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Advanced Design of Multi-story Reinforced Concrete Building

Conception moderne d'un bâtiment à plusieurs étages en béton armé

Fortschrittlicher Entwurf vielstöckiger Stahlbetongebäude

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SUMMARY
The design concept and design technique to make a multi-story reinforced concrete building a structure of high earthquake resistance are described, and a 30-story reinforced concrete model building is designed using the technique. The results of dynamic experiments on the frame and static experiments on columns of the model building are given, and it is indicated that the model building possesses ample ductility and excellent earthquake resistance.

RESUME
L'article présente la conception et les techniques de construction employées en vue de rendre hautement résistant aux tremblements de terre un bâtiment élevé en béton armé. Un modèle d'un bâtiment en béton armé de 30 étages est réalisé selon cette technique de construction. Les résultats des essais dynamiques sur le cadre et des essais statiques sur les colonnes de ce modèle sont présentés. La ductilité et la résistance aux tremblements de terre du modèle sont aussi présentées.

ZUSAMMENFASSUNG
Das Entwurfskonzept und die Entwurfstechnik, die zum Bau von vielstöckigen Stahlbetongebäuden mit hoher Widerstandsfähigkeit gegen Erdbeben nötig sind, werden beschrieben, und ein 30-stöckiges Stahlbetongebäude wird unter Verwendung dieser Technik entworfen. Die Ergebnisse der dynamischen Versuche mit dem Rahmen und der statischen Versuche mit den Stützen des modellierten Gebäudes werden beschrieben, und es wird aufgezeigt, dass das Modellgebäude ein grosses Mass an Duktilität besitzt und hervorragende Widerstandsfähigkeit gegen Erdbeben bietet.

1. INTRODUCTION

Multi-story reinforced concrete buildings have been built up to the present in various parts of the world subjected to comparatively severe earthquakes, but it cannot necessarily be said that earthquake-resistant design giving thorough consideration to properties peculiar to reinforced concrete structures has been carried out. In Japan, which is one of the most earthquake-prone countries in the world, it has generally been considered that reinforced concrete construction is inferior in earthquake resistance compared with steel construction and steel and reinforced concrete composite construction, and there have been few cases of construction of multi-story reinforced concrete buildings. However, reinforced concrete construction is a structural form which is amply capable of withstanding large earthquake forces if carefully designed. This paper, taking into consideration the characteristics of reinforced concrete structures, proposes a design concept and a design technique for rendering a multi-story reinforced concrete building into a ductile structure of highly reliable earthquake resistance. A 30-story reinforced concrete model building designed employing this technique is described, and the results of dynamic experiments on the component frames and static experiments of members are presented.

2. STRUCTURAL PLAN AND DESIGN FLOW

2.1 Structural Plan

2.1.1 Total Collapse Form of Beam-Yielding Type

Designing is done to make the building one of pure frame construction for an ultimate collapse form of beam-yielding type having large deformability as shown in Fig. 1. This collapse form is one in which damage from earthquake is not concentrated at a single portion, and instead, the entire building takes on a collapse form, which does not change for the frames of both directions against input of external seismic forces from two directions, and therefore, there is a high margin of safety against earthquake. With a multi-story building, tensile stresses are produced in the outermost columns and ultimate bending strengths are lowered, but designing is done so that a beam-yielding type will result as much as possible even in such case.

2.1.2 Cross-sectional Shape of Beam

As shown in Fig. 2, beams of small depth and large width are adopted to make it easier for plastic flanges to be produced at beam ends along with maintaining adequate shear strengths of beams.

2.1.3 Cross-sectional Shape of Column

Round columns are adopted and spiral hoops are used as shear reinforcement. Actually, in consideration of constructability, the concrete of a column is made octagonal as shown in Fig. 1. A round column using a spiral hoop provides high restraint against the core concrete and main reinforcement of the column, maintains sufficiently large ductility under high axial stresses.

