Stories	Damage Degree	Outline of the Damages	Counter-Measure Soon Ater the Earthquake	Repair and Strengthening Works				Repair Cost	Repair Cost
						Plan	Term	Cost (Million Yen)	Objective Total Floor Area (m^2)	Initial Total Cost (Current Price) (%)	Objective Total Fl. Area (Tousand Yen/m^2)
S.U. Ware-house	No.1·2·4 Building	2	Serious	Large permanent drift in the span direction Fracture of braces in both direction	Off Limit	Demolision and reconstruction (No.1, 2) Repair and renewal of braces (NO.4)	April, 1979-Oct., 1979 (No.1,2) June, 1978-Sep., 1978 (No. 4)				
	No.3·8·9·10 Building	1	Half	Fracture of braces in the ridge direction	Off Limit	Repair, renewal and increase of braces	June, 1978-Sep., 1978				
	No.5·6·7 Building	2	Collapse	Collapse (No.5, 7) Large permanent drift (No.6)	Off Limit	Demolision and reconstruction	Sep., 1978-Feb., 1979				
G.S.K. (Office) Building		2	Half	Fracture of all braces, yield of H-shaped columns at the 1st fl.	Off Limit	Repair, renewal and increase of braces		3,000	2,494	30	12.0
N.K. (Office) Building		4	Serious	Large permanent drift at the 3rd fl. in the ridge direction	Stop to Use, Temporary Braces of Wire	Demolision of the 3rd and 4th floors					
Y.G. Warehouse		2	Slight	Welded porsions of steel-piped braces	Temporary Braces of Wire	Renewal of braces	June, 1978-Oct., 1978	2,300	6,092	12	3.8
Sendai Pubric Apartment House		11	Serious	Compressive failure of A.C. piles at the south side, inclined to the south	Off Limit	Change to direct foundations from pile support. Compensation of permanent drift by Jacking up	Jan., 1979-March, 1980	28,500	11,392	33	25.0

Note : Serious Damage - Permanent story drift $\geq$ 1/30
Half Damage - Permanent story drift < 1/30



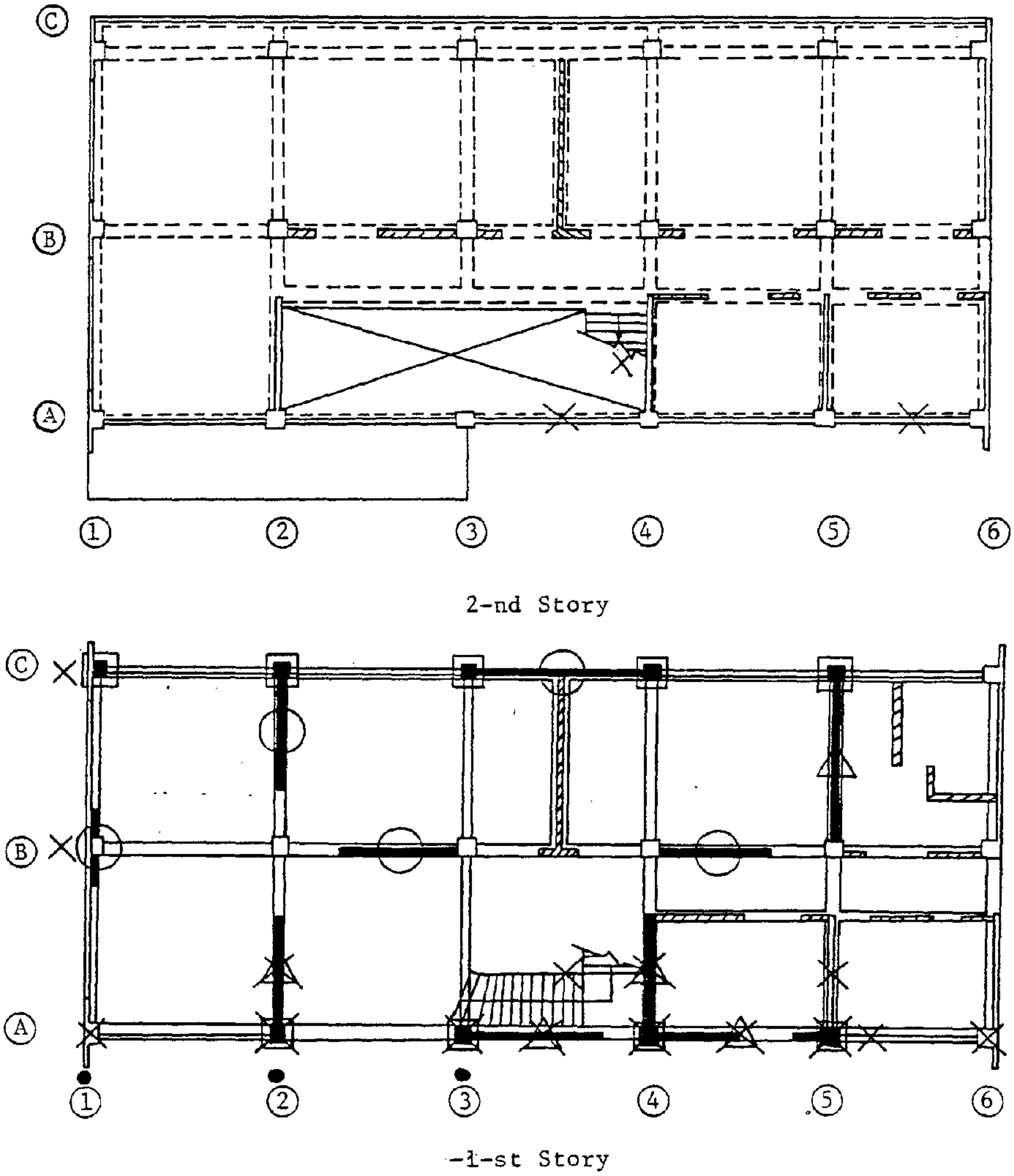
2-nd Story



1-st Story

<u>Note</u>
F : flexural failure
S : shear failure
(H) : heavy damge
(M) : medium damage
(S) : slight damage
(X) : X shape crack

Fig 1 Damage-Pattern
(due to Feb. Earthquake)



Note

- : new-wall placing, finished
- : wall repair, finished
- : column repair, finished
- : Crack repair, not yet
- : Steel pipe supporting roof
- : wall repair, not yet
- : column, repair, underway

Fig. 2 Damage-Restoration
(when June Earthquake occured)

THE REHABILITATION OF HISTORY CORNER OF THE STANFORD UNIVERSITY MAIN QUAD

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THE REHABILITATION OF HISTORY CORNER OF THE STANFORD UNIVERSITY MAIN QUAD

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HISTORY OF THE QUAD

The Main Quadrangle of buildings at Stanford, known simply as "the Quad", is composed of an inner quadrangle of generally one story buildings surrounding a large courtyard and a concentric outer quadrangle composed of larger buildings. Both the inner and outer quad buildings are connected by continuous arched arcades. Since the inception of the University, the Quad has been the focus of the campus and the center of undergraduate education. The architecture of the Quad, particularly the arcade arches, has become symbolic of Stanford.

The Quad was the original campus facility, construction starting in 1887, and, due to a variety of delays, was not completed until 1904. The structures on the quad as originally built have exterior load bearing masonry walls and interior wood bearing walls. The interior framing is supplemented as necessary for space requirements by wood or steel beams supported by wood or cast iron columns. Floors and roof were wood joists with straight sheathing. The two foot thick exterior masonry walls were brick with a sandstone facing. Steel rod anchors (known as "dog" or "government" anchors), typical of construction of the time, connected every third or fourth joist to the walls. Roofing material was red tile, which has since become traditional at Stanford.

Only two years after completion, the great San Francisco earthquake of 1906 did severe damage to the Quad buildings (Ref. 1). A few structures, notably Memorial Church and Memorial Arch, were never rebuilt to their original configuration. The balance of the buildings and arcades were repaired and strengthened in accordance with knowledge of the day. History Corner had several exterior walls rebuilt at that time with unreinforced concrete replacing the brick masonry of the typical construction, as well as some roof strengthening and extensive patching of interior plaster cracks. The cost of these repairs was estimated at the time as being $27,100 (Ref. 2).

Rehabilitation

Besides some remodeling, the major outer quad buildings were not touched until 1962, when a major reconstruction, including complete seismic upgrading, was undertaken on Physics Corner (also known as Math Corner). Reconstruction continued in 1967 with Jordan Hall and most recently with Margaret Jacks Hall and History Corner (See Figure 1). Work will also soon be underway on Building 120 in a long range project of complete Quad renovation which may take as long as twenty years (Ref. 3).

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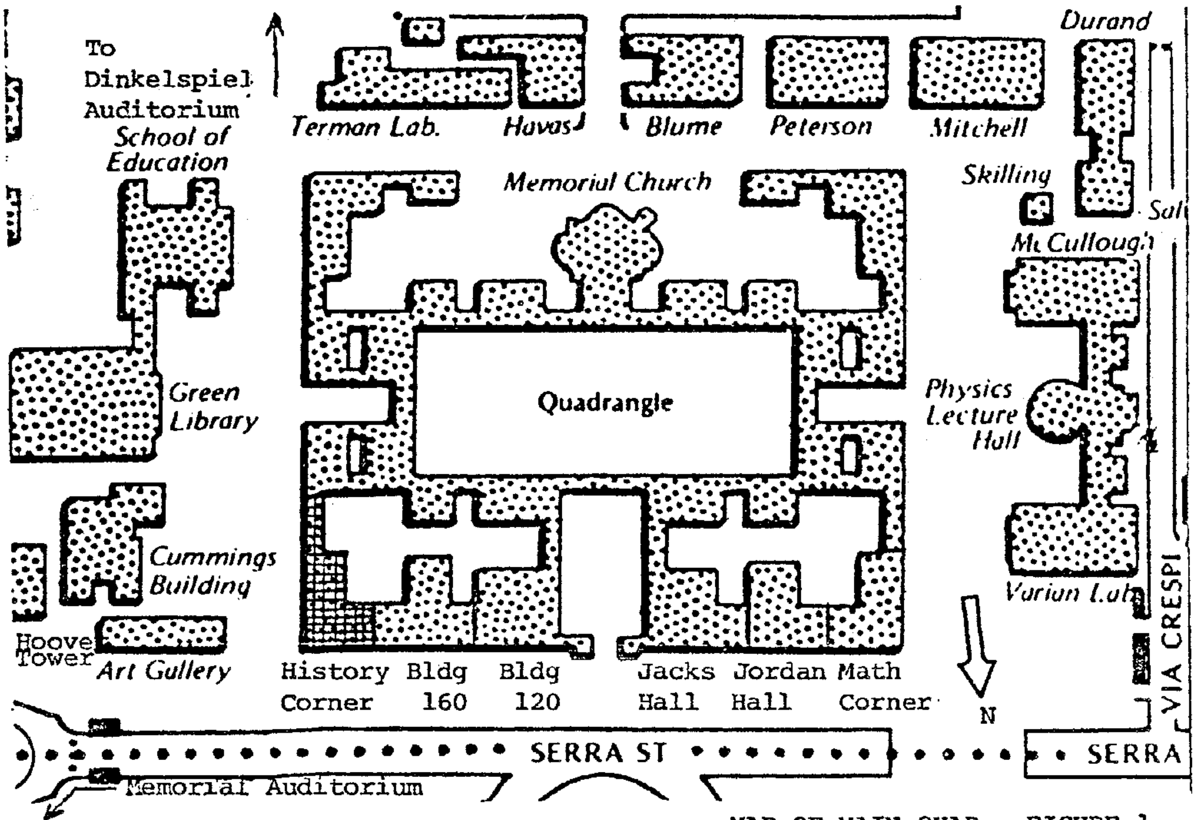


MAP OF MAIN QUAD - FIGURE 1

The general size and shape of History Corner can be seen in Figure 2. The original construction has no separation with adjacent buildings and major buildings such as History Corner and Building 160 shared common fire walls. The History Corner reconstruction is unique on the Quad in that the floors after reconstruction matched the original. The large story heights (See Section AA, Figure 2) have previously been taken advantage of and additional floors added during reconstruction (known at Stanford as "stuffing the building"). Here, only a mechanical mezzanine ("M" in Section AA) was added. In addition, the interior design will attempt to preserve the essential qualities of the original Quad construction although the building's room layout and circulation were completely revised.

Seismic Inadequacies

Problems with building types typical of Quad construction are well documented and have been often demonstrated in earthquakes in other areas since 1906. The unreinforced exterior walls are often incapable of spanning between floors (out of plane) for lateral loads and invariably have inadequate ties at the floor levels. The walls are also inadequate to act as primary shear walls (in plane) and the shear delivery is severely limited by the wood floors attempting to act as diaphragms. The failure of these walls in either mode is especially dangerous as they are load bearing.

In addition to the seismic inadequacies, the typical Quad buildings have outdated mechanical and electrical systems and in many cases are no longer functionally adequate due to ever changing physical requirements.

Due to this combination of inadequacies, the generalized solution in Quad upgrading has been to completely gut the interior and build a new structure within the existing exterior walls. This new interior structure could be structural steel or concrete and both materials have been successfully used. The exterior walls have been completely shotcreted on their interior side. This process solves both the out of plane bending and in plane shear problems as well as providing additional vertical load carrying capability and lateral stiffness. Shotcrete has been used in lieu of cast concrete primarily because of the economic advantages of saving forming. The process also has the advantages of low shrinkage against the existing wall, improved bond, and a better assurance of filling cavities in the brick. The new roofs have been framed in structural steel due to their complex shapes and slopes (See Figure 2).

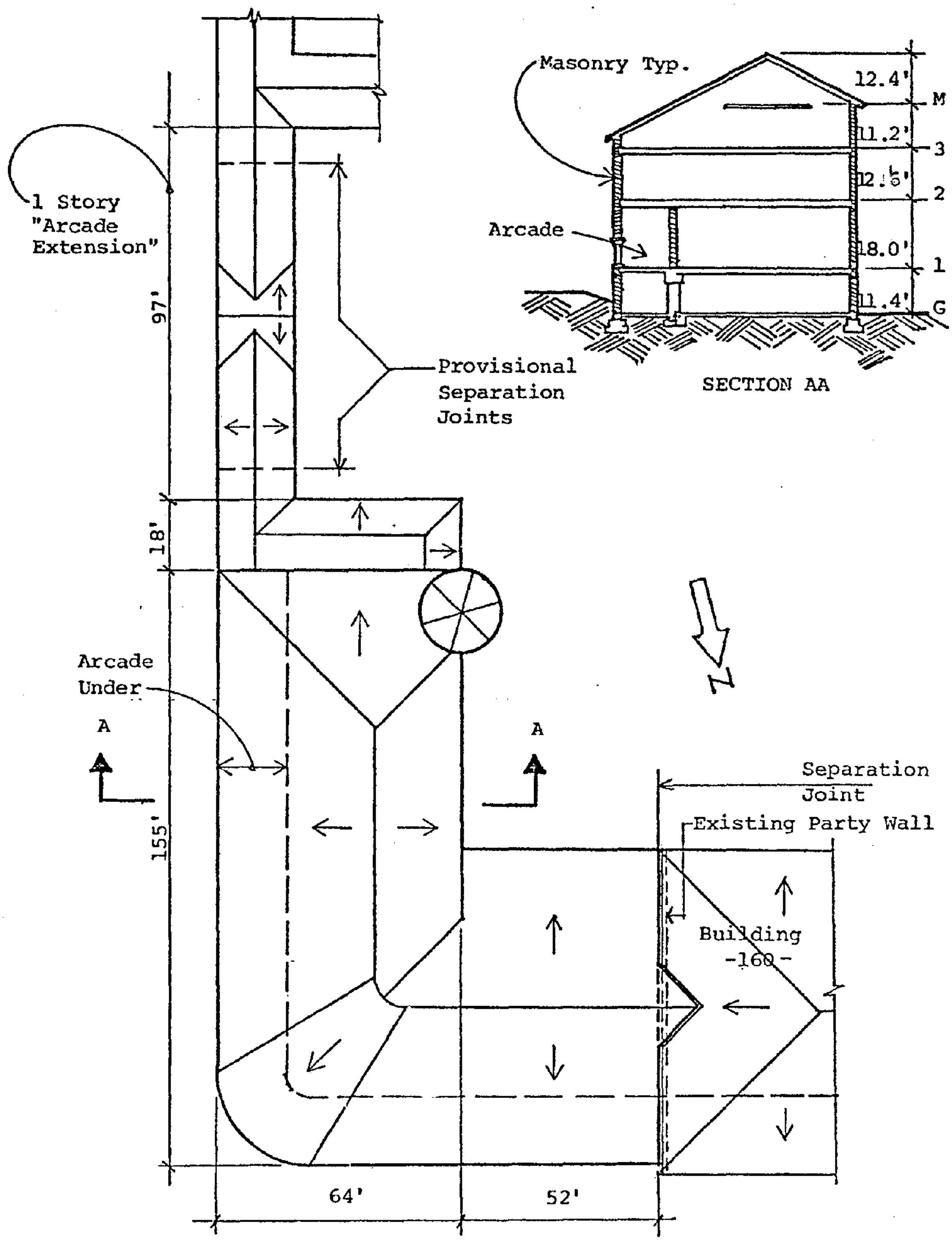
HISTORY CORNER

The History Corner Rehabilitation was similar to previous projects in an overall sense, but the fact that the building was not to be "stuffed" gave added flexibility to the design team. In addition, different programs, different architects, slightly different building configurations, and the continuous evolution of earthquake engineering all tend to make each project unique. One apparent consistency is that both early rehabilitations, Math Corner and Jordan Hall, had a new interior of structural steel, and both recent projects, Jacks Hall and History Corner, are of reinforced concrete.

Configuration Problems

Engineering problems in rehabilitation projects are generally related to the limiting affect of the existing building. In the case of History Corner, the shape of the exterior shell was given and could not be changed in any major way. The building was L shaped in plan and had a major vertical discontinuity where the interior arcade masonry wall did not continue through to the basement. The structure was also connected to other seismically inadequate buildings not included in this project.

It is becoming increasingly well recognized that configuration characteristics such as these have a significant effect on seismic performance even though there are no related code requirements. Mitigation of these configuration problems requires a great deal of cooperation from the architect, who is forced to consider seismic requirements in his development of internal planning and aesthetics. For example, problems from an "L" shape can be minimized by a seismic separation between the legs, or by providing substantial ties between the legs at the reentrant corner. At History Corner, a joint was unacceptable from an external preservation standpoint so the ties at the corner were required. The ties were provided, and a full height atrium and stair were designed around them. Similarly, a concrete shear wall was placed in the basement under the inner arcade wall to eliminate that discontinuity and this wall strongly affected the planning of the basement spaces. The configuration problem most difficult to completely solve is the lack of separation joints in the original quad construction. That is, History Corner, Building 160, and Building 120 are all essentially one building and the one story "arcades extension" to the south of History Corner is also built-in at both ends. A two foot thick masonry party wall was shared by History Corner and Building 160 at their junction (Figure 2).



ROOF PLAN AND GENERAL LAYOUT
FIGURE 2

Seismic Separations

Although no seismic separations had been provided between projects on the previous three upgradings on the northwest portion of the quad, a separation was placed between History Corner and Building 160 as part of this project. Consideration of damaging earthquake motion in both directions with and without the joint as following indicates the desirability of such a separation:

1. East West Direction:

   A. Without a separation: The added and reinforced walls furnished in rehabilitation will cause a large stiffness incompatibility between History Corner and Building 160. This incompatability will cause a load transfer from 160 to the new walls, either — overloading them or causing a failure somewhere along the transfer system. It is not economically feasible to either design the walls for this overload or upgrade the transfer system of an adjacent building.

   B. With a separation: No interaction would occur. The primary concern is whether Building 160, to be left unupgraded for an undertermined length of time, is weakened by its separation from History Corner. Because of the similarity of construction and of the exterior walls, it can be assumed that, prior to rehabilitation of History Corner, the two buildings would be reasonably compatible and each would therefore attempt to laterally support their own weight; a separation would therefore not increase lateral loads expected to be resisted by Building 160.

2. North South Direction:

   A. Without a separation: Similar to the east-west, stiffness incompatibility would cause either increased loads to the new walls or damage at the interconnection.

   B. With a separation: Since the joint is placed on the History Corner side of the preexisting party wall, that wall will now take no loads from History Corner and will therefore be more capable of laterally supporting Building 160. The Building 160 lateral resistance has therefore been increased in this direction.

The south edge of History Corner was also solidly connected to another element: a one story covered passage (the "arcade extension" See Figure 2) to the next group of quad buildings. There are two major requirements for support of the so called "free standing" arcades against seismic forces: 1) Provision of an overall support system, that is, the provision of adequate number and size of lateral force resisting elements to prevent collapse of the structure as a whole, and 2) provision of local support to prevent localized failures which may cause injury (i.e. falling blocks of sandstone), or more importantly, to prevent localized failures of structural elements which may cause collapse of a portion of the structure (i.e. local columns spanning to the diaphragm but not a part of the overall lateral system).

For the overall lateral support of the arcades there are two philosophically distinct methods possible: 1) Structurally separate the arcades from the buildings and make them self supporting by provision of ductile moment resisting frames or other lateral force resisting elements if architecturally acceptable, or 2) Provide a stiff roof of attic level diaphragm, attached to the buildings at each end of the arcade, which will take the arcade lateral forces to the shear walls of the buildings.

The structural separation of the arcade has the advantage of freeing the arcade from dependence on the adjacent buildings for structural support thereby allowing the arcades to be structurally upgraded on a time schedule independent of the buildings being upgraded. A disadvantage of this method is that the arcades are inherently without properly proportioned elements for really good independent lateral resistance. They can be made adequate to meet the minimum requirements of the Uniform Building Code but cannot be made stiff enough to prevent some damage from occurring in a large earthquake without violating the present architectural envelope.

The method of overall support relying on attachment of the arcades to the buildings can provide an inherently stiffer lateral resisting system since the buildings are quite stiff. This is only true in cases where the arcade diaphragm has reasonable length to width proportions of no more than 5 or 6 to 1. Even with these proportions some damage can be anticipated in a major earthquake. The shorter the arcade the less anticipated damage.

The major disadvantage of the attached method of support is that it makes little sense to upgrade the arcades by attaching them to buildings which are in themselves inadequate; therefore this type of solution should be recommended only where the buildings at both ends of the arcade have been previously structurally upgraded or are scheduled to be upgraded at the same time as the arcade.

At History Corner, a separate lateral force system was necessary for the arcade extension because the building at the south end would not be upgraded immediately and the length of the arcade was such that the roof diaphragm would be unable to span from one building to the next. A stiff reinforced concrete frame was therefore provided within several existing pilasters. However, seismic separation joints were only acceptable at the center of arch spans, which required substantial work on both sides of the joint. This was not possible at the south end of the arcade because it went beyond the scope of this project. It was therefore decided to install what was called a provisional joint at both ends. This included a separation in the new reinforcing structure, but no separation in the original, exposed sandstone and masonry wall. This maintains the architectural and structural symmetry for the arcade but requires acceptance of potential minor nonstructural damage at the joints until the building at the south end can be upgraded and the joints completed. The extent of these joints is so small and the existing material crossing the joints is so weak that the force interaction problem discussed in connection with Building 160 was not considered significant and the potential damage acceptable.

CONSTRUCTION SEQUENCING

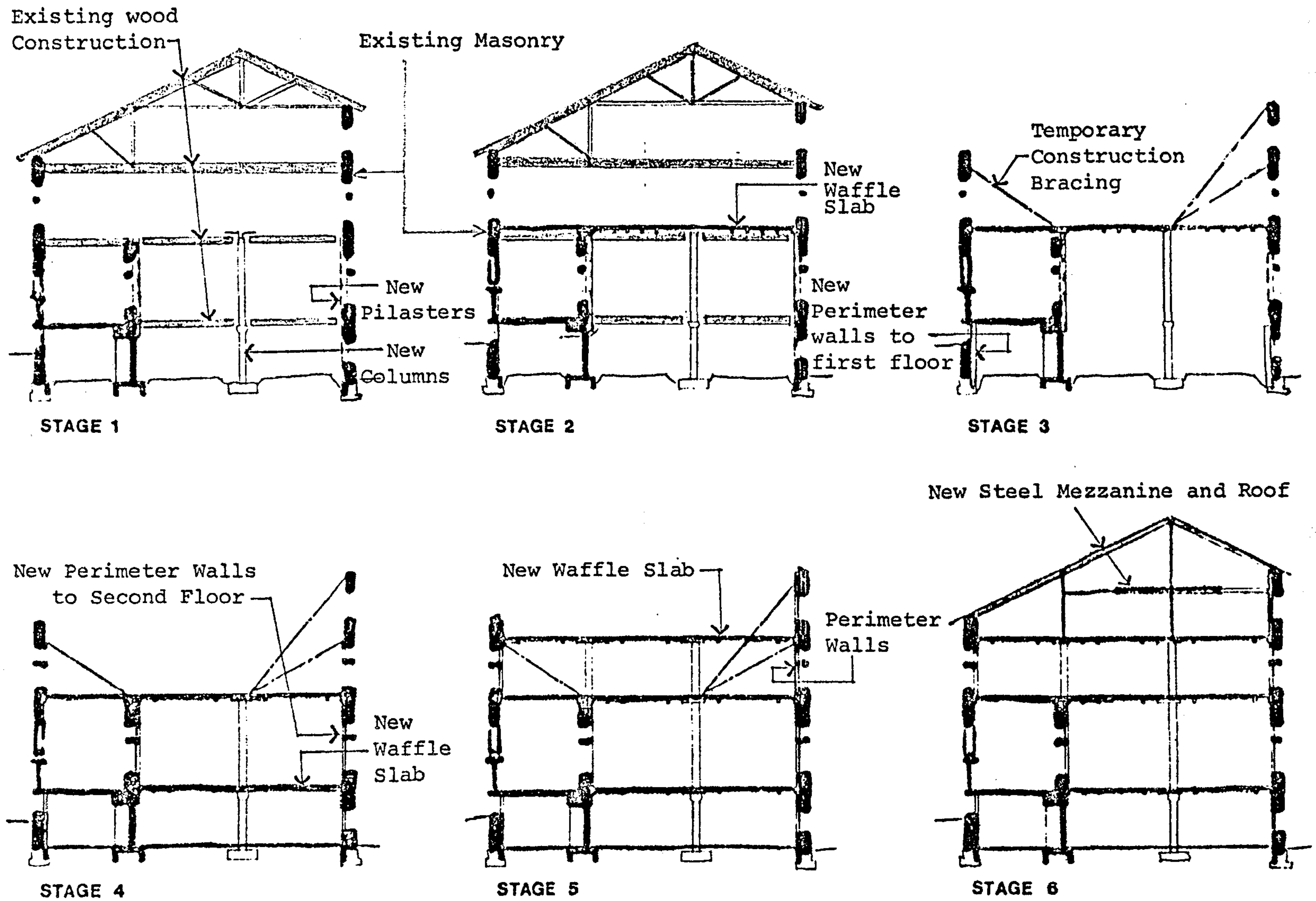
Conventional construction technique in historical upgradings like Stanford's has most often been to temporarily brace the exterior walls that are to be saved, gut the building, and then build the new internal structure. It was estimated that such temporary external braces at History Corner would cost $200,000, so efforts were made to eliminate their need. The first scheme considered consisted of supplying a minimum of new vertical supports and lateral elements and casting the new first and second floors on top of the existing wood floors (Refer to Section AA, Figure 2). Once in place, this new system would provide sufficient bracing for the exterior walls that the existing interior could be gutted and construction proceed from the second floor upward. Special procedures and temporary details required for such a sequence were estimated at only $80,000 so a substantial savings was possible compared to convential exterior bracing. However, the new first floor in this scheme would necessarily be located above the existing, thus causing misalignment with the existing arcade slab and forcing undesirable ramped or stepped entrances. The final scheme was therefore devised to take advantage of the potential savings and also create a first floor flush with the arcade. This scheme is shown in Figure 3.

Stage 1 consisted of providing sufficient vertical supports through the existing floors to support the new second floor, which would be cast using the existing second floor as a form. See Figure 4. Perimeter pilasters were cast in chases cut into the brick walls that would both vertically support the new floor and also span approximately thirty feet from ground to the second floor, providing temporary support for the exterior walls. Portions of the complete perimeter shotcrete wall system were placed in Stage 1 to provide lateral support for this intermediate structure. It was also necessary to install at this time two story high trusses which were used to span a large classroom in the basement. At no time, however, was sufficient existing flooring to be removed to significantly reduce the pre-existing integrity of the building.

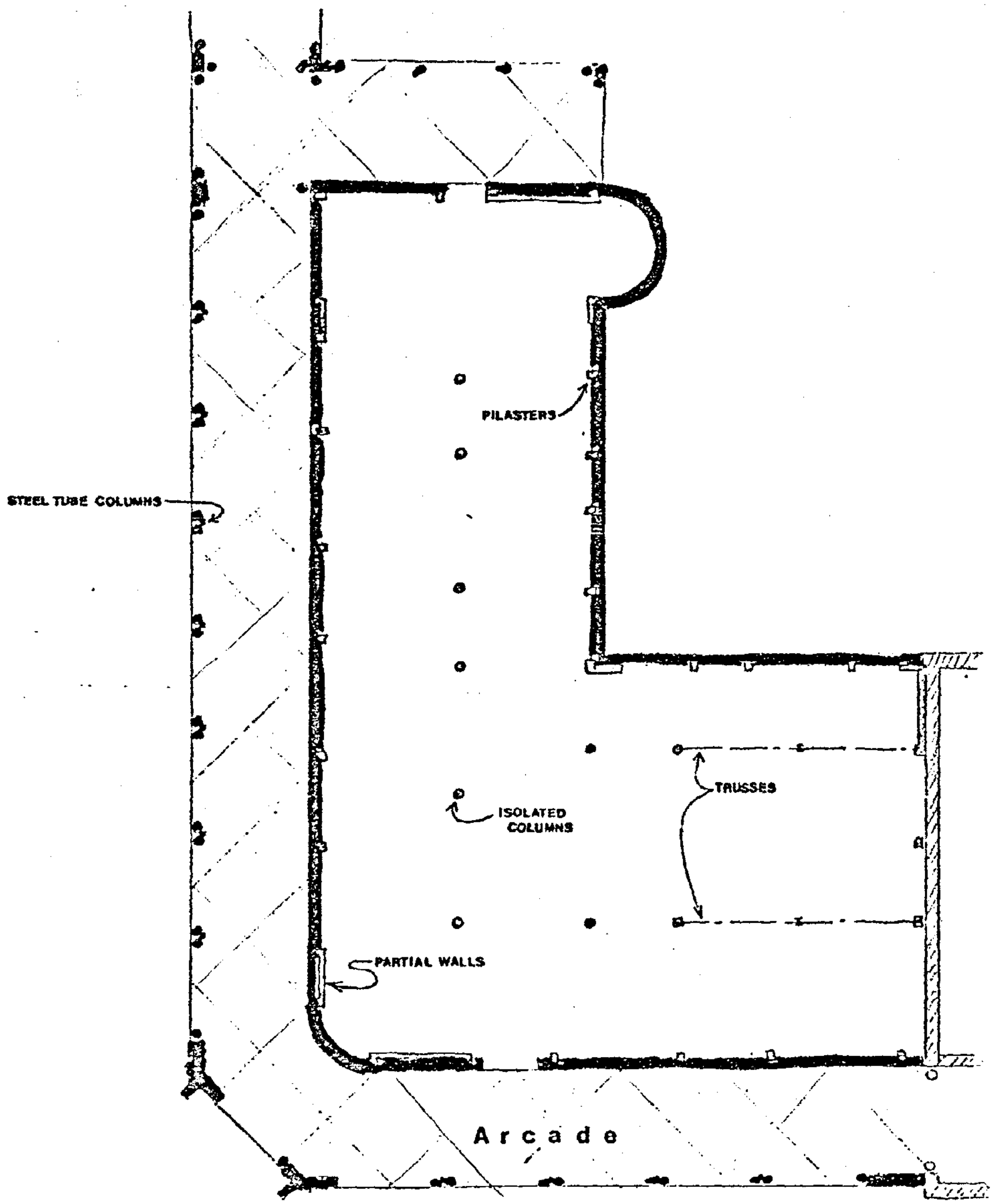
After the new second floor was cast (Stage 2), and minor temporary braces installed from the second floor up (Stage 3), the existing interior could be completely removed. The perimeter shotcreting could then be accomplished in the basement, and in Stage 4, the new first floor waffle slab formed and cast in a conventional manner. Construction would then proceed from the second floor up (Stage 5) and the temporary braces would be removed upon completion of the new third floor. Installation of the steel mezzanine and roof framing completed the process in Stage 6.

The special two story work in Stage 1 reduced the savings from the original scheme, but final estimates put the savings at $100,000. Several additional advantages included provision of additional storage (on the existing floors) on the very small site, the ability to work almost continuously during the winter months, and a significant construction time savings when the contractor opted to perform most of Stage 4 and 5 concurrently.

The project is scheduled to be completed during the summer of 1979.



CONSTRUCTION SEQUENCING - FIGURE 3



EXTENT OF STAGE 1 WORK

FIGURE 4

PROJECT COSTS: Bid August 2, 1977: ENR 1720; Area: 56,000 S.F.

Category	Total	Cost Per Square Foot
Structural	$1,252,377	$22.13
Underpinning	23,000	.41
Demolition, Bracing & Shoring	441,613	7.80
Architectural, Site Work	1,791,039	31.65
Mechanical/Electrical	996,961	17.61
Field Overhead, Bond	350,000	6.18
	$4,854,990	$85.78

PROJECT ACKNOWLEDGEMENTS

Architect: SMP/EHDD - A Joint Venture
(Stone Marraccini & Patterson/Esherick Homsey Dodge & Davis)
Structural, Soils Engineers: Rutherford & Chekene
Mechanical, Electrical Engineer: Hayakawa Associates
Landscape Architect: Richard A. Vignolo
Construction Consultant: Rudolph and Sletten, General Contractor
Contractor: Carl W. Olson and Sons Co.

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BUILDING 120 RECONSTRUCTION
SOCIOLOGY & COMMUNICATIONS DEPARTMENTS
STANFORD UNIVERSITY

Introduction

This latest project in a continuing series of projects at the Quadrangle which began in 1977. The basic building plan is to reconstruct new academic facilities inside the north and west facades of the original 1898 structure. The completed facility (still under construction) will house two departments, and will include a TV studio, small library, and various offices and teaching space. The roof profile as seen from the north will remain exactly as before to preserve the original Quadrangle line. Construction will be completed in 1982 at a cost of approximately $7,500,000. For general historical setting, refer to "The Rehabilitation of History Corner of the Stanford University Main Quad" paper.

Reconstruction Strategy

The results of a long planning process concluded that the new building should contain more square footage than could be contained within the limits of the existing walls and roof, and therefore the only choice was to go down. Since the building already had a basement, this decision meant that a new basement, one level lower, was required to accommodate the necessary floor area. At the same time, the program called for a large television studio, whose space requirements were such that two-story height was necessary. Therefore, the studio was a logical choice for some of this new sub-basement space.

In addition to space requirements, adjacent existing facilities such as valuable trees, a covered arcade constructed with sandstone arches, and adjacent buildings were required to remain in place without damage. New construction was required to meet the requirements of 1976 UBC, as well as being tough enough to provide lateral support for the existing exterior masonry walls, which are 24" thick.

Foundation Designs

Because of the complexity of supporting multi-story unreinforced masonry walls, and accommodating excavation of some 15' below existing grade, five different foundation systems were utilized:

1. Underpinning of existing gravity concrete basement walls supporting masonry above

2. Sequentially-loaded underpinning of basement walls

3 Vertical drilled friction piers, with haunches and anchor piers to support new walls and existing walls

4. Battered piers for underpinning of adjacent 4-story masonry wall (party wall with Building 160)

5. Vertical drilled piers with pre-tensioned tiebacks to support a 26' deep vertical cut at the TV studio area.

In order to sequence the removal of all existing interior construction, while properly supporting the masonry walls, it was necessary to drill the piers from the existing basement level first. Following completion of the underpinning and piers, the excavation was completed, tiebacks stressed, and construction of sub-basement walls, footings, etc. initiated. Drilled piers were generally 30" in diameter, heavily reinforced, with anchor piers of 18" ∅ diameter. Anchor piers were required to account for the significant eccentricity of vertical loads on the piers.

Lateral Force Design

The completed structure is considered a shear-wall structure, K=1.33, consisting of generally cast-in-place or shotcreted concrete walls, with many openings. The previously established technique of the application of shotcrete to masonry has been used many times before, and was the same here, wherein sandblasting, epoxied dowels and other anchors are used to create a homogeneous or anchored wall assembly, in some cases 36" thick. For a discussion of questions of separation from existing adjacent buildings refer to page 5 of the reference History Corner paper, which is essentially applicable to this project as well.

Precast Concrete Structural Elements

In order to make the completed facades on the south and east portions compatible with the general quadrangle qualities, a technique was developed to utilize precast concrete window elements with a stone-like appearance. In this case these elements consisted of massive sill and mullion elements, which were reinforced with bars designed for spandrel action of the shear walls. Erected on temporary pipe supports, the window elements were cast in with the balance of the shear walls, creating an integral element of the shear wall.

Exterior appearance of the precast units was made by special casting techniques against special handcrafted molds. Colored concrete was used to match the natural earthen tones of the surrounding sandstone. Following stripping of the wall forms, veneer units will be installed, which are also concrete, to simulate the stone. These individual concrete "stones" also have specially created surface textures.

Project Acknowledgements

Architect: SMP/EHDD - A Joint Venture
Structural, Soils Engineers: Rutherford & Chekene
Mechanical, Electrical Engineers: Hayakawa Associates
Landscape Architect: Richard A. Vignolo
Construction Consultant: Peter Adamson & Assoc.
Contractor: Ralph Larsen Co.

Harold A. Davis
Rutherford & Chekene
San Francisco, California

ON THE DEVELOPMENT OF POST-EARTHQUAKE MEASURES FOR BUILDING AND STRUCTURES DAMAGED BY EARTHQUAKES

MAY, 1982

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Presented at the Third Joint Meeting of Repair and Retrofit of Existing Structures, UJNR, San Francisco, California, U.S.A., May 13-15, 1982

ABSTRUCT

An extensive research project entitled "Post-Earthquake Measures for Buildings and Structures Damaged by Earthquakes" was started in five-years' program form April, 1981 by the Ministry of Construction, Japan.

The objective of this project is to develope guidelines for measuring and inspection method of damaged structures, for repair and strengthening method and for selecting optimum repair and strengthening method.

The Building Research Institute (BRI) is in charge of the development for such building structures as of reinforced concrete, of steel and of wood and for grounds of building sites.

This paper describes the outline of research results done for the first years, and also introduces an example of the succeeding four-years' program.

1. BACKGROUND OF THE PROJECT

Japanese regulations on seiemic design method for all kinds of buildings were revised in 1981 from working stress design with the base shear coefficient C = 0.2 to ultimate strength design with C = 1.0 including effect of ductility. Before that, in 1977 and 1978, evaluation methods for seismic performance of existing R/C and steel buildings were compiled respectively under supervision of the Ministry of Construction. And at the same time, several critical quantitative values for their seismic performances were proposed basing on experience of recent severe earthquakes.

However, we have experienced not so few minor damages and partial failures at every earthquakes although severe structural damages have extremely decreased.

But now there are no guidelines for inspection method of damaged structures and assessment method to repair and strenghthen them.

On the other hand, it was reported that a tremendous lot of people were killed by the aftershock of the Tangshan Earthquake in 1976 and the similar reports to this are not so few in the world.

Basing on the above-mentioned background, this project has been intiated. Table 1 shows subjects and their contents in the project for building structures.

2. RESEARCH ORGANIZATION

To develop the project, a research committee including the staffs of the Tsukuba University was organized in the Building Research Institute (BRI) and BRI made research contract with the Research Center for National Land Development Technology (RCNLDT). The center organized a Committee on Post-Earthquake Measures for Building Structures Damaged by Earthquakes chaired by Prof. Hajime Umemura, as well as four subcommittees, i.e., Subcommittee on Reinforced Concrete Building chaired by Prof. Tsuneo Okada, Subcommittee on Steel Buildings chaired by Prof. Koichi Takanashi, Subcommittee on Wooden Houses chaired by Prof. Hideo Sugiyama and Subcommittee on Grounds of Building Site chaired by Prof. Yasunori Koizumi.

Functions of the committees in BRI and RCNLDT are as follows.

i) Functions of the committee in BRI

a. Compilation of drafts for entire research program and anual plan for the project.

b. Formulation of budget requirement.

c. Compilation of specification for annual research trust contract.

d. Decision of persons on duty and details of jurisdictional research.

ii) Function of the committee and subcommittees in RCNLDT

a. Revision of the drafts for entire research program and anual plan of each branch.

b. Decision of a person or groups on duty for the trusted research.

c. Compilation and assessment of annual research products.

Accordingly, all of related researches are to be carried out fundamentally by researchers or research groups from governmental or private research institutes or universities under the counsel of the committees.

The total budget of the project will amount to about one million dollars in the branch of buildings. Almost half of the budget will be allocated to the BRI staffs including those of the Tsukuba University and the remainder will be trusted to the Center.

3. ACHIEVEMENT IN THE FIRST YEAR, 1981

3.1 Progress

The research in the fiscal year of 1981 includes the followings in order to help establish the systematic methods of earthquake damage inspection.

a. Planning of entire research program

b. Literature survey on measuring method and inspection method of buildings and grounds damaged by earthquakes

c. Study of the actual cases of buildings and land damages caused by the 1978 Miyagi-ken-oki Earthquake, for investigating the method of existing damage surveys and the criterion used to judge the damage degree

Concering item a, a draft was proposed by the committee in BRI and it was partially revised and authorized as a tentative one by the committee and the subcommittees in RCNLDT.

The research on item b was carried out by the BRI staffs including a staff of the Tsukuba University.

The researches on the item c were carried out by a group of researchers of Tohoku University and Tohoku Institute of Technology on buildings and by Prof. A. Asada, Tohoku Institute of Technology on lands.

Outline of the research products are summarized hereinafter.

3.2 Entire Research Program

Entire research program for this project was compiled in this fiscal year, and yearly plans of the research themes for individual research fields and research plans for the fiscal year, 1978 were also decided. Main contents are as follows.

(1) Fundamental Policies to Enforce the Project

i) Common Items

a. It must be actively done to call for contribution from related individual researchers and private institutes.

b. Extent of experimental studies shall be limited to the followings. However, guidelines for the remaining other structures are to be conpiled and reffered to basing on results of literature surveies.
(Concrete buildings) reinforced concrete framed structures except steel framed reinforced concrete, prestressed concrete and wall-typed or prefabricated concrete structures
(Steel buildings) structures with welded or bolted connections

except those with rivetted connections

(Wooden buildings) buildings by the Japanese traditional system except ones by the 2 x 4 system or prefabricated ones

(Grounds) natural or artificial slopes and retaining walls except piles

(Repairing and strengthening structural systems) selected systems by preliminary experiments or literature surveies

c. Specimens used to get data for damage inspection are to be also planed to get data for repair and strengthening after adequate restoration.

d. As damages of buildings due to the collapse of underground structures are recently observed, items on foundation structures must be considered in guidelines for damage inspection.

e. Objective value of seiemic performance for retrofitting design of damaged buildings may be analytically decided. The value may be tentatively considered smaller than that regulated in the revised enforcement order of the Building Standard Law.

Sufficient discussion with related administrative officials is necessary to decide the final value.

f. This entire program was compiled in the first year of the research project and not so few points remain uncertain. As many researches progress hereafter, some partial change of the program may become necessary. The details of research plans shall be discussed in the subcommittees and chage of the program shall be correpondingly discussed in the committees.

ii) Images of the Final Products

a. Inspection Criteria on Damage Degree

Two inspection criteria will be compiled, one is for urgent another is for restoration.

(Urgent inspection criteria)

These criteria are used to distinguish buildings to be demolished and ones to be restored,within few days after earthquake.

Also such items as follows shall be carefully considered to compile the criteria, i.e. qualified inspectors, inspection of falling exterior finishing and habitants' information on damages.

(Inspection criteria for restoration)

The ofjectives of the criteria are to make clear exact damage degree by detailed investigation and to know residuary aseismic ability of damaged buildings relative to objective ability of retrofitting design.

b. Guidelines for Temporary and Lasting Rehabilitation Technics

In case of restoration of a building damaged by earthquake, it is necessary to estimate adequately the presumed seismic performance after retrofitting as a whole building.

Assessment method for seismic performance as a whole structure is to be treated in the third theme and estimation methods for seismic performance of retrofitted structural members by certain method will be mainly treated in this theme keeping close contact with compilation of the third theme.

In the guidelines, certain retrofitting technics for structural members and so on, to be developed properly for damaged degree of buildings and grounds, will be shown with the following items.

- Outline of the retrofitting technic
- Objective characteristic performance : ductile or strong, temporary or lasting
- Retrofitting plan : buildings, damage degree and damaged portions adequate for the technic
- Retrofitting technic and details
- Retrofitting calculation : calculation procedure, calculation method for strength, rigidity and ductility of retrofitted portion
- Related item : cost, term and important points of the retrofitting work

Also the following items shall be carefully considered to compile the guidelines, i.e. treatment of new material and patent right, easy operation and easy acquisition of materials for temporary restoration.

c. Assessment Methods of Repair and Strengthening

Under this theme assessment methods will be developed in order to estimate collectively various performances of damaged buildings after repair and retrofitting.

Objective performance for assessment will be divided into two categories, i.e., structual performance and others.

Assessment on seismic performance will be done fundamentally by comparing external seismic force index with aseismic ability index of a building after repair and retrofitting. Accordingly, retrofitting design procedure will be described here basing on the research products of the second theme.

On the other hand, whether a damaged building had better be retrofitted and used again or not depends on not only feasibility of seismic retrofitting but also several social and economical factors. Assessment method on such factors as period and workability of repair works, functional damage during the period, spectacles impaired by repair works, repair cost and other economic benefits shall be also developed.

Finally by developing synthetic assessment method basing on assessments for each factors, it will become possible to sellect optimum repair and retrofitting method.

3.3 An Example of Research Plan in the Case of Steel Structures

i) Typical Patterns of the Damage of Steel Structures

The typical damage patterns of steel structures are generally classified into the followings

a. Damage of nonstructural elements
b. Concentration of damage into a particular story
c. Damage due to insufficient strength of jointed parts
d. Damage due to insufficient buckling strength of primary members

ii) Repair of Damaged Steel Structures

It may be usually estimated that damaged steel structures will be repaired by the following methods. There is, however, no certain inspection criterion and method for restoration at this stage.

a. Damaged nonstructural elements are replaced by new ones which can follow larger story drift.
b. Damaged building is reused as it stands in case when observed permanent story drift is small. In case of large permanent story drift, the building is reused after adequate repair and retroritting, or is demolished.
c. In case of the fracture of joints or connections, secondary members may be replaced, while primary members may not be replaced economically.
d. Girders and columns with local buckling may be generally left as

they are. Members which buckle compressively or laterally may be replaced or adequarely retrofitted.

iii) Planning of the Entire Research Program on Steel Structures

a. Inspection Methods on Damage Degree

(Urgent inspection criteria)

These criteria are used for judging mainly whether there is a danger of nonstructural elements falling down or not. Furthermore, as for the structures such as warehouses and factories whose structural members can be easily observed, the damaged state of them should be taken into consideration as a matter of course.

(Inspection criteria for the restoration)

These criteria are consisted of two phase inspections. In the first phase inspection, the damage degree of buildings will be estimated relatively and classified into several damage categories. In the second phase inspection, the residual seismic ability of damaged buildings will be quantitatively evaluated in comparison with the objective ability of restoration design.

In order to perform the inspection described above, it may be necessary to develop instruments which can be used to observe damage states of structural members inside the finishings.

b. Restoration Technique

(Objective ability for restoration and restoration design)

The objective ability for restoration will be the seismic ability reguired by the existing Japanese Building Code. However, they may not be necessary to be exactly identical.

In restoration design, alternative restoration methods or their combinations may be adopted. One is the restoration by repairing the damaged parts, and the other is the restoration by strengthening the damaged building. It may be difficult to specify the restoration methods according to the damage degree.

(Research plan for restoration methods)

The main subject of experimental study on the restoration methods is limited to the primary structural steel members and the steel structures with welded and bolted connections. The research needs for the inspection criteria and the repairing methods are recommended as follows ; buckled members (columns with local buckling and girders with lateral buckling), bolted and welded connections, and column-to-base connections.

3.4 Other Products

Beside the above-mentioned product on the research program, the following research products were obtained in the first year.

(1) Case Study on Buildings Suffered form the 1978 Miyagi-ken-oki Earthquake

This case study was carried out on twelve-buildings damaged seriously by the Miyagi-ken-oki Earthquake and the following items were investigated.

- Outline of the buildings
- Contents of the investigations carried out soon after the earthquake
- Outline of the investigated damages
- Countermeasures and the times immediately after the earthquake, and persons in charge of the decision
- Outline of the second investigation
- Planning and method of retrofitting, and persons in charge of the decision

The investigated results are described in the refference[1].

(2) Case Study on Lands Damaged by the 1978 Miyagi-ken-oki Earthquake[2].

By the study, artificially developed lands suffered from the earthquake were classified into three types according to the damage patterns and the modes of topographical alteration from hilly land. And, it was found that the fill-up ground and the cut ground could be identified by the measurement of ground's microtremor.

(3) Literature Survey on Measuring Method and Inspection Method of Buildings and Grounds Damaged by the past Earthquake in Japan[3),4)].

The products obtained by the survey are as follows.

(Measuring Method)

a. As for the measuring apparatus for earthquake damages of R.C. buildings, measures and plumb bobs have been widely used for measuring displacements and uneven settlements .

Other than the above, measurement of fundamental periods of damaged building and of concrete strength by Schmidt hammer which seems important to invesigate causes of earthquake damages have been occasionally made.

b. There are no remarks on the measuring apparatus which have been used for steel buildings. However, there are some reports which refer to the measurement of story drift or uneven settlement. For these measurement, measures or plumb bobs seem to have been used.

c. As for wooden structures, there are also no remarks on measuring apparatus. However, there are many reports which refer to the measurement of inclination of wooden houses. Clinometers on the market seem to have been used for that purpose.

d. As for the damage to grounds at buildings site, extent of damage is judged firstly by eyes. Transit surveying, leveling or aerial surveying are made in special cases to get technical informations about failure mode, scale and distribution. In the Miyagi-ken-oki Earthquake , one case of ground slide observation by measn of clinometers and strain-guages set in the ground to investigate the method of preventing second large sliding is reported.

e. Many kinds of propositions and suggestions have been made for the determination of so-called attennation laws of earthquake ground motions revising the classical formulae proposed by T.Tsuboi and K.Kanai.

In recent years, in addition, research works on the characteristics of earthquake ground motions in source region based on the concept of causative faults are remarkably increasing.

(Inspection Method)

a. The purposes of inspection for earthquake damages to RC buildings are damage-extent classification and investigation on damage-causes, etc.

Most of the inspection methods seem to be entrusted to inspector's subjectivity. So the inspection standard in which there are no rooms for inspector's subjective judgements is considered to be favourable.

b. Purposes of inspection on steel buildings are statistical survey for damaged buildings, investigation of damage causes and evaluation for damage insurance payments.

4, 6 and 2 examples of inspection methods corresponding to the purposes stated above have been collected respectively.

In most of the cases, however, the details for practical use are not clear and therefore the inspection results might definitely be affected by inspector's subjectivity.

c. As for wooden structures, 5 examples of inspection methods used in the past have been collected, the objectives of them are to compile damages statistics, to estimate repair cost and to investigate damage causes. Likely to other structures judgement might not be free from inspector's subjectivity because the application rules were not clear.

Studies on the inspection method have been done also in Architectural Institute of Japan and the Marine and Fire Insurance Association of Japan. The results may be considered to be of use for the present project.

d. The after-earthquake inspection for home lands has never been made. Most investigations are made for classification of failure modes in order to compile satistics of home land damages.

In the Miyagi-ken-oki Earthquake, however,regions which the first and second level hazard were designated in order to prevent secondary disasters. The judgement was made not through the specified standard but through results of discussion by experts.

ACKNOWLEDGMENTS

The witers wish to thank Prof. Hajime Umemura, all staffs of the Committees and the Subcommettees in RCNLDT and BRI and the staffs of RCNLDT for their contribution to carry out the first year programs.

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4) B.R.I. Report, 56.I.2.(2) B.R.I., M. Hirosawa "Literature Survey on the Inspection Methods Applied for the Structures Damaged by Earthquakes", (in Japanese)

Table 1 Program of Future Studies for Building Structures

Subjects			Contents					Final Output
Themes	Sub-themes	Subjects	1981 FY	1982 FY	1983 FY	1984 FY	1985FY	Results
I. Inspection Method for structures Damaged by Earthquake	I-1. Measuring Method	Building Structures and Ground of Building Site	Collection and analysis of existing measuring methods	·Selection of factors for assessing damage degree ·Inspection of past measuring instruments for damage degree	·Development of measuring technics for damage degree on building structures, piles and ground	·Manual of measuring technics for damage degree		Guideline for measuring method and inspection method
	I-2. Inspection Method	Building Structures and Ground of Building Site	Collection of past experiences	·Analysis and grading of factors for assessing damage degree ·Preliminary experiments of building elements for assessing damage degree of building structure	·Continuumation of experiments ·Framevork of inspection method	·Continuation of experiments ·Mennals of inspection methods (temporary and lasting)		
II. Repair and strengthening Method	II-1. Building Structures	Building Structures		·Collection and analysis of existing R-S technics ·Selection of effective R-S technics ·Schedule for experimental studies and preliminary tests	Experimental studies for assessing selected temporary and lasting R-S technics (R.C., Steel, Wooden structures)	·Continuation of experimental studies ·Framework of manuals of temporary and lasting R-S technics for building structures	·Continuation of experimental studies ·Manuals of temporary and lasting R-S technics for building structures	Guidelines for R-S technics
		Ground of Building Site		·Investigation of damaged grounds and retaining walls ·Schedule for experimental studies and preliminary tests for R-S technics	Experimental studies on seismic performance of slopes and retaining walls	·Experimental studies on rain resistance performance ·Framework of manuals of R-S technics for grounds and retaining walls	·Manuals of R-S technics for grounds of building site	
III. Assessment method of Repair and Strengthening	III-1. Factors to be constdered for Repair and strengthening methods	Building Structures		·Investigation on the factors for assessing total performance of repaired or strengthened buildings damaged at past earthquake	·Institution of factors for optimum repairing and strengthening procedure ·Weighting of the factors			Guidelines for assessment of repair and strenghtening
	III-2. Assessment method of repair and streng-thening	Building Structures			·Development of assessment methods for each factors	·Continuation of development of assessment method for each factors ·Framework of guideline to select optimum repairing and strengthening technics	·Guideline of method to assess repairing and strengthening technics considereing seismic, functional performance and investiment effect etc.	

EARTHQUAKE BRACING PROGRAM

UNITED STATES GEOLOGIC SURVEY

Menlo Park, California

The large extent of damage caused by the 1971 San Fernando Valley Earthquake raised serious concern about the adequacy of the then prevailing seismic design codes and the safety of existing buildings.

The United States Geological Survey in 1973 commissioned URS/J.A. Blume to evaluate the three major buildings at the Menlo Park, California, site, Buildings No. 1, 2 & 3.

Building No. 1, designed in 1953, is a 2 story steel frame building with timber floor and roof construction. Lateral resistance was provided by a combination of steel frames and diagonally wood sheathed walls.

Buildings 2 and 3, designed in 1955 and 1958 respectively, are two story, post-tensioned concrete lift slab buildings. Lateral resistance was provided by a combination of concrete block and gunite walls.

This analysis showed that although the force level used in the original design exceeded that of the 1973 UBC requirements by over 20%, and the Uniform Building Code, since 1961, did not require triangular vertical distribution for two story buildings, the buildings had weaknesses in structural design and detailing. Substantial upgrading beyond the requirement of the 1973 UBC was recommended.

In 1976 the buildings were re-evaluated in light of the revised 1976 Uniform Building Code requirements. Substantial seismic reinforcement was again recommended.

In 1979 Forell/Elsesser Engineers were retained by the General Service Administration, a federal agency which has jurisdiction for the USGS facilities, to make final recommendations and prepare construction drawings for the seismic reinforcement of the three buildings.

The General Service Administration (GSA) ruled that the building reinforcement be designed to meet the GSA Design Guidelines, using the Analysis Method No. I, which is the current (1979) Uniform Building Code method. Methods II and III which require the use of two design spectra with increasing sophistication of analysis were not authorized. The major change from the 1976 evaluation was the recommendation to use a K of 1.33 in the lateral force determination. The reason for this recommendation was that the Commentary of the SEAOC Blue Book strongly recommends that a vertical load carrying frame exhibit a nominal moment resistance to qualify for a K=1 structure. This was judged not to be the case in a lift slab structure.

The strategies for each individual building were worked out in conjunction with the project architect, Richard C. Marshall, FAIA, taking into consideration client program needs and aesthetics and users program requirements. In all buildings work disruption of occupancy was to be eliminated as much as possible. External bracing was therefore desirable.

Building No. 1

In view of the above considerations, an external bracing was selected. Projected future expansion of the building suggested an external extension of the existing frames on both sides of the structure. These single bay frames are designed and detailed as ductile moment frames capable to resist the design forces. The new frames are sized to accommodate future vertical and horizontal loads, should these bays be infilled to expand the building to the limits of the new frames. Except for some modification to the exterior existing longitudinal frame lines to increase connection capacity, collector capacity and shear transfer to the existing wood diaphragm no internal modification was required.

Building No. 2

The seismic reinforcement for this building is a combination of transverse interior concrete shear walls and longitudinal exterior steel braces. The building is divided into three elements, north and south wings and core element, by separation joints. In view of the brittle and sensitive nature of the post-tensioned lift slab construction, the use of moment frames was discarded. Rather a stiff system that would minimize building drift was selected. The separation joints were widened to accommodate relative building distortions due to earthquake forces. New shear walls were connected to floor and roof slabs by means of drilled dowels and epoxy grout. Existing wall elements, such as walls at stairs, which would create undesirable stiffness or stress concentrations were disconnected by means of saw cutting.

The exterior longitudinal steel braces were supported on independent footings and connected to floor and roof diaphragms by means of bolts and epoxy bonding. A new collector chords extends for the length of the building element. The steel brace is left exposed as an architectural element.

Building No. 3

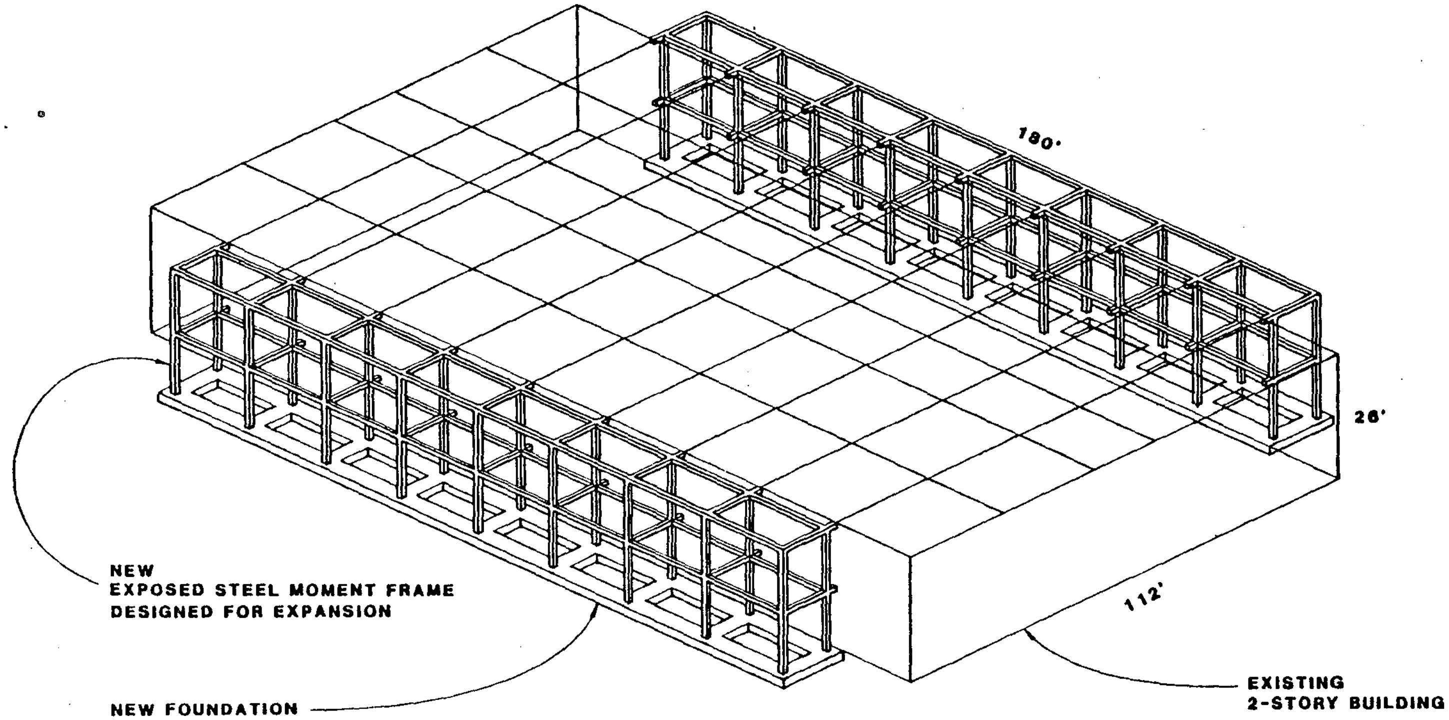
This building is now in design.

