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PROCEEDINGS OF THE SEMINAR ON REINFORCED CONCRETE HIGH-RISE BUILDINGS IN JAPAN WITH SPECIAL CONCERN ON ASEISMIC DESIGN OF BEAM-COLUMN JOINTS

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ABSTRACT

As part of a collaborative project on beam-column joint performance involving researchers in New Zealand, Japan, China, and the United States, a special seminar was arranged in Tokyo to acquaint researchers and design engineers with Japanese design and construction practice and philosophy in reinforced concrete high-rise buildings. The purpose of this report is to disseminate information from that seminar to designers and researchers involved with reinforced-concrete ductile frame construction. The papers presented at the workshop on design criteria and construction techniques are complemented with a preface giving the background of reinforced concrete high-rise buildings in Japan. The focus of the seminar and this report is on beam-column joints.

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The editors gratefully acknowledge the time and effort provided by the authors of the papers included. Their open and detailed discussions will provide their colleagues all over the world an opportunity to learn from their experience. The support of the National Science Foundation was instrumental in enabling the U.S. portion of the project to be developed. Finally, the report would not have been possible without the support of Shimizu Construction Company, Ltd. for Mr. Kurose's research assignment to participate in the project at the University of Texas.

PREFACE

BACKGROUND OF REINFORCED CONCRETE HIGH-RISE BUILDINGS IN JAPAN

by

Yukinobu Kurose

Social Background

Recently construction of high-rise residential buildings has become a matter of social concern in Japan. Actually, demand for high-rise residential buildings has arisen mainly for the following reasons:

(1) Projects utilizing large sites containing unused facilities, such as a factory, are planned to redevelop cities.

(2) People prefer to live in the center of a city rather than spend many hours commuting from the suburbs. Therefore, urban condominiums sell well.

(3) It is so expensive and difficult to acquire a large site for a housing complex in a city that effective utilization of land becomes one of utmost importance.

(4) Regulations regarding volume and occupancy of residential buildings in an urban area have been eased recently.

One of the cases corresponding to the first reason mentioned above is the big housing project called "Shinjuku Nishitoyama," now underway in Tokyo (see Appendix B). Many construction companies, real estate companies, and design firms have joined the project and established a new company for developing the housing complex. In this project, an RC (reinforced concrete) 25-story residential building was designed by a design firm in association with five construction companies and was approved by the Minister of Construction. Seismic tests on beam-column joints modeling connections in the building were conducted in cooperation with those companies.

High-rise residential buildings must not transmit undesirable sounds or vibrations nor undergo excessive sidesway during wind or seismic actions in order to guarantee residents' privacy and their confidence in the structure. Therefore, the buildings must have moderate mass for soundproofing and sufficient stiffness for reducing sidesway response. From this viewpoint, structures cast with concrete are particularly suitable for residential buildings.

In Japan, SRC (steel composite reinforced concrete) structures have been developed extensively and applied to the majority of high-rise residential buildings. Generally it has been believed that RC structures are inferior to SRC structures in earthquake resistance, since RC buildings were severely damaged in past earthquakes. The height of RC buildings has been restricted by building codes. For example, special approval is required to construct an RC building higher than 31 m according to the building code in Tokyo. However, SRC structures generally cost more and require longer construction periods than RC structures because of the structural steel fabrication.

Japanese construction companies have been making their utmost efforts to cut costs and to shorten the construction period. Consequently, they have developed RC structural systems for high-rise residential buildings which are expected to have excellent earthquake resistance.

Legal Background

According to the building standard law in Japan, special permission from the Minister of Construction is required to construct a building higher than 60 m. In order to enforce this regulation, the Building Center of Japan has organized a review committee for high-rise buildings. Professors of architectural engineering constitute most of the committee which reviews high-rise building designs in detail, focusing their attention mainly on the structural design. After passing review, the building designs are approved by the Minister of Construction.

Sometimes local administrative offices require a design review even if the building is lower than 60 m. For example, an RC building higher than 31 m needs to be reviewed by the committee, according to the building code in Tokyo.

The Building Center of Japan has recently organized a special committee for RC high-rise buildings. The special committee, chaired by Professor Hiroyuki Aoyama (The University of Tokyo), does not review designs for actual buildings but reviews structural systems or concepts for RC high-rise buildings. Although the special committee evaluates the structural system in detail, the evaluation does not have any legal stature. Therefore, in order to construct a specific RC high-rise building, design review by the committee mentioned previously is required to get permission from the Minister of Construction, even though the structural system for the building was approved by the special committee. However, the design review might be less rigorous in that case.

Construction Background

Large construction companies in Japan include not only construction divisions but also design devisions, research activities, and computer centers. A construction division consists of many departments, including those which operate manufacturing and warehousing operations. Precast concrete products are often produced in factories and shipped to the site. A design division includes licensed engineers working on architectural design, structural design, and design of mechanical and electrical equipment. Engineers supervise construction of their designs from inception to completion. Construction corporation research institutes generally involve several laboratories which include a variety of testing facilities. Research activities cover not only civil and architectural engineering, but also widespread fields in engineering and science. Computer centers are equipped with large-scale computer systems and engaged in developing software. The divisions work together to develop the structural system for RC high-rise buildings in the following manner.

Design criteria for structural systems are established mainly by design engineers referring to AIJ (Architectural Institute of Japan) Standards and/or other codes. Although AIJ Standards do not cover high strength concrete (F_c > 360 Kg/cm^2), an RC high-rise building, 30 stories high generally requires concrete strength of 420 Kg/cm^2 or higher. Actually, design criteria for such concrete strengths have been extrapolated from design requirements for normal strength materials and verified by structural tests.

Structural tests on members and frames have been conducted at the research institutes. Specimens were cast with high strength concrete and reinforced with large size reinforcing bars typically used in construction. Cyclic loading (simulating seismic effects) was applied to the specimens. Dynamic tests using a shaking table were also conducted on lower story frames of a high-rise building.

The computer centers have developed several advanced programs for static and dynamic analysis in order to back up structural design work. For example, three-dimensional RC frames subjected to gradually increasing loads in an arbitrary direction are analyzed in the inelastic range by modeling the bidirectional interaction surface for column yielding. In addition to mass-spring-dashpot models, RC frame models having inelastic springs at member ends are used in dynamic response analysis to evaluate member end forces and rotations.

Construction procedures for the structural system have been established mainly by construction division staff. Procedures were aimed toward construction with high quality control and minimizing site work. Such procedures were verified by experimental work. For example, procedures utilizing high strength concrete evaluated in the laboratory.

Fabrication and assembly of reinforcement in beam-column joints was checked using full-scale models to alleviate on-site congestion problems.

The resulting structural system for RC high-rise buildings is the product of cooperative programs involving all divisions of a construction corporation.

The Present Situation

In Japan, 18-story and 25-story RC buildings have been constructed to date and 25-story and 30-story RC buildings are now under construction. All are residential buildings constructed by Kajima Corporation. Although other companies have not constructed RC high-rise buildings yet, they have developed structural systems for RC high-rise buildings. The system has been approved by review committees described in the previous section.

Some trends developing in Japan can be mentioned. There is pressure for structural members in residential buildings to have dimensions as small as possible to utilize space effectively. For example, a column section of 90 cm x 90 cm may represent an upper limit. A column supporting an area of 24 to 30 m^2 may be appropriate for high-rise residential buildings. High strength concrete and large-size rebars are being used to reduce beam and column sections in RC high-rise buildings. Therefore, beam-column joints might be critical in the following points:

(i) shear stresses acting on joints; and

(ii) development of straight bars through joints.

INTRODUCTION TO SEMINAR

by

Hiroyuki Aoyama, Seminar Chairman
The University of Tokyo

Whenever I had a chance to talk with our colleagues in foreign countries on our mutually interesting subject of the earthquake resistant design of reinforced concrete buildings, I used to feel that we are talking on different types of structures. I used to emphasize that we Japanese are using reinforced concrete only for buildings with the number of stories less than 6 or 7, while it is used in the high rise construction as well in overseas countries, and it is high rise reinforced concrete construction that receives much interest in the research work on beam-column joints, because as the building gets higher, it becomes, in general, more difficult to design.

However, for some reasons which I am not familiar with, the construction environment in Japan has changed quite rapidly. So far only two high-rise reinforced concrete buildings were constructed in Japan, one with 18 stories and another with 25 stories. But very soon construction will start of one building with 25 stories and another with 30 stories. Several other buildings were also designed. These high-rise buildings were all designed by design sections of Big Five construction companies. They are all apartment buildings, bearing very similar outlooks. Structurally they are all frame buildings, and were designed using nonlinear time-history dynamic response analysis for earthquake motions.

As we do not have an effective building code to cover high-rise reinforced concrete construction, each company has to set up their own building code in the form of self-regulation. A preliminary survey of these self-regulations has shown that they look almost similar in general, but they also contain considerable differences in detail. At present, the design of each high-rise apartment building is reviewed by experts in the Building Center of Japan, as individual design proposals not conforming to the prevalent design codes, for which a special permission of the Minister of Construction is necessary. However, I personally feel that very soon we would come up to the stage where a unified building code or something similar for this kind of construction is necessary.

Recently, the design method and self-regulation of each of five companies became quite open and well known to others of these five companies. So I felt it is now possible to ask five companies to assemble in a room, and to present and explain their design details to experts from foreign universities. It is hoped that the engineers in five companies would make the utmost use of this valuable occasion, and use the lessons in the future design of high-rise reinforced concrete frame buildings.

ORGANIZATION

"Seminar on Reinforced Concrete High-Rise Buildings in Japan"

Time: May 28, 1985 - 14:00-17:00
Followed by Reception 17:00-19:00

Place: "Keyaki" at Norinnenkin-Kaikan
(Reception to be in "Shirakaba")

Organized by: The Building Center of Japan

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1. STRUCTURAL SYSTEM FOR REINFORCED CONCRETE HIGH-RISE BUILDING

by

Yasuo Inada

Yukinobu Kurose

May 28, 1985

Shimizu Construction Company Ltd.

1. INTRODUCTION

Shimizu Construction Company has been investigating the behavior of RC (reinforced concrete) structures under seismic load conditions by both analytical and experimental studies. Consequently we have succeeded in developing the new RC structural system for high-rise buildings, which can provide excellent earthquake-resistant capability. The buildings constructed by this new structural system are expected to reveal good ductile behavior and sufficient strength when seismic forces act on them.

Besides, we have attempted to prefabricate as many members as possible in order to shorten the construction term and make the quality control easier. Thus, the majority of structural and nonstructural members are designed to be made of precast concrete. However, we have to firmly connect precast concrete structural members to each other with in-situ concrete in order to unify them. Especially floor slabs, which act as diaphragms, are made thoroughly with topping in-situ concrete cast over precast concrete decks. On the other hand, nonstructural precast concrete members such as partitions are effectively separated from the frame action in order to ensure that the building behaves as assumed in the analysis. Connections of such members are well designed to allow the seismic movement of the frame to take place. The structural system for RC high-rise buildings has been examined to a large extent by both analytical and experimental studies. The outline of the system is presented hereinafter.

2. DESIGN CONCEPT

Buildings will be subjected to various forces such as gravity forces, wind forces, seismic forces, etc. The structural design has to deal with all those forces which are expected to act on buildings. Especially in the case of RC high-rise buildings, seismic forces are generally dominant compared with the others. Therefore, we shall focus our attention on the earthquake-resistant capability of such buildings.

Architectural Planning

In order to ensure the safety of the buildings against seismic motions, we shall take account of the following items:

(1) Buildings should have a simple layout in plan. The building having a wing or wings in a plan, such as L-, T-, or Y-shape will behave in a complicated way, mainly due to torsional oscillation.

(2) Structural members such as beams, columns, and walls should be arranged in a manner where their centroid of rigidity will coincide with the center of gravity.

(3) Each floor should have the same shape in plan. Masses and rigidities should be distributed uniformly among stories.

Structural Design Criteria

In the structural design, we have adopted the following criteria to provide the buildings with excellent earthquake-resistant capability.

(1) Ductile Behavior

i) Hinge mechanism. Plastic hinges should be formed at beam ends. As is shown in Fig. 2.1, columns should not fail before beams except in the following cases:

a) Exterior columns subjected to axial tension;

b) Bottom of columns in the lowest story; and

c) Top of columns in the uppermost story.

ii) Failure mode. Failure should be in flexure rather than shear. Premature failure of beam-column joints should be prevented.

iii) Interstory deflection. Interstory deflections should not exceed the allowable values (for example - drift angle of 1/120). The maximum response of interstory deflection should be calculated by the dynamic response analysis using several acceleration records.

(2) Sufficient Strength

i) Force magnification for columns. As is shown in Fig. 2.2, the design forces for columns should be determined from the hinge mechanism of beam ends and be magnified appropriately by considering the dynamic behavior of the building.

ii) Ultimate lateral strength. The ultimate lateral strength of the building should not be less than the maximum shear response.

Structural Design Procedure

Figure 2.3 shows the structural design flow chart with special concern on the earthquake-resistant design. As is shown in this figure, the structural design consists of two different levels which are related to different intensity levels of earthquakes.

The first level corresponds to severe earthquakes which may occur during the serviceable term of the building. In this level, members should be designed for elastic stresses by the working stress method. The second level corresponds to the destructive earthquake whose return period is expected to be 100 to 200 years. In this level, members should be designed for stresses at the plastic hinge mechanism by the ultimate strength method. The story shear-deflection relationship should be also examined by the static load analysis based on inelastic characteristics of members.

Subsequently, the dynamic response analysis should be carried out for both levels. If the maximum response predicted by the analysis exceeds the allowable value, the structural design should start again from the beginning.

3. RC 30-STORY BUILDING

We have designed several buildings in accordance with the design concept described in the foregoing sections. We introduce one of them herein.

Figure 3.1 shows the perspective view of the designed building. The building is an RC 30-story apartment house. Figure 3.2 shows the typical floor plan of the building. As is shown in this figure, the building has five spans in both X and Y directions. All columns and beams of PG1 to PG6 are made of precast concrete. The other beams (G1, G2) are made of cast-in-place concrete. Precast concrete decks are used to construct composite concrete floor slabs. The building consists of moment-resisting frames without any shear wall.

Figure 3.3 shows the framing elevation of an E-Frame. As is shown in this figure, the concrete strength changes from 210 to 420 kgf/cm^2 according to the floor level. All members below the roof level are made of normal weight concrete, although the penthouse is made of light weight concrete. Exterior columns in the lower five stories are reinforced with steel skeleton as well as rebars to carry large axial force due to overturning of the frame.

All members are reinforced with deformed bars. The maximum size of rebars used in the building is a nominal diameter of 41 mm. The yield strength of rebars is as follows:

Longitudinal reinforcement	4,000 kgf/cm^2
Lateral reinforcement	3,500 kgf/cm^2
Slab reinforcement	3,000 kgf/cm^2

4. CONSTRUCTION METHOD

The construction procedure is illustrated in Fig. 4.1 and is summarized as follows:

(1) Precast concrete columns are erected on the floor. Columns shall be plumb and true. The column-column joint is located at the bottom of the erected column where longitudinal rebars are welded to those developed from the lower column by gas pressure welding (see Appendix A). All rebars in each column are welded automatically at once, as shown in Fig. 4.2. Then the joints are filled with in-situ concrete in order to unify the columns. As shown in Fig. 4.3, concrete is poured into the joints.

(2) Precast concrete wall panels are installed in position. Precast concrete beams are placed on the columns.

(3) Form panels for in-situ concrete beams are set. Precast concrete decks are placed on the edge of beams and forms.

(4) Prefabricated rebars for in-situ concrete beams are set in position.

(5) Top rebars in slabs and beams and hoops in beam-column joints are arranged. As shown in Fig. 4.4, bottom bars in precast concrete beams are developed by bending to be anchored in the beam-column joint, although the other bars in beams are made continuously straight in the joint.

(6) Topping concrete is cast over precast concrete beams and decks to make composite concrete floor slabs. At the same time, in-situ concrete beams are made and beam-column joints are poured with concrete.

5. EARTHQUAKE-RESISTANT DESIGN

The earthquake-resistant design for RC high-rise buildings is presented briefly herein by taking the case of the RC 30-story building.

Table 5.1 shows the summary of the earthquake-resistant design. As is shown in this table, the design consists of two different levels described previously. The base shear coefficient, which is defined as the base shear force divided by the total weight of the building, is 0.130 for the first level and 0.195 for the second level. The maximum velocity of the input ground motions in the dynamic response analysis is 25 cm/sec for the first level and 40 cm/sec for the second level. The allowable values of the maximum response are defined in terms of the story shear and the interstory drift angle, as is also shown in this table.

Figure 5.1 shows the result of proportioning sections. A pair of spirals are arranged as lateral reinforcement in beams and columns, although column-column joints and beam-column joints are reinforced laterally with closed hoops.

Figure 5.2 shows details of reinforcement in a C-Frame, which is composed with precast concrete beams and columns. Rebars at the exterior end of beams are developed by bending to be anchored in the beam-column joint which is enlarged with the stub. Bottom bars at the interior end of the beams are also developed by bending in the joint, although top bars are developed straightly in the joint. The bent bar development length in the joint should not be less than 30 times the bar diameter, while the straight bar development length should not be less than 20 times the bar diameter.

The shear strength of the beam-column joint should be ensured according to the following equation recommended by the Architectural Institute of Japan.

$$2f_s \cdot \psi + \rho_w \cdot \sigma_{wy} \geqq \frac{Q}{A_e} \text{(kgf/cm}^2\text{)}$$

where

f_s = allowable shear stress of concrete (kgf/cm^2)

ψ = restraint coefficient

ψ = 3 for an interior joint

ψ = 2 for an exterior joint

ρ_w = lateral reinforcement ratio

σ_{wy} = yield strength of lateral reinforcement (kgf/cm^2)

Q = shear force acting on the joint (kgf)

A_e = effective sectional area of the joint to carry the shear force (cm^2)

The static load analysis has been carried out based on inelastic characteristics of members. Figure 5.3 shows the structural model in the analysis. The plane frame, which consists of flexible line elements and rigid zones, has been analyzed.

Figure 5.4 shows the typical force-deformation relationships of the members. The moment-curvature relationship is idealized by the trilinear curve and the shear force-deflection relationship by the bilinear curve. Axial deformation of beams is assumed to be zero. The results obtained from the analysis are shown in Fig. 5.5. This figure demonstrates that the ultimate lateral strength of each story exceeds the required story shear force calculated from the base shear coefficient of 0.195 for the second level design. As is also shown in this figure, plastic hinges are formed at beam ends, the exterior columns at line-1, and the bottom of columns in the first story.

The dynamic response analysis has been carried out by using several different acceleration records. As is shown in Fig. 5.6, the building has been idealized by the multi-degree-of-freedom system which consists of masses, springs, and dashpots. Each mass is lumped at each floor level and is connected to each other with springs whose restoring force characteristics are shown in Fig. 5.7. Each spring also has elastic flexural stiffness to resist overturning moment. The viscous damping is assigned to each dashpot depending on the elastic stiffness. The damping factor is assumed to be 3% in the first mode. Considering the behavior of the substructure such as piles, we add both rotational and translational springs to the base. The selected input ground motions corresponding to the second level design are tabulated in Table 5.2. The amplitudes in each acceleration record are scaled in the manner where the maximum velocity value of the record reaches 40 cm/sec. The eigenvalues of the building have been analyzed and are shown partly in Table 5.3 and Fig. 5.8. The fundamental natural period, which has been evaluated based on the stiffness of cracked members, is as long as 2.07 seconds.

The response of the building to the input ground motions has been calculated by the step-by-step integration of the differential equation of motion. Some results obtained from the analysis are shown in Fig. 5.9 to Fig. 5.11 and are summarized as follows:

(1) The maximum response of the story shear force is smaller than the ultimate lateral strength at each story.

(2) The maximum response of the interstory drift is 1.54 cm, which corresponds to the drift angle of 1/185.

(3) The maximum response at each story does not reach the third stage of the trilinear backbone curve.

Throughout the static load analysis and the dynamic response analysis, we have verified that the designed building has good earthquake-resistant capability which satisfies the design criteria described previously.

6. SEISMIC TESTS ON COLUMNS AND BEAM-COLUMN SUBASSEMBLAGES

Summary

Experiments of 1/2 reduced models, columns and beam-column subassemblages, have been conducted for the RC 30-story apartment house at the Technical Research Institute of Shimizu Construction. These specimens are modeled for the first and the fourth stories of this building.

The main purposes of these column experiments are as follows:

(1) Proof for sufficient bearing strength and ductile behavior of the first story columns, whose bottom has formed a plastic hinge under lateral loadings.

(2) Proof for safety of the fourth story columns, in which plastic hinges should be formed at beam ends, until the ultimate strength of the beams is reached, and also for sufficient maximum strength and ductile behavior of the columns.

Therefore, we have made five experiments for the exterior and interior columns of both the first and fourth stories. The main objective of the beam-column subassemblage tests is to prove sufficient strength under the cylic lateral loadings and to understand the structural behavior of these joints until the ultimate condition.

Results of Experiments

Results of two models of the first story columns will be described below. Details of the specimen of the interior column (1G-IN) are shown in Fig. 6.2, and those of the exterior column (1G-OUT) are shown in Fig. 6.3. The axial loads of the interior column are constant and those of the exterior column are changeable from compression to tension according to the lateral load conditions. The loading device of the column experiments is shown in Fig. 6.1. There are four jacks used for the lateral loading, three jacks for the axial compressive loading and two jacks for the axial tensile loading. Both Figs. 6.4 and 6.5 show lateral load-deflection relationships of each column. Figures 6.6 and 6.7 illustrate crack patterns of each column specimen. From these results, it is proved that these column specimens have sufficient bearing strength and ductile behavior.

Results of two models of the beam-column subassemblage will be presented below. Details of these specimens are shown in Figs. 6.9 and 6.10. Both specimens are constructed with precast concrete beams and slabs. The loading device of the beam-column subassemblage is shown in Fig. 6.8. Lateral loadings for beams are controlled to maintain the vertical deflections of each beam equal in opposite directions. The axial loads to the interior column are maintained constant, while those to the exterior column change from zero to the design compressive force. Responses of the beam-column subassemblage to cyclic loadings are shown in Figs. 6.11 and 6.12. These responses indicate the relative story deflections of these specimens, which include both deflections of beams and columns, and also deformation of a joint-panel. Crack patterns of the two specimens are shown in Figs. 6.13 and 6.14. Each specimen shows good structural behavior and proves that both column and joint-panel behave satisfactorily from the state where joint-ends of the beams form plastic hinges, until the ultimate state of the beams.

7. SUMMARY

The new structural system for RC high-rise buildings has been presented herein. The practices in the earthquake-resistant design and the results obtained from the seismic tests have been demonstrated briefly by taking the case of the RC 30-story building. These analytical and experimental results have verified that the designed building has excellent earthquake-resistant performance.

The majority of members in the building will be made of precast concrete. It has been ensured by extensive experiments that precast concrete members behave in the same manner as in-situ concrete members. The precast concrete members except nonstructural elements are firmly connected to each other to be unified. Therefore, the buildings constructed with such precast concrete members are expected to have the same earthquake-resistant capability as in-situ concrete buildings.

Shimizu Construction Company will continue to make extensive studies on the reinforced concrete structures in order to develop the advanced system further.

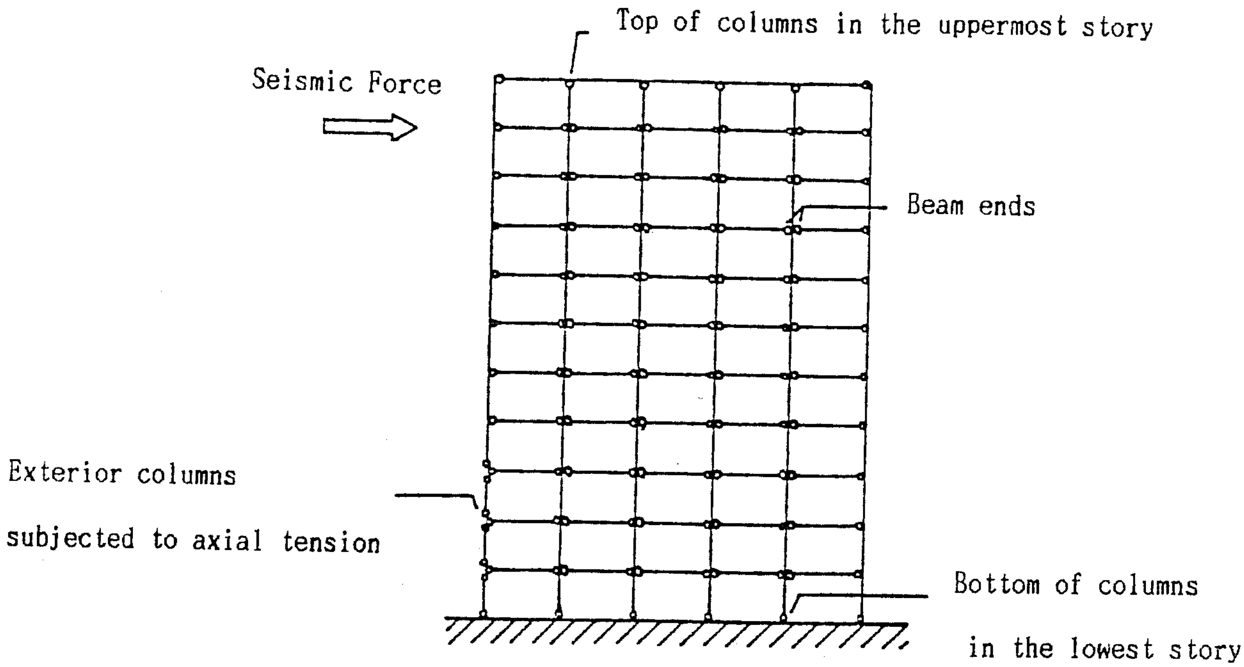
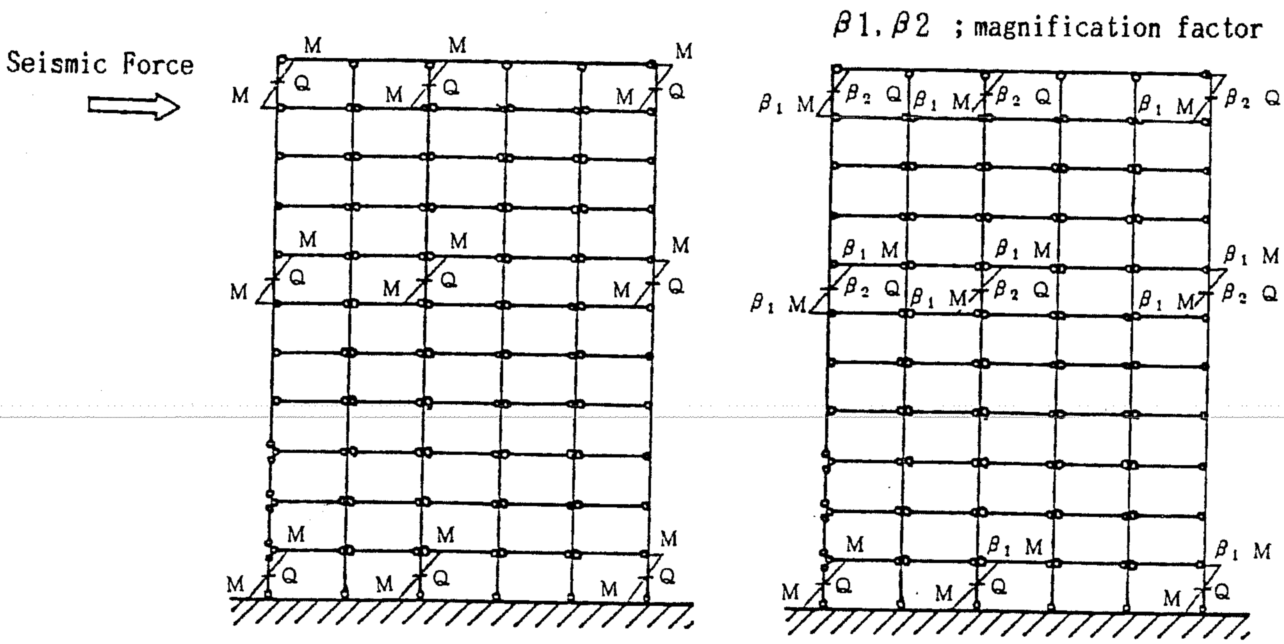


Fig.2.1 Allowable Locations of Plastic Hinges



(a) Forces Acting on Columns at Plastic Hinge Mechanism

(b) Magnified Forces

Fig.2.2 Force Magnification for Column Design

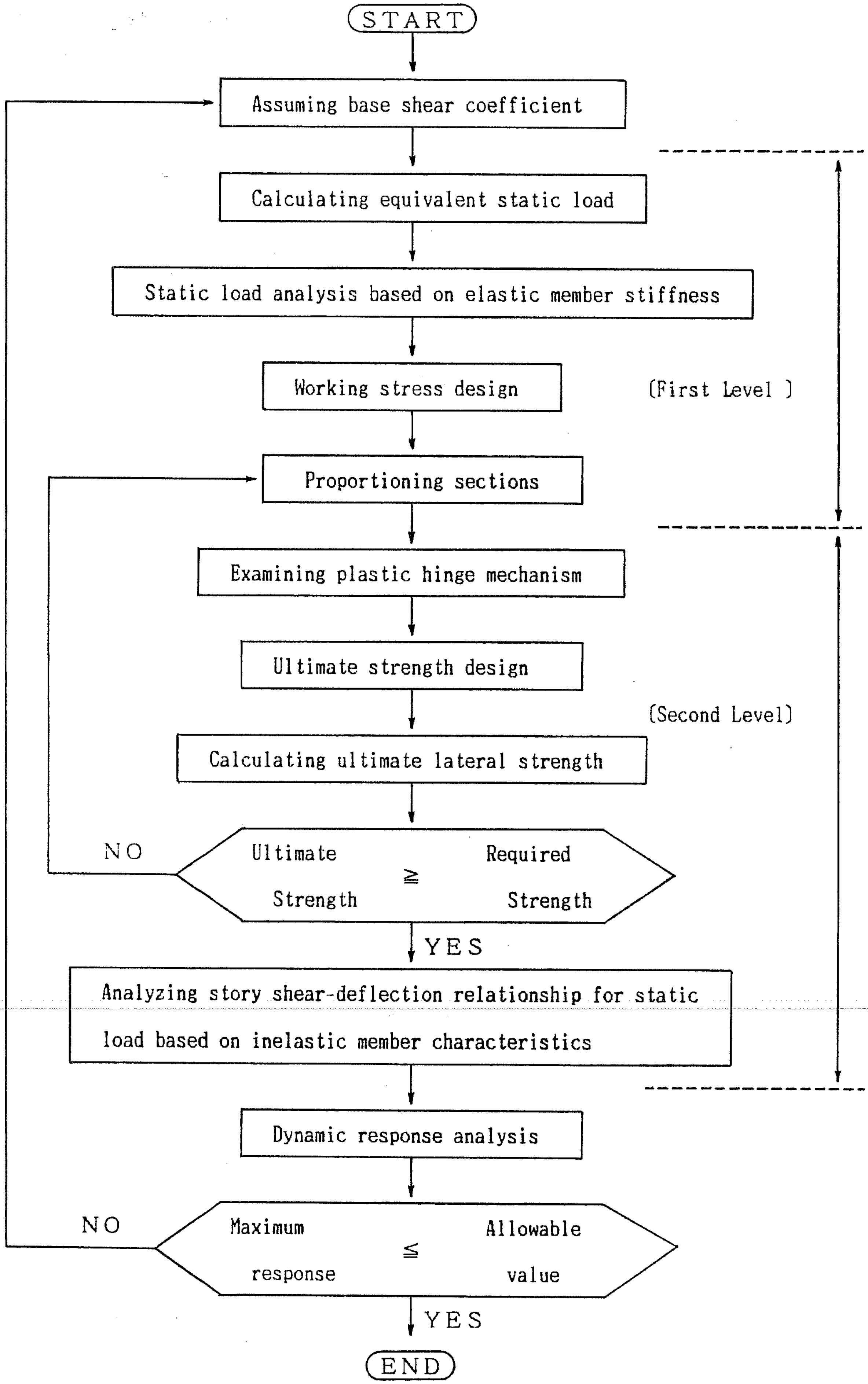


Fig.2.3 Structural Design Flow Chart

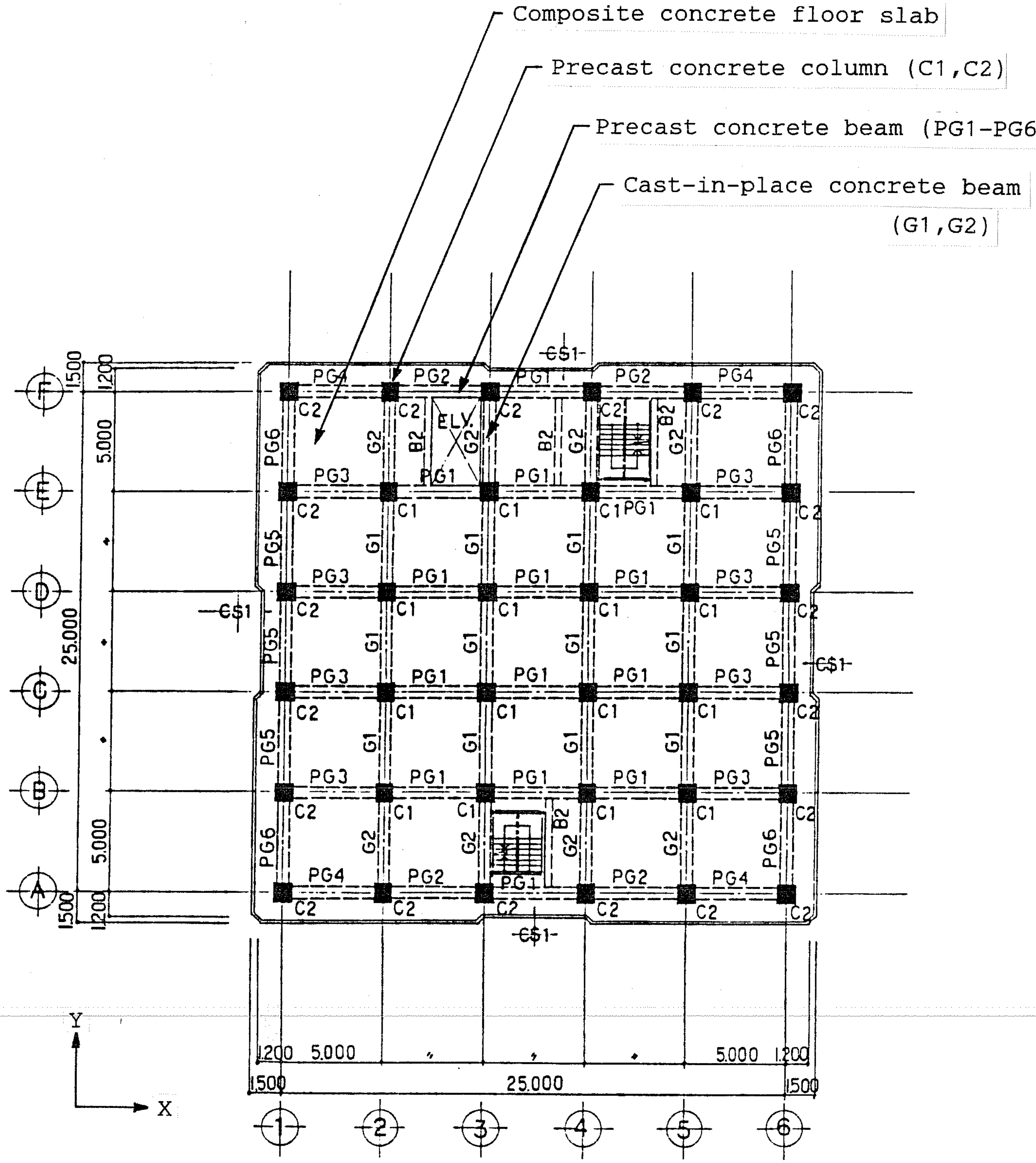


Fig.3.2 Typical Floor Plan

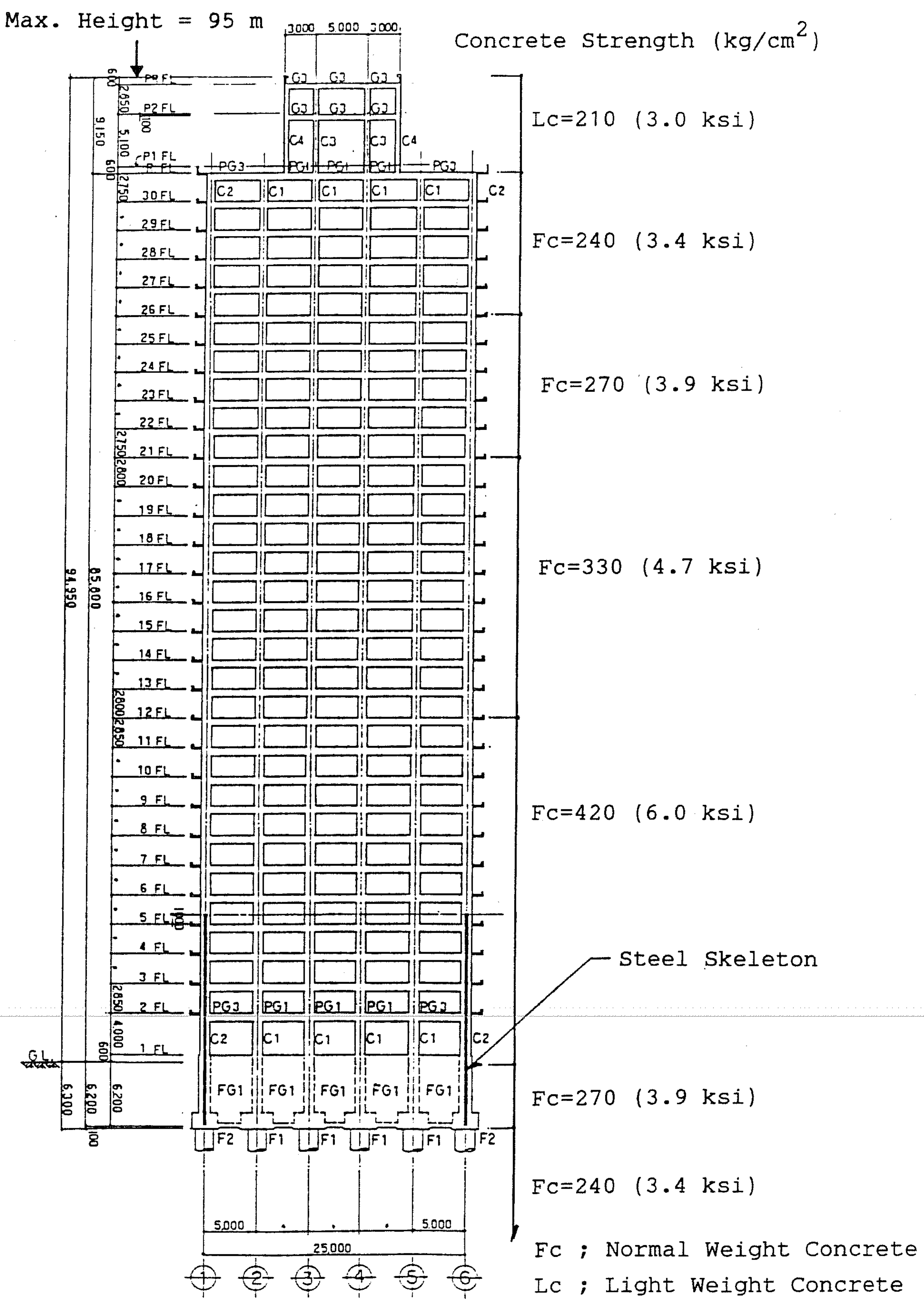
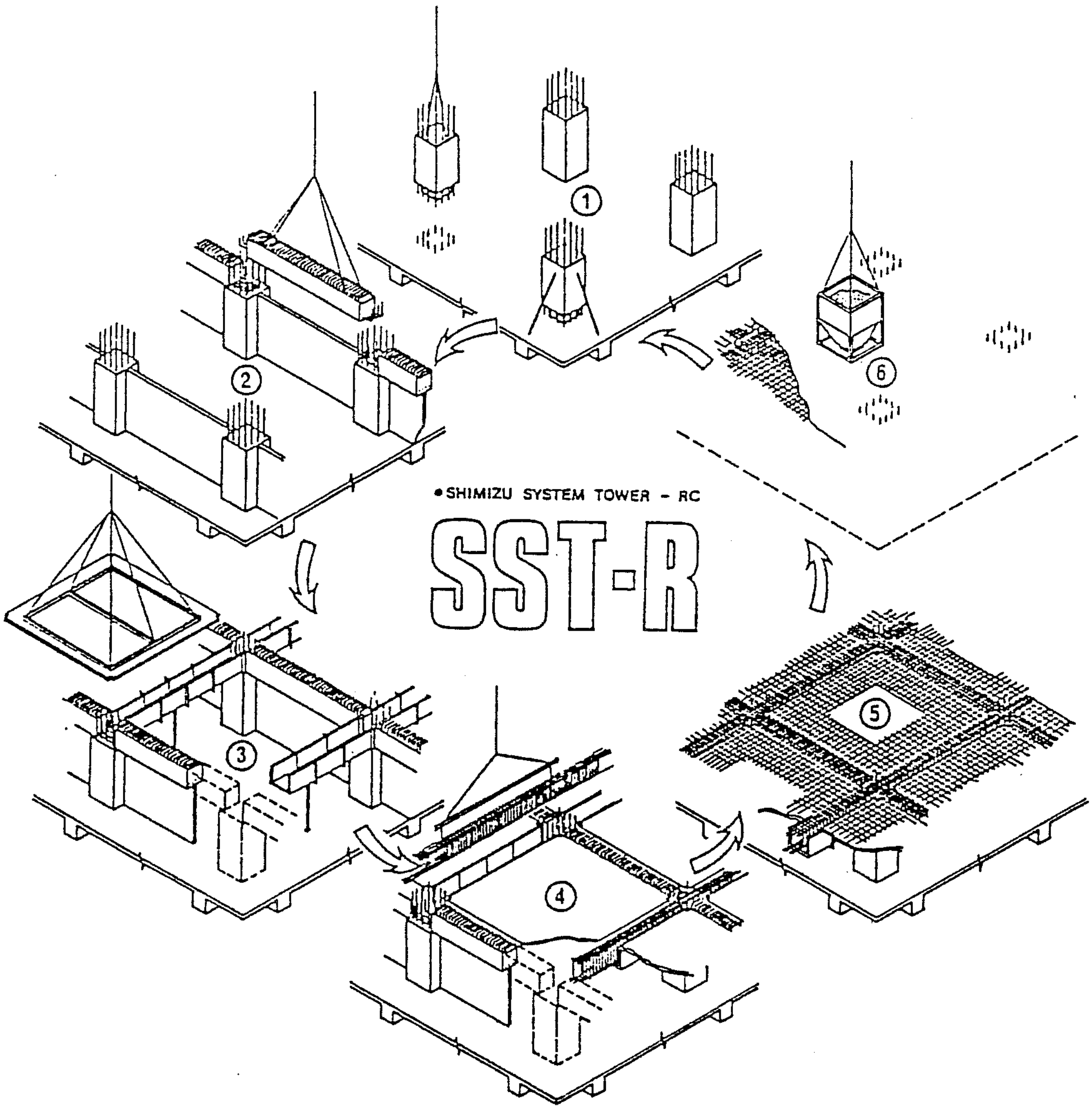


Fig.3.3 Framing Elevation (E-Frame)



① Erection of precast concrete columns

② Installation of precast concrete wall panels

Placing of precast concrete beams

③ Setting of form panels

Placing of precast concrete decks

④ Setting of prefabricated rebars for cast-in-place concrete bemas

⑤ Arrangement of rebars

⑥ Casting of in-situ concrete

Fig.4.1 Construction Procedure

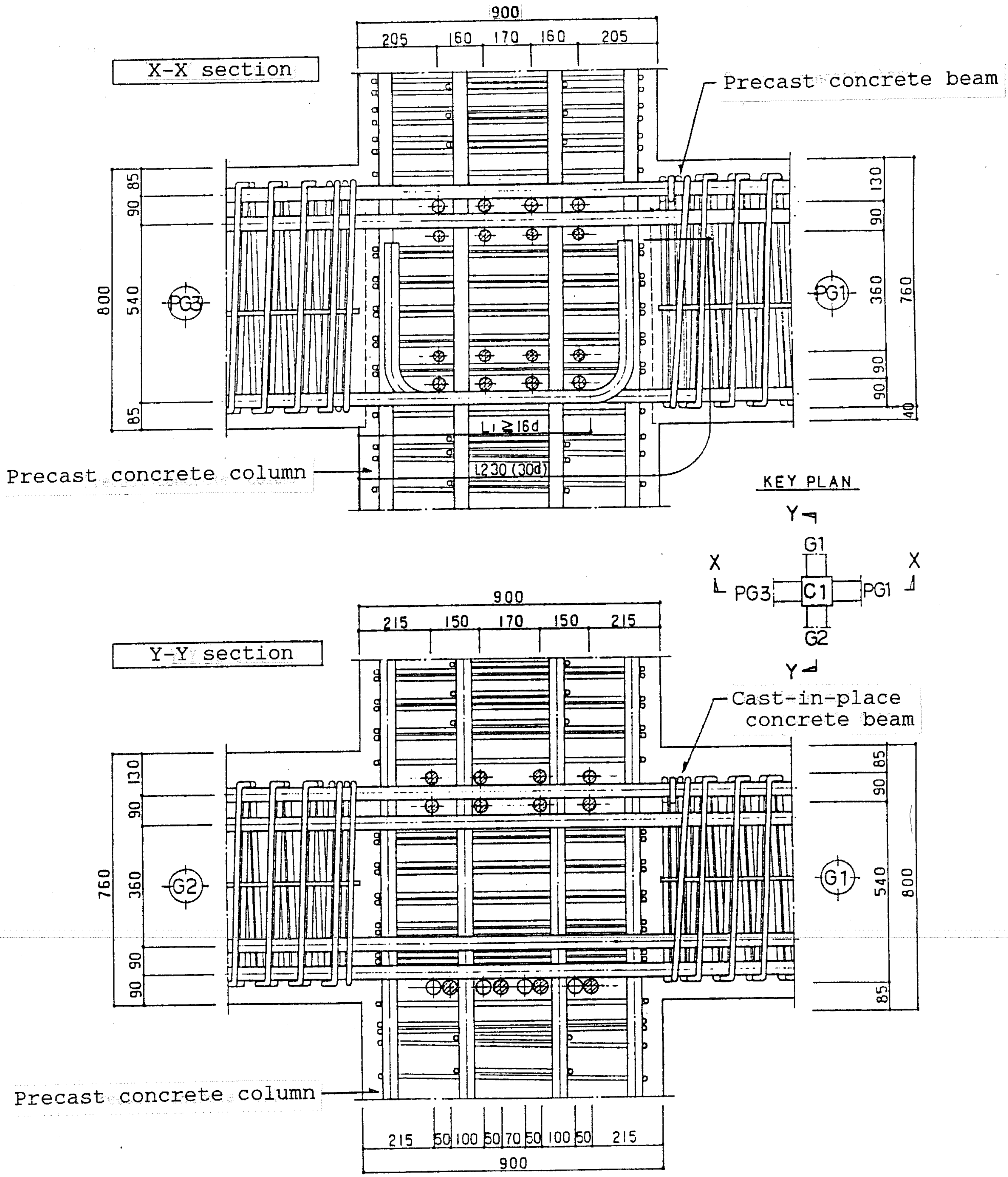


Fig.4.4 Beam-Column Joint Details

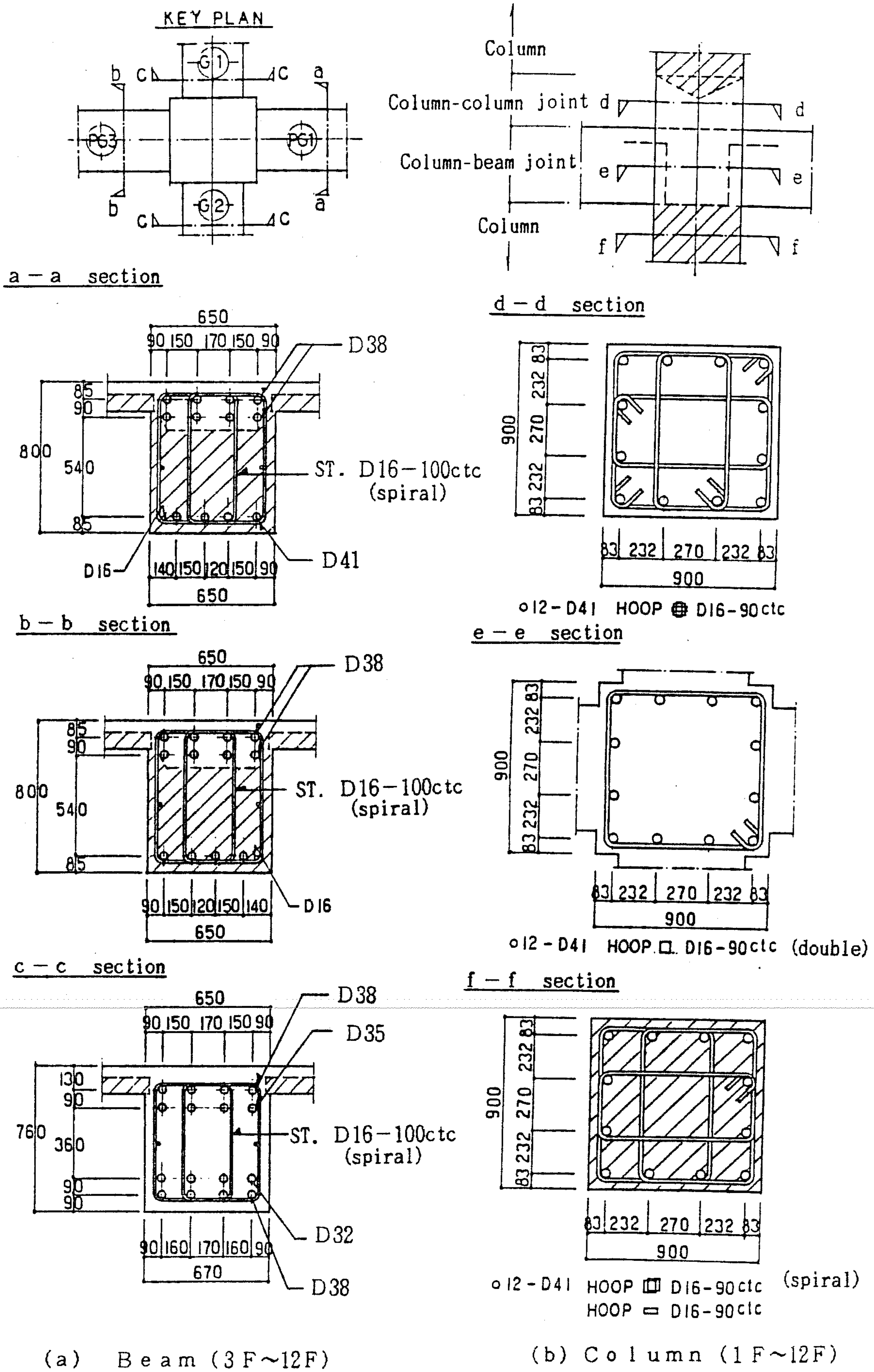
Table 5.1 Summary of Earthquake-Resistant Design

Items	First Level	Second Level
Base shear coefficient	0. 1 3 0	0. 1 9 5 ※1
Member characteristics in static load analysis	Elastic	Inelastic
Design method	Working stress method	Ultimate strength method
Maximum velocity of input ground motion	2 5 kines (kine=cm/s)	4 0 kines
Allowable interstory drift angle	1／2 0 0	1／1 2 0
Allowable story shear force	※2 Elastic limit of story shear	※3 Ultimate lateral strength

※1 The required lateral strength of the building is calculated on the basis of this value.

※2 The elastic limit of story shear is defined as the shear force acting on the story at the stage when the first plastic hinge is formed at any member in the story.

※3 The ultimate lateral strength should be calculated by moment distribution method.



Shaded portion ▨ shows precast concrete.