2.1.4 Beam-to-Column Panel

Horizontal haunches are provided at beam-to-column panels as shown in Fig. 3 to secure anchored lengths of main reinforcing bars of beams. The horizontal haunches alleviate slipping phenomena of main reinforcement of beams and are useful for increasing shear strengths of panels.

2.1.5 Considerations of Two-direction Input

(1) Limitations to Axial Stresses of Columns

Corner columns are sometimes subjected to seismic forces in two directions to result in large axial forces, and therefore, the following limitations are provided:

- Under long-term load: ${}_{L}\sigma_{0} \leq 0.2F_{c}$
- Ultimate: ${}_{U}\sigma_{0} = {}_{L}\sigma_{0} + 1.5{}_{M}\sigma_{0} \leq 0.6F_{c}$

For columns in general:

- Under long-term load: ${}_{L}\sigma_{0} < 0.333F_{c}$
- Ultimate: ${}_{U}\sigma_{0} = {}_{L}\sigma_{0} + {}_{M}\sigma_{0} < 0.55F_{c}$

(2) Calculation of Main Reinforcement of Column

The flexure reinforcement at column end is computed using the ultimate strength equation for bending moment 1.5 times that for collapse mechanism in order that the column will not yield even when subjected to input from two directions and the beam-yielding type collapse form will be maintained as much as possible.

2.1.6 Extra Calculation of Design Shear Forces of Beams and Columns

Design shear stresses are increased by extra amounts as follows in order that adequate ductility, that is, excellent deformability of beams and columns of ultimate ductility factors not less than 6 will be secured.

- Beam: ${}_{D}Q_{G} = {}_{L}Q_{G} + 1.5{}_{M}Q_{G}$
- Column: ${}_{D}Q_{C} = 1.5{}_{M}Q_{C}$

2.2 Design Flow

The earthquake-resistant design flow of a multi-story reinforced concrete building consisting of a pure frame structure is shown in Fig. 4. Firstly, in preliminary calculations, the cross section is assumed taking into consideration axial stress coefficient σ_{0}/F_{c} of the column, and correspondence with the response spectrum, configurations of displacement modes, etc., are verified performing eigenvalue calculations. Next, earthquake loads, namely, base shear coefficient and shear force distribution, are assumed, and in addition, the entire structure is replaced by a one-mass-point system and a prediction is made of the response. If the results are satisfactory, the earthquake load is decided, and the process moves on to primary design. Here, allowable stress intensity designing is done for stresses under long-term load and under earthquake load. Next, a check is made of the axial stresses of columns at beam-yielding type collapse. The strength possessed is calculated carrying out computations of the column cross section by ultimate strength equation.

When the static design above has been completed, a response prediction is again made by the substitute one-mass-point system, and checks are made of displacements. Upon confirming that the results of the checks are satisfactory, the next step is elasto-plastic seismic response analyses, and response analyses by a mass-point system and frame response analyses are performed. It is ascertained by frame response analyses that the ductility factor of the story is not more than 1 for a medium-scale earthquake of 250 gal and not more than 2 for a large earthquake of 400 gal.

3. OUTLINE AND DYNAMIC DESIGN OF MODEL BUILDING

3.1 Outline of Building

The model building made the object of design was a 30-story reinforced concrete residential building, and a pure frame structure of uniform spans of 5.55 m as shown in Figs. 5 and 6. The standard story height was 2.85 m with only the first story 4 m. The columns were octagonal columns using spiral hoops with diameters 95 to 80 cm. Beam depths were the two kinds of 70 cm and 75 cm, with beam width made a wide 50 cm. Horizontal haunches were provided at beam-to-column panels. The concretes used were normal-weight concrete of F_c = 4,119 - 3,236 N/cm^2 and lightweight concrete of F_c = 2,942 ~ 2,648 N/cm^2.