This report has been prepared by Nicholas F. Forell, President, Forell/Elsesser Engineers, Inc., San Francisco.

USGS MENLO PARK, CALIFORNIA

BUILDING NO. 2 - NORTH AND SOUTH WINGS

1953 UBC	1973 UBC	1976 UBC	GSA 1979
$V = \frac{C \times 0.15}{N + 4.5}$	$V = ZKCW$	$V = ZIKCSW$	$V = ZIKSCW$
$C = 4$ $N = 0.1$	$Z = 1.0$ $K = 1.0$ $C = 0.1$	$Z = 1.0$ $I = 1.0$ $K = 1.0$ $CS = 0.14$	$Z = 1.0$ $I = 1.0$ $K = 1.33$ $CS = 0.14$
$V = 0.12W$	$V = 0.1W$	$V = 0.14W$	$V = 0.186W$
$V = 240^k$	$V = 197^k$	$V = 277^k$	$V = 367^k$
Roof: 137^k	Roof: 103^k	Roof: 188^k	Roof: 250^k
2nd Fl: 103^k	2nd Fl: 97^k	2nd Fl: 89^k	2nd Fl: 117^k

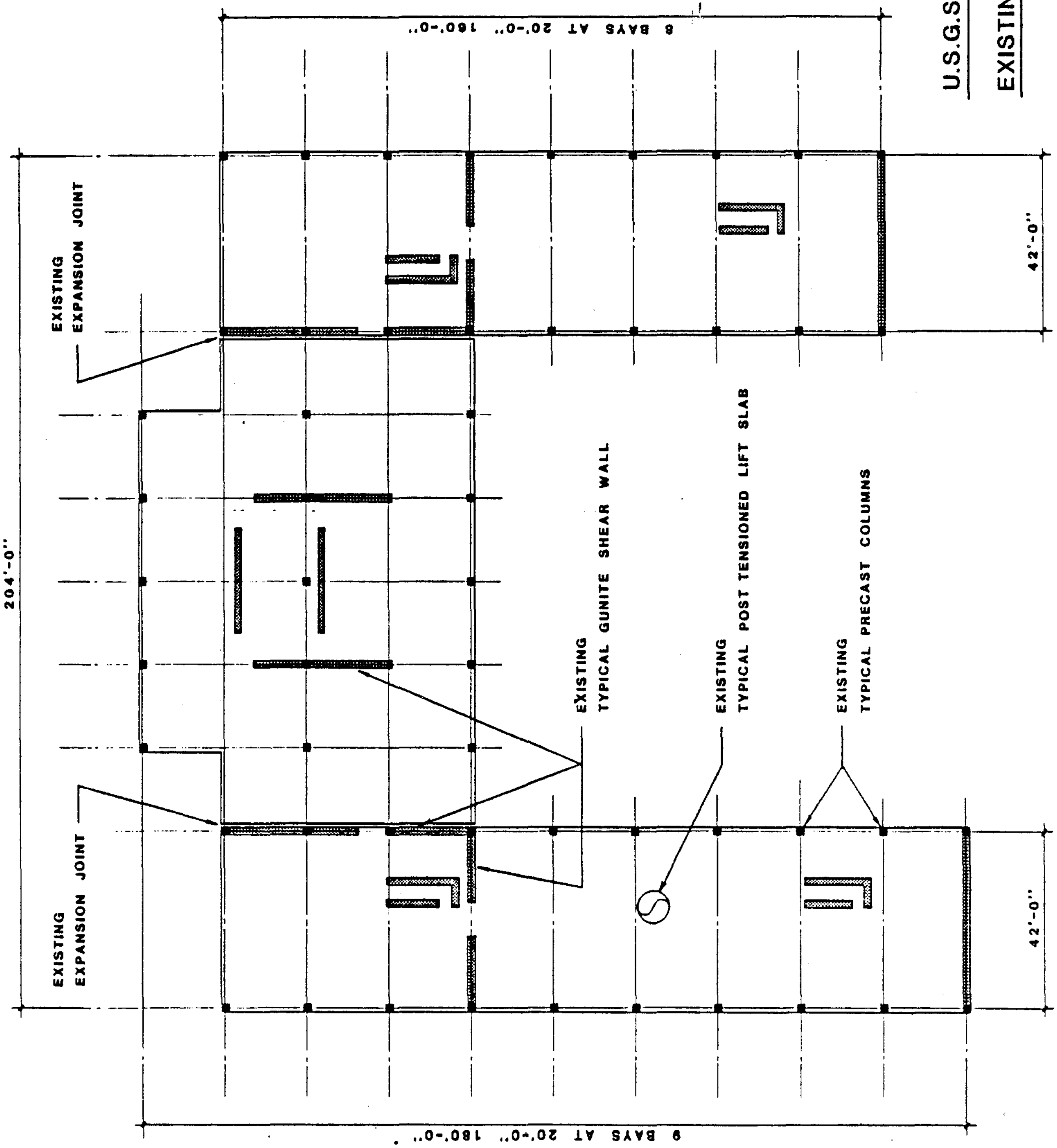


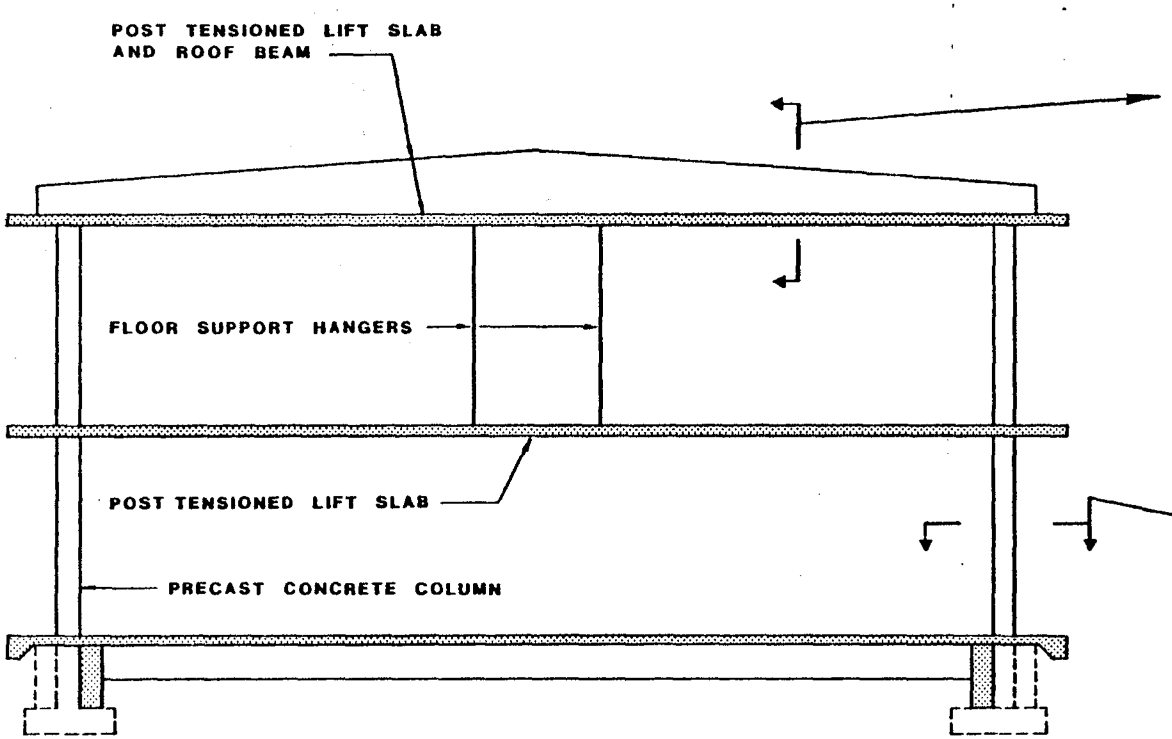
SCHEMATIC VIEW OF SEISMIC BRACING

U.S.G.S. BUILDING 1

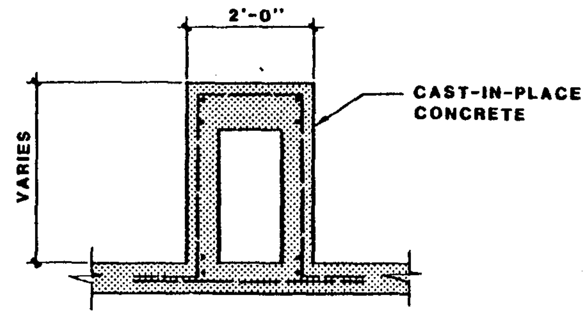
U.S.G.S. BUILDING 2

EXISTING SYSTEM

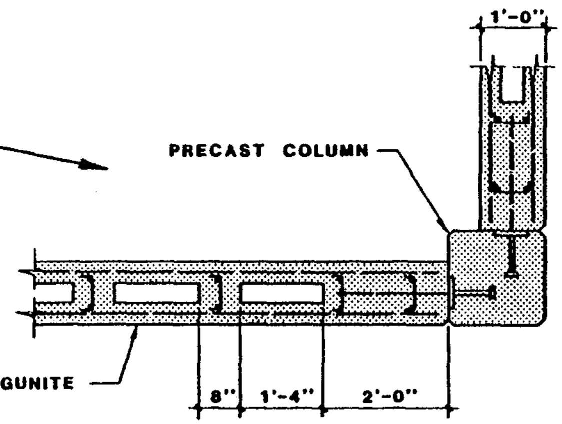




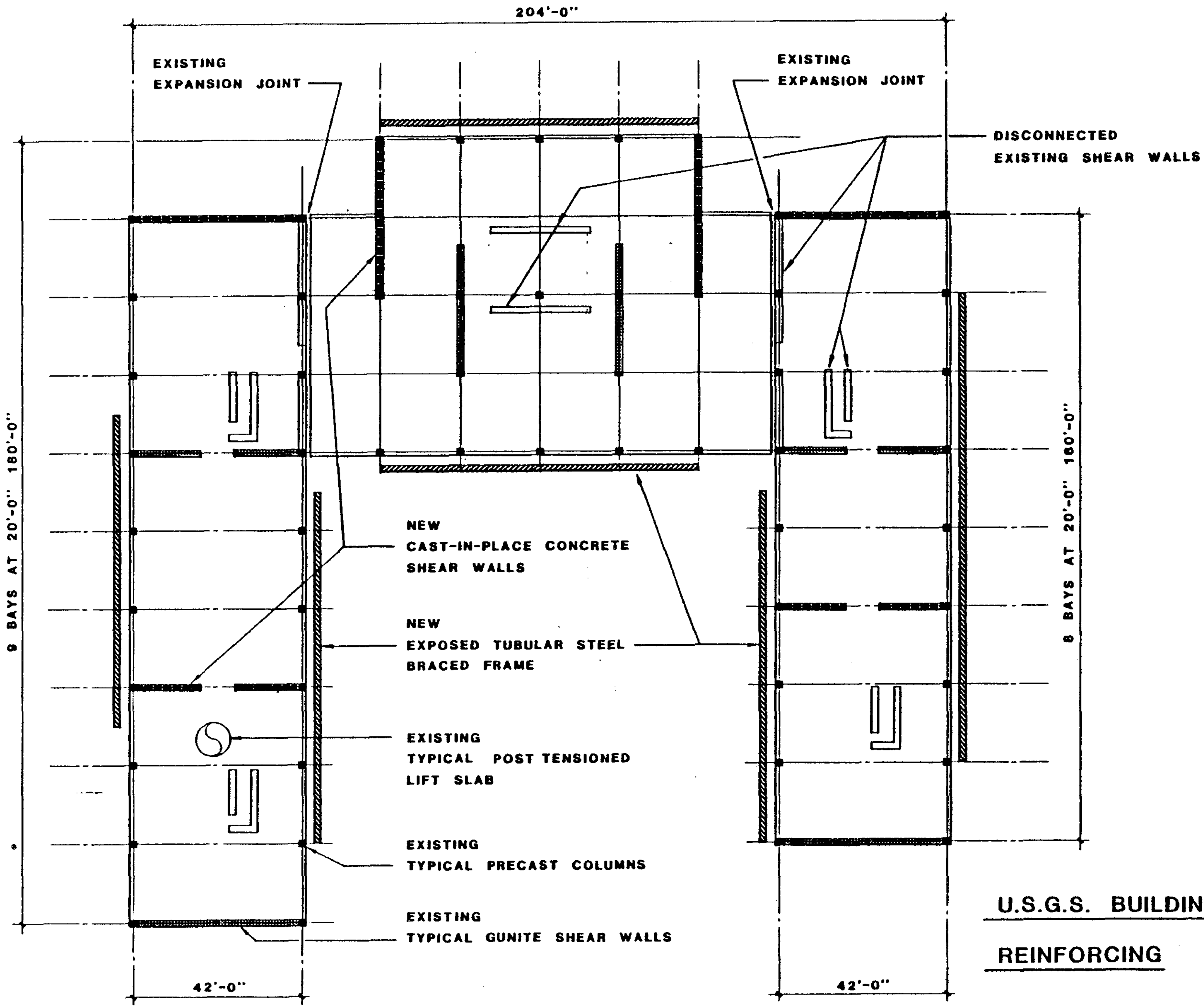
TYPICAL BUILDING WING SECTION



ROOF BEAM SECTION

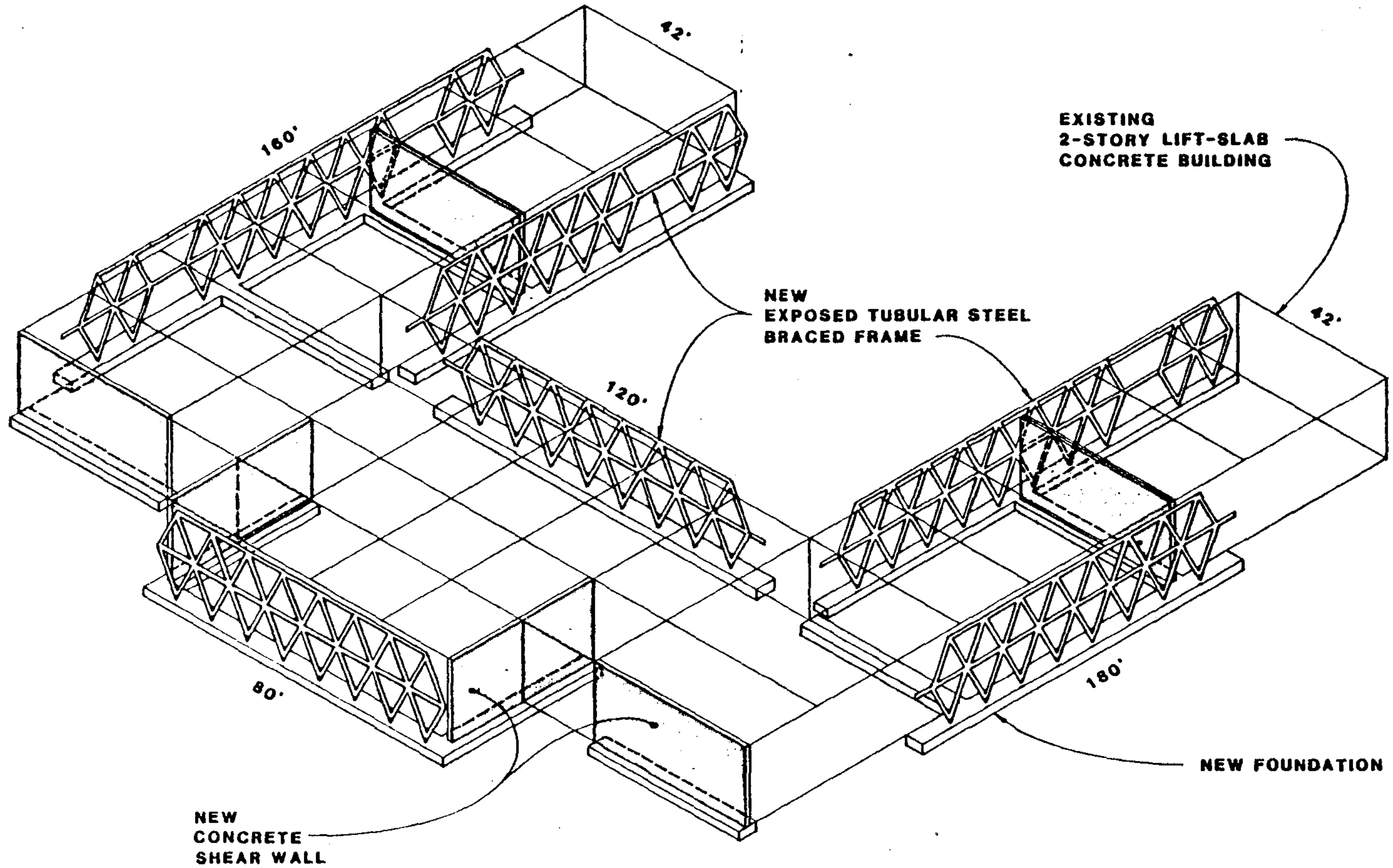


TYPICAL EXISTING SHEAR WALL SECTION



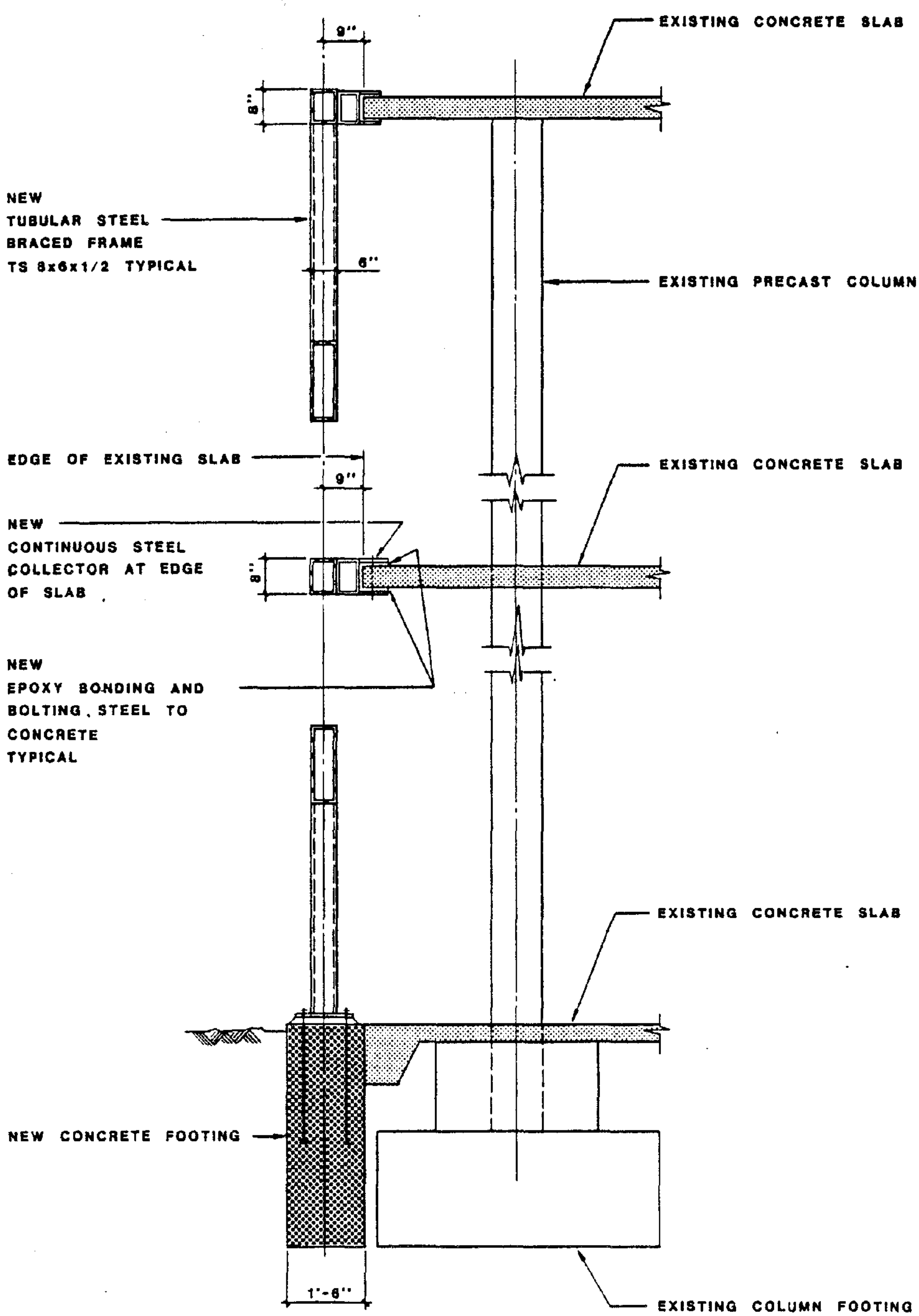
U.S.G.S. BUILDING 2

REINFORCING

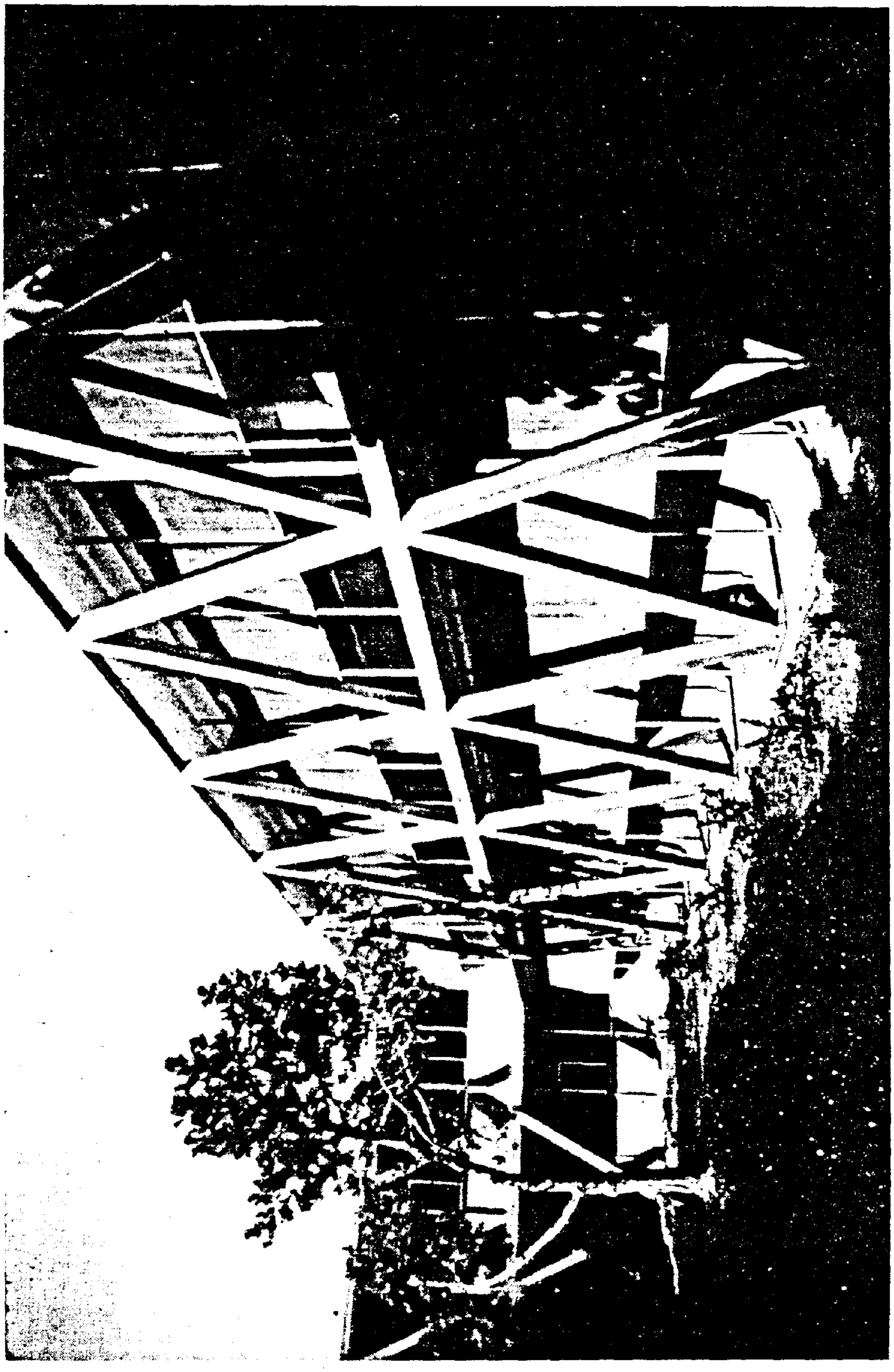


SCHEMATIC VIEW OF SEISMIC BRACING

U.S.G.S. BUILDING 2



BRACED FRAME SECTION



EXAMPLES OF REPAIR AND RETROFIT WORK
ON ROAD BRIDGE SUBSTRUCTURES

by

Hideya Asanuma
Public Works Research Institute
Ministry of Construction

1. INTRODUCTION

Japan has a network of some 1,110,000 km public roads covering all the territory including remote islands, and administrates as many as 580,000 bridges ranging a 2m-long culvert to a long suspension bridge.

These bridges subjected to severe natural conditions, heavy traffic loads and sometimes strong motion of earthquakes are liable to sustain injuries and harm in years after their construction.

Repairing and retrofitting damaged bridges against further harm is a work of great importance from the socio-economical point of view amid a low-growth economic circumstance of the country.

Here, the author introduce general concepts of repair and retrofit works on road bridge substructures with an emphasis on concrete structures, and then show some examples of the works.

2. DAMAGE TO BRIDGE SUBSTRUCTURES

Damages which concrete piers and abutments are apt to sustain are confined to some categories by portion of their location. Most liable cases are

(1) Damage around a support
(2) Crack and fracture of a column, wall and beam
(3) Damage to a foundation including a footing

Among the above items, the third one is very hard to be found or detected and also very difficult to be repaired or retrofitted because they exist under ground level. So, this problem would be omitted from the following discussions. Causes which bring damage are,

(1) Deterioration of the materials due to weathering
(2) Unequal settlement or consolidation of the ground
(3) Experience of an unexpected strong force like what hit by an earthquake.

3. GENERAL CONCEPT OF REPAIR AND RETROFIT

1. Support-related Damage

(1) Appearance of Damage

Most damages concerning supports are concentrated to those of shoes; breakage of a slide confining devise, dislodgement or fall-down of rollers, fracture of a lower shoe, fracture of a pin or rollers, rusting of a sliding surface or rolling surface, erosion of shoe members and so forth. However, here we will deal with only concrete structures beneath shoes, and also exclude anchorbolts.

Fig. 1 shows an example of fracture of mortar filled between a shoe and its bed. In a case of slight damage, the fracture degree is low and only cracks develop, while the mortar apparently breaks in a severer case.

Fig. 2 shows how a crest of pier or abutment breaks.

This type of fracture often takes place around a movable shoe. The reason would be that a stronger force than expected is liable to be transmitted through a movable shoe because of greater friction due to rusting of its sliding surface or rollers.

(2) Repair and Retrofit

Against cracking or a fracture of shoe mortar, such measures of filling up with fresh mortar as shown in Fig. 3 are effective. (a) is a likeliest case of filling with fresh mortar, but should be paid much attention to make sure if the mortar is filled up sufficiently. (b) shows an example of grouting in case a gap between a sole plate and its bed is narrow. And, (c) is an additional case of using pre-packed concrete as a filler.

Measures against a break incurred to shoe bed concrete should be chosen in compliance with the seriousness as well as the cause of break.

Common methods for repair or retrofit would be

1) wrapping the pier top by steel plates to strengthen a damaged portion as shown in Fig. 4
2) placing fresh concrete with reinforcing steel bars at the breakage. Fig. 5 is one example of this method.
3) attaching steel frames with anchors embedded into the column or abutment as shown in Fig. 6

In Japan, the Specifications for Highway Bridges provides that any pier or abutment should have sufficient length of overlapping with a corresponding superstructure so that even in the worst case a superstructure could escape falling down, unless it has a special device to prevent dislodgement or even falling-down of a beam.

However, bridges constructed before these provisions were established have not necessarily enough overlapping length. In order to enhance the safety of these relatively old bridges, such retrofit works were required as enlarging the crest of a sub-structure and having it overlapped sufficiently.

Fig. 7 shows some examples of the works done for this purpose.

2. Damage to Columns or Walls

(1) Appearance of Damage

Columns and walls are generally made as reinforced concrete structures except steel piers of elevated bridges constructed in a downtown. These structures are exposed to drying, humidity, frost and melt, as well as salty water, acid, alkali, heat, industrial drainage and so on. These factors cause concrete structures gradually deteriorate and lose their strength year by year. This effect is known as weathering.

However, damage due to weathering is very slight if compared with that due to an earthquake. So, here we lay emphasis on quake-related damages.

The damage sustained by sub-structures due to strong motion can be classified into following degrees from the lightest to the heaviest.

1) Cracking of concrete

Vertical cracking as seen in Fig. 8 and horizontal cracking as seen in Fig. 9.

2) Flaking of concrete and exposure of steel bars as seen in Fig. 10.

3) Complete destruction of concrete and buckling of steel bars as seen in Fig. 11.

Typical features of the damage are illustrated in Fig. 8 through Fig. 11.

The cause of damage can be considered as

1) excessive tensile stress induced in concrete due to bending moment, which causes tensile cracks on a surface of the structure.
2) buckling of reinforcing steel bars due to an excessive alternate force. As the case may be, the buckling causes flaking-off of the concrete outside the steel bars.
3) shear failure of concrete. Since a concrete member is brittle against shear failure, this type of collapse is mostly destructive and fatal.

(2) Repair and Retrofit

1) Measures against cracks

As regards repair of cracks, grouting or injection of such adhesive materials as cement mortar, resin mortar, epoxy resin etc. is effective. When cracking is not so profound, surface treatment or grouting of mortar as illustrated in Fig. 11 are sufficient. Against extensive and deep cracking, grout should be infiltrated under some pressure up to about 8kgf/cm^2. Fig. 12 is one of most progressed methods of this type. Another measure is to introduce prestressing across the cracks through PC steel bars or wires as seen in Fig. 13.

2) Measures against breakage

Heavier damages to a column or wall than cracking are flaking of surface concrete, exposure or even buckling of steel bars, and finally complete collapse of the structure. In the worst case of collapse, reconstruction of a new structure will become easier and more economical.

These breaks not only reduce a cross sectional area of the structure, but cause rust of steel bar. Therefore, we should make up for the reduced section and reinforce strength of the members. For this purpose it is common to wrap the column or wall with some materials.

Fig. 14 indicates an example of wrapping with steel plates or fiber reinforced plastic plates. Void between the plates and the concrete surface is filled with resin mortar or epoxy resin.

Fig. 15 shows how to thicken a faulty column by placing concrete with reinforcement and anchorage into the original column.

When durability against future strong motion is estimated insufficient, construction of a new member for additional support or reinforcement of the existing structure becomes of course necessary, even if there is no substantial damage on it.

4. EXAMPLES OF REPAIR AND RETROFIT WORKS

1. The Sendai Bridge

The Sendai Bridge, completed in 1965, is located in south part of Sendai City, and is crossing over the Hirose River as a part of the National Highway No. 4. The general side view is shown in Fig. 16. Superstructures are 9-span simply supported composite steel-plate girders, with span length of 9×33.840m, total length of 310m, and width of 19m. Substructures are T-shape columns (6.1m high) founded on rigid well foundations (9 to 18m deep) embedded into rather stiff sands. Bearing supports are of type of line bearings. Since this highway connecting Kanto and Tohoku regions is an important one, Sendai Bridge carries very heavy traffic (54,000 cars daily).

Due to the earthquake (the epicentral distance to the bridge is Δ=120km), all of the nine pier columns sustained damage. Piers 1 through 4 cracked horizontally at the column bases and surface concrete pieces separated heavily from the columns near the bases. Piers 5 through 8 had similar damage near the haunches which connect columns and beams. Pier 6 which has the lowest free height sustained the severest cracking at both sides (see Fig. 17). Concrete pieces separated at the haunch and reinforcing bars buckled. Near the haunch volume of reinforcing bars as well as concrete sectional area change rapidly. It is estimated that relative displacements between adjacent girders were 1 to 2.5 cm on the pier caps and that displacements at the pier caps of Piers 1, 2 and 6 were 11 to 18 cm.

Fig. 18 shows temporary frame works supporting the girders near Pier 6. Since the bridge is very important, damaged piers were repaired without stopping traffic even for a short time. Fig. 19 illustrates an example of permanent repairing work at Pier 6. The thickness of added concrete was 50 to 70 cm, and vertical reinforcing bars were fixed by epoxy adhesive into the well foundation and lateral bars were fixed to the columns. Moreover, chemical resin was placed into small cracks. It took only one month to completely repair the damage to this bridge.

2. The Abukuma Bridge

The Abukuma Bridge, constructed in 1932 and annexed a sidewalk in 1967, is spanning the Abukuma River on National Highway No. 6.

The general plan is shown in Fig. 20. The bridge had beared heavy traffic over 46 years and the possibility of replacement was just studied so that it might be accommodated to modern and heavy road traffic.

The Miyagi-ken-oki Earthquake hit this bridge and 8 of 16 piers, which all supported trusses, suffered cracking, separation of concrete and exposure of steel bars.

Fig. 22 shows how these piers were repaired. The piers of slight damage with only hair cracks underwent the repair of expoxy resin injection. Since design data of this bridge was missing and not available, the repair work was designed to stand on as safe side as possible; it means enough reinforcement and sufficient cross sectional area. Concrete with reinforcement was newly placed surrounding the existing piers after chipping their surfaces and taking anchorages into them.

Additionally, the crests of piers were widened to keep enought over-lapping length with the corresponding girders for future safety.

3. The Ten-noh Bridge

The Ten-noh Bridge is spanning the Kitakami River with 367.7 m length and 8.0 m width, completed in 1959 and annexed a sidewalk in 1975. A main structure is a Langer-type arch with simple supported side spans.

A pier which supports the arch sustained heavy damage. Many cracks propagated outward from the shoe positions and a long vertical cracking occurred at the center of the pier as seen in Fig. 24.

From the viewpoint of importance for regional traffic and the degree of damage, the bridge was opened to public traffic under some restrictions. Taking such a situation into consideration, the bridge was temporarily supported by H-shape steel frames, resting on the footing, with some stiffening members.

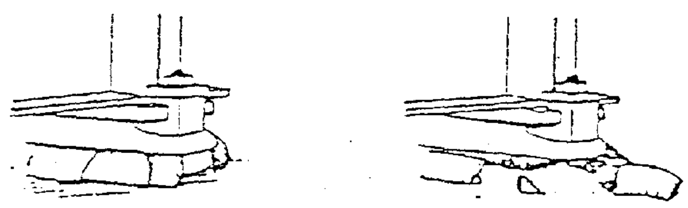
Therefore, as a permanent repair, the pier was designed to be enclosed by a new reinforced concrete structure with stiffening H-shape steels and afterward tied with the original pier by tie-rods.

In this case, the surface of the footing was under water table and a measure to prevent water from infiltration was needed. Fig. 26 shows the construction of a cut-off wall on the footing.

Table —1 Present Status of Roads in Japan as of April 1, 1980

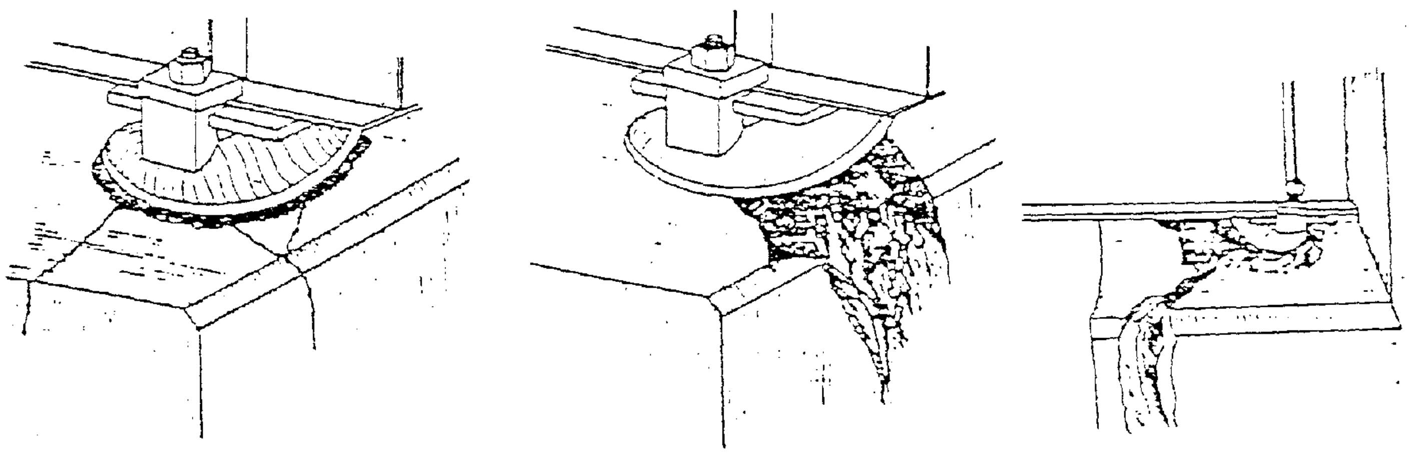
	Classification			Length of road, bridge, tunnel and ferry						
	Total (km)	Improved roads (km)	Un-improved roads (km)	Length of roads (km)	Bridges		Tunnels		Ferries	
					Number	Length (km)	Number	Length (km)	Number	Length (km)
National expressways	2,579	2,579	–	2,225	2,941	288	97	66	–	–
(Urban expressways)	(253)	(253)								
Sub-total	2,832	2,832								
National highways	40,212	35,298	4,914	33,666	36,306	1,058	1,720	488	8	41
Principal local roads	43,906	32,287	11,619	43,026	36,113	722	872	159	5	3
Prefectural roads	86,930	45,680	41,250	35,517	65,662	1,278	882	135	29	11
Municipal roads	939,760	252,327	686,933	935,880	440,058	3,752	1,410	129	140	126
Sub-total	1,110,808	356,093	744,715	1,103,089	578,139	6,809	4,884	911	182	181
Total	1,113,388	358,672	744,715	1,105,314	581,080	7,097	4,981	977	182	181

Note: 1. Referred to the Annuals of Road Statistics 1981.
2. The length of urban expressways is included in either principal local roads or prefectural roads.



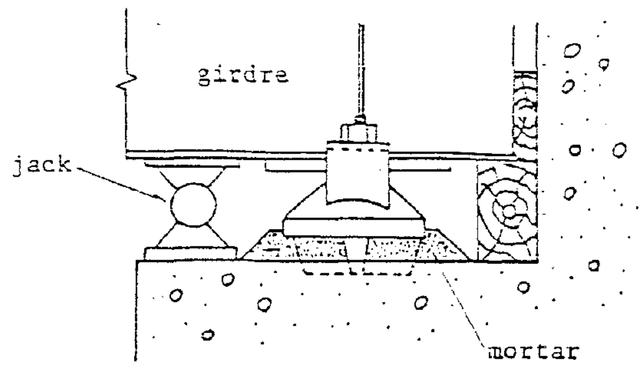
(a) (b)

Fig-1 Fracture of shoe mortar

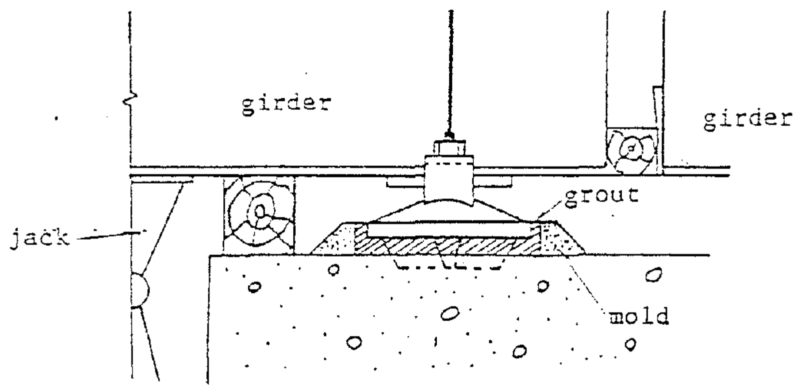


(a) (b) (c)

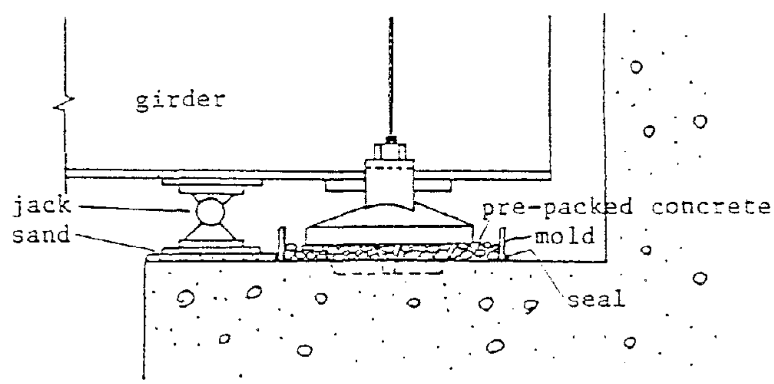
Fig-2 Fracture of shoe bed concrete



(a)

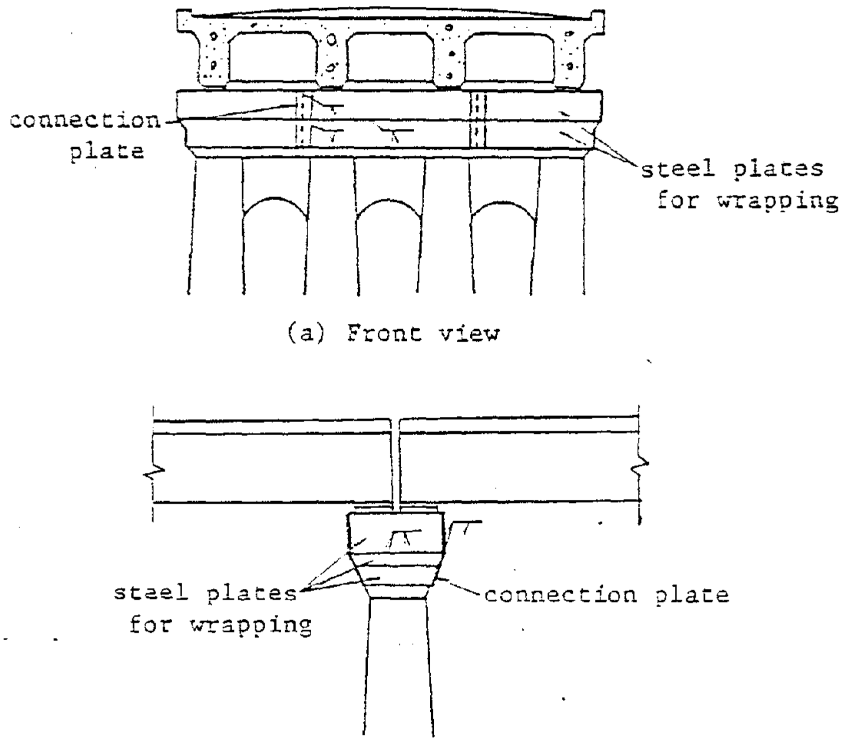


(b)



(c)

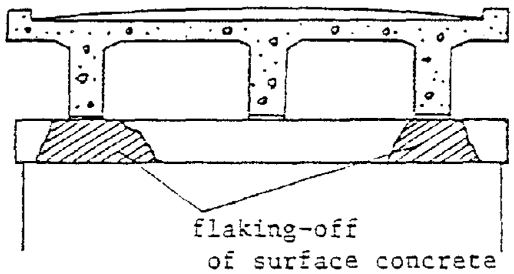
Fig-3 Methods of repair for shoe mortar



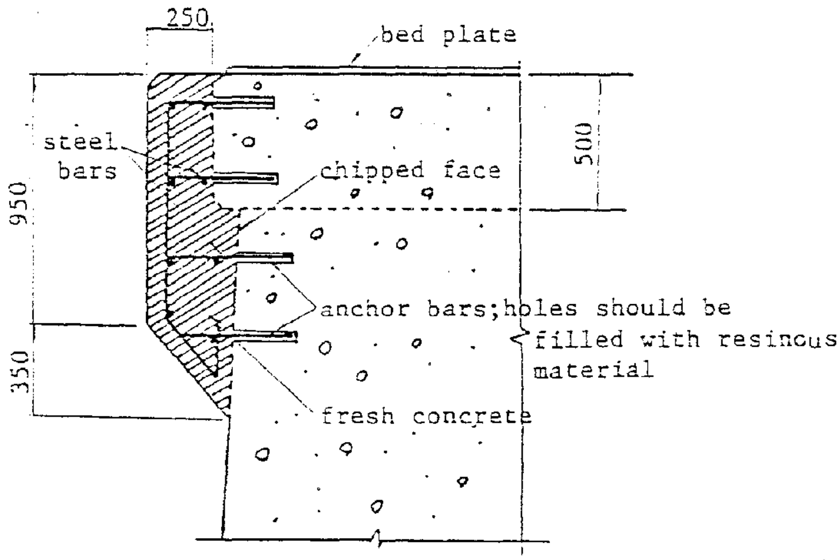
(a) Front view

(b) side view

Fig-4 Wrapping with steel plates

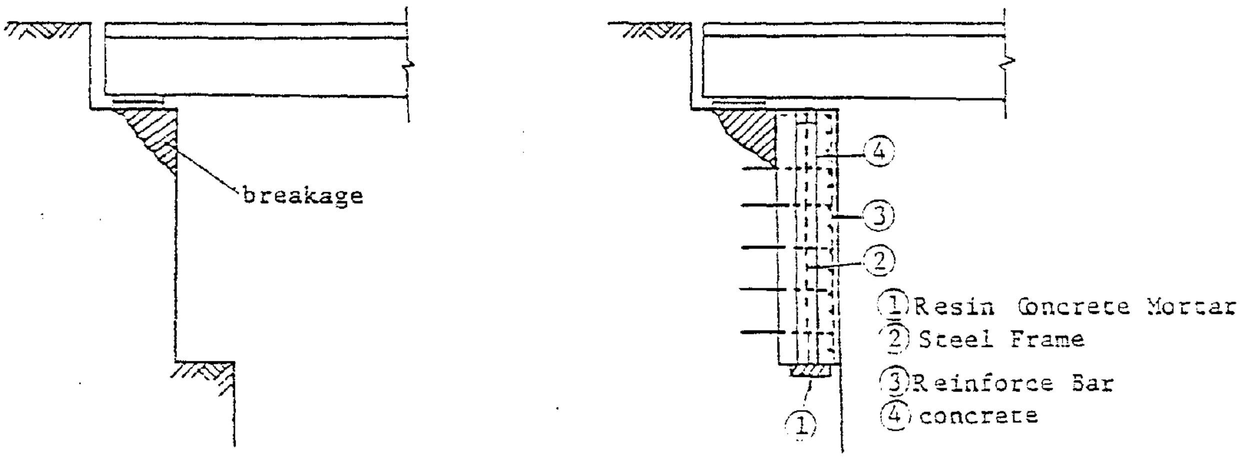


(a) General view of damaged portion



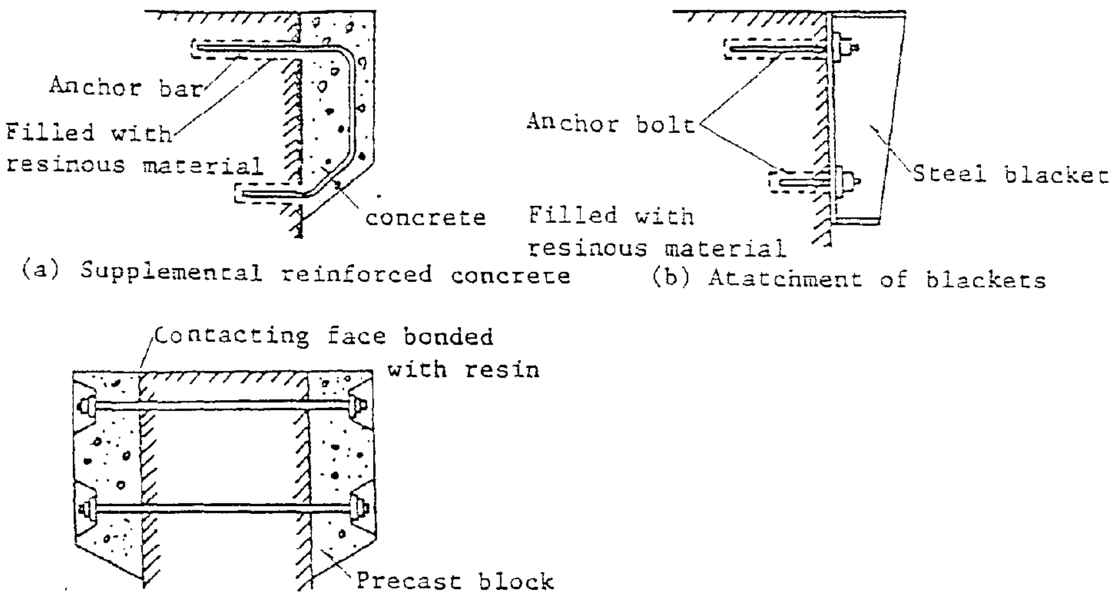
(b) Detail of the repair

Fig-5 Reinforcing by new concrete



(a) Side view of the damage (b) Repair work

Fig-6 Reinforcing by steel frames



(a) Supplemental reinforced concrete (b) Atatchment of blackets

(c) Supplement of PC precast blocks

Fig-7 Enlargement of the Crest of a Substructure

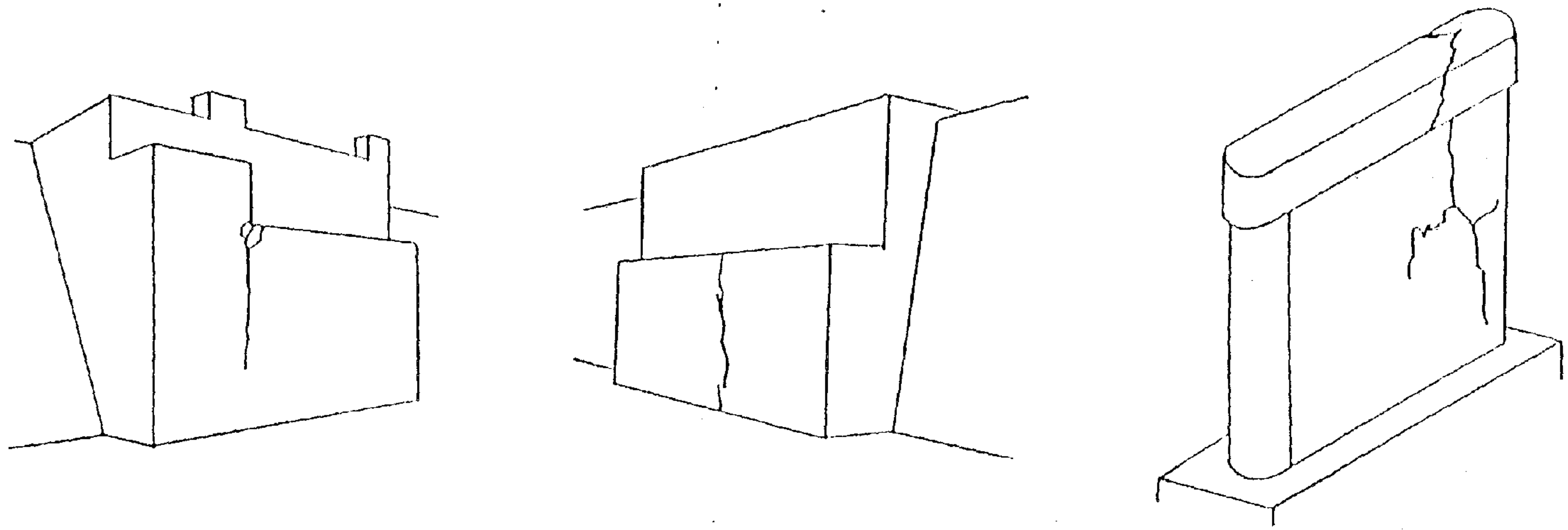


Fig-8 Vertical cracking

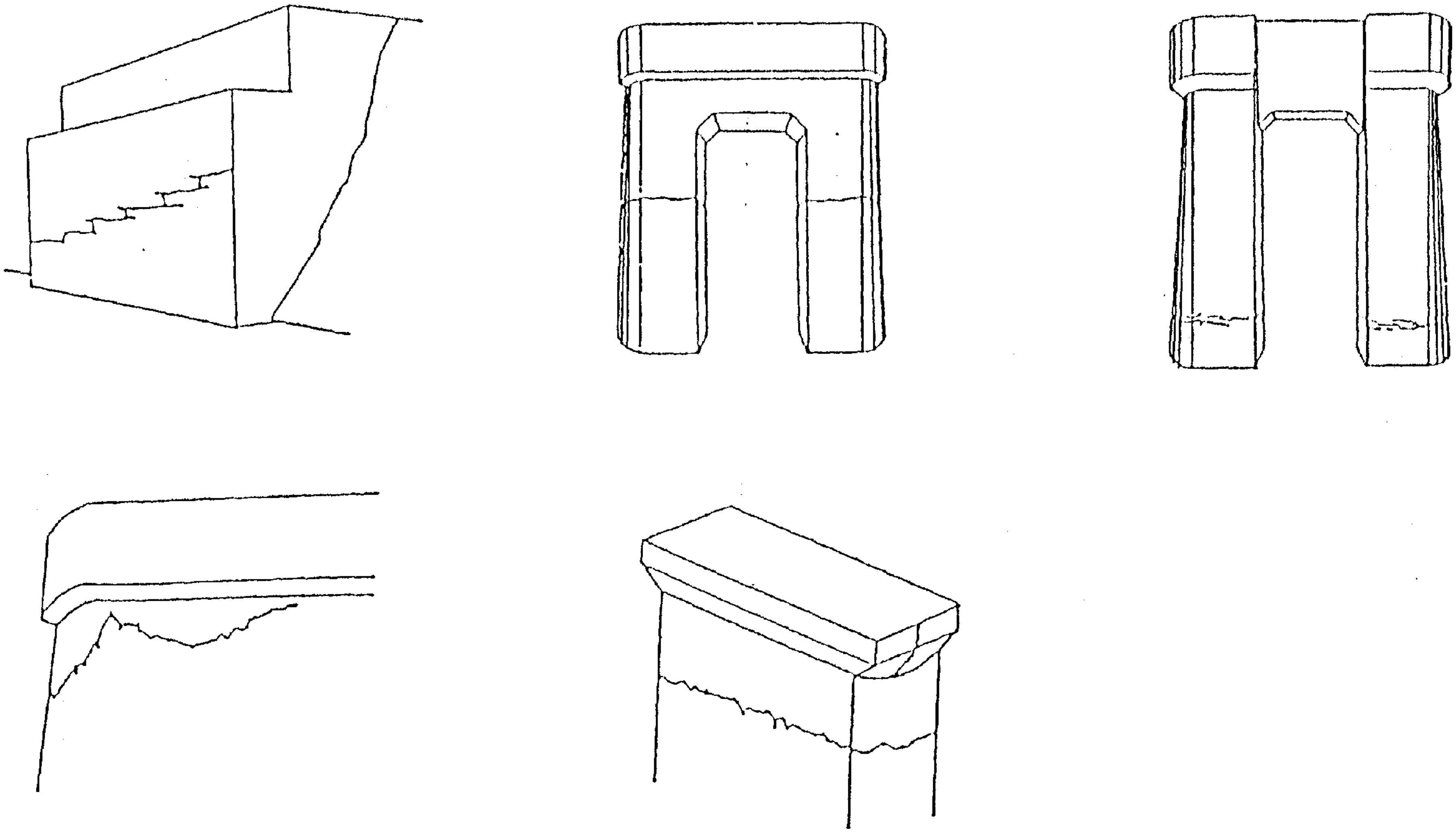
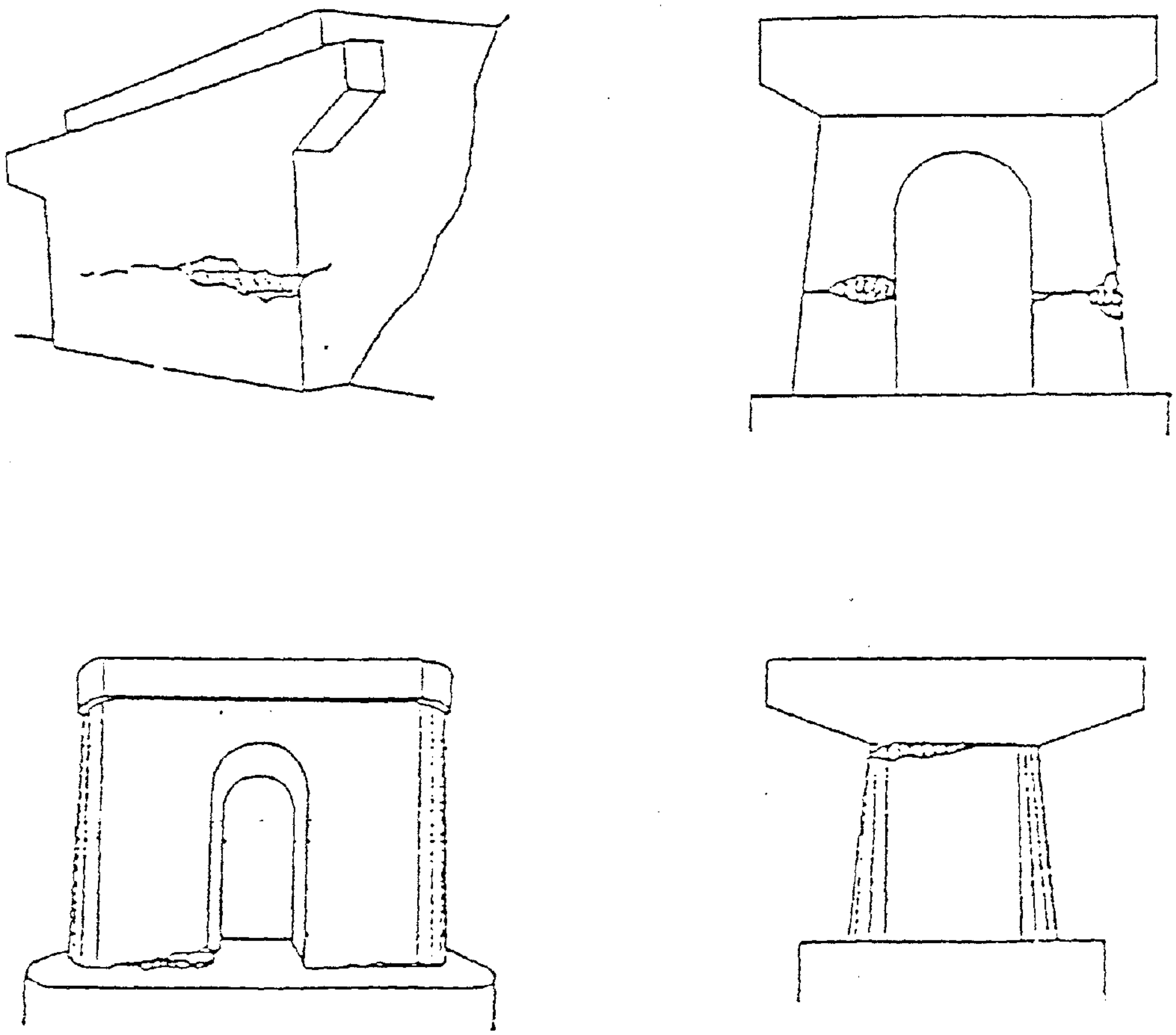


Fig-9 Horizontal cracking



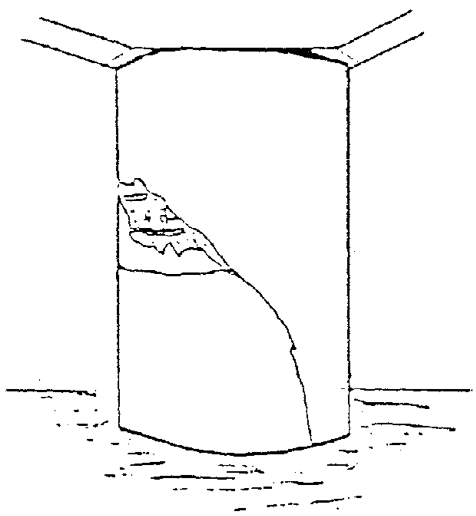


Fig-10 Flaking off of concrete and exposure of steel bars

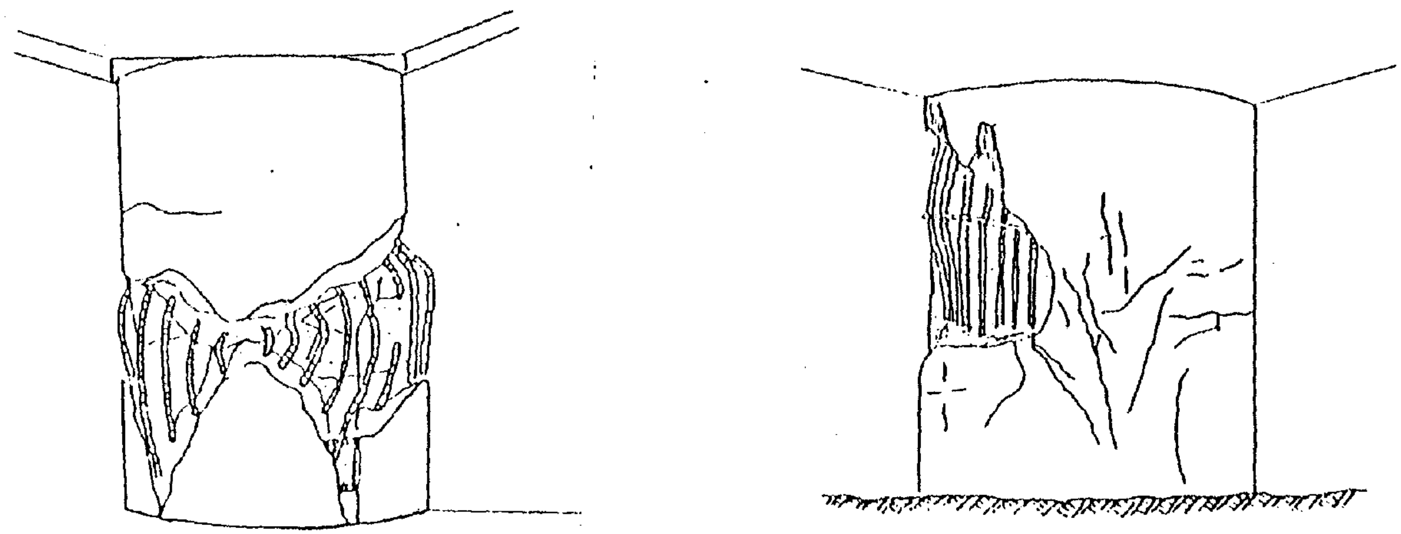
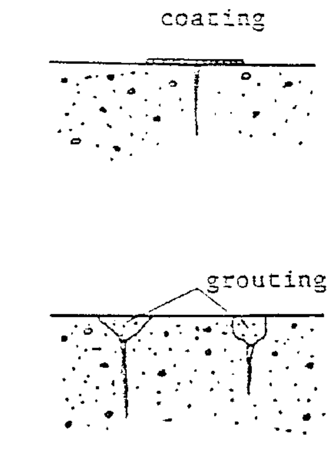
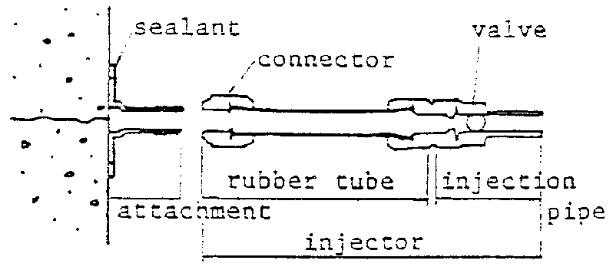


Fig-11 Conplete destruction

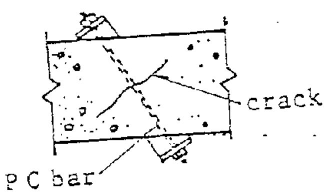


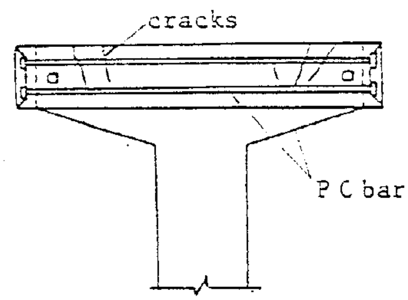
(a) Surface treatment



(b) Pressured injection

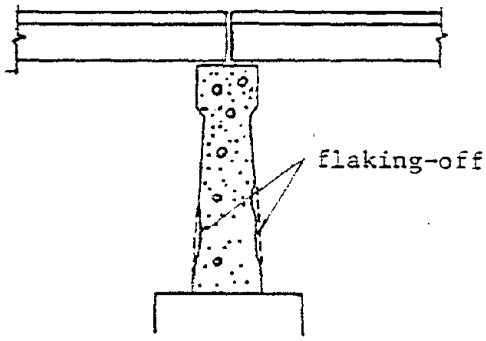
Fig-12 Grouting



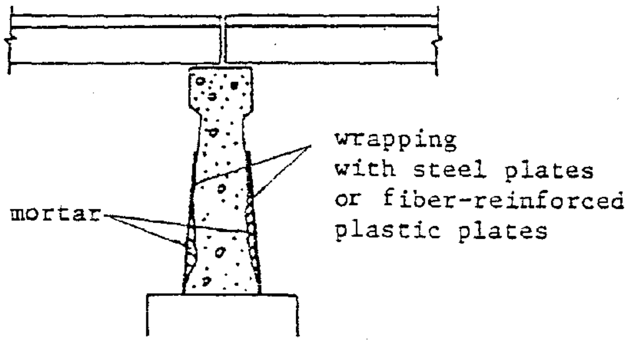


(b)

Fig-13 Prestressing

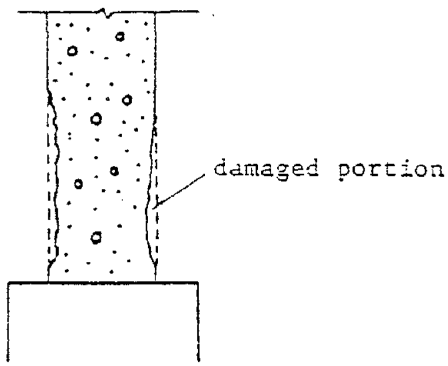


(a) damaged state



(b) repair work

Fig-14 Wrapping



(a) damaged state

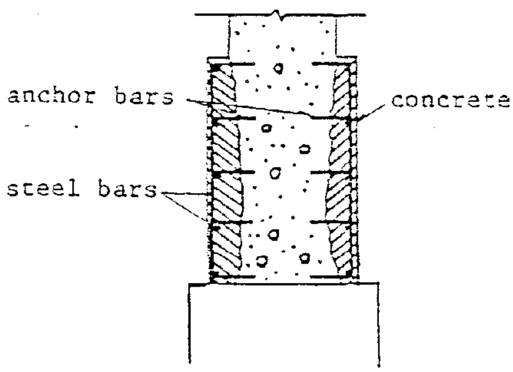


Fig-15 Supplemental reinforced concrete

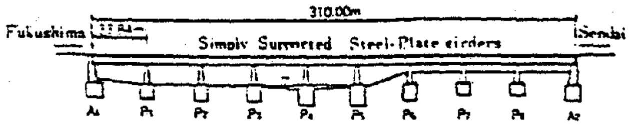


Fig. 16 General View of the Sendai Bridge.

