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COMPARISON OF BUILDING SEISMIC DESIGN PRACTICES IN THE UNITED STATES AND JAPAN

ATC APPLIED TECHNOLOGY COUNCIL

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A team of thirteen building design professionals from the United States met with a group of ten engineers and researchers from Japan to develop a cooperative U.S.-Japan program for the improvement of building seismic design and construction practices. Workshop groups reviewed design and construction practices in both countries, developed recommendations pertaining to improved seismic design and construction requirements and procedures, and identified areas of mutual concern, including topics where there is need for future communication and exchange of information. This report contains the workshop recommendations and conclusions as well as the technical papers presented at the workshop. As such the papers provide an overview of current dasign practices, including project case studies from both countries.

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Prepared by

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1984

i.a

PREFACE

In March 1984 Applied Technology Council (ATC) organized a team of thirteen building design professionals to meet with a group of ten engineers and researchers from Japan to develop a cooperative United States-Japan program for the improvement of building seismic design and construction practices. The groups agreed to meet in Hawaii because that location involved approximately equal travel distances for the two groups. At the Hawaii meeting, which was conducted in workshop format, the groups reviewed design and construction practices in both countries, developed recommendations pertaining to improved seismic design and construction requirements and procedures, and identified areas of mutual concern, including topics where there is need for future communication and exchange of information.

This report contains the workshop recommendations and conclusions as well as the technical papers presented at the workshop. As such the papers provide an overview of current design practices, including project case studies from both countries.

ATC gratefully acknowledges the efforts of Mr. Walter Lum, who assisted in the organization of the Hawaii meeting, and the encouragement and cooperation provided by Dr. John B. Scalzi, Program Director for Dynamic Structural Experimentation, Civil and Environmental Engineering Division, National Science Foundation.

The material presented in this report is based upon work supported by the National Science Foundation under Grant No. CEE-8307976. Any opinions, findings, and conclusions or recommendations expressed in this publication are those of the authors and do not necessarily reflect the views of the National Science Foundation.

Christopher Rojahn
Executive Director
Applied Technology Council

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INTRODUCTION

The existence of earthquake source zones within and near the borders of Japan and the United States and the occurrence of numerous damaging earthquakes has led both countries to develop earthquake hazard mitigation programs. These programs have enabled Japanese and U.S. researchers and engineers to perform extensive seismic-related research and to conduct post-earthquake investigations to evaluate building design and construction practices.

Much of the research data and results obtained to date (1984) have not been translated into improved workable design and construction requirements and procedures. Where research results have been utilized to improve design and construction practices, each country has largely focused on utilization of its research results and little advantage has been taken of the extensive research in other countries.

The need for closer cooperation and communication between Japan and the United States became increasingly apparent during meetings over the last five years between U.S. and Japanese engineers and researchers involved in the joint U.S.-Japan pseudo-dynamic testing program involving large-scale structures as Tsukuba, Japan. These meetings clearly demonstrated that both countries could benefit materially by cooperating on their respective programs for improving building seismic design and construction practices. As a result, the Applied Technology Council (ATC) working in conjunction with representatives of the Japan Structural Consultants Association (JSCA) developed a preliminary plan for development of a cooperative U. S.-Japan program.

The primary purposes of the program are:

- to establish a mechanism whereby both countries could improve current seismic design and construction practices through the utilization of information developed in the other country,
- to establish a channel for the rapid interchange of research ideas (and needs) relating to the improvement of building design and construction practices, and
- to minimize the potential for future duplication of costly and time-consuming research.

The first meeting of the U.S.-Japan Cooperative Program for Improvement of Building Seismic Design and Construction Practices was held at the Kaimana Beach Hotel, Honolulu, Hawaii, March 13-15, 1984. Thirteen U.S. and ten Japanese building design professionals plus several observers participated in the meeting.

During the first one and one-half days of the meeting (Workshop), the participants made detailed presentations on seismic design and construction practices for structural steel and reinforced concrete buildings in the United States and Japan (see Workshop Program, Table 1). Following these presentations, parallel working sessions were held to (1) review and discuss the technical presentations, (2) identify research efforts needed to improve seismic design and construction requirements and procedures, (3) identify areas of mutual concern and the need for further communication and exchange of information, and (4) develop a framework for future U.S.-Japan cooperative efforts. At the close of the Workshop a final session was held in which all participants considered and adopted conclusions and recommendations prepared during the working session meetings.

Planning for a second meeting was initiated in July, 1984. Current plans are for the next meeting to be held in Japan.

The conclusions and recommendations emanating from the Honolulu Workshop and the Workshop technical presentations are included in the main body of this report. A list of Workshop participants is provided in Appendix A. Appendix B contains information on the Japan Structural Consultants Association, and Appendix C, Applied Technology Council projects and report information.

REFERENCE

Earthquake Engineering Research Institute (EERI), 1984, Proceedings, 8th World Conference on Earthquake Engineering, Vol. 7, Berkeley, California, pp. 595-650.

TABLE 1

Workshop Program

Tuesday, March 13, 1984

Session 1: Opening Session

Co-Chairpersons: Roland L. Sharpe, U.S.
Masakazu Ozaki, JAPAN

8:30 am to 8:50 am	Opening Remarks and Welcome Roland L. Sharpe, U.S. Masakazu Ozaki, JAPAN John B. Scalzi, National Science Foundation, U.S. Ajit S. Virdee, Applied Technology Council, U.S.
8:50 am to 9:00 am	Review of Workshop Objectives Christopher Rojahn, Applied Technology Council, U.S. Hiroshi Inoue, Japan Structural Consultants Association
9:00 am to 10:20 am	Seismic Design Approach and Philosophy Yuji Ishiyama, JAPAN Roland L. Sharpe, U.S.
10:20 am to 10:40 am	- Coffee Break -

Session 2: High Rise Steel Buildings

Co-Chairpersons: Yuji Ishiyama, JAPAN
Christopher Rojahn, U.S.

10:40 am to 11:20 am	Takayuki Teramoto, JAPAN
11:20 am to 12.00 noon	Clarkson W. Pinkham, U.S.
12:00 noon to 1:30 pm	LUNCH Invited Address: Evolution of Seismic Design in Hawaii Walter Lum, Walter Lum Associates, Inc., Honolulu

Session 3: High Rise Concrete Buildings

Co-Chairpersons: Ajit S. Virdee, U.S.
Takayuki Teramoto, JAPAN

1:30 pm to 2:10 pm	Donald R. Strand, U.S.
2:10 pm to 2:50 pm	Masakazu Ozaki, JAPAN
2:50 pm to 3:10 pm	- Coffee Break -

TABLE 1 (Continued)

Tuesday, March 13, 1984 (Continued)

Session 4: Mid-Rise Concrete Buildings

Co-Chairpersons:	Hiroshi Inoue, JAPAN	
	Donald R. Strand, U.S.	
3:10 pm to 3:50 pm		Hiroyuki Aoyama, JAPAN
3:50 pm to 4:40 pm		Raj T. Desai, U.S.

Session 5: Mid-Rise Steel Buildings

Co-Chairpersons:	Clarkson W. Pinkham, U.S.	
	Hiroyuki Aoyama, JAPAN	
4:40 pm to 5:10 pm		Chris D. Poland, U.S.
5:10 pm to 5:50 pm		Toshiharu Hisatoku, JAPAN

Wednesday, March 14, 1984

Session 6: Mid-Rise Composite or Precast and Low-Rise Steel Buildings

Co-Chairpersons:	Toshiharu Hisatoku, JAPAN	
	Chris D. Poland, U.S.	
8:00 am to 8:40 am		Yoshio Murata, JAPAN
8:40 am to 9:20 am		Gerard Dixon, U.S.
9:20 am to 10:00 am		Toshihiko Kimura, JAPAN
10:00 am to 10:20 am		- Coffee Break -

Session 7: Low-Rise Steel and Concrete Buildings

Co-Chairpersons:	Gerard Dixon, U.S.	
	Toshihiko Kimura, JAPAN	
10:20 am to 11:00 am		Melvyn H. Mark, U.S.
11:00 am to 11:40 am		Hiroshi Inoue, JAPAN
11:40 am to 12:20 pm		William Rumberger, U.S.
12:20 pm to 2:00 pm		LUNCH Invited Address: Volcanism and Earthquakes in Hawaii Robert Koyanagi, U.S. Geological Survey, Hawaii

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TABLE 1 (Continued)

Wednesday, March 14, 1984 (Continued)

Group Meetings

2:00 pm to 5:00 pm	Group 1:	Steel Buildings Develop recommendations for improved seismic design and construction requirements.
	Group 2:	Concrete Buildings Develop recommendations for improved seismic design and construction requirements.
	Group 3:	Common Problems a. Develop recommended research and/or investigations for modifying code requirements. b. Determine need for future meetings.

Thursday, March 15, 1984

Session 8: Group 1 and 2 Reports

Co-Chairpersons: Roland L. Sharpe, U.S.
Masakazu Ozaki, JAPAN

8:45 am to 9:30 am	Group 1 Report
9:30 am to 10:15 am	Group 2 Report
10:15 am to 10:45 am	- Coffee Break -

Session 9: Group 3 Report and Closing Session

Co-Chairpersons: Roland L. Sharpe, U.S.
Masakazu Ozaki, JAPAN

10:45 am to 11:30 am	Group 3 Report
11:30 am to 12:00 noon	Closing Discussion

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RECOMMENDATIONS

The overall goal of the meeting was to develop a U.S.-Japan Cooperative program for timely exchange of information and ideas for improving seismic design and construction practices in both countries. Based on the presentations made and the interactive discussions held, the following recommendations for research and investigations are made:

Common Problems

1. The U.S. and Japanese seismic codes specify different force levels and other design requirements. Research studies should be conducted by both countries to determine whether the apparent differences result in signficantly different building designs. These studies should be coordinated between the two countries.

2. The limitations on story drift are based largely on engineering judgment. Research should be conducted to determine what drift limitations should be specified.

3. Observations of past earthquake damage indicates that primary damage is often concentrated in one story. Further research is needed to better determine potential damage concentration. The effects of torsional moments on potential damage should also be studied.

4. The fundamental period of a building determined for calculating the seismic coefficient for building design often is not representative of the actual dynamic characteristics of the building. Further research is needed to develop a more accurate simplified method for calculating building periods, particularly for buildings with torsion.

5. The currently available descriptive guidelines for developing mathematical models for use in dynamic analysis of buildings do not provide adequate guidance. Research is needed to develop practical guidelines for use by the analyst and design engineer.

6. The repair and strengthening of existing buildings has become important in both countries. Further research is needed to develop procedures and criteria for evaluating the seismic resistant capacity of buildings and for their strengthening.

7. The concept of base isolation for buildings and/or equipment and systems appears to be feasible for reducing induced seismic forces. However, further research is needed to evaluate this concept and to develop guidelines for its application.

Structural Steel Buildings

The recommendations pertaining to structural steel are to:

1. Provide an annotated English translation of new Japanese seismic design procedures and relevant material design standards (concrete, steel, etc.)

2. Perform comparative trial designs of representative buildings using both Japanese standards and U.S. standards, with groups from both countries using both procedures. Compare understanding of these standards and compare resulting quantities.

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3. Develop modeling methods and guidelines for both elastic and nonlinear analysis.

4. Develop guidelines for the analysis of moment-frame/braced-frame interaction in both the elastic and inelastic ranges.

5. Develop a classification of seismic resistant systems and members, including their functions.

6. Develop a description and classification of dynamic analysis procedures as normally applied in practice in both countries.

7. Clarify the procedures for determining appropriate damping levels to be used in analysis.

8. Interchange U.S. and Japanese concepts and experience with base isolation.

9. Study design fees in both countries, including both total and structural fees.

10. Develop a catalog of seismically acceptable steel-framing-to-foundation connections.

11. Outline current U.S. quality control practices for welding and laminations. Investigate applicability of this practice to seismically related demands.

12. Provide available research and testing data for large weldments involving $2\frac{1}{2}$" (60 mm) and thicker material. U. S. engineers should review Japanese test data.

13. For existing buildings:
    a. Develop seismic rating schemes
    b. Develop appropriate seismic strengthening methodology

14. Develop damage level descriptions and classifications suitable for use with nontechnical individuals and groups. Assemble nonstructural damage data.

15. Drift:
    a. Develop the philosophical objectives for drift limitations.
    b. U. S. engineers should review Japanese design rules and the backup for these rules.
    c. Develop research needs based on the above.

16. Moment frame column panel zones:
    a. Summarize available research data.
    b. Identify additional needed research, including, but not limited to, systems, analytical studies, and shaking-table testing.

17. Review and present design recommendations on the strong-column/weak-girder concept.

18. Develop bracing requirements for yielding members with emphasis on lateral-torsional buckling. Review available research data. Compare the methods used in both countries.

19. Develop design rules for the required torsional rigidity of columns used in braced frames.

20. Develop design rules for partial penetration welded column splices.

21. After reviewing Japanese design rules and both U.S. and Japanese test data, develop appropriate U.S. design rules for the bracing members of braced frames, including, but not limited to:
    a. Slender members
    b. Stiff members
    c. Tension only members
    d. Compression only systems
    e. Connection details

22. Identify and develop basic research regarding the nonlinear behavior of braced frame systems with initial emphasis on K-braced types.

23. Study the effect of possible column buckling in braced-frame systems. U.S. engineers should review Japanese design rules on this subject.

24. U.S. engineers should review Japanese design rules with respect to overturning prior to development of specific research projects on this subject.

25. Compare U.S. and Japanese test data and design recommendations on eccentric bracing. Codify design procedures.

26. Develop design rules and representative details for connections of steel with other materials using concept that connections shall either have excessive strength (compared to member strength) or they shall be ductile.

27. Review the U.S. 25-percent-backup-frame concept with respect to its ability to serve its intended function in various building configurations.

28. Study the effect on low building earthquake response of varying the stiffness of diaphragms from flexible to semi-rigid.

Reinforced Concrete Buildings

The recommendations pertaining to reinforced concrete buildings are to:

1. Compare Japanese and U.S. design procedures with respect to detailing and specifications for confinement, anchorage, splicing, etc.

2. Compare U.S. and Japanese codes.

3. Exchange drawings of typical concrete details between the two countries.

4. Compare confining requirements of U.S. and Japanese codes.

5. Study the bases for period determination and modal analysis.

6. Modeling Studies:
    a. Develop guidelines for modeling that take into account slab influence on the beam section, gross or cracked sections, and elastic or inelastic properties.
    b. Research the influence of nonparticipating structural or nonstructural elements and the degree of fixity at the base.

c. Develop modeling procedures for considering the effect of flexural deformations, shear deformations, and panel zone deformations.
d. Investigate damping and building periods; correlate with observed data.

7. Evaluate the use of the following materials in seismic-resistant construction:
   a. f'_c = 6,000 psi maximum (stone concrete)
   b. f_y = 80,000 psi maximum in ties
   c. lightweight concrete with f'_c = 5,000 psi.

8. Develop procedures for defining acceptable and repairable damage for frames and for defining minimum required redundancy.

9. Evaluate required overstrength of columns versus beam strength (strong-column/weak-girder). Conversely, study use of weak-column/strong-beam systems at certain locations and in low buildings.

10. Study the lateral stability of large columns at slabs (without beams).

11. Evaluate the use of full capacity connectors for rebars without stagger between connectors.

12. Investigate the transfer of seismic shear at foundations.

13. Identify confinement requirements for grade beams and precast concrete piles.

14. Evaluate the flexibility of precast plank diaphragms; determine design assumptions.

15. Compare U.S. and Japan detailing practices for boundary members in shear wall design.

16. Redesign a building with Japanese and U.S. codes to compare results.

17. Provide copy of Japanese code (in English).

18. Meet in each country so that reference materials, design practices, and construction procedures can be observed first hand.

General

There are numerous areas of structural design where further communication between structural engineers and researchers in the United States and Japan would greatly benefit the seismic design process. These include the following:

1. Technical
   a. Design of foundations
   b. Building configurations—regular and irregular buildings
   c. Use of computers in the design process
   d. Design for damage control versus life safety
   e. Use of base isolation
   e. Modeling methods and guidelines for both elastic and nonlinear analyses
   f. Appropriate damping levels

2. Regulatory and Contractual

A comparison should be made between U.S. and Japan regulations and practices for the following:

a. Buildings requiring design by a structural engineer
b. Permit processes and procedures
c. Insurance requirements
    (1) Errors and omissions
    (2) Building (during construction)
    (3) General liability
d. Structural engineering licensing
e. Fee structure and type of contracts

CONCLUSIONS AND RESOLUTIONS

1. The presentations and discussions of prepared papers provided a first step toward a mutual understanding of engineering problems and design and construction practices in both countries. It was concluded that this was a successful first meeting.

2. The following resolutions were adopted:
    a. There should be a continuing exchange of information between the two countries.
    b. The feasibility of exchange of personnel should be assessed.
    c. A second meeting should be held within 18 months.

3. A Joint Steering Committee was appointed to coordinate information exchange and to plan a second meeting. The members are T. Kimura, T. Murata, M. Ozaki, C. Poland, C. Rojahn, and R. Sharpe.

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WORKSHOP TECHNICAL PAPERS

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NEW ASEISMIC DESIGN METHOD FOR BUILDINGS IN JAPAN

Yuji ISHIYAMA*

ABSTRACT

New aseismic design method for buildings has been enforced since June 1, 1981 in Japan. This method is stipulated in Building Standard Law, Building Standard Law Enforcement Order, Notifications of Ministry of Construction and other relevant regulations. The following is the summary of these codes and regulations which may facilitate the clear understanding of the method.

1. GENERAL

1.1 Purpose

The purpose of this aseismic design method is that buildings shall withstand moderate earthquake motions, which would occur several times during the use of the buildings, with almost no damage and shall not collapse nor harm human lives during severe earthquake motions, which would occur less than once during the use of the buildings.

1.2 Scope

Buildings shall satisfy one or more of the design procedures specified in Sec. 2 (Design Procedure), according to the structural type, floor area, height, etc. (See Table 1 & 2).

Buildings exceeding 60 meters in height will require special permission from the Minister of Construction following a detailed review of the dynamic behavior of the structure by the board of technical members.

2. DESIGN PROCEDURES

2.1 Structural Requirements

Buildings shall meet the relevant structural requirements specified by the Building Standard Law Enforcement Order, Notifications of Ministry of Construction, the Specifications of Architectural Institute of Japan, etc.

* Head, Building Engineering Division, International Institute of Seismology and Earthquake Engineering, Building Research Institute, Ministry of Construction.

Table 1 Design Procedures Required

	Buildings		Design Procedures Required
A	(1) One or two story wooden buildings not exceeding 500 square meters in total floor area. (2) Single story buildings other than wooden, not exceeding 200 square meters in total floor area. (3) Special buildings used for school, hospital, etc. not exceeding 100 square meters in total floor area.		2.1
B	Buildings not higher than 31 meters	(1) Buildings listed in Table 2.	2.1 and 2.2
		(2) Others	i) 2.1, 2.2, 2.3 and 2.4 or ii) 2.1, 2.2, 2.3 and 2.5
C	Buildings higher than 31 meters		2.1, 2.2, 2.3 and 2.5
D	Buildings higher than 60 meters		Special permission from the Minister of Construction

2.2 Stresses

The stresses caused by the lateral seismic shear for moderate earthquake motions prescribed in 3.1, 3.2 and 3.3 shall not exceed the allowable stresses for temporary loads.

2.3 Story drift

The drift of each story of the building caused by the lateral seismic shear for moderate earthquake motions prescribed in 3.1 shall not exceed 1/200 of the story height. This value can be increased to 1/120 if the nonstructural members shall have no severe damage at the increased story drift limitation.

2.4 Eccentricity, Stiffness, etc.

A. The following eccentricity of stiffness R_e of each story shall be less than 0.15.

$$R_e = \frac{e}{r_e} \qquad (1)$$

where, e = the eccentricity of the center of stiffness from the center of mass.

r_e = the elastic radius, which can be defined as the square root of the torsional stiffness divided by the lateral stiffness.

B. The following variation of lateral stiffness R_s of each story shall be

Table 2 Buildings Which Need to Satisfy only Design Procedures 2.1 and 2.2

A	One, two or three story buildings of conventional wooden construction, reinforced concrete block construction or masonry construction that shall meet the structural requirements stipulated in the relevant regulations.
B	Buildings of steel construction which shall meet all of the following items. (1) Stories above the ground level shall not exceed three. (2) Maximum height shall not exceed 13 meters and the eaves height shall not exceed 9 meters. (3) Maximum span of beams shall not exceed 6 meters. (4) Total floor area shall not exceed 500 square meters. (5) Stresses caused by the lateral seismic shear in which the standard shear coefficient C_o in Eq. (8) becomes 0.3 shall not exceed the allowable stresses for temporary loads. (6) Every joint of braces shall satisfy the following formula: $A_j \cdot \sigma_u \geqq 1.2A_g \cdot {}_b\sigma_y$ (T.1) where, A_j = the effective cross sectional area of the joint σ_u = the stress when the joint material fails A_g = the cross sectional area of the brace ${}_b\sigma_y$ = tensile yield stress of the brace
C	Buildings not exceeding 20 meters in height of reinforced concrete construction or steel encased reinforced concrete construction, if each story shall meet the following formula in the longitudinal and transverse directions. $25 \cdot \Sigma A_w + 7 \cdot \Sigma A_c + 10 \cdot \Sigma A_c' > Z \cdot A_i \cdot W$ (T.2) where ΣA_w = the sum of horizontal cross-sectional area in square centimeters of reinforced concrete shear walls in the direction concerned. ΣA_c = the sum of horizontal cross-sectional area in square centimeters of reinforced concrete columns, and reinforced concrete walls except shear walls in the direction concerned. $\Sigma A_c'$ = the sum of horizontal cross-sectional area in square centimeters of steel encased reinforced concrete columns. Z = the seismic hazard zoning coefficient as shown in Fig. 1. A_i = the lateral shear distribution factor as shown in Fig. 3. W = the weight (See Section 3.1) in kilograms of the building above the story concerned.
D	Buildings of a combination of different constructions, which satisfy the A - C requirements given above, shall meet the following items. (1) Stories above ground level shall not exceed three. (2) Maximum height shall not exceed 13 meters and the eaves height shall not exceed 9 meters. (3) Total floor area shall not exceed 500 square meters.
E	Buildings specially permitted by the Minister of Construction.

greater than 0.6.

$$R_s = \frac{r}{\bar{r}} \tag{2}$$

where, r = the lateral stiffness, which shall be defined as the value of the story height divided by the story drift caused by the lateral seismic shear for moderate earthquake motions prescribed in 3.1.

$\bar{r}$ = the mean lateral stiffness, which shall be defined as the arithmetic mean of r's above ground level.

C. (1) Buildings of steel construction shall satisfy all of the following items:

i) The lateral seismic shear for moderate earthquake motions of steel structures shall be increased, according to the following formula.

$$Q_b = (1 + 0.7\beta)Q \tag{3}$$

where, Q_b = increased lateral seismic shear.

β = the ratio of the lateral shear of braces to the total lateral seismic shear of the story. The value of $(1 + 0.7\beta)$ need not be more than 1.5.

Q = lateral seismic shear for moderate earthquake motions prescribed in 3.1.

ii) Each brace shall meet the following formula.

$$_JP_u \geqq 1.2\ _MP_y \tag{4}$$

where, $_JP_u$ = the ultimate strength of the joint of the brace.

$_MP_y$ = the yield strength of the brace.

iii) The width-thickness ratio of plate elements of columns and beams subjected to bending moment shall satisfy the requirements in Table 3.

Table 3 Width-Thickness Ratio of Steel Columns and Beams

Members	Section	Portion	Steel	Width-Thickness Ratio: Standard	Width-Thickness Ratio: Maximum
Columns	H	Flange	SS41*	9.5	12
			SM50**	8	10
		Web	SS41	43	45
			SM50	37	39
	□	—	SS41	33	37
			SM50	27	32
	○	—	SS41	50	70
			SM50	36	50
Beams	I	Flange	SS41	9	11
			SM50	7.5	9.5
		Web	SS41	60	65
			SM50	51	55

* Steel conforming to SS41, SM41, SMA41, STK41 and STKR41 of JIS.
** Steel conforming to SM50, SMA50, SM50Y, STK50 and STKR50 of JIS.

iv) Every beam to column connection subjected to bending moment shall satisfy the following formula:

$$M_u > \alpha M_p \qquad (5)$$

where, M_u = maximum bending strength of the connection.

M_p = full plastic moment of the column or beam.

α = safety factor (1.2 - 1.3)

C. (2) Buildings of reinforced concrete or steel encased reinforced concrete constructions shall satisfy one of the following items:

i) Each story shall meet the following formula in the longitudinal and transverse directions.

$$25 \cdot \Sigma A_w + 7 \cdot \Sigma A_c + 10 \cdot \Sigma A_c' \geqq 0.75 \cdot Z \cdot A_i \cdot W \qquad (6)$$

where A_w, A_c, A_c', Z, A_i and W are the same as in Table 2.

ii) Each story shall meet the following formula in the longitudinal and transverse directions.

$$18 \cdot \Sigma A_w + 18 \cdot \Sigma A_c \geqq Z \cdot A_i \cdot W \qquad (7)$$

where A_w, A_c, Z, A_i and W are the same as in Table 2.

iii) Ultimate shear strength of each reinforced concrete member shall be greater than the ultimate flexural strength of the member.

2.5 Ultimate Lateral Shear Strength

The ultimate lateral shear strength of each story shall not be less than the necessary ultimate lateral shear Q_{un} determined in accordance with the following formula.

$$Q_{un} = D_s \cdot F_{es} \cdot Q_{ud} \qquad (8)$$

where, Q_{ud} = the lateral seismic shear for severe earthquake motions prescribed in 3.1.

D_s = the structural coefficient given by Table 4a and 4b.

F_{es} = the shape factor which shall be determined as follows.

$$F_{es} = F_e \cdot F_s \qquad (9)$$

where F_e is given in Table 5 as a function of eccentricity of stiffness R_e defined in 2.4A. F_s is given in Table 6 as a function of variation of lateral stiffness R_s defined in 2.4B.

3. LATERAL SEISMIC SHEAR

3.1 Lateral Seismic Shear above the Ground Level

The lateral seismic shear, Q_i, of i-th story above the ground level shall be determined in accordance with the following formula.

Table 4a. Structural Coefficient D_s for Buildings of Steel Construction

Behavior of Members	Type of Frame		
	(1) Ductile moment frame	(2) Frame other than listed in (1) and (3)	(3) Frame with compressive braces
A. Members of excellent ductility	0.25	0.3	0.35
B. Members of good ductility	0.3	0.35	0.4
C. Members of fair ductility	0.35	0.4	0.45
D. Members of poor ductility	0.4	0.45	0.5

Table 4b. Structural Coefficient D_s for Buildings of Reinforced Concrete or Steel Encased Reinforced Concrete Construction*

Behavior of Members	Type of Frame		
	(1) Ductile moment frame	(2) Frame other than listed in (1) and (3)	(3) Frame with shear walls or braces
A. Members of excellent ductility	0.3	0.35	0.4
B. Members of good ductility	0.35	0.4	0.45
C. Members of fair ductility	0.4	0.45	0.5
D. Members of poor ductility	0.45	0.5	0.55

* Values are decreased by 0.05 for steel encased reinforced concrete construction.

Table 5 Shape Factor F_e by Eccentricity of Stiffness R_e

R_e	F_e
less than 0.15	1.0
$0.15 \leqq R_e \leqq 0.3$	linear interpolation
more than 0.3	1.5

Table 6 Shape Factor F_s by Variation of Lateral Stiffness R_s

R_s	F_s
more than 0.6	1.0
$0.3 \leqq R_s \leqq 0.6$	linear interpolation
less than 0.3	1.5

$$Q_i = C_i \cdot W \tag{10}$$

where, C_i = the lateral seismic shear coefficient of the i-th story as determined in accordance with Formula (11).

W = the weight of the building above the i-th story.

The weight of the building shall be the sum of dead load and the applicable portion of live load. In heavy snow districts, the effect of snow load shall be considered.

The lateral seismic shear coefficient of the i-th story, C_i, shall be determined in accordance with the following formula.

$$C_i = Z \cdot R_t \cdot A_i \cdot C_o \tag{11}$$

where, Z = the seismic hazard zoning coefficient as shown in Fig. 1.

R_t = the design spectral coefficient, which shall be determined by the type of soil profile and the fundamental natural period of the buildings, as illustrated in Fig. 2.

A_i = the lateral shear distribution factor, which shall be determined by the fundamental natural period and the weight distribution of the buildings, as shown in Fig. 3.

C_o = the standard shear coefficient, which shall be not less than 0.2 and 1.0 for moderate earthquake motions and for severe earthquake motions, respectively.

The fundamental natural period of the building, T, to determine the design spectral coefficient and the lateral shear distribution factor, shall be determined in accordance with the following formula.

$$T = h(0.02 + 0.01\alpha) \tag{12}$$

where, T = the fundamental natural period of the building in seconds.

h = the height of the building in meters.

α = the ratio of the total height of stories of steel construction and the height of the building.

3.2 Lateral Seismic Shear of Appendages

The lateral seismic shear, q, for penthouses, chimneys, towers, cisterns, parapets and other appendages on buildings shall be determined in accordance with the following formula.

$$q = k \cdot w \tag{13}$$

where, q = the lateral seismic shear of the appendage.

k = the seismic design coefficient of appendages which shall be 1.0, but the value can be reduced to 0.5 in such cases where no harm to human lives will occur.

w = the weight of the appendage.

3.3 Lateral Seismic Shear of the Basement

The lateral seismic shear of the basement, Q_B, shall be determined in accordance with the following formula.

$$Q_B = Q_p + k \cdot W_B \tag{14}$$

where, Q_p = the portion of the lateral seismic shear of the first story that will extend to the basement.

k = the seismic design coefficient of the basement as determined in accordance with Formula (13).

W_B = the weight of the basement.

The seismic design coefficient of basement, k, shall be determined in accordance with the following formula.

$$k \geqq 0.1\left(1 - \frac{H}{40}\right)Z \tag{15}$$

where, H = the depth of the basement in meters. The value of H shall be fixed at 20 meters in such cases where the basement depth exceeds 20 meters.

Z is the same as defined in 3.1.

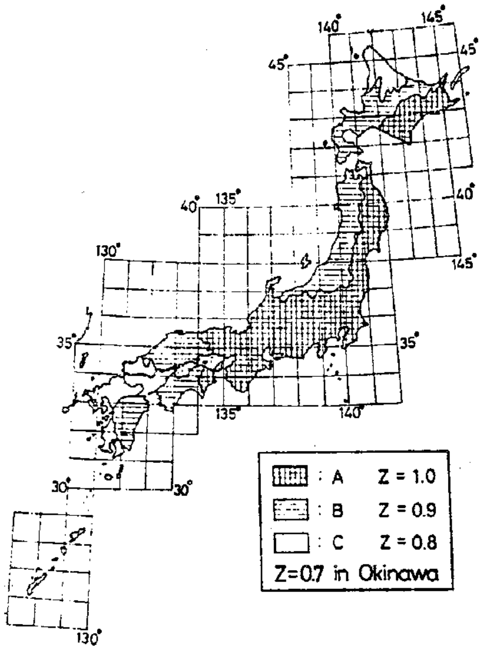

Fig. 1 Seismic Hazard Zoning Coefficient, Z

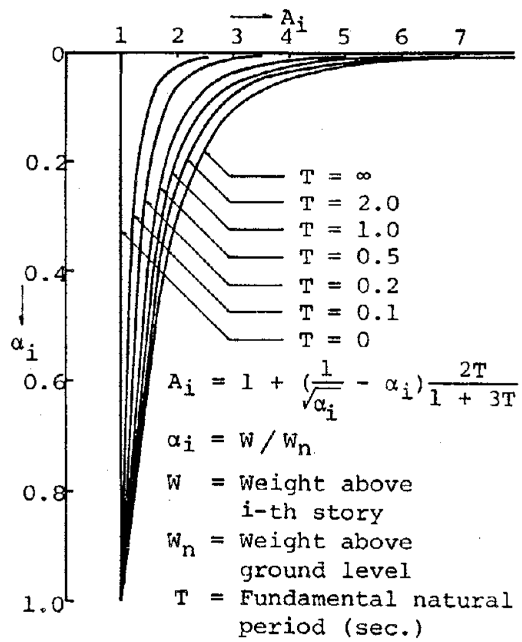

Fig. 3 Lateral Shear Distribution Factor, A_i

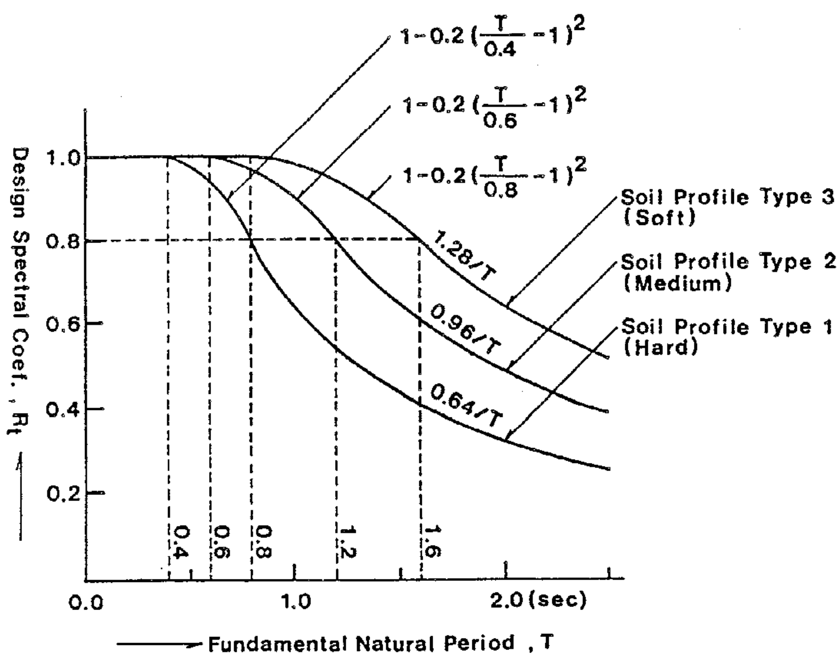

Fig. 2 Design Spectral Coefficient, R_t

Through the precise analysis of the structure, the foundation, the soil, etc., the value of R_t can be reduced to 0.75 of the value given by this figure. But the value shall not be less than 0.25.

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SEISMIC RESISTANT DESIGN PHILOSOPHY AND APPROACH

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INTRODUCTION

The purpose of this paper is to summarize the development of United States seismic building codes, their philosophy and the approach generally followed in the seismic design of buildings.

Seismic codes in the United States were not written until the late 1920's, although the San Francisco earthquake of 1906 did considerable damage, San Francisco was rebuilt on the basis of a 30 pound per square foot wind loading. There was little known about earthquakes and the way structures respond to them. The Santa Barbara earthquake of 1925 provided the impetus for the Uniform Building Code in 1927 to spell out a code coefficient $C=0.075$ times the weight of the structure. Other codes followed that requirement closely. For example, after the 1933 Long Beach earthquake, Los Angeles adopted a C factor of 0.08, roughly the same as the Uniform Building Code.

Further work was done by engineers in Los Angeles and San Francisco. In 1943, Los Angeles code provisions started to deal indirectly with the period of vibration by using a formula $C=60/N+4.5$, where N was the story height. It was realized that the response of the structure to earthquake motions was a function of its dynamic properties or period. A limit of 13 stories on buildings was also adopted.

Later the San Francisco Section of the American Society of Civil Engineers (ASCE) and the Structural Engineers Association of Northern California (SEAONC) formed a joint working group and in 1952 published the first seismic recommendation where the period of the building was explicitly introduced; and K was a constant that varied depending on the type of building.

However, it was realized that since earthquakes are dynamic phenomena and period is just one measure of the dynamic characteristics of the structure, more was needed. Therefore in 1957 the Structural Engineers Association of California (SEAOC) appointed a Lateral Forces Committee. The name was later changed to the Seismology Committee. After two years work, in 1959 the committee published the Recommended Lateral Force Requirements for Buildings, the so-called "Blue Book." The SEAOC provisions recognized that the seismic forces induced in a structure are related to its period of vibration. The "Blue Book" has been extensively studied and modified since then; the latest major changes appearing in 1974. The current Uniform Building Code contains almost verbatim the 1974 SEAOC recommendations plus a few changes developed since 1974.

In 1970, SEAOC organized a committee to look at the "Blue Book" and earthquake codes in general. This committee recommended that a group be put together to make an extensive survey of existing design practices, research data, and codes. This was in recognition of the fact that the "Blue Book" is a limited document that deals with building structures only. The report published in the Proceedings of the American Society of Civil Engineers, provided impetus

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others to see if federal support could be obtained for an extensive study of seismic design provisions.

In 1973 the National Science Foundation granted initial planning money. A group was formed of some 20 people from throughout the United States. This group developed a program, a budget, and an optimistic prediction--that it could be done in two-and-a-half years. It took a little over three. There were 85 participants representing structural engineers, mechanical and electrical engineers, architects, code officials, representatives from governmental agencies, and a number of professors and researchers from various universities throughout the United States. The provisions were essentially completed in 1976. After reviews by numerous reviewers, they were published in 1978 as the "Tentative Provisions for the Development of Seismic Regulations for Buildings"(ATC-3-06).

Currently the ATC provisions are being tested by seventeen structural design firms throughout the United States who are making comparative designs of numerous types of buildings. The buildings are first designed for the code in effect in the locality and then redesigned to meet the ATC-3 provisions. The results of this program will be used to modify the provisions as appropriate. The amended document will then be presented in 1985 to code promulgating groups for consideration and adoption.

In addition to the ATC-3 program the Structural Engineers Association of California Seismology Committee is currently working on a revision of the "Blue Book". They are considering the ATC-3 provisions together with research and other data that have been developed since the ATC-3 provisions were developed in 1976. The revisions are scheduled for completion in 1985.

CODE PHILOSOPHY

The primary philosophy of U.S. seismic codes is to protect life safety--of occupants and of those adjacent to the exterior of the building. The SEAOC Blue Book, Uniform Building Code and other seismic building codes generally are designed so that most buildings will:

1. Resist minor earthquakes without damage.

2. Resist moderate earthquakes without significant structural damage, but with some nonstructural damage.

3. Resist major or severe earthquakes without major failure of the structural framework, but with some structural as well as nonstructural damage.

In addition to the above, it is essential that the occupants be able to leave a damaged building and that rescuers be able to enter. Further, in case of a major earthquake, certain essential facilities should be functional during and immediately after the event. These include emergency command centers, communication centers, hospitals, ambulance facilities and similar. There also has been increasing concern that damage to nonstructural components and systems can cause injury to people and in addition, the cost of repair can often greatly exceed the cost of structural repair.

As a result of the above, requirements have been added to seismic codes relating to elevators, stairwells, entrances and exits. Essential facilities are designed to more stringent provisions than other buildings.

In the development of the U.S. seismic codes currently in use, it was recognized that the specified design forces are considerably smaller than those that might be encountered in moderate or major earthquakes. Primary consideration is given either explicitly or implicitly to the effects (both good and bad) of interior partitions, exterior cladding, different types of materials, and damping.

However, this approach has caused misunderstandings that can result in building designs with serious deficiencies such as inadequate connections or tying together of building components and inadequate provision(s) for the occurrence of building deformations in excess of those calculated for the design forces.

It is important to recognize that design forces derived from code formulas represent the best judgements of competent groups of professional engineers. However, the seismic deformations to which the building may be subjected are really unknown and in most cases may greatly exceed the design deformations obtained using the code formulas. In such cases the building structure could be severely overstressed as compared to stresses calculated for code seismic design loads.

The structure and its appendages must remain stable when undergoing horizontal deformations which could considerably exceed yield deflections. It is basic good design that, together with providing the minimum design strength, the performance of the structural system at very large overloads and deformations be carefully considered. To ensure adequate performance, the detailing of joints and members must be done to ensure that the structure will remain as a unit, even while subjected to these very large deformations. It is also important that some redundancy be provided and that structural elements be designed so less critical elements fail first, thus absorbing and dissipating energy and providing protection for more critical members. A building or structure having this capabiltiy is said to have ductility. Finally, it is essential that a continuous load path (or paths) having adequate strength and stiffness be provided so that all forces will be transferred from the point of application to the final point of resistance.

APPROACH TO SEISMIC DESIGN

The design of various types of buildings in the U.S. is being presented by other workshop participants. Therefore, this paper will only address the approach normally followed and the factors considered in seismic resistant design of buildings. The determination of seismic criteria, the types of construction materials and the general design considerations will be discussed.

Seismic Criteria

Several factors should be considered in developing seismic criteria for a site, the most important being general site characteristics, site seismicity, expected peak ground motions, structure or facility category use, type of structural system, and applicability of building code formulas.

When selecting a new site, an investigation should be made to determine the existence of any active earthquake faults. The potential for landslides, liquefaction, or consolidation of foundation soils when subjected to vibratory ground motion should be determined. For an existing site, all available foundation investigations and geological reports should be reviewed for such information. Large differences in elevations should also be studied. Soils-foundation reports should be reviewed with the assistance of a qualified professional to determine the possible liquefaction potential of underlying soils during a seismic event. Similarly, the data should be reviewed for potential soil consolidation when subjected to vibratory motion. Detailed field investigations might be necessary to determine existence and extent of any of the above such possible hazards. The extent to which such investigations are employed will depend on the size and importance of the proposed facility, probability of safety-related or large economic risks, and the possibility of or suspicion of potential site hazards.

For a normal-usage facility, it may be appropriate to select design coefficients based upon the seismic zoning shown in the building code. The importance of the proposed facility and/or the potential risks involved may warrant a special study of the site seismicity. If so, it should include a review of the historical seismicity within the surrounding area to a 50 mile radius. Probable frequencies of earthquake occurrence and probable ground motions at the site should be determined. An important factor in determining site seismicity is the degree of acceptable risk: should the facility be able to function after the occurrence of the maximum earthquake predicted to occur within 50 years, 100 years, or 500 years, or should the facility be designed to sustain only minimum damage and maintain functionability if any of these occur? This decision should be made by management, not left to the seismic consultant or engineer.

A facility that must be able to function during and after a major earthquake, or still be functional only <u>after</u> a major event, should have its peak ground motions (PGM) for design determined as part of the study. For most sites the possible PGM could be several times more than the building code coefficient. Therefore, the PGM amplitude used in design should depend on the degree of risk considered acceptable for the facility, i.e., the PGM for a 50-year return period would be much less than for a 500-year return period. After the PGM is determined, response spectra or acceleration time histories would be developed considering the soil and foundation conditions at the site and the distances to potential earthquake sources.

Structural System

The structural system used for the building, such as a moment space frame, shear wall, dual frame (moment plus braced frames or shear wall), or bearing wall system, largely governs the type of response a building will exhibit during an earthquake. A frame structure is usually the most flexible and dissipates energy by deforming in bending or flexure. If the frame structure is ductile, i.e., has redundancy and capability to remain stable when stressed beyond yield levels, then a lower total seismic force coefficient can be used in the design. However, the nonstructural components and systems must be designed to accommodate the expected building frame deformations. Such accommodation would include providing connection and support details so that the frame could move without damaging exterior walls, windows, or interior partitions. Similarly, provisions to accommodate building deformations must be provided in the design of mechanical and electrical systems in a frame structure. These provisions can add to construction costs; however, a moment space frame may be the best solution, based on building function and use, if the building deformations can be accommodated.

A shear wall structure on the other hand, is quite rigid and deflects less than a comparable moment frame. Therefore, more energy is imparted to the structure and a higher total seismic force coefficient is used. Most of the seismic-induced energy is dissipated by shearing distortion and less ductility is available. Since shear wall structures deform considerably less than comparable frame structures, savings on connections of exterior walls, windows, and interior partitions to the building structure can result. Shear wall structures may have a structural frame for supporting gravity loads.

Structural framing systems can be stiffened by using cross bracing or eccentric bracing, or shear walls to reduce deformation induced by seismic motion. For such systems, nonstructural components are less likely to be damaged. However, braced steel framing has much less ductility than a structural steel frame. Bracing members act primarily in tension: when failure occurs it is somewhat abrupt. Unlike a moment frame system, a braced system has little capability to dissipate energy by continued deformation. A recent development that is gaining favor is the eccentric-braced steel frame, which does allow some ductile deformation. Code requirements for this type of framing are being developed.

Design Considerations--Structural

The design factors generally considered are building shape and geometry, framing type, basic construction materials, arrangement and type of nonstructural components, and adequacy of connections of various building parts and components. To ensure an efficient, economical, seismic-resistant building, it is essential that the architect and engineers collaborate as a team during the conceptual design stage. Too often the architect working alone develops the conceptual plan, arrangement, and aesthetic design, and then presents the concept to the structural, mechanical and electrical engineers for design of the appropriate systems. By involving the engineers in the conceptual design stage, the design team can avoid expensive and sometimes inadequate solutions for effective seismic resistance.

During the conceptual design phase the design team should consider certain factors:

- A building's inherent resistance to seismic forces is determined to a large extent by the basic layout. It is desirable that the building be symmetrical or have symmetry about each axis. The symmetry should be considered in the arrangement of wall openings, location of shear walls, size and spacing of columns and other potential lateral force-resisting elements. When seismic force effects are considered in the initial layout, significant cost savings can be made without detracting materially from the building's function or appearance.

- Re-entrant corners, such as those concurring in L, T, or U-shaped plans, are locations of great stress and should be avoided or reinforced appropriately.

- The effect of components, such as interior partitions, filler walls, exterior glazing, or exterior wall panels should be considered in the initial layout. For example, filler walls not symmetrically located could interact with the framing system inducing torsional rotational moments in the structure, resulting in excessive stresses in columns or other shear-resisting elements. In addition, nonstructural components can stiffen a building and initially induce higher seismic forces in the building than may be contemplated in the design.

- Relative stiffnesses of the various stories in the building should be considered. Often the first story is made taller than the others with many of the interior walls deleted to give a more open appearance. As a result, the first story could be considerably more flexible than the others and excessive deformations could occur with accompanying adverse affects. An example of this situation was vividly demonstrated by the Olive View Hospital during the 1971 San Fernando earthquake. The upper stories were stiffened with shear walls, but the first story depended on columns in flexure to resist seismic forces. The large deflections of the first story caused excessive damage and almost total loss of the carrying capacity of the columns. Although only four months old, the structure had to be razed and a new facility designed and constructed.

- While the building layout is being determined, the type of construction material is also evaluated. Many factors other than seismic resistance affect selection of the particular construction material, including total building life construction cost and requirements for fire resistance. The architect and the engineer normally examine various framing systems and construction materials to determine which will provide the required function at the lowest cost.

- The use of a moment-resisting space frame (MRSF) versus a shear wall system should be evaluated. For buildings which have no brittle finishes, such as a warehouse or shop, either a MRSF, braced frame or a shear wall system will work equally well. Where there will be many relatively brittle elements such as interior partitions, stairwells, and exterior glazing, the cost of stiffening the MRSF and/or designing connections to accommdate relative movement between the structural frame and these elements should be considered. Often, dual frames or shear walls can be utilized to provide a stiffened structure without sacrificing function.

After the layout, type of framing, number of stories, and construction material are selected, the following design steps are taken:

- a preliminary value of the building fundamental period is calculated.

- the seismic coefficient is determined (in UBC, $C = 1/\sqrt{15T}$).

- the horizontal force factor, K, is selected based on the type of framing and varies from 0.67 to 1.33.

- the occupancy of the building determines the value of the factor, I (varies from 1 to 1.5).

- the site-structure resonance factor, S, is estimated.

- preliminary sizes of the structural framing system and the building weight, W, are determined.

- the base shear (V = ZIKCSW) is calculated.

- the base shear is distributed over the height. The UBC formula is:

$$F_x = \frac{(V-F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

where F_t = that portion of V considered concentrated at the top of the structure in addition to F_n.

w_i, w_x = that portion of W which is located at or is assigned to level i or x, respectively.

h_i, h_x = height in feet above the base to level i, n, or x, respectively.

- The gravity loads and the seismic shear forces calculated at each level are used to recalculate the vertical and horizontal member sizes. The seismic shear force and torsional shear forces are distributed to the seismic resisting system with consideration given to the relative stiffnesses of the vertical components and the floor or roof diaphragm. In addition to any calculated torsional moment, an accidental torsional moment equal to the story shear times a distance equal to not less than 5% of the maximum dimension of the building at that level is distributed to the vertical lateral-force-resisting system.

- The building should be designed to resist overturning effects caused by wind or earthquake forces. Using the calculated seismic shear forces, the overturning moment at each floor is calculated and the moment distributed to the various resisting elements (columns or walls).

- The design should be reviewed to see if there is adequate redundancy in the structural framing system. There are many uncertainties in the amplitude and frequency characteristics of the earthquake ground motions, in the detailed behavior of materials and systems as they resist seismic loadings, and in the methods of analysis. Therefore, it is considered good earthquake engineering to provide as much redundancy as possible in the building's seismic resisting system. In a structural system without redundant components, every component must remain operative to preserve the integrity of the building structure. On the other hand, in a system that has considerable redundancy, one or more components may fail and yet the structural system will retain its integrity and continue to resist lateral forces, although perhaps with some reduced effectiveness. In a frame system, redundancy can be obtained by making all of the joints of the load-carrying frame not only moment resisting, but also part of the seismic resisting system. A moment-resisting space frame has considerable load carrying ability even when stressed beyond yield deformations. Redundancy can also be provided by using more than one type of seismic resisting systems in any one building; thus a back-up system can prevent catastrophic effects if the primary resisting system undergoes excessive deformations.

- The design should be checked to determine if there are significant discontinuities in strength between adjacent stories, which can cause adverse response in a building. Normal practice is to determine the size, length, or strength requirement of a resisting member. If more than the required strength is provided, so much the better. The extra strength in a story, if significantly different than the strengths in adjacent stories, can produce responses which vary greatly from those calculated.

- The drift in each story should be calculated. Story drift should be controlled so as to ensure building stability under maximum earthquake conditions. Large horizontal deflections can cause secondary stress effects due to eccentricity of the gravity load inducing moments and forces in the members. The UBC limitation on story drift is 0.005 times the story height. The total building deflection is important when determining seismic separation between adjacent buildings. Generally, in areas of high seismicity, seismic-drift considerations will control for buildings up to medium height. In areas having low seismicity and for very tall buildings in high risk seismic zones, wind loading can control, at least in the lower stories.

- Member sizes and seismic resisting elements on each floor should be reviewed for conformance with the initial design assumptions. If they do not conform, the prodedure described above should be repeated, using the new sizes. If significant differences in mass and stiffnesses of adjacent stories exist, a dynamic analysis should be made.

The detail design of structural steel and reinforced concrete buildings are covered by the UBC. For structural steel the UBC follows the requirements of the American Institute of Steel Construction (AISC) specifications Parts 1 and 2, for the Design, Fabrication and Erection of Structural Steel for Buildings with certain modifications for connections of beams to columns, development of full plastic capacity of the beam, and special consideration of possible local buckling in members stressed beyond yield.

The requirements of the American Concrete Institute (ACI 318-83, Appendix A) with some modifications will govern the design of reinforced concrete buildings. The basic objectives are to minimize the possibility of concrete compressive failure, concrete shearing failure, or loss of reinforcing anchorage. Compression failures are controlled by requiring confinement of special transverse reinforcing of longitudinal reinforcing bars. Confinement increases the strain capacity, and compressive, shear, and bond strengths of concrete. Maximum confinement is required near beam and column connections. Shear failures are controlled by providing sufficient shear reinforcement and stirrup-ties or hoops. Anchorage failures are controlled by following the special anchorage requirements given in the building code.

In addition to the above, other important factors to consider include:

- The building components should be tied together to act as a unit. As a general requirement, a section passed through any part of a structure should be tied to the rest of the structure so as to resist a force at least equal to 5 percent gravity and, in higher seismic zones for at least 10 percent gravity times the weight of the portion of the building being connected. In addition, beams should be tied together, to their supports or columns, and columns tied to the footings for a minimum of 5 percent of the dead and live load reaction.

- Concrete and masonry walls should be anchored to all floors and roofs for lateral support. As a minimum, such walls should be anchored for a force equal to at least 200 pounds per lineal foot or the appropriate building code requirement, whichever is larger.

- Shear wall or other bracing elements in buildings are often not uniformly spaced around the floor or roof diaphragms. Collector or drag bars should be provided to collect the shear forces and transmit them to the shear resisting elements. These collector or drag bars are composed of reinforced concrete beams in concrete slabs, steel members in steel diaphragms, and continuous wooden members in timber structures.

- Diaphragms act as horizontal deep beams or trusses. They distribute the lateral loads to the vertical resisting elements, and are subject to shears, bending moments, direct stresses and deformations. In some cases deformations must be controlled because they could overstress the walls to which they are connected. Diaphragm deflection must not exceed the ability of the walls that are normal to the direction being analyzed to deflect without failure. Wall anchorages tend to tear off diaphragm edges and therefore ties must be extended into the diaphragm to develop adequate anchorage. For openings in shear walls and diaphragms, chord stresses must be provided for, and the chord members anchored (to develop chord stresses) by embedment. A diaphragm should be tied together so it will act as a unit.

- Bearing walls, like concrete and masonry walls, should be anchored to floor and roof diaphragms. It is important that the wall elements and interconnections have sufficient ductility or rotational capacity or strength to remain as a unit. Consideration should be given to shrinkage or settlement effects on this capability.

- Walkways into buildings or interconnections between buildings are often constructed with a roof slab and a single row of columns. These are referred to as inverted pendulum-type structures because a large portion of their mass is concentrated near the top. Where such structures incorporate heavy concrete slabs, lateral seismic motion may cause a rotation of the slab that can result in vertical accelerations acting in opposite directions on the slab overhang. Hence, a bending moment is induced at the top of the column. One way to cope with this is to apply one-half of the calculated foundation bending moment at the top and vary the moments along the column from 1.5 times the base moment at the base to 0.5 times the base moment at the top. This recommendation is based on background work performed during the development of ATC-3-06.

Design Considerations--Nonstructural

Seismic design requirements contained in most building codes are only for the structural framing system. Recent earthquakes have demonstrated that damage to architectural components and systems and to mechanical and electrical systems and components can, in some instances, exceed the total structural system cost; furthermore, significant damage can occur without major damage to the structural framing system. Enclosure systems (such as infill walls, curtain walls, spandrel beam covers and precast panels), finish systems (such as partitions, ceilings and veneers), and service systems (such as heating, lighting, air conditioning, communications and transportation) all can affect and possibly alter the response of a building and its components during an earthquake. Any of the components may act structurally, whether designed as part of the structural framing system or not. These systems are traditionally referred to as nonstructural components, but can behave structurally and improve or impair the building's ability to endure an earthquake without damage.

The degree to which any structural or permanent nonstructural component may interact with any other or all of the building's component parts should be considered in determining whether a given component can be incorporated into the lateral force-resisting system. If so, this could reduce the initial cost of the structural system and enhance building performance during an earthquake. The architect should collaborate with the engineers during the conceptual design stage so that such components can be incorporated into the traditional structural system to improve the building's response to earthquakes and help all components to better endure the induced forces and deformations.

Several categories of building components are typically not incorporated into the structural systems of most buildings. These categories include:

- components that are not considered permanent;
- permanent or major components for which structural incorporation would be too expensive;
- components having mass, stiffness or configurations that would probably have a detrimental effect on the building response, or would cause unacceptable problems in the building's functional layout or aesthetic concepts.

Nevertheless, some of the components in the above categories, present in a given building, will interact with others and affect the building's earthquake response. As in most aspects of providing a high degree of seismic safety in building design, the architect should work closely with the structural engineers when designing and detailing such systems.

Regardless of whether the component is part of the structural system or not, consideration should be given in its design to improved capability for earthquake-resistance. For example, a partition which is connected to the floor and ceiling must be able to accommodate the differential motions between the slab or floor above the floor on which it is supported, as well as be compatible with motions that may be induced in the ceiling, or in mechanical or electrical equipment systems.

As a minimum, architectural components and mechanical and electrical systems and components should be designed to resist seismic forces to which they may be subjected; this is especially true in UBC seismic zones 3 and 4.

Generally, the component's anchorage or attachment should be designed in accordance with the formula:

$$F_p = Z\ I\ C_p W_p,$$

where F_p is the seismic force applied to the component at its center of gravity, C_p is the seismic coefficient for the component, and Z,I, are the seismic zone coefficient and occupancy importance factors, respectively, as specified in the Uniform Building Code.

In the design of architectural, mechanical and electrical components and systems, design consideration should be given to the differential motion in each story (or story drift) during the earthquake. Since story drifts are relatively small in a shear wall or braced frame building, they probably do not need to be considered except in seismic zones 3 and 4. However, for most frame structures, provisions should be made to accommodate the story drift. As noted previously, consideration should also be given to possible interaction between architectural, mechanical, and electrical systems when the building deforms.

Partitions, ceilings, and filler walls should be designed to resist seismic forces normal to their plane. The Uniform Building Code gives minimum factors for which these elements should be designed.

The above presents an overall view of seismic-resistant design philosophy in the United States. The presentations that follow will discuss in detail the design approaches used by the several firms.

SEISMIC DESIGN CODES IN THE UNITED STATES

POST 1906 SAN FRANCISCO REBUILT TO 30 PSF WIND

1927 UNIFORM BUILDING CODE ($C = 0.075$ TO 0.10)

1933 LOS ANGELES CITY CODE ($C = 0.08$)

1943 LOS ANGELES CITY CODE ($C = \frac{60}{N + 4.5}$, $N \leqslant$ STORIES)

1952 ASCE - SEAONC ($C = \frac{K}{T}$, $K = 0.015 - 0.025$)

1959 SEAOC $V = KCW$, $C = \frac{0.05}{\sqrt[3]{T}}$

$$\left.\begin{array}{ll} \text{1974} & \text{SEAOC} \\ \text{1976} & \text{UBC} \end{array}\right\} V = ZIKCSW\ , \ C = \frac{1}{15\sqrt{T}}$$

1978 ATC TENTATIVE PROVISIONS

Figure 1.- An Overview of the Evolution of Seismic Design Codes in the U.S.

CODE PHILOSOPHY

- RESIST MINOR EARTHQUAKES WITHOUT DAMAGE
- RESIST MODERATE EARTHQUAKES WITHOUT STRUCTURAL DAMAGE, BUT WITH SOME NONSTRUCTURAL DAMAGE
- RESIST MAJOR EARTHQUAKES, OF THE INTENSITY OF SEVERITY OF THE STRONGEST EXPERIENCED IN CALIFORNIA, WITHOUT COLLAPSE, BUT WITH SOME STRUCTURAL AS WELL AS NONSTRUCTURAL DAMAGE
- ESSENTIAL FACILITIES CONTINUE TO FUNCTION
- CONSIDERATION OF NONSTRUCTURAL ELEMENTS

Figure 2.- U.S. Code Philosophy.

FACILITY CRITERIA

- Site characteristics
- Site seismicity
- Expected peak ground motion
- Facility use
- Type of structural system
- Applicability of code provisions

DESIGN CONSIDERATIONS - STRUCTURAL

- Basic layout - symmetry is important
- Avoid re-entrant corners or similar locations of stress
- Consider effects of nonstructural elements - partitions, windows, and filler walls
- Relative stiffness of building stories
- Consider moment frames versus shear wall or frame/ shear wall construction
  - Moment frames deform more but induce lower seismic forces
  - Shear walls deform less but have higher seismic forces
  - Moment frames exhibit better ductility than shear walls
  - Dual framing - moment frame/shear wall often good solution
- Tie all components together - beams to supports, walls to floors and roofs
- Consider torsional effects
- Provide sufficient space between separate parts of building to avoid impact

Figure 3.- Facility Criteria and Design Considerations.

- CALCULATE T
- $C = \frac{1}{15\sqrt{T}}$
- HORIZONTAL FORCE FACTOR, K
- OCCUPANCY FACTOR, I
- SITE - STRUCTURE RESONANCE FACTOR, S
- SIZE MEMBERS AND CALCULATE WEIGHT, W
- BASE SHEAR, V = ZIKCSW
- DISTRIBUTE BASE SHEAR VERTICALLY
- CHECK MEMBER SIZES, INCLUDE TORSION
- CHECK DRIFT
- CHECK OVERTURNING
- REVIEW FOR REDUNDANCY
- REVIEW FOR DISCONTINUITIES

Figure 4.- Design Steps.

STRUCTURAL STEEL

- **Excellent stress-strain characteristics**
- **Moment steel frames**
  - **Lower seismic forces**
  - **Higher lateral deformation can cause nonstructural damage**
- **Braced steel frames**
  - **Higher seismic forces**
  - **Lower lateral deformation**

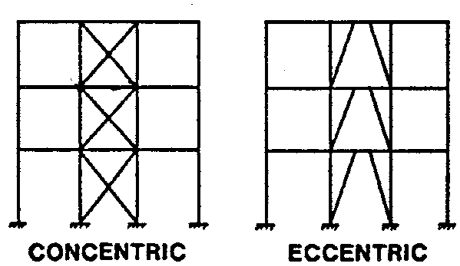

Figure 5.- Structural Steel Design.

REINFORCED CONCRETE

- Properly designed R.C. exhibits good ductility
- Design to avoid:
  - Shear failure
  - Anchorage failure
  - Compression failure
- R.C. moment frames - design for ductility
  - Confine longitudinal reinforcement at joints in columns and beams
- Shear wall structures
  - Boundary members
  - Anchor to framing
- Moment frame/shear wall structures

Figure 6.- Reinforced Concrete Design.

DESIGN CONSIDERATIONS - NONSTRUCTURAL

- Consider whether nonstructural will be overstressed by deformation of building frame - if so provide connections to minimize or design nonstructural to resist induced forces or displacements
- Design connections of precast concrete or other nonstructural element to withstand differential building deformations
- Nonstructural elements
  - Provide seismic restraints for partitions and ceilings
  - Provide seismic restraints for air ducts, fans, boilers, elevator equipment, light fixtures, etc.

Figure 7.- Nonstructural Design Considerations.

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HIGH-RISE BUILDING WITH 130M HIGH SQUARE ATRIUM

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1. Introduction

The Shinjuku NS Building, in the Shinjuku subcenter of Tokyo, is a 30-story high-rise building. Compared to the other 130 to 200 m high skyscrapers towering over the Shinjuku subcenter, this 130 m building is not really very tall, but its enormous 45 m long, 60 m wide, and 130 m high atrium in the center of the building is indeed striking.

In designing it, three-dimensional dynamic response analysis was done taking the deformation of floors into consideration and aiming at the retention of the rigidity and strength of the floor structure. Industrialized space-trusses (NS truss) were selected as the roof trusses to cover the atrium space.

2. Outline of the Building

This building features a new architectural configuration --- an enormous space for the atrium in the center of the building. On each standard floor, office space surrounds the atrium space and elevator shafts are located in the well. The well is surrounded by restaurants (the top floor), office space and two elevator cores --- each core consisting of three banks. From the inside of the building it is possible to see other parts through the atrium space.

The office space on each floor is divided into four 12.95 m wide and 64.15 m long (830 m^2) rooms and elevators and machine rooms are provided for every two rooms.

The atrium is a new spatial concept for high-rise buildings. An important advantage is that it can be used as a convenient passageway for pedestrians.

The outline of the building is as follows:

• Name	Shinjuku NS Building
• Location	2-4 Nishi-Shinjuku, Shinjuku-ku, Tokyo, Japan
• Owners	Nippon Life Insurance Co. and Sumitomo Realty & Development Co., Ltd.
• Design and Supervision	Nikken Sekkei Ltd.
• Contractor	Taisei Corporation
• NS truss supplier	Nippon Steel Corporation
• Construction period	Dec. 1979 to Sept. 1982
• Total floor area	166,233 m^2
• Standard floor area	4,461 m^2
• Number of floors	Underground: 3 Above ground: 30
• Eaves' height	121.45 m
• Usage	For offices, shops, parking lots, etc.
• External walls	Precast concrete panels (finished with tiles)

Figs. 1, 2, and 3 show the appearance of the building, the standard floor plan, and the section and elevation of the building, respectively.

3. Structural Planning

For the floors above the fourth floor, structural system was designed to be rigid frame steel structures, with each floor having almost the same open-center square floor plan. Bearing walls were used for both the outer and inner sides of the open-center square floor plan. These walls are rigid frames with 3.2 m spans consisting of large H-shaped steel columns and girders. The rigid frames were designed so that only the in-plane strength and rigidity of the frame (in the plane of the strong axis of an H-shaped steel column) would be ensured and so that almost all the horizontal forces acting on the building would be borne by the bearing walls.

Rolled H-shaped steel with an 800 mm high and 400 mm wide cross-section was used for columns and welded H-shaped steel of 750 mm deep was used for girders. The open-center square shaped floor structures by which bearing walls were to be connected horizontally, were designed such that reinforced concrete slabs having sufficient strength and rigidity in consideration of the in-plane stresses of floors due to horizontal forces, would be used. The top three floors are made of reinforced concrete slabs with steel plates attached, as considerably large horizontal forces act on these floors.

The fourth and lower floors were designed with consideration given to the fact that they would be used for specific purposes, i.e. for shops, parking, and exhibition halls. These floors are composite structures --- reinforced concrete structures have been partially used --- and the span between columns was extended to 9.6 m. Reinforced concrete shear walls were provided for these floors so that their strength and rigidity, as the podium of the upper part of the building, would be ensured.

The large roof covering the top of the atrium space was designed to be a space truss structure (in triple layers) simply supported by the columns of the inside bearing walls. Steel pipes were used as the chords and diagonal members of the space truss. All these members and the cast steel nodes are prefabricated at the factory and assembled by high strength bolts at the site. This space truss system called the NS truss system was developed through a joint venture by Nippon Steel Corporation and Nikken Sekkei Ltd.

4. Materials Used

4.1 Concrete

Fifth, and higher floors	LC 180	(Light weight concrete)
Compressive strength:	180 kg/cm^2	(Cylinder test)
Specific gravity:	1.6	
Fourth and lower floors	FC 240	
Compressive strength:	240 kg/cm^2	(Cylinder test)
Specific gravity:	2.3	

4.2 Reinforcing bars

D10-D19 (ϕ10-19 mm)	SD 30
Yield strength:	3,000 kg/cm^2
D22-D29 (ϕ22-29 mm)	SD 35
Yield strength:	3,500 kg/cm^2

4.3 Structural steel

Main material	SM 50
Yield strength:	3,300 kg/cm^2 (thickness: 6-55 mm)
Other materials	SS 41
Yield strength:	2,400 kg/cm^2

5. Design Loads

5.1 Dead load

The average dead load was calculated to be about 500 kg/m^2.

5.2 Live loads

It was estimated that the live loads on offices would be 300 kg/m^2 for slabs, 180 kg/m^2 for frames, and 80 kg/m^2 for the seismic load.

5.3 Wind load

The wind load was calcualted in the direction Y, in which the projected area was larger, in compliance with "Standards on Loads Acting on Structures" issued by the Architectural Institute of Japan (AIJ) in 1975.

a) $P = C \cdot q \cdot A$

where,

- C: Shape coefficient (=1.4)
- q: Wind pressure
- A: Projected area (= floor height x 87.5 m)

b) Wind pressure

$q = q_o \cdot Z_w \cdot L \cdot I$

where,

- q_o: Basic wind pressure
- Z_w: Regional coefficient (= 0.85)
- L: Coefficient of surface on which pressure acts (= 0.8)
- I: Occupancy importance factor (= 1.0)

When the environmental coefficient E was assumed to be 1.0, the values for q_o varied according to the height, as follows:

Height (m)	q_o (kg/m²)
30-230	280 + 11 (h - 30)
10-30	120 + 8 (h - 10)
0-10	120

Fig. 7 shows the results of calculation. From these results, the wind load was found to be 1/2.8 of the seismic load.

5.4 Seismic load

The values for the seismic loads acting on the upper part of the building were estimated by referring to "Guidelines on High-Rise Building Design

Techniques" by the AIJ, etc. and then the base shear coefficient (C_B) and the distribution of shear were determined from the results with the response analysis of various seismic waves.

Consequently, the design shear was assumed to be 255 t, 8,590 t, and 13,200 t on the 30th, 4th, and 1st floor, respectively. The story shear coefficient was assumed to be 0.326 on the 30th floor and 0.120 on floors 1-4.

The seismic load of the basement was determined by the application of linear interpolation, in which the lateral seismic factors on the first floor and the third floor of the basement were assumed to be 0.2 and 0, respectively.

The first natural periods (T) of the building were estimated to be 2.5 and 2.6 sec. and the base shear coefficient (C_B) on the fourth floor was designed to be 0.30/T and 0.31/T.

Fig. 6 shows the relationship between the first natural period and the base shear coefficients of the high-rise buildings of steel structures designed to date in Japan. From the figure it may be seen that the value 0.12 of this building is the average of other buildings. Fig. 7 shows the design values of the shear coefficient and the story shear.

6. Analysis and Design of Frames

6.1 Frame analysis

"BUILDING", a structural calculation system for building structures developed by Nikken Sekkei, was used for structural calculations.

This system is an integrated, computerized calculation system by which the configuration of a building can be found, frame models can be made, the weight of and various loads on the building can be calculated, frame analysis can be done, the cross-sections of members for design loads can be examined, and the ultimate lateral shear strength of the building also, can be calculated.

6.2 Deflection of building due to the seismic load

A matrix deformation method in which bending, shearing, and axial deformations are taken into account, was used for the analysis of the stresses generated when seismic loads are exerted on the building.

The horizontal deflection of the top due to the seismic load was 36.9 cm and 42.8 cm (approx. 1/336 and 1/290, respectively, of the overall height of the building) when the seismic load were exerted in directions X and Y, respectively.

Maximum story drift were 1.57 cm and 1.80 cm on the 17th floor when seismic loads were exerted in directions X and Y, respectively. The story drift was less than 1/200 of the story height in either direction (see Fig. 8).

6.3 Design of members

The large rolled H-shaped cross-section members and welded H-shaped cross-section members shown in Fig. 8 were used for columns and girders, respectively, as the standard members composing the main frame of the upper part of the building.

Column members had basically H-shaped cross-sections of 800 mm by 400 mm; the thicknesses of the flange and web were within 22 and 55 mm.

Girders of 750 mm deep were used on standard floors and were friction-jointed with high strength bolts at the center of the spans of 3.2 m long girders. Other girders were also friction-jointed, but at the end of brackets. The ends of girders and columns were shop-welded. Each column was three to four stories long and jointed by site welding.

6.4 Ultimate lateral shear strength of frames

It was confirmed that each member could adequately withstand the seismic load and that stress would be within those allowable for temporary loads. To obtain data for elasto-plastic dynamic response analysis, the yielding behavior and ultimate lateral shear strength of each member and frame were examined and the yielding strength and the order of yielding of each member were confirmed.

Yielding shear (Q_y)

The yielding shear of a story was defined as the story shear when the end of a member in a story reaches the full plastic moment for the first time as stresses increase in proportion to the stresses due to the seismic load. In the case of the frames of this building, it was determined mainly by the yielding of the ends of girders.

Collapse shear (Q_B):

To define the collapse shear (Q_B) of a story, four cases involving the collapse mechanisms of the top and bottom ends of columns, as shown in Fig. 10, were considered. The collapse shear (Q_B) of a story was obtained by totalling up the minimum shear --- out of the values calculated for the four cases --- for each of the columns in a story. The collapse mechanism of a standard floor of this building was assumed to be the yield type shown in Case 4. The collapse shear was 1.2 to 1.8 times the yielding shear.

Based on the values for Q_y and Q_B above, it was assumed that this building would have tri-linear type load-deflection properties as shown in Fig. 11.

Figs. 12 and 13 show the design shear, yielding shear, and collapse shear in directions X and Y.

7. Dynamic Response Analysis

7.1 Vibration models

One-dimensional mass system models in which a story is represented as one mass were set and dynamic response analysis was done to investigate the dynamic behavior of the building. Also, from the standpoint that this building would have open-center square-shaped planes, a three-dimensional model was set and dynamic response analysis was done to dynamically verify the floor structure designed for static forces.

The models for dynamic response were as follows:

a) 28-mass full matrix model

This model is a 28-mass model; it was assumed that the fourth floor was fixed and it was used to obtain the rigidity of the frame in the form of the reaction force matrix of the frames on any floor when a unit deformation was applied to a floor. The stiffness matrix was calculated in the form of the stiffness in directions X and Y at the center of gravity of the building. Up to tenth mode were considered in modal analysis.

b) 28-mass equivalent shear model

In this 28-mass equivalent shear model, the spring constant obtained when the story shear caused by the seismic load is divided by the story drift, is assumed to be an apparent shear spring (equivalent shear spring).

c) 31-mass equivalent shear model

This model is a 31-mass model; it was assumed that the first floor was fixed so that the frames of the lower part were taken into consideration.

d) Three-dimensional model

This model has six masses in the vertical direction and ten masses on each of the six open-center square planes; the fourth floor is assumed to be fixed. It as assumed that the masses were linked by the in-plane rigidity of the floor in the horizontal direction and by the rigidity of the frames in the vertical direction.

The natural period of each model was as shown in Fig. 15. The damping coefficient was assumed to be viscous damping type and was estimated to be 2% of the critical damping for the 1st natural period.

7.2 Input seismic waves

The six input seismic waves listed below were used for analysis.

The levels of these seismic waves were evaluated with the velocity of the waves, as the natural period of the building was long. The velocity of each seismic wave was obtained from the maximum response velocity of the one-mass system, in which the natural period was 10 sec. and the damping coefficient h^2 was 0.5. The acceleration of each wave was thus modified so that the velocity would be 25 or 50 kine. Fig. 17 shows the response spectra (25 kine, h = 0.02) of these seismic waves.

For the full matrix model and the equivalent shear models, these seismic waves were input in both directions.

For the three-dimensional model, the El Centro NS wave was input in the Y direction and a case in which NS and EW waves were input simultaneously was also examined.

	Name of seismic wave	Date of occurrence	Component	Acceleration: Maximum recorded (gal)	Acceleration: At 25 kine (gal)	Acceleration: At 50 kine (gal)	Period of response analysis (sec.)
1.	EL CENTRO CALIF.	1940. 5.18	NS	319.2	201	402	15.0
2.	TAFT,CALIF	1952.12.21	EW	155.0	210	420	15.0
3	TOKYO 101	1956. 2.14	NS	74.0	256	512	12.0
4.	SENDAI 501	1962. 4.31	NS	50.0	264	528	17.0
5.	TOKYO 141	1968. 7. 1	NS	30.3	248	496	10.5
6.	TOKYO 141	1968. 7. 1	EW	27.2	215	430	11.5

7.3 Elastic dynamic response analysis

Elastic dynamic response analysis was done when velocity was 25 kine.

Fig. 18 shows the results of the response analysis on the 31 mass equivalent shear model. As is shown in Fig. 18, the response of the El Centro, Calif. NS wave was extremely high in both X and Y directions. Although the base shear on the fourth floor of response analysis exceeded the design values by 34% in direction X and 29% in direction Y, each frame could remain in the elastic range.

The maximum story drift in direction X was 1.83 cm (story height/199) and this occurred on the 10th floor. That in direction Y, which was observed on the 13th floor, was 1.95 cm (story height/187). The maximum deflection at the top was 36.7 cm (building height/337) in direction X, and 40.6 cm (building height/305) in direction Y. These drifts were observed when the El Centro, Calif. NS wave was input.

This building was designed so that the rigidity in directions X and Y, respectively, would almost equal. As the difference in the rigidity of the X and Y directions is approx. 5% when measured in the first natural period (xT_1 = 2.46 sec, yT_1 = 2.60 sec), and the dynamic behavior and the response values in both directions are almost the same.

For the purpose of a comparison, response analyses were done using 28-mass full matric models and 28-mass equivalent shear models. The results of both analyses showed almost the same tendency, although there was a slight difference over the middle part of the response story shear. A 31-mass equivalent shear model and a 28-mass equivalent shear model were also analysed so that their responses could be compared and it was found that there were no significant differences in the response values and dynamic behavior of the upper part between the two models.

7-4 Elasto-plastic dynamic response analysis

Elasto-plastic dynamic response analysis was done using a 31 mass equivalent shear model which had the load-deflection properties mentioned in Section 6.4, and a velocity of 50 kine was input.

Fig. 19 shows the response results. With the elasto-plastic dynamic response, the response to the El Centro, Calif. NS wave was extreme. The maximum story drift in direction X was 3.65 cm (story height/100) and that in direction Y was 3.87 cm (story height/94); in both cases this occurred on the seventh floor. The maximum deflection at the top was 59.4 cm (building height/209) in direction X and 62.3 cm (building height/199) in direction Y. However, the usual extent of curtain wall installation details was sufficient to cope with this deflection. In addition, as the drift measured by such response analysis was a combination of the bending deformation and shearing deformation of the building as a whole, the shearing deformation influencing the installation of curtain walls, etc. was smaller than the combined value and therefore the safety of the building was ensured.

Ductility factor values were maximum when the El Centro, Calif. NS wave was input, giving 1.87 and 1.82 in directions X and Y, respectively, on the seventh floor.

7.5 Three-dimensional analysis

Dynamic response analysis using a three-dimensional model was done with a velocity of 25 kine.

The following points were considered during the course of this.

a) The planar eccentricity due to the weight of the building is given. (When there is no live load on one side of a floor, the eccentricity is 2.6%.)

b) The planar eccentricity due to the rigidity of the frame is given. (When the rigidity of the left frame and that of the right frame are deemed to be 1.05 and 0.95 times, respectively, the eccentricity is 5%.)

c) The earthquake waves are input in two directions.

From the results of these analyses, it was found that the stresses of the frame were almost the same as those obtained from the analysis in Section 7-3. The forces acting on the floors were obtained from these analyses and the safety of the floor structure was examined.

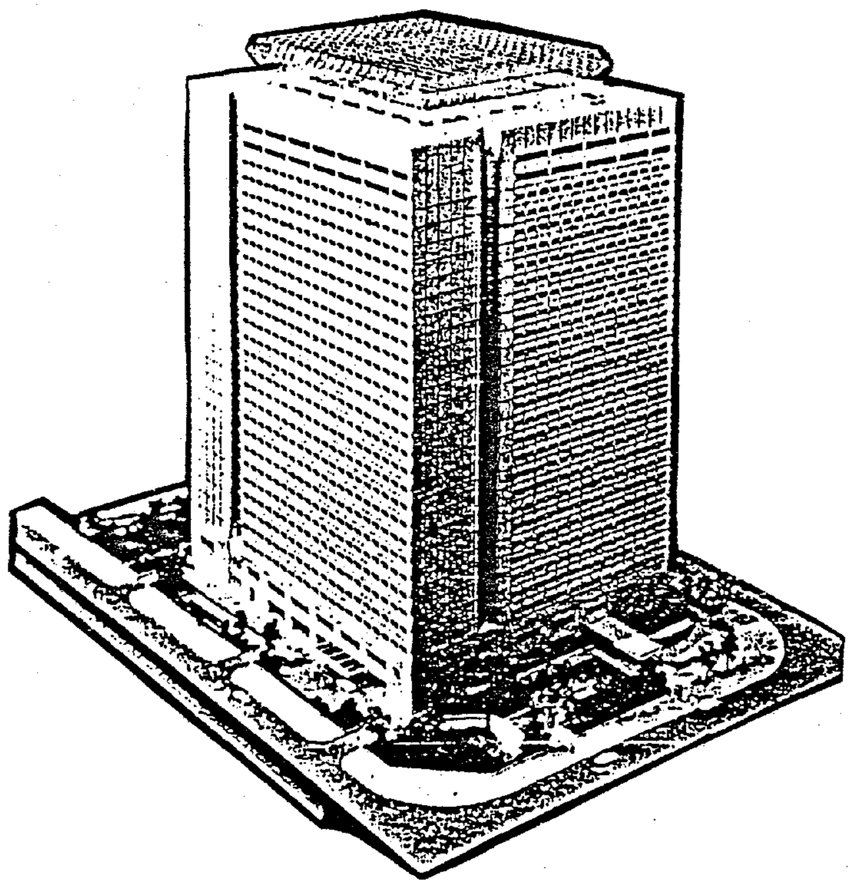


Fig. 1 Appearance of Building

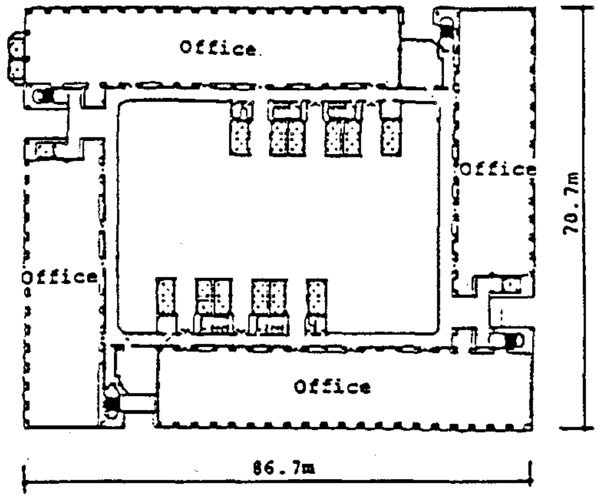


Fig. 2 Standard Floor Plan

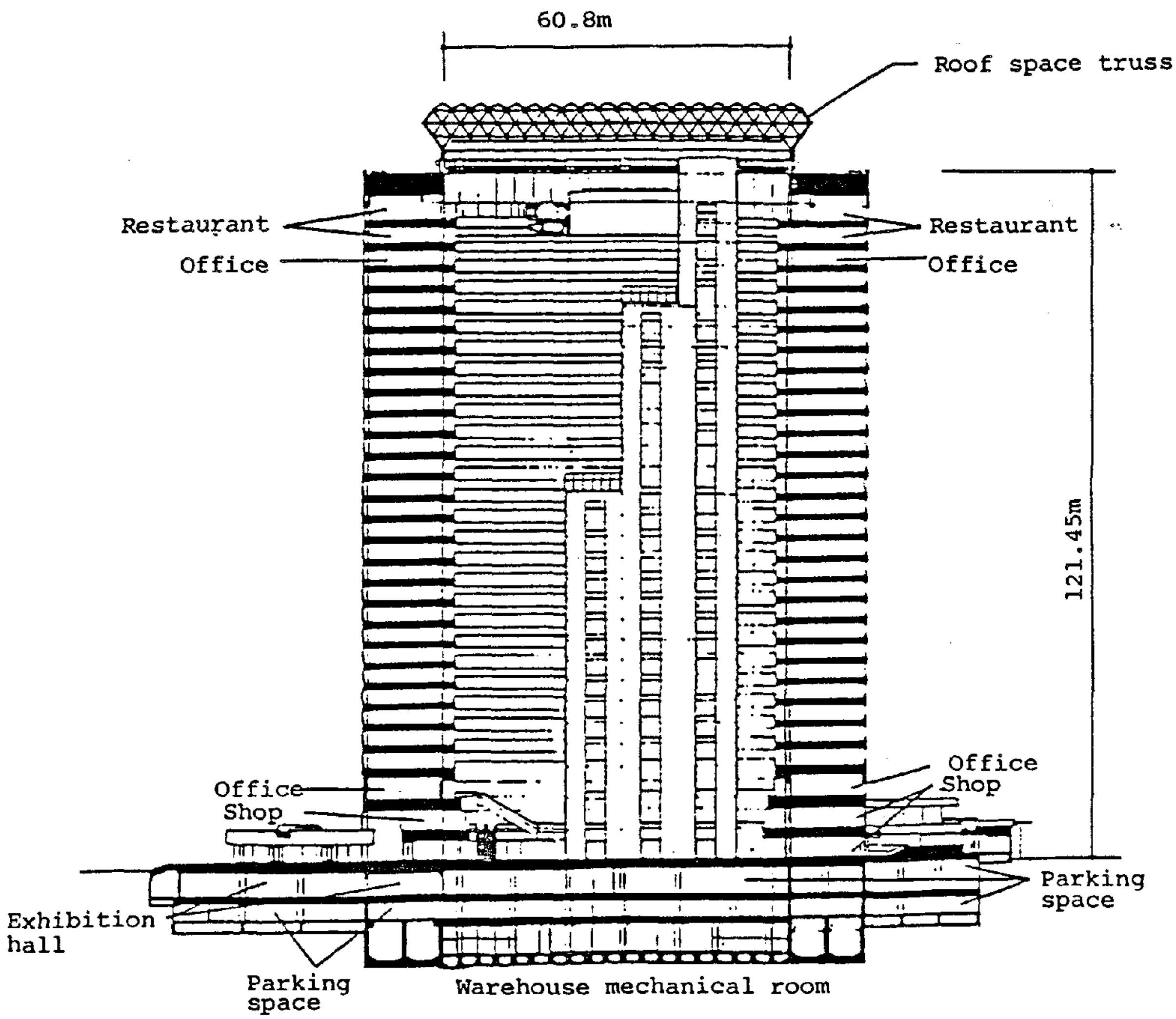


Fig. 3 Section and Elevation of Building

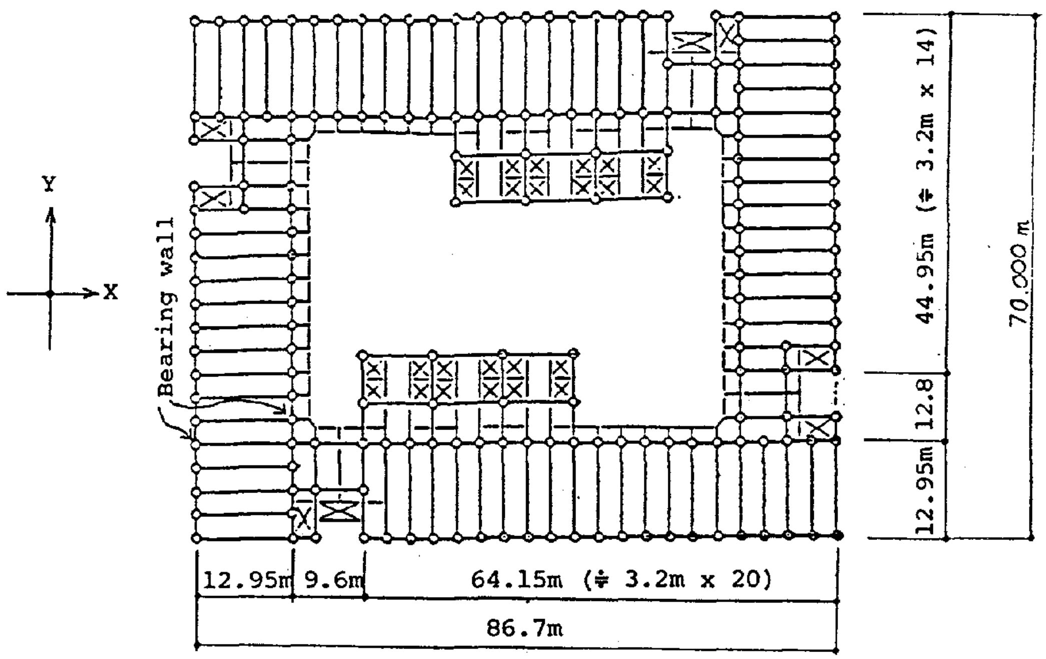


Fig. 4
Framing Plan

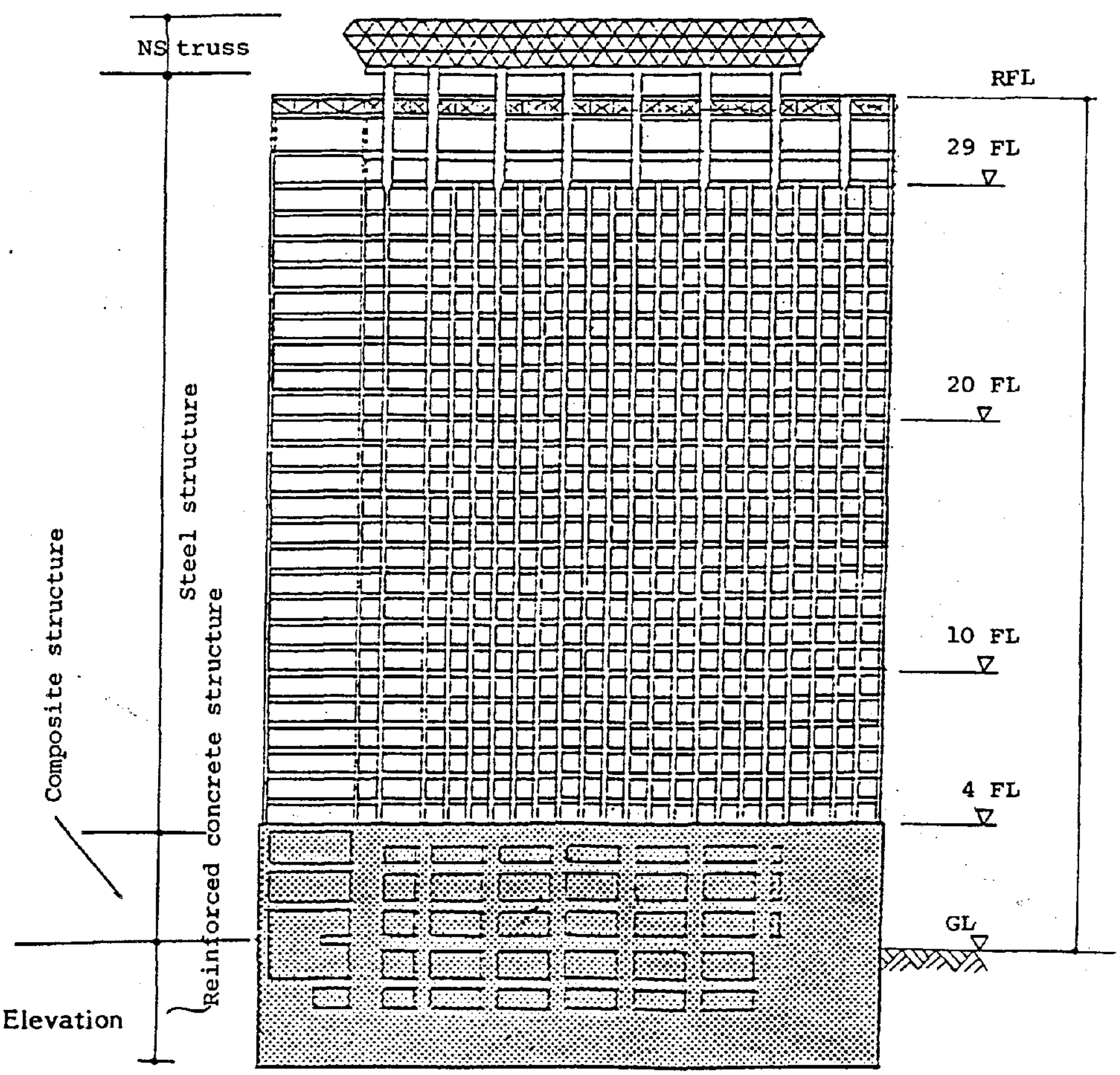


Fig. 5
Frame Elevation

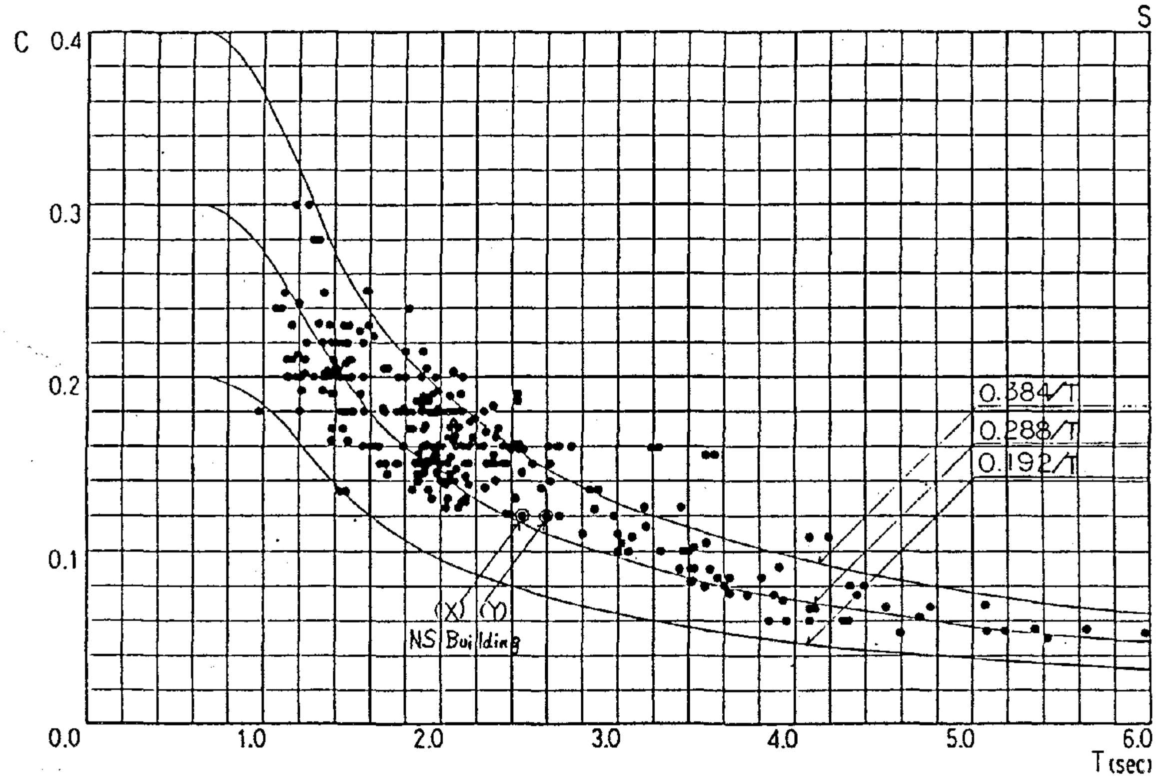


Fig. 6 First Natural Period (T) and Base Shear Coefficient (C) of Buildings

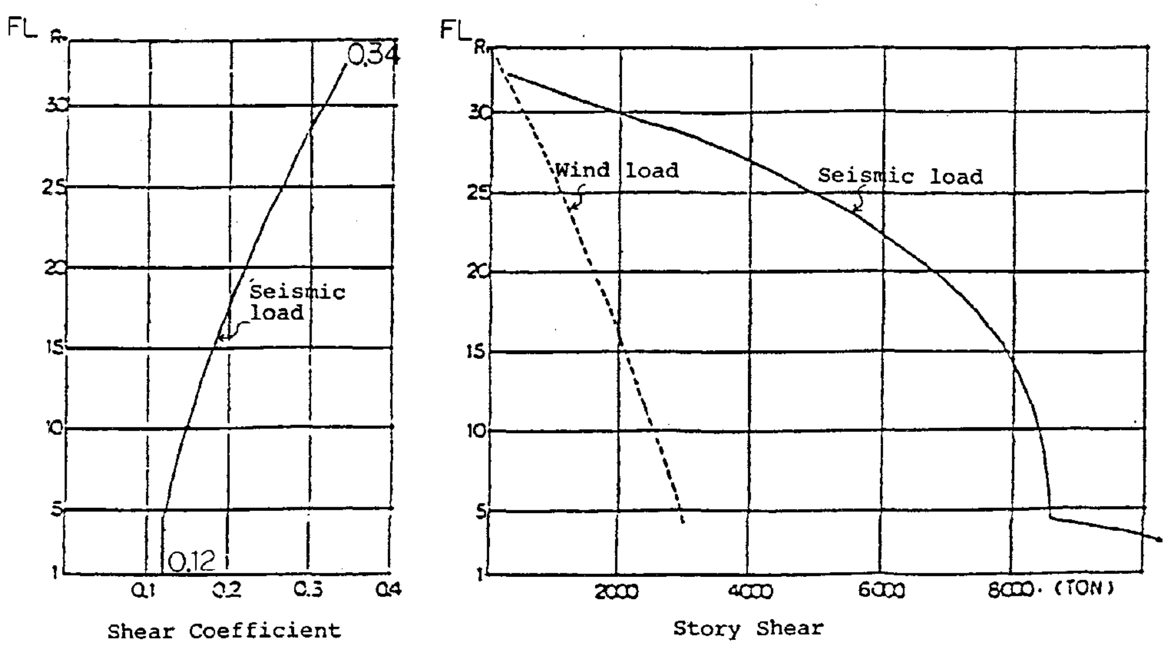


Fig. 7 Design Wind Load and Seismic Load

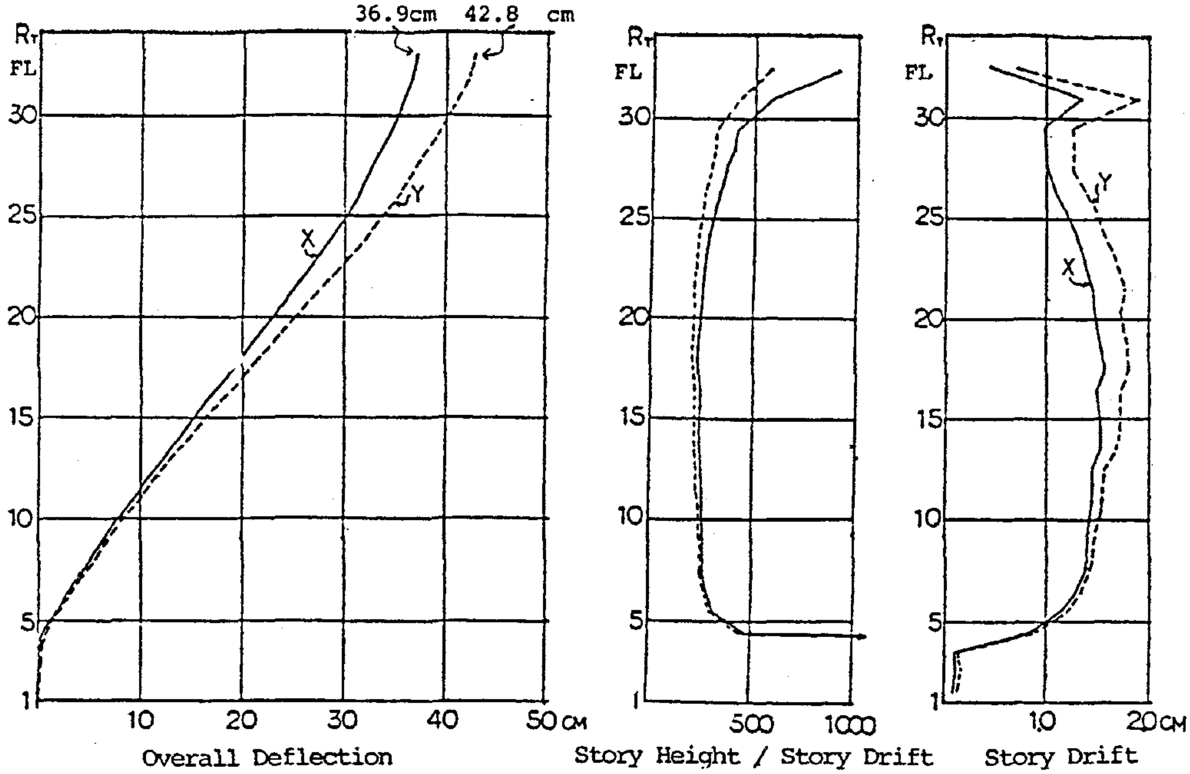
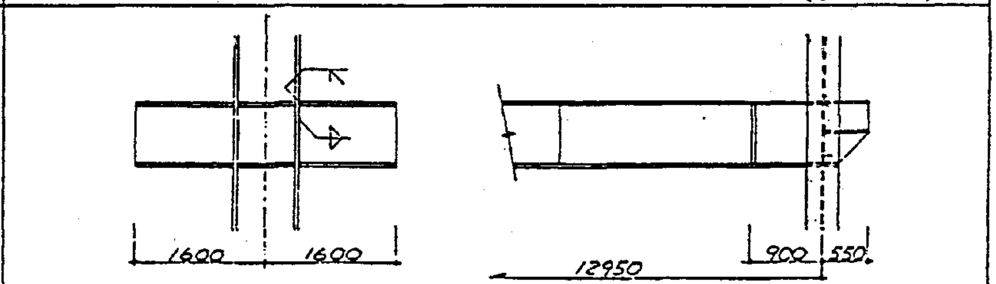
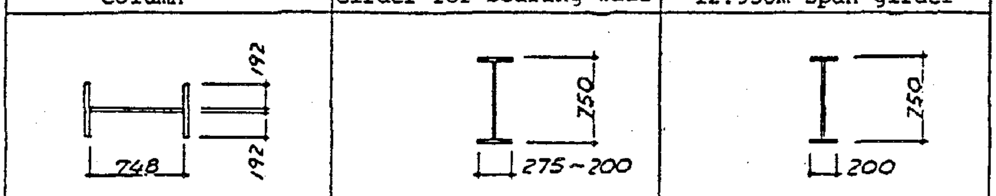