3.2 Earthquake Load

Designing was done in accordance with the design flow of Fig. 4. The natural periods of the building determined were as shown in Table 1. The primary natural period was 1.81 sec. Base shear coefficients C_B were 0.08 for exterior columns and corner columns, and 0.1 for interior columns. The design shear forces for the individual stories were determined smoothing out Taft EW of the largest story shear force distribution from elastic earthquake responses of 100 gal as shown in Fig. 7.

3.3 Ultimate Strength Design of Beams and Columns

The cross sections of beams and columns were computed using the ultimate strength equation and the reinforcement quantities were determined. The results are given in Table 2 and Table 3.

3.4 Lumped Mass System Response Analysis

The displacements between stories obtained from lumped mass system responses are shown in Fig. 8. The solid lines in the figure are yielding displacements δ_y. With responses of 250 gal and 400 gal, there are portions of responses exceeding δ_y, or 2 δ_y, but this is allowed considering that smooth response displacements would be obtained if frame analyses are performed.

3.5 Frame Response Analysis

Elasto-plastic earthquake response frame analyses were performed, and the vibration properties of the model building over the elastic and plastic ranges, namely, elasto-plastic behaviors of the entire building, stories, and beams and columns, were studied. Analysis was performed only on Frame c, and Hachinohe NS, with highest response level and primary vibration predominant, was used as the input seismic wave. The maximum input acceleration was 400 gal considering a great earthquake.

The distribution of the maximum response values of displacements between stories is shown in Fig. 9. The displacements between stories in frame analyses were largest at the 6th story with 3.06 cm (R = 1/93) and the 21st story with 3.22 cm (R = 1/89). The ductility factors of the stories determined by dividing displacements between stories by yielding displacement of a lumped mass system were μ = 0.52 - 1.88 to be within the range of design target values for ductility factors of stories of not more than 2.

The locations of yielding hinge occurrence of beams and columns in frame analyses and the ductility factors of yielded members are shown in Fig. 10. Except at the bases of the first-story exterior columns where axial forces due to earthquake force are on the tension side, yielding hinges did not occur at columns, with yielding hinges produced at beams in the 1st to 26th stories. The ductility factor μ of a member which had yielded was 1.0 - 3.0, and the target value of $\mu < 4$ was satisfied.

4. STATIC AND DYNAMIC TESTS OF MEMBERS

With the model building as the object, static experiments of columns including panels of beams and columns, and dynamic experiemnts of frames were conducted, and the results are shown below.

4.1 Experiments on Columns Including Beam-and-Column Panels

The specimen is shown in Fig. 11. This specimen considered an interior column at the first story and was approximately one half the size of actual. The column was octagonal, circumscribing a circle of 45-cm diameter, with horizontal haunches provided at the beam-to-column panel. Loading was done as shown in Fig. 12 in order that plastic hinges would be produced at beam ends and column bases to result in a collapse form. Axial forces equal to the limit to axial stress intensity of 0.55 F_c of interior columns described in Chapter 2 were made to act on the columns. The concrete strength of this specimen was F_c = 4,217 N/cm^2. The failure condition and load-deformation curve of the specimen are shown in Figs. 13 and 14. As expected, plastic hinges were produced at column bases and beam ends, and failure occurred in collapse form. There was hardly any strength reduction up to joint translation angle of approximately 1/20, and a large deformability was indicated.