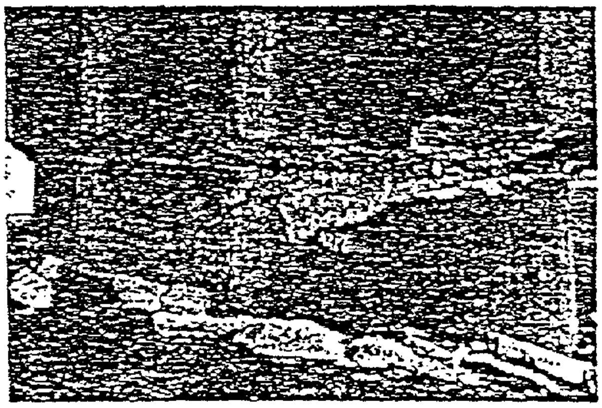


Fig. 17 Failure of Pier 6, the Sendai Bridge.

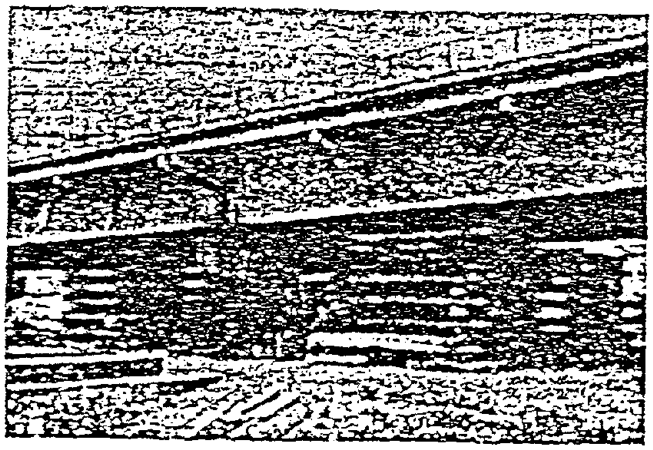


Fig. 18 Temporary Frames Supporting Girders near Pier 6, the Sendai Bridge.

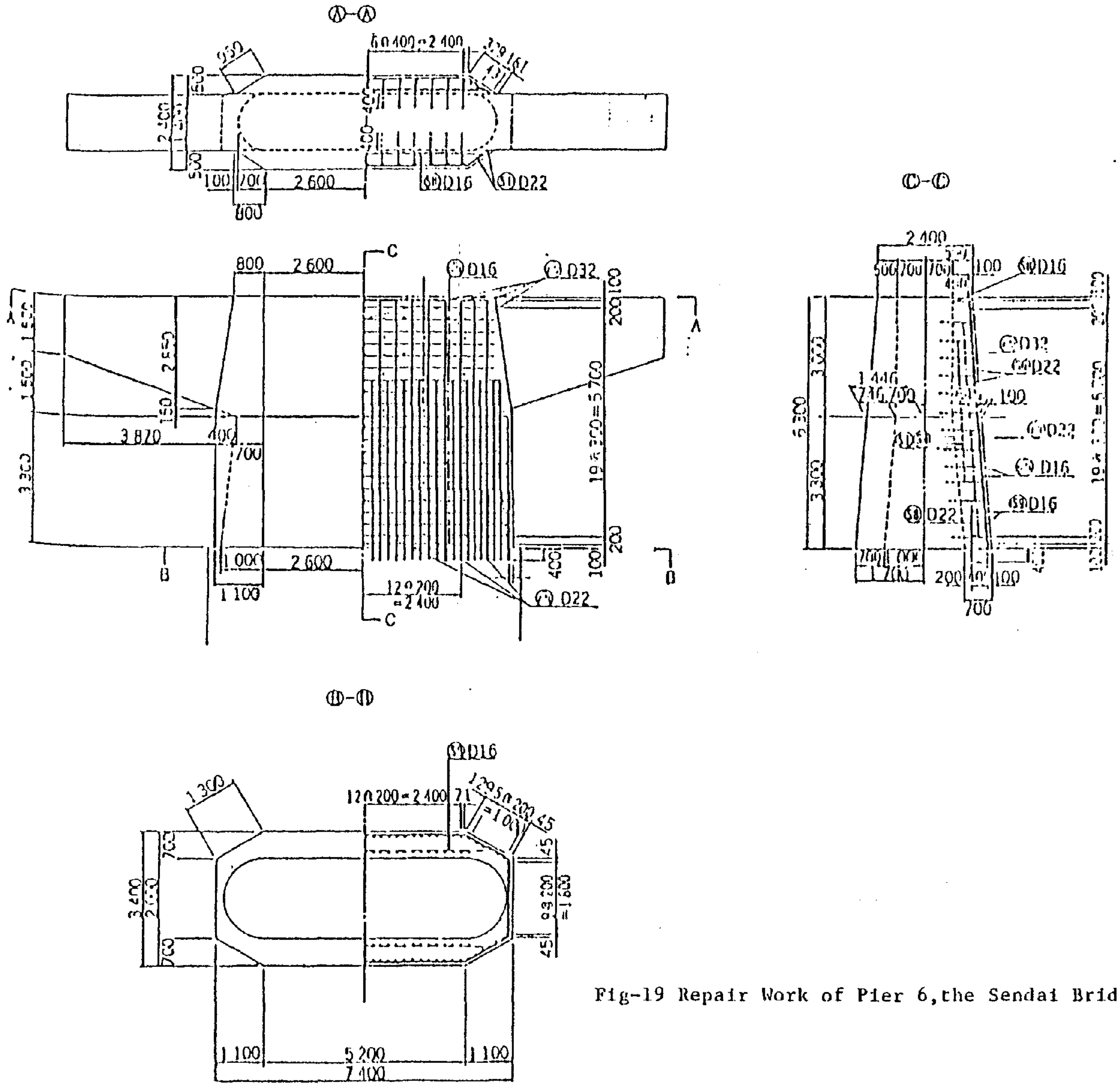


Fig-19 Repair Work of Pier 6,the Sendai Bridge

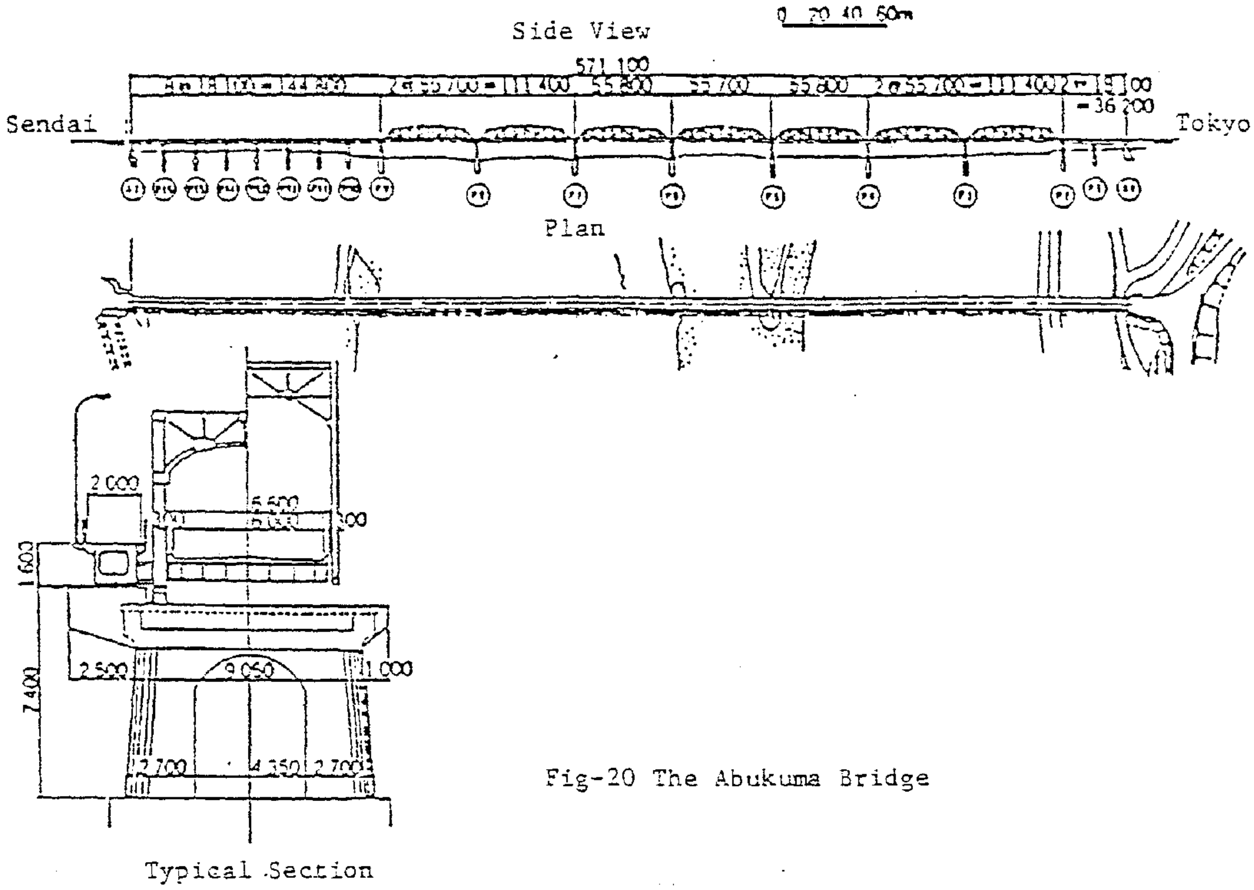


Fig-20 The Abukuma Bridge

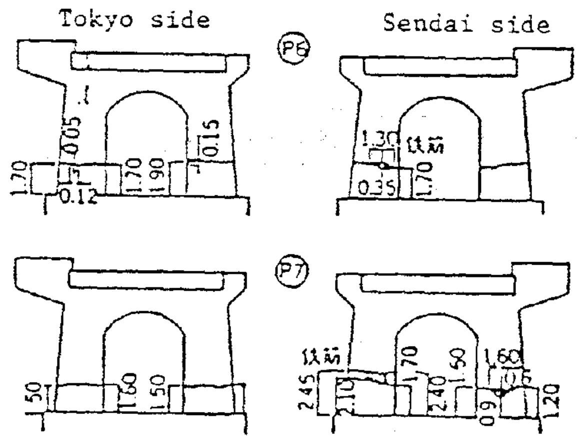


Fig-21 Damage to the Abukuma Bridge

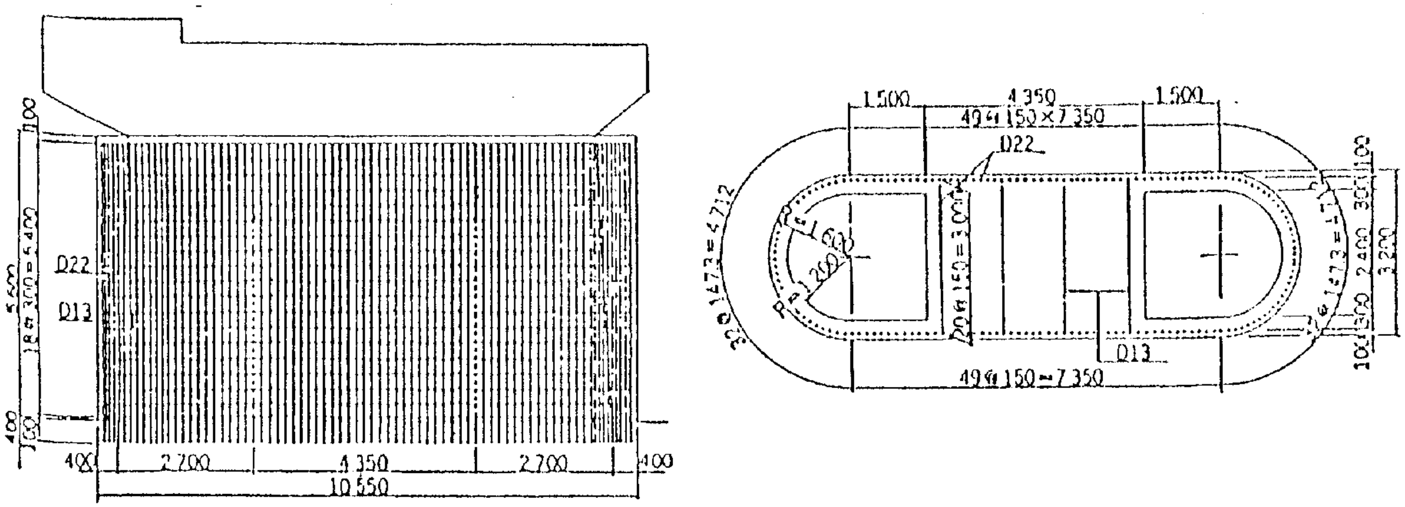


Fig-22 Repair Work of Pier 6 and 7,the Abukuma Bridge

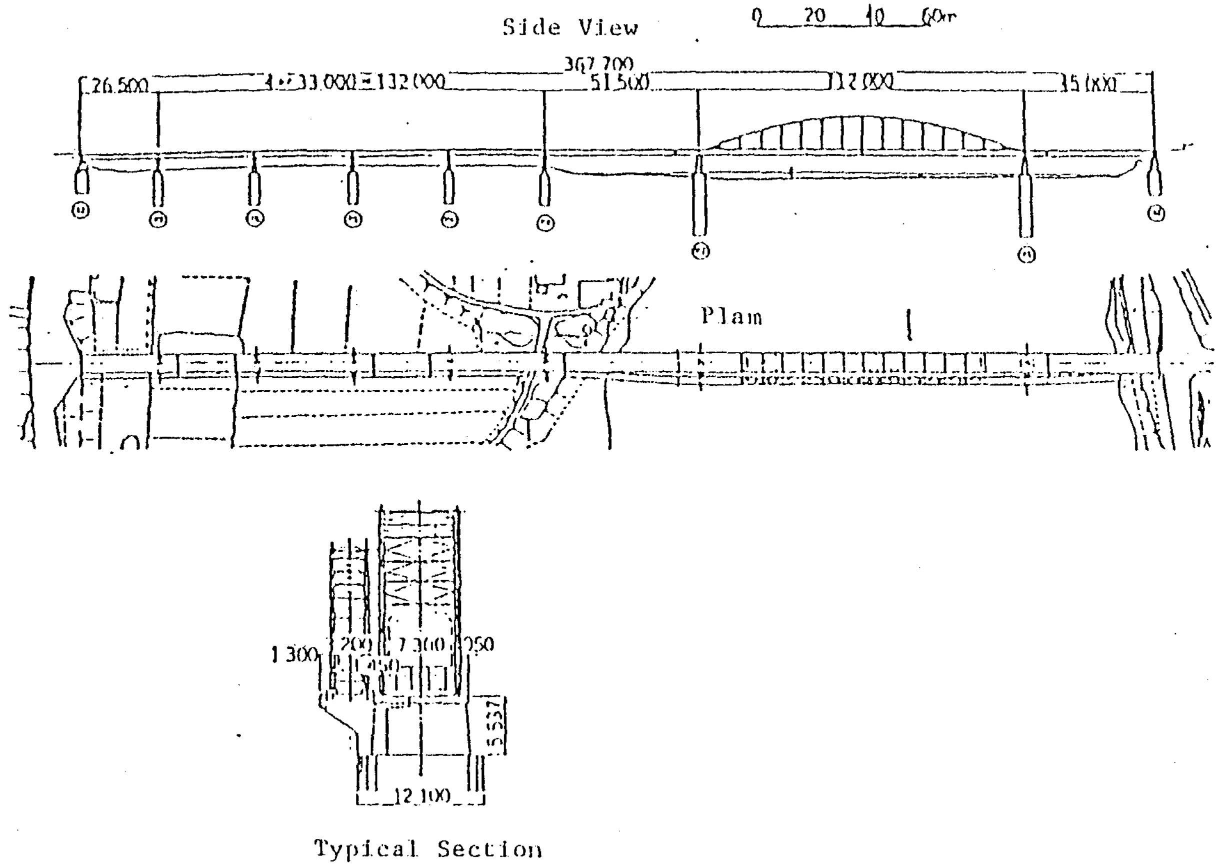


Fig-23 General View of the Ten-noh Bridge

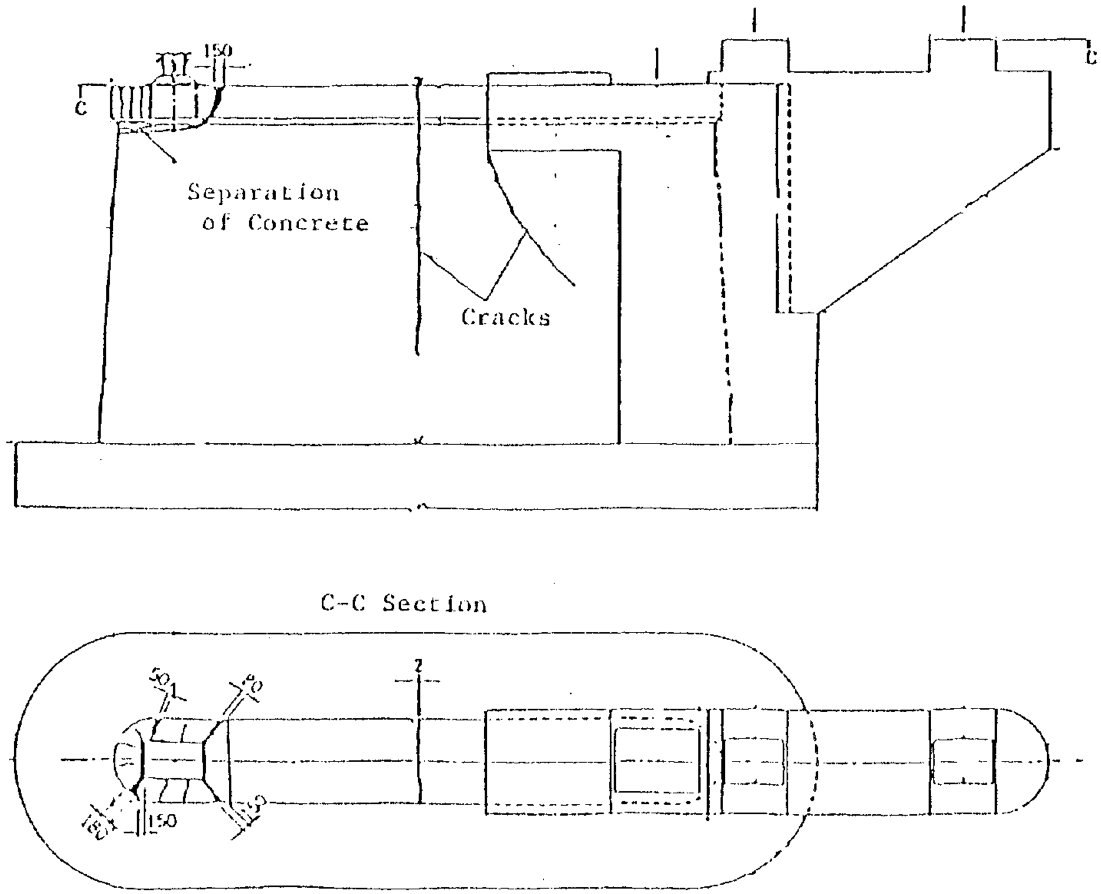


Fig-24 Damage to Pier P1, the Ten-noh Bridge

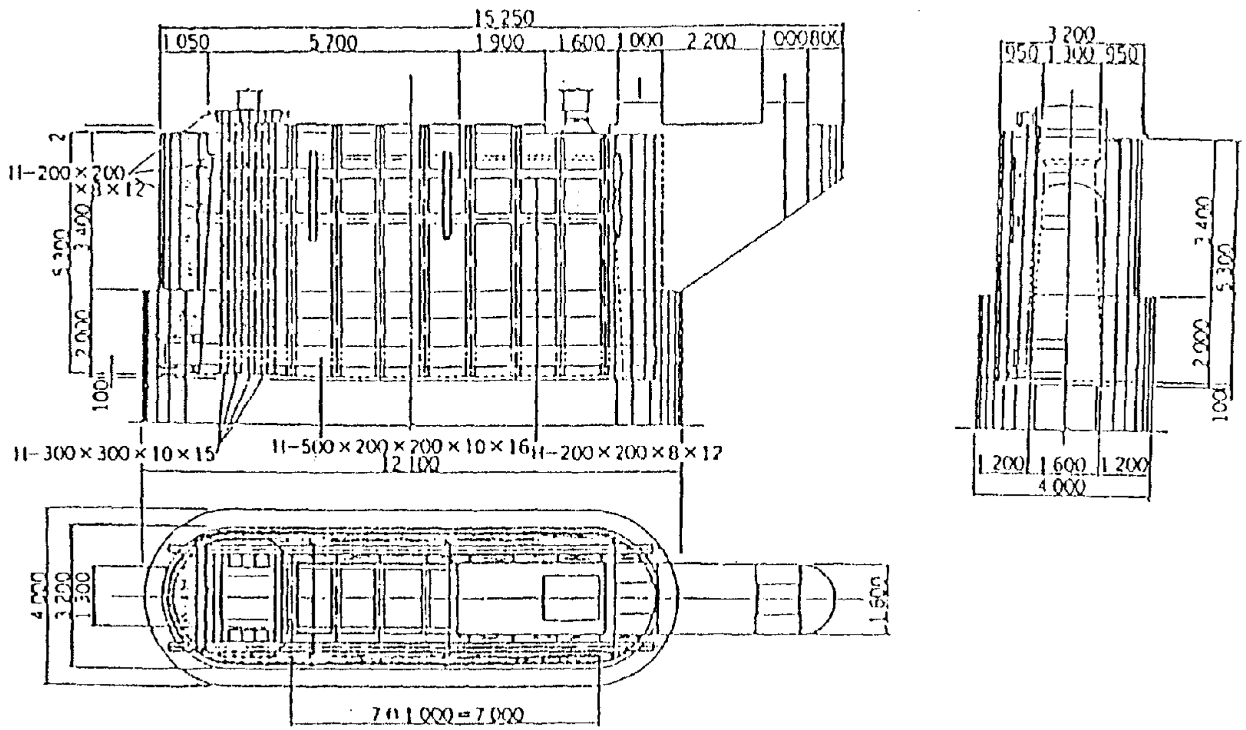


Fig-25 Repair Work of P1, The Sendai Bridge.

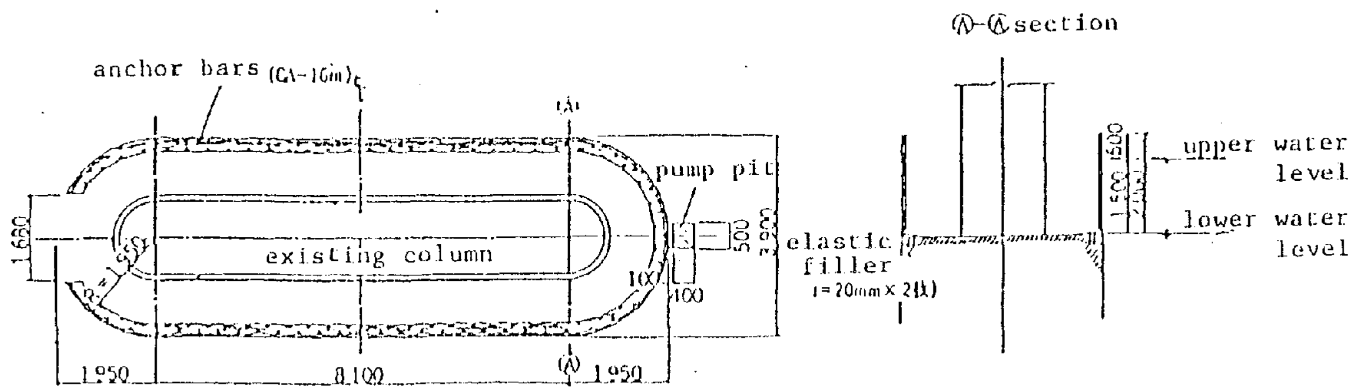


Fig-26 Detail of cut off wall

BRIDGE RETROFITTING DETAILS

Oris H. Degenkolb

INTRODUCTION

The 1971 San Fernando, California earthquake disclosed the fact that many existing bridges had serious seismic deficiencies. The State of California initiated a survey of all its bridges and determined that approximately 1220 could have their seismic resistance increased by retrofitting them to keep the structural segments from separating when shaken by an earthquake. The total program is estimated to cost approximately $50 million and is now slightly more than half completed.

DESIGN METHODS

California's criteria for designing restraining devices have changed a number of times since the retrofitting program was initiated in 1971. The first criteria was very simple and consisted of providing a restraining force equal to 25% of the dead load of the lighter segment of superstructure connected. An effort was made to use ductile materials.

Bridge seismic design specifications have been revised radically since that time, bridge designers are now able to take advantage of the advancements made in the field of computors and structural dynamics, and Load Factor Design methods have superseded the Working Strength Design method.

Restrainers are now basically designed in accordance with the American Association of Highway and Transportation Officials Standard Specifications for Highway Bridges. The equivalent Static Force Method may be used for designing restrainers for bridges with well balanced spans and supporting bents or piers of equal stiffness, but the Response Spectrum Method applied to the structure as a whole is generally preferred for more unusual structures and where contributing dead loads may come from beyond the immediate spans or frames.

Unless there are other limiting factors, such as the ability of a structure to accommodate the restraining forces, restrainers should be designed to resist forces equal to the acceleration, expressed as a percentage of the gravitational force, times the contributing dead load, but not less than 0.35 times the contributing dead load.

The dynamic analysis utilizes a modal analysis based on the application of the response spectrum of ground acceleration to a lumped sum mass space frame of the structure. This method considers the relationship of the site to active faults, the seismic response of soils at the site, and the dynamic response characteristics of the whole bridge.

Dynamic analyses sometime appear to give what seem to be erratic results. Minimum, or less than minimum, restrainers may be determined to be satisfactory, but more restrainer capability at the same location may be calculated as being insufficient. This apparent inconsistency is due to the fact that a stiffer element in a system will "attract" more force. In specific instances where this phenomena has been observed, restrainers with minimum or greater than minimum capacities have been considered to be the more appropriate. Considering the fact that different methods of analysis give drastically different results, none of which may represent what may happen when a structure is shaken by an actual earthquake, it should be realized that a designer must use a considerable amount of judgment.

RESTRAINER DETAILS

Many compromises must be made in designing seismic restrainers. Ideally, restrainers should:

- Be effective in an earthquake.
- Be economical.
- Dissipate energy.
- Keep units of a structure in their initial relative locations.
- Require no maintenance.
- Be accessible for inspection and repair.
- Be repaired or replaced by ordinary maintenance workers.
- Use ordinary tools for repair or replacement.
- Use commonly available parts which don't become obsolete.
- Not use liquids which can leak out or evaporate.
- Be foolproof.

The basic restrainer materials used for retrofitting California's bridges are 3/4" 6x19 cable (Federal Spec. RR-W-410C) and 1¼" Ø bars (ASTM A-722 with supplementary requirements). The end anchorages for the 3/4" cables (Figure 1) develop the full strength of the cable. Cables have a minimum breaking strength of 46 kips, are assumed to have a yield strength of 39.1 kips, and frequently test to 53 kips ultimate.

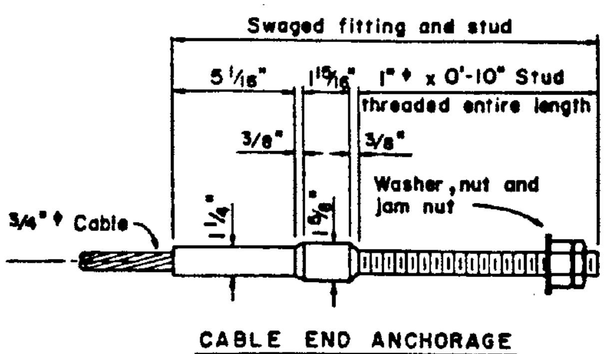


Figure 1

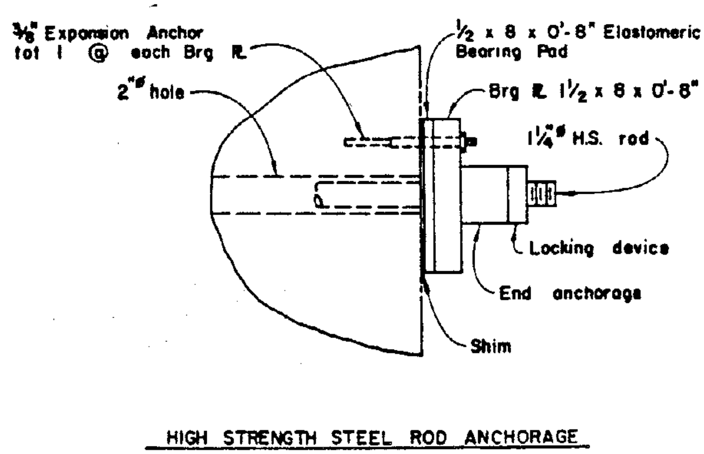


Figure 2

1¼" dia. high strength steel rods are required to have a minimum ultimate strength of 150 kips. The supplementary requirements of ASTM A 722 assure greater ductility. The 1¼ dia. size is readily available competitively, whereas the supply of other sizes may be somewhat limited. The design yield strength is 120 kips and two types of bars are commonly used: Dywidag threadbars, which have a continuous rolled-in pattern of threadlike deformations along their entire lengths, and smooth rods which are cut to length and have machine threaded ends. Although they frequently develop the full strength of the rod, couplers and anchorage devices are required to develop not less than 95% of the specified ultimate tensile strength of the rod. Figure 2 is a diagramatic sketch of a typical end anchorage for high strength steel rods.

Transverse restraining devices in the hinges of older concrete bridges are often considered to be inadequate for keeping the adjacent sections of superstructure aligned during an earthquake. Differential movement between the two sides of a hinge would shear high strength steel rod restrainers. Transverse restrainers (Figure 3) are added when required, to assist in keeping the two sides of a hinge in alignment. The concrete filled pipe transverse restrainers are placed in the direction of normal hinge movements.

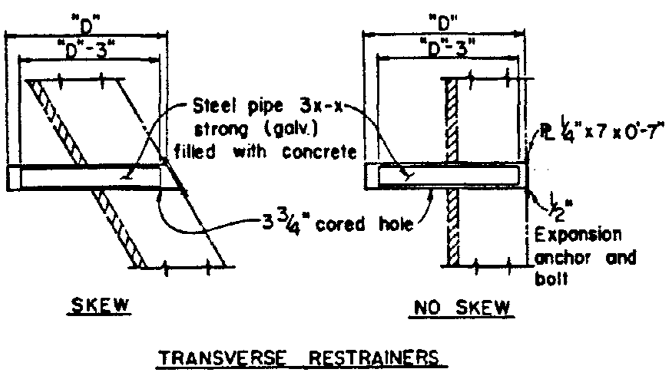


Figure 3

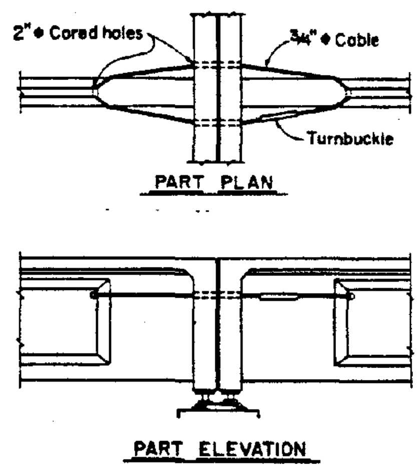


Figure 4

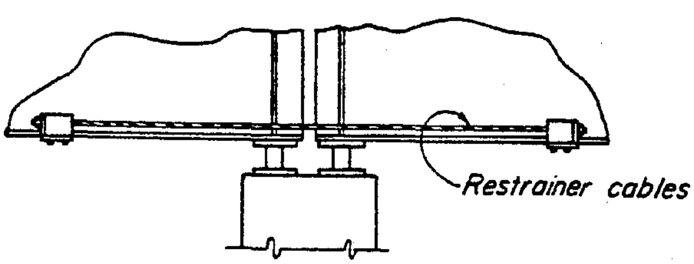


Figure 5

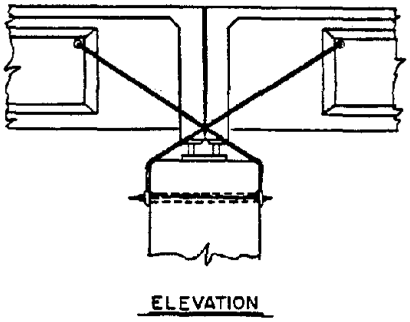


Figure 6

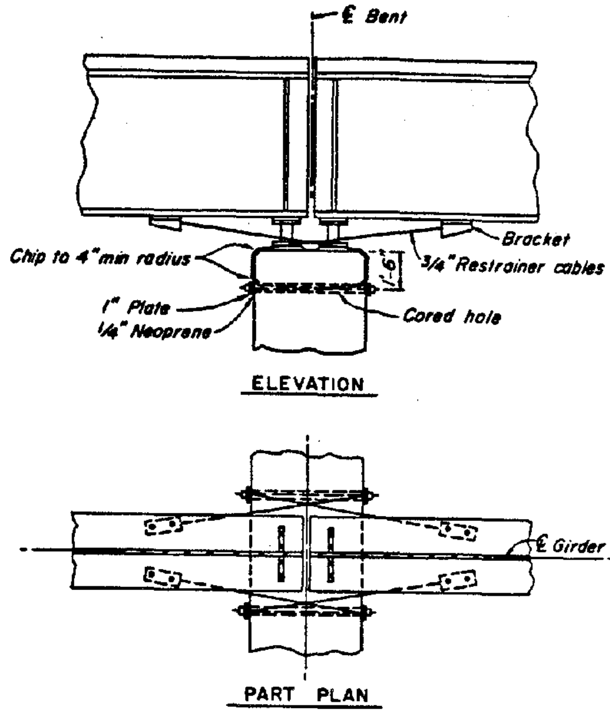


Figure 7

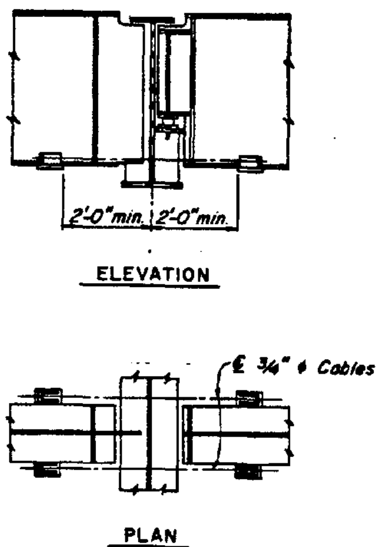


Figure 8

Figures 4 and 5 have been used for connecting segments of super-structures together and are suitable for relatively short structures with wide supports.

Figures 6 and 7 show methods of connecting precast-prestressed and steel girders to bent caps. Although these details are especially suited to long multi-span structures, they are also preferred for shorter structures with few spans. The detail illustrated in Figure 7 may not be suitable in some instances where vertical clearance underneath the structure is critical.

Figure 8 illustrates a method used for connecting steel girders which are supported on a steel girder cap where the girders are in line with each other. In cases where girders cannot be connected directly, because of curved or flared roadways, cable restrainers are attached to beams made up of steel channels which are connected to the bottom girder flanges as shown in Figure 9.

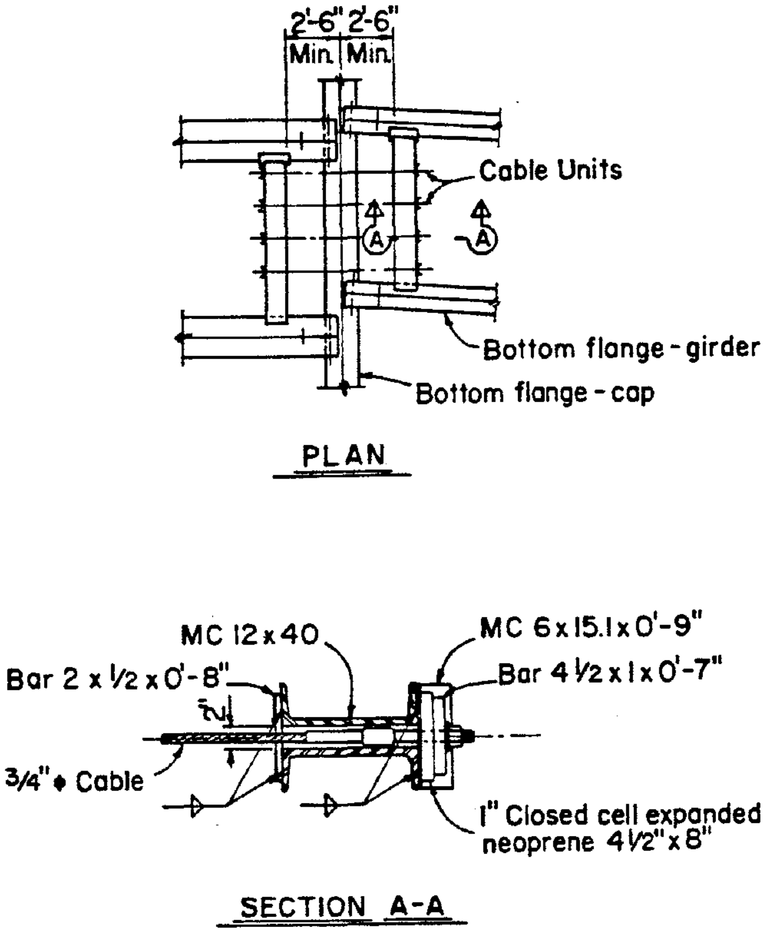
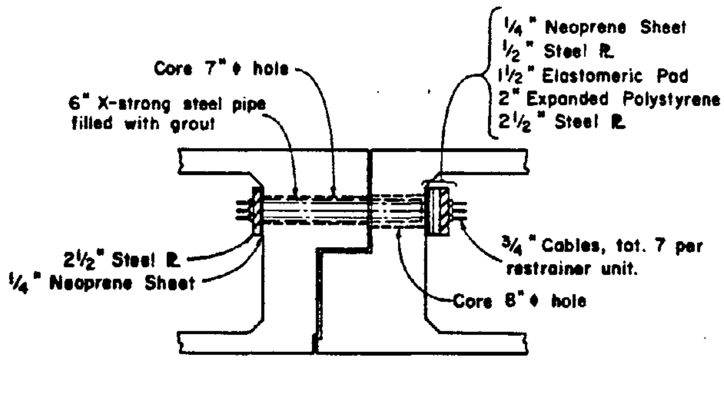


Figure 9



TYPE H-1A HINGE RESTRAINERS

Figure 10

An early type restrainer which was used in box girder hinges is shown in Figure 10. It had the advantage of providing a considerable amount of transverse and vertical, as well as longitudinal, restraint. It's use is limited, however, because many hinge diaphragms don't have the strength to resist the punching-out effect of the restrainer cable anchorages. Another disadvantage is that the grout, which is placed in the pipe to increase the transverse and vertical shearing capacity, reduces the stretching capacity (ductility) of the cables.

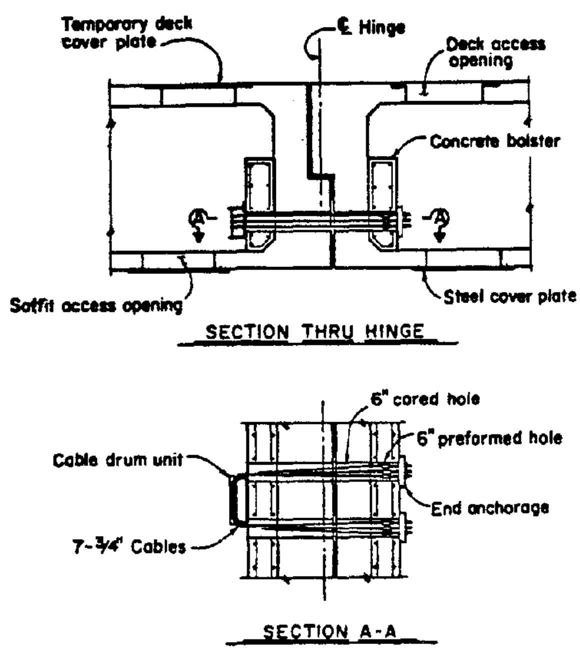


Figure 11

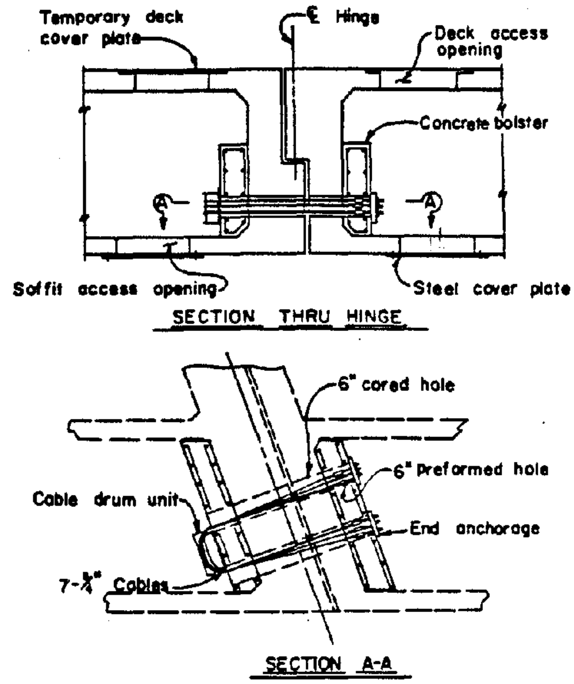


Figure 12

The most commonly used retrofitting detail in California is shown in Figure 11 because of the predominance of concrete box girder bridges. Reinforced concrete bolsters are used to spread out the anchorage forces which would otherwise destroy the hinge diaphragms. Figure 12 is a similar detail which aids in preventing rotation of superstructure segments caused by the skewed ends. The seven cables which are passed twice through the joint give the restraint of 14 cables. Seven cables passing through the hinge three times give the effect of 21 cables, as shown in Figure 13.

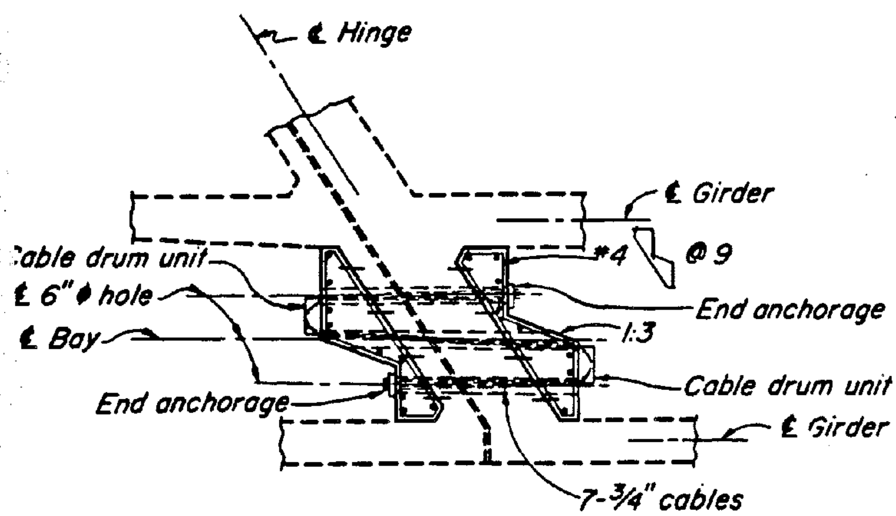


Figure 13

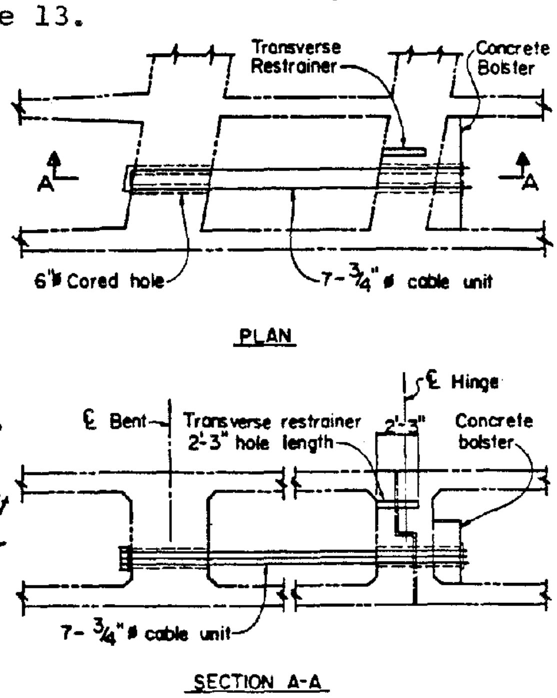


Figure 14

Hinges in box girder bridges are usually located about one-fifth of the span length from a bent. In cases where it is desirable for cables to stretch more than allowed in the previous details, cables are passed all the way through the cantilever end of the span and around the bent cap as shown in Figure 14. This same scheme is also used with rod restrainers. Rods must be longer than an equivalent cable restrainer which provides the same amount of restraint, because of the greater modulus of elasticity. A plan view of rods connecting a hinge to the bent cap of a skewed bridge is shown in Figure 15.

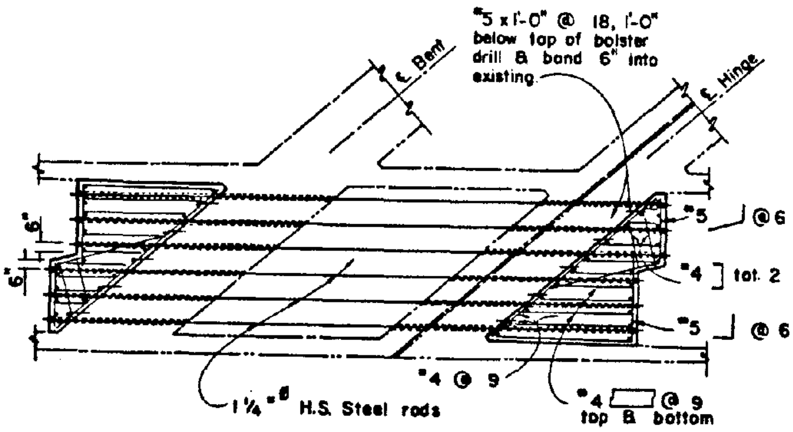
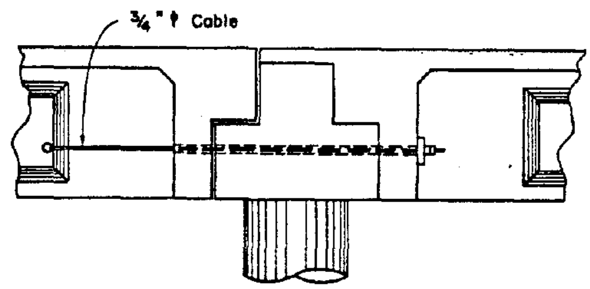


Figure 15



ELEVATION

Figure 16

Although access holes have been made in all three cells, in some contracts, some contractors have found it more economical to omit the access opening in the cantilever cell. They have been successful in aligning the holes through the cap and diaphragm and threading the restrainers through the cantilever cell from the first and third cells. Minor obstructions, caused by supports for the deck forms, can generally be pushed out of the way. Access to the cantilever cells is now optional and at the contractor's expense, if he prefers to use them.

Precast-prestressed girders supported on an "inverted T" cap can be restrained as shown in Figure 16. Coring holes near the ends of prestressed girders present no special problems if the girders are prestressed with strands or wires. Severing a few small tendons will have a negligible effect on the strength of a girder. If girders are tensioned with large rods, precautions should be taken to avoid damaging them.

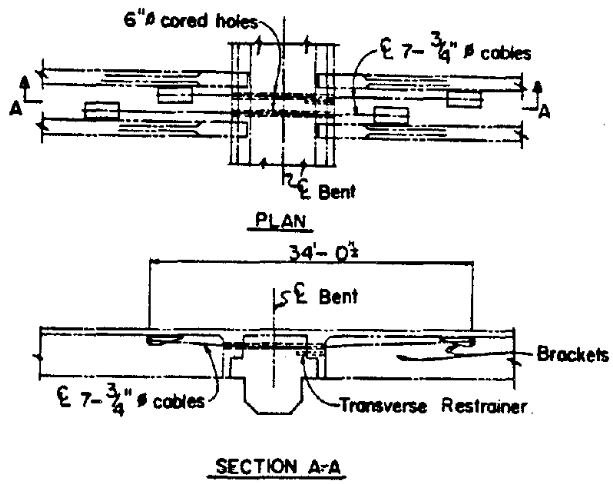


Figure 17

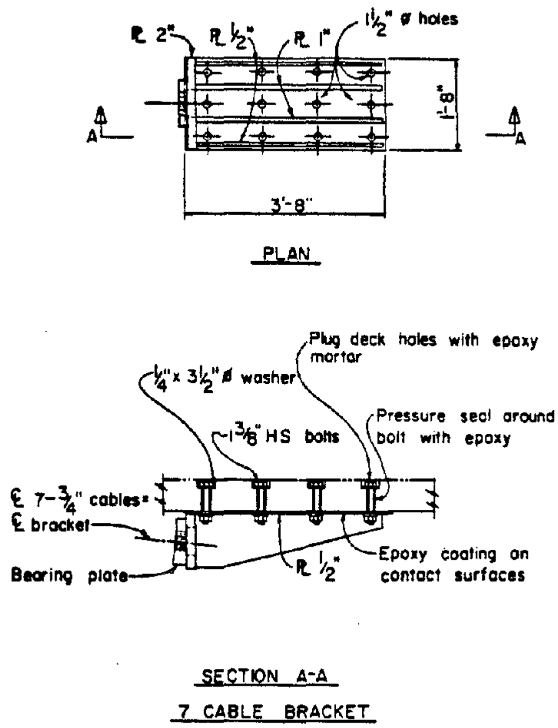


Figure 18

Figures 17 and 18 illustrate a method of restraining precast-prestressed girders at a bent by attaching cable anchorages to the underside of the deck and passing the cables through holes cored through the bent cap where it was considered impractical to attach the anchorages to the girders. Similar schemes have been used using rods in lieu of cables. Care must be taken to avoid damaging main cap reinforcement and not bending restraining rods excessively. Figure 19 shows the same general scheme where it was not practical to attach restrainers to the girders or inadequately reinforced diaphragms.

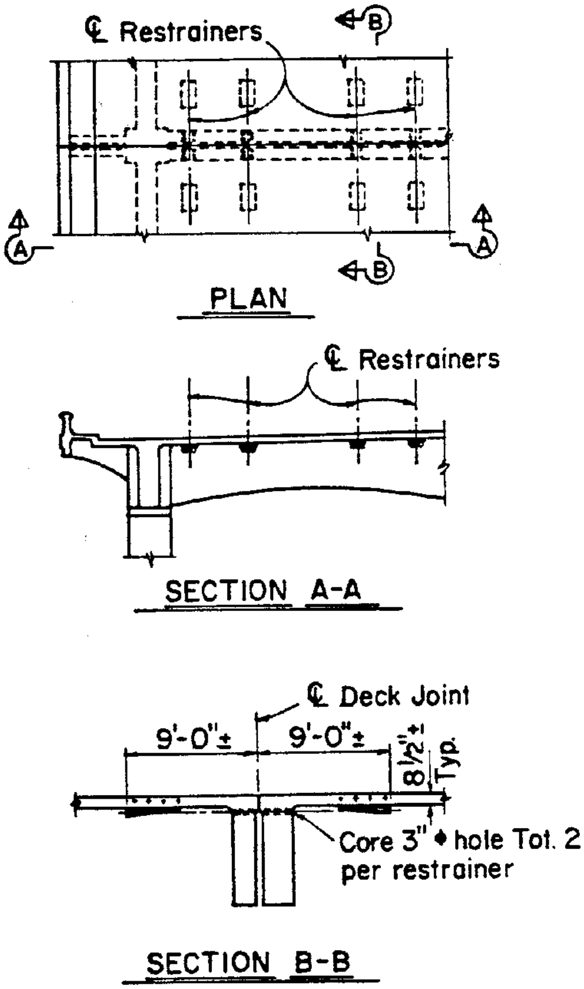


Figure 19

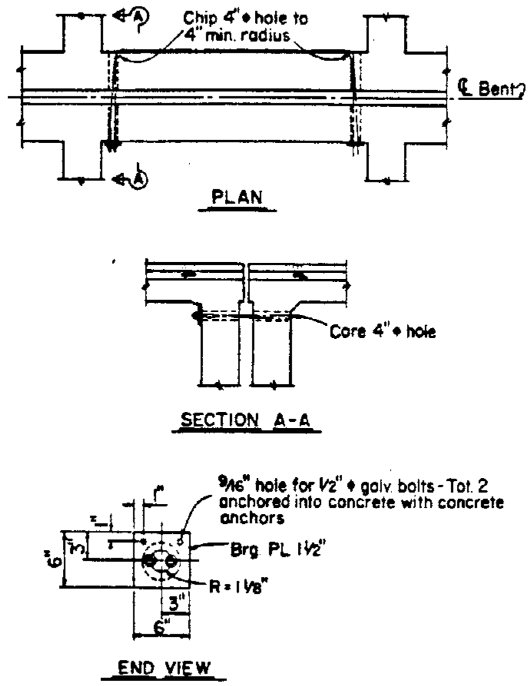


Figure 20

Figure 20 shows a typical restrainer for small T-beam bridges which have only a few spans with wide supports. An identical detail has been used for T-beam bridges with very narrow hinge seats.