Fig.5.1 Beam and Column Sections

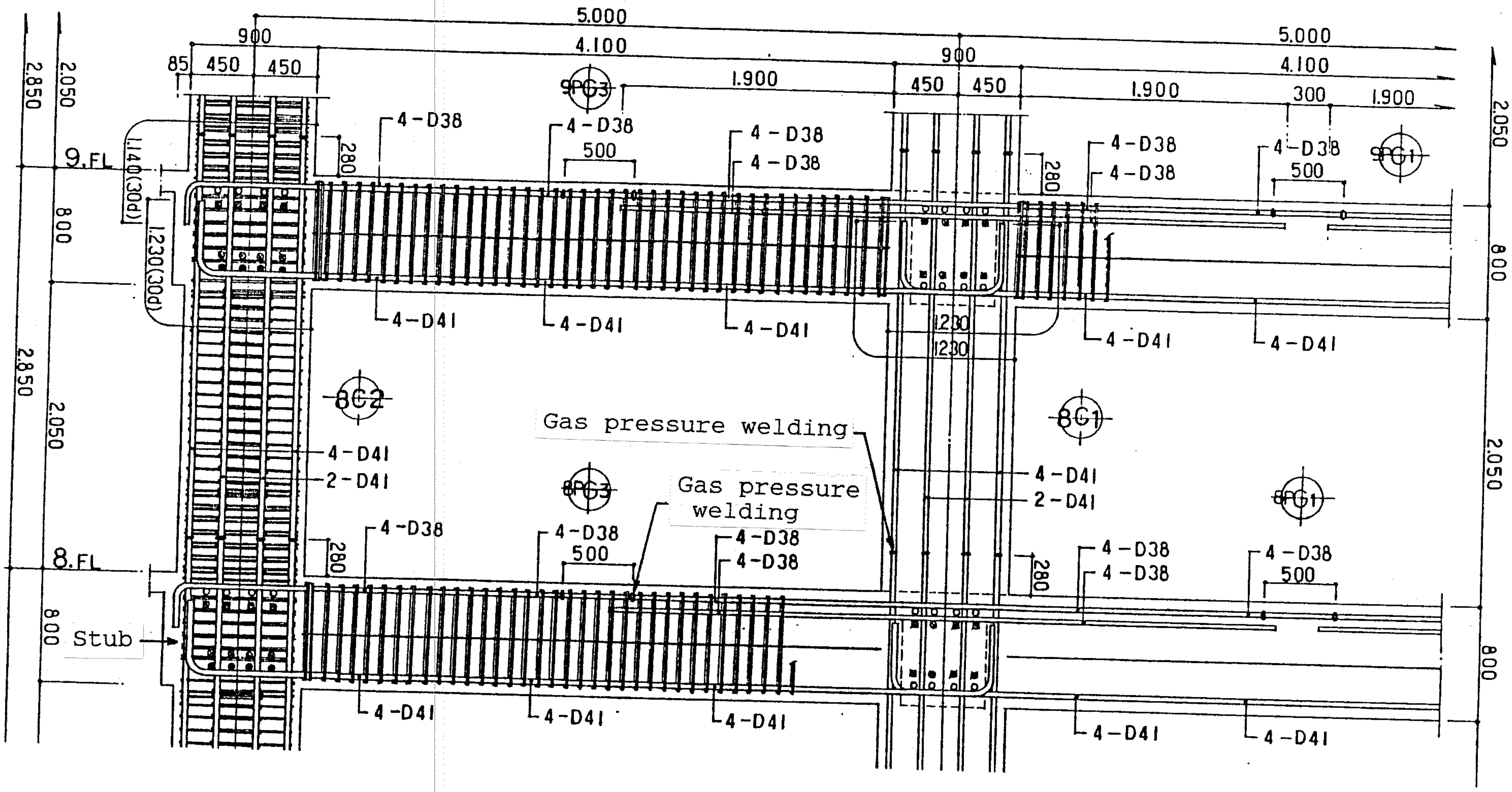


Fig.5.2 Reinforcement Details in C-Frame

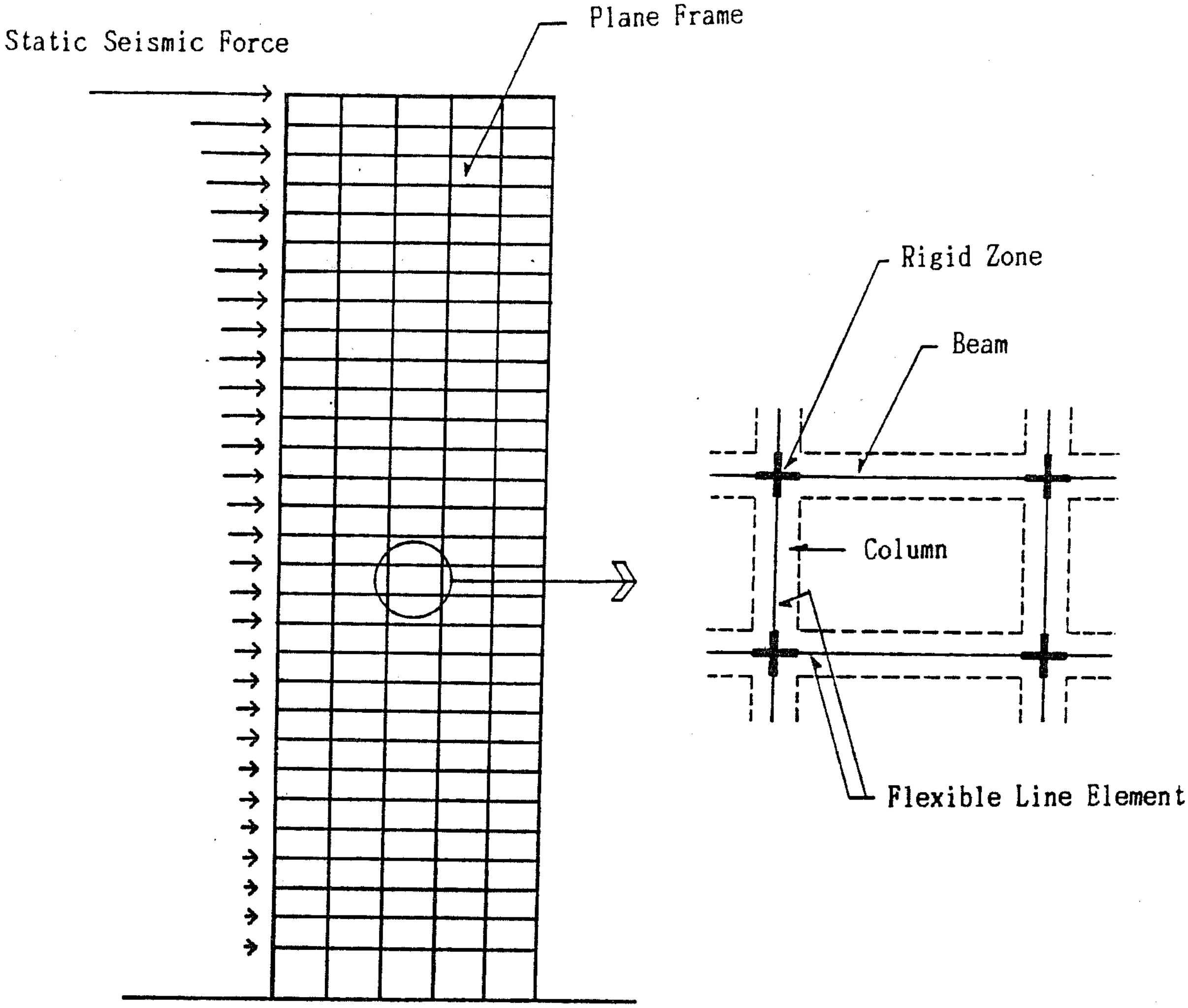


Fig.5.3 Structural Model in Static Load Analysis

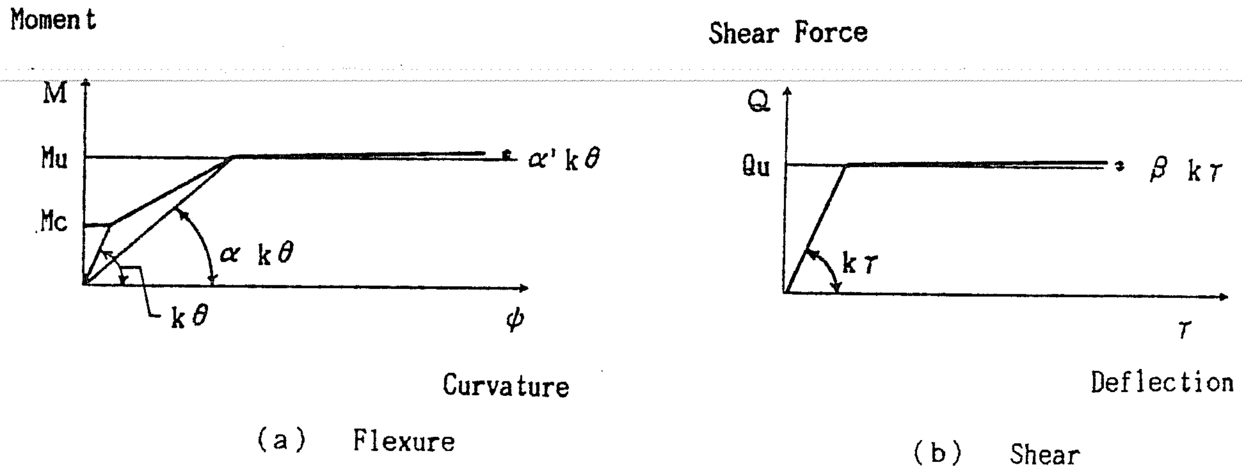


Fig.5.4 Force vs. Deformation Relationship for Line Elements

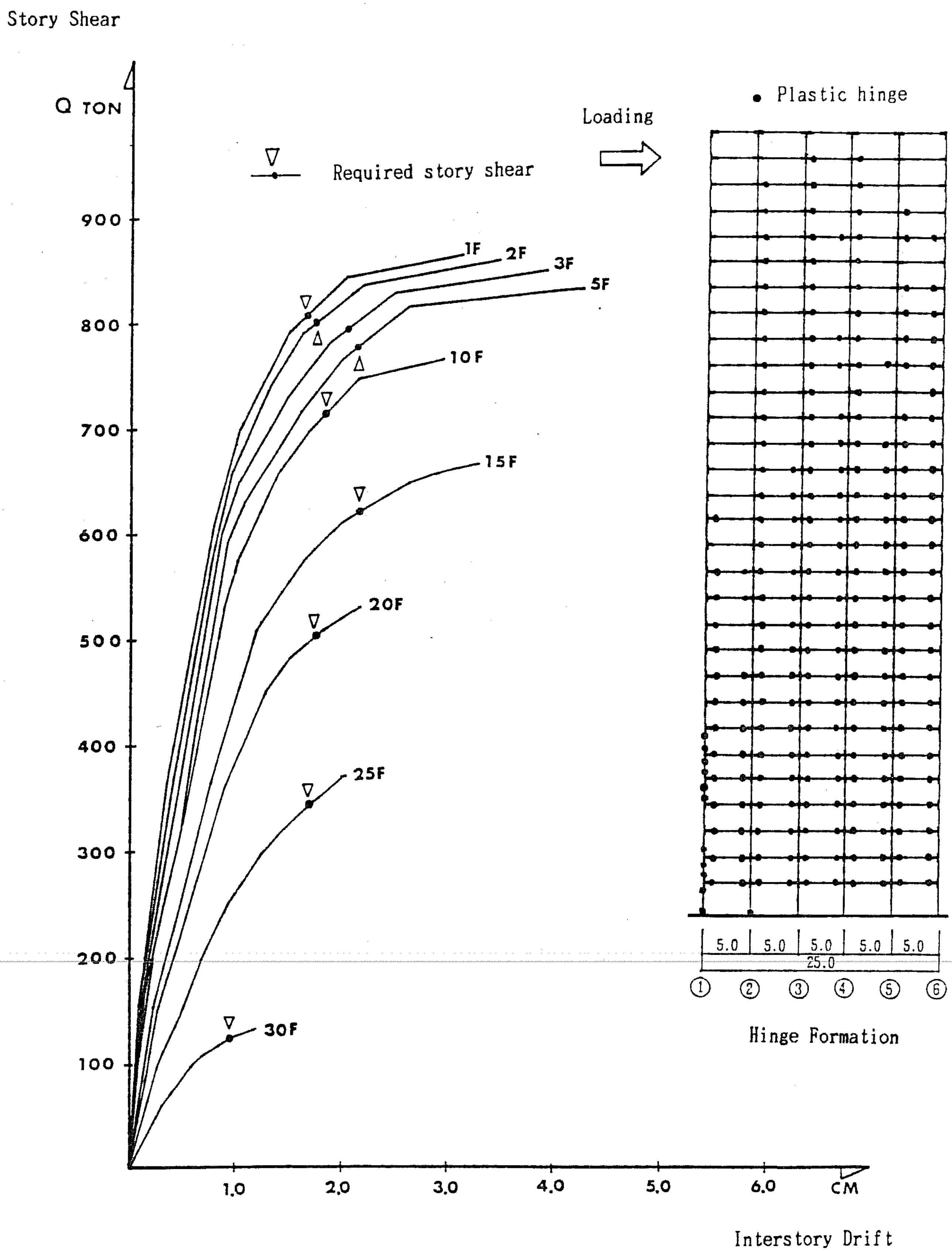


Fig.5.5 Results of Static Load Analysis

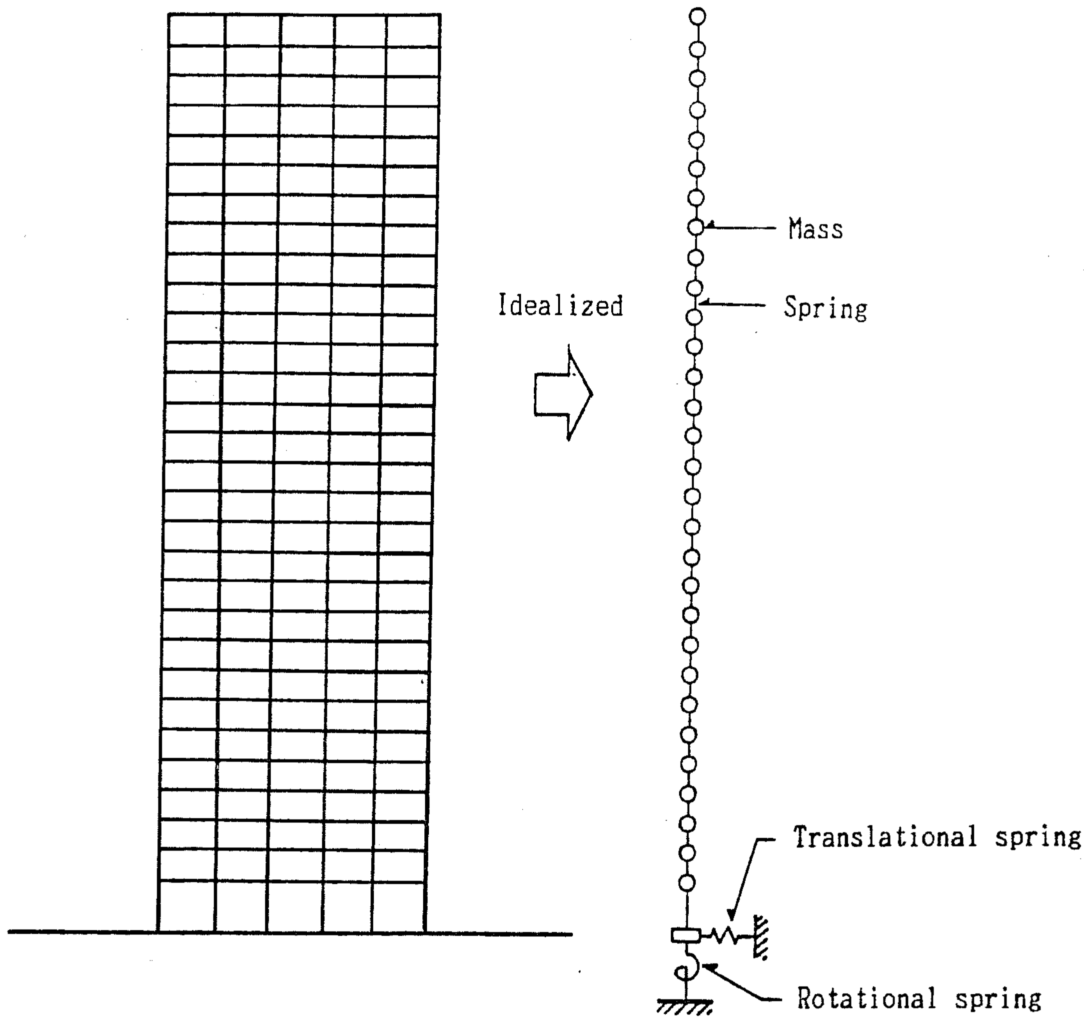
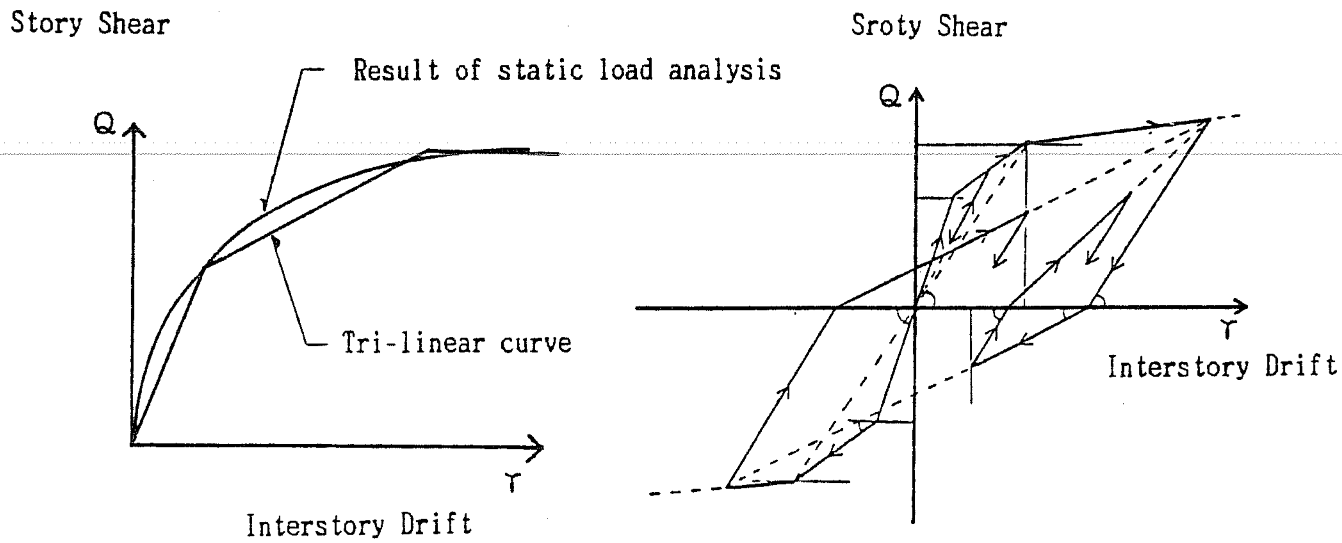


Fig.5.6 Multi-degree-of-freedom System in Dynamic Response Analysis



(a) Idealized Backbone Curve (b) Hysteresis Rules

Fig.5.7 Restoring Force Characteristics for Interstory Springs

Table 5.2 Input Ground Motions in Dynamic Response Analysis

Acceleration Record	Maximum Acceleration (gals)	Duration Time (sec)
El Centro 1940 NS	4 0 0	2 5
Taft 1952 EW	4 0 0	2 5
Tokyo-101 1956 NS	4 5 0	1 0
Hachinohe 1968 NS	2 4 5	4 0

Table 5.3 Natural Period T and Participation Factor ß

	1st	2nd	3rd
T (sec)	2.07	0.68	0.39
β	1.43	0.62	0.32

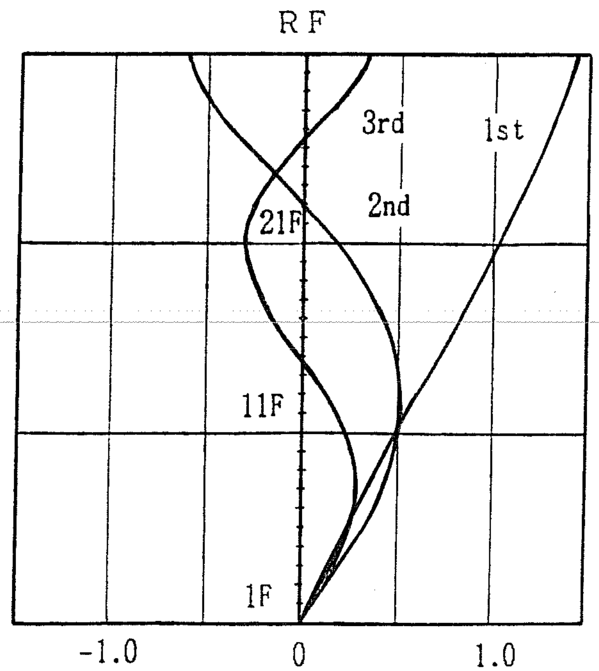


Fig.5.8 First Three Modes

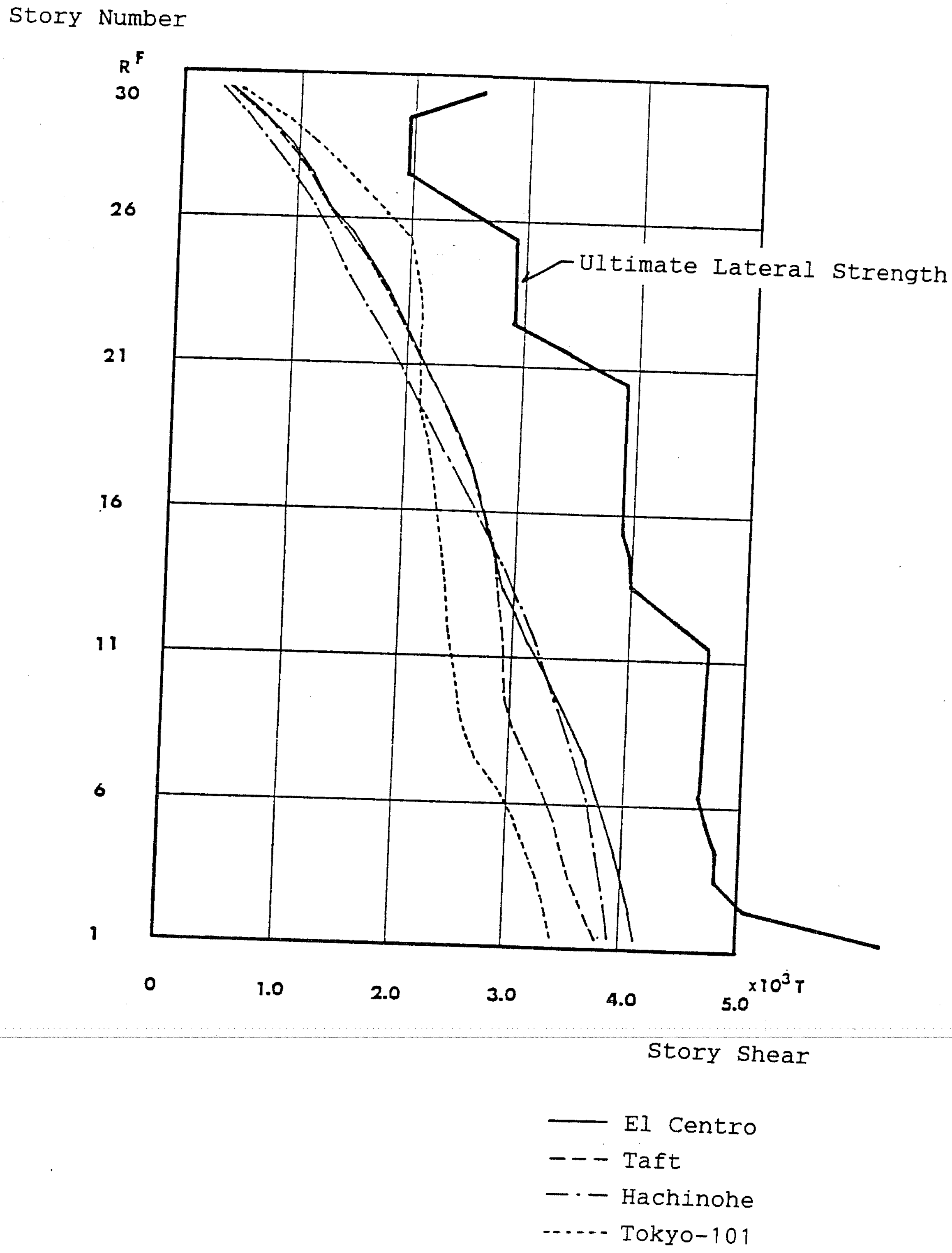


Fig.5.9 Maximum Response of Story Shear

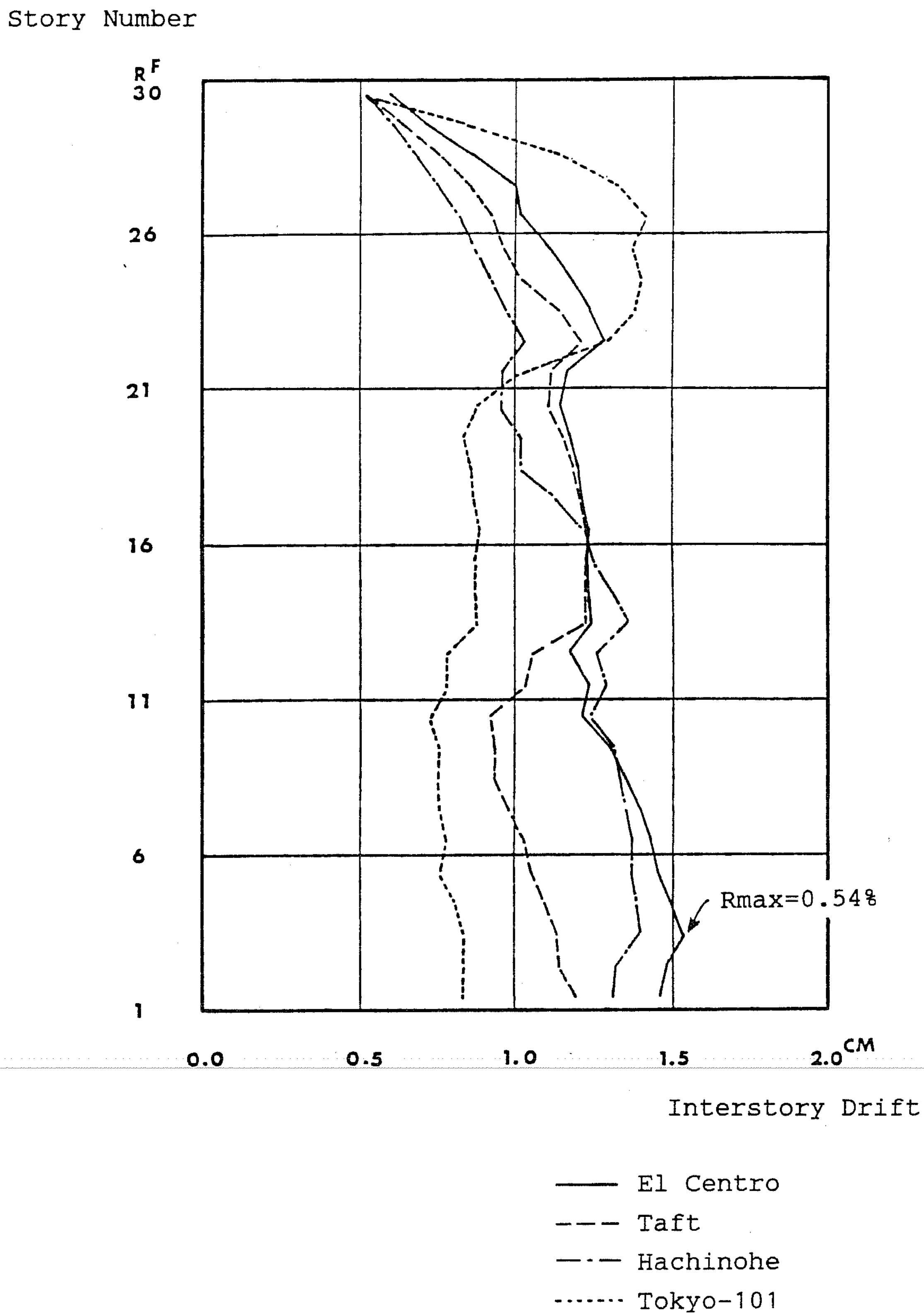


Fig.5.10 Maximum Response of Interstory Drift

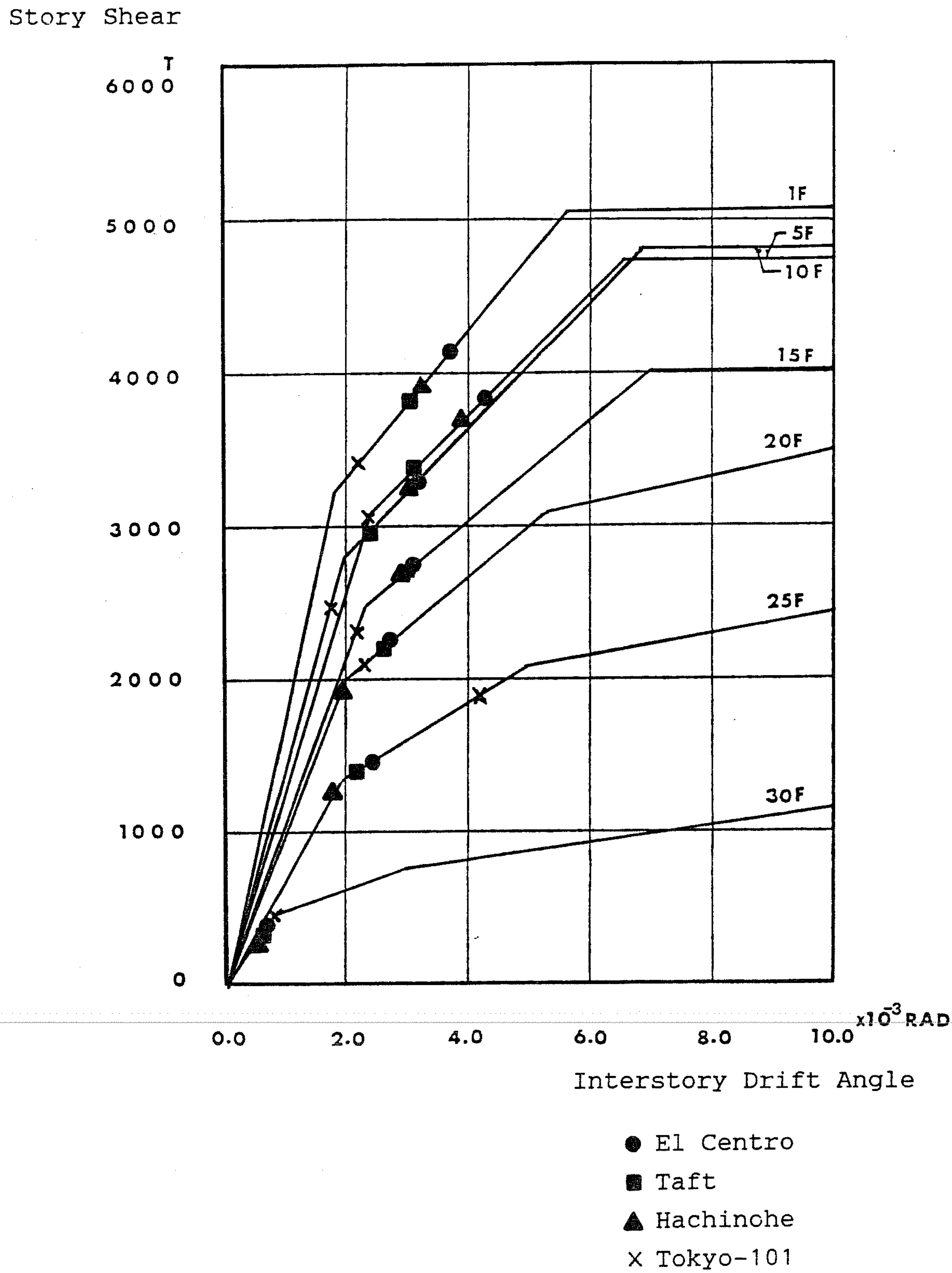


Fig.5.11 Maximum Response on Story Shear - Drift Angle Curves

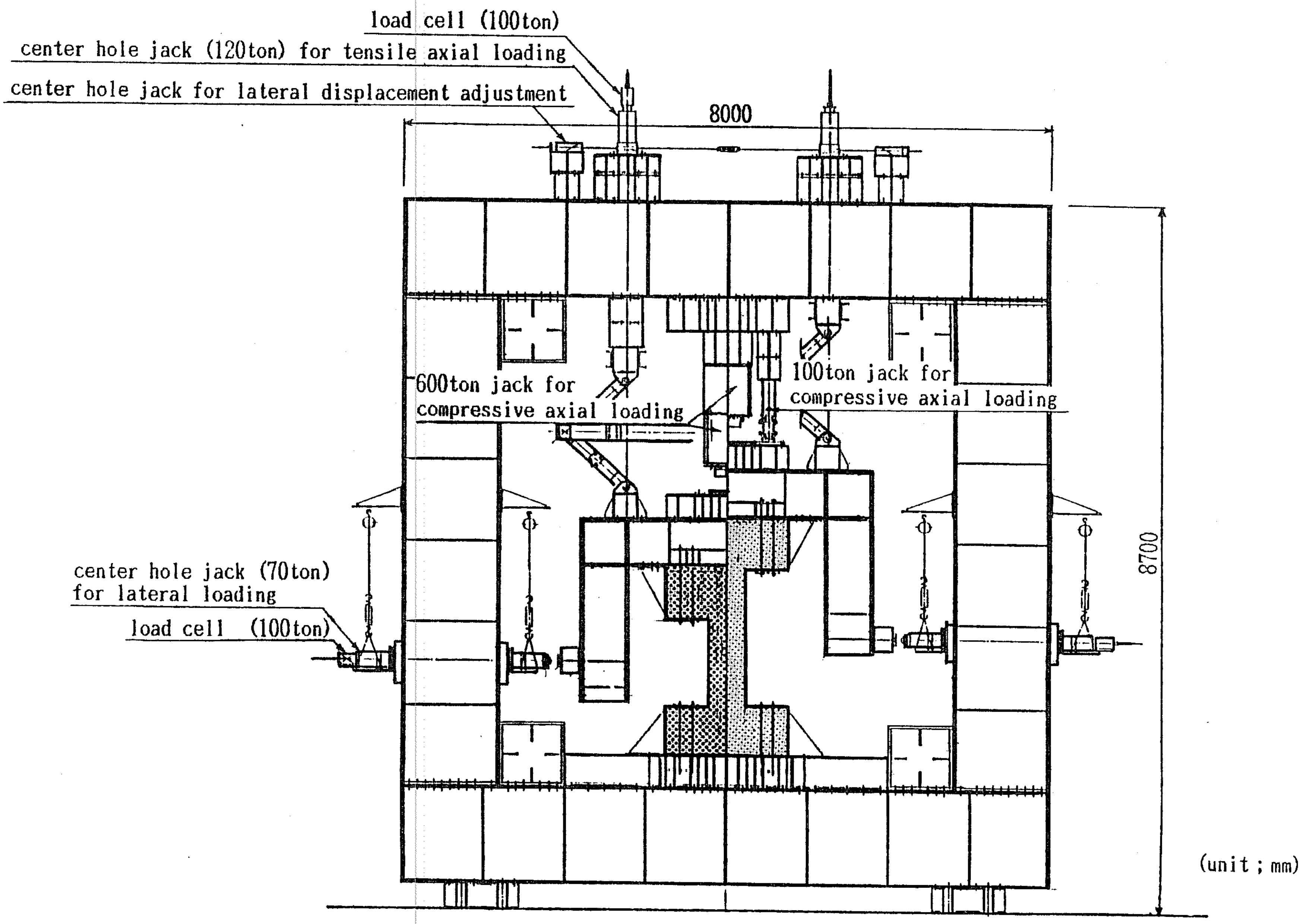
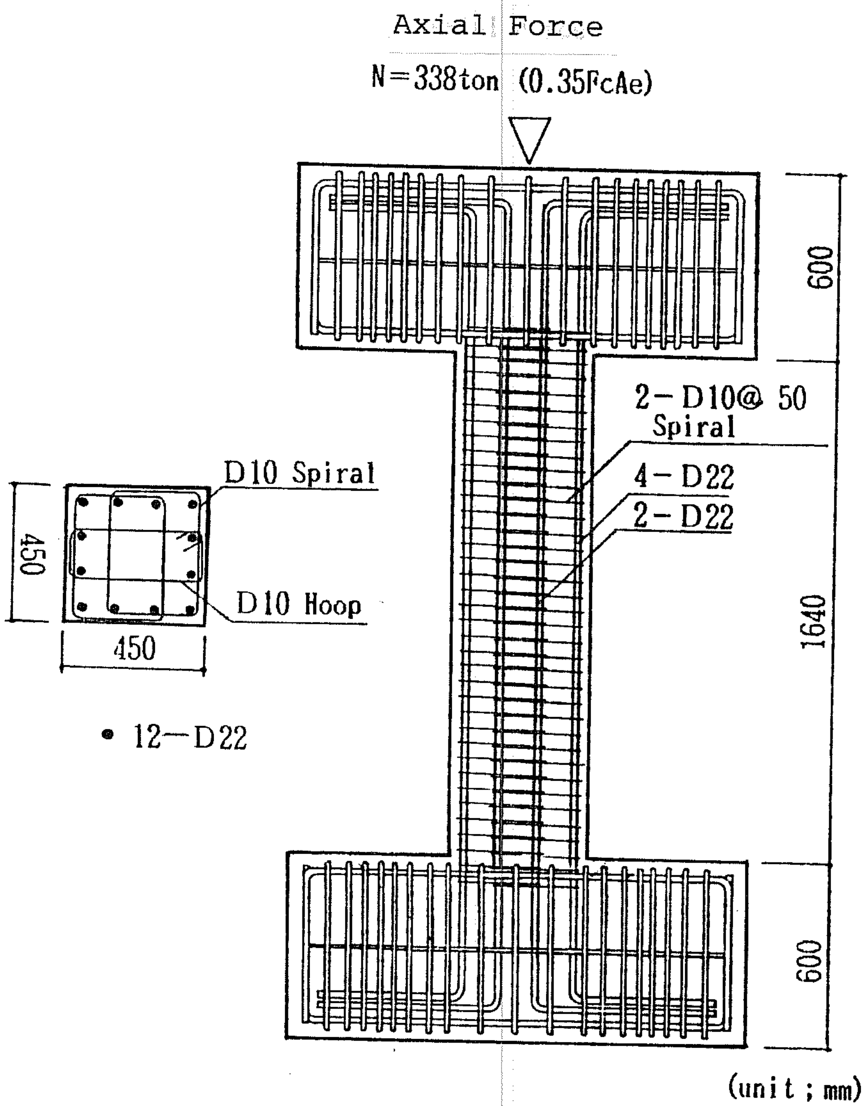


Fig.6.1 Loading Setup for Column Tests



Material Properties

(a) Reinforcement

	σy kg/cm²	εy $\times 10^{-6}$	σu kg/cm²	sE ton /cm²
D22	4113	2317	6563	1780
D10 Hoop	3878	2077	5329	1850
D10 Spiral	3029	1751	5075	1730

(b) Concrete

$c\sigma B$ kg/cm²	$c\sigma t$ kg/cm²	$c\varepsilon B$ $\times 10^{-6}$	cE1/3 ton/cm²	ν
430	30.0	2171	292	0.175

Fig.6.2 Details of Interior Column Specimen (1G-IN)

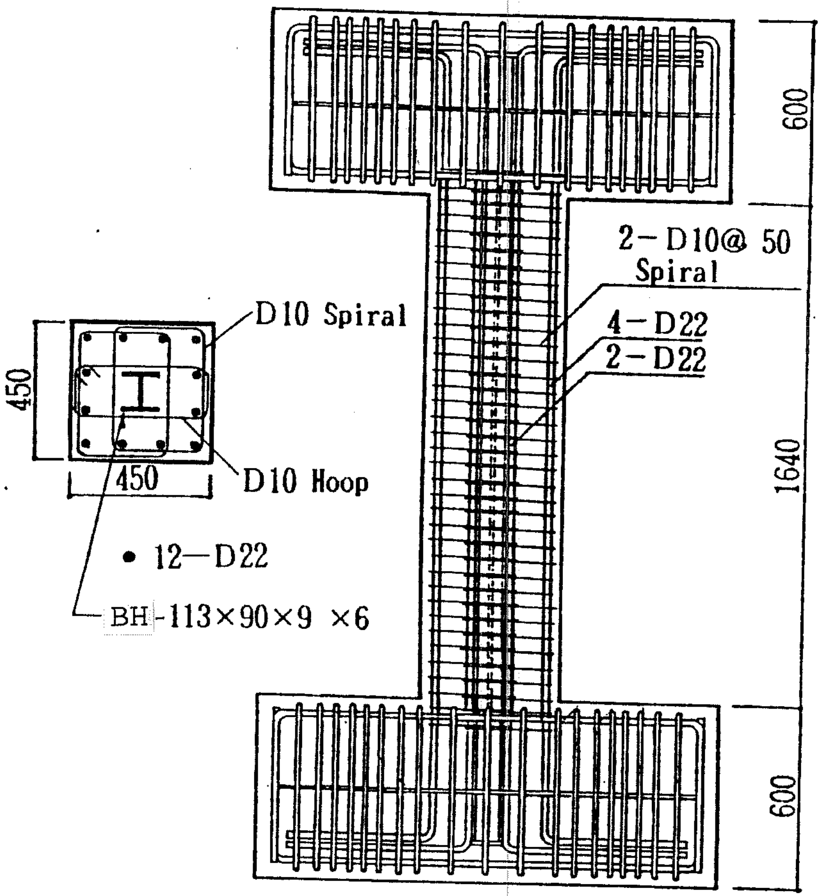
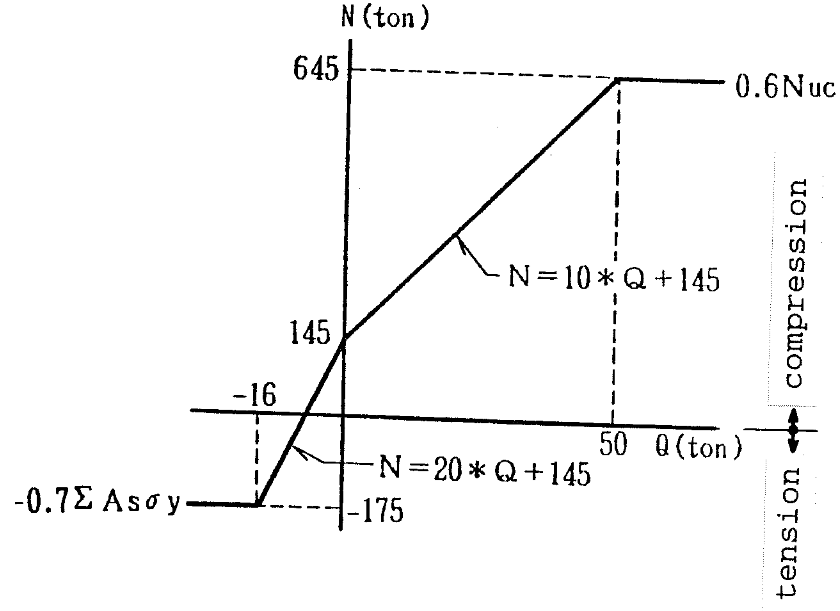


Fig.6.3 Details of Exterior Column Specimen (1G-OUT)

Axial Force N vs. Shear Force Q



Material Properties

(a) Reinforcement
(same as 1G-IN)

(b) Steel

	σy kg/cm²	εy $\times 10^{-6}$	σu kg/cm²	sE ton/cm²
Web t=8.6	4015	2012	5395	2030
Flange t=5.7	4266	2307	5462	2020

(c) Concrete

$c\sigma B$ kg/cm²	$c\sigma t$ kg/cm²	$c\varepsilon B$ $\times 10^{-6}$	cE 1/3 ton/cm²	ν
444	26.7	2206	299	0.144

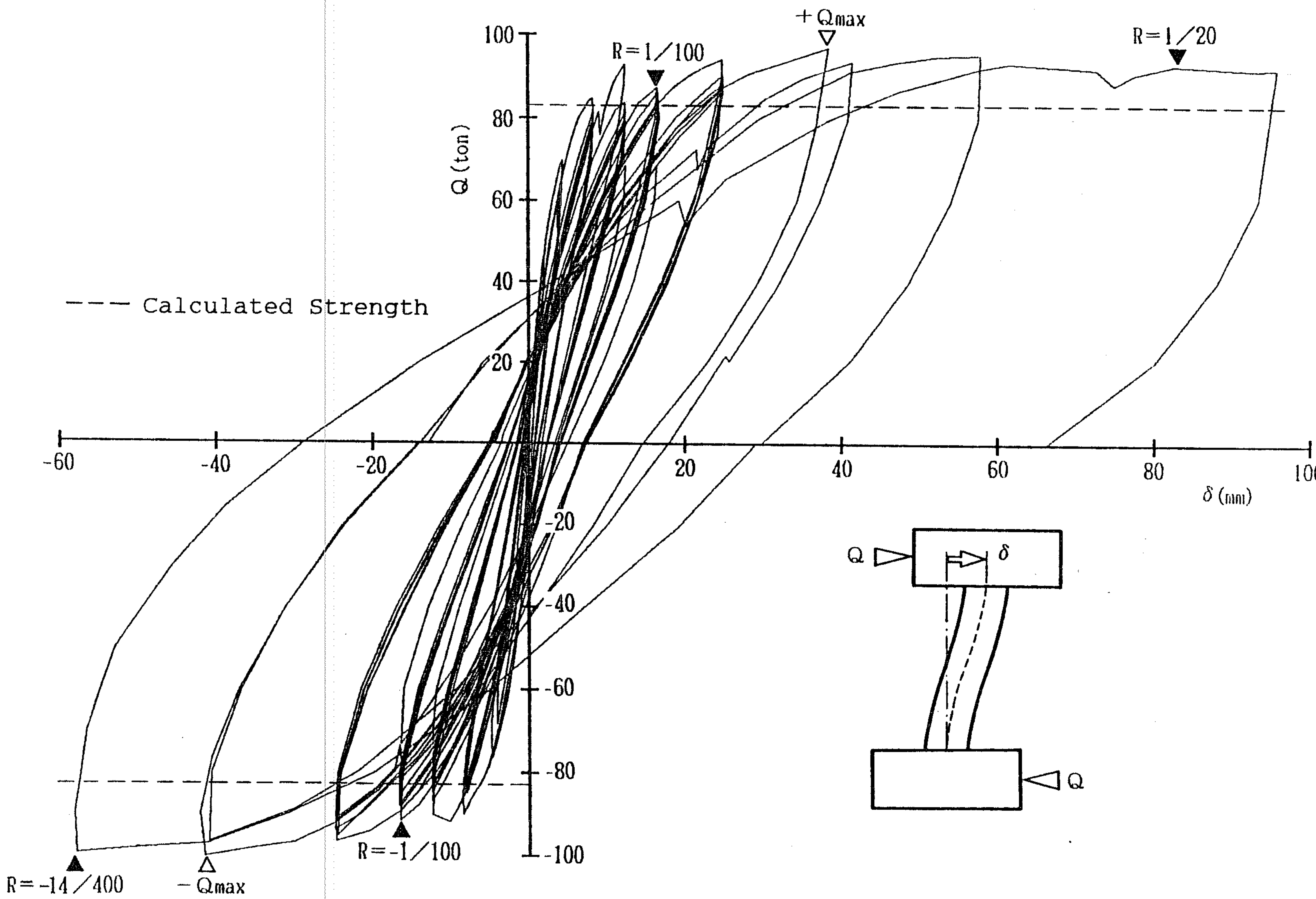


Fig.6.4 Lateral Load vs. Deflection Relation for Specimen 1G-IN

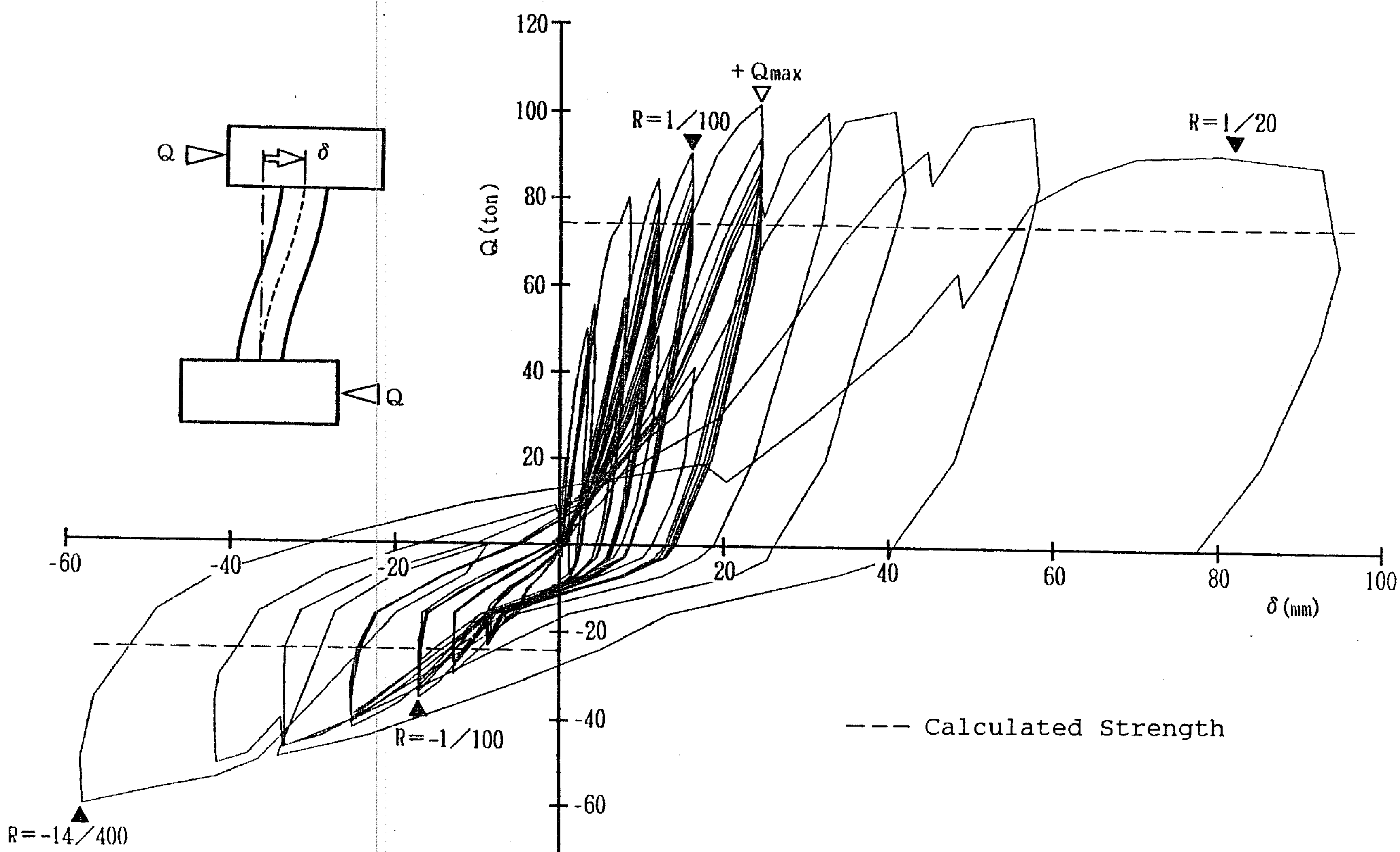


Fig.6.5 Lateral Load vs. Deflection Relation for Specimen 1G-OUT

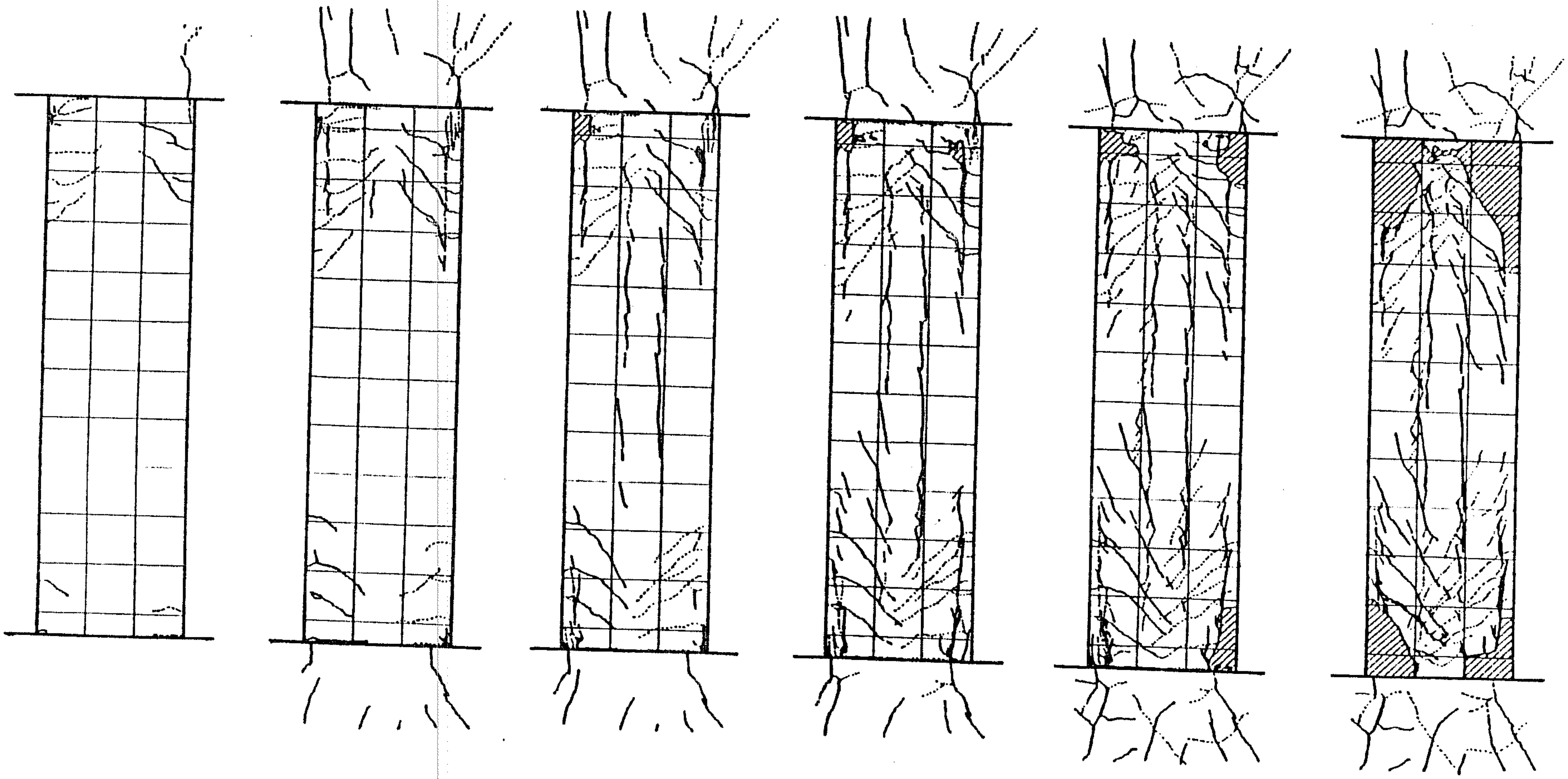
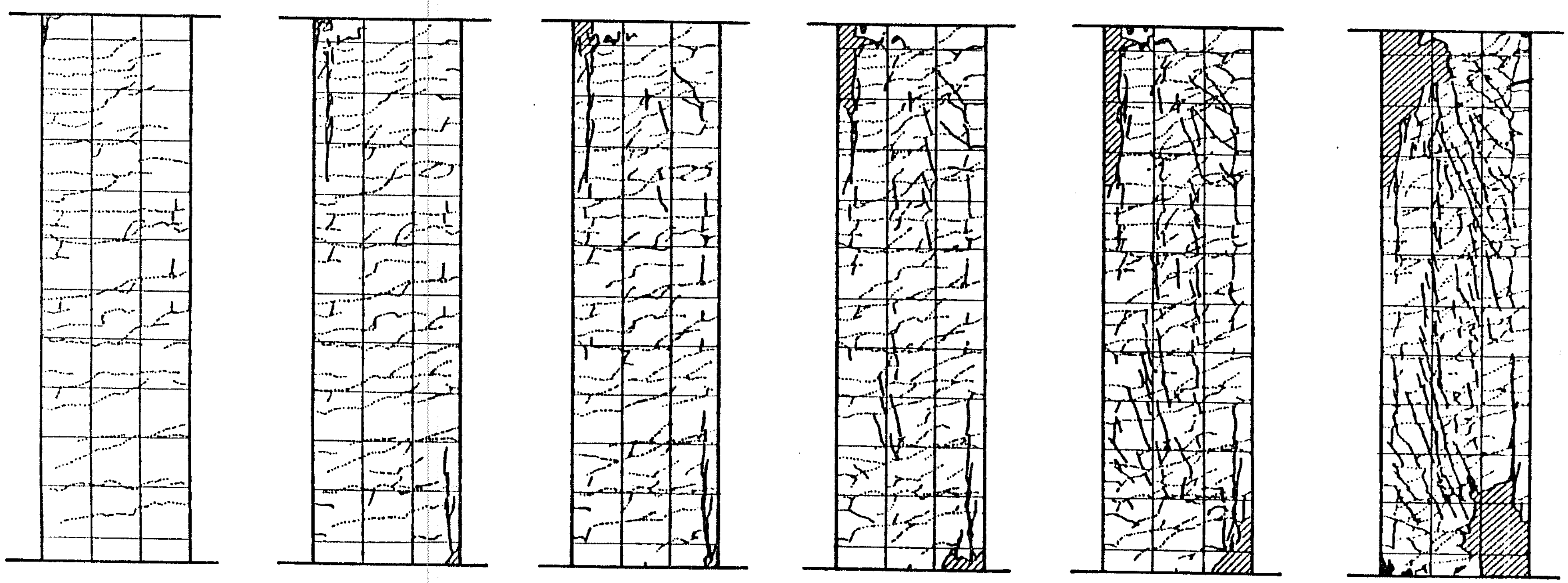


Fig.6.6 Crack Patterns for Specimen 1G-IN



R=-1/400 R=-2/400 R=-3/400 R=-4/400 R=-6/400 after testing

R ; Drift Angle

Fig.6.7 Crack Patterns for Specimen 1G-OUT

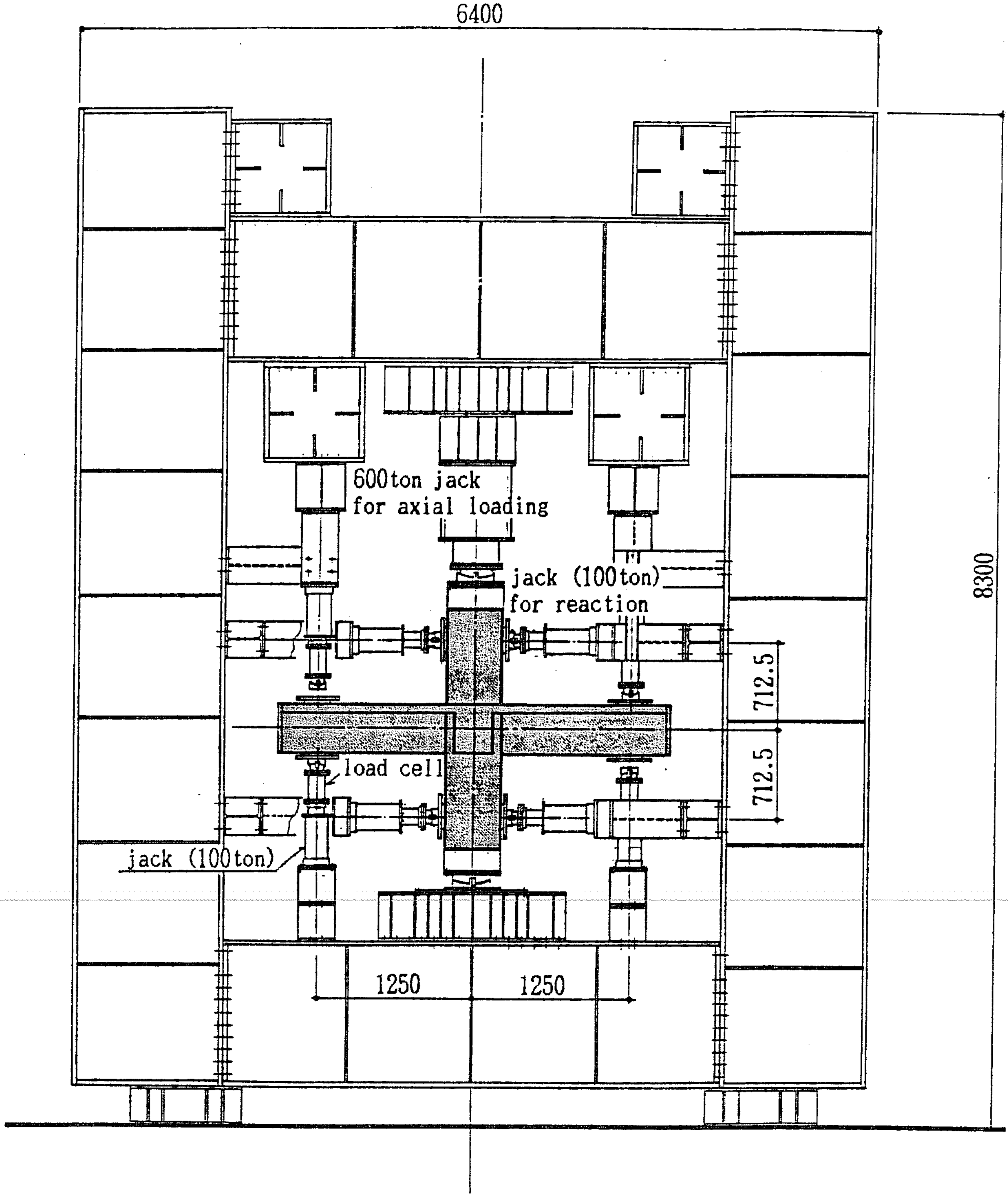
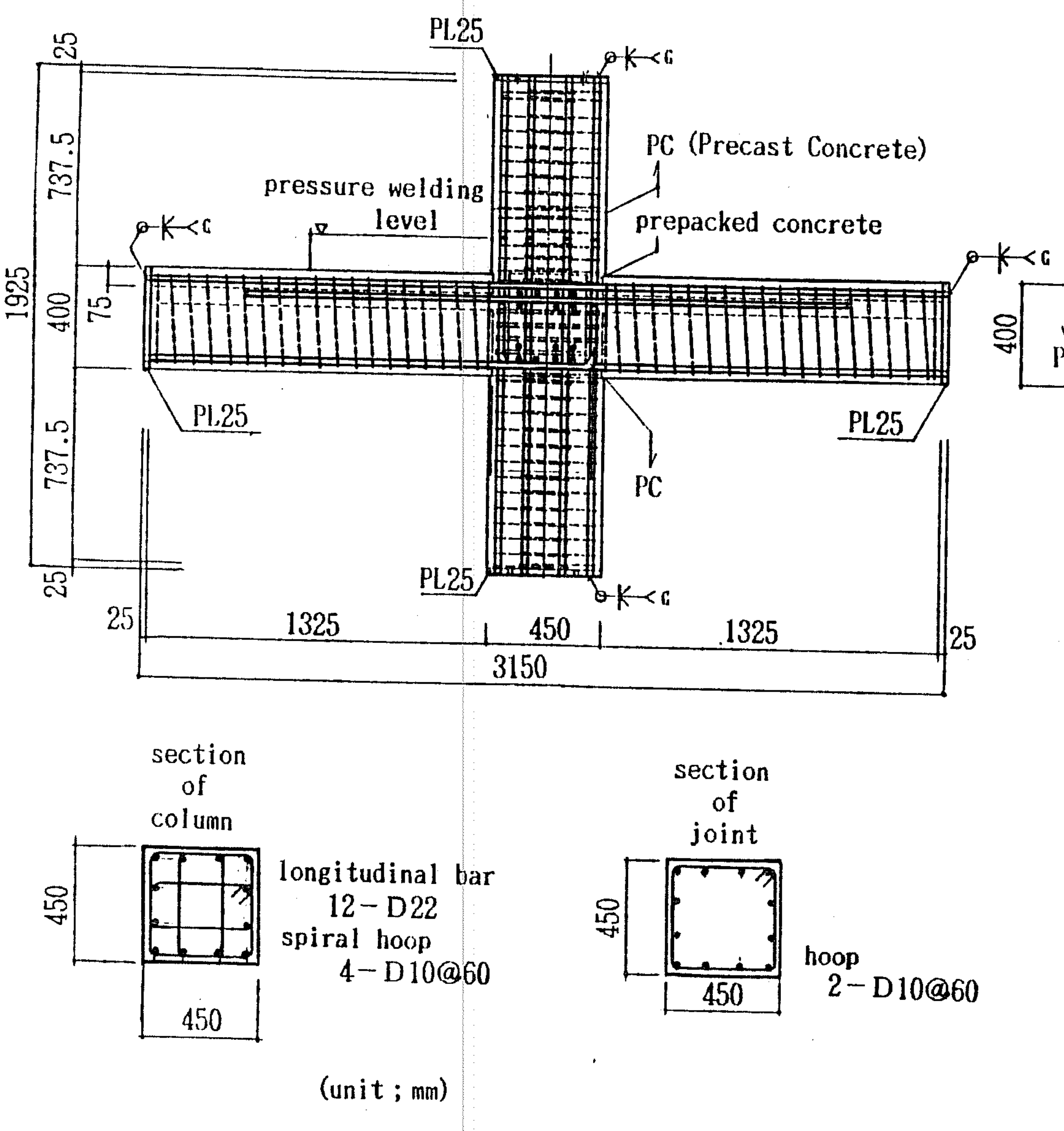