4.2 Dynamic Experiments of 6-Story Frame

Dynamic experiments of a 6-story frame were conducted in order to ascertain the safety during earthquake of a 30-story model building. The specimen is shown in Fig. 15. It is approximately 1/7 of actual in scale, the columns being octagonal to circumscribe circles of 13.5 cm. Beams were of width of 70 mm and depth of 105 mm, with horizontal haunches provided at beam-to-column panels. Added weight of 9 tons was applied to the columns, and axial force of σ_0 = 622 N/cm^2 was made to act. The specimen was a frame taken out from the bending inflection point of the beam, and in order to maintain this portion horizontal during vibration, steel supports were erected at beam ends and the beam and pin joints were provided with the beam. Hachinohe NS seismic waves were made to act on the specimen. From the correspondence between period of specimen, period of 30-story building and seismic waves, the time axis of input seismic wave was reduced to 1/4, and maximum acceleration was considered as 350 gal from the limitations due to the characteristics of the shaking table. The maximum response value is shown in Table 4, and the load-deformation curve, and the measured and calculated values of displacements and acceleration responses in Figs. 16 and 17, respectively. At acceleration of 350 gal, the joint translation angle of the specimen was 1/69, and the ductility factor 1.36.

5. CONCLUSIONS

The building designed according to the design concept indicated here, was recognized to be of amply high earthquake resistance during a strong earthquake according to an example of design of a 30-story reinforced concrete building and the results of experiments on component members of the building.

NOTATIONS

- ${}_L\sigma_0$: axial stress intensity under long-term load
- ${}_M\sigma_0$: axial stress intensity at collapse mechanism
- ${}_U\sigma_0$: ultimate axial stress intensity
- F_c: specified concrete strength
- ${}_DQ_G$, ${}_DQ_C$: design shear forces of beam and column, respectively
- ${}_LQ_G$: shear force under long-term load
- ${}_MQ_G$, ${}_MQ_C$: shear forces at collapse mechanism of beam and column, respectively

REFERENCE

1) Takeda, T., Yoshioka, K., Eto, H., and Tada, T., "Study on Aseismic Design of High Rise Reinforced Concrete Buildings, (Part 1), (Part 2), (Part 3)," (in Japanese), Trans. of AIJ, Extra, September 1983, pp. 1663-1668.