Fig. 8 Deformation of Building Due to Seismic Load

For Standard Floor (Unit : mm)



	Column	Girder for bearing wall	12.950m span girder



	Column	Girder for bearing wall	12.950m span girder
25 FL	CX1 WH-792×406×22×22 CY1 WH-792×406×22×22	GX1 WH-750×200×14×16 GY1 WH-750×250~200×14×22	G8 WH-750×200×9~6×14
15 FL	CX1 WH-798×409×25×25 CY1 WH-804×412×28×28	GX1 WH-750×250~200×14×19 GY1 WH-750×250~200×16×28	G8 WH-750×200×9~6×14
6 FL	CX1 WH-828×412×28×40 CY1 WH-838×412×28×45	GX1 WH-750×275~200×14×22 GY1 WH-750×275~200×19×32	G8 WH-750×200×9~6×14

Fig. 9 Cross-sections of Standard Members

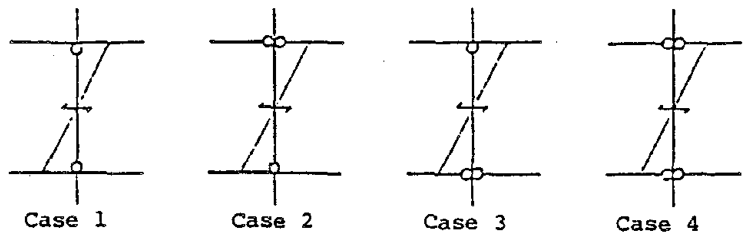


Fig. 10 Collapse Mechanism

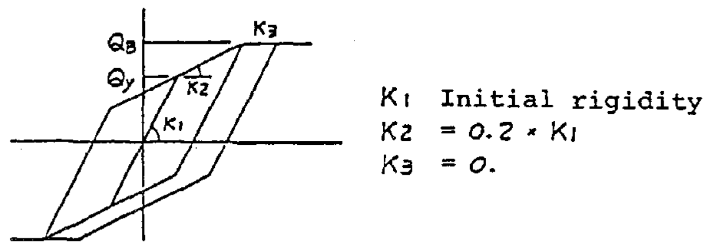


Fig. 11 Load-Deflection Properties

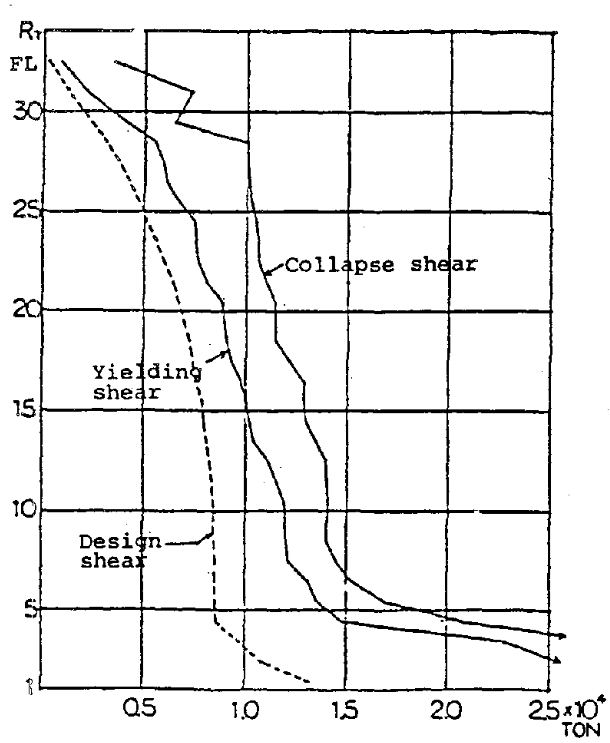


Fig. 12 Shear Strength of Frames in Direction X

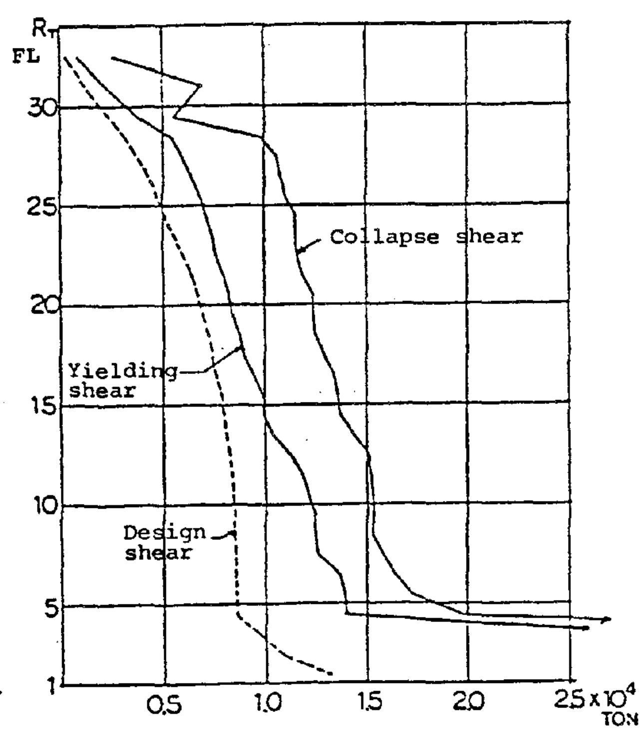
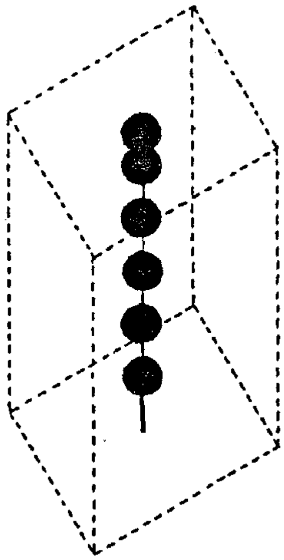
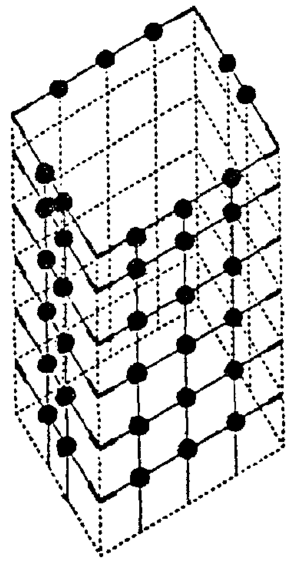


Fig. 13 Shear Strength of Frames in Direction Y



One-dimensional model for a) to c)



Three-dimensional model for d)

Fig. 14 Dynamic Response Analysis Models

Model		a) 28-mass full matrix model		b) 28-mass equivalent shear model		c) 31-mass equivalent shear model		d) 10-mass by 6 mass three-dimensional model	
Direction		X	Y	X	Y	X	Y	T (n) (Sec)	Remarks
N	n	T (N) (Sec)	T (N) (Sec)	T (N) (Sec)	T (N) (Sec)	T (N) (Sec)	T (N) (Sec)		
I	1		2.58		2.59		2.60	2.60	Y 1st mode
	2	2.45		2.45		2.46		2.45	X 1st mode
	3							2.19	R 1st mode
II	4		0.90		0.97		0.98	1.00	Y 2nd mode
	5	0.85		0.90		0.90		0.92	X 2nd mode
	6							0.83	R 2nd mode
III	7		0.53		0.61		0.61	0.64	Y 3rd mode
	8	0.50		0.56		0.56		0.58	X 3rd mode
	9							0.53	R 3rd mode

R : Tortional Mode

Fig. 15 Natural Periods of Models

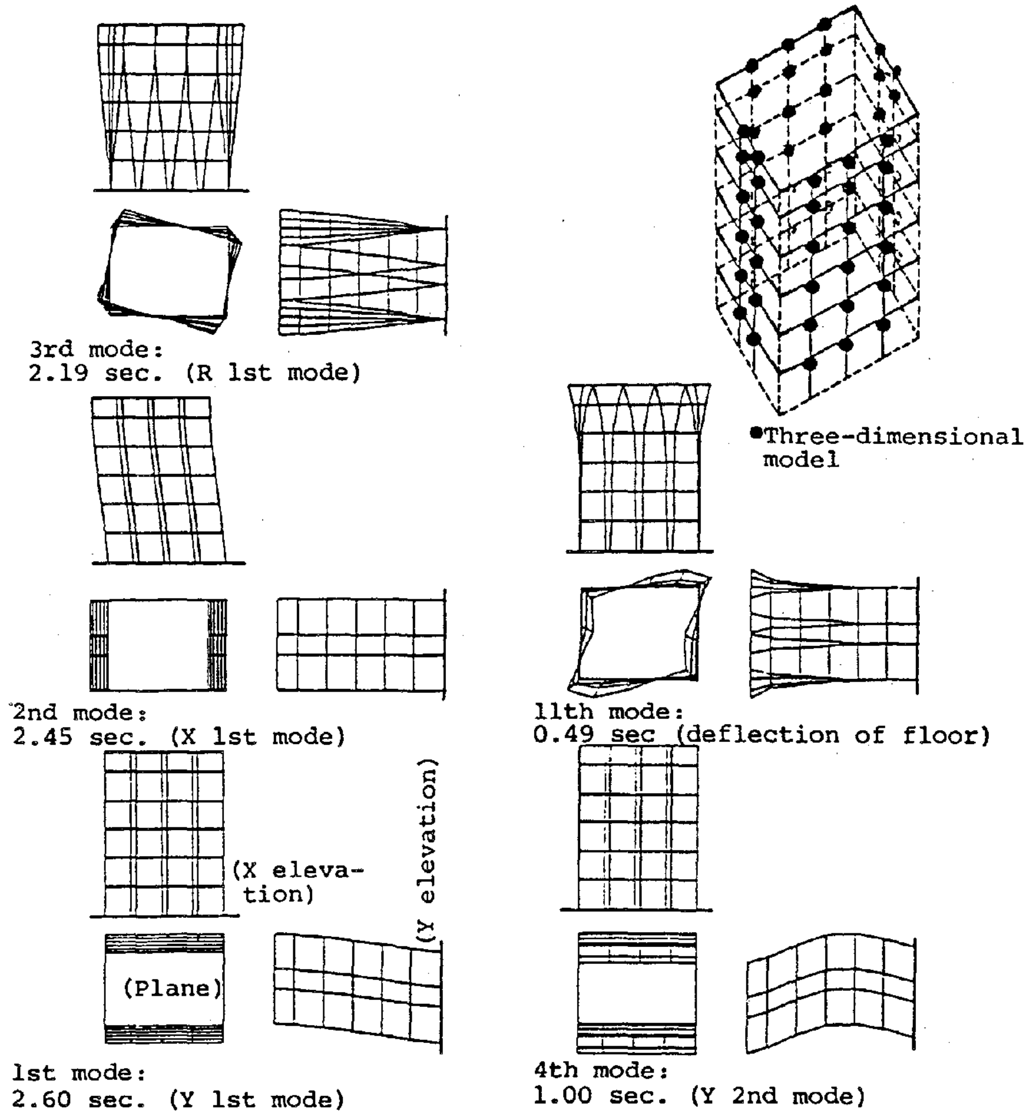


Fig. 16 Modes of Three-Dimensional Models

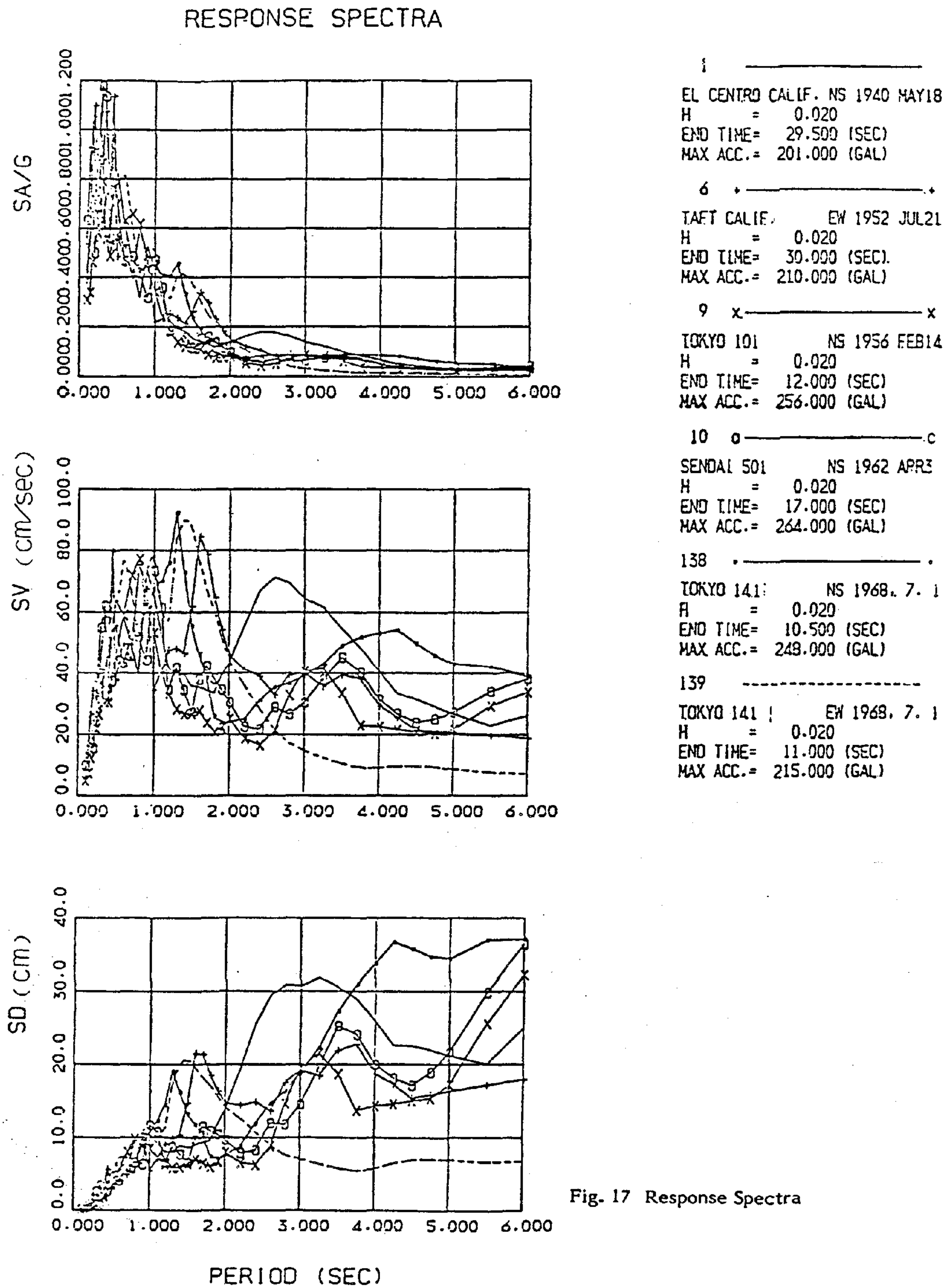
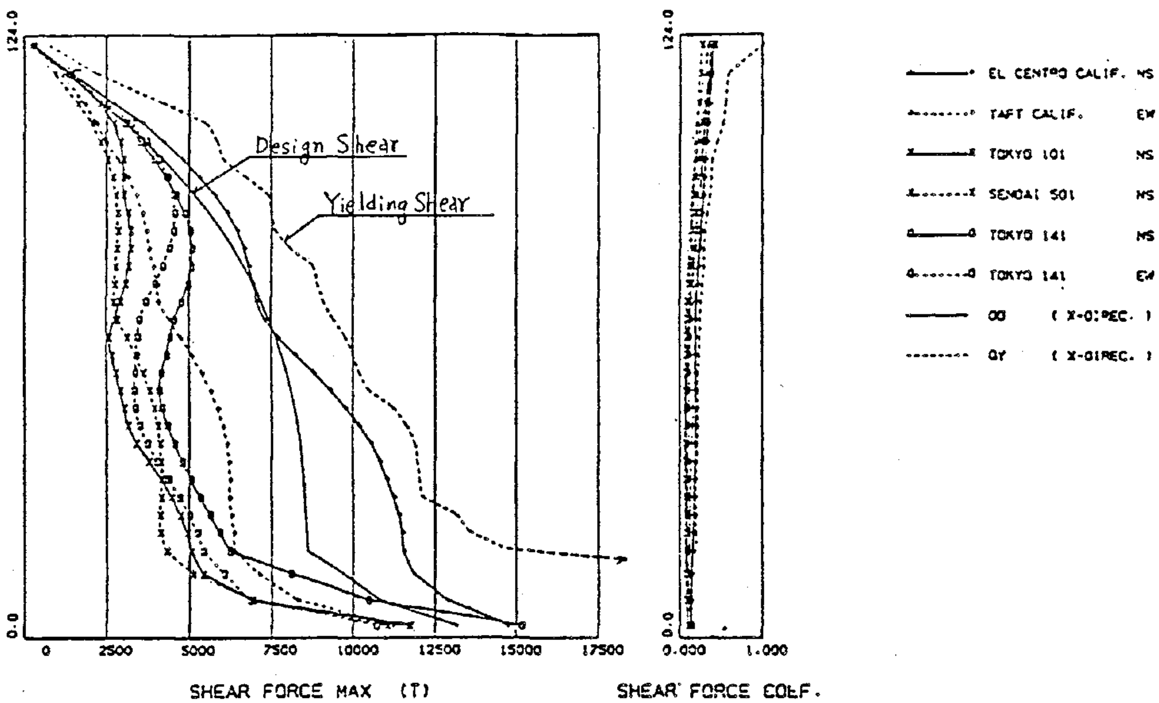


Fig. 17 Response Spectra

Response of 31-mass equivalent shear model in direction X



Response of 31-mass equivalent shear model in direction Y

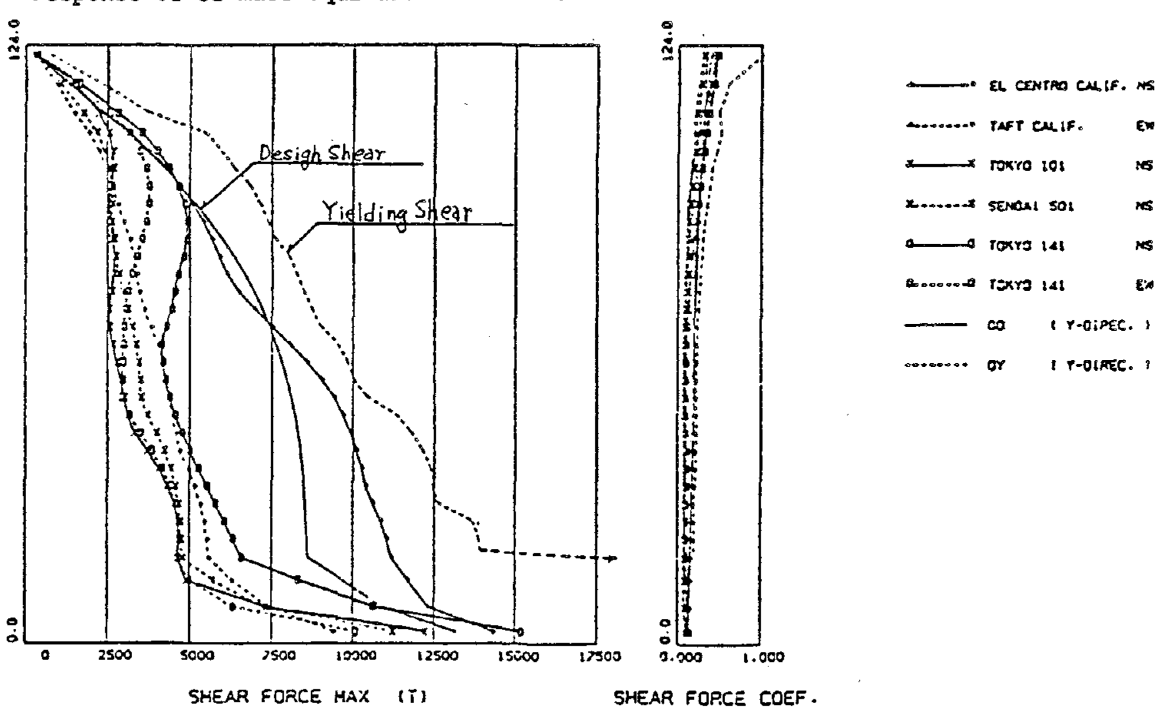
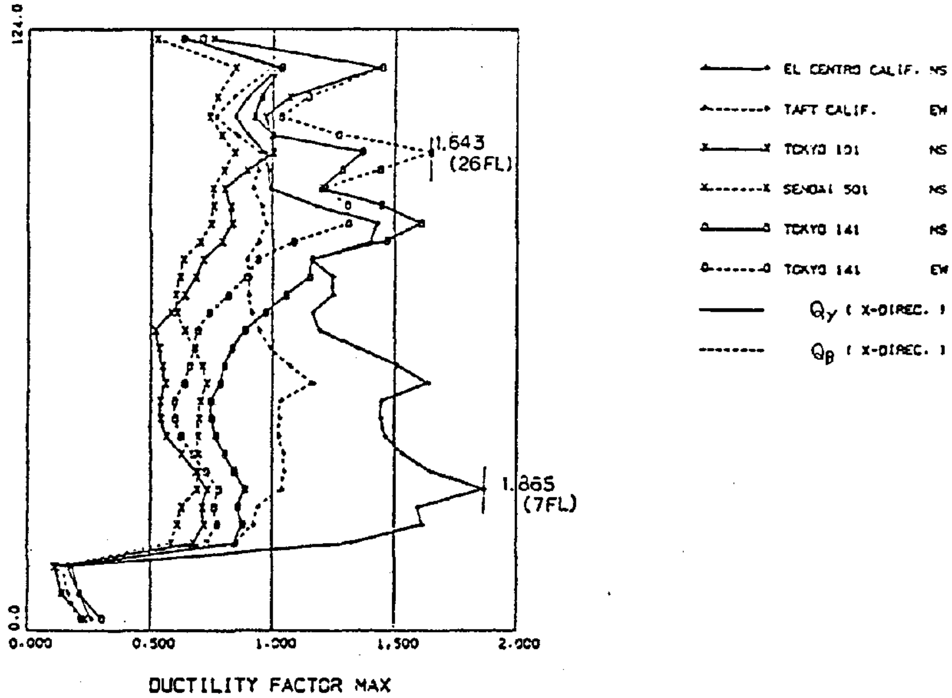


Fig. 18 Elastic Response of 31-Mass Equivalent Shear Model at 25 Kine

Response of 31-mass equivalent shear model in direction X



Response of 31-mass equivalent shear model in direction Y

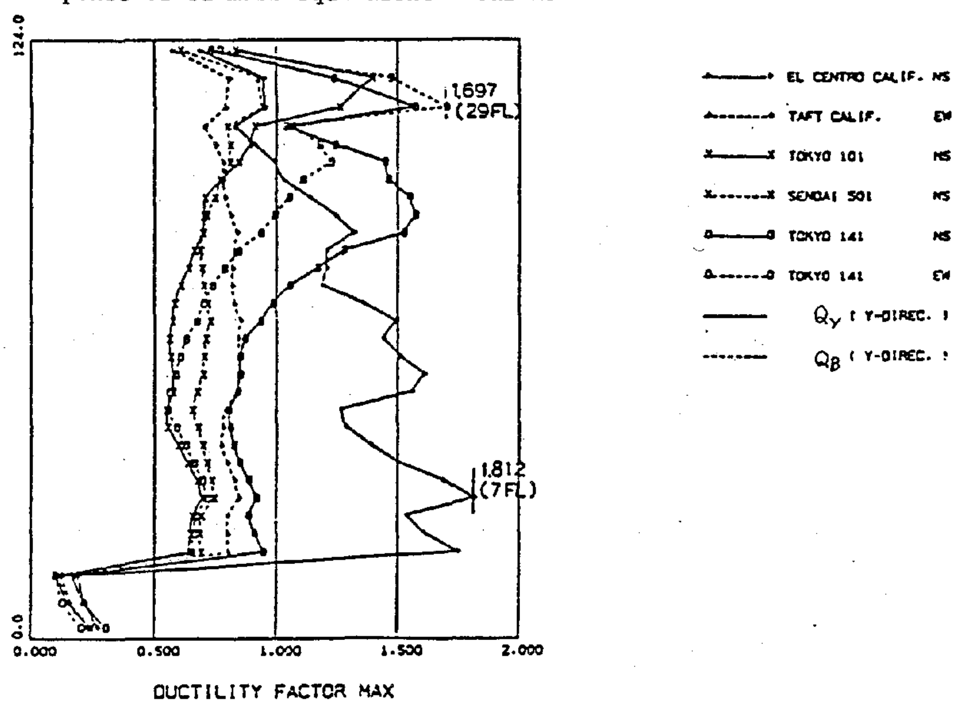


Fig. 19 Elasto-plastic Response of 31-Mass Equivalent Shear Model at 50 Kine

SEISMIC DESIGN PRACTICES IN THE UNITED STATES FOR HIGH-RISE STEEL-FRAMED BUILDINGS

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ANALYSES

The framing systems used in seismic design of high-rise structural steel buildings (over 20 stories) in the United States can be categorized into the following types: (1) those for shear resistance, including steel ductile moment resisting frame systems, steel eccentrically braced frame systems, steel concentrically braced frame systems, and steel frame systems with shear walls; and (2) those for augmented flexural resistance, such as tube framed systems.

Prior to a discussion on the details of each of these systems, a review of the methods of analyses, design philosophies, and constraints imposed by building codes is needed. The design process invariably requires a preliminary approximate sizing of members with a verification of the required strengths of members and deformations by a more sophisticated method. Examples of these systems are shown at the end of this paper.

The various methods of analysis, in order of increasing degree of sophistication, are as follows:

1. A static elastic analysis based on the prescribed distribution of forces specified by the building code. This presumes a reasonable degree of uniformity in the mass-stiffness relationship of the building.
2. An elastic modal analysis of the system using the code-prescribed base shear to describe the level of motion.
3. An elastic modal analysis using standard, area specific, smoothed spectra to describe the ground motion.
4. An elastic modal analysis using site specific, smoothed spectra.
5. An elastic time-history modal anlaysis using site specific earthquake records.
6. An elastic time-history analysis in which modes are continuously superimposed.

Methods 2 through 6 are used on two- and three-dimensional models. Three-dimensional models are used particularly if the building does not have a symmetrical, uniform arrangement and if the framing systems are significantly different in the orthogonal directions.

In tall buildings, nonlinear response analyses are seldom performed. This is because the code-imposed drift limitations frequently become the prominant limit state that analytically assures that only elastic response can be expected, even for maximum credible site earthquake motions.

Even though elastic response may be anticipated, it is part of the U.S. design philosophy to provide for tough potential nonlinear response by providing details such that the steel frame is capable, without significant loss of strength, to achieve either (1) hinging in beams, (2) hinging in columns, (3) shear or flexural hinging in link beams, or (4) axial nonlinear response in bracing members.

The Uniform Building Code (UBC82) requires a limitation on lateral deformation (drift) of the building under the prescribed working level forces. All building systems are required to meet this requirement, and it frequently becomes the critical design parameter in tall steel-frame buildings.

DUCTILE MOMENT-RESISTING FRAME SYSTEMS

In the past twenty years there have been many research programs on the details of moment-resisting steel framing in the United States. The research has concentrated on collecting data for the development of design criteria for member connections, bracing requirements, and shape limitations. The goal is to inhibit the failure mechanisms of local buckling and crippling and the fracture of connections as well as to develop the design criteria so that beams, columns, link members, and panel zones will be able to deform nonlinearly in a stable manner. The research is not yet complete enough to provide complete data on all design criteria. UBC82 contains only a few arbitrary and vague provisions. Based on the current information available, these criteria could be more definitively codified and work is proceeding on them. Special requirements for bracing of members deforming nonlinearly and the limitations on location of hinge formation are not given. The current design code contains only three special requirements for the design of "ductile" systems:

1. Steels are limited to lower yield steels ($F_y < 55$ ksi).
2. Member connections are required to develop the plastic strength of the section unless it can be shown that adequate joint displacement can be obtained with lesser connections.
3. Members in which hinges can form during inelastic displacements of the frame shall be limited in the width thickness ratio of elements in order to preclude local buckling.

Current studies are being made to provide changes to the moment frame seismic provisions. These include:

1. Limitations on the axial load (both tension and compression) that can be carried by columns.
2. Assignment of the column slenderness ratio factor K to be equal to 1 as long as the moment frame meets the prescribed drift limits.
3. Limits on the strength of partial penetration welds in column splices.
4. Design provisions for beam-to-column welded connections using butt-welded flanges but with webs either bolted to a fin plate using high-strength bolts or welded to the plate with fillet welds.
5. Design requirements for column-beam panel zones and requirements for reinforcing doubler plates. Doubler plates can be used both for increased shear strength or to assist in limiting the portion of the frame displacement contributed by panel zone deformations.

6. Special depth-to-thickness limitations for frame members without web stiffeners.
7. Definition of the conditions under which hinging in columns would be acceptable.
8. Definition of the bracing requirements for beams, columns, and beam-column joints expected to deform inelastically.
9. Definition of models to be used in frame analysis.

Additional studies are needed to provide information on the appropriate level for the design drift limitations. To date, various proposals have been made without adequate studies to determine truly appropriate limitations.

CONCENTRICALLY BRACED STEEL FRAMES

Concentrically braced systems have been the subject of considerable research and study in recent years. The main interest in seismic design studies has been to develop a methodology to eliminate a sudden or fracture type of post-elastic behavior. UBC82 contains a number of items that are intended to inhibit this method of failure. These provisions include the following:

1. Braced frames are designed for 1.25 times the force level for comparably framed shear walls.
2. Even though the margin of safety in steel connections is 1/3 greater than the margin of safety of the members they connect, the current Code UBC82 requires this to be increased by another 1/3, giving the connections approximately 1.75 times larger margin of safety than the members.
3. In buildings over 160 ft (50 m) in height, a moment frame capable of resisting at least 25 percent of the design shears is required in the building system. This is to provide an indeterminant amount of toughness to the building frame.

In addition, it has been proposed that the following provisions be added to further inhibit sudden collapse:

1. Columns in braced frames should be designed for an axial load of such magnitude that buckling during a major earthquake will not occur. This may be achieved by sizing the columns to be either (a) capable of carrying loads to develop the bracing members used, or (b) capable of carrying an amplified earthquake lateral force.
2. Limit bracing members to a slenderness ratio not exceeding 120.
3. Components of built-up bracing members shall be so fastened (stitched) so that the strength of the member as a whole will be less than the strength of the individual elements between stitching.
4. Local buckling shall not occur in outstanding elements in compression.
5. Connections shall be designed to either (a) be sure to develop the strength of the brace, or (b) be strong enough to resist an amplified lateral force.
6. Along any braced line, two braces shall be required--one in tension and one in compression. It is not intended that "K" or "Chevron" type bracing should qualify for this requirement. It is intended that one complete braced frame should always exist on any braced line where at least one diagonal would be in tension.

ECCENTRICALLY BRACED FRAMES

There has been considerable interest in the development of design criteria for this framing system. Both medium- and high-rise steel buildings have been built using the method.

In eccentrially braced frames, the brace does not interesect at the beam-column centerline, or adjacent brace centerlines do not meet at a point on the beam centerline. The length of the beam over the distance of the eccentricity is called a link beam. These link beams can either be shear or moment link beams, depending on whether shear transfer nonlinearity is accomplished by shear plasticity in the beam web or by flexural plasticity of the beam section.

Currently, in order to use this system, design criteria have to be taken from the reports of the researchers, as no code provisions are available.

It is generally felt that there are adequate data available now to codify this system. Current studies are being conducted to provide this information to the code agencies for adoption. The criteria being developed include such items as:

1. Limitations on design axial strength of columns.
2. Design of braces including a requirement that the brace strength shall be 1.5 times that required to develop the shear transfer in the link beams.
3. Design requirements for brace-to-beam connections.
4. Stability requirements of link members, including connections to columns and lateral bracing.
5. Design and placing of web stiffeners in link beams.

Eccentrically braced systems can be employed to minimize the flexibility of the building frame. The added cost of increased connection, stiffening, and lateral bracing may tend to reduce the economy of this system, as opposed to the use of concentrically braced framing systems.

STEEL FRAMES BRACED WITH SHEAR WALLS

Steel frames braced with concrete shear walls have frequently been used in the construction of high-rise buildings. Also, in special cases, steel shear walls have been used. Little, if any, research has been conducted on the effectiveness of the various methods of fastening concrete shear walls to frames.

The main design problem encountered is how to effect continuity between steel and concrete without weakening the system at the joints. The only specific UBC82 requirements dealing with composite systems of this type specify that the "horizontal wall reinforcement in the walls shall be fully anchored to the vertical elements." The details of how to accomplish this anchorage are left to the ingenuity of the engineer. Different anchorage problems exist when the wall is centered on the steel framing, when the wall is moved to one side sufficiently for one layer of reinforcing to pass the framing, and when the wall is essentially remote from the framing.

As with concrete buildings, all concrete shear walls are designed to a higher than normal margin of safety for seismic shear and diagonal tension. Since these code provisions were first used, there has been quite a lot of testing performed on concrete walls. It can be assumed that the design criteria for shear in composite shear walls will follow the lead of criteria for concrete. One additional feature required on tall buildings is that the frame be designed to be capable of resisting 25% of the total building shear for the same reason that it is required on concentrically braced systems.

Three types of shear wall systems have been used in tall buildings: (1) core wall buildings with separate 25% minimum frame, (2) wall coplanar with the 25% moment frame but slender enough so that there is pronounced wall-frame interaction, and (3) full-length exterior shear wall buildings with the 25% frame imbedded therein.

In tall buildings, full use of the strength of composite construction is seldom used in the United States for resisting seismic forces except for this combination of concrete walls having structural steel flexural reinforcement. The effect of composite beams and columns on the behavior of the steel frame has been used only when trying to assess the stiffness of the building system.

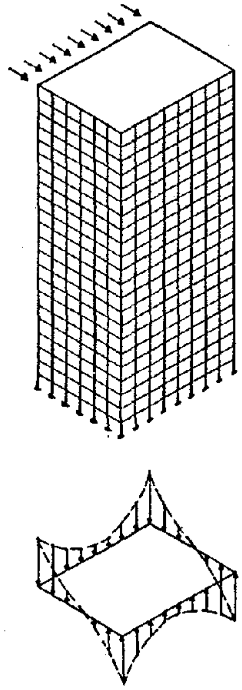
TUBE FRAMED SYSTEMS

As the framing system of a building becomes more and more slender, the deflected shape of the building under lateral loads assumes a flexural shape, and the proportion of the total deflected shape contributed by shear deformation becomes smaller. This is true even in moment-frame systems. As a result of this flexural behavior, the sizes of two-dimensional frames becomes increasingly large. One solution to this problem is to make full use of the column area on the perimeter of the building or tubular segment by having the columns relatively closely spaced with connecting spandrel elements stiff enough to spread the flexural load across the face of the building or tubular segment. The distribution of these flexural forces has been analyzed using normal analytical procedures.

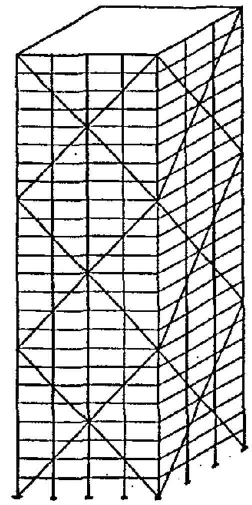
All four of the shear resisting systems previously reviewed can be used in a tube configuration.

CONCLUSION

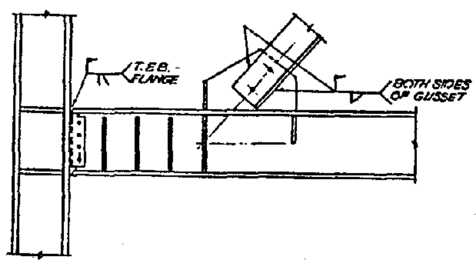
In general, it is felt by the design profession that all of these methods, when used with the current state-of-the-art design procedures, produce building systems well adapted to resisting the anticipated seismic forces in the United States.



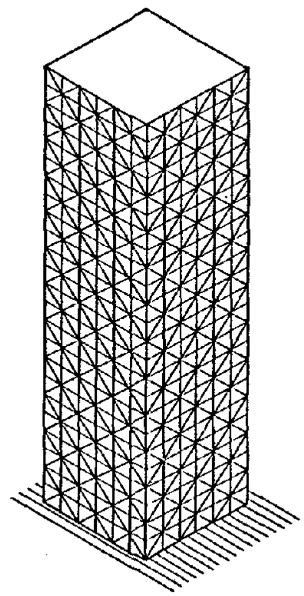
TUBULAR SYSTEM



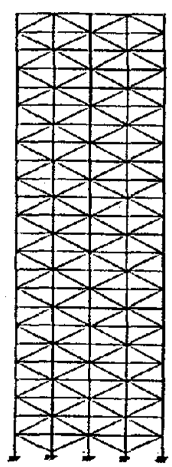
COMBINED MOMENT FRAME-
CONCENTRIC BRACED FRAME



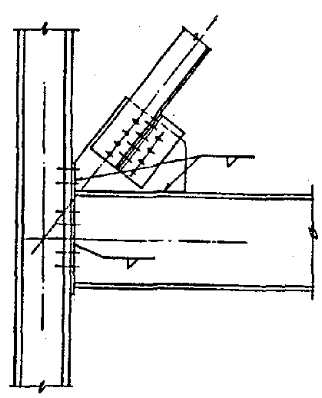
ECCENTRIC BRACED FRAME CONNECTION



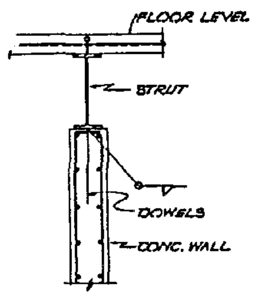
BRACED FRAME TUBULAR SYSTEM



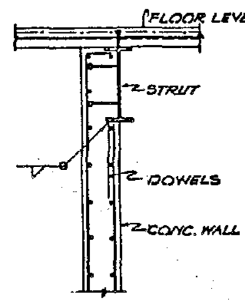
CONCENTRIC BRACED FRAME



CONCENTRIC BRACED FRAME CONNECTION



CONCRETE SHEAR WALL CONNECTION



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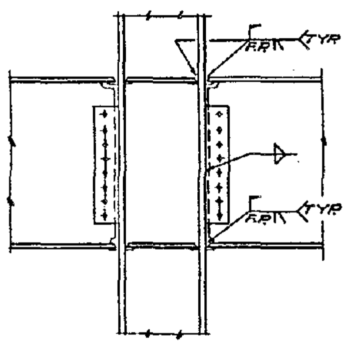
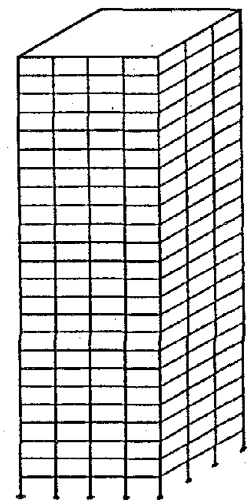
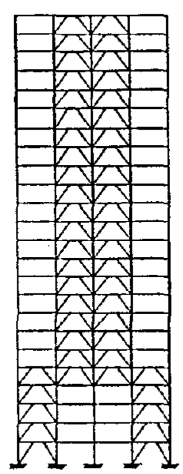


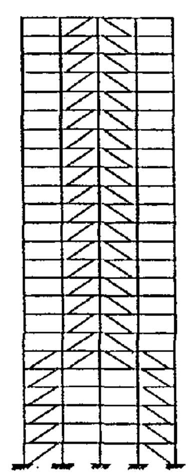
MOMENT FRAME CONNECTION



MOMENT FRAME SYSTEM



ECCENTRIC BRACED FRAME



ECCENTRIC BRACED FRAME

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DESIGN AND CONCEPTS FOR REINFORCED CONCRETE HIGH-RISE BUILDINGS

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INTRODUCTION

Reinforced concrete, for areas of high seismicity, has proven to be quite good in many instances. However, improper use of the material can result in total collapse. In this respect, design concepts and building codes constantly are being upgraded after major seismic events. But the most significant factor is that ductility must be built into the elements, and tying of the horizontal elements is necessary for survival of the structure by preventing sudden brittle failures.

A review of past performance indicates that few major high-rise structures have been built of concrete over the imposed 13-story or 160-ft height limit in high seismicity areas. Some buildings have been of structural steel framing with concrete shear walls in the core or as a cladding of the exterior wall. After considerable time and effort by the concrete industry, tests and design recommendations were published as "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions" (1961). At that time ductile provisions of means and columns acting as frames required that the shear capacity of members was to be based on the ultimate moments that could be generated at each end of the members.

Soon after this publication was available, the Structural Engineers Association of California revised their "Recommended Lateral Force Requirements and Commentary" (1966) to require ductile reinforcing for all concrete frames. They also removed height limits on buildings of reinforced concrete. These recommendations were adopted into the Uniform Building Code (1967) and other code jurisdictions as the state-of-the-art. Since that time, refinements have been made to both the design approach and the materials resistance, primarily as a result of the February 1971 San Fernando earthquake near Los Angeles, California. More recent laboratory tests have now produced better data on proper bar development, confinement requirements, and allowable shear stresses.

This paper will discuss the evolution of concrete design parameters and look at the various structural schemes being presently used in construction in California. Because of the time delay between testing, the translation of data into codes, and adoption by the design professional, it is found that many structures do not completely follow the latest techniques; they generally lag two or three years behind the most recent tests or data.

CRITERIA

Member Criteria

The use of reinforced concrete frames for high-rise buildings was slow in starting because little test data of cyclic loadings for such members was available. The first efforts to provide ductile provisions for concrete framing were done by the Portland Cement Association in 1966. The tests were instigated to show that concrete members could be made as ductile as structural steel members, especially for high-rise construction. Further testing and analytical studies resulted in the "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions" (1961). Through studies by the Structural Engineers Association of California, the first provisions for concrete ductile frames were published in their "Recommended Lateral Force Requirements" (1966) and the Uniform Building Code (1967).

The changes to code provisions for concrete frames has been from (1) working stress design of static code forces, to (2) ultimate design of the beam members at elastic capacity and in turn the column design, to (3) capacity design of the beam members based on realistic inelastic overstrength of reinforcing steel. These, in turn, are resisted in the beam-column joint and finally into the column, which is expected to behave generally in an elastic manner. A summary of this information is shown in Table 1, and includes basic strength and/or capacity criteria.

A review of Table 1 indicates that since 1962 the allowable shear stress in the element is 10 $\phi\sqrt{f'c}$ and has changed little except for the increase in shear stress in beam-column joints. The major increase of shear loads to be resisted is caused by using realistic overstrength of longitudinal reinforcing steel in the beam members. Present design based upon ACI-ASCE Committee 352 (1976, 1984) is to use 125 percent over the bar specified yield strength to figure maximum moment capacities to be encountered. This percentage of overstrength is based on actual supplied reinforcing steel yield strength and also on strain hardening effects.

A recent study by the Applied Technology Council in their report ATC-11 (1984) reviewed the parameters contributing to the beam-column joint design for ductile concrete frames. The evolution from the diagonal shear concept, to the compression strut, and to the compression strut-truss analogy concept was studied. Table 2 tabulates the various codes, and the analysis shows the impact on column sizes required for shear transfer or anchorage of the beam longitudinal bars in the beam-column joint.

Code Requirements

The approach used for design of buildings utilizing concrete ductile frames or frames in conjunction with shear walls is to follow the code seismic formulas for a minimum base shear force. These formulas take into account the seismic zoning, importance of the structure's use, the framing system, the building period, and the total mass contributing to the lateral loading. There are also story-to-story drift limitations, which apply to seismic loads but are optional for wind load conditions. Because of the

Table 1. Concrete frame design criteria.
(Based on the Uniform Building Code)

CRITERIA	Prior to 1961	1962 – 1971	1972 – 1976	1977 – 1983	1984
Ductile Requirements	No Provisions	Optional	Required	Required	Required
Allowable Ultimate Unit Shear – p.s.i.	$0.08 f'_c$ (Working Stress)	←	$10 \phi bd\sqrt{f'_c}$		→
Beam Shear	V_{d+l+e}	$\frac{M_{ur}+M_{ul}}{l} + 1.4 V_{d+l}$	←	$\frac{1.25(M_{ur}+M_{ul})}{l} + 1.4 V_{d+l}$	→
Column Shear	V_s	$\frac{M_{ut}+M_{ub}}{h}$	←	$\frac{1.25(M_{ut}+M_{ub})}{h}$	→
Allowable Ultimate Unit Joint Shear – p.s.i.	No Provisions	← $10 \phi bd\sqrt{f'_c}$	→	Varies 10 to 16 $\phi bd\sqrt{f'_c}$	Varies 12 to 20 $\phi bd\sqrt{f'_c}$
Joint Shear	No Provisions	← $f_y(A_s + A'_s) - V_{col}$			→
Area Of Required Confinement – sq. in.	No Provisions	← $0.22 \frac{sh_c f'_c}{f_y h}\left(\frac{A_g}{A_c} - 1\right)$	→	$0.12 \frac{sh_c f'_c}{f_y h}$ or $0.30 \frac{sh_c f'_c}{f_y h}\left(\frac{A_g}{A_c} - 1\right)$	or $0.09 \frac{sh_c f'_c}{f_y h}$
Approximate Seismic Base Shear	$\frac{0.046 N W}{N + .9(N-8)}$	← $\frac{0.67 \times 0.05 W}{(0.1 N)^{1/3}}$	→	$\frac{0.67 W}{15(0.1 N)^{1/2}}$	$\frac{0.67 W}{15(0.03 h^{3/4})^{1/2}}$

Table 2. Comparison of various procedures for designing reinforced concrete frame joints (from ATC-11).

Joint type	Location	Item	VETERANS ADMINISTRATION 1975	ACI-ASCE COMMITTEE 352 1976	ACI 318-77 APPENDIX A	ATC-3-06 1978	STANDARD ASSOC. OF NEW ZEALAND 1982	ACI 318-83 APPENDIX A	ACI-ASCE COMMITTEE 352 (IN PREPARATION)
TYPE 1 JOINTS	EXTERIOR	Max V_u		$10\phi bd\sqrt{f'_c}$		$10\phi bd\sqrt{f'_c}$			$20\phi bh\sqrt{f'_c}$
		v_c (N_u/A_g=0)		$3.5x1.4\sqrt{f'_c}$		$3.5\sqrt{f'_c}$			-
		l_s		$\frac{0.04A_b(f_y - f_h)}{1.4\sqrt{f'_c}}$		$\frac{0.04A_b(f_y - f_h)(0.8)}{\sqrt{f'_c}}$			$\frac{f_y d_b}{50\sqrt{f'_c}} - 5d_b$
		Min. Col. Size (in.) for f'_c = 4,000 psi		23		28			26
		Transverse Ties (V_u=min.; 26"x26" col.)		4 - #4 @ 6"		4 - #4 @ 4½" 28"x28" col. req'd.			3 - #4 @ 6" (arbitrary)
	INTERIOR	Max V_u		$10\phi bd\sqrt{f'_c}$		$10\phi bd\sqrt{f'_c}$			$24\phi bh\sqrt{f'_c}$
		v_c (N_u/A_g=0.05f'_c)		$3.5x1.4\sqrt{1.4f'_c}$		$3.5\sqrt{1.4f'_c}$			-
		l_s		N/A		N/A			-
		Min. Col. Size (in.) for f'_c = 4,000 psi		N/A		N/A			N/A
		Transverse Ties (V_u=min.; 26"x26" col.)		4 - #4 @ 8" (arbitrary)		2 - #4 @ 12" (arbitrary)			2 - #4 @ 6" (arbitrary)
TYPE 2 JOINTS	EXTERIOR	Max V_u	$20\phi A_{core}\sqrt{f'_c}$	$20\phi bd\sqrt{f'_c}$	$10\phi bd\sqrt{f'_c}$	$16\phi bd\sqrt{f'_c}$	$18bh\sqrt{f'_c}$	$15\phi bh\sqrt{f'_c}$	$15\phi bh\sqrt{f'_c}$
		v_c (N_u/A_g=0)	$3.5x1.4\sqrt{f'_c}$	$3.5x1.4\sqrt{f'_c}$	$3.5\sqrt{f'_c}$	$3.5\sqrt{f'_c}$	0	-	-
		l_s	$\frac{0.04A_b(f_y - f_h)1.4x0.8}{\sqrt{f'_c}}$	$\frac{0.04A_b(1.25f_y - f_h)}{1.8\sqrt{f'_c}}$	$\frac{0.04A_b(f_y - f_h)1.4(2/3)}{\sqrt{f'_c}}$	$\frac{0.04A_b(f_y - f_h)1.4(2/3)}{\sqrt{f'_c}}$	$\frac{f_y d_b}{70\sqrt{f'_c}}(0.8) + 5d_b$	$\frac{f_y d_b}{65\sqrt{f'_c}} - 5d_b$	$\frac{1.25f_y d_b}{75\sqrt{f'_c}} - 5d_b$
		Min. Col. Size (in.) for f'_c = 4,000 psi	27	22	30	28	28	22	24
		Transverse Ties (V_u=407K; 24"x24" col.)	4 - #4 @ 2½"	4 - #4 @ 3½"	4 - #4 @ 4½" 30"x30" col. req'd.	4 - #4 @ 2 3/4"	4 - #4 @ 2½" V_u=407x1.4/1.25=455K	4 - #4 @ 4"	4 - #4 @ 5½"
	INTERIOR	Max V_u	$20\phi A_{core}\sqrt{f'_c}$	$20\phi bd\sqrt{f'_c}$	$10\phi bd\sqrt{f'_c}$	$16\phi bd\sqrt{f'_c}$	$18bh\sqrt{f'_c}$	$20\phi bh\sqrt{f'_c}$	$20\phi bh\sqrt{f'_c}$
		v_c (N_u/A_g=0.05f'_c)	$3.5x1.4\sqrt{1.5f'_c}$	$3.5x1.4\sqrt{1.5f'_c}$	$3.5\sqrt{1.5f'_c}$	$3.5\sqrt{1.5f'_c}$	0	-	-
		l_s	$\frac{0.04A_b(f_y)1.4x0.8}{\sqrt{f'_c}}$	N/A	$\frac{0.04A_b(f_y)1.4(2/3)}{\sqrt{f'_c}}$	$\frac{0.04A_b\ f_y\ 1.4(2/3)}{\sqrt{f'_c}}$	$h_c \geq 35d_b$	$\frac{3.5f_y d_b}{65\sqrt{f'_c}}$	$h_c \geq 20d_b$
		Min. Col. Size (in.) for f'_c = 5,000 psi	N/A	N/A	N/A	N/A	44	N/A	25
		Transverse Ties (V_u=730K; 26"x26" col.)	4 - #4 @ 3½" 28"x28" col. req'd.	4 - #4 @ 4"	4 - #4 @ 7" 32"x32" col. req'd. V_u = 730/1.25	4 - #4 @ 4½"	4 - #4 @ 3½" V_u=730x1.4/1.25=817K	4 - #4 @ 6"	4 - #4 @ 7½"

NOTES TO TABLE:

1. This table was prepared to provide information on how the various procedures used in the design of reinforced concrete frame joints compare with each other. Because the procedures have extensive, and sometimes complex requirements, many assumptions and interpretations were made in preparing the table. Therefore, the reader is urged to refer to the original sources for the formulas, precise definitions of terms and parameters, and applicable assumptions.
2. All TYPE 1 joints in the table are assumed to be not confined by adjoining beams.
3. All TYPE 2 joints in the table are assumed to be confined by transverse beams covering 3/4 of the column width.
4. In the case of interior joints, all procedures except the Standards Association of New Zealand (1982) and ACI-ASCE Committee 352 (in preparation) allow the development of longitudinal beam bars through the joint and into the beam on the opposite side. Therefore, in these cases the minimum column size is not determined by bar anchorage or development.
5. For exterior joints, and for the two exceptions in Note 4, the minimum column size shown is determined by the beam bar anchorage or development requirements.
6. The effective joint area for shear stress computation, i.e., (A_{core}), (bd), (bh), is defined differently by different procedures. The reader should refer to the original documents for the definition of the effective area.
7. In all cases beam longitudinal bars are assumed to be #10, f_y = 60 ksi.
8. In all cases ties are assumed to have f_y = 60 ksi.
9. Transverse ties shown on the table are computed for the column sizes and shears shown on the left (previous page), regardless of the minimum column size dictated by the anchorage requirements. Wherever the maximum allowable ultimate shear stress v_u was exceeded, the column size required for shear stress reasons were shown on the table together with the transverse joint reinforcement associated with that column size. Ultimately, the designer may choose to increase the concrete strength rather than go to a larger column or section.
10. Sections of table with diagonal lines have not been completed because these procedures are applicable for structures in high seismicity areas only. In such cases, routine procedures such as those specified in ACI-318 would apply.

greater mass of a concrete building, generally drift and wind crieteria are not predominant and stress in the numbers dictates the design.

Because all concrete seismic load-resisting frames must be ductile, there are certain arbitrary limitations on dimensions or proportions of width and depth of members. Further, because longitudinal bars must be developed or transferred through the beam-column joint, such as in Figure 1, the joint may dictate the size of the column over that from axial loads and moments to be resisted in the members. The various types of joints (i.e., interior, exterior, corner) confined by adjoining beams or otherwise, as suggested by ACI-ASCE Committee 352 (1984), has led to different concepts and approaches to the proper design.

After the seismic base shear is calculated, present American design requires that (1) this base shear be distributed over the height of the building, basically in a triangular distribution; (2) minimum member sizes be estimated to adequately carry the induced forces, with a check for drift; (3) beam-column joints be checked for anticipated longitudinal bar development and induced shear through the joint based on the nominal yield stress; (4) the column generally should behave elastically while beams behave inelastically; and (5) all loads be properly resisted (conformance to allowable code stresses). In certain instances a dynamic analysis may be required.

Figure 1 illustrates the accepted approach to the member force transfer. Although ATC-3-06 (1978) was usually used for the allowable stresses to the beam-column joint, the ACI 318, Appendix A (1983) will now be the basis.

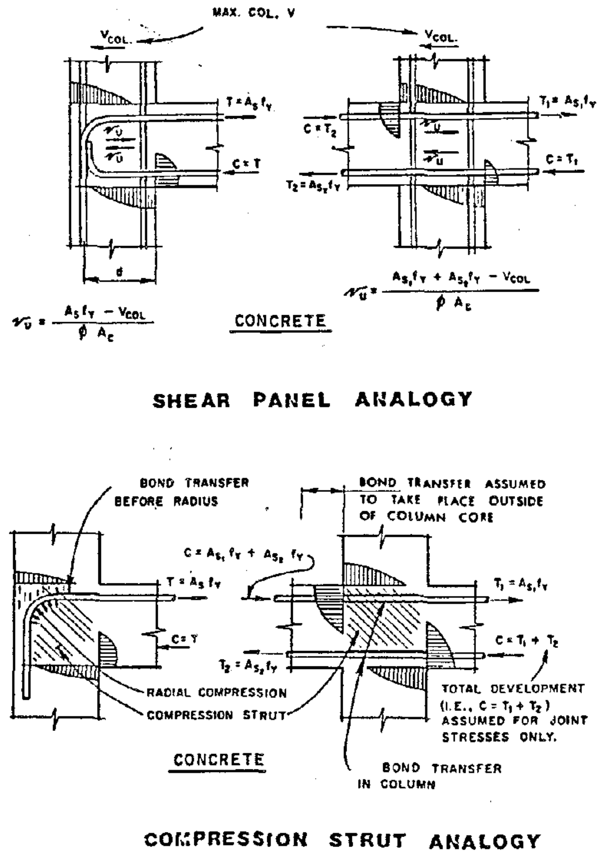


Figure 1. Beam-column shear transfer (from SEAOC Blue Book).

Dynamic Analysis

Dynamic analysis of a structure is normally an optional method of design and is based on site-related response spectrum data. Such data are generated by known faults in the area relating to the appropriate geologic, tectonic, and foundation material at the site. However, in some areas, such as the City of Los Angeles, a dynamic analysis is required of all buildings over 160 ft in height and all buildings that would be considered irregular.

The response spectrum data are usually broken down into two criteria, the maximum probable (design-level) earthquake, and the maximum credible (expected) earthquake. Representative spectra are shown in Figure 2. The design-level earthquake is generally based on a return period of 100 years and a damping of 5%; the credible earthquake has a return period of 475 years and a damping of 10%. Further, for concrete frames, the members are to remain elastic under the design-level earthquake with a drift not to exceed 0.0075 in./in., and may behave inelastically under the credible earthquake with a drift not to exceed 0.015 in./in.

DESIGN PARAMETERS

There have been a number of studies made of how concrete buildings behave in earthquakes. Unfortunately, there are no specific guidelines available for a "properly designed" building. In some respects researchers and designers are not talking about the same set of parameters, and thus differences in results are to be expected. To this end, the following information is presented regarding the designer's approach to selection of materials and strength and their contribution to building period, drift, and non-seismic resisting frame elements.

Concrete Materials

In general, selection of lightweight concrete aggregates helps in reducing the tributary lateral inertia load and reduces gravity loads to the foundations. However, this same material has not shown the best behavior as part of the lateral resisting frames, or it may not be enough mass to minimize floor vibrations under service loads. More recent tests now indicate that properly designed members would be acceptable, and thus cost of materials would be the factor to consider regarding choice of this material rather than rock aggregate. Because the beam-column joint usually controls the design, it has been found that column concrete strength should be a minimum of 5,000 psi.

The longitudinal reinforcing steel used for ductile frames must conform to ASTM A706 to assure the ductile strain requirements. The ties and stirrups generally need 60 grade strength to carry imposed confinement requirements.

Building Period

Determination of the building period for concrete structures is more difficult than for steel framing. This is caused by the increase in modulus of elasticity with time and the in-situ variations in strength over that specified. Further, most concrete structures have continuity of the elements to the columns, wherein steel members may be pin-ended. The situation is further complicated by whether sections are cracked or uncracked.

It is generally agreed that the building period should be based on the overall building participation. In this regard, each column line frame must be evaluated along with the lateral resisting frames and each is considered to have uncracked gross section properties when using formulas such as the

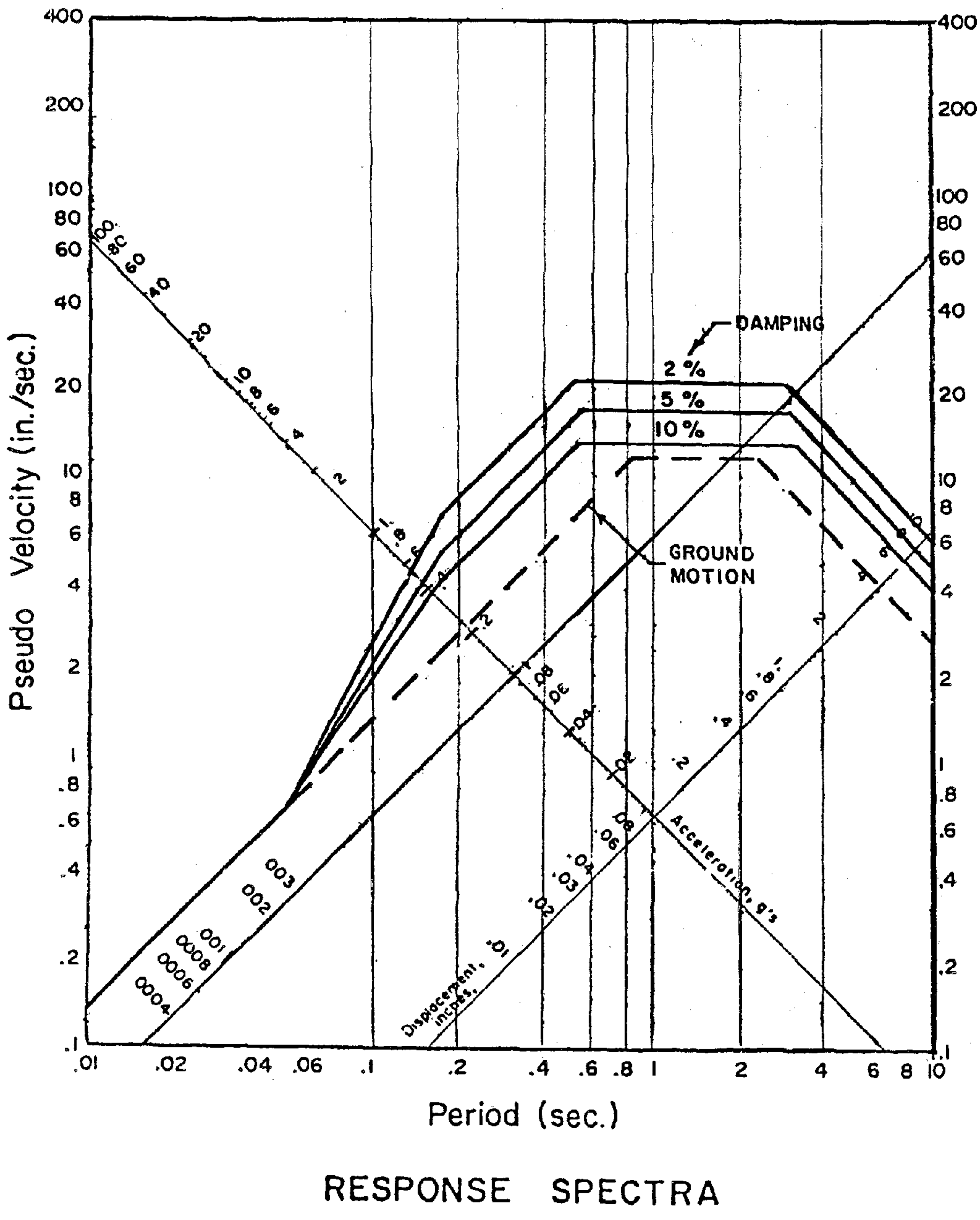


Figure 2. Typical credible response curve in Los Angeles Basin (probable response similar).

Rayleigh formula for period determination. Generally, the slab is not included with the beams when establishing the section properties. It has also been suggested that the calculated period be reduced by 25 to 35 percent to allow for nonstructural element resistance. Figure 3 indicates typical recorded building periods from the 1971 San Fernando earthquake and has been used as a guide for design.

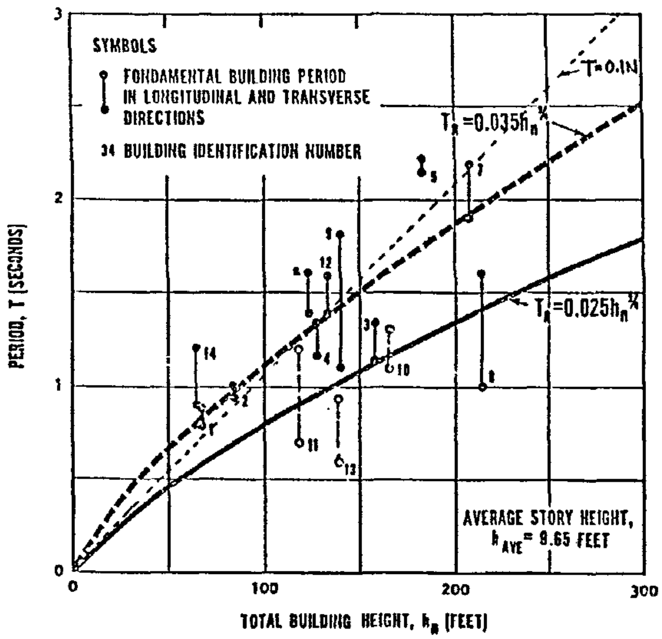


Figure 3. Recorded building periods for reinforced concrete buildings in the 1971 San Fernando earthquake (from ATC 3-06).

What has not been resolved is the modeling of the framing. Two items need further study: (1) inclusion of the shear deformation into the beam and column and (2) the shear distortion into the beam-column joint. Because many frame programs are based on flexural properties, and joints are assumed to be infinitely stiff, shear deformations must be added to the criteria. Added to this would be the effect of cracking in the members, which could reduce the stiffness by 25 to 50 percent of the uncracked section. Some recommendations are currently being finalized by ACI-ASCE Committee 442 (1984) regarding the modeling techniques.

Drift

A dichotomy with the period determination is the drift limitations. When using uncracked sections for member properties, one finds that drift limitations generally are not important, and that stress is. However, if cracking occurs, drift will become larger and possibly more important. With larger drifts, the effect on nonparticipating elements may impose hazardous conditions to these elements. A case in point would be the moment and shear transfer of a flat slab to a nonlateral resisting column at the 3/K loading required by the code to such elements. For this reason it is imperative that studies, such as by the ACI-ASCE Committee 352, be further developed on proper load transfer to nonparticipating elements.

BUILDING FRAME SYSTEMS

The various concrete frame systems that have been used in recent years are a complete departure from earlier designs. In past years, a proper design for concrete frames was based on the judgment of the designer. In more recent years, test data, earthquake deficiencies, and studies by code bodies are changing the design and detailing to provide safer structures.

The earliest high-rise structures were generally of steel or concrete frames, with or without concrete or masonry walls, and were designed for wind loads or nominal seismic loads. About 25 years ago, the height limit for buildings in seismic areas of California was removed. Generally, these newer buildings had structural steel taking a proportion of the lateral load and concrete incasement designed for the full load. Figure 4 depicts such a building. It can be seen that the exterior wall (Figure 5) was in reality a shear wall with nominal window openings. Ductile provisions of the concrete portion generally only conformed to allowable stresses and judgment.

In the 1960s, ductile frames were relatively unknown, and design of concrete frames was again based on judgment. Many major structures had a degree of confinement to the beams and columns through closely spaced ties or stirrups, but hoops were not used. The San Fernando earthquake in 1971 clearly illustrated the need for ductile concrete members to resist seismic loads. Engineering studies of this earthquake and prior research developed the new concepts being used for buildings.

Beam and Slab Framing

The most common concrete framing system previously used was beam and slabs supported on concrete columns. The designated frames to resist seismic loads are generally on the perimeter, but beam members framing to interior columns must be checked for their contribution in resisting lateral loads at realistic drifting of the building from an expected earthquake. Figure 6 shows one such building. It was found that, because of their relative stiffness, the interior transverse frames each carried approximately 5 percent of the total seismic loads. This necessitated that all interior frames were to have ductile provisions for confinement, but arbitrary proportions of member sizes could be waived.

Beam and Flat Slab Framing

To relieve the contribution of interior frames, many systems now use a two-way flat slab for the interior framing and either frames or frames with walls as the lateral resisting system. This trend is also used to alleviate the removal of the forms out of the sides of the building during construction. After these forms are moved to the floor above, the spandrels are poured with the columns. In many cases, the slabs are post-tensioned, which allows early removal of forms after stressing rather than reshoring for conventional slab systems.

Some of these systems with flat slabs and frames are illustrated in a review of the following buildings.

Figure 7 shows isolated frames located as convenient for architectural and/or structural layout. The heavy columns and beams tend to make the frame act similarly to a cantilever shear wall with large opening, and thus it has large overturning forces at each end. For high buildings, code drift requirements may be exceeded, but generally the drifts are lower than that obtained from spectrum analysis limitations. A principal advantage of this system is that simple frames can be designed and that joint shears are dictated by development of the top longitudinal beam bars rather than the combination of top and bottom bars in continuous frames. Further, mechanical

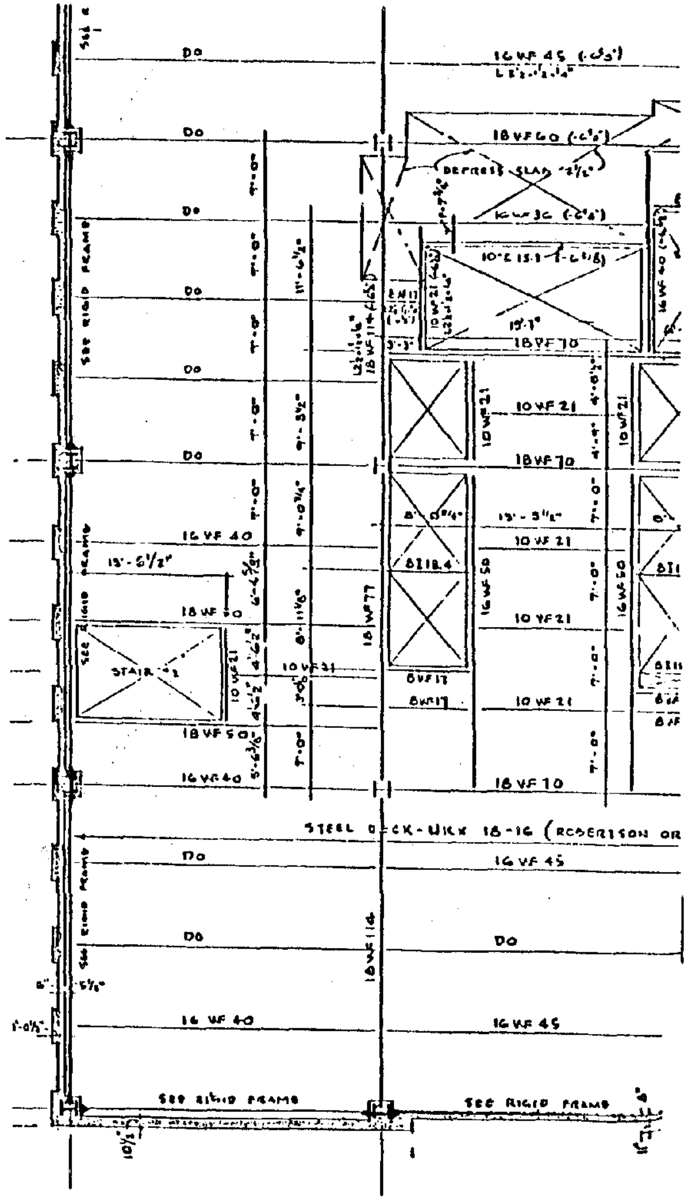


Figure 4a. Plan of pre-1960 building with exterior frames and concrete piers.

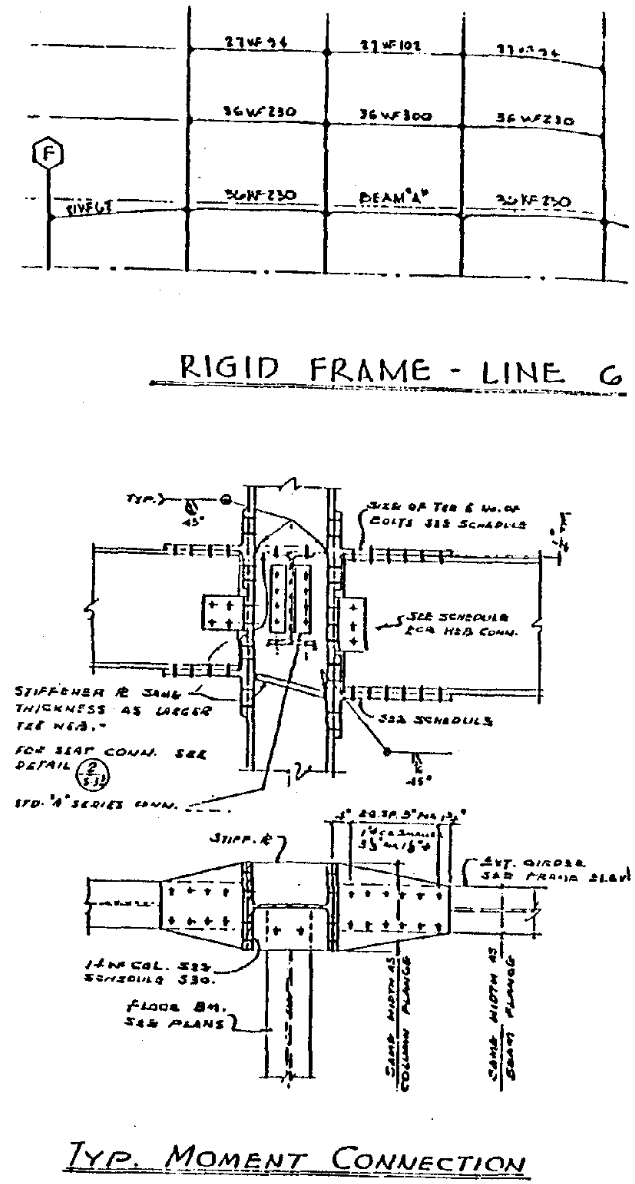


Figure 4b. Structural steel frame connection detail.

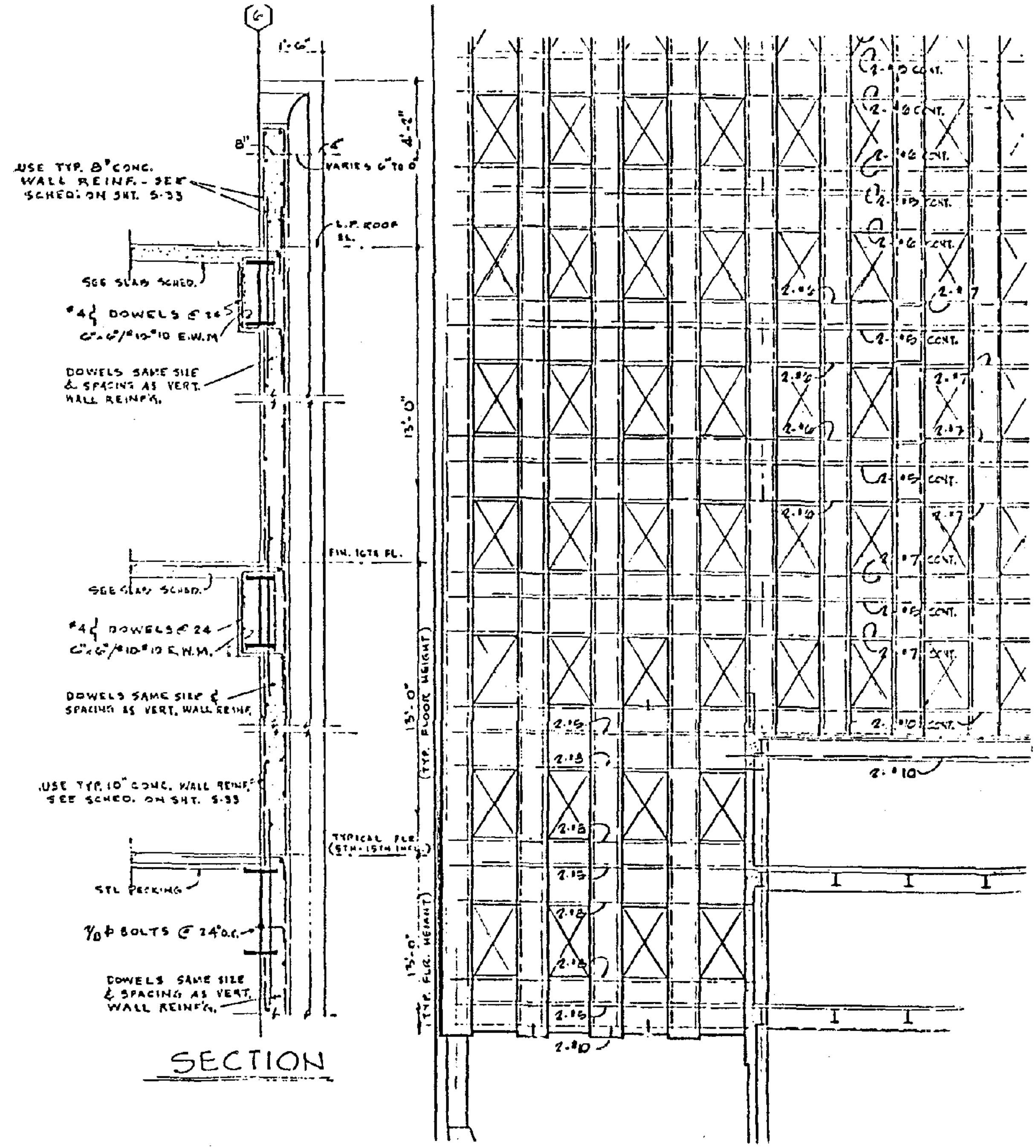


Figure 5. Exterior wall framing.

ducts and plumbing layouts generally can be placed so as not to penetrate the beam, and thus story heights are set by the beam depth only. Some of the questionable areas needing further study in this scheme are the lack of redundancy in the building system, especially at the perimeter; the shear and

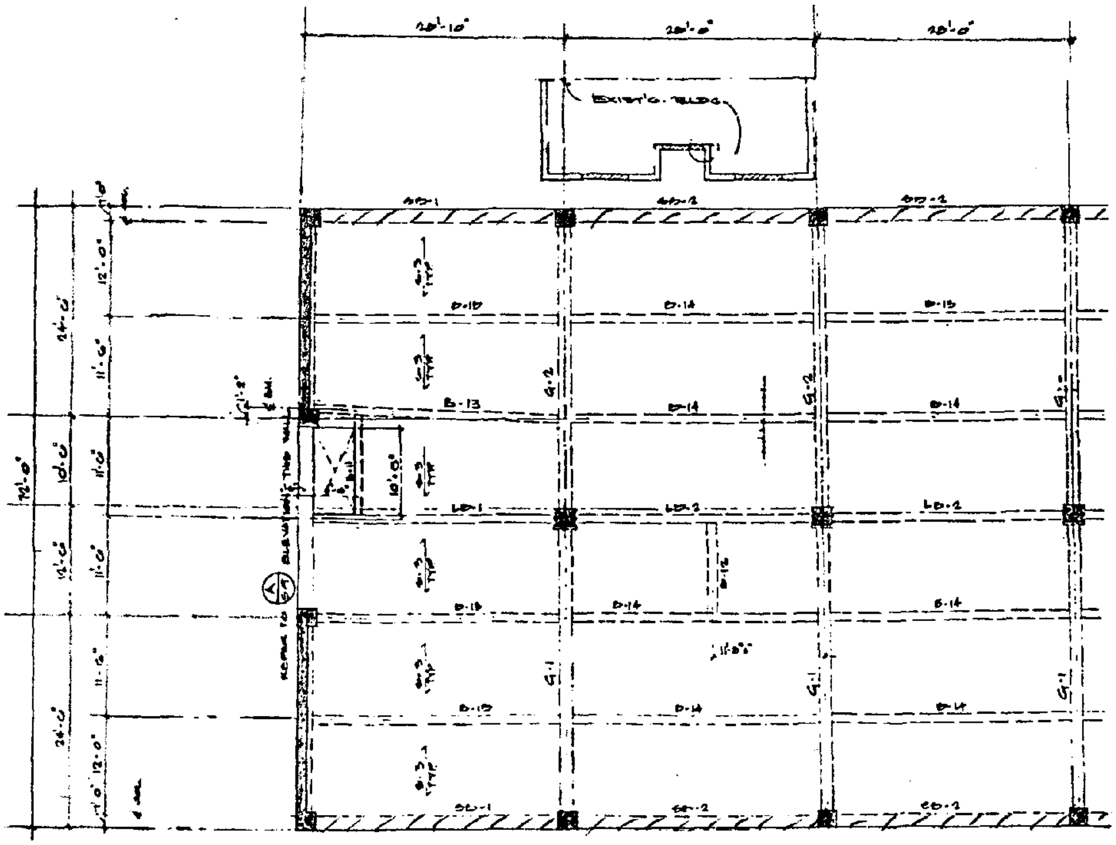


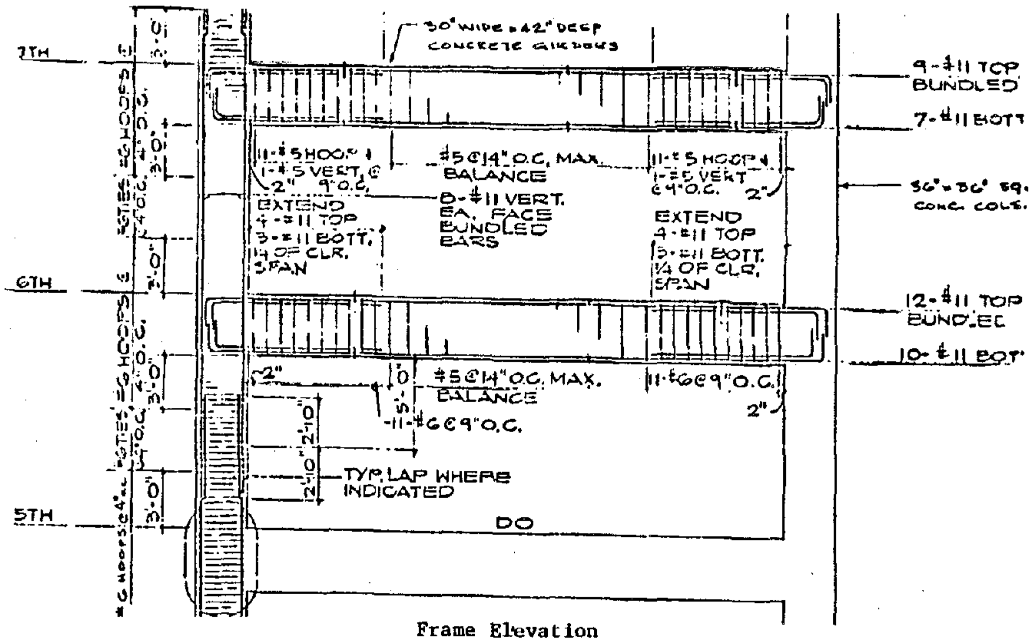
Figure 6. Beam and slab framing plan with frames and shear walls.

moment interaction of the cracked slab between the frames during lateral deformations; and the resolution of transfer of loads at the base or foundation.

A recent innovation of this system is a single bay precast frame shown in Figure 8. Details of this system are still to be fully developed.

Figure 9 shows perimeter beams in one direction and shear walls in the other. This system is used for many hotel structures and allows an exposed slab. This eliminates the need for a suspended ceiling, and the wall is a separation between rooms. As noted previously, the exterior spandrel is poured separately after the slab forms are moved upward. Because the spandrel may be placed above the slab and also is normally eccentric to the column centerline, the true frame behavior from lateral movements and torsional effects needs to be designed carefully. Further, the restraint by the slab to keep the columns stable under both high axial and seismic behavior has not been tested.

Figure 10 illustrates a combination concrete core with frames to carry the imposed loads. The frames are required to carry at least 25 percent of the seismic code loads. In general, when the core and frames are linked together for analysis, the frames will carry most of the seismic loads at the



Frame Elevation

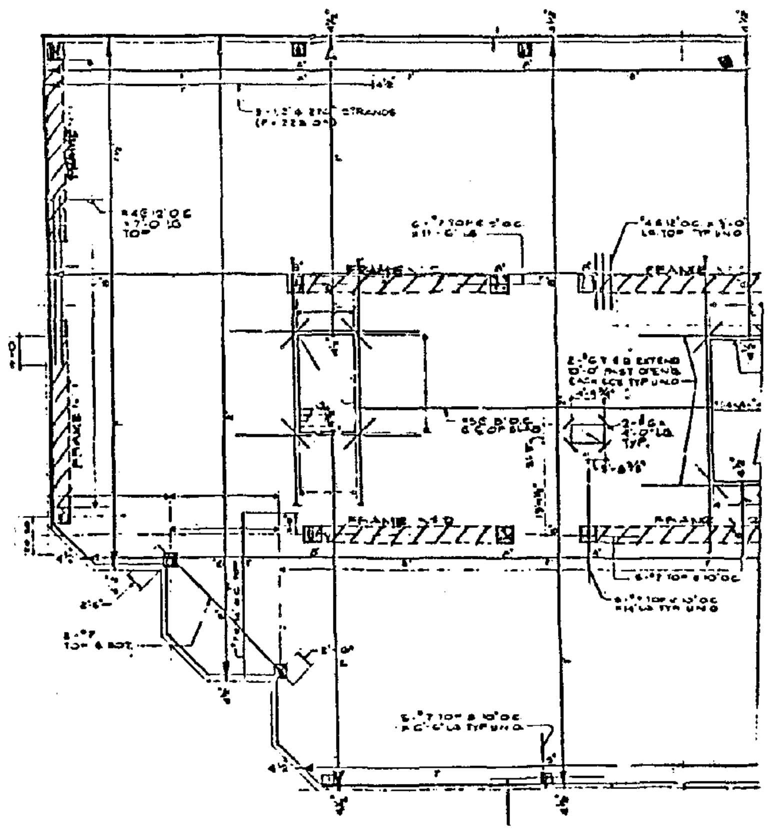


Figure 7. Isolated frames and flat slab plan .

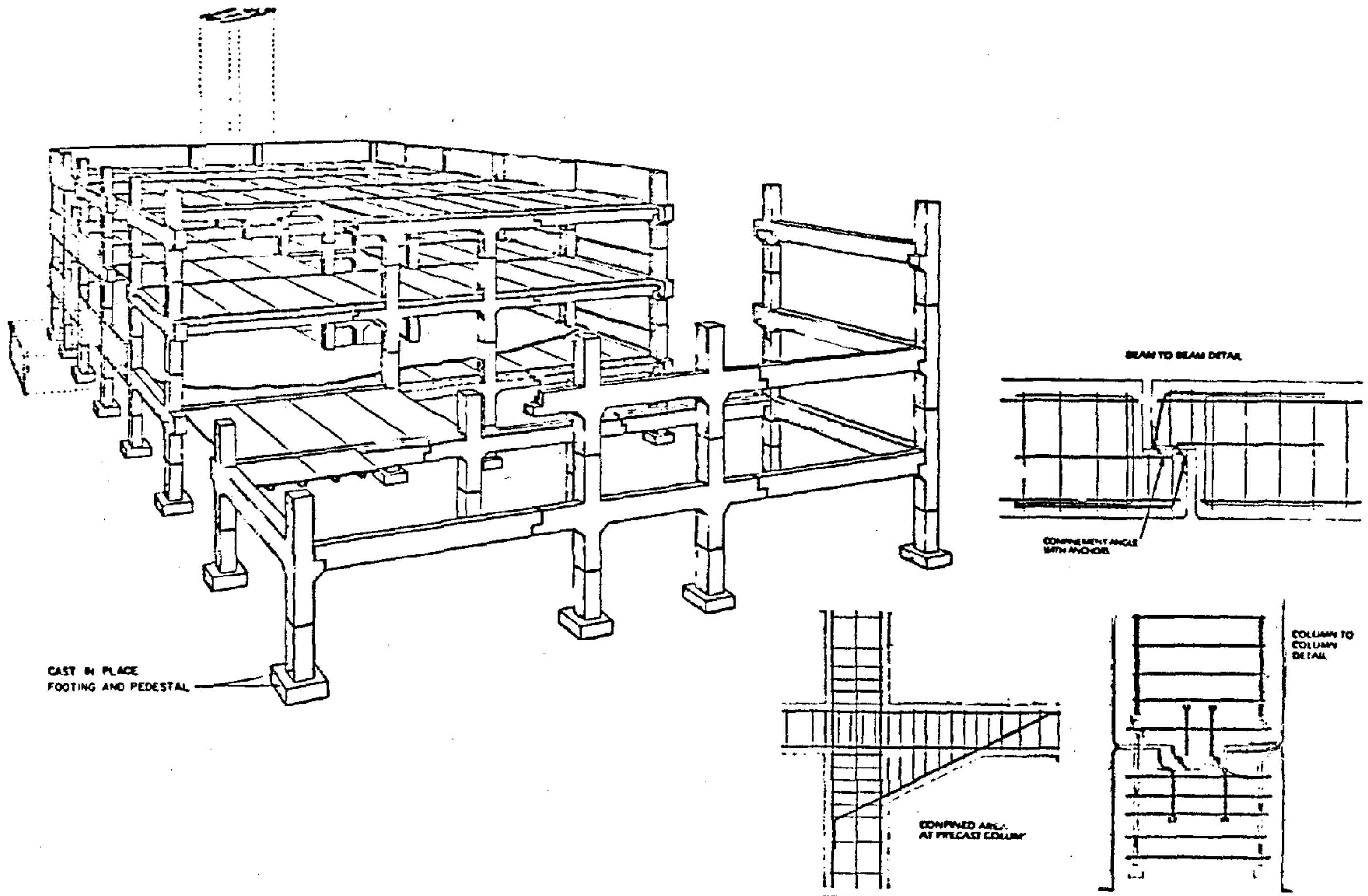
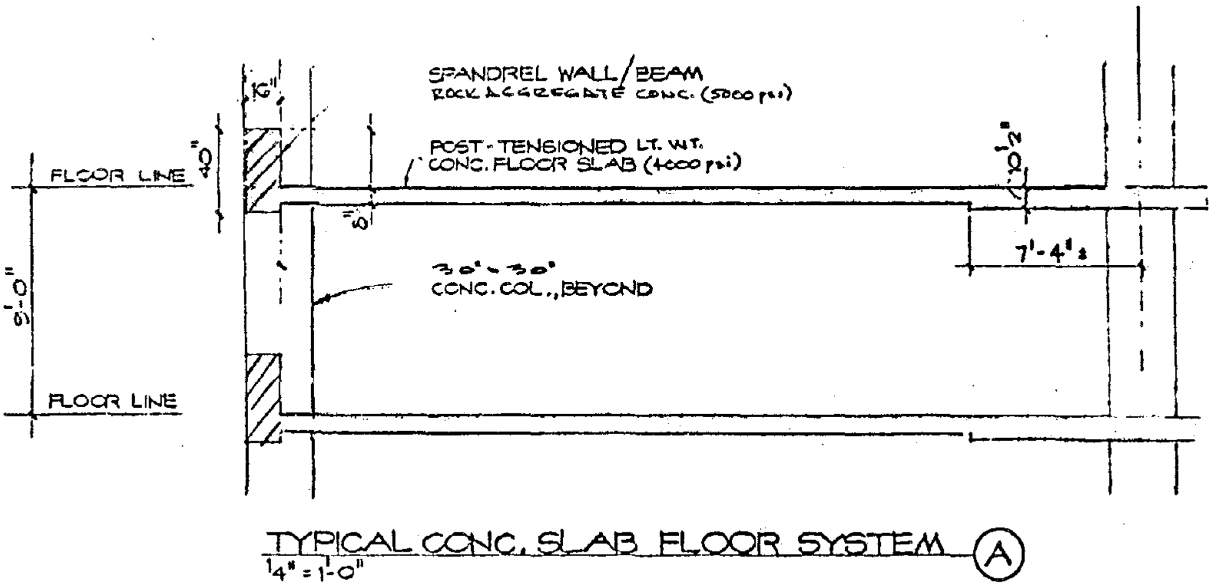


Figure 8. Precast frames.



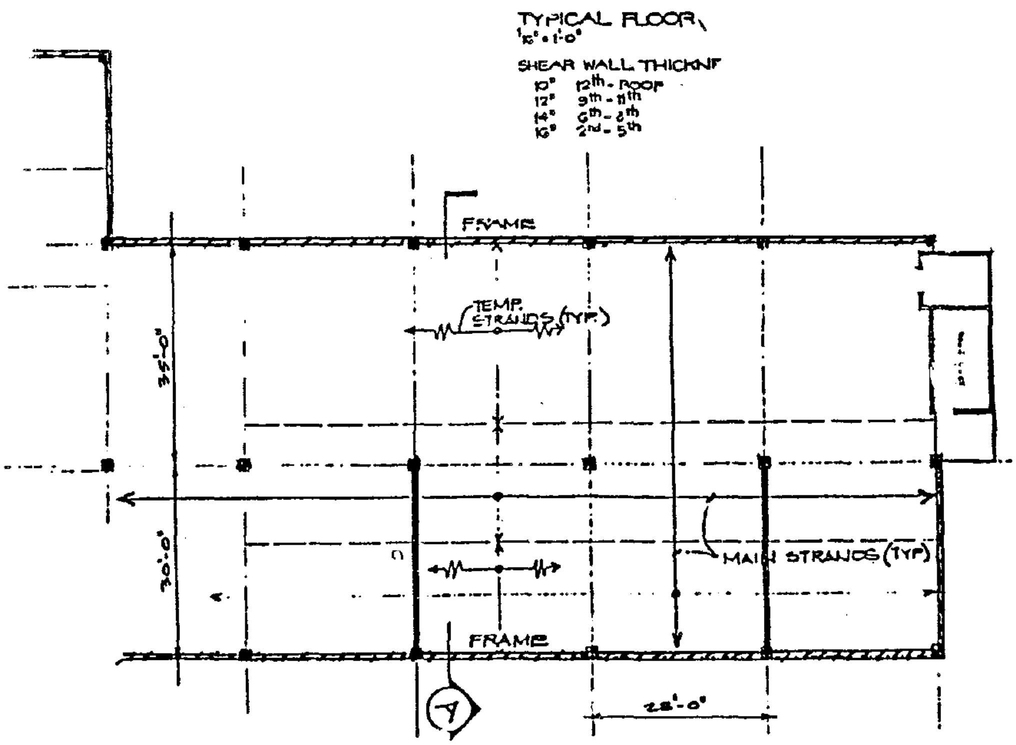


Figure 9. Frames and shear walls with flat slab.

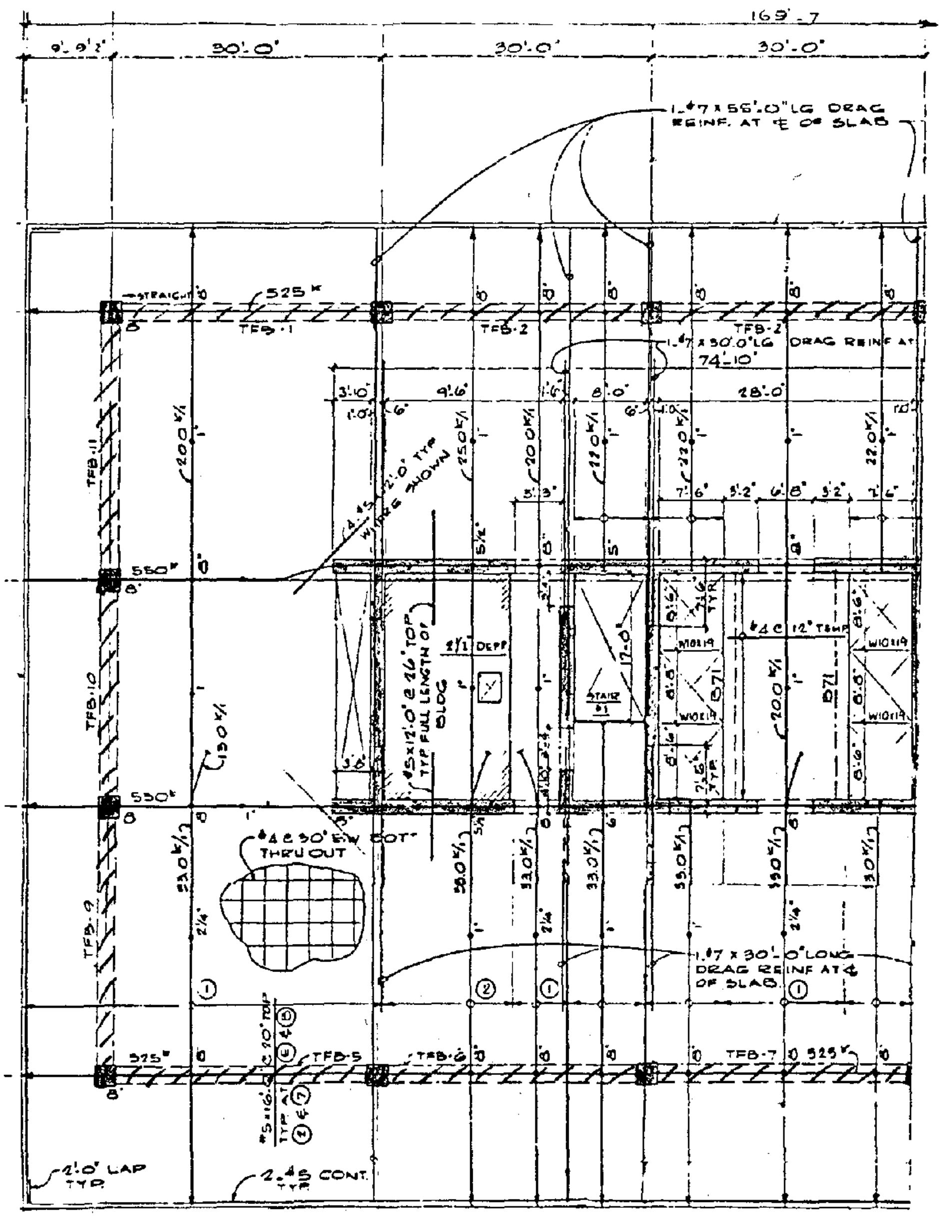


Figure 10. Frames and concrete core plan.

upper floors, and the shear walls will carry most of the seismic loads at the lower floors. Stress in the members will control the design.

An alternate to this system is to use precast beams spanning from the exterior frames to a slip-formed core. The precast beams are assumed to be pin connected at their ends and this removes any additional frame action and limits redundancy within the system.