Fig.6.8 Loading Setup for Beam-Column Subassemblage Tests



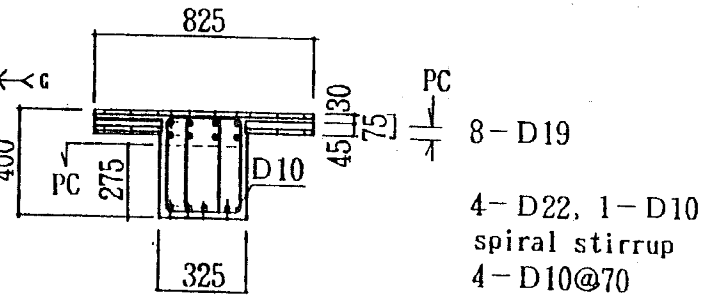


Fig.6.9 Details of Interior-type Subassemblage (CG-1)

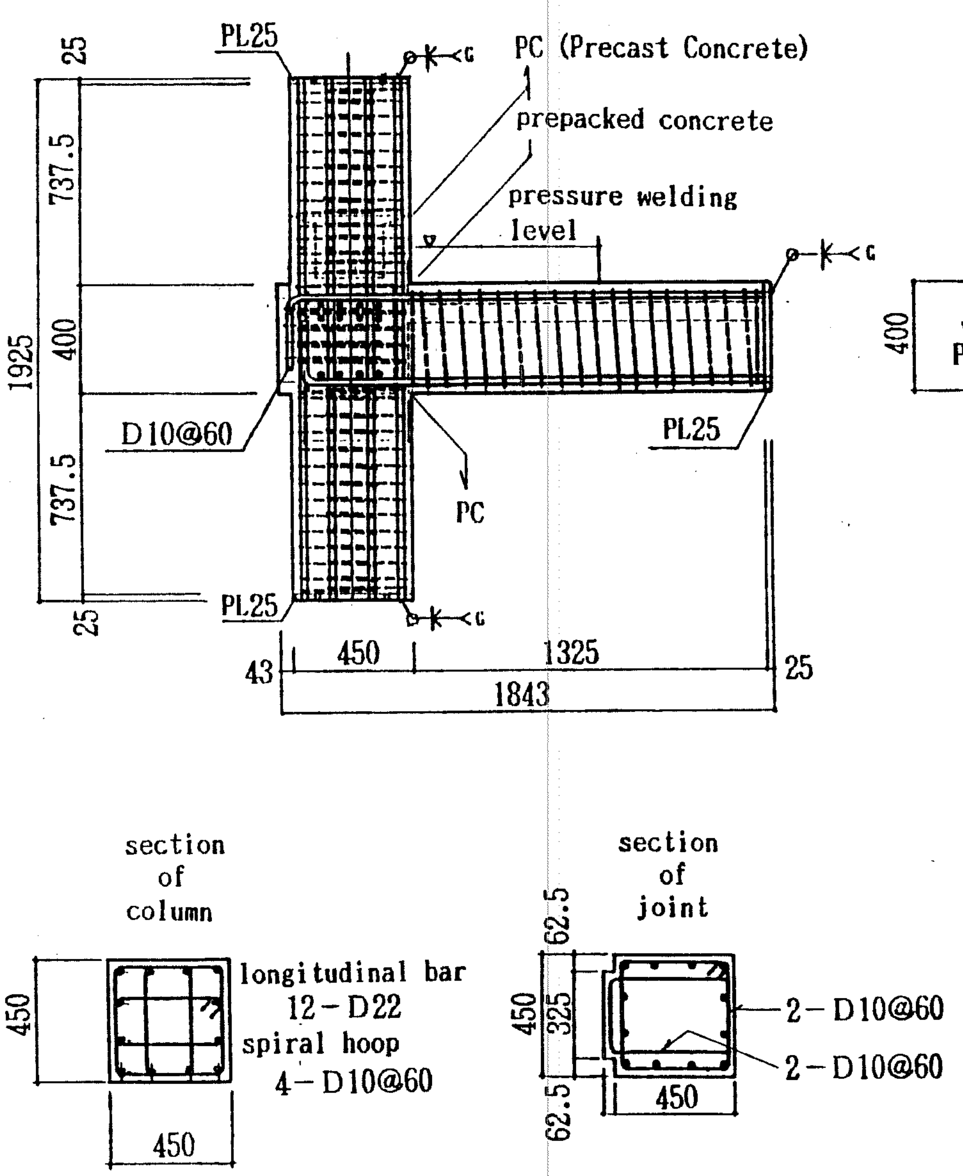
Material Properties

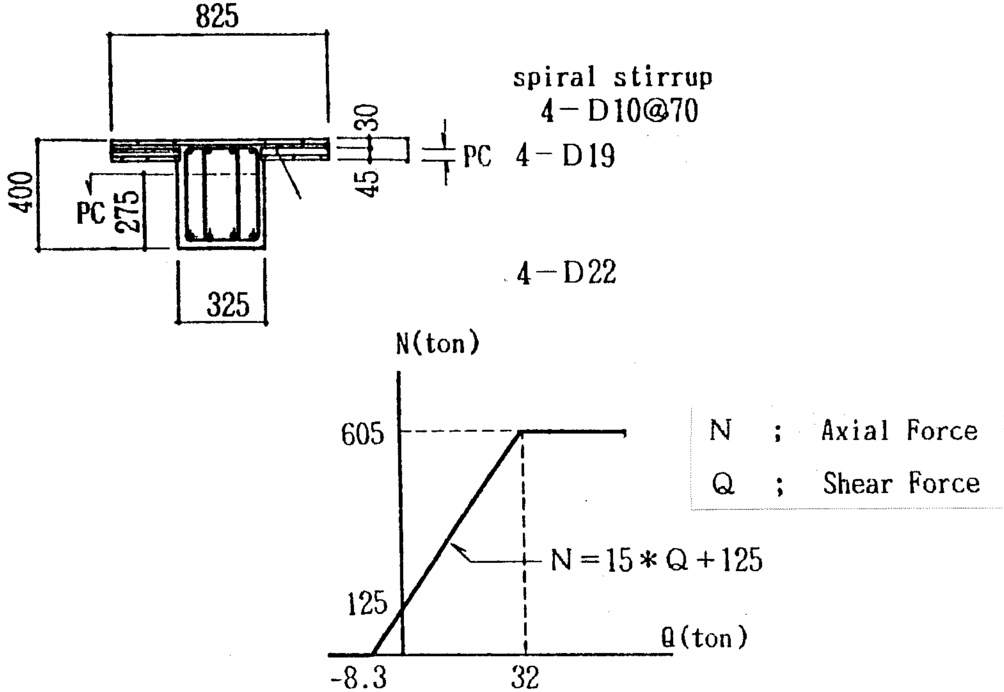
a) steel

	σy kg/cm²	εy ×10⁻⁶	σu kg/cm²	sE ton/cm²
D22	4113	2317	6563	1780
D19	4178	2377	6202	1760
D10(spiral)	3029	1751	5075	1730
D10 (hoop)	3949	2072	5378	1910
D 6 (slab)	3868	2254	5010	1720

b) concrete

	cσB kg/cm²	cσt kg/cm²	cεB ×10⁻⁶	cE1/3 ton/cm²	ν
precast	448	36.7	2436	266	0.161
cast-in-place	372	26.7	1900	280	0.166
prepacked	457	28.0	2703	213	0.178





Material Properties

a) steel (same as CG-1)

b) concrete

	$c\sigma_B$ kg／cm²	$c\sigma_t$ kg／cm²	$c\varepsilon_B$ $\times 10^{-6}$	$cE_{1/3}$ ton／cm²	ν
precast	487	30.6	2353	292	0.179
cast-in-place	329	27.4	1828	248	0.164
prepacked	552	36.7	3532	228	0.199

Fig.6.10 Details of Exterior-type Subassemblage (CG-4)

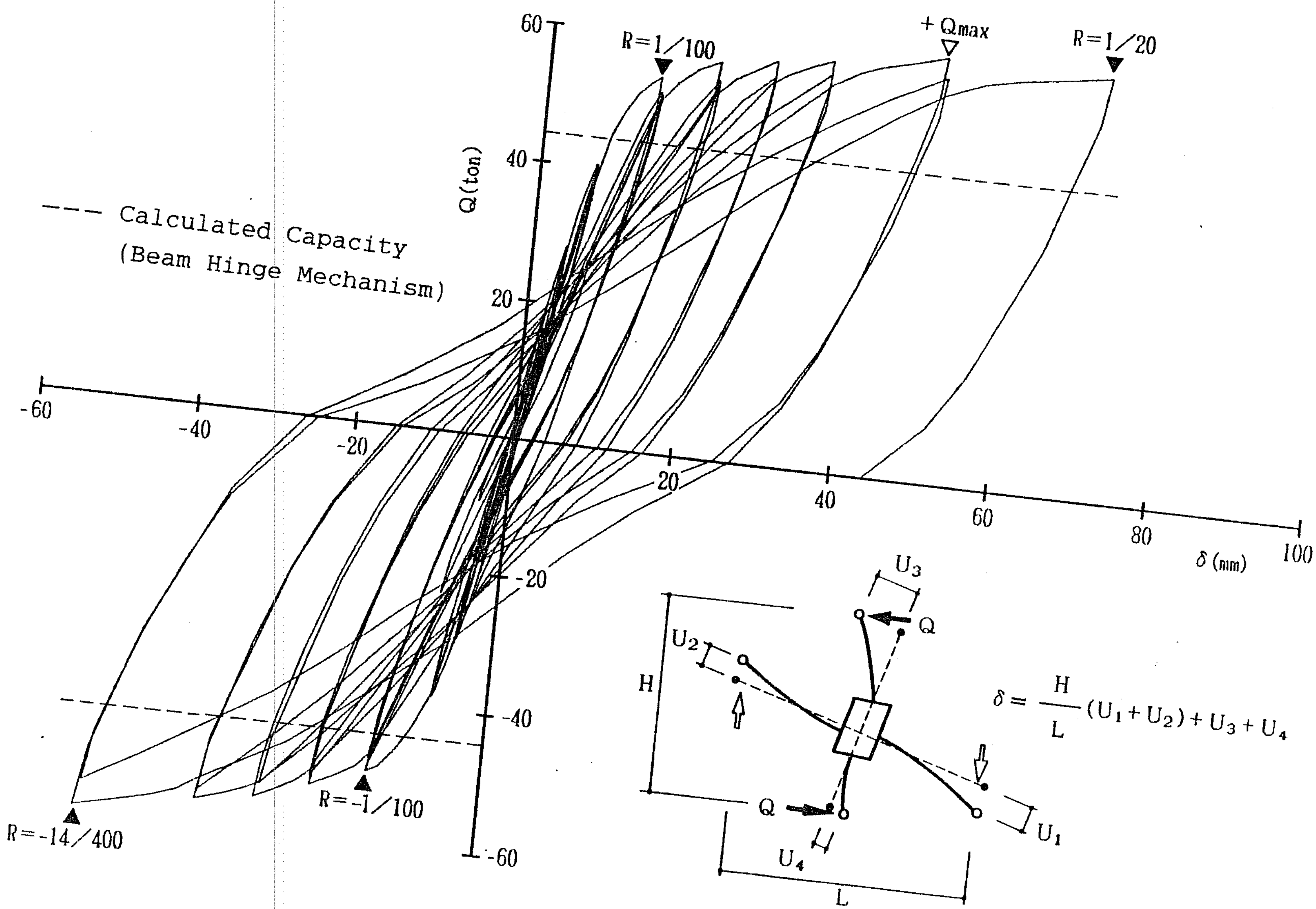


Fig.6.11 Story Shear vs. Drift Relationship for Specimen CG-1

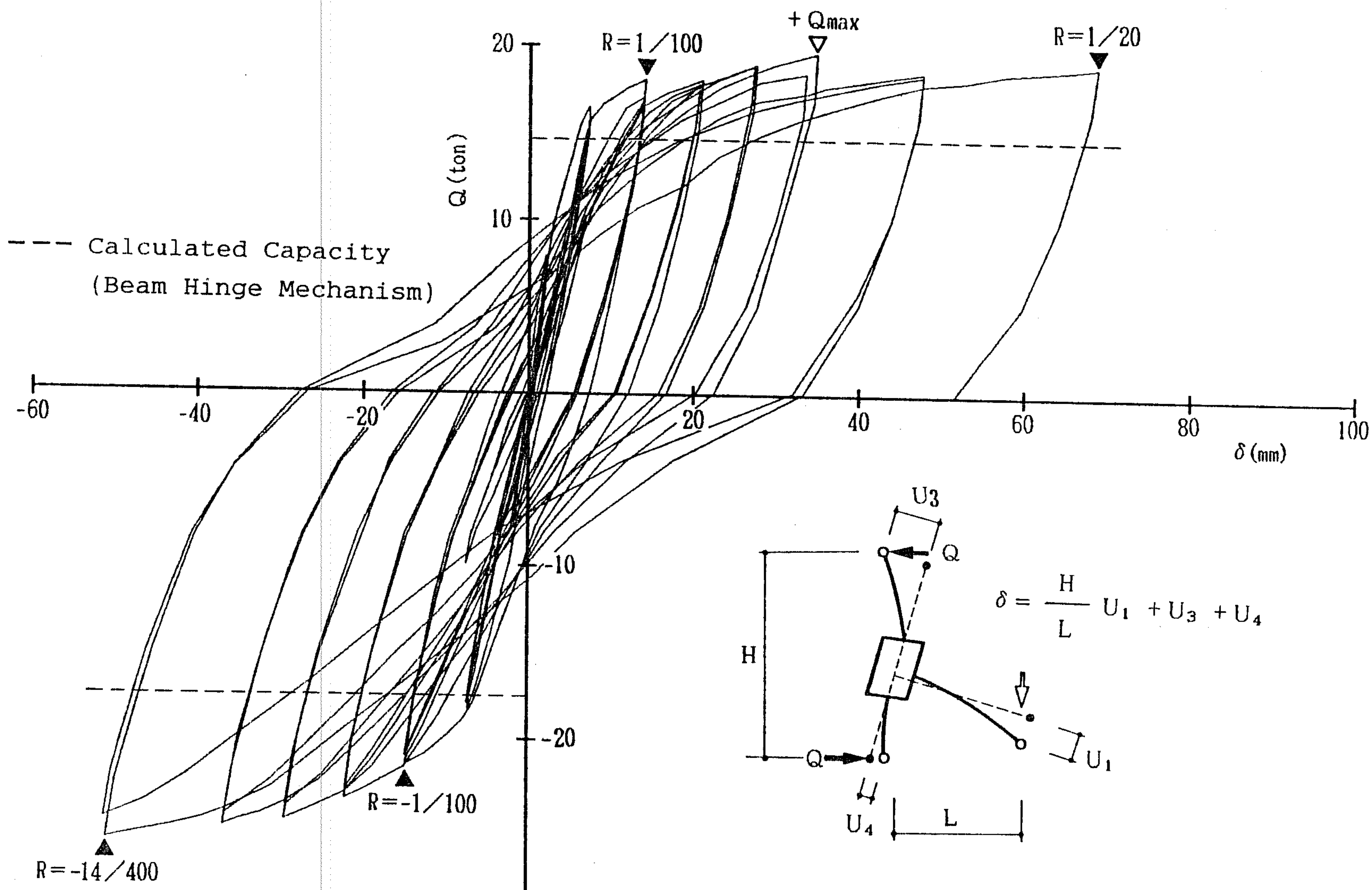


Fig.6.12 Story Shear vs. Drift Relationship for Specimen CG-4

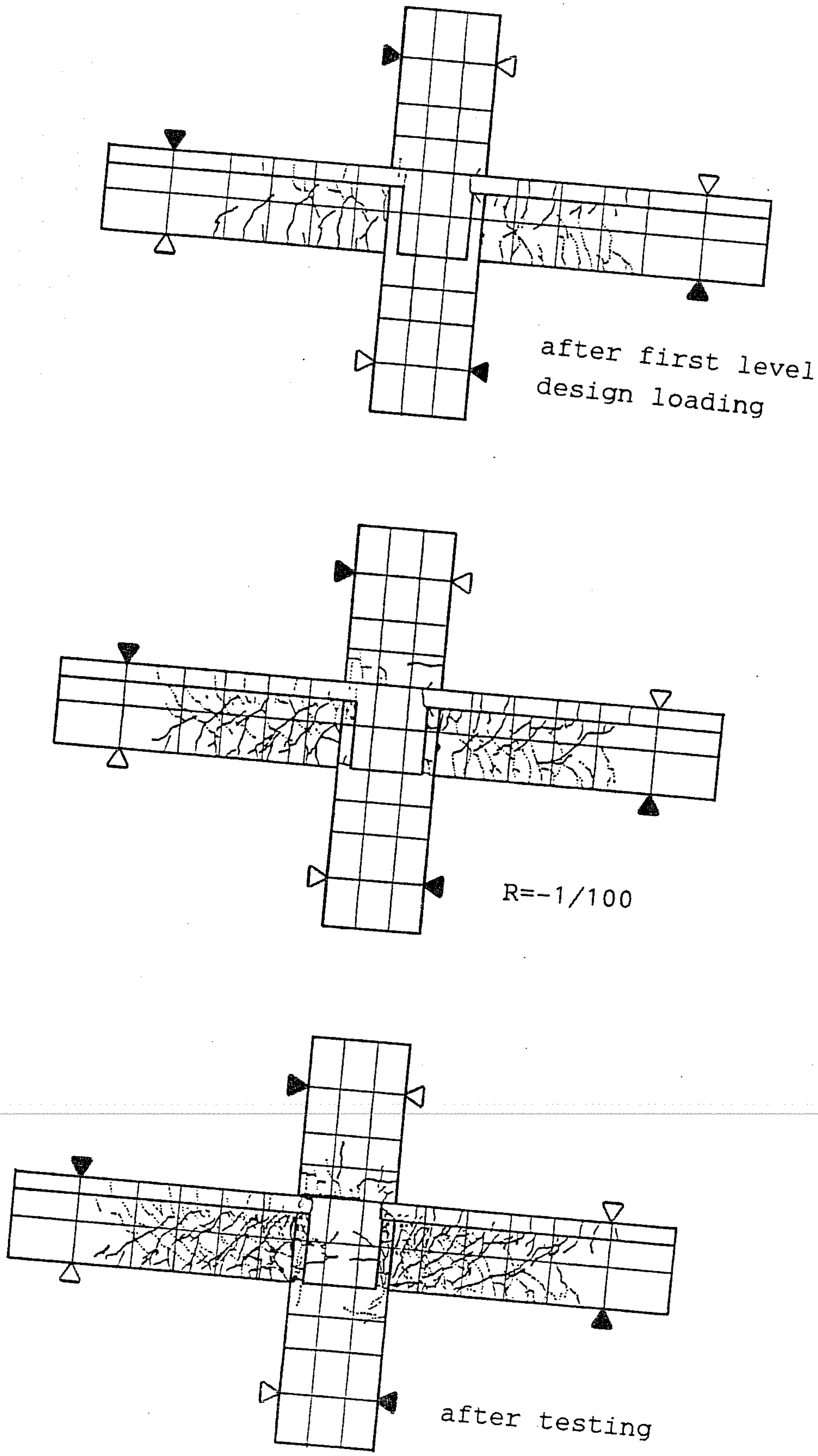
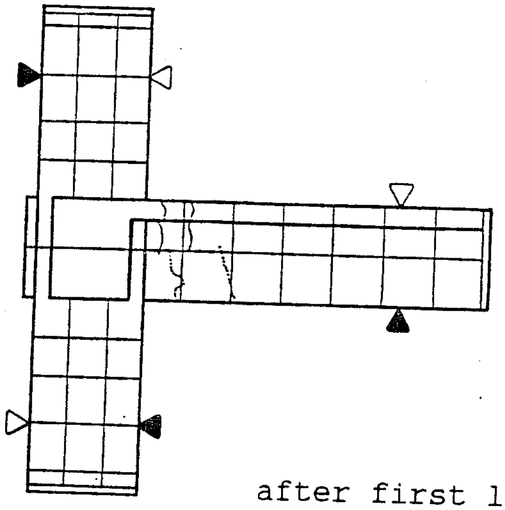
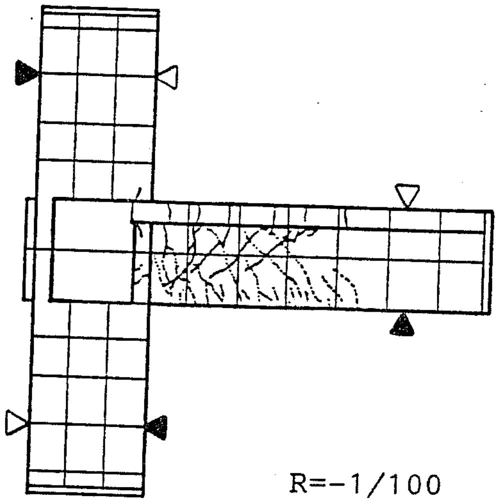


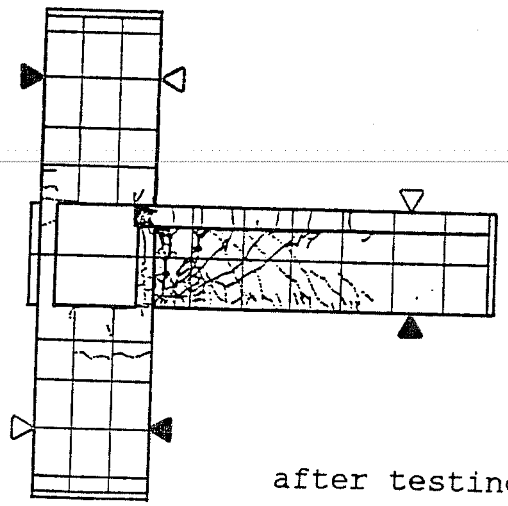
Fig.6.13 Crack Patterns for Specimen CG-1



after first level
design loading



R=-1/100



after testing

Fig.6.14 Crack Patterns for Specimen CG-4

2. STUDY CONCERNING THE DESIGN OF REINFORCED CONCRETE BEAM-COLUMN JOINTS

by

Minoru Fukushima

Satoshi Bessho

PART 1: The latest high-rise reinforced concrete building in Japan

PART 2: Design equation for joint shear force and its background

May 28, 1985

Kajima Corporation

PART 1: THE LATEST HIGH-RISE REINFORCED CONCRETE BUILDING IN JAPAN

1.1 Introduction

Kajima Corporation has constructed a number of high-rise reinforced concrete buildings since constructing the first 18-story reinforced concrete apartment house in Japan. The latest high-rise reinforced concrete building in Japan was designed and is being constructed by Kajima Corporation. The structural design of the Park City Shinkawasaki Building is outlined below.

1.2 Park City Shinkawasaki Building

Table 1.1 contains an outline of the latest high-rise reinforced concrete building in Japan. The building is a 30-story apartment house near Tokyo area and is called Park City Shinkawasaki Building. A perspective view of the building is shown in Fig. 1.1. Figure 1.2 shows the typical floor plan of the building. The floor plan is symmetric with respect to its center. The bay length is 4.8 to 5.3 m in both directions. Figure 1.3 shows the framing elevations. As shown in this figure, normal weight concrete with compressive strength of 240 to 420 kgf/cm^2 was used to cast the building.

1.3 Earthquake Resistant Design

Table 1.2 summarizes the structural design concept with emphasis on earthquake resistance. The structural design consists of three steps. The first two steps are based on static loads with different levels of base shear. The last step is a dynamic response analysis. The design criteria in the response analysis are also tabulated in this table.

Figures 1.4 through 1.6 show aseismatic details of structural members. Columns are laterally reinforced with spiral hoops and tie hoops, as shown in Fig. 1.4. Beam ends are heavily reinforced with stirrups to confine concrete in expected hinge zones, as shown in Fig. 1.5. Beam bars at exterior ends are anchored by bending in a U-shape, as shown in Fig. 1.6. Figure 1.7 shows sectional properties of beams and columns in the building. Exterior columns have an additional set of longitudinal bars placed in the center of the section, in order to carry large axial forces due to overturning of the building.

Figure 1.8 shows the distribution of design story shear forces. The ultimate lateral load-carrying capacity is also shown in this figure.

Figures 1.9 and 1.10 show results of the dynamic response analysis. The maximum story shear was less than the ultimate lateral load-carrying capacity, as shown in Fig. 1.9. The maximum story drift angle was 1/130 and less than the allowable value of 1/100.

1.4 HiRC Construction System

Kajima Corporation has developed the advanced construction system for high-rise reinforced concrete buildings (HiRC System) and achieved good results in construction of 18-story and 25-story buildings in Japan. At the time of writing, a 25-story building and 30-story Park City Shinkawasaki Building are under construction by Kajima Corporation using the HiRC System.

In the HiRC System, reinforcing bar cages are prefabricated and connected to each other with mechanical bar splices (a sleeve squeezed over the bars, see Appendix A) at the site. Large-scale shored forms are used for beam-slab floor systems in order to simplify the site work. High strength concrete is used to cast all structural members at the site.

Table 1.1 Outline of The Latest High-rise Reinforced Concrete Building in Japan

Name	: Park City Shinkawasaki Building
Place	: KASHIMADA, Kawasaki-Shi, Kanagawa Prefecture, JAPAN
Term of works	: Jan. 16,1985～Mar. 31,1987
Number of houses	: 230 houses
Story	: 30 stories
Eaves height	: 87.15 m (Max. height 97.65 m)
Total floor area	: 24400 m^2
Area of typical floor	: 948.5 m^2

Table 1.2 Structural Design Concept

1st Step

First phase design for earthquake

Base shear coefficient $C_B=0.12$

Vertical distribution factor Ai

$$Ai = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{2T}{1+3T}$$

α_i: non dimensional weight (or height)

All columns and beams should be under allowable design capacity

↓

2nd Step

Second phase design for earthquake

Base shear coefficient $C_B=0.18$

Vertical distribution factor Ai

The ultimate lateral load carrying capacity under flexural yielding of beams is not less than the story shear force for the second phase design

(Ultimate shear capacity of beams) ≧ (Ultimate shear force calculated by the flexural yielding of beams) X 1.1

(Ultimate shear capacity of columns) ≧ (Ultimate shear force calculated by the flexural yielding of beams) X 1.25

Structural provisions for arrangement of bar to assure the ductile frame

↓

3rd Step

Earthquake response analysis

Check aseismatic capacity

	major earthquake	worst earthquake
Input maximum acceleration	250gal	400gal
Maximum response shear force	not more than allowable capacity	not more than ultimate capacity
Maximum response story drift	$R \leqq 1/200$	$R \leqq 1/100$

R : Story deformation angle

Fig.1.1 Perspective of Park City Shinkawasaki Building

Fig.3.1 RC 30-story Apartment House

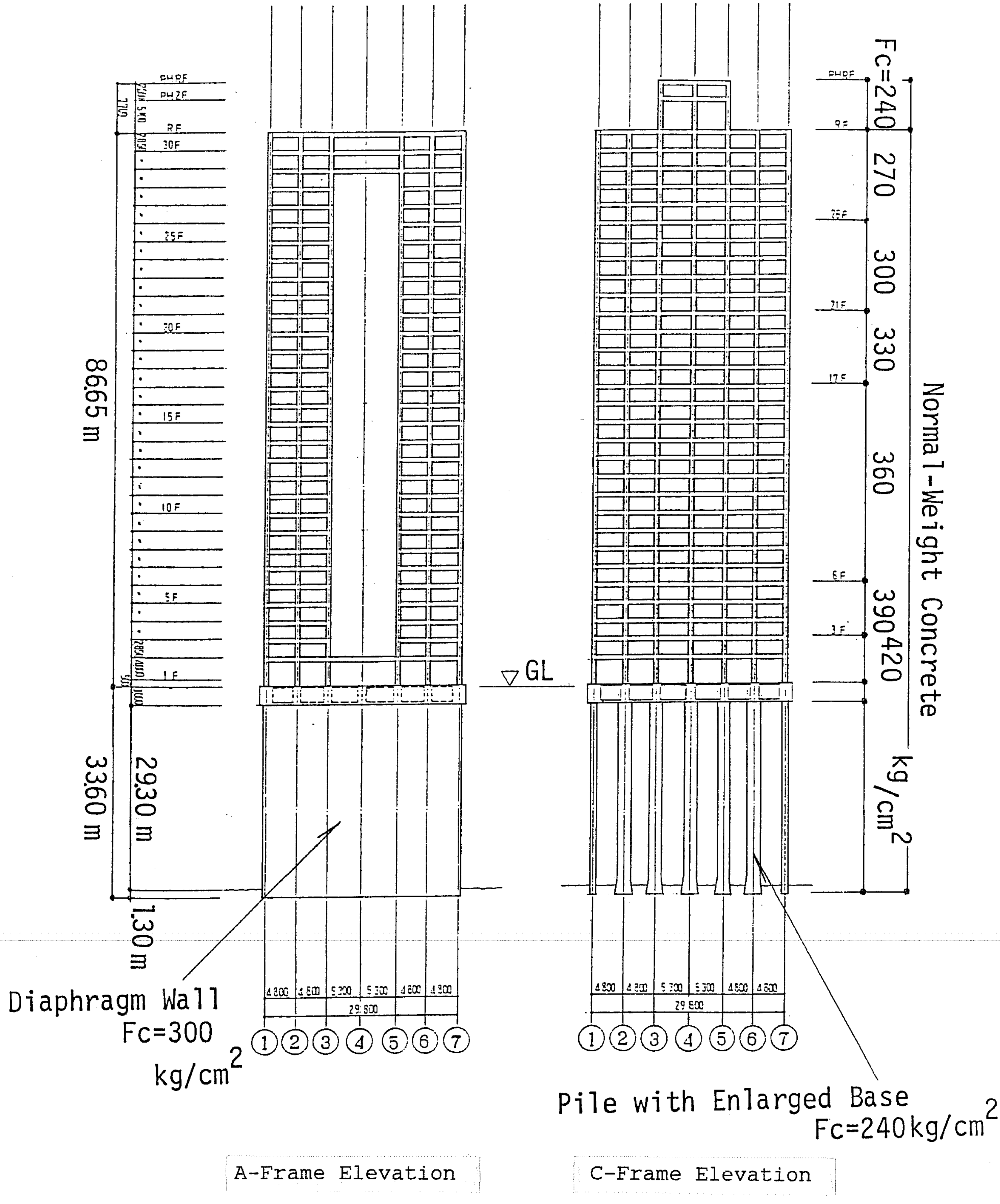


Fig.1.3 Framing Elevations

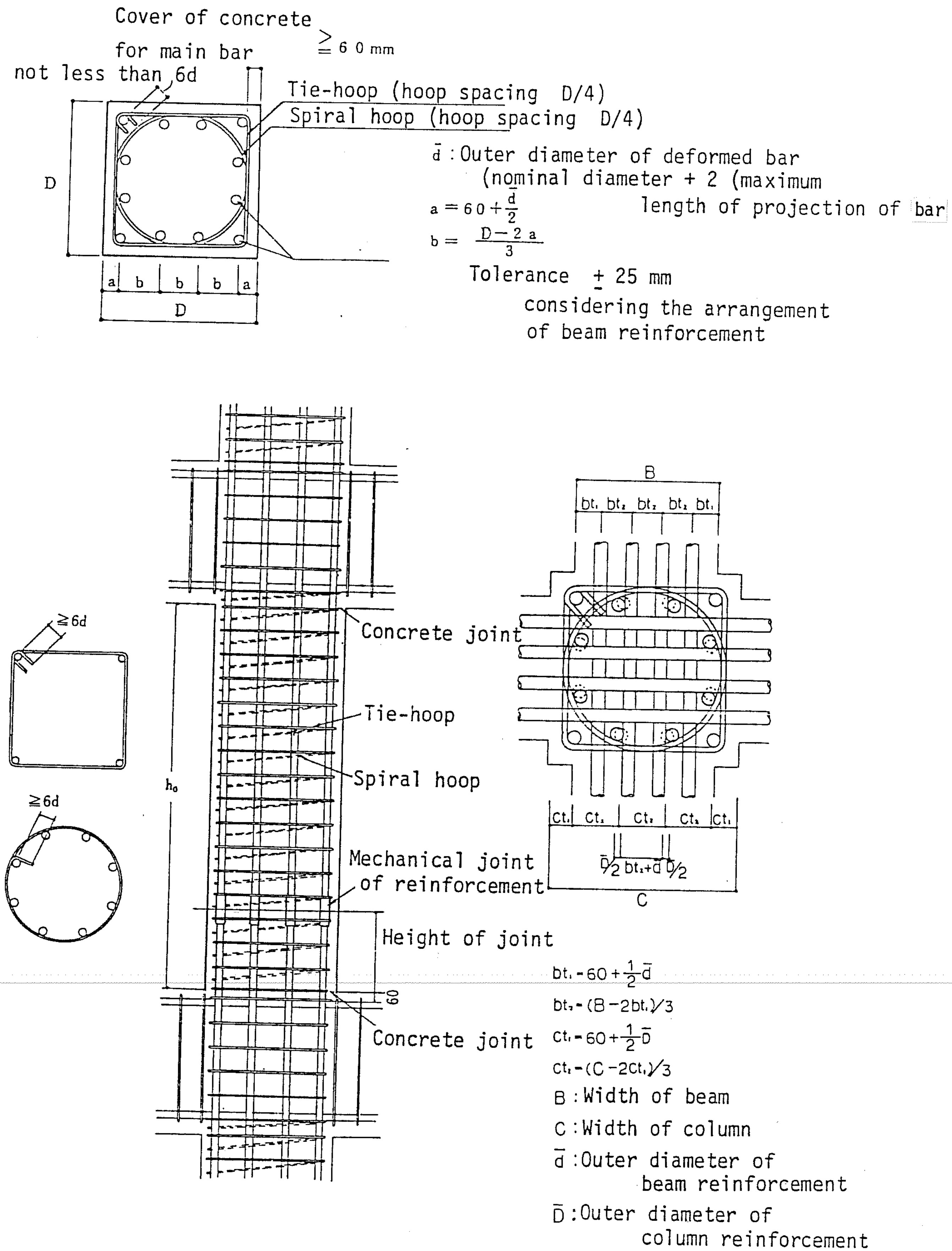


Fig.1.4 Aseismatic Details of Columns

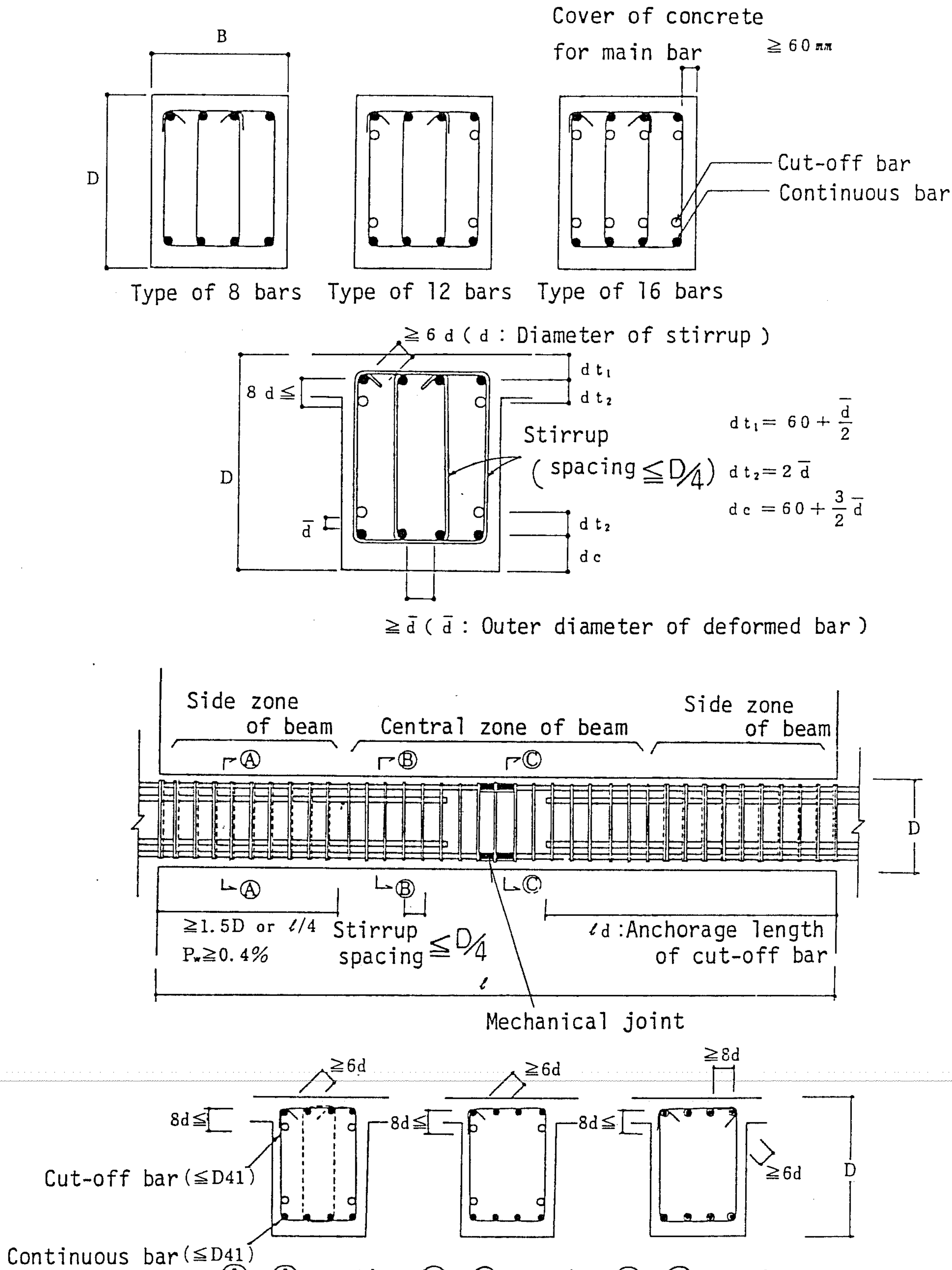
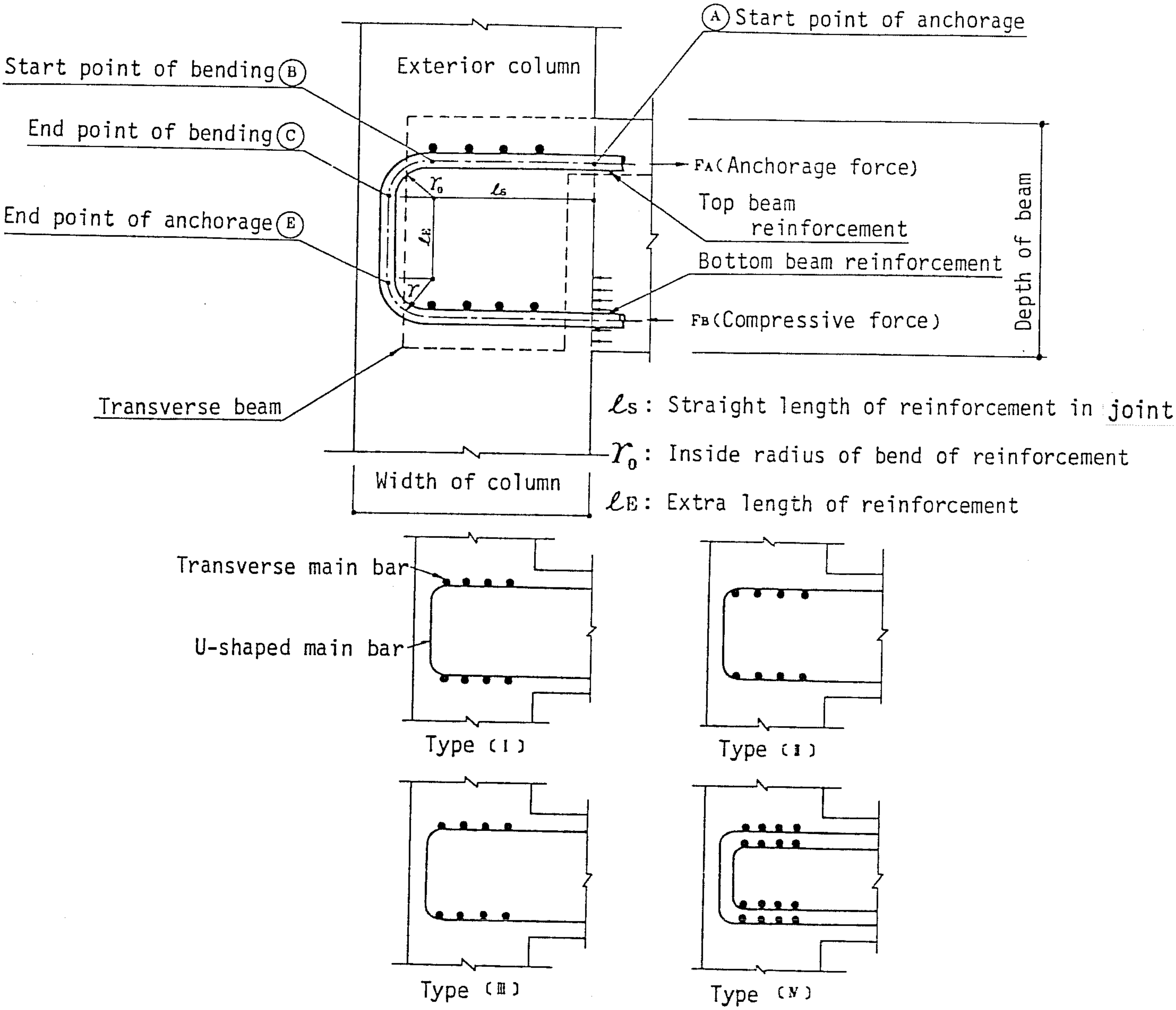


Fig.1.5 Aseismatic Details of Beams

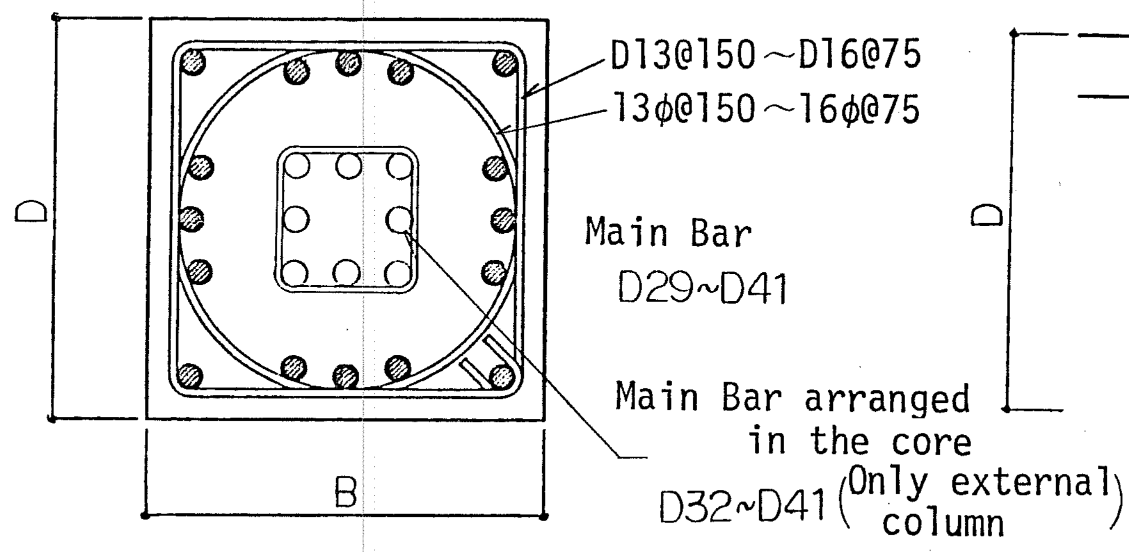


Minimum length of reinforcement

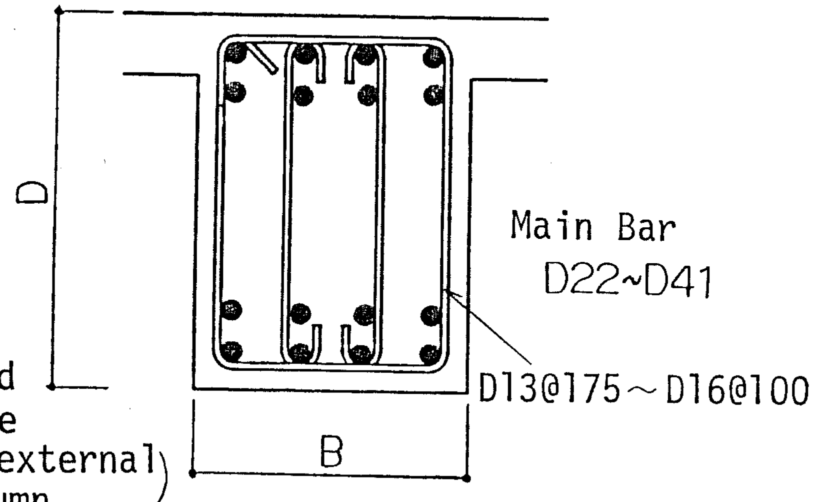
Type of anchorage method		minimum length of ℓ_s (mm)	minimum length of r_0 (mm)	minimum length of ℓ_E (mm)
Type [I~III]	D41,D38	12d	3d	4d
	not more than D35	10d		
Type [IV]	the first layer bar	15d	3d	6d
	the second layer bar	14d	1.5d	4d

d : the value of nominal diameter

Fig.1.6 Aseismatic Details of Exterior Beam-Column Joints



COLUMN



BEAM

(mm)

Story	B	D
21～30	750	750
	800	800
2～20	800	800
	850	850
1	850	850

Type of reinforcing bar

D22～D32	SD35
D35～D41	SD40
13φ,16φ	SR30
D13,D16	SD30

(mm)

Story	B	D
16～R	500	750
	550	600
	600	
3～15	550	800
	600	600
	650	
2	550	1,000
	600	
	650	

Fig.1.7 Design Section of Beam and Column

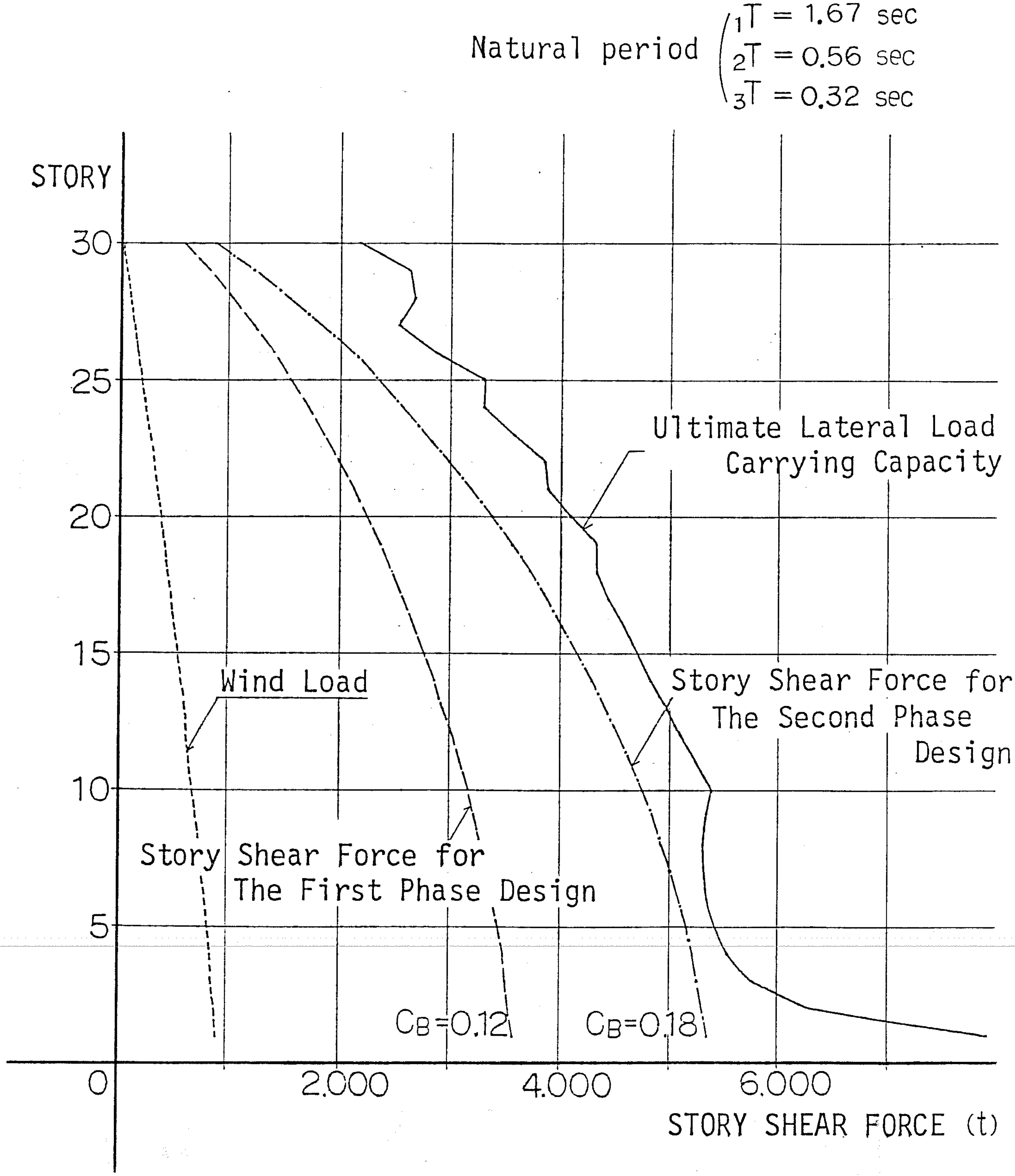


Fig.1.8 Design Shear Force and Ultimate Lateral Load Carrying Capacity

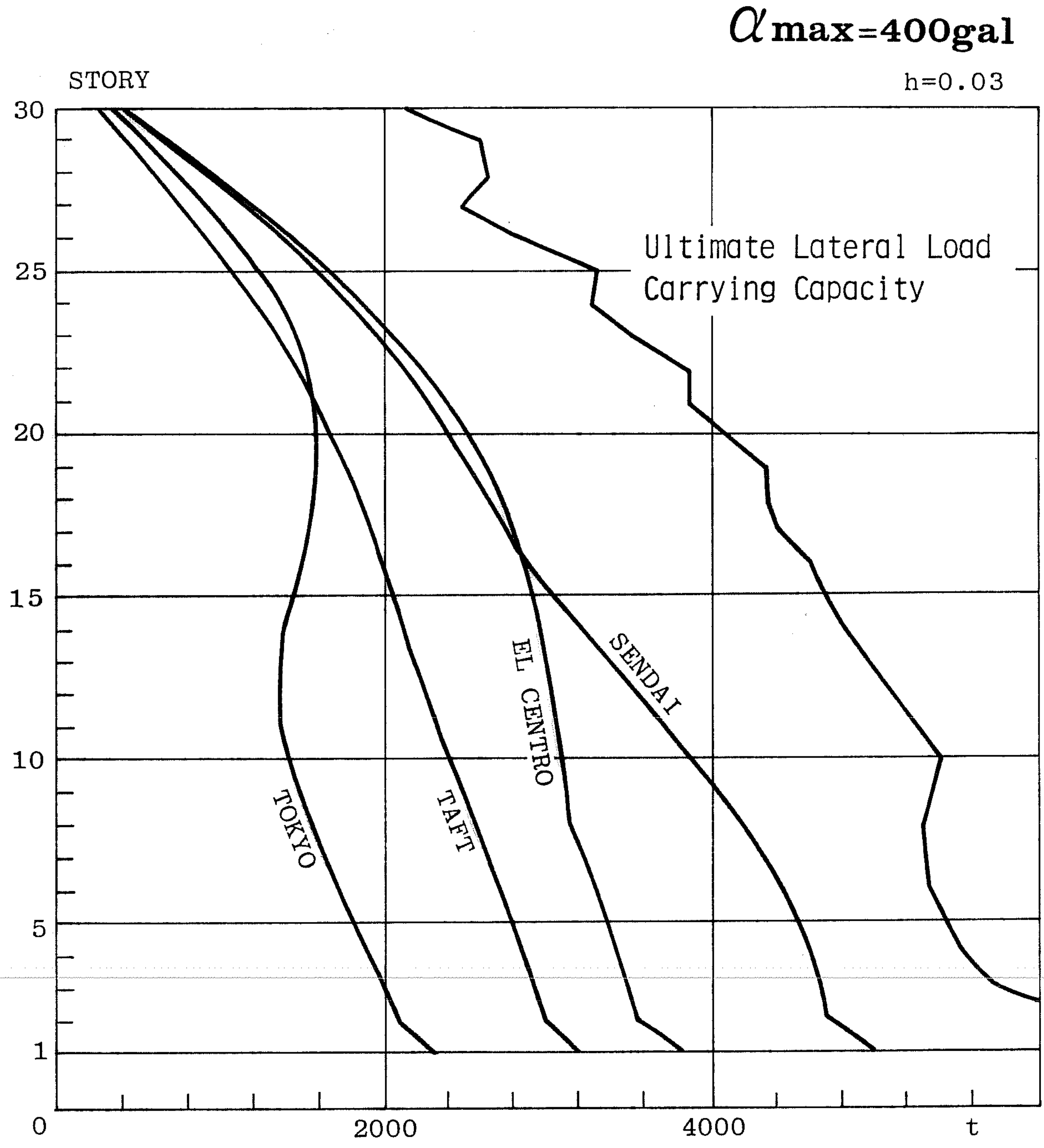


Fig.1.9 Maximum Response Story Shear Force

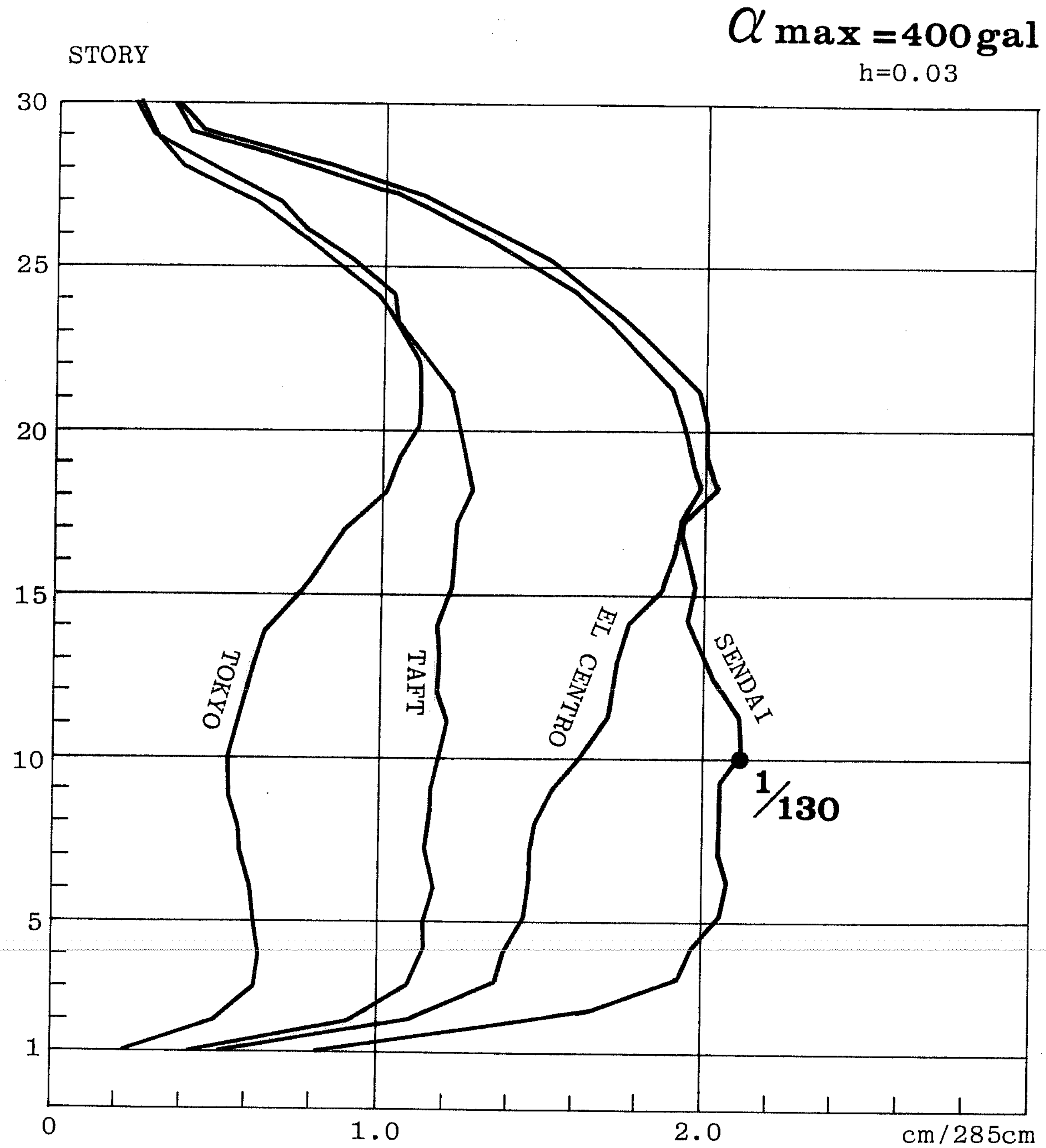


Fig.1.10 Maximum Response Story Drift

PART 2: DESIGN EQUATION FOR JOINT SHEAR FORCE AND ITS BACKGROUND

2.1 Design Equation for Joint Shear Force

Table 2.1 summarizes the structural design for joint shear forces. The design shear stress $_p\tau_D$ should be calculated from the beam hinge mechanism. The joint shear strength $_p\tau_u$ may be the sum of concrete contribution $_c\tau_u$ and steel contribution $_s\tau_u$. The equation for $_p\tau_u$ was proposed by the authors. Figure 2.1 shows the maximum values of joint shear stress $_p\tau_D$ in the building. The joint shear strength $_p\tau_u$ based on the authors' equation is also shown in parentheses.

2.2 Seismic Tests on Beam-Column Joints

Seismic tests on reinforced concrete beam-column joints were conducted at Kajima Institute of Construction Technology to verify the structural design method. The results of the tests are briefly described below.