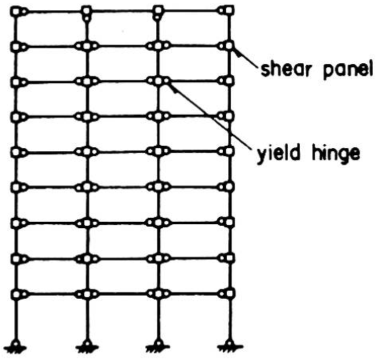

Fig. 1
Frame of Beam-Yielding Type

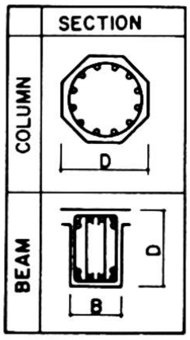

Fig. 2
Cross Section of Column and Beam

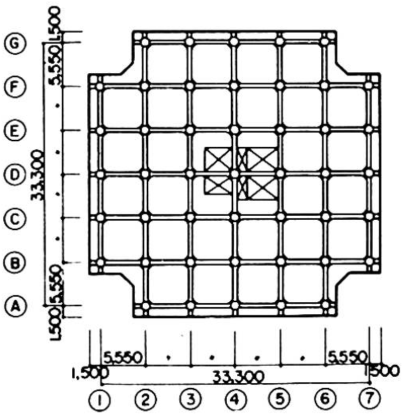

Fig. 5
Prototype Building Plan

Table 1
Natural Period of Prototype Building

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

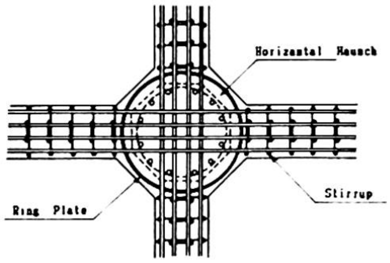

Fig. 3 Column-Beam Connection

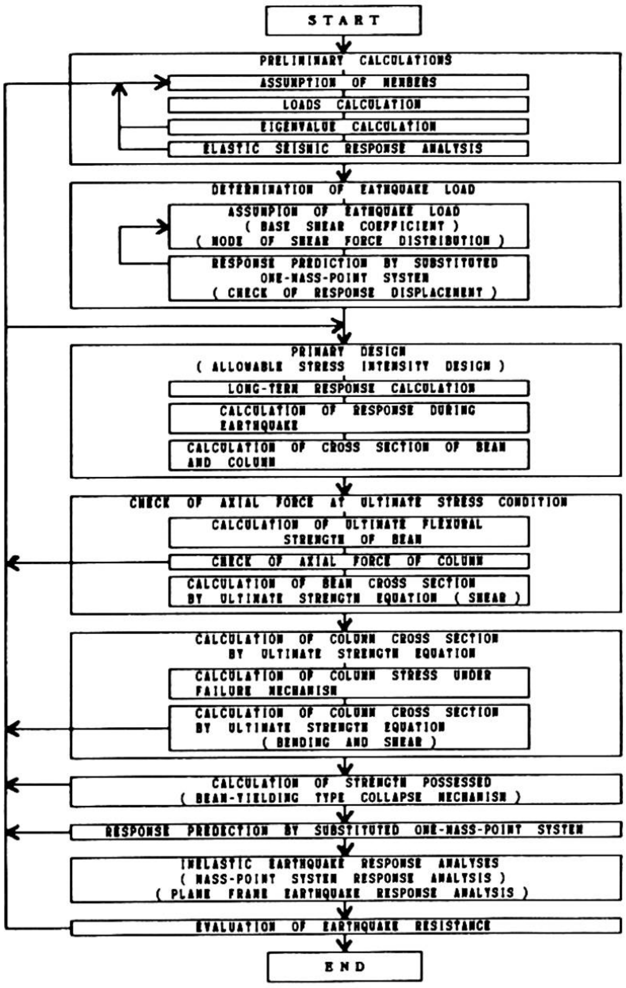

Fig. 4 Design Flow

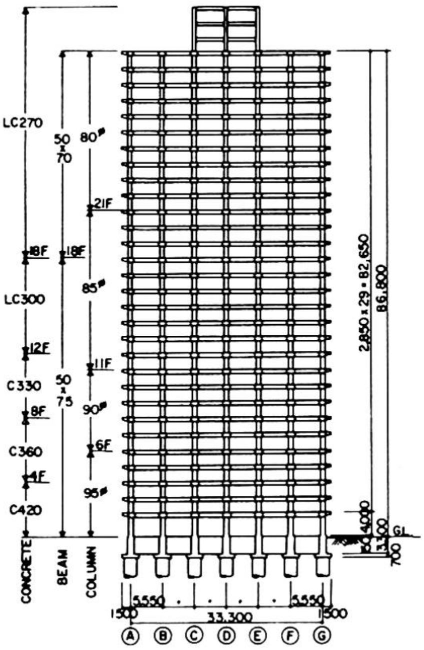

Fig. 6 Prototype Building Section

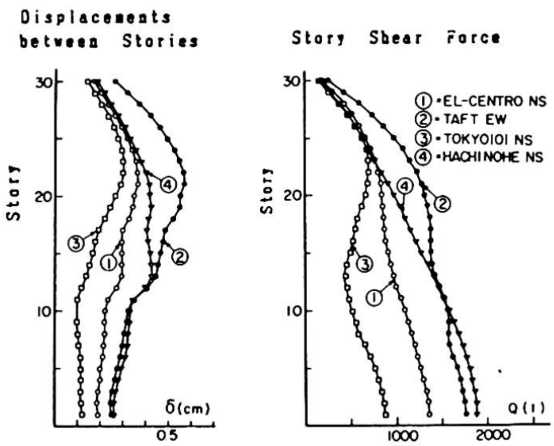

Fig. 7
Displacements between Stories and Story Shear Force Calculated by Elastic Earthquake Response Analysis