Figure 21 is commonly used for T-beam bridges with diaphragms too lightly reinforced to safely resist the required restrainer forces.

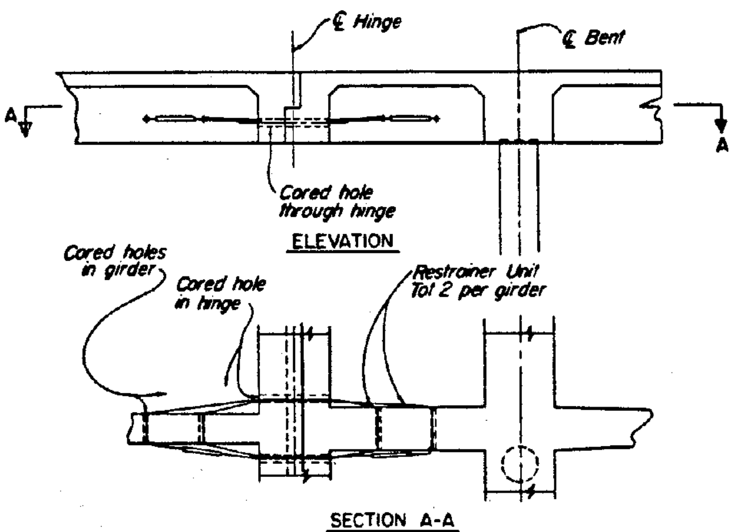


Figure 21

Continuous longitudinally reinforced concrete bridges with hinges are seldom retrofitted because the spans, if unseated in an earthquake, will not fall down under the influence of their own dead load. It is presumed that the problem will be recognized and temporary shoring placed or other remedial action taken before any serious problems occur. Simple spans, drop-in spans, and some specially designed slab bridges which are certain to drop if they become unseated, have been retrofitted.

Figure 22 shows the detail used for restraining a drop-in slab adjacent to a T-beam span. Hinges of special slab bridges have been restrained in an almost identical manner with diagonal holes being cored on both sides of the hinge (similar to the right hand side of Figure 22). The cable ends in those cases were either anchored in the deck or connected with a turnbuckle underneath the deck.

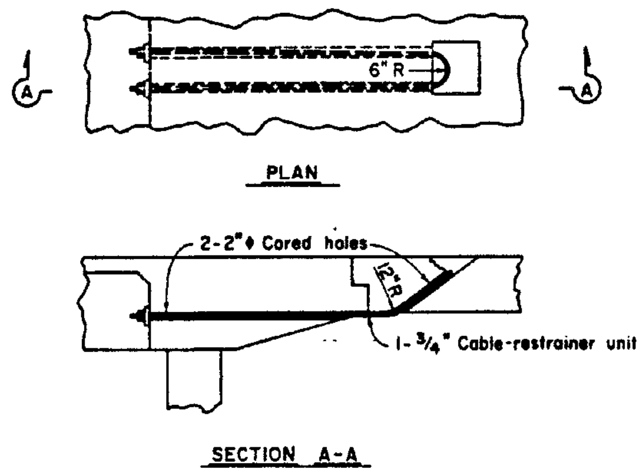


Figure 22

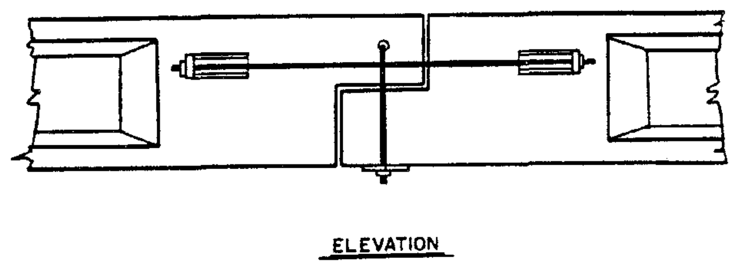


Figure 23

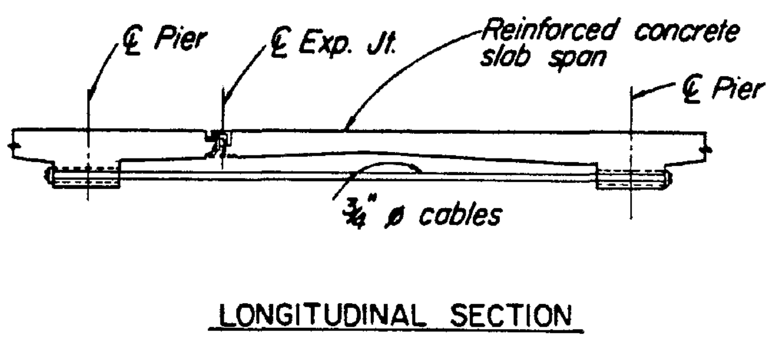
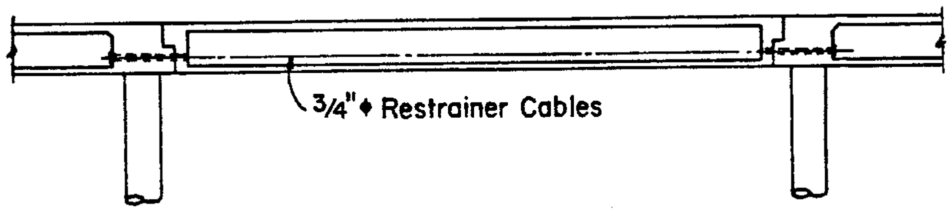


Figure 24

Precast-prestressed girder hinges have been restrained longitudinally and vertically as shown in Figure 23. The possibility of differential lateral movement should be considered also -- especially if the spans are skewed.

Hinges in relatively short spans or short drop-in spans can be restrained by connecting the adjacent piers with restrainers as shown in Figures 24 and 25. One of the main problems with this scheme may be excessive stretching of the tendons if the span is rather long. More tendons can be used to overcome that problem, but that also increases the cost.



ELEVATION

Figure 25

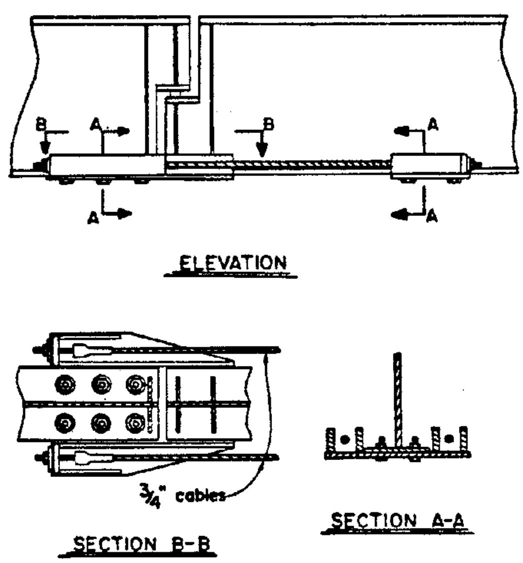


Figure 26

Figure 26 shows a detail for restraining a commonly used steel girder hinge. The welded plate assembly bolted to the bottom flange of the suspended side restrainers excessive transverse and vertical movement. The cables connecting that assembly and a bracket attached to the bottom flange of the cantilever limit the longitudinal opening of the hinge.

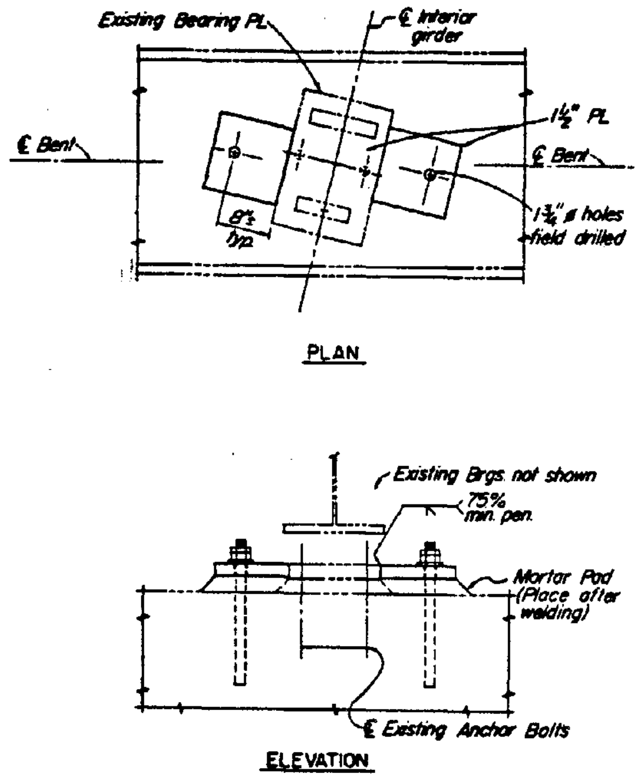


Figure 27

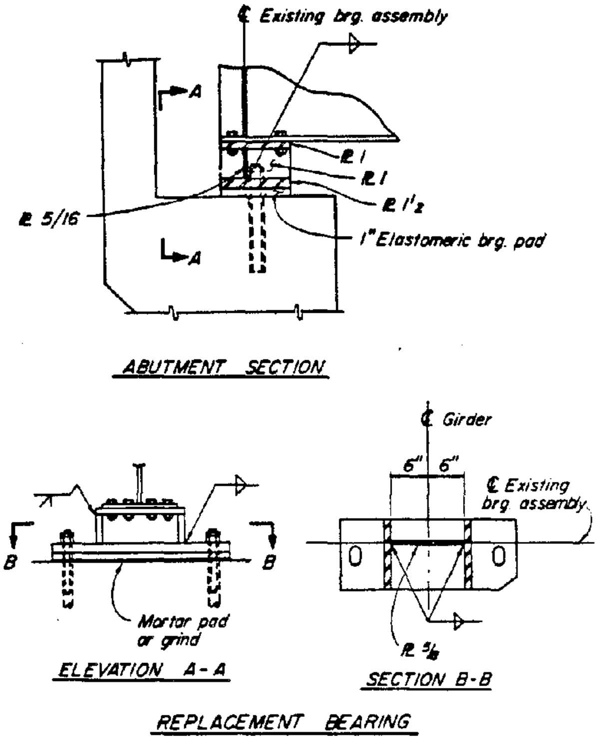


Figure 28

Masonry plate anchor bolts are one of the most seismically vulnerable details. Additional horizontal support has been given to some masonry plates by welding steel plate extensions with additional anchor bolts to existing bearing assemblies, as shown on Figure 27. Other portions of the bearings should be investigated and additional corrective action taken, if necessary, to make certain that there are no equally vulnerable deficiencies remaining. Steel bearings and anchor bolts should be designed to resist at least twice the calculated force that they may be subjected to.

It is sometimes advisable to replace existing steel bearings -- especially if they might allow the bridge to drop more than 6 inches or lead to other failures. This has been done as shown in Figures 28 and 29. Figure 28 shows a steel rocker bearing that was replaced with a welded steel pedestal and elastomeric bearing pad. It may also be necessary to provide additional longitudinal and/or transverse restraint in addition to this detail. Figure 29 illustrates how steel rocker bearings have been replaced by using elastomeric pads for new bearings and reinforced concrete to support the pad and provide transverse restraint. Longitudinal restrainers may also be required in addition to this detail.

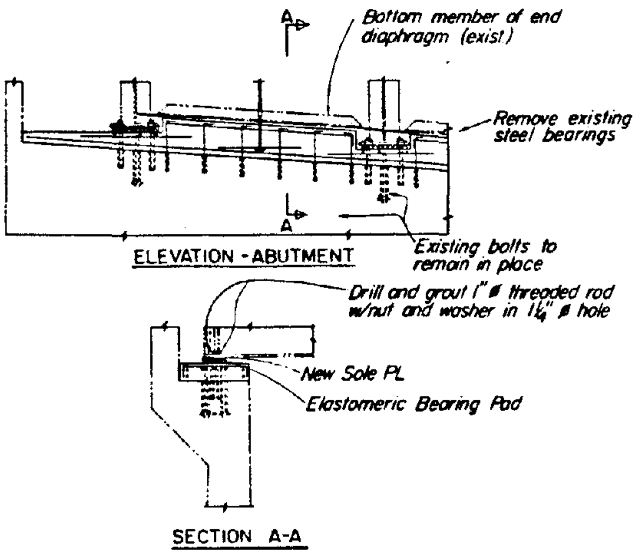


Figure 29

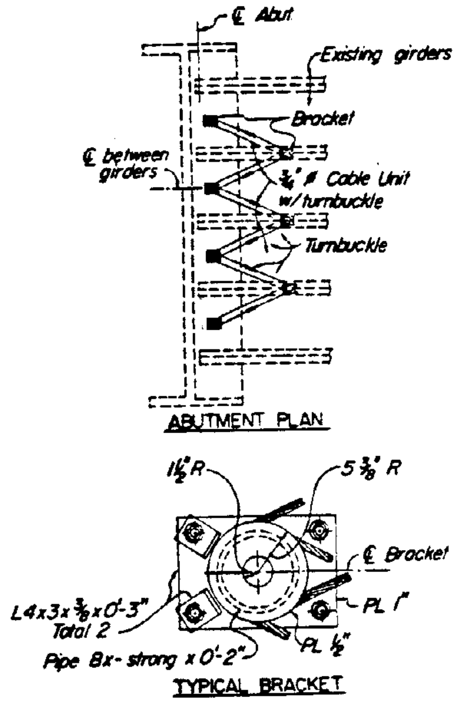


Figure 30

Figure 30 shows how steel girders can be given additional longitudinal and transverse restraint at an abutment. A method used for restraining steel girders transversely is shown in Figure 31.

The solid steel pin shown on Figure 32 will provide transverse and longitudinal restraint at an abutment. One of the main limiting factors for this detail is the strength of the abutment seat and end diaphragm. It is also a good detail for new construction where the concrete can be reinforced to develop the forces imposed by the steel pin. Although this scheme can also be used at intermediate supports there may be large amounts of negative reinforcement in bent caps that would be cut or damaged by coring the vertical holes.

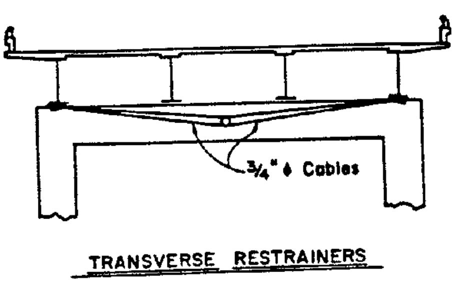


Figure 31

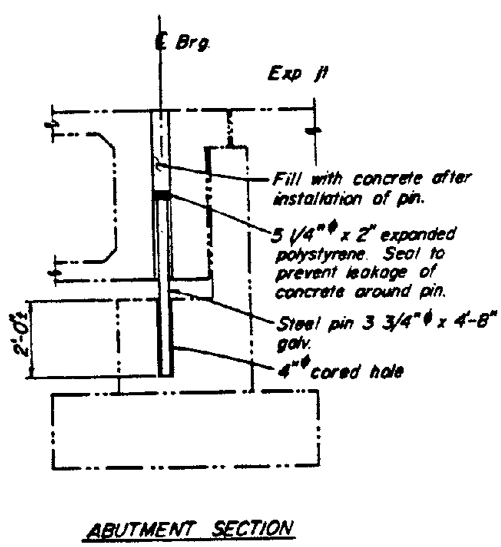


Figure 32

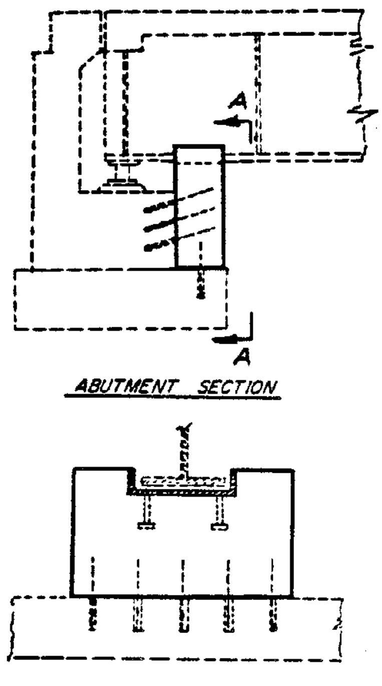


Figure 33

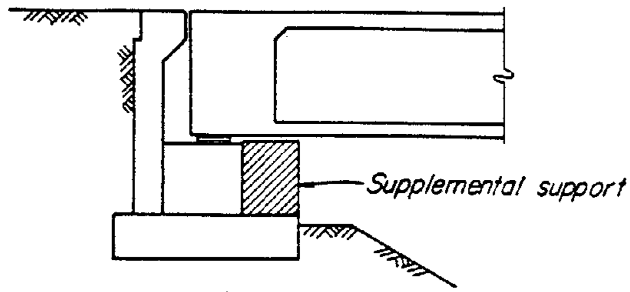


Figure 34

Figures 33 and 34 illustrate methods used for providing supplemental support under the ends of steel or concrete box girders, respectively, when longitudinal movements might exceed the capacity of the bearings.

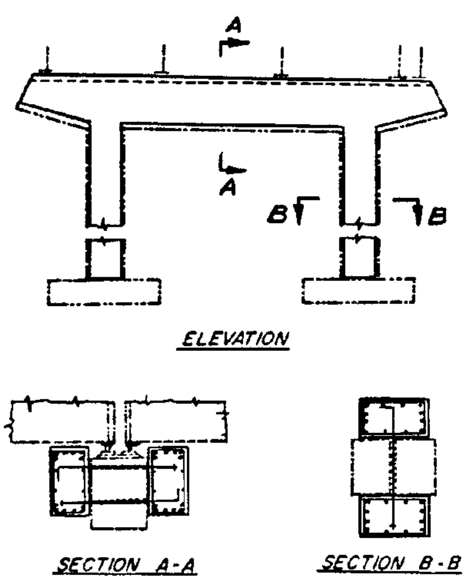


Figure 35

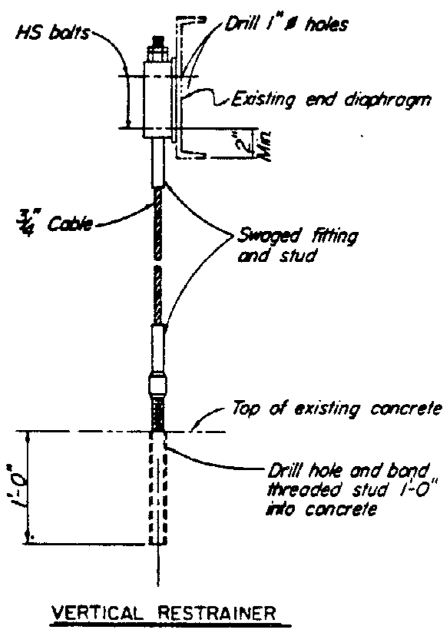


Figure 36

The support width of one bent in the center of a long viaduct was increased as shown in Figure 35. Based on the results of a dynamic analysis, it was felt that it would be advantageous to permit the viaduct to work as two short structures rather than one long one, with provisions for extra movement taken at this bent. Extensions were made on both faces of the cap and two columns to make the bent architecturally compatible with the rest of the structure.

Figure 36 shows a type of vertical restrainer commonly used in steel girder bridges. The upper end is attached to the end diaphragm and the lower end grouted into the top of the bent cap. Care should be taken to avoid the negative reinforcement in the bent cap.

CONCLUSIONS

The wide variety of bridges constructed over a period of many years in a large and varied area such as California makes it virtually impossible to use a few standard details for accomplishing an extensive retrofitting program. The details described in this paper are not necessarily complete in themselves. It is frequently necessary to use more than one detail to restrain a segment of a structure adequately. The combination of different construction materials, span lengths, skews, alignment, framing, vulnerable details, etc., makes it necessary to examine every structure as a unique problem.

AN INVESTIGATION INTO
METHODS OF NONDESTRUCTIVE EVALUATION
OF MASONRY STRUCTURES

By James L. Noland[1], Richard H. Atkinson[1], and John C. Baur[1]

INTRODUCTION

Objective of the Research

The objective of the research program described herein was to assess the applicability of selected nondestructive evaluation (NDE) methods to the strength and quality evaluation of masonry (38)[2].

NDE methods have been used extensively and with varying degrees of success on many other materials, e.g., concrete, metal, composite, and epoxy (1,2,4,6,9,10,11,15,17,21,23,24,25,26). Among NDE methods applied to other materials are: hardness, rebound, penetration, insert pull-out, vibration, radioactive, ultrasonic pulse velocity, mechanical pulse velocity, electrical, microwave absorption and acoustic emission. The application of NDE methods to masonry has been very limited with some applications in field situations (18,31).

Research was therefore needed under controlled conditions, i.e., the laboratory, using carefully constructed specimens to determine the effectiveness of selected NDE methods applied to masonry considering a range of material parameters and in unflawed and flawed conditions. The degree of success, data obtained, experimental methodology developed provide a basis for identifying additional research requirements and guidance for field applications.

Why the Research Was Conducted

Retrofit and strengthening of existing masonry buildings to increase their service life consistent with proper concern for safety has become an important issue because of several factors. There is a large number of such structures in the national inventory which may or may not be safe for continuing or changing use. Demolition and replacement of all of them is not feasible because of the scope of the effort and cost of new construction. Increased awareness of seismic hazards and technological developments leading to more stringent code requirements (32,33) for new construction have also caused attention to be focused on the adequacy of existing masonry buildings (16,29).

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[1]Atkinson-Noland & Associates, Inc., Boulder, Colorado.

[2]Numbers in parentheses refer to references in Appendix I.

Viable NDE methods would enable more comprehensive assessments of existing structures to be made for the purposes of retrofit and strengthening. The structural assessment of existing masonry structures is generally based upon visual observations and data obtained from destructive tests of small specimens taken from the structure (7,16,28,29). These methods are limited because visual observations can only reveal gross defects, and testing of a sufficient number of specimens taken from the building to permit a comprehensive assessment may be prohibitive due to cost, time and aesthetic considerations (18).

Basis for Selection of NDE Methods for Assessment

NDE methods were selected for review anticipating the conditions and constraints which would affect any NDE method applied to a masonry building. Of the many NDE methods which exist, first consideration was given to those which actually or potentially are amenable to field application on a general scale.

NONDESTRUCTIVE EVALUATION METHODS CONSIDERED

Schmidt Rebound Hammer

The Schmidt Rebound Hammer is basically a surface test apparatus developed for concrete testing (17). The device uses a spring activated hammer which impacts a steel plunger that is in contact with the test surface. The rebound of the hammer is measured on a scale on the device and the measurement is termed the "rebound number".

The rebound hammer has been used to evaluate concrete (11) and rock (1,5), and to provide data used to predict performance of rock tunnel boring machines (21).

Mechanical Pulse Velocity

The basis of the mechanical pulse velocity method is the correlation of compression stress wave velocity to material properties where the stress wave is generated by a single impact by a hammer, piezoelectric crystal, or explosion. Wave velocity is primarily a function of elastic modulus, Poisson's ratio, and density, however correlations have been found between various strength properties of concrete and wave velocity (11,26). The method has also been observed to be a prediction of deformation modulus of rock (5). The velocity of mechanically-induced shear waves has been correlated to shear modulus in foundation material studies (15).

Wave velocity is also affected by discontinuities in the material, i.e., flaws of various kinds (4,11). Thus the method is potentially amenable to flaw detection.

Ultrasonic Pulse Velocity

The basis of the ultrasonic pulse velocity method is the same as that of the mechanical pulse velocity method. The primary difference is in the means of generating stress waves. While the mechanical pulse velocity method relies upon a single impact, the ultrasonic pulse velocity method utilizes an electroacoustic transducer to produce high frequency stress waves. This method provides more control over the type and frequency of stress waves generated. Usually the transducers are designed to generate compression waves (11,26).

The velocity of stress waves generated by this method is also affected by flaws and voids (4,11). The method is therefore amenable to flaw detection (37).

Vibration

Free vibration induced by initial displacement or impact may be analyzed to yield natural frequency, modulus, and damping values which can be related to material quality. Vibration measurements have been used to characterize the overall properties of tall masonry buildings (12,13). Vibration characteristics are sensitive to continuity, hence may be amenable to flaw detection.

Penetration

The strength and stiffness of the material are among the factors which determine the depth of penetration of a probe of a given mass, shape, and impact velocity. The relationship between material strength and probe penetration is the basis of penetration methods for NDE of brittle materials. Penetration methods have been used in an experimental evaluation of masonry in field conditions (31).

Acoustic-Mechanical Pulse[1]

The acoustic-mechanical pulse method is an adaptation of the acoustic-ultrasonic technique (6,23,24,25) which in turn is an adaptation of the well-known acoustic emission technique (11).

Acoustic emissions are small amplitude elastic stress waves caused by local deformations in a material at a point or points being strained beyond the elastic limit. Stress waves thus generated are detected by sensors with high sensitivity to surface displacements. Various characteristics of the stress waves (see Figure 1) may be measured by suitable equipment. Data thus obtained has been used to qualitatively

[1]The number of tests performed with this method was extremely limited. Its evaluation was not included in the original research plan. The tests performed were made possible through the courtesy of the Acoustic Emission Technology Corporation, Sacramento, CA.

assess material with respect to flaw location and growth, and general condition (11,24).

The acoustic-ultrasonic method is based upon the replacement of energy released by local deformations of the material with energy introduced in the form of ultrasonic pulses. The stress waves thus generated mimic the stress waves caused by local deformations of a material under a state of stress (23,24,25).

The acoustic-mechanical pulse method, used in this research, relies upon introducing energy into a specimen by a single mechanical impact. The stress wave induced also mimics stress waves caused by local deformation, as shown in Figure 1.

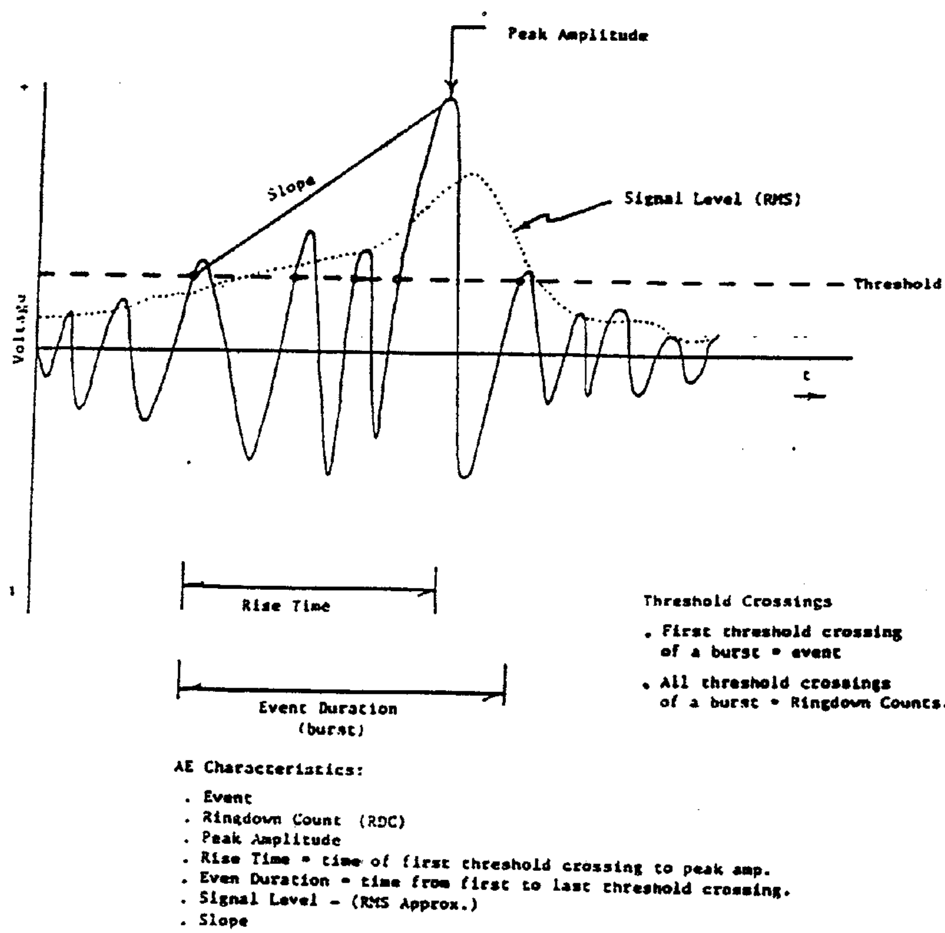


Figure 1 Stress Wave Characteristics

RESEARCH PLAN

Basic Approach

NDE measurements from large masonry specimens were compared to strength measurements from destructive tests of small companion masonry specimens. Statistical methods were used to quantify the correlation between NDE measurements and strength data.

Nondestructive Tests on Masonry Wall Specimens

Nondestructive tests were performed on large masonry wall specimens to enable the physical aspects of using the equipment and the sensitivity of the measurements to masonry material variations and flaws in masonry to be assessed. The walls were constructed in the laboratory in the cantilever condition. Construction in the laboratory enhanced quality and the cantilever configuration provided simple boundary conditions. The walls were constructed of three different types of solid clay unit each of a different compressive strength. Walls of each kind of unit were built using each of five different mortar mixes in order to be able to assess the capability of each NDE method to detect differences in masonry due to unit and mortar variations.

Subsequent to NDE measurements, flaws in the form of bed joint delaminations were introduced to assess the ability of NDE methods to detect such flaws.

Destructive Tests of Companion Small Masonry Specimens

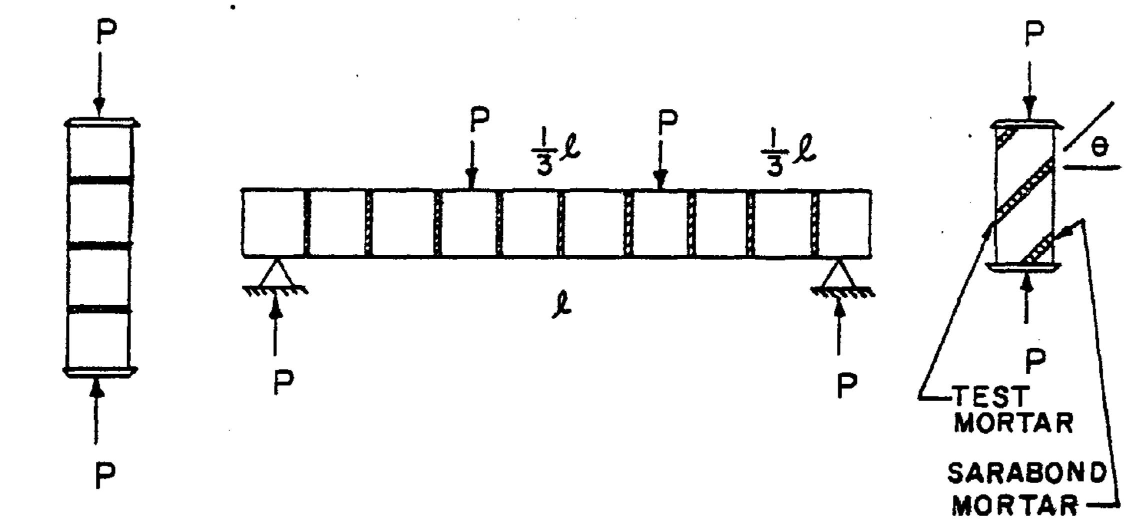
Destructive tests of small masonry specimens were conducted to obtain strength data for correlation with NDE measurements. The small specimens were built of the same materials and at the same time as their corresponding wall specimens. The tests and specimens included:

1) compression tests of stack-bond prisms (Figure 2a),
2) flexural tests of stack-bond beams (Figure 2b), and
3) shear test of inclined-bed-joint prisms (Figure 2c).

Compressive tests were also done on individual masonry units and mortar cubes of each mix to provide strength data of the masonry components and are documented elsewhere (38).

Destructive Tests of Core Specimens

Subsequent to NDE tests, cylindrical cores eight inches in diameter were removed from the wall specimens. Core specimens were loaded in compression across a diameter 15 degrees from the diametral bed joint as shown in Figure 3. This test is used in Los Angeles as a partial means of strength evaluation of existing masonry buildings (16,29).



COMPRESION a) FLEXURE b) SHEAR c)

FIGURE 2 SMALL SCALE SPECIMAN TESTS

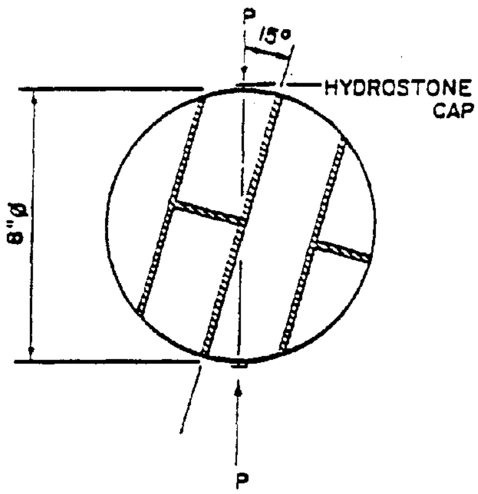


FIGURE 3 SHEAR-BOND TEST BY DIAMETRAL COMPRESSION

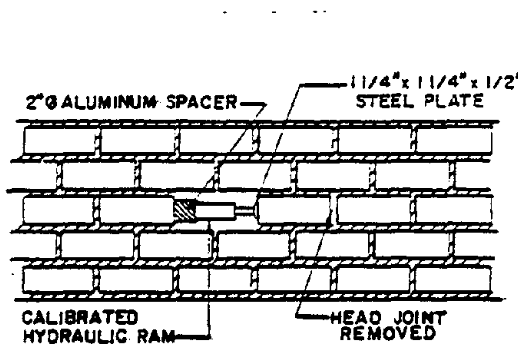


FIGURE 4 IN-PLACE JOINT SHEAR TEST

In-Place Joint Shear Tests

Also subsequent to NDE tests, bed joint shear strength was determined for a limited number of walls by the in-place shear test. The test consists of laterally displacing a single unit in the outer wythe relative to adjacent units in that wythe. Room is made for displacement by removing the head joint of one end. Force is applied by a jack placed in the void created by removal of the unit on the opposite end of the unit to be displaced as shown in Figure 4. This test is also being used in Los Angeles as a partial means of evaluating older existing masonry buildings (29).

Correlation of Nondestructive and Destructive Measurement

Linear bivariate regression analyses were done to assess the correlation between the various NDE measurements and the destructive test results. The computer routine used for the analyses provided the equation of the best-fit line and values for the coefficient of determination and correlation coefficient.

PREPARATION OF SPECIMENS

Construction of Wall Specimens

Thirty wall specimens were constructed of solid clay units (pavers with zero void area) on concrete pedestals 20 inches wide and 7 inches thick which had been poured onto an existing 5 inch slab. The pedestals were anchored to the slab by ½ inch diameter anchor rods.

All pavers used had nominal dimensions of 4 in. x 8 in. x 2¼ in. Actual dimensions varied somewhat among the three types used as seen in Table 1.

TABLE 1

DIMENSIONS OF SOLID CLAY UNITS

Unit	Length (in.)	Width (in.)	Height (in.)
Antique Rustic (A.R.)	7 5/8	3 9/16	2 1/4
Iron Spot (I.S.)	8	4	2 1/4
Hard Pressed (H.P.)	8	3 15/16	2 3/8

The walls were two wythes thick and seven units wide with a ½ inch collar joint and 3/8 inch bed joint. Those built with Antique Rustic

[1]Antique Rustic and Iron Spot units were modern, extruded, wire-cut clay units. The Hard Pressed units were molded, reclaimed units circa 1920 and were relatively soft.

and Iron Spot units were 28 courses high while, due to height restrictions, those built with Hard Pressed units were 26 courses high.

The bed joints on one surface were tooled and the bed joints on the opposite side were struck flush to represent inner and outer wythe construction.

Mortar ingredients, i.e., portland cement, type S lime, sand and water were measured by weight according to mix proportions to a flow of 130 ± 5%. The mortar was mixed for 3-5 minutes in batches of approximately 1½ cubic feet in an electric powered, paddle type mixer.

Preparation of Small-Scale Specimens

Small-scale specimens for destructive tests were constructed of the same materials used in the wall specimens on the same day as each wall specimen, and by the same mason. Mortar was taken from the same batch used for the wall specimen. Subsequent to completion of NDE tests, i.e., after 28 days age, core specimens were removed from each wall specimen and shear-test specimens prepared on the wall. Compression, flexural, and inclined bed joint shear specimens were built in a laboratory and left undisturbed for 12 hrs prior to removal to a 100% humidity fog room. After 7 days the specimens were stored in laboratory conditions, i.e., 70°F± and 40%± humidity until 28 days of age.

SUMMARY OF TESTS

The aggregate of wall specimens and NDE tests performed is summarized in Table 2. Because of the very low bond strength of 0:1:3 mortar, it was not possible to test flexural, inclined bed joint, nor 8-inch diameter cores; the specimens failed during handling. Three flexural specimens made with mortar containing cement were also destroyed during handling. Conversely, the in-place shear test was not accomplished for walls built with mortar of 1:¼:3 proportions because the bond strength of the unit to mortar exceeded the capacity of the 20,000 pound jack. The in-place shear test was only done for a limited number of walls early in the project.

Small-scale specimens built with each mortar-unit combination used for wall specimens included 53 compression prisms, 31 flexural prisms, 63 inclined-bed-joint shear specimens, 58 eight-inch cores, and 24 in-the-wall shear specimens (38). Small-scale specimens were nominally built and tested in sets of three. Additional specimens were built and tested if the results of the initial sample were erratic. All results were used in subsequent analyses, however. In two cases, specimens were accidentally damaged resulting in a sample size of two. The lost specimens were not replaced because of the consistent results from the remaining specimens.

TABLE 2

WALL SPECIMENS

Wall Series No.	Clay Unit Type	Mortar Type	*Nondestructive Tests Performed
IA	A.R.	0:1:3	SH, V, UP, MP
IB	A.R.	0:1:3	SH, V, UP, MP
IC	A.R.	0:1:3	SH, V, UP, MP
ID	A.R.	0:1:3	SH, V, UP, MP, DP
IIA	A.R.	1:¼:3	SH, V, UP, MP
IIB	A.R.	1:¼:3	SH, V, UP, MP
IIC	A.R.	1:¼:3	SH, V, UP, MP
IID	A.R.	1:¼:3	SH, V, UP, DP
IIIA	A.R.	1:1:6	SH, V, UP, MP
IIIB	A.R.	1:1:6	SH, V, UP, MP
IIID	A.R.	1:1:6	SH, V, UP, MP
IVA	A.R.	1:2:9	SH, V, UP, MP, DP, AMP
IVB	A.R.	1:2:9	SH, V, UP, MP, AMP
IVC	A.R.	1:2:9	SH, V, UP, MP, AMP
IVD	A.R.	1:2:9	SH, V, UP, DP
X	A.R.	1:3:12	SH, V, UP, MP, DP
VA	I.S.	1:¼:3	V, UP, MP, DP
VB	I.S.	1:¼:3	V, UP, MP
VC	I.S.	1:¼:3	V, UP, MP
VIA	I.S.	1:1:6	SH, V, UP, MP, DP
VIB	I.S.	1:1:6	SH, V, UP, MP
VIC	I.S.	1:1:6	SH, V, UP, MP
VII	I.S.	1:2:9	SH, V, UP, MP, DP
VIII	I.S.	1:3:12	SH, V, UP, MP, DP
IX	I.S.	0:1:3	SH, V, UP, MP, DP
XI	H.P.	1:¼:3	SH, V, UP, MP, DP
XII	H.P.	1:1:6	SH, V, UP, MP, DP
XIII	H.P.	1:2:9	SH, V, UP, MP, DP
XIV	H.P.	1:3:12	SH, V, UP, MP, DP
XV	H.P.	0:1:3	SH, V, UP, MP, DP

*Code:

SH = Schmidt Hammer
V = Mechanical Vibration
UP = Ultrasonic Pulse Velocity
MP = Mechanical Pulse Velocity
DP = Densicon Penetrometer
AMP = Acoustic Mechanical Pulse

EXPERIMENTAL PROCEDURE AND EQUIPMENT

Procedures for destructive tests of small-scale specimens followed standard methods (34) and methods developed in previous research (3,14,16,24). Procedures for nondestructive tests were based upon methodology described in the literature (1,4,9,11,13,15,23,24,25, 26,31) and were adapted to masonry during the early stages of the research reported herein (38).

NDE measurements were initially made on the wall specimens in the "as-built" condition, i.e., no known defects. Bed joints between selected courses were subsequently delaminated to determine the ability of NDE methods to detect such flaws in masonry.

Rebound numbers were obtained at each of ten points on the surface of each wall specimen as shown in Figure 5. The number recorded was the average of ten obtained at each point.

Mechanical pulse velocities were determined both vertically and horizontally in the plane of the wall specimens. The pulses (stress waves) were induced by hammer impact on the top and side of a specimen opposite wall-mounted horizontally and vertically oriented accelerometers. Impact points and accelerator locations are shown in Figure 5.

Test points denoted by letters A through M used for ultrasonic pulse velocity measurements are shown in Figure 5. Three types of ultrasonic pulse velocity measurements were made: 1) through-the-wall at each point (see Figure 6a), 2) semi-direct with one transducer on the end of unit A and the other successively placed on points B through G as in Figure 6b, and 3) indirect with one transducer on point H and the other successively on units I through M as in Figure 6c.

Wall specimen vibration measurements were made using a siesmometer placed on the top center of the walls as shown in Figure 5. Vibrations were caused by a light tap with a hammer on the wall centerline two courses from the top.

Penetration tests into units and mortar joints of wall specimens were made at random locations.

Acoustic-mechanical pulse measurements were made with a wall-mounted sensor in the same location as the accelerometers used for pulse velocity tests. Stress waves were induced by applying the Schmidt hammer to two units in the top course of the wall, and one location on each of courses 5, 6, 8, 16, and 19 (38).

Subsequent to delamination of a bed joint, mechanical pulse velocities were obtained in the vertical direction by successively striking the surface of the wall at the uppermost course and at each lower course on a vertical line directly above the vertically oriented accelerometer. The location of the delaminated bed joint was such that several of the courses which were struck were below the flaw, but above

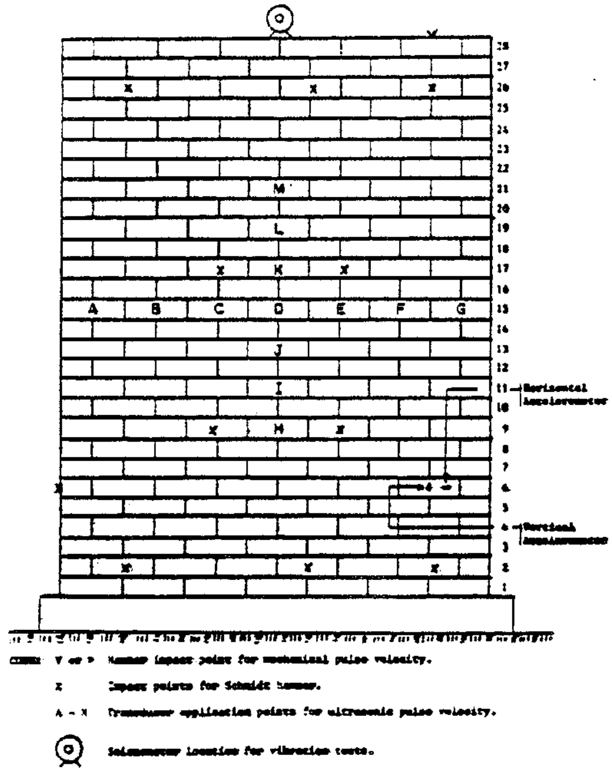


FIGURE 5 NDE TEST POINTS ON WALL SPECIMENS

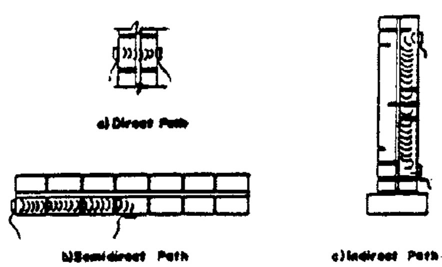


FIGURE 6 TYPES OF WAVE PATHS THROUGH THE WALL SPECIMENS

the accelerometer. Essentially the same procedure was followed with the acoustic-mechanical pulse and ultrasonic pulse velocity methods.

Equipment used to obtain NDE data was commercially available and with the exception of acoustic emission signal analysis equipment, was of a type available from several sources (38). The equipment and associated NDE tests are:

1) "Hardness" (Rebound Number) - Schmidt Hammer, Type N.

2) Mechanical Pulse Velocity - Wall-mounted accelerometer, hammer-mounted accelerometer, signal boost units, storage oscilloscope.

3) Ultrasonic Pulse Velocity - Ultrasonic concrete tester with CRT display (James V-Meter).

4) Vibration - Seismometer and strip-chart recorder.

5) Penetration - Handgun type of instrument using a powder charge to drive a steel probe (Densicon Penetrometer).

6) Acoustic-Mechanical Pulse - Stress wave signal amplifier/processor (Acoustic Emission Technology Model 140B/204GR).

ANALYSIS OF EXPERIMENTAL DATA

Methodology

Relationships between NDE measurements and strength properties as determined by destructive tests of small-scale specimens were obtained by bivariate, linear regression, least squares methods. Each analysis yielded an equation of the form:

$$y' = A + Bx \quad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which: y' = the predicted value of the dependent variable, a strength property in this case.

x = the independent variable, a NDE measurement in this case.

A = the x-axis intercept

B = a constant

Accompanying statistics include the coefficient of determination, R^2, the correlation coefficient, R, and the standard error of estimate, S_{yx}.

The effect of flaws upon NDE measurements was noted by percentage changes and discontinuities of plotted results.

Because of limited data, results of acoustic-mechanical pulse tests were observed only to evaluate consistency of results and sensitivity to flaws. Data obtained and its evaluation are presented elsewhere (38).

Results of Analyses

Because the intent of the research was to assess the capabilities of and reliability of NDE applied to masonry, the information of primary interest was the correlation coefficient (and coefficient of determination) associated with each linear regression expression.

Table 3 is a presentation of the coefficients of determination and correlation coefficients corresponding to each regression equation obtained. Typical plots of data and regression expressions are presented in Figures 7, 8, and 9 along with the accompanying statistics.

Comparisons of damped[1] natural frequency for unflawed wall specimens versus that for flawed specimens is presented in Table 4.

A plot of stress wave arrival time at an accelerometer mounted on the sixth course versus course number of impact points for the mechanical pulse method is shown in Figure 10. The influence of a delaminated bed joint between courses 14 and 15 is evident. The percent increase in arrival time varied considerably among the 14 wall specimens which were flawed, but was always noticable. The average increase was 100% (38).

Results of the in-the-wall bed joint shear tests performed on specimens made with 0:1:3 mortar clearly reveal the influence of normal stress upon shear capacity. For wall specimen IC, for example, failure stress increased from 7 psi to 15 psi, to 25 psi for specimens located in the 25th, 13th, and 5th course from the bottom of the wall. Results from specimens made with cement-based mortar were erratic and in most cases energy released at specimen failure caused damage to the adjacent regions of the wall specimen.

CONCLUSIONS

General

The results and conclusions herein must be tempered by the realization that they are based upon NDE and destructive tests of one kind of masonry, i.e., dry, two-wythe, solid clay-unit masonry in a laboratory environment. The Investigators have little doubt, however, that NDE methods can be currently useful for strength and quality assessment of field-built masonry and could be increasingly so with continued development. For the present, use should be to establish <u>relative</u>

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[1]The term "damped" refers to natural internal damping of the masonry wall specimens.

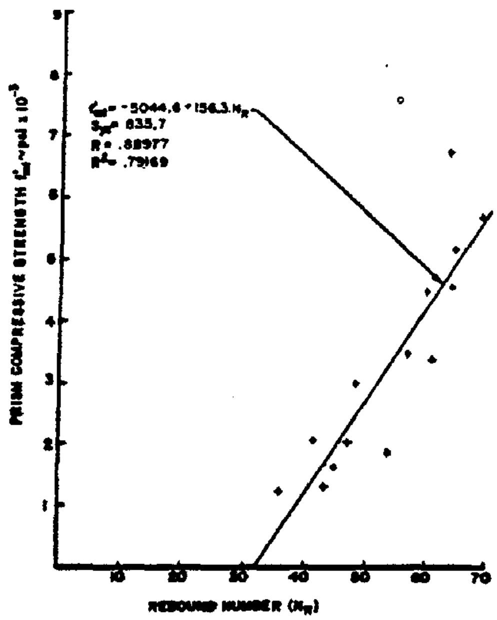

FIGURE 7 f'_{m} vs. REBOUND NUMBER

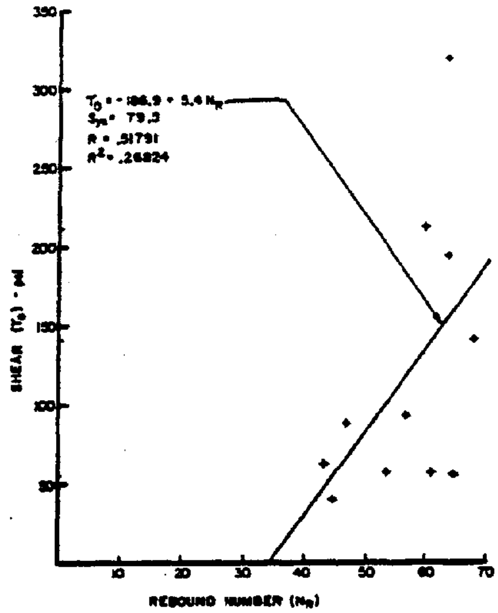

FIGURE 8 τ_0 vs. REBOUND NUMBER

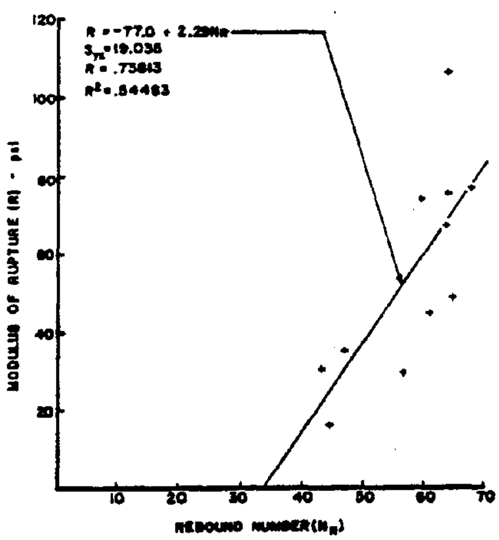

FIGURE 9 R vs. REBOUND NUMBER

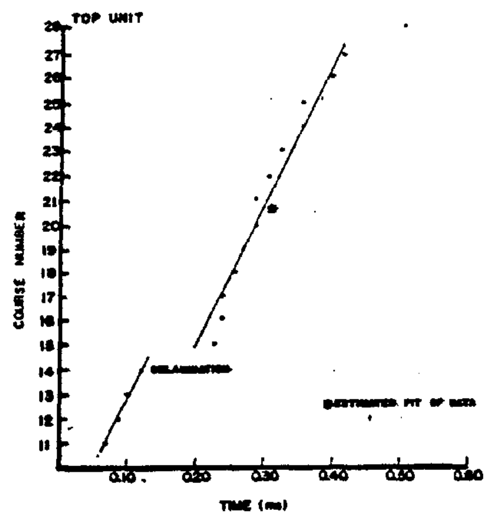

FIGURE 10
DISTANCE vs. MECHANICAL PULSE ARRIVAL TIME FOR FLAWED WALL SPECIMEN VI A

TABLE 3

RESULTS OF DESTRUCTIVE TESTS ON SMALL SPECIMENS
VS. NDE MEASUREMENTS ON UNFLAWED WALL SPECIMENS

Dependent Variable y'	Independent Variable[1] x	Correlation Coefficient R	Coefficient of Determination R^2
f'_{mt}	NR	.890	.792
	WD	.768	.590
	HUV	.772	.596
	VUV	.849	.721
	UV	.776	.603
	VMV	.703	.495
	HMV	.734	.539
	BP	.620	.390
	MP	.530	.280
R	HUV	.690	.476
	VUV	.675	.455
	UV	.672	.452
	WD	.644	.415
	NR	.738	.545
	VMV	.447	.200
	HMV	.910	.828
T_0	NR	.518	.268
	WD	.177	.031
	HUV	.257	.066
	VUV	.392	.154
	UV	.220	.049
	VMV	.340	.115
	HMV	.326	.106
CS	NR	.607	.368
	WD	.355	.126
	HUV	.476	.227
	VUV	.567	.322
	UV	.481	.231
	VMV	.566	.321
	HMV	.761	.578

TABLE 4
COMPARISON OF DAMPED NATURAL FREQUENCY
FOR UNFLAWED WALL SPECIMENS VERSUS THAT FOR
FLAWED WALL SPECIMENS

Wall	Unit	Mortar	WD Unflawed	WD Flawed	% Reduction
IID	A.R.	1:¼:3	17.6	8.5	53
VA	I.S.	1:¼:3	20.7	17.2	17
VB	I.S.	1:¼:3	21.5	12.3	43
VIA	I.S.	1:1:6	19.8	16.9	15
VIB	I.S.	1:1:6	21.9	16.8	23
XI	H.P.	0:1:3	10.4	9.3	11
XII	H.P.	1:1:6	12.4	10.8	13

[1]See Appendix II.--Notation.

quality of masonry with some destructive tests to provide a strength reference.

A general characteristic of all the NDE methods considered is that many data points are needed to produce a good statistical level of confidence in the results. This implies a more complete examination of the structure; it is one of the major advantages of NDE methods that large amounts of data can be readily accumulated without serious aesthetic effects nor structural damage. The Investigators believe that, based upon the variability of results from destructive specimens especially constructed and from specimens removed from completed wall specimens, that structural evaluation by destructive methods also requires a large number of data points. Because destructive specimens are removed from the structure, this could quickly become counterproductive.

The consistency and variability of results were generally better from tests on specimens made with modern, extruded clay units and cement-based mortar than were the results from the reclaimed Hard Pressed units. This suggests that NDE may be more successful if applied to more recently built masonry than to older construction of molded units and sand-lime mortar.

None of the NDE methods considered produced measurements well correlated to joint shear T_o. Only in one case was shear strength of eight-inch cores even moderately well correlated to a type of NDE measurement. Considering the poor correlation between T_o and prism compressive strength observed (38), one could tentatively conclude that joint shear strength may not be amenable to indirect measurement.

Conclusions regarding each NDE method and the correlation of NDE methods to strength properties follow. An overall relative evaluation of the NDE methods is also presented.

Schmidt Rebound Hammer

The rebound hammer method appears to offer the best means for immediate application of NDE to masonry. Because the rebound numbers are apparently determined by combined unit and mortar properties, both material and geometrical, determining relative quality of masonry should not be attempted between different kinds of masonry at this time.

The usefulness of this method on masonry made with soft units may be somewhat affected by tendency of the hammer to excessively pit the surface. This defaces the masonry and seems to affect the consistency of results.

The rebound hammer did not seem to be sensitive to delaminated bed joint flaws. No tests were made to determine whether or not collar joint flaws could be detected.

Mechanical Pulse Velocity

The correlations obtained between mechanically induced pulse velocities and compressive and flexural strength properties were similar to those obtained from the ultrasonic pulse velocity method. However, because of experimental difficulties encountered (38), the Investigators could not advise use of this method without extensive prior experience.

This method seems amenable to detecting the delaminated bed joint type of flaws. Detection of collar joint flaws was not attempted, but should be possible. Some difficulties in obtaining a meaningful oscilloscope trace may be anticipated due to the relatively short through-the-wall path length.

The Investigators believe that this method is potentially viable and could offer a means of evaluating large amounts of masonry, i.e., large path distances between the point of impact and the sensor, with equipment improvements or changes.

Ultrasonic Pulse Velocity

The method was reasonably successful in that correlations between ultrasonic pulse velocities and compressive and flexural strength were moderately good. The most detracting factor was the short "testing range", i.e., path distance between transducers, over which measurements could be taken particularly for masonry built with lower cement content mortar, e.g., 1:2:9. This limitation would be aggravated by the practical necessity of making many measurements by an indirect path (see Figure 6).

The transducer crystal operated at its natural frequency of 50 kHZ which is associated with long testing ranges in concrete (11) but evidently not in masonry. Other devices operate at a lower frequency and are capable of long test ranges in concrete (11) and may be more suitable for masonry evaluation.

The ultrasonic pulse velocity method has been demonstrated to be effective in detecting collar joint flaws (37). Its capability to detect delaminated bed joints was confirmed by a limited number of observations to be the same as that of the mechanical pulse velocity method.

That smooth surfaces of about three inches diameter are required on which to press the transducers could be a detracting aspect of this method. Firstly, the smooth surfaces must be ground on many masonry surfaces and is a time-consuming operation. Secondly, depending on the particular case, the smooth areas could be aesthetically degrading.

Mechanical Vibration

The natural frequency of the cantilever wall specimens was moderately well correlated to compressive and flexural strength of the companion small-scale destructive specimens and poorly to shear strength.

The method was relatively simple to apply to cantilever walls and therefore offers a possible means of quality assessment of masonry under construction. Most masonry walls are in a cantilever condition for a period soon after completion. The contribution of the wall support to frequency response should be considered in any application; true fixity is not possible. Response of walls and relative quality should only be compared between walls with similar support conditions unless steps are taken to enable wall response only to be determined.

It is because the support conditions of a given masonry element in an existing structure could significantly affect dynamic response that the Investigators do not consider this method potentially effective for NDE of masonry at the element or local level. Dynamic methods have been used for overall assessment of complete systems (12), however.

Densicon Penetrometer

The probe penetration method yielded only moderate correlation to compressive strength and exhibited a fairly high degree of variability. The Investigators are not prepared to discount this method completely, but would not endorse further study at the expense of the development of other methods at least for application to evaluation of clay-unit masonry. The penetration method may prove to be viable for evaluation of concrete unit masonry, but its history of use applied to concrete (11,19,20) does not portend dramatic success.

Acoustic-Mechanical Pulse

Based upon admittedly limited experience, the Investigators nevertheless believe that this method can be developed into a successful NDE method for masonry. The measurements taken are a more complete description of stress wave characteristics (see Figure 1) and could well be correlated to strength properties as they apparently are for other materials (23,24,25).

RECOMMENDATIONS

For Continued Development

The Investigators believe that the results obtained are sufficient to justify continued research and development efforts to improve experimental procedures and equipment and to build a data base to enable general use of NDE methods for masonry. This is reinforced by limitations on the extent and usefulness of destructive testing as a means of evaluation due to variability of results, and physical and aesthetic damage to the structure being evaluated which restrict the number of such tests which can be done. Field experience with NDE methods is necessary to assess the influence of site factors upon results and procedure.

For Current Applications

The specific NDE measurements obtained in this study and correlations to strength properties are applicable only to the single type of masonry used. Because it was done in a laboratory, the results should be confirmed for any other application.

A mixed-mode of NDE and destructive methods is suggested for structural assessment at the present time. NDE measurements made in the same location or locations from which destructive specimens are removed can be correlated to the destructive test results and then serve as reference data for other NDE measurements in other locations. It may be desirable to use more than one NDE method to take advantage of each method's strongest capability.

The basic philosophy of the mixed-mode approach suggested is therefore to use NDE methods extensively to establish a comprehensive assessment of the relative quality of masonry coupled with as small a number of destructive tests as is necessary to adequately relate NDE measurements to strength properties.

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APPENDIX II.—NOTATION

Unit Terms

A.R. ≡ Antique rustic solid clay unit

I.S. ≡ Iron spot solid clay unit

H.P. ≡ Hard pressed solid clay unit

Statistical Terms

x ≡ The independent variable in linear regression equations.

y' ≡ The estimated value of y, the dependent variable in linear regression equations.

A ≡ x-axis intercept.

B ≡ constant.

$R \equiv$ Correlation coefficient (also modulus of rupture).

$R^2 \equiv$ Coefficient of determination.

$S_{yx} \equiv$ Standard error of estimate, i.e., the standard deviation of actual values of the dependent variable, y, from predicted values, y'.

Stress Terms

$f'_{mt} \equiv$ The failure stress of 4-unit, stack-bond prisms based on gross area.

$R \equiv$ Modulus of rupture, i.e., the flexural tensile stress at failure based on a linear-elastic behavior assumption.

$T_o \equiv$ Ultimate bed joint shear strength assuming a uniform shear distribution over the bed joint determined by the inclined-bed-joint specimen test.

Nondestructive Evaluation Measurements

BP $\equiv$ The amount of probe penetration into brick.