2.2.1 Tests on Interior Subassemblages. Three specimens of reinforced concrete beam-column subassemblages were tested under cyclic loading. As shown in Fig. 2.2, the specimens were half-scale models for interior-type connections. Sectional properties of beams and columns were common among the specimens. Compressive strength of concrete $_c\sigma_B$ and yield strength of rebars $_s\sigma_y$ are also shown in this figure. Figure 2.3 shows details of the specimens. Main variation among the specimens is the number of transverse (confining) beams.

Figure 2.4 shows the test setup. Cyclic loading was applied to the beam tips and controlled by interstory drift angle. Constant axial load of 40 tons was applied to the column. Beam tip deflections and joint distortion were measured with respect to the reference frame linking column inflection points.

Figure 2.5 shows the crack patterns after completion of the tests. The beam-column joint in J-1 (without transverse beams) was more damaged than in J-2 and J-3 (with transverse beams). All specimens showed stable load-deflection relations, as shown in Fig. 2.6. Transverse beams might have confined the joint considerably, because J-2 and J-3 showed higher strength than J-1. However, there was no great difference in strength between J-2 (one transverse beam) and J-3 (two transverse beams).

Table 2.2 shows the test results in comparison with calculations for joint shear strength. The test results exceeded the calculated values based on the authors' equation. The authors' equation is plotted with other experimental data in Fig. 2.7. The equation seems appropriate as a design formula for joint shear strength.

2.2.2 Test on Exterior Subassemblage. An exterior-type specimen of a reinforced concrete beam-column subassemblage was also tested under cyclic loading. As shown in Fig. 2.8, the specimen was a full-scale model having a slab and transverse (side) beams. Beam bars were anchored in the joint by bending in a U-shape. Figure 2.9 shows the loading setup. Cyclic loading was applied to the specimen. Figure 2.10 shows the crack pattern for the specimen. The specimen showed flexural failure in the beam and no severe damage was observed in the joint. Figure 2.11 shows load-deflection curves. The specimen showed stable response to cyclic loading and the load was increasing even at 5% drift. Figure 2.12 shows strain distribution along beam bars. Tensile strains decreased along beam top bars within the joint. The U-shaped bend seemed to provide good anchorage.

Table 2.1 Design Equation for Joint Shear Force

${}_p\tau_u \geqq {}_p\tau_D$

${}_p\tau_u$: Ultimate Shearing Stress (kg/cm^2)

${}_p\tau_D$: Design Shearing Stress (kg/cm^2)

(1) $${}_p\tau_D = \frac{\sum {}_B M_U}{\left(1+\frac{D_B}{h_0}\right) {}_eV_c}$$

$\sum {}_B M_U$: Total Ultimate Bending Moment of Both End of Beam (kg·cm)

D_B : Depth of Beam (cm)

h_0 : Length of Clear height (cm)

${}_eV_c$: Effective Volume of Joint (cm^3)

$$= j_B \cdot j_C \cdot \frac{b_b + b_c}{2}$$

(2) ${}_p\tau_u = {}_c\tau_u + {}_s\tau_u$

$${}_c\tau_u = {}_p\alpha_s \cdot {}_p\beta_s \cdot {}_p\gamma_s 5.1\sqrt{Fc}$$

$${}_s\tau_u = 0.5\,{}_pP_w \cdot {}_s\sigma_w$$

${}_p\alpha_s$: The coefficient for type of concrete

Normal-Weight Concrete 1.0

Light-Weight Concrete 0.9

${}_p\beta_s$: The coefficient for frame

"┼" shaped frame 1.0

"├" shaped frame 2/3

"┌" shaped frame 1/3

${}_p\gamma_s$: The coefficient for transverse beam

no transverse beam 1.0

one or two transverse beams 1.4

Fc : Specified design strength of concrete (kg/cm^2)

${}_pP_w$: Ratio of shear reinforcing bar

${}_s\sigma_w$: Specified design strength of shear reinforcing bar (kg/cm^2)

Table 2.2 Observed and Calculated Shear Capacities of Joint

		J-1	J-2	J-3
Maximum Observed Stress	${}_{ob}\tau_{max}$ (kg/cm^2)	135.8	151.2	154.0
Our Proposed Equation 1)	${}_{1}\tau_{u}$ (kg/cm^2)	91.7	128.4	142.7
	${}_{ob}\tau_{max}/{}_{1}\tau_{u}$	1.48	1.18	1.08
Meinheit-Jirsa's Equation 2)	${}_{2}\tau_{u}$ (kg/cm^2)	98.9	166.8	186.6
	${}_{ob}\tau_{max}/{}_{2}\tau_{u}$	1.37	0.91	0.83

Failure mode; J-1 Shear failure of beam-column joint

J-2, J-3) Flexural failure of beam

1) $${}_{p}\tau_{u} = {}_{c}\tau_{u} + {}_{s}\tau_{u}$$

$${}_{c}\tau_{u} = {}_{p}\alpha_{s} \cdot {}_{p}\beta_{s} \cdot {}_{p}\gamma_{s} 5.1\sqrt{Fc}$$

$${}_{s}\tau_{u} = 0.5 {}_{p}P_{w} \cdot {}_{s}\sigma_{w}$$

	J-1	J-2	J-3
${}_{p}\alpha_{s}$	1.0		
${}_{p}\beta_{s}$	1.0		
${}_{p}\gamma_{s}$	1.0	1.4	

2) $${}_{p}\tau_{u} = 2.1\beta\gamma\zeta(Fc)^{2/3}$$

	J-1	J-2	J-3
β	1.0	1.69	
γ	1.0		
ζ	1.0		1.08

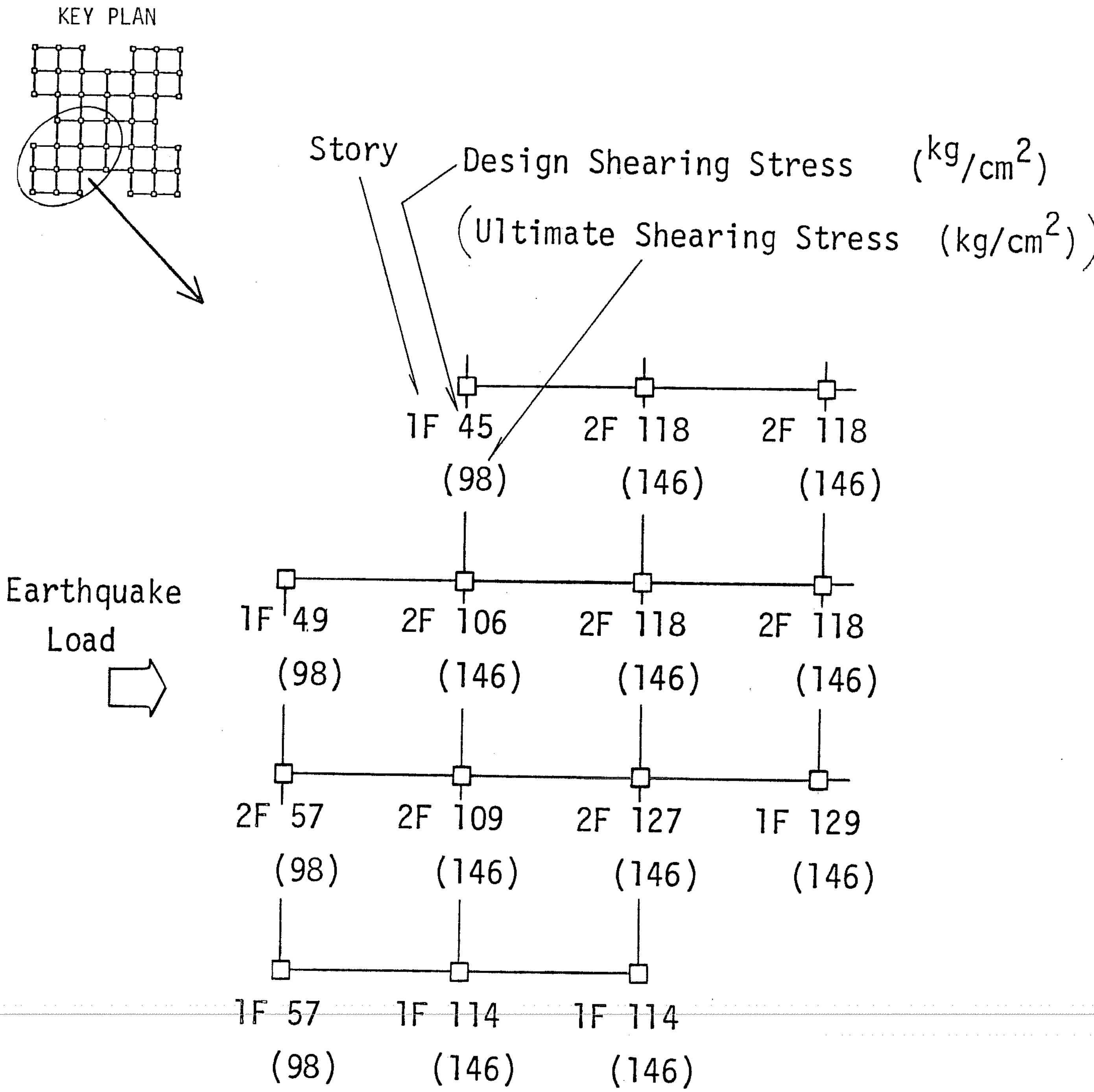


Fig.2.1 Maximum Ultimate Shear Stress of Panel Zone
(Park City Shinkawasaki Building)

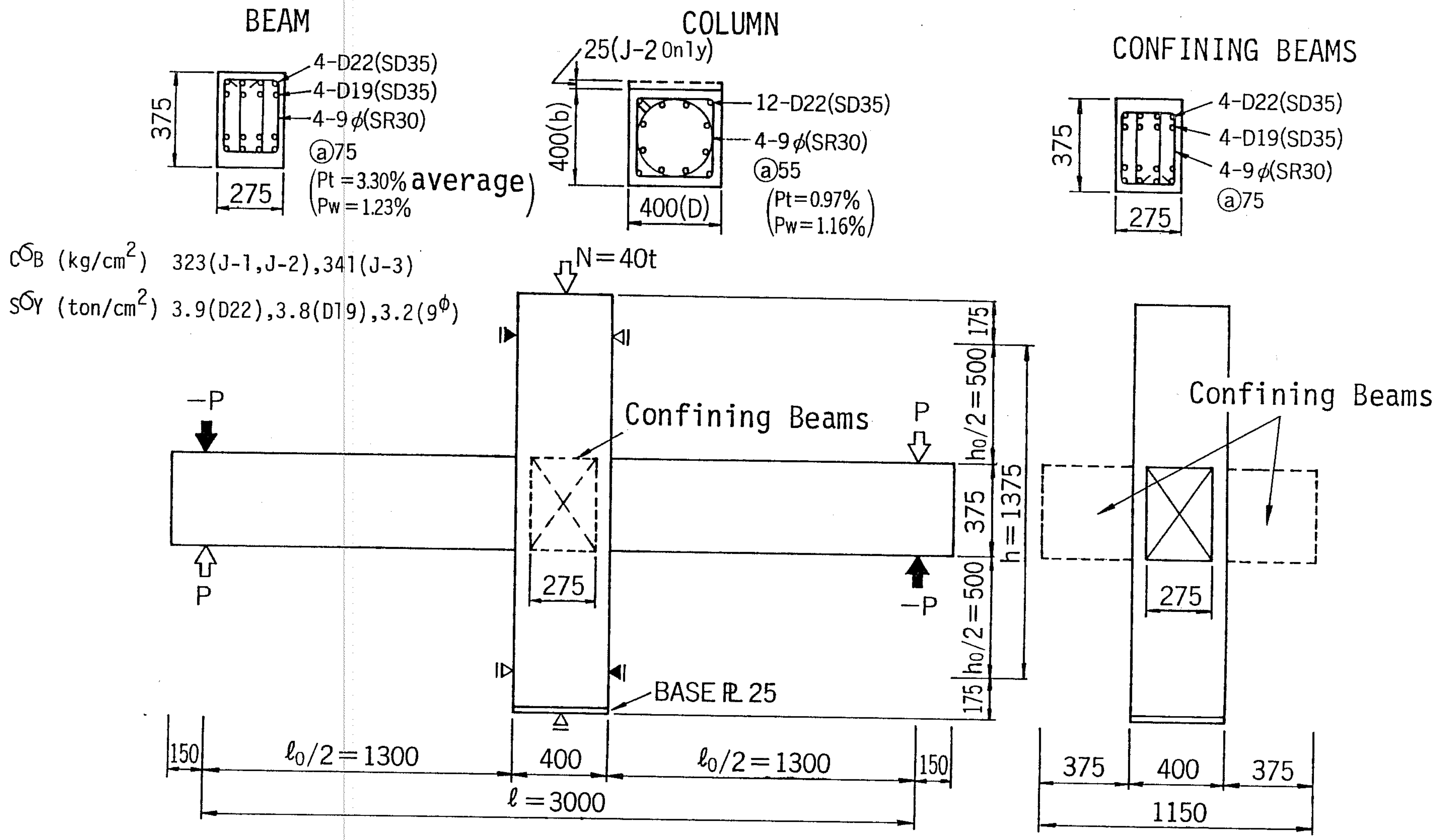


Fig.2.2 Outline of Beam-Column Joint Specimens

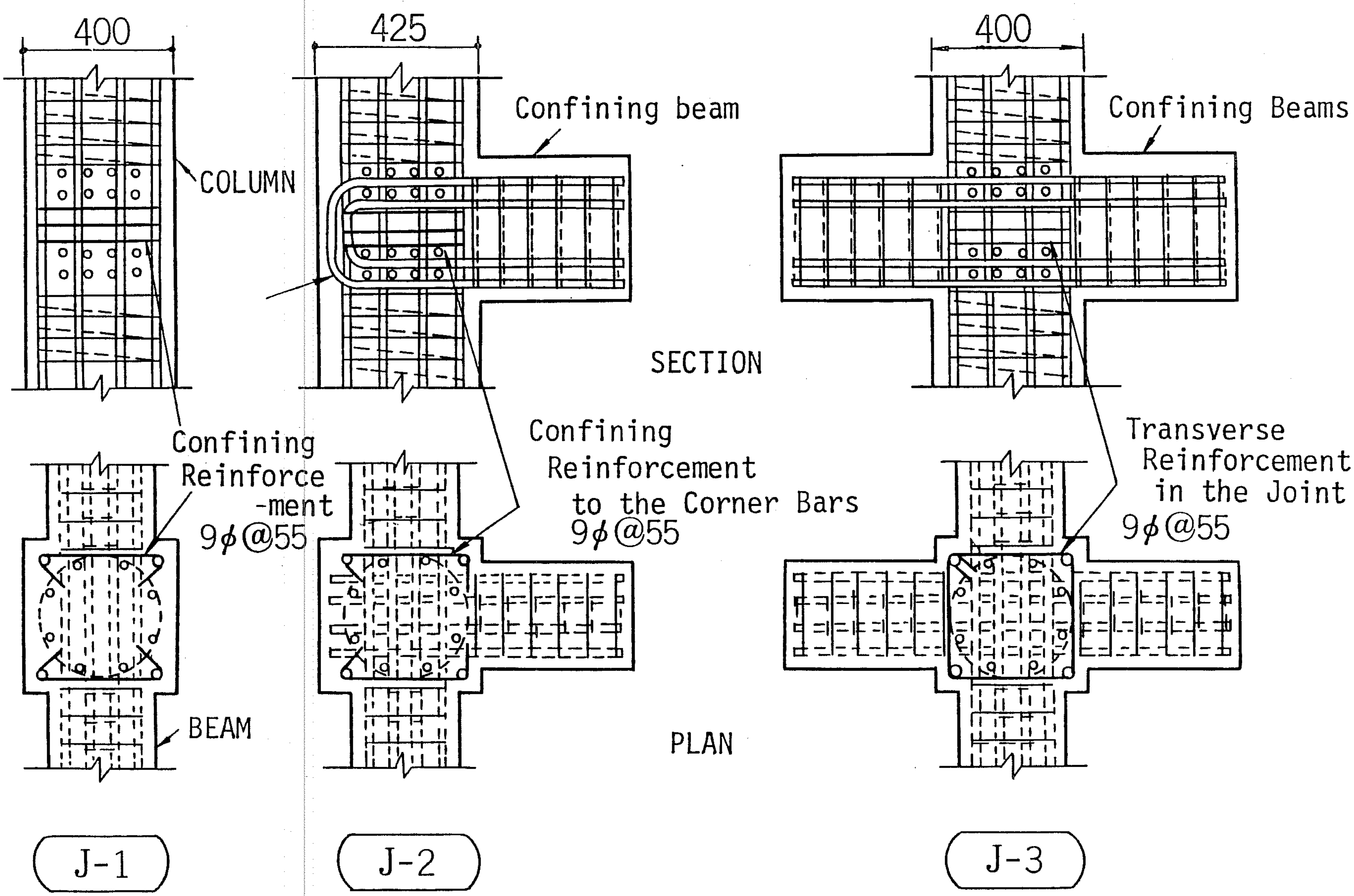


Fig.2.3 Details of Beam-Column Joints

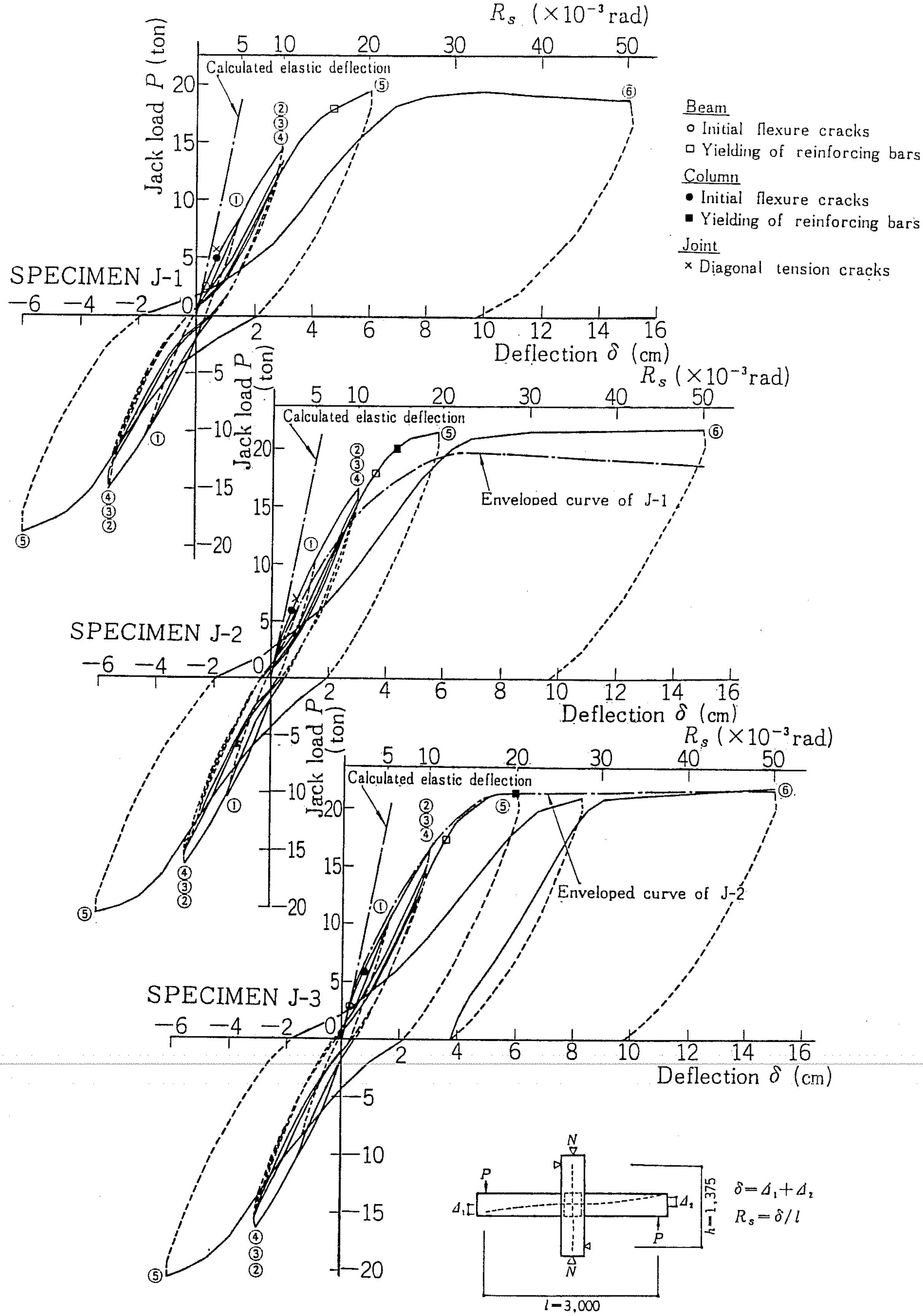
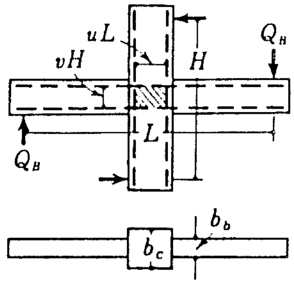


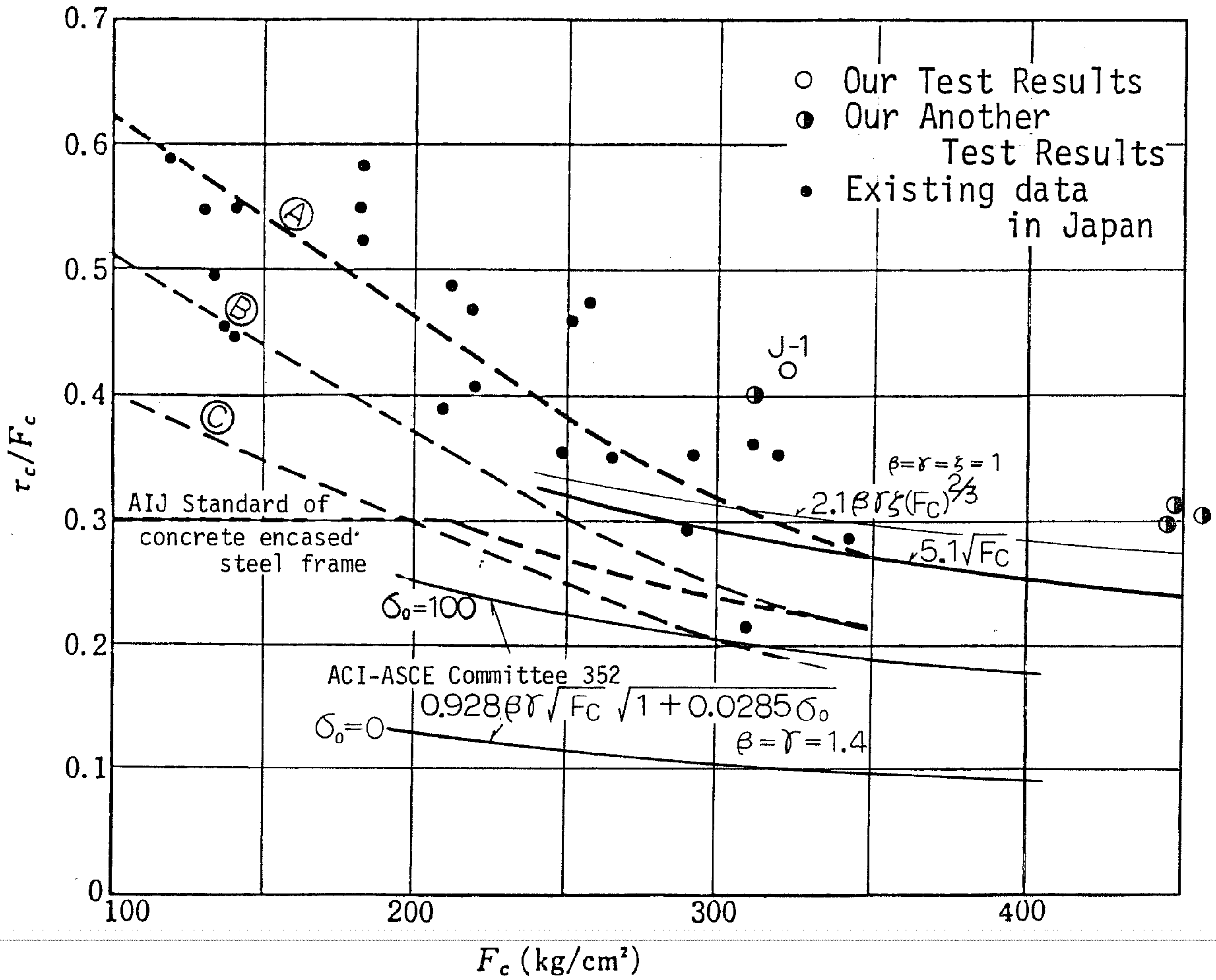
Fig.2.6 Load-Deflection Curves

$$\tau_p = \frac{(1-u-v)\,L \cdot Q_B}{{}_vH \cdot {}_uL \cdot t_p}, \qquad t_p = \frac{b_b + b_c}{2}$$

$$\tau_p = \tau_c + \tau_s$$

$$\tau_s = \frac{1}{2}\, p_w \cdot {}_w\sigma_y$$





Ⓐ KAMIMURA'S Equation $\begin{cases} \tau_c/F_c = 0.78 - 0.0016F_c \cdots\cdots F_c \leq 244 \\ \quad {}'' \quad = 95.1/F_c \cdots\cdots\cdots\cdots\cdots F_c > 244 \end{cases}$

Ⓑ ENDO'S Equation $\begin{cases} \tau_c/F_c = 0.65 - 0.0014F_c \cdots\cdots F_c \leq 232 \\ \quad {}'' \quad = 75.4/F_c \cdots\cdots\cdots\cdots\cdots F_c > 232 \end{cases}$

Ⓒ OHWADA'S Equation $\begin{cases} \tau_c/F_c = 0.50 - 0.0010F_c \cdots\cdots F_c \leq 250 \\ \quad {}'' \quad = 62.5/F_c \cdots\cdots\cdots\cdots\cdots F_c > 250 \end{cases}$

Fig.2.7 Relationship between Compressive Strength Fc and τc/Fc

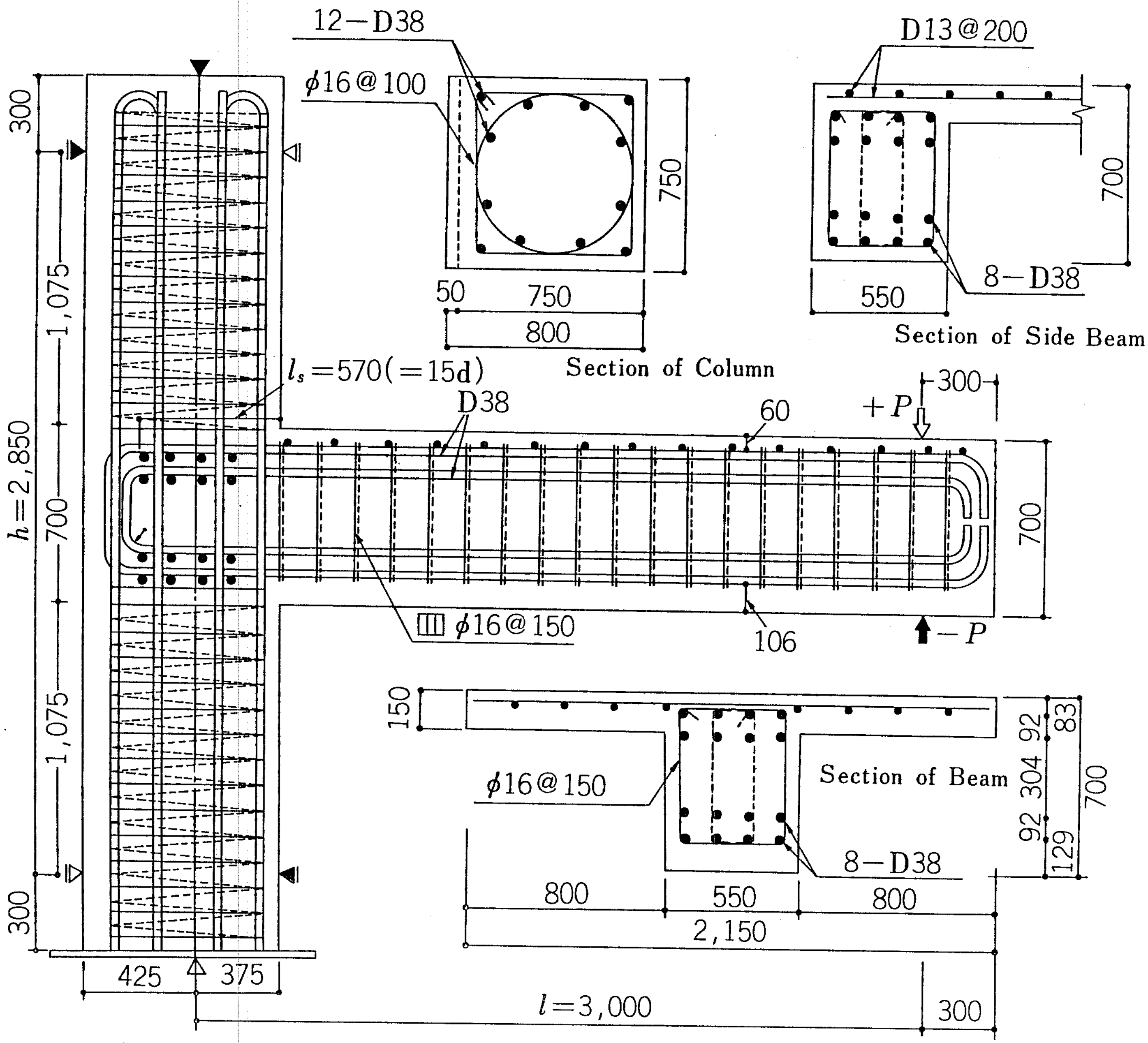


Fig.2.8 Details of Test Specimen

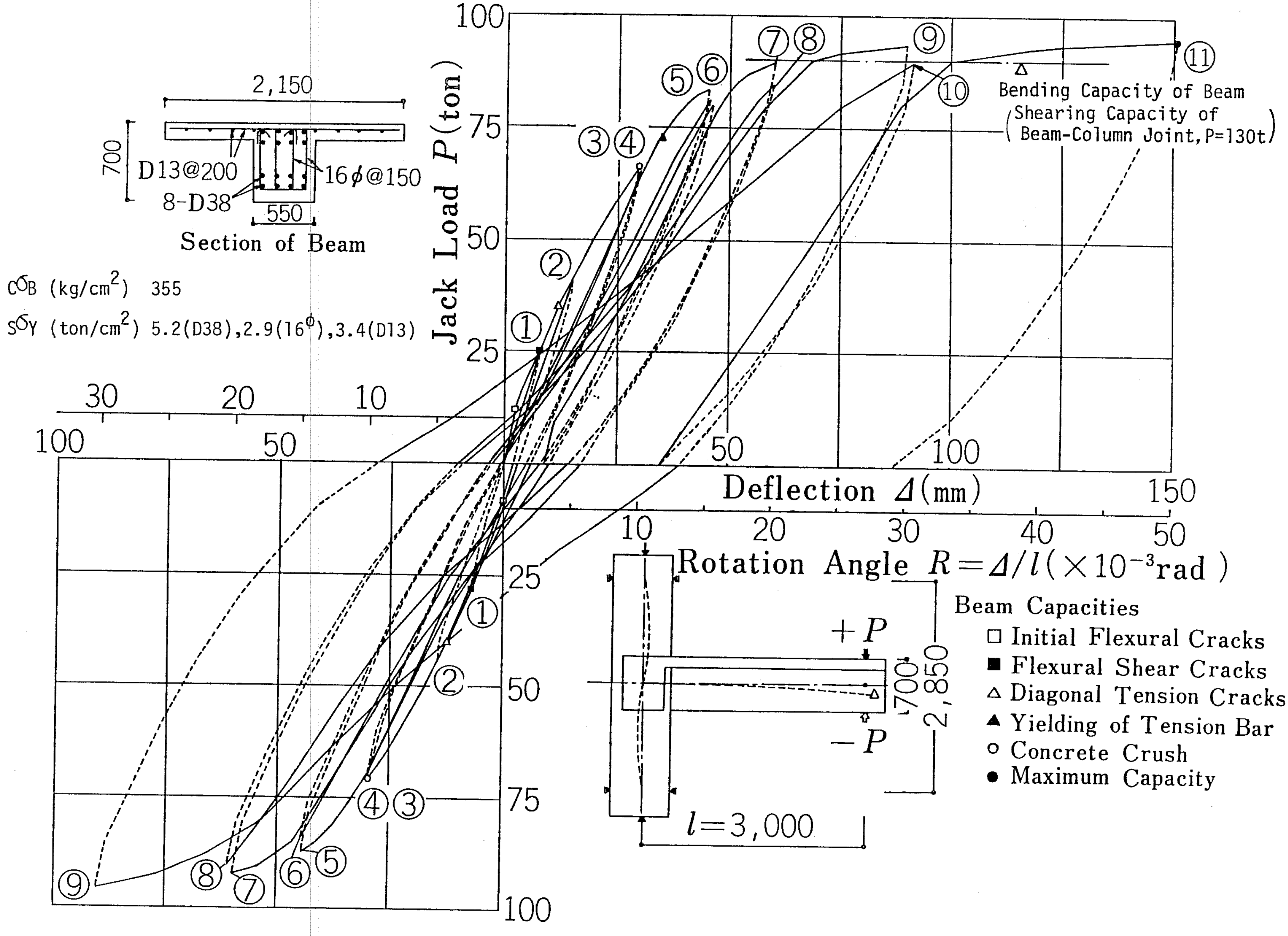


Fig.2.11 Load-Deflection Curve

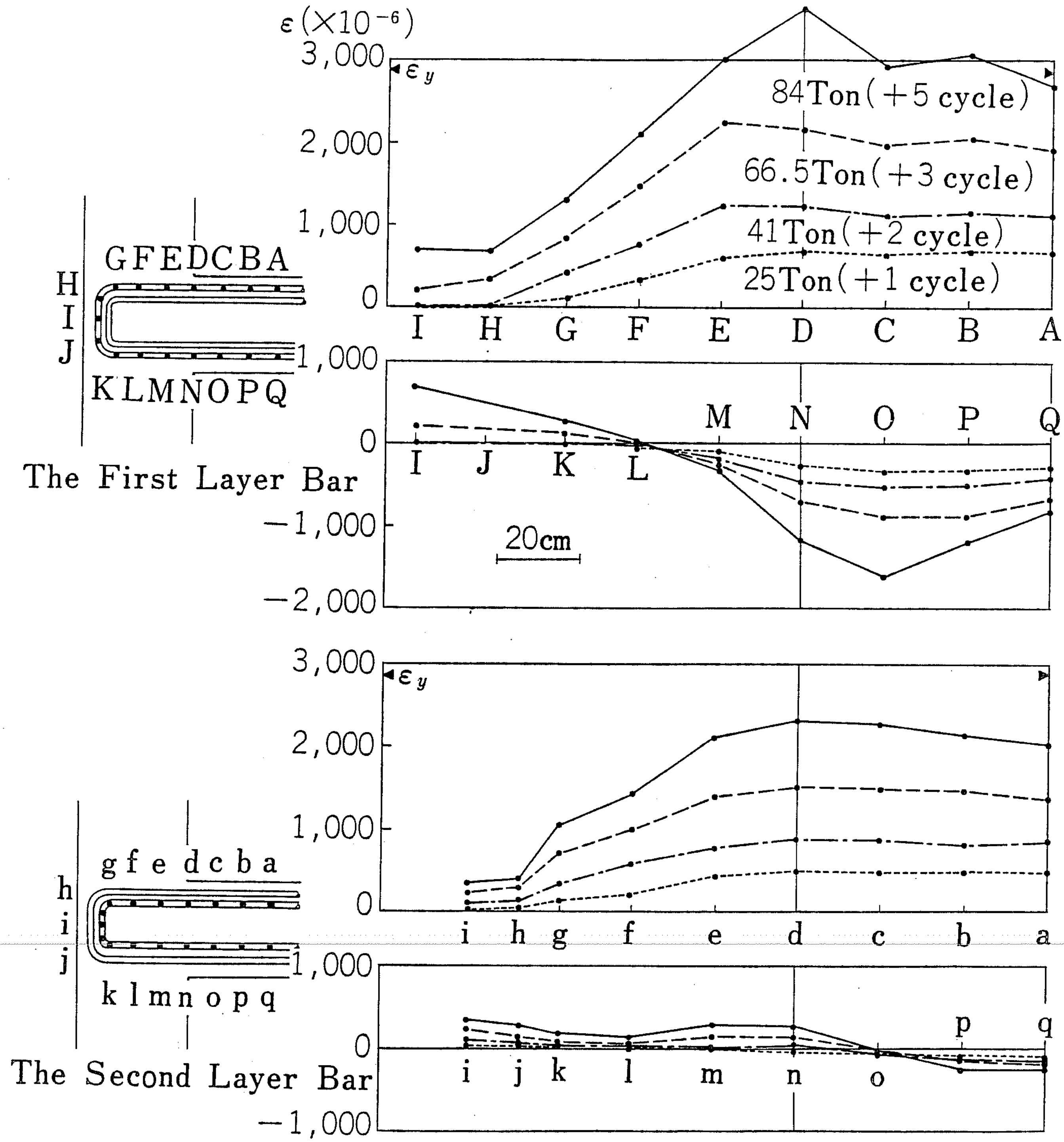


Fig.2.12 Strain Distribution of Beam Reinforcing Bars

3. STUDY ON ASEISMIC DESIGN OF HIGH-RISE REINFORCED CONCRETE BUILDINGS

by

Kenzoh Yoshioka
Toshikazu Takeda

PART 1: Advanced design of multi-story reinforced concrete buildings, by Toshikazu Takeda, Kenzoh Yoshioka, and Hiroaki Eto (see Proceedings, 12th Congress IABSE, Vancouver, B.C., Sept. 3-7, 1984)

PART 2: Simulated earthquake tests of reinforced concrete frame structures with columns subjected to high compressive stress, by Hiroaki Eto and Toshikazu Takeda (see Proceedings, 8th World Conference on Earthquake Engineering, San Francisco, July 1984)

PART 3: Experiments on beam-column joint subassemblages subjected to seismic loading

May 28, 1985

Ohbayashi Corporation
Technical Research Institute

PART 1: ADVANCED DESIGN OF MULTI-STORY REINFORCED CONCRETE BUILDING

by

Toshikazu Takeda
Kenzoh Yoshioka
Hiroaki Eto

Abstract

In this paper, a design concept and design technique for rendering a multi-story reinforced concrete building into a ductile structure of highly reliable earthquake resistance is proposed. In effect, the multi-story reinforced concrete building is made of pure frames without any shear wall, and the structure forms a beam-yielding mechanism with excellent energy absorption properties. The collapse mechanism of a beam-yielding type is one in which damage during earthquake is not concentrated at a single part of the building and the entire building becomes a collapse type so that it is superior in earthquake resistance.

In order to make a building a ductile structure it is necessary for the component members to have high degrees of deformability. For this purpose, the columns are to be round columns using spiral hoops, while beam-to-column panels are to be provided with horizontal haunches for a structure to prevent pull-out of the main reinforcing bars of beams, and in addition, to secure ductility of columns by limiting axial forces of the columns at the time of the collapse mechanism. Further, the design shear forces of beams and columns are increased by extra amounts for ultimate strength design to impart deformability after yielding in flexure. Also, in order to assure that it will be a beam-yielding type collapse mechanism, the columns are designed for extra bending stresses.

In seismic response analysis of the building, displacements between stories and story shear forces are checked along with ductility factors of members performing not only lumped mass system response analyses, but also frame response analyses.

A case of design of a 30-story reinforced concrete building in accordance with the above design technique was indicated, dynamic experiments of a 1/7-scale partial frame and experiments of a 1/2-scale column including beam-to-column panel with a model of the building as the object were performed, and it was confirmed that the designed building would be amply safe even during a severe earthquake.

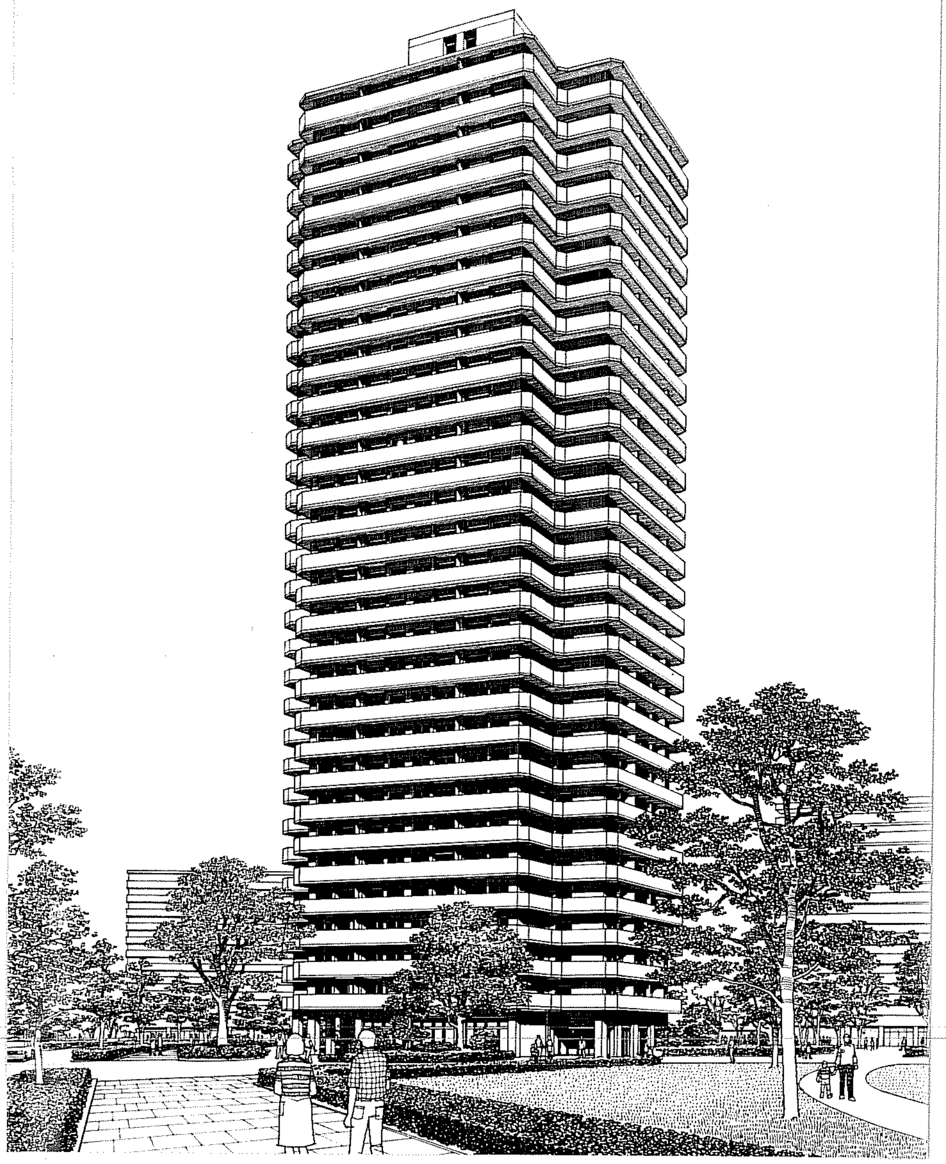


Fig.1 RC 30-story Apartment House

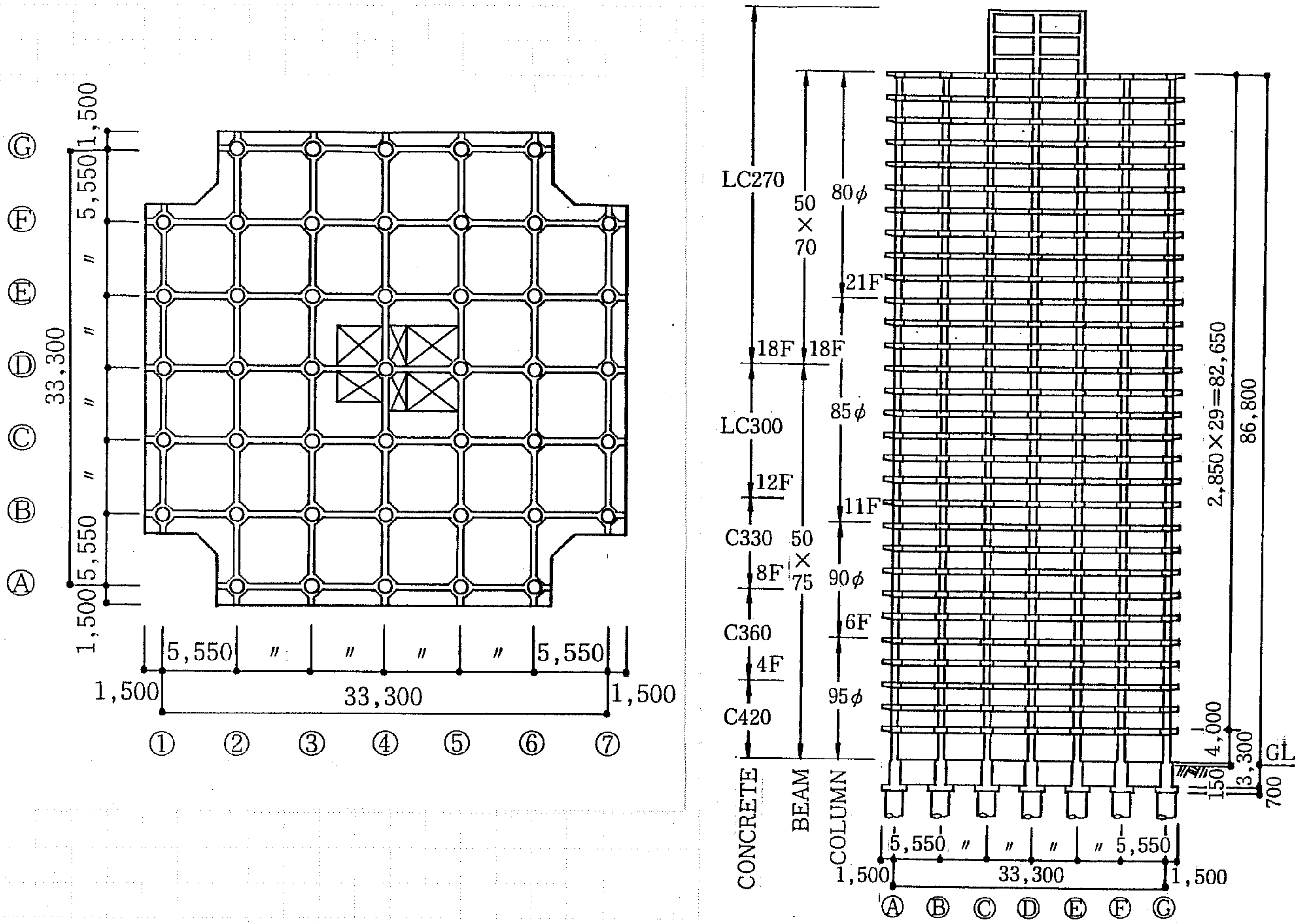


Fig.2 Plan and Flaming Elevation

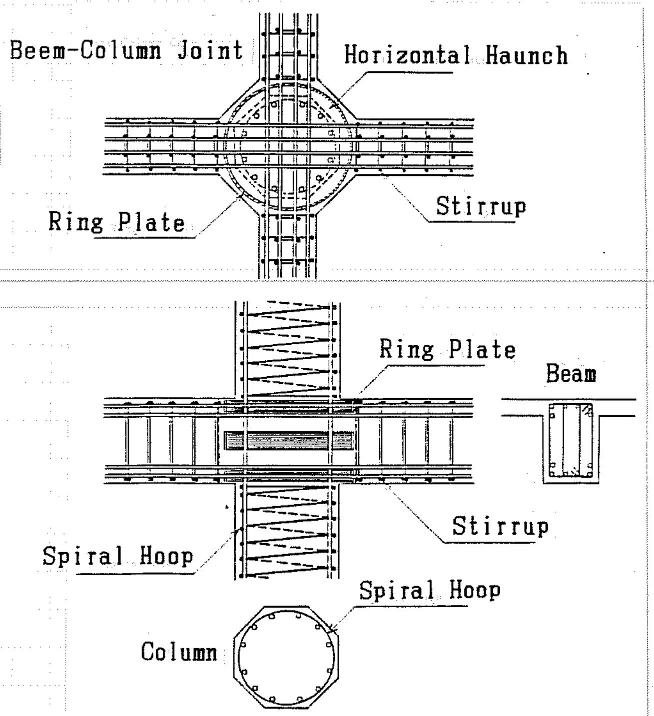


Fig.3 Details of Beam-Column Connection

Table.1 Natural Period

T_1 (sec)	T_2 (sec)	T_3 (sec)
1.813	0.638	0.376

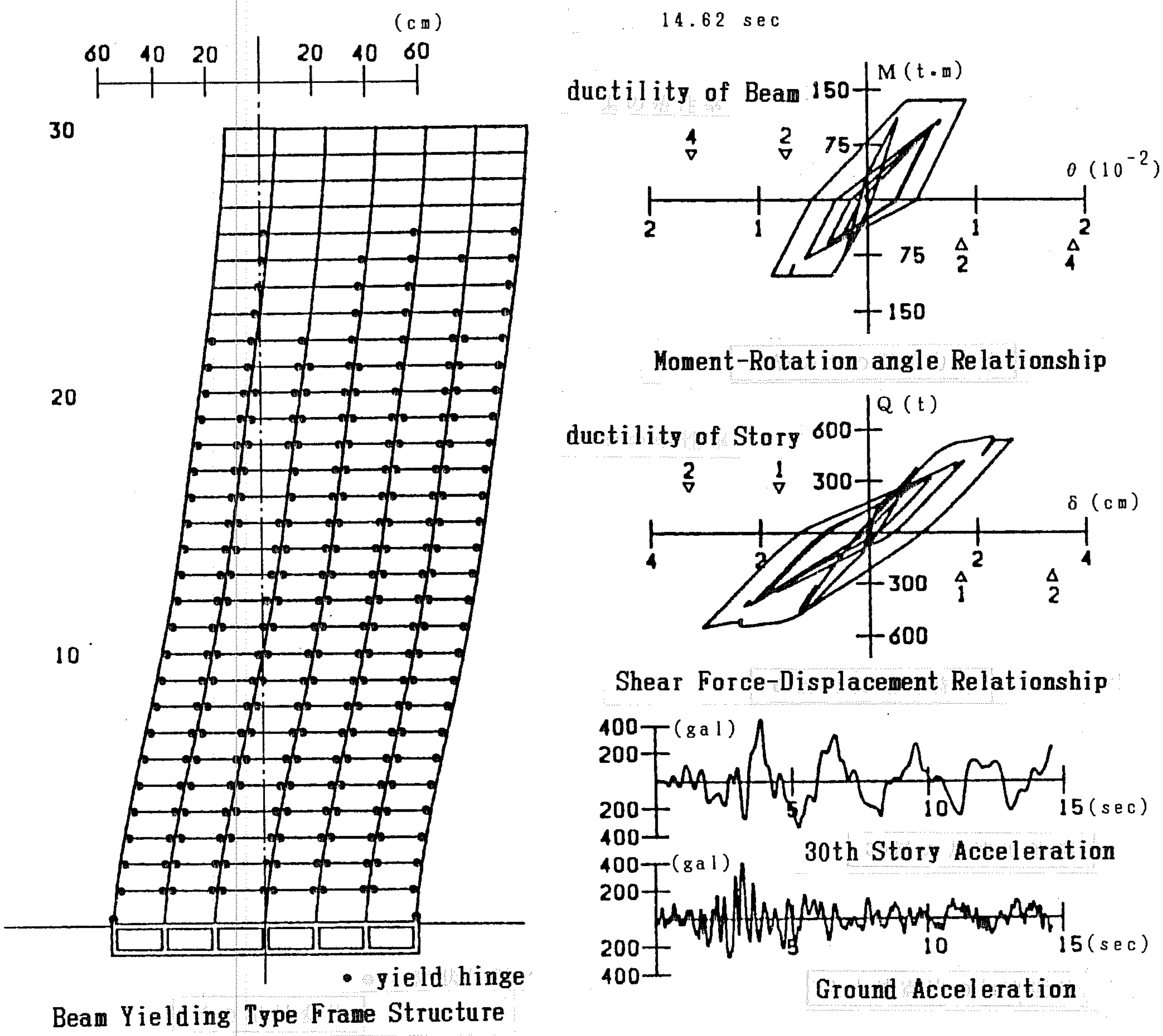


Fig.4 Inelastic Plane Frame Earthquake Response Analysis
HACHINOHE NS 400 gal

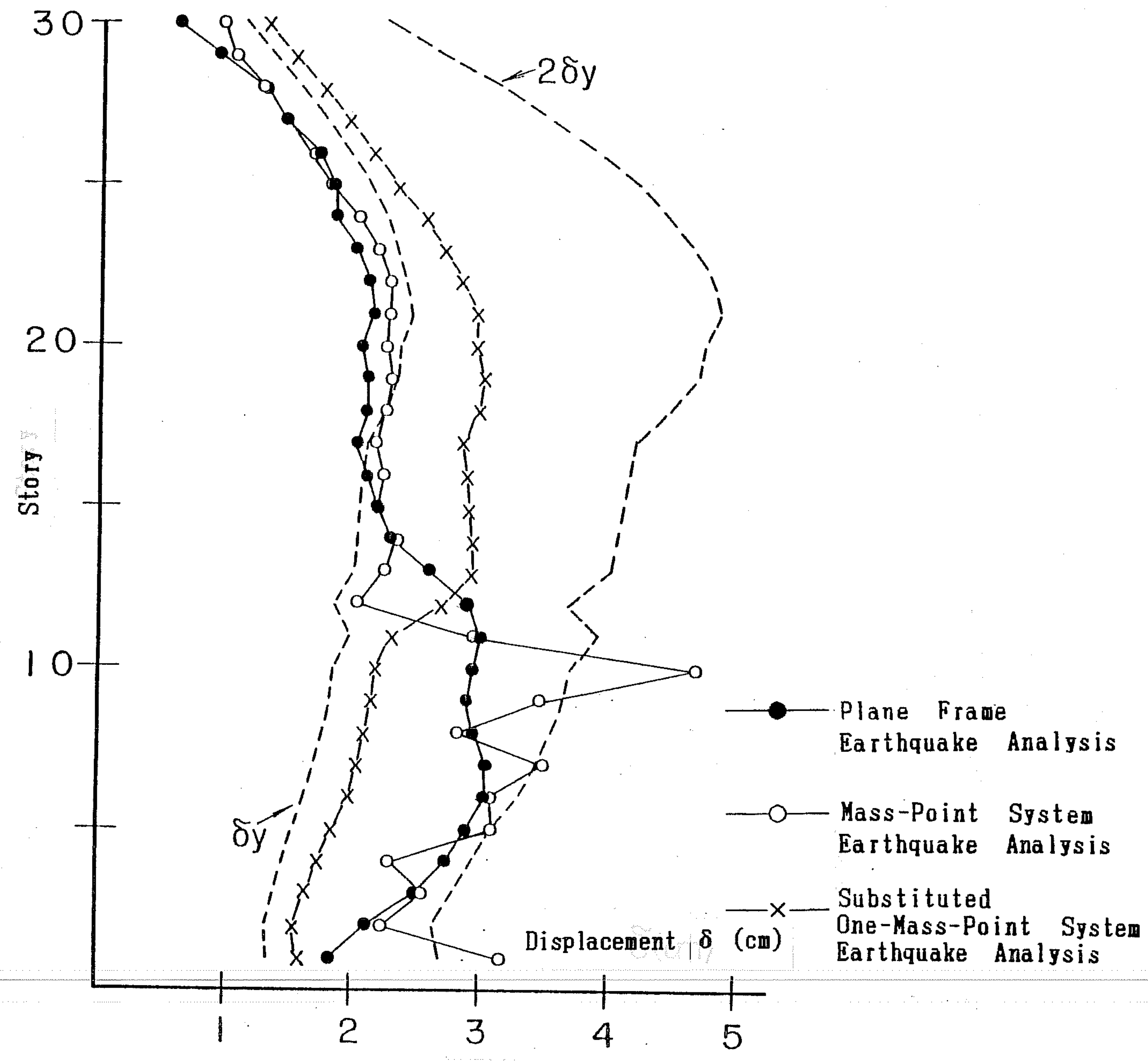


Fig.5 Maximum Response Story Drift
HACHINOHE NS 400 gal

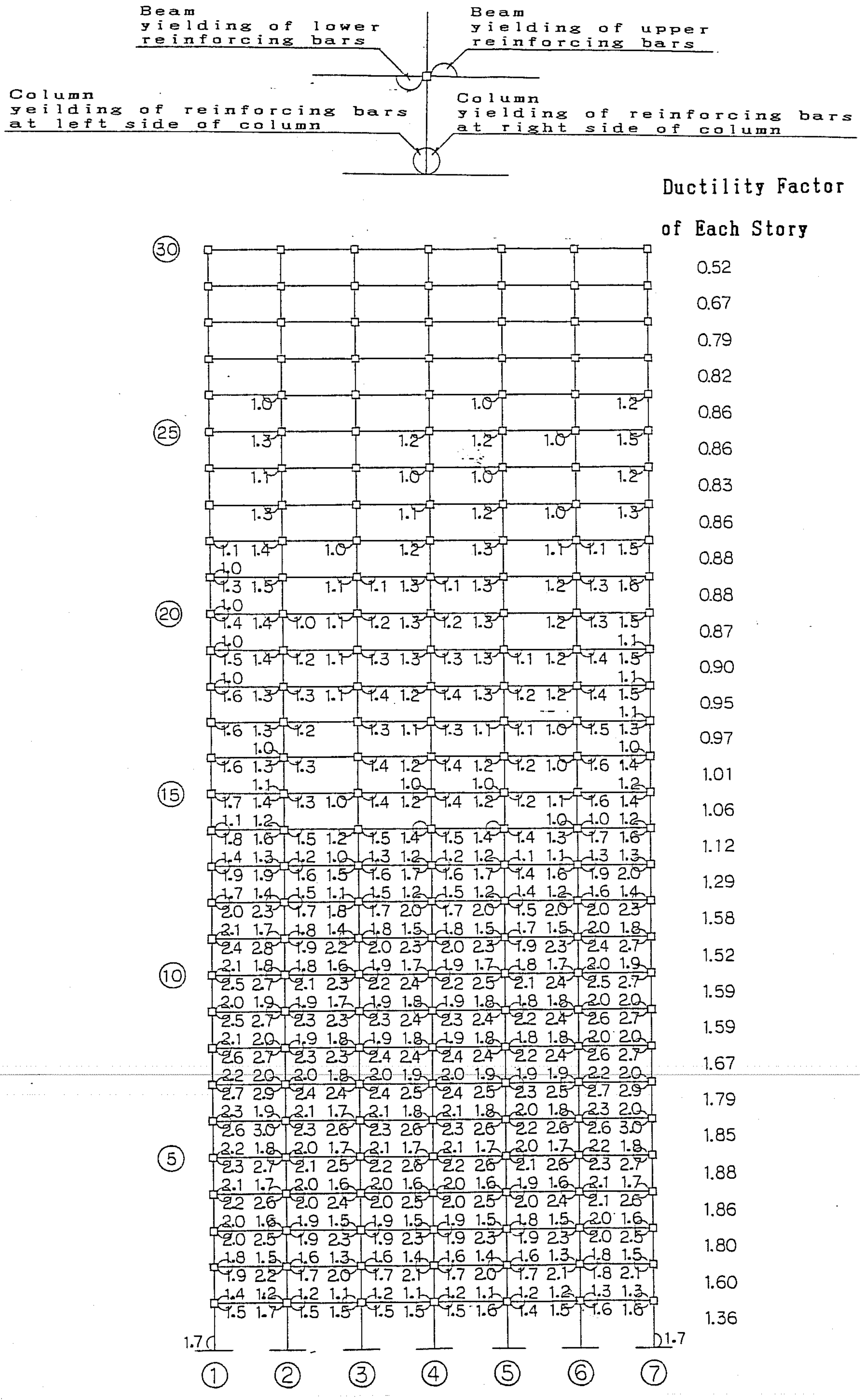


Fig.6 Ductility Factors of Yield Hinges

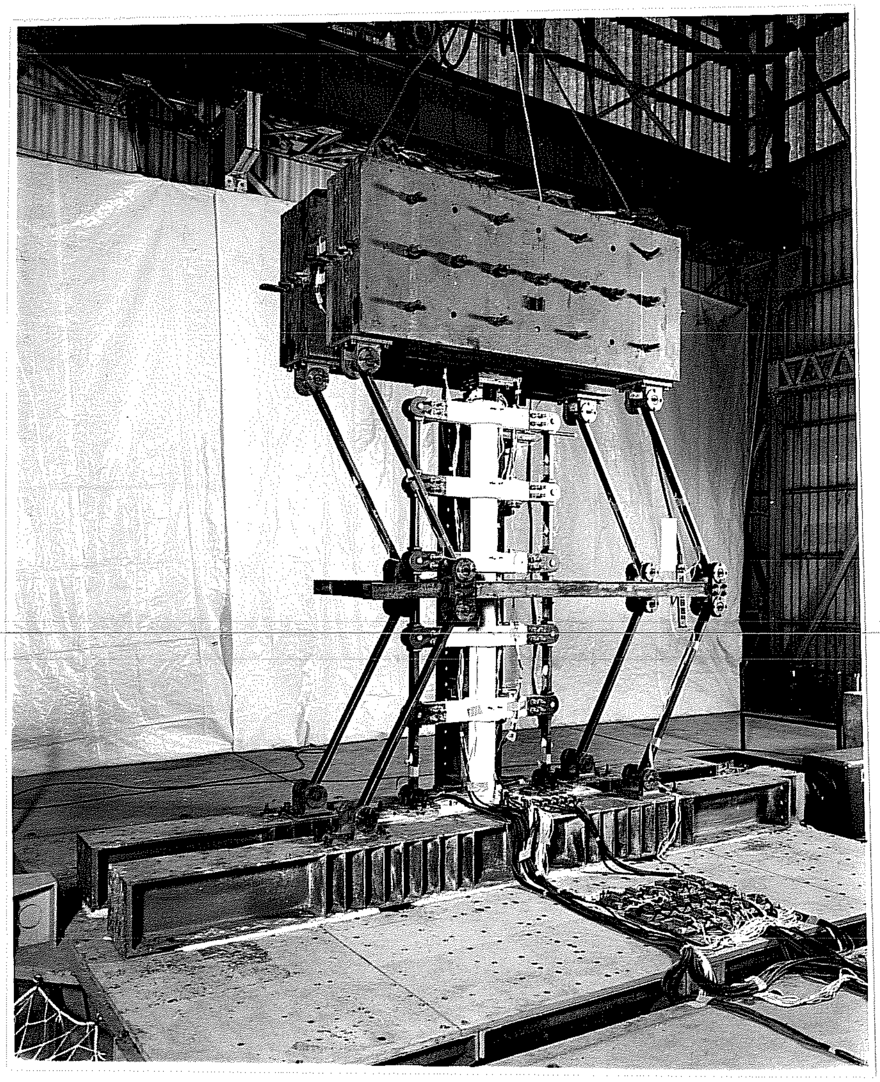
PART 2: SIMULATED EARTHQUAKE TESTS OF REINFORCED CONCRETE FRAME STRUCTURES WITH COLUMNS SUBJECTED TO HIGH COMPRESSIVE STRESS

by

Hiroaki Eto
Toshikazu Takeda

Summary

In order to confirm the aseismic safety of a 30-story reinforced concrete building possessing superior ductility which is of a type with beams yielding under bending, simulated earthquake tests, static loading tests and their analyses were carried out on 6-story reinforced concrete frame structures modelling the bottom part of this building. This paper describes the results of these tests and analyses.