Table 2 List of Beams

story	cross section b×D (cm × cm)	location	exterior frame: main reinforcement, exterior end	exterior frame: main reinforcement, exterior end	exterior frame: stirrup tie	interior frame: main reinforcement, both ends	interior frame: stirrup tie
30 ~ 28	50×70	upper	3 - D22	3 - D22	4 - D10 @ 150	5 - D22	4 - D13 @ 150
		lower	2 - D22	2 - D22		4 - D22	
27 ~ 25		upper	3 - D25	2 - D25 1 - D22		4 - D25 2 - D22	
		lower	3 - D22	3 - D22		3 - D25 2 - D22	
24 ~ 21		upper	2 - D29 2 - D25	3 - D29	4 - D12 @ 150	4 - D29 2 - D25	4 - D10 @ 150
		lower	2 - D25 2 - D22	2 - D25 1 - D22		3 - D29 2 - D25	
20 ~ 18		upper	2 - D29 2 - D25	2 - D29 2 - D22		6 - D29	
		lower	2 - D29 1 - D25	2 - D29 1 - D22		5 - D29	
17 ~ 11	50×75	upper	2 - D32 2 - D25	2 - D32 1 - D29		2 - D32 4 - D29	
		lower	2 - D29 2 - D22	2 - D29 1 - D22		4 - D29 2 - D25	
10 ~ 6		upper	2 - D32 2 - D29	2 - D32 2 - D25		2 - D32 4 - D29	
		lower	2 - D29 2 - D25	3 - D29		4 - D29 2 - D25	
5 ~ 1		upper	2 - D32 3 - D25	2 - D32 2 - D29		2 - D32 4 - D29	
		lower	2 - D29 3 - D25	4 - D29		4 - D29 2 - D25	

* main reinforcing bar: SD40, stirrup • tie: SD35
at the both ends of beams ties are providid at the space of 100 mm within the same extent as beam depth

Table 3 List of Columns

story	cross section D (cm)	C_1 (exterior column): main reinforcement	C_1 (exterior column): hoop	C_2 (interior column): main reinforcement	C_2 (interior column): hoop
30 ~ 28	80φ	12 - D35	spiral hoop D10 @100	12 - D35	spiral hoop D10 @100
27 ~ 25					spiral hoop D10 @100
24 ~ 21					spiral hoop D10 @60
20 ~ 18	85φ				
17 ~ 11					spiral hoop D10 @60
10 ~ 6	90φ		spiral hoop D10 @60		
5 ~ 1	95φ				

* Main reinforcement : SD40, spiral hoop : SD35
At the bottom of columns of the first floor, sub hoops D10 are provided at the space of 60mm within the extent of one and half times column diameter

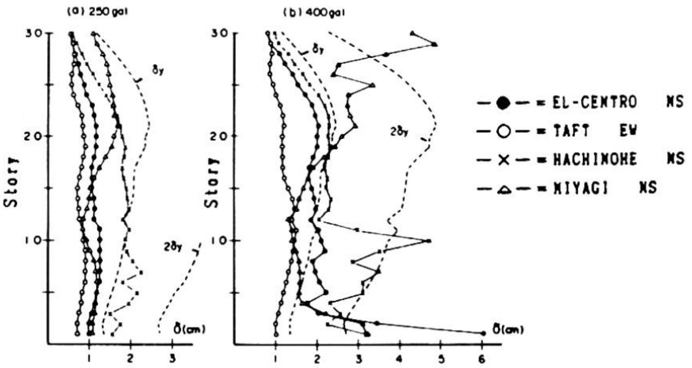

Fig. 8 Displacements between Stories Calculated by Inelastic Mass-Point System Response Analysis