CS $\equiv$ The failure shear stress, assumed uniform, of an 8-inch diameter, single-wythe core specimen in diametral compression.

HMV $\equiv$ Horizontal mechanical pulse velocity, fps, obtained by a hammer blow on the side of a wall specimen on the same wythe and same course and opposite a horizontally-oriented, wall-mounted accelerometer.

HUV $\equiv$ Horizontal ultrasonic pulse velocity, fps, based on measurements of ultrasonic velocity with the transducers placed on the same course and wythe of the wall specimen.

MP $\equiv$ The amount of probe penetration into mortar.

ms $\equiv$ Time in milliseconds.

$N_R \equiv$ Rebound number as determined by the Schmidt Hammer.

UV $\equiv$ Ultrasonic pulse velocity, fps, measured through the thickness of wall specimens, using transducers placed directly opposite each other.

VMV $\equiv$ Vertical mechanical pulse velocity, fps, obtained by a hammer blow on the top of a wall specimen on the same wythe and directly above a vertically-oriented wall-mounted accelerometer.

VUV ≡ Vertical ultrasonic pulse velocity, fps, based on measurements with the transducers placed one above the other, separated by various distances, on the wall specimen surface.

WD ≡ Natural frequency of the cantilever wall specimen as damped internally.

EFFECTS ON BEHAVIORS OF REINFORCED CONCRETE FRAMES

BY ADDING SHEAR WALLS

Y. Higashi*, T. Endo** and Y. Shimizu***

Introduction

A number of buildings of reinforced concrete were strengthened by adding infilled shear walls or wing walls. Higashi, one of the authors, and Kokusho studied on these walls and reported the results in 1975[1). The authors have studied on the effects of these walls since then. The test results on one-story one-bay frames strengthened by adding those walls were published on the 7th World Conference on Earthquake Engineering in 1980.[2) Thereafter the test results on three-story one-bay frames and the comparison between three-story and one-story were lectured at the last Joint Meeting.[3)

In order to investigate the effect of partial strengthening on multi-story multi-bay frames, three-story two-bays models strengthened by adding shear walls in only one-bay were tested.[4) The test results and comparison of them, and frame analysis are reported in this paper.

Specimens

The four three-story one-bay models in '79 series and the four three-story two-bays models in '81 series are discussed. Figure 1 illustrated the eight specimens. The length of one-bay is 630 mm and the height of one-story is 385 mm, which correspond to approximate one eighth of those of the common building.

The specimens No. 1 - 3F ('79) and No. 1 - 3F2 ('81) are frames without any strengthening, corresponding to the existing frames. 4-D10 bars are provided in beams in longitudinal direction and rectangular stirrups of 4 mm bar spaced 40 mm. The columns are reinforced by 4-D6 bars in longitudinal direction and by rectangular hoops of 2 mm bar spaced 40 mm.

The specimens No. 2 - 3PW ('79) and No. 2 - 3PW2 ('81) are strengthened by post-casted shear walls, 5 cm thick, without openings. The boundary surface

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between the beam and the wall is connected by wedge anchors of 6 mm dia. spaced 40 mm. The detail of the connection is shown in Figure 2. Reinforcing bars of 4 mm spaced 100 mm in both vertical and horizontal directions are arranged in wall panel. They do not across the above surfaces.

Precast wing walls are connected to the specimen, No. 3 - 3C2A ('79), but post casted wing walls are provided to the specimen No. 3 - 3C2A2 ('81). The connection between the beams and the wing walls are shown in Figure 3.

The last pair of specimens, No. 8 - 3FW ('79) and No. 4 - 3FW2 ('81) have monolithic walls, designed same as No. 2 - 3PW and No. 2 - 3PW2 respectively, except their details of boundary surfaces between frames and walls. The reinforcing bars of wall are anchored into the surrounding frame.

The strength of the cement mortar for the specimens are shown in Table 2. Expansive material was mixed to the mortar for the post casted walls and for filling between precast walls and frames. The yield and maximum strength of the steels are shown in Table 2 (b).

Testing

The all specimens are applied the constant axial force at the top of each column, corresponding to σ_0 = 30 kg/cm^2 (2.94 MPa) compressive stress. Cyclic horizontal loading, uniformly distributed to each story, was controlled manually so as to follow the same program about the bottom story deformation angle; the relative story displacement of the bottom story divided by the story height, shown below.

Sequent of the bottom story deformation angle:

R = 1/500	1 cycle
R = 1/200	4 cycles
R = 1/100	4 cycles
R = 1/50	4 cycles
Positive large deformation	

The "positive load" is defined as the wall is pulled outside. The loading arrangements of two series of tests are shown in Figure 2.

Relative horizontal and vertical displacements between gage holder, fixed at the base of the specimen, and the intersection points of the center lines of the beam and the column are measured by electric transducers. Strains of reinforcing bars and wall panells are also measured.

Test Results

Shear force - bottom story displacement curves are shown from Figure 4-1 to Figure 4-4. The final failure patterns are illustrated in Figure 5. The critical loads and their bottom story displacements given by the tests are listed in Table 3.

General Behaviors

No. 1 - 3F and No. 1 - 3F2:

Although the bottom columns of these frames failed in shear finally, the shear force - bottom story displacement curves have long flat part after yielding in bending. The ratio of the maximum shear force at bottom story in No. 1 - 3F2, to that of No. 1 - 3F is 1.7.

No. 2 - 3PW and No. 2 - 3PW2:

The specimen No. 2 - 3PW, one-bay, yielded in bending as a canti-lever. Thereafter the curve was almost flat up to 10 mm displacement (angle 1/35) and then load suddenly dropped by shear failure at the bottom story. On the other hand, the curve of No. 2 - 3PW2, two-bays, had only short flat part and dropped after 7 mm displacement by shear failure of wall at the bottom story. However, more than 60 percent of the maximum load was kept at the 27 mm displacement (angle 1/13). Then the wall panel in the latter specimen finally splitted along the anchor bolts and the independent column failed in shear.

The ratio of the maximum shear force of No. 2 - 3PW2 to No. 2 - 3PW is about 1.5. The maximum positive shear force of No. 2 - 3PW2 was a little different from negative one, and their ratio is about 1.15.

No. 3 - 3C2A and No. 3 - 3C2A2:

The crash of mortar in No. 3 - 3C2A, using precast wall panels, occured after large displacement.

Diagonal cracks occurred in wall panels of No. 3 - 3C2A2. The horizontal load capacity decreased gradually after that. The ratio of No. 3 - 3C2A2 to No. 3 - 3C2A is about 2.2.

No. 8 - 3FW and No. 4 - 3FW2:

The curves of the both specimens were quite similar. The maximum shear force of No. 4 - 3FW2 is 1.65 times that of No. 8 - 3FW. Wall panels of the both specimens failed in shear at the same displacement, 6 mm. The positive load of No. 4 - 3FW2 was quite different from negative one.

Deflection Mode

Measured horizontal and vertical displacements at the intersection points under major critical loads were plotted in Figure 6 - 1 and Figure 6 - 2. Pure frame specimens, No. 1 - 3F and No. 1 - 3F2, moved in horizontal direction only. Story displacement of each story was roughly proportional to each story shear force.

One-bay shear wall models, No. 2 - 3PW and No. 8 - 3FW, both post casted wall and monolithic wall, deformed as canti-lever beams. It can be obviously observed from Figure 6 - 1 that their deflection modes were almost same and that their vertical displacements were not so small compared with horizontal displacements.

On the other hand, shear walls in two-bays specimen; No. 2 - 3PW2 and No. 4 - 3FW2, had different deflection modes. Namely less vertical displacements were found than one-bay shear wall models, that means more percentage of shear deformation to the total deformation. In addition to that, inclinations of boundary beams; beams connected to walls, were found in two-bays specimen only under positive loads. Under negative loads, however, almost no inclination were found. As a result, it can be estimated that different plastic works under positive and negative loads at the ends of boundary beams give the different ultimate loads.

The beams of the specimen, No. 3 - 3C2A and No. 3 - 3C2A2, moved in parallel, although a little vertical displacement were found.

Envelope Curves

Four envelope curves of 79 series tests were drawn in Figure 7 (a) and four of 81 series tests were in Figure 7 (b). In Figure 7 (a), two curves; No. 2 - 3PW with post casted walls and No. 8 - 3FW with monolithic walls, were clearly coincident. That is one example that behavior of post casted wall is almost same as that of monolithic wall. In Figure 7 (b), however, the envelope curve of the specimen No. 4 - 3FW2, with monolithic wall, was quite different from that of No. 2 - 3PW2. Monolithic wall in 81 series had 9 percent greater strength than post casted wall in the same series, while the former was more brittle.

According to these figures, wing wall specimens, No. 3 - 3C2A and No. 3 - 3C2A2, were more ductile than shear wall specimens.

Their ductilities seemed to be enough for the common building.

Deflection of each story

About No. 2 - 3PW2 ('82), shear force - deflection curve of each story is shown in Figure 8 and story shear force - story deflection curve is shown in Figure 9. Proportion of each story displacement were almost constant up to 5 mm bottom story deflection (angle 1/77). After that point only bottom story deflection progressed.

Analysis

Inelastic Frame Analysis

The behavior of the all specimens are analyzed by using inelastic frame models. Every beam or column in the specimen is treated as a line member considering axial deformation and flexure, while every wall is treated as a diagonal brace considering only axial deformation. The line members assumed in the all specimen are shown in Figure 11.

Direct stiffness matrix analysis is used for solving the displacement corresponding to the increment of the horizontal load.

Moment and rotation relation at the end of the column or the beam is shown in Figure 12 (a). Yield bending moment in the figure is determined by the interaction curve illustrated in Figure 12 (b). Nonlinear part of the interaction curve was computed by the following equation, proposed by AIJ [5].

$$My = 0.8\ a_t\ \sigma_y\ D + 0.5\ N \cdot D\ (1 - N/b \cdot D \cdot F_c)$$

a_t : area of tension reinforcement

σ_y : yield strength of reinforcement

N : axial force

D : overall thickness of member

F_c : compressive strength of concrete

Stress - strain relation of the brace was assumed as illustrated in Figure 12 (c), reffering to the shear deformation characteristics of the corresponding wall based on beam theory.

When the shear force of the beam or the column reached the critical shear force computed by the following equation, two yield hinges are formed at the ends of the member.

Shear strength proposed by Arakawa and modified by Hirosawa[6]:

$$Q_{su} = \left\{ \frac{0.092\ k_u \cdot k_p\ (180 + F_c)}{M/Q \cdot d + 0.12} + 2.7\sqrt{P_w \cdot {}_w\sigma_y} + 0.1\ \sigma_o \right\} b \cdot j$$

in which

k_u : modification factor from magnitude of depth

k_p : $0.82\ p_t^{0.23}$

$M/Q \cdot d$: shear span ratio

P_w : shear reinforcement ratio

${}_w\sigma_y$: yield strength of shear reinforcement

σ_o : axial force divided by area of the member

The computed shear force to bottom story displacement curves are drawn from Figure 4-1 to Figure 4-4 by broken line. In general, the computed lines fitted the envelop curves given by the experiment.

Limit analysis

The differences between positive maximum load and the negative maximum load were not so small on No. 2 - 3FW2 and No. 4 - 3PW2 in '81 series. In order to know the reason, computation of ultimate load is tried asuming collapse mechanism shown in Figure 13. The computed positive ultimate load was about 57 percent (No. 2 - 3PW2) or 51 percent (No. 4 - 3FW2), while that of experimental value was 15 percent or 25 percent respectively.

Concluding remarks

Based on the experimental and analytical results, several conclusions may be deduced as follows:

1) The effect of the boundary frame is large. According to test results on post casted walls, the difference between the maximum shear force of the pure three-story two-bays frame and that of strengthened frame by one-bay post casted wall was 9.4 ton. In case of one-bay frame, the difference was 6.6 ton which is about two third of the above.

2) The behavior of the post casted wall using wedge anchors on the boundary surface between the frame and the wall was quite similar to that of the monolithic wall in case of one-bay frame, but was a little

different in case of two-bay frame strengthened by one-bay wall.

Reliability of the post casted wall changes according to shear span ratio of the wall.

3) In general, the frame strengthened by wing walls can be expected ductility.

4) The results of the inelastic frame analysis adopted in this paper relatively agreed with that of experiment on both individual wall and wall with boundary frame. This analysis can be easily used for the common building.

5) The difference between the positive maximum load and the negative maximum load was not so small. It was a little smaller than the difference between computed values by means of limit analysis.

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Table 1. Specimen

79 series	81 series	
No.1-3F	No.1-3F2	Pure frame
No.2-3PW	No.2-3PW2	Post casted shear wall
No.3-3C2A	No.3-3C2A2	Adding side walls
No.8-3FW	No.4-3FW2	Monolithic wall with frame

Table 2. Physical Properties of Materials

(a) Mortar

Mortar	$_c\sigma_B$	kg/cm^2
frame	79 series	144
	81 series	202
filled in	79 series	246
	81 series	205

(b) Steel

Steel	79 series				81 series			
	2φ	4φ	D6	D10	2φ	4φ	D6	D10
a_t cm^2	0.0315	0.124	0.32	0.71	0.030	0.120	0.32	0.71
$_s\sigma_y$ kg/cm^2	2520	4180	3700	3850	2840	3890	3580	3680
$_s\sigma_m$ kg/cm^2	3370	4850	5550	5630	3760	5140	5330	5730

$_c\sigma_B$; mortar compressive strength

a_t ; area of steel

$_s\sigma_y$; yielding strength of steel

$_s\sigma_m$; maximum strength of steel

Table 3. Test Results

Specimen		81-No.1		79	81-No.2		79	81-No.3		79	81-No.4		79
		P	N	No.1	P	N	No.2	P	N	No.3	P	N	No.8
Initial Stiffness	Ke (ton/cm)	37		6	250		150	63		13	273		155
Yield	load Py (ton)	2.6		1.1	12.8		7.6	5.2		1.5	13.6		7.6
	deflection δy (mm)	3.5		2.2	3.5		2.3	4.0		2.1	3.5		2.9
Maximum	load Pm (ton)	3.4	3.4	2.0	12.8	11.1	8.6	6.7	6.4	3.0	14.0	11.2	8.6
	deflection δm (mm)	15.1	7.0	9.4	5.3	6.8	7.0	11.1	7.0	16.8	5.3	3.5	7.0
Ultimate	load Pu (ton)	-			12.0		8.3	6.4		-	13.7		8.3
	deflection δu (mm)	-			7.0		9.8	12.6		-	6.9		9.0

N.B. P ; positive loading

N ; negative loading

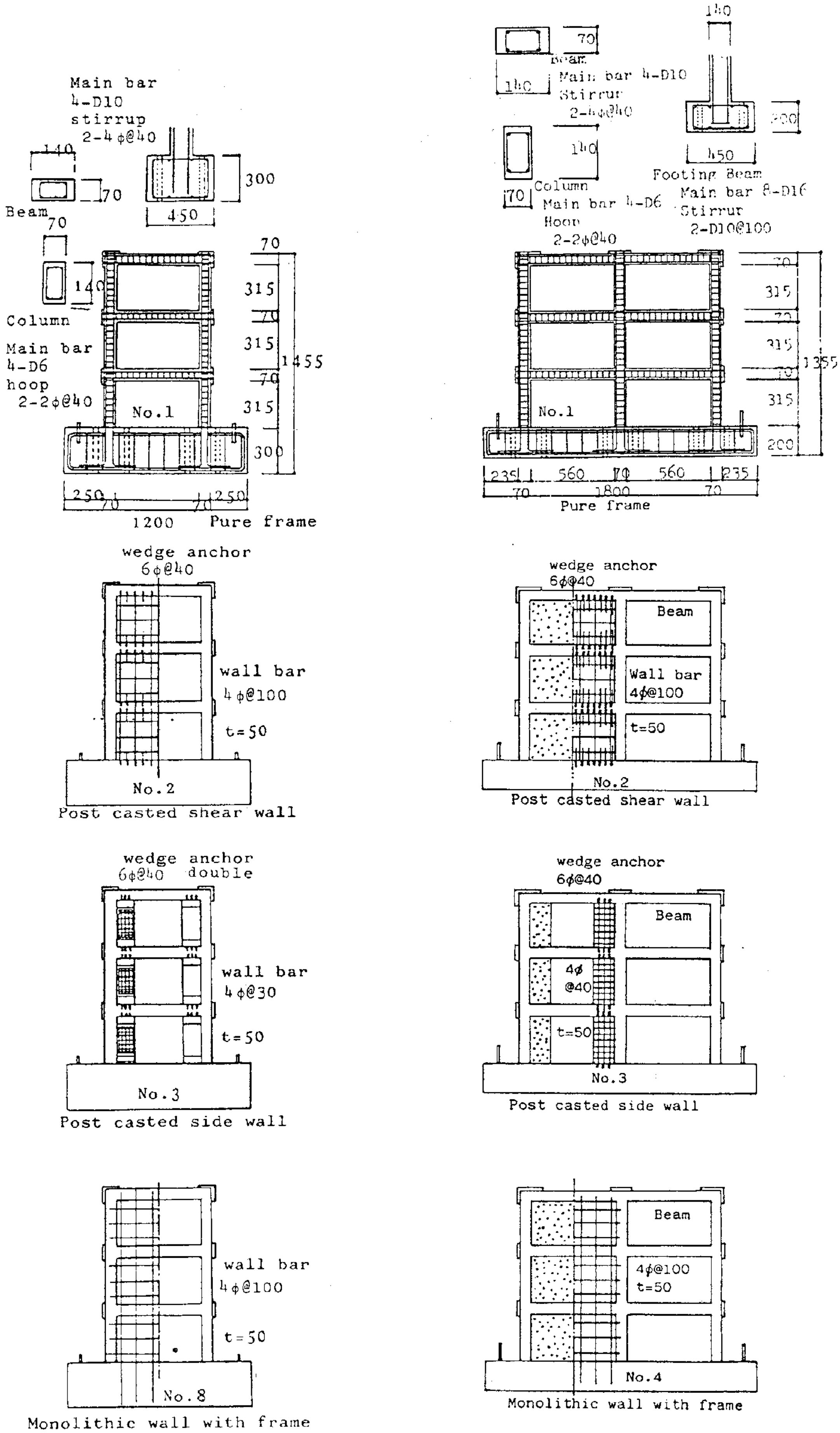
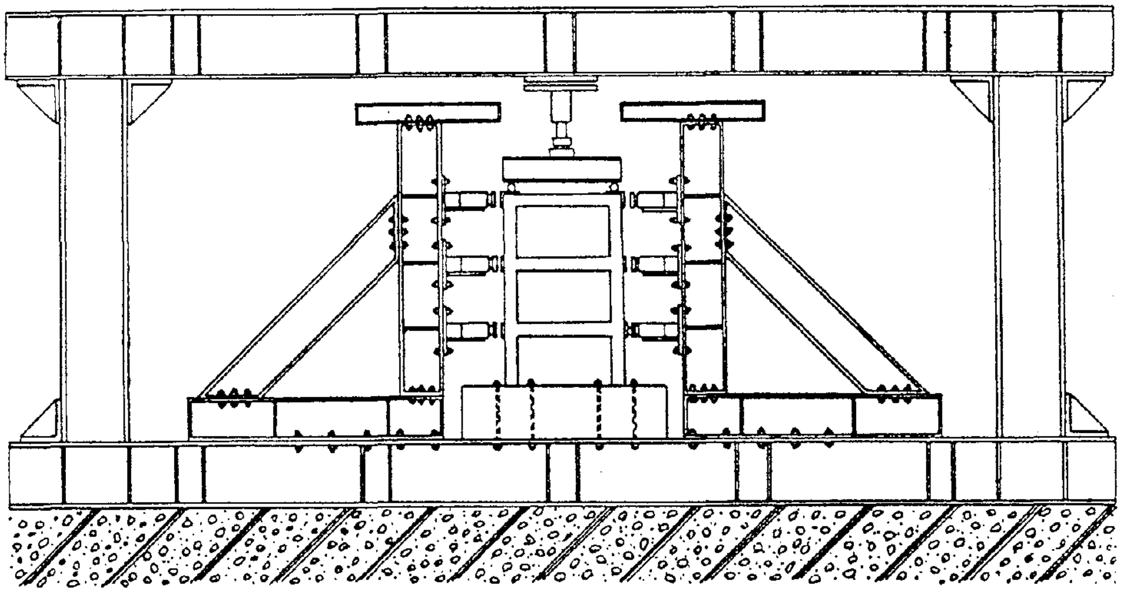
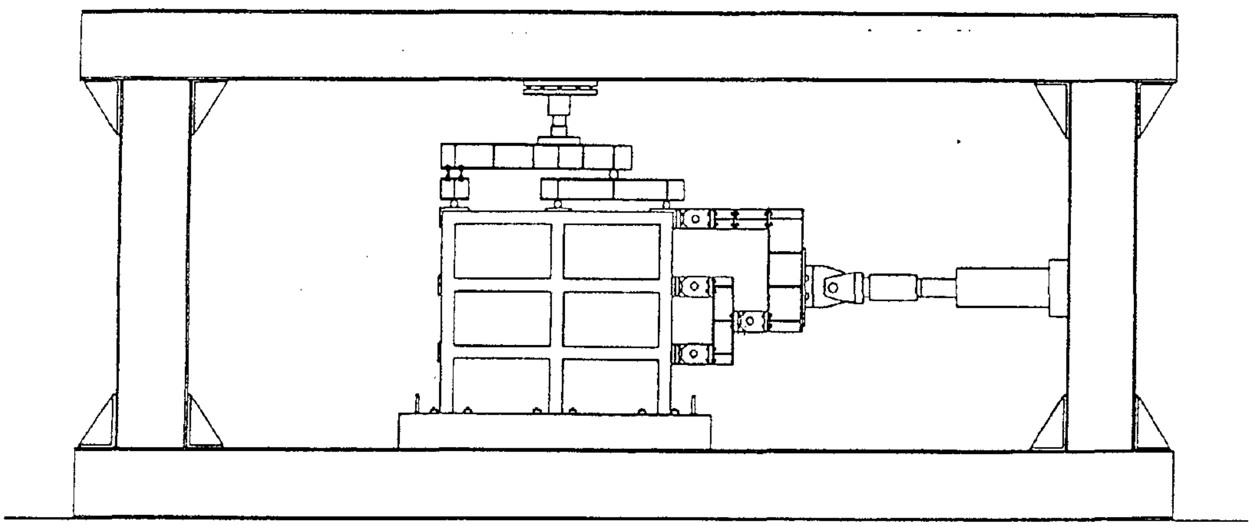


Fig. 1 Specimens



(a) 79 series



(b) 81 series

Fig.2 Loading Arrangement

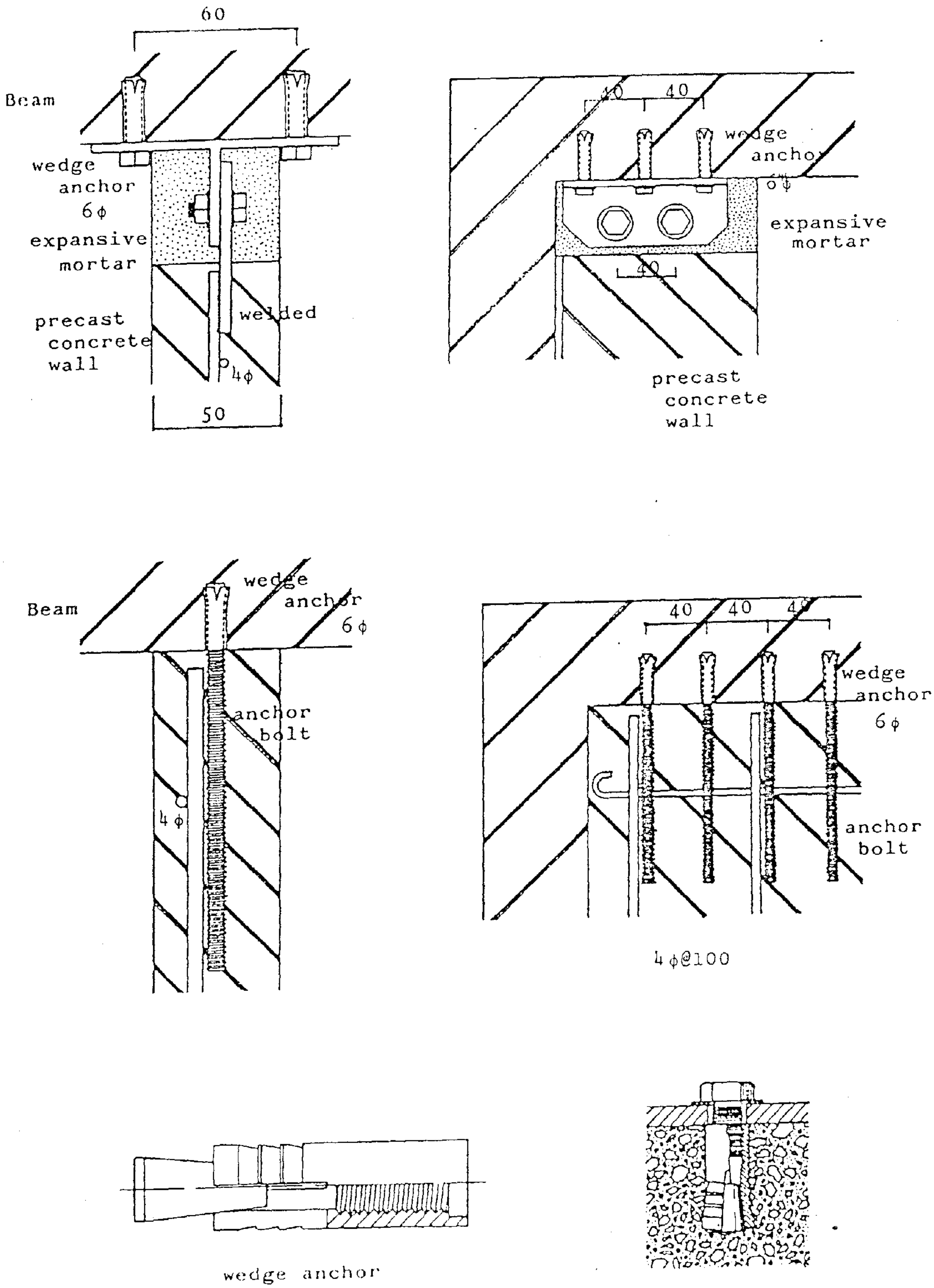
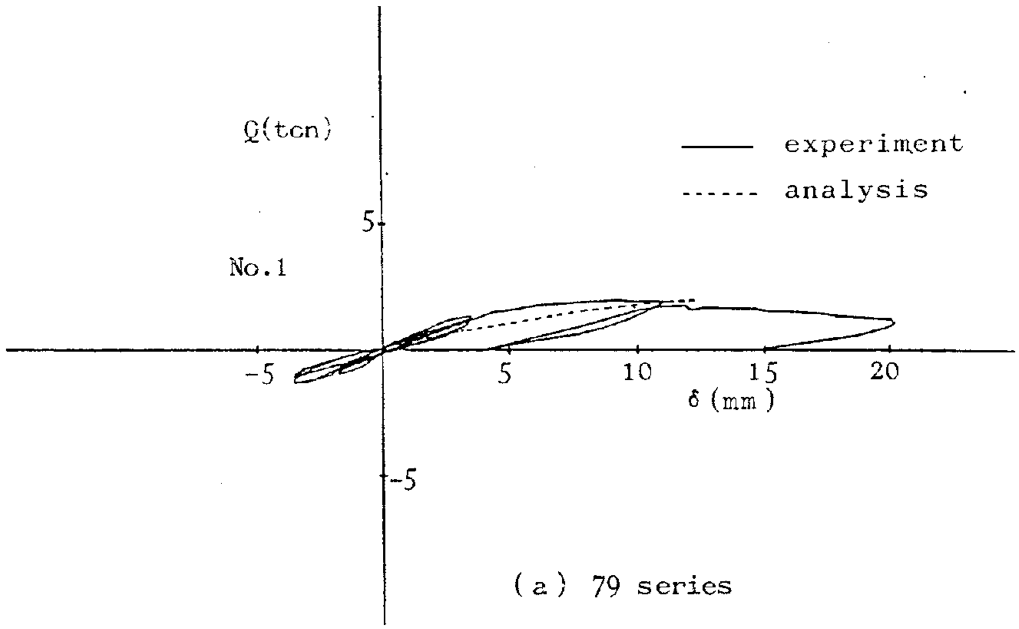


Fig. 3 Contact Point between Post Casted Shear Wall and Frame



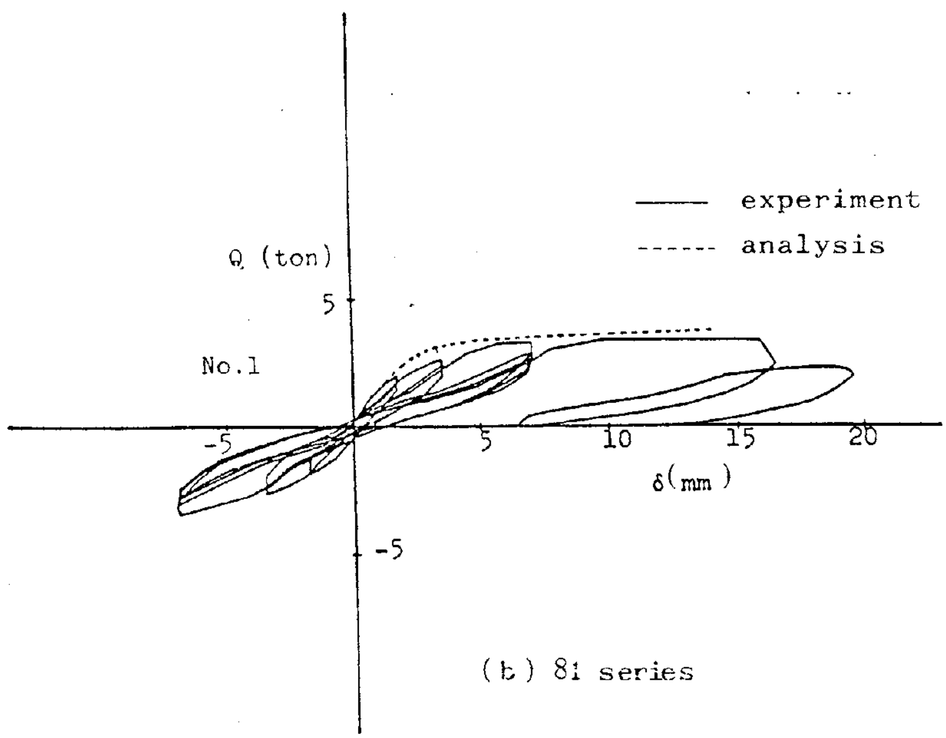
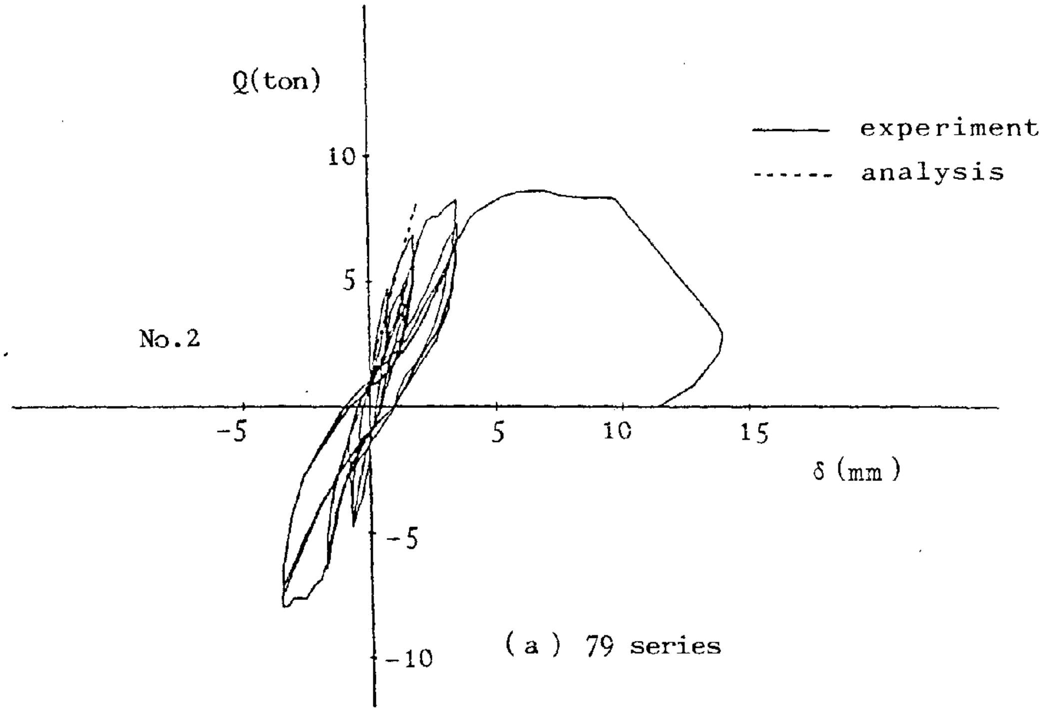
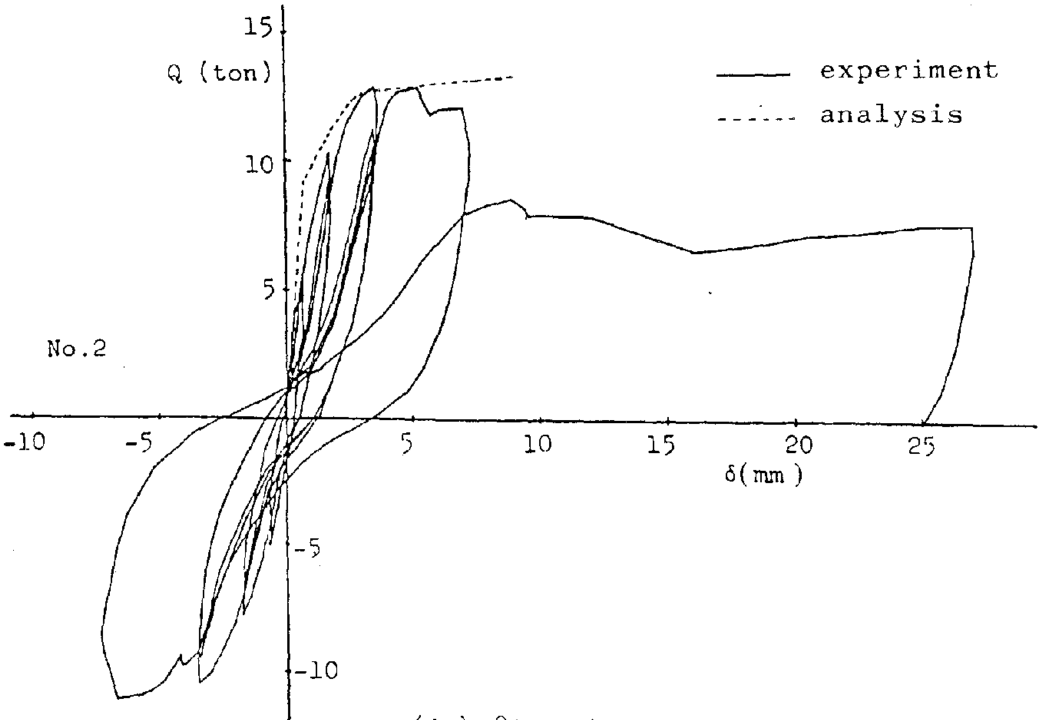


Fig. 4-1 Load - Deflection Curve

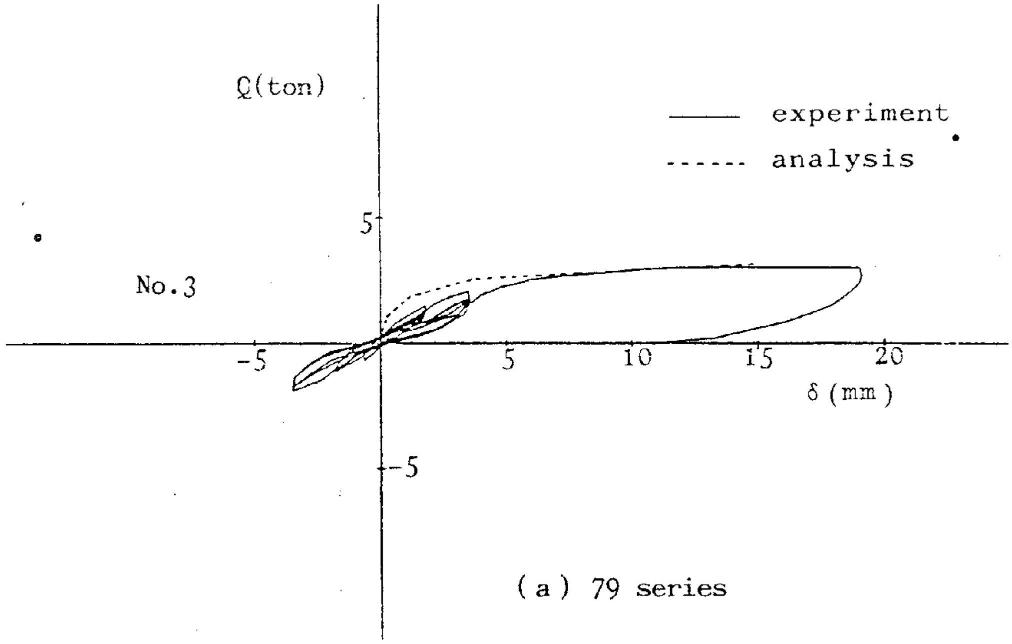


(a) 79 series



(b) 81 series

Fig. 4-2 Load - Deflection Curve



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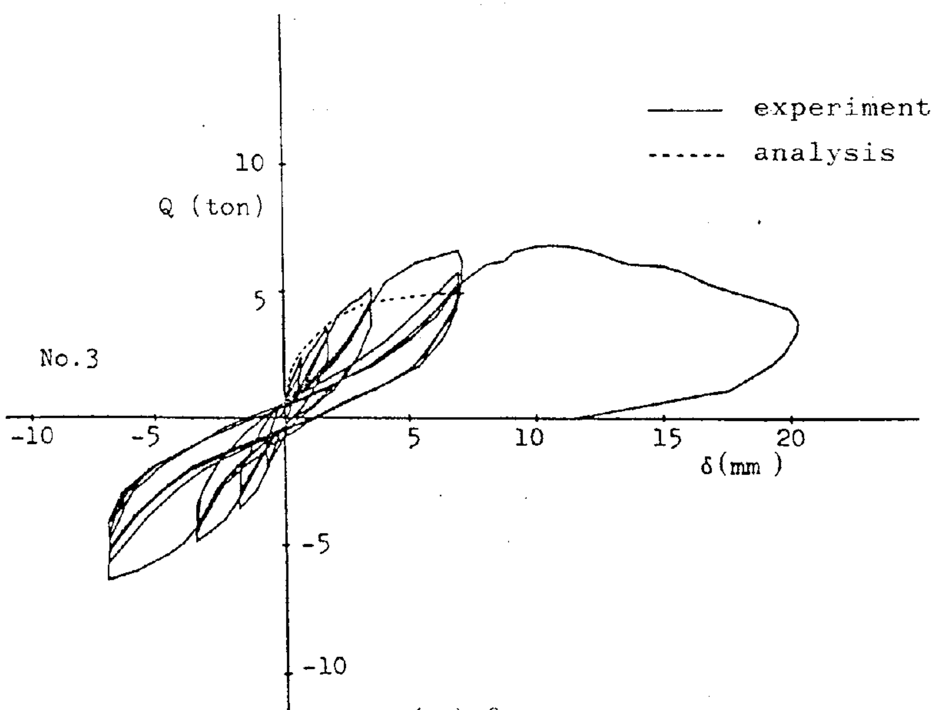
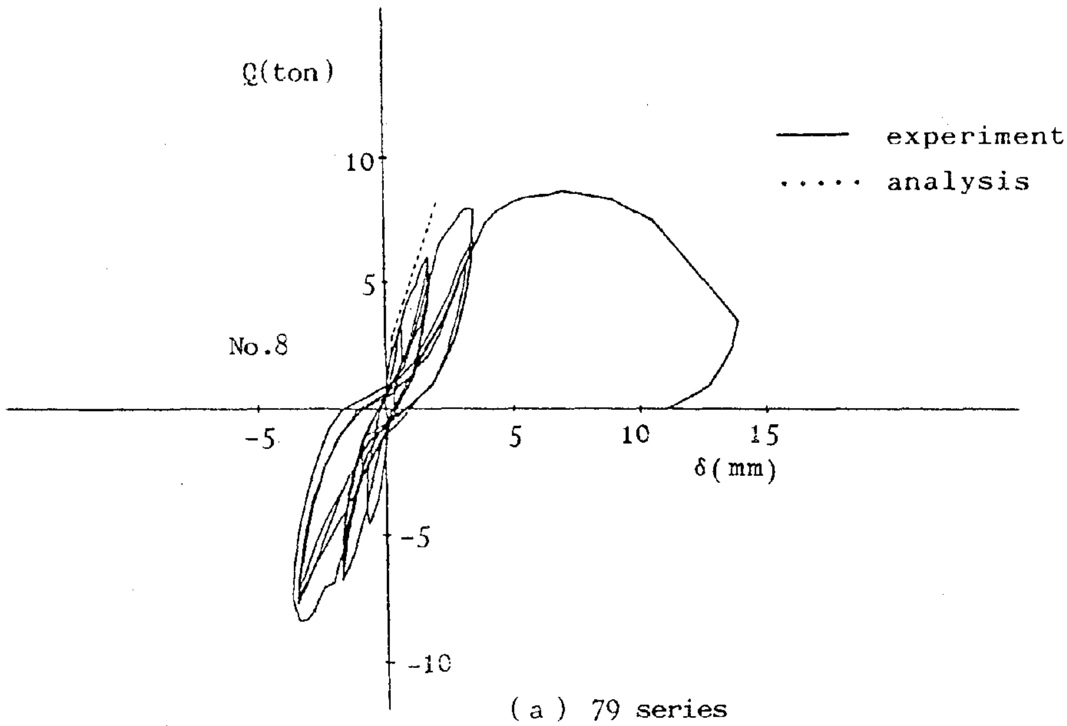
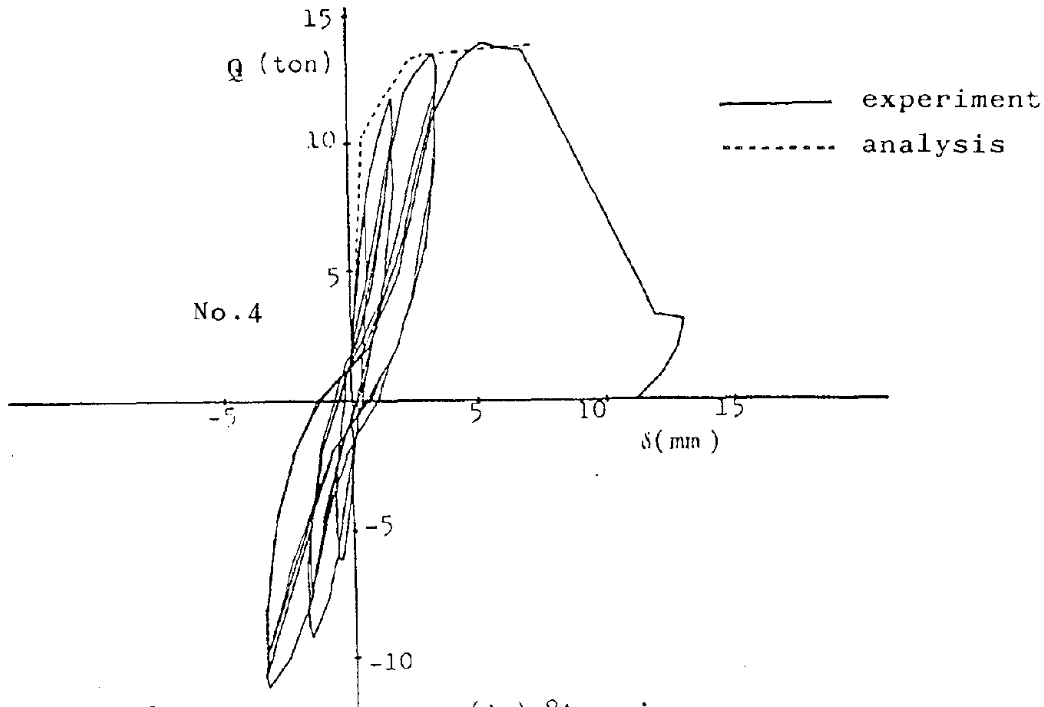


(b) 81 series

Fig. 4-3 Load - Deflection Curve



(a) 79 series



(b) 81 series

Fig. 4 - 4 Load - Deflection Curve

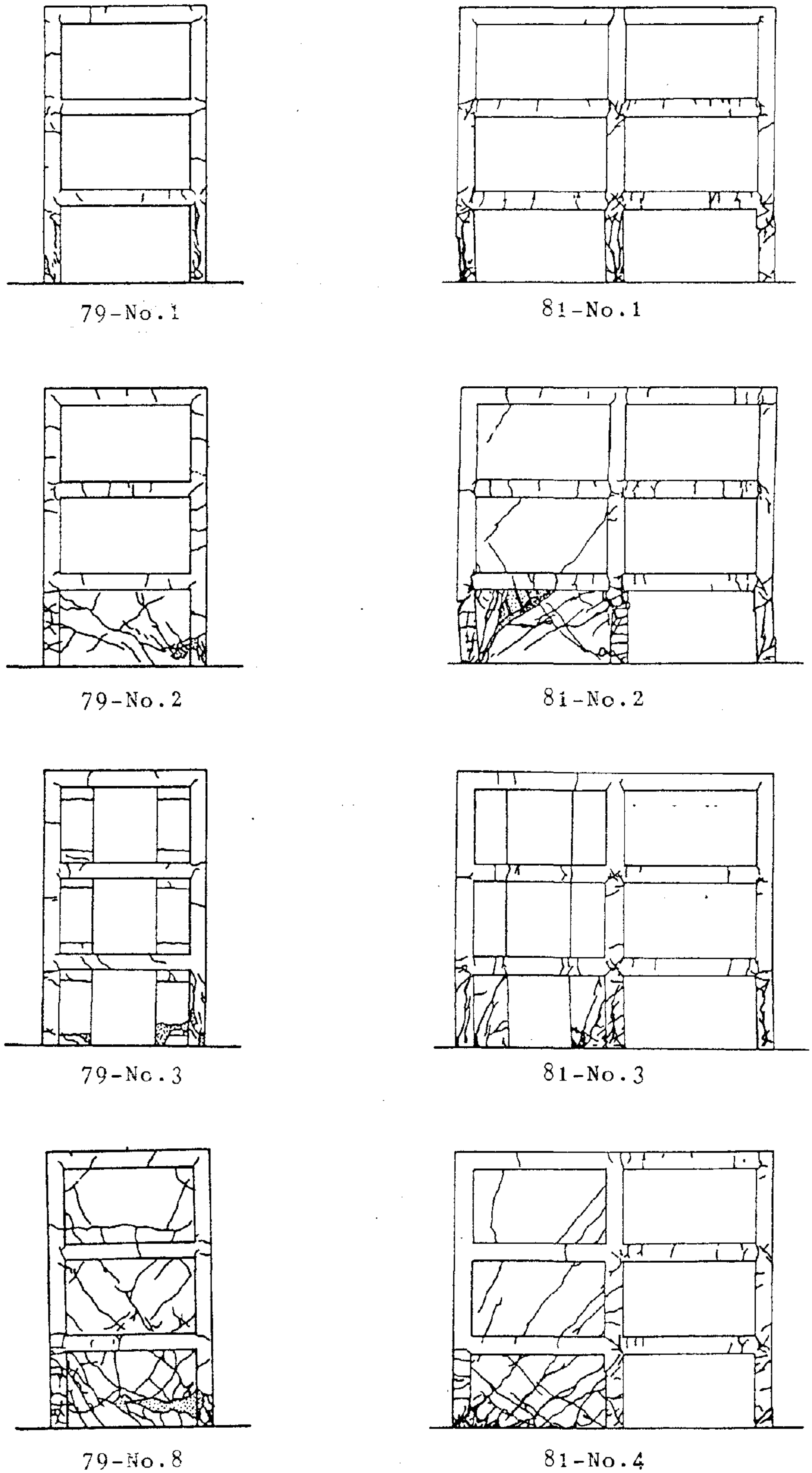
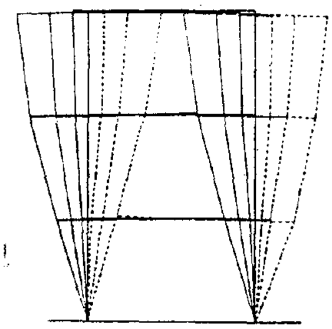


Fig. 5 Final Cracking Pattern



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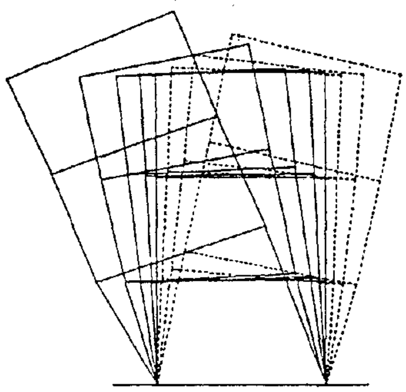

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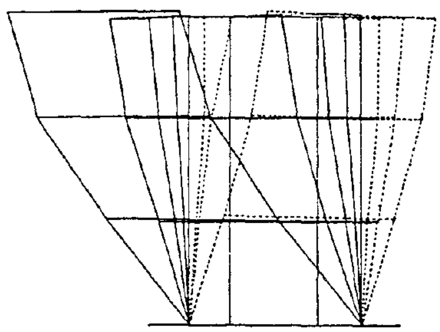

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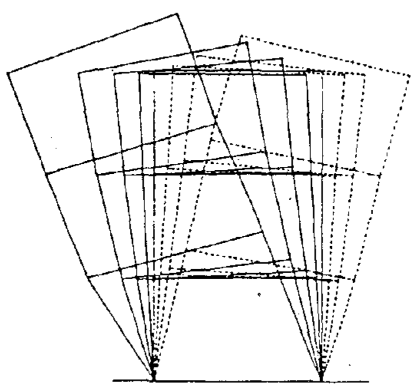
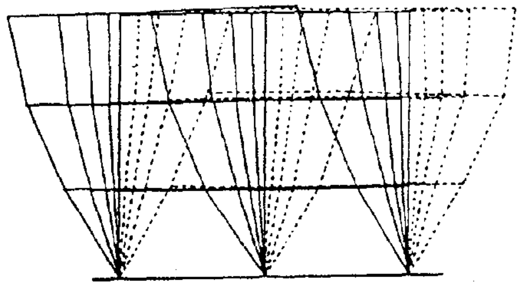
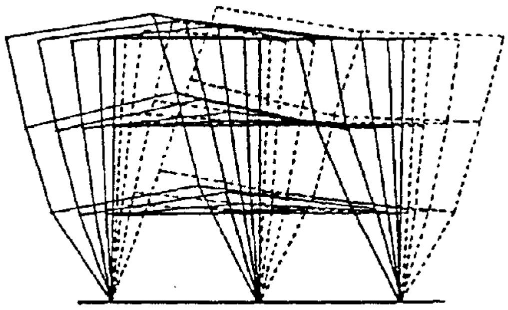


No.8

Fig. 6-1 Deflection Mode (79 series)



No.1



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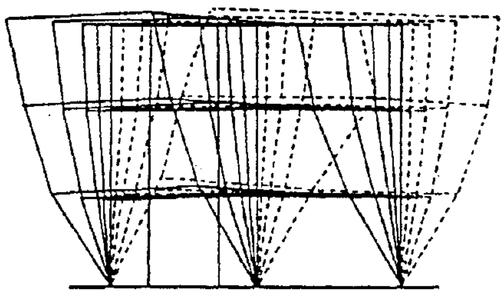

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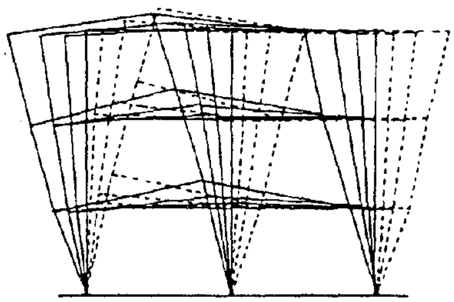
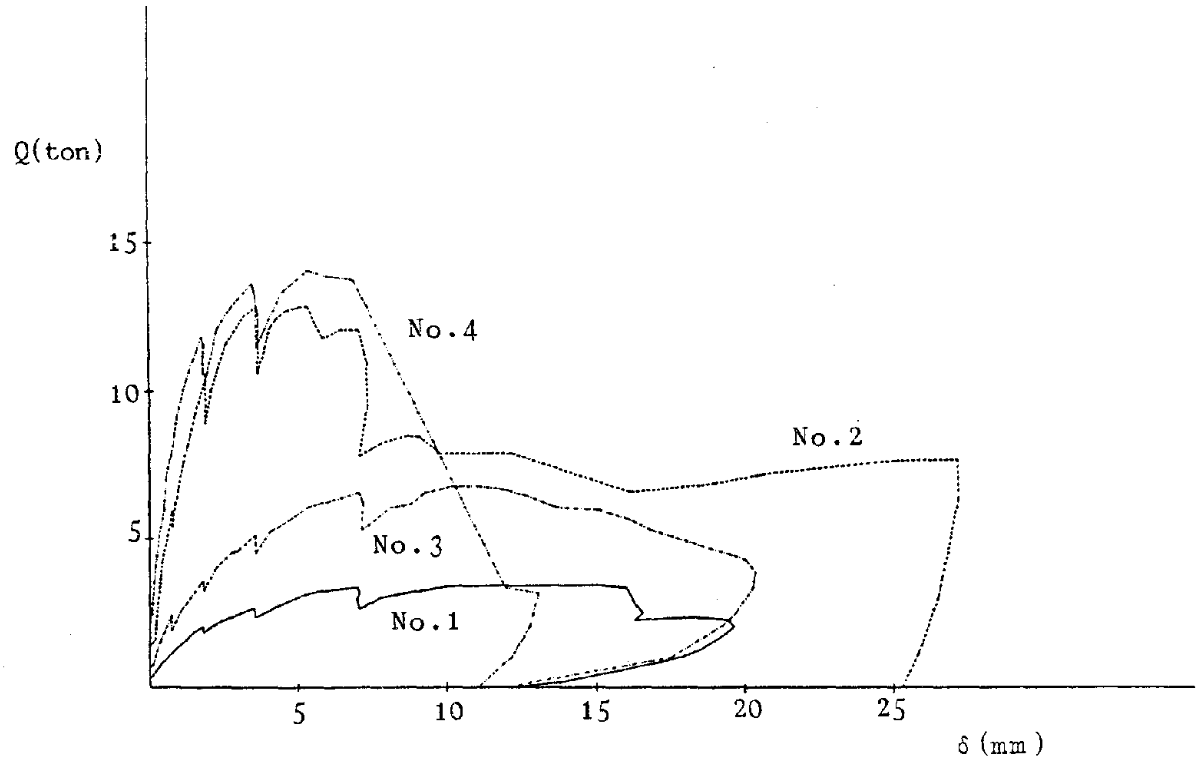
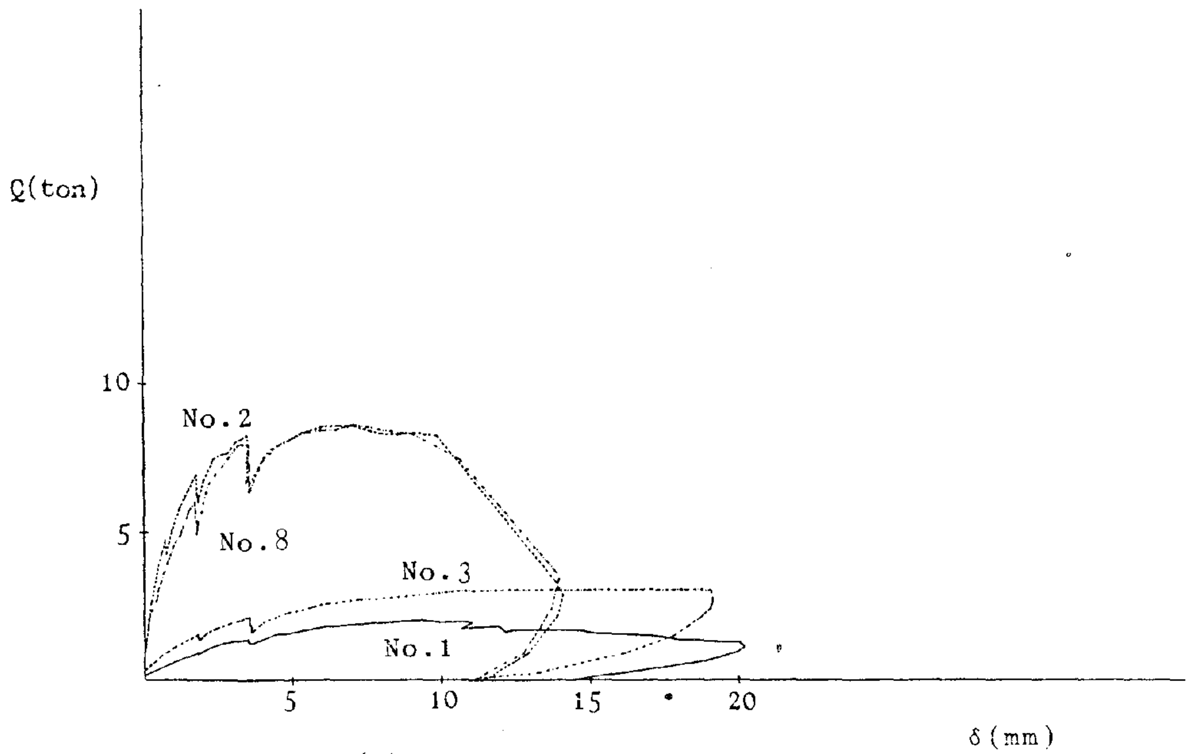


No.4

Fig. 6-2 Deflection Mode (81 series)



(b) 81 series



(a) 79 series

Fig. 7 Envelopes of Hysteresis Curves

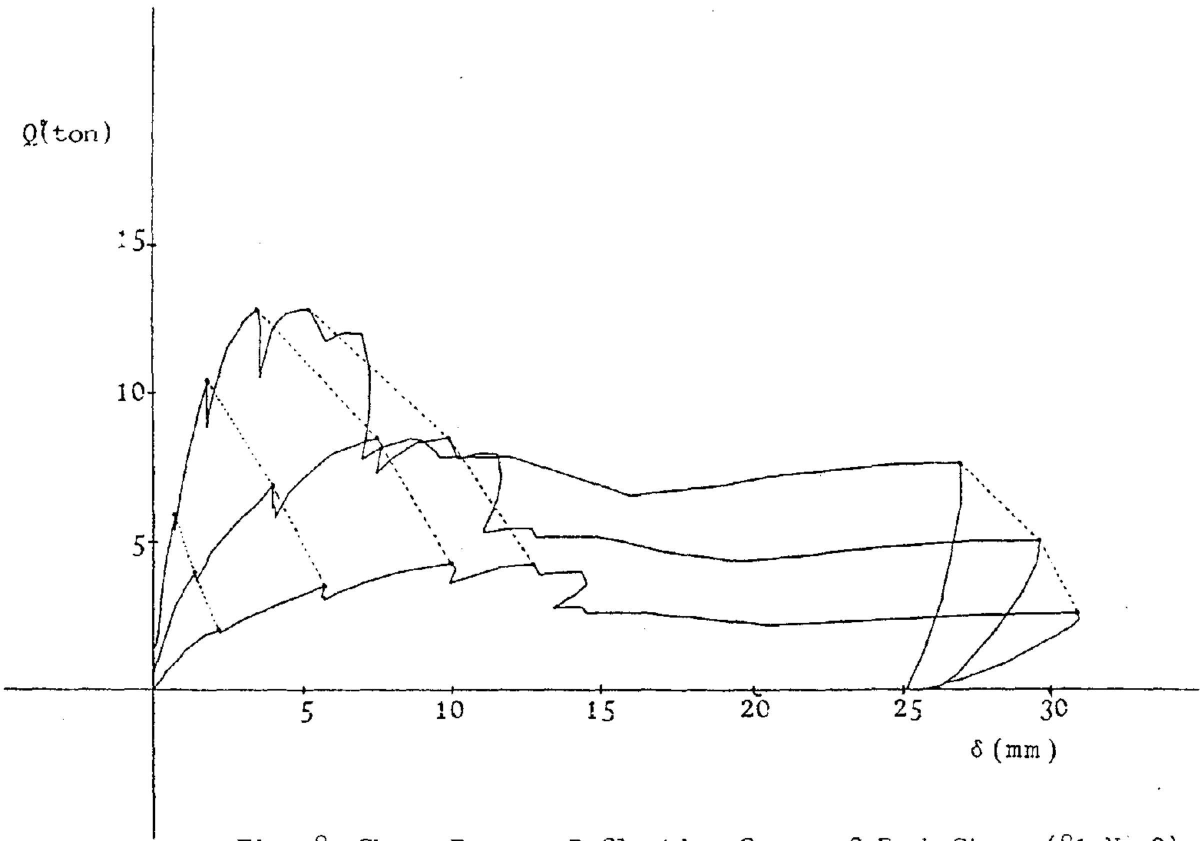


Fig. 8 Shear Force - Deflection Curve of Each Story (81-No.2)