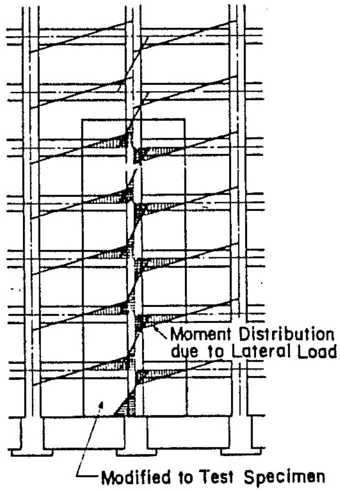


Fig. 1 Modelled Prototype Building

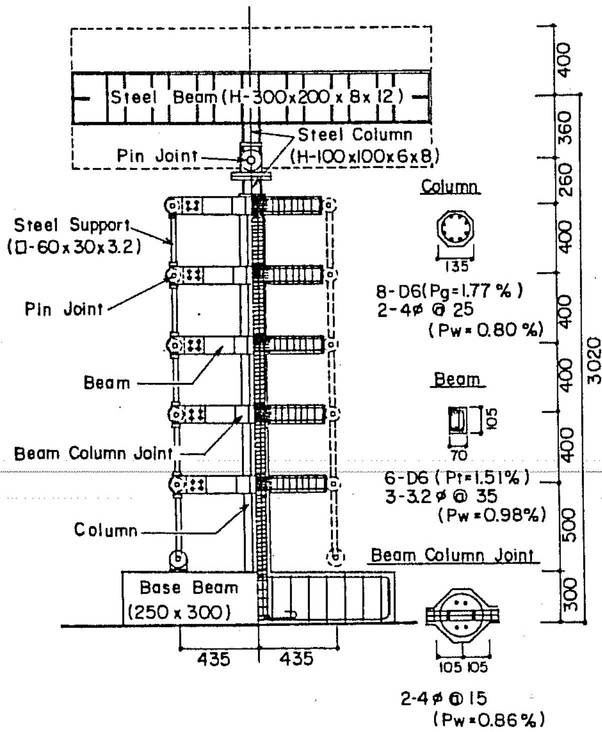


Fig. 2 Test Specimen

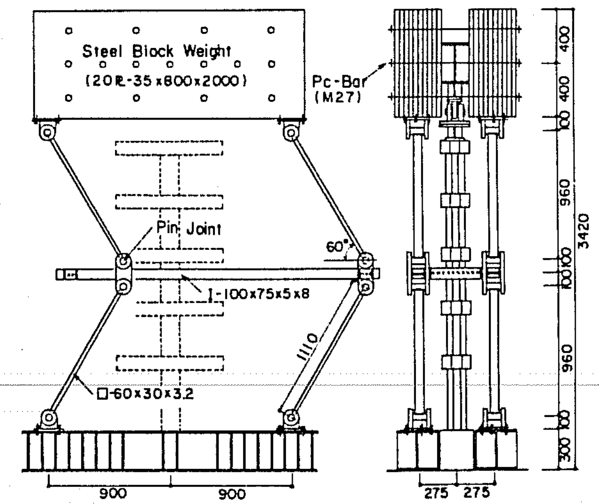


Fig. 3 Horizontal Movement Equipment

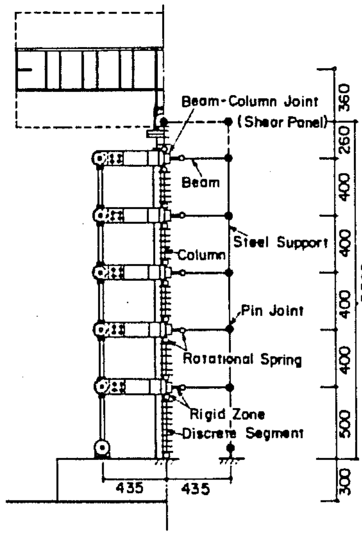


Fig. 4 Mathematical Model of Specimen

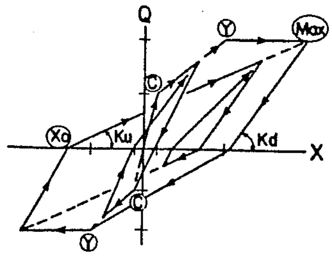


Fig. 5 Hysteresis Loops

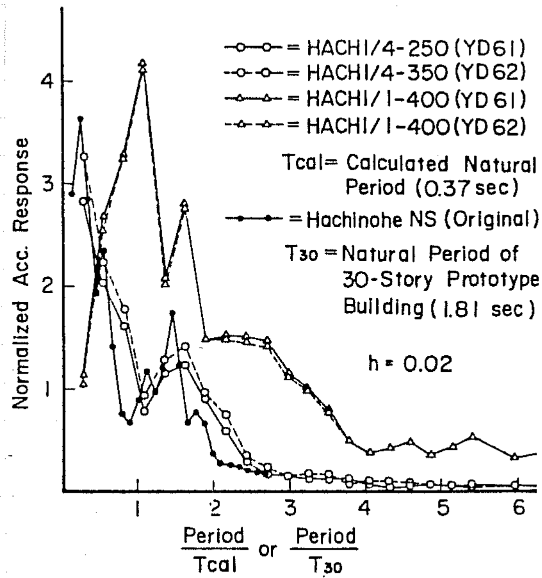


Fig. 6 Normalized Response Spectrum

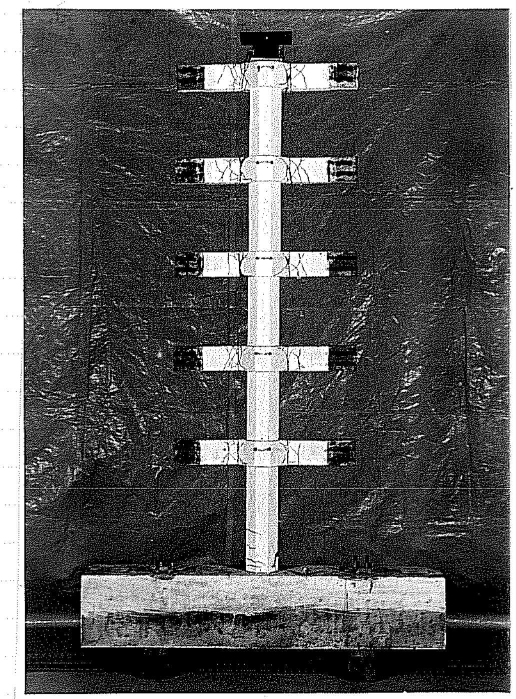
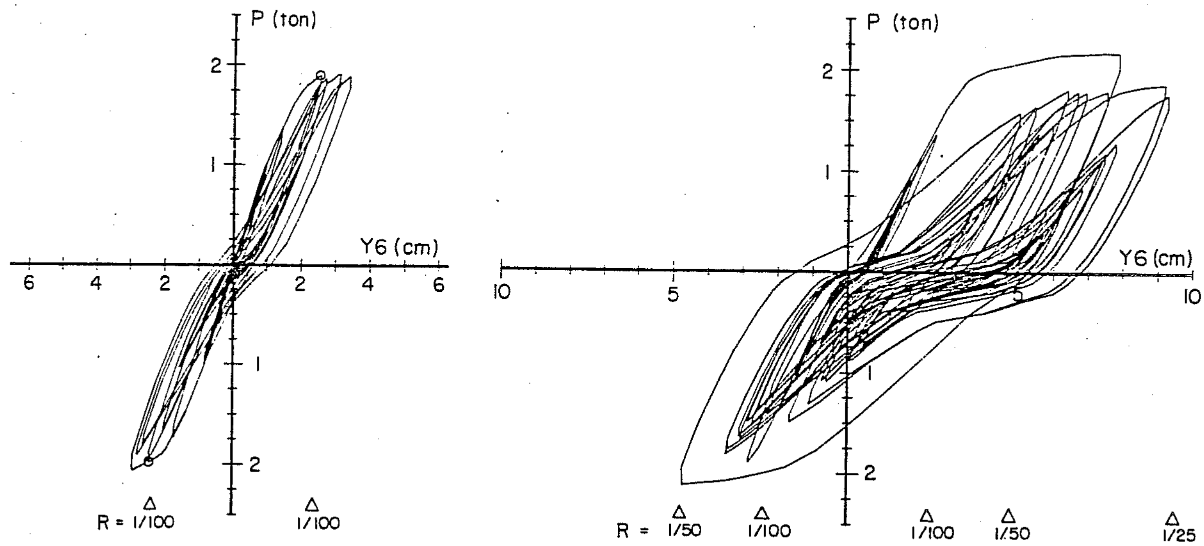


Fig. 7 Final Failure State of YD62

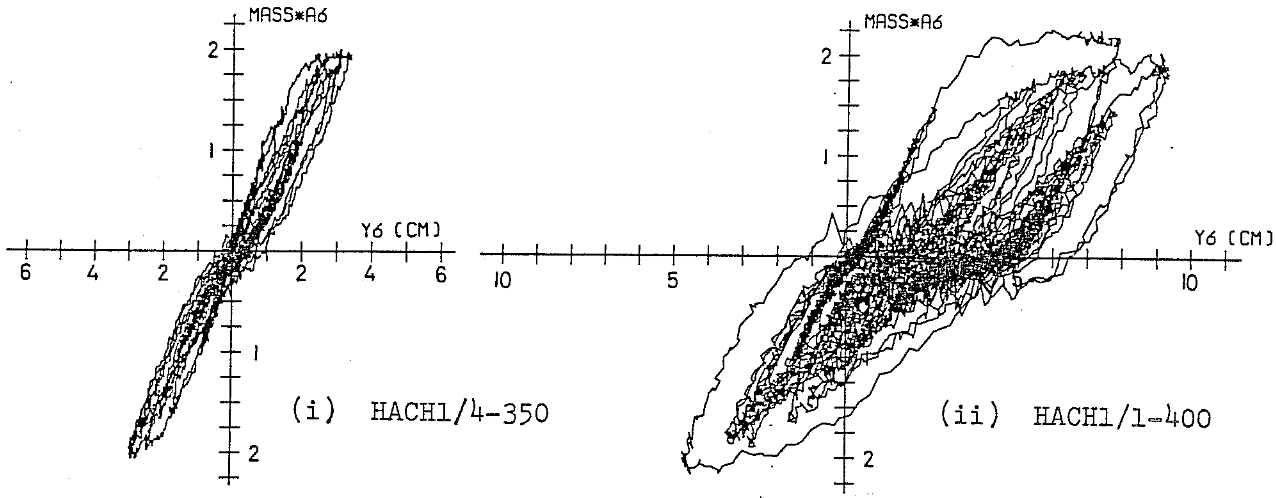
Table 1 Maximum Values of Measured Response

Specimen	Input Earthquake Wave	Acc. (gal) Base Motion [A0]	Acc. (gal) 6th Story [A6]	Disp. (cm) 6th Story [Y6]	Rotational Angle [R]	Ductility Factor [μ]
YD 61	HACHI/4 - 250	246 (5.764)	214 (9.012)	2.40 (9.026)	1/98	0.96
	HACHI/1 - 400	395 (18.170)	255 (18.300)	8.57 (36.815)	1/28	3.43
YD 62	HACHI/4 - 350	342 (5.162)	226 (7.528)	3.40 (9.302)	1/69	1.36
	HACHI/1 - 400	406 (18.175)	249 (18.305)	9.31 (36.875)	1/25	3.72

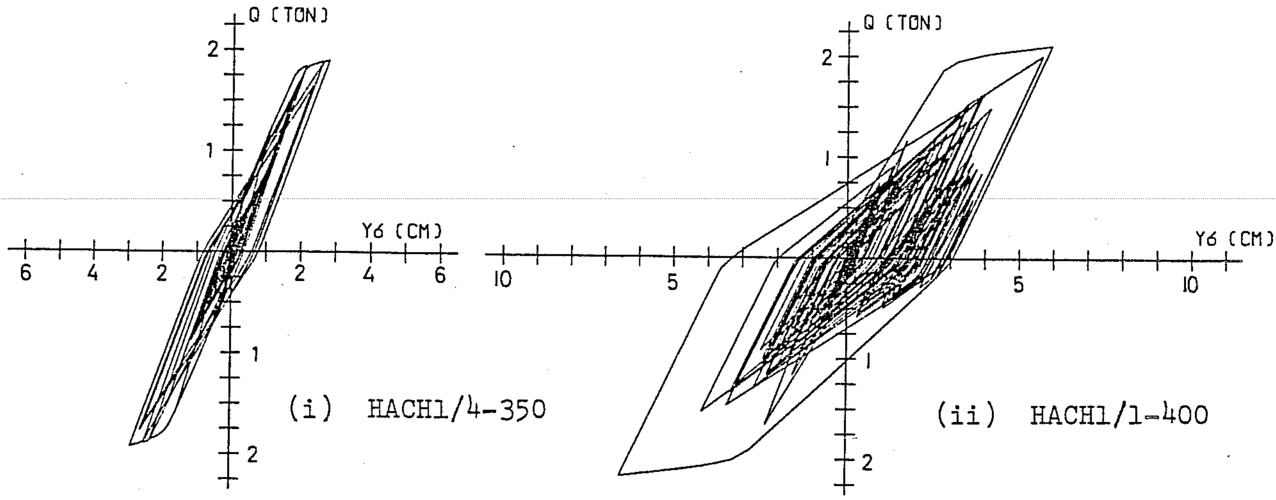
( ) = Occurrence Time (sec)



(a) Measured in Static Test (YS61)



(b) Measured in Dynamic Test (YD62)



(c) Calculated in Dynamic Analyses (YD62)

Fig. 8 Force-Displacement (Y6) Relationship in Tests and Analyses

PART 3: EXPERIMENTS ON BEAM-COLUMN JOINT SUBASSEMBLAGES SUBJECTED TO SEISMIC LOADING

by

Toshikazu Takeda
Toshimasa Tada

Abstract

This paper describes an experimental study concerning reinforced concrete beam-column joint subassemblages subjected to seismic loading. For realization of moment-resisting reinforced concrete high-rise buildings to withstand catastrophic earthquakes, it is necessary to obtain ductile members for the frame. With regard to beam-column joints, it is necessary to prevent shear failure of the joints and bond slip of beam reinforcement through the joints to guarantee ductile beam-end plastic hinges having sufficient energy absorbing capacities. Seven specimens were tested under repeated reversible loading. A number of these specimens were provided with threaded bar beam reinforcement, ring plates in joints, and horizontal haunches around joints. Through these experiments, joint details showing good performance against repeated loading were obtained.

Experimental Program

Seven reinforced concrete beam-column joint subassemblages were tested under cyclic loading. The specimens were half-scale models having dimensions shown in Fig. 1. Material properties are listed in Table 1 and specimen details are shown in Fig. 2.

Specimen No. 1 had standard details in which the joint was laterally reinforced with a spiral as in the column. In Specimen No. 2, beam bars in the middle of the layers were anchored by bending at a right angle in the joint, which was laterally reinforced with steel ring plates. In Specimen No. 3, beam bars at the corners were threaded and fastened to ring plates with nuts, while the other beam bars were anchored in the expected hinge zones by bending at an angle of 45 degrees. Specimen No. 4 had the same details as in No. 3 except that the middle ring plate was reduced to half-depth of that in No. 3. The joint in Specimens No. 5 and No. 6 was enlarged with a horizontal haunch which was laterally reinforced with ring plates. All beam bars in No. 5 were developed continuously in the joint, while middle beam bars in No. 6 were anchored outside the joint in the same way as in No. 3. Specimen No. 7 modeled an exterior connection where beam bars were anchored by bending at a right angle and the joint was reinforced with ring plates. The amount of joint reinforcement with ring plates in Specimens No. 2, 3, 5, 6, and 7 was approximately equivalent to two times that of the spiral in No. 1.

Cyclic loading was applied to beam tips, as shown in Fig. 3. An axial load of 80 tons was applied to the column in No. 4, and 160 tons to the other specimens.

Test Results

Load-deflection curves and final crack patterns are shown in Figs. 4 and 5, respectively.

Specimen No. 1 failed at the beam ends where cracks opened widely and concrete crushed, mainly due to pullout of beam bars from the joint. Specimens No. 2 through 7 (except No. 4) failed in beam shear after yielding of beam bars. The shear failure occurred within the plastic hinge zone in No. 2, 5, and 7, and outside the hinge zone in No. 3 and 6. Specimen No. 4 failed in joint shear after yielding of beam bars.

Although Specimen No. 1 showed inverse S-shaped hysteresis loops because of the bond-slip, the other specimens showed stable hysteretic behavior. Particularly, the load-deflection curves for Specimens No. 3, 4, 5, and 6 were spindle-shaped and showed excellent capability to absorb energy.

Test results in the other series are shown in Fig. 6 for reference. The specimen with a straight bar development length of 37.5 bar diameter showed better hysteretic behavior than with that of 28 bar diameters.

Table 1. Material Properties

	$_s\sigma_y$ (kg/mm^2)	$_s\sigma_u$ (kg/mm^2)
D16 (SD40)	43.4	63.0
D19 (SD35) *	36.5	54.8
D13 (SD30)	32.5	50.0
D10 (SD40)	46.7	63.0
PL9 (SS41)	28.3	44.2
CONCRETE (Normal Weight) (kg/cm^2)	Fc = 378 Ft = 34.3 Ec = 2.4 $\times 10^5$ (6 - week)	

* D19 is threaded bar

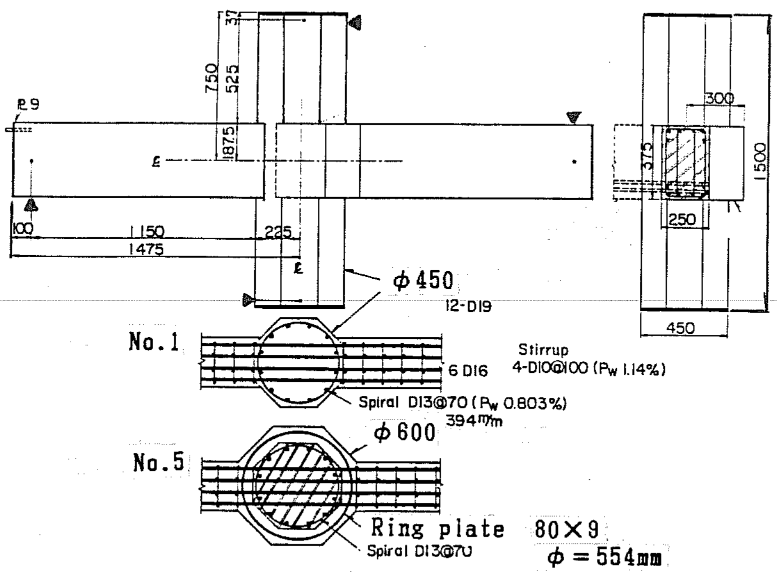


Fig.1 Test Specimens

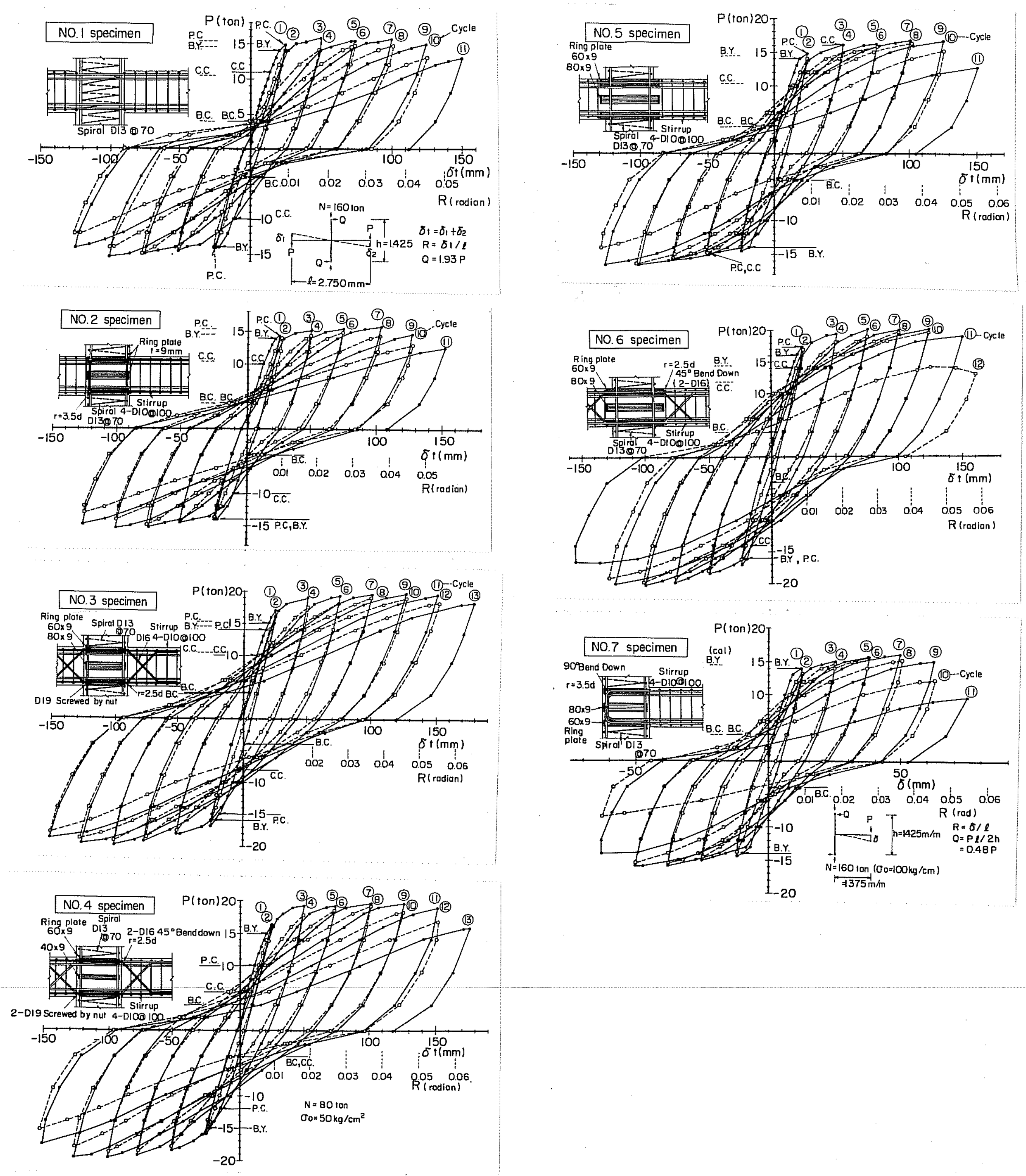


Fig.4 Load-Deflection Curves

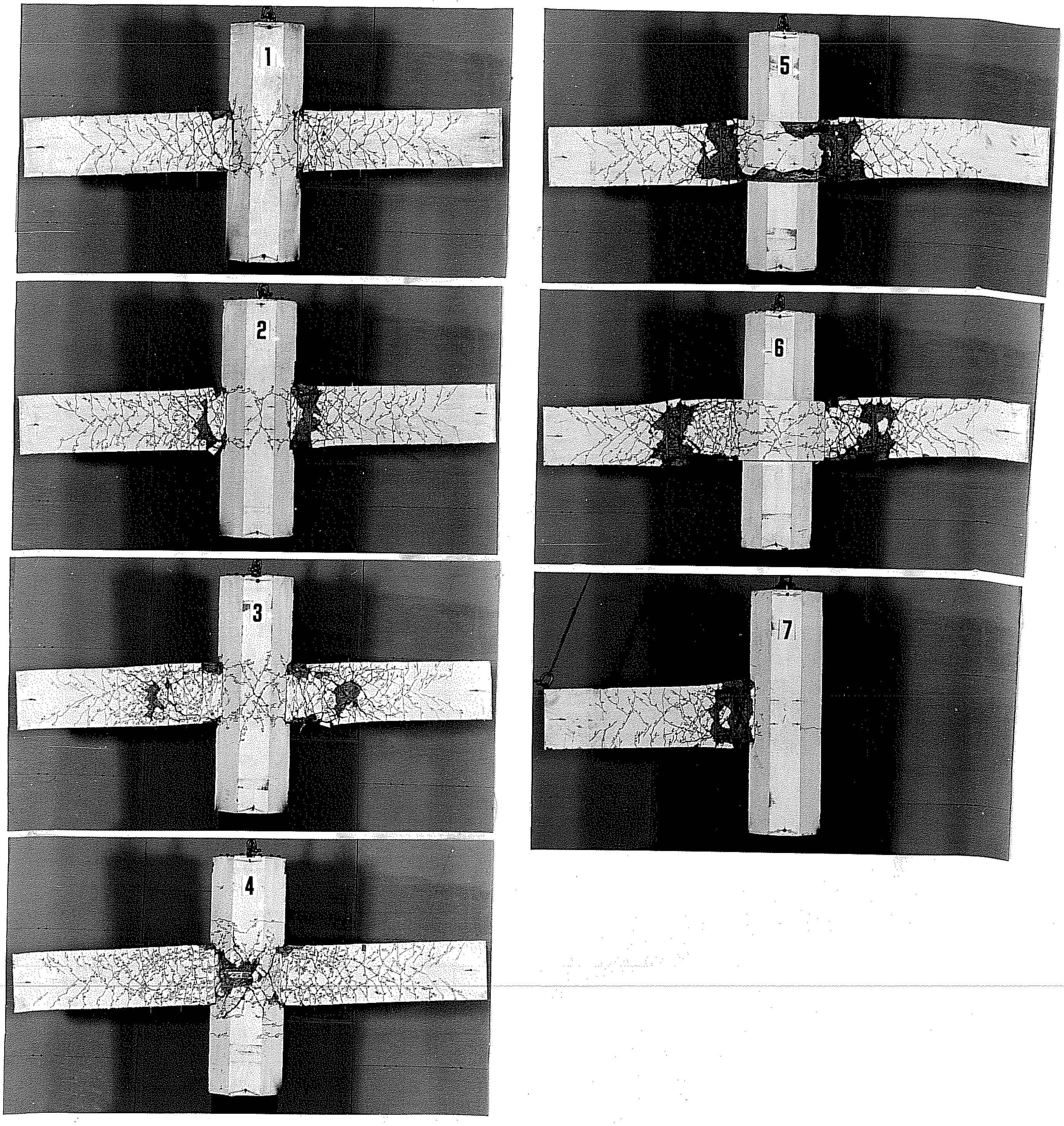


Fig.5 Crack Patterns

4. STRUCTURAL DESIGN AND RESEARCH OF REINFORCED CONCRETE TALL BUILDINGS

by

Shunsuke Sugano

Toshio Nagashima

Part 1 : Seismic Resistant Design, Analysis and Construction of Tall Reinforced Concrete Framed Buildings

Part 2 : Experimental Studies on Columns and Frames for Tall Reinforced concrete Buildings

Part 3 : Experimental and Analytical Studies on The Seismic Behavior of R/C Moment Resisting Frames Which Have Wide Columns by Ikuo YAMAGUCHI, Shunsuke SUGANO, and Yasuo HIGASHIBATA (see Proceedings of the Eighth World Conference on Earthquake Engineering, San Francisco, July 1984: also, Takenaka Technical Research Report No.32, November 1984)

May 28, 1985
(Revised in Oct. 1987)

Takenaka Komuten Co., Ltd.

PART 1: SEISMIC RESISTANT DESIGN, ANALYSIS AND CONSTRUCTION OF TALL REINFORCED CONCRETE FRAMED BUILDINGS

1. INTRODUCTION

This paper describes the philosophy and procedures proposed by the authors for the seismic resistant design of tall reinforced concrete framed buildings, and their application. A 30-story residential building which has been constructed in suburban Tokyo since October 1986 was designed employing the methods described here and the seismic performance of the designed structure was assessed through both analytical and experimental investigations.

2. DESIGN PHILOSOPHY AND PROCEDURE

2.1 General Design Philosophy

For the design procedures presented herein, the following philosophy and criteria are adopted to provide a building with sufficient seismic resistant capacity.

2.1.1 Hinge mechanism of a structure. The philosophy of so-called strong column and weak beam is adhered to. The desirable collapse mechanism of a multi-story framed structure must be a whole collapse mechanism rather than a partial sidesway mechanism (soft story mechanism) so as to provide the structure with enough energy-dissipating capability as well as adequate strength and stiffness. Plastic hinges are designed to be formed only at beam ends, bottom of columns in the lowest story and top of columns in the uppermost story. The formation of plastic hinges of exterior columns under axial tension may be allowed since such columns will behave in a ductile manner.

2.1.2 Design of members. Beams are designed and detailed to possess enough ductility (displacement ductility of more than four) as well as sufficient shear capacity. In a potential hinge region, closed stirrups are arranged so that each longitudinal reinforcement may be restrained against buckling, as shown in Fig. 2.1.

Columns are designed to have sufficient flexural and shear capacities in order to ensure the intended formation of plastic hinges in beams. Columns having potential plastic hinges are designed and detailed also for ductile behavior. Special attention must be paid on the level of axial compression and lateral reinforcement to ensure the required ductility. The lateral reinforcements are arranged so as to confine the core concrete and to restrain longitudinal bars against buckling, as shown in Fig. 2.1.

Beam-column joints are designed to possess sufficient shear capacity to prevent brittle failure. Adequate development length of longitudinal reinforcement of beams passing through interior joints must be provided to prevent the deterioration of energy-dissipation capability due to the loss of bond inside the joint panel subjected to cyclic-reversed, inelastic deformation.

2.1.3 Seismic analysis and performance criteria. Both static and dynamic inelastic analyses are conducted to determine design lateral forces, to discuss the seismic response of the designed structure and/or to assess the seismic capacity of the structure.

Two levels of earthquake input, the maximum probable and maximum credible levels with the intensity of 25 and 50 cm/s in terms of the maximum ground velocity, respectively, are considered for the dynamic response analysis. Note that a special study on the local seismicity must be performed to determine the specific input for the site when the site has special soil conditions such as deep soft soils.

Under the first level input, the structure should remain within the pre-yielding range. There should be no occurrence of plastic hinges in any structural members sustaining an interstory drift less than 0.5%. It is also intended that under the second level earthquake, post-yielding displacement may occur but should not exceed 1% in terms of the interstory drift. For the member level displacement, ductility of not more than four is allowed.

It should be noted that the levels of seismic input and performance criteria mentioned here were determined referring to design practices for tall buildings of other structures of steel or steel-encased reinforced concrete.

2.2 Design Flow

The design procedure is outlined as follows:

Step 1. To determine design lateral forces and their distribution based on preliminary inelastic dynamic response analysis. The existing codes and recommendaitons are referred.

Step 2. To design members for combined stresses under gravity loads and design lateral forces using an allowable stress design method based on the A.I.J. Structural Standard.

Step 3. To compute the flexural capacity of each potential plastic hinge, and to determine corresponding moment, shear and axial forces in each member at the plastic hinge mechanism of a total frame.

Step 4. To design beams for shear based on the ultimate strength.

Step 5. To design columns for flexure, shear and axial forces based on the ultimate strength.

Step 6. To carry out the nonlinear static-frame analysis to check the hinge mechanism of a total structure and to estimate the ultimate strength of the structure.

Step 7. To determine the seismic response of a total structure through the nonlinear dynamic analysis for several recorded and/or artificial earthquake ground motions and to assess the seismic capacity of the designed structure.

2.3 Special Design Requirements

The special requirements proposed by the authors for the design of members are described as follows. The experimental studies done by the authors (see Part 2), as well as other researchers, were reflected to those requirements.

2.3.1 Design of beams for shear.

The design of cross section of a beam for shear is based on

(a) $Qsu \geqq QL + 1.1\ Qmu$ for potential plastic hinge regions

(b) $Qsu \geqq QL + Qmu$ for other regions

where

Qsu = ultimate shear strength

QL = shear force due to gravity loads

Qmu = shear force calculated from the flexural strength at both ends of the beam considering the effects of slab reinforcement and the overstrength of beam flexural reinforcement.

2.3.2 Design of columns for flexure and shear.

The design of a column for flexure is based on

$$cMmu \geqq 1.3\ cMu$$

where

cMmu = ultimate flexural strength of the column under the given axial force

$_cM_u$ = column moment at the hinge mechanism of the structure.

The design of a column for shear is also based on

$$_cQ_{su} \geqq 1.3\ _cQ_u$$

where

$_cQ_{su}$ = ultimate shear strength

$_cQ_u$ = column shear force at the hinge mechanism of the structure.

2.3.3 Limits for column axial force. Axial forces on columns are limited as follows:

(1) Under gravity loads alone

$$N_L \leqq 0.3\ bDF_c$$

(2) Under gravity loads and design lateral forces

$$-0.1\ bDF_c \leqq N_s \leqq 0.5\ bDF_c$$

(3) At hinge mechanism

(a) for exterior columns

$$-0.25\ bDF_c \leqq N_u \leqq 0.65\ bDF_c$$

(b) for interior columns

$$N_u \leqq 0.4\ bDF_c$$

where

N_L, N_s, N_u = column axial forces under gravity loads alone, gravity loads and design lateral forces, and at the hinge mechanism, respectively

b, D = width and depth of the column

F_c = specified compressive strength of concrete.

2.3.4 Design of beam-column joint for shear. The design of beam-column joint for shear is based on

$$_p\tau_D < {}_p\tau_u = {}_c\tau_u + {}_s\tau_u$$

$$c\tau u = \begin{cases} Fc(0.78-0.0016\ Fc) & (Fc \leqq 244\ kg/cm^2) \\ 95\ kg/cm^2 & (Fc > 244\ kg/cm^2) \end{cases}$$

$$s\tau u = 0.5\ Pw \cdot s\sigma y$$

where

$p\tau D$ = nominal shear stress in the joint at the hinge mechanism

$$p\tau D = \frac{\Sigma bMu}{(1+\xi)eVc} \qquad \xi = D_b/h', \quad eVc = t_p \cdot j_b \cdot j_c$$

$$t_p = \frac{b_b+b_c}{2}, \quad j_b = 7/8d_b, \quad j_c = 7/8d_c$$

$p\tau u$ = ultimate shear strength of the joint

$p\tau c$ = shear strength provided by concrete (multiplied by 2/3 for exterior joints)

$p\tau s$ = shear strength provided by shear reinforcement

Pw = ratio of shear reinforcement

$s\sigma y$ = specified yield strength of shear reinforcement

bMu = ultimate flexural capacity of beams framing into the joint

h' = clear height of column

D_b = depth of beam

b_b, d_b = width and effective depth of beam

b_c, d_c = width and effective depth of column.

2.3.5 Development of beam flexural reinforcement. The development length of beam flexural reinforcement passing through interior joints is not less than 20 times the bar diameter.

3. DESIGN AND ANALYSIS OF A 30-STORY BUILDING

3.1 Building Description

The building is a 30-story condominium being constructed in suburban Tokyo. The perspective view, typical floor plan and section of the structure are shown in Photo 3.1, Figs. 3.1 and 3.2, respectively.

The structure consists of moment-resisting space frames having six spans in both directions, while it has shear walls at the basement story. The structure is supported by cast-in-place reinforced concrete piles extending 27 m below the ground level into a firm gravel layer.

The structure utilizes the concrete of the specified strength ranging from 420 kg/cm^2 (41 Mpa, 6000 psi) at the lower portion to 210 kg/cm^2 (21 Mpa, 3000 psi) at the upper portion of the building. It also utilizes steel bars of the specified yielding strength 4000 kg/cm^2 (392 Mpa, 57 ksi) and of the diameters from 41 [D41(#13)] to 25 mm [D25(#8)] as flexural reinforcement. It is noted that welded-wire fabrics and flash-welded, closed stirrups are utilized as lateral reinforcement in the columns and beams, respectively (see Fig. 2.1).

All columns and beams are cast-in-place reinforced concrete, while semi-precast concrete decks are used to construct composite floor slabs.

3.2 Design of Structure

3.2.1 Design earthquake loads. In order to determine design earthquake loads, preliminary nonlinear-response analysis was carried out for some recorded earthquake ground motions factored to meet the peak velocity of 25 cm/s, idealizing the structure as a lumped MDOF system. The estimated fundamental period of the structure was 1.98 sec and 2.02 sec for longitudinal (X) and transverse (Y) directions, respectively.

Referring not only to the result of the preliminary response analysis shown in Fig. 3.3, but also to available design recommendations for tall buildings and design practices, the value of design base shear coefficient was taken as 0.12. The distribution of lateral forces was determined, as shown in Fig. 3.3, based on the result of the preliminary analysis. Note that it was aimed at providing the structure with the ultimate lateral load-carrying capacity, approximately 1.5 times the design lateral forces.

3.2.2 Structural design. The structure was designed based on the A.I.J. Structural Standard and the special provisions described in Sec. 2.3. Typical cross sections of beams and columns are listed in Table 3.1.

3.3 Analysis of Structure

3.3.1 Static analysis. Nonlinear static frame analysis in member-to-member level was conducted to investigate the formation of plastic hinges and the inelastic displacement of the designed structure. The results are shown in Figs. 3.4 and 3.5. It was indicated that the hinge mechanism would be formed as designed, that is, whole collapse mechanism. The obtained ultimate capacity of the structure in terms of the base shear coefficient was 0.18, which is 1.5 times the value of the design lateral forces.

3.2.2 Dynamic response analyses.

(1) Response analysis of a lumped MDOF system. Idealizing the structure as a lumped MDOF system and considering the soil-structure interaction, nonlinear dynamic analysis was carried out to investigate the response of an overall structure to earthquake motions. The shear force vs displacement relationship of each story was determined based on the static inelastic frame analysis, and was idealized as shown in Fig. 3.6.

The viscous damping ratio of 3% for the first mode was assigned and proportioned to the initial stiffness. The recorded earthquake ground motions listed in Table 3.2 were used. The amplitude in each acceleration record was scaled in the manner where the maximum velocity reaches 25 and 50 cm/s, corresponding to the maximum probable and the maximum credible levels of earthquake motions, respectively.

As shown in Fig. 3.7, the results indicated that the maximum response of the interstory drift would be 1/337 and 1/172 when subjected to the maximum probable and the maximum credible motions, respectively. It was also indicated that the maximum shear force would not reach the ultimate capacity at any story.

(2) Member level response analysis of frames. In order to determine the seismic behavior of not only the whole structure but also structural members in detail, the nonlinear dynamic analysis in member-to-member level was conducted for a frame to represent the whole structure. The recorded ground motions of EL CENTRO 1940 NS with the scaled maximum velocity of 25 and 50 cm/s were used as input motions.

The obtained maximum interstory drifts corresponding to the two levels of motions were 1/520 and 1/211, respectively, as shown in Fig. 3.8. No plastic hinge was developed in any beams or columns.

(3) Torsional response analysis of a 3-D frame. Because the framing system of the building is not symmetric to the X direction due to void space, the structure has somewhat eccentricity to the X direction. Therefore, the nonlinear response analysis was conducted idealizing the structure as a quasi-three-dimensional model to investigate the torsional response of the structure. The result, however, indicated that the structure would respond to severe earthquakes without any significant torsional effect.

4. CONSTRUCTION OF THE 30-STORY BUILDING

The construction of the building began on October 1987 and it will be completed on May 1988. Several recently developed techniques have been used for the construction. The features of the construction system are described as follows:

(1) Reinforcing steel cages for all the columns and beams are prefabricated at the ground level of the site and lifted to their positions by a tower crane. Photo 4.2 shows three-dimensional prefabricated steel cages being lifted.

(2) Semi-automated enclosure welding is used to connect large-size longitudinal bars of beams and columns. Main bars of a column are connected at the position of one-meter high from the floor level and those of a beam are connected at mid-span (see Photos 4.3 and 4.4).

(3) Systematic large metal forms, as shown in Photo 4.5, are used for beams. For columns, semitransparent firm plastic forms are used together with metal forms (Photo 4.6).

(4) Large semi-precast concrete decks are used to construct composite floor slabs (Photo 4.7). The decks are also prefabricated at the site.

The outline of the construction procedure is illustrated in Fig. 4.1.

5. CONCLUDING REMARKS

The philosophy and procedures of the seismic-resistant design for tall reinforced concrete framed buildings were presented here. Emphases were put on the hinge mechanism, the special provisions for members subjected to flexure, shear and axial force to assure the ductile behavior of the total structure, and the nonlinear static and dynamic analyses.

The design and construction of a 30-story building were also described. It was assured by the analytical investigations that the designed structure would have sufficient margin of the seismic capacity and it would resist severe earthquake motions without any major structural damage.

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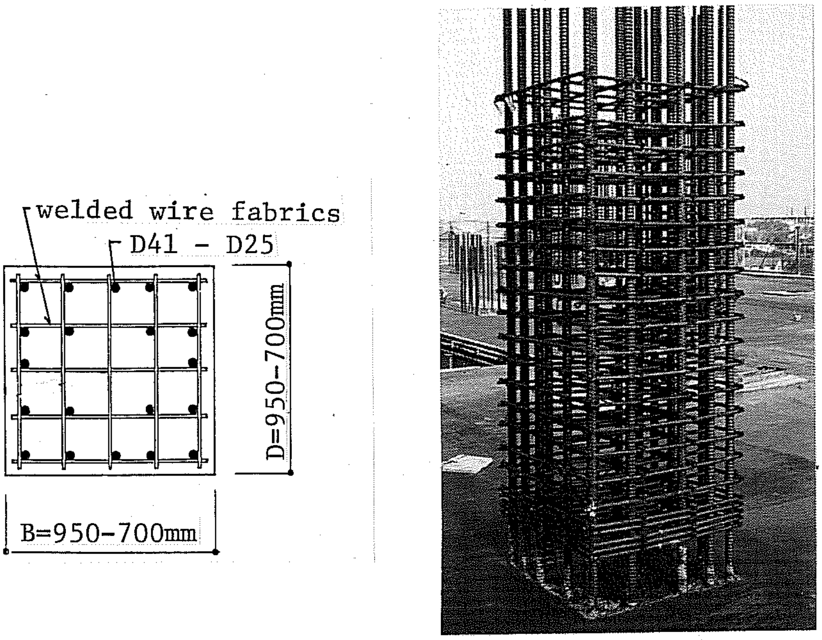
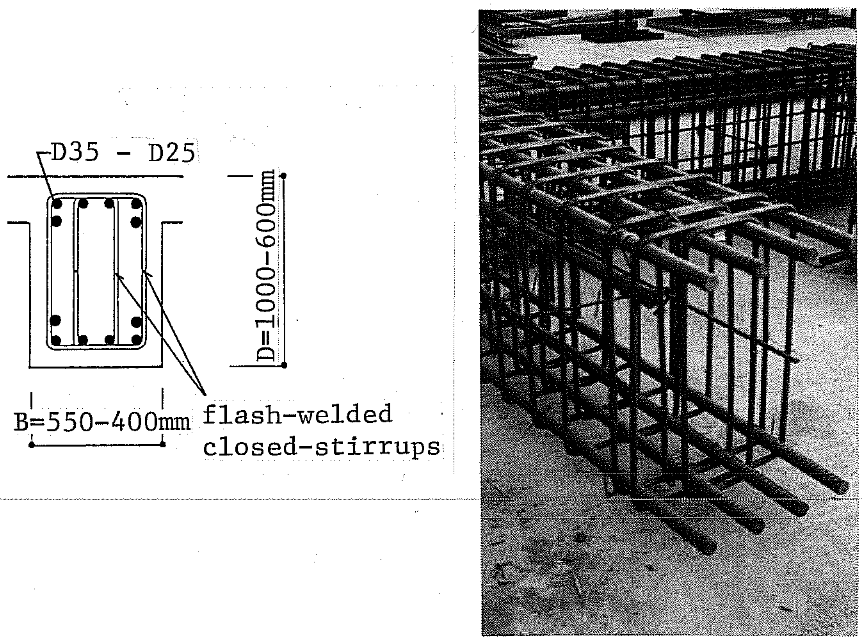


(a) Column



(b) Beam

Fig. 2.1 Typical Cross Section of (a) Column and (b) Beam

Photo 3.1 Perspective View of A 30 Story R/C Building

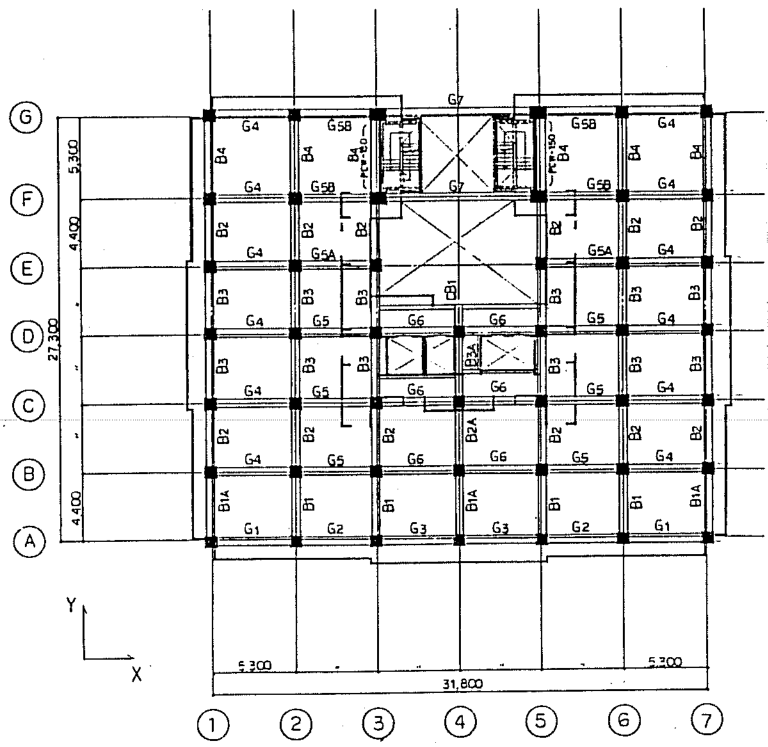


Fig. 3.1 Typical Floor Plan of the Building

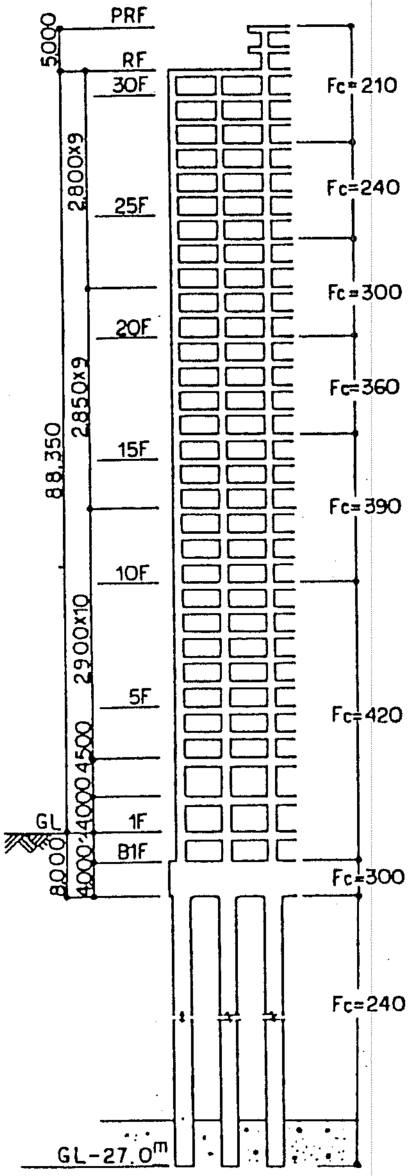


Fig. 3.2 Section of the Building

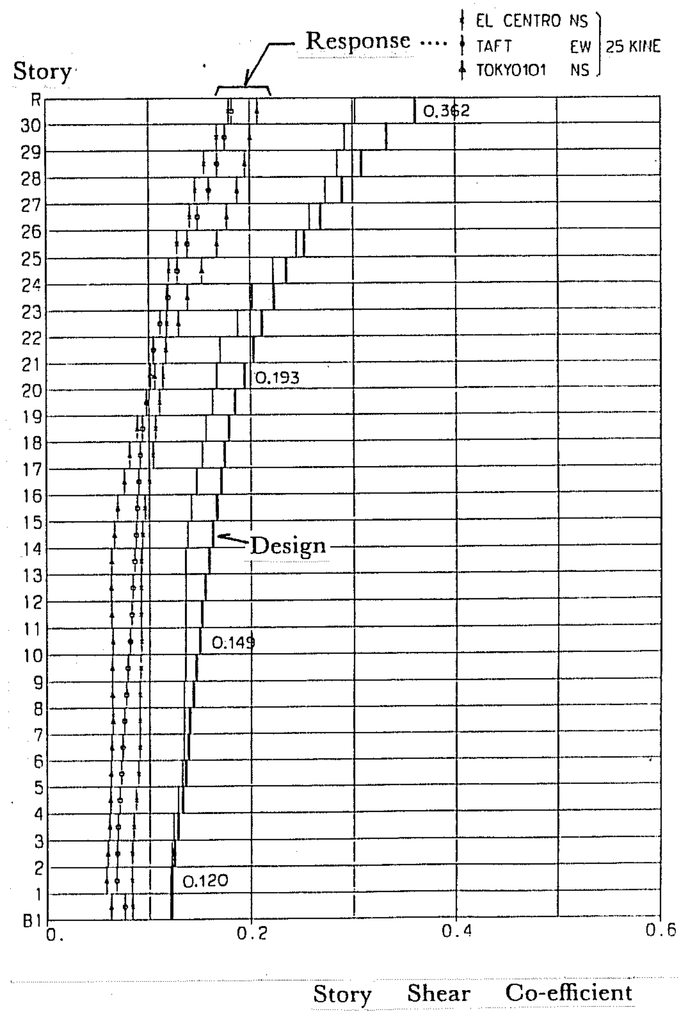
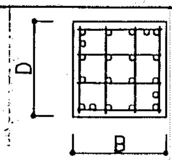
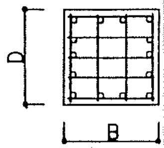
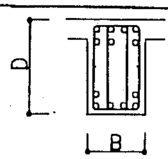


Fig. 3.3 Story Shear Co-efficient Calculated by Preliminary Inelastic Earthquake Response Analysis and Design Story Shear Co-efficient

Table 3.1 Typical Cross Section of Columns and Beams of The Building

Unit: mm

		Exterior Column	Interior Column	Beam
Story	Section			
25	BxD	700×700	750×750	500×700
	Main Bars	14-D29	14-D29	4+3-D29 4+2-D29
	Hoops & Stirrups	4,4-D10@100	5,4-D13@100	4-D13@150
15	BxD	750×750	800×800	550×750
	Main Bars	16-D35	16-D35	4+4-D32 4+3-D32
	Hoops & Stirrups	4,4-D13@125	5,4-D16@100	4-D13@150
5	BxD	850×850	850×850	550×800
	Main Bars	16+4-D38	16-D38	4+3-D35 4+2-D35
	Hoops & Stirrups	4,4-D16@125	5,4-D16@100	4-D16@125
1	BxD	950×950	950×950	550×1000
	Main Bars	20+8-D41	16-D41	4+1-D35 4+1-D35
	Hoops & Stirrups	5,5-D16@80	5,5-D16@80	4-D16@200

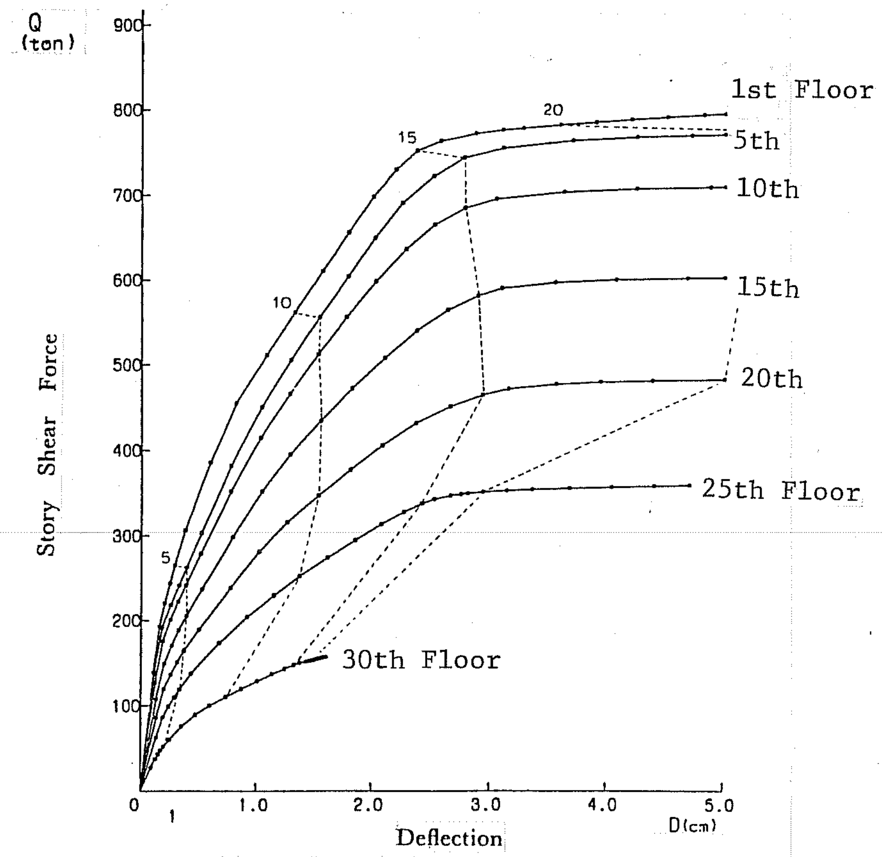


Fig. 3.4 Story Shear Force-Deflection Relationships Under Static Lateral Loads Obtained from In- elastic Frame Analysis