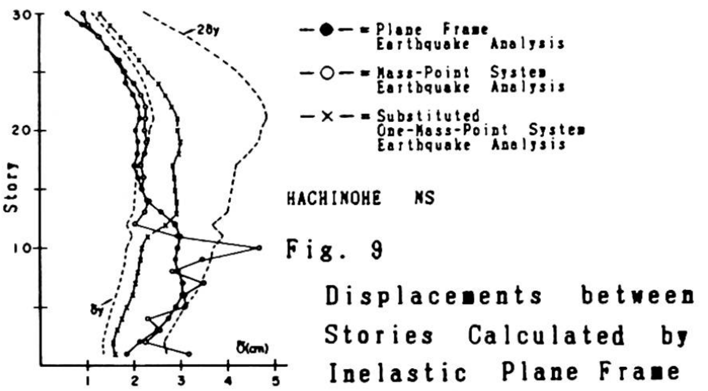

Fig. 9 Displacements between Stories Calculated by Inelastic Plane Frame Earthquake Response

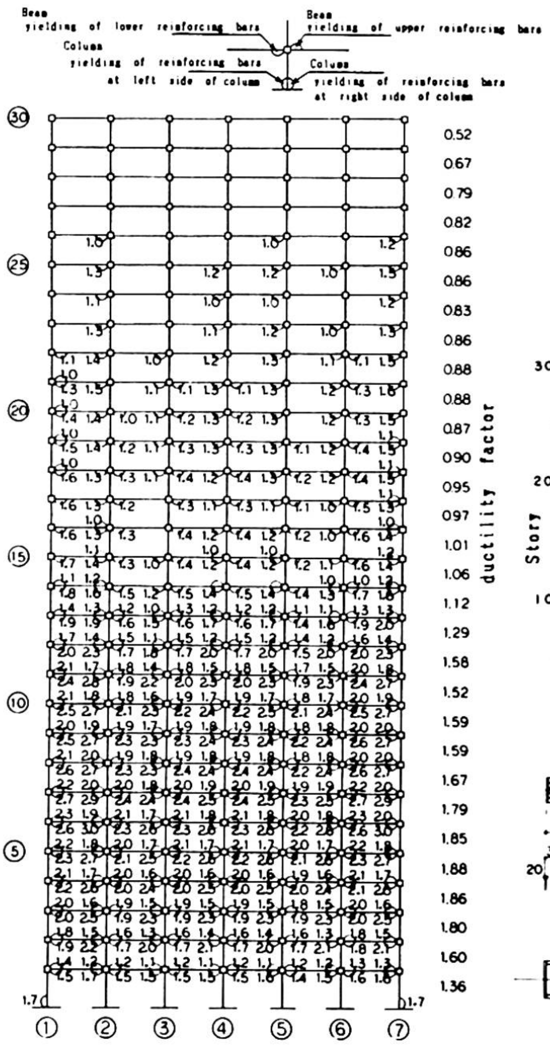

Fig.10 Yield Hinge of Beam and Column and Ductility Factor of Each Story

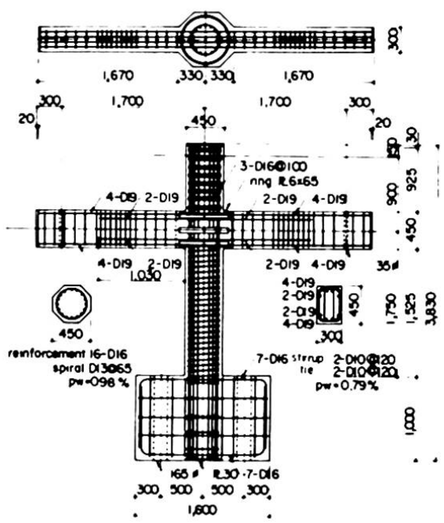

Fig.11 Specimen of Static Test

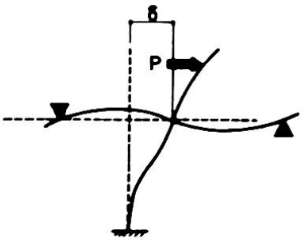

Fig.12

Loading Method and Deformation of Specimen

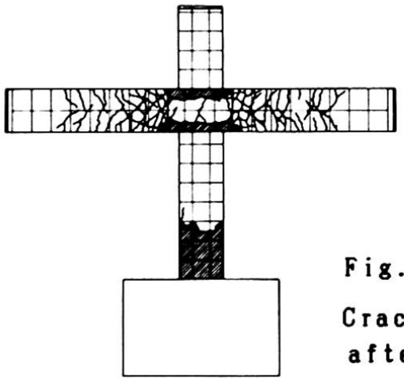

Fig.13

Cracking Pattern after Test