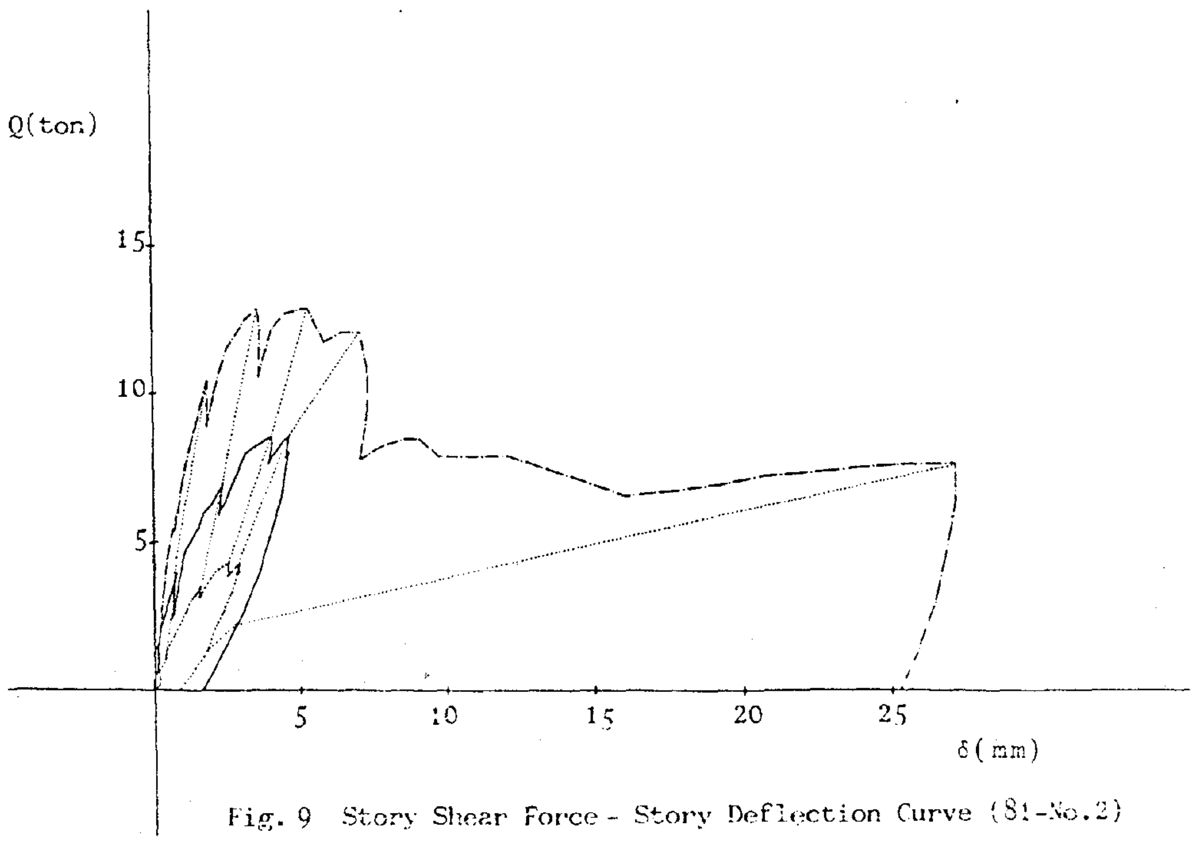
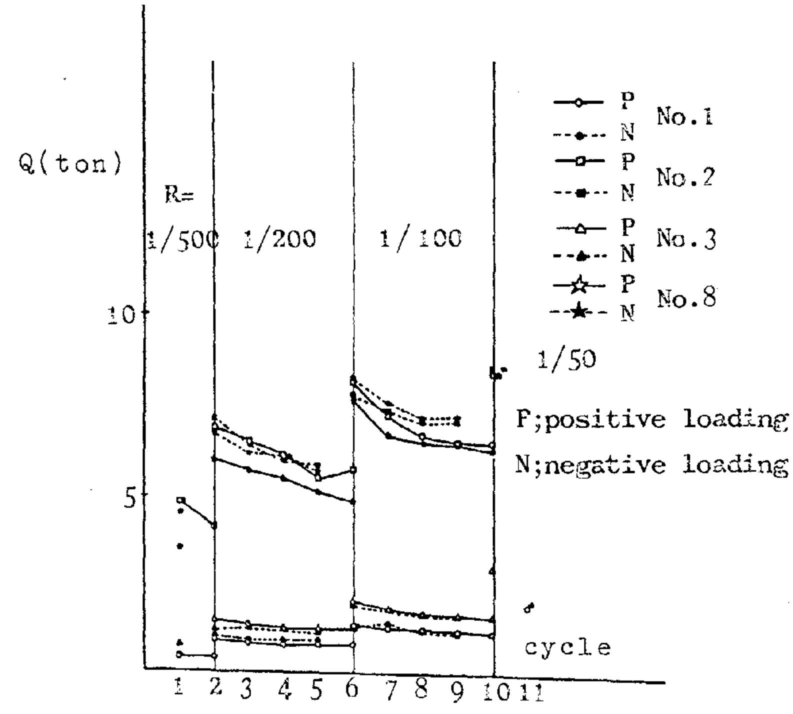
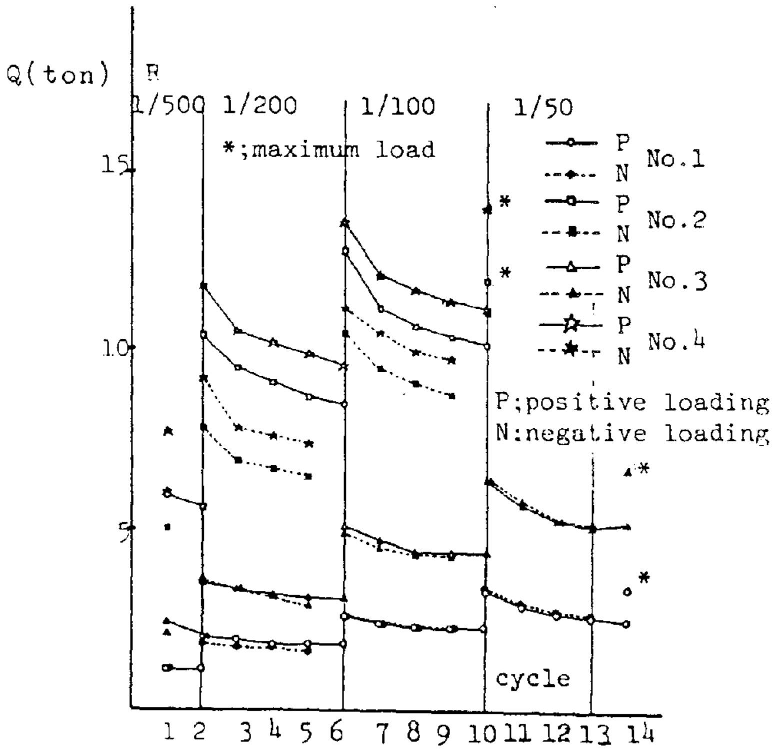


Fig. 9 Story Shear Force - Story Deflection Curve (81-No.2)



(a) 79 series



(b) 81 series

Fig. 10 Influence of Cyclic Loading

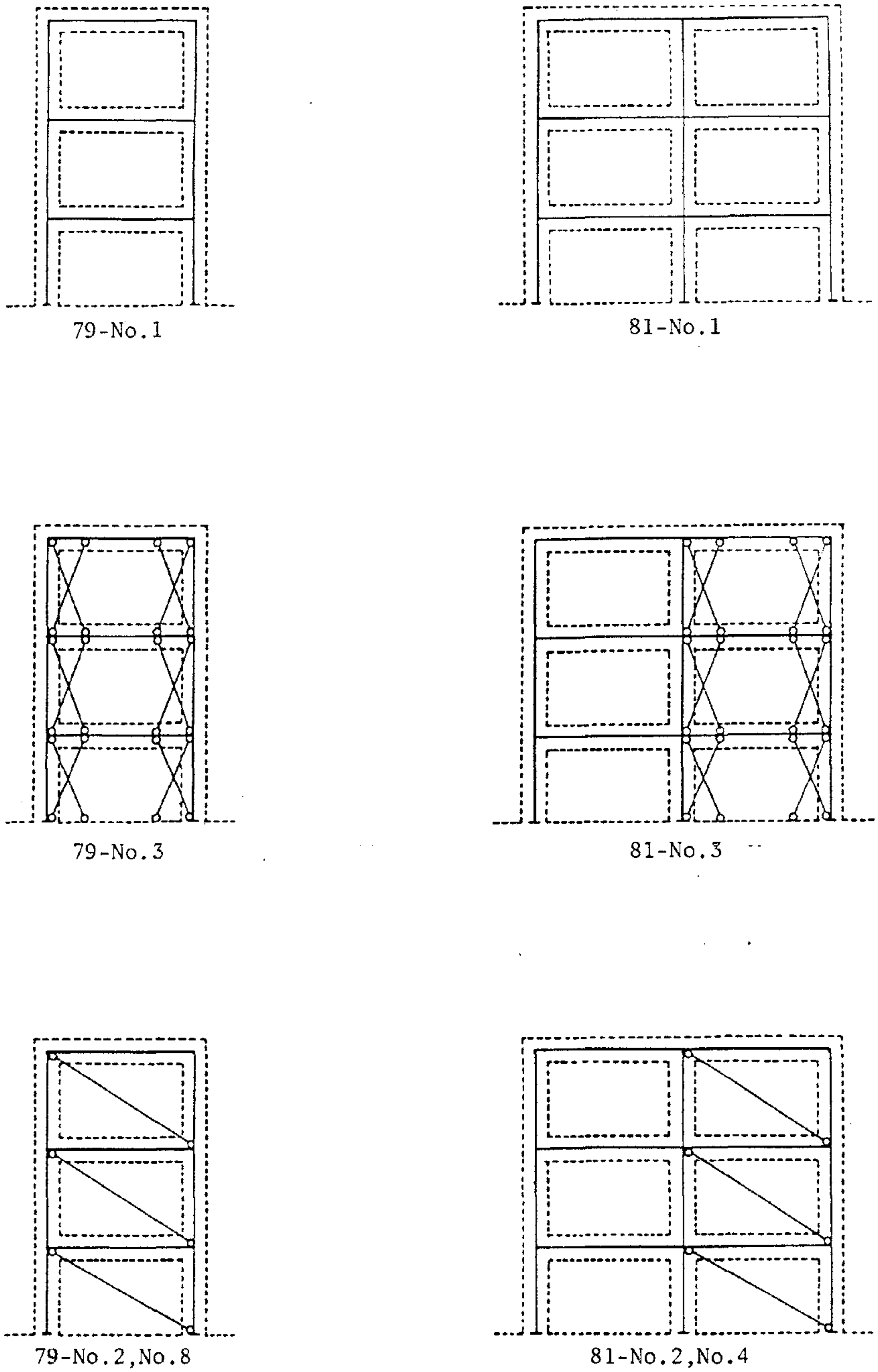
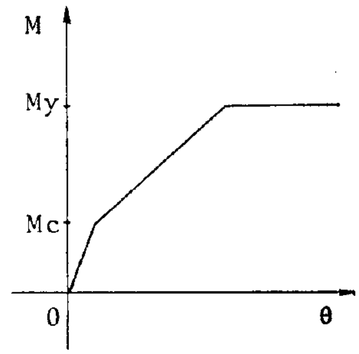


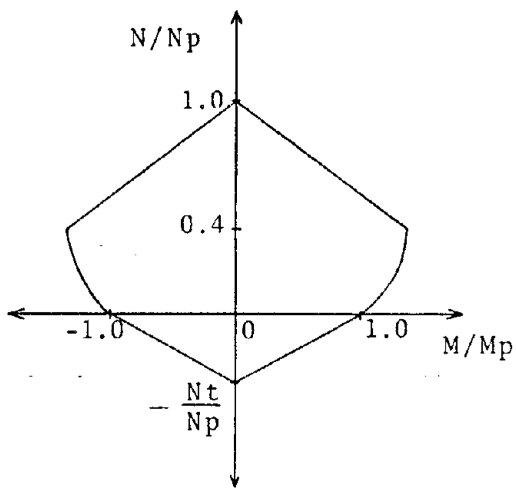
Fig. 11 Displacement Model



$$Mc = 1.8\sqrt{Fc}\cdot Ze + N\cdot D/6$$

$$My = 0.8\cdot at\cdot \sigma_y D + 0.5N\cdot D(1-N/bDFc)$$

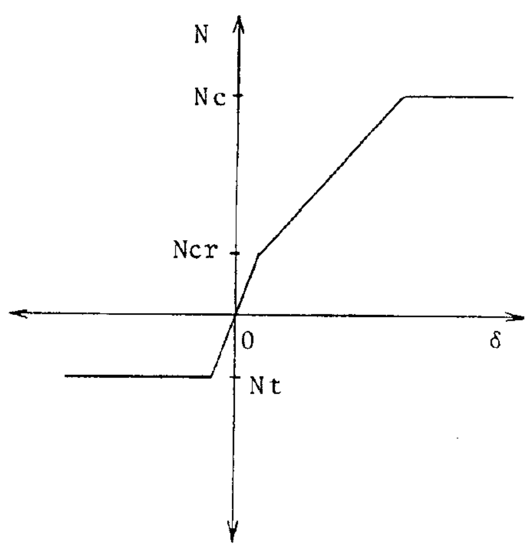
(a) Moment and Rotation Relation at the End of Members



$$Mp = 0.8at\cdot \sigma_y \cdot D$$

$$Np = Fc\cdot b\cdot D$$

(b) Interaction Curve



Nc ; compressive strength

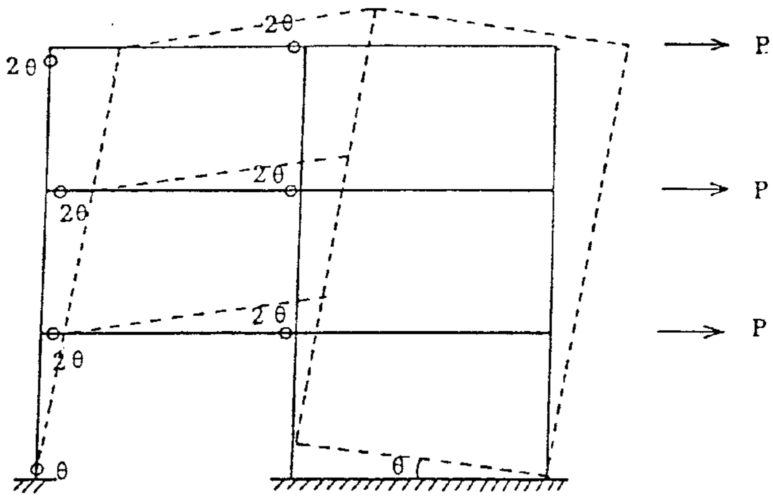
Ncr ; axial force at crackin

Nt : tensile strength

(c) Stress-Strain Relation of the Additional Wall as Bracing

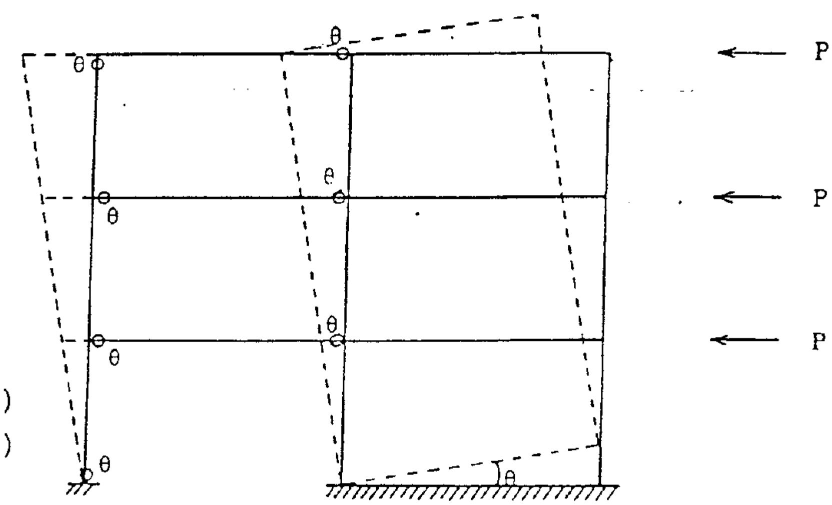
Fig. 12 Assumptions of Analysis

Shear Force
= 13.1 (No.2)
= 14.2 (No.4)



(a) Positive loading

Shear Force
= 8.3 (No.2)
= 9.4 (No.4)



(b) Negative loading

Fig. 13 Failure Mode ('81-No.2,No.4)

STUDIES REGARDING REPAIR AND RETROFITTING OF THE IMPERIAL COUNTY SERVICES BUILDING, EL CENTRO, CALIFORNIA

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Professor of Civil Engineering, University of California, Berkeley

INTRODUCTORY REMARKS

On November 21, 1979, five weeks after the earthquake of October 15, 1979, the Imperial County Board of Supervisors outlined the Scope of Work for Phase 1, to determine whether to undertake repair and strengthening of the County Services Building (CSB) or to demolish and replace the existing structure. The Scope of Work for Phase 1 was as follows.

(1) Review contract documents, plans, and specifications, calculations, and other documents to determine the structural makeup of the CSB.

(2) Visit the damaged structure and review and list the areas of structural damage. (Also review and verify the adequacy of the present shoring system.)

(3) Develop and implement a program to assess the extent of structural damage to the building. (Retention of a testing laboratory in connection with such program will be desirable.)

(4) Review building codes and other requirements which would have to be met for the building to meet seismic resistance standards and other requirements.

(5) Determine which portions of the CSB are irreparably damaged.

(6) Complete a preliminary analysis to assess structural (including foundation) strengthening necessary to meet requirements referred to under (4) above, if such strengthening is determined to be feasible. Such analysis shall include hand calculations.

(7) Make a preliminary determination of costs which would be required in replacing irreparably damaged portions of the building and in any structural strengthening deemed necessary and feasible for the remaining portions

of the CSB.

(8) Make a preliminary determination of costs which would be required for the demolition of the CSB and replacement thereof in accordance with applicable standards.

(9) Complete a report which summarizes conclusions and findings arrived at under the aforesaid tasks.

By December 1979 the Board of Supervisors had selected the firm of Blaylock-Willis and Associates to conduct the necessary work. Specialists in structural, mechanical, and electrical engineering, architecture, cost estimating, personnel specializing in epoxy injection repair, building demolition, and elevator repair, visited the CSB during the first week of January 1980 for a detailed field investigation. The Blaylock-Willis and Associates firm prepared a detailed report of the damages and estimate of costs to repair and/or construct a new facility.

The report, submitted February 1980, expressed the opinion that it was possible to repair and strengthen the CSB. A preliminary scheme for structural, architectural, mechanical, and electrical modifications, was presented, however, it stated, "it is doubtful whether it would be practical from an economic standpoint to undertake such repairs because the repair costs would nearly equal the costs for demolition and construction of a new facility containing the same space and capacity." A January 1, 1981 estimated cost for repair and strengthening was $4,995,000. A January 1, 1981 estimated cost for demolition and construction of a new facility was $5,037,000.

The author, with S. A. Mahin, has been involved in a study of the performance of the CSB. A detailed field survey of the building damage was conducted and analyses of the building performance were made. Repairing and retrofitting the structure were also investigated and found to be feasible.

Studies of the CSB emphasize that in dealing with the feasibility of repairing and retrofitting buildings for seismic resistance, it is necessary to investigate the technical and economical problems of repair and retrofitting the structure, and also to consider other problems which will be discussed briefly herein.

REQUIREMENTS TO BE CONSIDERED IN THE REPAIR AND RETROFITTING OF THE CSB

The CSB was designed according to the 1967 Uniform Building Code and related mechanical and electrical codes. Considerable changes have occurred in the codes since the original building was designed. These are especially true in the areas of seismic structural design, fire safety, energy standards, and handicapped persons' requirements. It is also noted that the applicable codes generally require that when alterations and repairs are made equal to more than fifty percent of the value of the existing building or structure, such building shall be made to conform to the requirements for new buildings and structures.

excess of fifty percent of the value of the building and, therefore, must comply to the new codes and standards in existance at the present time.

The Department of Housing and Urban Development regulations for federal disaster assistance require that any repairs or construction shall be in accordance with the applicable standards of safety, decency and sanitation that were in effect at the time of the disaster. They further indicate that the facility shall conform with current local applicable codes, specifications and standards, and that any reconstruction shall result in a safe and usable facility.

Current Applicable Standards: In accordance with the current application standards in the City of El Centro and in the Imperial County, the standards used to evaluate the CSB are as follows:

(1) 1976 Uniform Building Code

(2) 1976 Uniform Plumbing Code

(3) 1976 Uniform Mechanical Code

(4) 1978 National Electrical Code

(5) Title 24 of the California Administrative Code for Energy Standards

(6) Chapter 7, Division 5, of Title 1 of the Government Code, Access to Public Buildings by Physically Handicapped Persons

BUILDING DESCRIPTION

REFERENCES

1. Allawh, N., "A Study of the Behavior of the Imperial County Services Building during the 1979 Imperial Valley Earthquake," Master of Engineering Thesis, Department of Civil Engineering, SESM Division, University of California, Berkeley, June 1982.

2. Blaylock-Willis and Associates, "Damage Evaluation for Imperial County Services Building, El Centro, California," San Diego, CA, February 1980.

3. Wosser, T., et al., "On the Earthquake Induced Failure of the Imperial County Services Building," in Reconnaissance Report, Imperial County, California Earthquake, October 15, 1979, EERI, February 1980, pp. 159-172.

4. Selna, L., "Performance of the Imperial County Services Building, El Centro," Proceedings, 49th Annual, 1980, SEAOC, Sept. 1980, pp. 37-68.

5. Bertero, V. V., and Mahin, S. A., "Investigation of Performance of the Imperial County Services Building during the 1979 Imperial Valley Earthquake." A Summary of Research Progress Report to National Science Foundation, August 24, 1981. (Copy attached)

BUILDING DAMAGE

Extensive nonstructural and structural damage occurred. These damages are described in detail in Refs. [1] - [4].

REPAIR AND RETROFITTING OF BUILDING

GENERAL REMARKS

It is convenient to distinguish the repair and retrofitting of the structure from the repair of the nonstructural elements (architectural) and of the mechanical and electrical components.

REPAIR OF STRUCTURAL ELEMENTS (See Ref. 2 for detailed discussion)

Repair by Epoxy Injection. All the concrete structural elements, except for those located in the east end bay and the columns of the ground story, could be repaired by epoxy injection. One of the greatest difficulties in this repair method is that it requires that all exterior surfaces of the cracked structural elements be available to proper epoxy injection repair. This requires removal of any nonstructural components attached to the surfaces. Most of the walls and slabs were covered by tiles, plaster, insulation material, carpets, electrical panels, or equipment. Removal of these nonstructural components made the repair very expensive. The cost estimation of epoxy repair was $340,000. This includes the epoxy grout repair.

Epoxy and Concrete Grout Repair. In the top portion of most ground story columns and some of the connecting girders, there was some small spalling whch could be repaired by chipping out loose concrete, cleaning the remaining cavity and then placing epoxy or concrete grout.

Partial Concrete Removal and Reconstruction. The concrete of all the ground story columns had crushed and spalled at the ground level. Furthermore, in several columns the bars that were exposed showed permanent distortion (buckling). In the lower part of these columns the spacing of the ties was so large that it did not provide confinement to the concrete. Therefore, the concrete of the lower part of the columns would have to be removed, and new longitudinal and lateral reinforcing steel would have to be provided. This would have been very expensive as it would require shoring the building.

Demolition and Reconstruction of the East Wing of the Building. The floor slabs and girders on the east end bay suffered considerable structural damage throughout the whole height of the building as a consequence of the 1 ft shortening of the east end ground story columns. Although the east wall could have been jacked up until all the floors recovered their original level, it was considered that for restoring the structural continuity it would be convenient to demolish this end bay and build new structural members as required for proper retrofitting of the whole building.

The main problem in demolishing the east wing was to establish the limits of demolition to facilitate the structural integration of the new construction with the existing one. In establishing these limits, careful attention had to be paid as to how the main reinforcing bars would be exposed and then spliced with the new bars.

NONSTRUCTURAL COMPONENTS

Architectural Elements. Windows, sills, plaster, stairwells, partitions, and all types of building finishes suffered damage, some of an extensive nature. Furthermore, to facilitate structural repair, a large amount of architectural finish work would have to be removed and replaced.

Mechanical. Not only did many of the air conditioning system fixtures need to be checked and repaired, but the entire air conditioning system had to be upgraded to comply with new codes. The plumbing system also needed to be checked and repaired. Although the elevators suffered minor damages, existing code requirements were not met and so additional work was needed.

Electrical Components. The lighting fixtures needed cleaning and to be installed with better supporting fixtures. It was also necessary to upgrade the electrical system.

RETROFITTING OF THE BUILDING

Analysis of the building performance clearly indicated the need for retrofitting the ICSB structural system. Bertero and Mahin considered various methods of retrofitting by using R/C shear walls. See Figs. 1 and 2. Blaylock-Willis and Assoc., after studying the strengthening of the ICSB, concluded that it was structurally feasible to repair and strengthen the building by adding X braced frames on the longitudinal north and south elevators (see Fig. 3), and by extending east and west walls down to the foundation (Fig. 4). However, the estimated cost to repair and strengthen the ICSB would have been $4,995,000. The cost to demolish the ICSB and to build a new facility with a floor area similar to the existing one as of January 1981, would have been $5,037,000. Thus it was concluded that it was prefereable to demolish the building and to build a new facility.

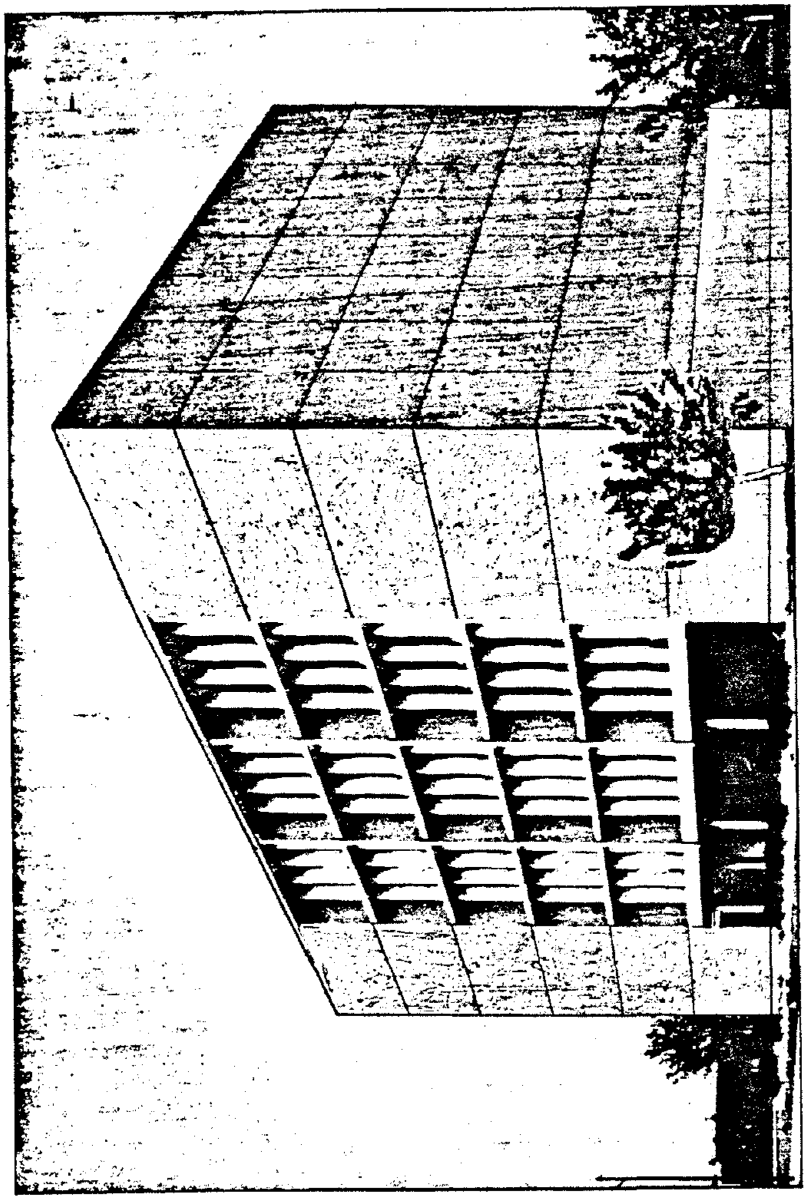

Fig. 1

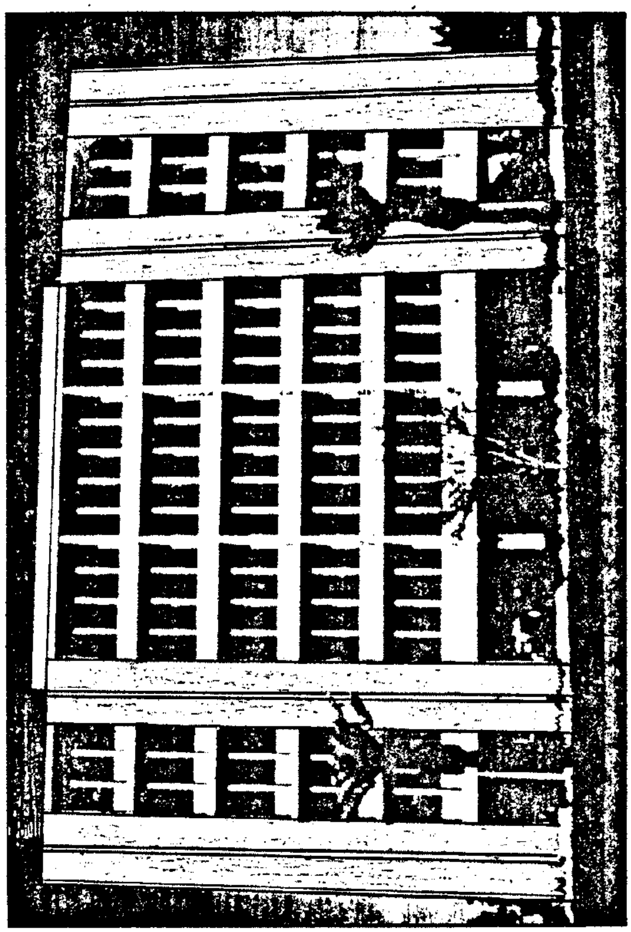

Fig. 2

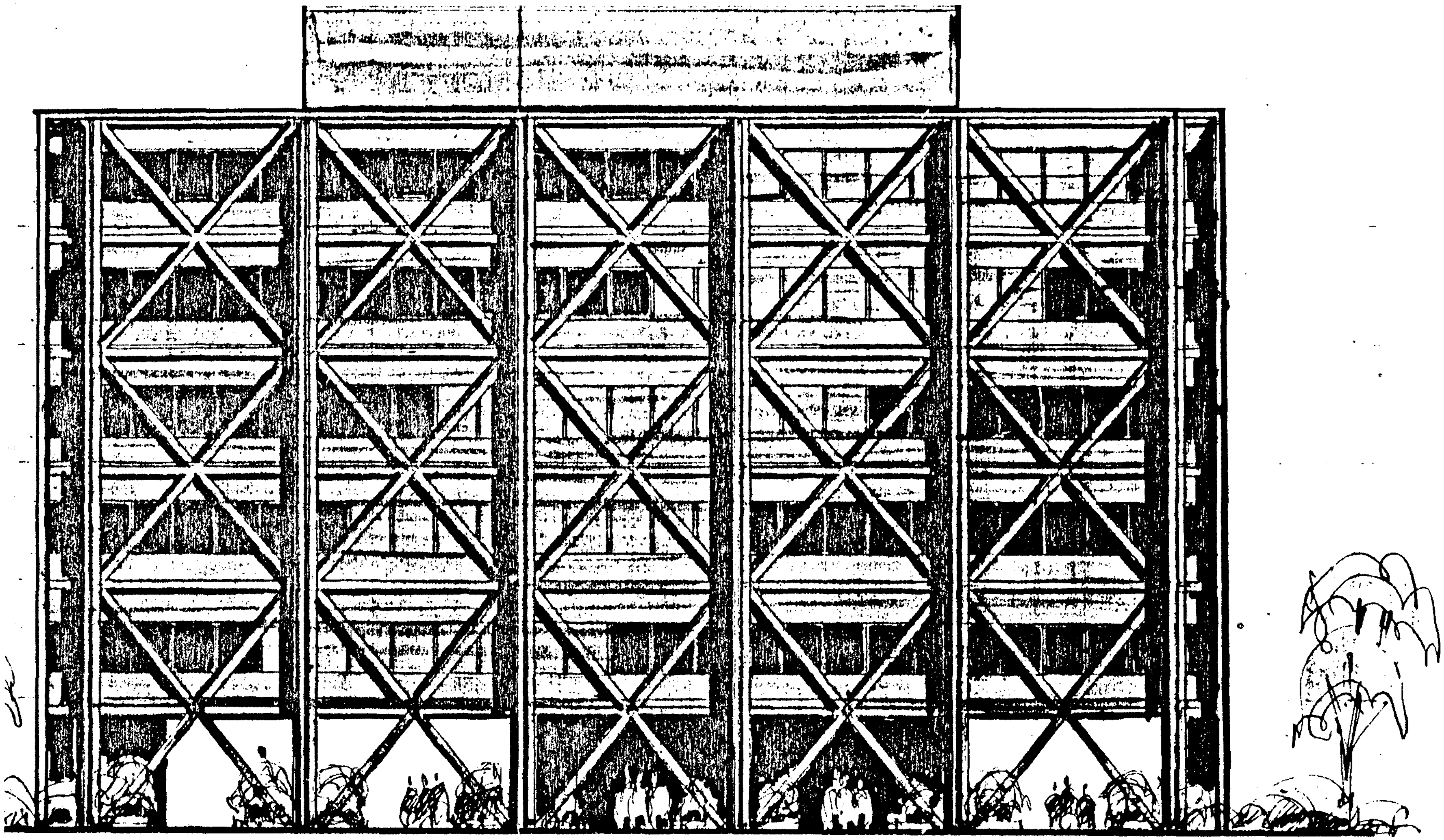

Fig. 3 PRELIMINARY STRENGTHENING STUDY

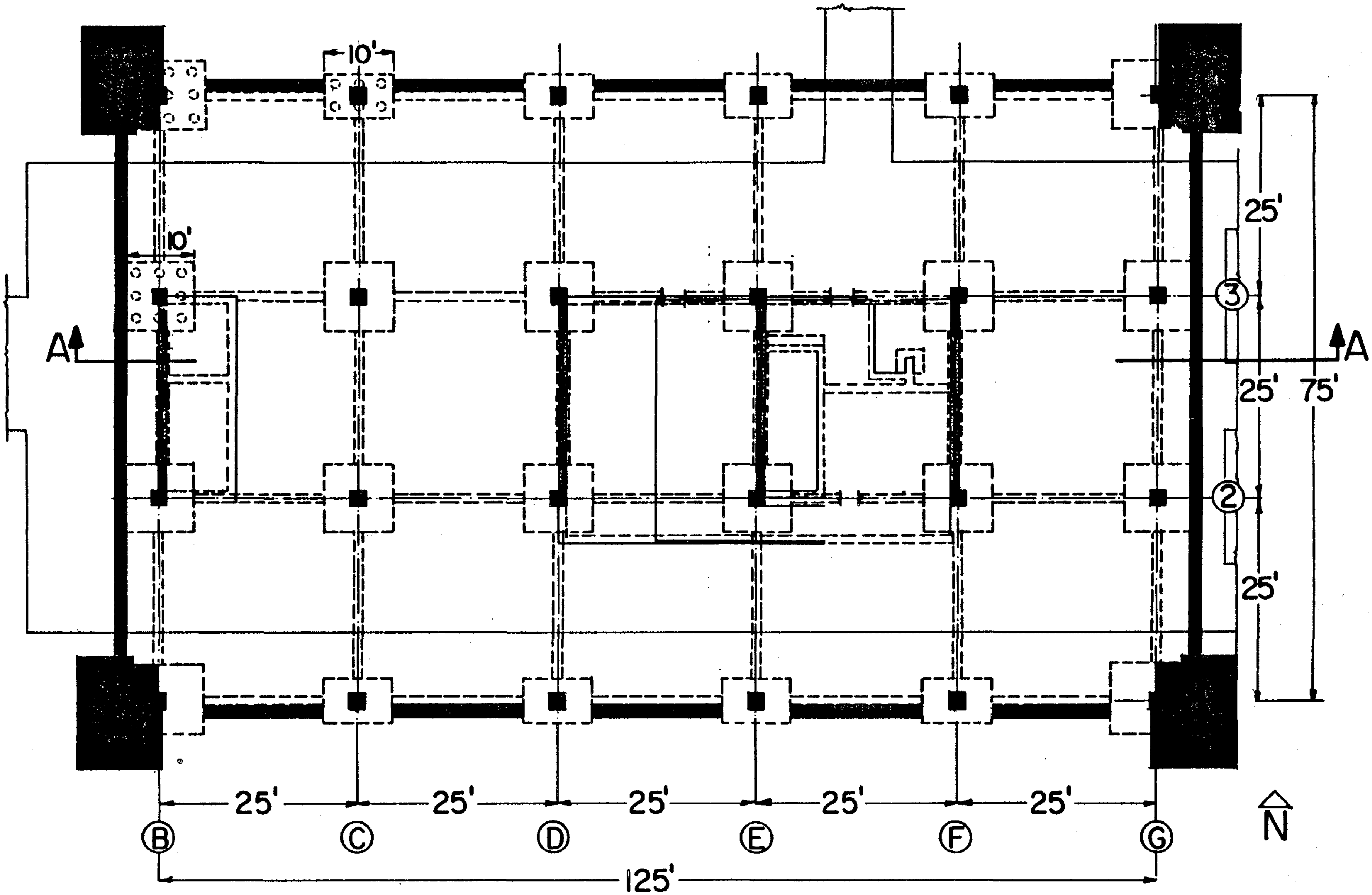

Fig. 4 FOUNDATION PLAN OF IMPERIAL COUNTY SERVICES BUILDING EL CENTRO, CALIFORNIA

ON THE DEVELOPMENT OF POST-EARTHQUAKE MEASURES FOR CIVIL ENGINEERING STRUCTURES DAMAGED BY EARTHQUAKES

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Presented at the Third Joint Meeting of Repair and Retrofit of Existing Structures, UJNR, San Francisco, California, U.S.A., May 13-15, 1982

ABSTRACT

The Ministry of Construction is conducting an extensive research project entitled "Post-Earthquake Measures for Buildings and Structures Damaged by Earthquakes," from April, 1981 to March, 1986.[1] This project is to develop guidelines on post-earthquake measures for practical engineers by presenting procedures of inspection and measurement of structures affected by strong motions, assessment of damage extent, and methods of repair and strengthening of damaged structures. The Public Works Research Institute (PWRI) is in charge of the development of post-earthquake measures for civil engineering structures such as slopes and earth structures, highway bridges, underground structures, etc. This paper describes the results of studies done by PWRI for the first year of this five-year project, and also introduces the program for the succeeding four years.

INTRODUCTION

Locating on the Circum Pacific Seismic Belt, Japan is one of earthquake-hazardous countries in the world and has suffered many times from major seismic disasters in her histories. The recent advancement in structural and earthquake engineering, however, has enable us to safely construct huge structures such as long-span bridges and high-rise buildings. In addition, the research project "Development of New Seismic Design Methods for Buildings and Structures"which was performed by the Ministry of Construction form 1972 to 1977 in the light of the lessons from the Tokachi-oki Earthquake of 1968 and the San Fernando Earthquake of 1971, has led our seismic design codes to a higher level.

As a result, severe damages such as entire collapses of buildings and structures have extremely decreased, and accordingly human lives have become to be kept comparatively in safe during recent large earthquakes. Minor damages and partial failures, however, may still occur in future earthquakes, as observed in the recent ones such as the Izu-Ohshima Kinkai Earthquake of 1978 (M = 7.0) and the Miyagi-ken-oki Earthquake of 1978 (M = 7.4). Post-earthquake measures for partially damaged structures become significant concerns to administrative governmental offices.

In two recent earthquakes brought off in Algeria (October, 1980) and Southern Italy (November, 1980), some buildings which had been weakened by the main shock were throughly collapsed by aftershocks and also delays of urgent help and post-earthquake inspection and repair caused secondary disasters which were very serious. The facts suggest the importance of appropriate post-earthquake countermeasures.

Basing on the background, the Ministry of Construction has initiated a project entitled "Development of Post-Earthquake Measures for Buildings and Structures Damaged by Earthquakes," in order to develop post-earthquake measures by presenting procedures of inspection and measurement, assessment of damage extent, and methods of repairing and strengthening of damaged structures.

Figs. 1 and 2 indicate the concept of the project and the flowchart of the development of the project, respectively.[1]

STUDIES FOR THE FIRST YEAR, 1981

To develop post-earthquake measures for civil engineering structures, the Public Works Research Institute has made a contract with the Research Center for National Land Development Technology. The Center has organized the Committee on Post-Earthquake Measures for Civil Engineering Structures Damaged by Earthquakes (Chairman : Professor Shunzo Okamoto), as well as three Subcommittees, i.e., Subcommittee on Slopes and Earth Structures (Chairman : Professor Kenji Ishihara), Subcommittee on Bridge Structures (Chairman : Professor Motohiko Hakuno), and Subcommittee on Underground Structures (Chairman : Professor Tsuneo Katayama). The Committee and the Subcommittees have discussed post-earthquake measures with emphasis on the experiences of past earthquake damages and presented an interim report.[2] The followings are summaries of the results obtained in the fiscal year of 1981.

SLOPES AND EARTH STRUCTURES

Regarding post-earthquake measures for slopes and earth structures, the following studies were conducted.

1) Existing measurements of extent of seismic damages to slopes and earth structures were examined; and practical applications of measuring instrument were considered. Furthermore, methods and instruments measurement including the diagonal aerial photography and the improved sounding mehtod are indicated as ones to be newly developed. The program for the succeeding years of the five-year project were also discussed in details.

2) Types of seismic damages to slopes and earth structures were classified into several groups. Using this classification, past seismic damages were systematically tabulated. Fig. 3 illustrates an example of types of damages to earth embankments resting on horizontal ground surfaces.
3) Documents of seismic damages and repair of slopes, embankments, and coastal banks were collected and summarized. These documents will be a basic information for the formation of repair methods of slopes and earth structures, according to damage extent.
4) With view of damage experiences of the Miyagi-ken-oki Earthquake of 1978, important factors to be considered in assessing damage extent were pointed out, and survey items were tabulated chronologically in relation to elapsing time from the break-off of an earthquake. Fig. 4 shows an example of survey items for assessing seismic damages to earth structures.

BRIDGE STRUCTURES

The following studies were done regarding post-earthquake measures for bridge structures.

1) Past experiences of seismic damages to bridge structures were extensively collected for several earthquakes including the Niigata Earthquake of 1964, the San Fernando Earthquake of 1971, the Miyagi-ken-oki Earthquake of 1978.
2) Types of seismic damages to bridge structures were classified into several groups, and bridge damage features were investigated. Fig. 5 illustrates the time-dependent variation of traffic restrictions in terms of causes of highway damages. It can be seen that repair of bridge structures takes longer time than damages to other structures. Table 1 shows a classification of types of seismic damages to bridge structures. Fig. 6 shows contents of damages to individual parts of highway bridges. (A), (B), and (C) of Fig. 6 correspond to substructures, superstructures and bearing supports of bridges, respectively, for two different earthquakes : the Niigata Earthquake and the Miyagi-ken-oki Earthquake.
3) Damage contents and repair methods were studied in details for 17 highway bridges which were severely damaged during the Miyagi-ken-oki Earthquake.

UNDERGROUND STRUCTURES

Seismic damage features and post-earthquake measures were studied for water supply systems, sewage systems, gas pipelines, electric cable ducts, telephone ducts, water gates and conduits, and highway tunnels. The followings are derived from the studies.

1) Cracks anf failures of pipes and ducts, failures of joints, damages to manholes and their connecting portions are found to be frequent types of damages, as already indicated from previous studies.
2) Since underground structures spread very widely in cities, most of damages were discovered by residential people and reported to responsible organizations. For those underground structures for which discovery of damages is difficult by sight, television cameras and other instruments are recently introducted to point out damages.
3) Repair works are implemented in different ways for various underground structures. Table 2 is an example of reapir works for sewage ducts.[3)]

PROGRAM OF FUTURE STUDIES FOR CIVIL ENGINEERING STRUCTURES

Program of future studies was discussed to further develop comprehensive post-earthquake measures for civil engineering structures damaged by earthquakes. Table 3 briefly shows the time table of the program. As shown in the table the project will be completed in four years.

References

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2) Research Center for National Land Development Technology, "Report on the Development of Post-Earthquake Measures for Structures Damaged by Earthquakes (In Japanese)," March, 1982
3) Nishida, Sugano, Takegawa, "Report on Damages to Sewage Facilities during the Miyagi-ken-oki Earthquake of 1978 (No. 1) (In Japanese)," Journal of Japan Sewage Association, Vol. 16, No. 180, May, 1979

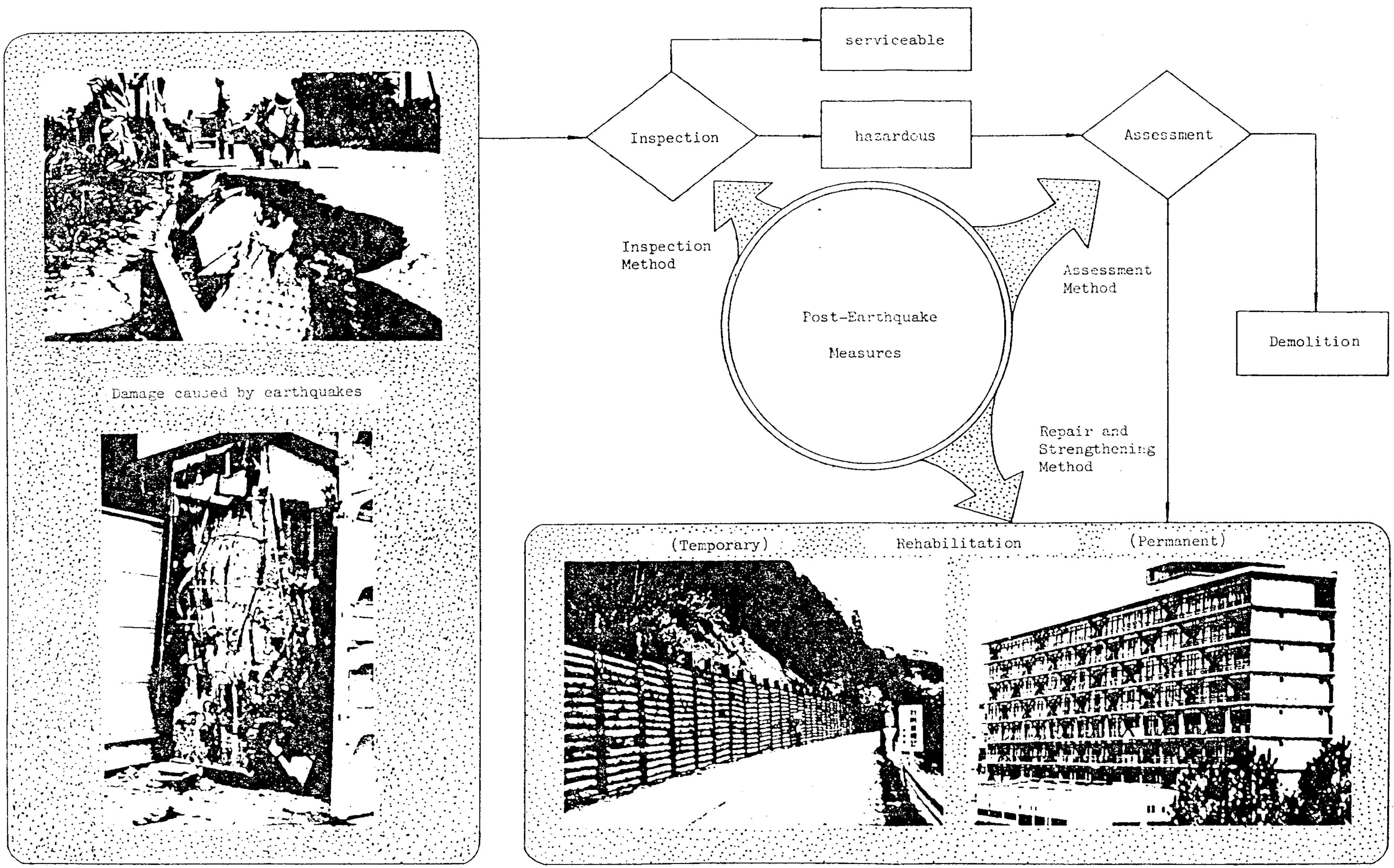


Fig. 1 Concept of Development of Post-Earthquake Measures for Buildings and Structures Damaged by Earthquakes

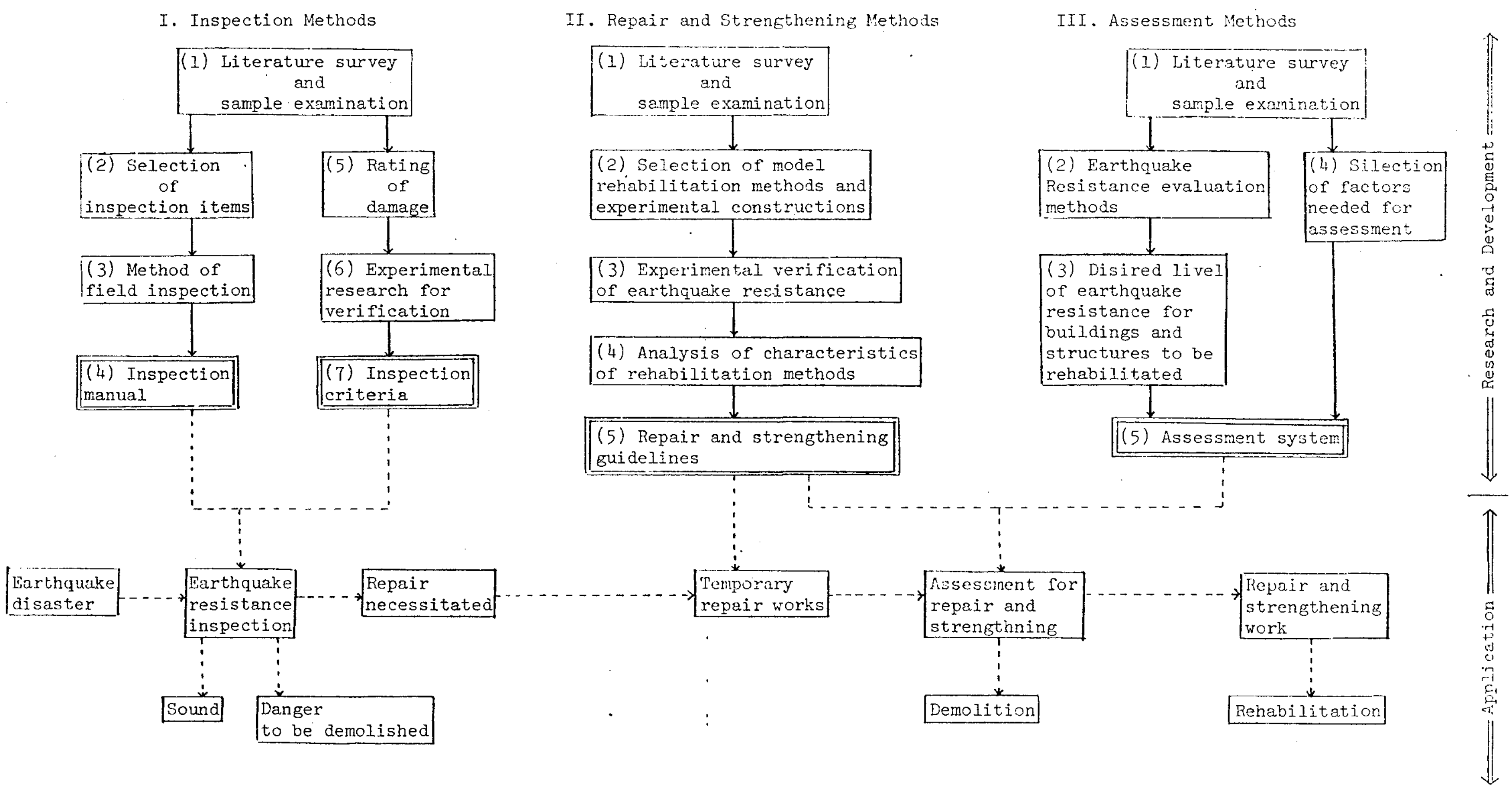
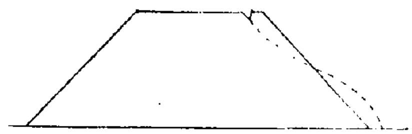
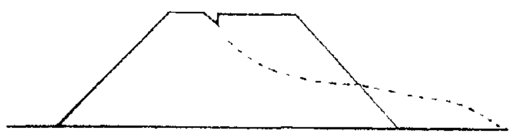
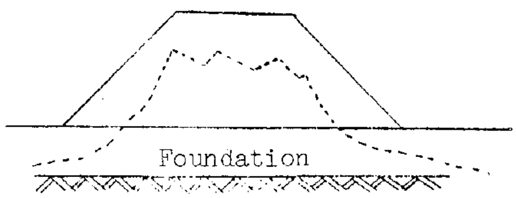
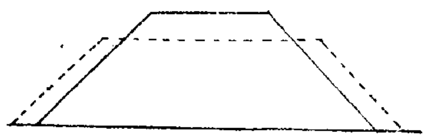


Fig. 2 Flowchart of the Project

Type	Illustration	Descriptions of Damages
1		Light slope failure —— (Opening of crack is near the shoulder)
2		Heavy slope failure —— (Opening of crack is near the center of the crest)
3		Severe Settlement (due to the failure of foundation)
4		Uniform settlement (without foundation deformation)

Damage Extent

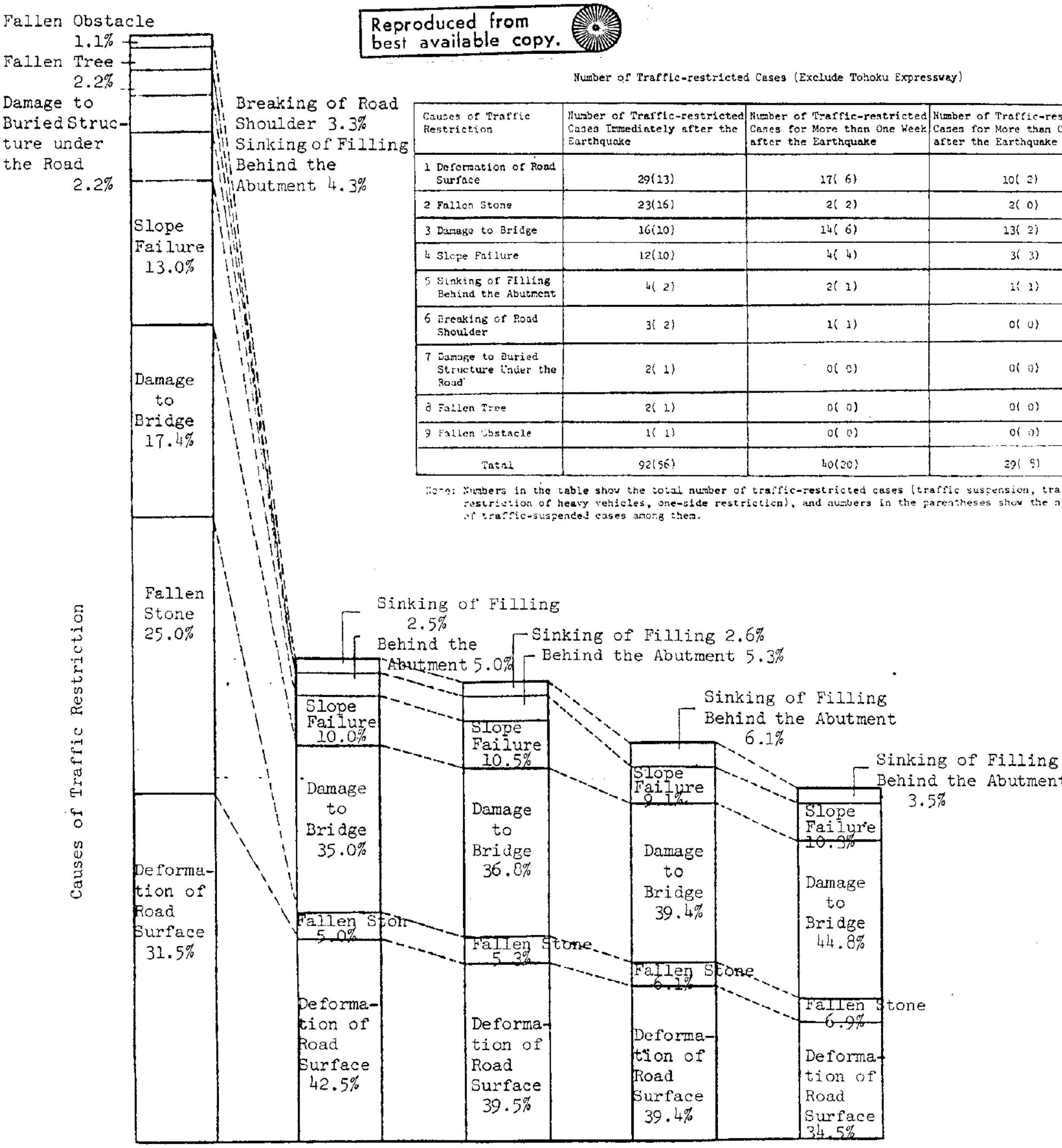
Type	Rank of Damage Extent	Explanations of Damage Extent
1	B	Cracking of 15 cm wide or more or Differential Settlement of 20 cm or more
	C	Cracking of less than 15 cm, and Differential Settlement of less than 20 cm
2	A	Cracking of more than 30 cm and Differential Settlement of more than 50 cm
	B	Cracking of 30 cm or less, or Differential Settlement of 50 cm or less
3	A	————
4	B	Settlement of 50 cm or more
	C	Settlement of less than 50 cm

Fig. 3 Types of Damages to Embankments on Horizontal Ground

First-stage survey → (Planning of quick repair works → Implementation) → Second-stage survey → Third-stage survey

	First-stage survey	Second-stage survey	Third-stage survey
Time:	Immediately after the quake	When planning rapair works	When designing repair works
Objectives:	1) Outline of damages (locations, damage extent, damage types) 2) Judgement of quick repair works 3) Judgement of detailed survey 4) Judgement of for restricting or closing traffic to avoid secondary damage	1) Details of damages 2) Estimate of cost and volume of repairs 3) Planning of geographycal survey 4) Planning of geological survey } Decision of repair works	1) Implementation of geological survey 2) Survey on different route when necessitated (in case of difficulty of repair) 3) Besic investigations for improving seismic design
Methods:	Field survey (By sight)	Field survey (sight), sounding geographycal survey, collection of relevant information	Geographical survey, soil boring sounding, seismic survey (measurement of S-wave velocity), geological survey

Fig. 4 Survey items for assessing seismic damages to earth structures

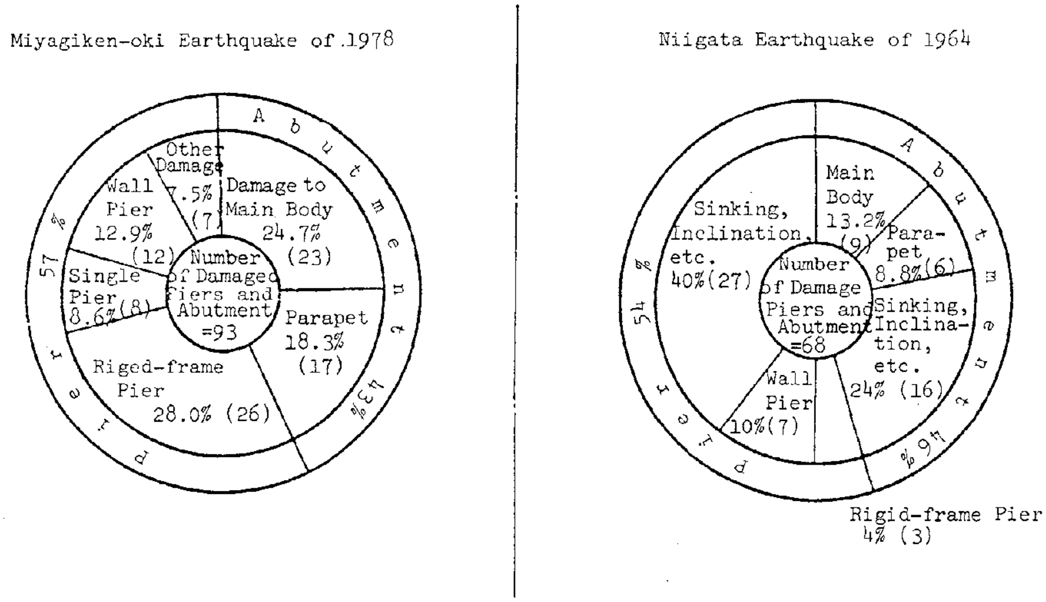


Number of Traffic-restricted Cases (Exclude Tohoku Expressway)

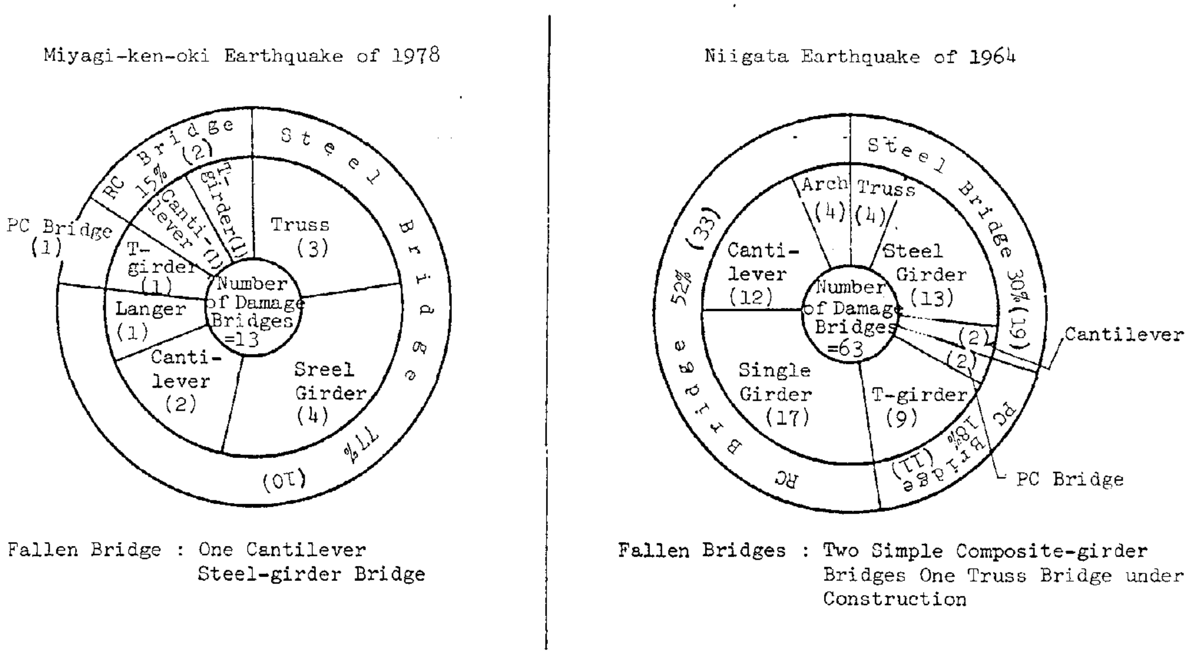
Causes of Traffic Restriction	Number of Traffic-restricted Cases Immediately after the Earthquake	Number of Traffic-restricted Cases for More than One Week after the Earthquake	Number of Traffic-restricted Cases for More than One Month after the Earthquake
1 Deformation of Road Surface	29(13)	17(6)	10(2)
2 Fallen Stone	23(16)	2(2)	2(0)
3 Damage to Bridge	16(10)	14(6)	13(2)
4 Slope Failure	12(10)	4(4)	3(3)
5 Sinking of Filling Behind the Abutment	4(2)	2(1)	1(1)
6 Breaking of Road Shoulder	3(2)	1(1)	0(0)
7 Damage to Buried Structure Under the Road	2(1)	0(0)	0(0)
8 Fallen Tree	2(1)	0(0)	0(0)
9 Fallen Obstacle	1(1)	0(0)	0(0)
Total	92(56)	40(20)	29(9)

Note: Numbers in the table show the total number of traffic-restricted cases (traffic suspension, traffic restriction of heavy vehicles, one-side restriction), and numbers in the parentheses show the number of traffic-suspended cases among them.