Figure 11 is the first major high-rise ductile frame building built

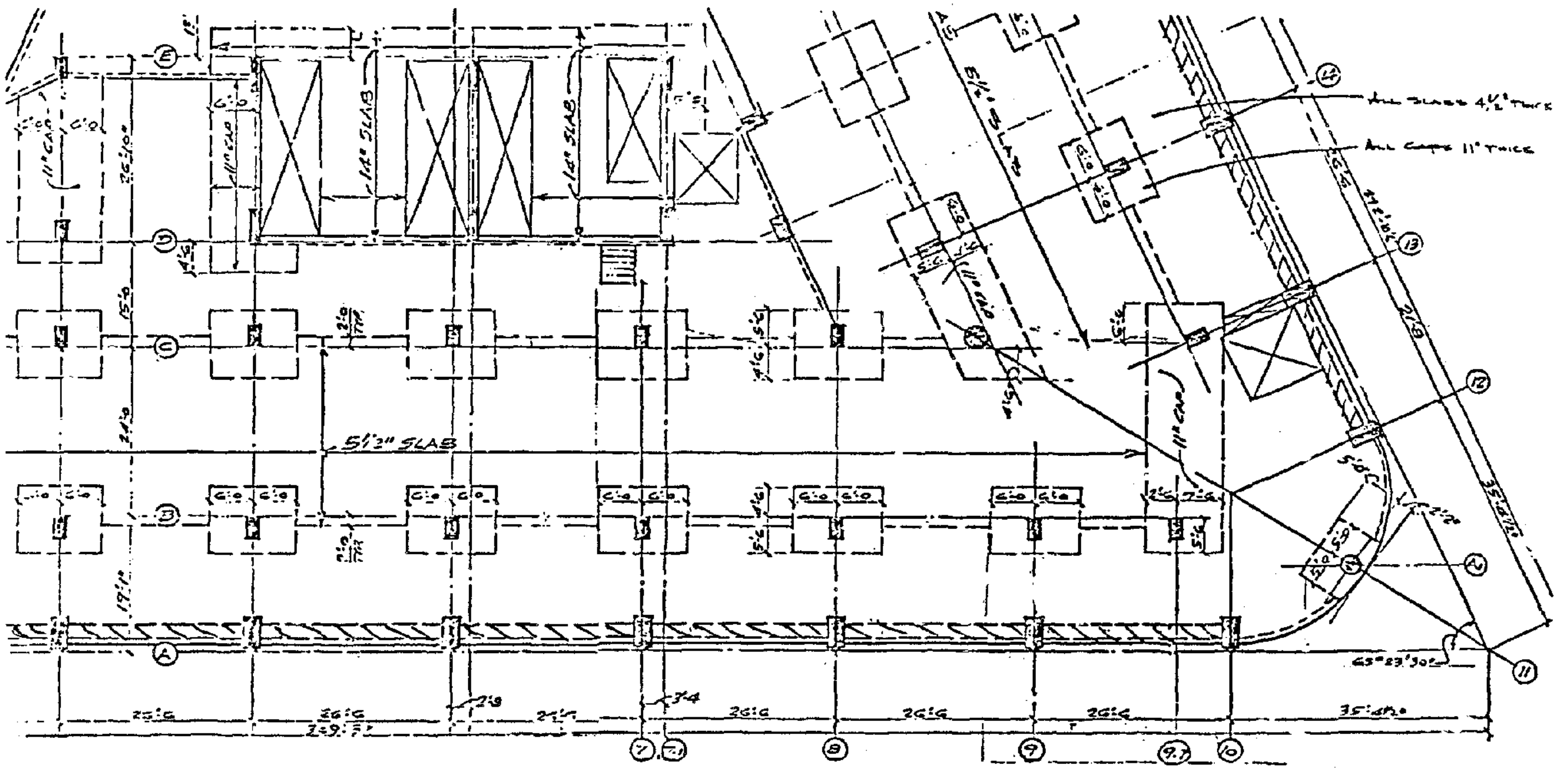


Figure 11. High-rise perimeter frame plan.

recently in Los Angeles. The 29-story, 270-ft-high building is generally triangular in shape with perimeter frames and had to conform to the dynamic analysis limitations of the code, both for being over the 160-ft height limit and also because of its irregular shape. The major problem for the design results from inablity to conform to the allowable shear in the beam-column joint. Based on recommendations of an advisory committee of SEAOSC, the designers calculated the building period assuming uncracked sections and using all of the building elements, the probable earthquake loads could be reduced by a ductility factor of two to account for some cracking, and the joint shear could conform to the ATC-03-6 requirements. It should be noted that the drift limitations were not critical.

Figure 12 is another major high-rise building, in the San Francisco Bay area. The 30-story, 310-ft-high building is "Y" shaped with 60 ft wide by 120 ft wings projecting from the central core. Because of its proximity to the active Hayward fault and its irregular shape, a dynamic analysis was provided in the design. As was evident in the previous building in Los Angeles, drift was not critical and the joint shear limitation was critical in the design.

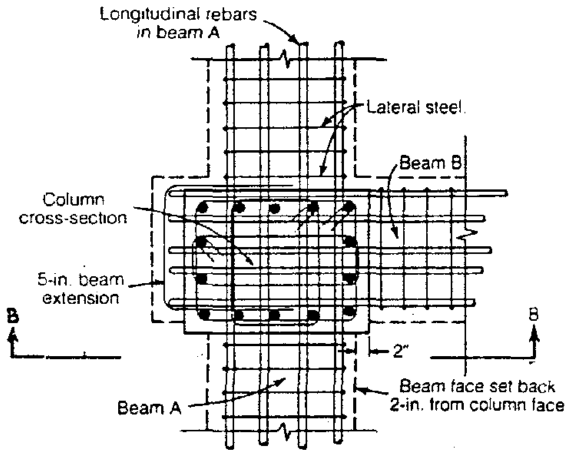
SUMMARY AND DISCUSSION

The use of concrete ductile frames as lateral resisting elements is now becoming a viable system to resist strong seismic forces. The modeling techniques used in the design process still need further study to be more in line with measured building periods, but the methodology for design is now becoming more rational. This rationale is being developed on the basis of past earthquake experience and a reduction of very high deformations developed in laboratory tests to realistic expected lateral deformations in the design.

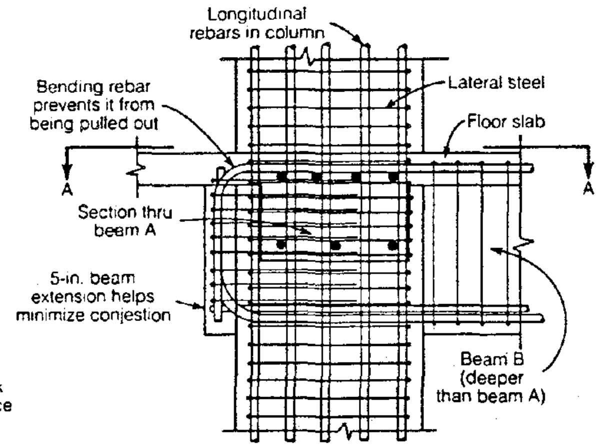
Accelerometers have recorded more recently built concrete buildings in earthquake events, but few have been recorded in some of the older, composite buildings. These older concrete buildings generally did not exceed 2.5 to 3 times the least frame width, although some of the more modern buildings are of 5 or more. With these more slender buildings, higher recorded building periods would be expected over previous events, and the contribution of higher overturning axial loads could also be expected. Further, because seismic design drift limits generally are not governing, the imposed wind drift limits may be more critical. In any event, there appears to be a need for limits on building periods used in the design equations as well as the drift limitations.

A review of numerous buildings recently built in high seismic areas indicated certain parameters that the designer can use for initial consideration of the framing. These include:

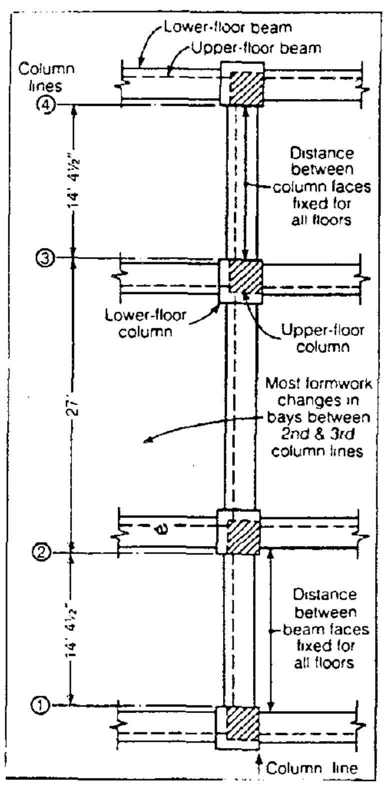
1. Use of 5,000 psi concrete strength in columns and beam-column joints.
2. Longitudinal and transverse reinforcement should be of 60 ksi nominal strength.
3. The beam-column joint shear and bar anchorage requirements will be critical.



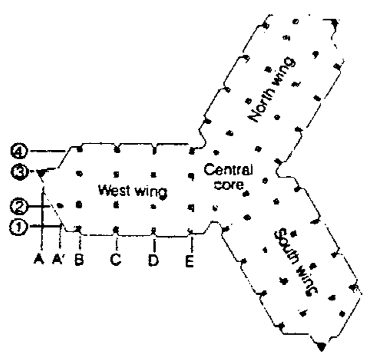
Column Plan



Beam-Column Section



Blow-up Plan



Overall Plan

Figure 12. High-rise beam and slab framing.

4. The minimum transverse reinforcement in the beam-column joint may be advisable for the full height of the column because splice areas also require closer spacing of ties.
5. A two-way flat slab system appears to be more economical for the interior floor framing system because this system induces less stiffness to the overall building.
6. The frame-beam-depth to clear-span ratio generally varies between 6 and 10.
7. The frame columns for high buildings are from 30 in. to 40 in.

There are, however, other areas in the design or detailing that need further research. These include:

1. Realistic modeling techniques of member properties, seismic response, and damping effects.
2. Better information about the scale effect of test models and real sizes used in buildings.
3. Whether large columns at close centers and deep beams act as frames or heavy walls with openings, and how they will perform. (The area of the columns has an equivalency of a thick wall spanning between column lines.)
4. The type of strut required at the floors to stabilize these heavy columns under lateral deformaions, especially with induced beam-column eccentricities.
5. Verficiation of supplementary tie behavior where lapped in the beam-column joint.
6. The shear transfer mechanism of flat slabs around nonseismic columns under realistic lateral deformations, especially for short spans.

In closing, it can be safely said that ductile concrete frames can be designed, detailed, and constructed as long as the various limitations are considered. These considerations should also include adequate space between reinforcing bars, access for good placement of concrete, need for the contractors' early input into the construction procedure, and an inspection of the progress of the work by the designer. Improper considerations have too often been shown in failures during catastrophic events.

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DAMAGE DISTRIBUTION AND CONCENTRATION IN BUILDINGS WITH DEGRADING STIFFNESS SYSTEMS

by Masakazu OZAKI
Prof. Chiba University

1. Introduction

It is well known that significant damage such as collapse of steel, reinforced concrete and steel reinforced concrete (composite) structures is often caused by severe damage concentration in a particular story of multi-story buildings. This paper deals with the evaluation of damage distribution and concentration for 1∿12 story buildings with degrading stiffness systems subject to strong motion earthquake excitation by utilizing non-deterministic response analysis.

In addition, damage distribution and concentration of multi-story buildings with torsional effect is also examined by a proposed yield level ratio for torsional effect, based on a step-by-step time integration response analysis.

2. Input Ground Motion

The predicted earthquake ground motion used for the response analysis is the modified specific ground motion based on the average of recent seismograph recordings observed in Japan. This modified ground motion has the predominant period $T_g=0.4$ sec., damping ratio of ground $\xi_g=0.5$ and stationary duration $T_D \fallingdotseq 20$ sec. The linear response spectrum for single-degree-of-freedom systems with damping ratio $\zeta=0.05$ obtained by non-deterministic response analysis is compared with the response spectrum obtained by step-by-step time integration response analysis using 20 artificial earthquake ground motions as shown in Fig. 1. In this Figure, the response values are approximately proportional to 1/T, when the natural period of systems T is longer than the predominant period T_g.

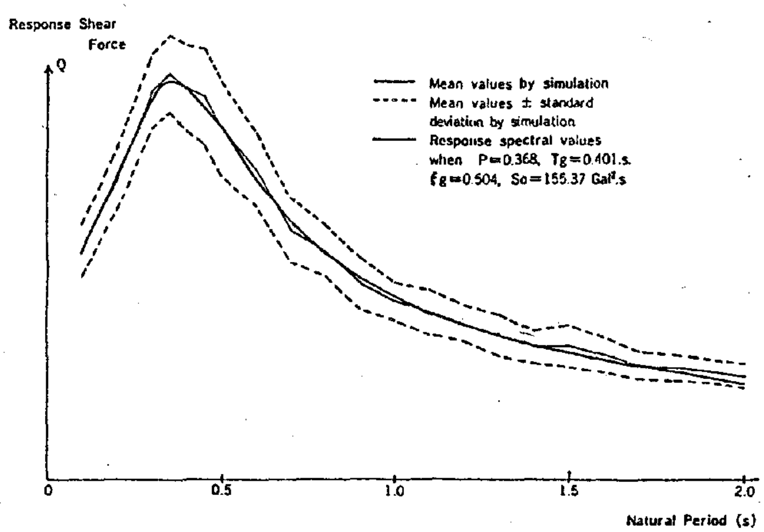


Fig. 1 Linear response spectrums

3. Restoring Force Characteristics with Degrading Stiffness Systems

Various kinds of patterns of non-linear models with degrading stiffness systems have been proposed by many researchers, idealizing the force-deflection characteristics of reinforced concrete and steel reinforced concrete structures obtained by the static or dynamic loading tests. The perfect

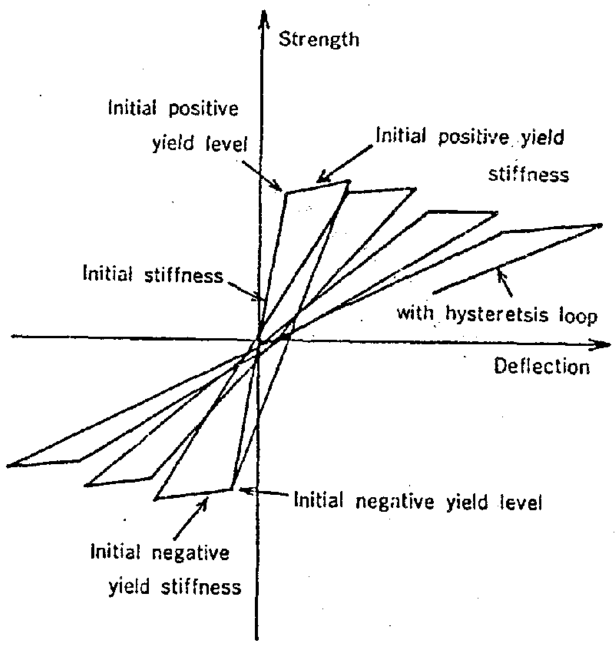


Fig. 2 Idealization of degrading stiffness system

patterns of non-linear models, however, are not established, because of the complex characteristics of the structures after yielding.

In this paper, non-linear models with degrading stiffness systems are idealized as follows.

1) The non-linear models with degrading stiffness systems have the positive and negative yield levels with the almost equal amplitude and relatively flat positive yield stiffness after each step of yielding as illustrated in Fig. 2.

2) After a single plastic excursion of response beyond the positive or negative yield level, the system has the second stiffness with the hysteresis loop estimated by the equivalent viscous damping crossing an arbitrary point on the line between the origin and the opposite side yield point.

For origin-oriented degrading stiffness systems the second stiffness crosses the origin. The response process crossing the opposite side yield point is defined as peak-oriented degrading stiffness systems.

In this paper, the bi-linear-type peak-oriented degrading stiffness systems with hysteresis loops estimated by the equivalent viscous damping are used for non-linear response analysis of one-story and multi-story buildings for the reason of simplicity.

The equivalent viscous damping ζ_{eq} after yielding is evaluated by the following equation considering the testing results of reinforced concrete and steel reinforced concrete columns subjected to repeated and reversed loadings,

$$\zeta_{eq} = C_\zeta \sqrt{(\mu^+ - 1) + (\mu^- - 1)}$$
$$(\mu^+ - 1) \geqq 0, \quad (\mu^- - 1) \geqq 0$$

where

C_ζ: coefficient of increasing equivalent viscous dampi

μ^+: ductility factor at each positive response displacement in non-linear response processes

μ^-: ductility factor at each negative response displac ment in non-linear response processes

4. Maximum Ductility Factor in Non-linear Response Analysis

Non-linear response analysis was carried out for one-story and multi-story buildings with the natural period $T=0.5$ sec.

Fig. 3 and Fig. 4 show non-linear response values of one-story and the lowest story in multi-story shear-type buildings with various values of yield level ratio R, envelope slope ratio α and coefficient of increasing equivalent viscous damping C_ζ,

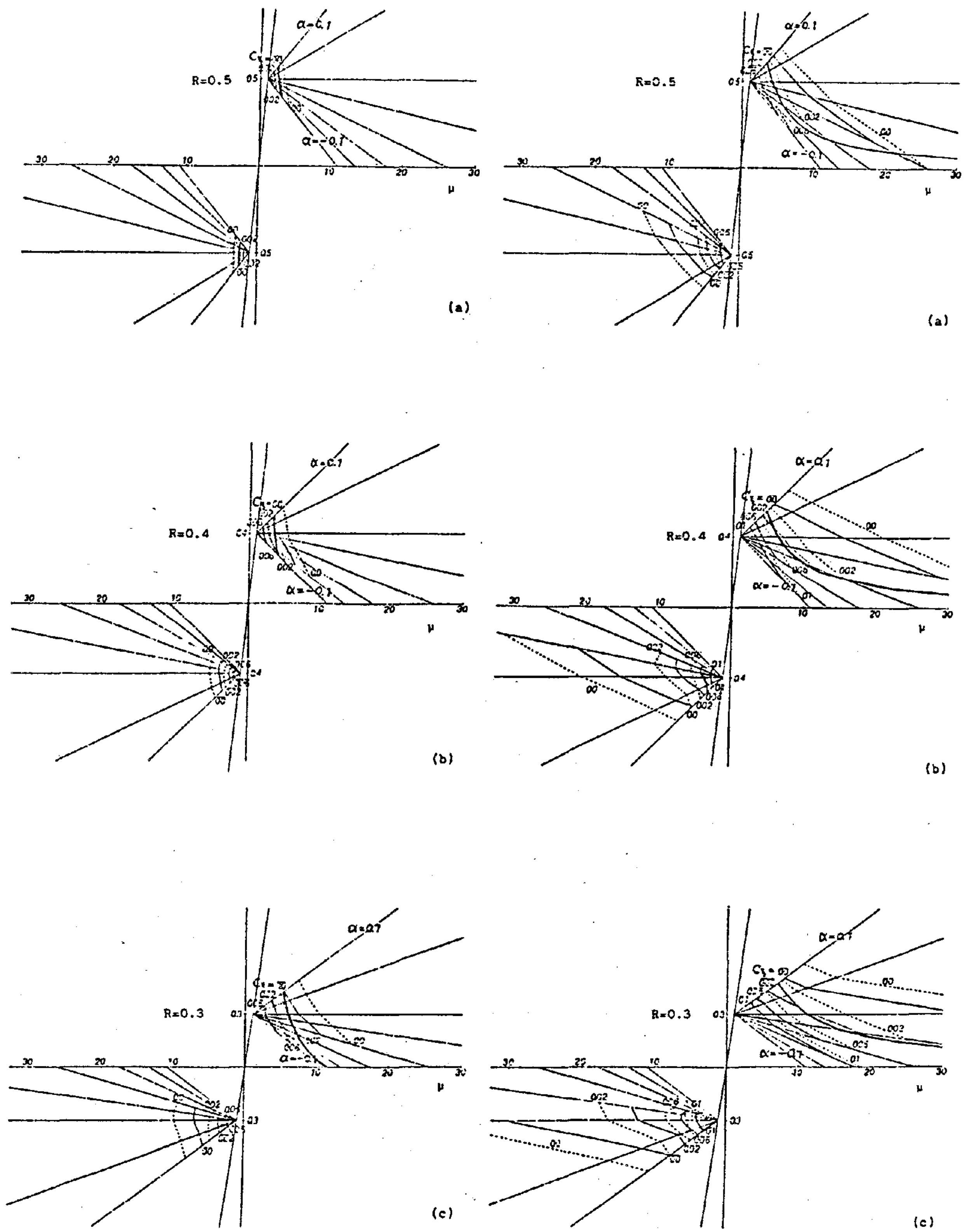
where

R = yield level Q_Y/linear response Q_L

α = yield stiffness/initial elastic stiffness

For non-linear response analysis of multi-story buildings, the yield level ratios R of the other stories are 20% higher than that of the lowest story.

It is observed that damage concentration in multi-story buildings is very large compared with one-story buildings when the yield level ratio R in a particular story of multi-story buildings is slightly smaller than those of the other stories.


R=0.5
(a)
R=0.5
(a)
R=0.4
(b)
R=0.4
(b)
R=0.3
(c)
R=0.3
(c)
μ
30
20
10
10
20
30

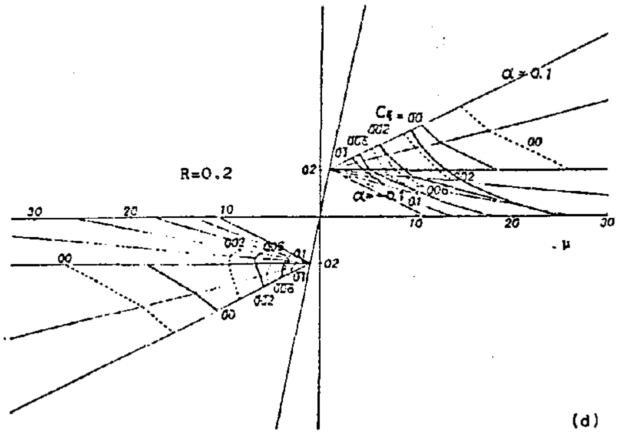


Fig. 3 Non-linear response of single-degree-of-freedom systems

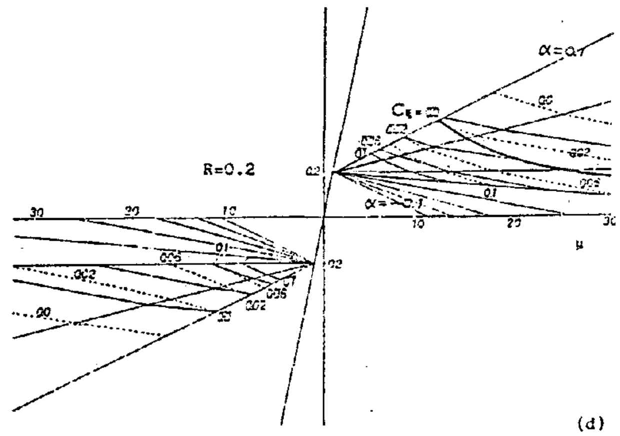


Fig. 4 Damage concentration in the lowest story of multi story shear-type building

(——— mean value
------- mean + standard deviation)

5. Damage Distribution

The elastic and plastic response of a building is almost predictable by that of a building with one-story (correctly called, elastic and plastic response of a single-degree-of-freedom system) in case, the first vibrational mode of the multi-story building subjected to a strong earthquake motion is predominant, maximum response of each story attains the yield level of each story simultaneously, the restoring force characteristics of each story almost same and each story has plastic deformation by the ductility factor of the same degree.

This is called here a condition of simultaneous yielding.

In case a condition of simultaneous yielding is established, as the building absorbs the energy of earthquake as a whole, it may be presumed as the most efficient way of the earthquake resisting systems. There are many patterns of destructions with the condition of simultaneous yielding as shown in Fig.5 ~ Fig. 9.

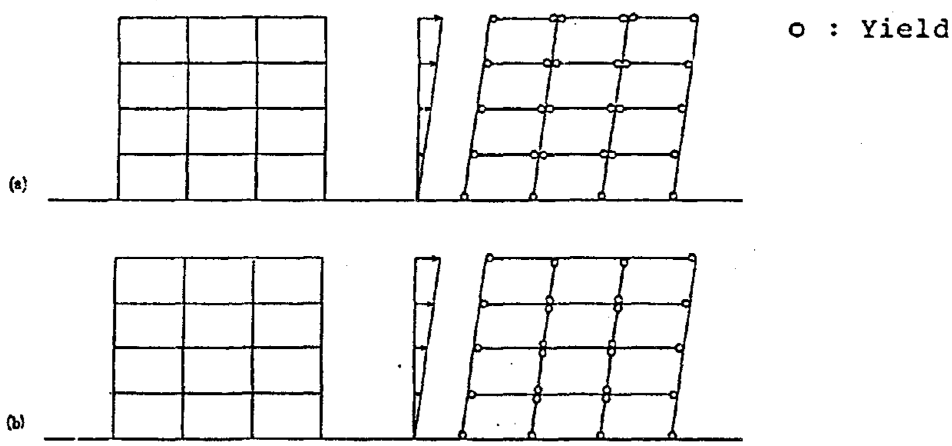


Fig. 5 Simultaneous yielding pattern (1)

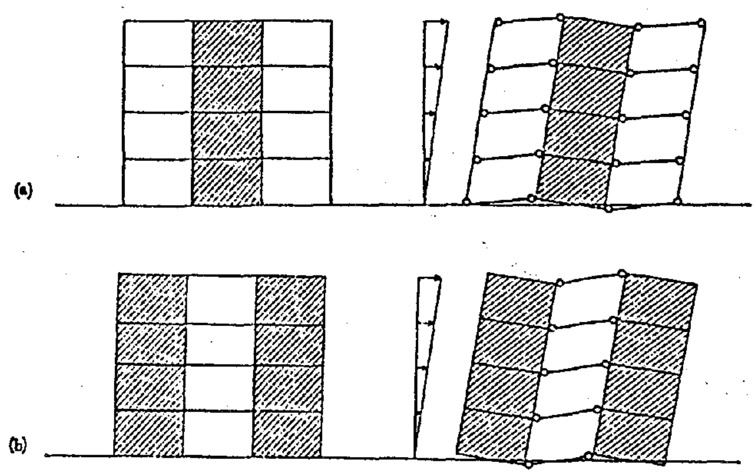
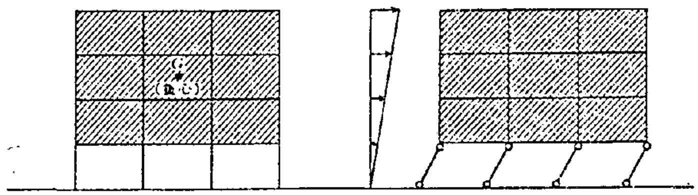


Fig. 6 Simultaneous yielding pattern (2)



o : Yield

Fig. 7 Piling type pattern (3)

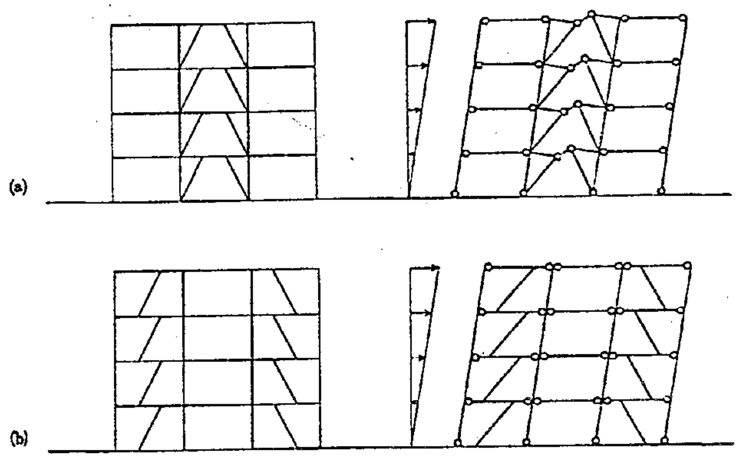


Fig. 8 Simultaneous yielding pattern (4)

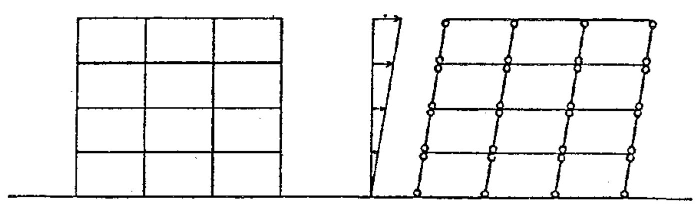
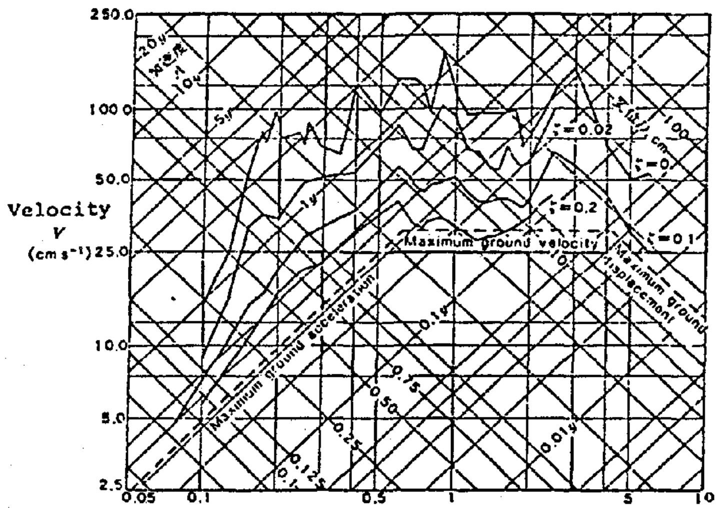


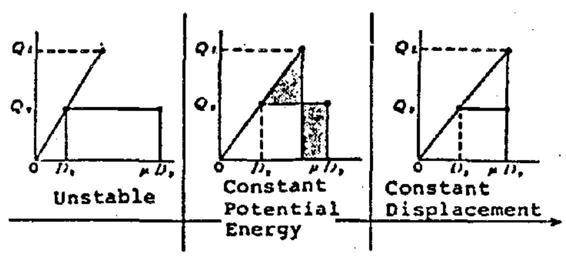
Fig. 9 Simultaneous yielding Pattern (5)

The third destruction pattern shown in Fig. 7 is not that o the simultaneous yielding, but there is a building of similar type as an example of the multi-story building which can be handled as that of one-story. This example shows that the structure above the first story has much stiffness and strength compared with those of the first story. In such a case, it is possible to calculate just like a building of one-story, when the portion above the first story is taken as a large mass considering the earthquake force acting on the center of the portion. Of course, it is quite same to evaluate the story shear force of the first story based on the horizontal shear force distribution for the multi-story structures. The fact that many damages occurred on these buildings comparatively is because the natural period is shortened by the effect of the upper wall.



Natural Period of Building

Elastic Response Spectra El-Centro NS (1940)



Range (I) Range (II) Range (III)

Natural Period of Building

Fig. 10 General tendency of single-degree-of-freedom system (Newmark)

Dr. Newmark made a study on the general inclination of elastic-plastic response by the earthquake response analysis of a model with one-story having the restoring force characteristics with degrading stiffness systems and the summary of this is shown in Fig. 10.

Looking at Fig. 10, the bearing capacity (strength) of a building is more effective than to expect the ductility of the building, when the natural period T of the building is shorter than the predominant period T_G (in the case of El-Centro NS 1940, it is around 0.4 sec.).

This is called the Period Range (I). When the natural period T of a building is longer than the predominant period T_G of earthquake motion, there is the Period Range (II) in which the theory of the potential energy is established with safe side estimation. In this case, the following formula by Dr. Newmark can be applicable.

$$\frac{\text{Yield Shear Force } Q_Y}{\text{Elastic Maximum Shear Force } Q_L} = \frac{1}{\sqrt{2\mu - 1}}$$

(μ: Ductility Factor)

In addition, when the natural period of a building is very long (T= 4 ∿ 5 sec.), there is the Period Range (III) in which the elastic response deformation is the same as the plastic deformation, so the following expression can be used.

$$\frac{\text{Yield Shear Force } Q_Y}{\text{Elastic Maximum Shear Force } Q_L} = \frac{1}{\mu}$$

The writer made a further, narrow investigation about the elastic and plastic response of one-story building in the Period Range (I) and (II) above mentioned.

The restoring force characteristics used for the investigation is the peak-oriented degrading stiffness systems with zero yield stiffness and the equivalent viscous damping C_ζ= 0.02. Non-linear response is compared with the elastic-response as shown in Fig. 11.

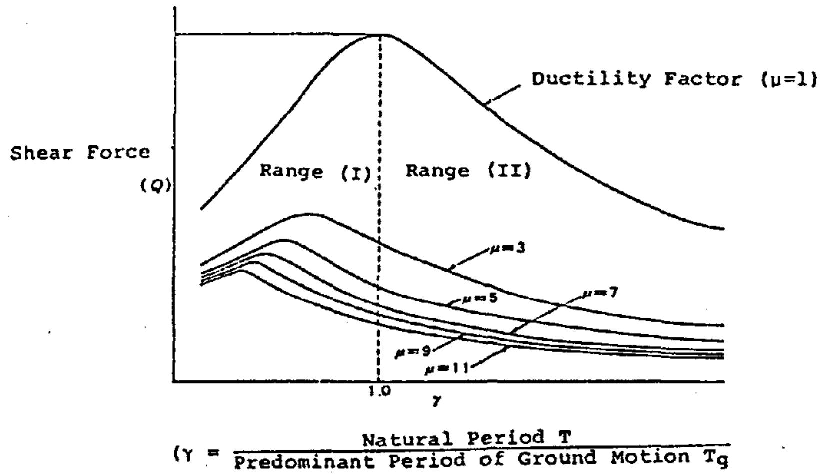


Fig. 11 Non-linear response

The ratio of the yielding shear force to elastic shear force (μ=1) doesn't become so small when the natural period of the building is in the Period Range (I), where γ in Fig. 11 (the value of the natural period T divided by the predominant period of earthquake motion T_G) is smaller than unity, even if the ductility factor μ is quite large.

So design shear force (base shear) must be raised to the level higher than the horizontal line at the peak of elastic shear force. Accordingly, a building in this Period Range (I) should be designed so that the shear force capacity of the building can be enough to resist against the shear force due to strong earthquake motion, for example, utilizing seismic resisting shear walls.

When the natural period T of a building is in the Period Range (II), that is γ in Fig. 11 is equal to, or larger than the value of unity, Q_Y divided by Q_L is equal to $1/\sqrt{2\mu - 1}$ by the theory of potential energy and this expression examined by Dr. Newmark can be evaluated fairly with the safe side estimation as shown in Fig. 3 by the thick lines.

In this range, there is a case that ratio of yielding shear force to the elastic shear force is smaller than 0.15, when the ductility factor μ is quite large.

6. Damage Concentration

The difficulty of realizing a building with the beam yielding type is pointed out by Dr. H. Akiyama as follows, "It involves the following difficulties in making the beam strength smaller than the column strength:

1) It is difficult to evaluate the accurate strength of beams on account of the floor slabs.
2) In the case of an ordinary rigid-frame structure consisting of orthogonal plane frameworks, the strength ratio of the beams to the columns increases $\sqrt{2}$ times when the horizontal seismic force is acting at the direction with an angle of 45° to the plane frameworks."

As far as the author knows, there are very few studies having been conducted on the ultimate strength of the beams by the combined effect of floor slabs.

If a multi-story building with column yielding type has smaller strength in a certain story, e.q., the first story than that of the other stories, concentrated damage will occur in the first story.

As mentioned above, Dr. Newmark's constant potential energy formula (Area of triangle OAB = area of trapezoid OCDE) can be applied, when the natural period T of one-story building is greater than the predominant period T_G of earthquake motion as shown in Fig. 12.

On the other hand, the author's response study on multi-story shear-type buildings with degrading stiffness systems shows the following tendencies.

1) When the fundamental natural period T of the building is greater than the predominant period T_G of earthquake motion (Period Range II) and damage concentration occurs in a specific story of the multi-story shear-type building, the energy of triangle OCD shown in Fig. 13 seems to be consumed to reduce the response energy of other stories. This tendency was suggested by Prof. E.Vanmarcke based on simuletion study.

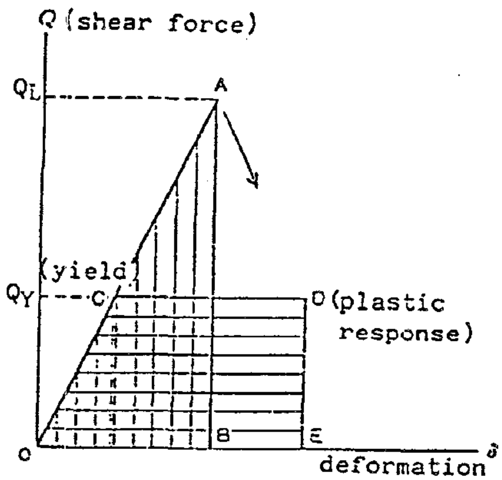


Fig. 12

Constant potential energy for one-degree-of-freedom systems

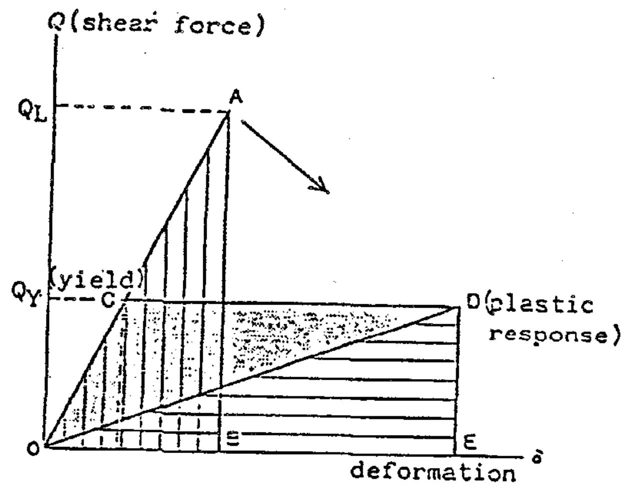


Fig. 13

Constant potential energy for multi-story shear-type buildings

In this case, the area of triangle OAB is equal to that of triangle ODE and, therefore, the constant potential energy can be written for multi-story building with zero stiffness after yielding as

$$\mu = \frac{1}{R^2}$$

The maximum ductility factur μ obtained by the above expression for multi-story buildings with various yield stiffness slopes is shown by the thick lines in Fig. 4.

2) When damage concentration occurs at the top story of a building, the response ductility factor μ is larger than $\mu = 1/R^2$ for the multi-story building with zero yield stiffness.

3) Such a great damage concentration may not occur with high-rise buildings, if a proper response analysis is made to scatter the plastic deformation throughout the stories.

4) When the fundamental natural period T of the building is smaller than the predominant period T_G of earthquake motion (Period Range I), damage concentration estimated by maximum ductility factor μ is much larger than that of multi-story shear-type buildings with longer fundamental natural period $T > T_G$.

7. Damage Distribution and Concentration in Multi-Story Buildings with Eccentricity

The above concept can be extended to multi-story shear-type buildings with one-axis eccentricity by using a proposed yield level ratio $\overline{R}$ for torsional effect corresponding to each story of the buildings.

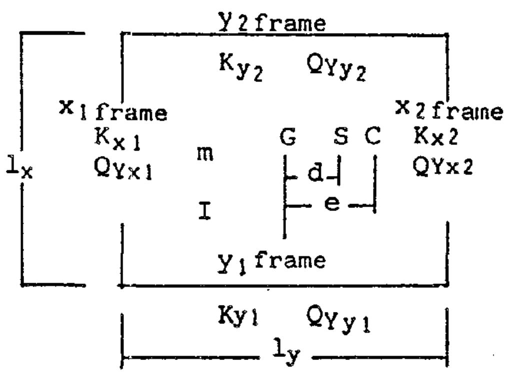


Fig. 14 Floor plan with eccentricity

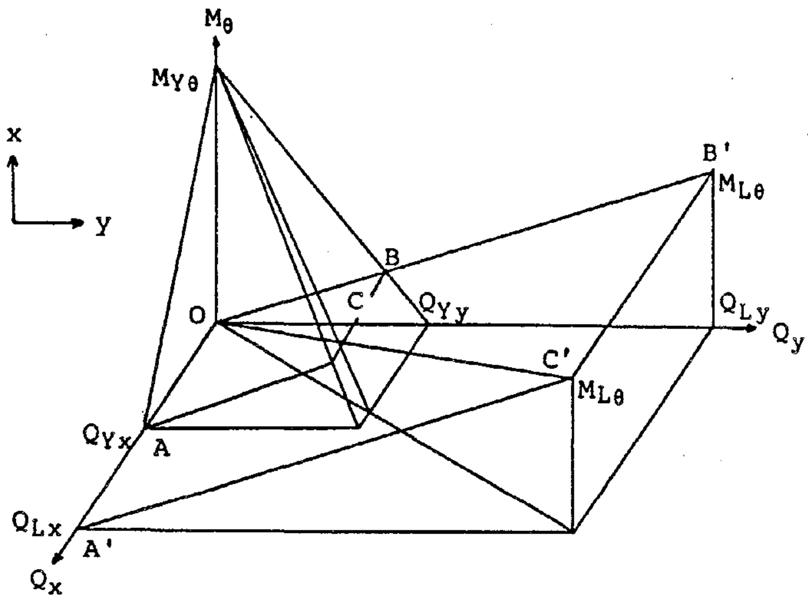


Fig. 15 Damage prediction surface

Fig. 14 shows a floor plan of a story in a multi-story shear-type building with one-axis eccentricity in the x direction, where

G: center of mass

c: center of rigidity

s: center of yield shear strength $Q_{yx2}(\frac{ly}{2} - d) = Q_{yx1}(\frac{ly}{2} + d)$

e: eccentric distance

d: distance between the acting point of story shear force in linear system in the x direction and the center of yield shear strength

lx, ly: plan dimention in the x or y direction, respectively

QYxi, QYyi: yield shear strength of xi or yi shear resisting frame (i=1, 2)

The torsional yield level ratios $\bar{R}x$ and $\bar{R}y$ which correspond to the values of yield shear force / maximum linear shear force in the x and y directions can be defined as follows by using a proposed damage prediction surface for torsional effect as illustrated in Fig. 15 ;

1) In case the maximum linear shear force in the x direction happens to occur when the linear shear force in the y direction is zero,

$$\bar{R}x = OA / OA'$$

2) In case the maximum linear shear force in the y direction happens to occur when the linear shear force in the x direction is zero,

$$\bar{R}y = OB / OB'$$

3) In case the maximum linear shear forces in the x and y directions happen to occur at the same time,

$$\bar{R}x = \bar{R}y = OC / OC'$$

In general, the torsional yield level ratios $\bar{R}x$ and $\bar{R}y$ are in the following ranges as the above cases are the extreme cases,

$$\bar{R}x = OA / OA' \sim OC / OC'$$

$$\bar{R}y = OB / OB' \sim OC / OC'$$

In Fig.15

$$Q_{Yx} = Q_{Yx1} + Q_{Yx2}$$
$$Q_{Yy} = Q_{Yy1} + Q_{Yy2}$$
$$M_{Y\theta} = \min(Q_{Yxi} \cdot l_y) + \min(Q_{Yyi} \cdot l_x) \quad (i = 1, 2)$$

and

$$Q_{Lx} = Q_{Lx1} + Q_{Lx2}$$
$$Q_{Ly} = Q_{Ly1} + Q_{Ly2}$$
$$M_{L\theta} = Q_{Lx} \cdot d$$

where

Q_{Lxi}, Q_{Lyi}: maximum total shear force for the shear resisting frames (i=1, 2) in the x or y direction in linear systems

$M_{L\theta}$: maximum moment with respect to the center of yield shear strength in linear systems

Table 1 Two story model

Story	Mass m	moment of interia I	stiffness $K_{xi} = K_{yi}$	linear shear force $Q_{Lx} = Q_{Ly}$
	$(t \cdot s^2/cm)$	$(t \cdot s^2/cm)$	(t/cm)	(t)
2	0.1	10666.7	20.68	59.74
1	0.1	10666.7	20.68	96.14

(i=1, 2)

Non-linear response analysis is carried out for two story shear type building models with rigid diaphragm in each floor level shown in Table 1 by a step-by-step time integration response analysis. The two story models with the fundamental natural period T=0.5 sec. are subjected to the 20 pairs of the modified ground motions shown in Fig. 1 in the x and y directions simultaneously, and each resisting frame has origin-oriented degrading stiffness systems.

Maximum response ductility factor in each shear resisting frame for the two story model without eccentricity is shown in Fig. 16.

R=0.25
μ=5.35

R=0.25 μ=5.35 R=0.25 μ=5.35 $\bar{R}x$=0.25 $\bar{R}y$=0.25

R=0.25
μ=5.35

μ =ductility factor

R =yield level ratio Q_L/Q_Y

$\bar{R}x$, $\bar{R}y$=yield level ratio for torsional effect in the x or y direction

R=0.25
μ=7.13

R=0.25 μ=7.13 R=0.25 μ=7.13 $\bar{R}x$=0.25 $\bar{R}y$=0.25

R=0.25
μ=7.13

(Damage Distribution)

Fig. 16 Two story model without eccentricity

To avoid the complex behavior in linear response in the case of the two story models with eccentricity each shear resisting frame has equal rigidity, but unequal yield shear strength with respect to the x direction as shown in Fig. 17 and Fig. 18.

The torsional yield level ratios $\bar{R}x$ and $\bar{R}y$ of each floor of the models and the corresponding maximum response ductility factor of each shear resisting frame with yield level ratio R are shown in Fig. 17 and Fig. 18, respectively.

As shown in Fig. 17 and Fig. 18, damage concentration in the multi-story buildings with eccentricity is extremely large when the yielding level ratio $\bar{R}x$ or $\bar{R}y$ for torsional effect in a particular story of the multi-story buildings is slightly smaller than those of the other stories.

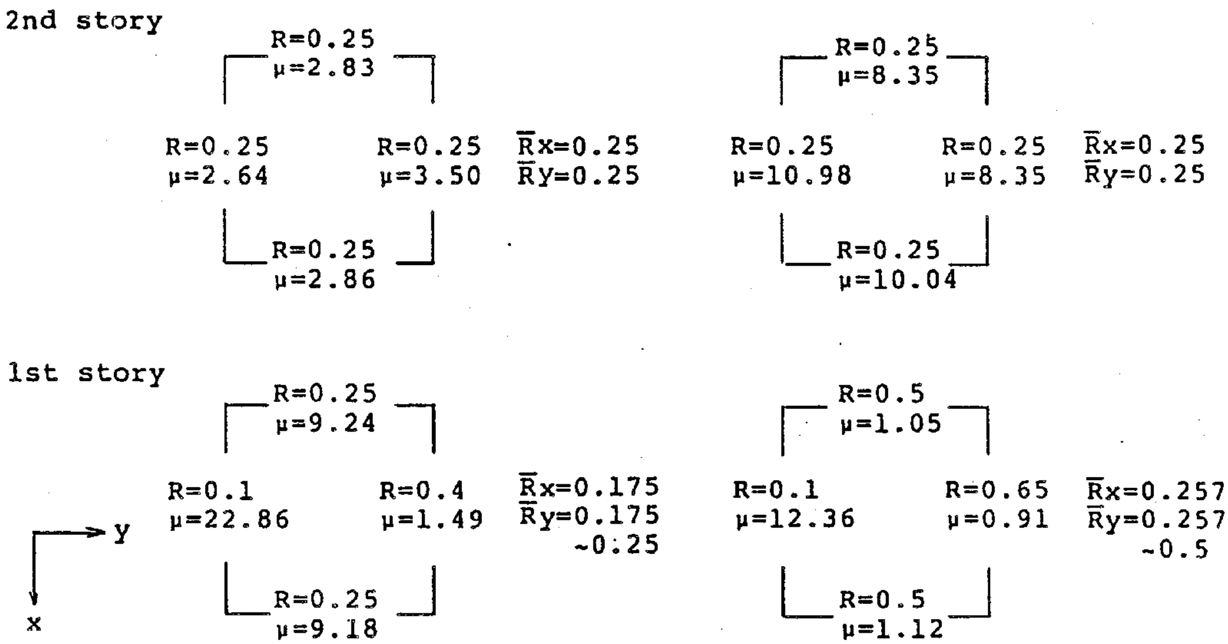


Fig. 17 Two story models with eccentricity in the 1st story

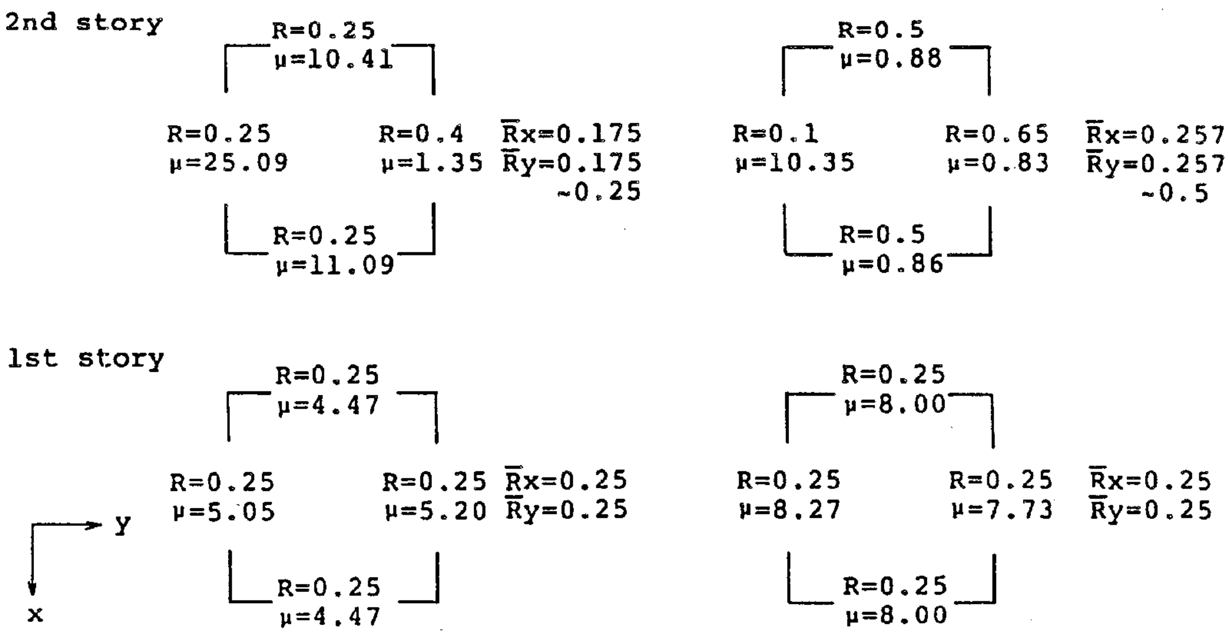


Fig. 18 Two story models with eccentricity in the 2nd story

8. Conclusions

1) There are two kinds of damage, distributed damage and concentrated damage.

2) Damage concentration is extremely large for a multi-story shear-type building when the yield level ratio R or the torsional yield level ratio $\bar{R}$ in a particular story is slightly smaller than those of the other stories.

3) It will be recommended that higher design base shear coefficient shuld be used for the buildings, when damage concentration is predicted.

4) It will also be recommended that higher design story shear should be used at least for the story with torsional effect in addition to higher design shear coefficient for the buildings.

9. Remarks

Study on damage distribution and concentration in multi-story shear-type buildings with one-axis eccentricity has been carried out by Dr. Satsuya SODA, Assistant Prof. and Mr. Seiichiro YASUDA, graduate student, Chiba University under the guidance of Prof. Masakazu OZAKI. This study is not completed so far and is extensively continuing.

PROBLEMS ASSOCIATED WITH "WEAK-BEAM" DESIGN OF REINFORCED CONCRETE FRAMES

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SUMMARY

"Weak-beam strong-column" design has been recommended for the better seismic performance of reinforced concrete frames, but several problems need be clarified before such design methods can be effectively applied in practice. This paper reviews some problems associated with the design of reinforced concrete frames; i.e., (a) the evaluation of the beam flexural capacity including the contribution of floor slabs, (b) the construction of spandrel walls, (c) the determination of required column strength, and (d) the design of beam-column connections. Finally, the Japanese Buiding Code Requirements and the proposed ATC-03 Seismic Regulations for Buildings are compared by designing simple frame structures.

INTRODUCTION

A structure can be designed earthquake resistant if the structure is provided with either (a) a large lateral load resisting capacity or (b) a large inelastic deformation capacity. The former design concept is sometimes called "strength design", while the latter is called "ductility design".

When the ductility design method is applied to a structure, the structural members must be carefully designed so that the premature brittle failure, such as shear and anchorage failure, should be prevented even under the most severe earthquake conditions. In order to reduce the inelastic deformation demand at the critical sections, the vibrational energy of the structure should be efficiently dissipated through stable hysteresis at the plastic hinges. It is not desirable to develop plastic hinges in columns because the columns have to support upper stories and because the columns are hard to develop large inelastic deformation. It is more desirable to develop plastic hinges in beams because (a) large inelastic deformation can be easily developed, (b) stable and large hysteretic energy can be dissipated without significant loss of resistance, (c) failure of some beams will not cause the total collapse of the frame, and (d) plastic hinges can be formed at all beam ends simultaneously. Therefore, a frame structure should be designed to develop "the beam sidesway mechanism (Fig.1.a)" rather than "the column sidesway mechanism (Fig.1.b)". In the beam sidesway mechanism, the hysteretic energy can be dissipated uniformly throughout the structure. This is achieved by designing the beams to be weaker than the columns or by designing the columns to be stronger than the beams at all beam-column joints. The concept is normally called as weak-beam strong-column frame design. Some of the problems associated with the weak-beam strong column design are reviewed in this paper.

FLEXURAL CAPACITY OF BEAMS WITH SLAB

In order to visualize realistic nonlinear behavior of three-dimensional reinforced concrete frames at or near the ultimate resistance for the weak-beam design, it is necessary to incorporate the effect of slabs on hysteretic behavior of beams under reversal of simulated earthquake loading.

The slab cast monolithically with beams is believed to contribute to the flexural stiffness as well as the flexural resistance of the beams. There have been a significant number of research efforts in the past to estimate the degree of slab contribution to the beam stiffness in the elastic stage (Ref. 8,12,19). On the other hand, it is not well understood how much width of slab could participate in developing the ultimate capacity of the beam. Some studies suggested that the entire width of slab could be effective at the ultimate stage (Ref. 14). However, most design codes (Ref. 1,5,15) permit the use of a limited width of slab in evaluating the ultimate strength of a beam, especially in case of horizontal loading. For example, present Architectural Institute of Japan Standard for Structural Calculation of Reinforced Concrete Structure (AIJ Standard) specifies that the slab width equal to one-tenth of the span length on each side of a beam may be considered effective in contributing to the stiffness of the beam if the clear span width to the adjacent parallel beam is greater than one-half of the span length. This conservatism is largely due to the difficulty in transferring slab forces to the joint through torsion of transverse beams. In other words, it has generally been believed that the effective width of slabs for the ultimate strength of beams can not be very large as long as the torsional stiffness and strength of transverse beams are limited.

Half-scale three-dimensional reinforced concrete interior beam-column subassemblages with floor slabs were tested at the University of Tokyo to clarify the effective width of slabs that contributes to the ultimate flexural capacity of the beams (Ref. 7,16,17). The geometry of the test specimen is shown in Fig. 2. The specimens represented a portion of interior beam-column-slab subassemblages removed from three-dimensional reinforced concrete frames by cutting off the beams and columns at arbitrarily assumed inflection points. In consideration of general construction process, the concrete was cast in the up-right position in two stages; i.e., first to the top of the slab, and then into the upper column. The specimens were designed so that the beams should yield prior to the columns under unidirectional lateral loading, and that the columns should yield simultaneously with the beams under bidirectional lateral loading.

Specimen SU20N was subjected to unidirectional lateral loading, while Specimen SB20N was tested under bidirectional lateral loading. The magnitude of column axial forces was maintained at 18 ton (average axial stress of 20 kgf/cm**2) during the test. Figure 3 shows a specimen placed in the loading setup. The base of the subassemblage was supported by a universal joint. The free ends of the beams were supported by vertical rigid members equipped with universal joints at two ends, creating roller support conditions at beam ends in the horizontal plane. Constant vertical and reversing horizontal loads were applied at the top end of the upper column by three servo-controlled actuators. The basic displacement paths given to the test specimens are shown in Fig. 4.

The effective width of slabs cast monolithically with beams is studied from the strains in the slab reinforcement parallel to the longitudinal beam (North end beam) at the face of the transverse beams (Fig. 5). In recognizing the difference in displacement histories of the two specimens, strain distribution was plotted at the peak of loading cycles under the same loading condition; solid lines for the negative beam moment (slab in tension) and dashed lines for the positive beam moment (slab in compression). The load stage numbers in circles correspond to the load stage numbers given in Fig. 4. Note that (a) in either unidirectional loading test (SU20N) or bidirectional loading test (SB20N), strain amplitudes increased toward the longitudinal beam, and decreased gradually with distance from the longitudinal beam, (b) strain amplitudes of both specimens increased with the deflection of the longitudinal beam, (c) the strains of all slab reinforcement remained in tension even under the positive beam moment, although some decrease in strain amplitudes was observed near the longitudinal beam. The region in which slab reinforcement yielded under negative moment spread with increasing longitudinal beam deflection, and finally almost all slab reinforcement exceeded the yield strain at the load stage at which the story displacement reached about one-twentieth of the story height.

The crack patterns of slabs at the final load stage of the two specimens are shown in Fig. 6; solid lines and dotted lines indicate the cracks developed under positive and negative loading, respectively. Specimen SU20N developed straight cracks in the slab perpendicular to the longitudinal beams (N and S beams) at a small story displacement. These cracks were observed as the extension of flexural cracks in the longitudinal beams, and developed along the entire length of the transverse beams, indicating the contribution of the entire slab width to the beam flexural resistance. The crack patterns in the bidirectional loading test (SB20N) are more complicated, but the cracks were clearly observed to develop along the entire length and on both sides of each beam.

The effective width of slab is examined analytically in Fig. 7. The observed beam shear-deflection relations (solid lines) are compared with those computed using different slab widths (dotted lines); i.e., (a) entire slab width of the specimen, (b) effective slab width recommended by AIJ Standard, and (c) null slab width. The plane section was assumed to remain plane after deformation in the analysis. Note that observed maximum resistance of a T-beam was nearly equal to the maximum value calculated with the entire slab width. On the other hand, the observed initial stiffness was close to the one calculated with the effective width of AIJ Standard.

Let us examine why the entire width of slabs can contribute to the flexural resistance of the beam although slab forces were generally believed difficult to be transferred to the joint through the shear and torsion of the transverse beams. In the test, the transverse beams were observed to deflect in torsion about its longitudinal axis and in bending in the horizontal plane as shown in Fig. 8. The beam deformed in the two modes because the transverse beams were pulled by the reinforcement of slab A in tension under bending. The slab B, stiff in its own plane, could not follow the beam curvature; hence, gaps must develop along the transverse beam. Therefore, tensile stresses from the slab reinforcement acted on both faces of the transverse beam as schematically shown in Fig. 8. The effect of the tensile stresses acting on the opposite sides of the transverse beam will be cancelled, the torsional moment and horizontal shear being reduced.

The preceding explanation applies only to an interior beam-column-slab subassemblage. In case of an exterior beam-column-slab connection, a transverse beam has a slab only on one side. In other words, there act no counteracting tensile stresses from the opposite slab reinforcement. Therefore, torsional moment can grow significantly large so that the width of slab that contributes to the beam flexural resistance is limited by torsional strength of the transverse beam.

From the above observation, it can be concluded that the effective width of slabs monolithically cast with interior beams spreads with increasing beam deformation, and the entire width of slabs can contribute to the ultimate flexural capacity of the beam even under bidirectional loading condition.

CONSTRUCTION OF SPANDREL WALL

One advantage of reinforced cocnrete construction is its ability to build various elements of the structure monolithically. In Japan, this advantage is greatly exploited, and it is a common practice for Japanese constructors to place concrete simultaneously into structural members as well as architectural elements, such as spandrels, parapets, and balconies. This practice creates a favourable condition for the reinforced concrete construction in the economic competition.

On the other hand, the architectural concrete often causes unfavourable interaction with the structural concrete; i.e., monolithically placed spandrels in reinforced concrete frames shorten the deformable length of columns, and force the columns to fail in shear. Earthquake experiences reveal such shear failure in reinforced concrete columns. From the weak-beam design point of view, the monolithic spandrel is also undesirable because it contributes to the flexural resistance of the girder and increases the column design actions. Therefore, it is clear that such nonstructural elements should not be introduced into the structure unless they are duely considered in the process of structural analysis, proportioning and detailing.

In order to avoid such failure in columns, it has been recommended that the spandrel wall should be separated from the adjoining columns by inserting vertical slits while the spandrel is attached monolithically to the girder. The vertical slit certainly recovers the column deformable length and also eliminates the contribution of the spandrel wall to the girder flexural resistance. A question arises as to whether such "slitted" girders can truly develop ductile plastic hinges with ample hysteretic energy dissipation capability at the critical sections.

Half-scale interior beam-column connection specimens were tested under constant vertical load and reversals of simulated earthquake loading, at the University of Tokyo, to study the effect of the spandrel walls on the beam end rotational capacity and energy dissipation capacity (Ref. 3,9). The variables of the test were (a) the width of vertical slits between the column face and spandrel walls, (b) the ratio of the amount of top and bottom beam flexural reinforcement, (c) the ratio of column width to diameter of beam flexural reinforcement, and (d) the anchorage of horizontal reinforcement of the spandrel walls into the column. All specimens were designed so that the beams yielded prior to column yielding and to shear failure in the beams and columns.

Let us first examine the behavior of three specimens SW00, SW01, and SW10. Specimen SW00 was not provided with a slit, while Specimens SW01 and SW10 had 1 cm and 10 cm wide slits, respectively. The horizontal reinforcement in the spandrel walls of Specimen SW01 was anchored into the column through the slits. The column width of the specimens was chosen large enough to protect the beam-column connection; i.e., the column width was approximately 30 times the diameter of beam longitudinal reinforcement.

The crack patterns of the specimes are compared in Fig. 9 at a story drift angle of 1/100, a displacement amplitude which may be expected in a design intensity earthquake. Solid lines and dotted lines indicate the cracks due to positive and negative loading, respectively. These crack patterns clearly show the stress flow in each specimen. Dominantly flexural cracks developed in the beams of all specimens. More cracks were observed in the column of Specimen SW00 indicating large input actions corresponding to increased flexural resistance by the monolithic spandrels. Diagonal cracks were observed in the spandrels in Specimen SW00, while no cracks developed in the spandrels in

Specimen SW10. Specimen SW01 developed vertical flexural cracks in the spandrels caused by forces introduced by the spandrel horizontal reinforcement anchored into the column. Once the edge of a spandrel started to make contact with the column in Specimen SW01, the diagonal cracks started to developed in the spandrel.

The envelope curves of the column shear-interstory deflection relations of the three specimens are compared in Fig. 10. Specimen SW00 without slit had the highest initial stiffness and attained the largest resistance, but the resistance started to decay at a story drift angle of 1/100 after crushing and spalling of spandrel concrete. Note that the spandrel walls were effective in increasing the stiffness and strength of the structure, but the high resistance could not be maintained after flexural yielding. The horizontal reinforcement of a spandrel wall anchored into a column contributed to the flexural resistance of the beam, hence Specimen SW01 showed higher resistance than Specimen SW10 with 10 cm wide slits. However, some decay of resistance was observed in Specimen SW01 after the maximum resistance was attained at point M in Fig. 10, primarily due to deterioration in anchorage of the spandrel horizontal reinforcement in the column. The resistance of Specimen SW01 started to increase when the spandrel wall made contact with the column. Specimen SW10 showed stable resistance after flexural yielding of beams, and attained large rotational capacity within the 10 cm wide slitted zone. The attained maximum resistances of the three specimens were approximately estimated by the routine flexural theory.

The required slit width w to prevent the contact between the spandrel and column at a given story drift angle R can be reasonably estimated by the following formula;

$$w = R \times (\text{Spandrel Height}) \times (\text{Nominal Span}) / (\text{Clear Span})$$

Hysteretic energy dissipated by specimens SW00, SW01, and SW10 in every half cycle of loading is compared in Fig. 11. The specimens were subjected to the same lateral displacement history at the top of the column. The strongest specimen SW00 with monolithic spandrels dissipated the largest hysteretic energy in every half cycle even after the crushing and spalling of concrete in the spandrel. Namely, if a monolithic spandrel beam is designed to yield prior to the column, the subassemblage is capable of dissipating a large hysteretic energy. However, the ratio of dissipated energy to the maximum resistance was observed to be smallest in Specimen SW00 which was attributable to the deterioration in resistance and the "pinching" in a hysteresis loop after crushing and spalling of spandrel concrete. Specimens SW01 and SW10 showed a stable spindle-shaped hysteresis.

The effect of anchorage of spandrel reinforcement in the column can be observed in Fig. 12, in which the story shear-interstory dispalcement relations observed in Specimens SW01 and SWB1 are compared. The latter was constructed under the identical specifications except for the anchoring of the spandrel reinforcement; i.e., the spandrel reinforcement was terminated within the spandrel. The resistance of Specimen SW01 was much higher than that of Specimen SWB1. Specimen SW01 also developed fat stable hysteresis loops, while Specimen SWB1 showed some "pinching" characteristic. This was attributable to reversals of high stresses and strains developed in the beam top reinforcement of Specimen SWB1. The deformation must take place within the narrow slitted zone of the girder, and the beam longitudinal reinforcement is severely stressed and strained within the critical zone. Therefore, the beam-column connection must be carefully designed to reduce the pinching behavior caused by the deterioration of bond stress transfer mechanism of the beam reinforcement within the connection.

From the above observations, it can be concluded that large deformation and energy dissipation capacities can be achieved by inserting a narrow vertical slit between the column and spandrel wall, provided that the depth of the column is large enough to prevent the bond deterioration and shear failure within the beam-column connection. The maximum resistance of spandrel beams with and without slit can be approximately evaluated by the flexural analysis. If the spandrel wall is actively taken into structural design, the spandrel can contribute to increase the stiffness and strength of the structure.

COLUMN OVERDESIGN FACTOR FOR UNIAXIAL EARTHQUAKE MOTION

In the weak beam design, columns should be provided with a high degree of protection against yielding in order to avoid the possibility of the column sidesway mechanism in any story even during the most severe earthquake. Note that calculated moments in columns and beams under code-specified lateral loads are not the ones developed during such an earthquake motion. Column moments deviate from those calculated by an elastic static analysis due to the dynamic contributions of higher modes. In addition, beam flexural strength may be enhanced beyond the calculated ultimate resistance by the following reasons; (a) actual yield strength of reinforcement might be greater than the specified strength, (b) additional reinforcement might be placed for the purpose of construction convenience, and (c) stress exceeds yield strength due to strain hardening after large inelastic deformation. The columns should be duely protected against the increased beam actions as well as the dynamic effect through the usage of column overdesign factors. In this study, required level of overdesign factors due to the dynamic effect was studied through nonlinear dynamic analyses.

In New Zealand, the weak beam design was practically adopted in the design procedure; i.e., the dynamic magnification factor ω, the ratio of the column design moment to that obtained by an elastic frame analysis for the code static loads, of floor levels at and above 0.3 times the height of the structure is given as follows (Ref. 15),

$$\omega = 0.6\ T + 0.85$$

and

$$1.3 < \omega < 1.8$$

where T = fundamental period of the structure. Below this specified level, a linear variation should be assumed. However, in no case should the value at the first story level be taken less than 1.3. The value of the dynamic magnification factor may be reduced to 1.0 at the roof level and at ground floor level.

In order to study the overdesign factor for columns, an analytical model consisting of one column with beams on both sides cut off at the inflection points was separated from an ideal planar frame. Beam strength in all stories except for ground and roof levels, and the column strength at the top and the base were determined from an elastic analysis under code specified lateral loads. The columns in intermediate stories were assumed to behave within the elastic range or permitting cracking only as shown in Fig. 13.a and b. The Takeda model was used to represent the hysteresis rules of beams and columns if plastic hinges were allowed to form. The nonlinear earthquake response analysis would present required level of column strength if the columns were to remain elastic. Two studies were made using different levels of seismic design loads.

<Case I> Four- and eight-story frames with 3.6 m story height, 6.0 m span width and 36.0 ton weight per floor, were analysed under the EW component of the Hachinohe Harbor motion recorded during the 1968 Tokachi-Oki earthquake (Ref. 10). The design base shear coefficient in this case was 0.2. The natural period was 0.29 sec or 0.65 sec for the four-story frame, and 0.64 sec for the eight-story frame. Figure 14 shows the results of the response analysis, in which α means stiffness degrading rate after cracking and β is damping coefficient. Story maximum response ratio is the sum of top and bottom column moments in a story as determined from the response analysis, divided by the corresponding design moments. Joint maximum response ratio means the sum of maximum response column moments above and below the joint, similarly divided by design moments. From the study, the followings were found

(a) The maximum response ratios increased with natural periods in frames having the same number of stories. The higher mode effect appears to be remarkable in the frame with a longer natural period (Fig.14.a and b).

(b) Comparing the response of four- and eight-story frames having the same natural period, the maximum response ratios are larger in the four-story frames. The higher mode effect is more noticeable in a shorter frame having less number of joints (Fig. 14.a and c).

(c) Increased damping makes the maximum response smaller and reduces the influence of the higher modes (Fig. 14.a and d).

(d) When column cracking is taken into account, the response ratios decreases because of increases hysteretic damping (Fig. 14. a and e).

<Case II> Four- and twelve-story frames with 3.3 m story height, 6.0 m span width, and approximately 35 ton weight per floor were analyzed under the NS component of the 1940 El Centro motion (Ref. 6). The design base shear coefficient was increased to 0.3 in this case. The natural periods of the structures were 0.6 and 1.2 sec for the four- and twelve-story frames, respectively. The results of this study are summarized in Fig. 15 and 16. Figures 15.a and 16.a show the overdesign factor at each floor level. This factor indicates the ratio of the maximum response column moment to the design values at a joint. Figures 15.b and 16.b present the ductility factor, the ratio of maximum response rotation to the yielding rotation, of top and bottom columns and and all floor beams. From this result, the following conclusions may be drawn;

(a) In the four-story frame, the overdesign factor tends to be larger in upper floor levels than in lower floor levels. When the ductility factor of beams reached about 2.0-2.5, the overdesign factor distribution seems to resemble that of the New Zealand Code. The overdesign factor at the third level becomes very large.

(b) In the twelve-story frames, the ductility factor distribution indicates the presence of the higher mode effect in the top and lower stories. The New Zealand Code design factor seems to be on the safe side in this case.

If the ductility factor of the most severely damaged beam is to remain within 3.0-4.0, the overdesign factor need be in a range of 1.5 to 1.8, regardless of natural period, to prevent any column hinging except at the top and the bottom of the frame.

COLUMN OVERDESIGN FACTOR FOR BIAXIAL EARTHQUAKE MOTIONS

The column is subjected to lateral loads simultaneously in the two principal directions during an earthquake. Due to this biaxial bending interaction in the two directions, the apparent flexural resistance in each principal direction is reduced from the value under unidirectional bending, which is normally recognized as the biaxial bending effect. The biaxial bending effect of columns and the dynamic effect of higher modes tend to cause yielding in columns although the columns are provided with resistances higher than those given by a static analysis of the planar frame under code specified earthquake loads. To avoid a collapse in the weak-column failure mode, a suitable column overdesign factor should be used in the design stage. This study aims at the selection of an appropriate column overdesign factor considering the effect of biaxial bending and higher modes. The seismic response of space frames were studied under two-directional earthquake motions (Ref. 13).

The column hysteresis model for biaxial bending was a modified Takizawa model (Ref. 18) which extended the concept of the ordinary uniaxial degrading trilinear model into the biaxial model making use of the Ziegler's hardening rule (Ref. 20). Analytical model of a three-story space frame was simplified into one column with beams in two perpendicular directions which were cut off at the inflection points. Story height was 3 m, weight was 20 ton at each floor. The stiffness of beams was made equal to that of columns. Eight different cases were studied in which the level of beam and column strengths Mb and Mc were varied as shown below. Yielding moments Mbo and Mco of beams and columns, respectively, in the standard model (Case 3) were determined from the elastic analysis using a base shear coefficient of 0.3.

	Case-1	Case-2	Case-3	Case-4	Case-5	Case-6	Case-7	Case-8
Mc/Mco	1.0	1.0	1.0	1.1	1.2	1.3	1.4	1.5
Mb/Mbo	1.2	1.1	1.0	1.0	1.0	1.0	1.0	1.0

The initial period of the structure was arbitrarily chosen to be 0.3 sec. Input earthquake motions were (a) the NS and EW components of the Hachinohe Harbor waveforms recorded during the 1968 Tokachi-Oki earthquake amplified by 1.8 and (b) NS and EW components of the El Centro waveforms recorded during the 1940 Imperial Valley earthquake amplified by 1.8 .

Figure 17 shows the maximum response interstory displacement in the NS direction of unidirectional and bidirectional earthquake response analyses. In the unidirectional analysis, the distribution of interstory displacement in Cases 1 through 3 indicates typical column sidesway mechanism; i.e., the first-story deflection occupies about 60 % of the total displacement. As the column strength increases from Case 4 to 8, deflections in the second and third stories increase in contrast to the decrease in the first-story displacement. In the bidirectional analysis, the first-story displacements in Cases 1 through 3 become more than three times as large as those of the unidirectional analysis because of a remarkable biaxial bending effect in the column. On the other hand, with an increase in column strength, for example in Case 7 or Case 8, slight difference is observed between unidirectional and bidirectional responses.

To investigate the extent of yielding in the beams and columns, the stiffness degrading rate (Deg) in the beam and column plastic hinges subjected to the most severe damage is shown in Fig. 18. As the column strength increases from Case 1 to Case 8, Deg value of columns increases and that of beams decreases. In other words, collapse mechanism shifts from the column sidesway mechanism to the beam sidesway mechanism. The ratio (Deg,col/Deg,beam) of minimum stiffness degrading rates of column and beam ends is defined to determine the collapse mechanism; i.e., if the ratio greater than 1.0 implies the beam sidesway mechanism. Figures 19 and 20 show calculated values in each case, varying the initial period from 0.2 to 0.5 sec. In the unidirectional analysis, beam yielding can be attained for any initial period when the column overdesign factor was greater than 1.2. On the other hand, in the bidirectional analysis, the factor must be greater than 1.4.

DESIGN OF BEAM-COLUMN CONNECTION

In order to expect effective energy dissipation in the plastic hinges at the ends of weak-beams for better seismic performance, beam-column connections must be designed, first of all, not to fail in shear before the flexural yielding of weak-beams, and furthermore, to sustain high shear caused by the yielding of framing members. It is also indispensable that bond of beam bars passing through beam-column connections is prevented from bond deterioration which lowers the capacity of energy dissipation as a total frame.

In Japan, AIJ Standard for reinforced concrete construction does not specify a method to design a beam-column connection against shear nor bond deterioration mainly because earthquake damage was rarely observed in the beam-column connection in Japanese wide-column construction. However, with the rationalization in design, use of higher strength materials, and possible application of the weak-beam design concept, a serious concern has been pointed out. After surveying beam-column test results, specimens of which failed in shear prior to beam flexural yielding, Kamimura proposed an empirical equation to evaluate the ultimate shear strength vu of beam-column connections (Ref. 11). The Kamimura's equation consists of two terms related to the concrete strength fc' (kgf/cm**2) and the shear reinforcement ratio pw in the connection;

$$vu = (0.78-0.0016\ fc')\ fc' + pw\ fy\ /\ 2$$

where fy (kgf/cm**2) is the yield strength of shear reinforcement.

In the United States, a simple method was adopted in the American Concrete Institute Building Code Requirements for Reinforced Concrete (ACI 318-83). In this approach, concrete strength and configuration of connections are considered to be effective on the shear capacity of beam-column connections; the nominal shear stress of the joint should not exceed values given below,

$$vu = 20\sqrt{fc'} \qquad \text{for confined joint,}$$

$$vu = 15\sqrt{fc'} \qquad \text{for others,}$$

in which fc' in psi.

In New Zealand, a fundamentally different approach was developed and is used in the code (Ref. 15), in which beam-column connection is designed against horizontal and vertical shear. The horizontal shear stress in the connection must be resisted by concrete vc and shear reinforcement vsh. The portion of shear carried by concrete is given as

$$vc = (2/3)\sqrt{Cj\ Pe / Ag - fc' / 10}$$

in which Pe/Ag (MPa) = minimum average compressive stress on the gross concrete area of the column above the joint, and Cj = a factor to allocate the effect of axial compression to the two principal directions. The horizontal shear resisted by shear reinforcement is given as

$$vsh = Ajh\ fyh / bj\ hc$$

in which Ajh = total area of horizontal shear reinforcement placed between the outermost layers of top and bottom beam reinforcement, bj = width of joint, hc = depth of column, fyh (MPa) = the yield strength of the shear reinforcement. In addition, the New Zealand Code limits the diameter db of longitudinal bars passing through a joint core as follows,

$$db = hc / 25 \quad \text{when} \quad fy = 275 \text{ MPa},$$

$$db = hc / 35 \quad \text{when} \quad fy = 380 \text{ MPa}.$$

In order to reconcile some of the differences in design methods used for reinforced concrete beam-column connections, half-scale interior beam-column connections were tested under a constant vertical load and reversals of simulated earthquake loading, at the University of Tokyo, to study (a) the effect of column width-to-beam longitudinal bar diameter ratio on bond deterioration of beam bars passing through beam-column connections, and (b) the effect of shear reinforcement ratio on connection shear strength. Variables of the tests were (a) column width-to-beam longitudinal bar diameter ratio, (b) column axial load, (c) amount of connection shear reinforcemet, (d) amount of beam flexural bars. Twelve specimens were tested, and they are summarized in Table 1. Specimens S1, S2, S3, S4, S5, S6, and J6 were designed so that the average horizontal shear stress in the connection would be kept less than the diagonal cracking stress calculated by the Mohr circle analysis at the time of flexural yielding of beams. Specimens J1, J2, J3, J4, and J5 were designed so that shear stress would not reach a stress level calculated by the Kamimura's equation, at weak-beam yielding.

First, let us examine the behavior of three specimens, J1, J3, and J6. The ratios of the gross beam longitudinal bars to the gross beam section were 2.9%, 2.9%, and 1.7%, and the ratios of the connection shear reinforcement were 0.28%, 1.14%, and 0.43%, respectively. In other words, shear stress developed by the yielding of beams in the connection of Specimens J1 and J3 was significantly larger than that in Specimen J6. Specimen J3 was heavily reinforced against shear in the connection compared to Specimen J1.

The crack patterns of the three specimens are compared in Fig. 21 at a story drift angle of 1/15. Connection cover concrete of Specimens J1 and J3 came off at this stage. In the connection of Specimen J6, there were only a few shear cracks and the damage of connection was not so remarkable as in Specimens J1 and J3. No yielding occurred in the horizontal shear reinforcement of Specimen J6, though for Specimens J1 and J3 it occurred at a story drift angle of 1/100. The protection of the connection against shear at the initial yielding of the framing beams is not sufficient to prevent shear failure of the connection.

Horizontal average shear stresses in connections normalized by the concrete compressive strength are compared in Fig. 22 for specimens J1, J2, J3, J4, and J5. The stress reached maximum shear stress (o) after beam yielding (+) by strain hardening effect. The maximum shear stress was approximately as high as a stress level given by Kamimura's equation. After several cycles at large story drift angles of 1/50 to 1/20 were experienced, remarkable damage was observed in beam-column connections. Connection shear deformation angles reached as large as 1/60 to 1/30 and the shear stress decreased by 20-40% from the maximum value at a story drift angle of 1/15 (•). The effect of horizontal shear reinforcement is seen on the maximum shear stress and shear stress at a story drift angle 1/15.

Comparing the connection shear stress-deflection angle relationships (Fig. 23), large amount of horizontal shear reinforcement (1.14%) is obviously effective on the control of connection shear deformation. Comparing the contribution of column, beam and connection deflections to the overall story drift of the specimen (Fig. 24), the effect of horizontal shear reinforcement on connection shear deformation control is remarkable at story drift angle of 1/200-1/15 especially in Specimen J3. On the other hand, the effect of column load on connection shear deformation is observed only at a smaller story drift angle of 1/200-1/50. In all specimens, connection deformation reach as much as 20-40% of the total story displacement at the of the tests.

Bond deterioration of continuous beam bars can be studied from the results of Specimens S1, S2, S3, and S4, in which bond length of the beam reinforcement was varied from 19 db to 30 db by using different diameter bars for the same column depth. Column load was maintained at 20 or 60 kg/cm**2. All specimens were designed to barely reach connection cracking shear stress calculated by the Mohr circle analysis at beam yielding.

Comparing bond stress distributions (Fig. 25) for Specimen S4 with bond length of 19 db and Specimen S3 with bond length of 30 db, in the connection of specimen S3 the point of maximum bond stress was always located at the pull out region of beam bars at a story drift angle of 1/400 to 1/200. The maximum bond stress was about 50 kg/cm**2 in the bottom bars. On the other hand, in case of Specimen S4, there was a tendency that the point of maximum bond stress moved towards the connection center at the same stage. This means that bond deterioration of beam bars in Specimen S4 started at rather a small story drift angle.

Effect of column load was not observed on the shape of the beam shear-deflection hysteresis loops of Specimens S1 and S2 (Fig. 26). The capacity of energy dissipation was deteriorated remarkably especially in specimen S4 with a bond length of 19 db after a load reversal at a story drift angle of 1/50. Little deterioration in strength was seen in either specimen.

From the above observations, it can be concluded that (a) higher connection shear capacity could be attained with connection shear reinforcement larger than 1%, (b) once connection shear stress reached a stress level calculated by the Kamimura's equation, connection shear resistance started to decay by 20%-40% after several cycles at story drift angles of 1/50 to 1/15, and the effect of connection shear reinforcement on shear capacity deterioration was small, (c) if column depth was smaller than 20 times beam longitudinal bar diameter, bond of beam bars passing through the connection was deteriorated at a stroy drift angle of about 1/200, to cause the deterioration in energy dissipation capacity, and (d) better energy dissipation capacity was attained by using beam longitudinal bar diameter smaller than a 25-th of the column depth.

ATC-3 AND JAPANESE EARTHQUAKE RESISTANT BUILDING DESIGN

The Building Standard Law Enforcement Order in Japan was revised and put in force from June 1981. Earthquake resistant building design procedure was significantly changed taking into account the recent development in earthquake engineering. The revision introduced the concept of the ductility design for the first time in Japan. The detailed description of the revision can be found in Reference 2.

It is difficult to compare the earthquake resistant design provisions of two different countries because of differences in (a) seismicity, and (b) social and economic background. Even limiting the comparison to the Japanese earthquake resistant design provisions and proposed ATC-03 provisions, the ATC-03 provisions are based on the ultimate strength design using load factors and strength reduction factors, while the Japanese provisions use allowable stress design for low level earthquake loading and the ultimate member strength in evaluating the collapse state resistance of the structure for high level earthquake loading. This fundamental difference will invalidate the examination and comparison of individual differences in definition and amplitude of some coefficients. Therefore, it will be more meaningful to compare the final product from the two design provisions.

Four-story and eight-story office buildings were designed using the new Japanese seismic design procedure and the proposed ATC-03 procedure for comparison. In order to simplify the design, only a single column with adjacent girders on both sides was removed from an imaginary structure with an infinite number of uniform 6 m spans. The story height was uniformly 3.6 m high throughout the building. The base of the first story column was fixed to the rigid base. The structure was classified as a reinforced concrete special moment frame to be constructed on ground of soil profile type S2 in Los Angeles or on type II soil in Tokyo. The compressive strength of concrete was 210 kg/cm**2 (3000 psi). and the grade of reinforcing steel was SD35 (Grade 50).

The dead and live loads specified in the two provisions are listed below. The dead load was estimated to be, on the average, 420 kg/m**2 per floor, and made common in all structure. The ATC-03 considers the weight of fixed partitions as part of the weight of the structure, but the fixed partitions were ignored in this study.

(a) Japanese Procedure (unit in kg/m**2)

Floor Level	Roof			General		
Type of Loads	Dead	Live	Total	Dead	Live	Total
Frame Analysis	420	130	550	380	180	560
Seismic Analysis	420	60	480	380	80	460

(b) ATC-3 Procedure (unit in kg/m**2)

Floor Level	Roof			General		
Type of Loads	Dead	Live	Total	Dead	Live	Total
Frame Analysis	420	78	498	380	244	624
Seismic Analysis	420	0	420	380	0	380

Note that the live load specified in an office building is larger in the United States, hence the total floor loads for the gravity load frame analysis is larger in the United States approximately by 10 percent. On the contrary, the total weight of a structure for earthquake loading is larger in Japan due to the neglection of possible live loads.

Parameters to determine the magnitudes of lateral loads are listed below. The period of the structure was evaluated by simple expressions in the two countries;

T = 0.025 h**0.75 for ATC-03

T = 0.02 h for Japanese Code

The ATC-03 expression gives approximately 1.3 to 1.6 times longer natural periods for the corresponding structures. The natural period of the Japanese buildings was less than the predominant period of the ground (=0.6 sec) to allow no reduction in base shear, while the period of the ATC-03 buildings was long and in a rapidly descending branch of the seismic design coefficient curve. Therefore, the Japanese buildings must be design using lateral loads approximately 1.6 to 2.2 times greater than the ATC-03 buildings.

Design Procedure	Japanese		ATC-03	
Buildings	Four Stories	Eight Stories	Four Stories	Eight Stories
Period, sec	0.29	0.58	0.45	0.76
Weight, ton	105	265	94	243
Base Shear Coeff.	0.20	0.20	0.14	0.10
Base Shear, ton	21.0	53.0	13.2	24.0

The design moments at the critical sections of the structures are given in Fig. 27 after combining loads in an appropriate manner. Both in four- and eight-story buildings, design actions were significantly greater in the Japanese buildings, especially in the columns where the magnitude of earthquake loads will affect design actions most.

The members of the ATC-03 buildings were proportioned using the ultimate strength design procedure with appropriate strength reduction factors, while those of the Japanese buildings were proportioned by the allowable stress design procedure. It was decided that tensile reinforcement ratios of the beams should fall within 0.004 to 0.025, and that the amount of bottom reinforcement be at least more than one-half the amount of top reinforcement in the beam critical sections. In the column, gross reinforcement ratios were chosen to be between 0.01 to 0.06, and the sum of the ultimate moments of beams framing into a joint should be less than the sum of the ultimate moment of the columns above and below the joint. In addition, the column reinforcement was made continuous through each beam-column joint. The final beam and column dimensions with reinforcement are listed in Tables 2 and 3. Reinforcing bars D22 and D25 are identical to No.7 and No. 8 bars in the United States. Japanese Building Code requires larger cross sections, but longitudinal reinforcement ratios smaller than the ATC-03 Regulations.

The design provisions of Japanese Building Code and the proposed ATC-03 Seismic Regulations for Buildings were examined through designing highly idealized simple frames. It was made clear that the Japanese Building Code tends to require higher stiffness and strength than the ATC-03 Regulations.

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Table 1 Variables in Beam-Column Connection Tests

(a) S-Series Specimens

Specimen	S1	S2	S3	S4	S5	S6
Beam						
Top Bars	4-D13	4-D13	7-D10	3-D16	4-D13	4-D13
pt (%)	0.94	0.94	0.92	1.11	0.94	0.94
Bottom Bars	3-D13	3-D13	5-D10	2-D16	3-D13	3-D13
pt (%)	0.71	0.71	0.66	0.74	0.71	0.71
Stirrups	2-D6	2-D6	2-D6	2-D6	2-D6	2-D6
@ (cm)	5.0	5.0	5.0	5.0	5.0	5.0
Column						
Total Bars	12-D10	12-D10	12-D10	12-D10	12-D10	12-D10
pg (%)	0.95	0.95	0.95	0.95	0.95	0.95
Hoops	2-D6	2-D6	2-D6	2-D6	2-D6	2-D6
@ (cm)	5.0	5.0	5.0	5.0	5.0	5.0
Load (ton)	54.0	18.0	54.0	54.0	54.0	54.0
(kg/cm2)	60.0	20.0	60.0	60.0	60.0	60.0
Connection						
Hoops	4-D6	4-D6	4-D6	4-D6	4-D6	2-D6
pw (%)	1.28	1.28	1.28	1.28	1.28	0.43

(b) J-Series Specimens

Specimen	J1	J2	J3	J4	J5	J6
Beam						
Top Bars	8-D13	8-D13	8-D13	8-D13	8-D13	4-D13
pt (%)	1.88	1.88	1.88	1.88	1.88	0.94
Bottom Bars	4-D13	4-D13	4-D13	4-213	4-D13	3-D13
pt (%)	0.94	0.94	0.94	0.94	0.94	0.71
Stirrups	2-D6	2-D6	2-D6	2-D6	2-D6	2-D6
@ (cm)	5.0	5.0	5.0	5.0	5.0	10.0
Column						
Total Bars	16-D13	16-D13	16-D13	16-D13	10-D13	12-D10
pg (%)	2.26	2.26	2.26	2.26	1.41	0.95
Hoops	2-D6	2-D6	2-D6	2-D6	2-D6	2-D6
@ (cm)	8.0	7.5	8.0	8.0	8.0	5.0
Load (ton)	18.0	18.0	18.0	72.0	18.0	54.0
(kg/cm2)	20.0	20.0	20.0	80.0	20.0	60.0
Connection						
Hoops	2-D6	4-D6	4-D6	2-D6	2-D6	2-D6
pw (%)	0.28	0.57	1.14	0.28	0.28	0.43

Table 2' Beams and Columns of Four-Story Frame

(a) Beam Sections

Procedure	Japanese Code			ATC-03		
Floor Level	Section	Top	Bottom	Section	Top	Bottom
Roof	35 x 60	3-D22*	3-D22*	30 x 55	3-D22	2-D22*
Fourth	40 x 65	4-D22	3-D22*	35 x 60	4-D22	2-D22*
Third	45 x 75	5-D22	4-D22*	40 x 65	5-D22	3-D22*
Second	45 x 75	5-D22	4-D22*	40 x 65	5-D22	3-D22*

* Minimum reinforcement requirement.

(b) Column Sections

Procedure	Japanese Code		ATC-03	
Story Level	Section	Total	Section	Total
		8-D22		8-D22
Fourth	50 x 50		45 x 45	
		8-D22		16-D22
Third	55 x 55		45 x 45	
		8-D22		16-D22
Second	60 x 60		45 x 45	
		8-D22		16-D22
First	65 x 65		50 x 50	
		16-D22		20-D22

Table 3' Beams and Columns of Eight-Story Frame

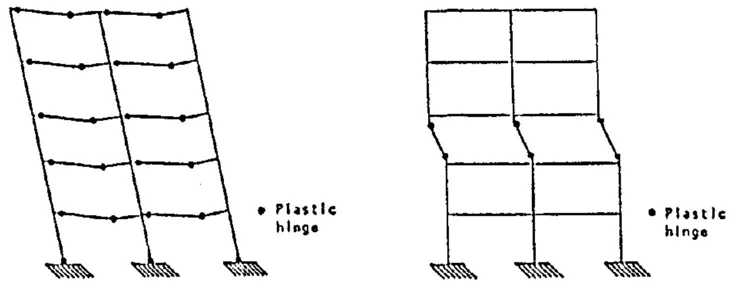
(a) Beam Sections

Procedure	Japanese Code			ATC-03		
Floor Level	Section	Top	Bottom	Section	Top	Bottom
Roof	35 x 60	3-D25	2-D25	30 x 55	3-D25	2-D25*
Eighth	45 x 70	5-D25	3-D25	35 x 60	4-D25	2-D25*
Seventh	45 x 80	5-D25	4-D25	35 x 70	4-D25	2-D25*
Sixth	55 x 85	6-D25	5-D25	40 x 70	5-D25	3-D25
Fifth	55 x 90	6-D25	5-D25	40 x 75	5-D25	3-D25
Fourth	60 x 90	7-D25	6-D25	45 x 75	6-D25	4-D25
Third	60 x 90	7-D25	6-D25	45 x 75	6-D25	4-D25
Second	60 x 90	7-D25	6-D25	45 x 75	6-D25	4-D25

* Minimum reinforcement requirement.