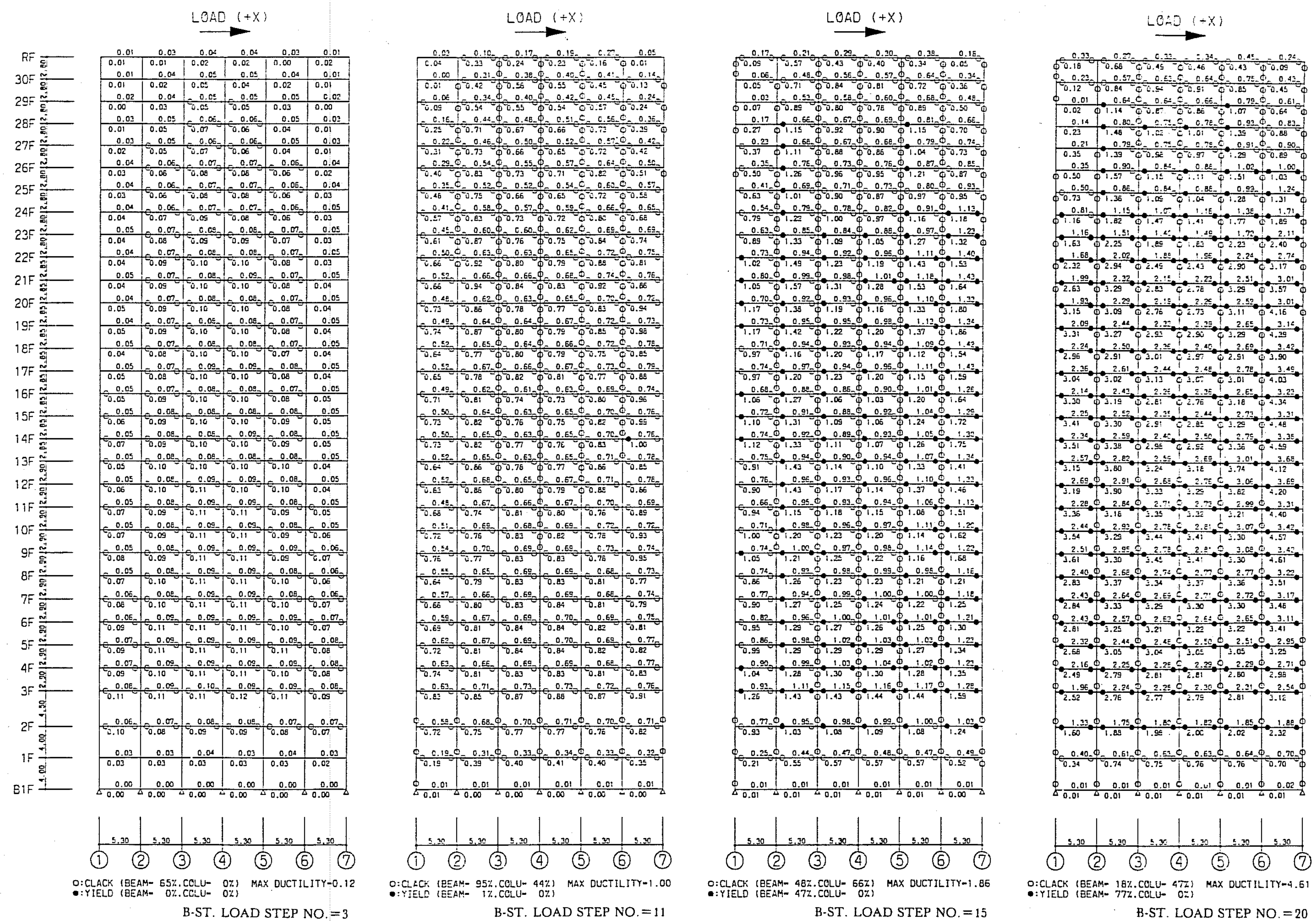
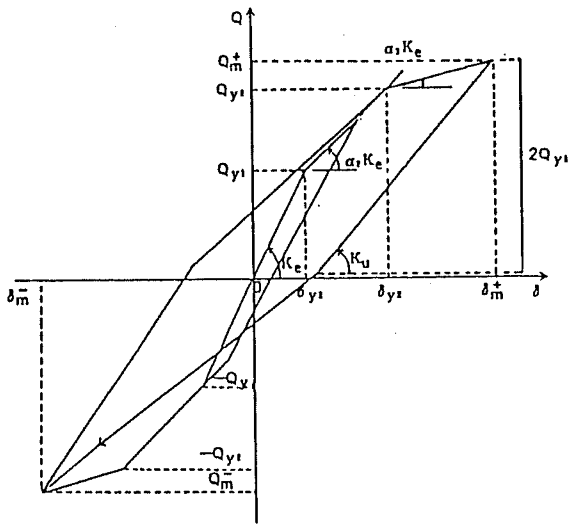


Fig. 3.5 Development of Plastic Hinges Under Static Lateral Loads

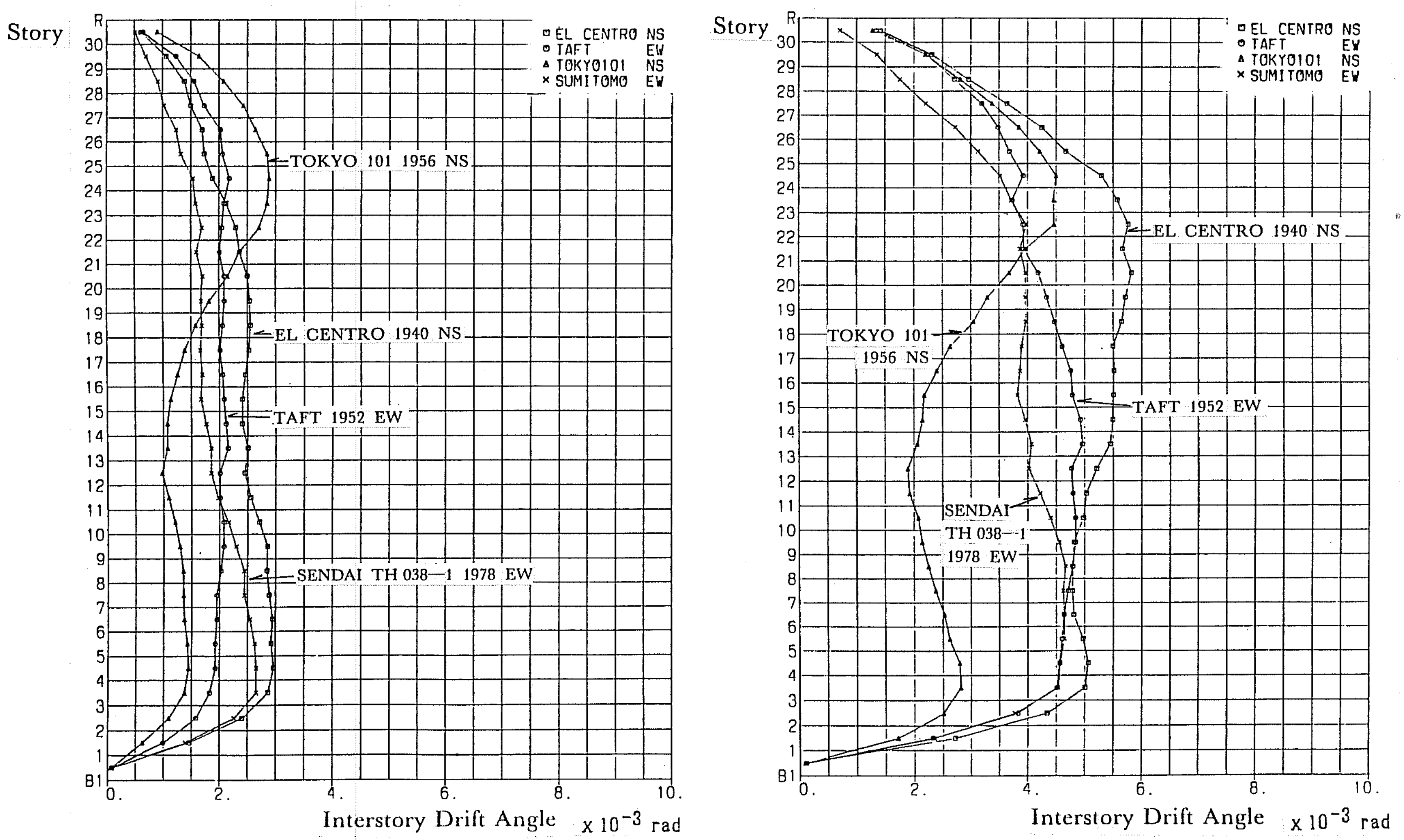


$$\begin{cases} \delta_{y1} \leq |\delta_m| \leq \delta_{y2} & : \quad K_U = \frac{1}{2}(K_e + Q_m/\delta_m) \\ |\delta_m| > \delta_{y2}\ , & : \quad K_U = \frac{1}{2}(1 + K_e \cdot \delta_{y2}/Q_{y2}) \cdot Q_m/\delta_m \end{cases}$$

Fig. 3.6 Hysteresis Rules for the Story Shear Force- Deflection Relationships

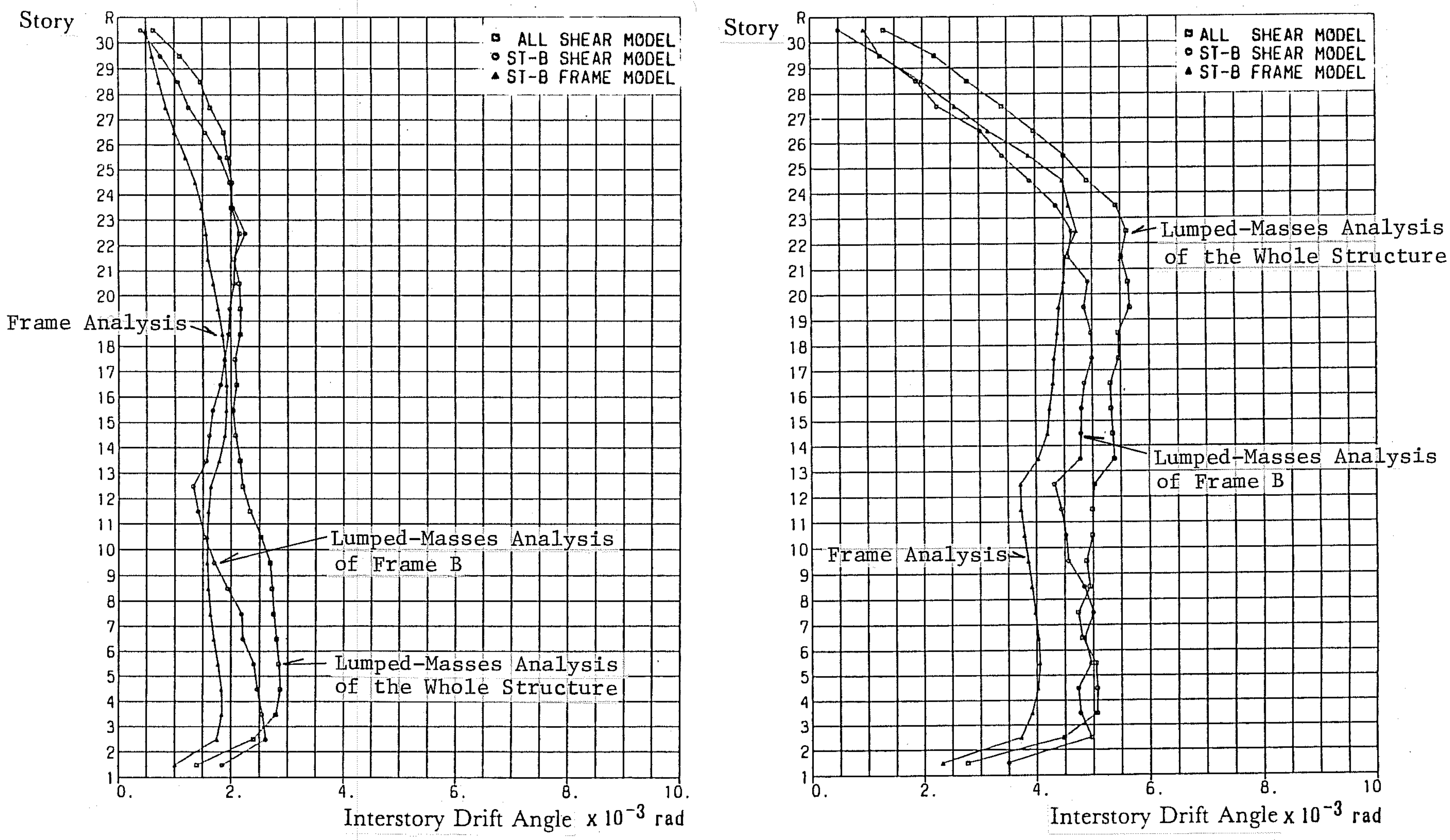
Table 3.2 List of Input Earthquake Ground Motions

Name of Recorded Earthquake Ground Motion	Scaled Max. Acceleration Corresponding to Max. Velocity=25cm/s(25Kine)	Scaled Max. Acceleration Corresponding to Max. Velocity=50cm/s(50Kine)
EL CENTRO 1940 NS	205.8	411.6
TAFT 1952 EW	216.2	432.4
TOKYO 101 1956 NS	256.4	512.8
SENDAI TH038-1 1978 EW	151.7	303.4



(25 Kine) (50 Kine)

Fig. 3.7 Maximum Response of Interstory Drift Angle



(25 Kine)

(50 Kine)

Fig. 3.8 Maximum Respose of Interstory Drift Angle Obtained from Frame Analysis

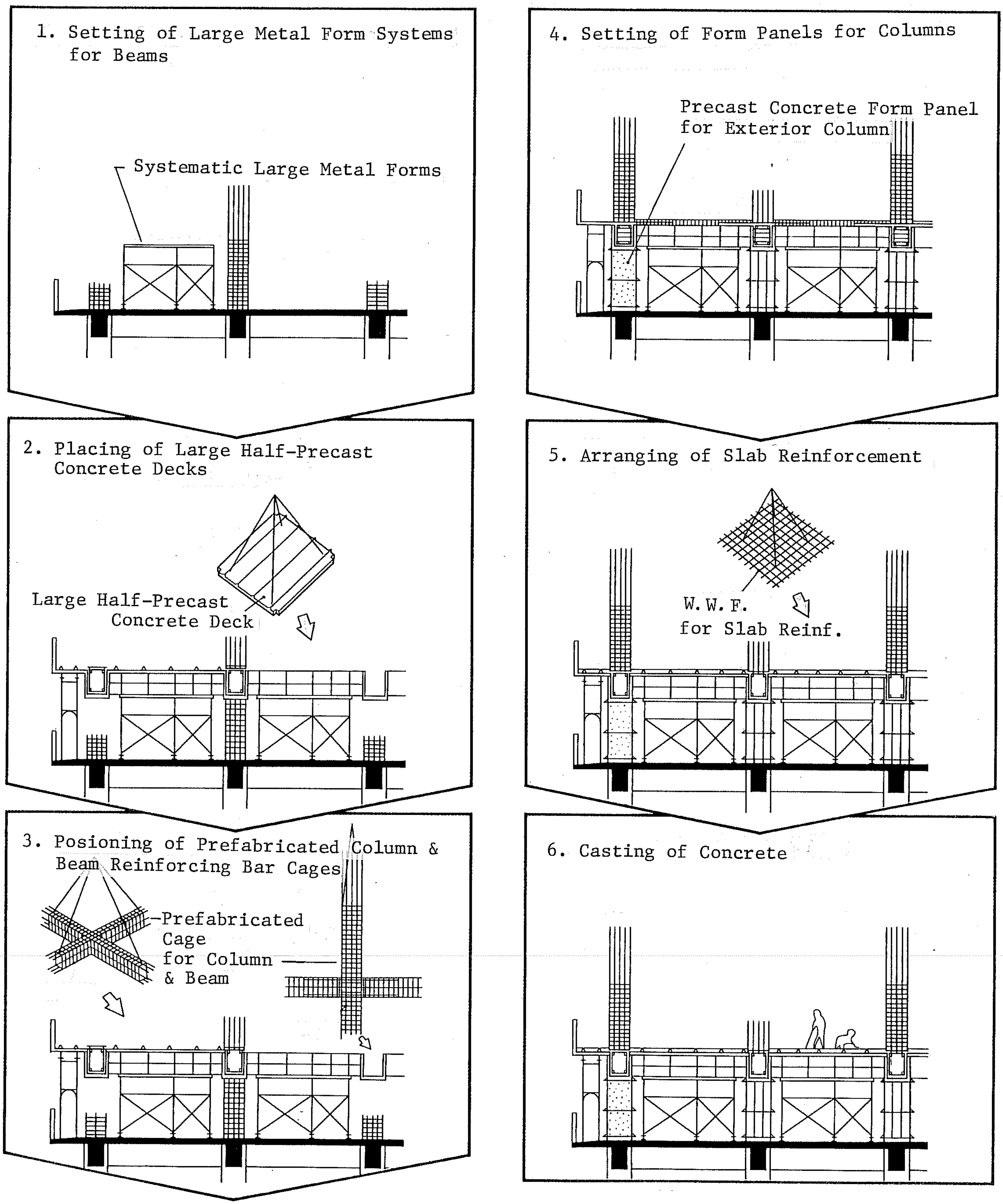


Fig. 4.1 Outline of Construction Procedure

Photo 4.1 A 30-Story R/C Building Under Construction

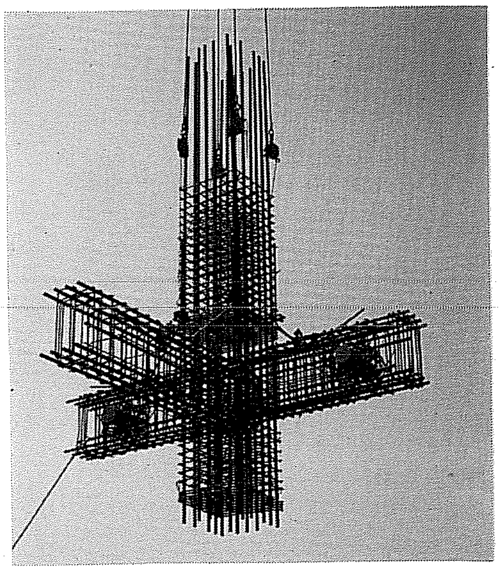


Photo 4.2
Three-Dimensional Prefabricated
Column & Beam Reinforcing Bar Cages

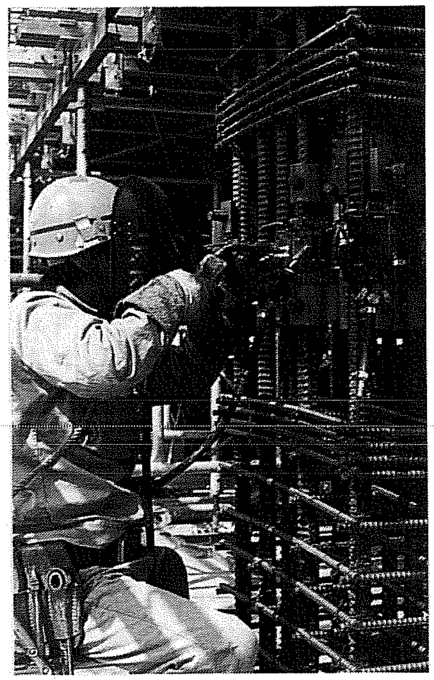


Photo 4.3
Semi-Automated Enclosure Welding
of Column Vertical Steel Bars

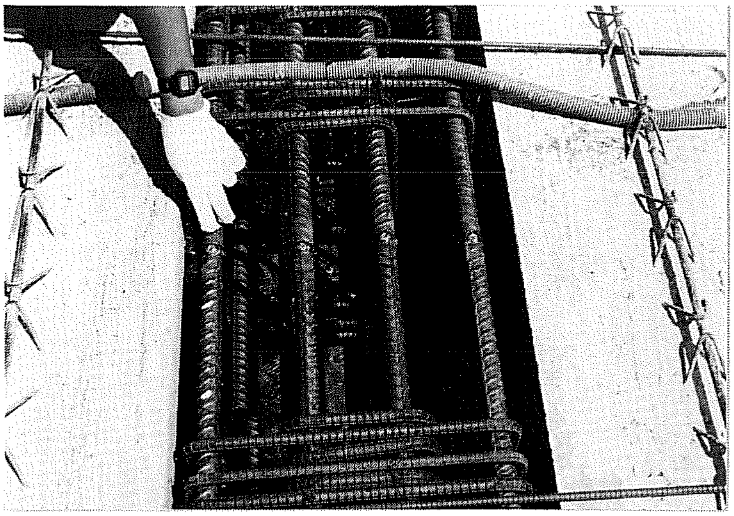
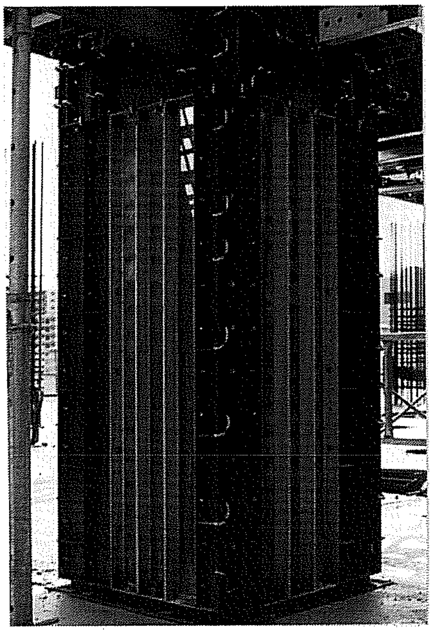


Photo 4.4
Beam Main Bars Jointed at Mid-Span Using Semi-Automated Enclosure Welding



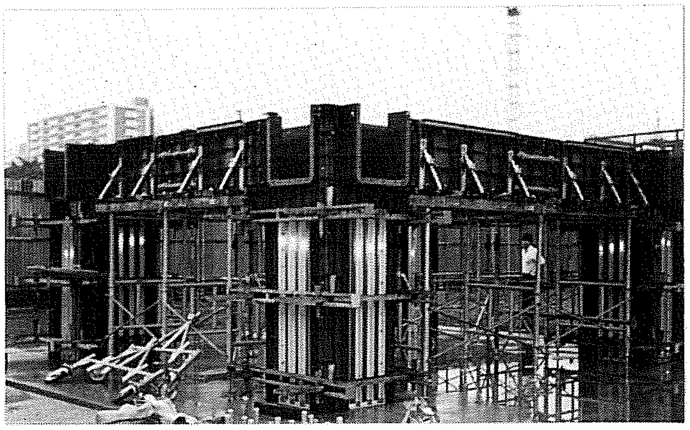


Photo 4.5 Systematic Large Metal Forms for Beams

Photo 4.6 Semitransparent Plastic Forms for Columns

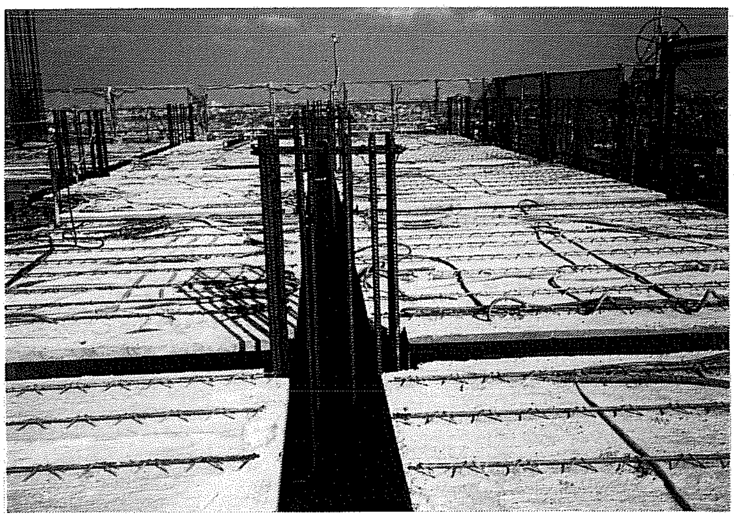


Photo 4.7
Large Half-Precast Concrete Decks

PART 2: EXPERIMENTAL STUDIES ON COLUMNS AND FRAMES FOR TALL REINFORCED CONCRETE BUILDINGS

1. INTRODUCTION

A series of experimental studies on columns and frames were conducted to obtain design guidelines for tall reinforced concrete framed structures. Prototype buildings of 25 to 30 stories were studied to establish the experimental program.

The experimental program consisted of a uni-axial compression test of columns, two series of tests of columns subjected to simulated seismic loads to failure, a test of subassemblies representing interior and exterior frames in the bottom story of a prototype building, and a test of beam-column joints. The summary of results obtained from these tests are described below.

2. UNI-AXIAL COMPRESSION TEST OF COLUMNS

As a preliminary study, short columns having special types of lateral reinforcement such as those of welded-wire fabrics and ultra-high strength bars and different amounts of lateral reinforcement, as shown in Table 2.1, were tested under monotonic uni-axial compression to failure.

As shown in Fig. 2.1, the effect of the amount of lateral reinforcement on the behavior of the columns was clearly observed. Their ductility was significantly improved with the increased amount of lateral reinforcement. The ductility was also affected by the type of lateral reinforcement and it was indicated that the columns laterally reinforced by welded-wire fabrics or ultra-high strength bars could be provided with larger ductility than those of ordinary reinforcement with standard hooks.

3. SIMULATED SEISMIC LOADING TEST OF COLUMNS

3.1 Series-I Test

A total of 18 column specimens, shown in Fig. 3.1, were tested under simulated loads to failure. The main objective of the test was to investigate the seismic behavior of columns laterally reinforced by welded-wire fabrics with special emphases on the ultimate flexural and shear capacities, ductility, and energy-dissipating capability. Main variables selected for the experiment were shear-span ratio (a/D), ratio of axial compression to the compressive strength of concrete (η), amount of lateral reinforcement (Pw), and type of lateral reinforcement (welded-wire fabric, ultra-high strength bar or plain bar). Figure 3.2 shows the utilized loading apparatus for the test.

Comparing the measured load-displacement hysteresis loops, the effect of the previously specified parameters on the overall behavior of the columns was clearly observed. As shown in Fig. 3.3., the columns laterally reinforced by welded-wire fabrics were superior to the columns with the same amount of lateral reinforcement of plain bars, with respect to the energy-dissipating capability, displacement ductility and the lateral load-carrying capacity. The result also showed that the ductility of the columns was significantly affected by the level of shear stress and axial force as well as the amount of lateral reinforcement.

Based on these results, it was recommended that the lateral reinforcement in columns should be increased in proportion to the level of both shear and axial stresses attained in columns so as to provide large displacement capability as shown in Fig. 3.4.

3.2 Series-II Test

Five 1/2.5-scale column specimens representing lower story columns of a 30-story framed structure were tested as shown in Fig. 3.5 and Table 3.1. All the specimens were laterally reinforced again by welded-wire fabrics. The main objectives of this test were to investigate the ductility and the shear capacity of such columns under high axial compression force corresponding to the upper limit for the design of exterior columns specified in Sec. 2.3.3 of Part 1, and the effect of biaxial loading on the behavior of the columns.

Specimens Nos. 1, 3 and 4, having a shear-span ratio of 2.5, were designed so as to behave in a ductile manner based on the guidelines described in previous Sec. 2.3 of Part 1. The other two specimens, Nos. 2 and 5, having a shear-span ratio of 1.25, were provided to investigate the ultimate shear capacity under high axial compression.

For all specimens, except No. 3, constant and high axial compression force (the nominal compressive stress of 280 kg/cm^2 (27 Mpa, 4000 psi) or 65% of compressive strength of concrete) was applied during the test. The axial force corresponding to the nominal compresssive stress of 30% concrete strength was applied to specimen No.3. Specimens Nos. 4 and 5 were subjected to cyclic lateral forces in the diagonal direction.

Figure 3.6 shows the measured lateral load vs displacement hysteresis curves. As was expected, the columns exhibited ductile behavior until the displacement of more than 3% without any drop in strength even under the high axial compression. It was also indicated that the columns had sufficient shear capacity. Only the minor effect of loading direction was detected.

4. TEST OF SUBASSEMBLIES

Two 1/3.5-scale beam-column subassemblies, shown in Fig. 4.1, representing interior and exterior frames in the bottom story of a prototype building were provided. The design of beams, columns and beam-column joints were based on the previously described guidelines except for some details for stirrups and ties in beams and for hoops in columns. Figure 4.2 shows the test setup for the interior frame subassembly. The constant axial stress of 40% concrete strength was applied to the column of the interior frame assembly while the axial force was alternately varied from tension to compression (-22% to 60% concrete strength) during the test of exterior subassembly.

Both frames developed plastic hinges at the beam ends and at the bottom of the column, as designed, and were capable of maintaining stable hysteresis loops up to the story drift of 3% as shown in Fig. 4.3. However, the buckling of column main bars, some of which were not restrained by lateral reinforcement, was observed in both the interior and exterior columns at the story drift of 3% and 2.5%, respectively. It was pointed out that each longitudinal bar must be restrained by lateral reinforcement against buckling.

5. TEST OF BEAM-COLUMN JOINTS

Four half-scale interior beam-column subassemblies representing a lower part of a 30-story framed structure were tested. The details of the specimens are shown in Table 5.1 and Fig. 5.1.

For all the specimens except No. 4, the design of joint for shear was based on the previously described provisions which were established extending the empirical formula proposed by Professor Kamimura. The joint of specimen No. 4 had two times the amount of shear reinforcement, as much as required by the formula. Deformed bars with the nominal diameter of 16 and 19 mm [D16(#5), D19(#6)] were provided as main bars in beams, providing the length of 21 and 25 times the bar diameter for the development of bars passing through the joint panel.

All the specimens exhibited ductile behavior up to the story drift of 8 to 10% as shown in Fig. 5.2. The joints had sufficient shear capacity and no significant pinching effect was observed on the hysteresis loops.

6. CONCLUDING REMARKS

The results obtained from a series of experimental studies for tall reinforced concrete framed structures are summarized as follows:

(1) The ductility of column under uni-axial loading was significantly enhanced when laterally confined with welded-wire fabrics or ultra-high strength bars. The columns laterally reinforced by welded-

wire fabrics or ultra-high strength bars also exhibited very ductile behavior under seismic loading.

(2) The amount of lateral reinforcement strongly affected the ductility of columns under both uni-axial compression and seismic loading. The ductility was significantly improved with the increased amount of lateral reinforcement.

(3) The levels of nominal shear stress and axial stress attained in a column were also an important factor in controlling the ductility of columns under seismic loading. The increased amount of lateral reinforcement in proportion to the level of both shear and axial stresses improved the ductility of columns.

(4) The columns laterally reinforced with welded-wire fabrics and designed based on the previously described provisions behaved in a very ductile manner up to the displacement of more than 3% even under the high axial compression of the nominal stress level of 65% concrete strength.

(5) Both interior and exterior frame subassemblies representing the bottom story of a prototype building were capable of maintaining stable hysteresis loops up to the story drift of 3%. However, it was pointed out that to obtain larger ductility each longitudinal bar of a column must be restrained by lateral reinforcement against buckling.

(6) The interior beam-column subassemblies, designed based on the provisions in Part 1 also exhibited very ductile behavior up to the story drift of 8 to 10% without any drop in strength or significant pinching effect.

From the results mentioned above, the following recommendations are suggested for the design guidelines described in Part 1.

(1) To use welded-wire fabrics as lateral reinforcement of columns.

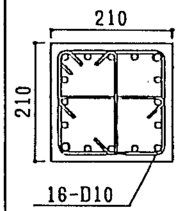
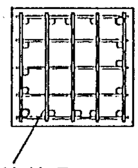
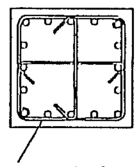
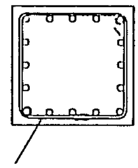
(2) To arrange lateral reinforcement of a column so as to restrain each longitudinal bar against buckling.

(3) To limit the level of axial force attained in a column.

(4) To determine the amount of lateral reinforcement in a column in proportion to the level of both shear and axial stresses.

Thus is was concluded that the framing members, beams, columns and beam-column joints designed according to the previously described guidelines in Part 1 would have sufficient margin of the seismic capacity against the displacement specified in the criteria presented in Part 1 and the displacement induced by the design earthquakes.

Table 2.1 Column Specimens (Uni-Axial Compression Test)

Specimen		S06	S12	M06	M12	U06	U12	BP12
Section								
Transeverse Reinf.	Type	Plain bar		Welded wire fabric		Ultra-high strength bar		Band Plate
	Amount	$\phi 5@50$ Pw=0.56%	$\phi 5@25$ Pw=1.12%	$\phi 4@50$ Pw=0.60%	$\phi 4@25$ Pw=1.20%	$\phi 5@50$ Pw=0.56%	$\phi 5@25$ Pw=1.12%	PL-25×2.3@50 Pw=1.10%

Section BxD=210x210 H=618

Longitudinal Reinf. 16-D10 Pq=2.58%

Materials

Deformed bar D10 : σy=4000kg/cm², σmax=5803kg/cm²

Plain bar $\phi 5$: σy=4207kg/cm²*, σmax=4505kg/cm²

W.W.F. $\phi 4$: σy=5535kg/cm²*, σmax=5661kg/cm²

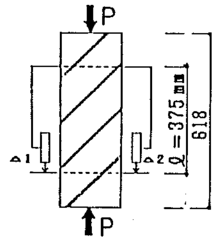
Shear strength of welded points 18.8kg/cm²

Ultra-high strength bar $\phi 5$: σy= 13970kg/cm²* σmax= 15281kg/cm²

Cocrete $c\sigma B$=247 ~277 kg/cm²

$c\sigma t$=19.1kg/cm²

E1/3cσB=2.19~2.34kg/cm²



$\varepsilon = \frac{\Delta 1 + \Delta 2}{2 \times 375}$: Average Axial Strain

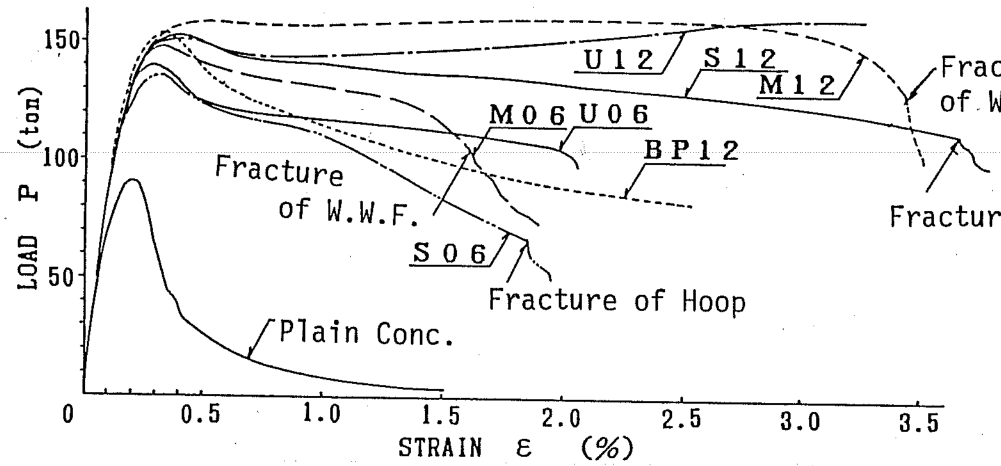


Fig. 2.1 Column Axial Load-Average Strain Curves

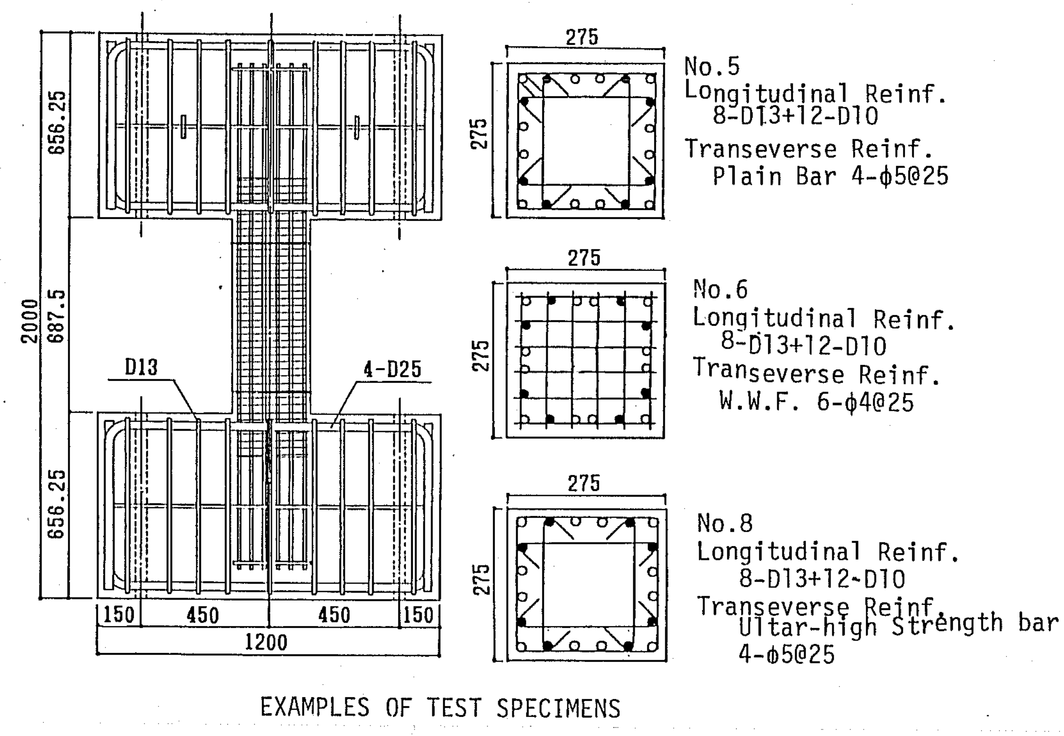


Fig. 3.1 Details of Column Specimens Subjected to Simulated Seismic Loads (Series-I Test)

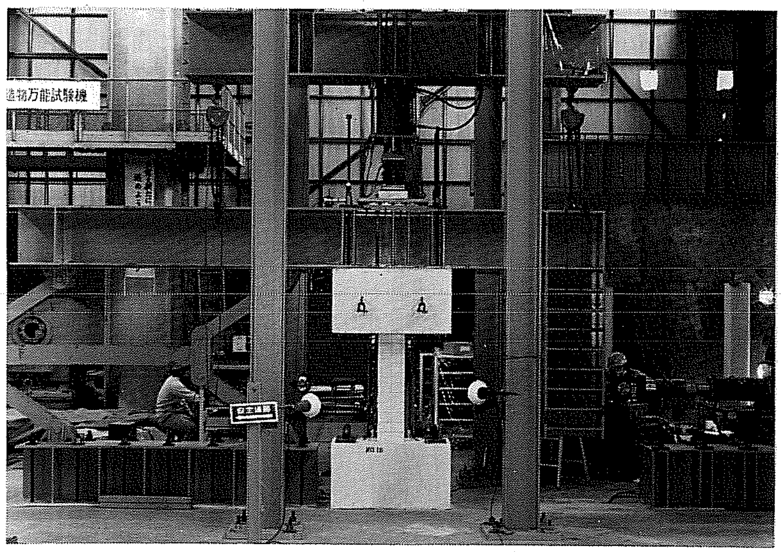
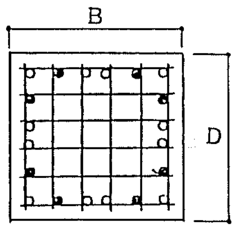
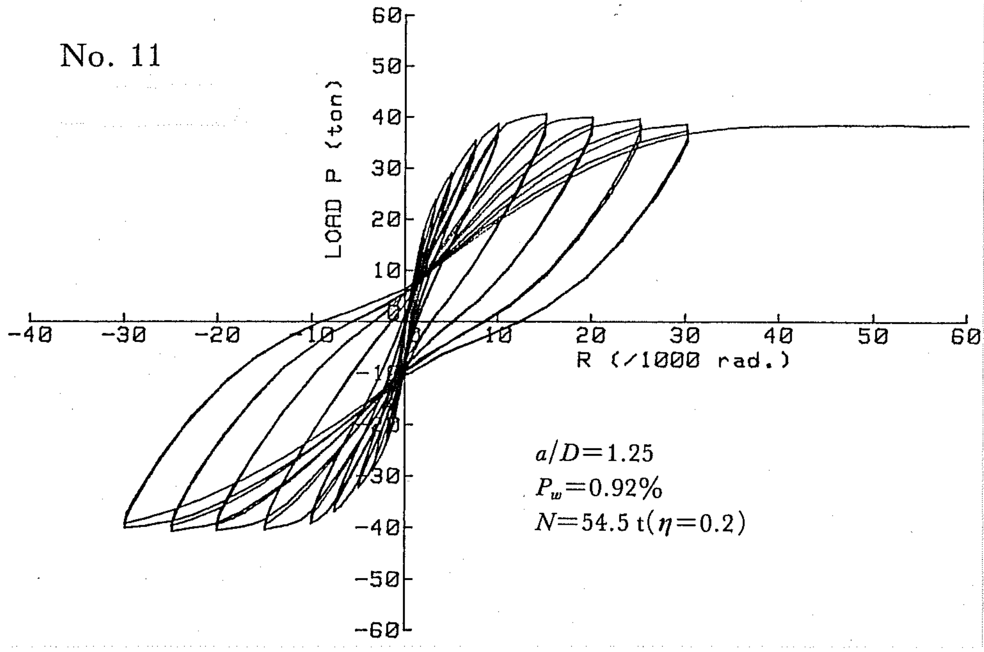


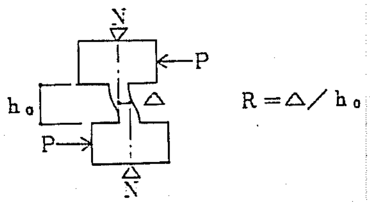
Fig. 3.2 Loading Apparatus for Column Seismic Loading Tests

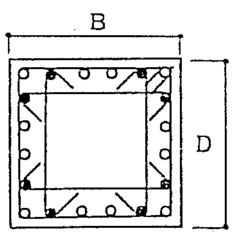


BxD=275x275mm
Longitudinal Reinf.
8-D13+12-D10
Transeverse Reinf.
W.W.F. 6-4ϕ@30
ho/2D=1.25, N=54.5ton

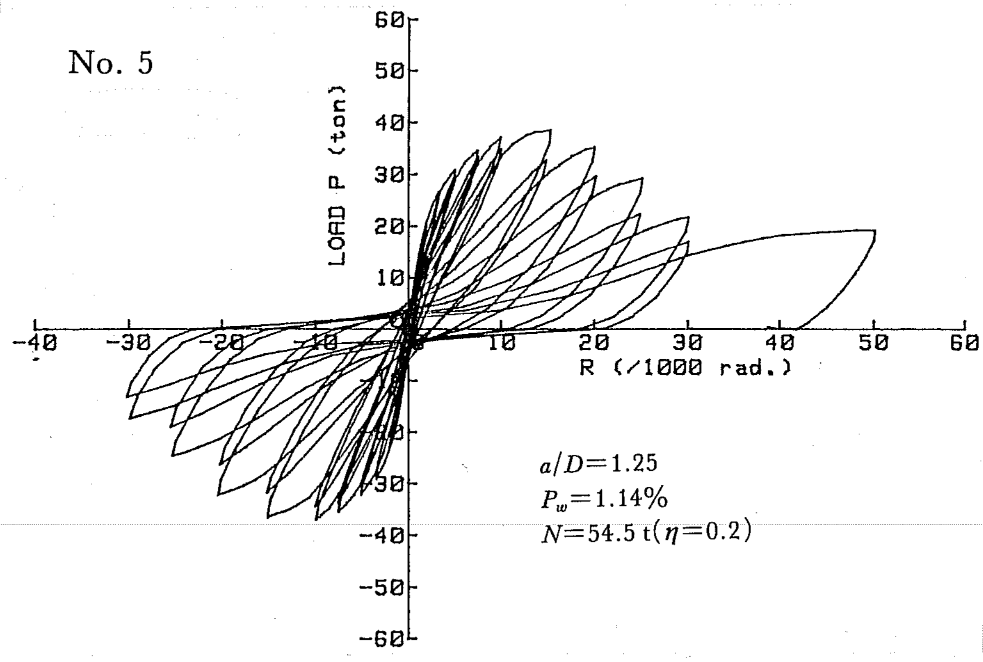


Specimen No.11 Column with W.W.F.





BxD=275x275mm
Longitudinal Reinf.
8-D13+12-D10
Transeverse Reinf.
Plain bar 4-ϕ5@25
ho/2D=1.25, N=54.5ton



Specimen No.5 Column with hoop & ties of plain bar

Fig. 3.3 Comparison of Measured Horizontal Load-Displacement Hysteresis Loops

Note Ru : lateral displacement capacity, or lateral displacement when the lateral load is reduced to 80% of the maximum lateral load.

Pw : ratio of lateral reinforcement.

wσy : yield strength of lateral reinforcement.

τmu.cal. : nominal shear stress calculated from the flexural strength at the both ends of the column specimen.

η : ratio of nominal axial compression stress to the concrete compressive strength.

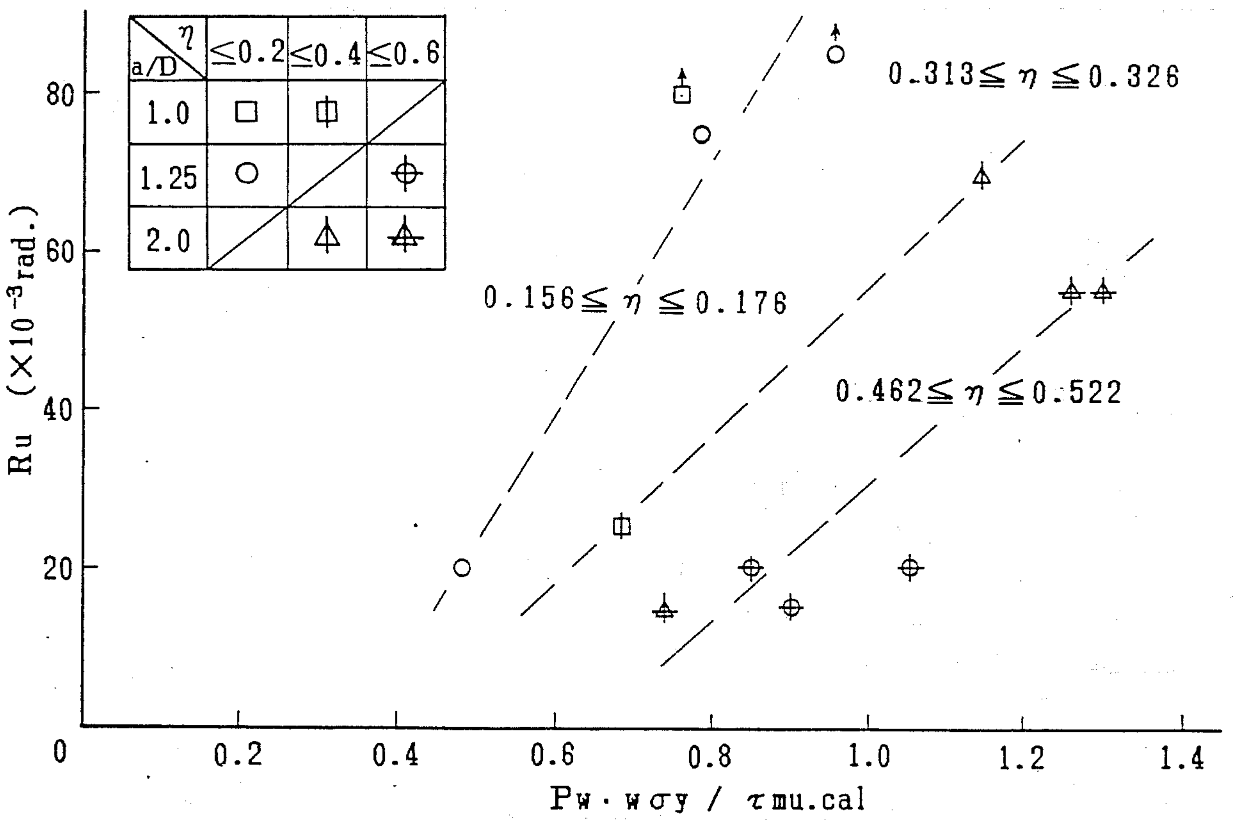


Fig.3.4 Lateral Displacement Capacity versus Amount of Lateral Reinforcement in The Columns

Table 3.1 Column Specimens for Simulated Seismic Loading Test (Series-II)

Specimen	B × D (cm)	Main Bars (SD35)	Pg (%)	Hoops (W.W.F.) (SD30)	Pw (%)	a/D	η	Note
No.1	35 × 35	20-D16	3.25	5,5-D6@37.5	1.22	2.5	0.65	
No.2	35 × 35	20-D16	3.25	5,5-D6@45.0	1.02	1.25	0.65	
No.3	35 × 35	16-D16	2.60	5,5-D6@37.5	1.22	2.5	0.30	
No.4	35 × 35	20-D16	3.25	5,5-D6@37.5	1.22	2.5	0.65	Diagonal Loading
No.5	35 × 35	20-D16	3.25	5,5-D6@45.0	1.02	1.25	0.65	Diagonal Loading

P_g : ratio of total longitudinal reinf.

B,D: width & depth

P_w : ratio of transeverse reinf.

a/D: shear span ratio

η : ratio of axial comp. stress to comp. strength of concrete

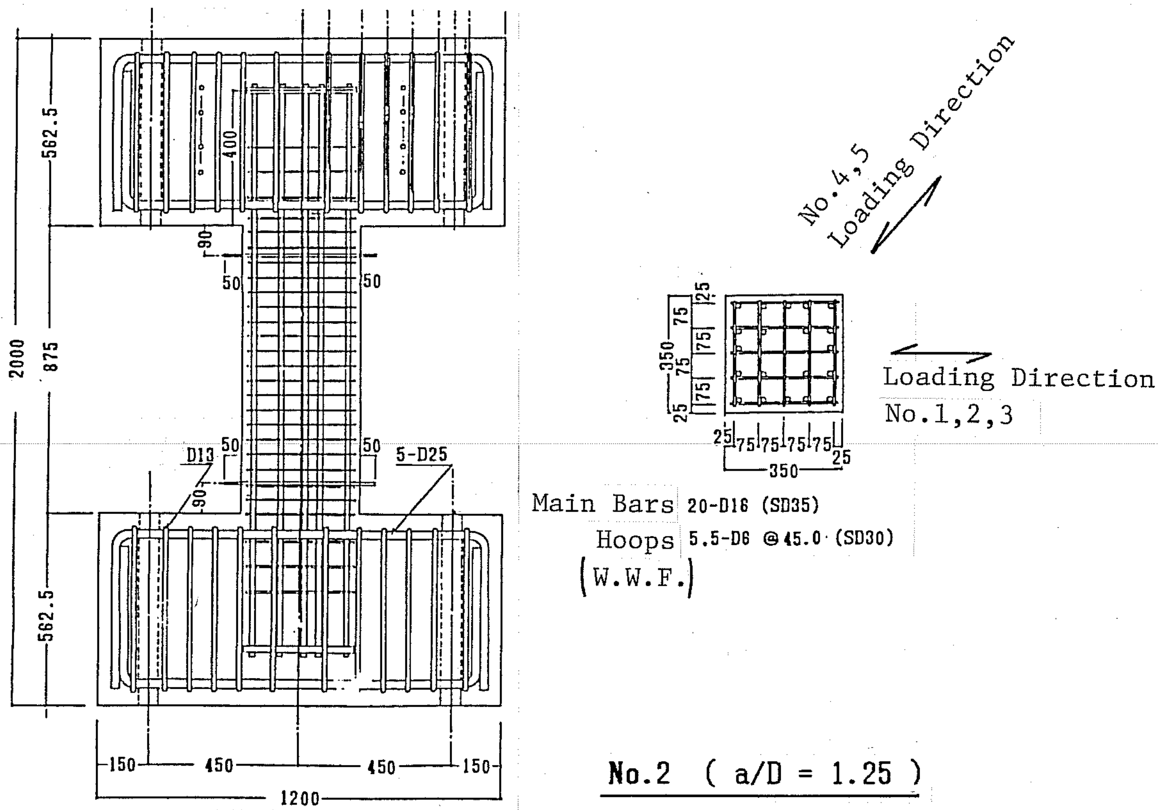


Fig.3.5 Details of Column Specimen for Simulated Seismic Loading Test (Series-II)

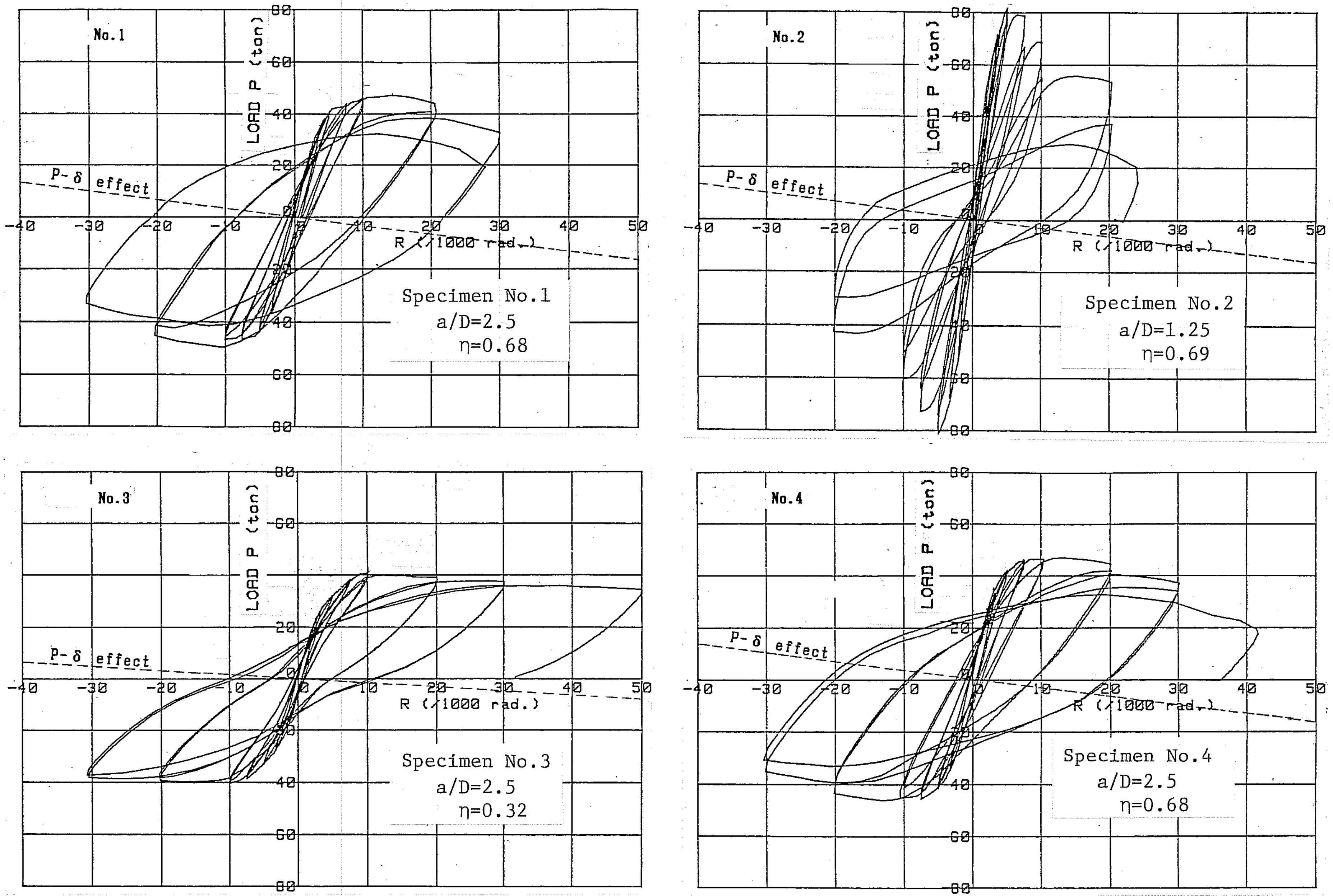
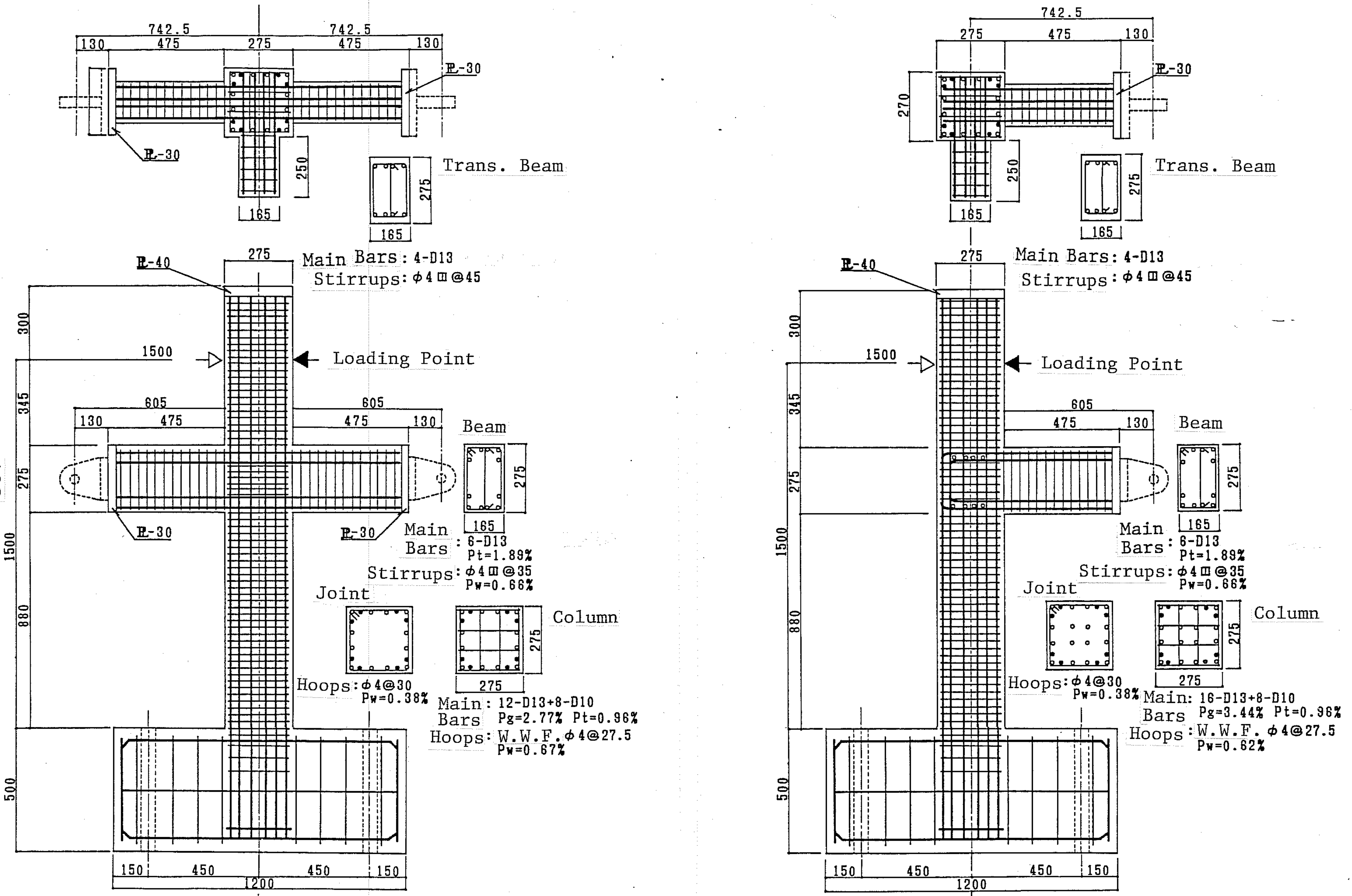