Fig. 5 Transition of Traffic Restriction after the Miyagi-ken-oki Earthquake of 1978 (Exclude Tohoku Expressway)

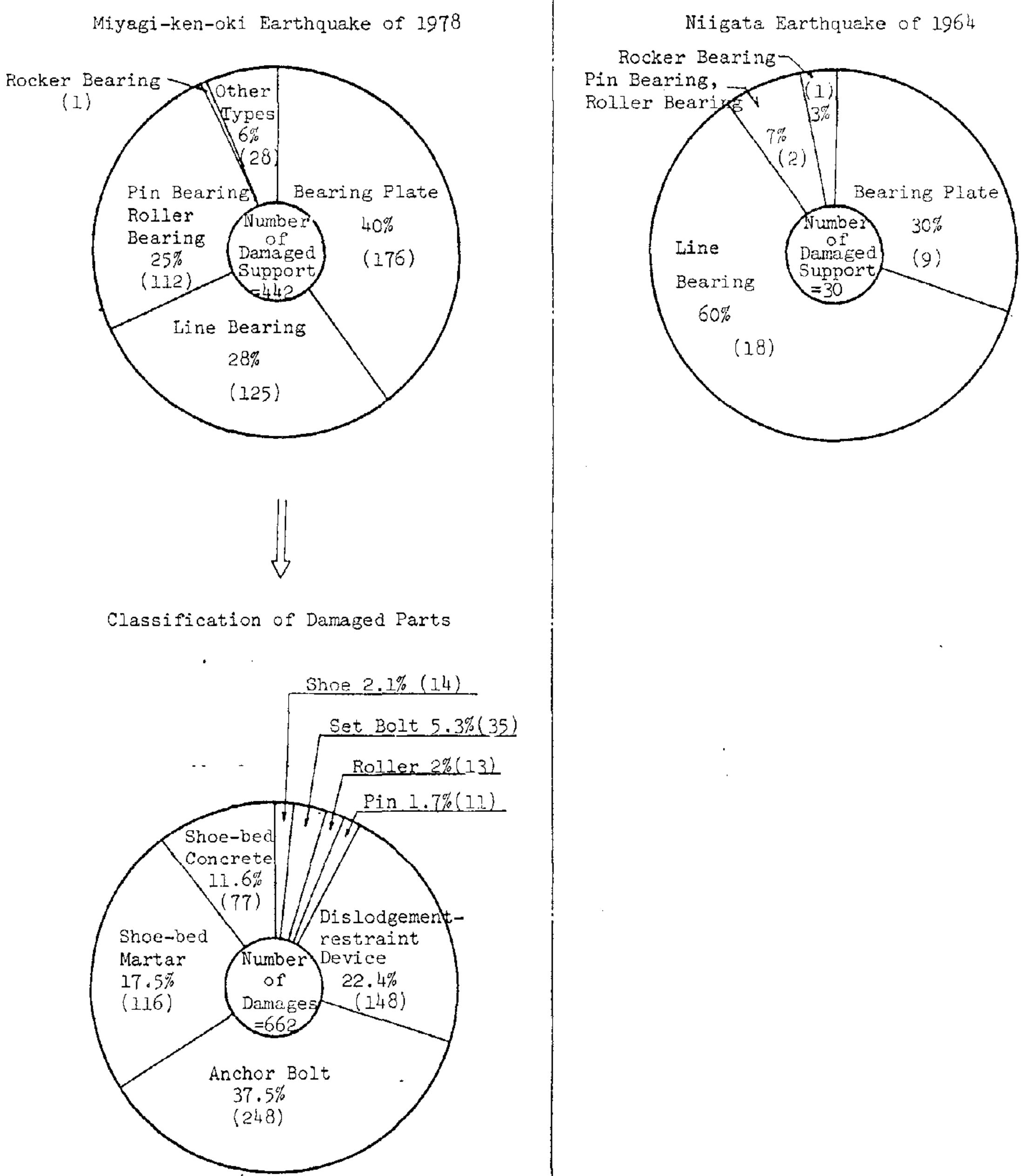


Damage to Substructures



Damage to Substructures

Fig. 6 Damage Features to Highway Bridges by the Past Earthquakes (No. 1)



Damage to Supports

Fig. 6 Damage Features to Highway Bridges by the Past Earthquakes (No. 2)

- Superstructure
  - Steel Bridge
    - Truss
      - Broken-out of a Main Member
      - Buckling of a Truss Member over the Fixed Bearing
      - Buckling of a Sway Bracing
    - Langer
      - Deformation of a Portal Bracing
      - Deformation of a Lateral Bracing
    - Plate Girder
      - Buckling of a Sway Bracing over the Support
      - Damage to a Main Girder (Collision with a Parapet)
      - Falling of a Suspended Girder (Movement in the Longitudinal Direction)
      - Movement in the Transverse Direction
  - Prestressed Concrete Bridge
    - Prestressed Concrete Girder
      - Crack in a Main Girder over the Support
  - Reinforced Concrete Brigde
    - Reinforced Concrete Girder
      - Crack in a Main Girder over the Support
      - Crack in a Main Girder
- Substructure
  - Abutment
    - Main Body
      - Crack in the Front of a Main Body
      - Crack in the Side of a Main Body
    - Crack in a Parapet
    - Other Damage
      - Damage to the Top of an Abutment
      - Sinking and Inclination of an Abutment
  - Pier
    - Rigid-frame Pier
      - Horizontal Crack in Column
      - Vertical Crack in a Beam
    - Single Pier
      - Horizontal Crack in a Column
    - Wall pier
      - Horizontal Crack
      - Vertical Crack
    - Other Damage
      - Damage to the Top of a Pier
      - Sinking and Inclination of a Pier
- Support
  - Shoe
    - Deformation and Broken-out of Upper and Lower Shoes
  - Set Bolt
    - Slackness of Bolting and Broken-out of a Bolt
    - Damage to a Sole Plate
  - Roller
    - Dislodgement of Rollers
  - Pin
    - Broken-out of a Pin
    - Pulled-out of a Pin Cap
  - Dislodgement-restraint Device
    - Deformation and Broken-out of a Stopper
    - Deformation of a Side Block
  - Anchor Bolt
    - Pulled-out of an Anchor Bolt
    - Broken-out of an Anchor Bolt
  - Shoe-bed Mortar
    - Crack and Demolition of Mortar
  - Shoe-bed Concrete
    - Crack and Demolition of Concrete

Table 1 Classification of Damage to Highway Bridges in the Miyagi-ken-oki Earthquake of 1978

Table 2 Damages to Sewage Ducts and Repair Methods in the Miyagi-ken-oki Earthquake*

Structures	Description of Damages		Repair Method
Manholes	Failures		Replacement new construction
	Settement/ Movement	Heavy	Replacement new construction
		Minor	Partial trimming
Connections between vertical walls of manholes and inclined walls	Differential movement		Excavation/Trimming (Replacement)
	Leakage		Excavation/Leakage stop (replacement)
	Failure		Replacement of wall
Ducts	Failure 1. Cross-sectional crack 2. Longitudinal crack 3. Failure of duct damage to duct body		Replacement of duct
	Leakage	Large duct	Leakage stop from inside
	No or small movement of joints	Small duct (ϕ450 mm or less)	Replacement of duct
	Movement of joing		Replacement of ducts when necessitated (hydraulic effects)

* After Nishida, Sugano, Takegawa, "Report on Damages to Sewage Facilities during the Miyagi-ken-oki Earthquake of 1978 No.1)," Journal of Japan Sewage Association, Vol. 16, No. 180, May, 1979

** When the leakage spot can not be pointed out, excavation may be permitted.

Table 3 Program of Future Studies for Civil Engineering Structures

Subjects			Contents					Final Output Results
Themes	Sub-themes	Subjects	1981 FY	1982 FY	1983 FY	1984 FY	1985 FY	
I. Inspection Method	I-1. Measuring Method	Civil Engineering Structures (Slopes, Earth Structures, Bridges, Underground Structures)	Collection and analysis of existing measuring methods	• Measuring methods for assessing damage extent • Aerial photograhy • Simplified soil sampling		Manual of measuring method		Guideline for measuring method and inspection method
	I-2. Inspection Mehtod	Civil Engineering Structures (Slopes, Earth Structures, Bridges, Underground Structures)	Collection of past experiences	• Inspection method for assessing damage extent • Preliminary experiment for assessing bridge damage	• Framework of inspection method • Experiment of bridge and earth structures	• Manual of inspection method • Experiment of earth structures		
II. Repair and Strengthening Method	II-1. Civil engineering Structures	Slopes and Earth Structures		• Preliminary study on R-S methods • Preliminary experiments and planning	Experiments on R-S method (Anchoring, sheet piling and retaining wall)	• Framework of manual of R-S methods • Experiments on R-S methods (slope protection work, pile)	• Experiments on R-S methods • Manual of R-S methods	
		Bridges		• Preliminary study on R-S methods • Preliminary experiments and planning	Experiments on repair methods (Injection of epoxy resin	• Framework of manual of R-S methods • Experiments on R-S method (Adding new concrete and bars to damaged piers)	• Experiments on R-S methods • Manual of R-S methods	Guidelines for R-S
		Underground Structures			Preliminary experiments and planning	• Framework of manual of R-S methods • Experiments on R-S methods (Adding new concrete and bars to damaged ducts	• Experiments on R-S methods • Manual of R-S methods	
III. Assessment method of Repair and Strengthening	III-1. Factors to be considered for assessment of Repair and strengthening methods	Civil Engineering Structures		Magnitudes and frequencies of aftershocks	• Optimum repairing procedures considering geometric distribution of structural systems • Factors for various structures			Guidelines for assessment of repair and strengthening
	III-2. Assessment methods of repair and strengthning	Civil Engineering Structures			• Assessment methods of repair and strengthening considering distribution of structural systems • Assessment methods for various structures	Damage assessment considering repair methods	• Manual of assessment of repair and strengthening	

REINFORCING STEEL CONSIDERATIONS UNIQUE TO REPAIR AND RETROFIT

by

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INTRODUCTION

The restoration of existing and addition of new reinforcing steel in the repair and retrofit of concrete and masonry structures requires many special considerations. In order to attain or maintain continuity of existing and new reinforcing, appropriate methods of joining must be utilized. Where new concrete or masonry is interrupted, such as by the framing members of a frame building where new infill panels are being installed, provision for the new reinforcing to penetrate or bypass the conflicting structural members must be made. Where new reinforcing terminates in existing concrete, suitable provision must be made to secure the new bars or dowels to the existing construction. In the United States, joining of individual lengths of reinforcing steel is most commonly done by lapping the bars, however, this method is often impractical and sometimes impossible, due to restricted exposure of existing steel as well as the often restricted working space inherent in repair and retrofit work. Welding is a frequently used alternate to lapping, however, special considerations must be directed toward details of the welding procedures to be utilized. The physical properties of the reinforcing steel must be determined, especially carbon content, and appropriate welding procedures selected based upon those properties.

ADDITION OF NEW REINFORCING STEEL

Reinforcing steel that has been excessively yielded or otherwise damaged must be replaced. Additionally, the existing reinforcing is often supplemented with new bars. In order to provide continuity for such new steel, the existing elements are often notched or drilled to enable bypassing or penetration of the new steel (Fig. 1). In preparing notches for new rebar, the preferred method is by utilization of chipping hammers fitted with pointed gads or chisels. Diamond sawing should be avoided as the diamond blades tend to polish the cut surface while being rotated. They also produce a relatively smooth surface that is not capable of accepting strong bond or transfer of shear to the new repair material. Likewise, the preferred method for drilling holes for the penetration of new steel, or installation of new dowels, utilizes percussion type drills fitted with bits that produce large cuttings and hollow drill steel that permits simultaneous blowing of compressed air providing immediate expulsion of the cuttings from the hole. In addition to producing clean,.roughened holes, the preferred drilling equipment tends to quickly detect the interference of existing reinforcing, thereby permitting relocation of the hole before any significant damage is done. Such is not possible with standard core drilling equipment, wherein rebar is often substantially if not fully severed prior to being detected.

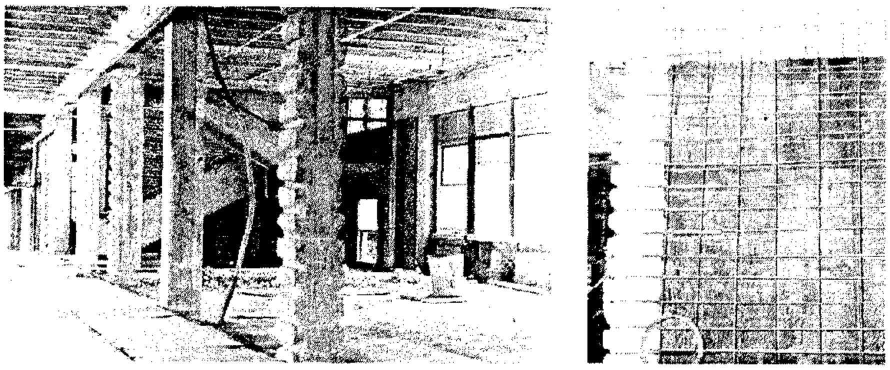

Figure 1 - Notching of existing columns to allow reinforcing for new infill shear walls to bypass. At wall termini, new steel is wrapped around existing column and returned into new construction.

Rebar Dowels

Where it is not possible to penetrate the element such as in corners or at termini, or where additional shear resistance is required, new reinforcing bars or dowels can be secured into drilled holes (Fig. 2).

Figure 2 - New rebar dowels epoxy cemented into drilled holes. Vertical bars at floor extend through floor to level below. Randomly spaced hooked bars are to provide additional shear resistance of existing and new section.

Drypack cement mortar, non-shrink cementitious grout sulphur, and epoxy resin materials have all been used for this purpose. The epoxy resin materials have proven most suitable on a number of large projects where a variety of different methods were evaluated (1, 2, 3, 4). This method requires a smaller hole into the existing construction, minimizing possible interference with existing reinforcing, as well as being more economical. Tests have shown that properly installed epoxy set dowels will retain their full yield capacity when embedded about ten times their diameter. Because increasing the embedment depth of epoxy set dowels entails only an infinitesimal amount of additional cost, it is practical and probably advisable to so do to at least fifteen bar diameters where thickness of the existing section permits. Field proof testing of grouted bars is frequently required, at a rate of from 10% to 50% of the total bars set. The frequency of such tests is often reduced, however, as the job progresses, if consistently satisfactory results are obtained. Proper performance requires that the holes be filled, preferably from the closed end outward, the bar then being pushed into the partially filled hole so that the resin material oozes out around it, insuring complete contact. The bar is usually twisted slightly as it is inserted in order to insure total bond.

The resin material can be injected with proportioning pump in-head mixing equipment or by hand caulking guns. In either case, the nozzle must be provided with a hose or tube of sufficient length to reach the bottom of the hole being filled. The installation of dowels in the horizontal or overhead locations is facilitated by covering the hole with masking tape. A split is then made in the tape through which the resin injection tube is inserted, followed by the bar, the tape acting as a barrier to prevent the material from running out. Somewhat thixotropic resin formulations are generally used for this work except where the holes are drilled in a downward position from the surface. In such cases, a low viscosity resin can be used and is simply poured into the hole followed by insertion of the bar. Optimal hole size is the smallest that can be readily drilled and yet enable insertion of the steel. Because of the creep potential of most epoxy formulations, hole sizes more than about 13 mm (1/2 in.) greater than the bar diameter should not be used.

The preferred drilling method for the holes is by rotary-percussion equipment, that continuously blows the drill cuttings out of the hole by means of compressed air impelled through hollow stem drill rods. The preferred drill bit encompasses a single chisel tooth which is continuously rotated during percussion. Such equipment results in a roughened hole that is relatively free of dust or other deleterious material resulting from drilling as the single tooth results in the largest possible cutting particles and they are immediately expelled from the hole by the continuous air circulation. Rotary drilling utilizing diamond core bits has been used in a limited amount of prior work. They should be avoided, however, as such bits require continuous circulation of water for cooling and removal of the drill cuttings, resulting in a wet hole. Whereas many epoxy formulations are compatible with wet surfaces, when moisture is entrapped within the confines of the hole, the exotherm heat of the resin often results in development of damaging vapor which significantly reduces the strength of the material. Additionally, the rotation of the diamonds tends to polish the sides of the hole which reduces bond potential. Electric impact hammers or other tools which do not provide for the immediate expulsion of the drill cuttings should also be avoided. Such implements result in pulverization of the cuttings into a fine dust that is difficult to remove and adversely affects the bond of the cementing material (1, 2).

WELDING

Welding of reinforcement is a frequently used expedient in repair and retrofit work. Prior to its consideration, however, it is important to determine that effective welds can be made. Historically, it was common practice to roll rebar from axel or rail steel; quality control was often lacking, and uniformity non-existent. Rebar found in old structures must be carefully evaluated in order to confirm its weldability. When joining new to existing steel, it is desirable to match as closely as possible the properties of the existing material. Unless there exists positive knowledge thereof, an analysis by a metallurgist should be made. Of particular importance is determination of the carbon content, which is readily made with a specimen of only a few grams. Such can be easily procured by filing. In the United States, welding of rebar is covered by the Structural Welding Code - Reinforcing Steel (AWS D1.4-79). Therein, if the weldability of the steel has not been predetermined by analysis, pre-heating of the bars to be welded to 260°C (500° F), and the use of low hydrogen electrodes is required.

Generally, full penetration butt welding is preferred, though lap welding is sometimes used. In all cases, due to the varying heat dissipating properties of the steel which is encased in concrete, and that which remains in the open, such welds will require close control of temperature. It is imperative to expose a minimum of 10 to 15 cm (4 to 6 in.) of steel prior to welding. Immediately upon completion, the weld area should be wrapped in asbestos to prevent rapid cooling. Careful attention to proper preparation as well as subsequent welding and controlled cooling cannot be overstressed. X-ray inspection of the completed welds is recommended, especially in the early stages of construction on a given project.

Gas Pressure Welding

A butt welding system which is widely used in Japan (6) involves holding the bars together in a special compressible jig, heating to a specified level with an oxyacetylene torch, and simultaneously applying pressure as facilitated by the jig. Conventionally, such work has employed manually held torches or burners, and manually actuated mechanical compressing jigs. The system is simple and economical, however, as with conventional welding, the quality of the weld depends largely upon the skill of the individual operator. As an improvement to the conventional system, the procedure has been fully automated within the last several years (7). The automated equipment is capable of welding bars of 25 to 51 mm (1 to 2 in.) in diameter. Extensive testing of bars utilizing the method have been performed, and welds of consistently high quality have been confirmed. The Gas Pressure Welding system requires steel of uniform known quality. It can be applied to steel complying with American standard, ASTM A 616-81 for billet steel, which is commonly used in the United States. The procedure has not yet found use in the United States to the writer's knowledge, however, it is available from at least one Japanese steel distributor with representation throughout the country. It offers obvious advantages over traditional welding procedures, especially in that excessive congestion can be substantially reduced by eliminating the customary laps.

CONCLUSIONS

Because of the need to supplement existing reinforcing in repair and retrofit work, often times with limited exposure of the existing steel and subject to restricted working space, special considerations are necessary. Proper preparations of notches or holes in existing construction to receive new steel is essential to insure proper performance of the repair. Welding is a useful method to join new to old steel, however, the properties of both must be determined and appropriate welding methods selected. The Gas Pressure Welding system commonly used in Japan offers great potential to application in repair and retrofit where the quality of the steel is adequate. Although these factors may appear to be relatively small details when compared to the overall project, rigorous attention to such details is essential to the optimal completion of any repair or retrofit project.

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Damage to Buildings from Urakawa-oki, Japan, Earthquake of March 21, 1982

Prompt Report

by

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1. Introduction

On Sunday, March 21, 1982, a destructive earthquake with magnitude of 7.3 occurred off the coast of Urakawa, Hidaka, Hokkaido, Japan. Although the effects were felt widely in north Japan, major damage was confined to Urakawa-cho and other few towns in Hidaka area and Sapporo City, capital of Hokkaido. Some of the present authors visited these area on March 24 and spent 5days to investigate the damage to buildings. This is a prompt report of the investigation.

2. Description of the Earthquake

Urakawa-oki Earthquake, 1982 occurred at 1132 JST, March 21, 1982 with Richter magnitude of 7.3. Its hypocenter was located at a point 42. 1°N latitude, 142.6°E longitude at a depth of 10km according to the announcement of Meteorological Agency of Japan. According to the observation by Local Observation Center for Earthquake Prediction, Hokkaido University, the hypocenter was located at the depth of 30km. The epicenter was located at a point apporaximately 20km west from the coast of Urakawa.

Ground motion were observed at Urakawa Weather Station and the records are shown in Fig.1. Although ground motions can not be known in detail because the major ground displacement exceeded the seismograms full scale, we can get a rough image of the ground motion from the records; for first 20 seconds the horizontal ground motion of EW direction is predominant and after that NS and up-down components become predominant for 25 and 10 seconds, respectively.

SMAC strong-motion accelerograms recorded the earthquake motion at Hiroo (epicentral distance 70km) and Sapporo (epicentral distance 150km). Maximum accelerations recorded there were as follows:

Hiroo (epicentral distance 70km)

EW comp.	200cm/sec.
NS	300
UD	70

Sapporo (epicentral distance 150km)

EW comp.	72.5cm/sec
NS	66
UD	30

3. Statistics of damage to buildings

As stated before, major damage caused by the earthquake was confined to few towns in Hidaka area and Sapporo. Table 1 and 2 shows the statistics of damage of buildings in those area. It will be noted that the effect of Urakawa-oki Earthquake on Hidaka area was quite minimal, considering it's magnitude and epicentral distance. This is considerd to be due to the following reason:

(1) Wood construction residential houses which formed about 90% of buildings in Hidaka area were provided enough resistance to the earthquake motion due to the following conditions:

a. Light weight roofs due to sheet metal roofings
b. Small opening in exterior walls
c. High rigidity and strength of continuous footings acquired by deep embedment (more than 60cm in Hidaka area) of footing to prevent frost heaving in winter.

(2) Towns in Hidaka area are located on peat deposits which are extremely soft ground containing much water. It's frequency response characteristics might be favourable for buildings to resist the strong earthquake.

4. Features of damage to buildings

4.1 Wood construction buildings

90% of buildings in Hidaka area are single family dwellings of wood construction. Althogh most of furnishings overturned and suffered severe damage, structures themselves in general performed very well in the earthquake.

The only major damage to wood construction was the inclination of the first story of tuo-storied shop-combined dwellings (Photo. 1 and 2). Those dwellings commonly had wide openings in the first stories of their facade which caused rigidity discontinuity between first and second stories and also large eccentricity in first stories. It has been recommended for carpenters to place bearing walls in those openings but Photo.3 shows that the recomendation was not necessarily effective.

Another damages to wood construction dwellings frequently observed were fall of wire-lath mortal wall finishing (Photo.4) and collapse of chimneys (Photo.5 and 6). Those damages were caused by the weak connection between wood frames and wire-lath or chimneys.

4.2 Reinforced concrete buildings

Reinforced concrete buildings in Hidaka area suffered more or less partial damages; shear failure of columns, shear cracks in wall or beams, damages to expansion joints, breakage of glazings, etc.

Fukushi Center Building Photo.7 shows the lateral buckling of a parapet wall of this three-story building.

Takasugi Department Store This was a three-story building with a reinforced concrete frame, two span by four. As shown in Photo.8 and 9, shear cracks occureed in the first- and second-story columns in the middle of the building. The second floor concrete slab connected to escalater was badly cracked.

Urakawa High School There observed few cracks in a column and a foundation beam near the evidence of ground settlement (Photo.10).

Red Cross Hospital This hospital was cosisted of several two- to four-story buildings connected by expansion joints. Many of medical facilities, medicine cabinets and furnitures overturned, which caused the stop of medical function of this hospital for three days. Damage to each of those buildings was quite minimal but expansion joints were badly damaged. Photo.11 shows shear cracks on second-story walls and Photo.12 shows a power supply unit of 800kg weight for a X-ray facility overturned in the earthquake.

4.3 Reinforced Concrete Block Masonry Buildings

About 10% of residential houses are of reinforced concrete block masonry construction. All of them with one exception performed very well in the earthquake. Photo.13 to 15 show the only one damage to residential house in Mitsuishi-cho area. The wall layout of this residence contained no special problem, but horizontal reinforcements were not sufficiently anchored into intersecting walls.

4.4 Steel Construction Buildings

Many of steel construction buildings also performed very well in the earthquake in spite that some diagonal bracings broke or buckled as shown in Photo.16 and 17. In Sapporo a three story office building (Nissho Building) suffered severe damage to exterior walls of ALC panels (Photo.18 and 19). This was caused by two reason; 1) breakage of diagonal bracings caused by poor welding work of their connections, 2) no clearance between upper and lower story ALC panels (Photo.20).

Table 1 Statistics of damage to buildings

	Administrative Districts							
	Shizunai		Mitsuishi		Urakawa		Total	
	Quantity	Value (1000yen)	Quantity	Value (1000yen)	Quantity	Value (1000yen)	Quantity	Value (1000yen)
Population	24833	-	7424	-	20213	-	52470	-
Household	7358	-	2013	-	6061	-	15432	-
1. Dead/Missings	0	-	0	-	0	-	0	-
2. Injured	35	-	30	-	94	-	159	-
3. Households suffering damage	31	-	42	-	150	-	223	-
4. Buildings wholly destroyed								
Residential	3	4110	3	32401	5	51048	11	87559
Non-residential	0	0	3	1800	6	48226	9	50026
5. Buildings partially destroyed								
Residential	2	8340	5	13000	9	40550	16	61890
Non-residential	0	0	4	1100	4	10599	8	11699
6. Partially damaged residential build	15	7267	31	12340	124	39227	170	58834

Table 2 Damage to buildings in Sapporo

Settlement of residential houses	8
Partially destroyed office build	1
Damage to chimmeys	
cracked	8
collapsed	7

Fig. 1 Seismograph at Urakawa Weather Station

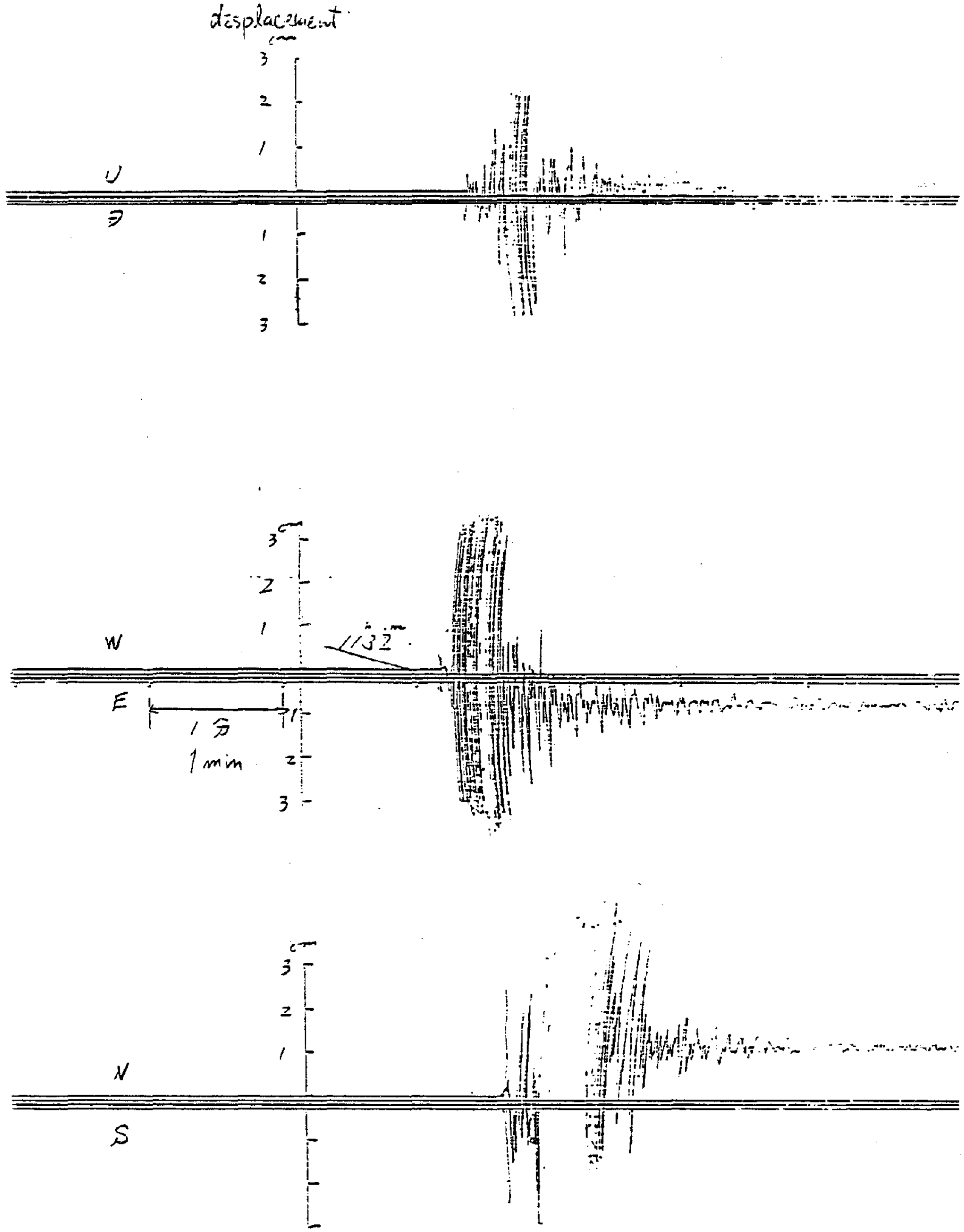

図2·2 気象庁1倍計の本震記録（浦河測候所 昭和57年3月21日11時32分

AN OVERVIEW OF THE STATE-OF-THE ART IN SEISMIC STRENGTHENING OF EXISTING REINFORCED CONCRETE BUILDINGS IN JAPAN

by

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SYNOPSIS

This paper describes an overview of techniques whereby current knowledge developed in Japan can be used to give increased resistance to existing substandard reinforced concrete buildings. First the general design and construction procedures are described and then brief reviews of some of the relevant experimental studies are given. Some application of strengthening techniques to existing buildings will also be described.

INTRODUCTION

Backgrounds of seismic strengthening of existing reinforced concrete buildings in Japan will be briefly reviewed in the following. Current activities related to seismic evaluation and strengthening of existing buildings is illustrated in Fig.1.

A number of reinforced concrete buildings, damaged by recent severe earthquakes, required extensive repair and also strengthening [1-4]. The Tokachi-oki Earthquake of 1968 heavily damaged a large number of low rise buildings. Some of these were strengthened by the addition of structural walls. These buildings are still in use. Because of the lack of guidelines, the design as well as the construction for strengthening was based on engineering judgement alone. This was practically the first experience for Japanese engineers to extensively strengthen existing buildings against severe earthquakes. The destructive 1978 Miyagiken-oki Earthquake was also followed by the strengthening of a number of buildings. However, in this case materials and techniques for construction were specifically selected and the design was based on experimental or analytical investigations or on guidelines, where these were available [5-7].

Current studies have indicated that there is a wide scatter in the level of seismic resistance of existing buildings [8-13]. It was found that a considerable number of low to medium rise buildings, designed and constructed in accordance with previous building codes, may need strengthening. Consequently a number of public and private buildings, considered hazardous, were strengthened or rebuilt.

It is intended to provide not only increased strength, so as to prevent collapse, but also increased stiffness to give increased protection against damage to non-structural building components. To establish guidelines for design and construction, several experimental studies, relevant to the seismic strengthening of existing structures, have been conducted [14-24]. However, test data, accumulated over some 10 years, have not been systematically reviewed as yet.

The necessity for strengthening of hazardous buildings was recognised for some time, and as a consequence an advisory committee for the Japanese Ministry of Construction prepared design guidelines in 1977 [25]. These design guidelines were intended to be used in conjunction with the method of evaluation of seismic safety of existing buildings, proposed by the same committee [26]. This method has been described in some detail in references 27 and 28. In a number of cases these guidelines have already been used in Japan.

Several projects on existing buildings have currently been in progress. In the Shizuoka Prefecture which is facing the potential Great Tokai Earthquake, a large number of buildings have been checked up since 1979 by the municipal officials and local engineers [11]. Substandard buildings were and are going to be strengthened in accordance with the level of their seismic resistance. The Japanese Ministry of Construction started in 1981 the five years research project in order to develope inspection, assessment and repair and strengthening methods for buildings and structures damaged by earthquakes [29]. Also in 1981, the Japan Concrete Institute organized a research committee for existing civil and building structures. Past experimental data and available examples of strengthening will be systematically reviewed by engineers from different fields and organizations.

DESIGN AND CONSTRUCTION

General Principles

The approach to the design and construction for the strengthening of hazardous buildings in Japan is summarized in the flow chart of Fig.2. Detailed discussions are held before the design commences and the construction technique is chosen. The results of the seismic evaluation and in certain cases also those of field investigations are required. Laboratory tests may be necessary to provide additional information for design and construction.

The aims of the strengthening are to provide:
(1) Increased strength with respect to lateral loading, or
(2) Increased ductility or
(3) A proper combination of these two features.
These concepts are illustrated in Fig.3. The combination of strength and ductility involve the proper balance between strength and stiffness.

To provide increased strength is considered as being the most promising approach in the strengthening of low to medium rise buildings. Even if ductility is provided, increased strength is expected to reduce the magnitudes of inelastic displacements. For ductile structures it is considered to be particularly important to reduce eccentricities resulting often from the irregular distribution of stiffness within a storey or throughout the entire structure. The separation of spandrel walls from columns may be effective to eliminate "captive columns".

Construction Techniques

Typical strengthening methods considered in Japan are assembled in the chart of Fig.3. Examples of construction techniques to meet both, the increased strength and increased ductility, criteria for strengthening are given in Fig.4. Generally new elements may be added to existing structures to give increased strength, or existing framing elements may be reinforced with new materials to improve their ductility. Infilled walls and wing walls, extensively used in Japan and shown in Figs.4(a) and (b), are cast in situ or precast wall elements which are attached to frames or to beams, as appropriate.

For the systems shown in Fig.4(a), (b) and (d), cast in situ or precast concrete is commonly used with the various connection techniques that are listed in Fig.3. Typical details for such connections are given in Figs.5 and 6 where additional information required in the design is also given. Careful attention must be paid to connections, because they will strongly affect the behaviour of the strengthened structure, as well as for the placing of concrete on the site. High pressure pumping of the fresh concrete or nonshrink material may be necessary to avoid the formation of cavities between the new and the existing elements. Detailing of bracing elements should be such as to avoid stress concentrations.

In the process of increasing the ductility of existing columns, such as shown in Fig.4(e), (f) and (g), one of the aims is to increase their shear strength. This is achieved by the wrapping techniques shown in these illustrations. A narrow gap at the ends of the encasement is provided to avoid the undesired increase of the flexural strength of the member at that section.

Design Procedures

The safety of strengthened building is assessed with the recently introduced Japanese evaluation procedure for existing reinforced concrete buildings [26]. If they are more detailed, other procedures are also permissible. In the evaluation procedure above, the judgement of safety of a building can be made based upon the experience of the damage to buildings due to recent earthquakes, such as 1968 Tokachi-oki Earthquake, as shown in Fig.7 [28]. It is suggested by the figure that the value of I_S-index of 0.6 will be the border between damaged and undamaged buildings experienced 25 - 30%g level of ground motion. The index indicates the ultimate horizontal strength of the building or equivalent strength when the ductile behavior is expected.

The guidelines [25] give specific calculation techniques for infilled walls, wing walls and encased columns. These calculation procedures are based on tests. As an example, the design strength of an infilled wall assembly of Fig.5(d) is given as the smaller of either:

(1) The total shear strength of the wall panel and both columns, treated as independent elements or

(2) The total shear strength of one column and the connection to the wall

along the beam and the punching (sliding) shear strength of the other column.

These procedures assume that failure will occur either in the wall panel or along the connections where shear is being introduced to the infill wall. The strength of the connections is evaluated either from theoretical considerations or from empirical equations applicable to the techniques shown in Fig.5(a) to (c). The punching or sliding shear strength of the column is given in terms of the principal tensile stress in the concrete.

The contribution to flexural and shear strength by a column strengthened by wing walls, as shown in Fig.6(a), is estimated to be 80% of that of a column with identical properties but cast monolithically with the wing walls. In the case of wing walls with dowel connections, an idealized truss system is used to model load transfer, as shown in Fig.6(c).

In the case of encased columns of the type shown in Fig.3(e), (f) and (g), the shear and flexural strength is evaluated as for ordinary monolithic reinforced concrete columns, using the increased dimensions as well as the contribution of the added steel elements.

For other strengthening methods, evaluation by testing is recommended.

Selection of Construction Methods

The selection of construction methods should be based on the overall considerations of the work involved on the site, weights and conveyance of elements to be handled, cost and also on the structural characteristics relevant to each alternative of structural solution. Table 1 illustrates these aspects. It is based on the author's experimental study of one-bay, one-storey simple frames which were strengthened by five different techniques [18]. The quantities in brackets in the table represent normalized values in terms of the infilled concrete wall construction which was taken as unity.

For this type of structure it was found, as Table 1 indicates, that reinforced concrete construction has merits in terms of cost, strength and stiffness while it is penalized for site work and conveyance of components. In general steel bracing was found to offer advantages in conveyance, weight and ductility but it was the most expensive solution.

RESEARCH

The number of strengthened structures that have been examined experimentally served as a background in formulating the guidelines for design [25]. Since the proposal of the guidelines, further experimental studies have been conducted.

The earliest tests, performed several years after 1968 Tokachi-oki Earthquake, were aiming at the improvement of ductility in columns [14, 23] by the techniques shown in Fig.4, and at the boosting of the strength of frames by the addition of precast and cast in situ walls [14-16]. Subsequently one storey infilled frames with various connection details [14, 15] and bracing systems [17, 18] were examined. Currently, three storey frames, strengthened by infilling and bracing, were also tested [30]. Tests were also conducted for infilled walls [19-22] and reinforced columns [24]. As an example of these studies, the author's test of strengthened frames by various techniques is shown in Fig.8. The scale of test frames was 1/3 the real size.

The test data of more than 100 strengthened frames and 40 columns have been accumulated up to now. Most frames were one-bay and one-storey, and had framing elements provided poor web reinforcement. Their scale was from 1/10 up to 1/2 the real size. Many of the frames were strengthened by cast-in-situ concrete walls providing different detailing of shear transfer. The rest used precast panels, concrete blocks, steel plates or steel braces. In the early tests, about 20 columns strengthened by wing walls were examined. Wrapping techniques such as those shown in Fig.4 were also selected for other columns by using different reinforcing elements.

A brief review of the findings of some of these studies is given in the following.

Infilled Concrete Walls

In a series of tests with three types of infilled walls [14, 15] it was found that:

(1) These provided significant increase in strength.

(2) Dowels connecting wall to frames failed simultaneously at the threaded shank by shear.

(3) It was effective to provide gaps along columns to allow walls to behave in a ductile manner.

(4) Chipped shear keys provided as good a shear connection as monolithic construction.

When adequate connection, placed continuously around all boundary members, were provided, infill frames exhibited the same strength as monolithic walls with identical boundary elements [18]. Recently tested multiple precast infill wall panels indicated good ductility properties [17], but, as expected, significantly less strength was attained. The predominance of bending behaviour in three storey infilled frames was observed in contrast to shear dominance that controlled the response of one storey infilled structures [30].

Using this data, the author proposed the following guidelines with respect to the strength of infill panels with dowel connections (Fig.9, [18]):

(1) When the required strength of a strengthened structure is more than 60% of that obtainable with an identical monolithic wall, Q_w, or more than 0.6 $\sqrt{f'_c}$ MPa, in terms of concrete shear stress, dowels should be designed to possess a shear stress equivalent to 1.0 MPa.

(2) Walls without any connection to a boundary frame may develop a strength corresponding to 0.4 Q_w or 0.3 $\sqrt{f'_c}$ MPa.

(3) Walls connected to beams but not to columns could be expected to develop strength up to 0.6 Q_w or 0.6 $\sqrt{f'_c}$ MPa in terms of shear stress.

For dowels to connect infilled panel with existing frame, mechanical anchors developed by wedge action into the existing concrete are generally used. When the screw part of dowels, however, was located on the shear plane, dowels failed simultaneously at the threaded shank by shear [15]. The joints, which used the improved anchor where the screw part of dowels was located inside the panel apart from the shear plane, demonstrated ductile behaviors [20]. The adhesive anchors to connect dowels inside the existing concrete by epoxy grouting were also tested [5, 15, 22]. Practically, there was no significant difference in the behavior of these two types of anchor.

Braced Frames

Available data on the response of braced frames is as yet limited. X, K and diamond shaped bracing systems were studied [17, 18]. These indicated moderate increases in strength but adequate ductility and ability to dissipate energy. The studies indicated that connection details require careful attention as they might strongly influence the overall hysteretic response.

Wing Wall Construction

The effect of small walls placed adjacent to existing columns or placed separately in the frames was also studied [14, 16]. Cast in place wall additions provided almost as much strength as identical monolithic construction. The addition of precast units resulted in less strength but it produced more ductility.

Reinforced Columns

The types of strengthening techniques shown in Fig.4 were examined [14, 15, 23, 24] experimentally. Both the strength and ductility of poor columns was significantly increased by these strengthening methods. While welded wire fabric wrapping resulted in considerable increase in ductility, the use of steel straps prevented shear failures and delayed the crushing of the concrete.

General Effects of Strengthening

Typical load-displacement relationships for the structures studied are presented in Fig.10. These are only qualitative indications of the order of strength and ductility that might be attained using different strengthening techniques. The lateral (shear) force, required to develop the full strength of a monolithic reinforced concrete wall with boundary elements, is denoted as Q_W, while that of the original frame or a column is given as Q_F and Q_C respectively. It is seen that, when adequate shear connectors are provided, frames strengthened by infilled wall panels can develop strength of the order of 0.6 Q_W or 3.5 Q_F respectively. Steel braces and multiple precast concrete panels gave less strength increase but resulted in improved ductility. Wing walls attached to columns have approximately doubled the strength of the original column while giving considerably increased ductility.

Figure 11 shows the dramatic improvement attained by strengthening of a column using welded wire fabric wrapping and mortar [14, 15]. Fig.11(a) shows the brittle failure of this type of short column that has been extensively used in Japanese construction. Displacement ductilities larger than 6 could be attained with the strengthening technique employed.

Figure 12 compares the observed response of strengthened three storey and one storey frames [17, 30]. As stated earlier because of the dominance of bending effects, the three storey frames exhibited larger ductility, while the one storey frames suffered brittle shear failures.

STRENGTHENING OF DAMAGED BUILDINGS

More than ten reinforced concrete buildings suffered very severe damage during the 1978, June 12th, Miyagiken-oki Earthquake (M = 7.4) in and around the city of Sendai. An earlier earthquake on February 20 of the same year (M = 6.5) also damaged several buildings in Northern Japan. Although some of these buildings were demolished and subsequently rebuilt, many were repaired and also strengthened.

Some examples of strengthening of damaged buildings are briefly described in the following.

School Building "A" [6]

A five storey college building, shown in Fig.13, and Fig.16(a) to (d) suffered severe damage during the June 1978 earthquake. Particularly "captive columns", such as seen in Fig.16(b), were affected. Severely damaged columns were replaced with new concrete and additional reinforcement has also been provided. Overall strength with respect to lateral loadings was increased by the addition of infilled walls and also by the thickening of existing walls. In the long direction of the building, cross bracing was added to the framing, as seen in Fig.16(c) and (d). Bracing members were connected at every floor to existing exterior beams by means of steel plates.

An experimental study was also undertaken to investigate the behaviour of these braces. Spandrel walls along the exterior of the building were drilled to reduce their interaction with columns. Microtremor measurements indicated that the reconstruction restored the stiffness of the building to almost its pre-earthquake value. The lateral load carrying capacity of the building in the longitudinal direction of the plan was increased to approximately 1.8 times the strength before the earthquake.

School Building "B" [5]

During the June 1978 earthquake three buildings of this school suffered moderate to severe damage, as shown in Fig.14 and Fig.16(e) and (f). Again "captive columns" at the North side were affected mainly by seismic actions in the East-West direction. Severely damaged columns were replaced as in the other School building, while those with lesser damage were repaired with epoxy or the spalled concrete cover was replaced. Major strengthening was achieved by adding concrete walls to the longitudinal frames in the North, to resist East-West seismic actions. Every second bay in the first storey and every fourth bay in the upper storeys, was infilled as seen in Fig.16(f). Epoxied dowels were used to connect new walls to existing frame members. This resulted in approximately doubling the lateral load resistance of the school blocks in the long direction.

City Hall [7, 31]

This simple two storey structure, shown in Fig.15, suffered moderate to severe damage during the earlier February 1978 earthquake. Details of the damage are recorded in Fig.15(a) and overall views can be seen in Figs. 16(g) and (h). Construction for repair and strengthening was under way when the June 1978 earthquake struck. (Fig.16(i)). The damage during this earthquake was only minor even though the ground shaking in the area was presumably much more intense than during the February event. It appears that the new walls installed, shown in Fig.15(b), contributed significantly to the increased strength of the building. This is one of the very few structures in which the effects of strengthening were put to test by a new earthquake. The minor damage experienced give considerable encouragement for further application of this technique.

STRENGTHENING OF UNDAMAGED BUILDINGS

A considerable number of undamaged, substandard buildings are supposed to be strengthened by various techniques over some 10 years since the Tokachi-oki Earthquake of 1968. Very few detailed data of these undertakings, however, was available to enable a reliable survey to be made.

Some data are currently becoming open in accordance with the progress of relevant projects on existing buildings and the increased number of practices. The data of some one hundred public and private buildings have been accumulated, up to now, in the research committee organized by the Japan Concrete Institute. Most of them were those of 3 or 4-storey school buildings. The rest included those of government buildings, office buildings and residential buildings. In most cases, infilled walls or infilled walls and wing walls were used. Isolated columns were also reinforced in some cases with wrapping techniques. Several buildings needed strengthening of foundations to support the increased weight resulted from strengthening or to provide new structural elements.

Some examples of strengthening of undamaged buildings are briefly described in the following.

Seismic Evaluation and Strengthening of Buildings - An Example of a Prefecture [11]

In the Shizuoka Prefecture which is facing the potential Great Tokai Earthquake, about 500 important buildings, out of 1,400 reinforced concrete buildings in the area, have been evaluated since 1979 with respect to their seismic resistance [11]. The evaluation was based on own standards, extended from the method of reference 26, or higher level procedures in some cases. According to the statistics as of the first two years, 22 percent of 180 buildings needed urgent strengthening (class D) and 6 percent needed rebuilding (class E), as shown in Table 2[11]. Note that most of the young buildings designed after the revision of the structural design code of reinforced concrete in 1971 [43] were ranked higher classes than D. Some of the low class buildings were already strengthened and the rest is planned to be strengthened in accordance with their seismic resistance and importance.

A Residential Building [32]

A four-storey residential building shown in Fig.17 had a "soft storey" and the indices of structure I_S of the storey (1st storey) were much lower than the border value (Fig.18). The strengthening was designed to improve both the strength and stiffness of the longitudinal frames and to improve the ductility of all the columns, at the soft storey, respectively. A cast-in-situ concrete wall was placed to longitudinal direction. Wing walls were also used not so as to disturb openings. All the columns were reinforced by wrapping techniques using steel plates at ends and welded wire fabrics through the length except narrow gaps at the ends. The non-linear dynamic response analysis indicated that the expected displacement will be small or tolerable, as a result of strengthening, during ground motions of the level of 0.45g. The indices I_S were doubly increased.

School Buildings and A City Hall [33, 34]

Two four storey school buildings were strengthened in the longitudinal direction by infilled concrete walls [34]. Subsequent forced vibration tests indicated some 25% increase in frequencies in comparison with those measured before the strengthening.

Both, elastic analysis and the seismic evaluation procedure, predicted that there would be local damage to columns in the three storey city hall buildings [33]. This was to be expected because of torsional effects. Because of the considerably lower stiffness of the upper storeys, the penthouse was expected to overturn. Additional walls were constructed to improve the modes of vibration and to rectify the eccentricity of stiffness.

CONCLUSIONS

Because of lack of data, insufficient experience with and understanding of seismic phenomena, a great deal of work is yet to be done in the area of strengthening of reinforced concrete buildings. Some of the problems to be solved before improved guidelines for seismic strengthening can be compiled, are as follows:

(1) Analytical and experimental approaches should be used to assess the effect of strengthening on the overall seismic behaviour of buildings. Further experimental verification of the behaviour of strengthened sub-assemblages is required. Workmanship and the detailing of connections greatly affects the response of strengthened structures. These aspects are difficult to model and for this reason it is desirable to test large if not full scale specimens.

(2) Existing test data must be evaluated more systematically to obtain reliable global information with respect to energy dissipation capacity, displacement capacity, stiffness deterioration and potential strength to resist earthquake forces.

(3) Additional information is required with respect to infilling techniques, the use of precast and bracing elements and methods of connections to existing structural systems.

(4) It is desirable that data for design and construction for seismic strengthening be integrated and compiled for the benefit of potential users. Engineering approach to strengthening will still have to rely on experience so gained and on judgement.