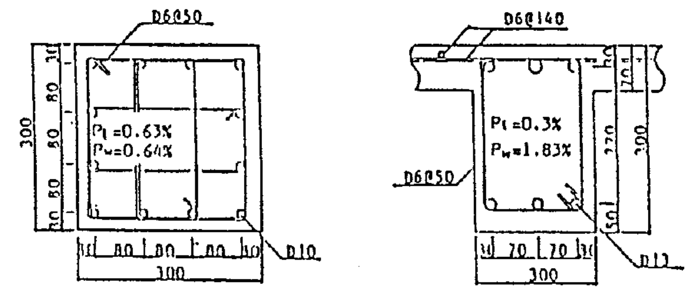
(b) Column Sections

Procedure	Japanese Code		ATC-03	
Story Level	Section	Total	Section	Total
		12-D25		8-D25
Eighth	50 x 50		50 x 50	
		16-D25		12-D25
Seventh	60 x 60		50 x 50	
		16-D25		16-D25
Sixth	70 x 70		50 x 50	
		16-D25		16-D25
Fifth	75 x 75		60 x 60	
		16-D25		16-D25
Fourth	80 x 80		60 x 60	
		16-D25		16-D25
Third	80 x 80		70 x 70	
		16-D25		16-D25
Second	80 x 80		70 x 70	
		20-D25		16-D25
First	80 x 80		80 x 80	
		28-D25		16-D25

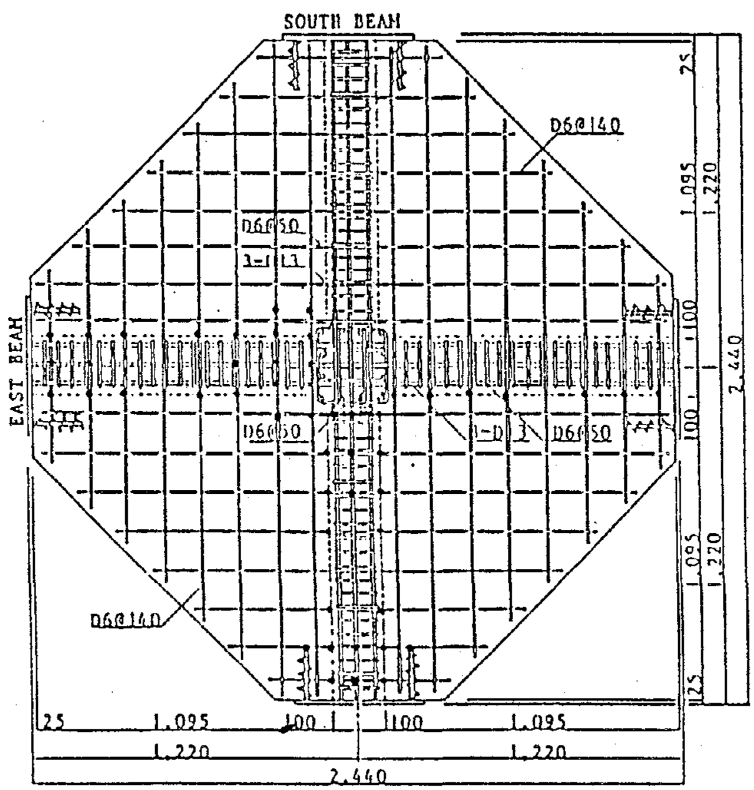


(a) Beam Sidesway Mechanism (b) Column Sidesway Mechanism

Fig. 1 : Energy Dissipation Mechanisms



(a) Column section (b) Beam section



(c) Plan

Fig . 2 : Beam-Column-Slab Subassemblage Specimen
(unit in mm)

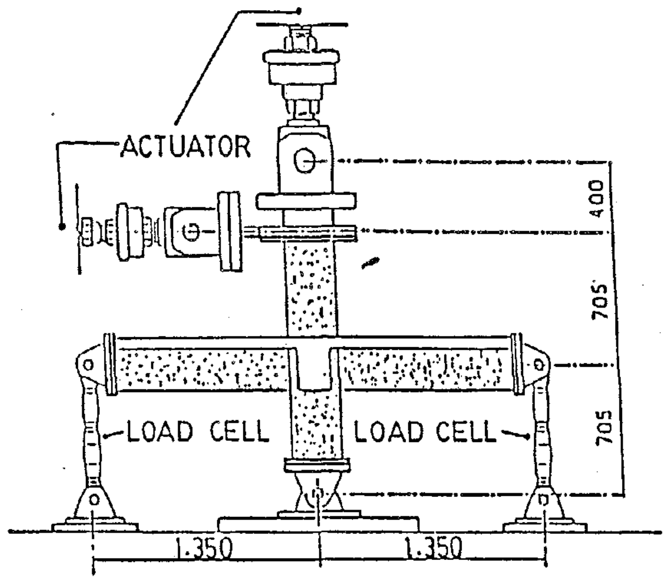


Fig. 3 : Specimen in Loading Apparatus (unit in mm)

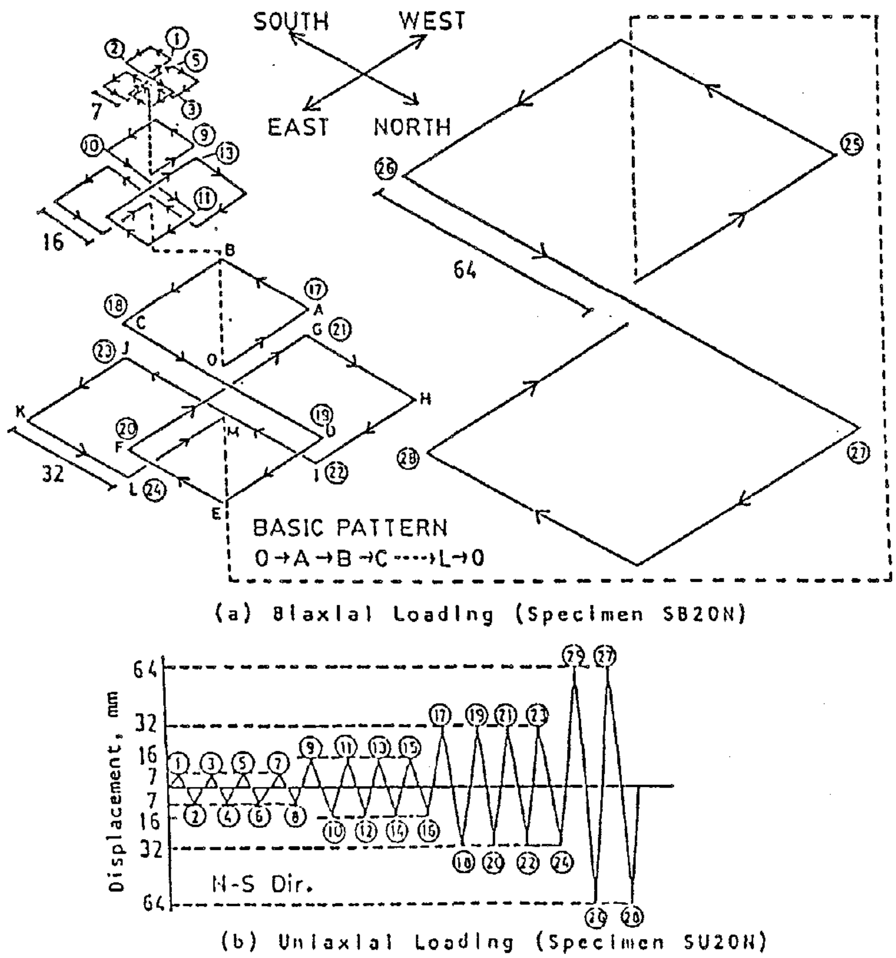
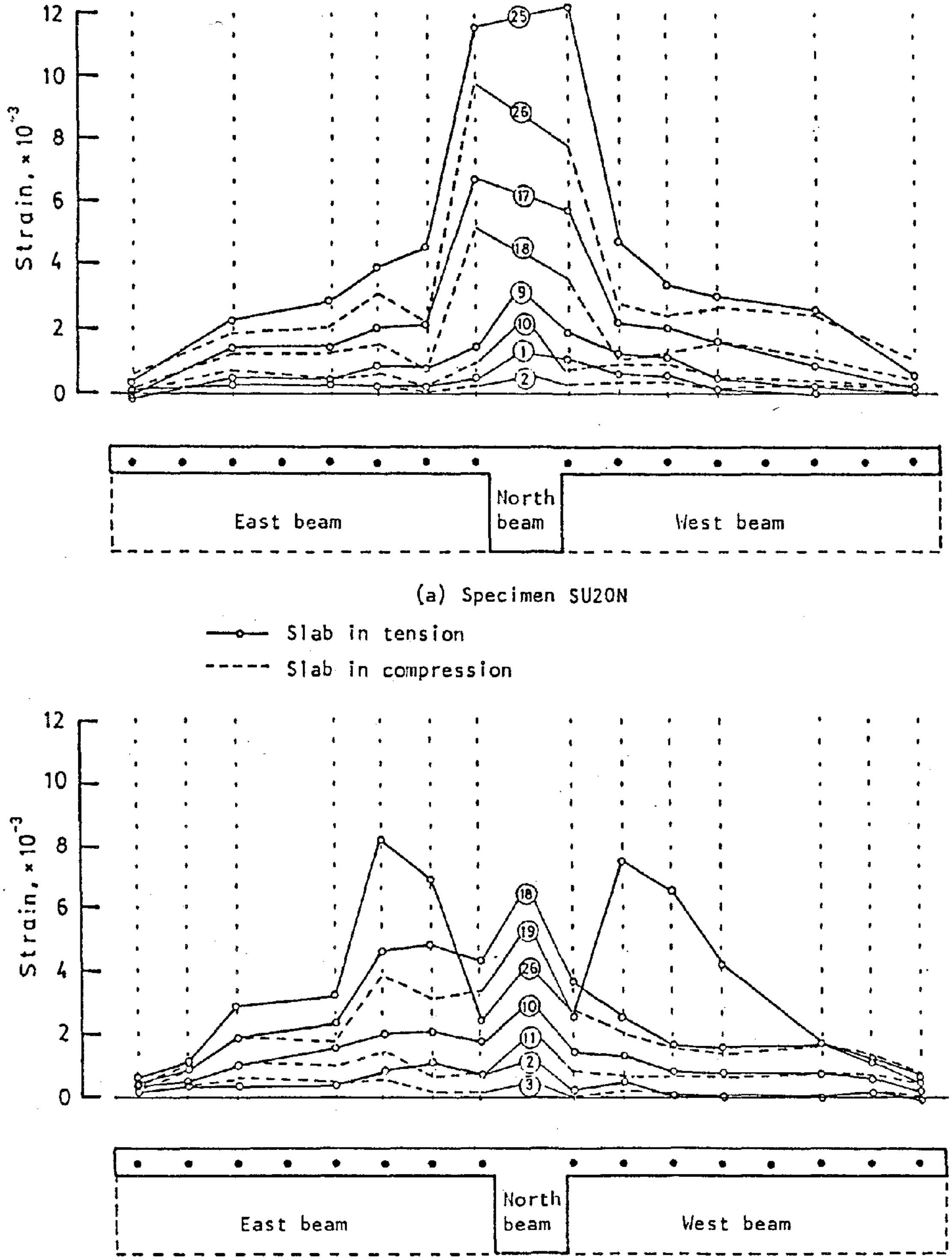
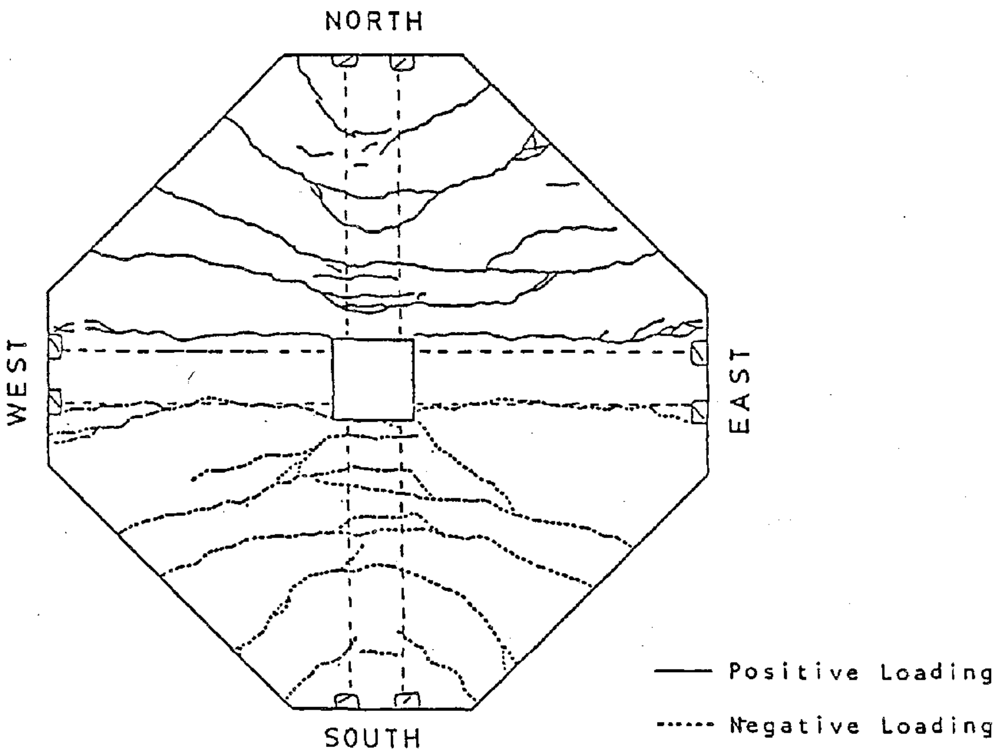


Fig. 4 : Displacement History during Tests (unit in mm)

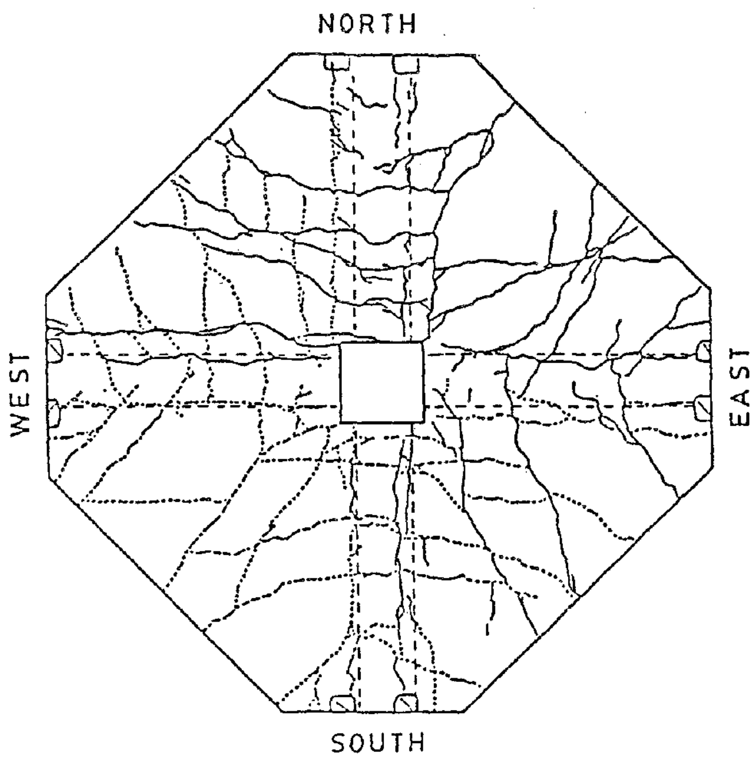


(b) Specimen SB20N

Fig. 5 : Strain distribution of Slab bars



(a) Specimen SU20N



(b) Specimen SB20N

Fig. 6 ;Crack Patterns in Floor Slabs

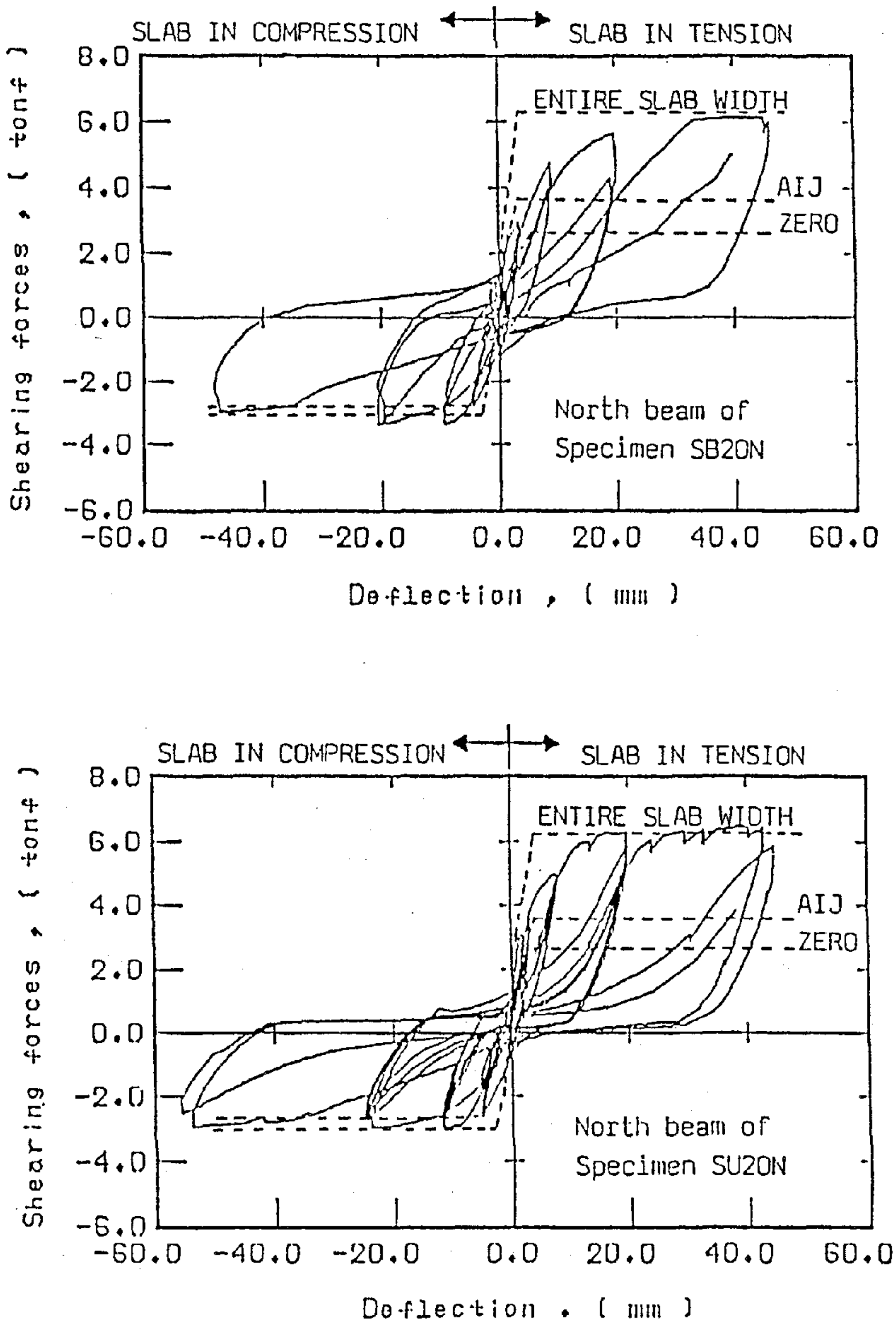
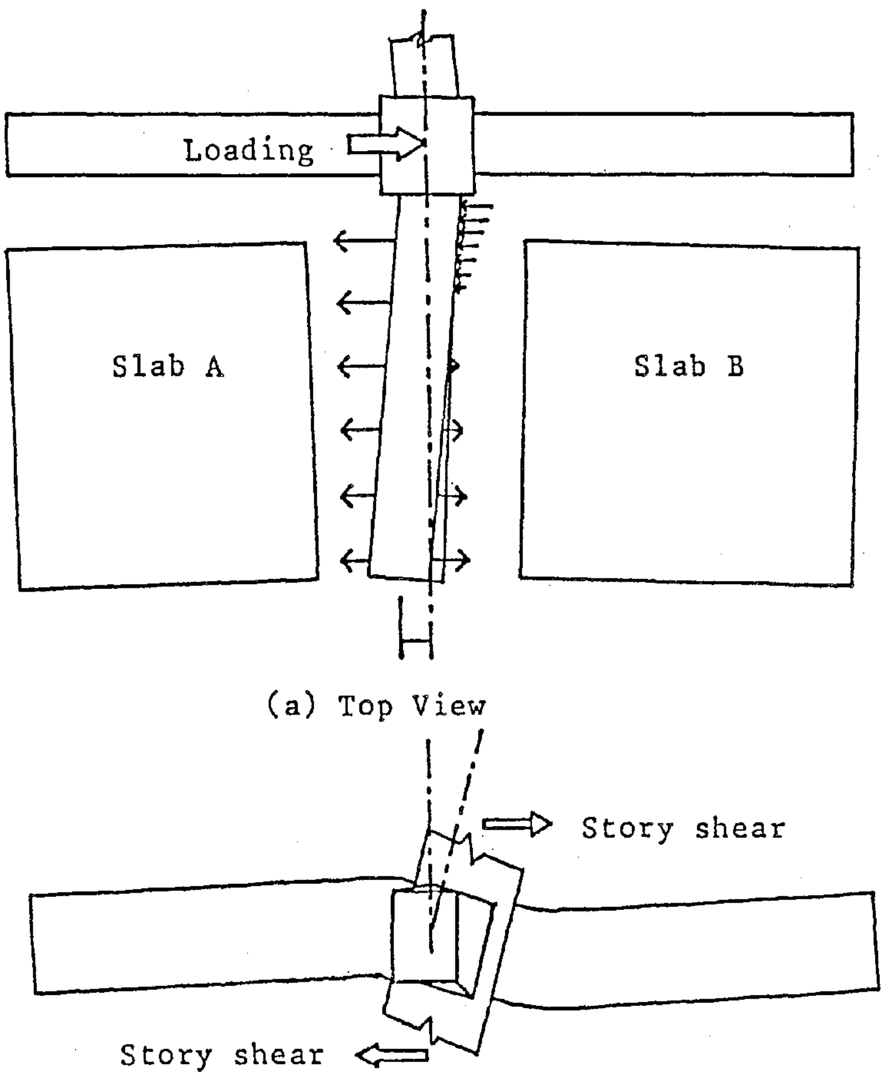


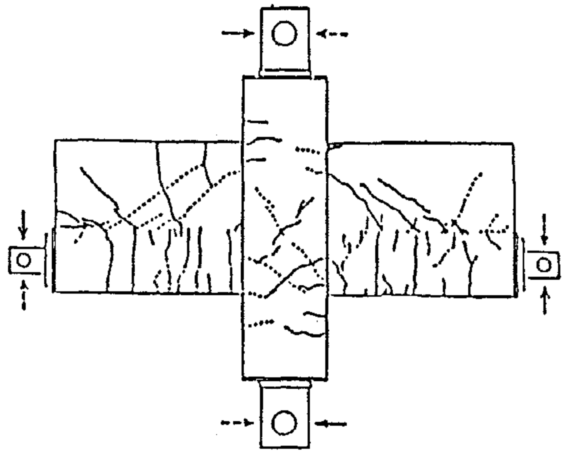
Fig. 7 : Shearing Forces - Deflection in North Beam



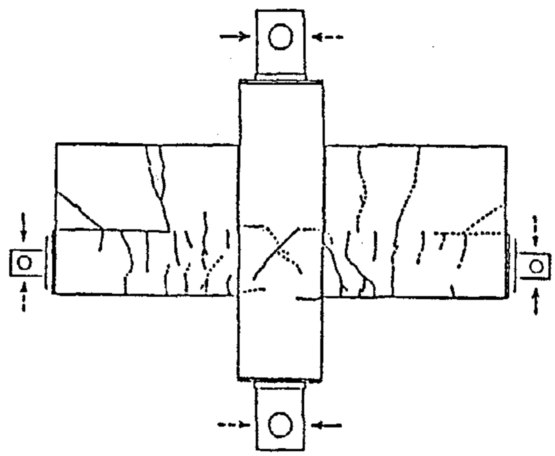
(a) Top View

(b) Side view

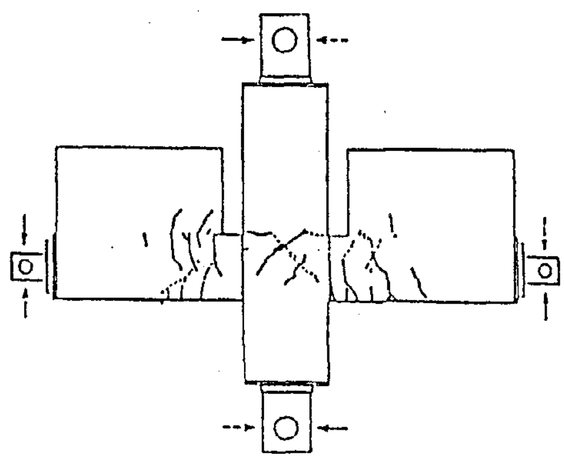
Fig. 8 : Deformation and Stresses of Transverse Beam



(a) Specimen SW00



(b) Specimen SW01



(c) Specimen SW10

Fig. 9 : Crack Patterns of Beam-Column Connections with Spandrels

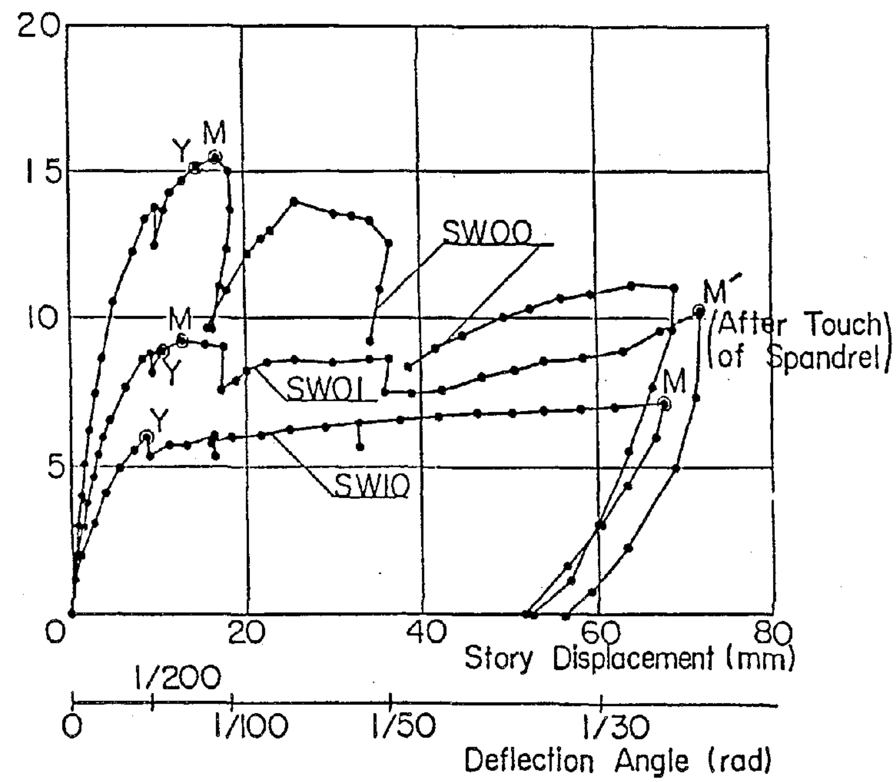


Fig. 10 : Envelope Curves of Story Shear - Displacement Relation

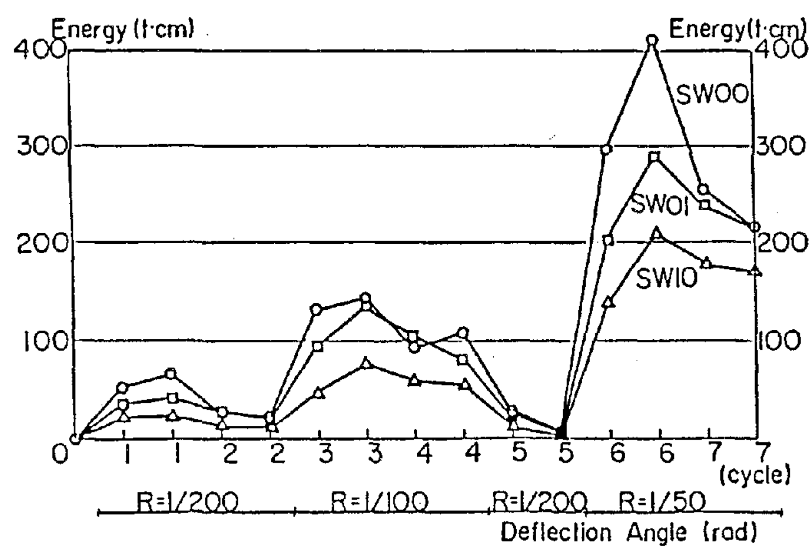
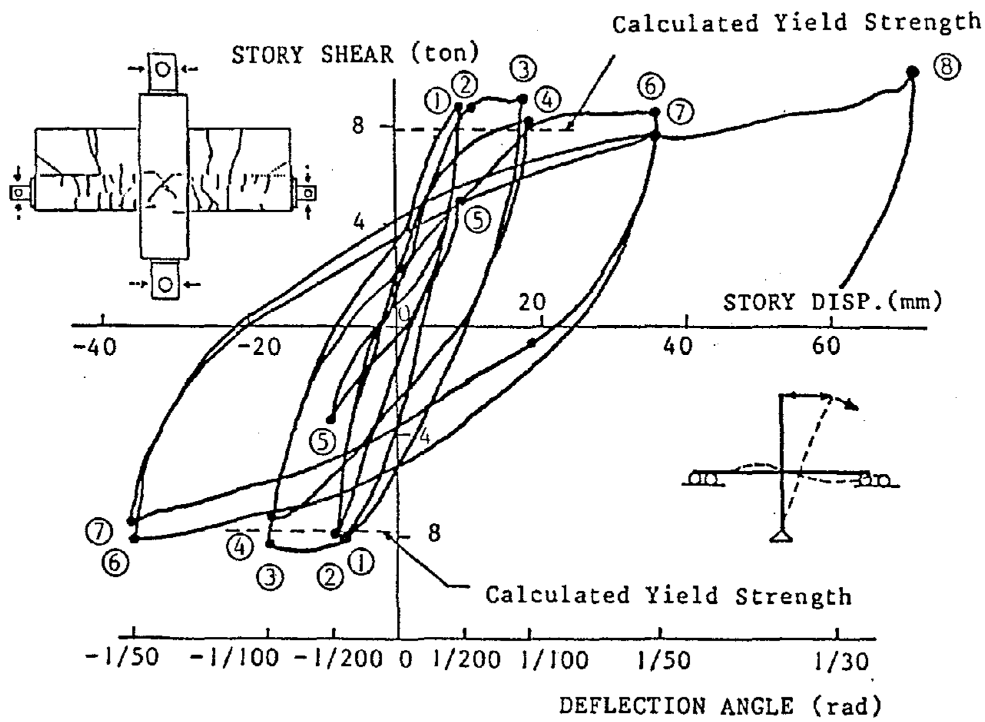
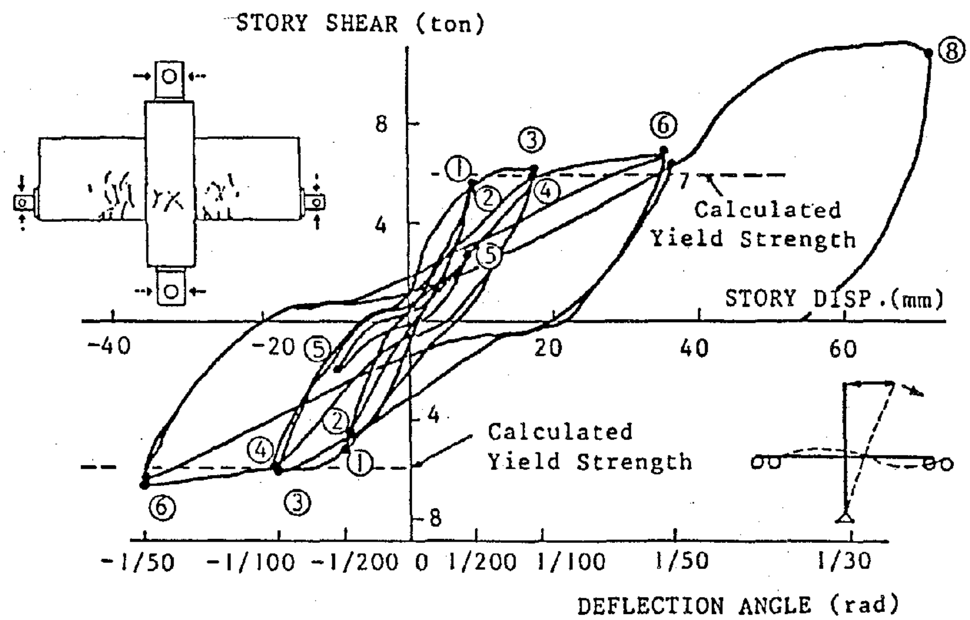


Fig. 11 : Energy Dissipation in Every Half Cycle

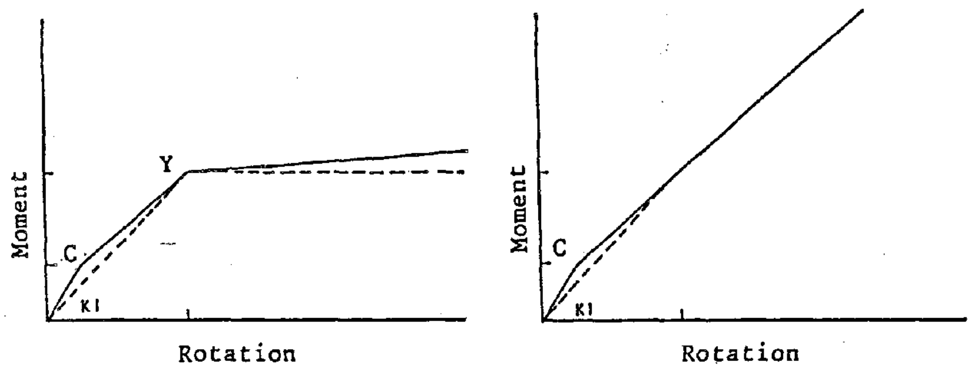


(a) Spacimen SW01



(b) Specimen SWB1

Fig. 12 Effect of Anchorage of Spandral Horizontal Reinforcement into Column.



(a) All Floor Beams and Top and Bottom Floor Columns

(b) Columns in Intermediate Stories

Fig. 13 ; Skeleton Moment-- Rotation Relations

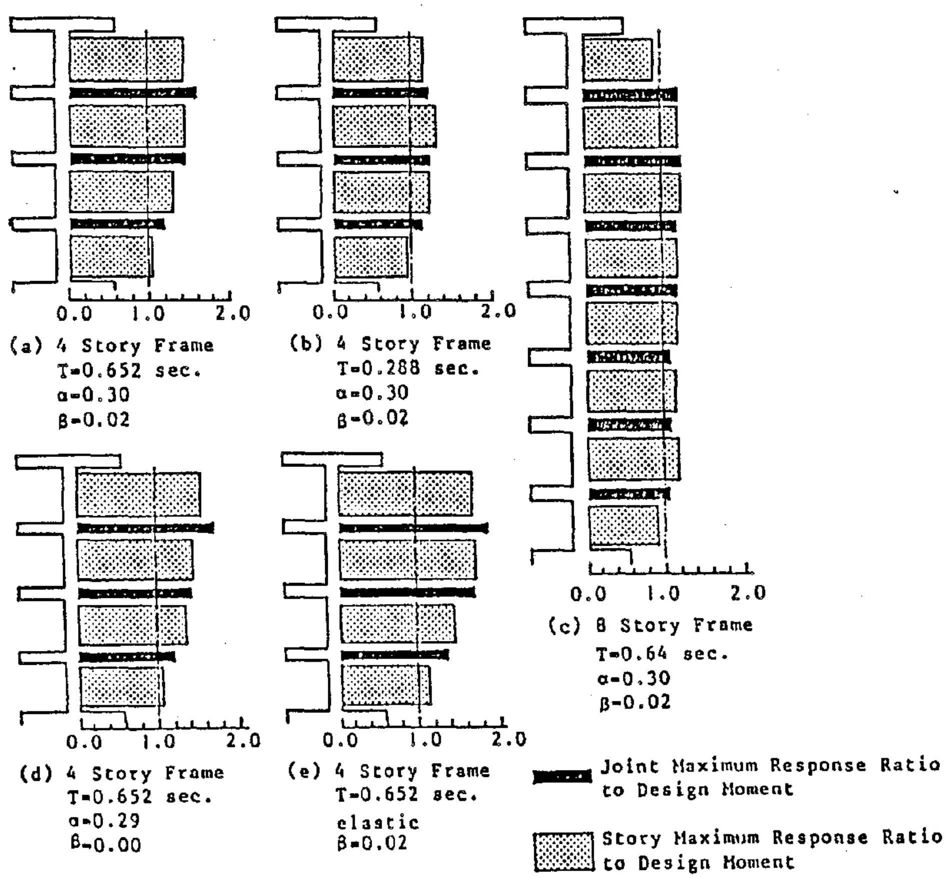
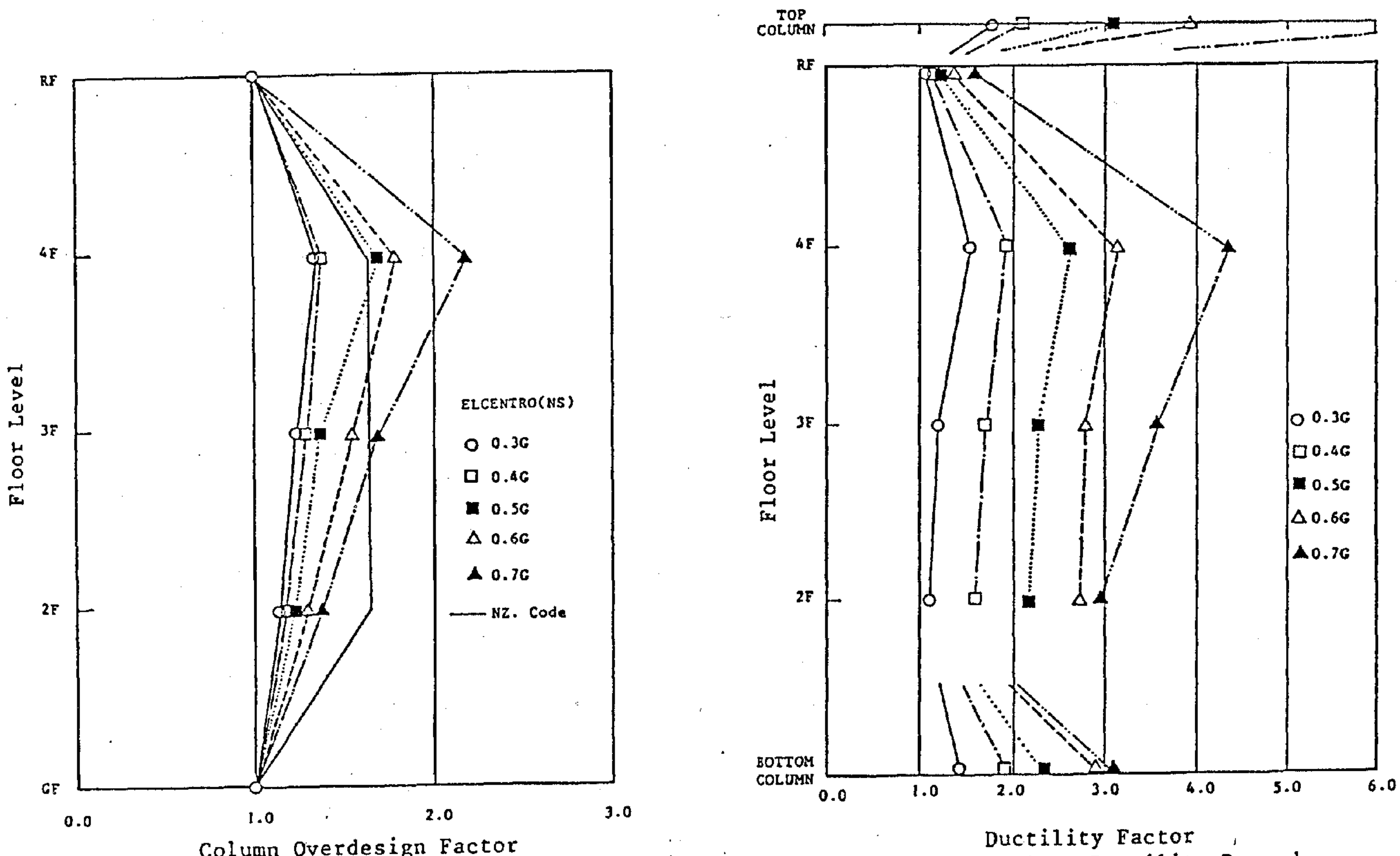


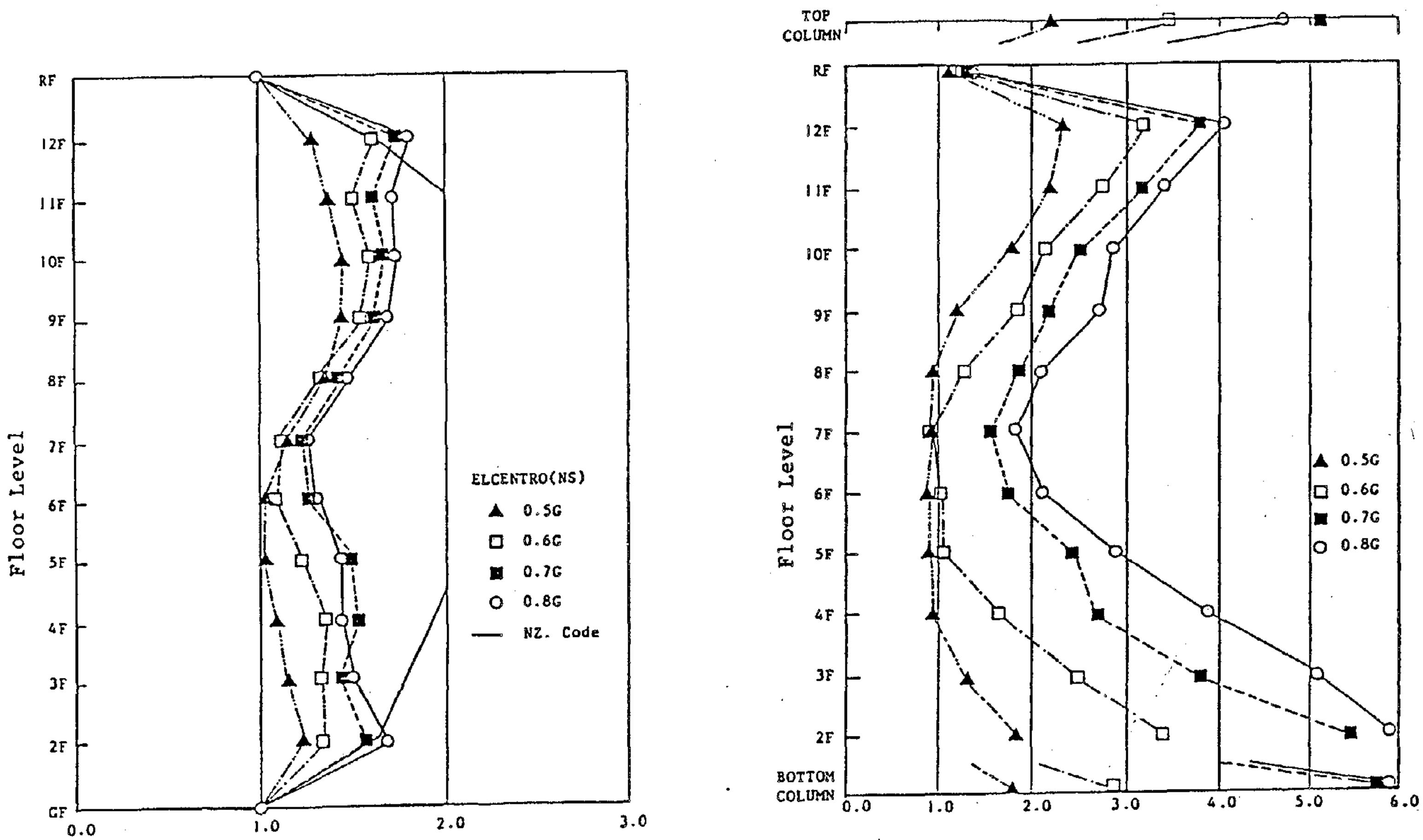
Fig. 14 : Joint and Story Maximum Response Ratio HACHINOHE(EW)



(a) Column Overdesign Factor

(b) Beam and Column Ductility Demand

Fig. 15 : Response of Four-Story Frame with Earthquake Intensity



Column Overdesign Factor
(a) Column Overdesign Factor

Ductility Factor
(b) Ductility Factor

Fig. 16 : Response of Twelve - Story Frame
with Earthquake Intensity

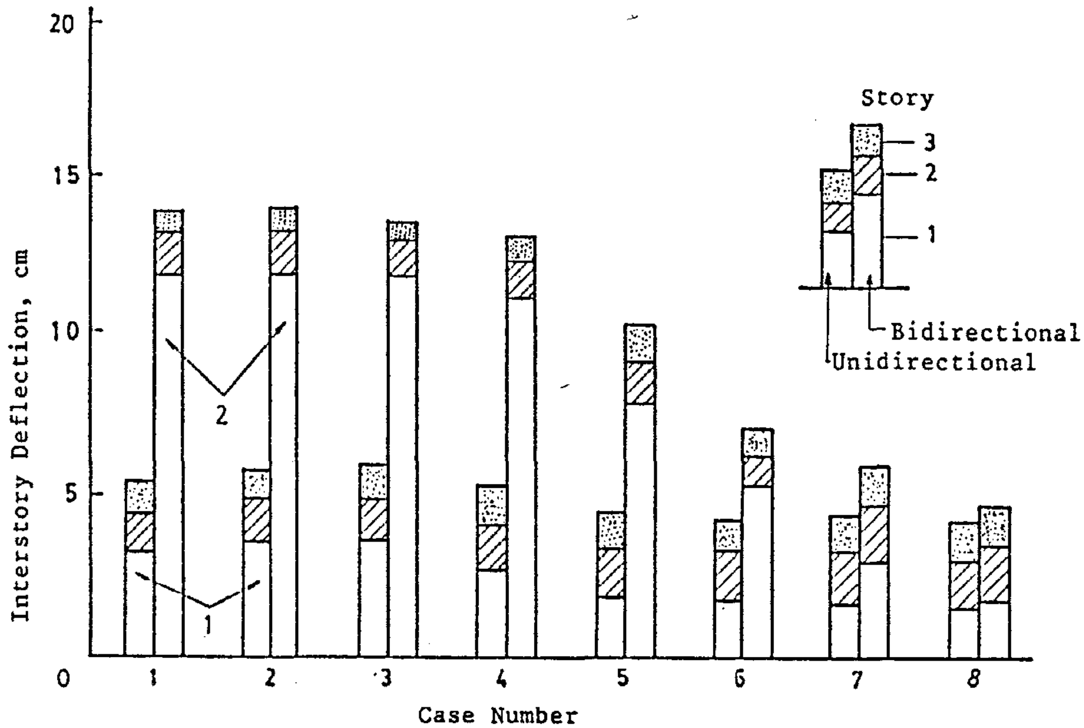


Fig. 17 : Distribution of Deflection along Story Height
(NS Direction)

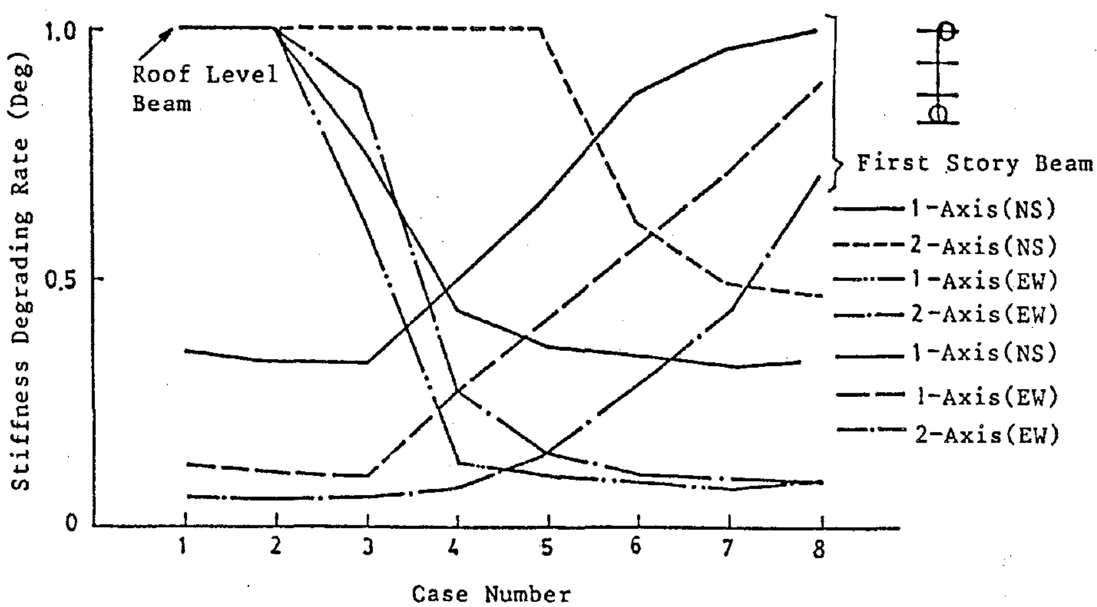
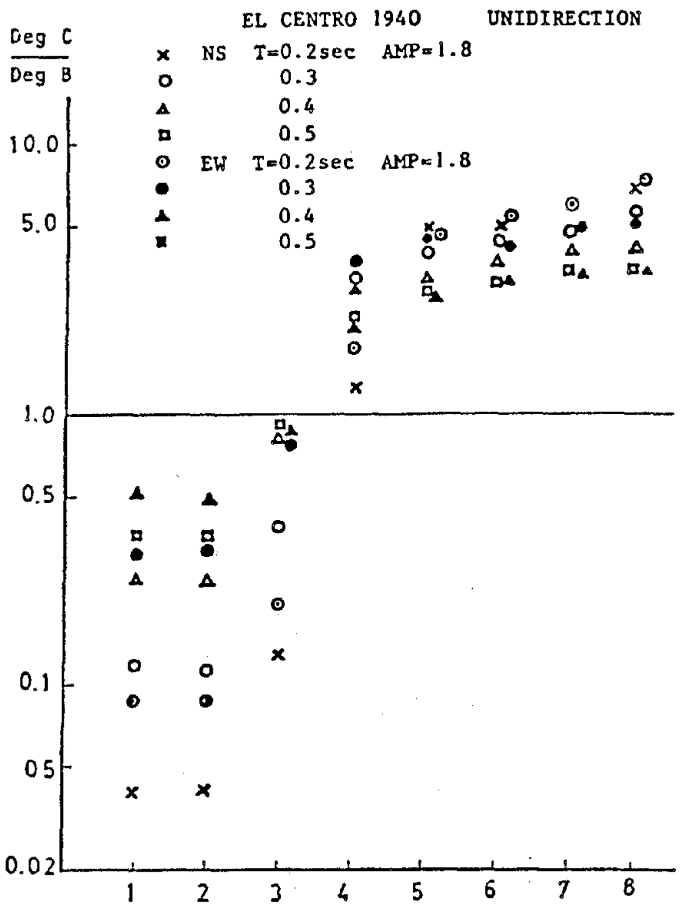
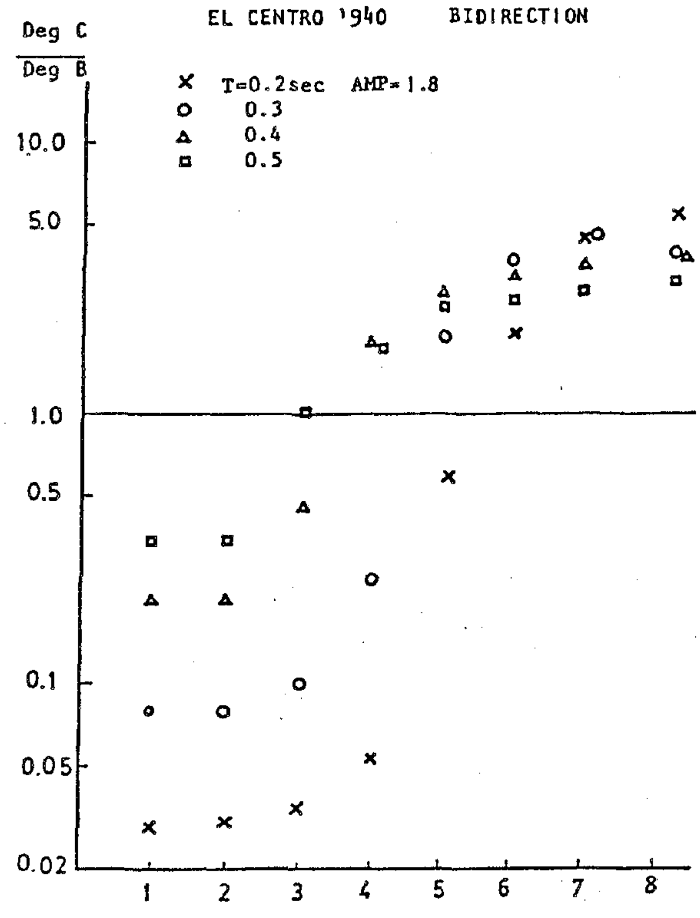


Fig. 18 : Stiffness Degrading Rate

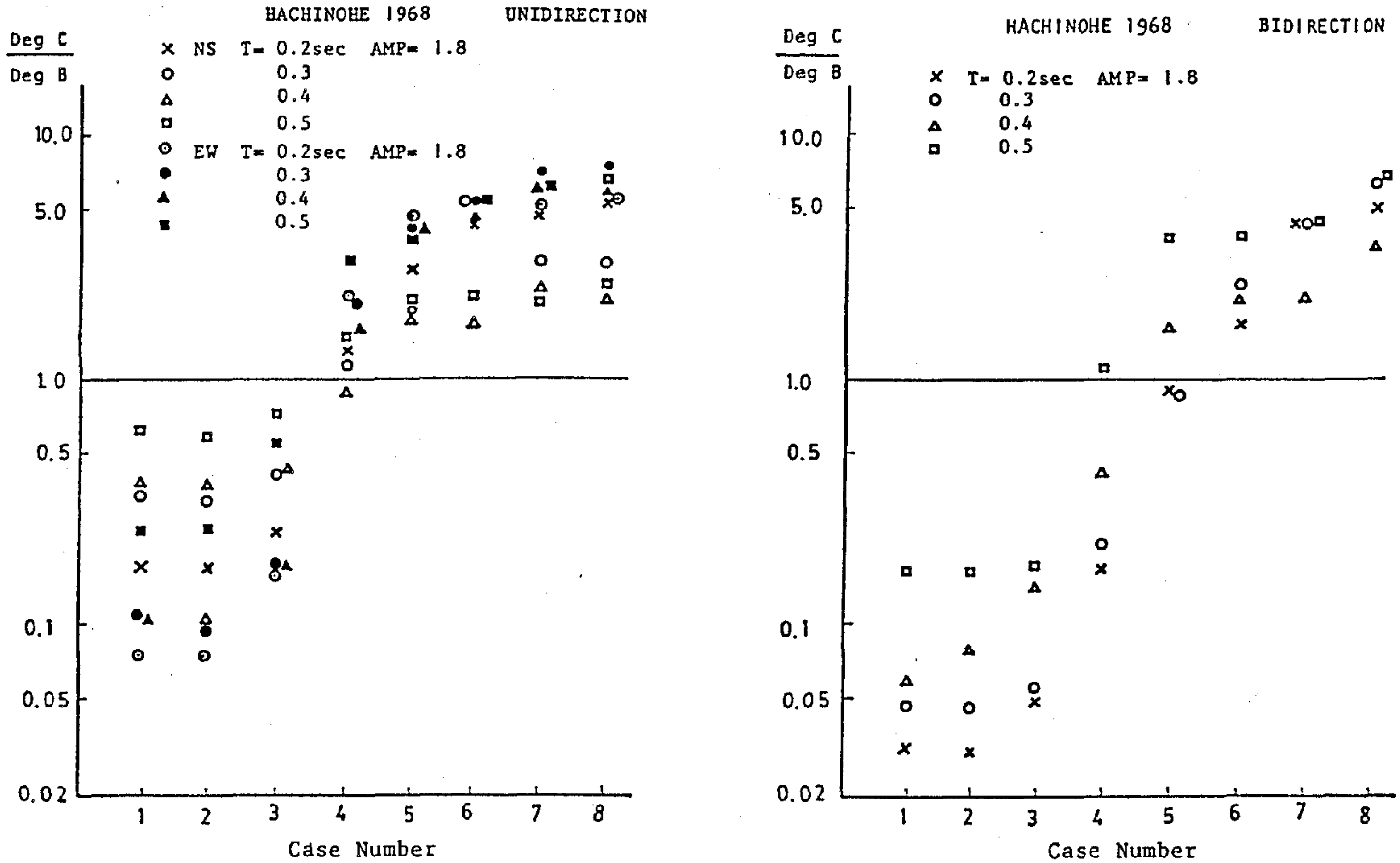


(a) Unidirectional Response



(b) Bidirectional Response

Fig. 19 ; Ratio of Column to Beam Stiffness Degrading Rates
(El Centro 1940)



(a) Unidirectional Response (b) Bidirectional Response

Fig. 20 : Ratio of Column to Beam Stiffness Degrading Rates

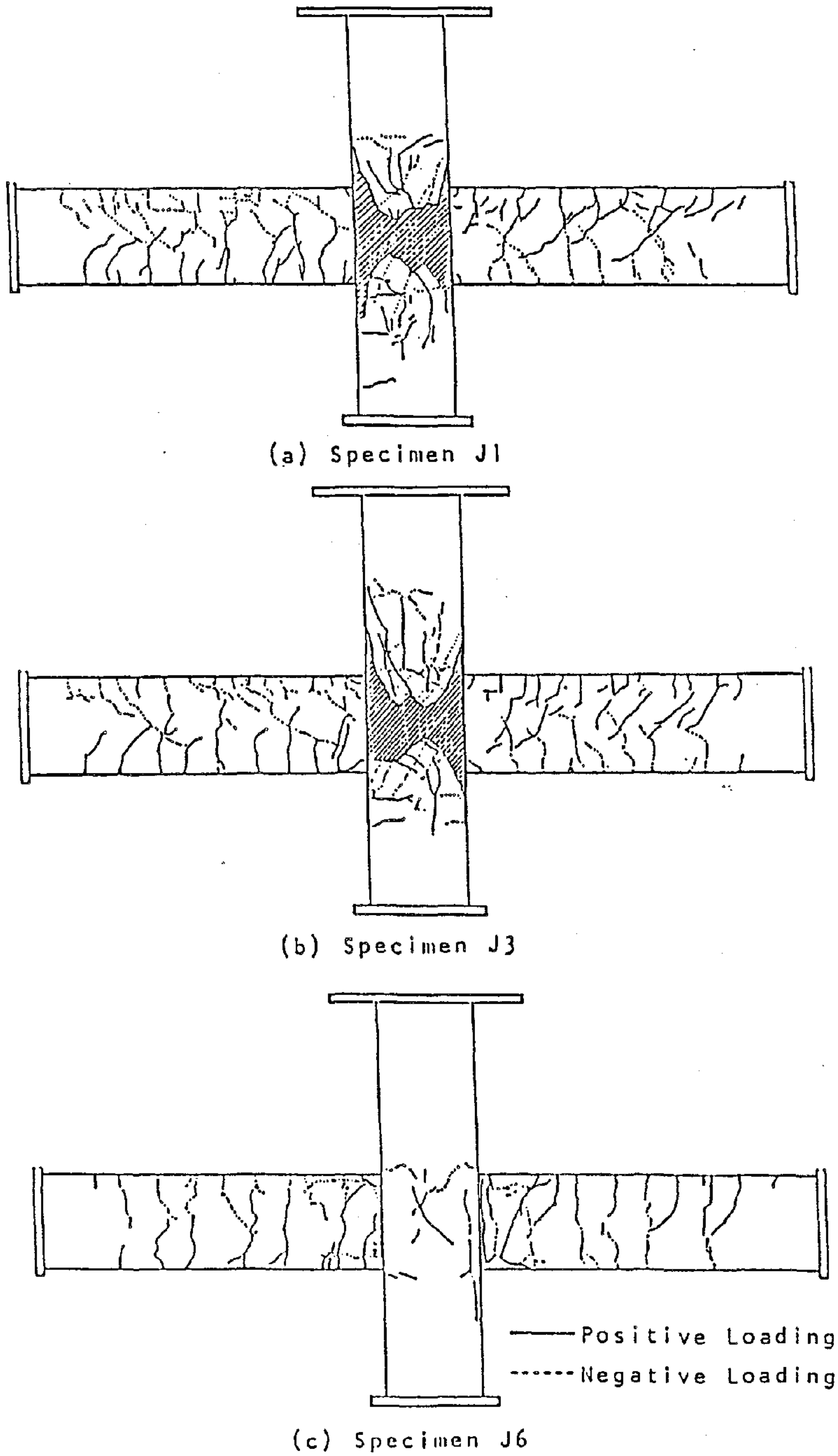
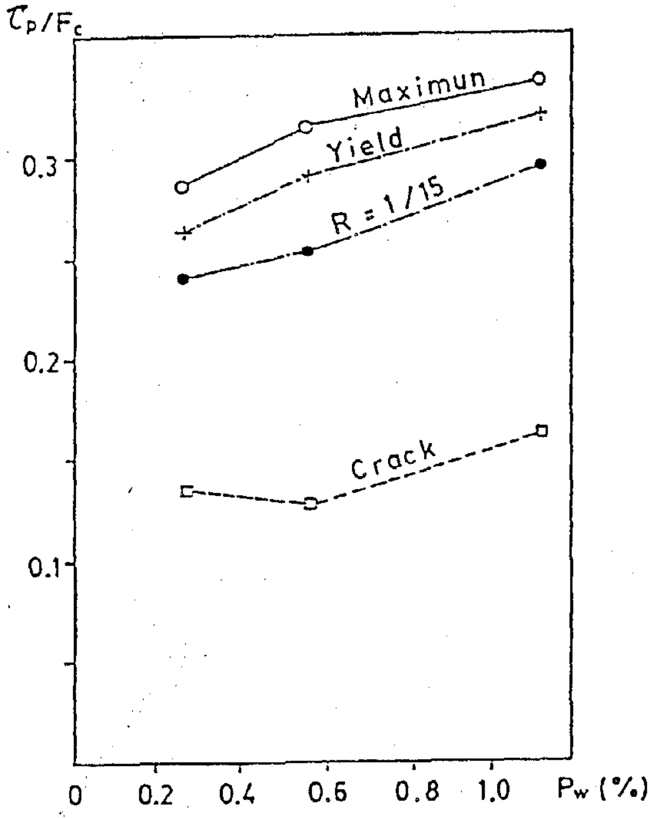
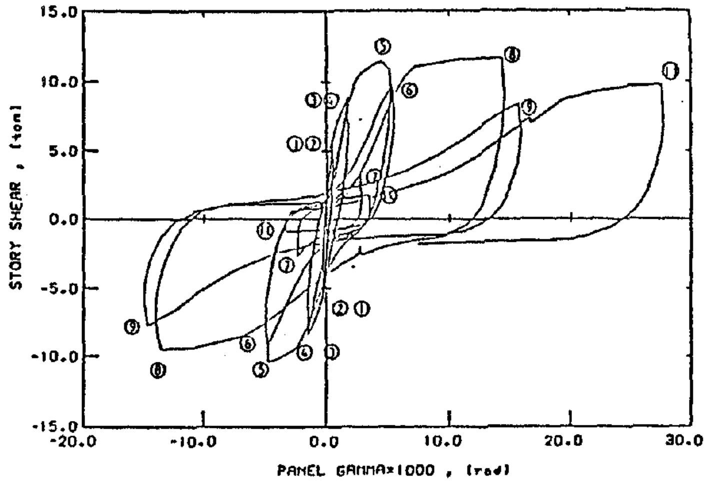


Fig. 21 : Crack Patterns of Specimens after Test

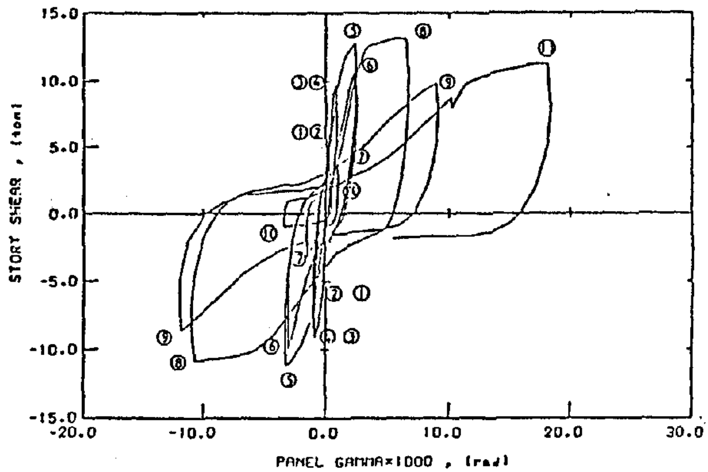


Shear Reinforcement Ratio

Fig. 22 : Effect of Shear Reinforcement



(a) Specimen J1



(b) Specimen J3

Fig. 23 : Story Shear - Shearing Angle
in Connection

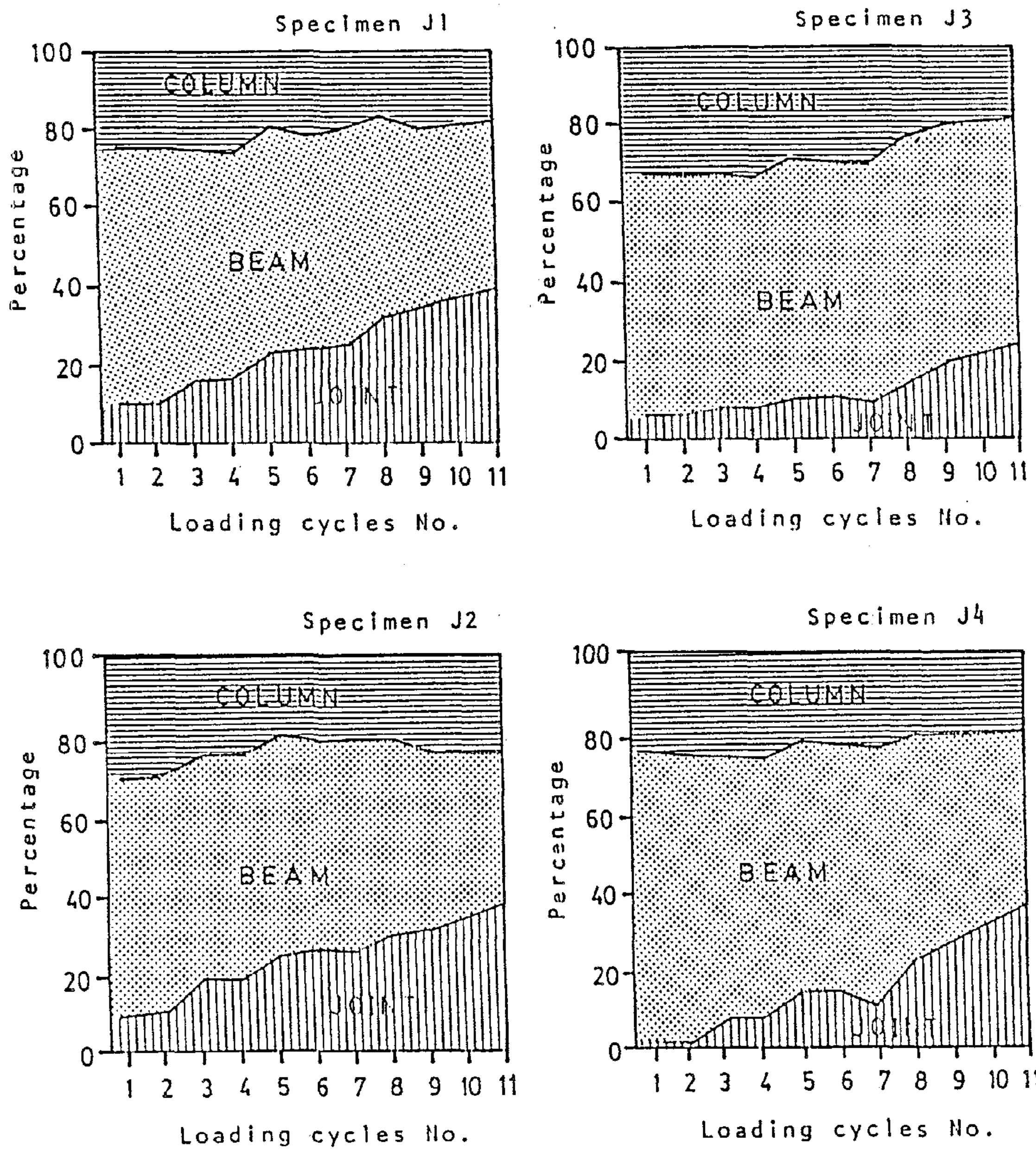


Fig. 24 : Components of Story displacement

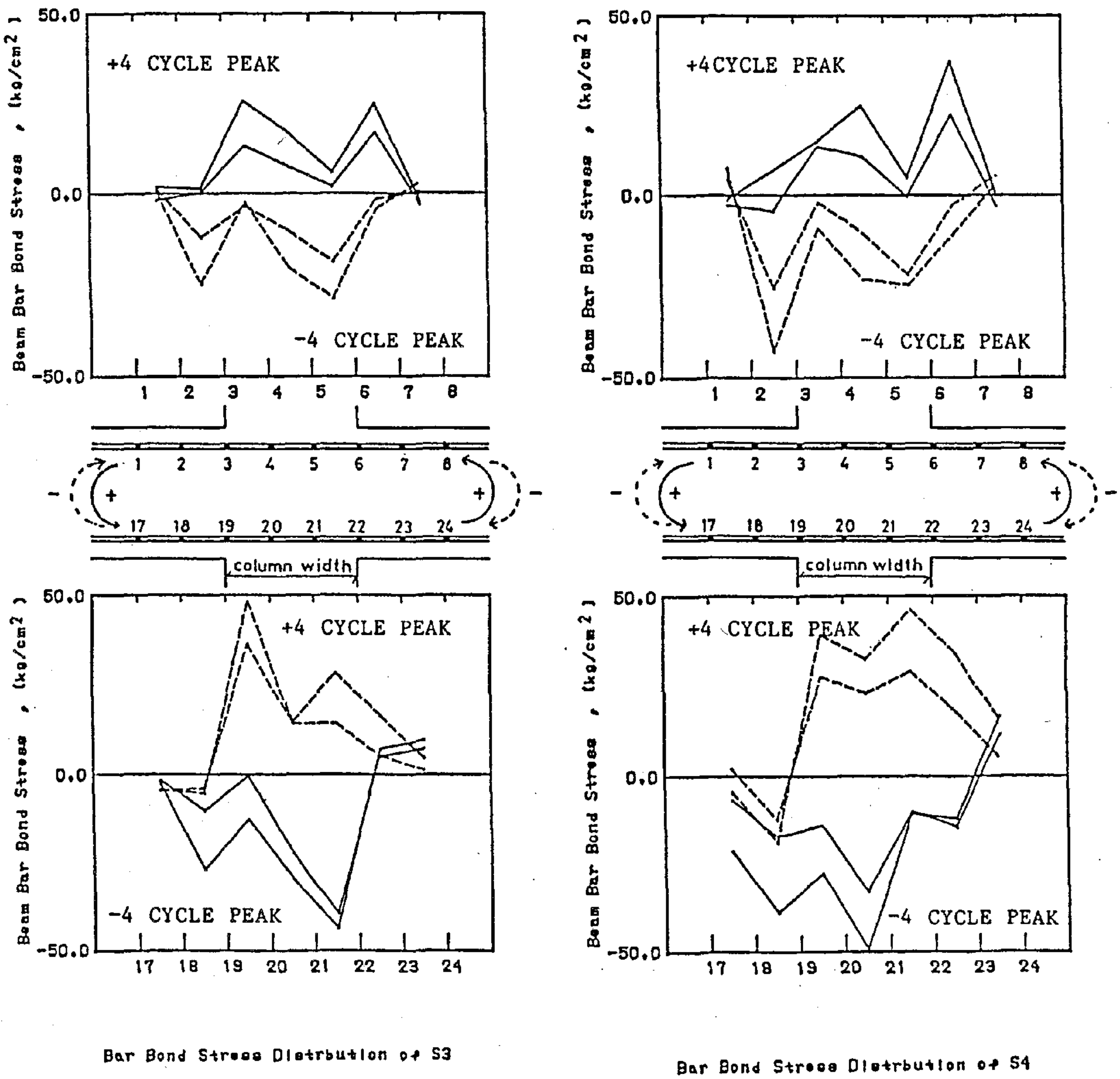


Fig. 25 : Bond Stress Distribution along Beam Reinforcement

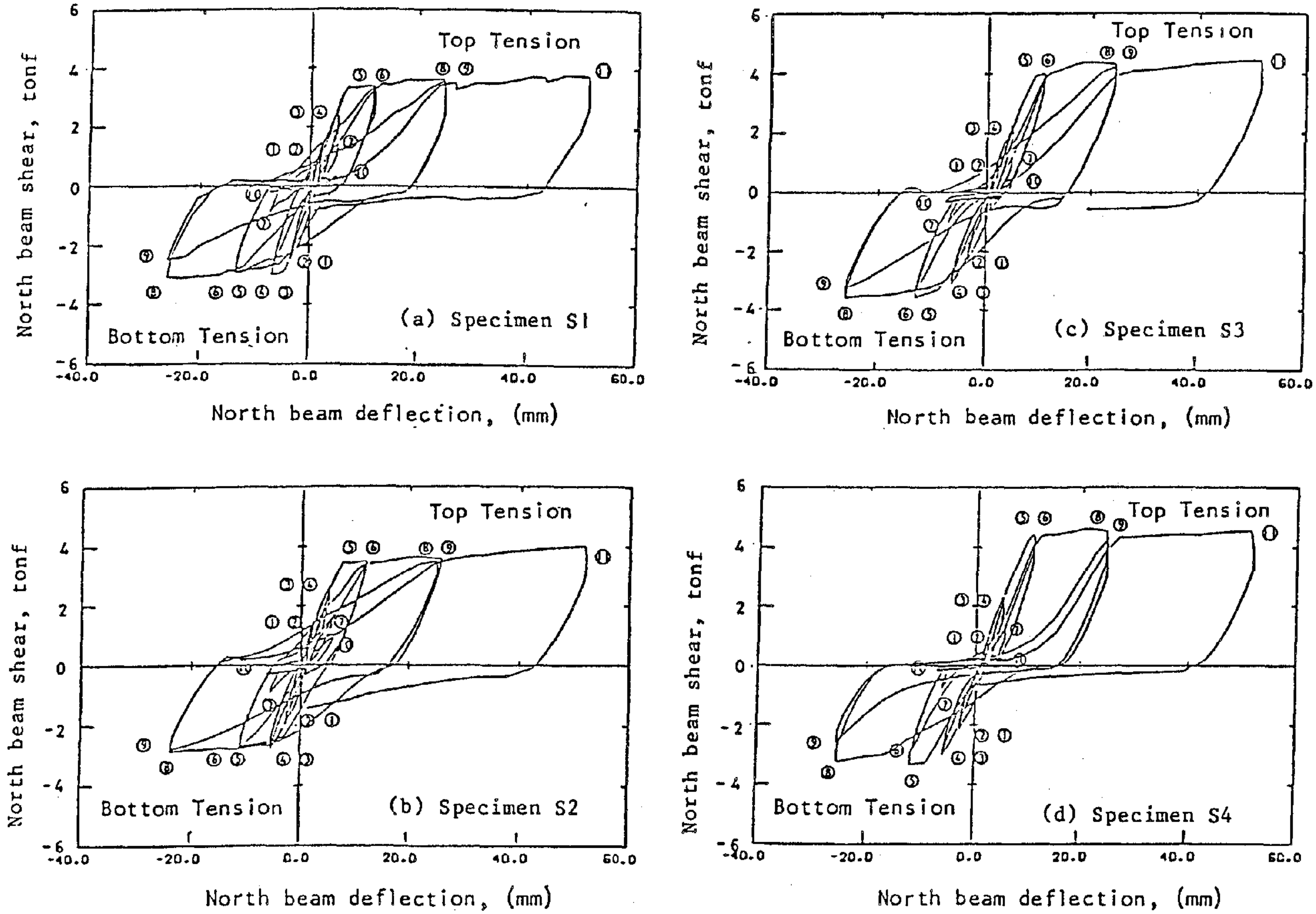
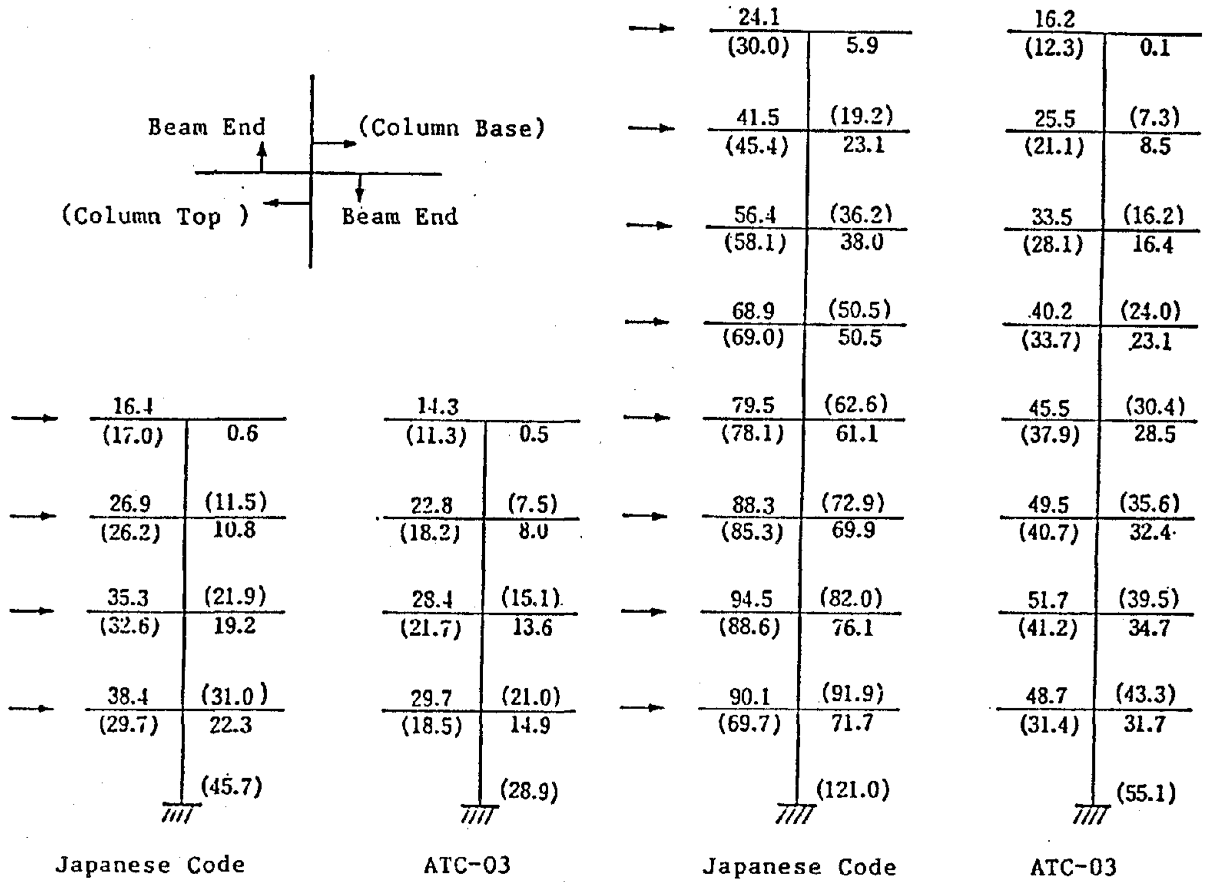


Fig. 26 : Beam Shear - Deflection Relations



(a) Four-Story Buildings

(b) Eight-Story Buildings

Fig. 27 : Design Moments (unit : t·m)

SEISMIC DESIGN CONSIDERATIONS
FOR MID-RISE REINFORCED CONCRETE BUILDINGS

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INTRODUCTION

This paper presents the evolution, from concept to completion, of a mid-rise reinforced concrete building. A mid-rise building is defined here as a building eight to nineteen stories high. The methods used are in the tradition of a California-based structural engineering office. The general concepts and controlling criteria basic to the design are discussed. Steps from concept to completion are outlined, including the factors considered in the conceptual design and selection of a framing system and the computer programs used for analysis and design. Economy in construction costs with the use of precasting and prestressing are shown, and the importance of large-scale details, careful checking of shop drawings, and reasonable construction review for minimizing field problems are discussed.

GENERAL CONCEPTS AND CONTROLLING CRITERIA OF SEISMIC DESIGN

In mid-rise buildings, possible seismic-resistant systems include:

1. shear walls that are also bearing walls, K = 1.33
2. non-bearing shear walls, K = 1.0
3. shear walls with special ductile coupling beams, K = 1.0
4. concentric diagonally braced frames, K = 1.0
5. ordinary moment frames (not allowed in earthquake Zones 2, 3, 4 as described in the Uniform Building Code (UBC)
6. concrete ductile moment frames (CDMF), K = 0.67
7. dual system: shear walls in combination with CDMF, K = 0.8

Systems 1, 2, 3, and 4 can be used only for buildings with a height of 160 ft or less. For the most part, this paper will concentrate on concrete ductile moment frames (Systems 6 and 7).

General design concepts can readily be understood by a review of the basic reasoning behind the word "ductile" as appied to concrete frames. This word encompasses both ductility and toughness.

At present, considerable information is available from theoretical studies based on records of earthquake motions, corresponding spectra and characteristics of structures. This information generally indicates that forces generated in the building during a major earthquake can be several times larger than design forces prescribed by the UBC (Figure 1). On the other hand, numerous observations of damage after various earthquakes reveal that buildings designed for considerably smaller forces have withstood major earthquakes with relatively minor damage. Absorption or dissipation of seismic energy through deformation of structure is considered one of the most important factors for the successful performance of these buildings. Some of the seismic energy is also dissipated by damping and rocking at foundations, as well as by uncalculated reserve strength of the structural and nonstructural elements on the building. The performance of a building is directly related to the quality of the overall structural system, such as symmetry, geometric uniformity, and absence of a large difference in lateral rigidity between adjacent floors. Also, meticulous detailing is essential to tie the building together so that it acts as one integral unit during earthquakes. Equally important are good workmanship and field inspections that assure compliance with contract documents.

Hence, it becomes evident that, in order to prevent collapse of the structure during earthquake, beams and columns of the moment frames must have ductility; that is, they must have the capability to undergo inelastic flexural deformations. The beams and columns also must be tough; that is, they must retain the ability to support gravity loads even with the deterioration that occurs due to cracking from inelastic deformation. The dissipation of seismic energy during inelastic flexural deformation occurs through rotation of ductile plastic hinges ($M_u\phi$). Initially, the plastic hinges should occur in beams and not in columns. Otherwise, with simultaneous hinging at the tops and bottoms of columns of a particular story, a side-sway mechanism forms. Therefore, hinge rotation demanded for energy dissipation may not be available without an accompanying high drift and a dangerous P-Delta effect.

Spectral Accelerations, S_a

Period, T	0	.050	.125	.31	.40	.50	.70	.90	1.2	2.0	3.0	5.0
SEAOC, MAX	.140	.140	.140	.156	.164	.168	.150	.132	.113	.084	.067	.040
SEAOC, MIN	.133	.133	.133	.147	.145	.134	.123	.112	.071	.054	.044	.035
EQ, 5%	.10	.15	.30	.30	.30	.26	.20	.16	.13	.07	.05	.03
EQ, 2%	.10	.16	.43	.44	.44	.39	.29	.25	.19	.10	.07	.04

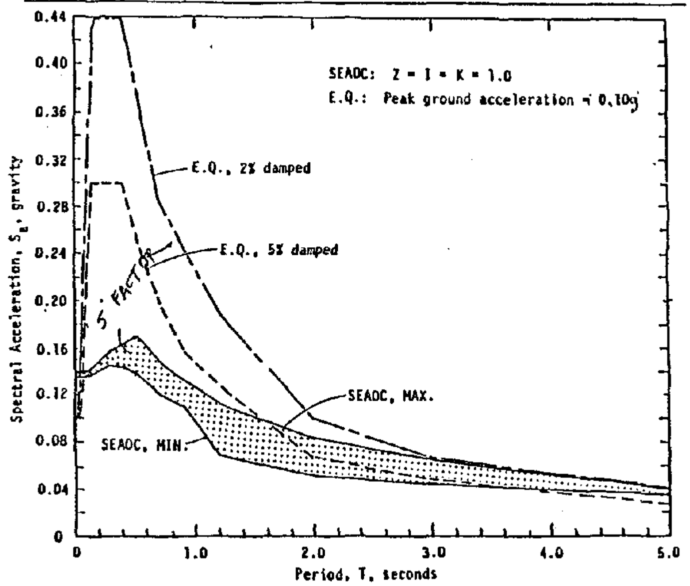

Figure 1. Equivalent response spectra.

From the above, the following controlling criteria for design can be derived:

1. flexural ductility
2. capacity design, i.e., the members are designed for maximum shear and compressive forces that can be developed due to flexural overstrength
3. strong column and weak beam
4. retention of the gravity-load-carrying capacity of beams and columns that are not part of the seismic resistant system

Laboratory tests and studies conclude that special specifications-type-design requirements in the UBC are essential to incorporate the above criteria. These special design requirements can be loosely grouped as follows:

1. confining of concrete by means of hoops to improve ductility and

toughness of concrete members; this has the beneficial side effect of increasing strength in compression, shear, and bond (see Figures 2 and 3)

2. insuring yielding of tensile reinforcement prior to occurrence of failure due to compression, shear, anchorage, or instability of noncompact cross sections of beams and columns.
3. making sure hinging in beams occurs prior to hinging in columns in most of the cases
4. checking that the non-moment frames can support gravity loads while also resisting stresses due to (3/K) x code displacements.

OUTLINE OF STEPS IN DESIGNING A MID-RISE CONCRETE BUILDING

1. Study characteristics of the proposed building, such as column layout, number of stories, floor-to-floor height, feasible framing for gravity loads.
2. Estimate seismic loads.
3. Initially, locate moment frames on the perimeter to resist most efficiently the seismic torsional moments of the building (this layout also allows optimum beam depth without impacting mechanical duct layout).
4. Find sizes of beams and columns so that the shear stress in the beam-column joint is within the allowable value.
5. Coordinate with the architect and mechanical engineer regarding column layout and beam depths if moment frames are necessary in the interior of the building.
6. Obtain information on foundation and period of soil.
7. Obtain information on penthouse layout and mechanical equipment loads.
8. Select computer programs.
9. Decide on computer model and number of case loads.
10. Carefully check the print-out of input data prior to running the program.
11. Make static check of output at various floor levels to ascertain the accuracy of the results.
12. Tabulate moments, shears, and axial forces for each member.
13. Check slenderness ratio for columns and its effect.
14. Design beam reinforcing.
15. Check columns.
16. Check non-moment frame members for 3/K deformation.
17. Show large-scale details on working drawings.
18. Add special note regarding reinforcement and concrete in specifications.
19. Make site visits and coordinate with full-time inspector on site.

ECONOMY IN CONSTRUCTION COSTS

Economy in construction costs can be attained by reducing field labor, especially carpentry and mason work, and by speeding up construction. Faster construction means lower financing costs and earlier occupancy of the building. Economy can be achieved in construction by incorporating the following ideas.

First, in projects with unusual plans, where pour-in-place concrete construction is necessary, maintain one size for beams and one size for columns in identical bays for as many floors as possible. Investigate the

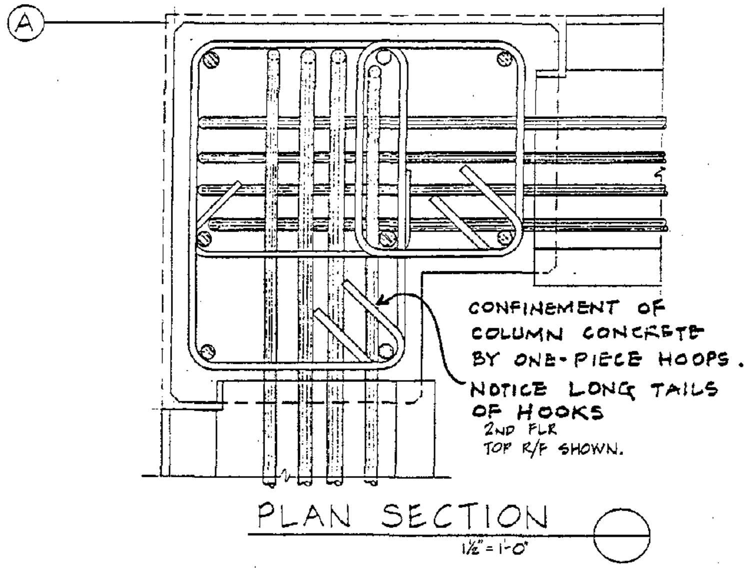

Figure 2. Confinement of column concrete by one-piece hoops.

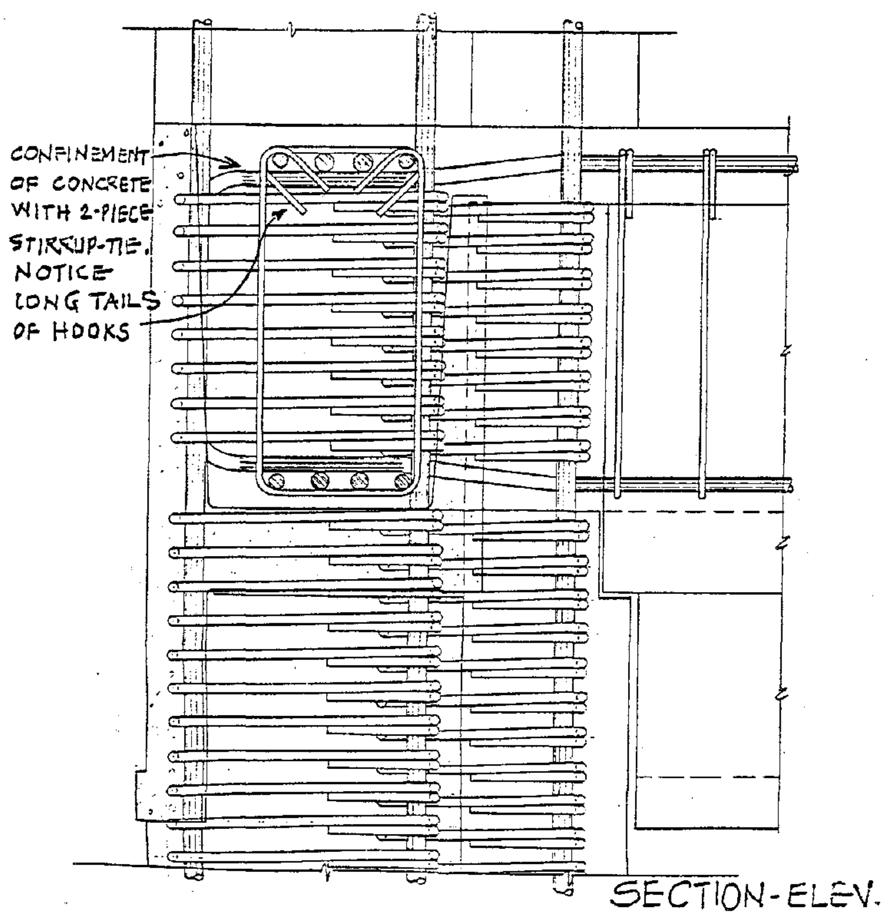

Figure 3. Confinement of concrete with two-piece stirrup tie.

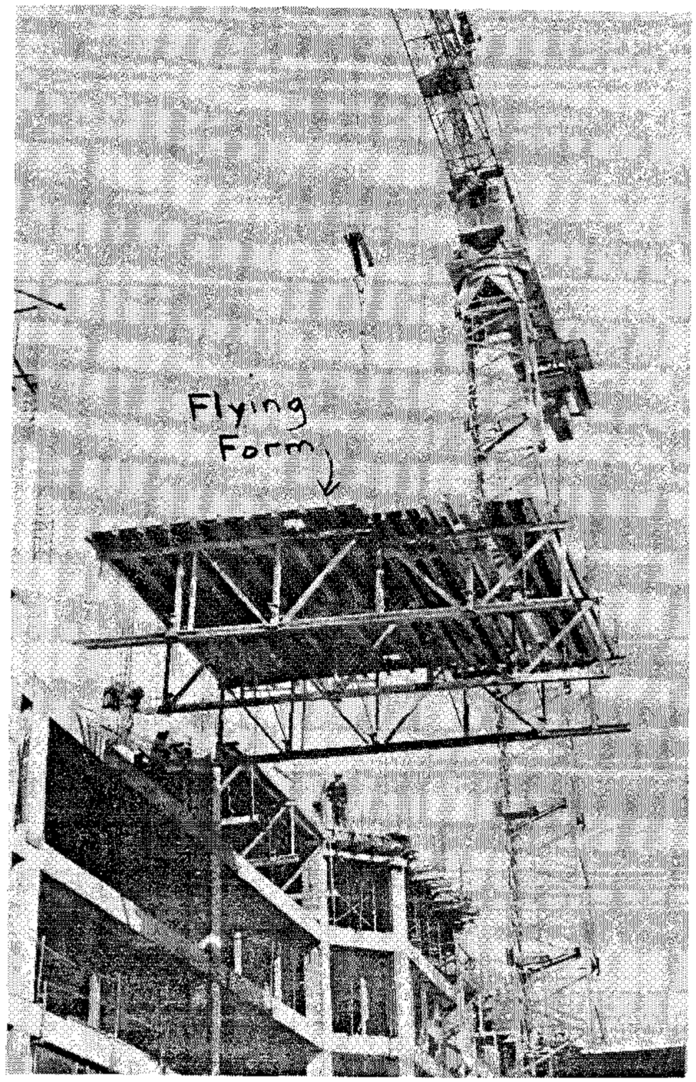

Figure 4. Flying form.

use of flying form, a one-piece platform type formwork for slab and oftentimes for slab and beam together. This one-piece formwork is removed as a whole from one floor and installed to the floor above (see Figure 4).

Second, in projects with reasonably regular plans, minimize the number of forms by maintaining one size for beams and one size for columns on all floors throughout the building. At non-moment frames, use precast beams with an added feature that can directly support formwork for slabs (see Figure 5). Notice in Figure 5 a continuous notch at the top of the beam. This notch is used to support light and shallow adjustable metal trusses that are spaced at 24-inch centers. Five-eighth-inch plywood is laid on top of these trusses as a form for a concrete slab. After the concrete attains the required strength, these trusses are easily removed and taken to the floor above. Use

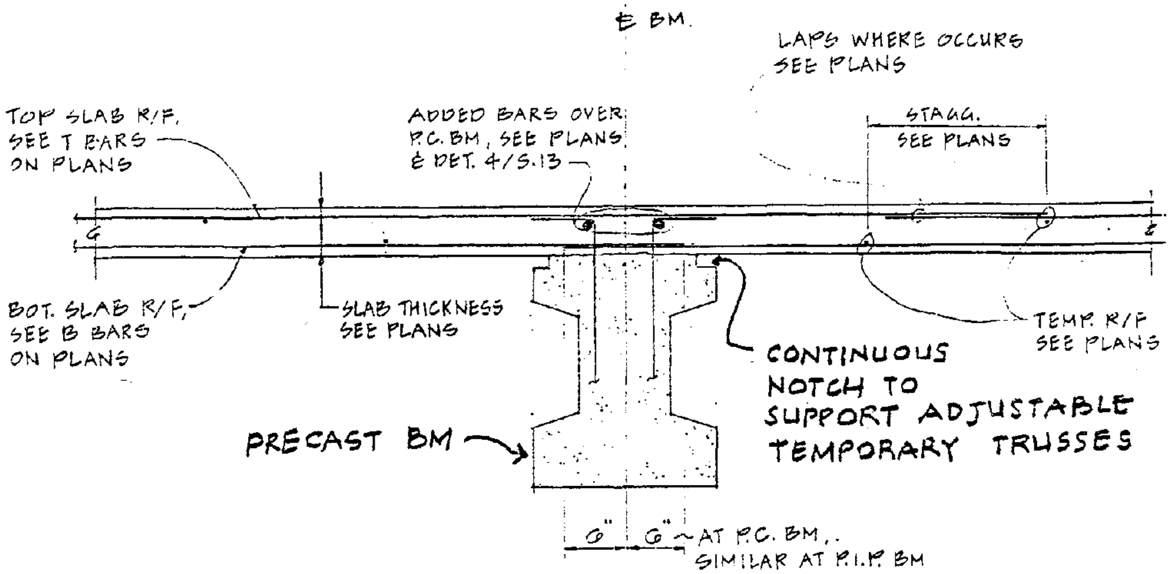

Figure 5. Use of precast beams.

prestressing in precast beams to span a longer distance with less depth and less mild reinforcing. Use precast elements on the perimeter. On the exterior, these precast elements can provide all types of architectural treatment. They also double as forms for moment frame beams and columns (see Figure 6). By careful planning, have precast elements fabricated and stacked on the site for use without delay.

Third, pre-assemble reinforcing cages for columns.

Fourth, machine-produce column ties as well as beam stirrups from continuous wire fabric sheets.

LARGE-SCALE DETAILS, SHOP DRAWINGS, AND SPECIFICATIONS

The importance of large-scale details cannot be overemphasized. Without such details, the designer cannot be certain that the reinforcement as designed will fit. Frequently, member sizes have to be altered, and large-scale detailing makes alterations easier. Additionally, with these details the general contractor as well as rebar detailer, fabricator, and erector are made aware of the complexity of the job. Thus, these details not only help make CDMF projects feasible to build, but they also help save valuable construction time. Further, they reduce the numerous emergency site visits by the design engineer. See Figures 2, 3, and 7 for examples of large-scale details.

Careful checking of shop drawings should be undertaken, preferably by an engineer familiar with the design. Errors in shop drawings of CDMF projects are costly because placement of reinforcement in the field becomes difficult due to tight clearances and the multitude of ties and stirrups. Make sure

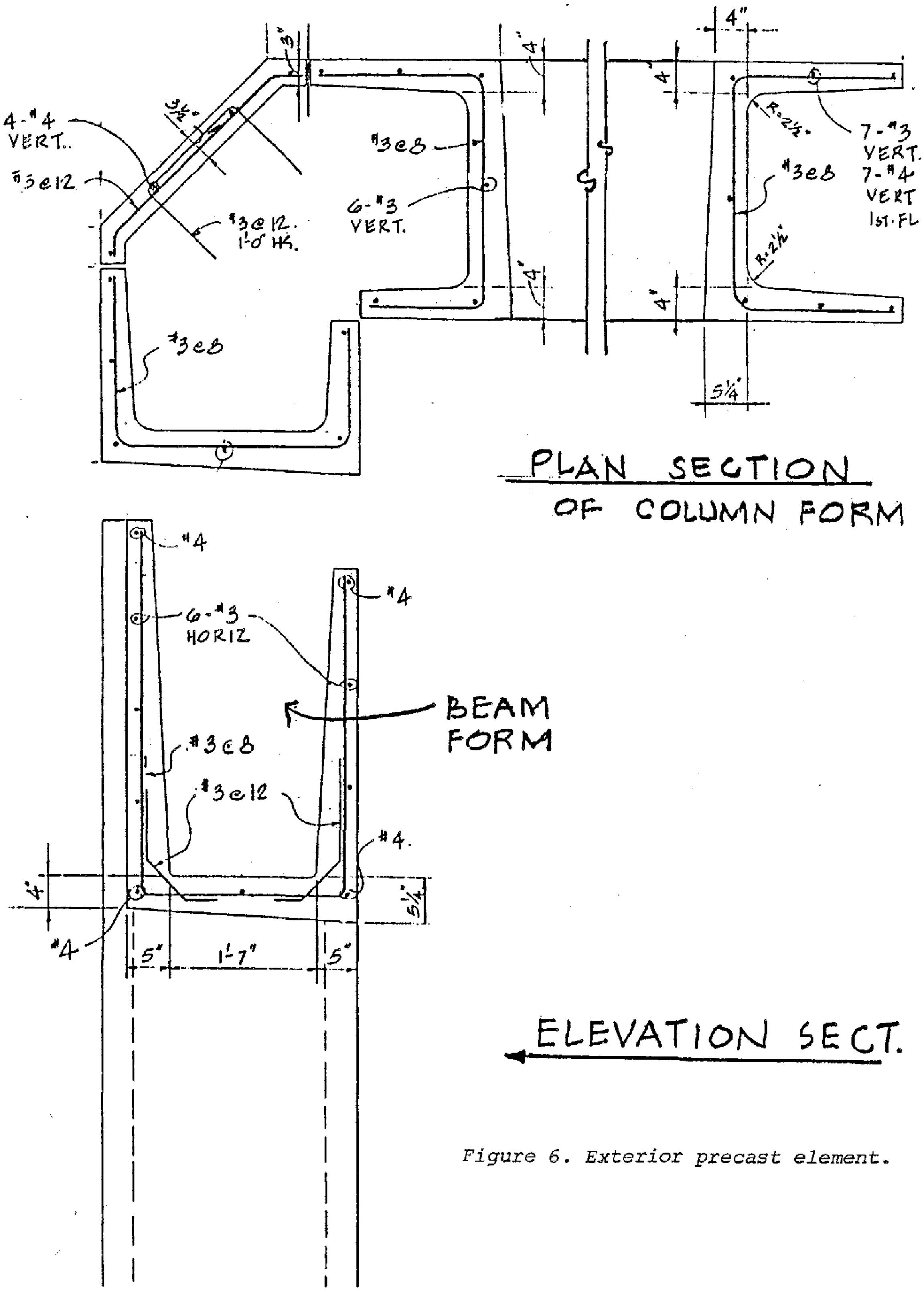

Figure 6. Exterior precast element.

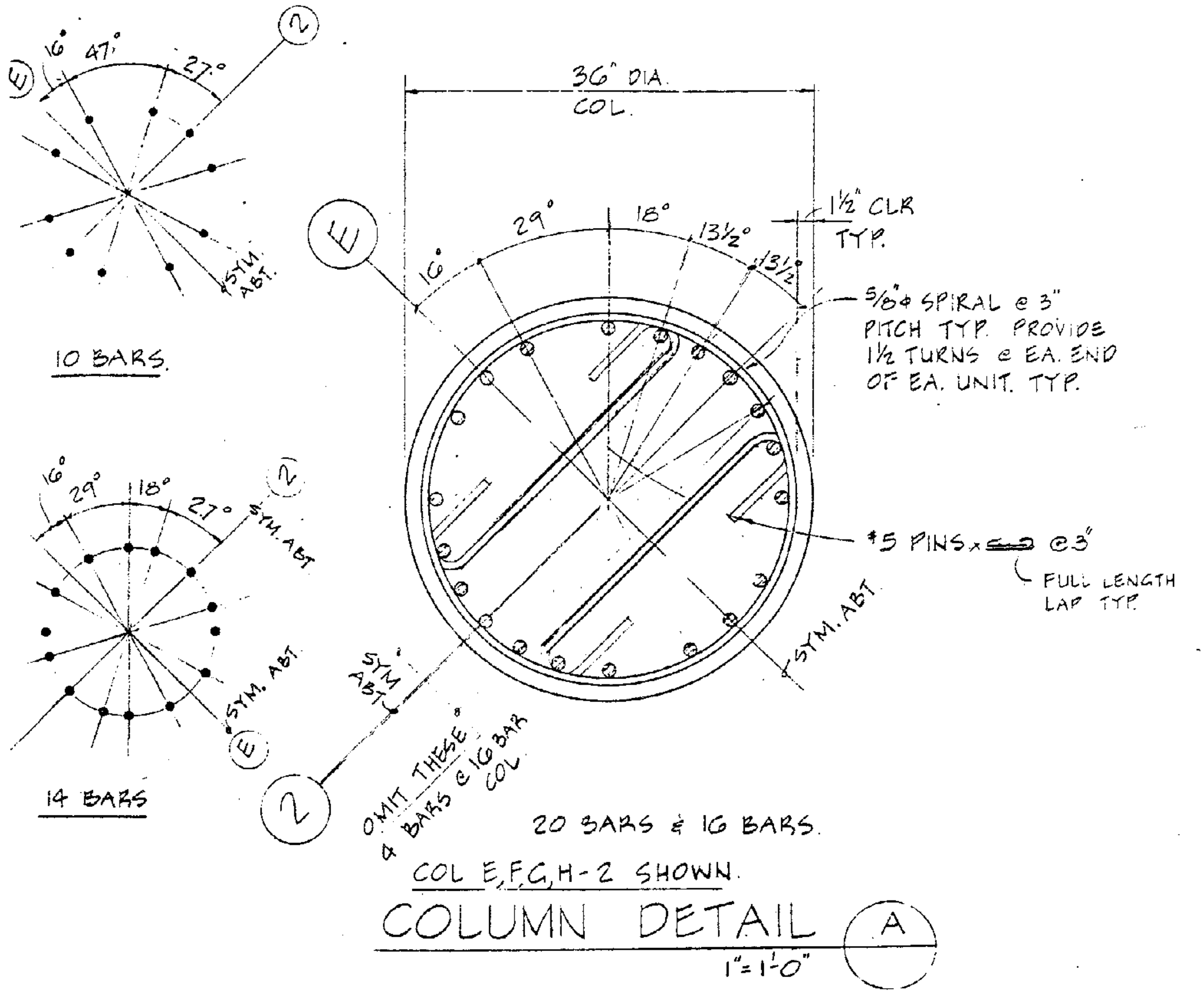

Figure 7. Column detail.

rebars are properly offset vertically when beams intersect at columns. Also insist that large-scale details are shown on shop drawings.

Certain notes on reinforcing and on concrete are important in specifications for CDMF projects.

1. Reinforcing
   - o The structure incorporates "concrete ductile moment frames." Therefore it requires close tolerances and extra care in detailing, fabricating, and placing of reinforcement.

- Prior to erection, submit mill test reports of reinforcing in beams and columns. Reports must show that the yield stress is not greater than 78,000 psi and that ultimate tensile stress is larger than 1.33 x yield stress.
- Provide templates for accurate placement of vertical reinforcement of moment frame columns (see Figure 8).

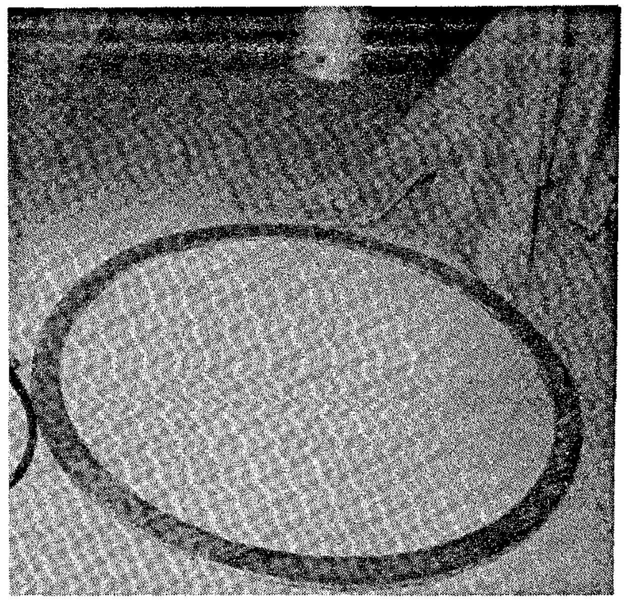

Figure 8. Two-piece template for positioning vertical rebars in CDMF column

INSPECTION

The owner must contract an independent inspection agency to inspect all reinforcing with special attention to moment frame beams and columns. The inspector must be specially qualified to inspect CDMF. Generally, it is advantageous to educate the general contractor, inspector, and rebar erector regarding the necessity of careful planning for placement of rebars, especially in moment frame beams and one-piece column ties at beam-column joints.

ADVANTAGES AND PROBLEMS

The advantages of a structure with perimeter moment frames are numerous. First, seismic resistance is provided in a fairly uniform and symmetrical manner. Next, the ill effects of torsion in the building are reduced to a minimum. And finally, chord forces, collector forces, and overturning forces are small in value and do not add to the cost of the building.