Fig.3.6 Measured Horizontal Load-Displacement Hysteresis Loops (Series-II Test)



(a) Interior Subassembly

(b) Exterior Subassembly

Fig. 4.1 Test Specimens of Interior and Exterior Beam-Column Sub-Assemblies

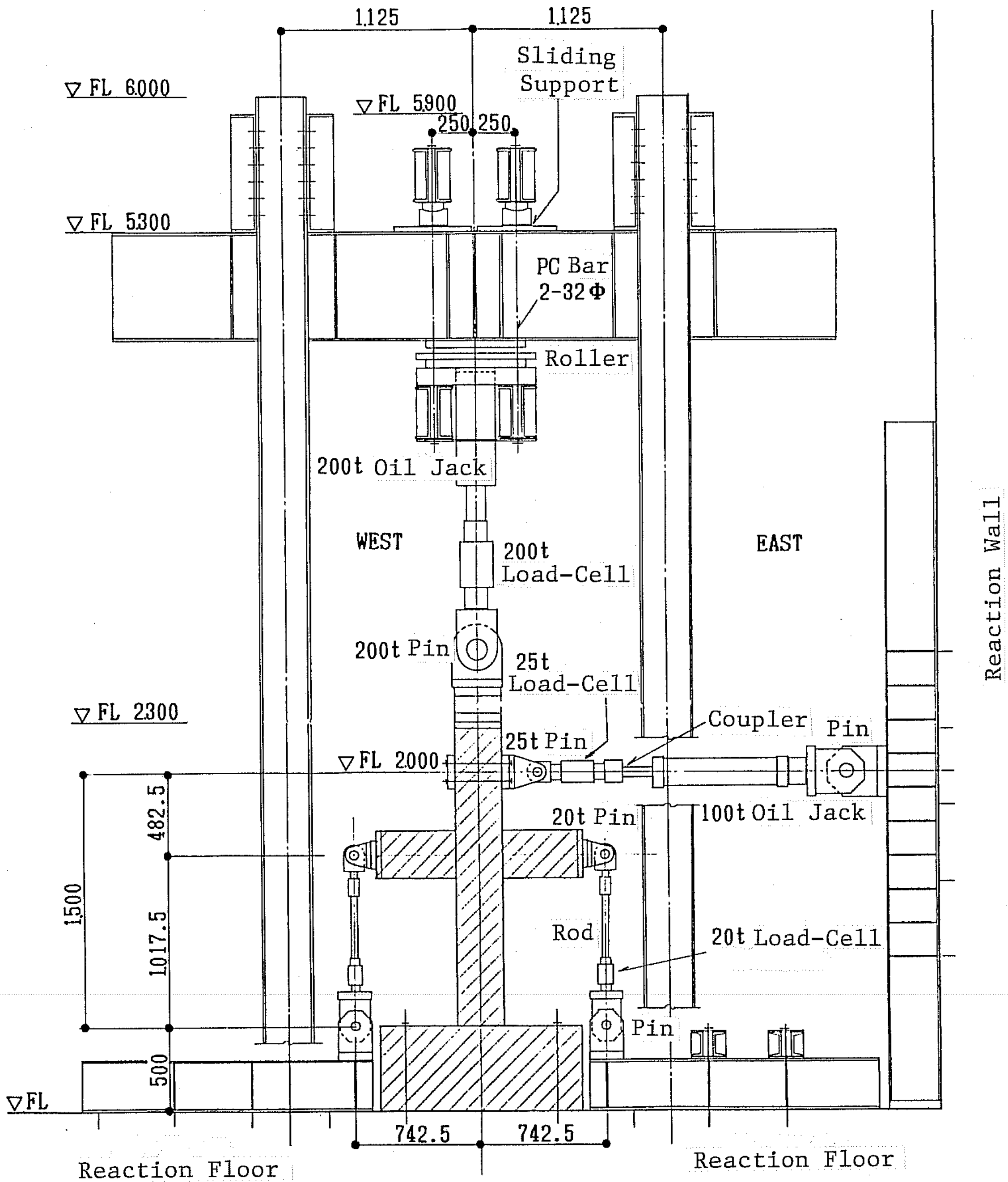
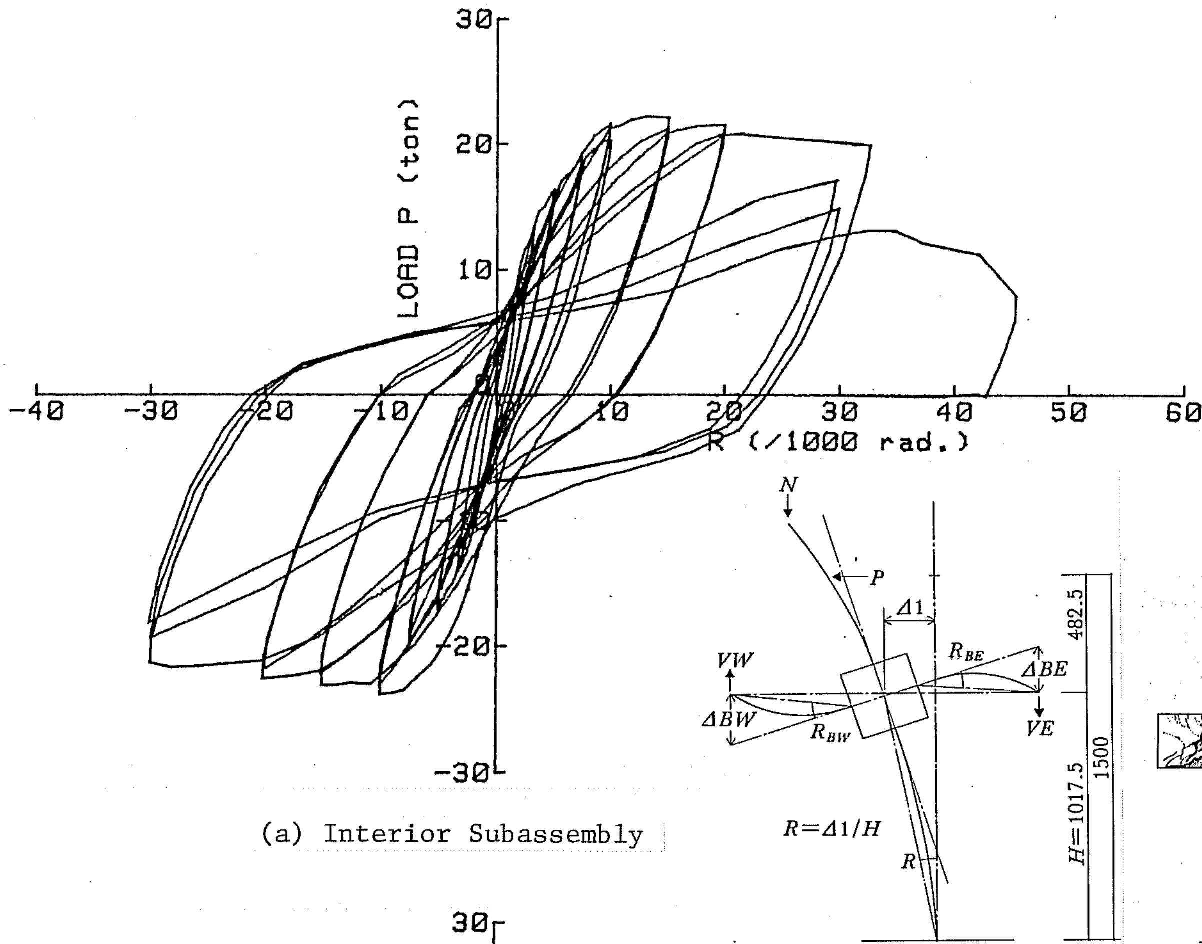
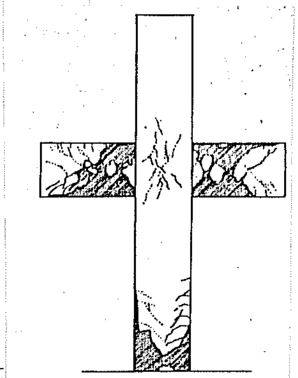


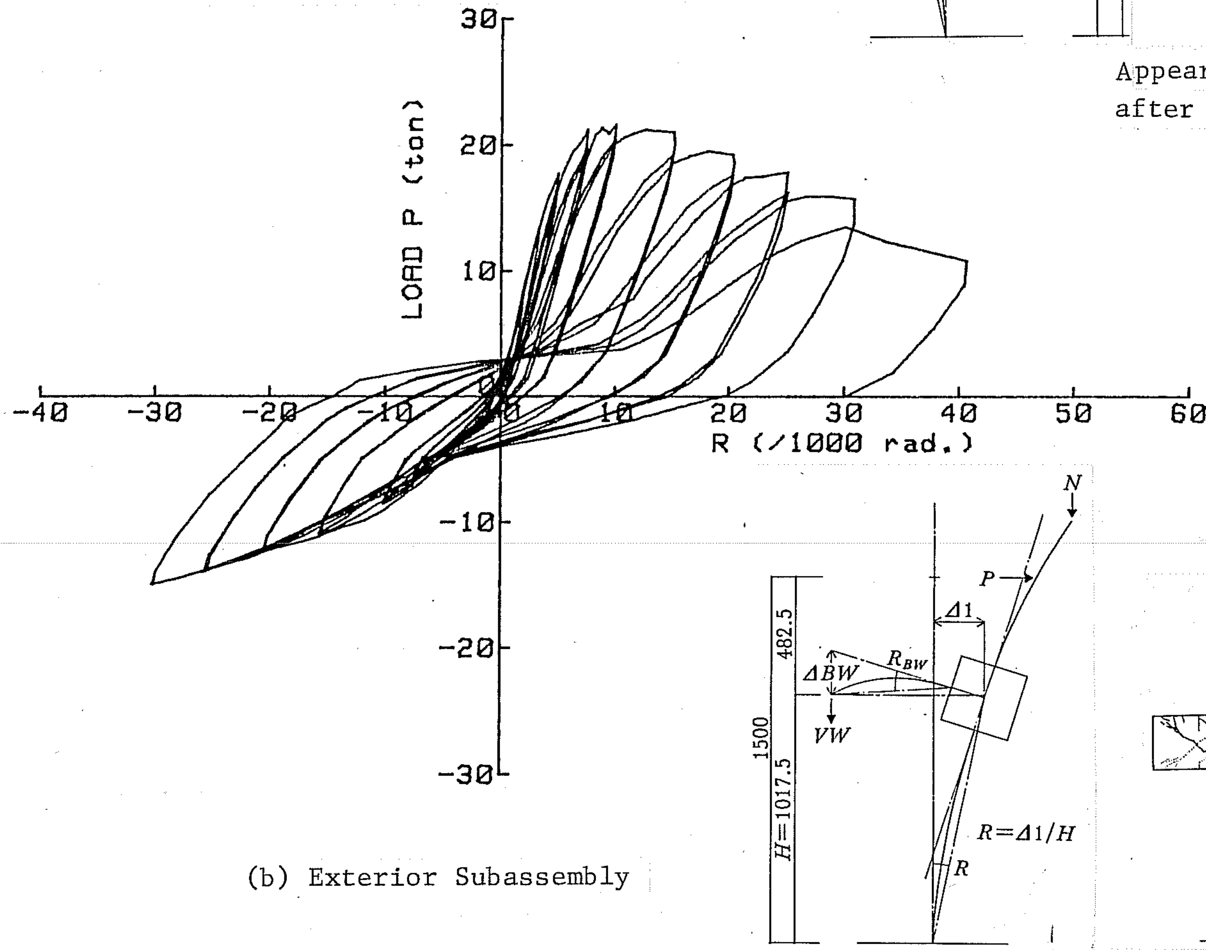
Fig. 4.2 Loading Apparatus for Interior Frame Sub-Assembly



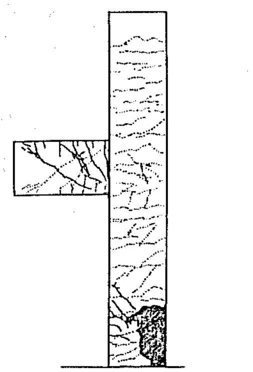
(a) Interior Subassembly



Appearance of Specimen after Test



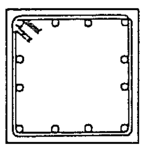
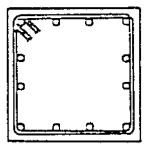
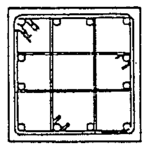
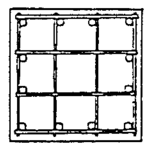
(b) Exterior Subassembly

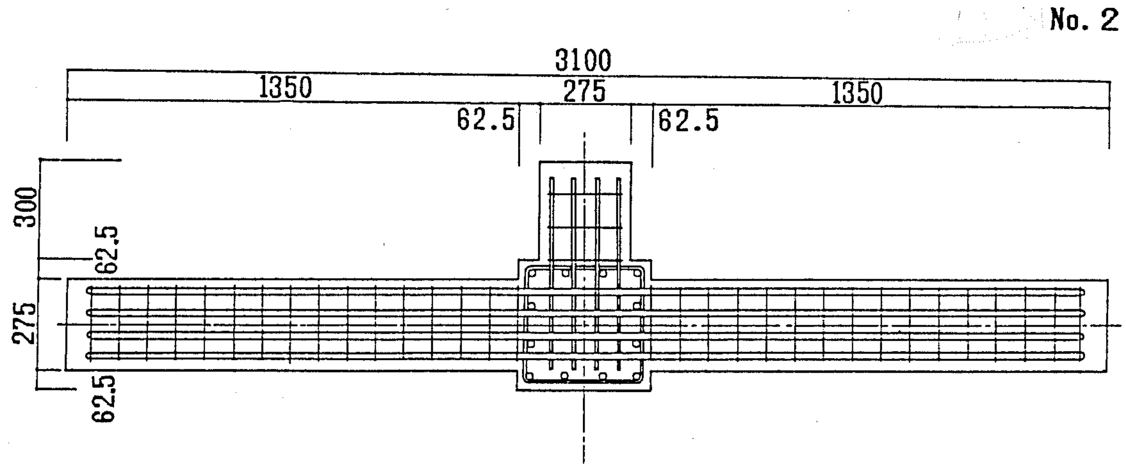


Appearance of Specimen after Test

Fig. 4.3 Measured Lateral Load-First Story Drift Curves

Table 5.1 Beam-Column Joint Specimens

	Specimen	NO.1	NO.2	NO.3	NO.4
Column	B x D	40×40cm			
	total longt. reinf. ratio Pg	2.15% (12−D19)			
	ten. reinf. ratio Pt	0.78% (4−D19)			
	hoop reinf. ratio Pw	0.79% (4−D10@90)			
Beam	B x D	27.5×37.5cm			
	ten. reinf. ratio Pt	1.56% (2-D19+4-D16)	1.81% (2-D19+5-D16)	2.06% (2-D19+6-D16)	1.81% (2-D19+5-D16)
	shear reinf. ratio Pw	1.03% (4−D10@100)		1.21% (4−D10@85)	1.03% (4−D10@100)
Joint	nominal shear stress	88kg/cm²	100kg/cm²	113kg/cm²	100kg/cm²
	shear reinf. ratio Pw	0.45% (2−D10@85.5)	0.77% (2−D10@42.5)	1.55% (4−D10@42.5)	1.55% (4−D10@42.5)
	type of joint hoops	 135° hook closed hoop	 135° hook closed hoop	 135° hook closed hoop & tie	 welded wire fabric



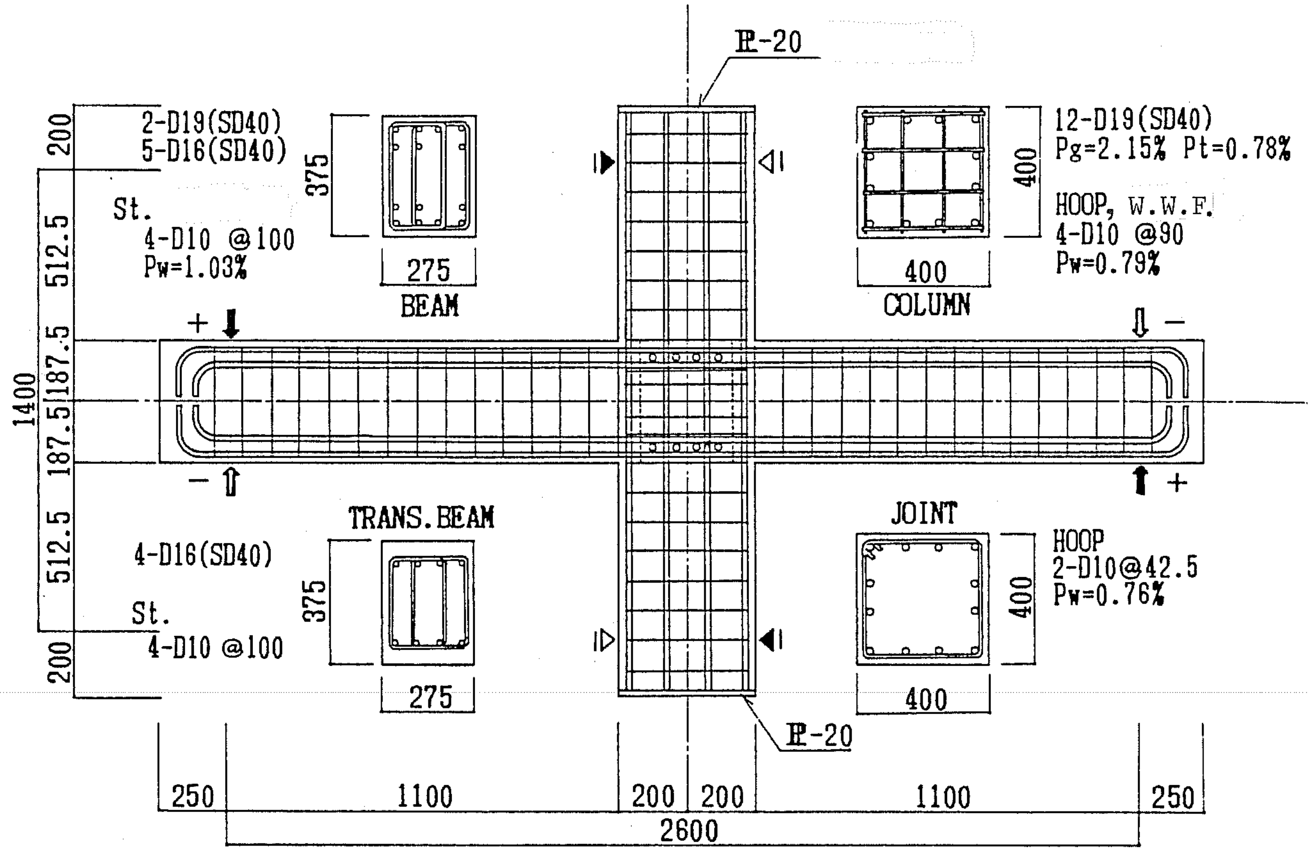
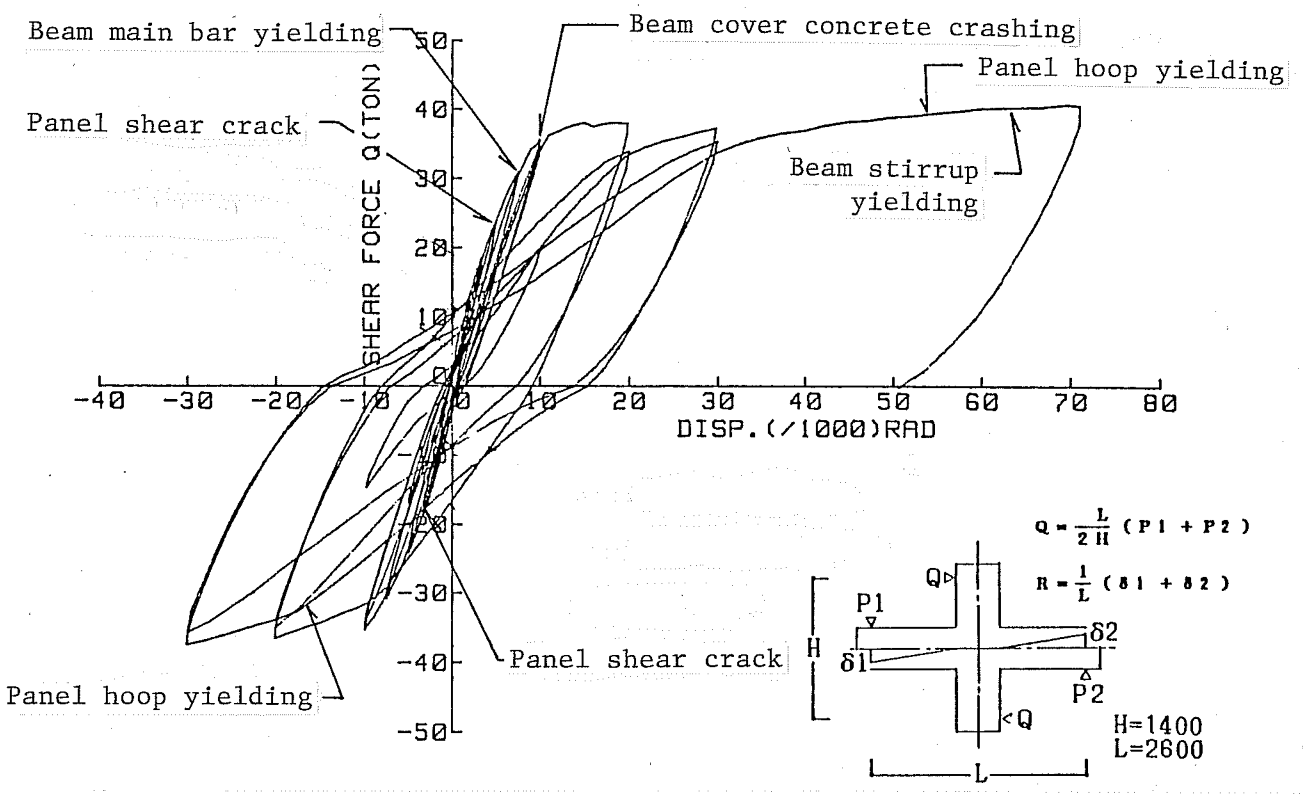


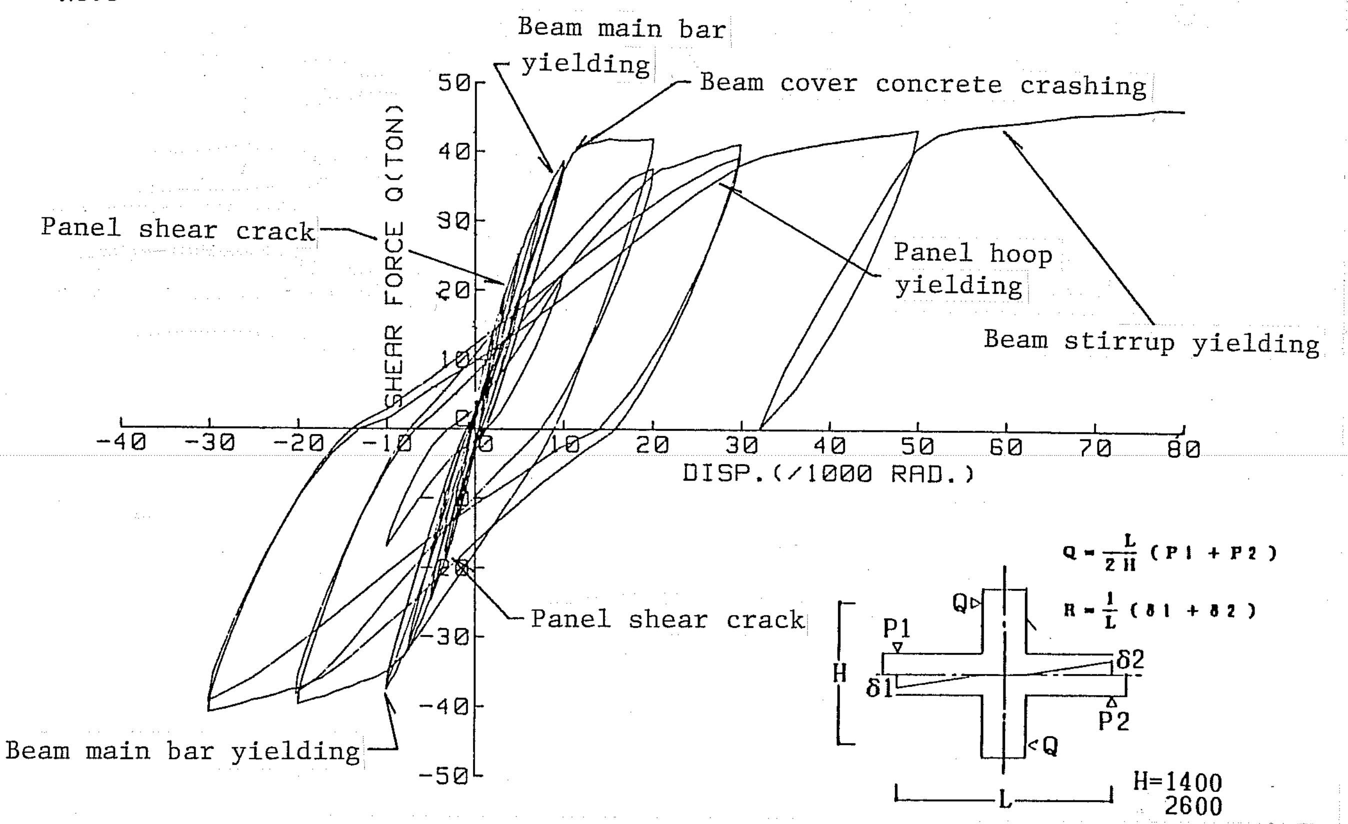
Fig. 5.1 Details of Beam-Column Joint Specimen (No.2)

No.2



(1) No.2 Specimen

No.3



(2) No.3 Specimen

Fig. 5.2 Measured Lateral Force-Displacement Hysteresis Loops

PART 3: EXPERIMENTAL AND ANALYTICAL STUDY ON THE SEISMIC BEHAVIOR OF RC MOMENT-RESISTING FRAMES WHICH HAVE WIDE COLUMNS

by

Ikuo Yamaguchi

Shunsuke Sugano

Yasuo Higashibata

Synopsis

The seismic resistance of reinforced concrete wall-type, moment-resisting frames, which have wide columns like wall-panel structures, was experimentally and analytically investigated to obtain design guidelines. Cyclic loading tests were conducted for models of subassemblages and members in medium to high-rise frames, and their ductile behavior was verified. It was indicated by the nonlinear static and dynamic analyses that the response of the structural system to severe ground motions would be much less than its seismic capacity observed in the experimental studies.

5. REINFORCED CONCRETE LAYERED CONSTRUCTION SYSTEM

by

Ichizou Kawabata

Seiji Yoshizaki

May 28, 1985

Taisei Corporation

Introduction

Taisei Corporation has constructed a number of reinforced concrete medium-rise buildings since Taisei developed the reinforced concrete layered construction system shown in Fig. 1. The layered system is partially prefabricated construction and combines the advantages of prefabrication and cast-in-place construction. The system aims for high quality control, low cost, and a short construction period.

Recently, Taisei Corporation has designed reinforced concrete high-rise buildings using this system. One of them is a 25-story apartment shown in Fig. 2. Earthquake resistance of the building has been verified by a series of seismic tests on the constituent members and joints. The structural profile of the building and some results of the seismic tests are briefly described hereinafter.

Reinforced Concrete 25-Story Building

As shown in Fig. 3, the typical floor plan of the building has seven bays in the longitudinal direction and 5 bays in the lateral direction. The building is supported by cast-in-place concrete piles as shown in the foundation plan. Normal weight concrete with compressive strength of 240 to 360 kgf/cm^2 is used to cast the building, as shown in the framing elevation.

Figure 4 shows details of columns in the building. The columns are laterally reinforced with high strength bars in combination with rectangular spirals and hoops. The lateral reinforcement consists of deformed bars used for prestress applications. The bars have a nominal yield strength of 13,000 kgf/cm^2 and are fabricated by bending (cold working). Outer columns are reinforced with longitudinal bars placed in the core in addition to bars placed around the periphery.

Figure 5 shows beam details. The beams are partially precast. The beams are connected to the slabs by cast-in-place concrete. Joints are also cast-in-place. High strength bars are also used as beam stirrups as in the columns. Beam bars at exterior ends are anchored in the joint by bending. The bent bar development length is 30 to 35 times bar diameter. Beam-to-beam joints are located at the midspan where beam bars are connected by enclosure welding (see Appendix A). Figure 6 shows locations of the beam-to-beam joints. The beams between the joints are prefabricated and handled as one unit.

Beam-column joints should be designed for shear forces developed when the beam hinge mechanism forms. The following equations may be used to evaluate joint shear strength.

$$p\tau u = \tau c + \tau s \qquad (\text{kgf/cm}^2)$$

$$\tau c = 95$$

$$\tau s = 0.5\ \rho_w\ s\sigma y$$

where

$p\tau u$ = joint shear strength

τc = shear stress carried by concrete

τs = shear stress carried by joint reinforcement

ρ_w = joint reinforcement ratio

$s\sigma y$ = yield strength of joint reinforcement

Rectangular ties fabricated with high strength bars are used as the joint reinforcement.

Beam bars at interior ends are developed continuously in the beam-column joint. In order to avoid undesirable bond slip in the joints, the beam bar diameter should be less than one-twentieth of the joint depth.

Figure 7 shows the construction procedure for the layered system. Columns are cast with in-situ concrete. Precast concrete beams are placed in position and connected to each other. Slabs are made of precast concrete decks (Omunia Board) and in-situ topping concrete is placed, as is the concrete to complete the joints.

Seismic Tests on Beams, Columns, and Joints

Seven series of seismic tests were conducted at Taisei Corporation Research Institute to obtain data to complement structural design work. The specimens were $1/\sqrt{2}$ to 1/3 scale models of columns, beams, and beam-column joints, as shown in Fig. 8. Cyclic lateral loads in combination with axial loads (except beam specimens) were applied to the specimens to simulate earthquake effect.

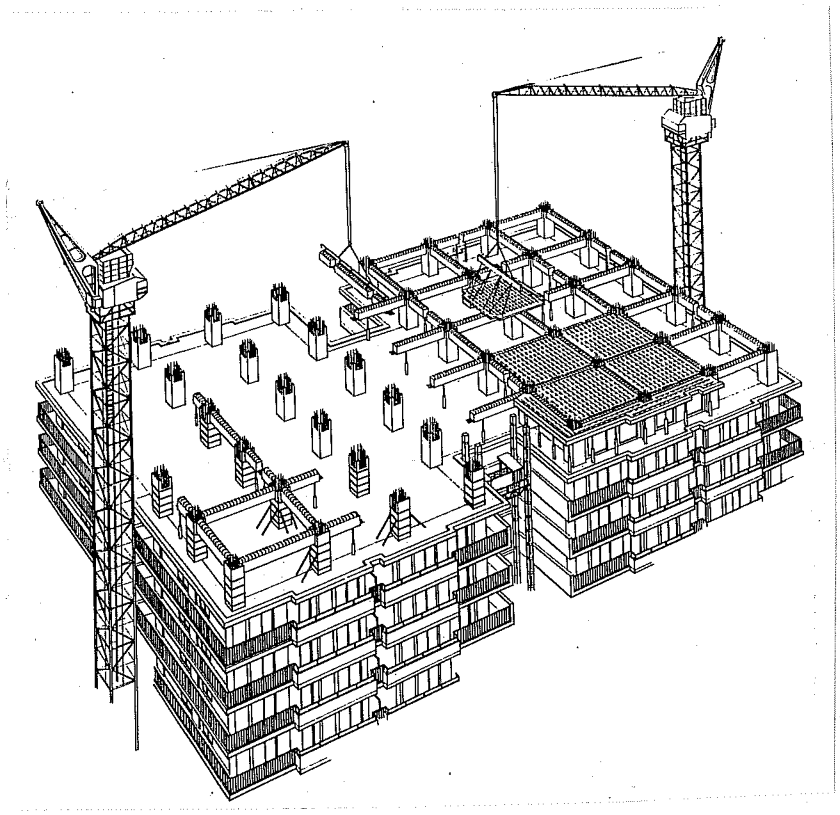
Figure 9 shows the loading setup for column and beam specimens. The top column stub of the specimens was fastened to a steel loading yoke which was deformed laterally. The loading and deformation produced is shown in the upper left corner of Fig. 10. Axial loads were applied to the column specimens through vertical loads on the yoke.

Figure 10 shows a load-displacement relation for an exterior column specimen. The specimen was subjected to an axial load varying from compression to tension and vice versa with the applied lateral load, in order to simulate the overturning effect on exterior columns. The relationship between the axial load N and the lateral load Q is also shown in this figure. The specimen showed higher strength and larger stiffness in positive loading than in negative loading because the specimen was subjected to compression in positive loading and tension in negative loading. Consequently, the load-displacement relation showed considerable pinching and unsymmetrical hysteresis.

Figure 11 shows a final crack pattern for the exterior column specimen. In positive loading, steeply inclined cracks were observed and concrete was considerably crushed. On the contrary, gently inclined cracks were distributed over the specimen length in negative loading.

Figure 12 shows a load-displacement relation for a beam specimen. The specimen was fabricated according to the layered construction system so that it had a construction joint between hardened concrete and topping concrete by slab depth. The specimen was set upright and loaded laterally in the same way as in the column test. The specimen had a stable load-displacement relation, although it had a small aspect ratio of a/D = 1.7 (a = shear span length, D = beam depth). Figure 13 shows the final crack pattern for the beam specimen. The specimen apparently failed in flexure.

Figure 14 shows the loading setup for a beam-column subassemblage. The specimen was laid on the reaction floor and held by jacks at the beam tips. Axial loads were applied to columns with prestressing strands stressed using centerhole jacks. Cyclic lateral loads were applied to the column tips where the columns were laterally linked to each other with steel yokes and jacks. Figure 15 shows a load-displacement relation for a beam-column subassemblage specimen. The specimen was a $1/\sqrt{2}$ scale model and fabricated using the same procedure as in the layered construction system. The specimen exhibited a stable load-displacement relation as shown in this figure. Figure 16 shows the final crack pattern for the beam-column subassemblage. Some inclined cracks were observed in the beam-column joints. The specimen apparently failed in beam flexure.



R.C. Layered Construction System is partialy prefabricated construction system.
This system has both advantages of prefabrication and cast-in-place.

Aims

- High quality
- Low cost
- Short construction period

Layered Construction System

Fig. 2 Perspective of 25-

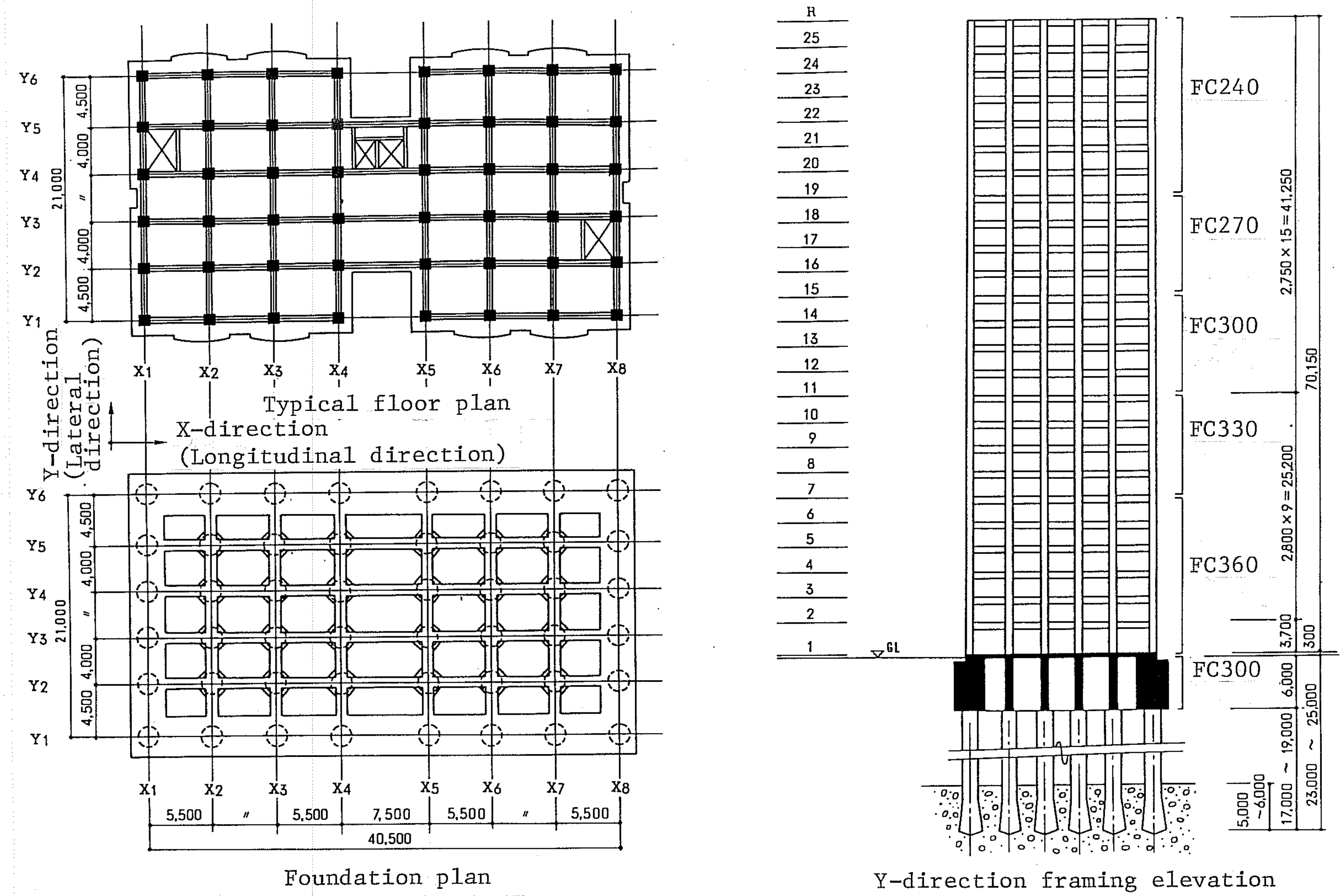
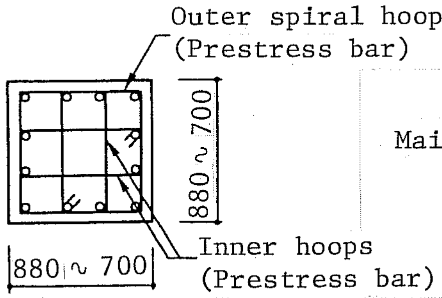


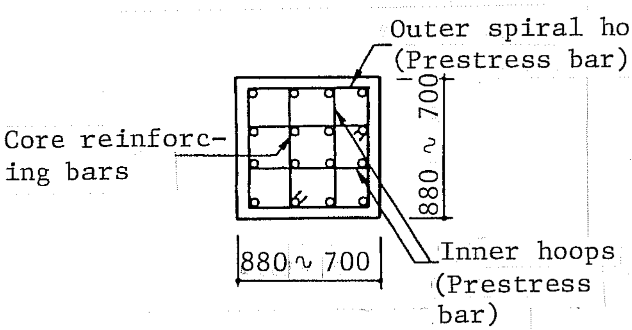
Fig. 3 Structural Plans and Framing Elevetion

o Inner columns



Main reinforcements			12-D32∿12-D41(SD40)
Hoops	Outer spirals		ϕ11-□-100@∿80@
	Inner hoops		ϕ9.2- -200@∿
			ϕ11-⊞-100@∿80

o Outer columns



Main reinforcements		12-D32∿16-D41(SD40)
Core reinforcements		4-D32∿ 8-D41(SD40)
Hoops	Outer spirals	ϕ11-□-100@∿60@
	Inner hoops	ϕ9.2-⊞-200@∿
		ϕ11-⊞-100@∿60

Fig. 4 Column Details

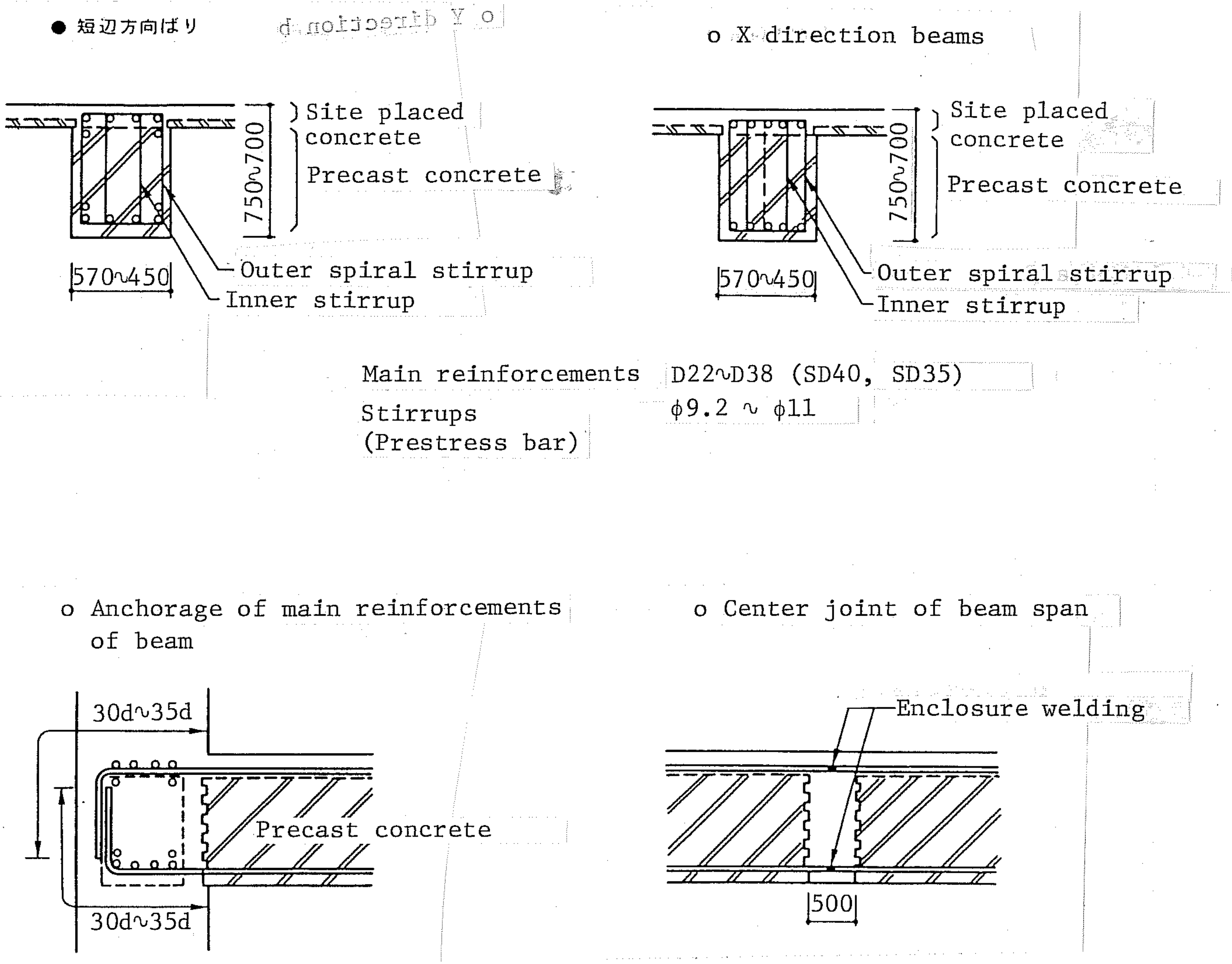
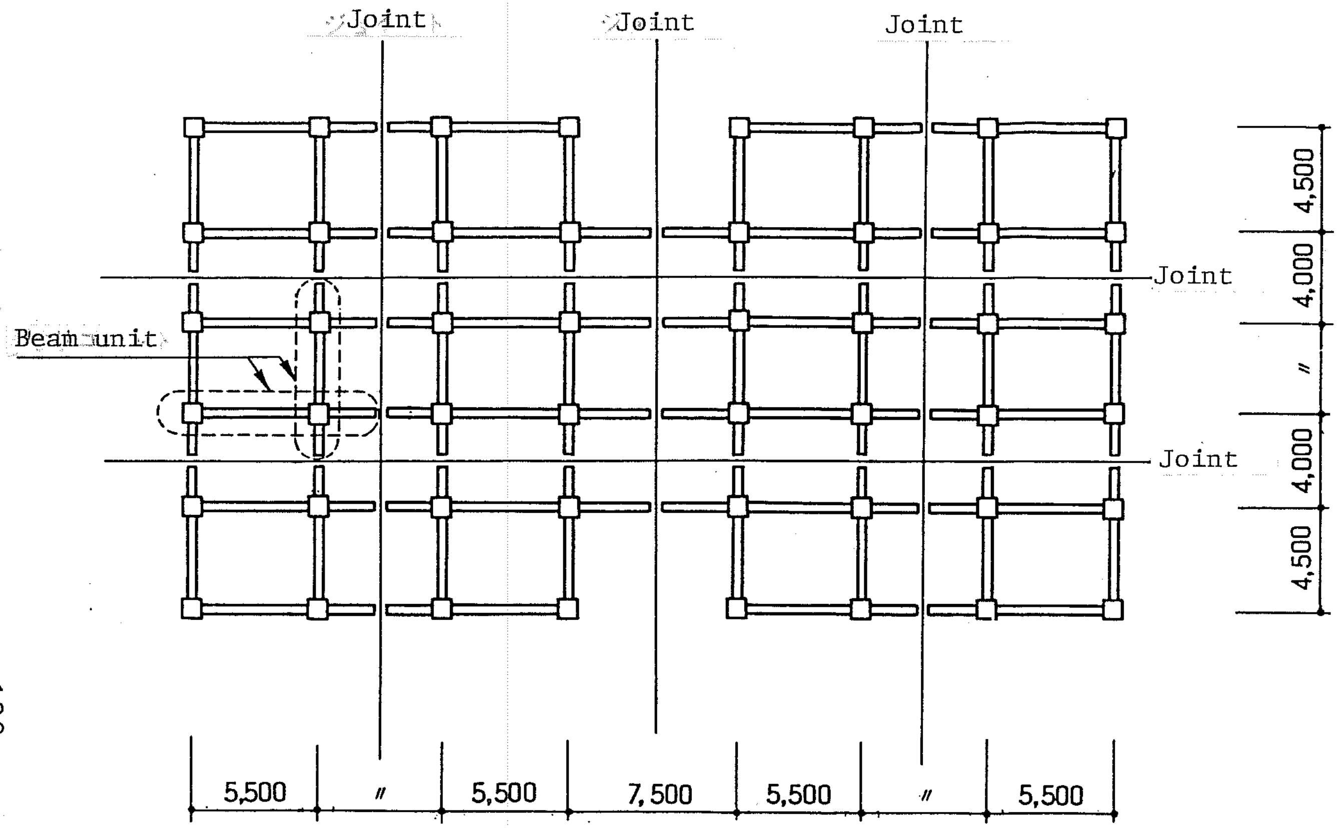
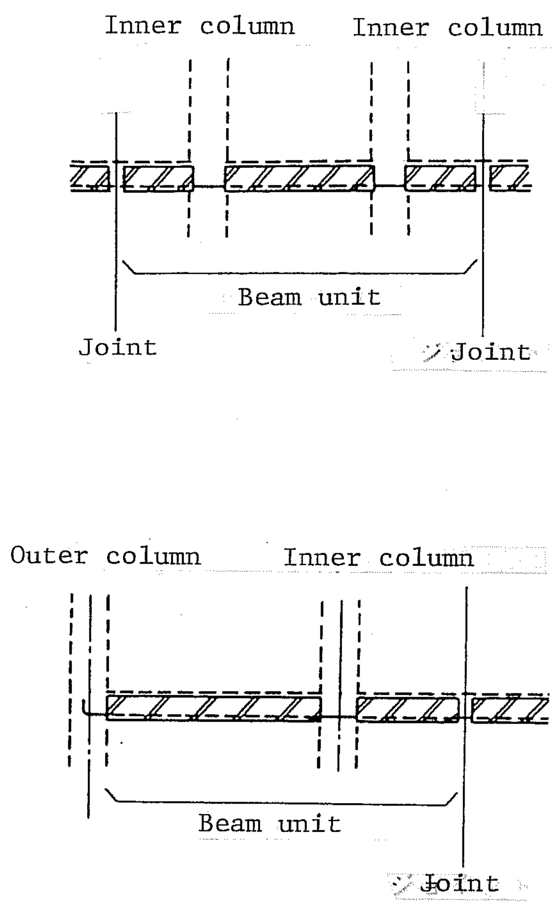


Fig. 5 Beam Details



o Arrangement of beam units and joints

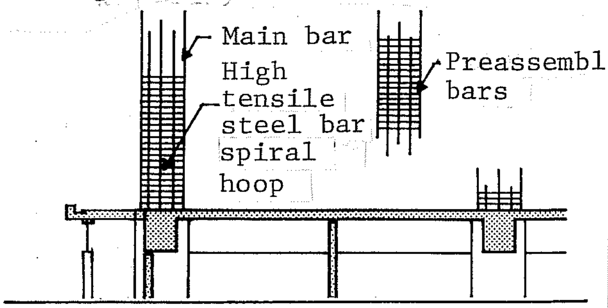


Main reinforcement of column : Pressure welding

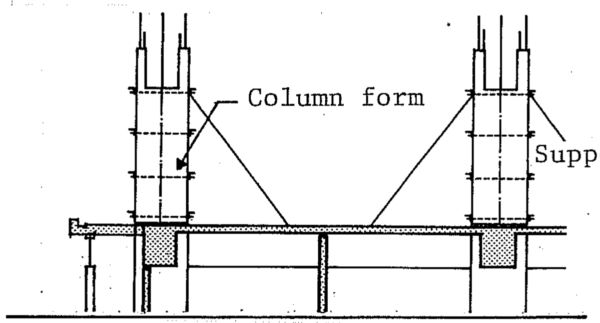
Main reinforcement of beam : Center joint, enclosure welding

Fig. 6 Locations of Beam-Beam Joints

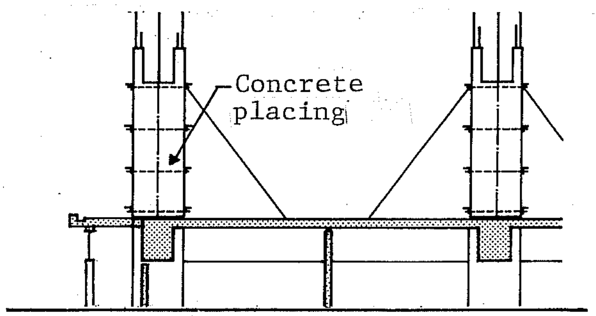
1. Assembly of column reinforcing bars (Automatic, simultaneous gas pressure-welding)



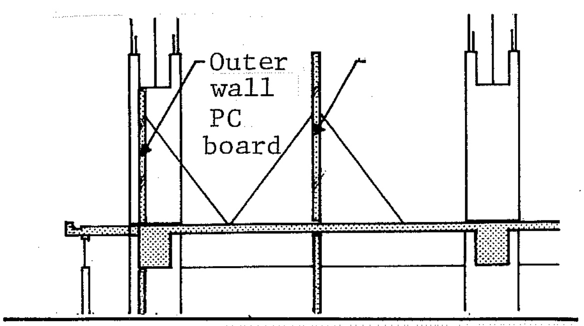
2. Setting column form



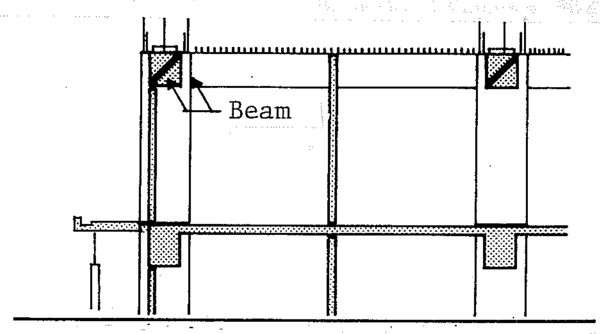
3. Placing column concrete (up to lower end of beam : VH method)



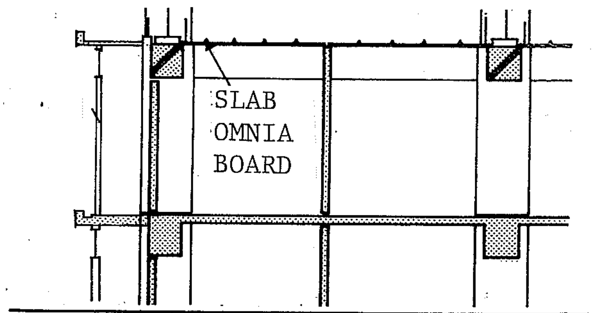
4. Fitting wall Panel



5. Fitting Beam and jointing beam center (main bars)



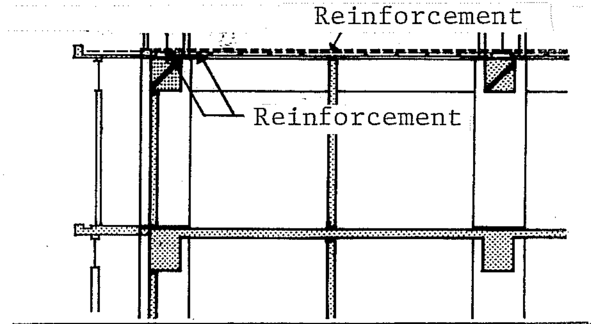
6. Laying Slab OMNIA BOARD



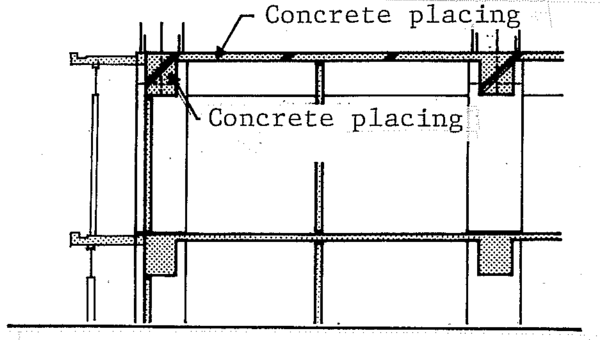
7. Arrangement of beam reinforcement, and beam-column connection reinforcement

8. Arrangement of floor slab reinforcement

9. Embedded equipment piping



10. Placing concrete Beam-column connection, Upper part of beam, and Upper part of floor slab



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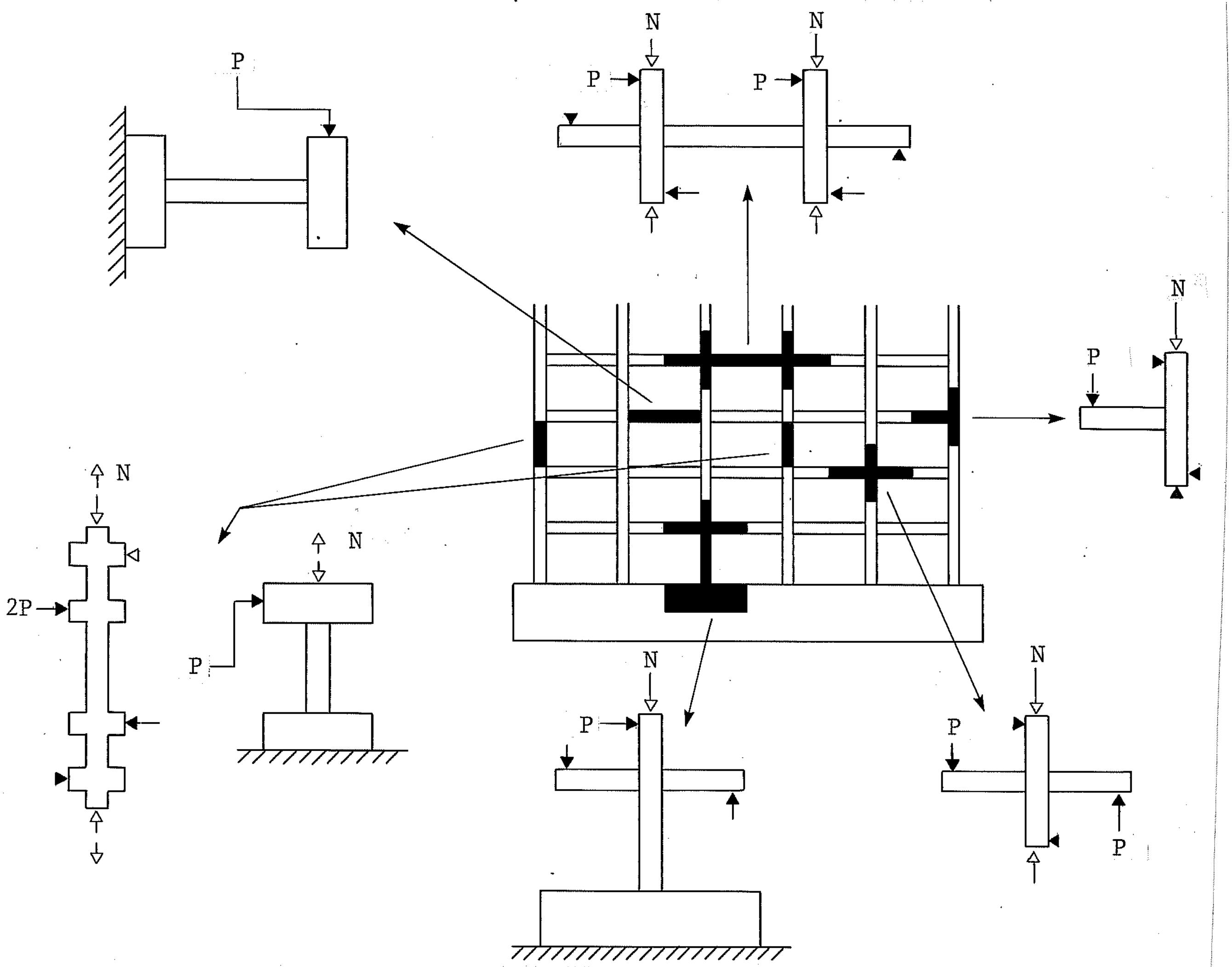