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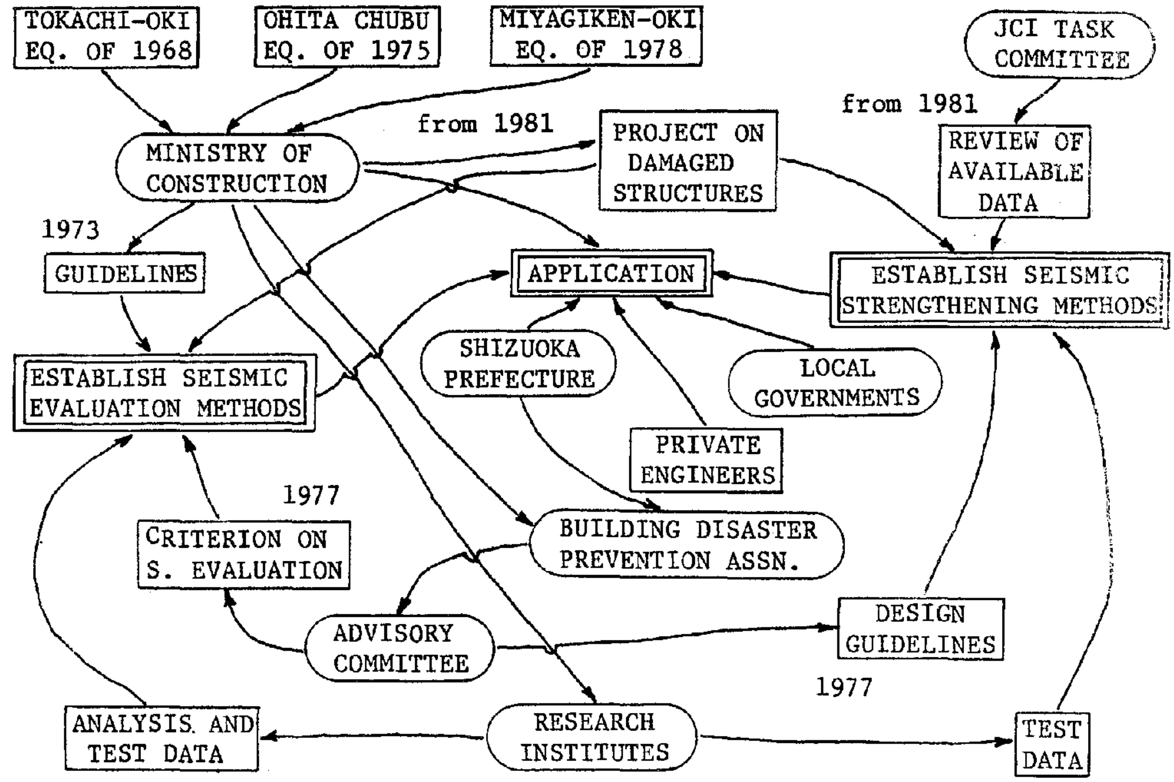


FIG. 1 CURRENT ACTIVITIES RELATED TO SEISMIC EVALUATION AND STRENGTHENING OF EXISTING R/C BUILDINGS

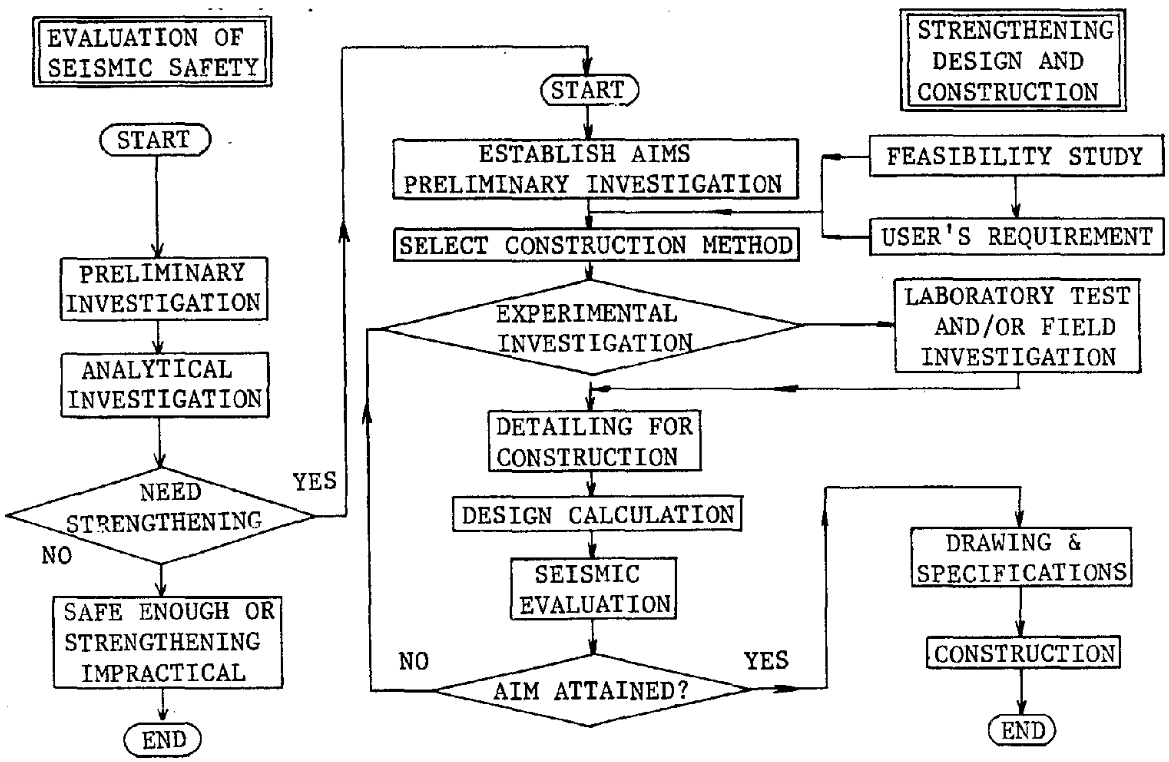


FIG. 2 FLOW CHART OF DESIGN AND CONSTRUCTION OF SEISMIC STRENGTHENING

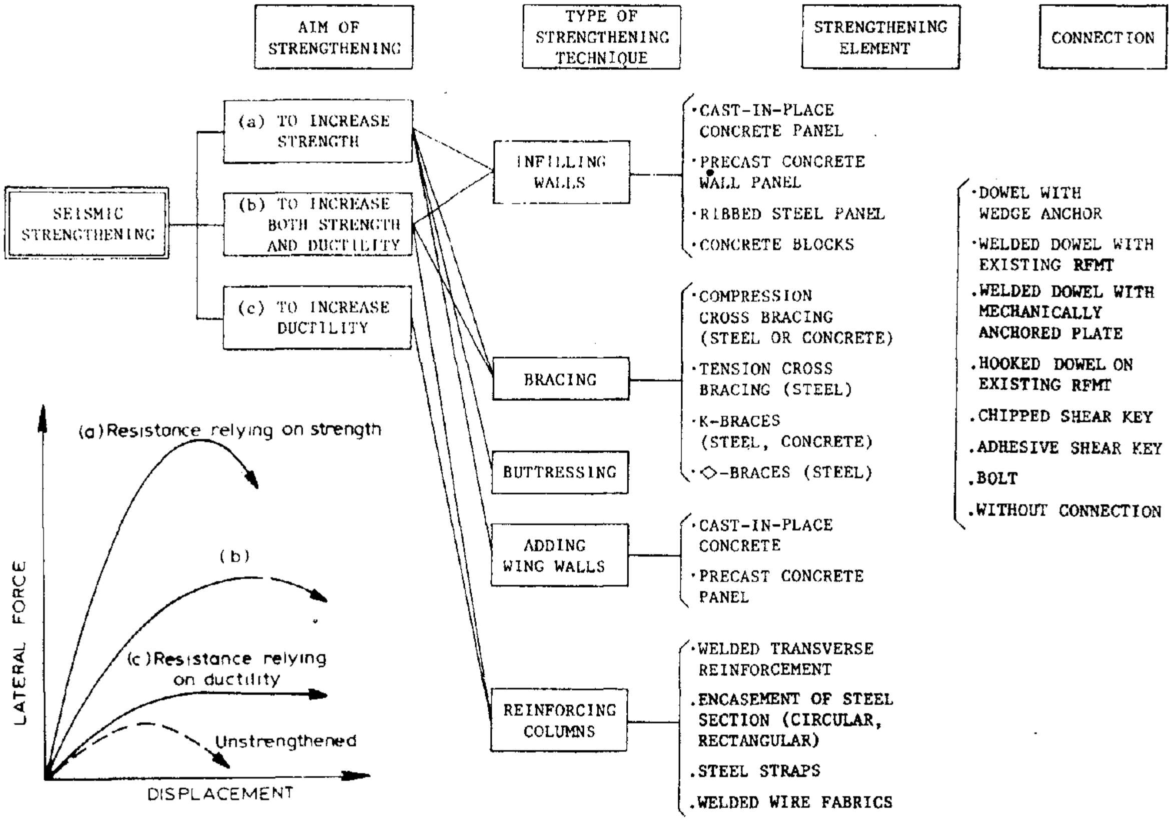
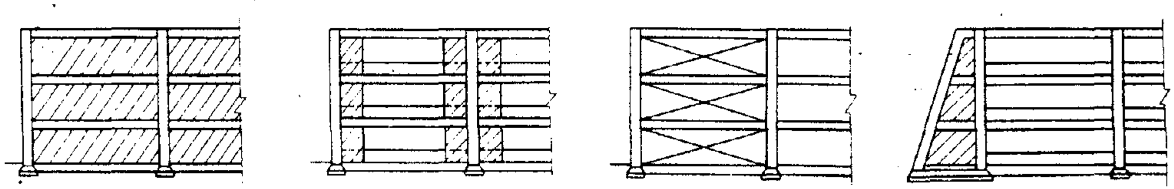


FIG. 3 CONCEPT OF SEISMIC STRENGTHENING AND TYPICAL STRENGTHENING METHODS



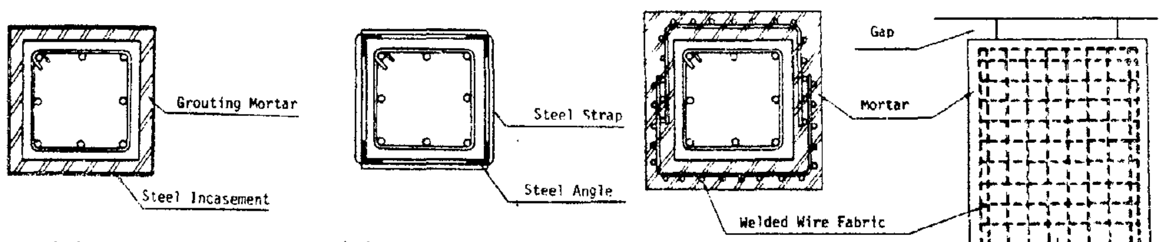
(a) BY INFILLED WALLS

(b) BY WING WALLS

(c) BY BRACES

(d) BY BUTRESSES

(1) TO INCREASE STRENGTH



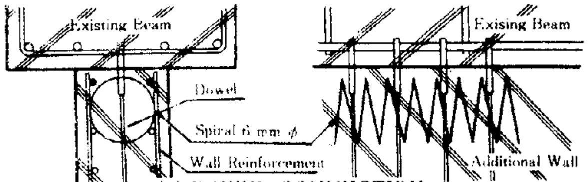
(e) BY STEEL INCASEMENT

(f) BY STEEL STRAPS AND ANGLES

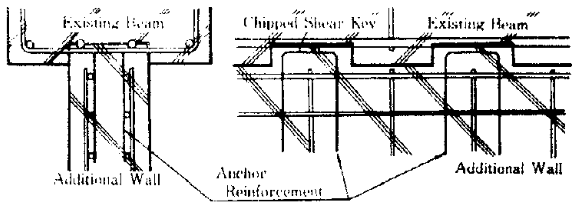
(g) BY WELDED WIRE FABRICS

(2) TO IMPROVE DUCTILITY

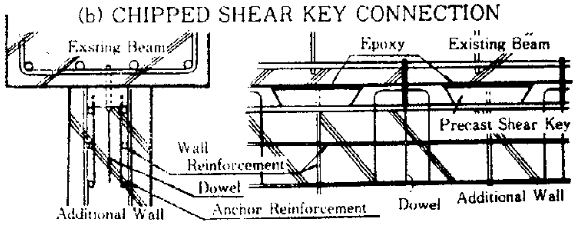
FIG. 4 TYPICAL CONSTRUCTION TECHNIQUES FOR SEISMIC STRENGTHENING



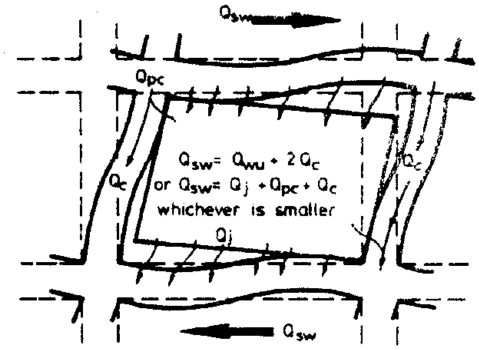
(a) DOWEL CONNECTION



(b) CHIPPED SHEAR KEY CONNECTION



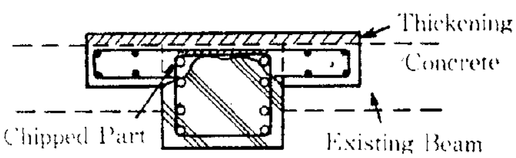
(c) PRECAST SHEAR KEY CONNECTION



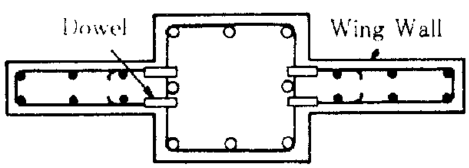
Q_{wu} = Shear strength of wall panel
Q_{sw} = Strength of an infilled wall
Q_{pc} = Punching shear strength at top of column
Q_j = Total strength of connections along a beam
Q_c = Strength of a column

(d) DESIGN OF INFILLED WALL

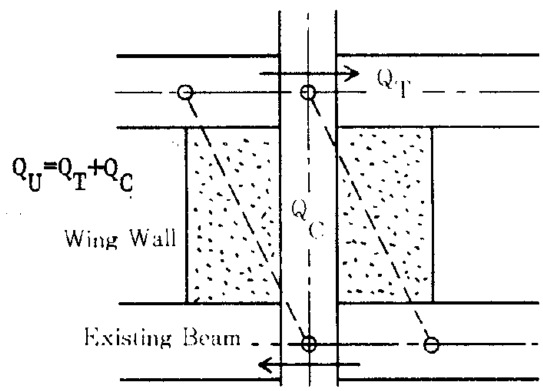
FIG. 5 DESIGN AND CONSTRUCTION OF INFILLED WALLS



(a) MONOLITHICALLY CAST WING WALLS

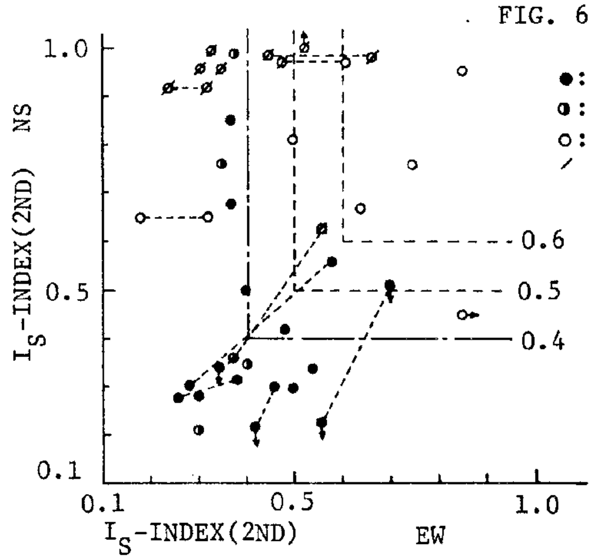


(b) WING WALLS WITH DOWEL CONNECTIONS



(c) DESIGN OF WING WALLS WITH DOWEL CONNECTIONS

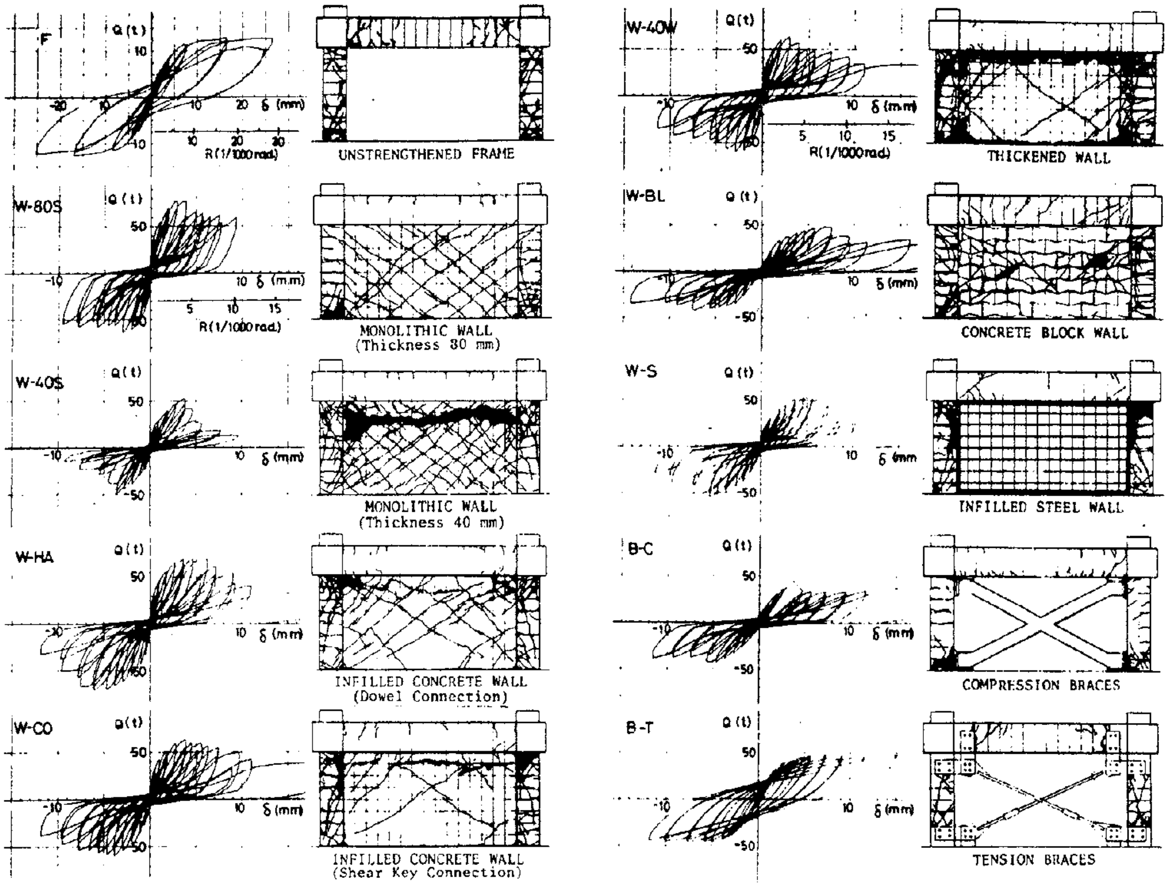
FIG. 6 DESIGN AND CONSTRUCTION OF WING WALLS



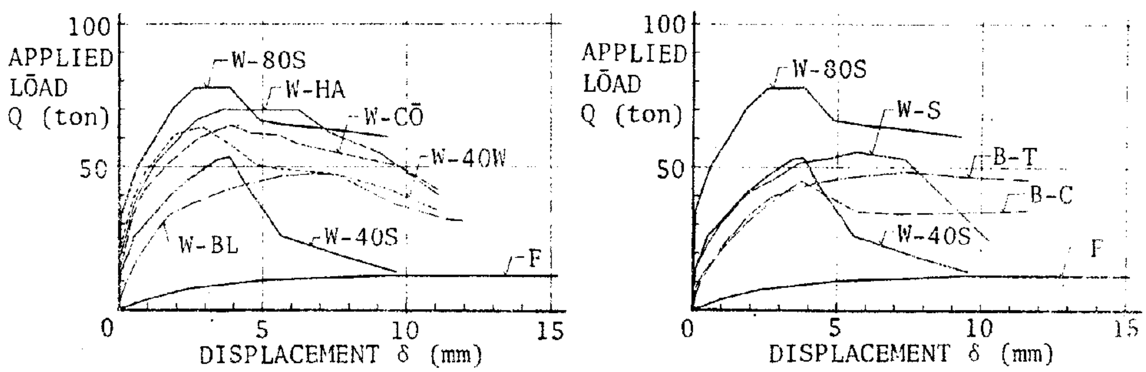
●: SEVERE DAMAGE
◐: MODERATE DAMAGE
○: NO DAMAGE
DUE TO 1968 TOKACHI-OKI AND 1978 MIYAGI-KENOKI EARTHQUAKES
/ 1978 IZUOSHIMA KINKAI EARTHQUAKE

○--○ REDUCTION DUE TO EXTREMELY SHORT COLUMNS
↓ REDUCTION DUE TO ECCENTRICITY

FIG. 7 SEISMIC EVALUATION OF DAMAGED BUILDINGS DUE TO RECENT EARTHQUAKES[28]



(a) HYSTERESIS CURVES AND FAILURE MODES



(b) ENVELOPES OF HYSTERESIS CURVES

FIG. 8 AUTHOR'S TESTS OF STRENGTHENED FRAMES BY VARIOUS TECHNIQUES[18]

TABLE 1 FEASIBILITY STUDY OF STRENGTHENED ONE-STOREY FRAMES

	Construction				Cost of Structures	Structural Capacity		
	Workability	Conveyance	Curing	Weight		Stiffness	Strength	Ductility
Infilled Concrete Wall	much work necessary	difficult conveyance of concrete	necessary	heavy (1.00)	cheap (1.00)	high (1.00)	high (1.00)	low (1.00)
Infilled Concrete Block Wall	easy work, few workman	easy	local curing necessary	heavy (1.00)	slightly expensive (1.61)	low (0.30)	low (0.30)	low (1.13)
Infilled Steel Wall	easy work, but accuracy needed	easy	local curing necessary	light (0.39)	expensive (2.41)	moderate (0.49)	moderate (0.78)	low (0.96)
Compres-sion Braces	simple connection, easy work	easy	local curing necessary	light (0.39)	slightly expensive (1.47)	low (0.27)	low (0.63)	high (1.7)
Tension Braces	easy work but accuracy needed	easy	necessary for chipped slabs	light (0.44)	expensive (2.93)	low (0.24)	low (0.67)	high (1.7)

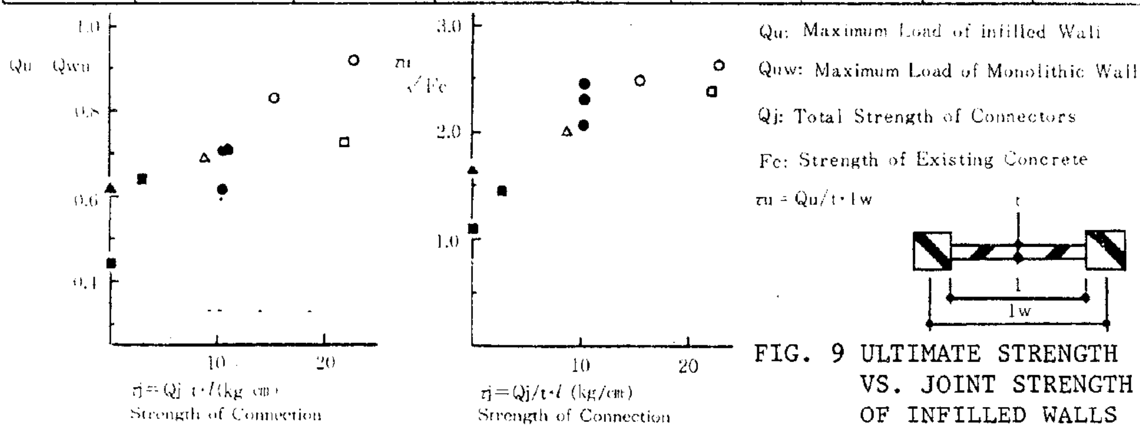


FIG. 9 ULTIMATE STRENGTH VS. JOINT STRENGTH OF INFILLED WALLS

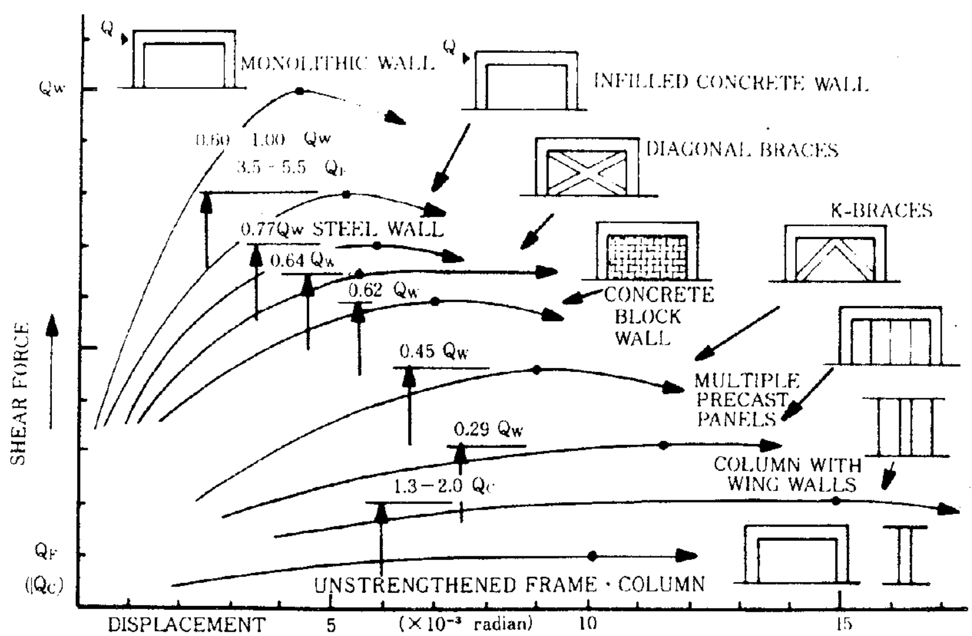
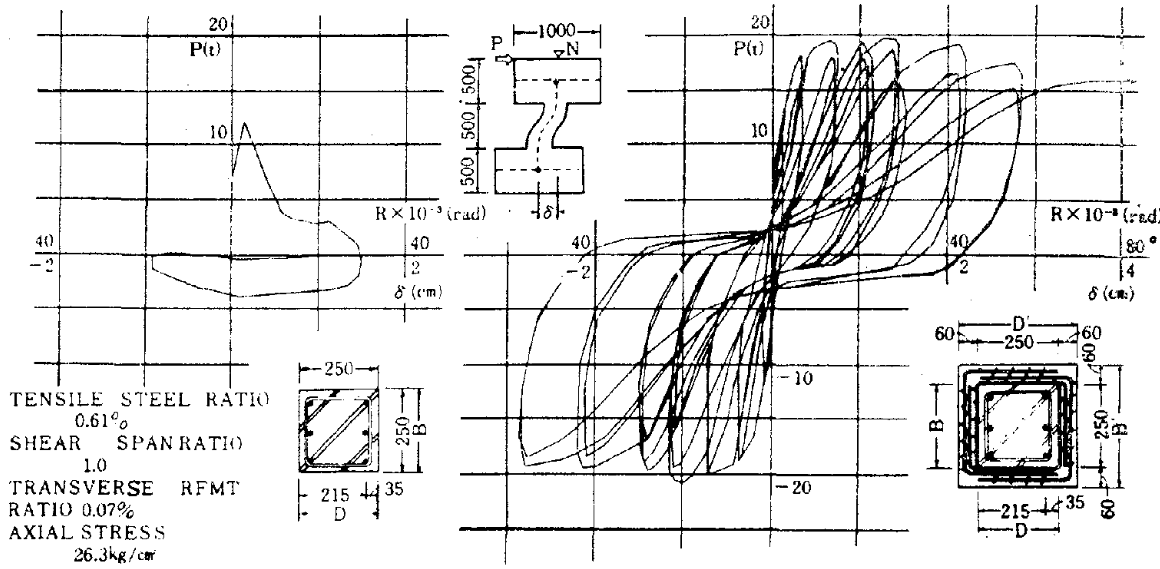


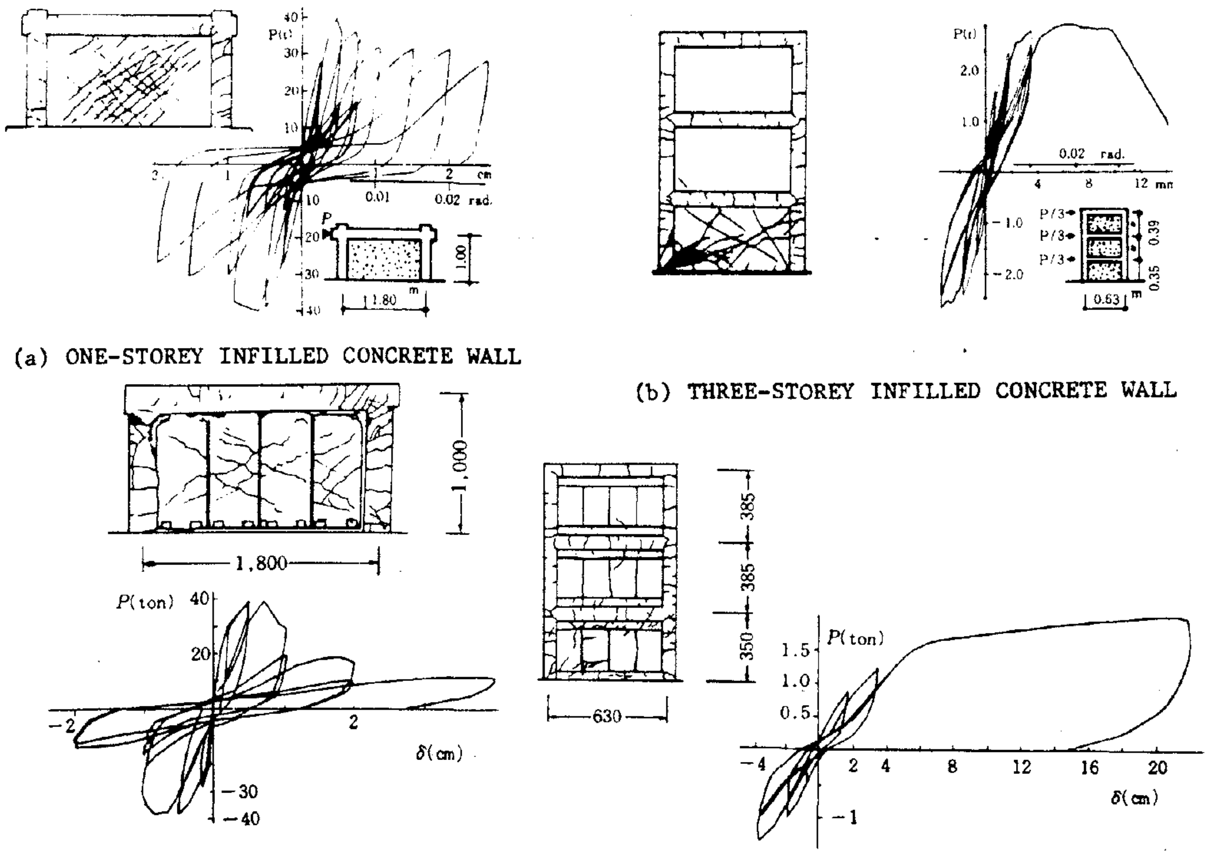
FIG. 10 TYPICAL LOAD-DISPLACEMENT RELATIONSHIPS FOR DIFFERENT STRENGTHENING TECHNIQUES



(a) UNSTRENGTHENED

(b) STRENGTHENED BY ADDITIONAL MORTAR AND WELDED WIRE FABRICS

FIG. 11 STRENGTHENING OF COLUMNS



(a) ONE-STOREY INFILLED CONCRETE WALL

(b) THREE-STOREY INFILLED CONCRETE WALL

(c) ONE-STOREY MULTIPLE PRECAST CONCRETE PANELS

(d) THREE-STOREY MULTIPLE PRECAST CONCRETE PANELS

FIG. 12 STRENGTHENING OF MULTI-STOREY FRAMES

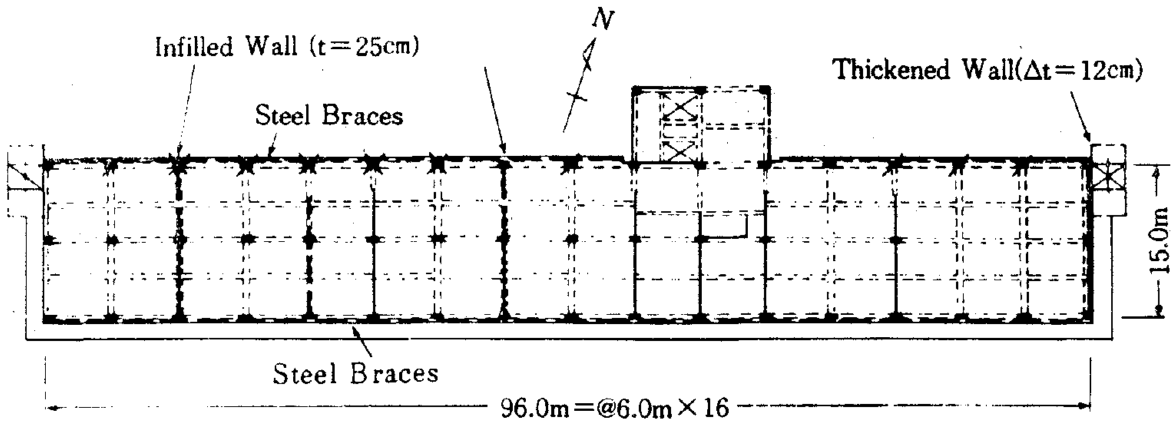


FIG. 13 STRENGTHENING OF THE DAMAGED SCHOOL BUILDING "A"

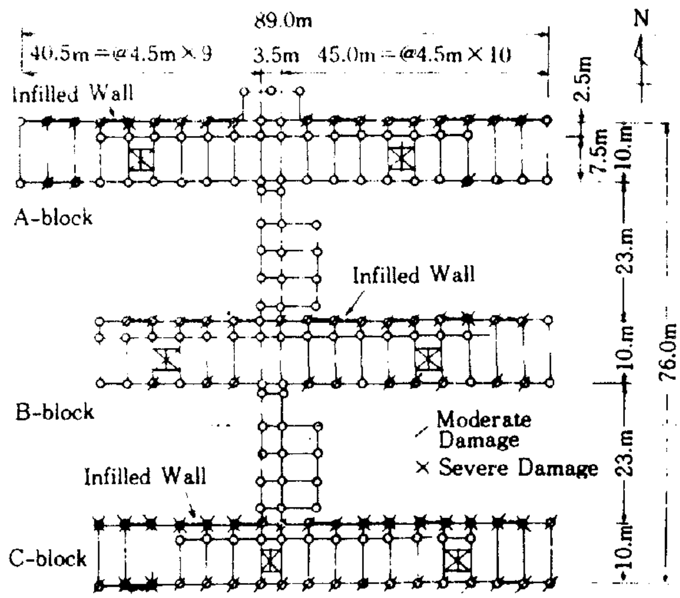
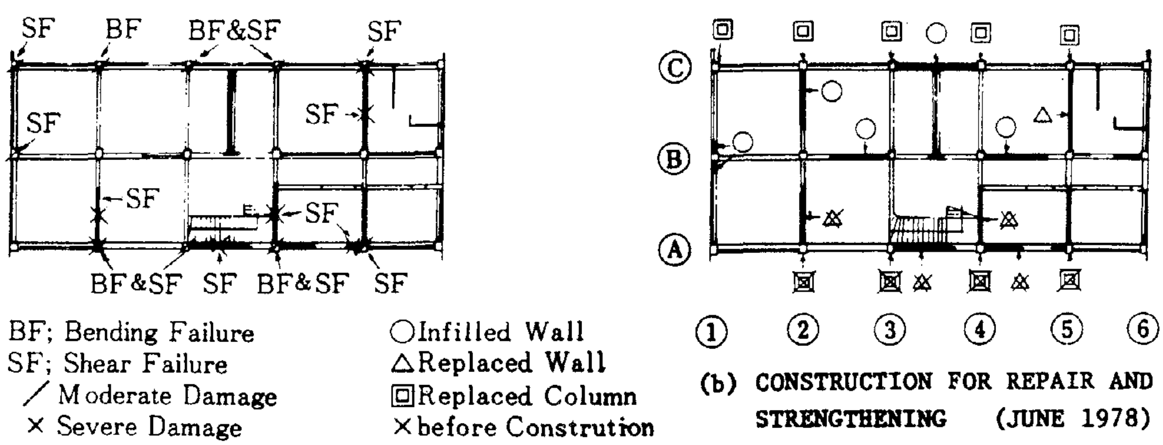


FIG. 14 STRENGTHENING OF THE DAMAGED SCHOOL BUILDING "B"



(a) FLOOR PLAN AND DAMAGE TO MEMBERS (FEBRUARY 1978)

(b) CONSTRUCTION FOR REPAIR AND STRENGTHENING (JUNE 1978)

FIG. 15 STRENGTHENING OF THE DAMAGED CITY HALL

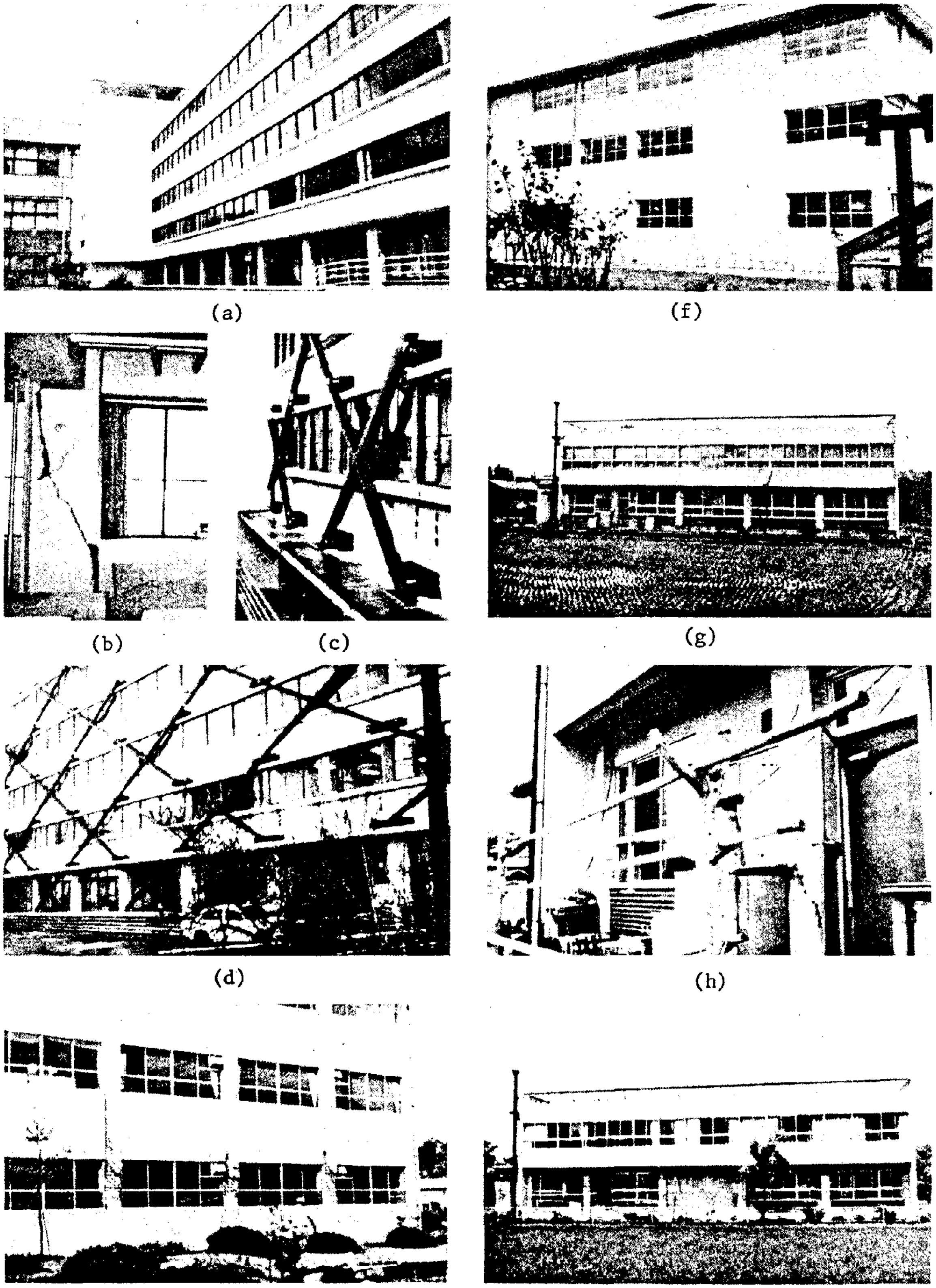
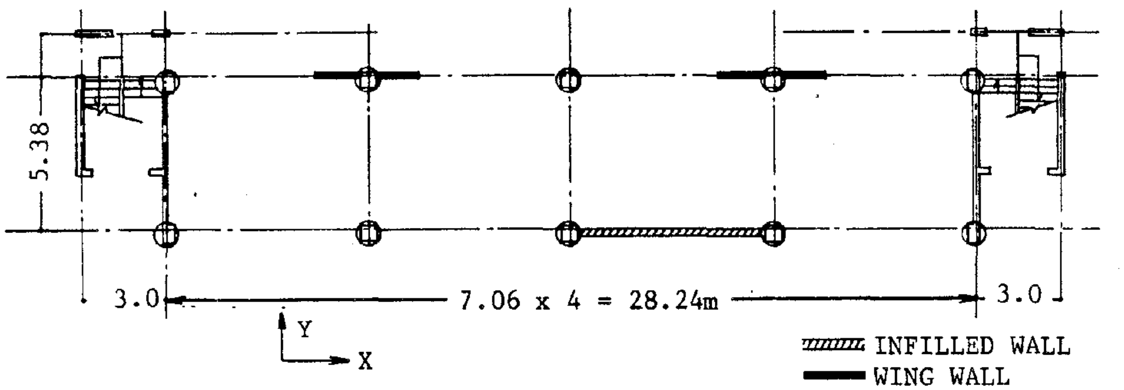


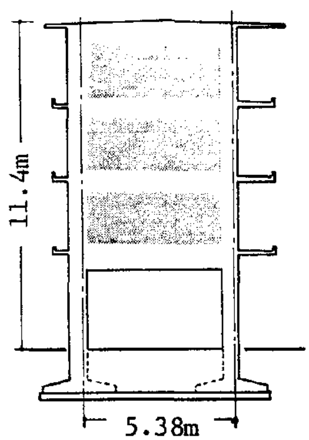
FIG. 16 PHOTOGRAPHS SHOWING DAMAGE AND STRENGTHENING OF VARIOUS BUILDINGS

TABLE 2 AN EXAMPLE OF SEISMIC EVALUATION OF EXISTING BUILDINGS[11]

Rank	Evaluation of Earthquake Resistance	Level of Seismic Index of Structure I_S	Number of Buildings	Percentage
A	strong enought	$I_S \geqq E_T$	17	9 %
B	need a check-up in detail	$I_S \geqq 0.7E_T$	21	12 %
C	need reinforcement	$0.3E_T<I_S<0.7E_T$	93	51 %
D	need urgent reinforcement	$0.3E_T<I_S<0.7E_T$	39	22 %
E	need rebuilding	$I_S \leqq 0.3E_T$	10	6 %
		Total	180	100 %



(a) PLAN OF FIRST STOREY



(b) SECTION

FIG. 17 STRENGTHENING OF A RESIDENTIAL BUILDING

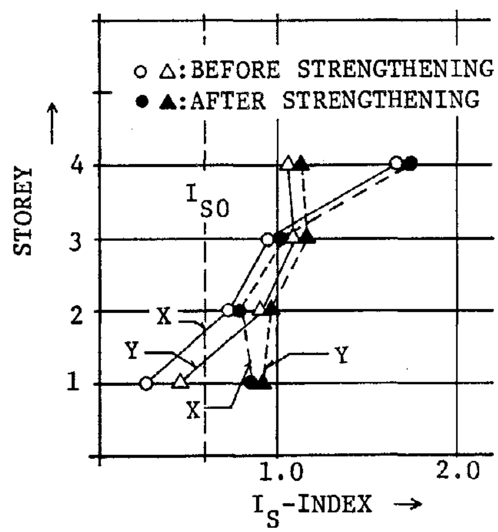


FIG. 18 SEISMIC EVALUATION OF A RESIDENTIAL BUILDING

VIBRATION TESTING OF AN EPOXY-REPAIRED FOUR-STORY REINFORCED CONCRETE STRUCTURE

by

Roger E. Scholl[I] and G. Norman Owen[II]

SYNOPSIS

A full-scale, 4-story reinforced concrete structure, deliberately damaged by forced vibration, was repaired by the epoxy-injection method and again deliberately damaged by forced vibration. At low-amplitude motions, the epoxy-repaired structure was slightly less stiff than the original, undamaged structure. However, at large deflections associated with severe damage, the epoxy-repaired structure was stiffer than the original structure. On the basis of these and other results, the epoxy-injection technique is considered to be an adequate method for repairing an earthquake-damaged structure.

INTRODUCTION

Epoxy-injection techniques were used extensively to repair cracking of highway bridges, buildings, and other reinforced concrete structures damaged by the 1964 Alaska, 1969 Santa Rosa, and 1971 San Fernando earthquakes, among others. In this method, a high-strength epoxy is injected into the cracked concrete, filling the voids and rebonding the fractured members. Experiments at the University of California, Berkeley (UCB), have shown this method to be quite effective for repair of laboratory specimens. However, the results of dynamic shaking-table tests conducted at UCB on two identical two-story reinforced-concrete-frame scale models suggested that epoxy repairs might not restore a damaged structure to its original stiffness. No information was then available concerning the effectiveness of epoxy repair for full-scale structures subjected to high-amplitude, destructive-level vibration.

THE FOUR-STORY REINFORCED CONCRETE STRUCTURE

In 1965-1966, two identical, full-scale, four-story reinforced concrete structures (see Figure 1), designed specifically for field investigation associated with a structure-response program conducted by URS/John A. Blume & Associates, Engineers (URS/Blume), for the U.S. Department of Energy, were constructed at DOE's Nevada Test Site. Each test structure is 12 ft by 20 ft center to center in plan and has four 9-ft stories. There are four rectangularly tied corner columns, 16 in. by 14 in. Spandrel beams are 16 in. by 15 in. in the 20-ft direction and 14 in. by 12 in. in the 12-ft direction. The floor slabs are 6 in. thick and are reinforced for two-way action.

Between 1966 and 1973, the two structures were subjected to ground motion caused by more than 50 underground nuclear explosions. In addition, they were subjected to numerous nondestructive vibration tests during that period, including pull-release tests, vibration-generator tests, and human-induced-vibration tests.

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II Project Engineer, URS/Blume

In 1974, one of the structures was deliberately forced into the range of inelastic response by means of a reciprocating-mass vibration generator. Structural damage was extensive, consisting of X-cracking and spalling at beam-column connections. The type and extent of the damage were similar to what might be expected from a major earthquake. The results of that test were published in 1976 (1).

RETEST OF THE EPOXY-REPAIRED STRUCTURE

In June 1975, the damaged structure was repaired by the epoxy-injection technique. The repaired structure provided an excellent opportunity to determine the effectiveness of epoxy-injection techniques in recovering the original physical properties of structures. In September 1979, the epoxy-repaired structure was again deliberately forced into the range of inelastic response.

Vibration Generator

To damage the test structure, it was necessary to build a hydraulic generator capable of producing a maximum force of 12,000 lb and with a frequency range of 1 to 50 Hz. The vibration generator was designed and assembled by Sandia National Laboratories, Albuquerque, New Mexico (see Figure 2).

The inertial force transmitted to the test structure was generated by a large oscillating mass driven by a hydraulic piston. A mass of about 15,000 lb was used in the 1974 tests and most of the 1979 tests, but it was increased to about 24,000 lb for the 1979 destructive test. The mass moved on four V-groove casters, with a maximum displacement of 3.9 in. from zero to peak. The supporting frame, constructed from two 12-in. wide-flange beams, distributed the load of the mass over a large floor area and transferred the inertial force to the building. Without the oscillating mass, the assembly weighed about 4,000 lb.

Instrumentation for controlling the vibration generator included: a displacement gage to measure the displacement of the oscillating mass with respect to the frame; a strain-gage-type force transducer attached to the hydraulic piston to measure the input force; and two accelerometers to measure the oscillating mass and frame accelerations. The vibration generator and associated hardware are described in more detail by Smallwood and Hunter (2).

Instrumentation

During the tests, 11 L-7 velocity meters measured the structural response. The input force applied to the building was recorded simultaneously with these 11 channels of response data on a 12-channel recording system. Additional L-7 velocity meters were also installed on the ground floor and at 5 ft, 30 ft, and 60 ft away from the building to measure transmission of vibration through the soil. This instrumentation consisted of 12 velocity meters and a 12-channel recorder. These response data were recorded on magnetic tape in analog form and later digitized for use in analysis.

The L-7 seismograph system is a compact, versatile unit capable of recording velocities over a range from 100 cm/sec to 9×10^{-5} cm/sec on either magnetic tape or paper strip charts. A detailed description of this instrumentation can be obtained from Navarro and Wuollet (3).

Five movie cameras were installed for the destructive test to provide permanent documentation of the structural response and damage. Four movie cameras were mounted on the test structure (at a beam-column connection at the top of each story), and one was located about 100 ft away from the structure to record the overall motion of the structure.

Test Procedure

The test sequence and procedures were, in general, the same as those followed for the 1974 tests. Although the testing was conducted principally to measure the dynamic-response behavior of the repaired structure in the inelastic range, low-amplitude testing in the quasi-elastic range was also conducted before and after the destructive test. With the vibration generator mounted on the roof, the structure was forced into low-amplitude motion in the 12-ft direction at frequencies corresponding to the first, second, and third modal frequencies for that direction. The vibration generator was rotated 90° on the roof, and the tests were repeated for the 20-ft direction. For the destructive test, the vibration generator was moved to the third floor and oriented along the 20-ft direction (see Figure 3). Following the test, the vibration generator was left in the same location, and the structure was again forced into low-amplitude motion at the lower modal frequencies.

OBSERVATIONS AND RESULTS

The results of this study show that, for low-amplitude motions, the epoxy-repaired structure was slightly less stiff than the original undamaged structure. This was expected because not all cracks could be repaired and also because the epoxy that was used is a more flexible material than concrete. However, a plot of the destructive-test data shows that, as the amplitude of structure's response increased, the difference in flexibility between the epoxy-repaired structure and the original structure decreased (see Figure 4). At large deflections associated with severe damage, the epoxy-repaired structure was actually stiffer than the original structure.

The cracking at the beam-column connections appeared to be much less severe in the 1979 test than in the 1974 test. Figure 5 shows the damage from the 1974 test at the northeast corner of the third floor, and Figure 6 shows the damage from the 1979 test at the same location. Figures 7 and 8 show the southeast corner of the third floor after the two tests. (Note that a construction error is evident in these two figures: a longitudinal reinforcing bar at the bottom of the beam was placed outside the confined region of the reinforcing steel in the column.)

The less severe cracking in the epoxy-repaired structure is consistent with the observed increase in stiffness at large deflections and may be the result of a better bond between the reinforcing bars and the concrete in the repaired structure than had existed in the original structure.

On the basis of these results, it is concluded that the epoxy-repair technique is an adequate method for repairing earthquake-damaged structures.

The 1979 test program and the current evaluation of the resulting test data were funded by the National Science Foundation under Grant No. PFR-7812714. The Nevada Operations Office of DOE provided the test structure, strong-motion instrumentation, and administration for the project.

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2. Smallwood, D. O., and N. F. Hunter, "A Transportable 56-kN, 200 mm Displacement Hydraulic Shaker for Seismic Simulation," *Proceedings*, Institute of Environmental Science, 1975.

3. Navarro, R., and G. M. Wuollet, *The L-7 Velocity Seismograph Shaking-Table Results*, Technical Report ERL 254-ESL 26, U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Environmental Research Laboratories, 1972.

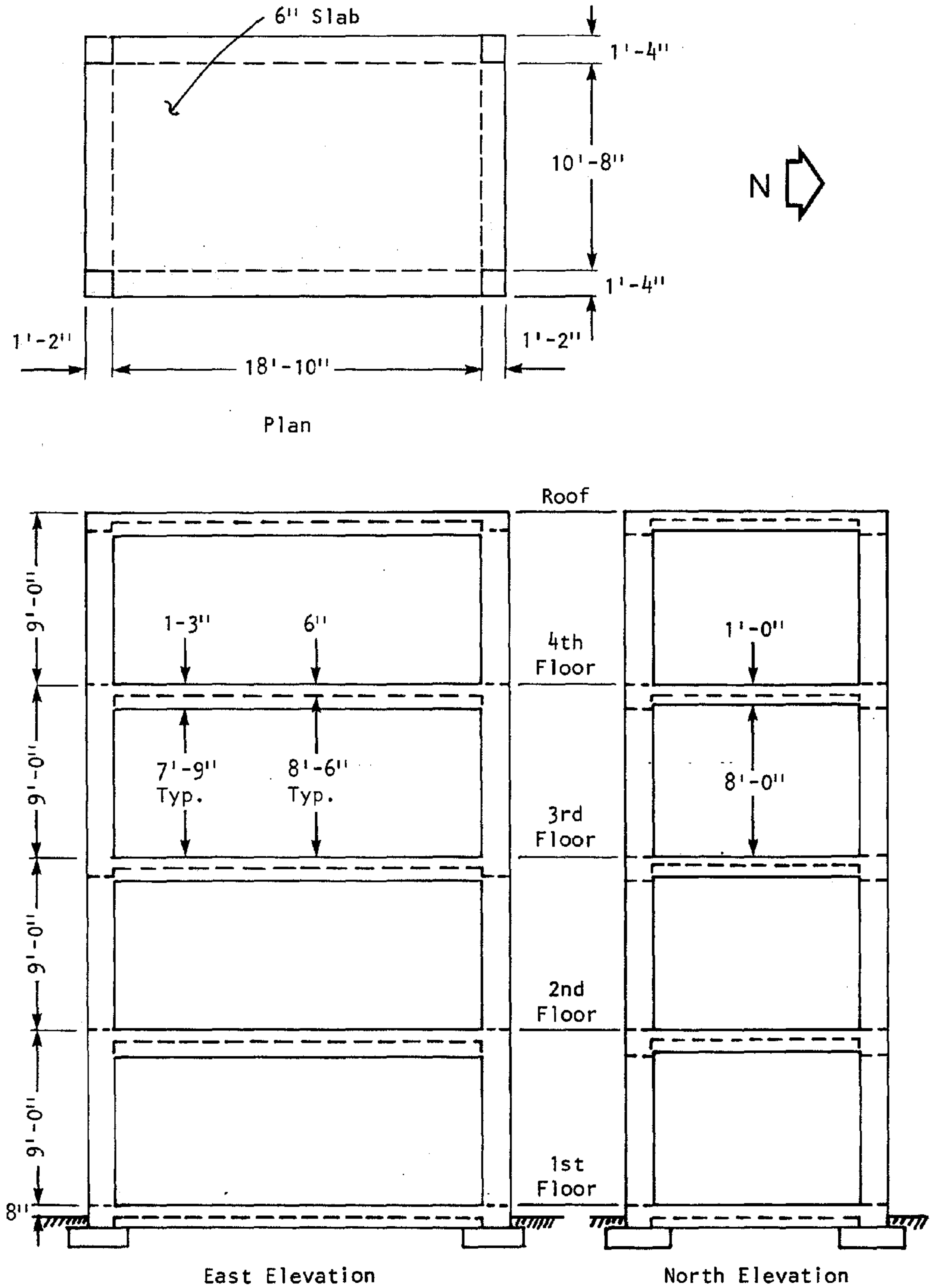

FIGURE 1 4-STORY CONCRETE TEST STRUCTURE

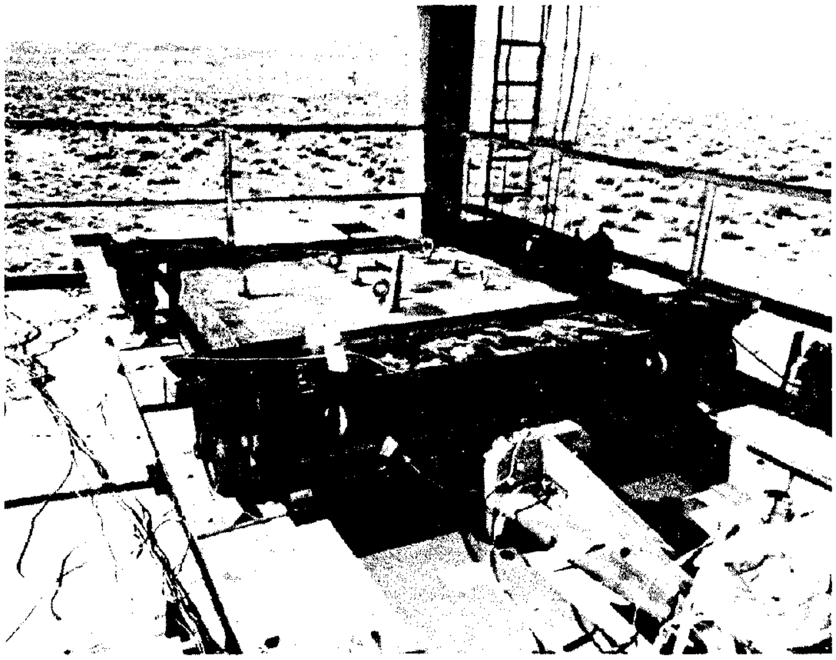

FIGURE 2 VIBRATION GENERATOR IN PLACE
ON THE TEST STRUCTURE

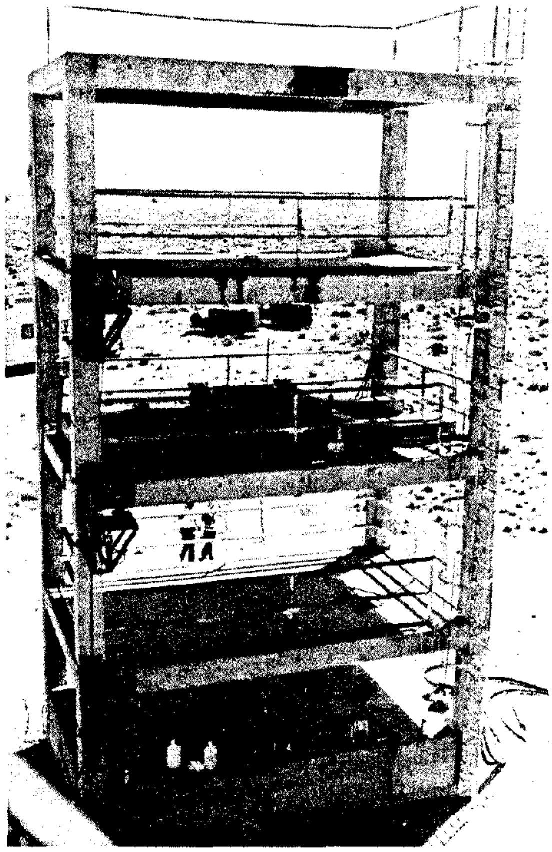

FIGURE 3 VIBRATION GENERATOR ON THE THIRD FLOOR OF THE TEST STRUCTURE

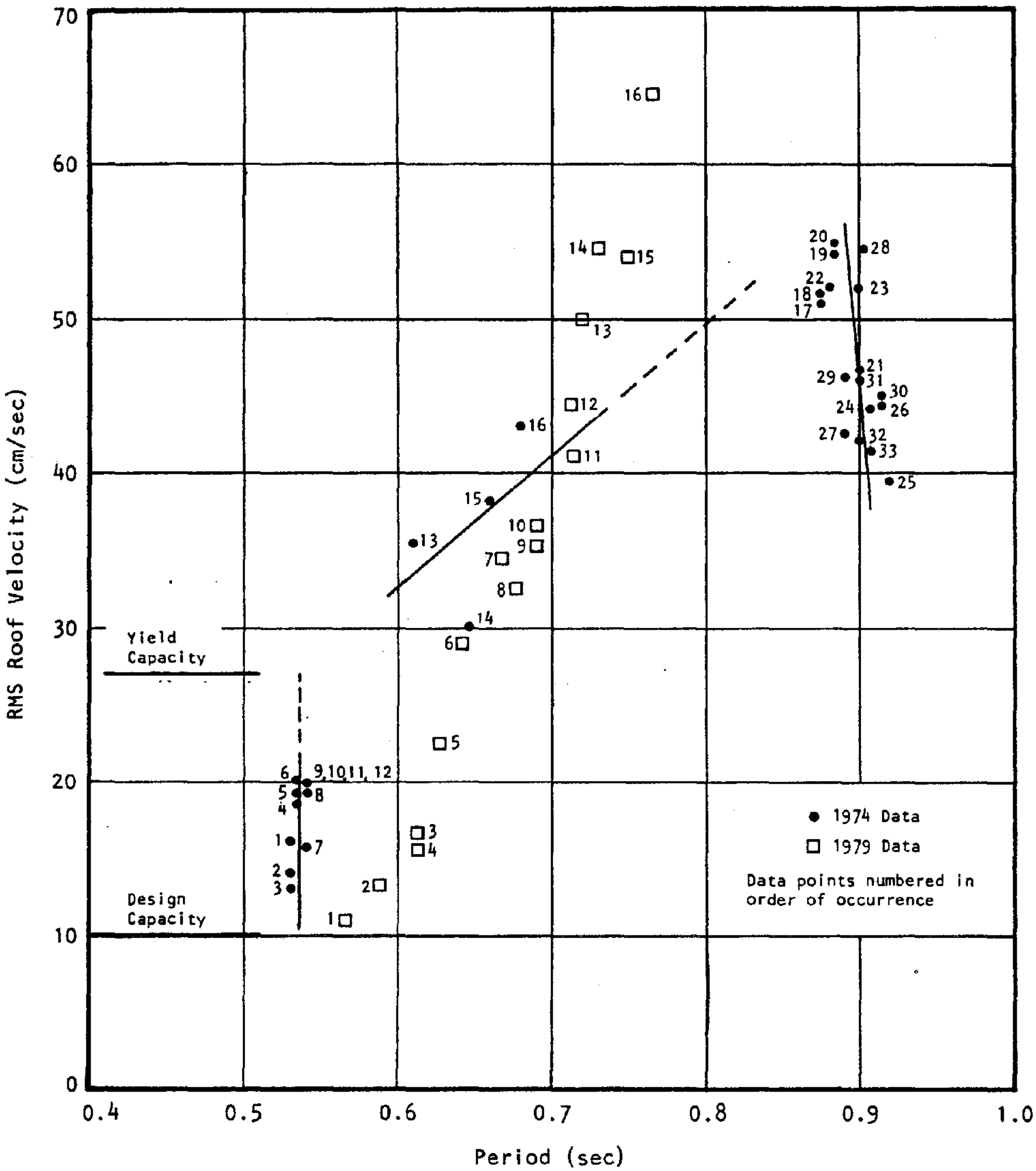

FIGURE 4 VELOCITY VERSUS PERIOD FOR THE DESTRUCTIVE TEST, LONGITUDINAL DIRECTION

FIGURE 5 DAMAGE FROM THE 1974 TEST AT THE NORTHEAST CORNER OF THE THIRD FLOOR OF THE ORIGINAL STRUCTURE

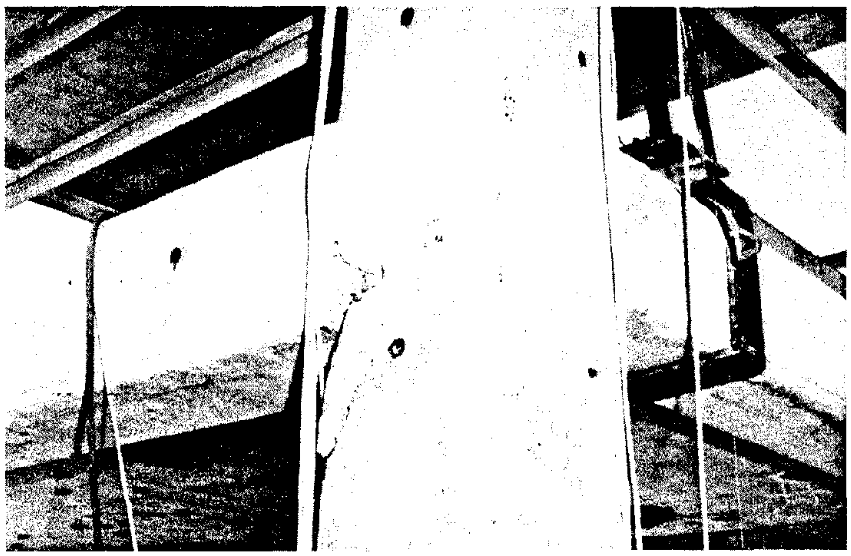

FIGURE 6 DAMAGE FROM THE 1979 TEST AT THE NORTHEAST CORNER OF THE THIRD FLOOR OF THE EPOXY-REPAIRED STRUCTURE

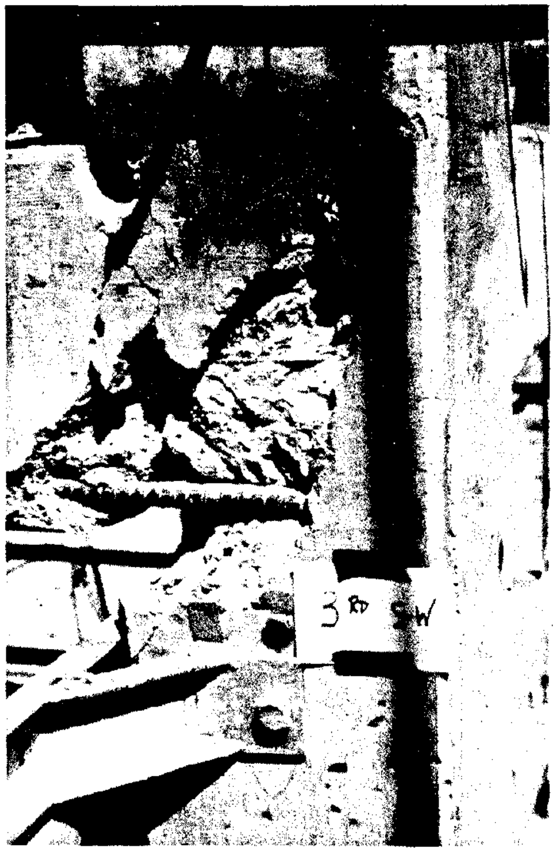

FIGURE 7 DAMAGE FROM THE 1974 TEST AT THE SOUTHWEST CORNER OF THE THIRD FLOOR OF THE ORIGINAL STRUCTURE

FIGURE 8 DAMAGE FROM THE 1979 TEST AT THE SOUTHWEST CORNER OF THE THIRD FLOOR OF THE EPOXY-REPAIRED STRUCTURE