There are some problems and some questions that need more study. The goal of design, according to the Structural Engineers Association of California, is that the building structure should resist a major earthquake without collapse, although it may experience some repairable structural and nonstructural damage. However, the definition of what is acceptable and repairable damage has not been worked out. Other questions are, How is the appropriate value of period of a structure calculated? This has not been answered. What is the rational method of analysis and design of beam-column joint? This problem needs more study. Design of columns with biaxial bending and uplift is tedious and time consuming.

Another problem concerns additional time. More construction time is required because careful sequencing of top and bottom bars of beams is essential to enable installing of one-piece column times in beam-column joints.

There are other problem areas. In the field, rock pockets in the concrete could occur due to congestion of rebars. Congestion is a result of very close spacing of column ties, longer hooks at times and stirrups, and 90-degree hooks of top and bottom bars in beams at corner columns. This problem can be alleviated by the use of superplasticizer. Finally, we must determine if full capacity couplers can be used without staggering to connect beams and column bars. If couplers could be used, this type of connection could promote use of precast moment frames and therefore provide additional economy.

CASE STUDY: RECENTLY COMPLETED PROJECT

To illustrate some of the steps in the conceptual design of a mid-rise concrete building, we will use a recently completed project for our discussion. Characteristics of the building, estimates of the seismic loads, location of the moment frames, and sizes of the beams and columns will be discussed.

Characteristics

This fifteen-story building in Salt Lake City, Utah, has an unusual plan that resembles the shape of a boat or a mouth (see Figure 9). The height from the top of the foundation to the second floor is 23 feet, and the height between floors from the second floor to the roof is 13 feet.

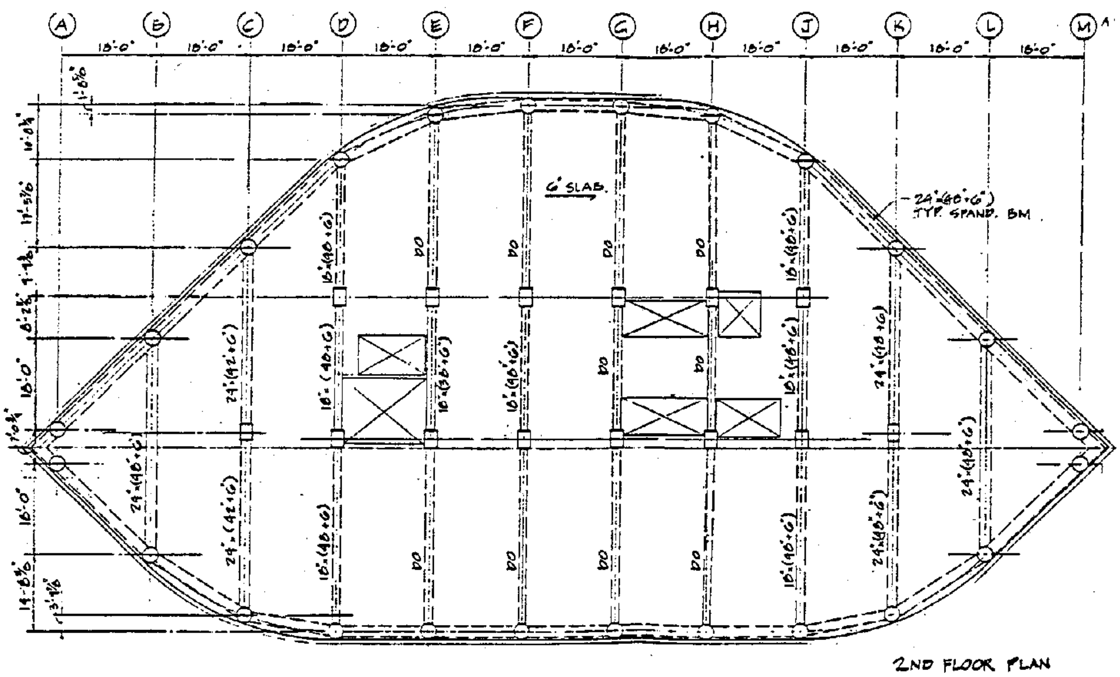

Figure 9. Second floor plan of case study building in Salt Lake City, Utah.

A metal and glass wall system is used for the perimeter of the building. Concrete construction is used due to easy availability of concrete materials and proven cost benefits compared to structural steel construction.

A bank will occupy the first floor, requiring flexible, open space. The remaining stories of office space must have flexibility to alter layouts of offices, with walls available at the elevator core only. The applicable code for the building is the 1979 Uniform Building Code.

Seismic Load Estimates

V = ZIKCSW where Z = 3/4 for Salt Lake City, which is in earthquake zone 3

I = importance factor = 1 for office buildings

K = 0.67 for CDMF

$$C = \frac{1}{15\sqrt{T}} = \frac{1}{15\sqrt{1.5}} = 0.0544$$

T = 0.1 x number of stories

= 0.1 x 15 = 1.5 = assumed period of structure

S = numerical coefficient for soil-structure resonance

= 1.5, which is the maximum value

CS = 0.14, which is the maximum value

W = weight of building

= 0.200 ksf (assumed dead load per square foot of floor area) x area per floor x number of floors

= 0.200 x 16360 square feet x 15 stories = 49000 kips

Therefore, V = 3/4 x 1 x 0.67 x 0.0544 x 1.5 x 49,000 = 2000 kips

Moment Frames

Because of the unusual shape of the building (see Figure 9), moment frames are required on the perimeter, and transverse moment frames are required through the interior of the building.

Sizes of Beams and Columns

There are 24 columns on the perimeter frames of the building for longitudinal seismic resistance, and 34 columns on 10 transverse grid lines for transverse seismic resistance. Because there are fewer columns, let us assume the moment frames on the perimeter are critical, and that the equivalent of 18 columns are resisting seismic load. Then

$$V \text{ per perimeter column} = \frac{2000 \text{ kips}}{18'} \times 1.05 \text{ multiplier for torsional effect}$$

$$= 120 \text{ kips approximately}$$

V for perimeter beam $= 120 \text{ kips} \times \frac{18'}{18' \text{ bay width}} = 120 \text{ kips}$

M per perimeter beam = 120 kips x 7.5' = 900 kip-feet at face of column

M_u perimeter beam = 1.4 x (900 kip-feet) = 1260 kip-feet

. Assume 24" x 54" beam size, d = 51", f'_c = 5 ksi, 60 grade reinforcing.

A_s top and bottom $= \frac{1260k'}{4.2 \times 51''} = 6 \text{ in}^2$

V_u beam-column joint $= \frac{1.25}{0.9}\left[(6 \text{ in}^2 + 6 \text{ in}^2) \times 54 \text{ ksi} - \frac{1260k' + 1260k'}{18'}\right]$

= 705 kips where 1.25 factor is used to allow for over strength in yeild stress of beam reinforcing

v $= \frac{V_u}{0.85 \times \text{gross column area}}$

$= \frac{705 \text{ kips}}{0.85 \times 3.14 \times 20^2}$ where column radius = 20"

= 660 psi which is less than allowable stress of $15\sqrt{5000}$ psi = 1060 psi

It is important to keep in mind that the procedure for analysis and design of the beam-column joint of CDMF is by no means precise. There is no definite consensus on the limit of allowable shear stress. Additionally, recent studies and testing point to the view that horizontal forces are also transferred by diagonal strut mechanism. Further study is necessary to arrive at a rational method for analysis and design.

CONSIDERATIONS IN THE CONCEPTUAL DESIGN OF THE CASE STUDY

1. Why use the CDMF system rather than shear wall system? Because the Salt Lake City building under discussion is higher than 160 ft, the shear wall system alone is inadequate. The feasibility of a dual system for seismic resistance was briefly considered, as follows:

V = ZIKCSW where Z = 3/4

I = 1

K = 0.8 for dual system

$C = \frac{1}{15\sqrt{T}} = \frac{1}{15\sqrt{0.7}} = 0.08$

$T = \frac{0.05H}{\sqrt{D}}$ where H = height = 200' and D = length = 200'

(length in the long direction is used, not transvere length, because the long direction is critical)

$= \frac{0.05 \times 200'}{\sqrt{200'}} = 0.7$ seconds

$S = 1.5$

$W = 49000$ kips

Therefore $V = 3/4 \times 1 \times 0.8 \times 0.08 \times 1.5 \times 49000 = 3520$ kips

$$L = \frac{3520 \times U}{0.85 \times t'' \times v \times 12''}$$ where L = required length of shear wall in feet

$U = 2$

t = wall thickness = 16"

v = allowable shear stress = $8\sqrt{5000 psi}$

= 566 psi

$$\text{Therefore, } L = \frac{3520 \text{ kips} \times 2}{0.85 \times 16'' \times 566 psi \times 12''} = 76'$$

Because of the bank requirements of open space, the available length of the shear walls would be 40 ft, far short of the 76 ft required. Similar conditions apply to the transverse direction. Thus, a dual system for seismic resistance was ruled out.

2. Why use T = 0.1N for the period of the structure? This value (T = 0.1N), prescribed by the UBC, compares well with the fundamental period of the structure obtained from computer results (see Table 1). These results are based on the gross uncracked sections of beams and columns for structure stiffness. This value of period gives smaller period and larger design seismic loads. It is generally believed that at the onset of an earthquake, the fundamental period of the building will be close to the 0.1N value. During the peak of the earthquake, the beams and columns of the moment frames will strain beyond the elastic limit, the structure stiffness will be reduced, and the fundamental period of the structure will become bigger in value. Then the imposed seismic loads will be smaller, enabling the building to safely "ride out" the earthquake.

3. Why use S = 1.5 for the soil-structure resonance factor? This value is used because soil engineers on the project were not able to furnish the value for the period of the soil. Thus, we chose this value, which is the maximum value.

4. Why use only 18 out of 24 columns in the perimeter frames for calculating seismic shear force? We arrive at this assumption because the perimeter frames follow the unusual shape of the building and some of the columns are less efficient than others in resisting earthquake loads (see Figure 10). Figure 11 shows seismic shear force in perimeter columns according to computer analysis.

5. Why use round columns on the perimeter frames? Because of the unusual organic layout of floor and columns, perimeter beams intersect columns at different angles. Circular columns were selected because it is easier to rotate vertical reinforcement within spirals to accommodate beam reinforcing than within ties of square or rectangular columns (see Figure 12).

Table 1. Comparison of period of concrete ductile moment-framed buildings.

(period by computer is based on uncracked gross sections of beams and columns; program TABS is employed.)

JOB	Height	No. of stories	Period (sec) UBC $T = 0.1N$	ATC $T = 0.025h^{3/4}$	COMPUTER	Remarks
Salt Lake	205'	15	1.5	1.35	1.58	Fixed at foundation
Gift Mart	155'	12	1.2	1.10	1.10	Fixed at foundation
Gateway	89'	7	0.7	0.72	0.65	Fixed by grade bm.
Gateway	64'	5	0.5	0.57	0.48	Fixed by grade bm.
Sunroad	89'	7	0.7	0.72	0.7	Fixed by grade bm.
Bayhill 6	50'	4	0.4	0.47	0..52	Fixed by grade bm.
Concord	82'	6	0.6	0.68	0.68	Fixed by grade bm.
Expos.	41'	3	0.3	0.41	0.5	Fixed by grade bm.
Bayhill 5	39'	3	0.3	0.39	0.48	Fixed by grade bm.
San Jose	52'	4	0.4	0.48	0.38	Fixed by grade bm.

Seismic shear stress in beam-column joints has been kept low because in this project all the perimeter columns are subjected to biaxial bending and hence additional shear stress during an earthquake. Bigger perimeter column size has beneficial side effects such as more than adequate space for anchoring beam bars of transverse moment frames at critical exterior joints. Also, bigger size provides enough space for placement of vertical column reinforcing.

Our first trial calculations were based on 36-in. diameter columns and 24-in. by 48 in. beams. First, beam-column joint shear stress was calculated:

$$M_u \text{ beam at face of column} = 120 \text{ kips} \times \left(\frac{18' - 2.5'}{2}\right) \times 1.4 = 1300 \text{ kip feet}$$

$$A_s \text{ seismic} = \frac{1300}{4.2 \times 45''} = 7 \text{ in}^2$$

$$V_u = \frac{1.25}{0.9}\left[(7 \text{ in}^2 + 7 \text{ in}^2) \times 54 \text{ ksi} - \left(\frac{1300 \text{ k}' + 1300 \text{ k}'}{18'}\right)\right] = 850 \text{ kips}$$

$$v_u = \frac{850 \text{ kips}}{0.85 \times 3.14 \times 18^2} = 980 \text{ psi which is about equal to allowable stress of } 15\sqrt{5000} \text{ psi} = 1060 \text{ psi}$$

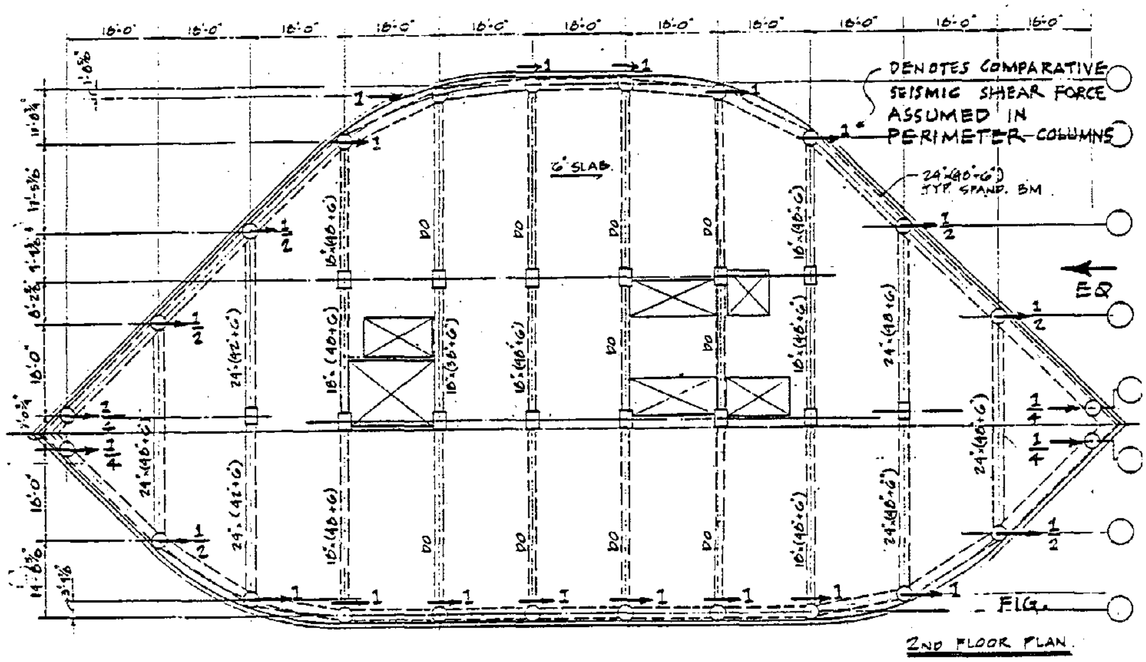

Figure 10. Perimeter frames and columns of building in Figure 9.

Next, column size and reinforcing for the first-floor columns were calculated:

P_u column = 15 floors x 400q. ft. x (1.4 x 0.200 + 1.7 x 0.050)

= 1680 kips + 510 kips = 2190 kips

M_u column = 1.4 x 120 kips x $\left(\frac{23' - 4'}{2} \right)$ = 1600 kf

eccentricity (e) for the design case of 1.4 (D + L + E) is as follows:

$$e = \frac{1600 \text{ kf} \times 12''}{2190 \text{ kips}} = 9''$$

Therefore the column size required is 36-in. diameter with 3 percent reinforcing. The 3 percent reinforcing amounts to a 28-in. square.

However, because the above calculations do not include effects of biaxial bending from transverse frames, seismic overturning force, slenderness and P-Delta effect, the decision was made to use 40-in. diameter columns and 24-in. by 54-in. beams. (The column reinforcing in this Salt Lake City project varied from 2.0% (16-#11) to 3.2% (26-#11) for the first floor columns.)

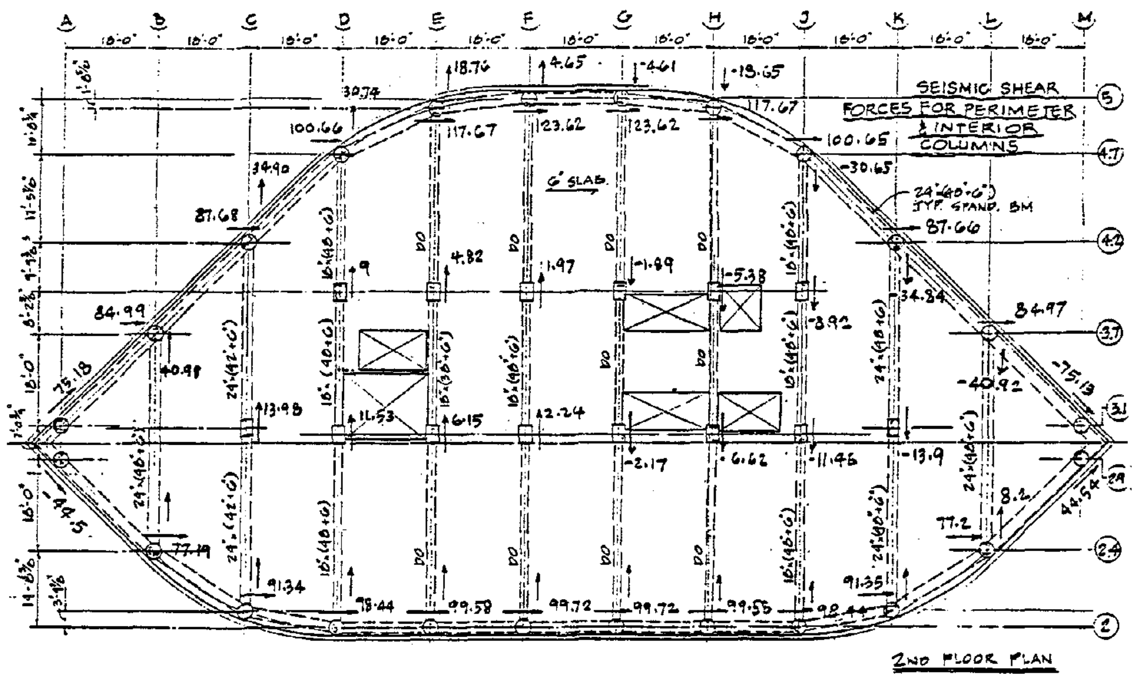

Figure 11. Seismic shear force in perimeter columns for building in Figure 9.

COMPUTER PROGRAMS USED IN THE SALT LAKE CITY PROJECT

For frame analysis of the Salt Lake City building, the computer program ETABS, one of the TABS (Three-dimensional Analysis of Building Systems) series, was used. The TABS series of computer programs are practical and efficient tools for static and dynamic analysis of multi-story buildings, with or without shear walls. Originally operating on main frame computers, these programs are now available for microcomputers with floppy or hard disks. These programs are simple and practical because their formulations take into account the following common characteristics of building structures:

1. buildings are usually of simple geometry with horizontal beams and vertical columns
2. many frames and shear walls in a building are typical, i.e., of constant dimensions
3. floors are rigid, i.e., their in-plane stiffness is very high
4. imposed loading is either vertical or horizontal
5. mass for dynamic analysis can be lumped at floor levels.

Additionally, member forces are given at the support faces of the members for direct use in design. Other features of these programs include a shear panel element for shear walls that are discontinuous or have openings, and a diagonal element for braced frames with X, K, or eccentric braces.

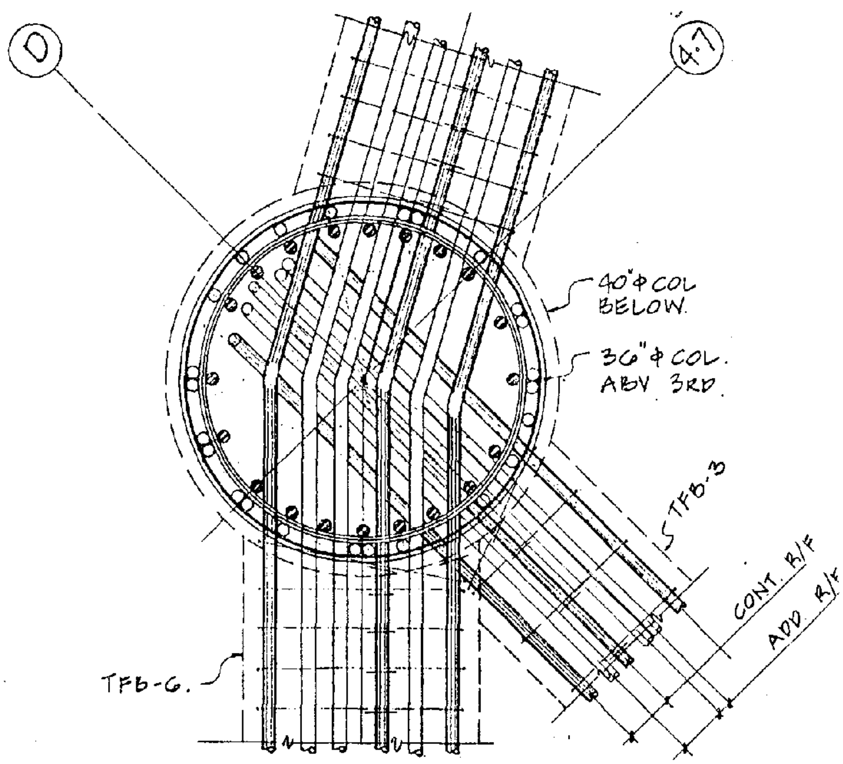

Figure 12. Detail of circular column.

For analysis and design of the building's foundation, mat program SAFE was used. SAFE is a large-capacity program that uses finite element analysis. It is also used for analysis and design of concrete slab systems including two-way flat slabs. It can economically solve systems with up to 100,000 degrees of freedom. It allows arbitrary geometry of slabs, thickness variations, and various types of supports, as well as loads. Output includes reactions, shears, moments, deflection, and required reinforcement. SAFE was an ideal program for the Salt Lake City project because it accommmodated the unusual footprint of the mat and column layout. Input had 54 spacings on I-grid, 105 spacings on J-grid, and 9 load cases. The computer printed soil pressures as well as required reinforcing in the format that followed configurations of the foundation mat. This manner of print-out allows easy checking and direct translating onto drawings.

CONCLUSION

Mid-rise concrete buildings are suitable for office buildings as well as apartment buildings. By understanding the basic concepts and complying with controlling criteria, concrete buldings can be constructed for necessary seismic resistance. With careful detailing and watchful inspections, buildings can be built for adequate performance during an earthquake. Economy in construction costs can be gained by use of present elements, repetitive forms, and prestressing.

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SEISMIC DESIGN CONSIDERATIONS FOR MID-RISE STEEL BUILDINGS IN THE UNITED STATES

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INTRODUCTION

Mid-rise steel buildings represent one of the most reliable types of construction available for seismic areas. To date, there have been no reports of significant structural damage or collapse to this type of building. Steel is inherently a highly ductile material that can withstand substantial yield and redistribution of forces with little evidence of damage. Structural steel is an adaptable material; it can be cut and welded to meet most configuration needs. When combined with a metal deck floor system, structural steel allows for very rapid building erection. Unfortunately, structural steel is an expensive material, requires considerable lead time for ordering and fabrication, and requires careful inspection and testing during construction.

It is convenient to consider the seismic design of mid-rise steel buildings in the same order in which they are designed and constructed. A mid-rise steel building starts with the selection of a design team, followed by the development of a building configuration and structural system. Using predefined or uniquely developed design criteria, the building is analyzed, designed, and detailed. A set of construction documents is developed that show the plan and elevation configurations in detail and the typical details of construction. When the building is constructed, it hopefully provides years of maintenance-free service.

STRUCTURAL CONFIGURATION

Given a mid-rise building, the selection of the structural system is actually a combined effort of the architect and the structural engineer. At issue is the selection of a framing scheme that will fit in the aesthetic concept of the building, will provide the least limitation to the functional layout of the building, and will provide a satisfactory performance under the various load conditions that it will experience. Very often, unfortunately, the system is selected because of its ease of design and adaptability to function, with little consideration given to the consequence of damage during earthquakes. To understand the structural system selection process, a few thoughts on design team organization and selection are required.

Design Team Organization

Mid-rise buildings are generally designed for a private owner by a team of design professionals. In most cases the prospective building owner will contact an architect for the design. The architect will serve as the single point of contact for the building owner, and the architect will employ various consultants to fill in expertise where it is lacking within his own staff. In most cases, the structural engineer has no direct contact with the building owner.

The capabilities of the architect are judged in terms of the appearance of his other projects, his ability to work with a building owner and achieve the desired functional goals for the project, and, perhaps, his ability to do work on time and within a specific budget. Only rarely will consideration be given to the ability of the architect and his consultants to design a structure capable of withstanding a major earthquake with a specified amount of damage.

The architect will develop the basic configuration of the building to match the functional needs of the client and develop an aesthetic treatment for the building. In all cases, the architect will employ a structural engineer as a consultant to design and detail a structural system capable of carrying the vertical and lateral loads. Other consultants involved in the project will usually include a mechanical engineer, electrical engineer, and others as required. Each professional is by law individually responsible for his portion of the work.

Structural engineers develop highly individualized opinions of the acceptable performance levels for their mid-rise steel structures. These opinions depend on their perception of the accuracy and intent of the code, their personal experience with earthquake damage, and their understanding of the owner's expectations regarding seismic performance. Our buildings are a direct reflection of these attitudes, which, in general, are uncoordinated with building owners.

Architects in California are developing a refined awareness of the importance of considering earthquake effects in the initial design of buildings. References on selecting a configuration complimentary to seismic performance are available and being used.

Building owners, in general, are unaware of the levels of damage expected by structural engineers in major earthquakes. Many expect that their buildings are "earthquake proof." Their cost-conscious attitude during the design phase leads them to insist on a minimum design. After an earthquake, however, they are often surprised and angry that more was not done to limit the damage.

Structural System Selection

The lateral force resistant system for a mid-rise steel building is generally configured as a moment-frame system or a braced-frame system. The braced-frame configuration may include steel braces, concrete shear walls, or, on occasion, steel plate shear walls. The decision whether to use steel braces, concrete walls, or steel plated walls depends on both the needed strength and on the aesthetics of the building. Steel braces provide the greatest clear space around them and provide the least obstruction to window walls. Eccentric steel braces can be detailed to provide greater ductility than conventional bracing. Concrete shear walls can be reinforced to be significantly stronger and more ductile than steel braces. Steel plated shear walls are used only in rare cases where very high shear strengths are required.

Moment frame systems are most often selected for mid-rise construction because of their adaptability to various architectural configurations, adaptability to future functional uses, ease of design, and simplicity of construction. Architects generally prefer moment frame systems for mid-rise steel buildings and often request that we develop this type of system.

From a structural engineering perspective, a moment frame system provides an efficient combination of vertical load- and lateral load-carrying ability. It provides a well-distributed lateral force resisting system that tends to eliminate any overturning problems in the structure. Because all beams and columns serve as part of the lateral system, the entire dead weight of the structure is available to resist overturning. The one major disadvantage to the system is the potential for large story-to-story deflections. These deflections are a direct result of the flexible nature of frame structures and have been shown to lead to substantial damage to the nonstructural elements of a building. In order to limit this damage, special isolation details are required for any element that extends from floor to floor. Examples of such elements are stairs, elevators, exterior curtain walls, or interior partitions.

The alternate to a moment frame lateral force resistant system in a mid-rise building is to provide some form of braced-frame system. This system counters the basic problem with moment-frame structures in that it dramatically limits the amount of story-to-story deflection that is expected. It does, however, create a number of problems in other areas that must be carefully considered and properly detailed. These include the added task of locating a balanced set of braced frames within a structural system, designing for the added shear and moment demand on the floor diaphragms, and designing for the large overturning forces which are created within the bracing lines.

Both the calculated and anticipated deflections in a braced-frame system are insignificant to the nonstructural elements. Little special consideration is required for any nonstructural elements that extend from floor to floor. These elements need only be connected to the structure for the anticipated accelerations that the building will experience. Naturally, because the braced-frame building is substantially stiffer, these accelerations will be higher than in a moment-frame building, and therefore require considerably stronger connections.

In order for a braced-frame structure to perform satisfactorily, a balanced set of continuous, full-height shear wall-type elements must be located in the structure. These elements must be located such that their center of rigidity is near the center of mass of the building and such that they provide a substantial polar moment of inertia for torsional stability.

The International Building (Figure 1) is a twenty-three-story mid-rise

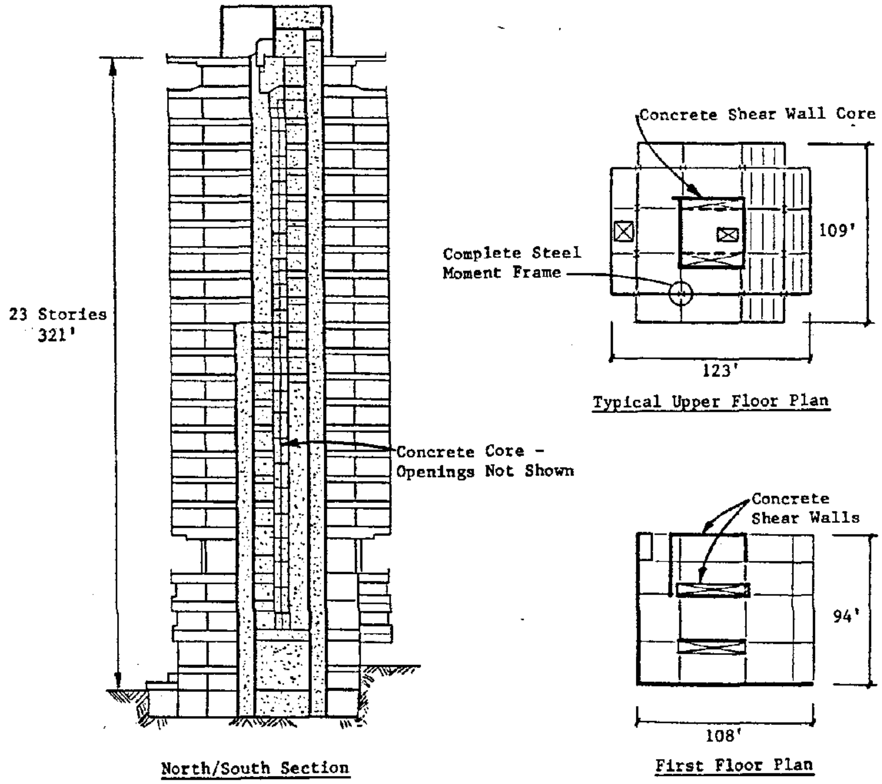


Figure 1. International Building, San Francisco, California.

steel-frame building constructed in San Francisco in 1958. It stands as a complete moment-frame structure with concrete shear wall core, and has a 109 foot by 123 foot tower and 36 foot square concrete core.

The 1978 addition to Moffitt Hospital (Figure 2) is a sixteen-story, steel moment-frame structure with plate steel shear walls. It is an irregular-shaped building that expands from a 75 foot by 216 foot tower in the upper floors to a 180 foot by 216 foot base.

Currently under design is the seismic strengthening of the nine-story Naval Hospital in Oakland, California (Figure 3). The strengthening scheme is composed of four, nine-story, 70-ft-square steel braced-frame structures acting as occupiable buttressing structures. These structures behave and are being designed as braced-frame buildings. They carry about three times the lateral force normally associated with buildings of this size.

Improvements Needed

For a mid-rise steel building, the selection of a moment frame lateral force resistant system or some form of braced-frame system should be based on a rational assessment of the damage potential of that system, given a particular building configuration and the consequences of that damage when viewed in light of the functional use of the building. In order for this to be properly done, structural engineers need a better understanding of the damage

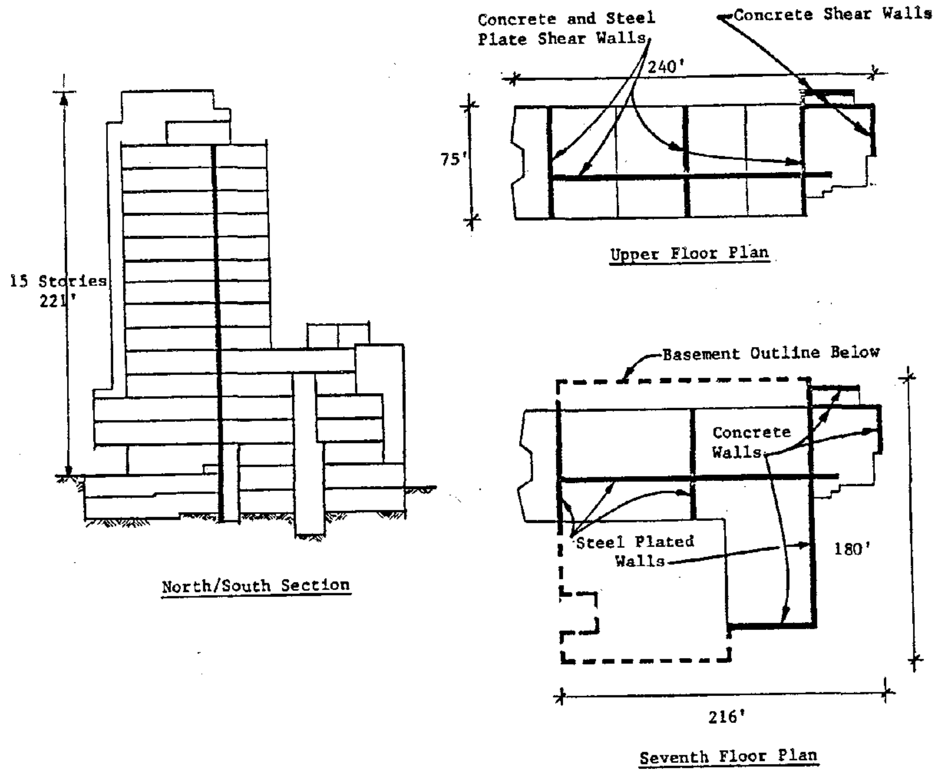


Figure 2. Addition to Moffitt Hospital, San Francisco, California

potential for the various steel frame systems that are used. In addition, other members of the design team and the building owner must become aware of this damage potential, participate in evaluating the consequence of that damage, and agree on acceptable levels of damage. Responsible decisions based on accurate information must be made by knowledgable design teams and implemented at the very beginning of the job if the goal of the lateral force resistant system is to be achieved. At present, the needed damage potential information is not available.

In order to accomplish this goal, a program of cataloguing all observed earthquake damage to mid-rise steel buildings needs to begin. It should be catalogued in a manner consistent with our current design criteria and analysis procedures. In addition, we need to actively pursue increasing understanding on the part of structural engineers, architects, and the public in general concerning the anticipated performance of mid-rise steel buildings when designed to the current lateral force requirements.

DESIGN CRITERIA

The lateral force design criteria use for most mid-rise steel buildings built in California is the Uniform Building Code (UBC) as developed by the International Conference of Building Officials. It is based on a set of recommendations developed and periodically updated by the Structural Engineers Association of California. The provisions include a minimum lateral force to be used in design, a recommended lateral force distribution for regular buildings, provisions for the lateral force analysis, and a number of special detail requirements aimed at achieving the needed ductility in the structural system. The code has been developed over the past fifty years and is based on a comparison of observed structural damage to the basic lateral strength available. In recent years, the approach has been tailored significantly to make it consistent with the available theoretical analysis procedures and the recorded strong motion records that have been obtained.

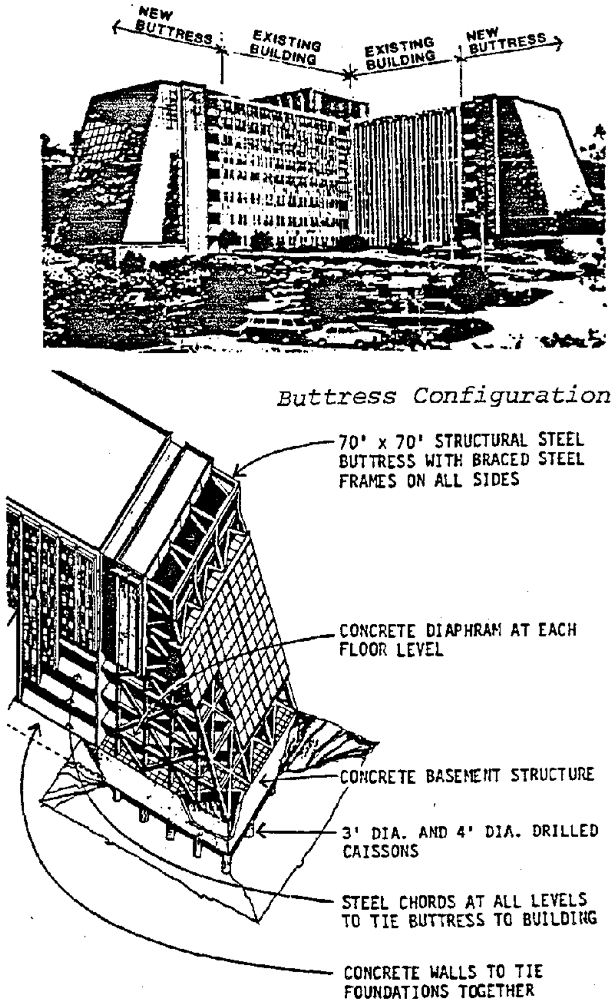


Figure 3. Seismic strengthening with steel structures, Naval Hospital, Oakland.

The current minimum lateral force level for a mid-rise steel building located in a zone of highest seismicity will vary from 4.4% g to 14% g. The design force is based on the building system, level of expected seismicity, and the function of the building. This variation is most directly related to the ductility in the framing system and to the period of the building. The design coefficient curves are based on and resemble recorded response spectra. These have been substanitally reduced to reflect available ductility, multimode effects, and material strengths beyond working stress levels.

An example of minimum lateral force levels for one class of mid-rise steel buildings is shown in Figure 4. The base shear is derived from the formula

$$V = ZIKCSW$$

where Z = zone factor ($0.19 \leq Z \leq 1.0$); I = importance factor ($1 \leq I \leq 1.5$); K = horizontal force factor ($0.67 \leq K \leq 1.33$); C = numerical coefficient, S = soil factor ($CS \leq 0.14$); and W = total dead load. The comparison shown assumes normal-use buildings (I = 1.0) in an area of highest seismicity (Z = 1.0). The International Building shown in Figure 1 was designed for 6% g with a period of 1.5 seconds.

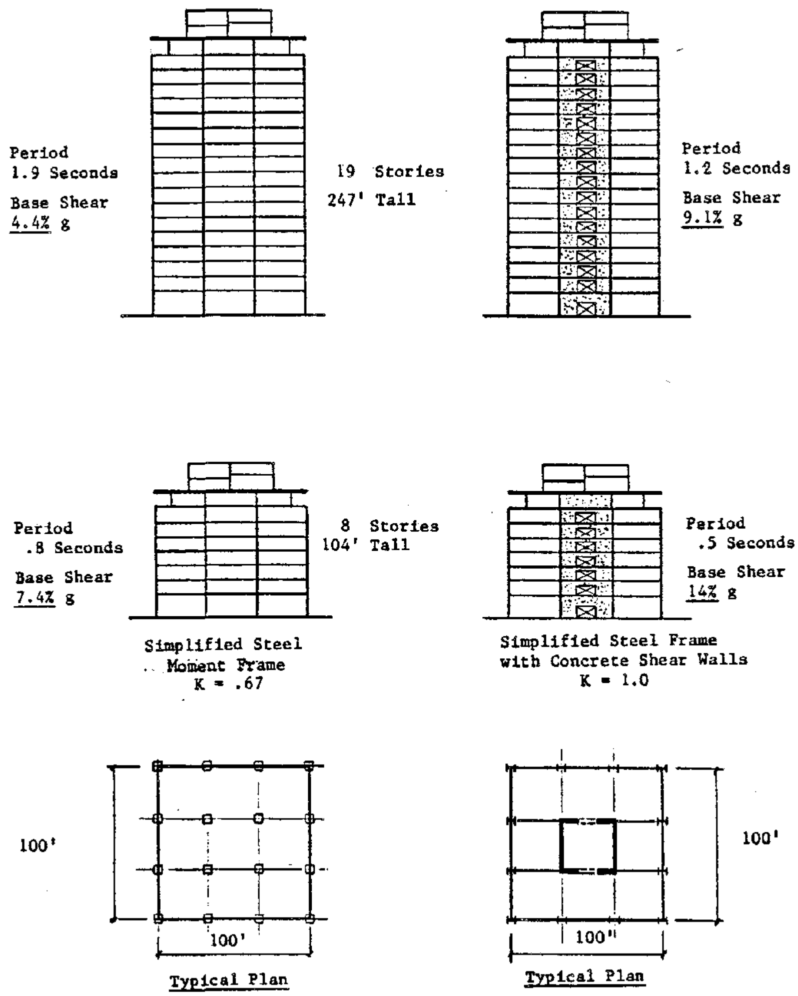
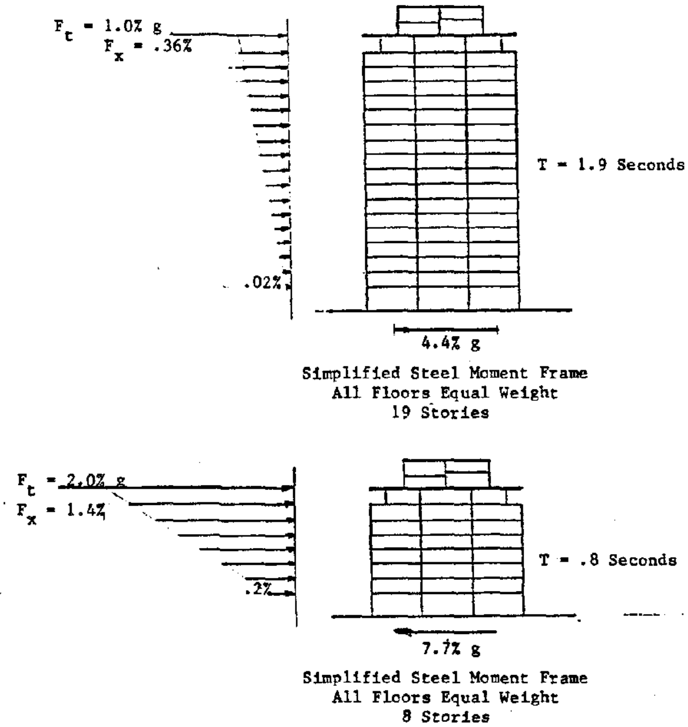


Figure 4. Ranges of code periods and base shears for mid-rise steel buildings.

The vertical distribution of the minimum lateral force is specified in the code as a triangular distribution with a concentrated load at the roof level. The concentrated load is intended to tailor the force distribution to more closely match the fundamental period of a mid-rise steel structure. The code limits the use of this procedure to structures having regular shapes or framing systems, although these terms are not defined. The triangular force distribution used in the code is based on principles of dynamic analysis. Experience has shown that for regular structures, it is a very close approximation to the results of actual modal analysis. The vertical distribution for the two example moment-frame buildings is shown in Figure 5.

For buildings of irregular configuration, the code-specified vertical force distribution varies considerably from the results of modal analysis. Figure 6 shows an example of the variation that occurred on Moffitt Hospital. Note that in this case the two distributions varied as much as 30 percent.

In the past twenty years, engineers have come to recognize the need to detail their structures so that they are capable of developing the levels of translational ductility assumed by the lateral force procedure. The Uniform Building Code now contains a number of special requirements that are tailored



Where: $F_t = .07TV$

$$F_x = \frac{(V-F_t)w_x h_x}{\Sigma w_i h_i}$$

Figure 5. UBC-defined vertical distribution of base shear example moment frame buildings (see Figure 4).

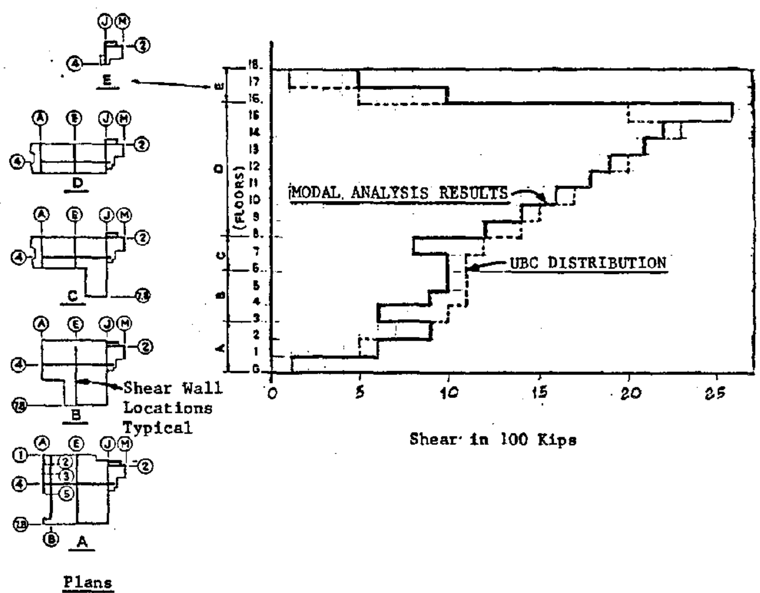


Figure 6. Vertical distribution of lateral force, Moffitt Hospital.

to make mid-rise steel buildings more than strong but also capable of redeveloping substantial inelastic capacity. These requirements involve the details necessary to develop plastic hinges, the added strength needed in brace design, the need for ductile boundary members in shear walls, and the need to control structural steel welding. Each one will be discussed, and examples shown, in the design and detailing section.

An occasional alternate procedure to the minimum lateral force requirements defined by the UBC is to develop a site-specific response spectra for use in the lateral force analysis. These are often developed within a geotechnical firm by geologists and geotechnical engineers. They have the advantage of being able to consider the local site conditions and distance from various faults in developing the expected site response. Unfortunately, the development of proper spectra for a given site is not an exact process, and often very different results can occur. Figure 7 shows three spectra developed for the same San Francisco site.

When using a site-specific response spectra, it is important not to overlook the other sound analysis and design considerations that are included in the UBC, nor to calculate an effective base shear below the minimum code levels. Site-specific response spectra should be used only to improve a structure beyond minimum code levels.

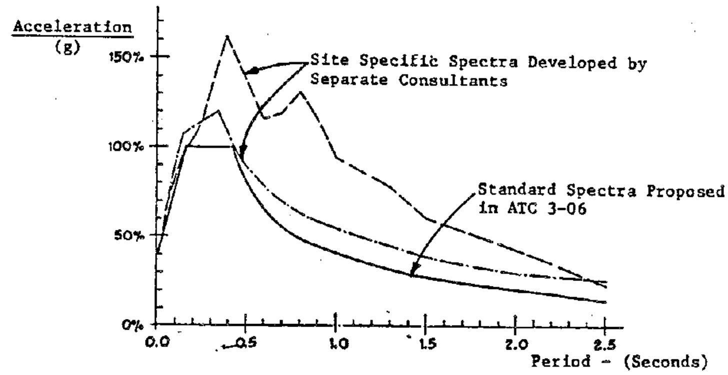


Figure 7. Response spectra for the design of Moffitt Hospital.

Moffitt Hospital was designed for an equivalent base shear of 26% g, based on the the largest spectra shown in Figure 7 and a modal analysis procedure. The higher force level, along with the shear wall system, were chosen in the interest of minimizing damage as much as possible.

The Naval Hospital is being strengthened for an equivalent base shear of 25% g based on a site-specific response spectrum and modal analysis. In this case, the higher force levels were chosen to dictate elastic behavior of the buttress under the design loading. It is important that the drift of the existing non-ductile concrete frame structure be limited to preserve the stability of the building.

Improvements Needed

By far, the largest unknown in the design criteria portion of the design of a mid-rise steel building, as with all buildings, occurs in estimating the tie between the calculated elastic response of a structure, the recorded response spectra, and the equivalent lateral force procedure. Recently proposed lateral force provisions have introduced the concept of an R factor to describe this reduction. Current UBC provisions include these reductions but do not specifically quantify them.

At present, these R factors have been derived based on experience, and considerable disagreement exists as to their validity. Although this basic analysis procedure appears to be adequate for mid-rise buildings, the magnitude of the R factors needs to be refined and related, if possible, to structural damage.

ANALYSIS PROCEDURES

There are at least two methods commonly used for the analysis of vertical and lateral loads in mid-rise steel buildings. Both techniques use some form of computer-assisted analysis. The first method involves a combination of computer-assisted analysis and hand calculations. In essence, the computer analysis is done only on typical frames and other statically indeterminate conditions. The other type of analysis is to develop a complete three-dimensional model for the entire structural system and analyze it using a single general-purpose program for all vertical and lateral loading conditions. The advantages and disadvantages of these techniques depend on the structural system, the complexity of the framing scheme, and the perception of the structural engineer.

Most mid-rise steel buildings as moment frame structures are analyzed using some form of three-dimensional analysis. Complete models are built of the entire structural system, which includes the moment frames, the foundation conditions, and any bracing elements or shear wall elements. In the case of the moment-frame buildings, this analysis technique is fairly straightforward and greatly simplifies the evaluation of the structural system. For braced-frame buildings, the analytical modeling techniques are more difficult and often cannot be adequately conceived in a single representation. Considerable care must be taken in three-dimensional modeling techniques. These must be reviewed at each milestone in the analysis and design process.

There are a number of areas in mid-rise steel building analysis that require careful consideration to avoid error. It is convenient to classify these in terms of their relation to the overall analysis of the structure, horizontal distribution of lateral forces to the various elements, and determination of the internal force distribution within elements.

Analysis of Overall Structural Response

The current UBC criteria do not define when a building is irregular and not suitable for application of the code-defined lateral force procedures. Structural engineers are responsible for this determination and exercise various applications of modal analysis to determine response. In carrying out this process, it is important to consider the effects of the foundation conditions, the accuracy of the available criteria, and the need to maintain a back check on the results.

It is convenient analytically to assume that the base of a structure is fully fixed. This may be fairly accurate for mid-rise moment-frame buildings, but must be used carefully on braced-frame structures. The results of a series of the modal analyses on Moffitt showed that the period varied 100 percent with the addition of appropriate foundation constants.

The often-used code lateral force requirements idealize the building as a concentration of mass at each floor with one translational degree of freedom at each level. There is a tendency, when using a three-dimensional model of a structure, to carry out the modal analysis using as many degrees of freedom as possible. Modal analysis is needed to describe the general building deflection characteristics in terms of floor translation and horizontal rotation. To consider more degrees of freedom leads to complex results with closely spaced modes and erratic mode shapes.

Structural engineering is a practice, not an application, of precise scientific knowledge. As such, every structure designed must represent a balance between analytical evaluation and the initiation and good sense of the design engineer. For this to be accomplished, the engineer must not allow any analysis to violate his understanding of force distribution and/or statics. The use of SRSS results for the design of individual members denies the engineer the benefit of verifying his results intuitively. Modal analysis procedures should lead only to refined estimates of base shear and vertical force distribution. These forces should then be applied statically along with all other load conditions to derive the design values for individual elements.

Horizontal Distribution of Lateral Forces

Given a lateral force and vertical distribution throughout the height of the building, the distribution of these forces to the various elements of the system must consider the stiffness of each element, the foundation stiffnesses, and the stiffness of the diaphragm. For moment-frame buildings, the typical concrete fill on metal deck diaphragm can be considered as rigid, and a rather simple analysis performed. For braced-frame buildings, however, the lateral force resisting elements are much stiffer than the floor diaphragms, and special consideration must be given to the flexibility of the diaphragm.

Figure 8 shows the floor geometry, location of shear walls, and resulting force distribution for two analysis assumptions used on Moffitt Hospital. The rigid diaphragm analysis, shown as a solid line, results in an erratic distribution of shears that is very sensitive to localized wall stiffnesses. The flexible diaphragm analysis, shown as a dotted line, results in a smoothed distribution that better represents the anticipated conditions.

The failure of a number of low-rise buildings with open fronts has led to their inclusion in the UBC Special Consideration of Torsional Forces. The provisions required that the actual torsion that is built into a system be accounted for and that a minimum amount be included.

In the normal procedures used in analysis of mid-rise buildings, it is rather inconvenient to add torsion as defined by the code or even determine the percent inherent in the system. Often an arbitrary torsional moment is added at each floor, equal to the minimum required, and no attempt is made to determine the actual torsion.

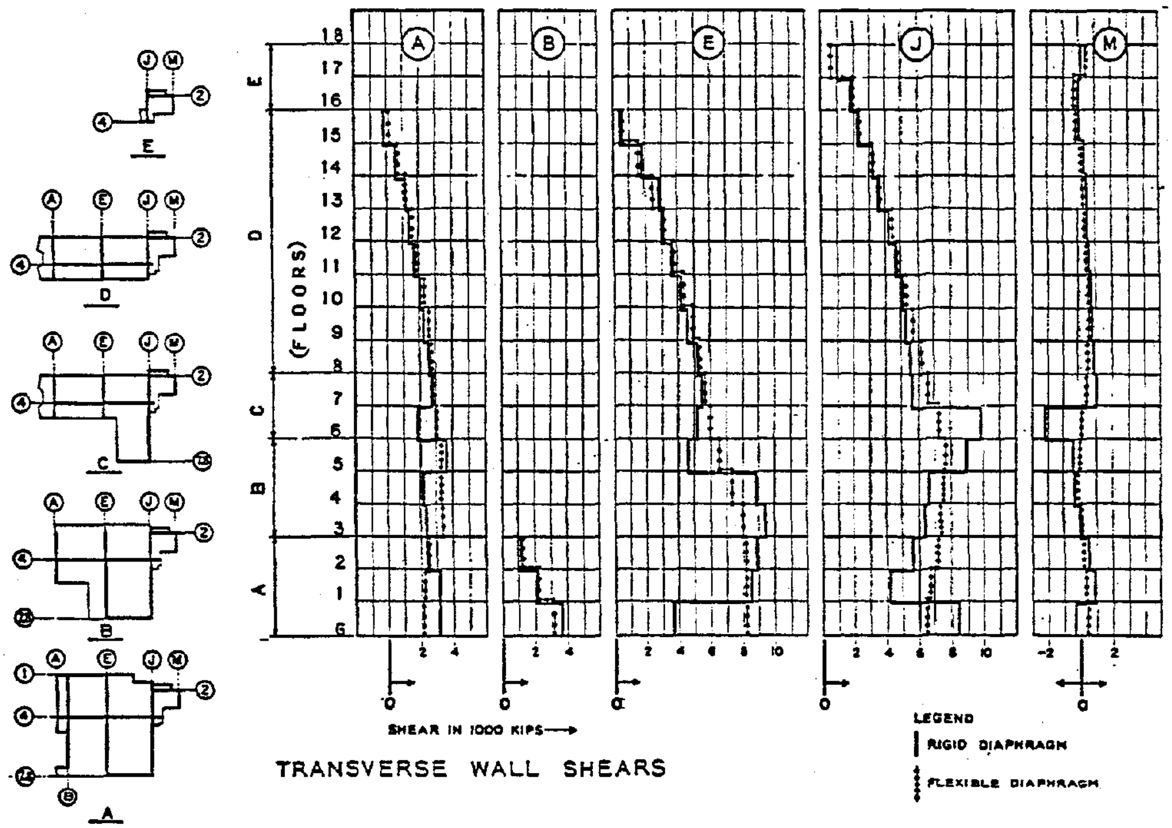


Figure 8. Comparison of rigid diaphragm and flexible diaphragm results, Moffitt Hospital.

It would be much more rational to recognize the limited value of this type of analysis to the performance of mid-rise steel buildings and eliminate it from the provisions. What is important, and what needs to be considered, is the development of a lateral force resisting system that is balanced and not subject to inherent torsion. A method needs to be devised to describe an acceptable level of inherent torsion in a mid-rise structure and to define acceptable levels.

Internal Force Distribution

Mid-rise moment frame systems and braced frame systems using steel bracing members are easily analyzed using general purpose stiffness method programs. In the case of moment frame and braced frame systems, the members can be modeled individually within the program assumptions. These models behave quite well and produce member forces that can be used directly in member design.

The analysis of shear walls is, unfortunately, not as straightforward. In shear wall analysis, the size of the members modeled becomes significant, as does the effect of the foundation conditions. In the case of a coupled, mid-rise shear wall analysis, it is important to model the foundation stiffness in order to properly value the spandrel design shear. Figure 9 shows the effect of improper assumptions on spandrel shears. Special steps also need to be taken to properly model the shear stiffness of the panel zones at the pier and spandrel intersections.

It is possible to analyze shear walls with openings using finite elements. Figure 10 shows such a model on one of the walls at Moffitt Hospital. The model used a minimum of four finite elements per pier and two per spandrel. The resulting stresses had to be post-processed into design shears and moments for each pier and spandrel to be useful.

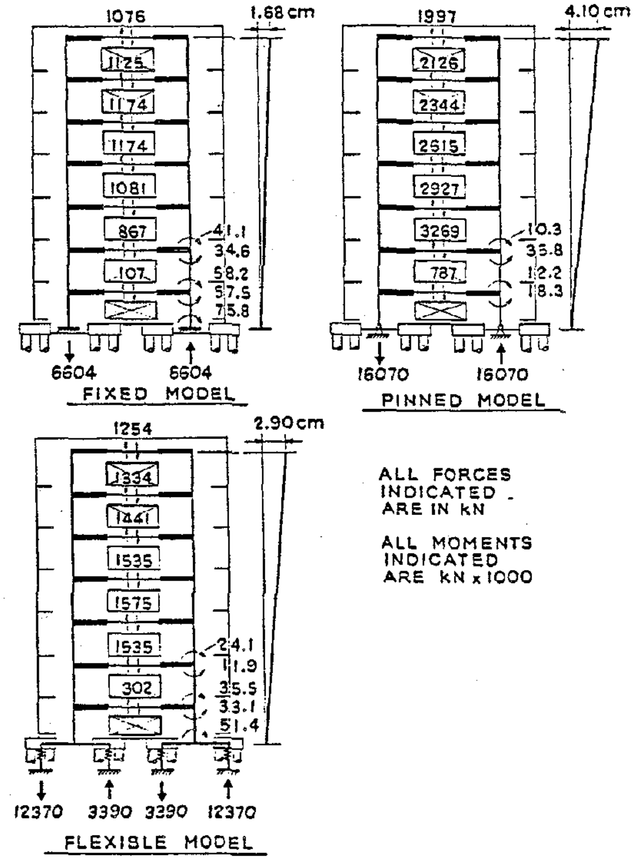


Figure 9. Effect of modeling techniques of pier and spandrel forces.

Improvements Needed

At present, the UBC set minimum standards for lateral force design. Because of the wide variety of computer analysis techniques available, these standards can be applied in a variety of ways. Unfortunately, these can be applied improperly, leading to an undesirable result.

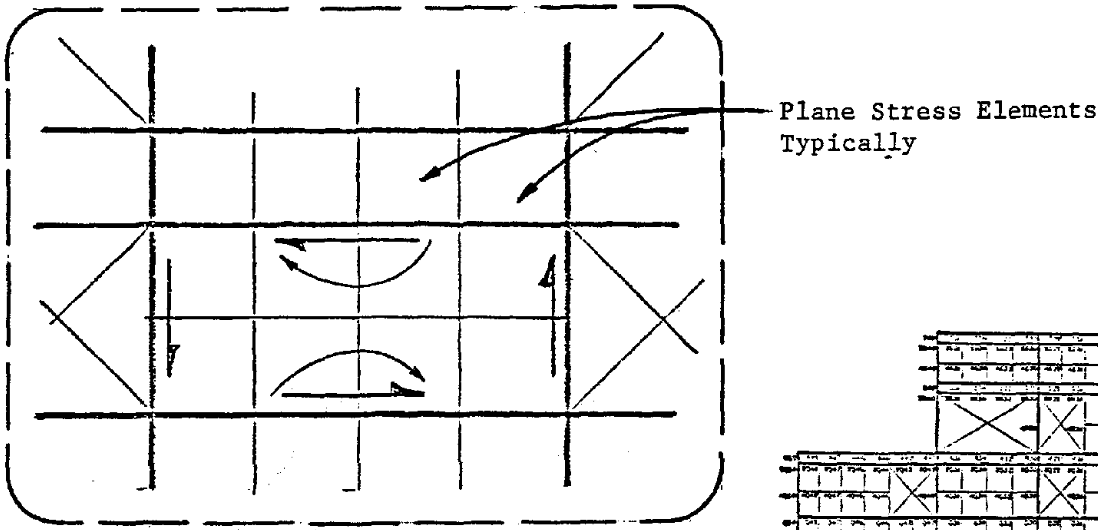
Because the code stands as a minimum, it should not be subject to a variety of interpretations for a particular building. The provisions need to be improved so that the minimums apply uniquely to each class of buildings.

Mid-rise buildings tend to lend themselves well to computer analysis of various levels of sophistication. It has become apparent that engineers occasionally use these analytical tools improperly, which results in seismic designs less than the code minimums. This action may be accidental or deliberate. In any case, improvements are needed in our modeling techniques and our understanding of the intent of the minimum lateral force standards.

There is a definite need to develop minimum standards for structural analysis and computer modeling. These need to relate to and support the minimum lateral force requirements of the code.

DESIGN AND DETAILING

Design projects are usually organized and develop as a four-part process. The client will retain the architect, go through the initial interviewing process, and develop a set of schematic drawings. These drawings



Note: 1. Must use a minimum of four elements per pier to determine points of inflection.

2. Finite element stress results must be converted to moments and shears for purposes of design.

3. This type of analysis is only beneficial for large walls with many irregular openings.

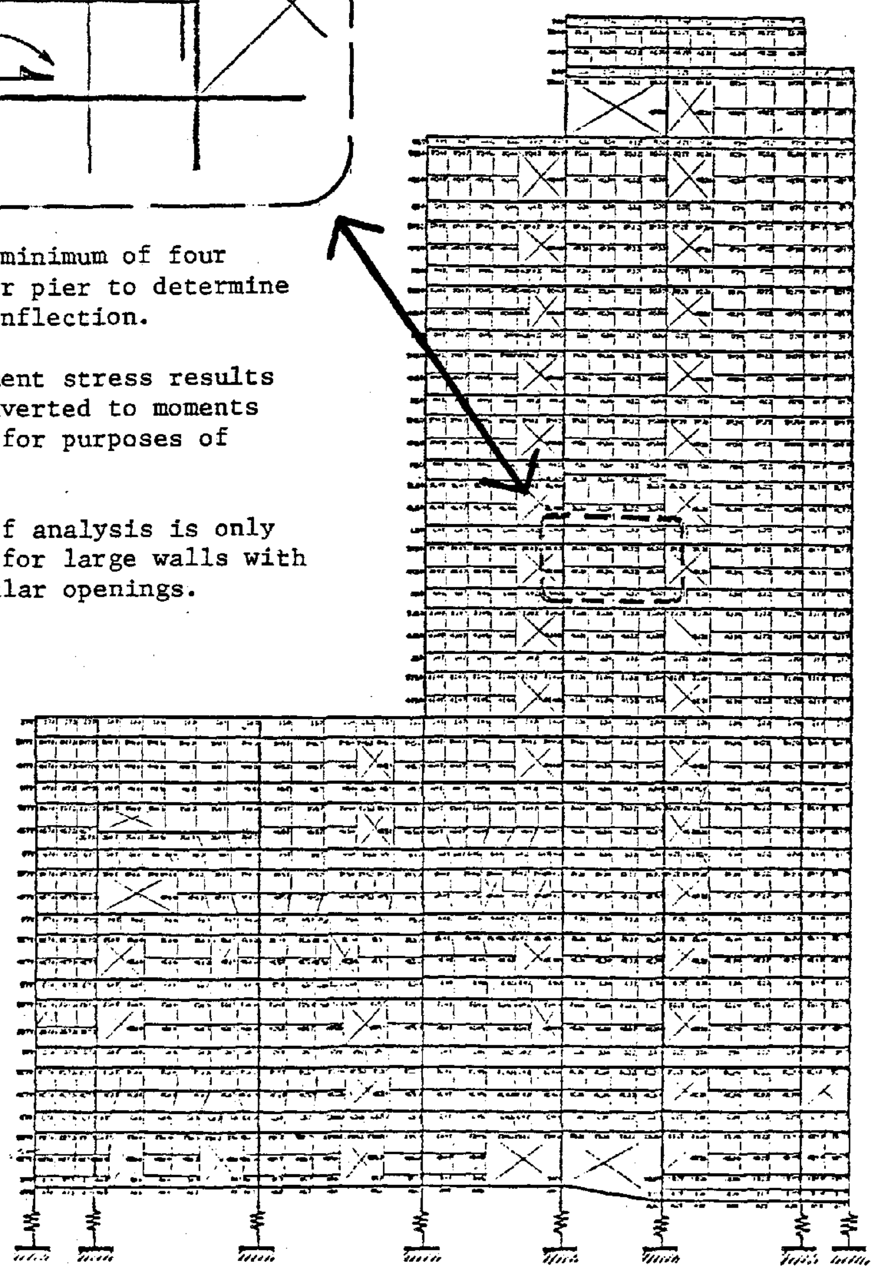


Figure 10. Finite element shear wall analysis, Moffitt Hospital.

will indicate the basic plan configuration of the building, its orientation on the lot, and the basic functional layout of the various elements. Once reviewed and approved by the client, a design team will be organized and the architect, structural engineer, and other consultants will develop the schematics into a set of design development drawings. These drawings will include plans, exterior elevations, basic framing member sizes, and details. Once approved by the client, the design team proceeds to completing their construction documents. These documents will include complete plans and specifications for the project. In general, the plans will indicate the basic geometry of the building and the typical details of construction. The specifications will contain the legal requirements for the construction of the project, definition of all the materials to be used, special procedures to be used during the construction, as well as all testing and inspection requirements. The final step, the development of as-built drawings, is normally delegated to the contractor.

Required Details

Many specific details are required by the UBC. These are intended to allow the building to achieve the level of ductility anticipated by the lateral force requirements. They include details for developing plastic hinges in steel frames, providing extra strength in steel braces, specifying boundary members in shear walls, and establishing appropriate procedures for welding and weld inspection.

In order for a steel moment frame to develop and maintain post-elastic capacity, it is important that the sections used maintain local stability and overall stability during plastic hinging. The UBC requires that steel moment frames be detailed in conformance with the plastic design criteria as established by the American Institute of Steel Construction (AISC). In general, the AISC criteria require the use of compact sections with additional limitations on depth thickness and width thickness ratios. It also includes special requirements for web stiffeners in beam-column joints, establishes stricter limits on unbraced lengths for columns, and requires lateral bracing at all anticipated hinge locations.

Research has shown that braced steel frame structures are subject to deterioration of the brace strength under repeated cycles. This deterioration is due both to the repeated buckling of the member and to the deterioration of the brace connection. The UBC requires that braces be designed for a load 25 percent greater than that calculated based on the minimum lateral force for the building. In addition, it requires that the braces be connected for their full capacity or at least 33 percent greater than the brace design load. We further limit the design of our braces to members with an l/r in the 50 to 80 range, in order to increase their cyclic ability.

Mid-rise steel buidings of both moment-frame and braced-frame configuration are nearly always built as fully welded structures. Structural steel welding is a difficult procedure and requires substantial testing and inspection. The UBC specifies detailed procedures for the various types of welds and also includes minimum standards for testing and inspection. As a practice, we specify that all welds will be performed under full-time inspection and that all of the complete penetration welds will be tested non-destructively.

A series of typical details of steel framing common to mid-rise steel construction is shown in Figures 11 through 15. These will vary among structural engineers, but in general are similar. In developing details of this type, it is important to consider that, in some areas, these details will experience yield level loads. The connections themselves should not be the weak links. For that reason, beam column connection should contain full continuity plates (Figure 11). Column splices should be full penetration if they carry uplift (Figure 12). All overturning forces should be carried fully into the foundations, so that any overturning-related yielding will occur at the foundation soil interface (Figure 13). Diaphragm should be detailed to be weak in shear capacity and not in interface shear between the concrete, metal deck, and steel beam. All diaphragm shear should be carried in the reinforced concrete and not the metal deck. Eccentric braced connections should yield in shear and not buckle the brace. Concentric brace connections should develop the member in both tension and compression, and the braces themselves should be limited to a kl/r of 80 (Figure 15).

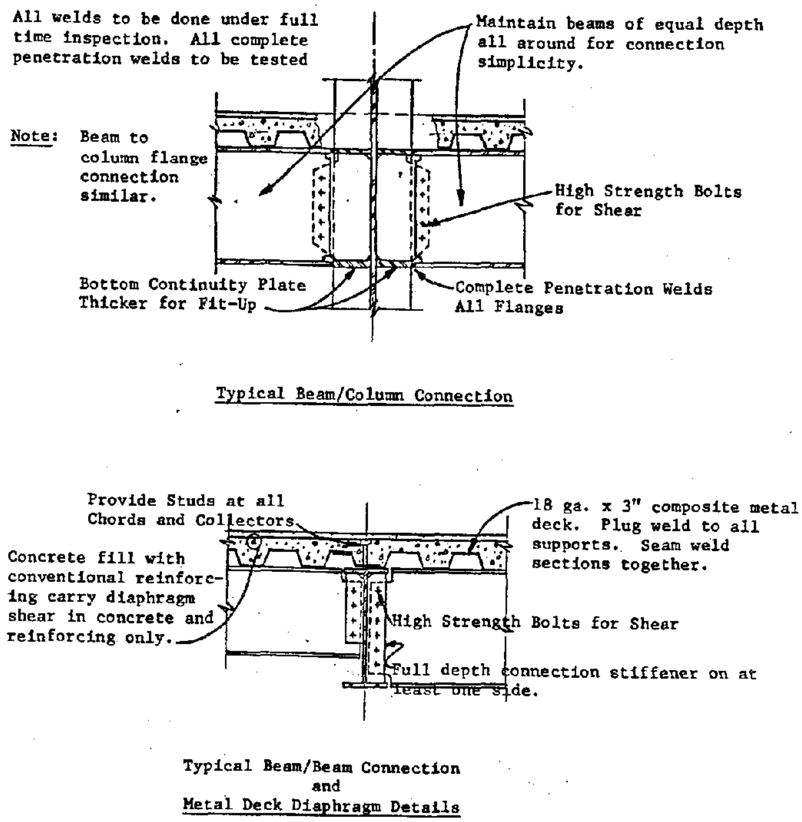
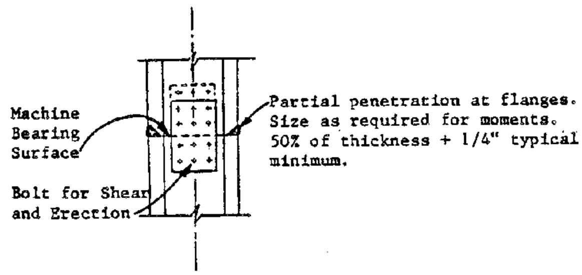


Figure 11. Steel beam and metal deck floor details.

The use of steel-plated shear walls is very effective for high shear conditions. The steel plates need to be stiffened to control shear buckling, and they must be fireproofed. On Moffitt, concrete walls were gunited on each side to accomplish both of these needs. The disadvantages in using steel-plated walls include the very high overturning forces associated with the large capacity of the wall and the lack of flexibility in making even minor future penetrations.



Improvements Needed

There is much concern about laminar tearing in structural steel due to welding. At risk are the large plates that are loaded perpendicular to grain due to weld shrinkage. Unfortunately, this is the same loading that a column flange will see in a moment connection under a major earthquake. We have yet to see whether these joints are as ductile as we believe. Additional research and testing are needed regarding the performance, yield capacity, and ductility of weld steel connections using large sections.

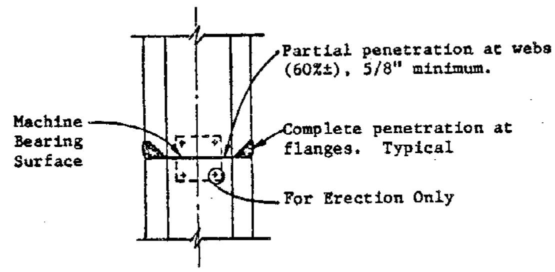


Figure 12. Column splices.

Metal deck diaphragms are ideal from a construction standpoint. They are adaptable, usable during construction for staging, and strong. The concept of a 4-1/2" to 6" concrete slab to distribute and redistribute lateral loads from major lateral force resisting elements is unproven. The analysis shows that the stiffness is there but the relative sizes of the elements question its rationality. At issue is not whether a metal deck diaphragm provides a safe system, but what its appropriate representation should be in a proper analytical model. Improvements are needed in our understanding of the strength and behavior of metal deck and reinforced concrete filled diaphragms.

SUMMARY

Structural steel is probably the most common form of material used for this class of buildings in California. Structural steel buildings are conceived as moment-frame, braced-frame, or shear-wall buildings. Each of these systems has various advantages and disadvantages that can be thought of in terms of building performance, architectural aesthetics, adaptability to future uses, and initial cost of construction. As a general statement, moment-frame buildings can lead to more damage to architectural elements in a

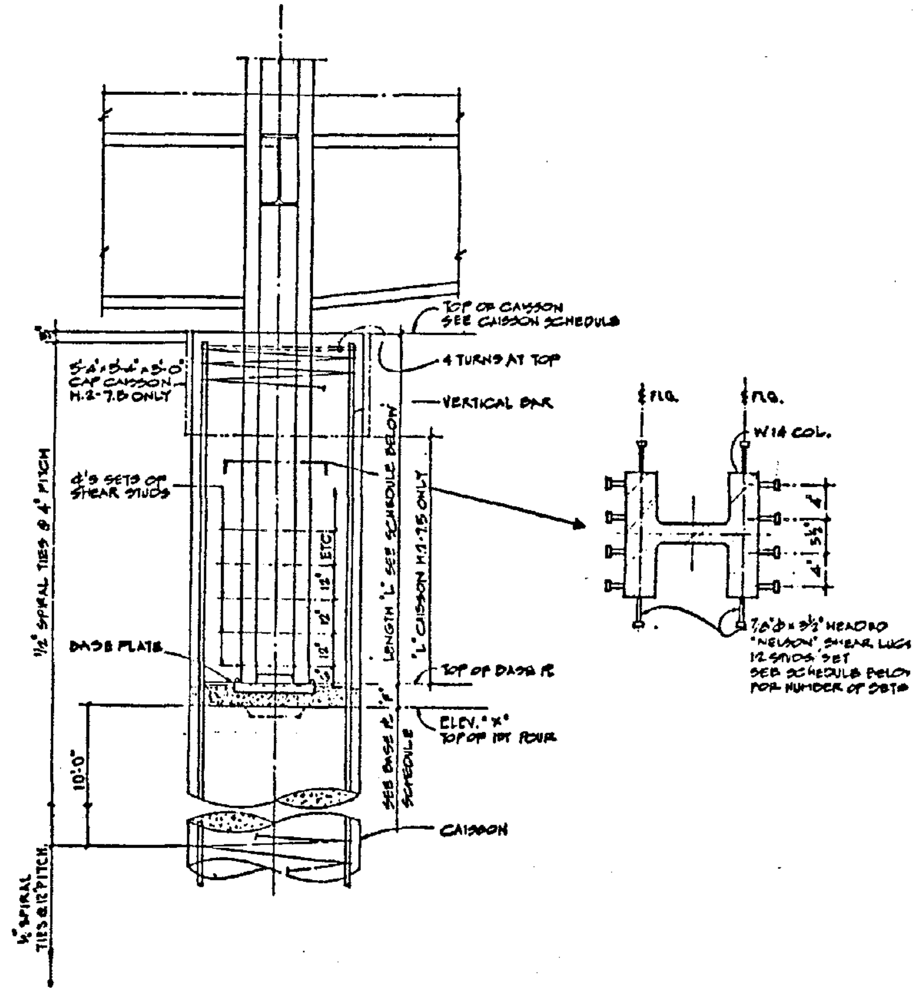


Figure 13. Column to caisson connection for overturning.

building, while braced-frame and shear-wall buildings need to be built stronger because of the increased lateral forces that they develop.

The selection of the structural system for a particular project is not solely the decision of the structural engineer. The project is designed by a team of architects and engineers, all of whom have some input into the building configuration and structural system. From the architectural point of view, the structural system must be durable, should not inhibit the architectural expression of the building, and should not inhibit any future reorganization of the building function. From the structural engineer's point of view, the structural system has to provide a complete vertical and lateral load-carrying ability and sustain only limited damage in a major earthquake. Very often, the fundamental use of the building will dictate the structural scheme and overshadow serious consideration of seismic performance. A better understanding of the vulnerability of mid-rise steel buildings to damage is needed.

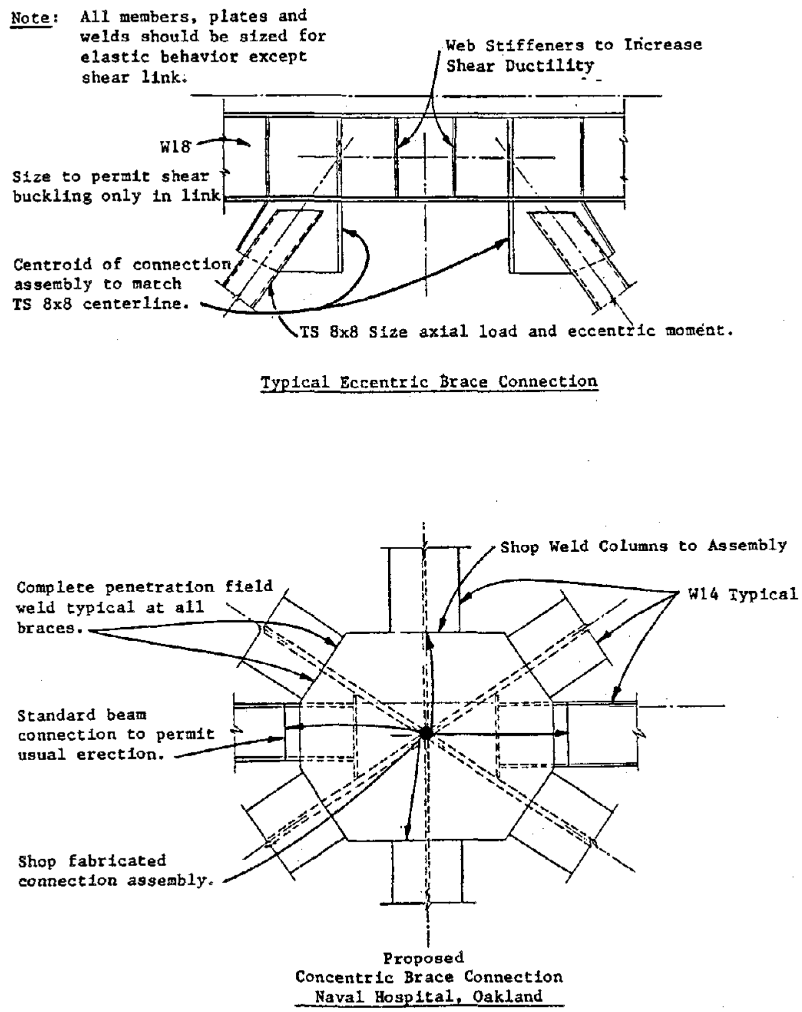
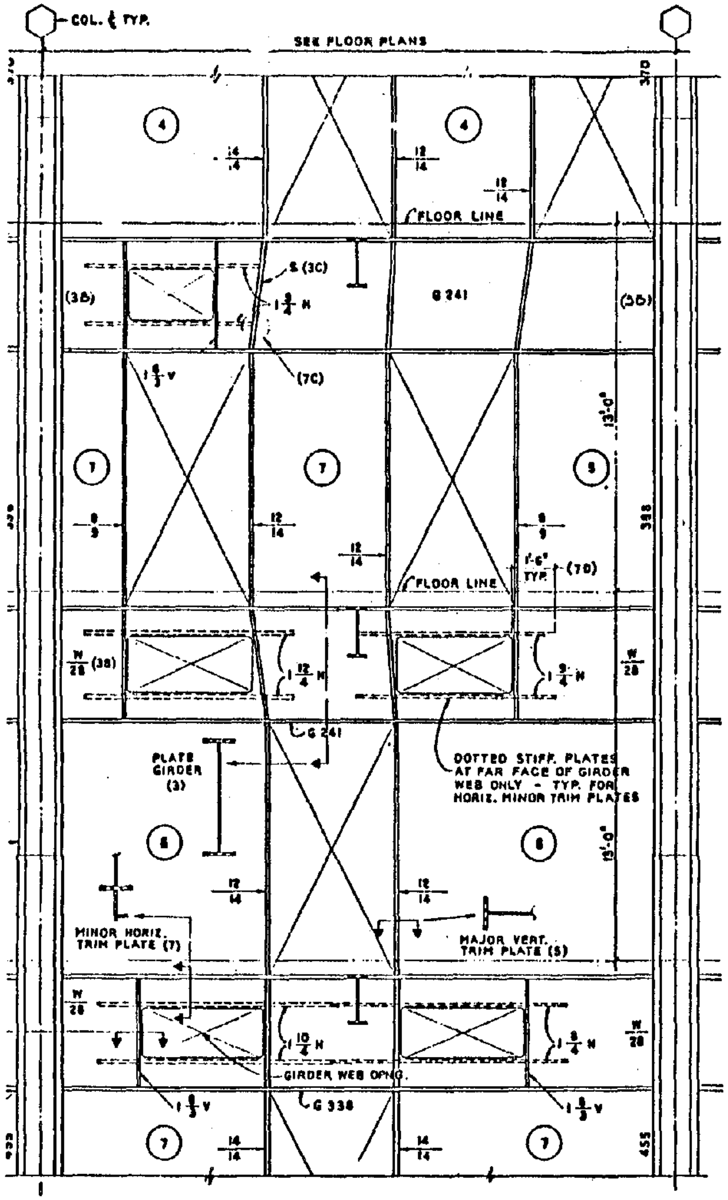


Figure 14. Selected brace frame connections.

Nearly all buildings in California are designed to the requirements of the UBC. This code is developed by the Association of City Building Officials. Their intent is to develop a document that sets a minimum standard of construction that can be used to review projects that are submitted for construction in their jurisdictions. The Association of Building Officials accepts proposed code provisions from various associations and agencies for inclusion in their building code. The lateral force provisions in the UBC were developed and continue to be updated by the Structural Engineers Association of California. In reality, then, the design criteria that are used in California are developed by the California structural engineers. Criteria basically involve a stated minimum lateral force resistant level for each type of building and a collection of special detail requirements that are intended to add the necessary ductility to the structure. Refined criteria that can better relate seismic performance to design are needed.

Mid-rise steel buildings are analyzed for lateral force requirements using a variety of hand and computer-assisted techniques. In general, most are designed for the minimum lateral force requirements in the UBC, although



Elevation of Steel Plates, Trim Bars, Stiffeners and Frame Members

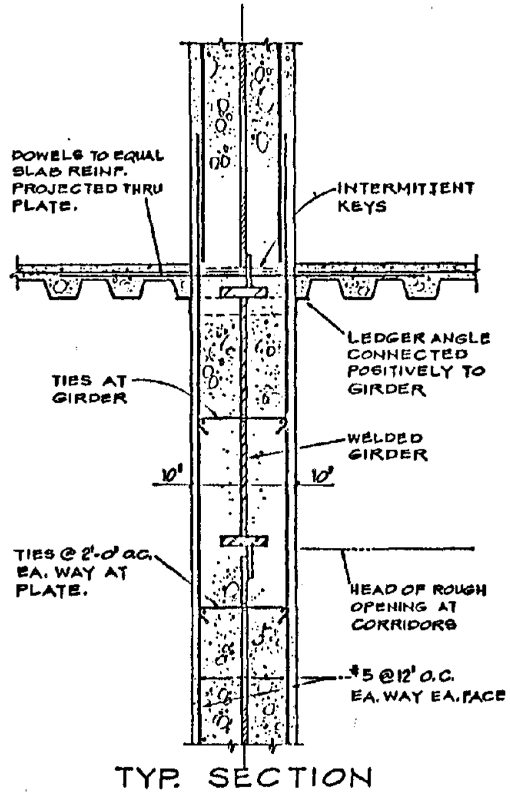


Figure 15. Steel plated shear wall details, Moffitt Hospital.

some are analyzed using modal analysis procedures and site-specific response spectra. In the standard code procedure, lateral forces are derived for each floor level, distributed to the various lateral force resisting elements and internal member stresses determined for all of those elements. These internal member forces are added to the vertical load requirements for the various members, and the members are designed for the appropriate combined loads. Computer analysis is often used for the modal analysis, to distribute the forces to the various members, and, on occasion, to design the members automatically for the forces that have been calculated. Improved guidelines are needed to establish appropriate computer analysis techniques.

The actual structural design of a building proceeds as follows: The structural analysis of the building is preceded by a preliminary sizing of all main members and followed by the final sizing of all the members in the structure. The connection details are subsequently developed and normally shown for all typical conditions. Plans, elevations, and sections are then drafted and, hopefully, checked carefully by the structural engineer. Finally, a set of specifications is developed to define the various types of materials that are to be used, the special procedures of construction that need to be followed, and the various testing and inspection requirements for the job. Many of the welding details and diaphragm results that are being used are untested and need to be watched carefully in future earthquakes.

Given a set of construction documents for the job, the owner will contract with an independent general contractor for the construction of his building. In almost all cases, this contractor is not related or associated with the professional design team. The contractor is selected either on a negotiated basis or through some sort of bidding process. The general contractor will hire a variety of subcontractors that perform the work. The subcontractors develop complete sets of shop drawings for the various structural elements. These shop drawings are based on the design drawings and are reviewed by the design team. If the building is to be built as envisioned by the design team, a considerable amount of field review and construction inspection is required.

The goal of mid-rise steel construction is to develop a building that will provide decades of maintenance-free service to its owners, and withstand earthquakes with minimum damage. To do less than this would pose an unnecessary hazard to the life and property of the occupants of the building. To do any more would be to impose unnecessary expense on the building owner in particular and society in general. In order to achieve these goals, all aspects of the design and construction must be performed properly. Poor consideration of the structural configuration, design criteria, structural analysis, design and detailing, or construction can cause the project to perform somewhat short of the desired goal.

To this end, the state-of-the-art for the design and construction of mid-rise steel buildings must continue to improve in many areas, including at least the following areas:

1. Develop rational and accurate assessment procedures for determining and communicating the damage potential of mid-rise steel buildings.

2. Refine and redefine, as necessary, the design criteria to accurately relate structural damage during major earthquakes to variable design criteria.

3. Develop minimum standards for structural analysis and computer modeling. These need to relate to and support the minimum lateral force requirements of the building code.

4. Demonstrate through research and testing the integrity of large-scale welded connections and the ability to perform as intended.

5. Develop appropriate modeling techniques and strength estimates for reinforced concrete fill on metal deck diaphragms.

A UNIQUE MECHANISM OF HIGH-RISE RESIDENTIAL BUILDINGS
BY LARGE STEEL STRUCTURAL FRAMEWORK

Toshiharu Hisatoku, Member of JSCA

SUMMARY

A newly modeled high-rise residential building has been developed by adopting a new construction method in which the architectural-planning solutions and structural mechanism intended for the industrialization are systematically integrated. An essential characteristic is a large structural steel framework consisting of stair cores serving as columns and communal floors as girders into which each dwelling unit made of precast concrete panels is incorporated, consequently creating ideal living environment with a high degree of amenities. This paper presents the outline of the structural design of a unique high-rise residential building and the design policy about the connecting joints of the structural elements.

INTRODUCTION

In order to create a good residential environment, residences of good quality must be supplied at stable prices and a comparatively large area of open space be secured. For the former, saving labour at the constructing site is considered an effective means and the prefabrication of residences are getting popularity. If residences are producted in a great quantity by such a method, the effectiveness of the method will increase. For the latter, constructing high-rise residential buildings in order to make the most of the limited land secures an area enough for the improvement of the surroundings around the residential buildings.

One of the characteristic features of the project described in this paper is a unique system in which architectural design requirements to improve the functions of the individual unit of high-rise residential buildings and the surroundings are united organically to the new construction method with large steel structural frameworks. The details of the unique character are summarized as follows:

(1) Residential space available for planning variety,
(2) Communal spaces serving the community,
(3) Residential unit for securing privacy of an essential requirement,
(4) Rational access system to residential units,
(5) Equipments and facilities for preventing or escaping from disasters.

These requirements are fulfilled by several distinctive ideas such as one staircase for every two residential units, communal floors and so on.

It is one of the social needs at present to develop construction methods of industrialized residence production which make possible the quality control and the cost reduction. Building elements in this project are prefabricated and manufactured in factories, which is one of the approaches to the industrialization from the conventional production system. The standardization and mass production of building elements can stabilize quality and cost, save labour and reduce construction time. From this point of view a new method is adopted in this project where the structural framework is separated from the residential units. Therefore, the dimensional changes in the framework members due to the building scale or the member location are absorbed by the structural frame only, and building elements of residential units are standardized as much as possible.

In this project are constructed the residential buildings with a total of some 3,400 dwelling units by several different clients on the reclaimed land off the coast as shown in Fig.1. Table 1 indicates the outline of the buildings which are 14, 19, 24 and 29 stories as illustrated in Fig.2 and have the variations of 11 in the type. Figure 3 shows an example of the plans of the residential units.

OUTLINE OF STRUCTURAL DESIGN

Structural frame

The structural frames of the residential buildings are given in Fig.4. The basic unit for the structural frame is a set of four residential units per floor as shown in the figure. In order to serve the free space for residential unit, the structural frame in the X (ridge) direction consists of two large rigid frames making the core with the stair columns and communal floor girders. In the Y (span) direction the structural frame consists of four rigid joint truss frames situated at the both sides of the staircases. The above-mentioned two frames in the X direction and four frames in the Y direction are designed so as to resist wind and earthquake loads as well as a vertical load whereas the remaining two frames at outer sides of the building in the Y direction are designed to bear a vertical load only. Horizontal braces installed in the communal floor and its upper floor ensure the lateral rigidity of the whole structure.

Structure of residential unit

Figure 5 gives the outline of the structure of the residential unit. The residential unit is composed of PCa (precast concrete) panels and the four-storied residential units lie on the beam located at the upper floor of the communal floor, except the lowest part of the building. The PCa panels composing the floor and the wall of the unit bear the vertical load. The load is transmitted from the floor panels to the wall panels and then the vertical load of the four stories is eventually supported by the beam of the upper floor of the communal floor. These PCa panels are participated against neither wind nor earthquake load.

Relationship between residential unit and structural framework

The characteristic features of the construction method are constituted by structural members, large frameworks and connecting joints. Since the structural framework is separated from the residential units in this construction method, the connection details are most important.

The connections of the PCa panels are largely classified into two, one between the PCa panels and the other between the PCa panel and the structural framework. Each connection is designed in order to fulfill the intended function properly as well as to transmit the working loads safely. The walls and floors of the residential unit are not only required to bear the vertical load but also to comply with the deformation of the structural framework when the horizontal loads are exerted on the structure. The deformation of residential units due to horizontal loads are sketched in Fig.6. The walls in the Y direction are particularly required to shift horizontally as they bear the vertical load and to remain rigid enough to prevent large shearing cracks. For this purpose, tetrafluoro-ethylene resins are placed on the top of the walls of every story to slide the upper floor and wall. The typical connections between the PCa panels and the structural framework are the connections which attach the PCa panel to the frame column and are designed to transmit the horizontal load of the residential unit to the structural framework in every stories preserving the necessary functions.

Substructure

Since the high-rise residential buildings are built on the reclaimed land, the substructure should have enough strength and rigidity. Special attention is called to the following matters as well as the fundamental considerations such as bearing capacity, settlement and so on.

(1) Liquefaction of a sand stratum of the reclaimed land under the earthquake, Improvement of the soil by the filled sand layers right below and around the building,

(2) Horizontal rigidity of the substructure considering the balance with that of the super structure, Foundation of a large diameter of steel pipe pile.

Figure 7 illustrates the outline of the substructure and the super structure in the Y direction.

Seismic design

Seismic design is executed by using the so-called dynamic analytical technique. In order to ensure the safety of the building against earthquakes, the necessary structural performances during appropriate levels of earthquakes are provided first. Then, taking into consideration the type and the scale of the structure, design shearing forces are determined by the dynamic analysis and the members of the tentative structure are designed. The dynamic characteristics of the mathematical model of the tentative structure are evaluated and the responses to earthquakes are analyzed. The responses obtained through this procedure are examined whether they can satisfy the various tolerance values of the structural performances set in the beginning. The members of the tentative structure are, if necessary, modified and the same procedure is repeated. In other words, this is the method for ensuring the structural performances during earthquakes by the so-called feedback loop. Table 2 presents the structural performances the building should have during the two levels of earthquakes. Table 3 gives the natural periods of the four types of the buildings of 14, 19, 24 and 29 stories.

The design base shear coefficient C_B is calculated from

$$C_B = \frac{0.33}{T} \quad \text{and} \quad 0.11 \leqq C_B \leqq 0.33$$

where T is the natural period of the structure. Both C_B and the shear coefficient of the highest story are shown in Table 4.

When designing members of the structural framework in steel, the design policy is determined in detail in order to ensure the structural performances. They are also designed so that they can have a good capacity of restoring force characteristics in both elastic and plastic regions by limiting width thickness ratio of plates, slenderness ratio of braces, axial force ratio to yielding strength of columns and so on. Figure 8 indicates the typical examples of the response analysis results under earthquakes.

DESIGN OF CONNECTIONS

The connections are largely classified into three kinds in this construction method as follows:

(1) Connections between steel members,
(2) Connections between PCa panels,
(3) Connections between PCa panels and steel members.

Connections between steel members are popular, so the other two types of connections are described below.

The residential unit is formed by assembling PCa panels. A PCa panel is connected to another panel at the building site with built-in connectors of each panel. A floor or a wall is formed by assembling several panels and is expected to be connected so that the in-plane rigidity can become as high as possible. The connections between the floor and the wall are also required to be deformable in order that the residential unit can be deformed as shown by Fig.6 under the horizontal loads. Connections between PCa panels are devided into two, the connection J_A between floor panels and the connection J_0 between floor and wall in the Y direction

The construction method which separates residential units from structural frames requires the connection J_1 between them to transfer forces or loads, to comply with deformation, to adjust inaccuracy during the execution of works and so on.

Connection between floor panels J_A

This connection is mainly required to have two kinds of capacity, one having the strength that enables a PCa panel to transmit the horizontal load per residential unit to another PCa panel under the horizontal load and the other having the rigidity enough to unite assembled panels. Each panel is connected to each other by welding the spliced plate to both of the built-in connectors of the panels. This method is often employed in the conventional PCa assembly construction method. In this project, however, stud bolts are used for anchoring the built-in connectors so as to save labour.

Figure 9 illustrates the detail of J_A connection and Fig.10 gives the results of a shearing test in a full-size model. It is clear from this test that the ultimate strength Q_U of in-plane shear force is about the double of the design load Q_D. The in-plane rigidity of the whole assembled panel is also examined to be high enough through the study based on the experimental results.

Connection between floor and wall in the Y direction J_0

The design requirement of the wall is not to develop a big crack under the horizontal load while the resistance against wind and earthquake is not expected. The wall in the Y direction has comparatively high in-plane rigidity and is bearing the vertical load of the residential unit. In order to satisfy the above-mentioned design conditions, the connection J_0 is designed so that the wall panels can comply with deformation of the structural framework in the Y direction as sketched by Fig.6. Every floor is attached to the structural framework by J_1 connections explained later and the wall panels in the Y direction are connected to the floor panels on the same floor so that they can move in a body. Consequently every residential unit can follow the deformation of the structural framework as if it were a drawer of a desk, so the restoring characteristics of the unit agree to those of the structural frame by J_1 connections.

The slide bearing pad of tetrafluoro-ethylene resins is inserted between a floor panel and a lower wall panel in the Y direction as shown in Fig.11 so that the wall can slide supporting a comparatively heavy vertical load. The relationship between the coefficient of friction and the intensity of compressive stress of this slide bearing material is given by Fig.12. This slide bearing material has a characteristic tendency that the higher the intensity of the compressive stress becomes the lower the coefficient of friction becomes. Taking into consideration the relationship of Fig.12 as well as the ultimate compressive strength, the creep characteristics and so on, the allowable compressive stress of the slide bearing pad is decided σ=140kg/cm^2. It is also decided that the coefficient of friction μ=0.1 after additional consideration on the accuracy in the execution of works and the degradation of contact surfaces. Since a high accuracy is required at the horizontal level of the top side of the wall to ensure the function of the slide bearing pad, the wall can be adjusted up or down by the height controlling bolt attached to the bottom side of the wall. After the adjustment some mortar is filled up between the bottom of the wall and the floor.

The durability and the resistance to weather of this kind of a slide bearing pad seem to have been proved sufficiently by the successful use in bridges or pipe lines with more severe conditions of environment. In order to employ this kind of a slide bearing material in this project, several experiments are carried out about the choice of the partner sliding materials and about the sliding efficiency under unfavorable conditions like eccentric loads or inclination of a contact surface, in addition to the basic property tests of the material and the mechanical characteristic tests.

Connection between PCa panel and steel member J_1

Horizontal loads exerted on the residential unit during an earthquake are transmitted to the structural framework through J_1 connections. The movement and restoring characteristics of the residential unit comply with those of the structural framework. Further, the incompatibility of accuracy between the election of the steel framework and the assembly of the residential units is concentrated and adjusted on this connection. In this construction method the connections J_1 are, therefore, most important of all three kinds of connections. A J_1 connection is required to have the following kinds of functions:

(1) Connection between a residential unit and the structural framework,
(2) Compliance with deformation of the structural framework in the X direction,
(3) Adjustment for horizontal accuracy in the execution of works,
(4) Adjustment for vertical accuracy and compliance with deformation.

The shape of a J_1 connection is illustrated in Fig.13. A steel plate with anchors at the concrete of a floor panel and an I-bracket welded to a steel column are connected by two L-spliced plates. The two L-spliced plates are connected to the steel plate with four high-tensile friction bolts and are also attached to the web of the I-bracket with pin bolts.

The horizontal load of the residential unit is eventually transmitted to the structural framework by the shear and tension of the pin bolt as well as by the bearing of the L-spliced plates. The connection should neither slip greatly nor be shaky so that the load of the residential unit can be transmitted smoothly. The shearing test and tension test of the full-size model are carried out so as to confirm the capacity of these kinds. Figure 14 shows the results of the shearing test, which indicates clearly that the ultimate strength Q_U is about the double of the design load Q_D calculated from the maximum response acceleration of the floor through the earthquake response analysis with input acceleration level of $0.3g$. Since any large slip is not observed in the load-deformation curve, the connection has comparatively high rigidity.

Errors or inaccuracies in the execution of works are adjusted when fixing the connection. A horizontal adjustment in the Y direction is made by the oversize hole (±10mm in the Y direction) for the high-tensile bolt holes of the L-spliced plate while it should be made in the X direction at the connection in the outer sides after fixing the connections in the staircases, which coincides with the structural design where elements resisting wind and earthquakes are structurally concentrated on the core part of the staircases. A vertical adjustment is carried out by making the pin bolt hole vertically long (±10mm is included for adjusting).

The different kinds of deformations at the connection after fixing are also absorbed by the rotation or translation of the pin bolt in the oversize hole. Figure 15 shows the pin bolt hole for a vertical adjustment designed by calculating an axial deformation amount of the steel column.

CONCLUSION

The outline of the structural design and the design of connecting media in a new construction method which intends industrializing a production of high-rise residential buildings has been presented. Although the relationship between design details and accuracy in the execution of works or reproducibility is very important in this kind of construction methods, it is often difficult to predict problems in the execution of works when it is at the designing stage. There are also some other problems. Some of them are the comfortableness of the high-rise residential buildings constructed by this new method, highly technical judgement for the substructure on the reclaimed land and so on. By solving these problems through the experiments both on and off the site, the new construction method should be developed.