Fig. 8 Tested Portion

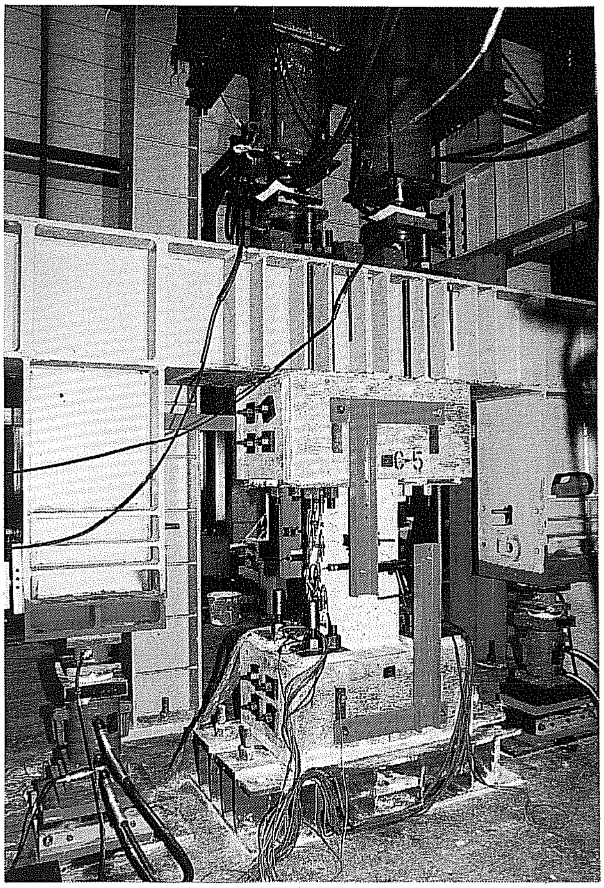


Fig. 9 Loading Set-up for Column or Beam Specimens

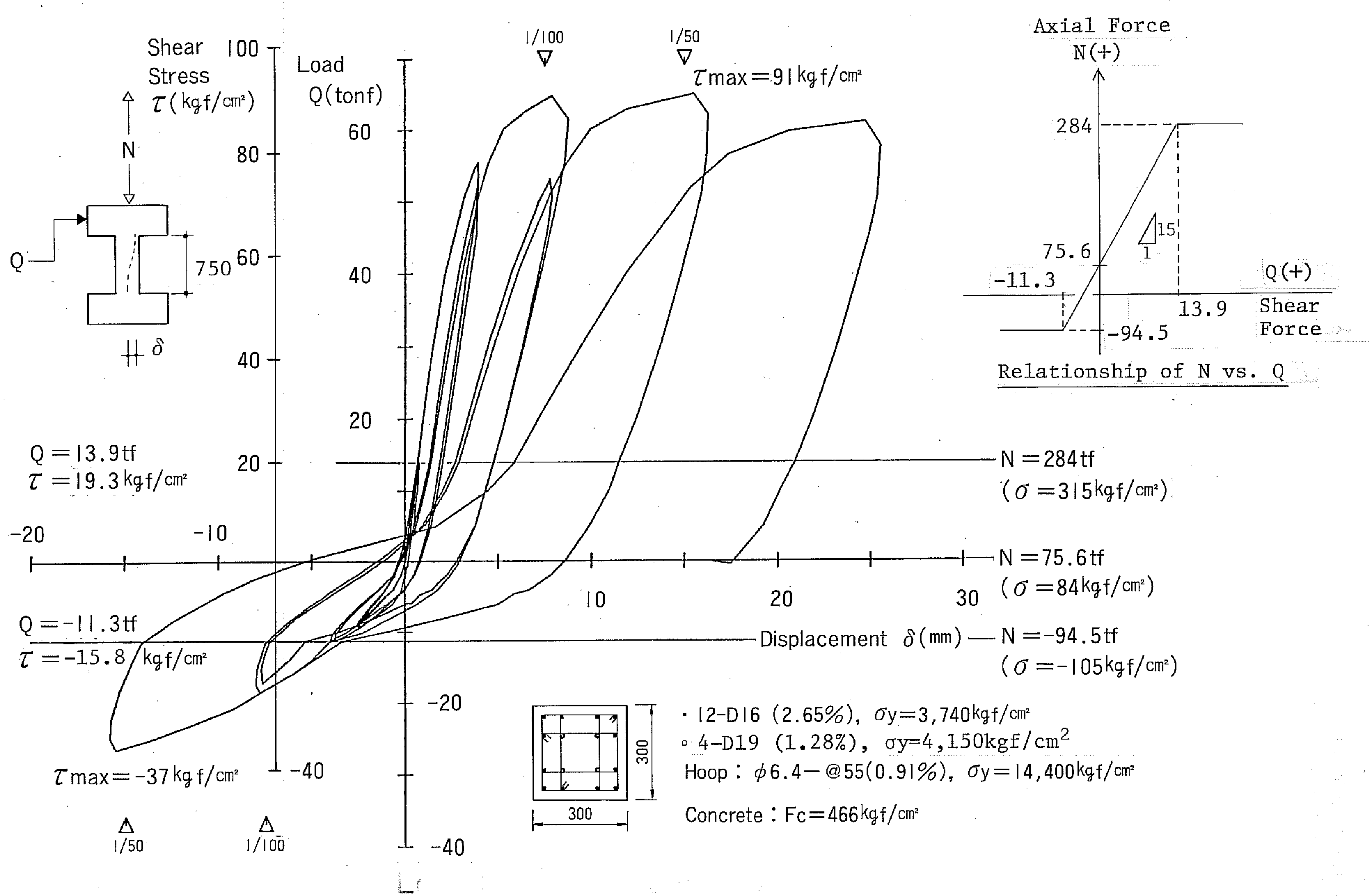


Fig. 10 Load-Displacement Relation for Column Specimen (C-4)

Fig. 11 Final Crack Pattern of Column Specimen

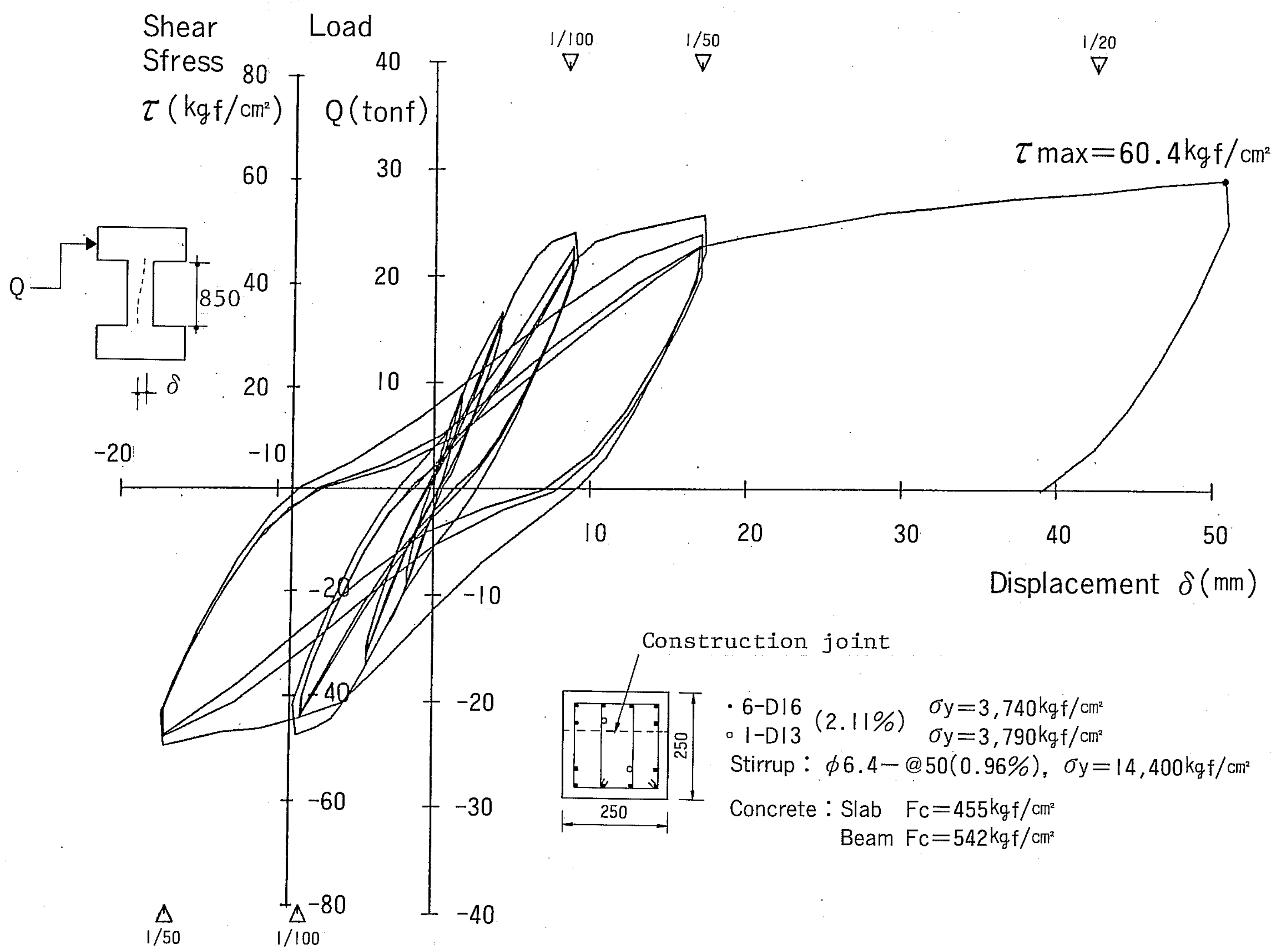


Fig. 12 Load-Displacement Relation for Beam Specimen (G-5)

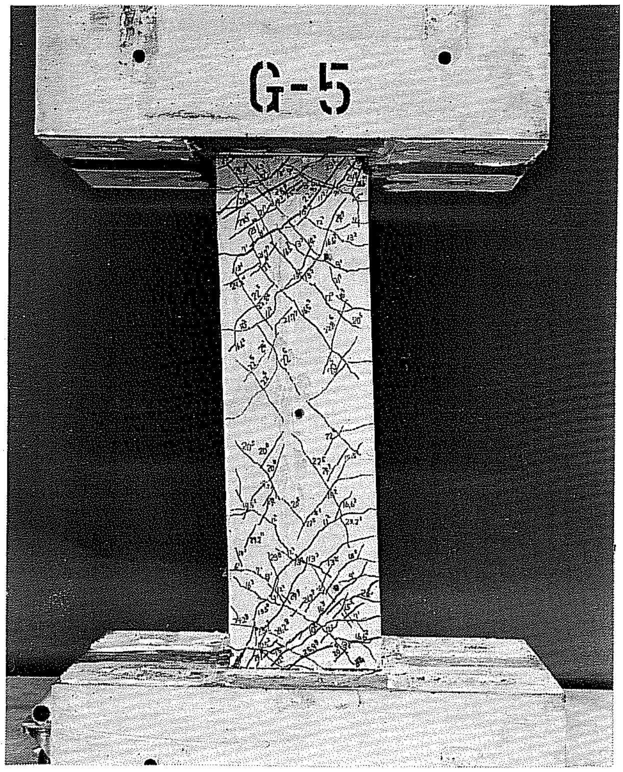


Fig. 13 Final Crack Pattern of Beam Specimen

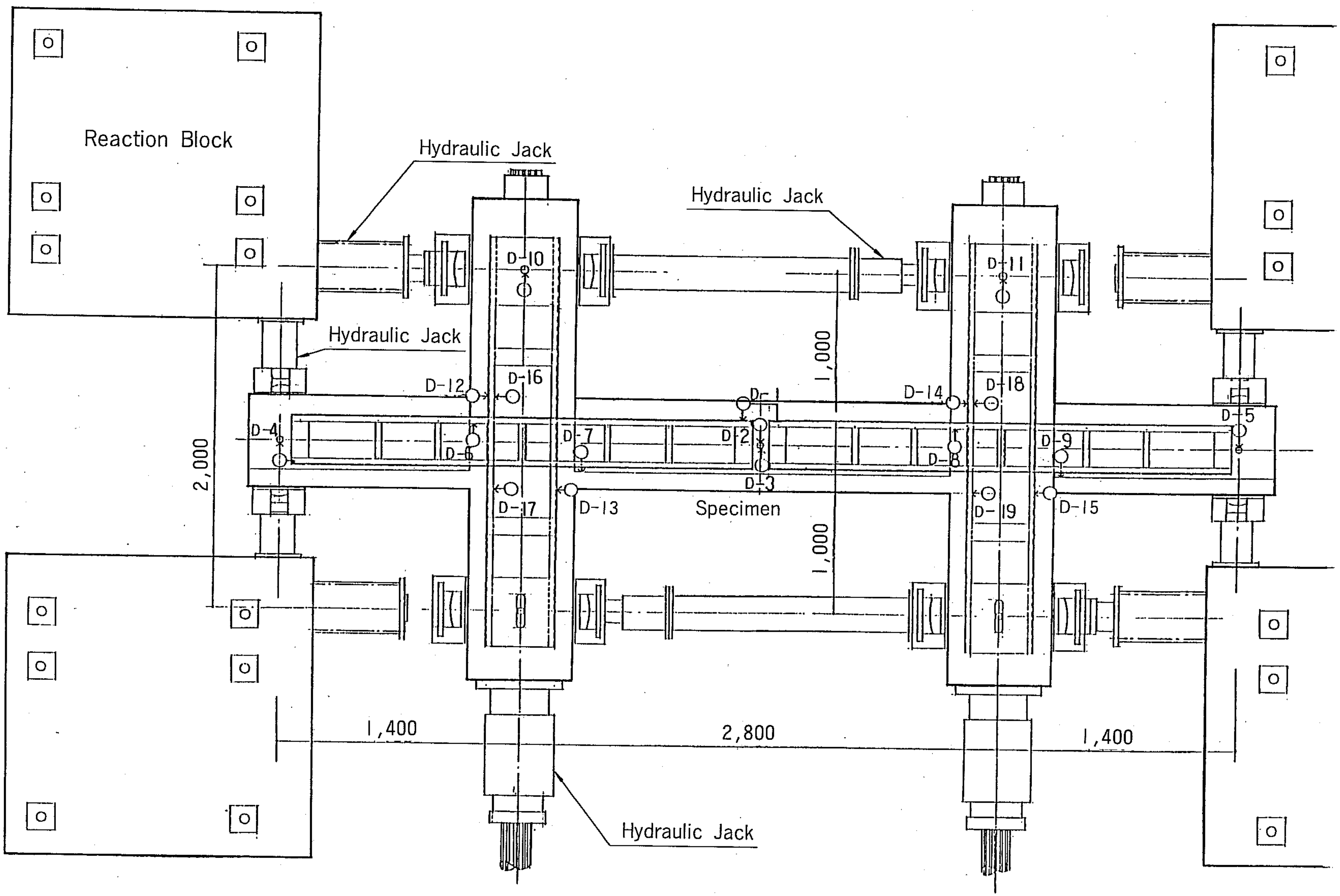


Fig. 14 Loading Set-Up for Beam-Column Subassemblage

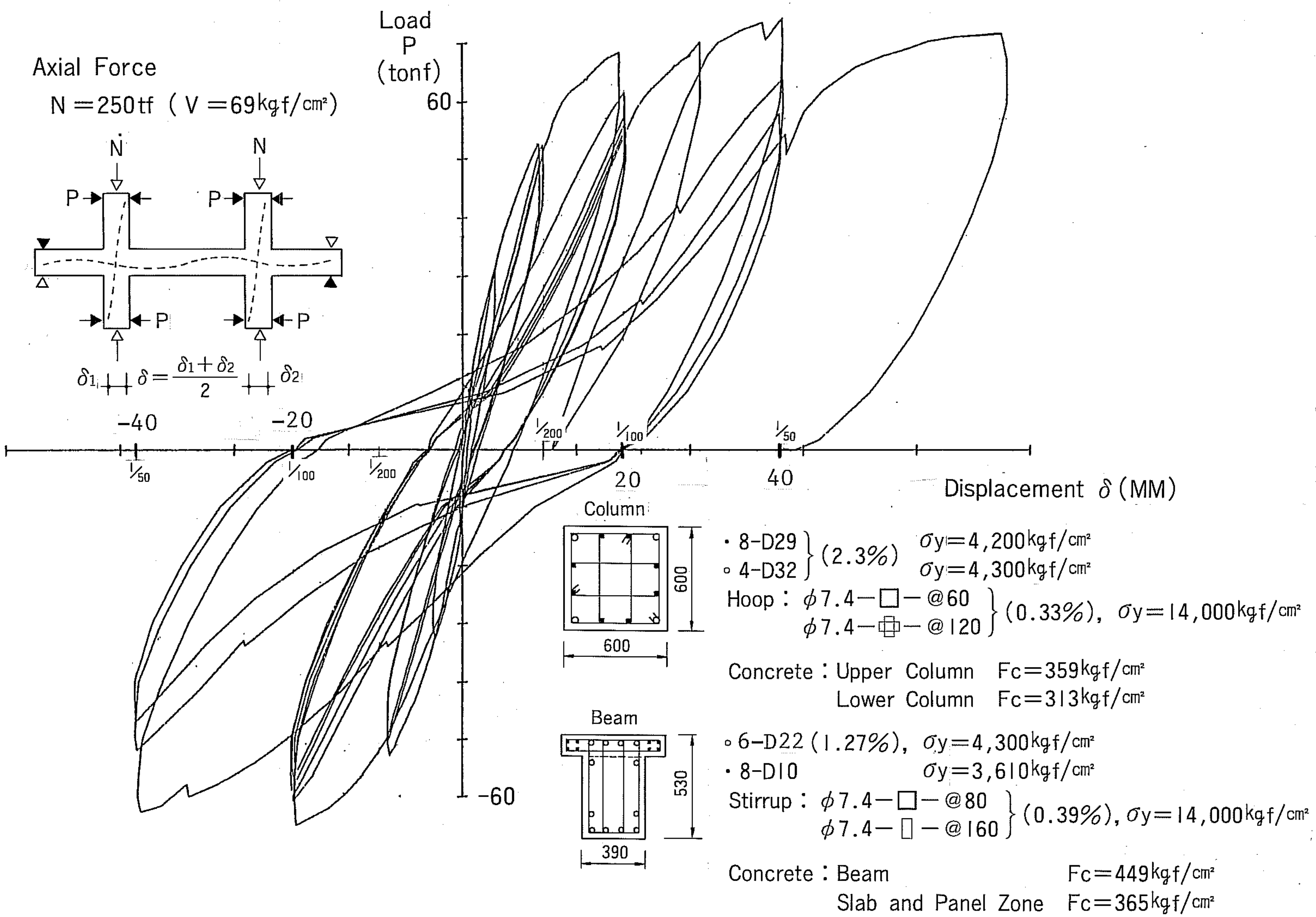
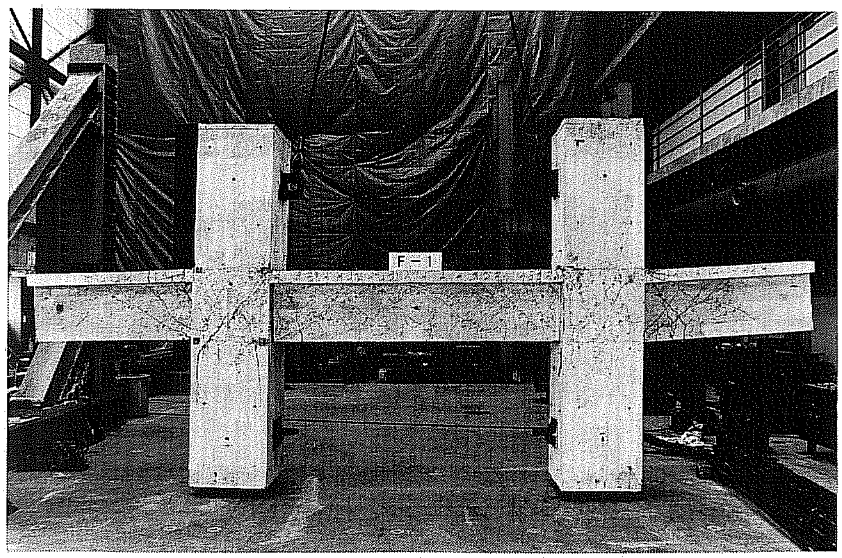


Fig. 15 Load-Displacement Relation for Beam-Column Specimen (F-1)


F－1

APPENDIX A

BAR SPLICE METHODS IN JAPAN

APPENDIX A: BAR SPLICE METHODS IN JAPAN

A.1 Gas Pressure Welding

Gas pressure welding is one of the most popular methods for joining reinforcing bars. The welding procedure is schematically shown in Fig. A.1 and summarized as follows:

(a) The ends of reinforcing bars are cleaned and sanded.

(b) The bars are aligned with a hydraulic cylinder.

(c) The ends of bars are heated with an acetylene torch and are clamped together with a pressure of 300 kgf/cm^2 or higher.

(d) Heating and clamping are applied to develop a bulge of at least 1.4 times the nominal bar diameter.

(e) Heating is stopped and the clamping device is removed after the bulge has formed correctly.

The ends of the bars are heated to 1200 to 1300°C and are joined without fusing. Note that gas pressure welding is considerably different from ordinary arc welding which needs a high temperature of 1900 to 2000°C to fuse base metal. The heat and pressure are controlled manually or automatically as shown in Fig. A.2. Generally, reinforcing bars made from blast furnace steel are upset two times and those from electric furnace steel are upset three times but over a shorter time.

Figure A.3 shows a computer-controlled welder used widely in Japan.

A.2 Enclosure Welding

Enclosure welding is an arc welding method for joining reinforcing bars. The butt zone of the bars is enclosed with a backing sleeve and arc-welded with a coated electrode. Two methods, shown in Figs. A.4 and A.5, are available in Japan. A copper sleeve is used in KEN method and a steel sleeve is used in SBR method. These sleeves have a cut for welding operations.

A.3 Mechanical Splice

Various mechanical splices have been developed and used in actual construction. Most of them join deformed bars by pressing a mild steel sleeve over the butt zone. Figure A.6 shows the method called "squeezed sleeve joint". The steel sleeve is squeezed to interlock the bar deformations in this method.

A.4 Chemical Splice

Figure A.7 shows the method called "NMB splice sleeve joint". A cast-iron sleeve is used to enclose the butt zone of the deformed bars and non-shrink mortar is grouted into the sleeve. This method is widely used in precast concrete structures.

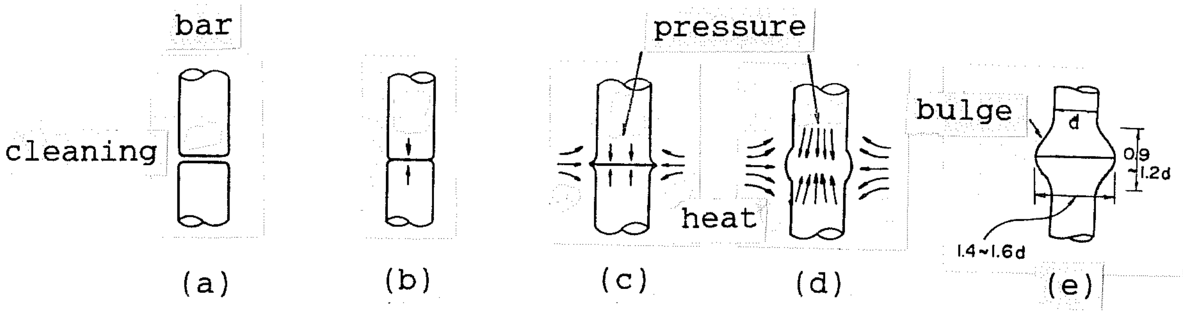


Fig. A.1 Procedure of Gas Pressure Welding

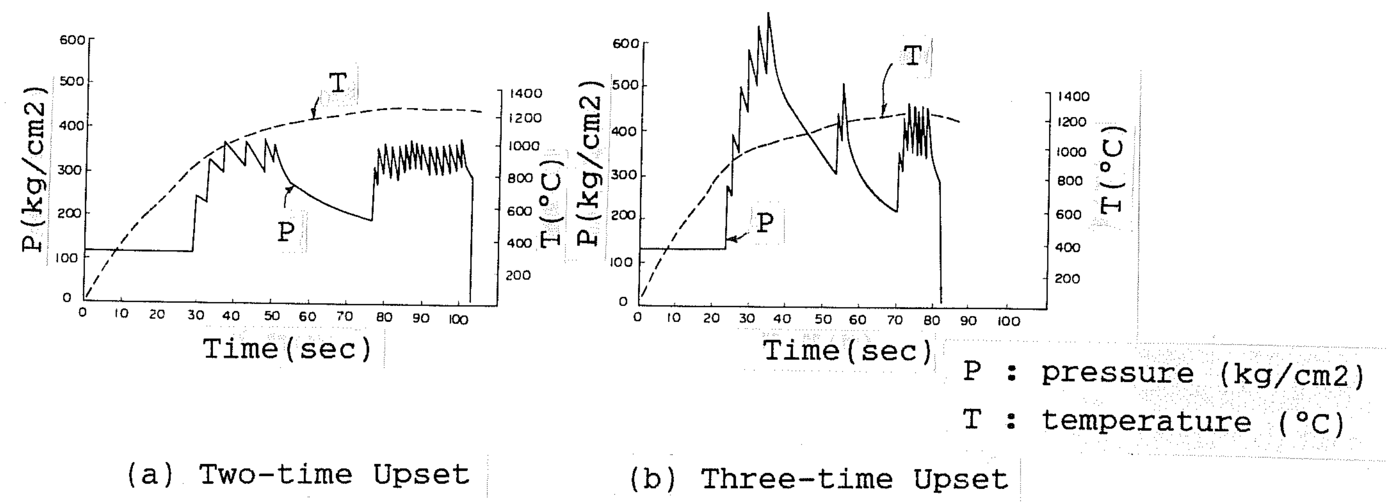


Fig. A.2 Pressure and Temperature in Gas Pressure Welding

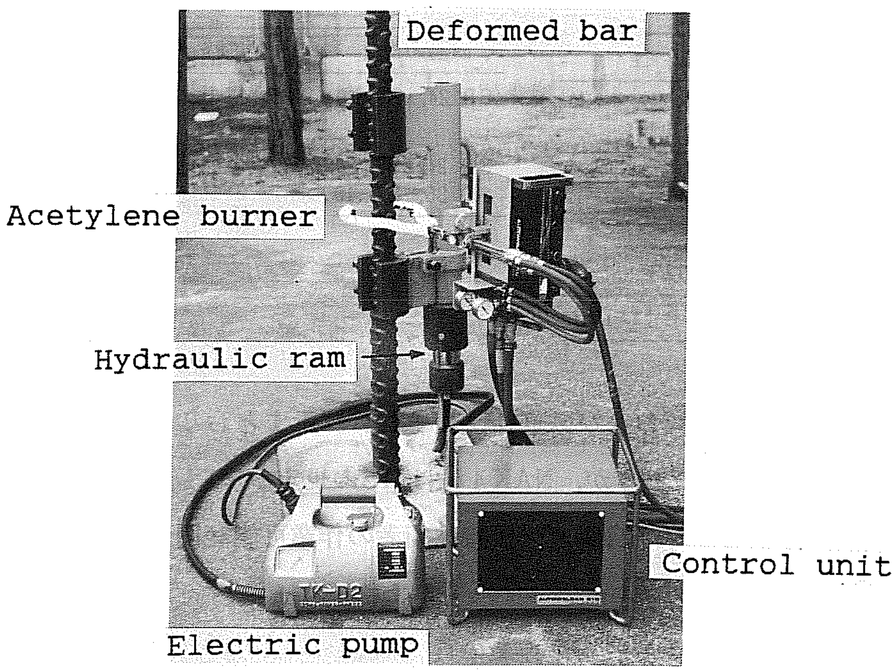
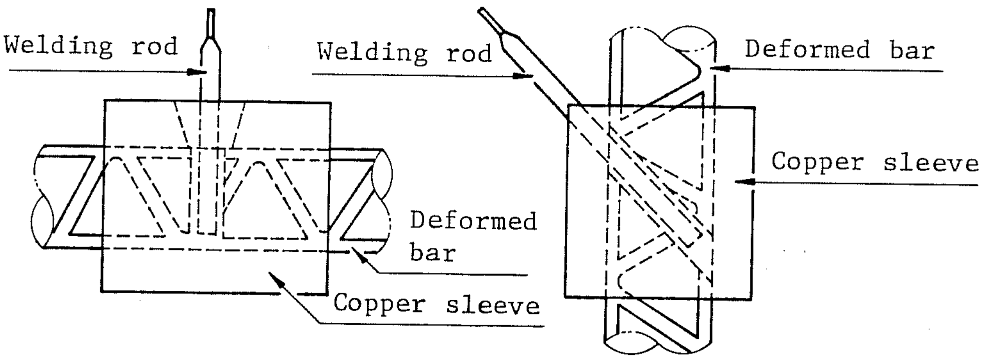
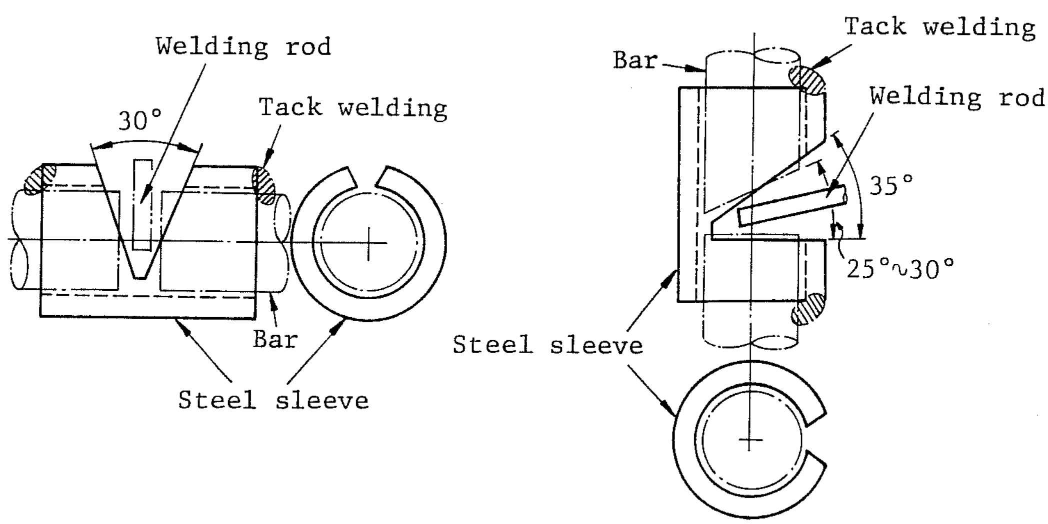


Fig. A.3 Automatic Gas Pressure Welder



(a) Horizontal position (b) Vertical position

Fig. A.4 KEN Method (Enclosure Welding)



(a) Horizontal position (b) Vertical position

Fig. A.5 SBR Method (Enclosure Welding)

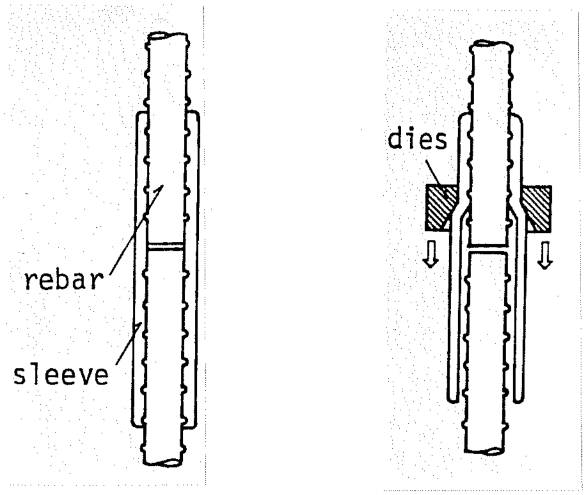


Fig. A.6 Squeezed Sleeve Joint

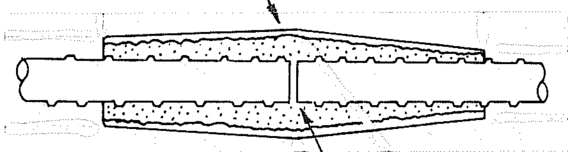


Fig. A.7 NMB Splice Sleeve Joint

APPENDIX B

Special Lecture

on

SUPER HIGHRISE BUILDING IN JAPAN

APPLICATION OF PRECAST REINFORCED CONCRETE STRUCTURE

(Design and Construction for Shinjuku Nishi-Toyama Housing Complex)

presented at

Seminar on Precast Concrete Construction in Seismic Zones

Tokyo, October 28, 1986

by

Toshikazu Takeda, Vice-Director

Ohbayashi Corporation, Engineering Research Institute

APPENDIX B

Super Highrise Building in Japan
— Application of Precast Reinforced Concrete Structure —
(Design and Construction for Shinjuku Nishi-Toyama Housing Complex)

Dr. Toshikazu Takeda
Vice-Director
Engineering Research Institute
OHBAYASHI CORPORATION

1. Introduction

A number of large-scale urban redevelopment projects have been recently planned and implemented in Japan, with a view to fully utilizing former factory sites, state-owned land, etc. as well as to expanding domestic demand. One of such projects is the construction of Shinjuku Nishi-Toyama Housing Complex ("Nishi-Toyama Tower Homes"), which is now under way at a site previously occupied by a housing complex for government workers. The project has attracted considerable attention as the first of its kind intended to utilize state-owned land in cooperation with the private sector.

Being constructed under the project on a site of approximately 20,000 m^2 are three super high rise collective dwellings, two low-storied auxiliary buildings and one multipurpose meeting hall as a community facility. All of the three collective dwellings will have 25 stories, being of pure rahmen structure built of reinforced concrete (hereinafter called "RC"). The RC structure has been adopted due to its superior characteristics in terms of dwelling conditions such as resistance against wind and sound insulation as well as economic conditions such as construction cost and work period. Another feature of those buildings is the extensive use of precast concrete (hereinafter called "PC") for their beams, floor slabs, external walls (non-structural) and other members, in addition to column forms -- which aims at improving work quality, shortening work period, and otherwise rationalizing the construction work. It is for the first time in

Japan that PC is to be used in such a way for super high rise buildings of RC structure.

The building owner under the project is Shinjuku Nishi-Toyama Development Co., Ltd. (established jointly by 66 developers), while the designer and supervisor is Shinjuku Nishi-Toyama Development Project Designing and Supervising Consortium (a joint venture of Mitsubishi Estate Co., Ltd. and other eight companies). The work has been started by a joint venture of Ohbayashi-Gumi Ltd. and other 15 builders in May 1986, and is scheduled for completion in March 1988.

This paper describes the structural designing and work execution for the super high rise collective dwellings.

2. Outline of the Building

The typical plan and elevation of the building are shown in Figures 1 and 2, respectively.

The three buildings are identical to each other in use, scale and structure, with all of them having 25 stories and one basement level. The typical floor has a size of 32.1 m in both X and Y directions. The end span is 6.15 m while the inner one (four spans) is 4.95 m. The floor height is 2.9 m on the typical floor, 4.5 m on the first floor and 4.0 m on the basement level. The eaves height is 75.0 m.

The floor of the building is U-shaped, with open space provided at the center toward the north. The northeast and southwest corners of the building are cut as shown in the typical plan, and the northwest corners of the 23rd and upper stories are set back as shown in the elevation. In structural terms, girders are provided across the open space on every even-numbered floor for frame ① in the Y direction (frames Ⓒ through Ⓔ) and on every floor for frame ②, thus preventing torsional deformation of the building and maintaining the in-plane rigidity of the floor.

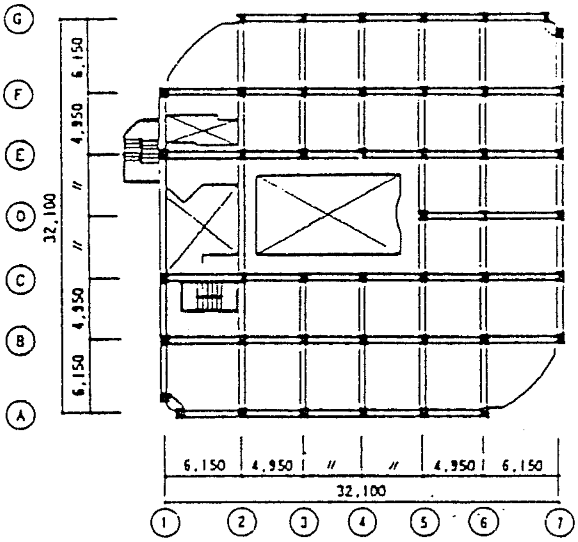


Fig. 1 Typical Plan

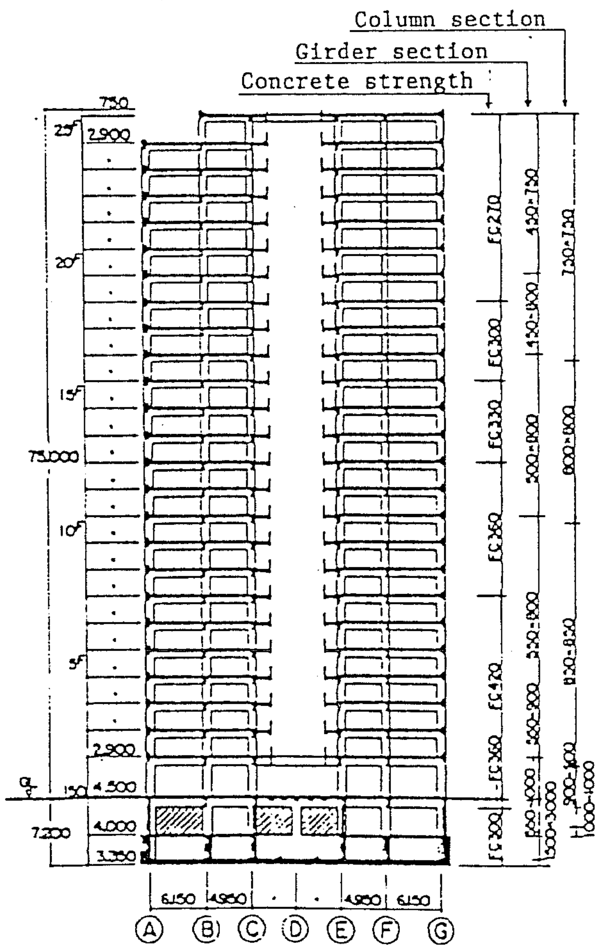


Fig. 2 Elevation

3. Structural Design

(1) Principles for aseismatic designing

The basic concept in aseismatic designing is to enable the building to absorb seismic energy by means of plasticity of its members. The building has been designed so that its collapse mechanism would be overall collapse through bending yield of beams. However, bending yield is allowed at the column capital on the uppermost floor, column base on the first floor and some of the outer columns supporting axial tension at the time of an earthquake. As a result, columns, beams, their connections, etc. are designed not to be subject to shear and other brittle fracture and to have superior restraint effects on core concrete and main reinforcement so that such members would have superior hysteresis characteristics with sufficient ductility.

Aseismatic characteristics of the building have been examined with the elasto-plastic response analysis. It has been required ultimately that the collapse mechanism of the building be of beam-yielding type even in a large earthquake, and that the deformation and ductility factors of stories and members satisfy the dynamic design criteria shown in Table 1.

(2) Structure of the main frame

The sections of the column and beam are shown in Figure 3.

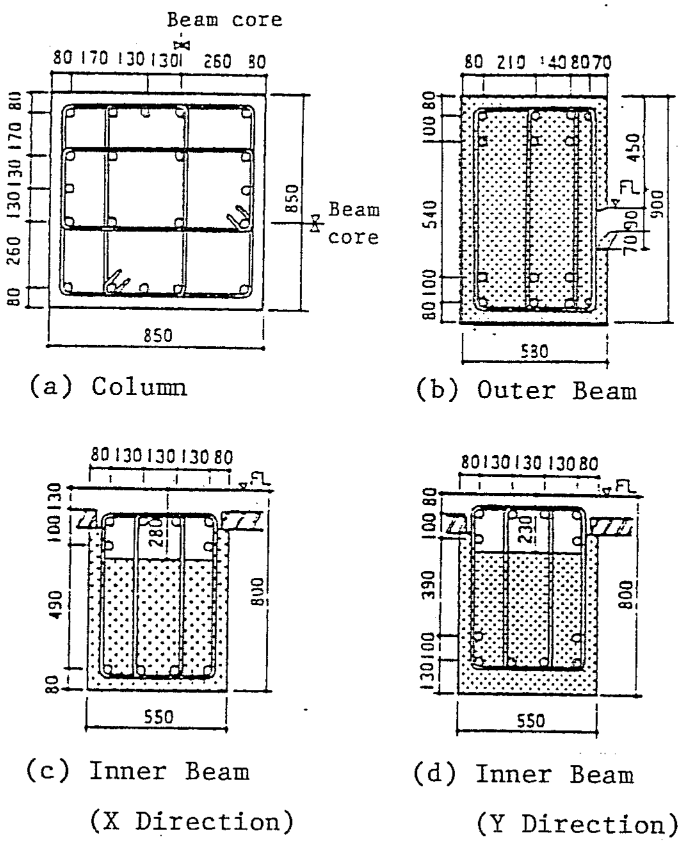
The section of each member, reinforcement, etc. has been designed on the assumption that PC would be used extensively. On aboveground stories, the column has a square section of 75 - 90 cm, while the beam section is 45 - 60 cm wide and 75 - 90 cm long. The outer beam has a raised head whose projecting part is to be connected with the floor slab at the middle of the beam length (top of the beam = FL + 45 cm). Main reinforcement for columns and beams are thick bars of D29 - D41, and core bars are used for some of the outer columns.

To provide members with superior ductility, deformed PC steel bars (9.2 and 11ϕ) are used as the reinforcement for columns and beams against shear. The columns are of the outer spiral + ╬ type, and the beams are of the double spiral type so that they would have superior restraint effects on core concrete and main reinforcement.

The concrete used is high strength normal concrete with design strength Fc of 270 - 420 kg/cm^2. The main reinforcement for columns and beams is SD 40 and the slab reinforcement is SD 30.

Table 1 Dynamic Design Criteria

	Input level	Deformation	Member ductility	Story ductility
Maximum earthquake	25 kine	1/200 or less	1.0 or less	1.0 or less
Limit earthquake	40 kine	1/100 or less	4.0 or less	2.0 or less



(a) Column

(b) Outer Beam

(c) Inner Beam (X Direction)

(d) Inner Beam (Y Direction)

Fig. 3 Typical Section

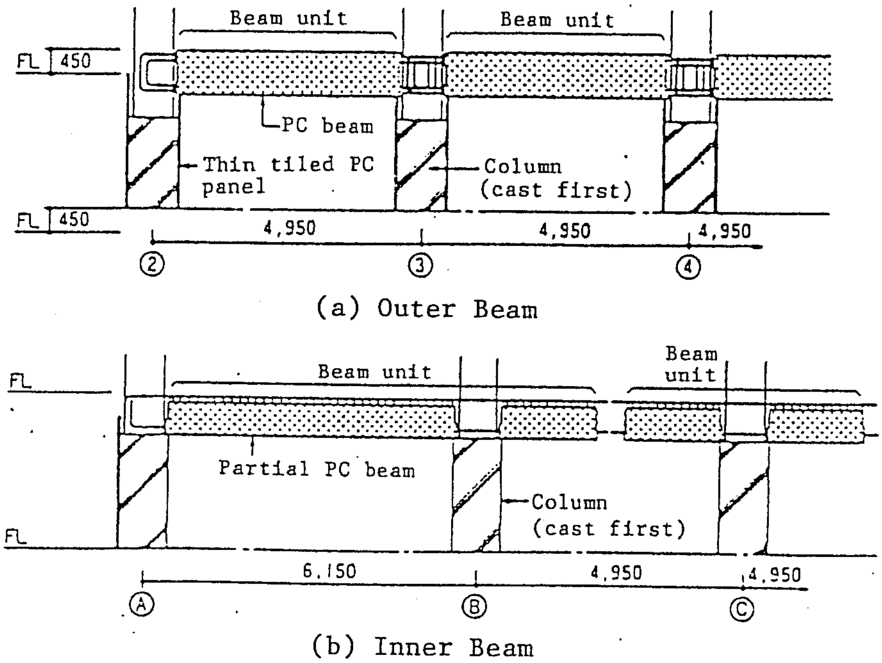


Fig. 4 Precast Unit of Beam

(3) Use of PC

Figure 4 shows the PC unit of beam.

The columns on aboveground stories consist of prefabricated reinforcement and cast-in-place concrete. For three sides of each outer column, thin tiled PC panels are used both as the form and finishing material.

Outer beams (with raised heads) are built of tiled PC, with each span constituting the PC unit for improving finishing precision. PC members are joined with each other at the column-beam connection (cast in place), and all of the main reinforcement is anchored at the joint with the double U method.

For inner beams, partial PC members (the column-beam connection not provided with concrete and only with the bottom reinforcement) are used. PC members are joined at the middle of the span. Those members have precast bottom reinforcement and stirrups, and the concrete for the upper reinforcement and other parts is cast in place. The main reinforcement is L-shape anchored at the outer end (with the upper reinforcement bent downward and the bottom reinforcement upward) and go through inner ends. Joints are welded with enclosed welding.

Floors are built of thin PC panels for synthetic floor. Those panels have precast bottom reinforcement (the panels being also used as forms), with the concrete for the upper reinforcement and other parts being cast in place.

4. Standard Designing

In primary designing, normal allowable unit stress has been obtained for sustained and temporary loads. Regarding the story shear for aseismatic designing, applicable values have been calculated in accordance with the relevant provisions of the Enforcement Order of the Building Standard Law (Base shear coefficient C_B = 0.130 / Story shear distribution factor = A_i). Stress at the time of earthquake has been obtained by means of two-dimensional elastic frame analysis, and locally concentrating stress has been reallocated.

5. Developed Designing

In developed designing, the ultimate strength and ultimate lateral load carrying capacity have been calculated. The building has been designed so that its collapse mechanism would be of beam bending yield type, i.e., that the ultimate shear strength of columns, beams, and their connections as well as the ultimate bending strength of columns not assumed to yield would equal or exceed a value obtained by multiplying the collapsing stress by an additional coefficient. The building has also been designed so that the ultimate lateral load carrying capacity would at least be 1.5 times as much as the design story shear.

The ultimate lateral load carrying capacity is shown in Figure 5.

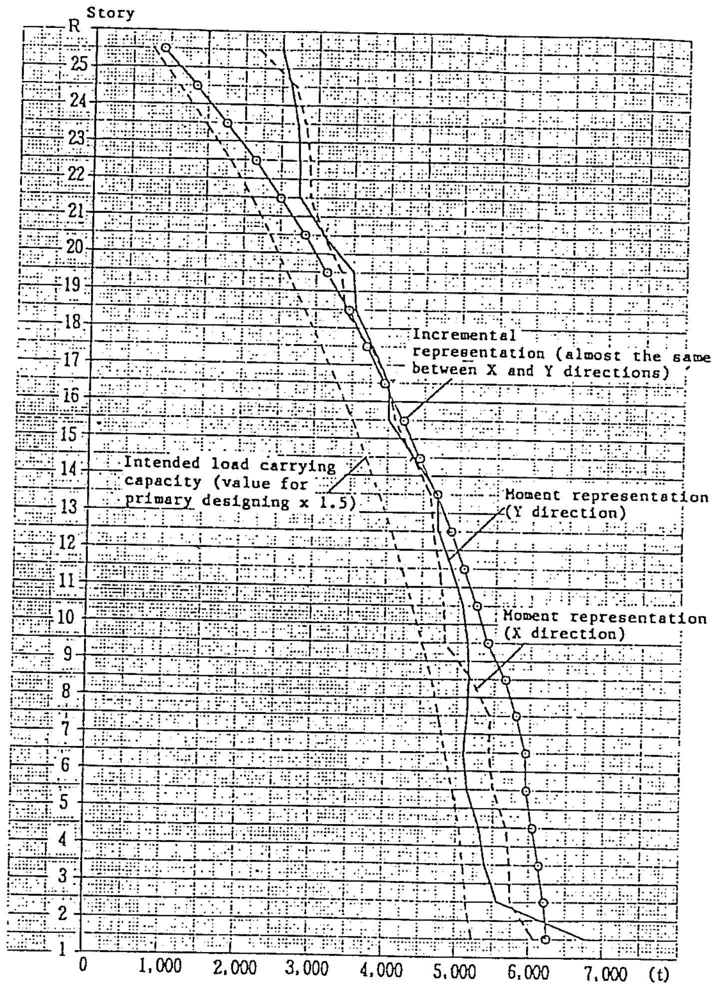


Fig 5 Ultimate Lateral Load Carrying Capacity

Table 2 Natural Period (Shear Model)

	X direction	Y direction
T_1 (sec)	1.253	1.362
T_2 (sec)	0.484	0.532
T_3 (sec)	0.297	0.327

6. Earthquake Response Analysis

The elasto-plastic dynamic analysis was carried out for the structure of 25 story building above ground using earthquakes of 25 kine and 40 kine. It was confirmed that this structure meet the design criteria shown Table 1. The earthquake response analysis has been conducted using lumped mass shear model (X and Y directions) and two-dimensional lumped mass shear model (Y direction). In the two-dimensional shear model, each plane of structure in the X and Y directions has been represented with a lumped mass shear model at the plane, and assuming that the floor is rigid floor, the analysis has been conducted at the center of gravity on each story considering three degrees of freedom (two horizontal and one rotational).

The hysteresis characteristics have been assumed to take a tri-linear form (Takeda Model). The skeleton curve of the hysteresis characteristics of each plane of structure on each story has been obtained by conducting the static elasto-plastic plane frame analysis (end rigid plasticity spring method) independently for each plane of structure, and then, converting the results of the analysis into a tri-linear form for each story. The skeleton curve has been used for analyzing the two-dimensional shear model, while the curve for the shear model has been obtained by summarizing the above-mentioned analysis results into a tri-linear form.

The natural period for elasticity as well as response results of shear model and two-dimensional shear model are shown in Table 2, Figure 6, and Figure 7, respectively. Figures 8 and 9 represent the yield hinges of beams and columns (at earthquake of 25 kine in Figure 8 and 40 kine in Figure 9), which have been estimated based on the results of the static frame analysis.

The earthquake response analysis has revealed that maximum story deformation angle R is 1/258 (Y direction) for 25 kine and 1/147 (Y direction) for 40 kine. Story ductility factor μ is 1.01 for 40 kine. All of the values are those of the two-dimensional shear model (frame 1), and as shown in Figure 7, they are larger than the values of the shear model by about 10%. Earthquakes of 25 kine are assumed not to result in the yield of any member, and maximum beam ductility factor μ at earthquake of 40 kine is estimated to be 1.31 (X direction).

All of the above results satisfy the dynamic design criteria, and the building can be considered to have sufficient strength against earthquakes of the maximum and limit scales.

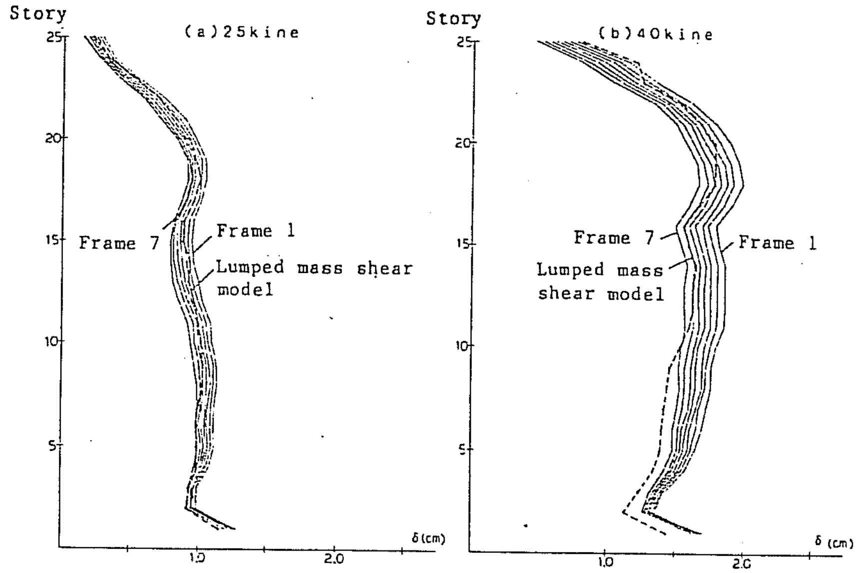


Fig. 7 Response Results of Shear Model
(Y Direction) -- Story Drift

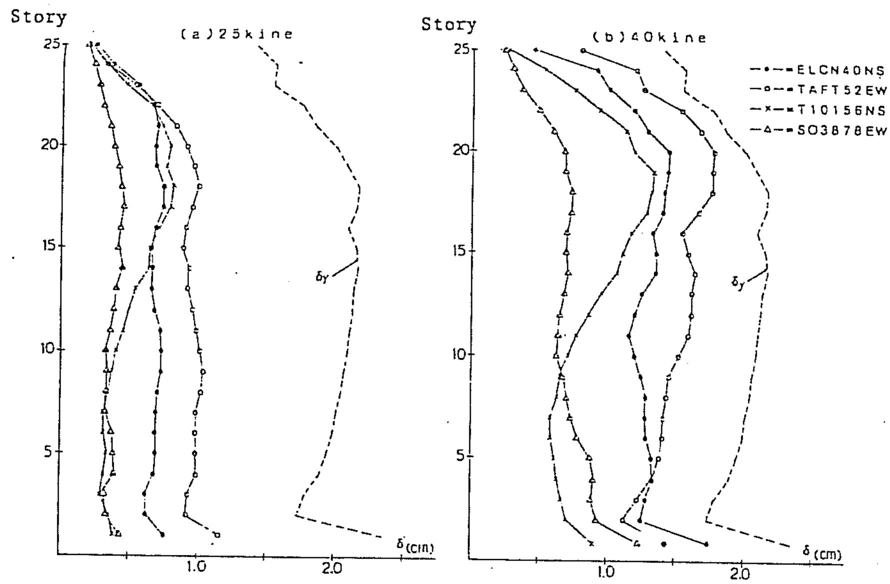


Fig. 6 Response Results of Two-dimensional Shear Model
(Y Direction, TAFT52EW) -- Story Drift

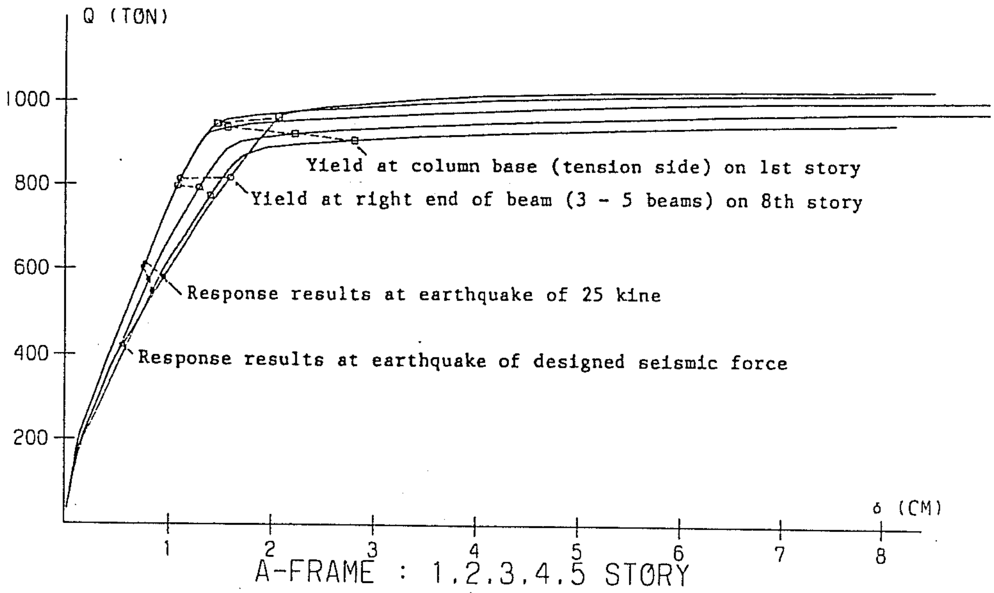


Fig. 8 Nonlinear Response of Beams and Columns to Earthquake of 25 Kine (Ⓐ Frame)

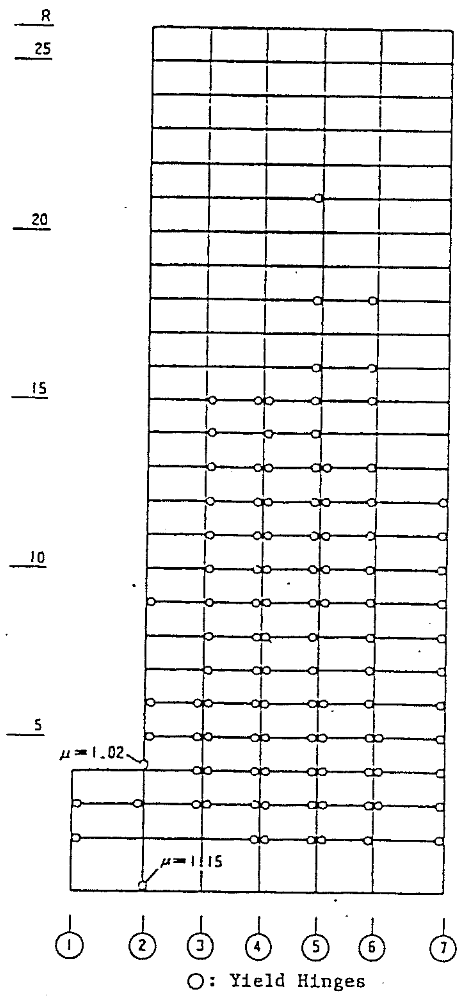


Fig. 9 Yield Hinges of Beams and Columns at Earthquake of 40 Kine (Ⓖ Frame)

7. Work Supervision

The construction work is being carried out using high strength concrete, PC members, etc., and conforms to the proposed revision (JASS 5) specified by the Japan Architectural Association. Further efforts, however, will be made to conduct more strict quality control on materials and work execution for ensuring stable quality and high precision of the building.

8. Concluding Remarks

We have explained the detailed design and construction of Nishi-Toyama Housing Complex with particular reference to the extentive use of PC members.

In the design of super highrise reinforced concrete structures, considerations are paid, regarding earthquake forces, to allow structures to maintain ductility such that the collapse mechanism of structures should be of bending yield type at each beam end. For the purpose of achieving that mechanism, structural members should be designed to be kept ductile and also the accurate prediction should be done on the large lateral displacement of structures which are subjected to severe earthquakes. Furthermore, when using the PC members for this type of structures, much consideration should be paid to proceed structural design from the initial stage regarding the actual construction on the site.

In designing the above building, the designed workability and aseismatic characteristics have been confirmed with mockups of the outer and inner frames.

(Acknowledgment)

This paper is based on the design materials prepared by Shinjuku Nishi-Toyama Development Project Designing and Supervising Consortium and approved by the Minister of Construction in March 1986. The author extends his sincere gratitude to all of the parties concerned for their kind cooperation.