Since this is a construction method which intends to be clear from unknown design elements, safety for design elements considered are increased. However, additional unforeseen strength of unknown elements is not expected. In this kind of construction methods, therefore, it is particularly important and essential that quality control should be carried out sufficiently, and materials and members of the structural framework should have the capacity expected. This is the reason why quality control is considered important particularly in the building production system with the future tendency in building constructing methods.

ACKNOWLEDGEMENTS

The plan of this project submitted by the ASTM won the first prize in the competition for High-Rise Housing Complex at Ashiyahama in 1973 and the construction was completed in 1979. The name ASTM is the combination of the first letters of Ashiyahama, name of the city where these buildings were built, and the five participating companies in Japan. The success of the project is inevitably dependent on contributions from many people and organizations. The author is grateful particularly to Mr. R. Tamura, Nippon Steel Corporation and Mr. Y. Kato, Takenaka Komuten Co., Ltd. for their excellent contributions. Thanks are also due to the staffs of the ASTM for their elaboration in accomplishing the project.

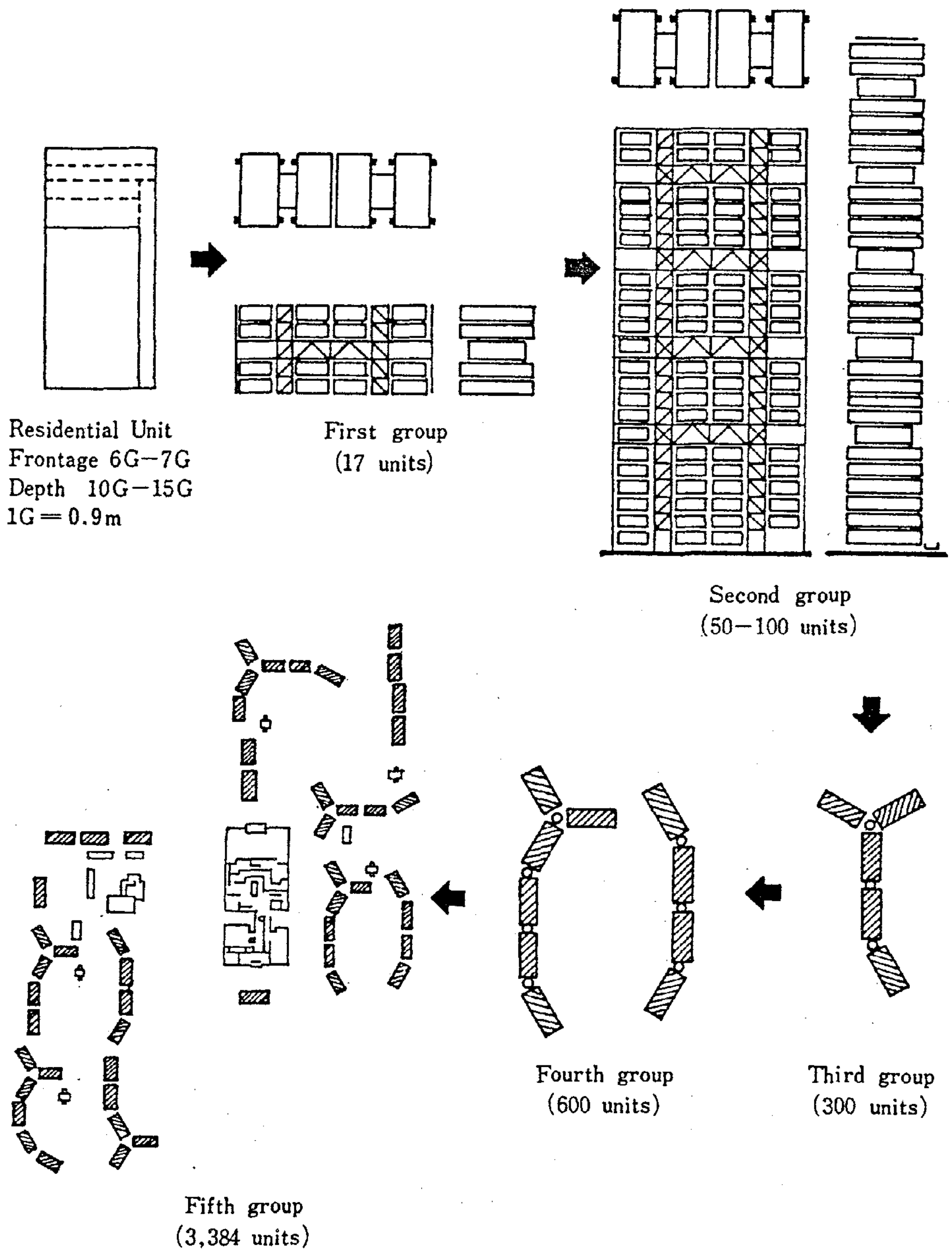


Fig.1 System of High-Rise Residential Buildings

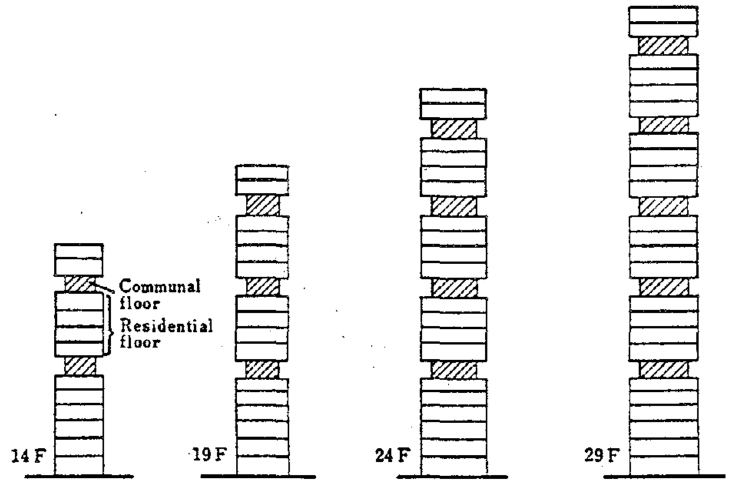
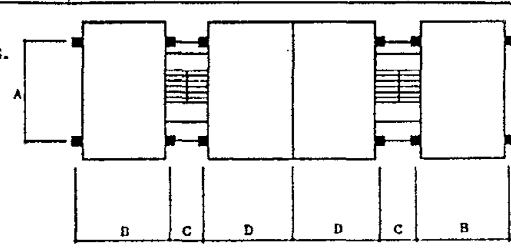


Fig.2 Composition of Stories

Table 1 Outline of Buildings

Owners of buildings		KE	KS		DA			DB		MA	MB		
Standard grid for residential unit (G)		6×10	6×12		6×13			7×13		7×14	7×15		
Area of residential units (m²)		48.6	58.3		63.1			73.7		79.1	85.1		
Number of stories (stories)		14	14	19	14	19	24	19	24	19	24	29	
Height (top of parapet) (m)		40.82	40.82	55.02	40.92	55.02	69.22	55.90	70.33	55.90	70.33	84.76	
Depth of foundation G.L-(m)		3.00	3.00	3.40	3.00	3.40	4.80	4.80	4.80	4.80	4.80	6.00	
Height of a story (m)	Residential floor	2.58						2.63					
	Communal floor	3.23						3.27					
Span (m)	A	7.70	8.60		9.50			9.50		10.20	11.30		
	B	6.250	6.235	6.250	6.240	6.255	6.315	7.205	7.245	7.205	7.245	7.255	
	C	2.450	2.450	2.510	2.450	2.510	2.510	2.950	2.950	2.950	2.950		
	D	5.910	5.910	5.920	5.910	5.920	5.950	6.870	6.870	6.870	6.870	6.870	
Standard floor area (m²)		229.05	268.50		277.59			322.56		345.64	368.73		
Total floor area (m²)		3,206.8	3,759.0	5,101.5	3,886.2	5,274.2	6,662.1	6,128.6	7,741.4	6,567.2	8,849.5	10,693.1	
Number of buildings (buildings)		12	4	6	5	5	5	4	4	2	2	3	52
Number of residential units		596	199	396	249	331	411	266	334	133	168	301	3,384

Note: 1. The standard grid is 1G=0.9m
2. Spans A, B, C and D mean the following lengths.



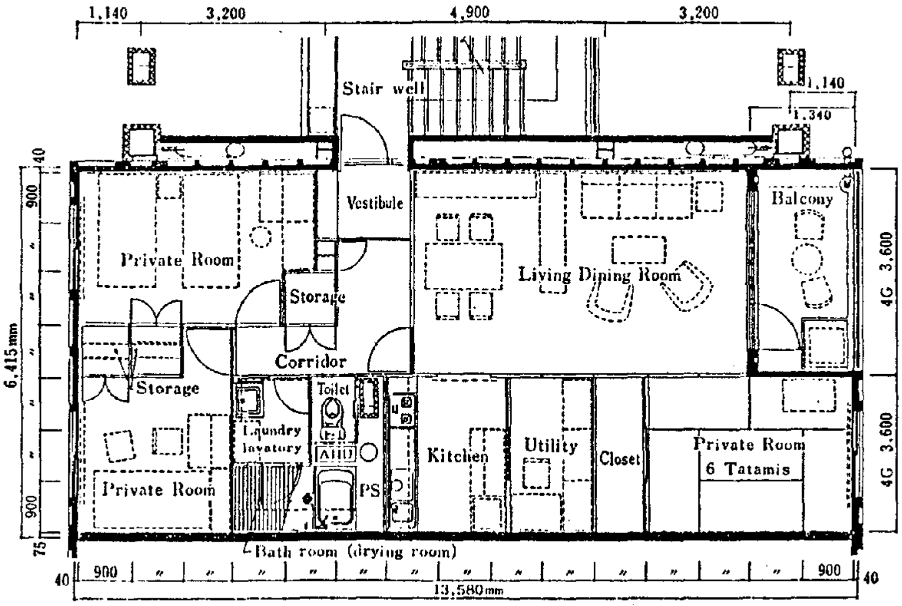


Fig.3 Plan of a Residential Unit

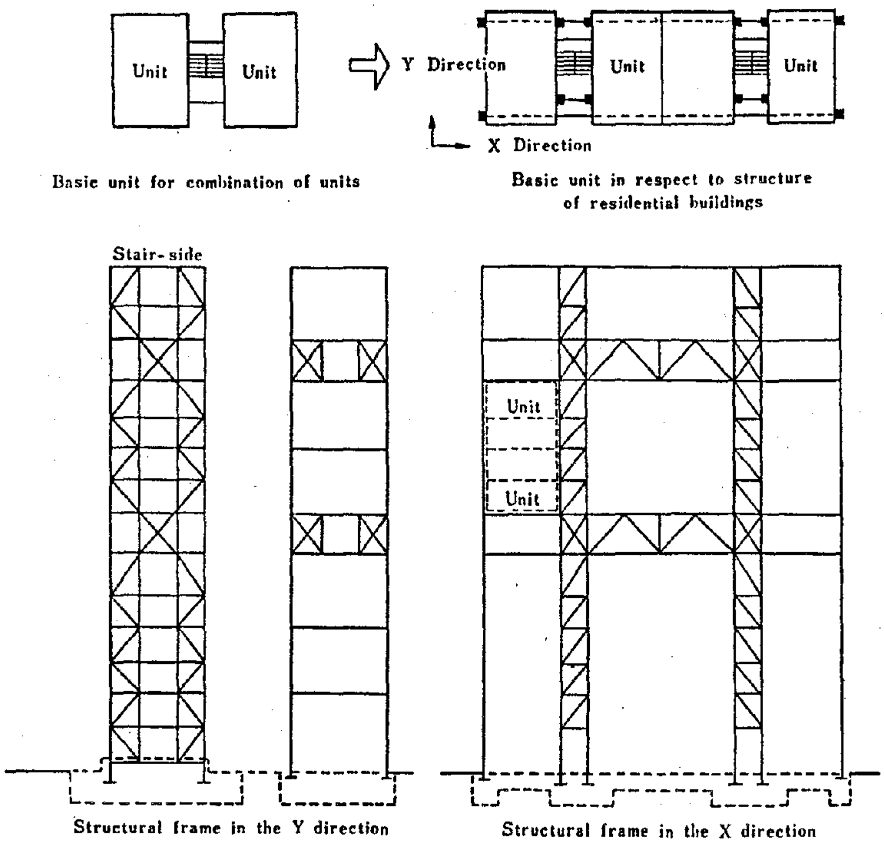


Fig.4 Structural Framework

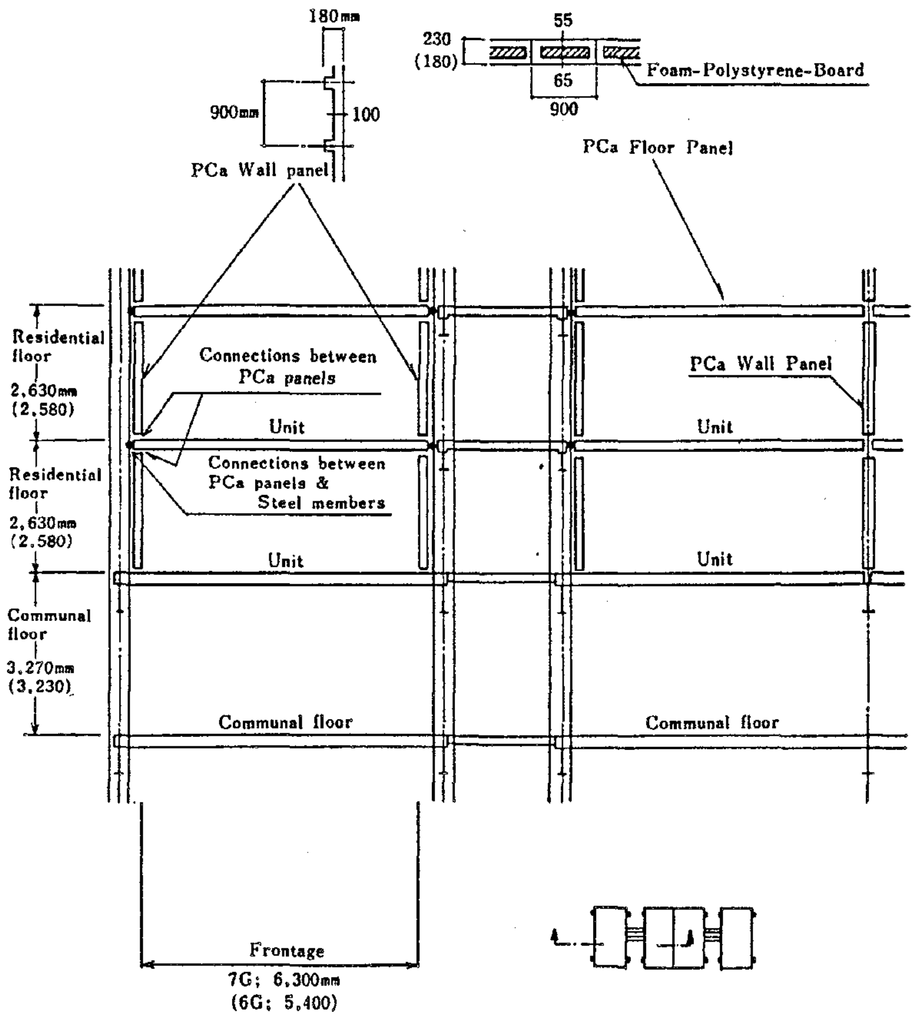


Fig.5 Structure of Residential Units

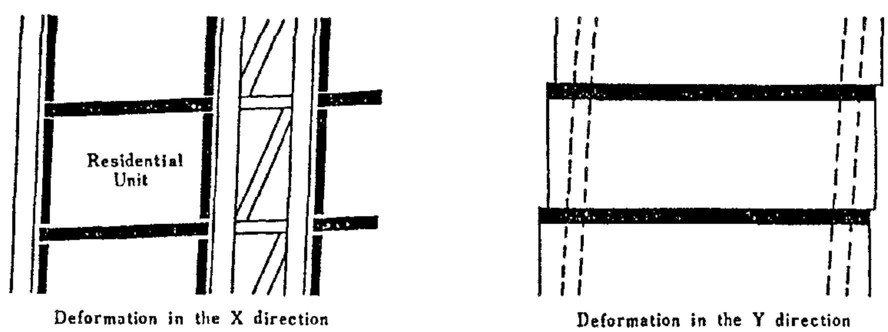


Fig.6 Deformation of Residential Units

Table 2 Criteria for Structural Performance

External Load Conditions			Structures Performance		Hypothetic Frequency
Seismic Force		Wind Pressure	Deformation***	Strength Ductility	
Intensity Scale of Earthquake	Acceleration*				
V Very strong	0.2g	Based on the technical instruction of high-rised buildings published by architectural institute of Japan.	$\leq \frac{1}{150}$ Rad.	Elasticity	Once fifty years
VI Disastrous	0.3g		$\leq \frac{1}{100}$ Rad.	μ=2.0** (Braces do not buckling)	Once a hundred years

Note: * Acceleration on the ground-surface
** μ means ductility
*** The maximum displacement between stories (Indicated by joint translation angle)

Table 3 Natural Periods

(SEC.)

Mode Number \ Building / Direction	KE 14 X	KE 14 Y	KS 19 X	KS 19 Y	DA 24 X	DA 24 Y	MB 29 X	MB 29 Y
1st.	1.41	1.50	1.75	1.97	2.30	2.32	2.66	2.70
2nd.	0.51	0.54	0.64	0.71	0.84	0.85	1.00	1.04
3rd.	0.32	0.50	0.42	0.59	0.56	0.70	0.70	0.83

Table 4 Design Shear Coefficients

Story \ Building / Direction	KE 14 X	KE 14 Y	KS 19 X	KS 19 Y	DA 24 X	DA 24 Y	MB 29 X	MB 29 Y
Highest story	0.520	0.510	0.515	0.555	0.420	0.510	0.285	0.360
1st story	0,236	0.230	0.190	0.172	0.145	0.150	0.126	0.125

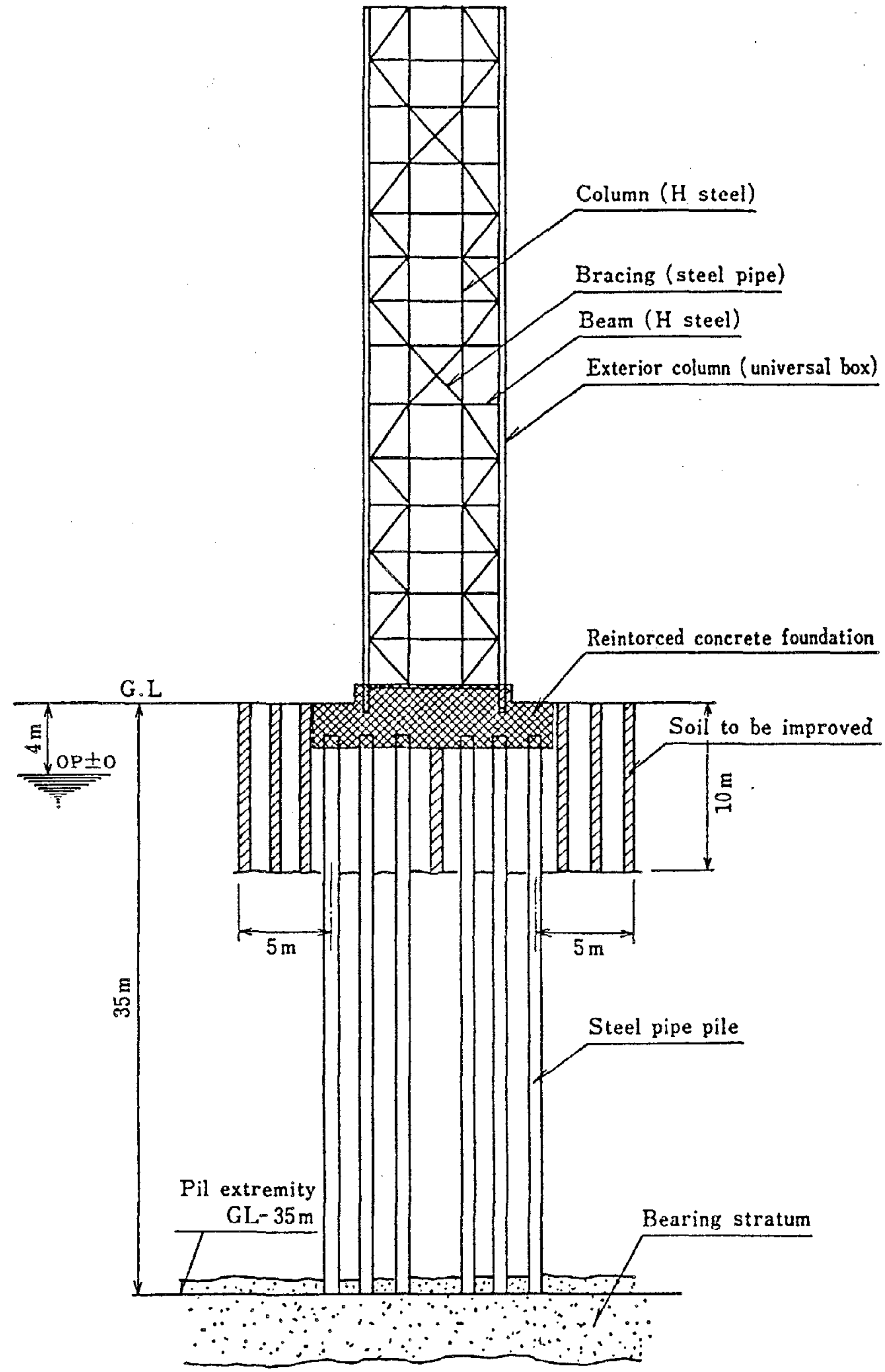


Fig.7 Outline of Structure

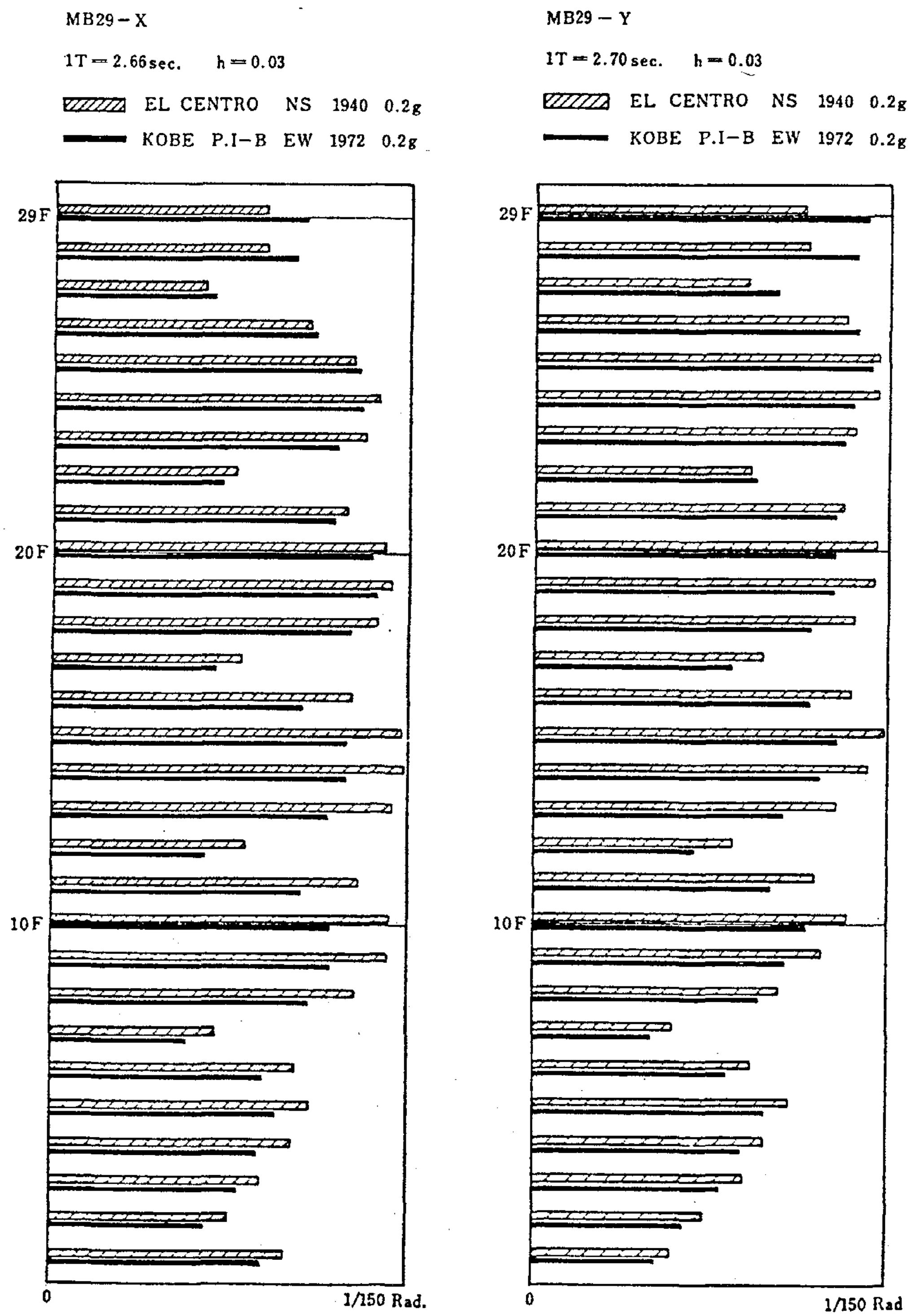


Fig.8 Maximum Response of Story Angle

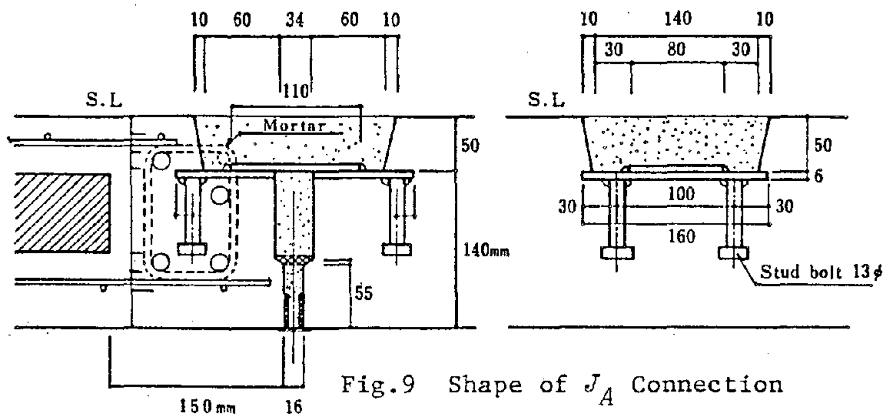


Fig.9 Shape of J_A Connection

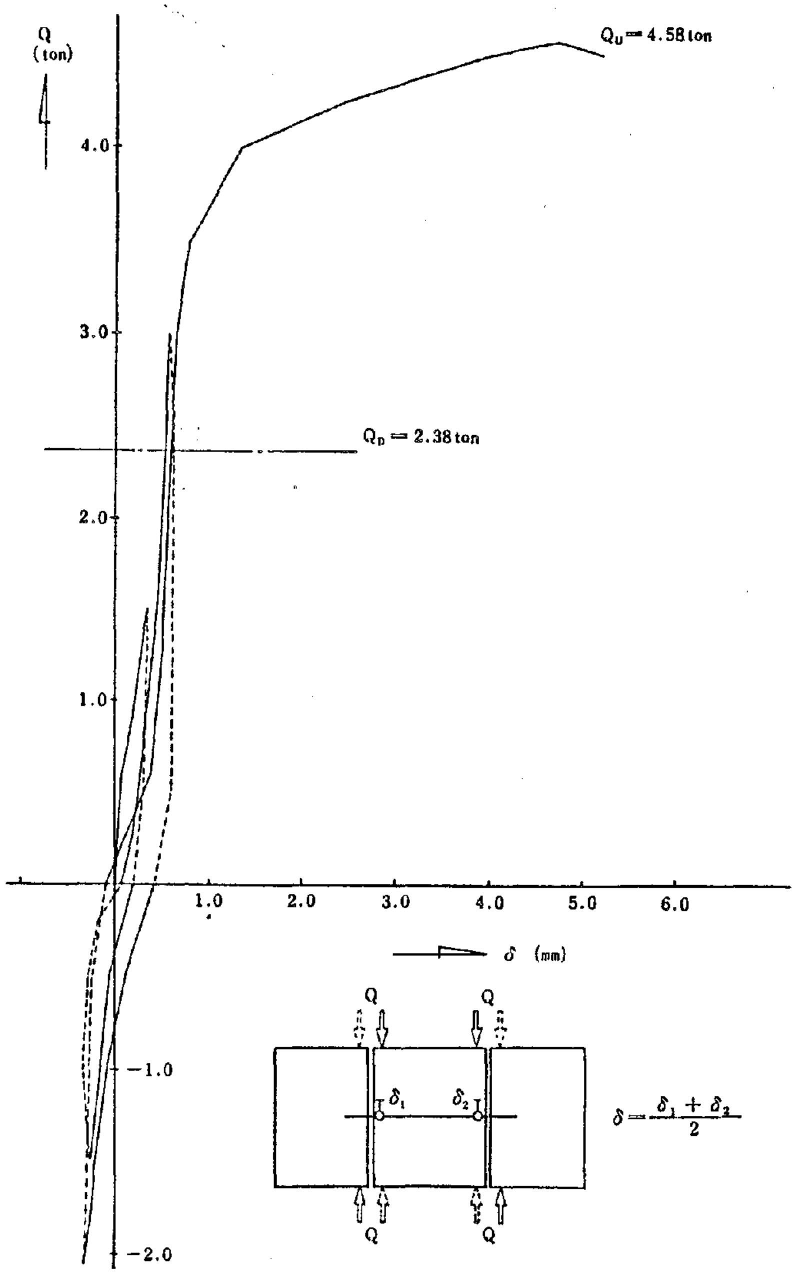


Fig.10 Experimental Result of J_A Connection

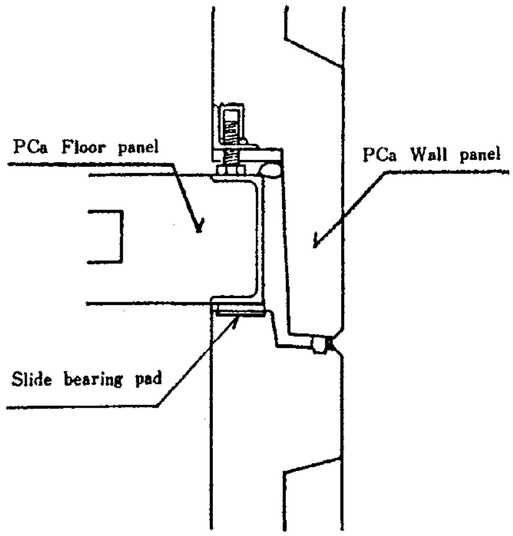


Fig.11 Shape of J_0 Connection

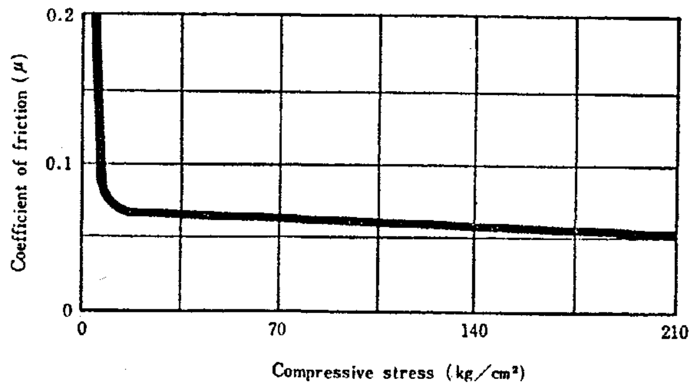


Fig.12
Relationship between Coefficient of Friction and Compressive Stress for Slide Bearing Pad

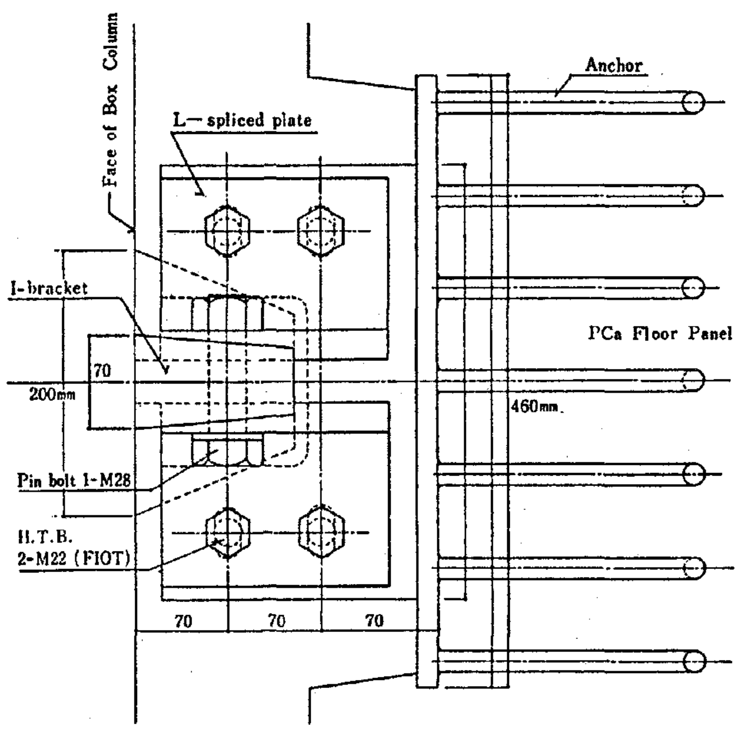


Fig.13 Shape of J_1 Connection

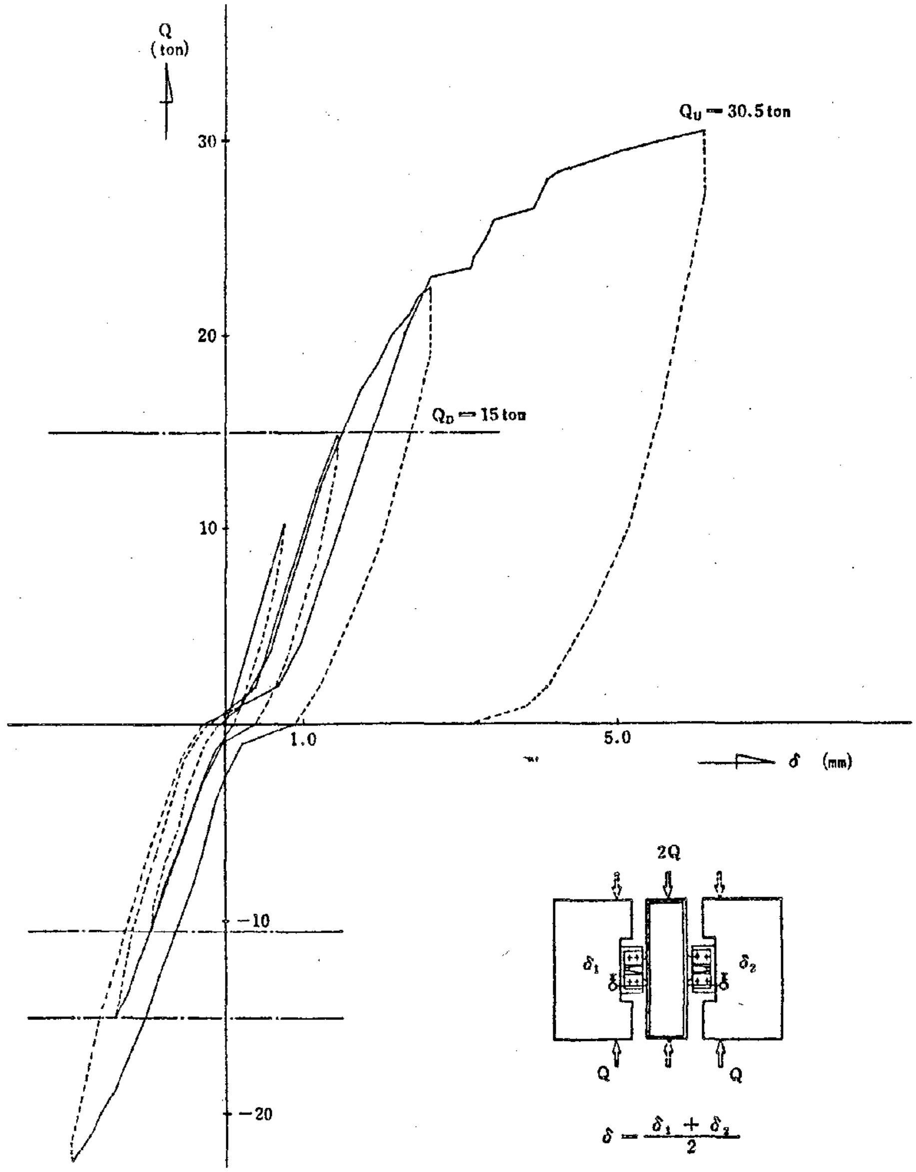


Fig.14 Experimental Result of J_1 Connection

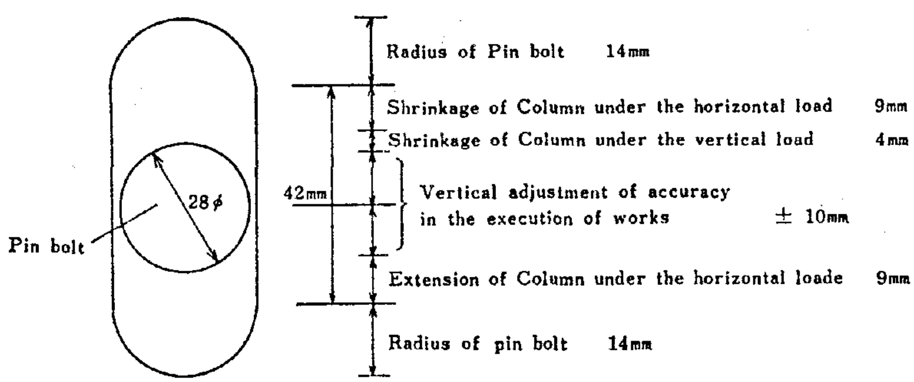


Fig.15 Vertical Adjustment of J_1 Connection

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A MID-RISE COMPOSITE STRUCTURE

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Japan

INTRODUCTION

This paper gives an outline of the structural design of a typical type of Japanese office building. The building we will use as the example is the head office of Tokyo Nissan Motor Sales Company. It is an 11-story building, excluding the penthouse and the basement floor, and it stands on approximately 3,300m^2 of land on the eastern side of Shinagawa railway station in Tokyo. The total floor area is approximately 10,000m^2, and each floor occupies roughly 840m^2.

The floor plan is basically rectangular (38.4m x 19.2m) and contains two side cores. The foundations are of reinforced concrete (RC), the cores and perimeter frame are constructed of steel-encased reinforced concrete (SRC) and the long-span beams across the width of the building are made of steel (S). The floor slabs were constructed on site (cast RC) on a metal framework without shoring. This type of structural composition is the most popular for buildings of this scale in Japan. A summary of the characteristics of the building is given in Table 1; the sectional elevation, the first floor plan and a typical floor plan are shown in Figs. 1, 2 and 3.

DESCRIPTION OF THE STRUCTURE

1. Soil Conditions

The site is on the eastern edge of a land area reclaimed from Tokyo Bay. It is 3.0m above sea level and is located 1.5km from the present shore of Tokyo Bay.

Each layer of earth under the site runs undisturbed in a lateral continuous form. The soil is composed as follows, from order of top to bottom: reclaimed soil, an alluvial silt layer, an alluvial fine sand layer, a diluvial silty layer and a diluvial gravel layer. The bottom of the bore hole contained a layer of Tertiary Mudstone. The depth of the diluvial gravel layer, known as the 'Tokyo gravel layer', was confirmed to a depth of 5m in test bores over the entire area of the site.

Almost all high-rise buildings in Tokyo rest on the Tokyo gravel layer, as does this building. The geological layers are given in Table 2. The boring log and a geological diagram of Tokyo are shown in Figs. 4 and 5.

2. Structural Members

A summary list of structural members is given in Table 3 and the materials used are given in Table 4.

STRUCTURAL DESIGN

1. Design Concept

In the design of office buildings, the planning methods from the architectural, structural and mechanical engineering perspectives differ greatly for head offices and for rental offices. In the case of head office buildings, money is made available for the construction budget in order to ensure that the building is spacious, blends in with the immediate environment and presents a positive image of the company concerned.

On the other hand, one of the most important requirements to be met for rental office buildings is that the planning should respond to the needs of undefined clients. For that purpose, space and performance will be considered for the average tenant's needs and construction costs will be kept down as much as possible to provide office space at a reasonable rent. There is a basic difference, then, between the two types of buildings, even though they are both used as offices.

Since the building we are discussing is a head office building, the floor plan is designed with column-free office space and is of the so-called 'side core' type. The office space is designed to be open and flexible in order to meet any changes in organization within the company.

The cores were placed at the north and south sides of the plan so that the building can command a wide view to the east and the west. The use of long span beams between the north and south cores provides spacious column-free room. To create this space it is necessary not to subject the long span beams to an excessively large seismic force and to determine the distribution ratio for each structural member for a lateral seismic force, while maintaining a balance between permanent and temporary stresses. The cores were designed with steel-encased reinforced concrete (SRC) in an attempt to balance the lateral force distribution.

The perimeter short span beams along the length of the building were constructed of SRC and the layout and thickness of the shear walls around and inside the core were adjusted to reduce horizontal torsional effects and to achieve a symmetrical structure. The shear walls are of the cantilever shear wall type to ensure ductility by flexural yielding (Figs. 7,8,9).

The long span beams were designed as unencased composite beams rigidly connected to SRC columns. The reasons are as follows.

When steel beams and concrete-encased steel (SRC) beams (of the same depth and elastic section modulus) are used for long span beams, as in this case, SRC is not economical because the additional section modulus produced by re-bars is cancelled by the additional weight of the concrete. The shorter depth of the steel beams increases the steel weight by about 15%, and the concrete formwork, re-bars and steel volume naturally increase.

It is common practice in Japan to cut holes in the beams for A/C ducts to reduce story height. Bigger holes can be cut in steel beams than in SRC beams.

Steel beams have the advantage of reducing the dead load, because light buildings are one of the most important design points in earthquake-prone areas.

On the other hand, steel beams suffer problems concerning stiffness, fire protection and stress at the joint of the steel beam and the SRC column. Low stiffness is one disadvantage of long span beams, but sufficient stiffness can be achieved by connecting steel beams to floor slabs. Mineral wool can be sprayed on to provide fire-proofing at a reasonable cost.

The main point of the stress mechanics problem is whether or not the shears and moments introduced in a steel beam can be carried fully to the SRC column. When the section modulus of the steel section in a reinforced concrete column is more than one third of the section modulus of a steel beam, the stress in the beam can be transferred fully to the SRC column, and the beam can achieve ultimate strength by lateral deflection. This is in accordance with reports of various experiments (Figs. 10, 11, 12).

2. Design Route and Design Criteria

a) Design Route

The structural engineer can choose the design route by considering the size, use and the type of building in accordance with a new aseismic design method for buildings in Japan.

The building under consideration here is categorized as a steel-encased reinforced concrete structure. This design was chosen based on the structure's height, the number of stories and the size of the floor area. The design route is indicated by the thick solid line in Fig. 13. This route is the so-called Route 3, which holds the lateral drift caused by a moderate earthquake force to under 1/200 of the story height. The SRC structure was developed in Japan, and is a composite structure with high earthquake resistance. It has the character of both steel and reinforced concrete, combines the ultimate strength of both materials and provides sufficient ductility. Such types of buildings which have experienced earthquakes have been found to suffer little damage. Therefore, SRC frames with proper shear walls will more and more play a leading role as aseismic structures in Japan.

b) Design Criteria

The design criteria of this building were specified as follows:

- The building shall meet the requirements stipulated in the Building Standard Law Enforcement Order, the Notification of the Ministry of Construction and the Specifications of the Architectural Institute of Japan.

- Permanent and temporary stresses at principal points of the structural frame shall be calculated by summing the Dead load, the Live load, the Snow load and the Seismic load. Because the Wind load is smaller than the Seismic load, it is neglected in the design of the principal members, but it is taken into account in the design of the secondary members (Fig. 14).

- The bending moment used for sectional design of a beam or column shall be combined with the permanent stress calculated at the intersection of the frame grid and with the temporary stress at the face of each connected structural member.

- Boundary effects of beams connected to shear walls in the same frame and perpendicular to shear walls shall be calculated.

- One hundred percent of the lateral seismic force on the basement floor, excluding the dead weight of the double slab foundation, shall be considered to act at the top of the piles, since the site is near a canal. The connection between the piles and footing pads shall be pin joints.

3. Structural System

The moment-resisting frames and shear walls are arranged in parallel fashion along the width of the building and serially along its length.

Concerning the direction along the width of the building, the core is divided into three bays of 6.4m width and the office space between the cores is supported by eight frames along the 19.2m span single bay. Multi-storied reinforced concrete shear walls are arranged at Y_1-Y_2 and Y_3-Y_4 on the grid lines X_1, X_2 and X_{11}, and at Y_1-Y_3 on X_{12} (Figs. 7,8,9). The distribution coefficient of lateral force of shear walls increases from top to bottom, caused by bending deflection. The percentage of lateral force borne by the shear walls is approximately 60% at the top and 80% at the bottom.

Concerning the direction along the length of the building, the cores are 4.8m wide and there are 9 bays of 3.2m width. The main aseismic structure consists of the Y_1 and Y_4 frames. Multistoried reinforced concrete shear walls are arranged at the core on the Y_1 and Y_2 grid lines. The percentage of lateral force borne by the shear walls is approximately 20% at the top and 60% at the bottom.

Disturbance due to floor vibration was checked along the composite beams of 19.2m width and it was confirmed as being within reasonable limits. The static deflection was calculated by dropping a weight of 6kg freely from a 5cm height, which was assumed to be equivalent to the walking of two people. The natural frequency and amplitude were also calculated along the composite steel beams with an effective slab width of 267cm.

The amount of disturbance was judged based on a sensitivity curve (Fig. 14). The natural frequency (f) was calculated using Eq. (1). The deflection δ_L (cm) at the middle of the beam was induced by a vertical load.

$$1/f = T = 0.175\sqrt{\delta_L} \qquad (1)$$

The amplitude δ_d was calculated by means of Eq. (2):

$$\delta_d = \delta_{st} \cdot \left(1 + \sqrt{1+(2H/\delta_{st}) \cdot (W/(W + kW_1))}\right)$$

where

δ_{st} : static deflection produced by W (cm)
h : height of free drop (cm)
W_1 : dead load of slab and beam (kg)

W : weight of dropped load (kg)

k : coefficient of end restriction of beam
fixed end = 13/35; simple beam = 17/35

The calculation was done both with and without a live load. The results are shown in Fig. 15 and it is seen that a disturbance from vibration will not occur under normal usage.

4. Design of Foundation

Because this building has a one-story basement and the Tokyo gravel layer runs for a depth of 14m under the grade of the building, a piling foundation was used. Cast-on-site concrete piles were adopted because of their greater bearing capacity to a lateral seismic force.

The pitch of the piles shall be more than two time the piles' diameter, assuming the bearing capacity is resisted by the combination of friction and end bearing. For the piles on the Y_1 and Y_4 grid lines, the pitch could not be made with sufficient spacing, as the 3.2m span is too short. Therefore, the stem of the caissons was made smaller and the bearing capacity given by the end bearing must be adopted there. The bearing pressure of the permanent force of the caisson is 50 t/m^2 at the end.

Dewatering by deep wells was required to excavate for the caissons. The water table of upper silt is approximately 1.5m under the grade and the head of water of the Tokyo gravel layer is approximately 7.5m above this layer. Since the water table is rather high, the foundation was designed by the so-called double slab foundation method and the bottom slab was verified as to its resistance to the uplift pressure of the ground water. Space between the two slabs is used for a water tank, sewerage tank and other.

STRESS ANALYSIS AND STRENGTH OF THE STRUCTURE

1. Load

a) Live Load

Normal-weight concrete was used in all stories. The live load for this building, shown in Table 5, was the same as the normal design load for general office buildings.

b) Lateral Force

i) The lateral seismic coefficient was calculated by Eq. 3 in accordance with the New Aseimic Design Code (1981),

$$C_i = Z \cdot R_t \cdot A_i \cdot C_o \qquad (3)$$

where:

C_i : lateral seismic shear coefficient of the i'th story
Z : zoning coefficient
R_t : design spectral coefficient
A_i : the lateral shear distribution factor
$\overline{W}$: weight above the i'th story

W_n : weight above ground level
T_c : fundamental natural period of the soil
T : fundamental natural period of the building

$$R_t = 1 \quad \text{for } T < T_c$$
$$R_t = 1-0.2\,(T/T_c-1)^2 \quad \text{for } T_c \leq T < 2T_c$$
$$R_t = 1.6T_c/T \quad \text{for } 2T_c \leq T$$
$$A_i = 1+(1/\sqrt{\alpha_i}-\alpha_i)\cdot(2T/(1+3T))$$
$$\alpha_i = \overline{W}/W_n$$

ii) The wind load was calculated by Eq. 4, in accordance with the AIJ Standard for Loads on Buildings (1981), but for the wider external area that is along the width of the building.

$$P = q\cdot C_f\cdot G_f\cdot A \tag{4}$$

where:
q : wind pressure
C_f : wind force coefficient
G_f : gust effect coefficient
A : projected area

A comparison of the seismic and wind shear forces is shown in Fig. 20. Vertical loads were calculated for each column of each story in order to check the torsional force in the stress analysis by computer.

2. Method of Elastic Stress Analysis

The stress analysis was done by using the following model:

- Springs were applied at the base of the building, taking into consideration the vertical stiffness of the piles.
- The slabs were assumed to be completely rigid. The lateral seismic force was assumed to act at the center of gravity of the building and the horizontal torsion was calculated.
- In the cores, because the stress acts on the walls in the direction perpendicular to the seismic force, a three dimensional analysis was done taking into account the stiffness of the walls.
- For shear walls, the approximation of decreased stiffness was taken to be half of the elastic modulus.

3. Story Drift, Eccentricity and Stiffness

The inter-story deflection caused by the design seismic force is less than 1/200 of the story height, as given in Table 6.

The variation of lateral stiffness (R_s) and the eccentricity of stiffness (R_e), which are necessary for estimating the required lateral shear strength, were calculated as given in Tables 7 and 8. The quantity R_s indicates the distribution of stiffness of all stories. All R_s values are greater than 0.6 for this building. The quantity R_e denotes the degree of eccentricity between the center of rigidity and center of gravity. All R_e values are less than 0.15, except in the direction of the X-frame on the 11th story.

4. Elastic Limit Strength

The strength of the structure in the elastic range, which is not shown in the structural design flow, was also evaluated. The elastic limit strength is the seismic shear force at the moment any structural member of a particular story reaches the temporary allowable stress, under the assumption that the seismic shear increases in proportion to the increases in the design load. Although some structural members reach their elastic limit, there is still reserve strength in some stories, so the effects on other stories can be neglected. Therefore, the elastic limit strength can be calculated for each story.

a) Elastic Limit Strength of Structural Members

The coefficient of elastic limit strength (α_y) is calculated as the ratio of the stress induced by the design seismic shear force and the temporary allowable stress of the structural member.

i) Columns: On an M-N graph (Fig. 21), the permanent load condition (M_1, N_1) is described as Point A, and the temporary load condition, a combination of permanent and seismic loads $(M_1 + M_e,\ N_1 + N_e)$, is described as Point B. The Point C is the cross point of the intersection of a straight line through A and B and the allowable strength curve. Therefore, α_y is defined as the ratio of the length AC/AB.

ii) Beams: The quantity α_y for beams is calculated using

$$\alpha_y = (M_y - M_1)/M_e \tag{5}$$

iii) Shear Walls: The quantity α_y for shear wall is calculated using

$$\alpha_y = Q_y/Q_e \tag{6}$$

iv) Braces: A wide flange brace is adopted only on the top story. The quantity α_y is calculated using

$$\alpha_y = N_y/N_e \tag{7}$$

b) Coefficient of the Elastic Limit of a Story, $\mathrm{Min}(\alpha_y)$, and Elastic Limit Strength

The coefficient of elastic limit strength of a particular story is calculated as the smaller value obtained from i) or ii) below, as follows:

i) Minimum value of α_y of a column, shear wall or brace of the particular story.

ii) Minimum mean value of α_y of the ceiling and floor beams of the particular story.

The elastic limit strength of a particular story is calculated by multiplying $\mathrm{Min}(\alpha_y)$ by the design seismic shear force.

5. Ultimate Lateral Shear Strength and That Required

The ultimate lateral shear strength in the final state of the structure was compared with the required maximum lateral shear strength as determined by law from many factors, such as the distribution of stiffness, and so on.

The temporary allowable stress is used to calculate the ultimate strength of the structural members. The ultimate lateral shear strength of each frame is calculated by the following method; that of the whole building is assumed to be the sum of the individual frame strengths.

a) Frames

The end of a beam or column is assumed to reach its maximum bending strength. Also, the end of a boundary beam on a multistoried shear wall is assumed to form a plastic hinge. The calculation procedure is as follows:

i) The bending strength of beams is calculated neglecting the floor slab working with them. The bending strength of columns is calculated according to the axial force induced by the shear wall when all connected beam ends are assumed to form plastic hinges.

ii) The sum of bending moments on the left and right sides of beams and at the top and bottom of columns are compared at a connection and the smaller moment is selected as the maximum moment of that connection.

iii) When a column-hinge collapse condition forms, a redistribution of the stress is carried out. The ultimate strength is calculated by repeating the above procedure. n this case, the beam-hinge collapse condition is formed for all major frames except for the top of the columns on the top story.

b) Shear Walls

When comparing the shear strength, bending strength and the limit of uplifting of the foundation in the state of all boundary beams forming plastic hinges, the smallest strength is selected as the maximum.

c) Braces

All braces are assumed to reach their ultimate strength.

The required maximum lateral shear strength is calculated by

$$Q_{un} = D_s \cdot F_{es} \cdot Q_{ud} \tag{8}$$

where:

D_s : structural coefficient
F_{es} : shape factor

Q_{ud} : lateral seismic shear for a severe earthquake which has the same distribution as the design seismic shear and is calculated by taking $C_o = 1.0$

As seen in Table 9, the ultimate lateral shear strength (Q_u) is greater than that required (Q_{un}).

RESTORING FORCE OF STRUCTURE

Similar to an elasto-plastic response analysis, the characteristics of the restoring force of the structure were determined by the following method. The lateral force is increased gradually, while maintaining the same distribution of the design lateral shear force, and a redistribution of the stress is allowed to occur at each yield hinge formed at all structural members until reaching the collapse condition. The stiffness of the structure is reduced step by step in the process of the stress redistribution. Here the restoring force is treated as a Tri-linear Model which is composed of primary and secondary stiffnesses and zero spring constant when the collapse condition is reached. The primary stiffness is calculated based on a static elastic analysis. The secondary stiffness is calculated based on the fact that yield hinges are formed at major boundary beams of shear walls and the ratio of reduced shear stiffness of shear walls becomes 0.3. The first folding point of the model is determined at the elastic limit shear strength and the second folding point is determined at the ultimate lateral shear strength.

DYNAMIC ANALYSIS

1. Method of Analysis

a) Elastic Response Analysis

The model consists of 12 masses with a bending and shearing deflection system fixed on the B_1 floor. The natural periods and vibration modes are calculated as the eigenvalues of the full stiffness matrix of the whole structure. Then a numerical integration response analysis was done.

We inspected the 1st to 5th natural modes. Because the torsional movement, which induces eccentricity between the center of the lateral force and the center of rigidity, was negligibly small, the response analysis was done for the two principal directions individually. The natural periods and vibrational modes are shown in Fig. 22.

The damping ratio was taken to be 0.05 for the first vibrational mode and was increased in proportion to the natural frequency for the other vibrational modes.

b) Elasto-plastic Response Analysis

The model consists of 11 masses with an equivalent shear deflection system of the tri-linear type fixed on the 1st floor, since the basement story has enough rigidity and strength in the plastic range. The restoring force of the building was discussed in Section 4 of the previous chapter. The value of the damping ratio was taken to be 0.05.

Five earthquakes were used and the maximum acceleration was 150 gal for the elastic response and 300 gal for the elasto-plastic response. The earthquakes were: El Centro (1940) NS; El Centro (1940) EW; Taft (1952) NS; Taft (1952) EW; Hachinohe (1968) NS.

2. Evaluation of Response

a) Elastic Response (Fig. 23)

The maximum response shear force was found to be less than the elastic limit strength in the directions of both the X frames and Y frames. The maximum inter-story deflection was 0.46 cm on the 6th story in the X-frame direction and 0.76 cm on the 4th story in the Y-frame direction. The drift ratios were 1/798 and 1/501 in the X-frame and Y-frame directions, respectively, which are much less than the design criteria of 1/200 of the story height.

b) Elasto-plastic Response (Figs. 24,25,26)

The maximum response shear force was less than the maximum shear force strength in the X-frame direction and it appeared to reach the maximum strength between the 1st and 6th stories in the Y-frame direction.

The maximum inter-story deflection was 1.07 cm on the 7th story in the X-frame direction and 2.05 cm on the 1st story in the Y-frame direction. The maximum drifts were 1/338 and 1/194 (3rd story) in the two directions, respectively, which are much less than the design criteria of 1/120 of the story height. The maximum ductility factor was 2.19 on the 7th story in the X-frame direction and 2.65 on the 3rd story in the Y-frame direction.

Although the maximum shear strength was reached in the Y-frame direction, serious problems are not foreseen 1) because the maximum strength of the structural members is calculated using the temporary allowable stress; therefore, the reserve strength until real failure remains, and 2) because the maximum response drift ratios do not attain large values.

CONCLUSION

As shown in Fig. 27, the design seismic load based on the new aseismic design method appears to be reasonable because the story shear coefficient calculated by using the lateral shear distribution factor, A_i, is almost the same as that resulting from a dynamic response analysis. The required ultimate lateral shear strength obtained with the method also appears to be reasonable because of the good results from the elasto-plastic response analysis. Therefore, buildings of this type meeting the new aseismic design code are structurally safe.

For this type of building, inter-story deflection, which is closely related to the performance of non-structural members, does not attain large values, as demonstrated in the results of a 300 gal elasto-plastic response, and damage to non-structural members will therefore not be serious. As a future step, a response analysis considering the interaction of building, soil and pile will be implemented and fed back into the design process.

Table 1 Outline of the project

Name of Project	Minato Office Building
Owner	Tokyo Nissan Motor Sales Company
Site	1-5-8 Kounan, Minato-ku, Tokyo, JAPAN
Use	Office
Design, Supervision	Nihon Architects, Engineers & Consultants, Inc.
Contractor	Joint Venture of Kajima Construction Company and Nitto Construction Company
Site Area	3,306.0 m^2
Building Area	842.3 m^2
Total Floor Area	10,105.4 m^2
Typical Floor Area	796.8 m^2
Story	11 Stories, 1 Basement
Height	Eave Height 42.93 m; Maximum Height 48.00 m
Story Height of Typical Floor	3.63 m
Ceiling Height of Typical Floor	2.50 m
Depth of Excavation	G.L. - 6.0 m
Actual Length of Pile	9.0 m

Table 2 Geological layer

Epoch		Layer		N-Value	Depth (m)
Quaternary	Alluvial	F	Reclaimed soil layer	1 - 3	1.5 - 2.5
		AC	Silt layer	0 - 5	3.0 - 4.5
		AS	Fine sand layer	-	6.25
	Diluvial	DC_1	Silt with gravel layer	6 - 21	4.6 - 4.7
		DSF	Silty fine sand layer	13 - 34	1.8 - 2.1
		DC_2	Silt with gravel layer	9 - 16	1.0 - 1.65
		DG	Gravel layer	50 <	5.0 - 5.6
		DS	Fine sand layer	50 <	0 - 0.6
Tertiary		T	Mudstone layer	50 <	5.5 <

Table 3 Outline of structural members

Pile	Cast-on-site pile (caisson)
Foundation	Reinforced concrete footing
Frame	Steel reinforced concrete Some parts of basement floor are reinforced concrete
Aseismatic element	Reinforced concrete shear wall at core part
Slab	Core: Normal reinforced concrete Office: Reinforced concrete on keystone plate form work
Column (Super-structure)	Steel reinforced concrete using several sectional types of steel members; built-up cross, double H, T and L
Beam (Super-structure)	Core and shorter spans; Steel reinforced concrete using built-up H shape steel Longer spans; rolled H shape (wide flange) steel composed with floor slab
Fire protection	Sprayed rock wool and formed silicic acid calcium board
Exterior wall	Aluminum framed curtain wall precast concrete curtain wall

Table 4 Materials

Steel	Columns, Girders -- SM50A Beams, Lateral suports -- SS41
High Strength Bolts	Special High Strength Bolts -- F10T; M16, M20, M22
Bars	D16 or smaller; SD30, Lap Joints D19 or greater; SD35, Press Weld Joints
Studs	JIS B 1198 Standard material
Concrete	Structure: Normal Concrete F_c = 210 kg/cm^2 Piles: Normal Concrete F_c = 240 kg/cm^2

Table 5 Main live loads

kg/m²

	For slab	For frame (vertical loading)	For frame (seismic loading)
Roof	180	130	60
Office	300	180	80
Computer room	500	300	150
Inside core (generally)	300	240	120

Table 6 Story drift and drift ratio according to a static analysis

Story	Story height (m)	X-frame direction		Y-frame direction	
		Story drift (cm)	Drift ratio	Story drift (cm)	Drift ratio
11	5.00	0.401	1/1246	0.405	1/1235
10	3.68	0.369	1/ 998	0.448	1/ 821
9	3.63	0.393	1/ 923	0.505	1/ 718
8	3.63	0.414	1/ 877	0.543	1/ 668
7	3.63	0.422	1/ 859	0.574	1/ 632
6	3.63	0.428	1/ 847	0.585	1/ 620
5	3.63	0.426	1/ 852	0.595	1/ 610
4	3.78	0.423	1/ 893	0.615	1/ 615
3	3.78	0.398	1/ 950	0.595	1/ 635
2	3.63	0.345	1/1050	0.521	1/ 696
1	4.68	0.374	1/1252	0.608	1/ 769

Table 7 Distribution of lateral stiffness

Story	Story height (m)	X-frame direction		Y-frame direction	
		$r_s (\times 10^2)$	Rs	$r_s (\times 10^2)$	Rs
11	5.00	12.46	1.275	12.35	1.694
10	3.68	9.98	1.021	8.21	1.126
9	3.63	9.23	0.945	7.18	0.985
8	3.63	8.77	0.898	6.68	0.916
7	3.63	8.59	0.879	6.32	0.867
6	3.63	8.47	0.867	6.20	0.850
5	3.63	8.52	0.872	6.10	0.837
4	3.78	8.93	0.914	6.15	0.843
3	3.78	9.50	0.972	6.35	0.871
2	3.63	10.50	1.075	6.96	0.955
1	4.68	12.52	1.281	7.69	1.055
		$\overline{r_s}$= 9.77		$\overline{r_s}$= 9.29	

Table 8 Eccentricity ratio of rigidity

Story	X-frame direction					Y-frame direction				
	Center of mass (m)	Center of stiffness (m)	Eccentricity (m)	Elastic radious (m)	Eccentricity ratio	Center of mass (m)	Center of stiffness (m)	Eccentricity (m)	Elastic radious (m)	Eccentricity ratio
11	18.45	4.11	14.34	52.68	0.272	9.37	5.64	3.73	32.86	0.114
10	18.71	20.25	1.54	24.88	0.062	9.43	8.50	0.94	21.69	0.043
9	18.81	21.03	2.23	23.43	0.095	9.46	8.67	0.79	21.23	0.037
8	18.86	21.25	2.39	22.67	0.015	9.47	8.69	0.78	21.53	0.036
7	18.90	21.43	2.53	22.21	0.114	9.48	8.74	0.74	21.50	0.035
6	18.93	21.41	2.49	21.54	0.115	9.49	8.70	0.79	22.16	0.036
5	18.59	21.26	2.31	20.67	0.112	9.49	8.70	0.79	22.48	0.035
4	18.96	21.22	2.26	20.39	0.111	9.50	8.63	0.87	23.73	0.037
3	18.97	20.77	1.80	18.78	0.096	9.49	8.67	0.83	23.96	0.035
2	18.98	20.92	1.94	19.33	0.101	9.50	8.85	0.64	22.66	0.028
1	18.99	20.45	1.46	17.88	0.082	9.50	9.02	0.47	24.09	0.020

Table 9 Ultimate lateral shear strength and that required

(ton)

Story	X-frame direction		Y-frame direction	
	Qun	Qu	Qun	Qu
11	1343.0	1884.5	795.4	2303.6
10	1673.0	2830.4	1374.2	3534.2
9	2058.4	3503.0	1715.4	3480.7
8	2398.6	3863.3	2398.6	3664.9
7	2700.3	4195.2	2700.3	3840.4
6	2970.6	4497.7	2970.6	3999.5
5	3738.4	4785.6	3204.3	4165.8
4	3977.3	4953.6	3409.1	4116.5
3	4183.5	5179.2	3585.9	4127.1
2	4361.0	5552.2	3738.0	4582.2
1	4491.0	5270.5	3849.4	4428.4

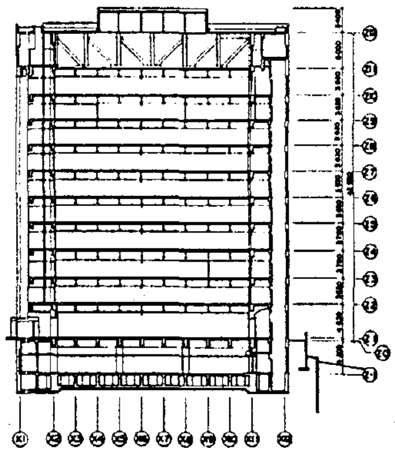


Fig. 1 Section

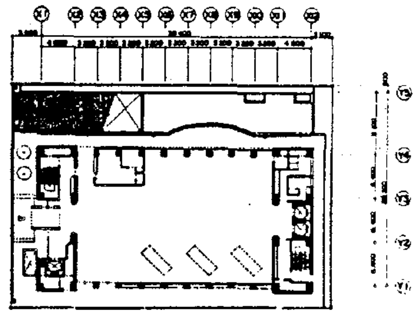


Fig. 2 Plan of 1st floor

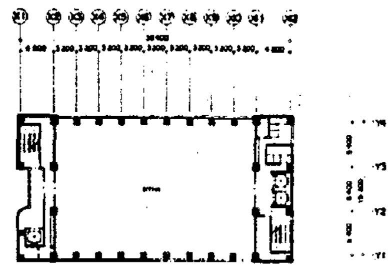


Fig. 3 Typical floor plan

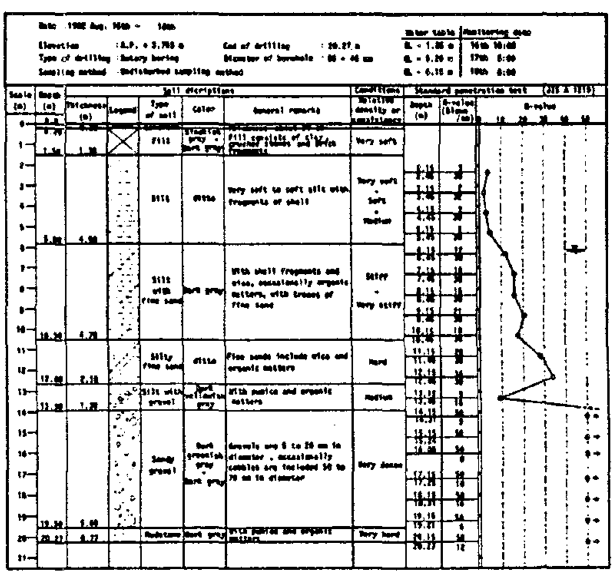


Fig. 4 Boring log

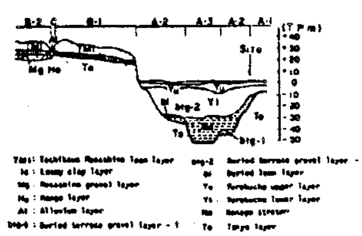


Fig. 5 Geological illustration of Tokyo

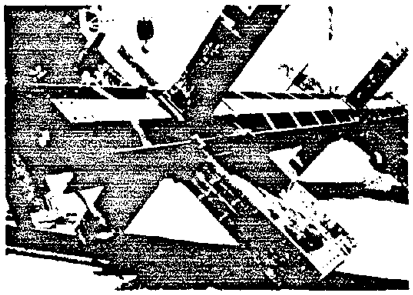


Fig. 6 Fabrication of a cross-section column with bracket beams (Detail shows cross-sectional steel column with bracket beam, which is the most popular detail of steel reinforced concrete in Japan)

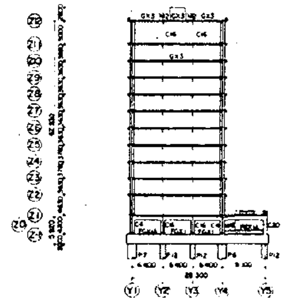


Fig. 8 Framing elevation of a typical along-width frame

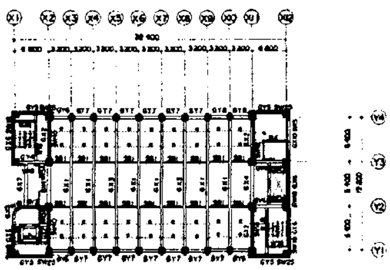


Fig. 7 Framing plan of a typical floor

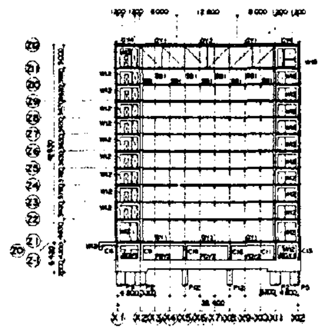


Fig. 9 Framing elevation of a typical along-length frame

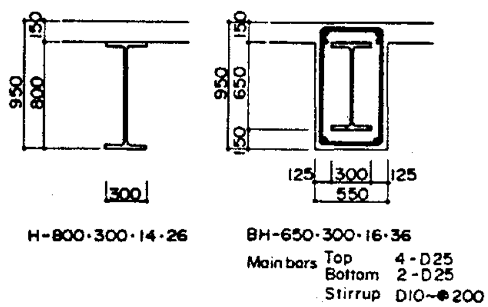


Fig. 10 Steel and SRC beams which have the same section modulus and the same depth

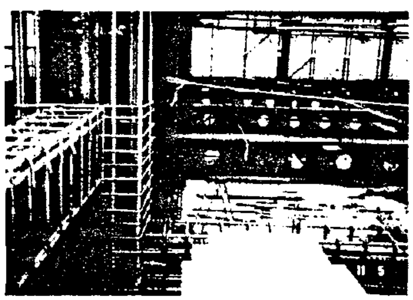


Fig. 11 Example of a building with long-span steel beams, SRC columns, and short-span SRC beams (The steel beams have openings for ducts and pipes)

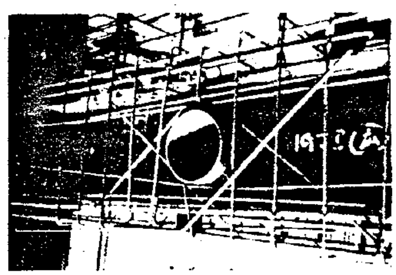


Fig. 12 Example of an SRC beam with sufficient reinforcement against web openings

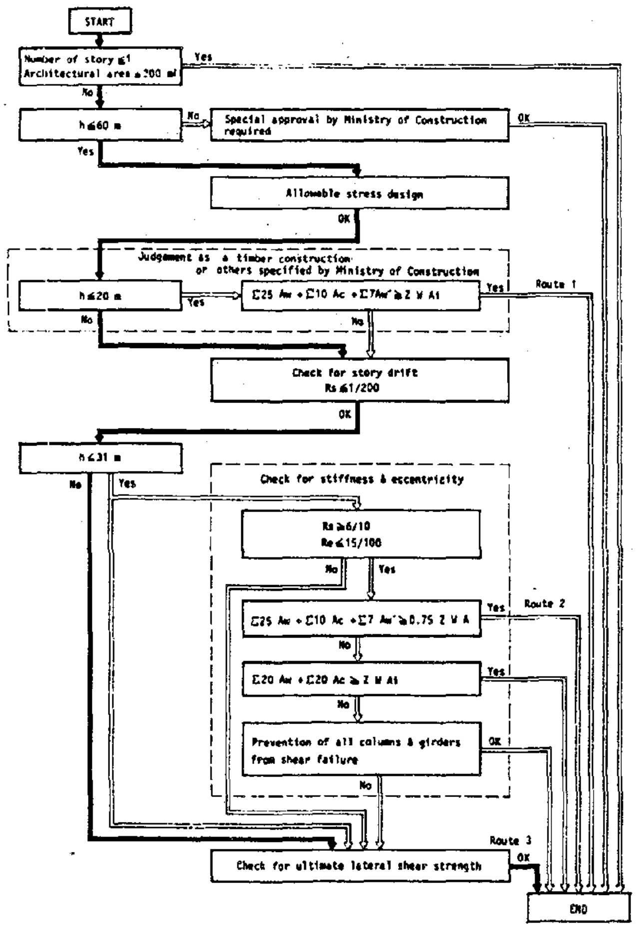


Fig. 13 Flow of structural design for SRC buildings

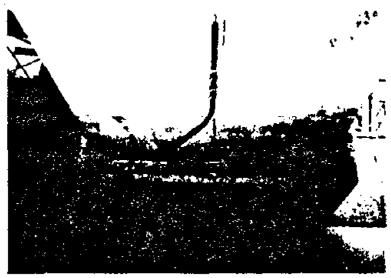


Fig. 14 Details for secondary structural elements
Stone veneer is set to conform to the story drift)

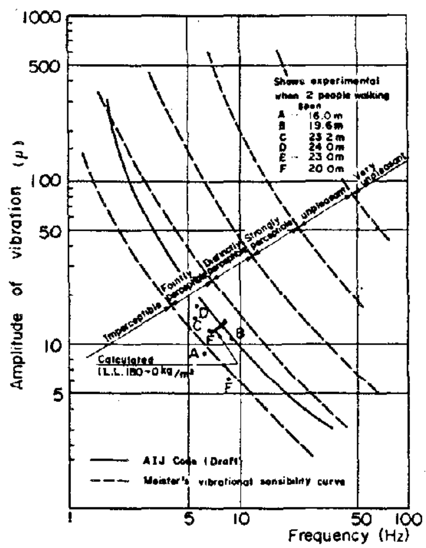


Fig. 15 Sensibility curve

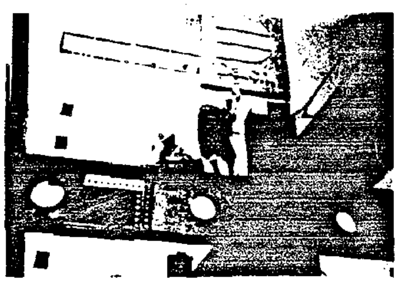


Fig. 16 Erection of steel structures

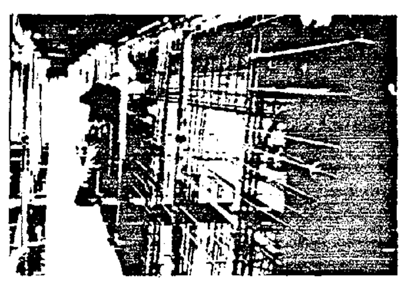


Fig. 17 Fabrication of bars of basement wall

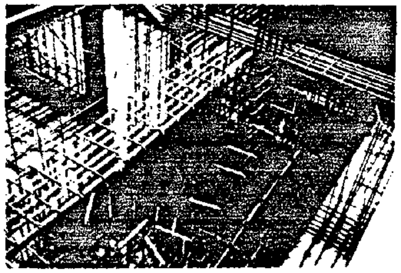


Fig. 18 Fabrication of bars of foundation and foundation beam

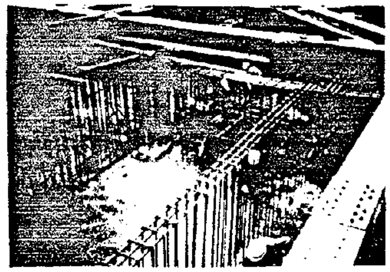


Fig. 19 Placing concrete in foundation slabs

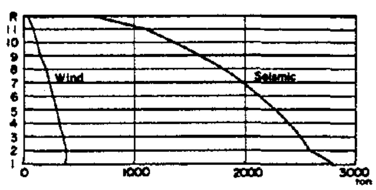


Fig. 20 Comparison of seismic and wind shear force

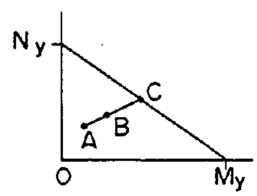
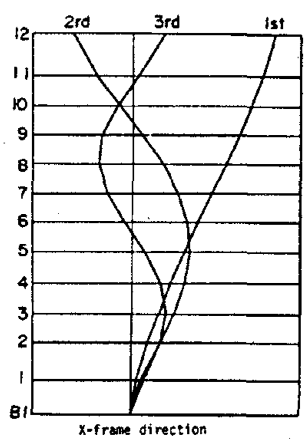


Fig. 21 M-N graph

Period (sec)	1st	2nd	3rd
	0.722	0.191	0.099



Period (sec)	1st	2nd	3rd
	0.876	0.250	0.133

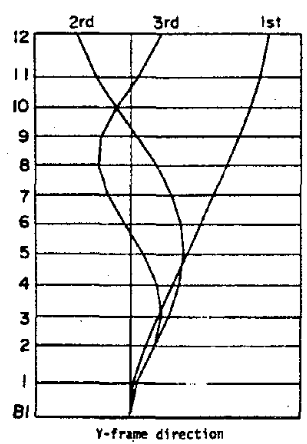


Fig. 22 Eigenvalues and mode shapes

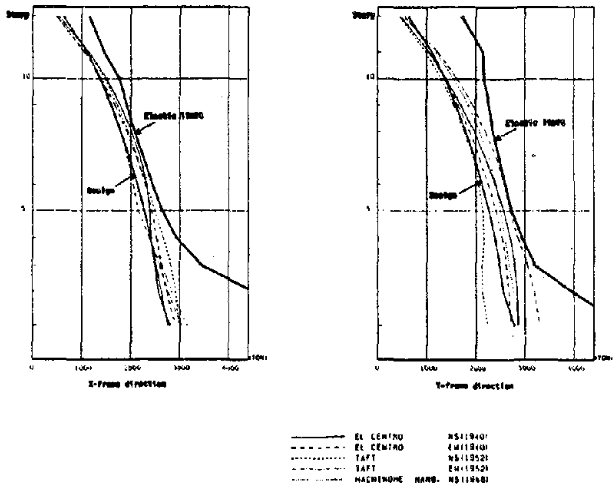


Fig. 23 Maximum response shear force (Elastic resp.)

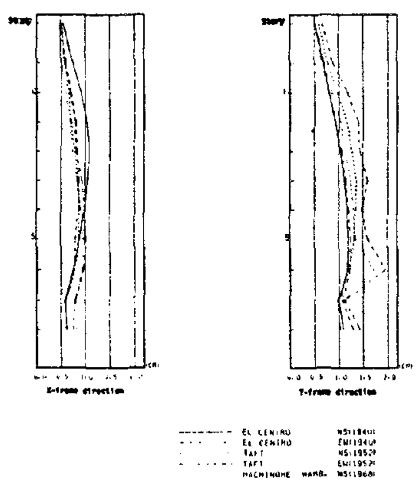


Fig. 24 Maximum response story drift (Elasto-plastic resp.)

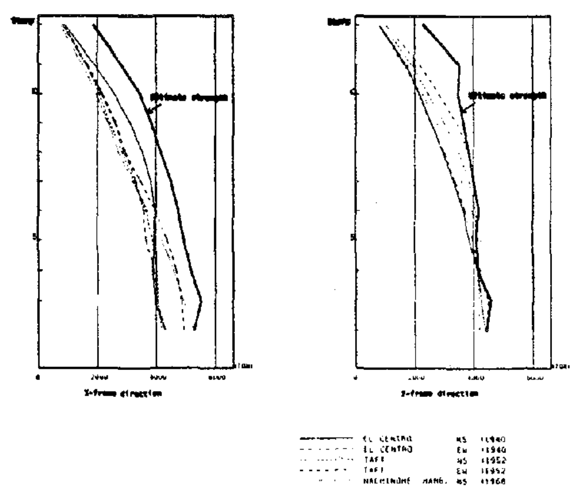


Fig. 25 Maximum response shear force (Elasto-plastic resp.)

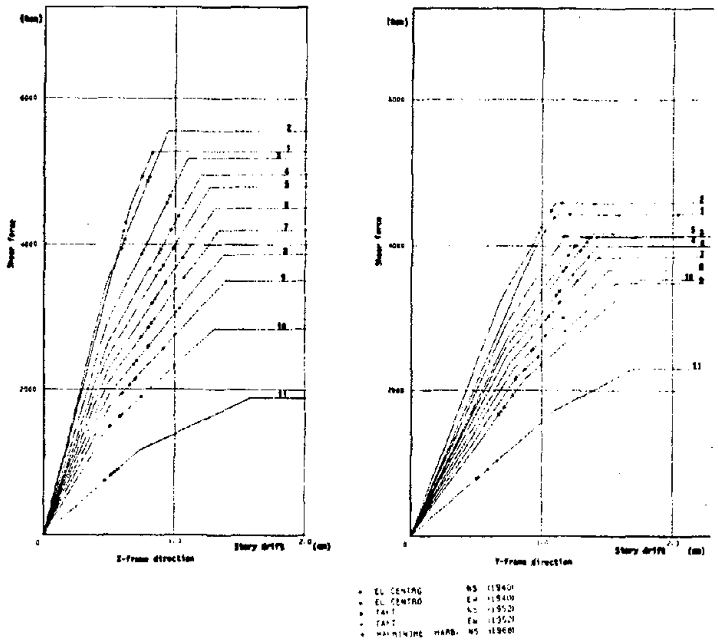


Fig. 26 Condition of plasticity (Elasto-plastic resp.)

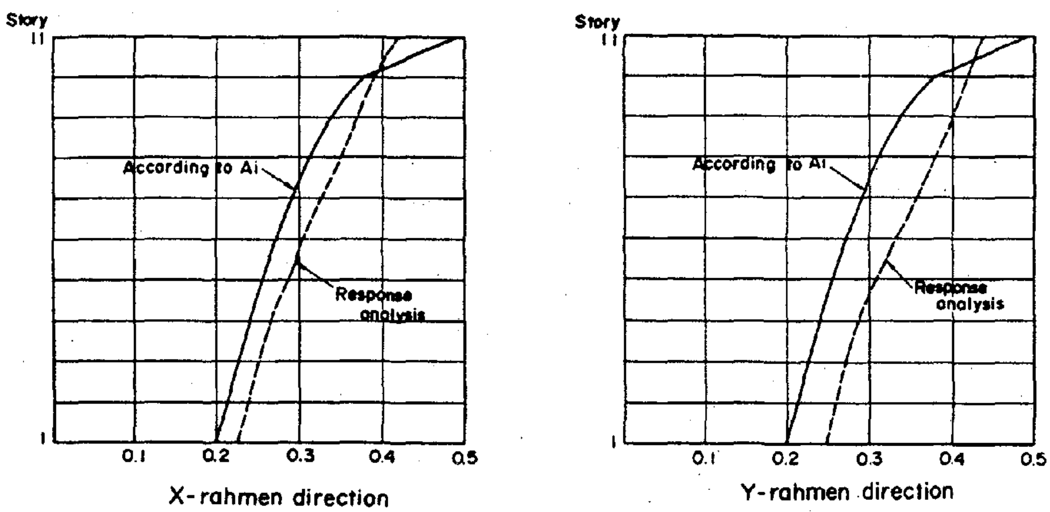


Fig. 27 Story shear coefficient calculated by using A_i and according to a dynamic response analysis

PLANNING, DESIGN, AND CONSTRUCTION CONSIDERATIONS OF PRECAST CONCRETE STRUCTURES FOR SEISMIC LOADS

Gerard Dixon, P.E.
Director of Structural Engineering
John Graham and Company
Seattle, Washington

INTRODUCTION

The precast concrete industry really started to grow in the United States in the late 1940s and early 1950s. Since that time, the whole field of precast, prestressed and post-tensioned concrete has expanded and is now a significant portion of the total construction carried out in the country. This recognition of the industry and the ongoing and increasing use of the material is justly deserved.

Like the suppliers of other construction materials, the precast industry is constantly striving to improve its product, to find new and different ways to use it economically, and to improve quality control and production techniques. The majority of precast concrete used is still produced in fabricating plants and shipped to the site or, if volume permits, a small production plant is set up directly on the site. This is a definite asset in maintaining quality and in cost control.

CURRENT RESEARCH

The Precast Concrete Institute (PCI), which directly or indirectly sponsors most of the research in the industry, has over the years made continuing efforts to determine the areas in which research is most needed to meet the requirements of engineers, architects, and the industry at large.

The PCI Journal of November/December 1981 presented the results of an industry-wide survey conducted by its research committee. Compiled from this survey was a listing of the ten topics that the participants felt most required research. These were rated in order of desirability. The topic leading the list was "economic moment resistant beam-column connections."

In addition to the rated list of topics, there were 62 further suggestions of additional research needs. Eighteen of these topics were unique to bridges; the remaining 44 were in the area of general precast and prestressed concrete construction. Heading this list was "seismic considerations." This subject area was broken down as follows:

- Behavior of prestressed members in response to seismic forces
- Ductility characteristics of welded connections
- Design criteria and test data for ductile embedded anchors
- Architectural precast connections for earthquake-resistant design
- Design criteria and methods for precast and prestressed frames and panels for earthquake-resistent design
- Seismic response of piling--favorable effects of confinement

None of these are very surprising, as the use of precast and prestressed concrete in areas of high seismic activity presents the engineer with some unique problems.

The design of steel and conventionally reinforced cast-in-place concrete buildings pre-dates the development of the present precast industry. The codes and standards were initially developed to cover these types of construction, and the "comfort level" of most structural engineers is higher when using these materials. As more research is carried out and precast, prestressed structural systems are developed and proved to be economical, the "comfort level" of the designing engineer with precast concrete will increase.

It is to the credit of the precast industry that it is meeting this challenge head-on rather than being satisfied to limit the use of precast concrete to only certain elements and building types.

Although the use of precast, prestressed concrete is widespread in high seismic areas, this use is not predominant in the mid- and high-rise rigid-frame type of construction. It is nevertheless significant.

Since the results of the 1981 survey were published, the PCI has been successful in reaching its funding goal for an expanded PCI research and development program. The following eight projects are in this program:

1. Economical moment-resistant beam-column connections
2. Exceptions of precast, prestressed members to minimum reinforcement requirements
3. Prestressed concrete column behavior
4. Load deformation of simple connections
5. Design of spandrel beams
6. Strength of members with dapped ends
7. Survey of precast, prestressed concrete parking structures
8. Collection and analysis of new fire data

Six of the ten highest priority topics identified in the 1981 survey are included in the above program. Projects 1 and 4 are currently underway in the Seattle area. An association of Concrete Technology Associates and ABAM Engineers, Inc., is carrying out these research projects. The schedule calls for testing of connections to proceed through 1984 (16 total) followed by a "full-scale" frame test in 1985. The results of this work should be available via the PCI Journal in late 1985.

The specific objectives of this project are (1) to evaluate the requirements for typical prefabricated concrete frames and (2) to determine economical, reliable, and competitive methods of making the connections. It is expected that the results of this study will be of value to engineers engaged in the design of precast structures, producers engaged in the fabrication of precast structures, building officials responsible for the approval of precast structures, and society in general, which will benefit from increased safety and reduced construction costs as economical precast structures gain increased acceptance.

FORECAST AND INDUSTRY TRENDS

Economics

Generally, the precast, prestressed market in the Northwest is still very competitive. Fiscal year 1982/83 was quite good, with sales 10 to 15 percent above expectations, and with an acceptable profitability. Fiscal year 1983/84 is down 10 to 15 percent below the previous year, and profitability is not as good, due to the amount of work available and quite keen competition.

The outlook for the second half of 1984 is expected to be better, with only nominal increases in the material and labor rates. The volume of work is expected to be up, which should help with a more profitable pricing structure.

Trends

Bridge Work: This area is expected to be up because funding is is available for both new and replacement work. Recently there has been a high rate of precast, prestresssed use by transportation departments in both Washington and Alaska.

Waterfront Structures: Construction is anticipated from both private companies and port authorities with need for piling and loading dock sections.

Parking Structures: Opportunities will exist for several significant parking structures. Recently, precast, prestressed in a variety of schemes and systems has been very popular for this type of structure.

Buildings: The use of poststressed, precast hollow-core plank has maintained a gradual growth over the past few years. This is expected to continue, and plant capacities have been expanding to keep pace.

Architectural Precast: This continues to be popular with the more conservative and institutional owners. In the Seattle area there is an increase in the use of glass-fiber-reinforced concrete precast panels, and this product is expected to gain popularity due to its economy and light weight.

On-Site Precast: One area somewhat unique to Western Washington is the amount of on-site precast work. This construction method is quite popular for warehouse, manufacturing, and commercial facilities. There are several specialty contractors who prefer this method of construction for this type of building. This is not always the case in other parts of the country.

Overall, the outlook for the precast industry in the Northwest is positive.

CASE STUDY: PLAZA CENTER GARAGE

General Description

The Plaza Center Garage is located in Bellevue, Washington, on an open, level site between two office buildings. The existing 10-story concrete-framed office buiding, constructed in 1977, used the garage site for required parking, and this parking area had to be at least partially usable during construction. The new 17-story steel-framed office building, built concurrently with the parking structure, required the construction of the new parking garage because the on-grade parking lot did not have sufficient capacity for both buildings.

The proposed garage was located between the two office buildings and connected to them by footbridges at the second level of each building. The overall plan dimensions of the garage were 226'6" x 255'4", and as originally planned had one basement level, one framed level at grade, and three elevated framed levels. The parking bay widths varied, with two bays approximately 60'0" wide and two bays approximately 52'0" wide. The northernmost bay was ramped to provide vertical access from grade (see Figure 1).

The City of Bellevue is located just east of Seattle, and the Belleview Building Code, which governs the garage design, is essentially the Uniform Buiding Code with some minor local modifications. The garage, therefore, is located in Seismic Zone 3 as defined by the Uniform Building Code.

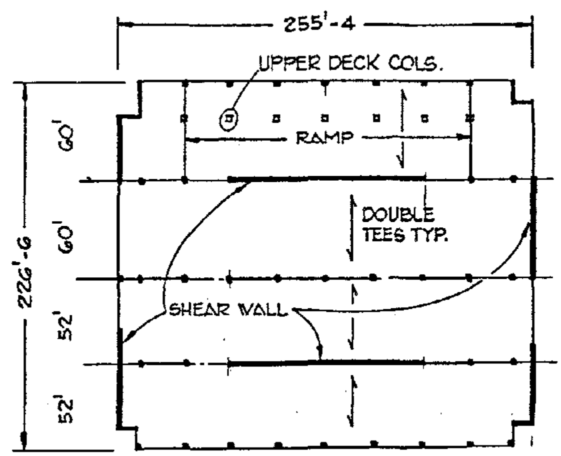


Figure 1. Garage plan.

Preliminary Design

Numerous framing schemes were considered in the conceptual design. These were then narrowed down to three schemes, which were presented to the contractor and client for their review and study.

Scheme 1 was precast, prestressed untopped double tees spanning the 52-ft and 60-ft bays and supported by precast, prestressed inverted tee girders and on continuous corbels on the center and perimeter shear walls. Most of the columns were also precast and were constructed with corbels to support the girders. The foundations, foundation walls, and shear walls were cast in place. The perimeter of each bay had cast-in-place closure strips, which would effectively enclose the tee decks for diaphragm action. The connections between adjacent tees were to be mechanical anchorages.

Scheme 2 was precast, prestressed girders spanning the 52-ft or 60-ft bays at approximately 20'0" o.c., supporting a cast-in-place post-tensioned slab. The columns were to be precast, with the balance of the foundation's basement walls and shear walls cast in place similar to Scheme 1.

Scheme 3 consisted of steel girders spanning the 52-ft and 60-ft bays at approximately 24' o.c. with 8" precast, prestressed hollow core slabs spanning 24'0". The columns were steel, and it was proposed to use steel-braced frames in lieu of shear walls. The ends of the hollow core slabs were to be grouted solid for composite action with the steel girders.

Foundations were no problem for any of the schemes, as the soils engineer recommended allowable bearing pressures in the range of 8 to 10 kips per sq ft. Based on several factors, but primarily on speed of construction and overall economy, Scheme 1 was selected.

Final Design and Revisions

Final design and construction documents were begun based on the agreed criteria. Because time was critical, it was agreed to layout and design the tees first, as they formed the bulk of the precast work. These preliminary plans and details were issued to the contractor so that the precaster could start preparing shop drawings and get a head start on the fabrication before the remainder of the design was completed.

During this time period, a formal Environmental Impact Statement was presented to the city for approval. It soon became apparent that the permit process would be difficult. An adjacent property owner on the north side raised strong objections to the height of the proposed garage because it would affect the future use of his property. It was probable that these objections could have been overcome, but the owner decided not to get involved in protracted hearings and legal maneuvers and to make some fairly radical changes to the design to satisfy the objections of the adjacent property owner. After deciding to make these changes, the big question was whether the design could be modified at this point, with the documents well along, and still meet the tight construction schedule.

The major changes were as follows:

1. The whole facility was lowered one more level into the ground to reduce overall structure above ground. This resulted in two framed levels now being affected by greater lateral earth pressures than previously. These, in combination with the lateral seismic forces, increased diaphragm shears significantly.

2. The 60'0" upper ramp bay was to be cut back 20'0" to further reduce the visual effect of the garage on the adjacent property. This resulted in the upper rear columns that supported the top deck having to be supported on 60-ft girder spans below. Because these loads were too great for precast sections of the same depth as the tees, cast-in-place post-tensioned girders were introduced in the tee system at 20'0" o.c. to support these columns.

3. The additional excavation and introduction of post-tensioned girders

were two of the reasons why the construction schedule became more critical. The owner and the city agreed that it would be necessary to construct the north half of the garage first, and occupy it before the south half construction could commence. In this way, the south half of the site could be available for parking for the existing building during construction. This change required the relocation of the main north/south shear walls, which had been located at the perimeter (see Figure 1), from being all in the south half to being partially in the north half, since this half structure was to be occupied and stand alone for some three to four months.

It is to the credit of all concerned designers, contractors, and the client that these quite radical changes were accommodated with little lost time while construction documents and shop drawings progressed.

It was possible to accommodate changes while still maintaining the integrity of the overall concept both structurally and aesthetically. The basic building elements of double tees, girders, and columns could be quickly rearranged to fit the new scheme and still be economical while not seriously affecting the schedule.

<u>Seismic Design and Details</u>

The key to a good seismic design in this case was in providing (1) sufficient shear walls to effectively resist the lateral force, (2) locating these walls to reduce calculated eccentricities to a minimum, and (3) providing diaphragms that would safely distribute the lateral forces into the shear walls.

One of the reasons that double tees with 4-1/2" flanges were selected was that these could be erected quickly at the site and connected by mechanical anchorages (see Figure 2) without having to cast the more traditional topping slab. The spacing of the mechanical connections between the tees could be adjusted to resist the varying shear forces between adjacent tees. The anchorages were spaced from a minimum of 2'6" to a maximum of 6'0".

It was helpful to be able to adjust the connection spacings when the garage was lowered one level, resulting in greater shears due to earth pressures on the diaphragm, and when diaphragms with a much reduced L/D ratio were introduced by erecting the north half of the garage as an independent structure.

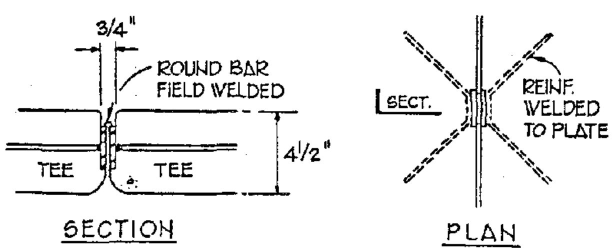


Figure 2. Typical tee joint.

The contractor did not want to use a topping system. He also wanted to eliminate as much flat forming as possible. Because the diaphragm design required a chord element, it was necessary to introduce a cast-in-place edge condition but still keep forming to a minimum. This was done by reducing the thickness of the tee flange from 4-1/2" to 2" for a 2-0" length at the ends of all tees. This resulted in a cast-in-place pour strip the full length of the bay into which mild steel reinforcing could be placed and enclosed with dowels and cross reinforcing (see Figure 3). At the edges parallel to the tees a similar closure pour was created by holding the edge of the tee back from the girder or wall. These strips at the perimeter on each side of the project required some forming, but only to a limited extent. The chord reinforcement at these locations was enclosed by dowel reinforcement from the edge of the tee and into the slab edge planter cantilever (see Figure 4).

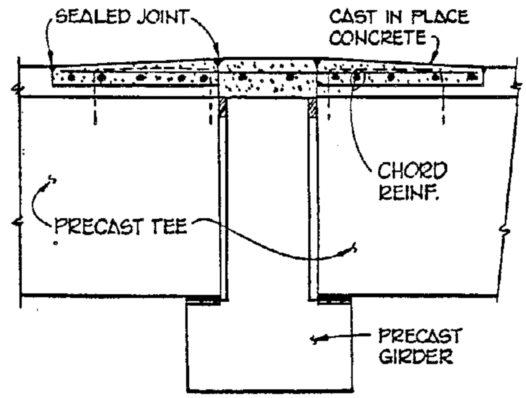


Figure 3. Typical girder section.

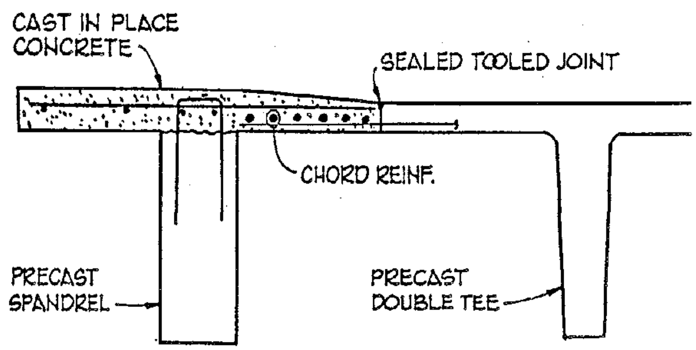


Figure 4. Edge condition.

The sloping ramp bay was handled in a similar manner. This 60-ft deep diaphragm at the grade level was also designed to resist the lateral earth pressure from the top of the foundation wall. Therefore, the completed diaphragm at each level is made up of four long bays 60' or 52' x 255'0". Each bay is completely enclosed within a continuous pour strip that effectively binds the whole together to form a rigid plate capable of transmitting the lateral forces to the supporting shear walls.

Various grades of elastomeric bearing pads were used at the bearings of the double tee stems and girder seats to account for any member rotation or ongoing movements due to creep or shrinkage. Finally, the joints between the tees and the grooved construction joints between the precast and cast-in-place concrete were filled with a polyurethane joint sealer. This sealant was one of several patented products available and was color matched to the finished concrete. The precast elements were sized and detailed so that the finished garage fitted visually into its surroundings and was aesthetically pleasing.

Construction

The cast-in-place construction and erection of precast went quite smoothly with only minor problems. At one location, dowels had inadvertently been specified to project from a shear wall, which made it almost impossible to erect the adjacent tees. This was remedied by removing the dowels and instead using drilled inserts to receive threaded rebar.

Another minor problem was at the upper ramp level, where the post-tensioned girders were used to support one level of columns. Even with the use of a computer, it is still difficult to predict deflection and camber with absolute certainty. The top of the post-tensioned girders did not match the top of the adjacent tees once all the loads were in place and post-tensioning was completed. Fortunately, the girder had deflected down approximately 3/4", and it was possible to add concrete to compensate for this sag. Had the girder cambered up after post-positioning, it would have been a more complicated problem.

The contractor helped to reduce the construction schedule during the south half of construction by electing to precast most of the foundation walls flat and tilt them up into position with vertical cast-in-place closure strips at the columns and pilasters. This required only minor modifications to the design and certainly saved construction time.

<u>Summary</u>

The initial selection of the precast system was a sound one. It was possible to design this garage for significant seismic loads, provide the long clear spans required by the owner, modify the design as it progressed in both the design office and the plant, and still provide a good-looking and economical facility almost on schedule.

The final garage was approximately 280,000 sq ft in area with parking for 1,117 cars, giving a ratio of 251 gross square feet per parking stall. The construction cost was $3,806,000, for $13.60/sq ft, or $3,407/stall, which is very economical by current standards.

CASE STUDY: AUBURN DOWNS GRANDSTAND

This project is currently in working drawings, and construction is planned for the fall of 1984. It is a four-story grandstand/clubhouse to be located in Auburn, Washington, just south of Seattle. The applicable code is the 1979 Uniform Building Code with minor local modifications. The siesmic zone is Zone 3.

The primary lateral load resisting system is a two-way ductile steel frame, and most floor framing, including the sloped seating areas, is 12-in. precast, prestressed hollow-core plank with a 2-in. cast-in-place topping slab (see Figure 5). This system was chosen for both economy and speed of construction. Both the structural steel and the precast plank could be pre-ordered on a fast-track basis. The grandstand is founded on 16-1/2" octagonal prestressed, precast piles with a useful capacity of 160 tons/pile.

There are no mechanical connections between adjacent planks. The joints are grouted with nonshrink grout and then the area receives the 2-in. topping slab with light steel mesh reinforcement.

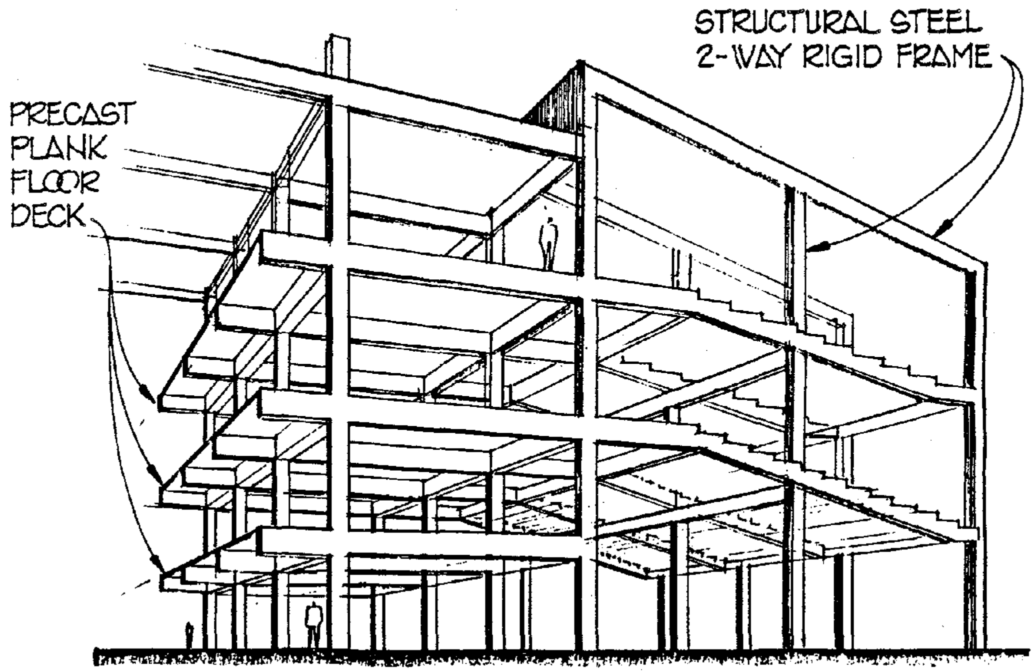


Figure 5. Idealized view, main frame grandstand.

Because the steel frame requires fireproofing, and because the structure is to be exposed in most areas, it was decided to encase the bottom flange of the steel frame members with cast-in-place concrete. This creates a bearing surface for the precast plank. The upper portion of the steel beams and the ends of the plank are then connected when the topping is placed. This provides the required fireproofing, and the addition of reinforcing stiffens the frame for both gravity and seismic loads (see Figure 6).

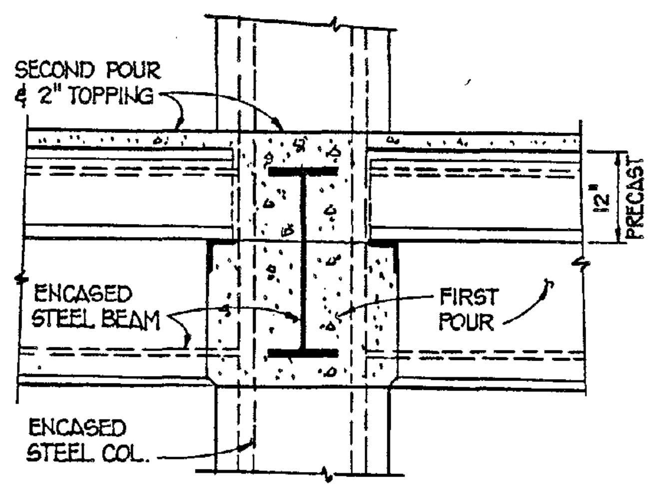


Figure 6. Typical frame section.

Boundary members for the floor diaphragms are provided by the continuous steel frame girders and cast-in-place concrete at perimeter conditions. In summary, this structure is a good, economical blend of structural steel, precast, prestressed plank, and cast-in-place concrete, intended to optimize the use of each material by using one material to complement the others for speed of construction.

CASE STUDY: SEATTLE SHERATON HOTEL

This 35-story hotel tower, constructed in 1982-83 in downtown Seattle, is triangular in shape and is one of the tallest ductile concrete frames on the West Coast. The primary lateral load resisting frame is located at the perimeter of the buiding (tube frame), and the outer face of the column and spandrel girders with inset windows forms the facade of the tower (see Figure 7).

To achieve a high-quality finish, maintain a tight construction schedule, and still be economical, the contractor opted for reusable steel forms for all frame members. Because the forms were to be used several times, it was decided to use a precast, prestressed concrete plank for the outer floor areas. This enabled the frame to be cast and forms lifted easily and quickly. The fairly simple connection of the precast planks to the frame could be made later with little, if any, down time in forming operations (see Figure 8).

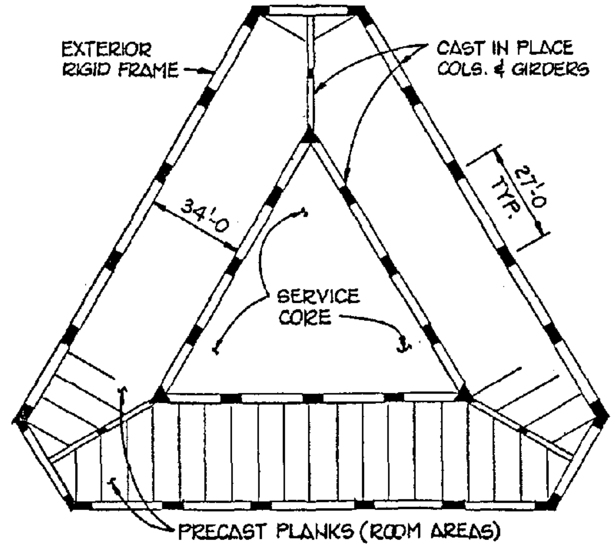


Figure 7. Typical guest room floor.

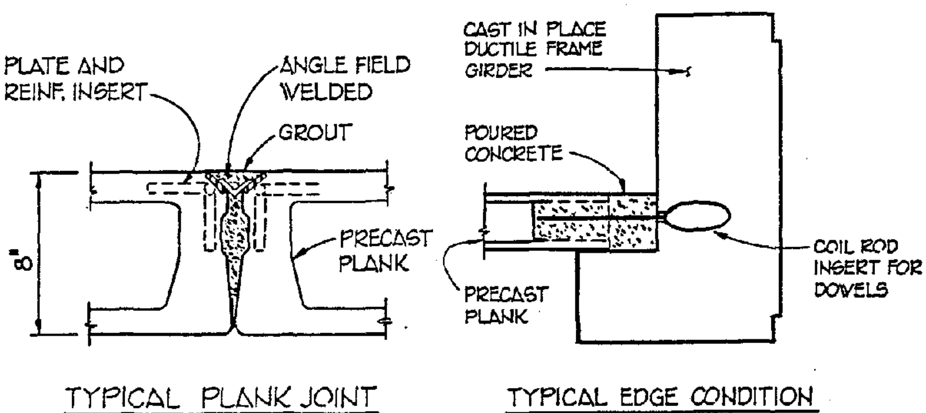


Figure 8. Connection of precast planks to frame.

By using coil rod inserts in lieu of dowels, the integrity of the steel forms could be maintained. The plank was untopped with mechanical anchorages and grout in the joints between the planks.

There was considerable emphasis on aligning the plank in order to level the soffits because there are no suspended ceilings in the guestrooms. In these rooms, a textured paint finish of the plank soffit was the finished ceiling. This was economical, not only due to the elimination of the usual hung ceiling, but also because it reduced the overall height. The floor-to-floor height for the tower portion was only 9-1". Minor irregularities in the top surface of the plank were leveled with a grout material, and the use of carpet and pad as the typical floor finish was perfectly adequate. The floor functions as a structural diaphragm to effectively transfer lateral loads into the exterior tube frame, which completely encloses it. The precast units were carefully located relative to the room layouts, and it was possible in most cases to accommodate plumbing chases and duct openings by blocking out the voids in the plank.

The connections of the plank ends to the outer frame and the cast-in-place core area were made by dowelling from the cast-in-place concrete into the plank voids and then grouting these ends solid.

SUMMARY

The Northwest is particularly fortunate to have several excellent fabricators and producers of precast, prestressed reinforced concrete. By any standards, their products are of high quality and their plants have good quality control. This produces a consistently reliable product both from a structural/strength standpoint as well as from the standpoint of an aesthetically pleasing and durable product when used for purely architectural requirements.

The analysis of prestressed and post-tensioned structural systems is no longer the problem or effort it used to be, thanks to the computer. Engineers can perform sophisticated analyses of almost any kind of building structure with remarkable accuracy compared to the manual methods in practice 15 or 20 years ago.

In Seismic Zones 3 and 4, where the seismic forces are usually controlling versus wind, special attention must be paid to the actual detailing of systems in order to maintain strength and ductility in resisting seismic forces. Fortunately, it is in this area that the PCI is concentrating much of its research efforts.

With efficient, responsible producers and an industry that is in tune with the needs of engineers, architects, owners, and the public, we can look forward to new and improved uses of all forms of precast concrete. It will require the efforts of all concerned to keep the building codes up to the state-of-the-art for the industry so that these potential new methods and techniques are not kept on the shelf due to lack of testing or confidence in their soundness.

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COOPERATIVE US-JAPAN PROGRAM FOR IMPROVEMENT OF BUILDING SEISMIC DESIGN AND CONSTRUCTION PRACTICES - 1st MEETING -

EVALUATION OF SEISMIC ACTION ON STEEL DOME

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(1)

Nowadays there are many types of architecture or structure in the world. My work has principally been on structures which play important parts in the artistic or aesthetic effects of architecture, for example, the International Conference Hall in Kyoto, the Metropolitan Concert Hall at Ueno, etc. Usually architects are very free and ambitious to create interesting, beautiful or attractive forms of structure and consult us on how to realize them. We, the structural engineers, should respond to their requests and try to find solutions to bring them into realization, doing our best as long as the architect's idea is socially acceptable.

Since the birth of a structural whole is the result of a creative process, the fusion of technique and art, of ingenuity and study, the collaboration between architect and engineer should focus on the practice of the structural design, and such studies may be rewarding to structural engineers, to ourselves.

The more severe the natural conditions, the more important is the role of the structural engineer. In Japan, there are high intensity earthquakes, violent typhoons, heavy rains and frequent floods and landslides. The sub-soil of most built-up areas is soft or muddy clay. So, in their design practices, structural engineers are overloaded with tasks so that it is difficult to maintain safety and to achieve economy. Besides, requests for large scale or new styles of buildings are coming in one after another.

Under these circumstances, the structure should still be sound. Naturally, it must be reasonable in its safety, in its economy and in its realizability. And, as far as possible, it should be attractive, beautiful or interesting, overcoming the restrictions of concrete science and reaching out beyond. This is my opinion.

(2)

Now I would like to show you a steel structure under construction in Fujisawa City. It is about 30 miles from Tokyo. The project is a gymnasium complex designed by Mr. Fumihiko MAKI, one of the most active architects in Japan and a professor at Tokyo University. The project will be completed in October of this year. I believe this structure will be met with satisfaction by Mr. Maki and all of its users in the district.

The gymnasium complex consists of three wings, a Main-Arena, a Sub-Arena and an Entrance-Wing. The Main-Arena, which is covered by a big dome, contains three Volley-ball courts and two Main-Stands with 2,000 seats for spectators (Fig. 1). The Sub-Arena, which is covered by a vault, contains Judô and Kendô Halls, a Training Center and other functions (restaurant, offices, etc).

The Main-Arena dome is supported by a couple of parallel keel arches which have a span of 265 ft from north to south and a rise of 75 ft at the center. The foundations of both ends are tied by a footing beam of prestressed concrete. The main trusses of the roof span these arches at interval of 21 ft, and side trusses are extended at the same intervals sloping from the arch to the outside ends where they are supported by the posts provided at the top of the Main-Stand cantilevers of prestressed concrete. The supporting system at this point has a sliding shoe, and the horizontal thrust of the sloping trusses are bound by a ring arch of steel running along the periphery of the Stand. The Main-arch is also a latticed truss of triangular section and its legs are covered with reinforced concrete for stiffness and for fireproofing. The roof is finished with thin stainless sheets. Thus, the dome is simply constructed with straight trusses and curved trusses.

Even though the structural concept is very clear and simple, owing to the curvature of the dome, the calculation of dimensions and the details of connections are very complicated. However, the subject of this report is not these difficulties, but the aseismic design of this dome, especially the evaluation of seismic forces.

Several years ago, the New Criteria for Aseismic Design including Building Codes and Regulations were established in Japan. These criteria indicate a certain mode of seismic action and are based on statistics of the results of dynamic analysis of many case-studies. But, as the cases concentrated on rectangular frame structures like office buildings or apartment houses, a dome such as this case cannot conform to the mode of the criteria. Therefore, I had to make a new study to find a particular mode of seismic action suitable for this structure.

(3)

Fortunately, we were able to take advantage of the symmetry of the dome with respect to the two axes passing through the center, from north to south and from east to west. We divided the whole dome into four quarters by these axes. The model of the three dimensional frame-work is shown in Fig-2. Besides, from a practical point of view, it is sufficient to consider only up to the third order of vibration modes. It means that only three mass points need be picked up so long as they can represent the characteristics of vibration of the whole structure. Among 27 joint points of the model, the most adequate points are No.5, No.11 and No.18 along the main arch.

In order to express the characteristics of the entire structure using only these three points, it is necessary to use the stiffness matrix of the whole structure and to eliminate the elements of useless points in the matrix and to condense it into

3 x 3. It means making a matrix of the following type.

$$\begin{bmatrix} k_{11} & k_{12} & k_{13} \\ k_{21} & k_{22} & k_{23} \\ k_{31} & k_{32} & k_{33} \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \\ u_3 \end{Bmatrix} = \begin{Bmatrix} P_1 \\ P_2 \\ P_3 \end{Bmatrix} \qquad (0)$$

In this formula, the suffixes 1, 2 and 3 means respectively points No.5, No.11 and No.18; u means the displacement of the point and P means acting or reacting force at the point. If we can find the matrix [K] of the left side of this formula, the natural periods of motion and the vibration modes will be obtained by solving the eigen values and the eigen vectors of this matrix. Furthermore, by investigating the response spectrums, it is possible to know the response level of each period to be composed and finally to evaluate the seismic forces which will act on the dome at the various points.

Firstly, as to the method of how to get the [K] matrix, that is, how to eliminate the elements of useless points in the matrix of the entire structure, and how to condense it into 3 x 3, there are several ways. Among them, the following method is recommendable as a quick and convenient one. It is to solve the static equation of the whole structure (of course this means the model of one quarter area) three times under three types of boundary conditions, taking into account the following relations:

$$\begin{bmatrix} K \end{bmatrix} \begin{Bmatrix} 1 \\ 0 \\ 0 \end{Bmatrix} = \begin{Bmatrix} P_{11} \\ P_{21} \\ P_{31} \end{Bmatrix}, \quad \begin{bmatrix} K \end{bmatrix} \begin{Bmatrix} 0 \\ 1 \\ 0 \end{Bmatrix} = \begin{Bmatrix} P_{12} \\ P_{22} \\ P_{32} \end{Bmatrix}, \quad \begin{bmatrix} K \end{bmatrix} \begin{Bmatrix} 0 \\ 0 \\ 1 \end{Bmatrix} = \begin{Bmatrix} P_{13} \\ P_{23} \\ P_{33} \end{Bmatrix} \qquad (1)$$

The first solution can be found by giving the forced displacements of 1, 0, 0 to the points No.5, No.11 and No.18. The reactions at these points will give the vector P_{11}, P_{21}, P_{31}. The second solution can be found under the condition of forced displacements of 0, 1, 0, and the reactions will give P_{12}, P_{22}, P_{32}. The third solution will give P_{13}, P_{23}, P_{33}. Provided that no loads or forced displacements are given to the other points to be eliminated, the elements of these points in the matrix can be eliminated automatically in the solution. Then, by uniting the above obtained three vectors of P into one matrix, we can get

$$\begin{bmatrix} K \end{bmatrix} \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \end{bmatrix} = \begin{bmatrix} P_{11} & P_{12} & P_{13} \\ P_{21} & P_{22} & P_{23} \\ P_{31} & P_{32} & P_{33} \end{bmatrix} \qquad (2)$$

Considering that the second matrix of the left side is a unit matrix, it follows that the necessary [K] matrix is exactly the same as the [P] matrix of the right side. Naturally, this matrix must be symmetric because of Maxwell-Betti's "Reciprocal Theory", and this fact can be used to check whether the result is correct.

The condensed 3 x 3 matrix can be obtained for one direction (for instance, X direction) by solving the static equation of the whole structure three times under three types of forced displacements in the direction. Thus, in the same way, the condensed [K] 3 x 3 matrices for the other directions can be obtained.

Secondly, we have to find the natural periods of motion T_1, T_2, T_3 and the eigen vectors V_1, V_2, V_3 up to the third order for each direction by using these matrices. For this purpose, there are also several methods, but generally we use Jacobi's method in the computor program. The results are shown in Fig-3.

These are the actual [K] matrix, natural periods and eigen vectors for each direction.

Thirdly, we have to examine the response level of each period (β_1, β_2, β_3) in each direction referring to the various response spectra. In this case, even the first period is sufficiently short in practical consideration, and it may be permitted to regard all responses of three orders as being approximately at the same level. It means: $\beta_i = 1.0$ (i=1, 2, 3).

Finally, we have to evaluate the expected seismic action on each zone of the dome by composing these modes. the simple summation amounts to the following:

$$V = \beta_1 V_1 + \beta_2 V_2 + \beta_3 V_3 \qquad (3)$$

But we should take the maximum level expected throughout the earthquake duration. However, if we use the absolute value of each V in the formula, the total V value estimate may be too high.
In such cases, the Square Root of Square Sum (SRSS) is employed.
That is,

$$V^* = \sqrt{(\beta_1 V_1)^2 + (\beta_2 V_2)^2 + (\beta_3 V_3)^2} \qquad (3^*)$$

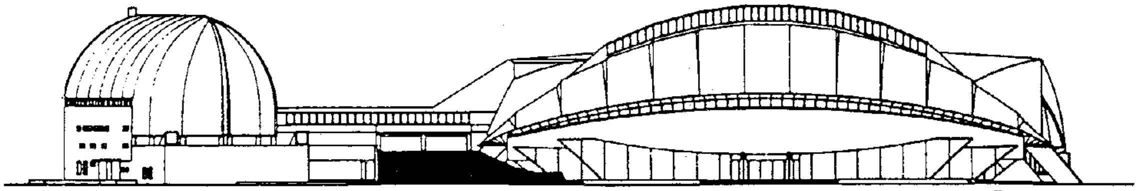
This is the resultant of seismic intensity. If the response means of acceleration, V* corresponds to the seismic action mode (seismic force mode) for each mass point.

The result should be compared with the Japanese New Criteria. Since the mode of the criteria is of the story shear, it should be converted to the action mode in order to compare these two modes.

Otherwise, V* should be converted to the shear force mode. As it is shown in Fig-4, we find a remarkable difference between these modes, especially for the Arch-axis direction (X direction). The reason may be because the obtained mode is for a closed shape such as a dome, while the Criteria mode is for an open shape such as a cantilever from the base. Such a model is generally used for the vibration analysis of a rectangular frame-work such as an office building or an apartment building.

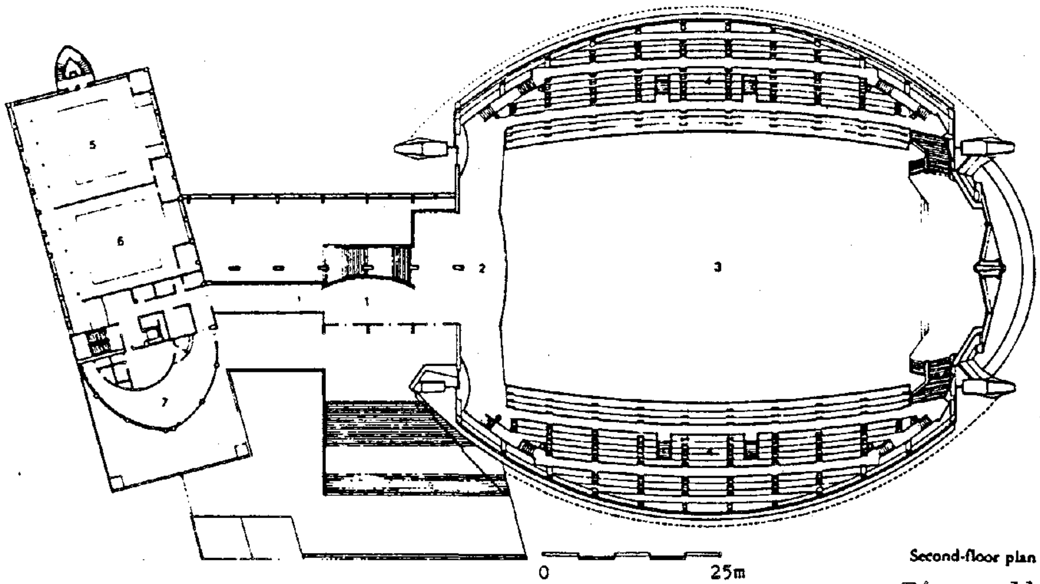
(4)

The outline I have just described is only one small example of the analytical processes which occur in the practices of aseismic design of a special structure. As I said at the beginning, there are many types of structure and most of them do not require such troublesome methods as this case. But, in certain instances, the regulations don't cover the problem that arises, and even the papers of the Architectural Institute cannot be directly applicable to the problem. When we encounter such cases, we, the structural engineers, have to find our own solutions beyond the regulations or the recommendations. I think that for the progress of the architecture, it is an important task of ours to respond to society's or architects' requests.



East elevation

Fig - 1a



Second-floor plan

Fig - 1b

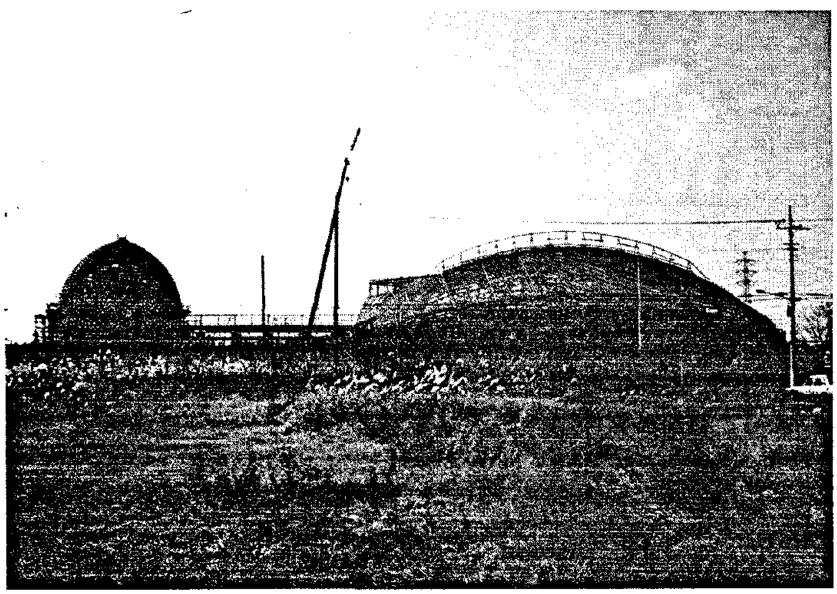


Fig - 1c

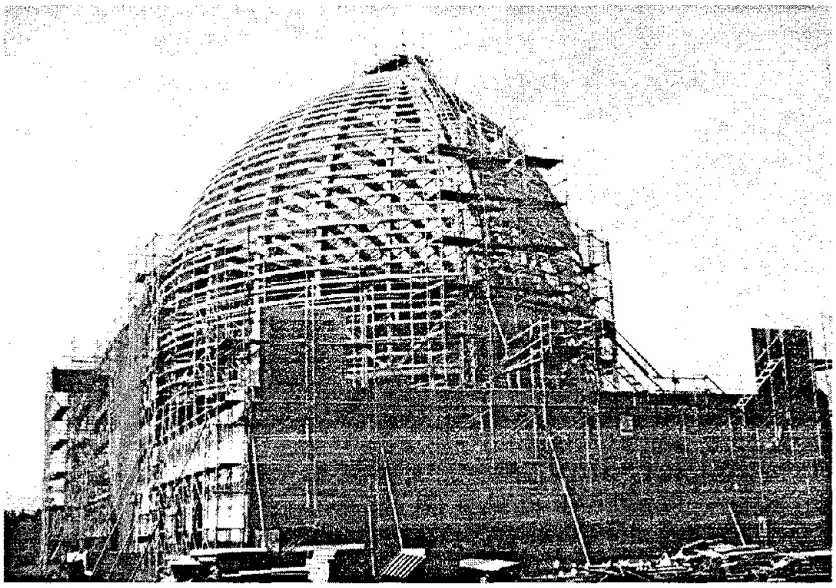


Fig - 1d

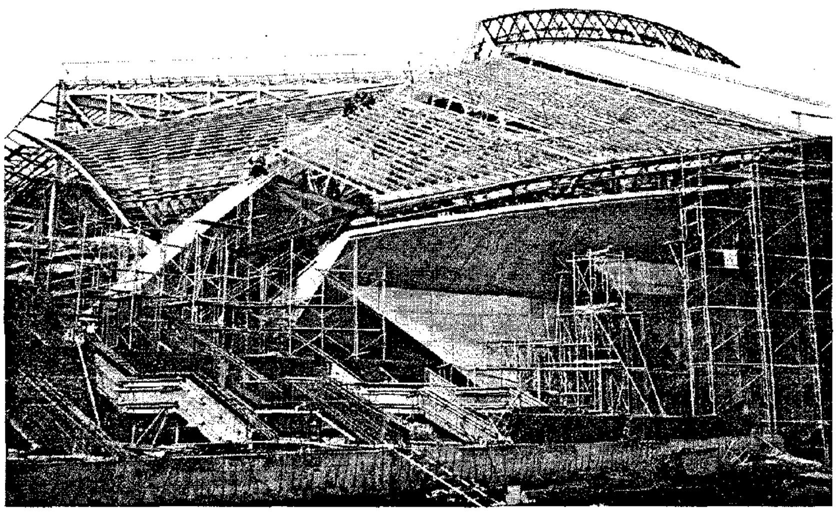


Fig - 1e

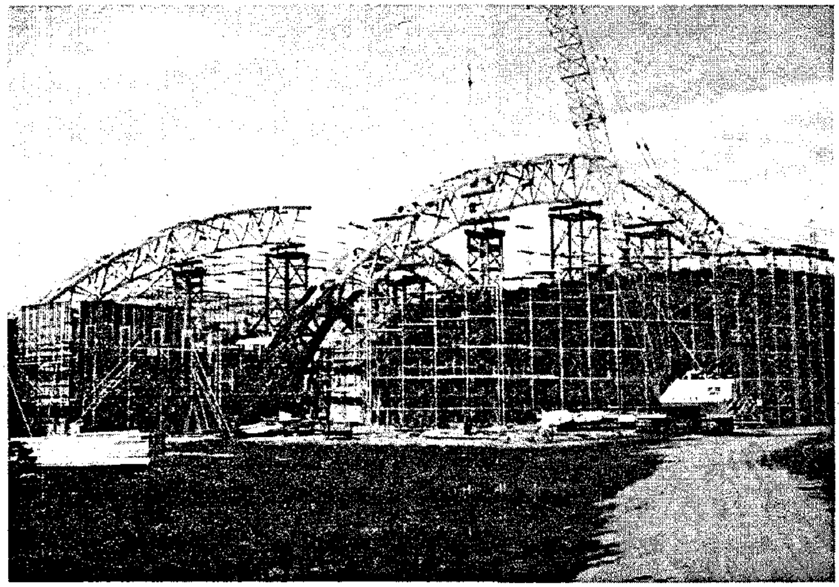


Fig - 1f

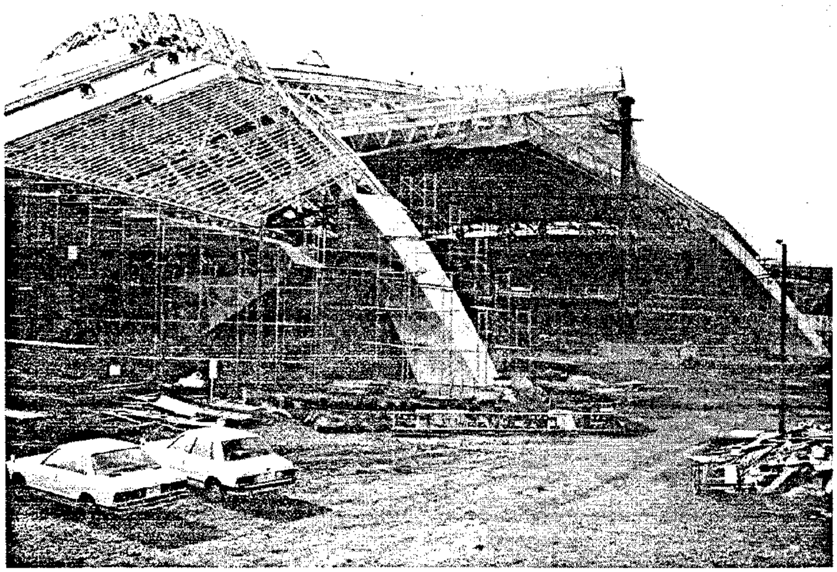


Fig - 1g

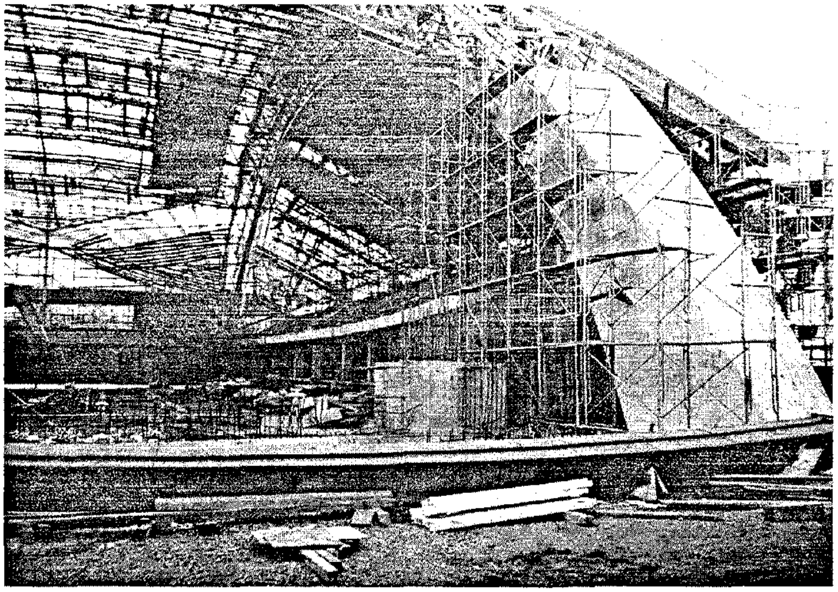


Fig - 1h

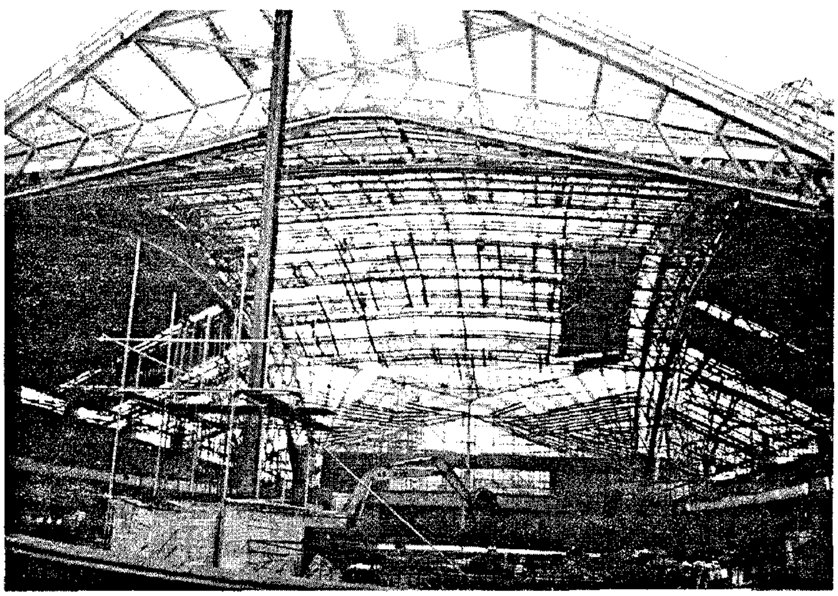


Fig - 1i

Fig -2 ANALYTICAL MODEL

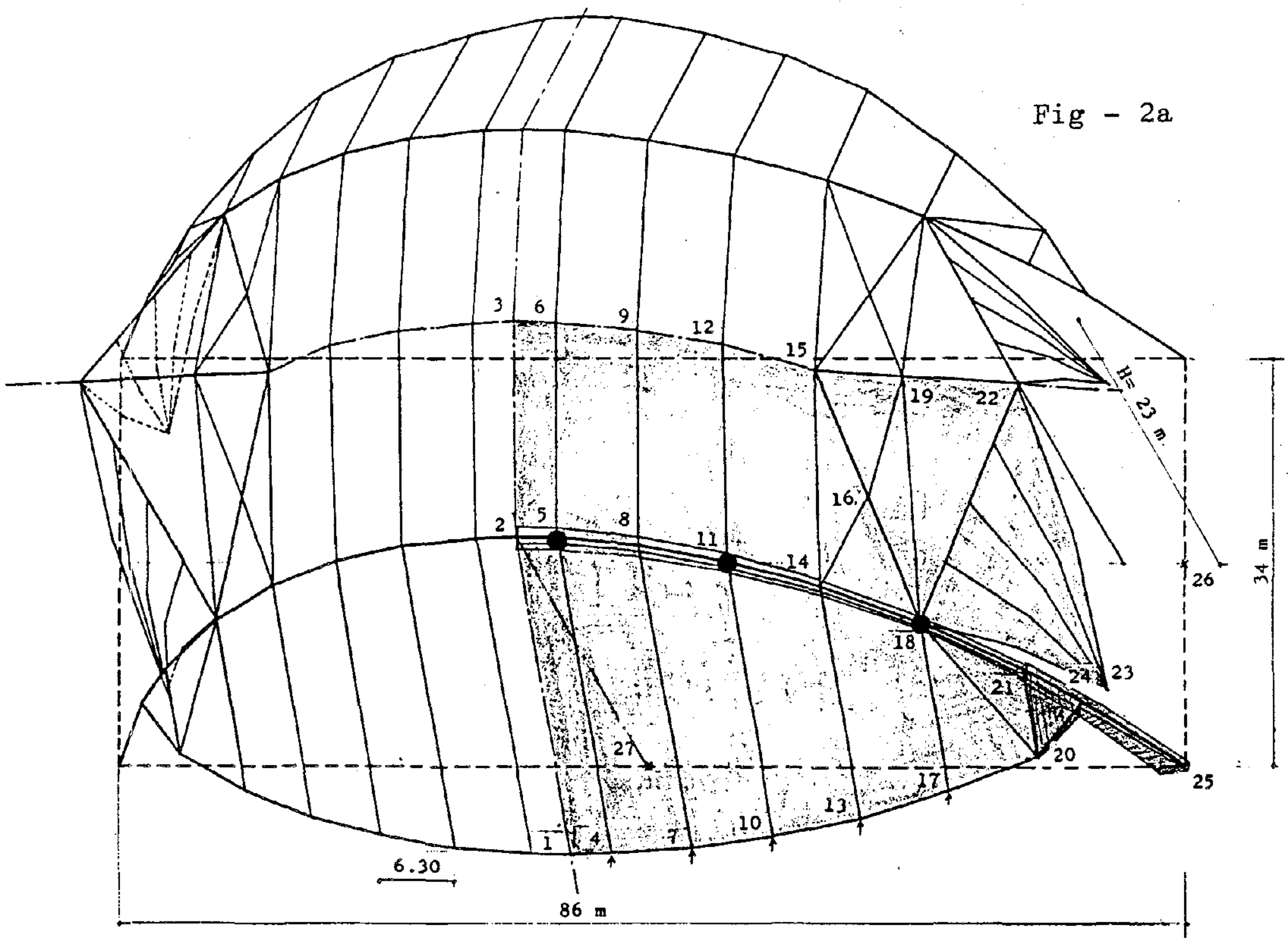


Fig - 2a

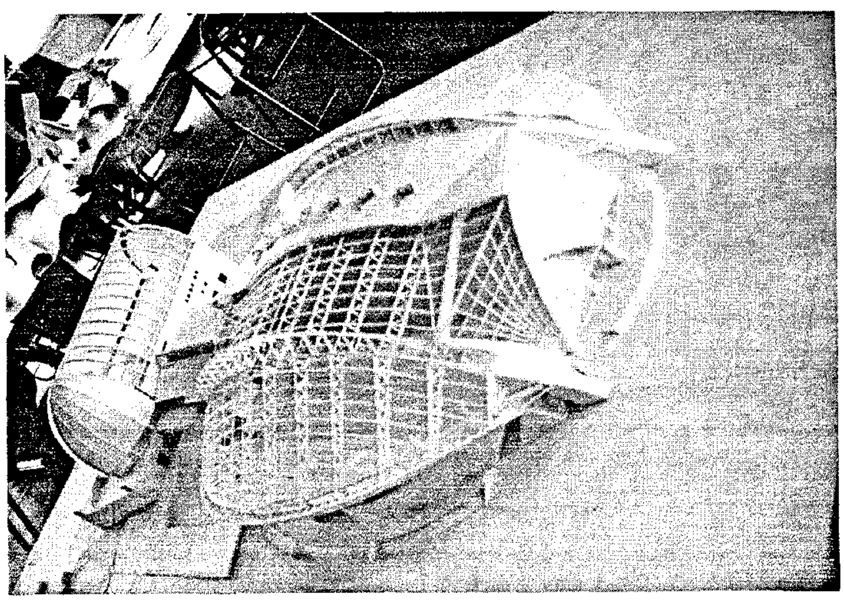
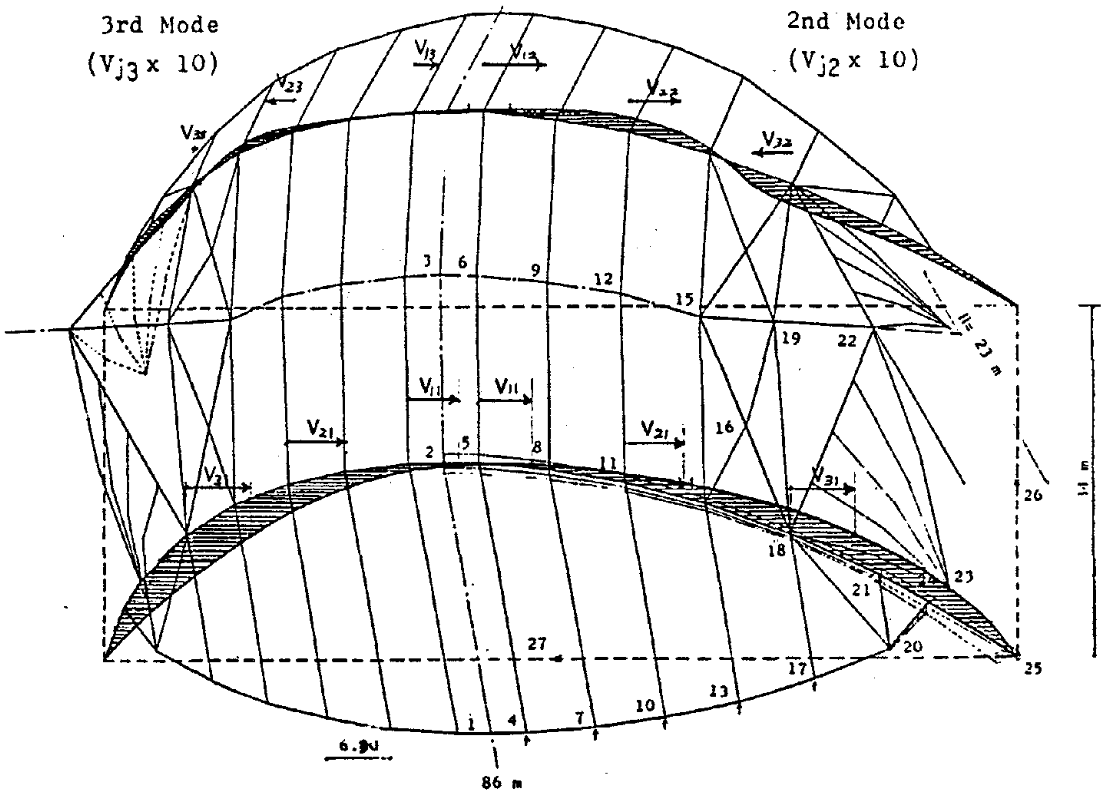


Fig - 2b

Fig - 3x MODE OF X DIRECTION



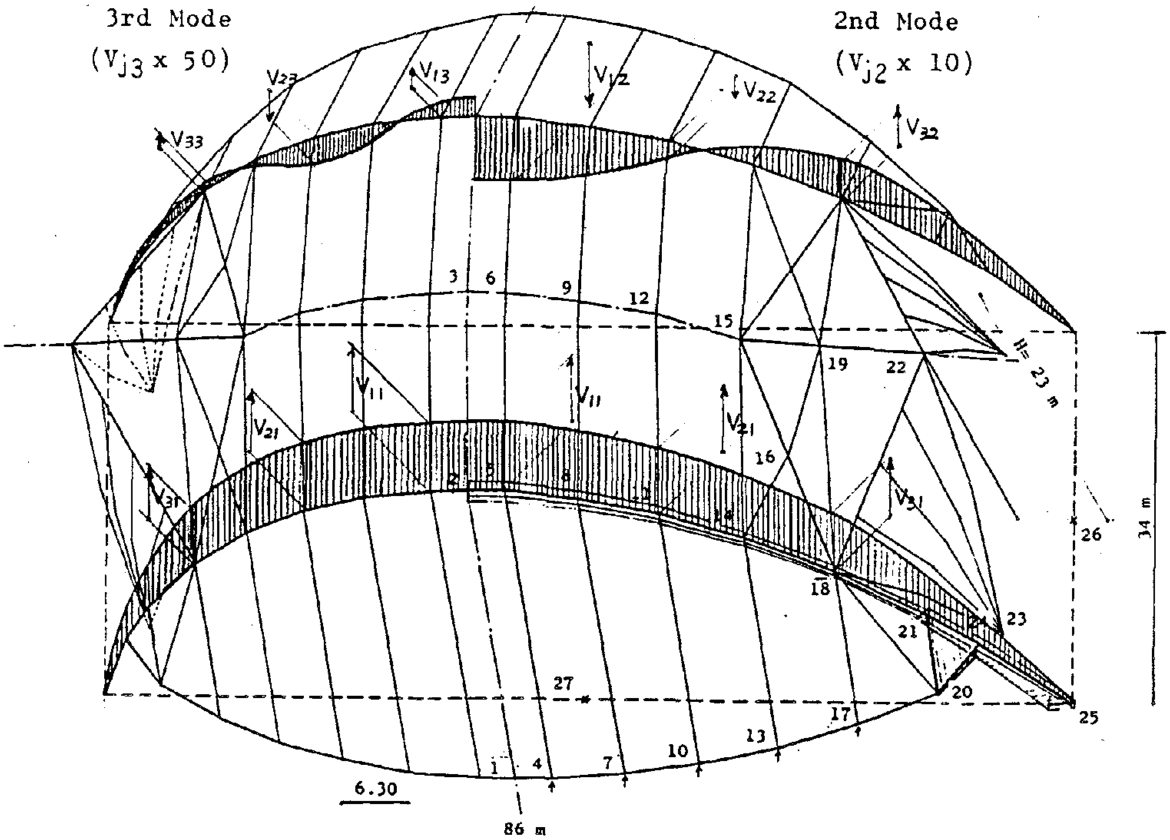
1st Mode (V_{j1} x 1)

Stiffeness Matrix

$$[K] = \begin{bmatrix} 52068.83 & -44158.15 & -1675.96 \\ -44158.15 & 57821.09 & -15624.34 \\ -1675.96 & -15624.34 & 17711.49 \end{bmatrix}$$

	1st Mode	2nd Mode	3rd Mode
Period	T_1 = 0.597sec	T_2 = 0.139sec	T_3 = 0.056sec
Point	V_{j1}	V_{j2}	V_{j3}
No. 5 (j=1)	0.8644	0.09886	0.03679
No.11 (j=2)	0.9611	0.08104	-0.04210
No.18 (j=3)	1.0603	-0.06294	0.00262

Fig - 3y MODE OF Y DIRECTION

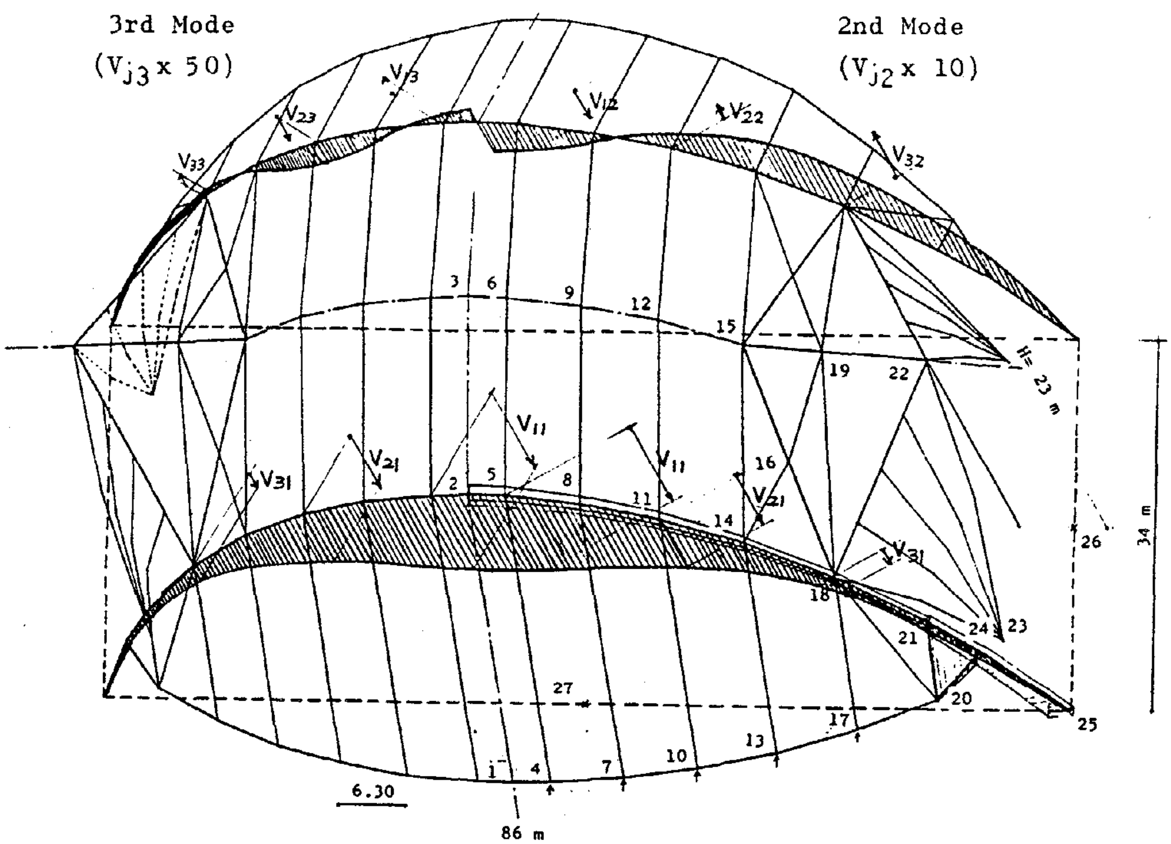


1st Mode (V_{j1} x 1)

Stiffeness Matrix $[K] = \begin{bmatrix} 5796.86 & -6334.64 & 641.55 \\ -6334.64 & 12296.33 & -5850.64 \\ 641.55 & -5850.64 & 6485.58 \end{bmatrix}$

	1st Mode	2nd Mode	3rd Mode
Period	T_1 = 0.969sec	T_2 = 0.299sec	T_3 = 0.136sec
Point	V_{j1}	V_{j2}	V_{j3}
No. 5 (j=1)	1.0928	-0.09723	0.00448
No.11 (j=2)	1.0345	-0.02616	-0.00829
No.18 (j=3)	0.9325	0.06583	0.00162

Fig - 3z MODE OF Z DIRECTION



1st Mode (V_{j1} x 1)

$$\text{Stiffeness Matrix} \quad [K] = \begin{bmatrix} 5062.32 & -6258.02 & 2707.31 \\ -6258.02 & 11785.18 & -5087.41 \\ 2707.31 & -5087.41 & 8039.23 \end{bmatrix}$$

	1st Mode	2nd Mode	3rd Mode
Period	T_1 = 0.434sec	T_2 = 0.322sec	T_3 = 0.121sec
Point	V_{j1}	V_{j2}	V_{j3}
No. 5 (j=1)	1.3336	-0.3728	0.03924
No.11 (j=2)	0.8768	0.1908	-0.06761
No.18 (j=3)	0.1718	0.8141	0.01406

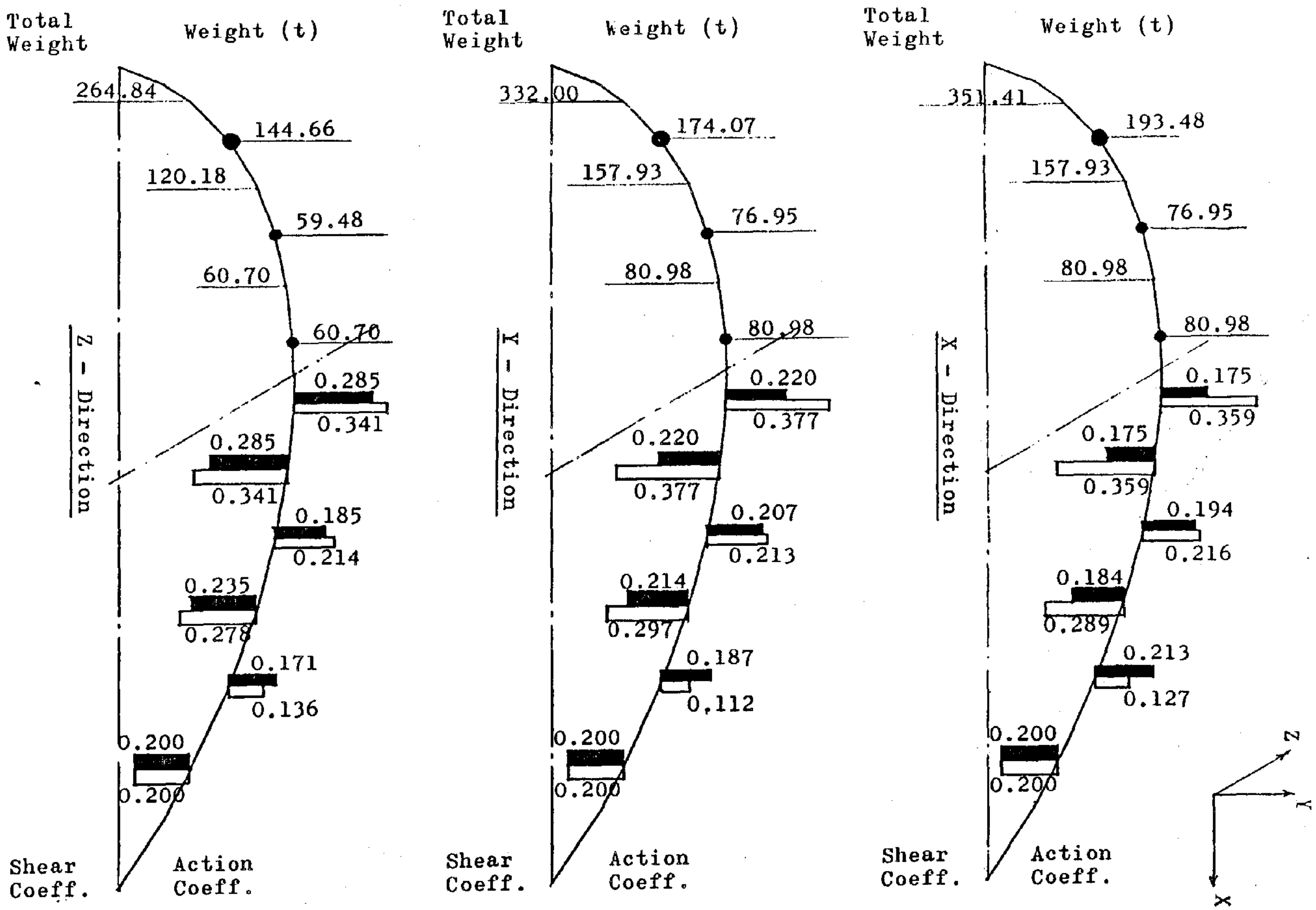


Fig - 4 COMPARISON OF MODE between ANALYSIS ▮ and CRITERIA ▯

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SEISMIC DESIGN CONSIDERATIONS FOR
LOW-RISE STEEL BUILDINGS

Melvyn H. Mark
Ferver Engineering Company
San Diego, California

Current and proposed building code seismic requirements present few problems to the structural design engineer working with low-rise steel buildings. When problems do arise, they generally can be traced to building configurations that will not accept rational, viable, and economical seismic resistant structural systems. The causes for this have more to do with individual perceptions and economic pressures than with engineering. The need for good seismic resistant systems is less often appreciated with low budget, low-rise buildings than with high-rise structures; consequently, the structural engineer is often not consulted early enough in the design process, when concepts and configurations are developed. The architect or developer may not have a commitment toward good earthquake engineering and may base his selection of the engineer more on the engineer's fee than on quality of design or service. Engineers who try to change and improve questionable systems proposed by these same clients are less likely to find themselves blessed with repeat commissions. Earthquake engineering for buildings is an art based on knowledge, experience, and talent. It achieves its best results when practiced, with a commitment, early in the design process.

Low buildings commonly involve steel in combination with other structural materials such as wood, masonry, and concrete. The one-story, wood-frame building shown in Figure 1a relies partly on plywood shear wall panels for seismic resistance; steel bracing is also incorporated, as shown. The architect in this case chose to feature the bracing. A similar situation is shown in Figure 1b, a wood building that incorporates exposed rod X-bracing. Low retail stores often have masonry shear walls on three sides and window walls at their fronts. Steel moment-resisting frames, sometimes knee braced, are frequently placed at these window walls to complete the systems. A key element in the design of these mixed systems is compatability of the deflections of the steel portions with the remainder of the construction. Sometimes it is difficult to devise strong, ductile and workmanlike details that tie the steel to the other materials.

Although steel plate shear walls have been used in a few buildings, vertical bracing systems generally can be classified as either braced frames, incorporating members that are axially loaded, or moment frames, where flexure predominates. The eccentrically braced frame, conceived by Prof. Popov at U.C. Berkeley, is a recent development and may be better suited to the taller structures; concentric bracing is most often used for the lower structures. Considering present design standards, on a relative scale of 0.67 to

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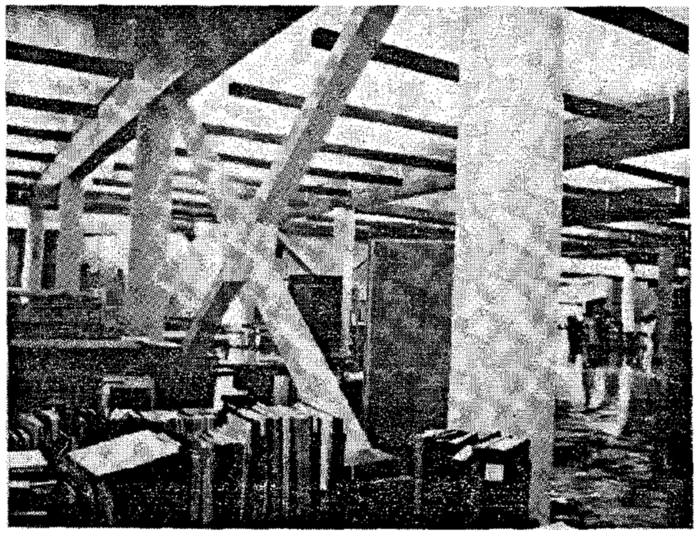
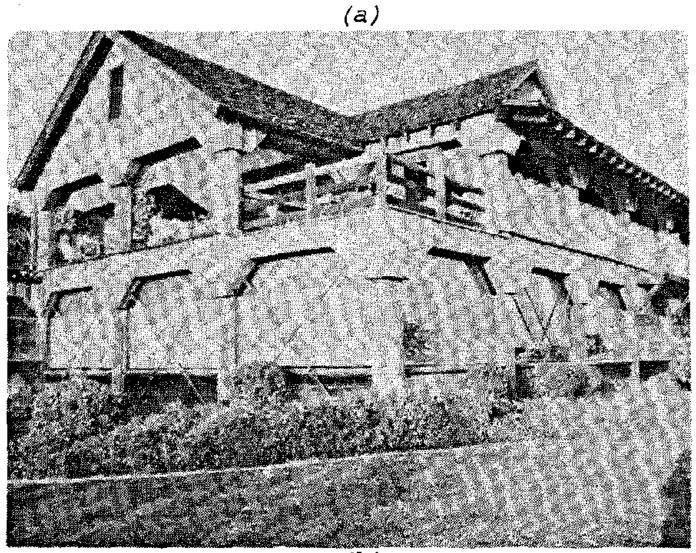
1.33, seismic design forces generally are 1.0 for buildings with concentrically braced frames without backup systems. To reduce inelastic demands on the members of the braced frames only, these forces are raised by 25%. To further reduce ductility demands on connections, they must develop the strength of the members or, alternatively, be designed without the normal 1/3 increase in allowable stresses.



(a)



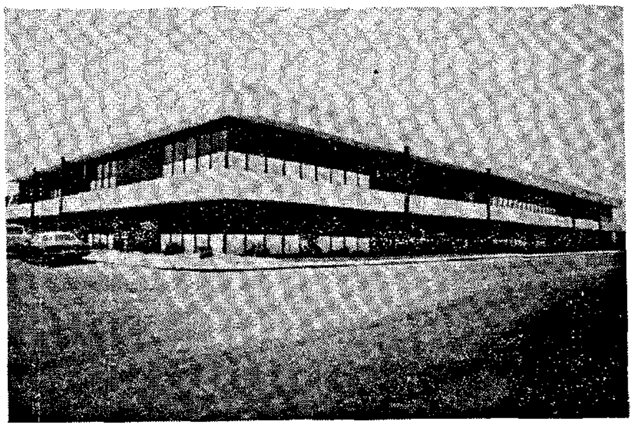
(b)

Figure 1. Examples of buildings combining steel with wood.

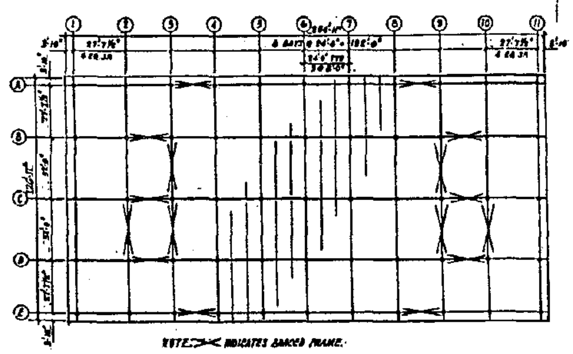
Figure 2 shows a two-story industrial office building that relies on braced frames. Figure 2b is a framing plan showing the location of the braced frames, and Figure 2c shows these frames in elevation. Single compression diagonals, X-bracing and K-bracing are used. All diagonals are double angles.

A small apartment building with one large apartment per floor is shown in Figure 3. Figure 3b shows elevations of two of the braced frames in this building, and Figure 3c is a typical connection detail incorporating rolled columns and beams with double angle diagonals. Overturning is a design problem with narrow braced frames. In this particular building, overturning effects necessitated tension connections between the columns and the foundation, as shown in Figure 3d. The response of buildings in earthquakes indicates that, although overturning may be a design problem, real buildings seldom overturn. This subject requires additional research.

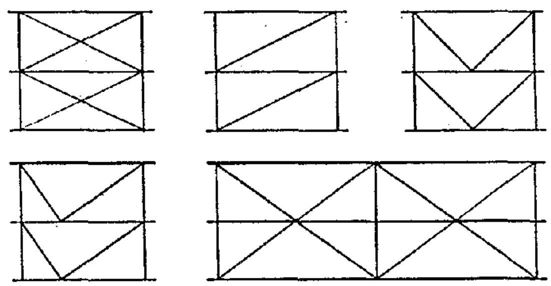
The large two-story building shown in Figure 4 is of mixed construction: the lower story and first floor above grade utilize concrete, and the second story, which is quite high, and the roof are of steel construction. Only the second story appears in Figure 4a. A typical steel braced frame connection is shown in Figures 4b and 4c. Wide flange members connected much like a



(a)



(b) ROOF FRAMING PLAN



(c) BRACING ARRANGEMENTS

Figure 2. Two-story building using braced frames.

welded truss are employed. Braced bents incorporating two columns with diagonals between them were shop fabricated as a unit. Frequently the development of the equivalent static design shear force from the braced frames to the foundation results in cumbersome details. Figure 5 is one such example. Note the large number of anchor bolts.

The previous examples illustrate several types of bracing--some slender like the angles and some less slender, like the wide flanges. Clearly, these different types have varying degrees of ductility and energy absorption, yet present design standards treat them equally. The monotonic stretching effects of rod bracing and similar systems is not addressed. The ATC-3 document does offer some moderate improvements. The possibility of columns buckling prior to other members of the bracing system attaining their strengths raises concerns. Some research on the cyclic inelastic behavior of axially loaded members has been performed; more is needed, especially with regard to system behavior. Design codes should be updated to incorporate available knowledge on this subject.

Moment frames are classified as either (1) "special" and "ductile" or (2) "ordinary." On a relative scale of 0.67 to 1.33, current seismic design forces for buildings are 0.67 when ductile frames are employed and 1.0 when ordinary frames are used. Presently employed rules for ductile frames are quite weak. Although ductile steel materials, welding inspections and lamination inspections are adequately covered, the regulations on joints are ambiguous. These state that the connection shall develop the full plastic capacity of the girders, but are silent as to the panel zone.

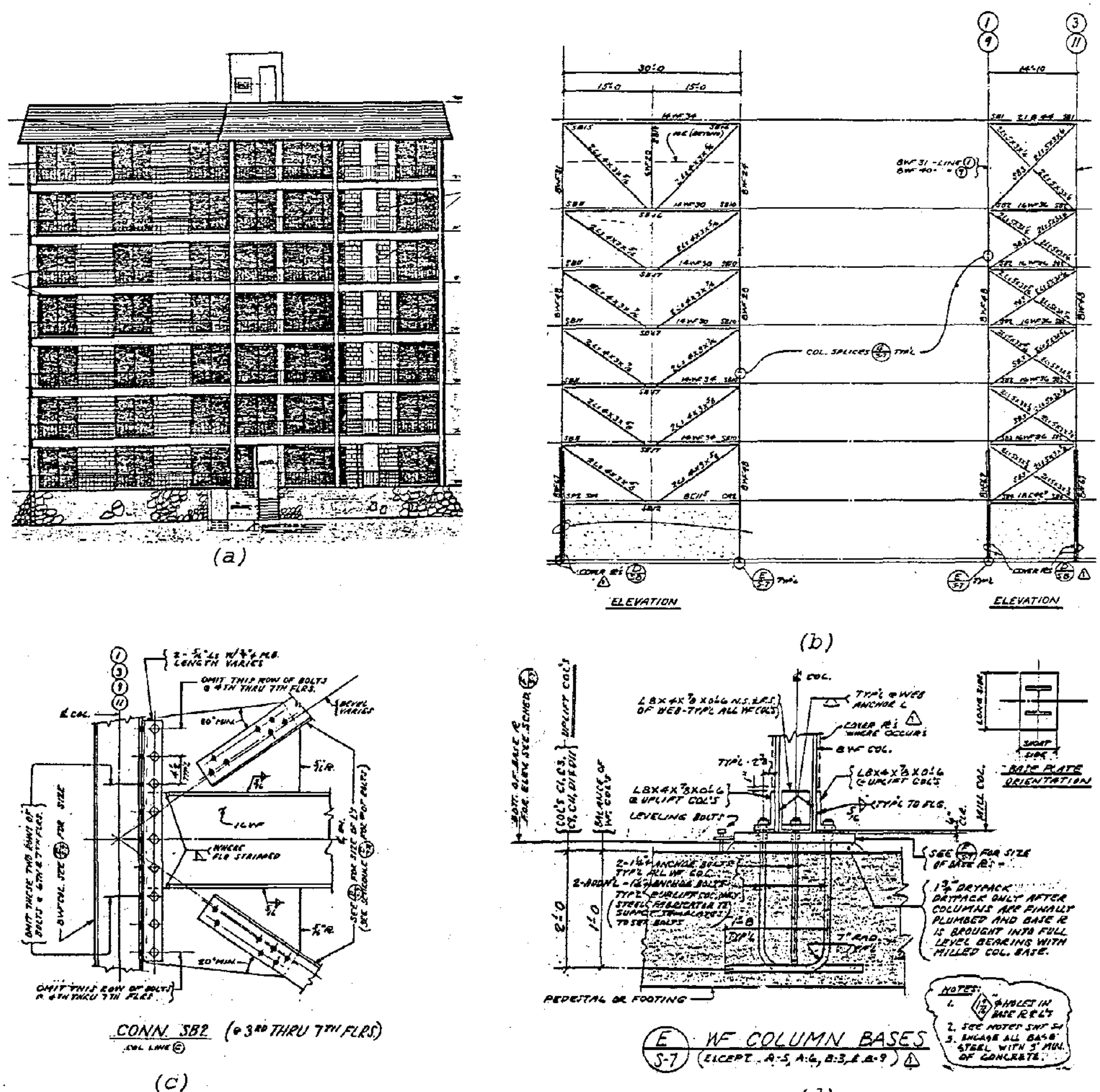


Figure 3. Apartment building using braced frames.

In the past, Ferver Engineering Company designed the panel zone for the shears resulting from plastic girder moments of opposite sense on both sides of the joint. This results in thick web doublers due to the light columns of low buildings. Recent research may show that this approach is not justified. Present design rules are also silent on the strong column-weak girder philosophy, which is generally employed by Ferver. Although this concept may be justified for tall buildings, it may not be valid for the columns of low building moment frames, which carry very small axial loads. Local buckling is addressed in the present standards, but lateral-torsional buckling is not mentioned.

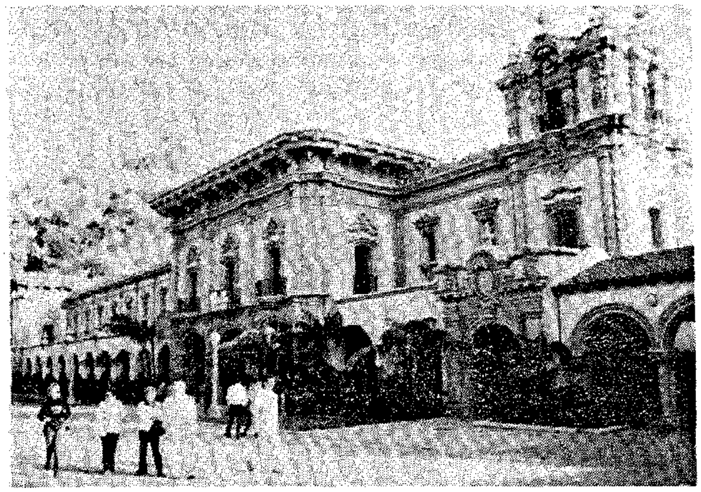
Moment frames are frequently employed on only selected column lines in low buildings. Figure 6a illustrates a plan of one such structure where the moment frames, in this case "ductile," are located only on the perimeter. Figures 6b and 6c show the finished appearance and steel frame of the building employing this plan. The columns are heavy beam shapes, not column shapes, to help in drift control.

Figures 7a and 7b illustrate building construction with a more complete ductile moment frame system. In this structure all the transverse frames and the exterior longitudinal frames are moment resistant. The interior longitudinal girders are simply connected. This avoids large weak axis bending in the interior columns and also avoids two-way moment connections, a significant economy.

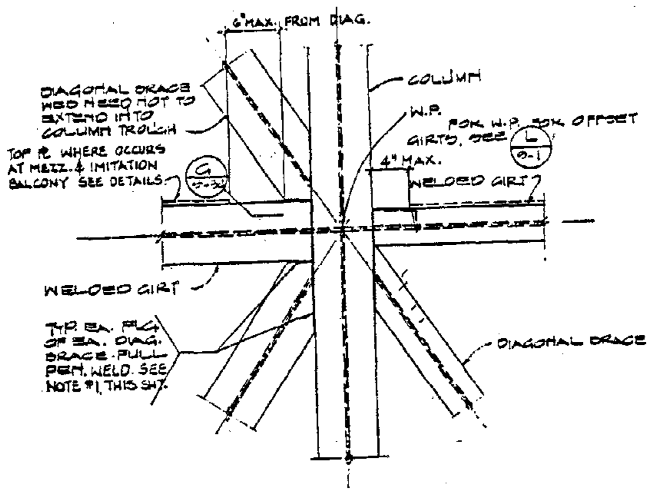
Figure 8 shows a moment connection employed when the column is only in strong axis bending. Web doublers are located on each side of the column web with a space between them. When web doublers are needed, this detail affords some economy, as horizontal stiffeners are generally not required and the connection of the floor beam is simplified. Horizontal stiffeners are frequently required in low buildings with conventional joints as the columns can be quite light. The type of joint of Figure 8 has received little testing.

Two-way moment connections are also utilized in low building frames, as illustrated in Figure 9.

Low buildings may not have basements. In such cases, flexural restraint for the bottoms of moment frame columns is needed to reduce column bending moments and to aid drift control. One way to



(a)



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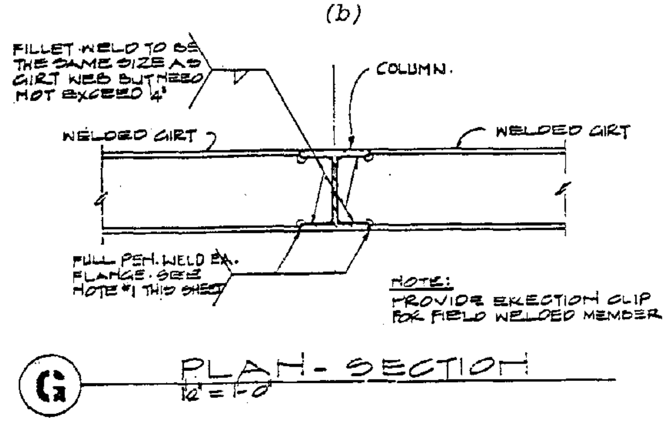


(c)

Figure 4. Two-story building of mixed concrete and steel construction.

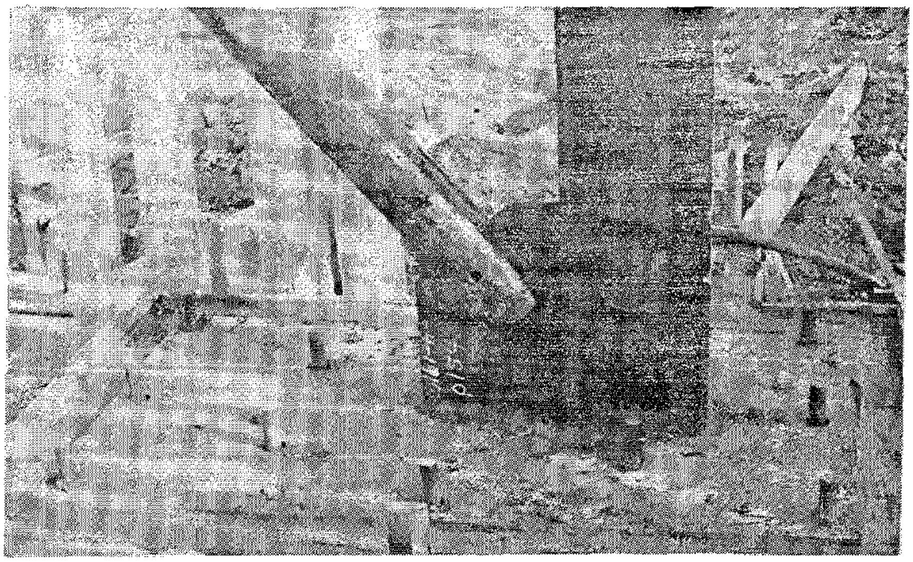
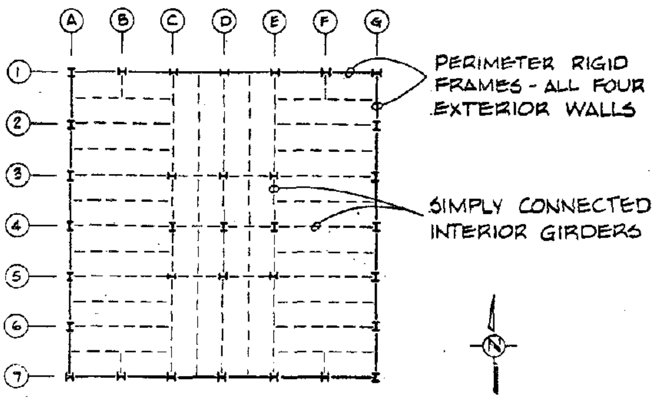


Figure 5. Example of braced frame construction details.



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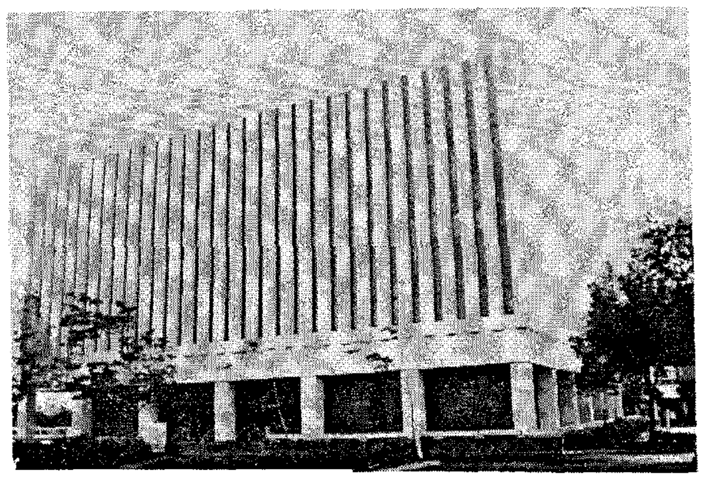
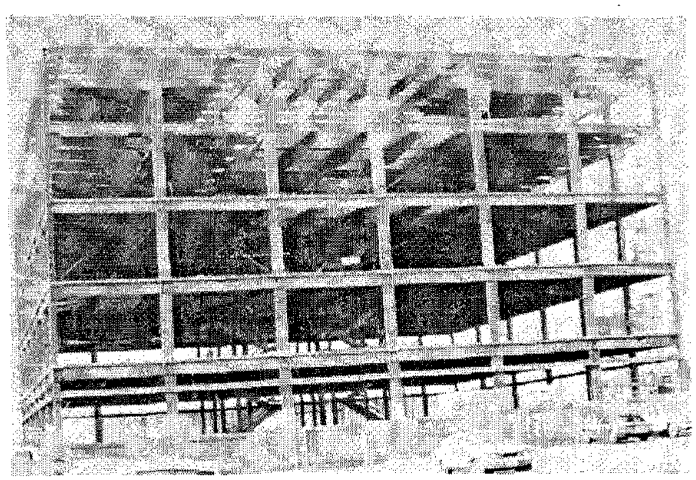


(b)

Figure 6. Building in which moment frames are located only on perimeter.



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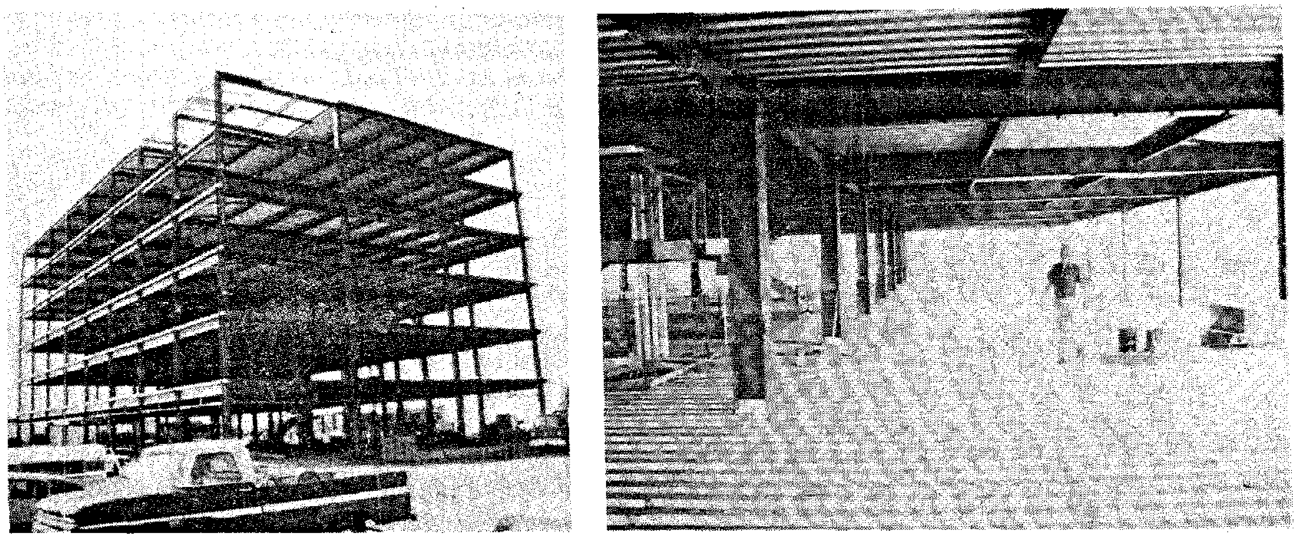


(a) (b)

Figure 7. Building with complete ductile moment frame system. All columns in single flexure.

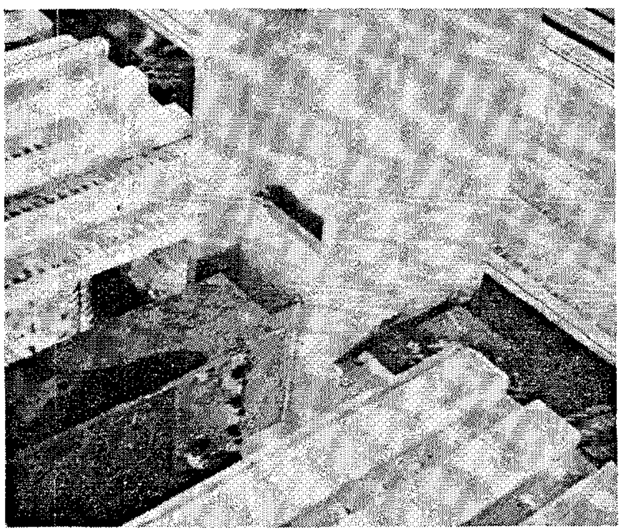


Figure 8. Example of moment connection when column is only in strong axis bending.

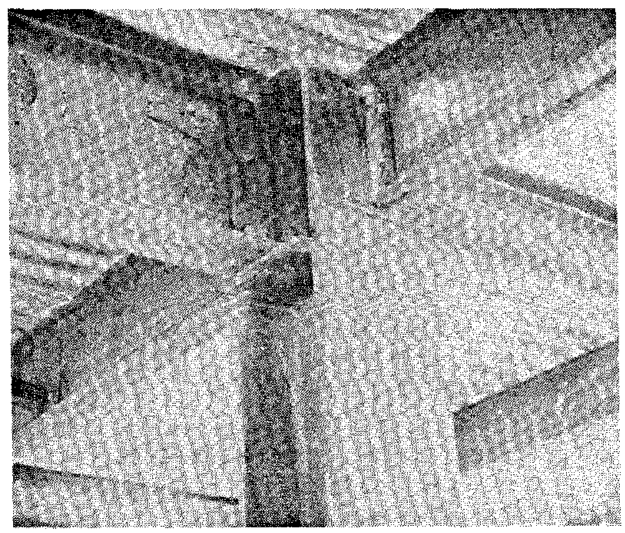


Figure 9. Example of two-way moment connections.

provide restraint can be in the form of a column-base moment connection to the foundation, as shown in Figure 10. In this detail, assessing the amount of flexural restraint offered by the footings presents a problem because soil parameters cannot be estimated reliably in most cases. The restraint is also nonlinear due to one edge of the footing lifting off the soil under high moments. Another way to provide restraint is the utilization of steel grade beams moment-connected to the bottoms of the columns, and shown in Figure 11. These beams are encased in concrete in the finished structure. They also can serve as foundation ties, which are required between pile-supported foundations.

Ferver Engineering Company employs the plastic design rules for lateral-torsional buckling assuming plastic moments of opposite sense at the ends of ductile frame girders. Frequently, bottom flange girder bracing has to be provided, as shown in Figure 12. Present design standards and ATC-3 require clarification of this item.

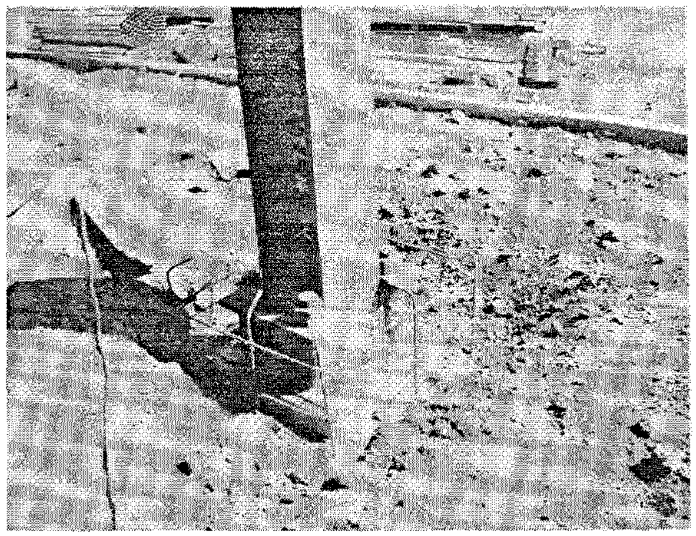


Figure 10. Example of column-base moment connection to foundation.

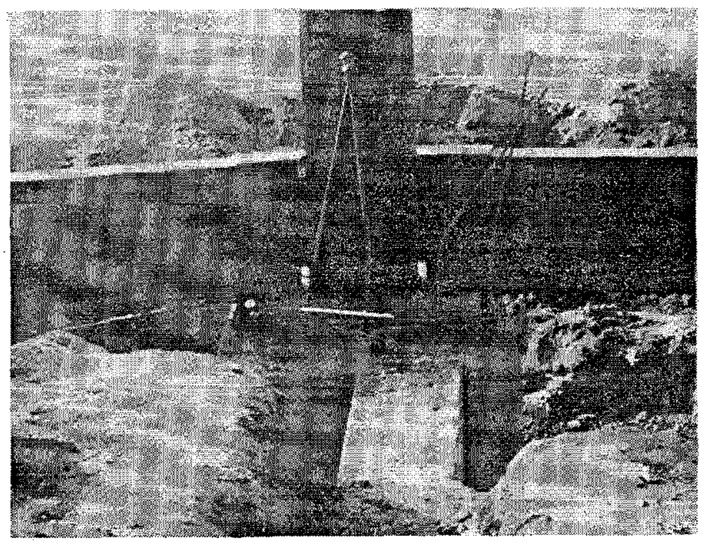


Figure 11. Example of steel grade beams moment-connected to bottoms of columns.

Ordinary or non-ductile steel moment frames are also utilized in the seismic resistant systems of low buildings. A one-story lightweight manufacturing building of this type is illustrated in Figure 13. Cross sections through this building are presented in Figure 13b. The top cross section shows the moment frames that provide the modified saw-tooth roof shown in the lower cross section. The "girders" of the moment frame are actually trusses. Figure 13c shows a connection of the trussed girders to the columns, and Figure 13d is an interior view. There is some controversy as to the use of non-ductile ordinary steel moment frames in areas of high seismicity; this building was chosen to illustrate an extreme example of this. Considering collapse, these structures have performed well in strong seismic events. Although this building was designed by present standards, calculations indicate that when the first members of the moment frame reach their maximum strength, the base shear would be about 1/4 g and the roof force would be about 1/3 g (based on the roof weight). Considerable redundancy and elastic strength remain after the initial attainment of member strength or first yield.

Story drift, which is the relative deflection between building levels, frequently governs over stress or strength considerations in the design of steel moment frames. Ferver Engineering Company's approach to design of these frames when drift does govern is shown in Figure 14. Conservatively, these drift calculations are based on a model that incorporates only the stiffness of the steel girders and columns; a "bare frame" model. The figures illustrate a four-story building with a triangular force distribution. Period is a function of the target drift. The approximate resulting periods are computed for present standards and the ATC-3 requirements as well as for damage control based on a realistic spectral velocity for a probable earthquake in a zone of high seismicity. The allowable drift for this last case is based on limiting interior partition damage to a moderate amount.

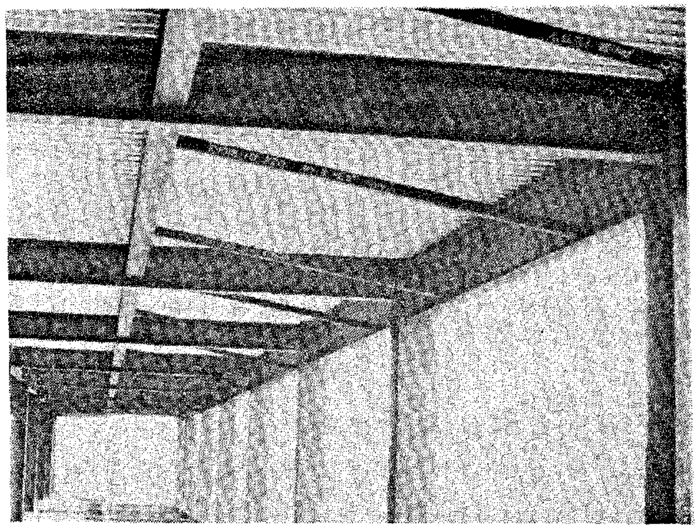


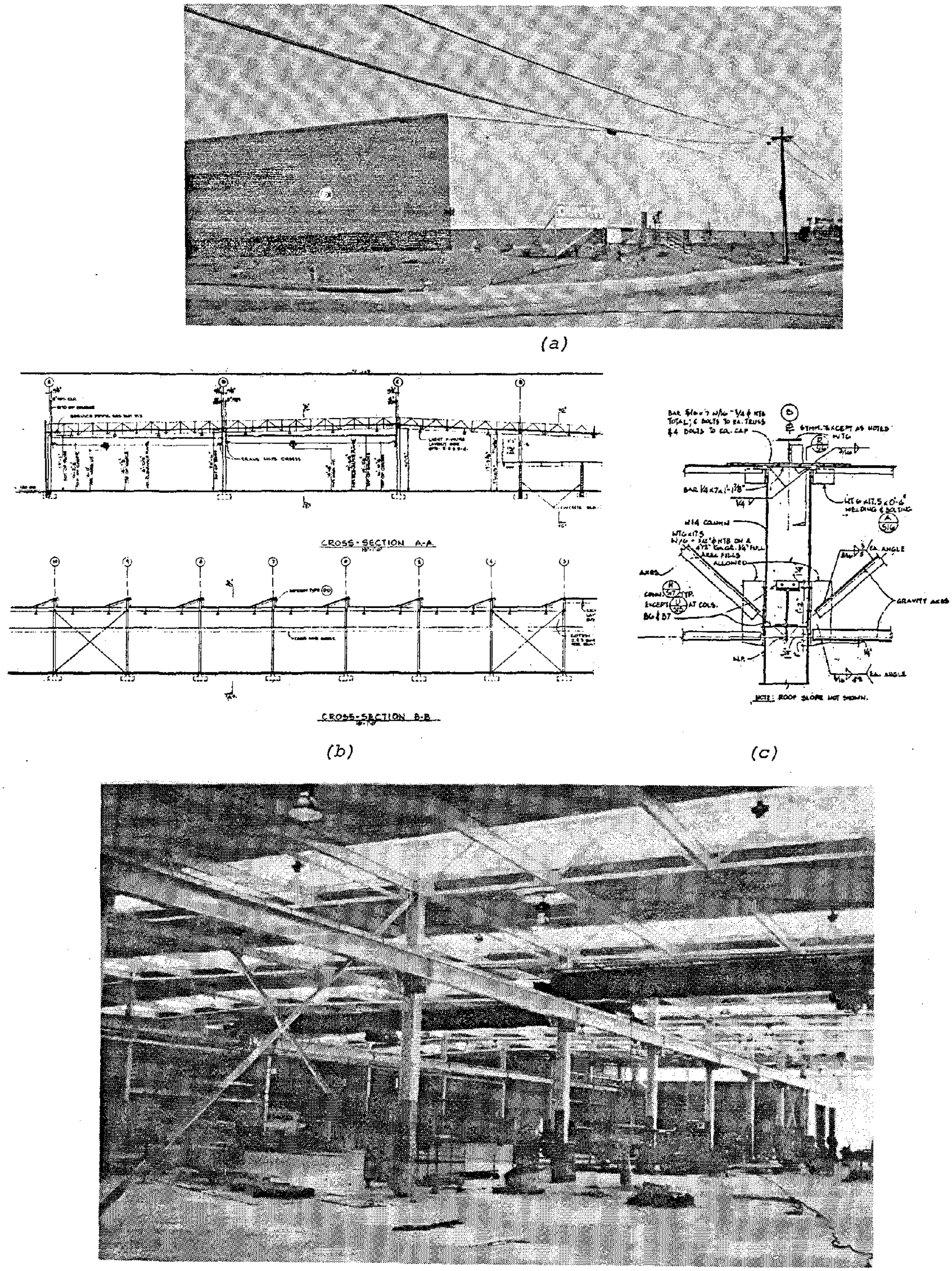
Figure 12. Example of bottom flange girder bracing.

The table at the end of Figure 14 compares the resulting periods and relative frame stiffness obtained by the three criteria. Note the wide difference in frame stiffnesses. Clearly, a need for reassessing drift limitations is warranted. The first step in this assessment would be philosophical: a clear consensus needs to be developed regarding the objectives of the drift limit; in particular, damage control, overall frame stability, and member strain. These philosophical issues were not adequately addressed in the development of both ATC-3 and present standards. In addition to arriving at this philosophical consensus, more technical research is needed, especially for damage control.

Of course, we would not use the periods developed by Figure 14 for stress and strength design. Forces for these considerations would be based on a period derived considering stiffnesses in addition to the bare frame.

Present standards require a consideration of dynamics for irregular buildings. Figure 15 illustrates a three-story structure, irregular in plan and vertical configuration. Originally, it was designed utilizing only braced frames and a lot of judgment. The braced frames at the front of the building were later revised to ordinary moment frames after the structural design was largely complete, as shown in Figure 16b. The redesign was also done with a lot of judgment; however, it was felt that to obtain approval for construction with this additional structural irregularity, a dynamic analysis was necessary. The only thing proved by the dynamic analysis was that judgment worked in this case; there were very few changes after the analysis.

The costs of the redesign were 45% of the original structural design fee. Had dynamic analysis been used for this building in the original design



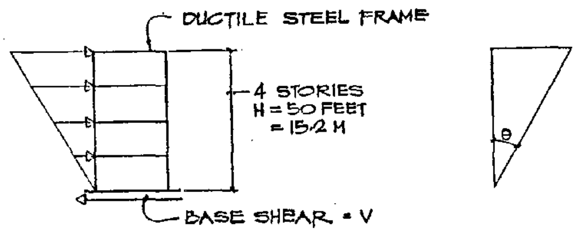
(a)

(b)

(c)

(d)

Figure 13. Lightweight manufacturing building with non-ductile steel moment frames.



BUILDING & FORCES — DESIGN DRIFT

IT CAN BE SHOWN THAT: $T = .783\left(\left[1+\frac{1}{N}\right]\frac{WH\theta}{V}\right)^{1/2}$

BUT INCREASE V BY 20% TO ACCOUNT FOR TORSION, P·Δ AND ACTUAL DRIFTS BEING LOWER THAN DESIGN LIMIT.

$$T = .783\left(\left[1+1/4\right]\frac{W\times 50\times\theta}{1.2V}\right)^{1/2} = 5.65\left(\frac{W\theta}{V}\right)^{1/2}$$

PRESENT CODE (SEAOC/UBC)

ZONE OF HIGHEST SEISMICITY $Z = 1.0$

$I = 1.0$ $\quad S = 1.2$

$$V = ZIKCSW = 1\times 1\times .67\times\frac{1}{15T^{1/2}}\times 1.2W = \frac{.0536\,W}{T^{1/2}}$$

$\theta = .67\times .005 = .00335$

$$T = 5.65\left(\frac{W\times .00335}{.0536\,W/T^{1/2}}\right)^{1/2} \qquad T = 1.58^{s}$$

ATC-3

ZONE OF HIGHEST SEISMICITY $A_v = 0.4$

$S = 1.2$

$$V = C_D C_S W = C_D\frac{1.2A_vS}{RT^{2/3}}W = 5.5\,\frac{1.2\times .4\times 1.2}{8\times T^{2/3}}W = \frac{.396\,W}{T^{2/3}}$$

$\theta = .015$

$$T = 5.65\left(\frac{W\times .015}{.396\,W/T^{2/3}}\right)^{1/2} \qquad T = 1.15^{s}$$

DAMAGE CONTROL FOR A PROBABLE EARTHQUAKE

ZONE OF HIGHEST SEISMICITY. USE SPECTRAL MODAL ANALYSIS. EARTHQUAKE WITH MODERATE POSSIBILITY OF BEING EXCEDED. SPECTRAL VELOCITY, $S_v \approx 18$ in/s (0.46 m/s)

SPECTRAL ACCELERATION, $S_a = \frac{18}{61.5T} = \frac{.293}{T}$

$$V \approx .9S_aW \approx .9\times\frac{.293}{T}W \approx \frac{.26\,W}{T}$$

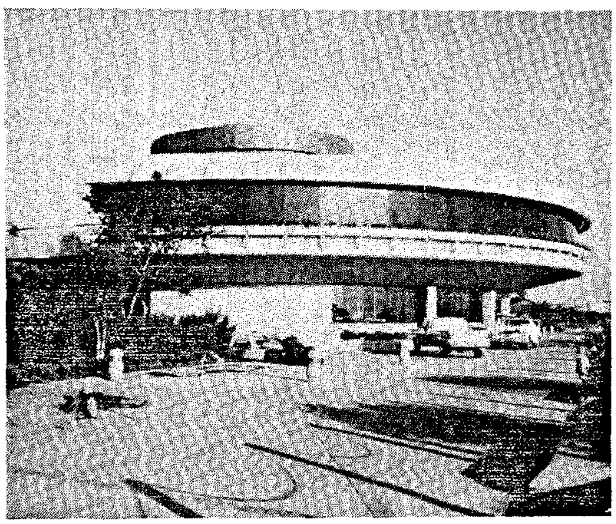
FOR DAMAGE CONTROL, $\theta \approx .0075$

$$T = 5.65\left(\frac{W\times .0075}{.26\,W/T}\right)^{1/2} \qquad T = .92^{s}$$

COMPARISON

PROCEDURE	T	FRAME RELATIVE STIFFNESS ∝ $1/T^2$
PRESENT CODE	1.58	1.0
ATC-3	1.15	1.9
DAMAGE CONTROL	.92	3.0

Figure 14. Design of steel moment frames considering story drift.



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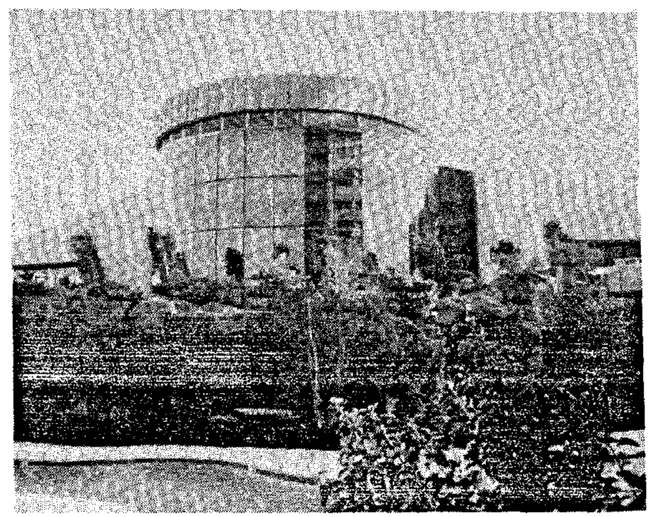
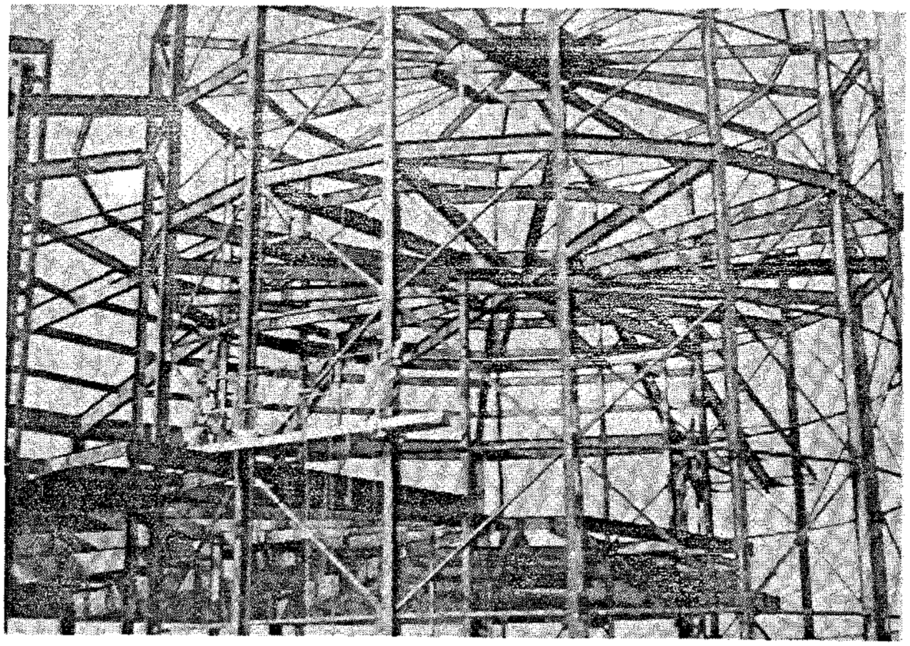


(b)

Figure 15. Irregular building.

process, the extra costs would have been about half as much. Also, review of the design by a small committee of experts, "peer review," would have been appropriate in this case, and would have averted dynamic analysis. Costs of peer review for this building would be about 25 percent of the redesign costs.



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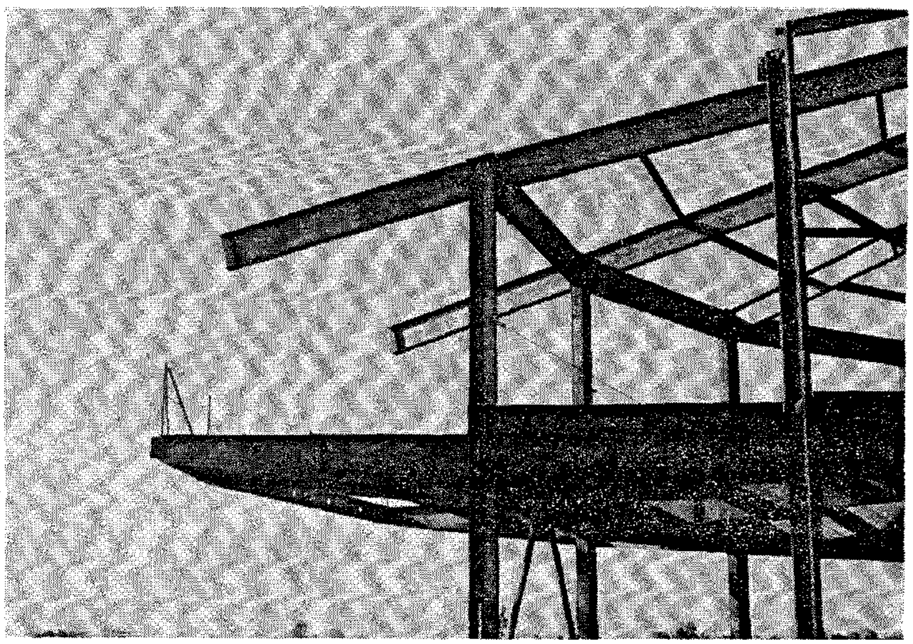

(b)

Figure 16. Frames for building shown in Figure 15.

Corrugated steel decking is often used with steel framing in floors and roofs, as shown in several of the illustrations. For roofs, it is often not topped with concrete or other fill material. These corrugated steel roof diaphragms can be more flexible than some of the vertical resisting elements of low buildings such as masonry or concrete shear walls, a point not normally considered in design.

It is hoped that this review has presented some insight on U.S. practice for low steel buildings and some suggestions to improve that practice.

CURRENT DESIGN AND CONSTRUCTION PRACTICES: LOW-RISE CONCRETE BUILDINGS

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Low-rise reinforced concrete construction embraces the majority of the concrete construction built in the western United States. Its use includes commercial structures, industrial structures, multi-story housing and hotels, warehouses, communication structures, office buildings, hospitals and clinics, and a great many special use buildings and structures. To illustrate the wide variety of construction types to which low-rise concrete construction is applicable, see Table 1 (Table 50 of the Uniform Building Code). Note that although low-rise concrete may be used in any of the construction types shown on the table, only Type I or Type II fire-resistant construction may be used for mid-rise construction, and only Type I construction may be used for high-rise construction.

Figure 1 is a seismic risk map from the 1982 Uniform Building Code. Areas to which this paper is applicable have been cross-hatched. Note that the area is entirely within the bounds of Seismic Zones 3 and 4, both areas of high seismic risk. As a matter of interest, note that the Zone 3 follows the Northern California boundary; this is for administrative purposes. A geologic map would indicate it as being more or less parallel to the Zone 1 line immediately to the north. Design details and construction methods are significantly different for buildings in Zones 3 and 4 from those used in buildings in Seismic Zones 1 and 2.

In addition to its economy, the reasons for the choice of reinforced concrete are its properties of long life, fire resistance, security, load carrying ability, nuclear shielding ability, and the ability to store and dissipate heat.

The entire spectrum of types of concrete construction are commonly used for low-rise construction. They break down into the following construction types:

1. Poured-in-place construction: Concrete is placed in forms in the position where it is to be used. This concrete is either reinforced with reinforcing bars or mesh, or may be prestressed using one of several post-tensioning systems.

2. Tilt-up construction: Concrete is precast on the site and transported to the position where it is to be used. The name derives from the original application where concrete was cast on slab-on-grade construction immediately adjacent to the location it was to be used and actually tilted up into place. The name is now applied to any site-cast concrete and used primarily for wall panels. See Figure 2.

Table 1. Maximum Height of Buildings

OCCUPANCY	TYPES OF CONSTRUCTION								
	I	II			III		IV	V	
	F.R.	F.R.	ONE-HOUR	N	ONE-HOUR	N	H.T.	ONE-HOUR	N
	MAXIMUM HEIGHT IN FEET								
	Unlimited	160	65	55	65	55	65	50	40
	MAXIMUM HEIGHT IN STORIES								
A-1	Unlimited	4	Not Permitted						
A) 2-2.1	Unlimited	4	2	Not Permitted	2	Not Permitted	2	2	Not Permitted
A) 3-4	Unlimited	12	2	1	2	1	2	2	1
B) 1-2-3[1]	Unlimited	12	4	2	4	2	4	3	2
B-4	Unlimited	12	4	2	4	2	4	3	2
E[2]	Unlimited	4	2[2]	1	2[2]	1	2[2]	2[2]	1
H-1	Unlimited	2	1	1	1	1	1	1	1
H) 2-3-4-5	Unlimited	5	2	1	2	1	2	2	1
I-1	Unlimited	3	1	Not Permitted	1	Not Permitted	1	1	Not Permitted
I-2	Unlimited	3	2	Not Permitted	2	Not Permitted	2	2	Not Permitted
I-3	Unlimited	2	Not Permitted[3]						
M[4]	See Chapter 11								
R-1	Unlimited	12	4	2[5]	4	2[5]	4	3	2[5]
R-3	Unlimited	3	3	3	3	3	3	3	3

[1]For open parking garages, see Section 709.
[2]See Section 802 (c).
[3]See Section 1002 (b)
[4]For agricultural buildings, see also Appendix Chapter 11.
[5]For limitations and exceptions, see Section 1202 (b).

N—No requirements for fire resistance
F.R.—Fire resistive
H.T.—Heavy Timber

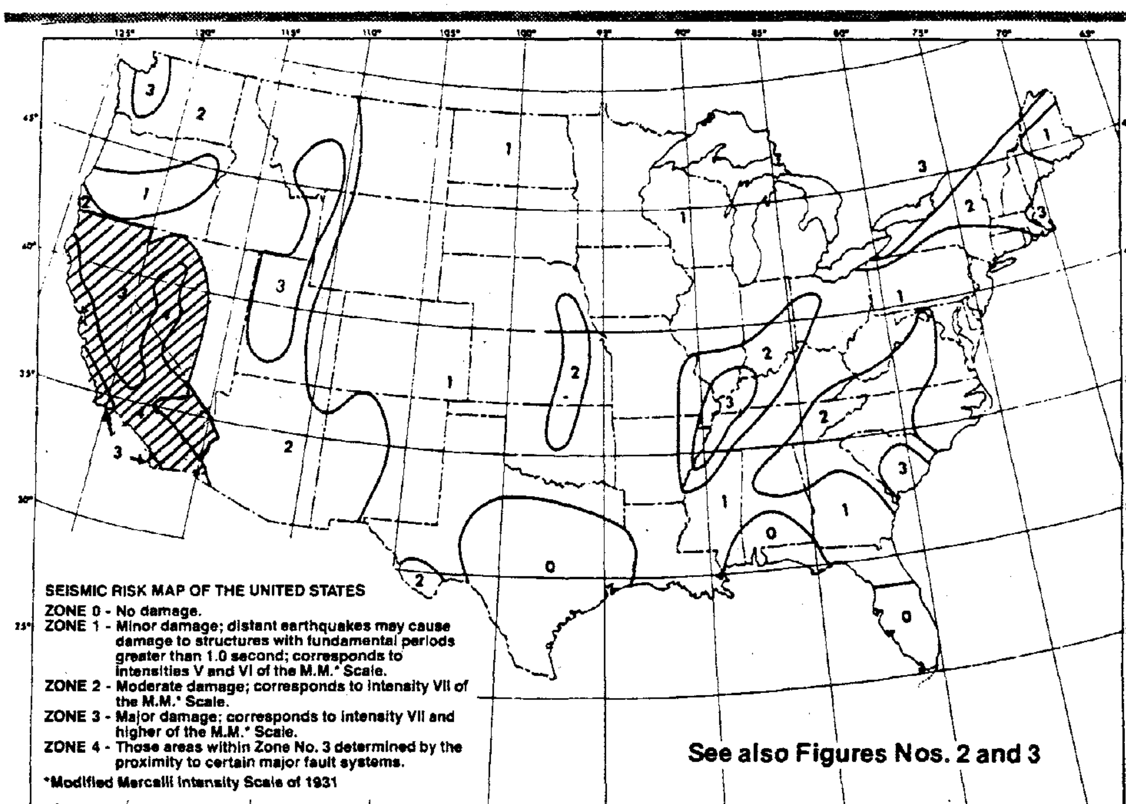


Figure 1. Seismic Zone map of the United States.

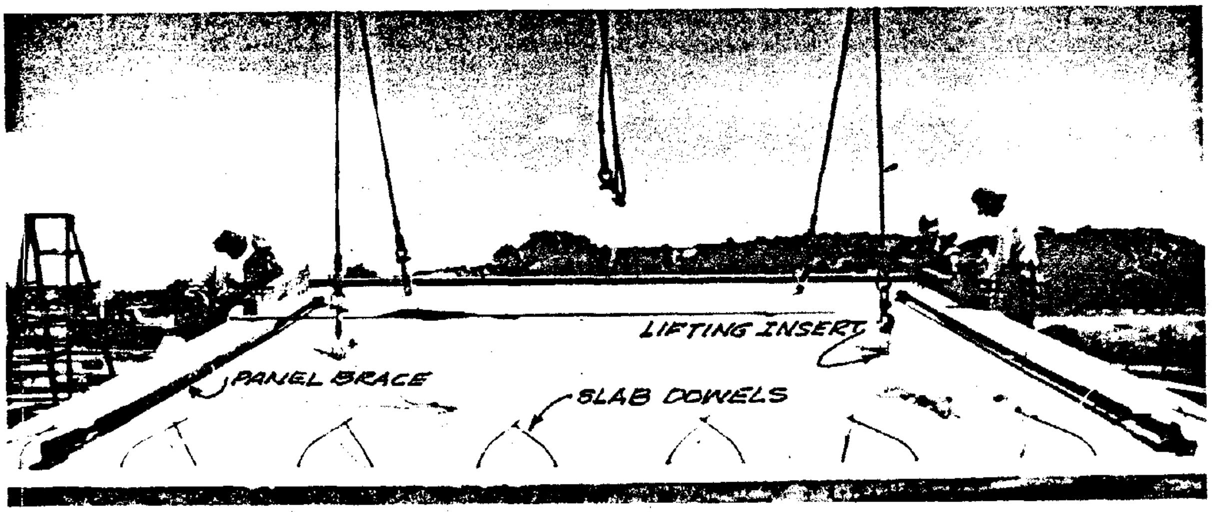


Figure 2. Tilt up construction--a typical installation.

3. Precast and precast, prestressed concrete: Concrete units are cast in a casting yard on casting beds and transported to the jobsite. Casting beds for wall panels may be of any of a variety of materials including wood, steel, fiberglass, or concrete. Casting beds for prestressed structural members are usually of steel.

4. Hollow concrete block (or hollow unit masonry) construction: Small modular concrete elements are mass-produced at a factory and laid up as masonry. The cells are normally reinforced and grouted. See Figure 3.

There are various types of low-rise concrete construction. For purposes of this discussion, three broad categories have been chosen: one-story construction; multi-story structures with heavy load-carrying capacity such as industrial manufacturing, operating, and storage facilities; and multi-story structures with light load-carrying capacities such as commercial office, retail sales, and parking structures.

ONE-STORY CONSTRUCTION

Most low-rise concrete construction falls into this category. Concrete is used primarily for slab-on-grade and exterior walls, which in most cases are tilt-up concrete or concrete unit masonry. It is generally more economical to construct the roof of steel or wood. Concrete is used only for the roof construction in special situations where fire resistance or security considerations are involved, and in most areas it will be cast in place. Precast, prestressed concrete is generally economical only in locations close to a casting yard.

Concrete strengths for this type of construction are generally in the 3,000 pounds per square inch (psi) range for both walls and foundations. Slabs on grade may use concrete strengths as low as 2,000 psi if wear resis-

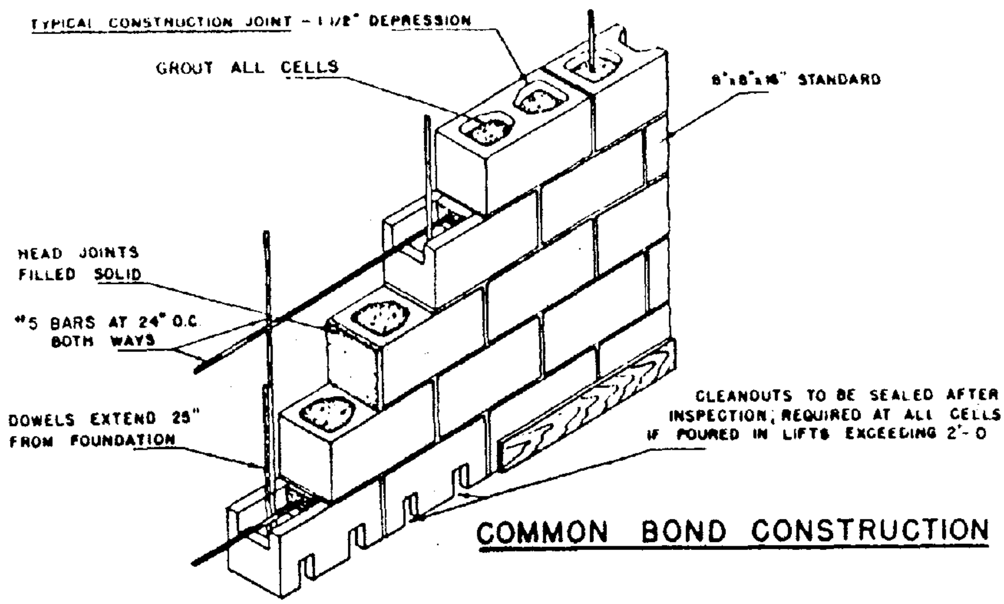


Figure 3. Typical reinforced hollow unit masonry construction.

tance is not required. Reinforcing steel is almost without exception ASTM A615 Grade 60, having a yield strength of 60,000 psi, or Grade 40, having a yield strength of 40,000 psi.

Design Considerations

Many one-story buidings are quite large, necessitating consideration for temperature expansion. Design for expansion stresses is not normally performed, but, especially in the warm interior areas, expansion joints must be considered in buildings greater than 200 ft in length.

Vertical load design is normally straightforward and simple, as the only loads to be considered are the weight of the structure plus a live load of 20 pounds per square foot (psf). (In mountain areas, where snow loads of as much as 240 psf must be accommodated, there are not many large structures.)

Current seismic design is based on the 1980 "Recommended Lateral Force Requirements" of the Structural Engineers Association of California. As it applies to low, single-story concrete, the simplified static force equivalent design provisions are used. Base shear is obtained using the formula

$$V = ZIKCSW$$

where Z = zone factor; I = importance factor; K = horizontal force factor; C = numerical coefficient; S = soil factor; and W = total dead load.

Almost without exception, buildings in this category are shear wall buildings having very short periods. Therefore, V = 0.14W or 0.19W in Zone 4, or V = 0.10W or 0.14W in Zone 3.

The appendix contains an example of lateral force design, excerpted from Fundamentals of Reinforced Masonry Design prepared by Dr. Ajit Virdee for the Concrete Masonry Association of California and Nevada. It is included because it is a simple statement of equivalent static force procedure. The example is for concrete masonry construction utilizing working stress design. Had the walls been concrete and had strength design been used, a load factor of 2 would have been used for shears and applied over 0.8 the length of the shear element, and a load factor of 1.4 would have been used for moment.

The SEAOC Seismology Committee is currently in the process of rewriting its recommendations, and it appears that the base shear formula will be revised to read:

$$V = ZICW/Rw.$$

In the new format, Z remains as it was, I is proposed to receive slightly lower values, C includes the effect of the present site period, S is numerically greatly changed, and Rw is a "structural quality factor" roughly equal to 8 times the present K values. The magnitude of the base shear to be designed for in buildings of the category being discussed is virtually unchanged.

MULTI-STORY STRUCTURES WITH HEAVY LOAD-CARRYING CAPACITY

Poured-in-place reinforced concrete is well suited to multi-story structures with heavy load-carrying capacity. Although framing and forming techniques vary according to building use, spans are usually short, about 30 ft maximum, and the construction is heavy. One-way slab and flat beam construction (Figure 4) seems to be well adapted to telephone equipment buildings (125 to 150 psf live and equipment loads); concrete joist and flat beam construction (Figure 5) is being utilized in a recently designed winery (400 psf); and flat slabs with dropped panels have been used in the design of a storage warehouse (450 psf).

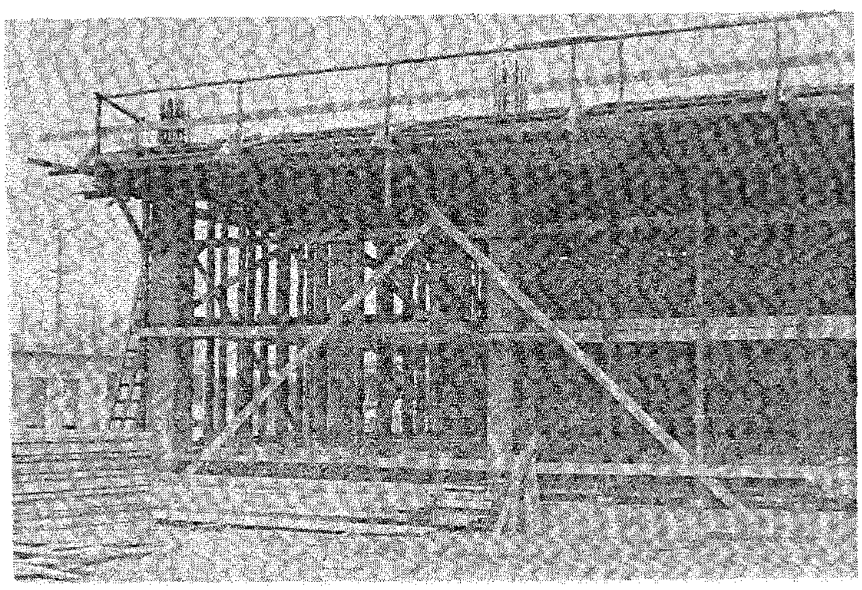


Figure 4. Forming method for one-way slab and flat beam construction in a telephone equipment building.

Concrete strengths for one-way slab and concrete construction are usually in the 4,000 psi class. ASTM A-615 Grade 60 reinforcing steel (60,000 psi yield) is usually used. Because these structures are so heavy, the practical method of resisting lateral forces is by utilizing shear walls. This results in stiff, short-period structures having K values of 1.0 or 1.33. Consequently, the appropriate lateral force design method is by static force equivalents, as it was in the case of the single-story construction. Distribution of base shear is illustrated in Figure 6.

An accepted method of distributing these shears is as follows. Centers of mass for each story are computed as they were in the example for one-story construction. Centers of rigidity are computed for each story as they were previously. Torsional moments, however, are computed using the center of mass of not only the floor under consideration, but the stories above as well. In other words, the problem is treated as though several one-story buildings were placed one above the other. Story shears are allocated to individual wall elements according to their rigidities, as previously computed.

Overturning moments in a building wall are computed using the differences in story shear between successive stories and applying them as loads at each floor line. These moments must be resisted in the structure in addition to the local moments in individual elements caused by story shear alone. This method is reasonable for many structures, but must be used with caution, as it is an approximate method. With the widespread use of computers, frame programs with provision for shear deflections are increasingly available and yield more accurate results.

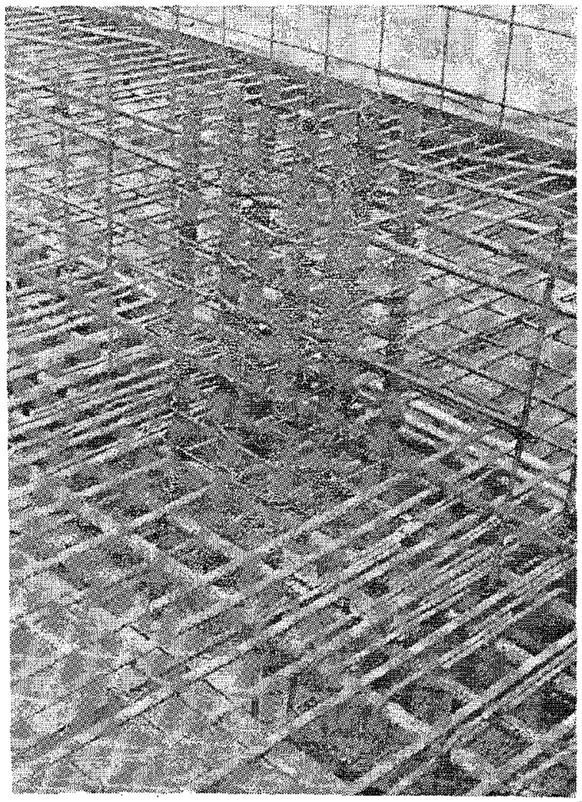


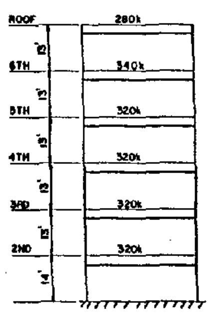
Figure 5. Concrete reinforcing arrangement at column-beam intersection.

DISTRIBUTION OF BASE SHEAR AT VARIOUS FLOOR LEVELS

Given: A six story reinforced concrete masonry building (shear walls) located in San Francisco area.

Each floor load is known. Plan dimensions are 100' x 60', longer dimension in the north-south direction.

Find: Lateral forces due to seismic loads at each story level.



SOLUTION:

Determination of base shear

V = ZIKCSW

Where:

$Z = 1.0$

$I = 1.0$

$K = 1.33$

$C = \frac{0.067}{\sqrt{T}} < 0.12$

$S = 1.5$

$CS < 0.14$

$W = 4(320.0) + 340.0 + 280.0 = 1900$ k.

$T_{e-w} = \frac{.05 \times 79}{\sqrt{100}} = 0.395$ and $C = \frac{.067}{\sqrt{.395}} = 0.11$

$CS_{e-w} = 0.14$ (governs)

$T_{n-s} = \frac{.05 \times 79}{\sqrt{60}} = 0.51$ and $C = \frac{.067}{\sqrt{.51}} = 0.094$

$CS_{n-s} = 0.094 \times 1.5 = 0.14$

$V_{e-w} = V_{n-s} = (1.0)(1.0)(1.33)(0.14)(1900)$

$= 354.6$ kips (say 360.0 kips)

Distribution of base shear in both directions

$F_t = 0$ (T is less than 0.7 seconds)

$V = 360.0^k$

Floor Level	h_x (ft.)	w_x (kips)	$h_x w_x$	F_x (kips)	
2	14.0	320.0	4,480.0	18.44	
3	27.0	320.0	8,640.0	35.56	
4	40.0	320.0	12,800.0	52.70	
5	53.0	320.0	16,960.0	69.82	
6	66.0	340.0	22,440.0	92.38	
Roof	79.0	280.0	22,120.0	91.10	
Total			87,440.0	360.0	OK

Figure 6. Distribution of base shear at various floor levels.

MULTI-STORY STRUCTURES WITH LIGHT LOAD-CARRYING CAPACITY

Most multi-story structures with light load-carrying capacity are office, retail sales, or parking structures. Apartment buidings occasionally fall in this range. Almost any construction type will be found. Prestressed concrete will almost certainly be used in the parking structures. Because structural steel currently seems to be more economical for most of the above occupancies, the choice of concrete must be made for some special property, or some local cost advantage.

Concrete strengths for this type of construction are generally in the 4,000 to 5,000 psi range for cast-in-place concrete, and 5,000 to 6,000 psi for precast and prestressed concrete. Reinforcing bars will be in the 60,000 psi yield range. Prestressing cable is usually ASTM A-416 Grade 40 or 270K. Lateral load resisting systems will be either shear wall or ductile moment-resisting space frame.

State of California Office Building

To illustrate current methodology, the recently completed four-story, 180,000 sq ft State Office Building 1B will be followed through the design process. This building was one of three "energy efficient" buildings built by the State of California to utilize various energy saving systems. This particular building was built of concrete to have the structure as a "heat sink."

The plan, including the 24-ft-by-24-ft column spacing, was predetermined by the State Architect's Office, based on the needs of the Energy Commission, for whose use the building was intended. The function of Rumberger-Haines-Virdee Associates, Structural Engineers, then, was to take their project development plans and provide workable construction documents. They had already determined that a ductile moment-resisting space frame should be used and that an expansion joint would be required in the location shown on the plans. The structure was originally conceived as a two-way slab-and-beam construction. Based on economic studies, the structural engineering firm determined that a waffle slab would be more economical, but they did not convince the State Architect's Office that they could design for temperature change stresses and eliminate the awkward expansion seismic joint near the center of the building. The 30-in. diameter concrete columns used were dictated by architectural design.

The basis of structural design was Title 24 of the California Administrative Code, which essentially was the 1976 Uniform Building Code.

Structural Information

Pertinent structural information is as shown in Table 2.

Design Procedure

The first order of business was to proportion the frame and to select floor thicknesses and column sizes that the work of the design team could

Table 2. Structural information

Foundation Data:	
Foundation Type:	Pile, end bearing
Pile Capacity:	40 ton
Length to Bearing:	35-45 ft
Pile Type:	Precast, prestressed
Building Data:	
Floor Construction:	Waffle slab
Columns:	Spiral
Structural Frame:	Ductile, moment resistant
Concrete Data (all concrete normal weight):	
Concrete Strengths:	
General use:	4,000 psi
Precast piles:	5,000 psi
Precast architectural:	5,000 psi
Reinforcing Steel Data:	
Reinforcing Bars:	ASTM A-706 ductile, 60 ksi yield
Alternate:	ASTM A-615, Grade 60 with special chemical and mechanical properties
Prestressing Strand:	ASMT A-416, Grade 270K
Design Live Loads:	
Penthouse Roof:	20 psf
Main Roof:	50 psf
Office Spaces:	80 psf (file load)
Public Spaces:	100 psf
Mechanical Penthouse:	125 psf
Lateral Forces:	
Seismic Zone:	III
Occupancy Factor:	I = 1
K Value:	0.67

proceed. From the outset, it was obvious that the 30-in. diameter architectural columns would be adequate and that the main problem was to select a slab depth that could easily accommodate frame moments and shears. From past experience it was known that a waffle slab thickness of 18 in. would easily accommodate the vertical loads; what was not known was how much additional depth would be required to reasonably accommmodate the laterals. This problem was actually solved by the economies of waffle pan construction and the fire requirements of minimum slab thickness. A minimum of 3 in. of slab thickness was required for fire. If a pan depth of 14 in. could not be used, then the next economically available depth was 18 in., giving us a floor thickness and beam depth of 21 in. Typical waffle slab layout is shown in Figure 7.

An interesting point arose during the preliminary design phase. For many years a form of the Manney-Goldberg method has been found useful in estimating column stiffnesses. (See Design of Multi-Story Reinforced Concrete Buildings for Earthquake Motions by Blume, Newmark, and Corning.) In the case of irregular structures (see Figure 8) such as this one, this method would be used to assign a stiffness to each column in each direction in plan for the purpose of computing a center of rigidity. In this particular case the system yielded ridiculous answers. It appeared that what the analysis

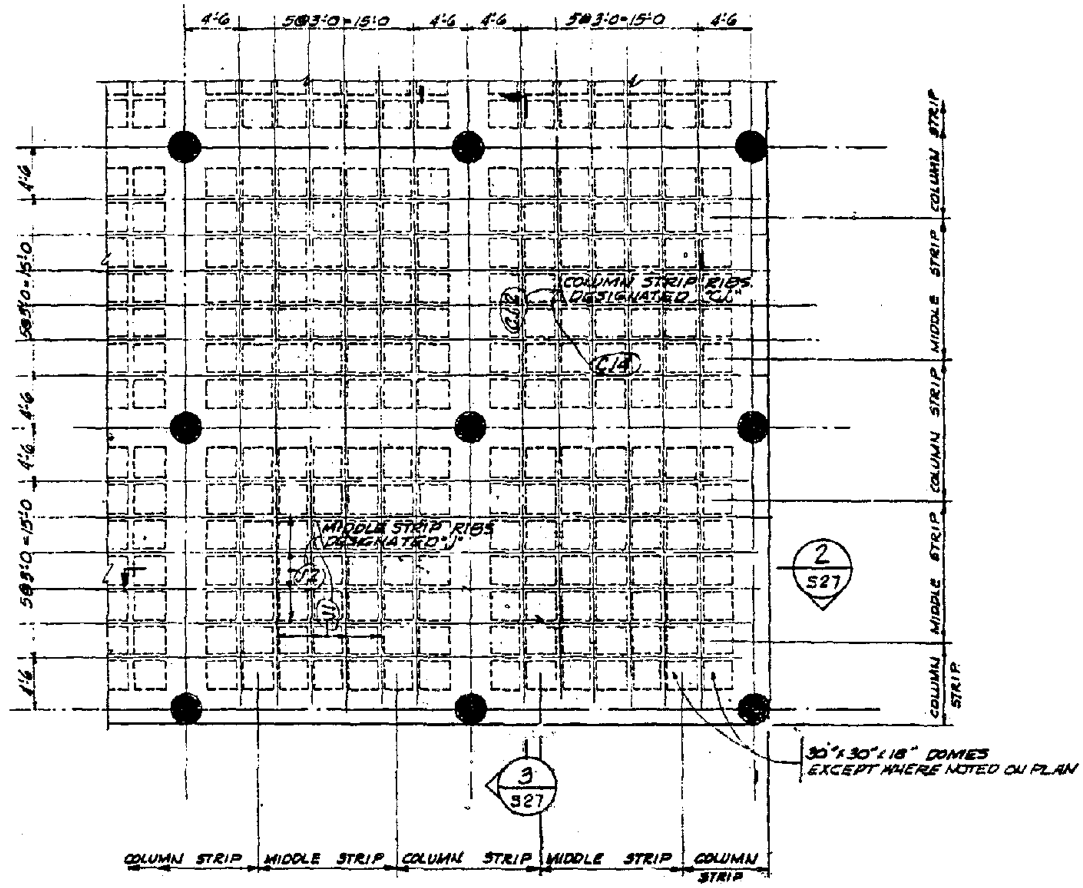


Figure 7. Typical layout: waffle slab construction.

was saying was that, due to their extreme stiffness, there were no inflection points in some of the building columns. The method, of course, could not be used. Therefore, an estimate of frame stiffness for torsional moments using the cantilever method was employed for this preliminary proportioning. If the method was inaccurate, it could at least be done rapidly.

Computer Analysis

Considering the above facts, it was obvious that one of the available computer programs utilizing modal frame analysis would be required if a reasonable seismic design was to be obtained in a reasonable time. A full-blown dynamic analysis was impossible because the required geologic data were unavailable. The output listed a response spectra and yielded deformations, axial forces, moments and shears that appeared consistent with the building geometry and mass. This section will concern itself with the input data.

Once the proportions of the framing members were decided upon, frame moments for vertical loads were computed on the Wang 700 Programmable Calculator. In order to use the program it is necessary to manually compute the moment of inertia of the columns and floor slabs. Using ACI 318-77 flat slab design methods, the floor stiffness is reduced to an effective stiffness to

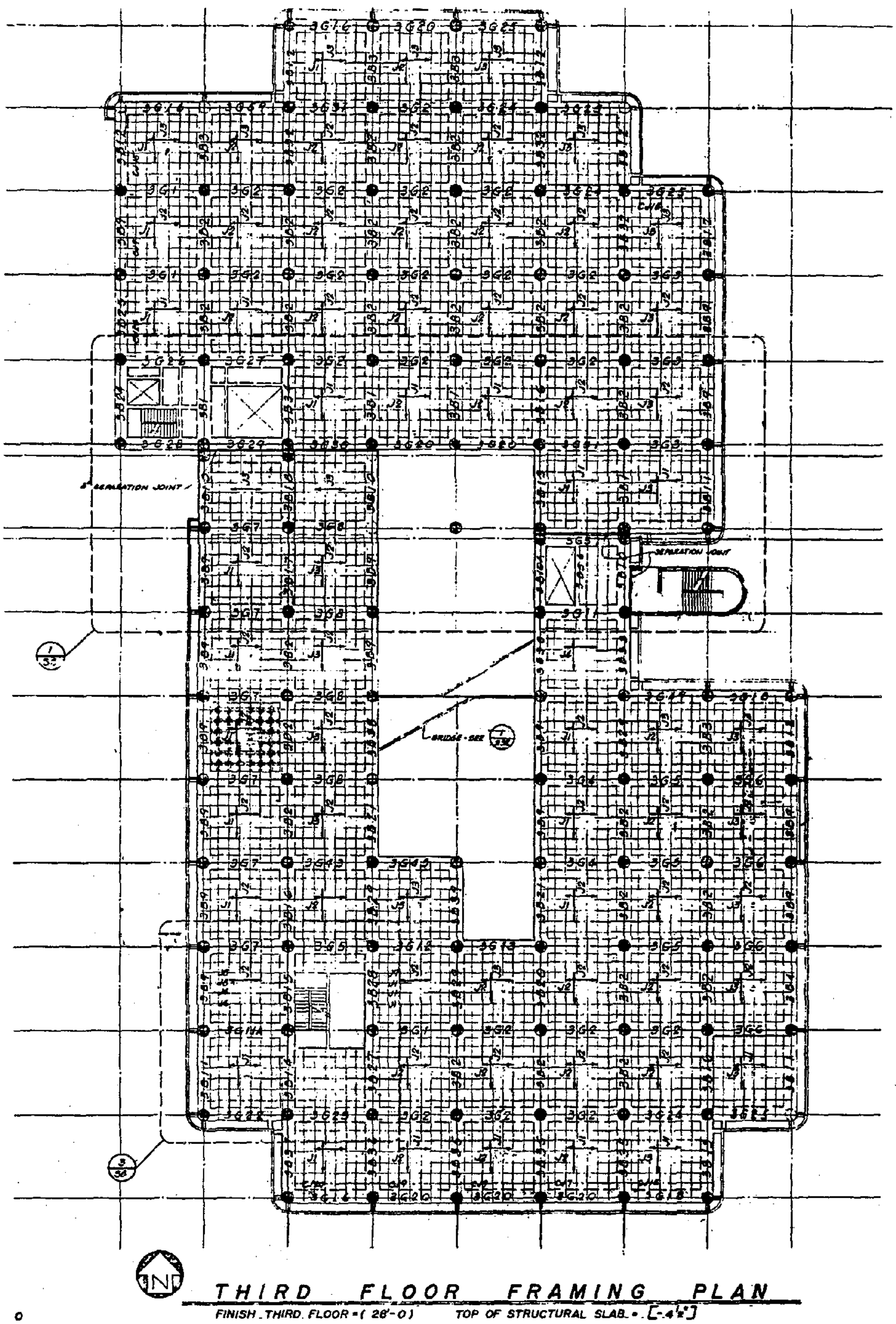


Figure 8. Third floor framing plan.

account for the fact that only part of the slab is effective in transferring moments into the columns. In the case of lateral forces, it is also necessary to provide information on the stiffness of each frame member. Inasmuch as earthquake loadings are generated in the earth, it was necessary to consider the columns as fully effective and to use their full moment of inertia. As equivalent (reduced) moment of inertia was calculated for the floor system by inverting the process used for calculating the reduced moment of inertia for columns. Diagrams for each frame showing lengths and listing the area and moment of inertia were provided for computer input (see Figure 9). The results of the computer print-out were later transferred to these same diagrams for ease in transferring them to the beam, slab, and column design sheets. In addition to the above, the weight of each floor and center of gravity load was computed as well as its mass moment of inertia (in slugs) as data for the computer input.

There was nothing sensational about the design of the columns, beams, or slabs. It was, however, necessary to add and distribute seismic and vertical load moments manually for the design of the beam and slab systems. The beam sections were designed for the entire seismic moment and shear as required by the code, but inasmuch as the adjoining column strip joists were included in the column strip, they were designed for their proportion of seismic loadings even though they are not a part of the ductile frame.

Floor diaphragms were designed by differencing the frame reactions at each level and, in the case of the lower two stories, increasing them proportionally to minimum code levels. Forces were low and had no influence on design except at the narrow slab throat near the elevator shaft.

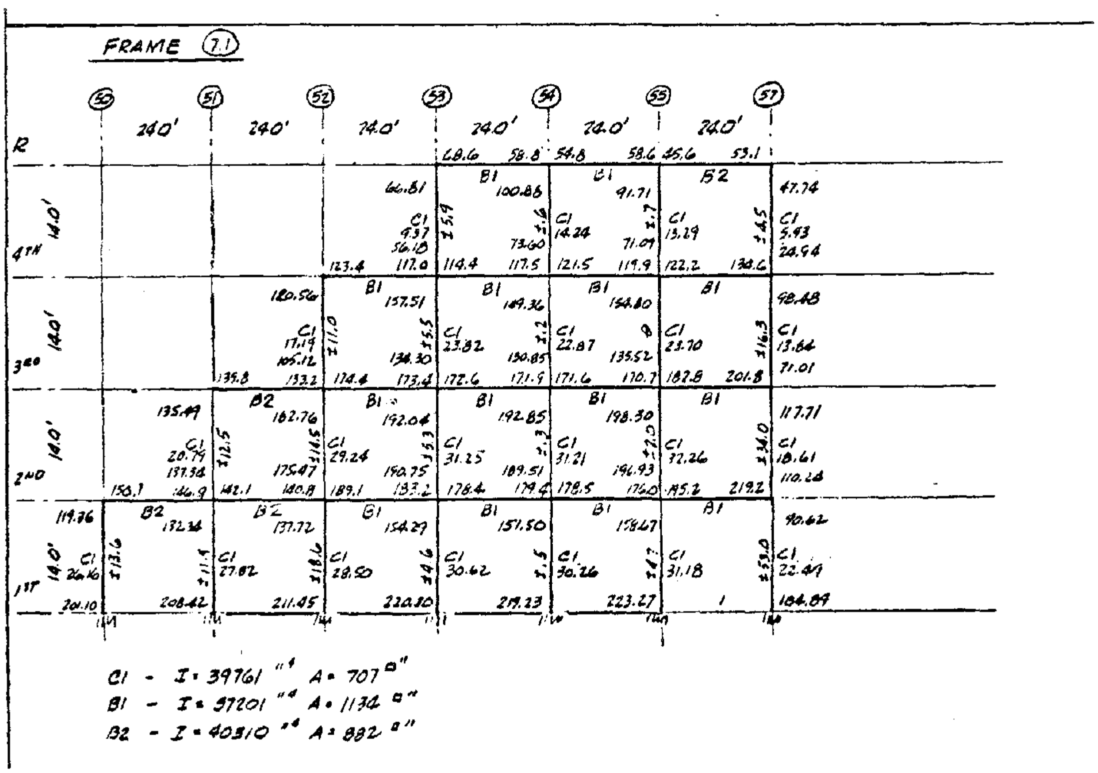


Figure 9. Frame properties, moments and shears.

Additional Details

It was decided to consider building columns as fixed at their base because pile caps and their tie beams are quite rigid. The tie beams were therefore designed as ductile concrete to take column moments, and detailed as shown in Figures 10a and 10b.

A method of avoiding end "Panel Zone" anchorage difficulties is shown in Figure 11. Note how advantage has been taken of the cantilever sunshade supports to anchor top and bottom reinforcement. A diagram of the arrangement of typical beam steel is shown in Figure 12.

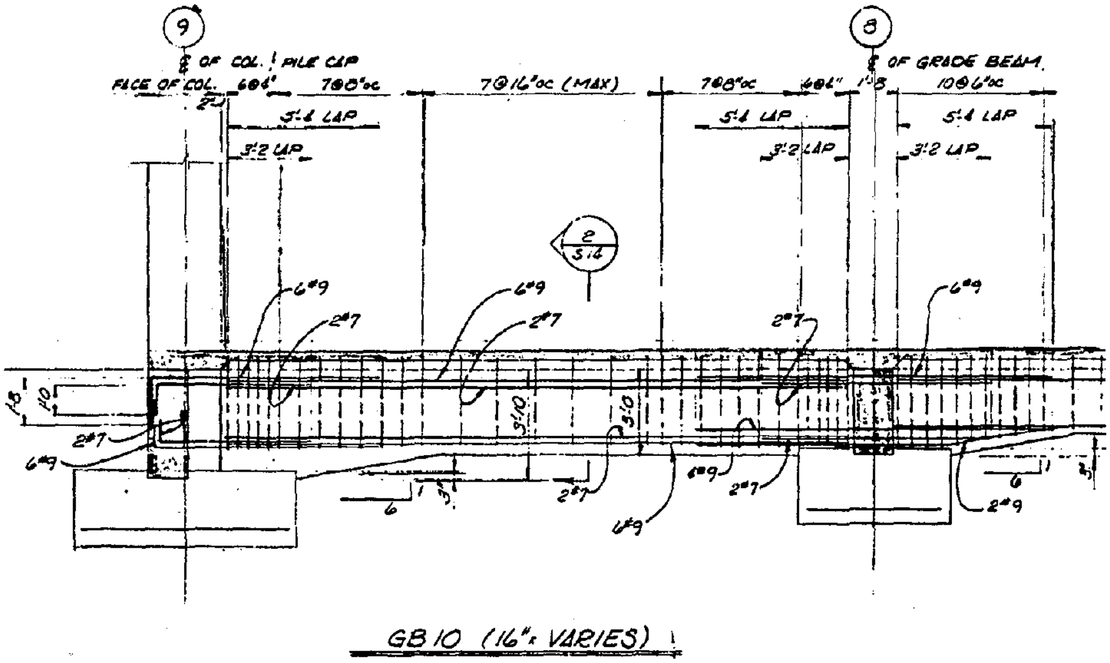


Figure 10a. Typical tie beam.

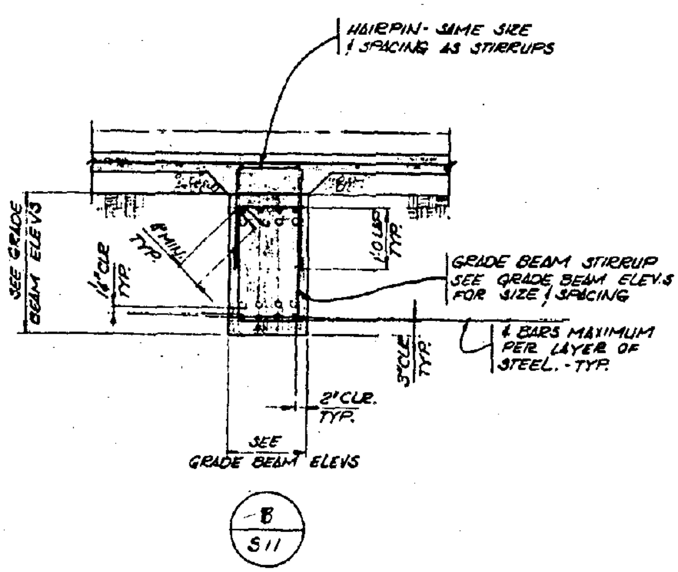


Figure 10b. Tie beam section.

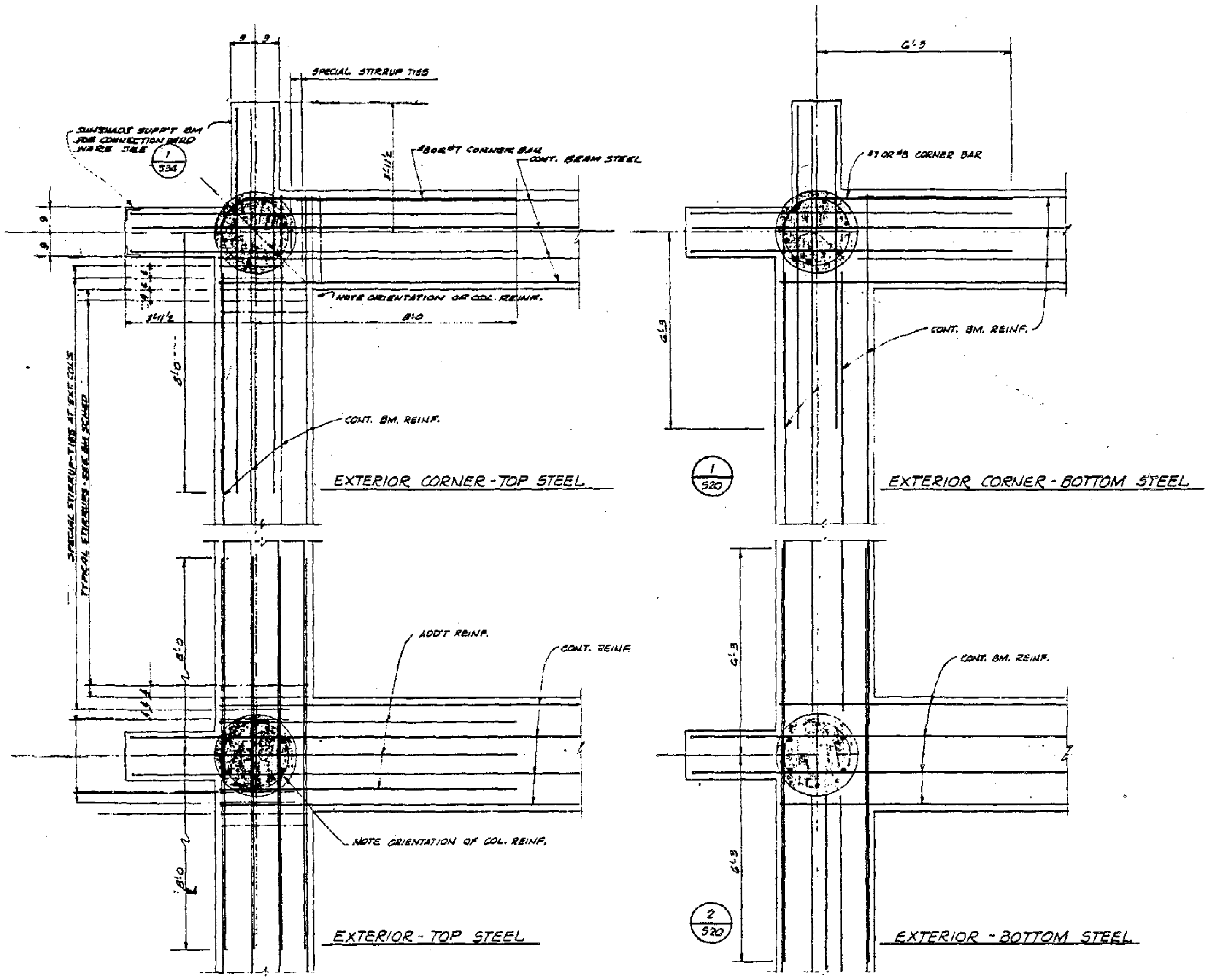


Figure 11. Method to avoid "Panel End" anchorage difficulties.

Construction went very well. Only 2 of the 640 concrete piles were broken during driving. Figure 13 shows some of the fiberglass column forms. The collars at the top are at the second floor level. Spiral reinforcing has been spread at floor zone to insert concrete pump nozzle. Note that column reinforcing extends two stories without a splice. The steel tubular shores holding column reinforcing are standard tilt-up panel braces.

Figure 14 shows 30 in. x 30 in. x 18 in. fiberglass domes and girder reinforcement. Cap stirrups were placed at the same time as top rib steel as they were in the same plane. Note the staggered bar laps at the column splices. Welded splices utilized the "Caldweld" process.

Figure 15 shows third floor construction. Note bends in column reinforcing at their tops. Cans for plumbing are shown in foreground. One-half inch electrical conduit was permitted in slabs; three-quarter inch in ribs.

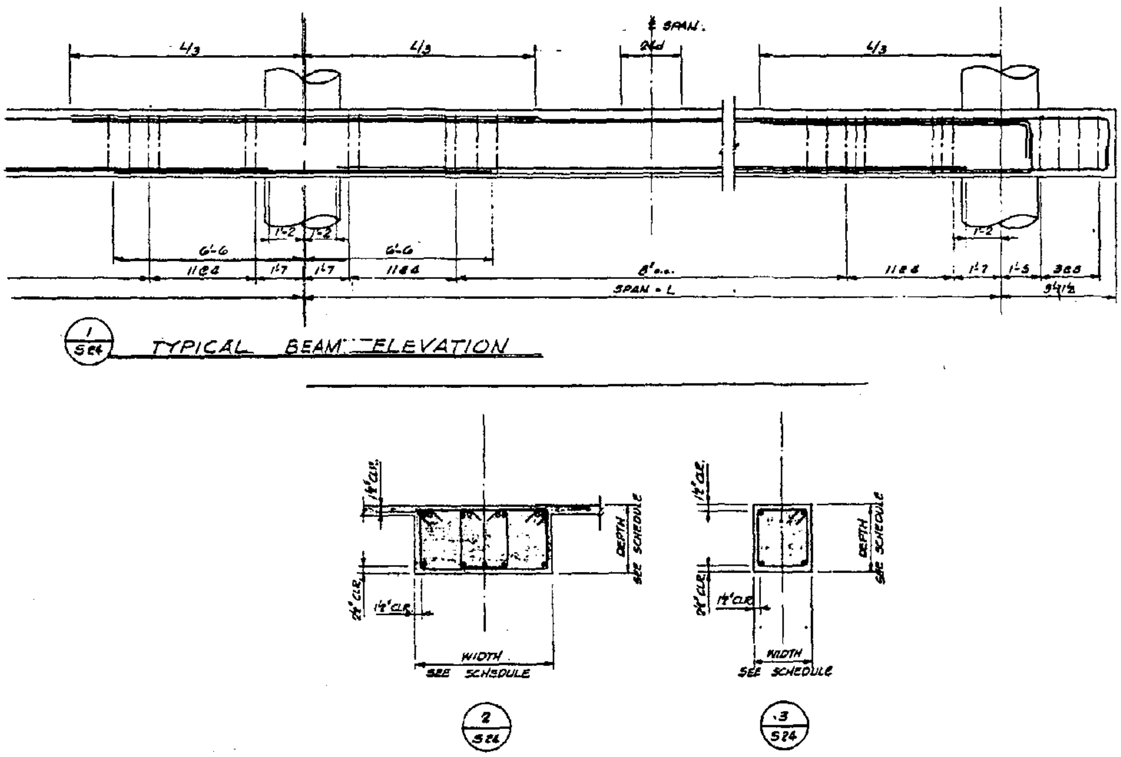


Figure 12. Arrangement of typical beam steel.

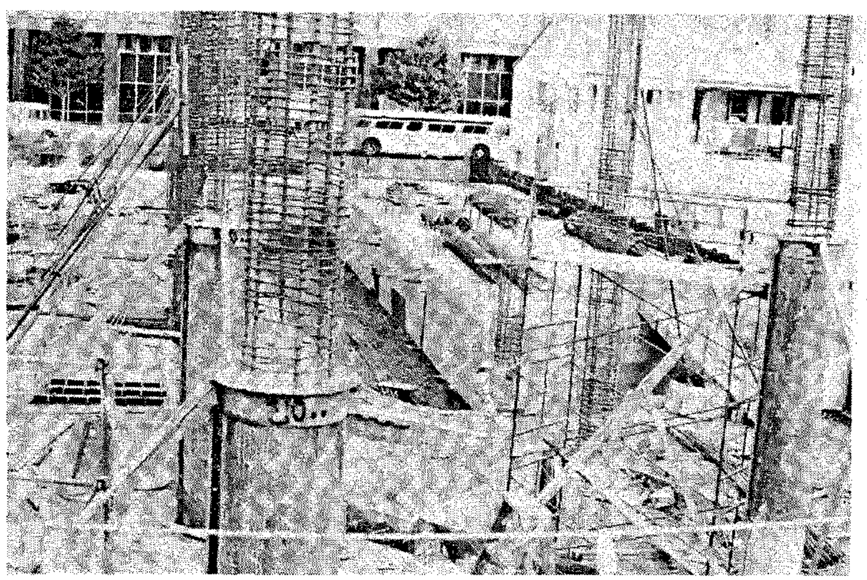


Figure 13. First and second story construction.

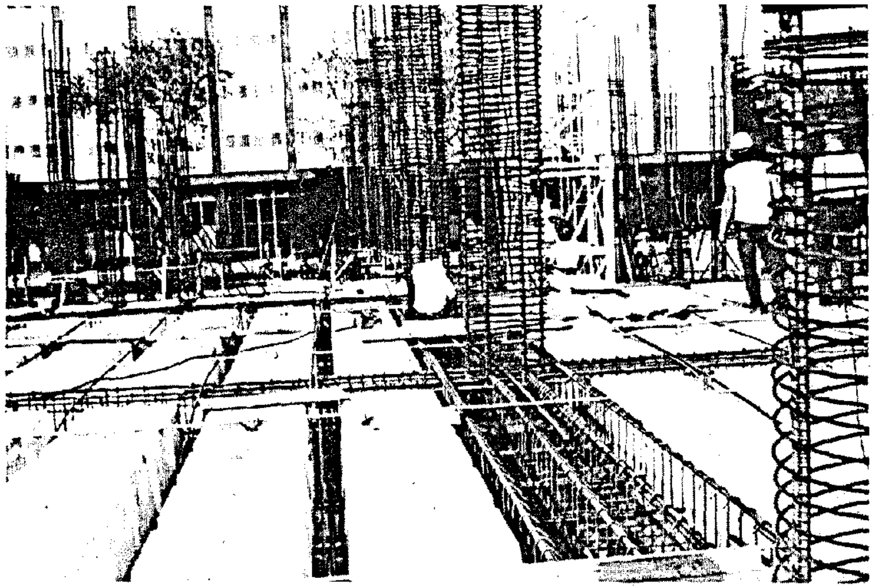


Figure 14. Dome construction.

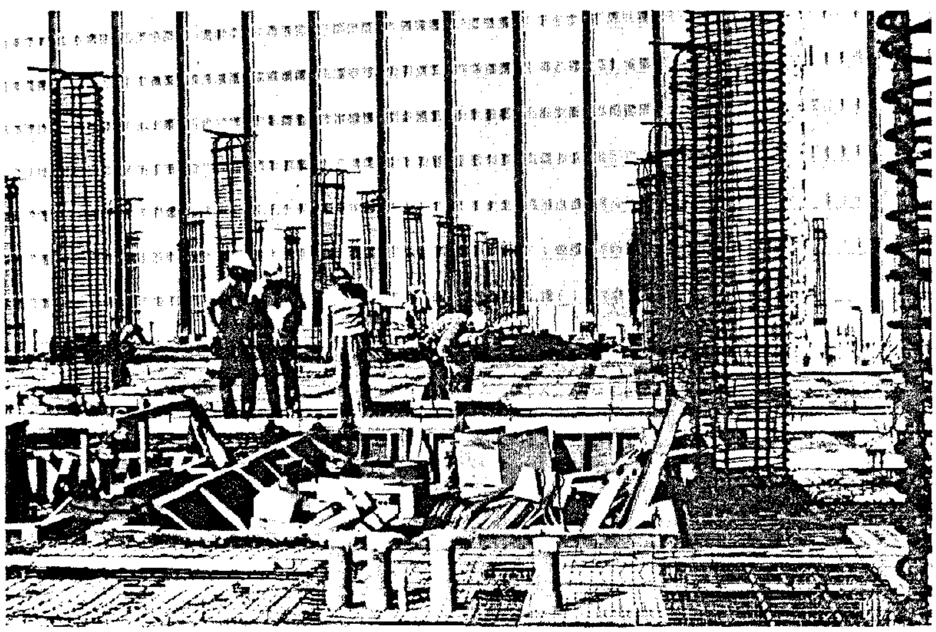


Figure 15. Third floor construction.

Figure 16 shows the forming method used to construct the concrete smoke tower on the east side of the building. This element was structurally separated from the rest of the building. Figure 17 shows building irregularity at the southeast corner.

GENERAL DESIGN METHODOLOGY RECOMMENDATIONS

Single-Story Construction

This is the category of low-rise concrete construction in which structural and architectural designers generally get their first experience, and in which beginning contractors unfortunately get their first experience as well. The combination of the two can be most harrowing. It is therefore important that the codes for this class of construction be kept very simple so that they can be administered without too much difficulty. Building officials may be without a great deal of experience as well. Actually, because a lot of dowels get left out, some design values should be developed for epoxy-bonded anchors and dowels. They seem to work very well, and manufacturers have come up with some data, but no generally accepted standards seem to have been developed.

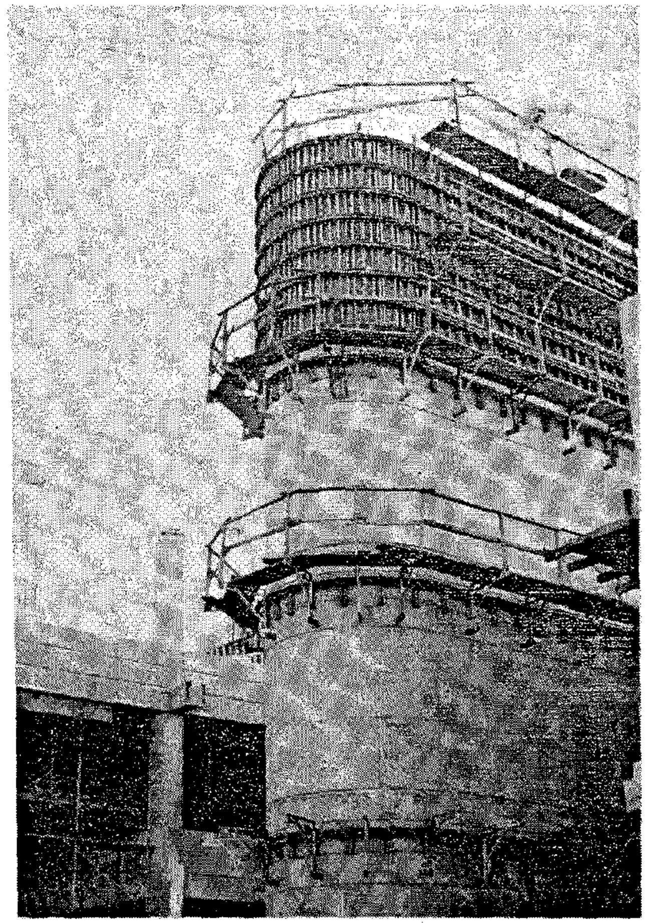


Figure 16. Smoke tower.

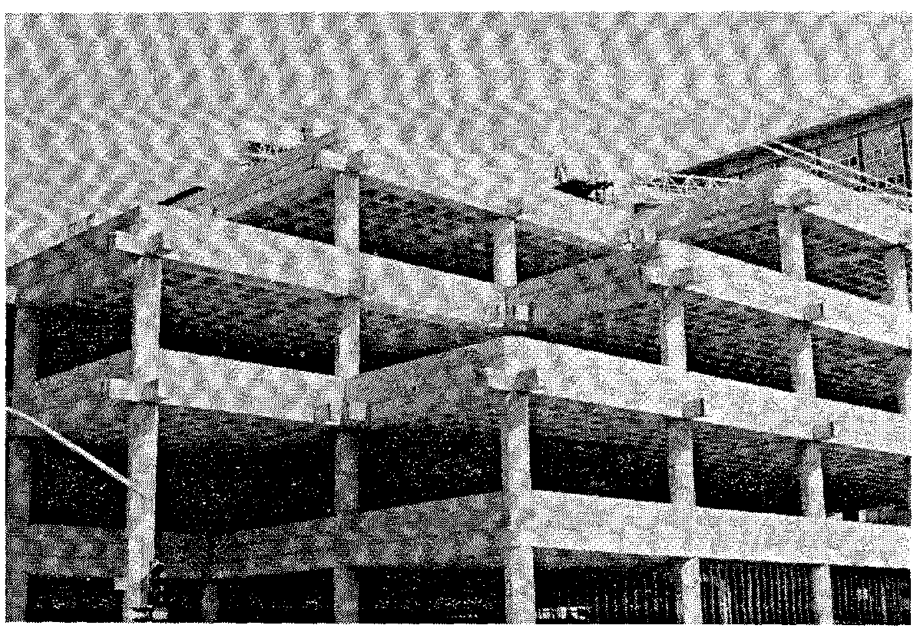


Figure 17. Building irregularity.

With regard to seismic phenomena, because some of the one-story structures are very large, and because areas, particularly in the Central Valley, are on very deep alluvial deposits, a half wavelength should be able to be determined for this special condition. This is due to that fact that earthquakes seem to develop "S" waves based more on site resonance than input frequencies. It may be that there is a critical length for structures in this location.

Heavy Low-Rise Construction

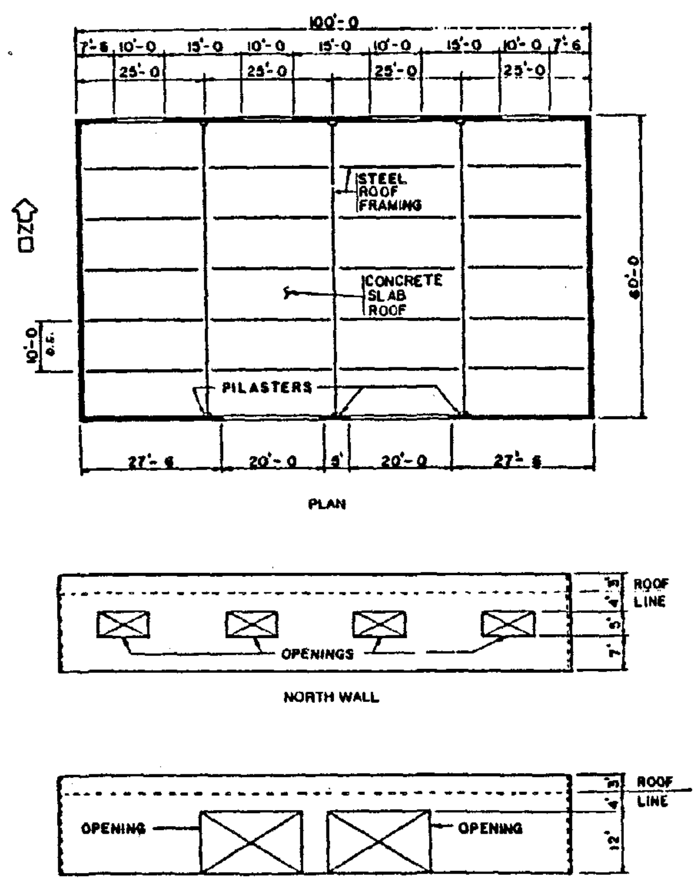
One facet of the seismic phenomena that has not been codified is vertical acceleration. It is acknowledged and measured, but no generally acceptable design criterion has been devised. It is needed for the heavily loaded concrete structure. Individual engineers have attempted to compensate for this by increasing the vertical load factors in column design, by deliberately overstating the unit design loads, or by simply oversizing members. It would be better if an accepted methodology were adopted.

Light Concrete Low-Rise Construction

Most of the research and study being conducted seems to be applicable to this construction category for buildings over three stories in height as well as high and mid-rise construction. This is addressed in another paper in this volume.

APPENDIX

PLAN AND ELEVATIONS



A CONCRETE MASONRY SHEAR WALL BUILDING WITH RIGID DIAPHRAGM

A single story warehouse 60′ × 100′ located in Seismic Zone 4. Plan and elevation are as shown and the roof is considered a rigid diaphragm. Design the masonry shear walls including the effects of torsion. (This Example is intended to show only the shear wall design of a building with a rigid diaphragm. The other portions of the design would follow the same procedures as presented in Examples 8.1 and 8.2.)

Roof dead load = 80 psf
Roof live load = 20 psf
Use 12″ (nominal dimension) concrete masonry wall - solid grouted. weight = 125 psf

Wind load = 15 psf

Seismic load - total building:
$V = Z\,I\,K\,C\,S\,W = (1.0)(1.0)(1.33)(0.14)\,W = 0.186W$
W = Dead load of roof plus weight of parapet plus weight of one-half the height of building.

f'_m = 1500 psi = no inspection - use half-stresses
f_y = 40,000 psi
F_s = 20,000 psi, except for compression in vertical column reinforcement: F_s = 16,000 psi
n = 40
$F_m = \frac{1}{2}(0.33)\,f'_m = 250$ psi

LATERAL LOAD DESIGN (Total Building)

Wind Load = 15 psf
North-South Direction:
$V = (15)(8 + 3)(100) = 16{,}500$ lbs.
East-West Direction:
$V = (15)(8 + 3)(60) = 9{,}900$ lbs.

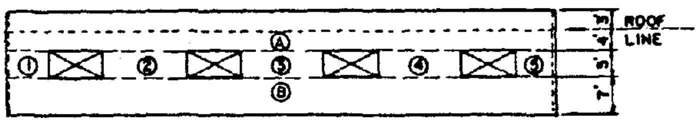
Seismic Load
W = 2 (½ wall weight plus parapet) and roof dead load
Roof dead load = 60′ × 100′ × 80 = 480,000 lbs.
North Wall = [(100 × 8.0) − (4 × 10 × 4.0)] 125 psf + (100 × 3)(125) = 117,500 lb.
South Wall = [100 × 8.0) − (2 × 20 × 4.0)] 125 psf + (100 × 3)(125) = 117,500 lb.
East & West Walls = 2 (60)(8 + 3) 125 psf = 165,000 lbs.
W = 880,000

V = (0.186)880,000 = 163,680 lbs. (governs for both directions)

As in Examples 8.1 and 8.2, the relative rigidities of the wall piers between the openings will have to be calculated in order to distribute the shear force within each wall. In addition, since the diaphragm is rigid, the total rigidities of the walls will have to be calculated in order to distribute the shear force to the walls and determine the torsional effects.

Note: For the purposes of this Example, even though the actual wall thickness has been increased to 12 inches, the calculation of rigidities will be made using t = 8 inches: Since the wall thickness is constant throughout, it has no effect on the relative rigidities. Therefore, we can make use of the previously calculated rigidities.

NORTH WALL



NORTH WALL

Pier rigidities: (from Examples 8.1 and 8.2)

Pier	Δ	R′	$\frac{R'}{\Sigma R'}$
1	0.957	10.45	0.116
2	0.432	23.14	0.256
3	0.432	23.14	0.256
4	0.432	23.14	0.256
5	0.957	10.45	0.116
		90.32	1.000

In order to determine the total rigidity of the wall, the deflections of Piers A and B must also be determined.

Pier A:
$$\Delta = \frac{1}{8}\left[0.333\left(\frac{4}{100}\right)^3 + \frac{4}{100}\right] = 0.0050$$

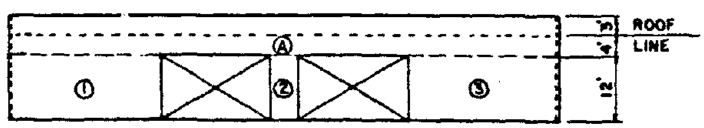
Pier B:
$$\Delta = \frac{1}{8}\left[0.333\left(\frac{7}{100}\right)^3 + \frac{7}{100}\right] = 0.0088$$

Total Δ = Δ of Pier A + Δ of Pier B + Δ of Piers 1-2-3-4-5

$$\text{Total } \Delta = 0.0050 + 0.0088 + \frac{1}{90.32} = 0.0248$$

$$\text{Total } R' = \frac{1}{0.0248} = 40.26$$

SOUTH WALL:



SOUTH WALL

Pier rigidities (from Examples 8.1 and 8.2)

Pier	Δ	R′	$\frac{R'}{\Sigma R'}$
1	0.0580	17.24	0.484
2	0.8754	1.14	0.032
3	0.0580	17.24	0.484
		35.62	1.000

Pier A:
$$\Delta = \frac{1}{8}\left[0.333\left(\frac{4}{100}\right)^3 + \frac{4}{100}\right] = 0.0050$$

Total Δ = Δ of Pier A + Δ of Piers 1-2-3

$$\text{Total } \Delta = 0.0050 + \frac{1}{35.62} = 0.0331$$

$$\text{Total } R' = \frac{1}{0.0331} = 30.23$$

EAST & WEST WALLS: (solid)

$$\Delta = \frac{1}{8}\left[0.333\left(\frac{16}{60}\right)^3 + \frac{16}{60}\right] = 0.0341$$

$$R' = \frac{1}{0.0341} = 29.31$$

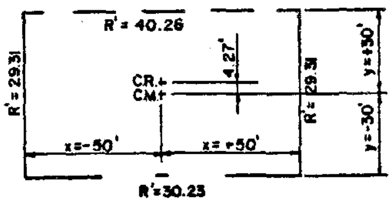
DETERMINE CENTER OF MASS AND CENTER OF RIGIDITY

This is done in order to determine the torsional effects. The lateral force acts at the Center of Mass. If the Center of Mass and Center of Rigidity do not coincide, then torsion will result.

Center of Mass:

Referring to the calculation of the Seismic Load (page 8.19), the weights (contributing to the Seismic Load) of the North and South Walls are equal, as are the weights of the East and West Walls. Therefore, the Center of Mass is at the geometric center of the building.

Center of Rigidity:



Wall	R'	y	R'y
North	40.26	+30	1207.8
South	30.23	−30	− 906.9
	70.49		300.9

Note: Center of Rigidity in x - direction is at center of building, since the rigidities of the East and West Walls are equal.

$y_{cr} = \dfrac{300.9}{70.49} = +4.27'$ (north of center)

DETERMINE TORSIONAL ECCENTRICITY

The U.B.C. specifies that in all buildings with rigid diaphragms the minimum torsional eccentricity in each direction shall be assumed to be 5 percent of the maximum building dimension.

$e_x = 0$, Min. $e_x = 0.05(100) = 5.00'$

$e_y = 4.27'$, Min. $e_y = 0.05(100) = 5.00'$

DETERMINE LATERAL FORCES IN WALLS

NORTH-SOUTH DIRECTION:

$V = 163{,}680$ lbs. $T = Ve_x = 163{,}680(5) = 818{,}400$ ft. lbs.

Force due to shear: $F_v = V \dfrac{R'_x}{\Sigma R'_y}$

Force due to torsion: $F_t = T \dfrac{R'd}{\Sigma R'd^2}$

Note: Torsion forces will have a positive or negative sign. The torsion force will always be positive on the wall that has the lesser rigidity for the particular direction being considered. The torsion force in the other wall will be negative but this force is never subtracted from the shear force.

Wall	R'_x	R'_y	d_x	d_y	R'd	R'd²	F_v	F_t	V
North	40.26			25.73	1036	26653		−4062	4062
South	30.23			34.27	1036	35503		+4062	4062
East		29.31	50		1466	73275	81840	−5749	81840
West		29.31	50		1466	73275	81840	+5749	87589
	70.49	58.62				208706			

EAST-WEST DIRECTION:

$V = 163{,}680$ lbs. $T = Ve_y = 163{,}680(5) = 818{,}400$ ft.lbs.

$F_v = V \dfrac{R'_x}{\Sigma R'_x}$ $F_t = T \dfrac{R'd}{\Sigma R'd^2}$

Wall	R'_x	R'_y	d_x	d_y	R'd	R'd²	F_v	F_t	V
North	40.26			25.73	1036	26653	93485	−4062	93485
South	30.23			34.27	1036	35503	70195	+4062	74257
East		29.31	50		1466	73275		−5749	5749
West		29.31	50		1466	73275		+5749	5749
	70.49	58.62				208706			

DISTRIBUTE LATERAL FORCES WITHIN EACH WALL
(In same manner as in Examples 8.1 & 8.2)

EAST AND WEST WALLS: (No distribution - solid)

$V = 87589$ lbs.

$v = \dfrac{1.5(87589)}{(60\times12)(11.63)} = 15.69$ psi

$v_m = 17 \times 1.33 = 22.6 > 15.69$ psi *OK*

NORTH WALL:

Piers 1 & 5:

$V = 93485(0.116) = 10844$ lbs.

$v = \dfrac{1.5(10844)}{(7.5\times12)(11.63)} = 15.54$ psi < 22.6 psi *OK*

Piers 2, 3, and 4:

$V = 93485(0.256) = 23932$ lbs.

$v = \dfrac{1.5(23932)}{(15\times12)(11.63)} = 17.15$ psi < 22.6 psi *OK*

SOUTH WALL:

Piers 1 & 3

$V = 74257(0.484) = 35940$ lbs.

$v = \dfrac{1.5(35940)}{(27.5\times12)(11.63)} = 14.05$ psi < 22.6 psi *OK*

Pier 2:

$V = 74257(0.032) = 2376$ lbs.

$v = \dfrac{1.5(2376)}{(5\times12)(11.63)} = 5.11$ psi < 22.6 psi *OK*

No additional reinforcement for shear is required in any wall.

OVERTURNING ON SOUTH WALL

Piers 1 & 3:

Assume piers are fixed at top and bottom.

$\text{OTM} = V \times \dfrac{H}{2} = 35940 \times \dfrac{12}{2} = 215640$ ft.lb.

Compute axial load stress:
Roof DL = 400 plf
Weight of ½ wall = 1000 plf
Weight of parapet = 375 plf

$$f_a = \frac{400 + 1000 + 375}{11.63 \times 12} = 12.72 \text{ psi}$$

Compute allowable axial load stress:

$$R = 1 - \left(\frac{h}{40t}\right)^3 = 1 - \left(\frac{16\times12}{40\times11.63}\right)^3 = 0.930$$

$F_a = (½)0.2F'_m\,R = ½(0.2)(1500)(.930) = 139.4$ psi

Assume section is uncracked and find bending stress in masonry:

$$S = \frac{tD^2}{6} = \frac{11.63(27.5\times12)^2}{6} = 211084 \text{ in}^3$$

$$f_m = \frac{M}{S} = \frac{215640\times12}{211084} = 12.25 \text{ psi} < 12.72 \text{ psi}$$

∴ Section is uncracked

Interaction Equation:

$$\frac{f_a}{F_a} + \frac{f_m}{F_m} = \frac{12.72}{139.4} + \frac{12.25}{250} = 0.14 << 1.33 \; OK$$

No additional reinforcing is needed for overturning

Pier 2:

$$OTM = 2376 \times \frac{12}{2} = 14256 \text{ ft.lb.}$$

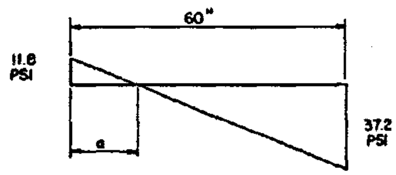
Axial load stress: (neglect pilaster)
$f_a = 12.72$ psi
Allowable axial load stress:
$F_a = 139.4$ psi

Assume section is uncracked and find bending stress in masonry:

$$S = \frac{tD^2}{6} = \frac{11.63(5\times12)^2}{6} = 6978 \text{ in.}^3$$

$$f_m = \frac{M}{S} = \frac{14256\times12}{6978} = 24.5 \text{ psi} > 12.72 \text{ psi}$$

∴ Section is cracked
Stress on Section:
$f = f_a \pm f_m = 12.72 \pm 24.5$
= 37.2 psi compression
to 11.8 psi tension



$$a = \frac{11.8}{37.2+11.8} \times 60 = 14.45"$$

Tension force = ½(11.8)(14.45)(11.63) = 991.5 lb.

Area of Steel:

$$A_s = \frac{T}{f_s} = \frac{991.5}{26{,}700} = 0.04 \text{ in.}^2$$

Typical wall reinforcement will be adequate to resist this force and no additional steel over the minimum required at openings is needed.

APPENDIX A

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APPENDIX B

JAPAN STRUCTURAL CONSULTANTS ASSOCIATION INFORMATION AND HISTORY

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HISTORY OF THE JAPAN STRUCTURAL CONSULTANTS ASSOCIATION

Hiroshi Inoue
Hiroshi Inoue Architects and Engineers
Tokyo, Japan

Before the second world war, there were no structural design offices in Japan. At the end of the war in 1945, Japan was in a state of total collapse with little remaining buildings.

At first, wooden houses of a somewhat barrack-type structure consisting of some 20 to 30 square meters began to be constructed. From around the year 1950, reinforced concrete structures began to be built on a small scale. At the same time, an increase was noted in the number of structural engineers, resulting in a small increase in structural design offices.

The approximate twenty year period beginning in 1955 and lasting until 1973, saw "a period of very large economic growth" in which architectural growth was very high as shown in Figure 1. In this period, the increase in the number of structural engineers was limited resulting in our being extremely busy with little time to think or even sleep. During this period, the need for an organization such as the Japan Structural Consultants Association was realized, but the pressures of work prohibited the setting up of such an organization.

The structural engineer in Japan works under the peculiarity of our land, which is the most severe earthquake area in the world, and typhoons and heavy snow fall also add to the underlying of the heavy responsibility of the structural engineering field. Even with this heavy responsibility, the structural engineering field is not generally recognized by the Japanese society, and our fees are usually inadequate. The lack of an organization such as the JSCA was the prime reason for this condition.

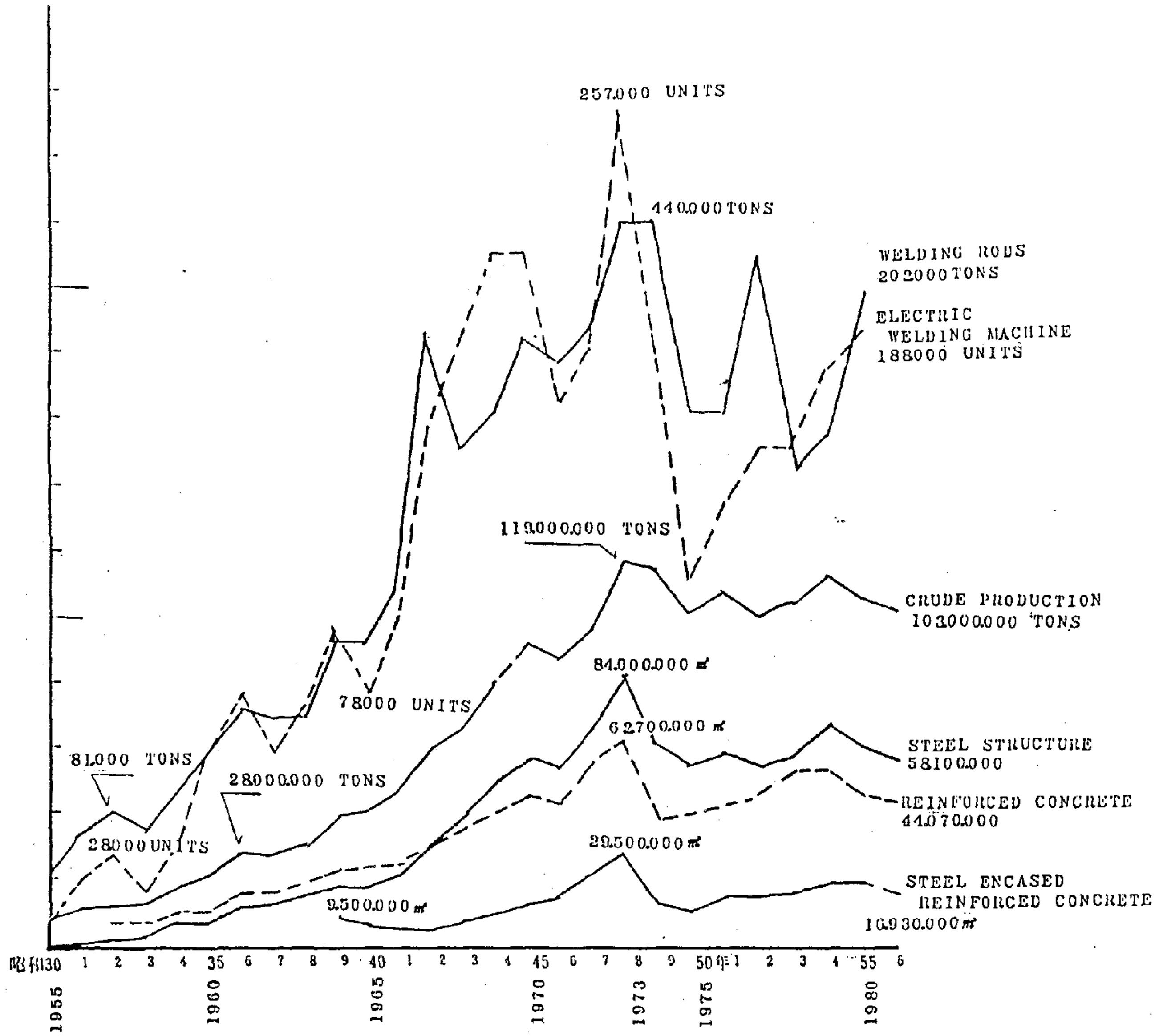
In 1973 with the occurrence of the first oil shock, the amount of construction activity dropped sharply, and the Japanese construction industry faced a very difficult period. So, at the same time, the structural engineers forgot about forming an association.

The new seismic design method for building enforced in 1981 provided the springboard for the founding of JSCA. At a gathering of one hundred structural engineers on May 29, 1981, the Association was founded. The present number of members today exceeds one thousand.

Until the founding of the Association, the members were employed by various companies with little opportunity for communication. However, the foundation of the Association has provided a means of communication for its members.

The constitution of JSCA provides for eight subcommittees as shown in the following section. Under the technological committee there are six separate meeting areas. The members from Japan attending the 1984 Hawaii Workshop are from the Seismic Structural meeting area. In Japan, there are seven branches ranging from Hokkaido in the north to Kyusyu in the south. Each of these branches sets up their own board of directors, committees, technological meetings, etc. according to their regional conditions. There is much communication between each of these branches.

In summary, the organization provides the means of communication for structural engineers in Japan. Although the organization is not completely satisfactory at present, it is moving toward the right direction.



Architectural Growth in Japan, 1955–1981.

FIGURE 1

JAPAN STRUCTURAL CONSULTANTS ASSOCIATION (JSCA)

Secretary
1-4-12 Hirakawa-Cho,
Sogo Dai 6 Bldg.,
Chiyoda-Ku, Tokyo 102
Japan
Tel. (03) 262-9498

PURPOSE OF JSCA

- JSCA was established in May, 1981 to satisfy the social needs as a professional group. It is formed of leading structural consultants.
- For more than thirty years after World War II, the structural design business has been recognized in the society as a key profession on the development of economy and culture. Structural consultants united their efforts to achieve many technological innovations.
- Harmonizing the art and economy, spirit and technology, important duties of accomplishing the functions of the structure and protecting human life and culture from the natural disaster are the very functions of the structural consultant.
- When we think about the peculiarity of our land (condition of the nature), which is the most severe earthquake region of the world and also has typoons and heavy snow fall, the seriousness of difficulty and responsibility of structural consultants are much heavier compared to those of other countries.
- This Association, as a group of professionals, has the responsibility to design and manage the structure of the organization, on the premise that intelligence, knowledge, technical ability, and sense of each member is focused to the activity.
- Externally, JSCA will be the interface of structural consultants and surrounding society, to cultivate a better understanding of the functions of the structural consultants, forming the basis for proper and smooth conduct of the business.
- Internally, JSCA intends to promote mutual contacts of the members, improve the technical level of structure design and management, prepare official opinion on structural problems, establish necessary standards, open a consulting facility, exchange information through the activities of publishing bulletins, etc., and enlighten the study and training of each other, and younger professionals.
- Though its external and internal activities, JSCA builds up a basis that realizes better structural design, and aims to contributing to the development of the culture and economy of the society through each member's design activity.
- We won't be able to achieve the goal without a systematic activity under the close cooperation of structural designers throughout the country.

JSCA ACTIVITY

- General Meetings, Board of Directors Meetings, Administrative Meetings

  Three types of meetings are held to make decisions on the activities and policy of JSCA and to cope quickly with specific situtions: (1) General Meetings by all regular members; (2) meetings of the Board of Directors; and (3) Administrative Meetings, which are attended by Representatives, Alternate-Representatives, Chairmen of Committees, and the Secretary.

- Committees

  The association consists of eight committees for organization, operation, technical, public relations, finance, legislation, preparation for incorporation, and business compensation. Each committee meets once a month to exchange views and prepare proposals for decision of the association. Each regular member is welcome to register and participate in committee action. Subjects which are discussed in committee sessions are communicated to all members through publications, bulletins, etc. The technical committee is divided into six working parties which probe into the closely related daily design activities.

- Bulletin Publication

  "Bulletin - Structure" is a quarterly magazine edited by the Public Relations Committee from contributions of members. It includes reader's columns and technical columns, member's information, theses, movements of the Association, etc. To deliver various information, it publishes membership lists, special technical reports, (e.g., reports regarding Japan-U.S. joint earthquake engineering experiments).

- Branch Activities

  To encourage activity of local structural consultants, JSCA is divided into seven regions: Hokkaido, Tohoku, Chubu, Kanai, Chugoku, Shikoku and Kyushu. Each branch selects regional subjects and works for intra-region communication.

- Social Meetings

  At least twice a year, a friendship party is held to cultivate mutual friendship of the members.

- Panel Discussions

  Panel discussions are held once every two or three months regarding various subjects (e.g. technical, legal, occupational in nature, social standing of structural consultants, etc.).

- Discussion Meetings

  Discussion meetings are planned upon occasion by regular members, who serve as meeting leaders. The subjects discussed are related to the various functions of JSCA. Meeting discussions/summaries are printed in the bulletin.

- Lecture Meetings

  Professionals in every field, as well as in the structural field, are invited to lecture meetings held from time to time. These meetings involve 20 to 30 people, or more.

- Field trips

  JSCA sponsors field trips to special structures, bridge construction sites and various production facilities, once or twice a year. Detailed information and explanation are given to participants.

- Inter-relationships with Other Architectural Organizations

  Close contact is kept with other architectural organizations including government agencies such as the Ministry of Construction; Tokyo Metropolitan Office and other government and public agencies throughout the country; scientific organizations including the Architectural Institute of Japan; various associations including the Japan Federation of Architects Office Association; and associations related to architecture, including the Architectural Industry Association.

- Requested Research and Overseas Relations

  JSCA will accept research requests if they agree with the JSCA purpose and policy. Members can participate in the research and own the results jointly. As an overseas relationship, a Japan-U.S. Aseismic Structure Council will be established in March, 1984.

JSCA MEMBERS

- Special Members - 27

  Persons who have special knowledge and experience in structural architecture. Membership is by approval of the Board of Directors.

- Academic Members - 104

  Persons who have established their professional field at their university or laboratory, and have academic achievements. Admission is by approval of the Board of Directors.

- Regular Members - 695

  First class architect; more than 10 years of career as a structural designer; should be 35 years of age or older. Must be recommended by a regular member; admission is by approval of the Board of Directors.

- Associate Members - 43

  First class architect; interest in architectural structure; no career needed. Must have introduction by a regular member.

- <u>Supporting Members - 196</u>

  Corporations in support of the purpose and business of JSCA: Design Offices; the Construction Industry, Foundation Industry, Iron and Steel Industry, and Concrete Industry; Research Offices; etc.

APPENDIX C

APPLIED TECHNOLOGY COUNCIL PROJECTS AND REPORT INFORMATION

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PROJECTS AND REPORT INFORMATION

One of the primary purposes of Applied Technology Council is to develop resource documents that translate and summarize research information into forms useful to practicing engineers. This includes the development of guidelines and manuals, as well as the development of research recommendations for specific areas determined by the profession. ATC is not a code development organization, although several of the ATC project reports serve as resource documents for the development of codes, standards and specifications.

A brief description of several major completed and ongoing projects is given in the following section. Funding for projects is obtained from government agencies and tax-deductible contributions from the private sector.

ATC-1: This project resulted in five papers which were published as part of Building Practices for Disaster Mitigation, Building Science Series 46, proceedings of a workshop sponsored by the National Science Foundation (NSF) and the National Bureau of Standards (NBS). Available through the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, VA 22151, as NTIS report No. COM-73-50188.

ATC-2: The report, An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings, was funded by NSF and NBS and was conducted as part of the Cooperative Federal Program in Building Practices for Disaster Mitigation. Available through the ATC office. (270 pages)

> Abstract: This study evaluated the applicability and cost of the response spectrum approach to seismic analysis and design that was proposed by various segments of the engineering profession. Specific building designs, design procedures and parameter values were evaluated for future application. Eleven existing buildings of varying dimensions were redesigned according to the procedures.

ATC-3: The report, Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC-3-06), was funded by NSF and NBS. The second printing of this report, which includes proposed amendments, is available through the ATC office. (505 pages plus proposed amendments)

> Abstract: The tentative provisions in this document represent the result of a concerted effort by a multidisciplinary team of 85 nationally recognized experts in earthquake engineering. The project involved representation from all sections of the United States and had wide review by affected building industry and regulatory groups. The provisions are tentative in nature and their viability for the full range of application should be established by test designs to determine their workability, practicability, enforceability and cost impact. The second printing of this document contains proposed amendments prepared by a joint committee of the Building Seismic Safety Council (BSSC) and the NBS; the proposed amendments were published separately by BSSC and NBS in 1982.

ATC-3-2: This project, Comparative Test Designs of Buildings using ATC-3-06 Tentative Provisions, was funded by NSF. The project consisted of a study to develop and plan a program for making comparative test designs of the ATC-3-06 Tentative Provisions. The project report is intended to be used by the Building Seismic Safety Council in its refinement of the ATC-3-06 Tentative Provisions.

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ATC-3-4: The report, Redesign of Three Multistory Buildings: A Comparison Using ATC-3-06 and 1982 Uniform Building Code Design Provisions, was published under a grant from NSF. Available throught the ATC office (112 pages)

Abstract: This report evaluates the cost and technical impact of using the 1978 ATC-3-06 report, "Tentative Provisions for the Development of Seismic Regulations for Buildings," as amended by a joint committee of the Building Seismic Safety Council and the National Bureau of Standards in 1982. The evaluations are based on studies of three existing California buildings redesigned in accordance with the ATC-3-06 Tentative Provisions and the 1982 Uniform Building Code. Included in the report are recommedations to code implementing bodies.

ATC-3-5: This project, Assistance for First Phase of ATC-3-06 Trial Design Program Being Conducted by the Building Seismic Safety Council, was funded by the Building Seismic Safety Council and provided the services of the ATC Senior Consultant and other ATC personnel to assist the BSSC in the conduct of the first phase of its Trial Design Program. The first phase provided for trial designs conducted for buildings in Los Angeles, Seattle, Phoenix, and Memphis.

ATC-3-6: This project, Assistance for Second Phase of ATC-3-06 Trial Design Program Being Conducted by the Building Seismic Safety Council, was funded by the Building Seismic Safety Council and provided the services of the ATC Senior Consultant and other ATC personnel to assist the BSSC in the conduct of the second phase of its Trial Design Program. The second phase provided for trial designs conducted for buildings in New York, Chicago, St. Louis, Charleston, and Fort Worth.

ATC-4: The report, A Methodology for Seismic Design and Construction of Single-Family Dwellings, was published under a contract with the Department of Housing and Urban Development (HUD). Available through HUD, 451 7th Street S.W., Washington, DC 20410, as Report No. HUD-PDR-248-1. (576 pages)

Abstract: This report presents the results of an in-depth effort to develop design and construction details for single-family residences that minimize the potential economic loss and life-loss risk associated with earthquakes. The report: (1) discusses the ways structures behave when subjected to seismic forces, (2) sets forth suggested design criteria for conventional layouts of dwellings constructed with conventional materials, (3) presents construction details that do not require the designer to perform analytical calculations, (4) suggests procedures for efficient plan-checking, and (5) presents recommendations including details and schedules for use in the field by construction personnel and building inspectors.

ATC-4-1: The report, The Home Builders Guide for Earthquake Design (June 1980), was published under a contract with HUD. Available through the ATC office. (57 pages)

Abstract: This report is a 57-page abridged version of the ATC-4 report. The concise, easily understood text of the Guide is supplemented with illustrations and 46 construction details. The details are provided to ensure that houses contain structural features which are properly positioned, dimensioned and constructed to resist earthquake forces. A brief description is included on how earthquake forces impact on houses and some precautionary constraints are given with respect to site selection and architectural designs.

ATC-5: This project will result in a report titled Recommended Guidelines for Seismic Design and Construction of Single-Story Masonry Dwellings in Seismic Zone 2 of the United States. The project is funded by HUD and involves review and evaluation of readily available research and earthquake damage reports in the subject area; close cooperation with the Earthquake Engineering Research Center of the University of California, Berkeley where masonry behavior is being evaluated through shaking table tests; and preparation of the recommended guidelines report based on the shaking-table test results. The project is scheduled for completion in 1985.

ATC-6: The report, Seismic Design Guidelines for Highway Bridges, was published under a contract with the Federal Highway Administration (FHWA). Available through the ATC office. (210 pages)

Abstract: The Guidelines are the recommendations of a team of sixteen nationally recognized experts that included consulting engineers, academics, state and federal agency representatives from throughout the United States. The Guidelines embody several new concepts which are significant departures from existing design provisions. An extensive commentary and an example demonstrating the use of the guidelines are included. A draft of the Guidelines was used to seismically redesign 21 bridges and a summary of the redesigns is also included.

ATC-6-1: The report, Proceedings of a Workshop on Earthquake Resistance of Highway Bridges, was published under a grant from NSF. Available through the ATC office. (625 pages)

Abstract: The report includes 23 state-of-the-art and state-of-practice papers on earthquake resistance of highway bridges. Seven of the twenty-three papers were authored by participants from Japan, New Zealand and Portùgal. The Proceedings also contain recommendations for future research which were developed by the 45 workshop participants.

ATC-6-2: The report, Seismic Retrofitting Guidelines for Highway Bridges, was published under a contract with FHWA. Available through the ATC office. (220 pages)

Abstract: The guidelines are the recommendations of a team of thirteen nationally recognized experts that included consulting engineers, academics, state highway engineers, and federal agency representatives. The guidelines, applicable for use in all parts of the U.S., include a preliminary screening procedure, methods for evaluating an existing bridge in detail, and potential retrofitting measures for the most common seismic deficiencies. Also included are special design requirements for various retrofitting measures.

ATC-7: The report, Guidelines for the Design of Horizontal Wood Diaphragms, was published under a grant from NSF. Available through the ATC office. (190 pages)

Abstract: Guidelines are presented for designing roof and floor systems so these can function as horizontal diaphragms in a lateral force resisting system. Analytical procedures, connection details and design examples are included in the Guidelines.

ATC-7-1: The report, Proceedings of a Workshop on Design of Horizontal Wood Diaphragms, was published under a grant from NSF. Available through the ATC office. (302 pages)

Abstract: The report includes seven papers on state-of-the practice and two papers on recent research. Also included are recommendations for future research which were developed by the 35 participants.

ATC-8: This project, Workshop on the Design of Prefabricated Concrete Buildings for Earthquake Loads, was funded by NSF. Project report available through the ATC office. (400 pages)

Abstract: The report includes eighteen state-of-the-art papers and six summary papers. Also included are recommendations for future research which were developed by the 43 workshop participants.

ATC-9: The report, An Evaluation of the Imperial County Service Building Earthquake Response and Associated Damage, was published under a grant from NSF. Available through the ATC Office. (231 pages)

Abstract: The report presents the results of an in-depth evaluation of the Imperial County Services Building, a 6-story reinforced concrete frame and shear wall building severely damaged by the October 15, 1979 Imperial Valley, California, earthquake. The report contains a review and evaluation of earthquake damage to the building; a review and evaluation of the seismic design; a comparison of the requirements of various building codes as they relate to the building; and conclusions and recommendations pertaining to future building code provisions and future research needs.

ATC-10: This report, An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance, was funded by the U.S. Geological Survey. Available through the ATC office. (114 pages)

Abstract: The report contains an in-depth analytical evaluation of the ultimate or limit capacity of selected representative building framing types, a discussion of the factors affecting the seismic performance of buildings, and a summary and comparison of seismic design and seismic risk parameters currently in widespread use.

ATC-10-1: This project, Seminar and Workshop on Earthquake Ground Motion and Building Damage Potential, was co-funded by the U.S. Geological Survey and the NSF. The combination Seminar/Workshop was held in San Francisco in March 1984. The overall objective of the project was to identify the critical aspects of ground motion and building response that should be considered in building design practice but currently are not. A report will be available through the ATC office.

ATC-11: The report, Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers, was published under a grant from NSF. Available through the ATC office. (184 pages)

Abstract: This document presents the results of an in-depth review and synthesis of research reports pertaining to cyclic loading of reinforced concrete shear walls and cyclic loading of joints in reinforced concrete frames. More than 125 reseach reports published since 1971 are reviewed and evaluated in this report, which was prepared via a consensus process that involved numerous experienced design professionals from throughout the U.S. The report contains reviews of current and past design practices, summaries of research developments, and in-depth discussions of design implications of recent research results.

ATC-12: This report, Comparison of United States and New Zealand Seismic Design Practices for Highway Bridges, was published under a grant from NSF. Available through the ATC office (270 pages).

Abstract: The report contains summaries of all aspects and innovative design procedures used in New Zealand as well as comparisons of United States and New Zealand design practice. Also included are research recommendations developed at a 3-day workshop in New Zealand attended by 16 U.S. and 35 New Zealand bridge design engineers and researchers.

ATC-13: This project, Earthquake Damage Evaluation Data for California, is funded by the Federal Emergency Management Agency (FEMA) and involves the development of damage-factor estimates, loss-of-function estimates, and other types of earthquake loss data for facilities in California. The project is scheduled for completion in 1985. A report will be available through the ATC office.

ATC-14: This project, Methods for Evaluating the Seismic Strength of Existing Buildings, is funded by NSF and will provide comprehensive methods for evaluating the seismic strength of existing buildings, both before and after strengthening. A team of 8 nationally recognized individuals from private design practice, research, and government regulatory agencies have been selected to guide the development of these methods. The project is scheduled for completion in 1986. A report will be available through the ATC office.