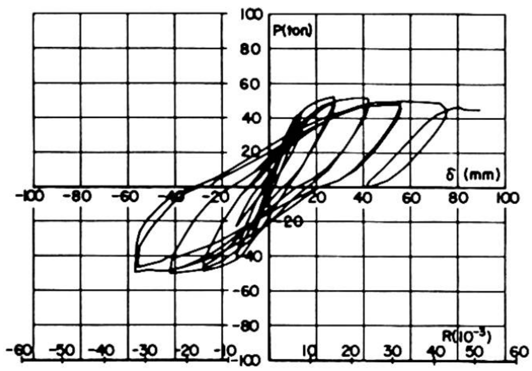

Fig.14

Load-Displacement Relationship

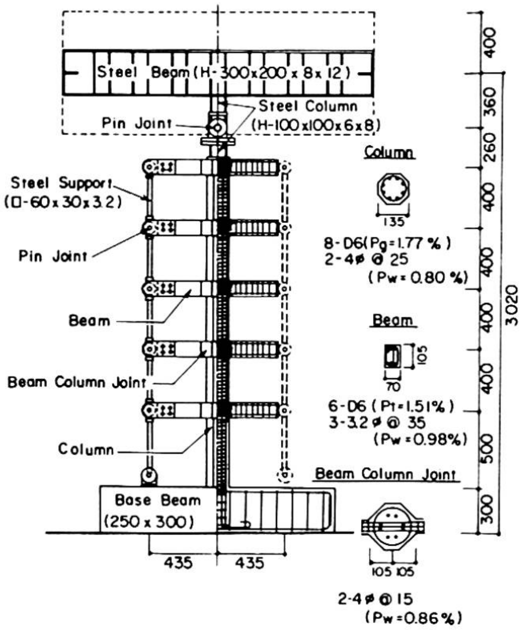

Fig.15

Specimen of Dynamic Test

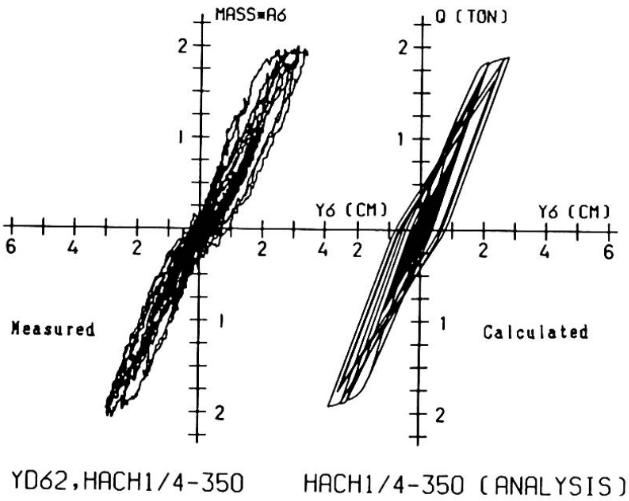

Fig.16

Force-Displacement Relationship in Test and Analysis

Table 4

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)
YD62	HACH1/4 - 350	342 (5.162)	226 (7.528)	3.40 (9.302)	1/69	1.36
	HACH1/1 - 400	406 (18.175)	249 (18.305)	9.31 (36.875)	1/25	3.72

( ) = Occurrence Time (sec)

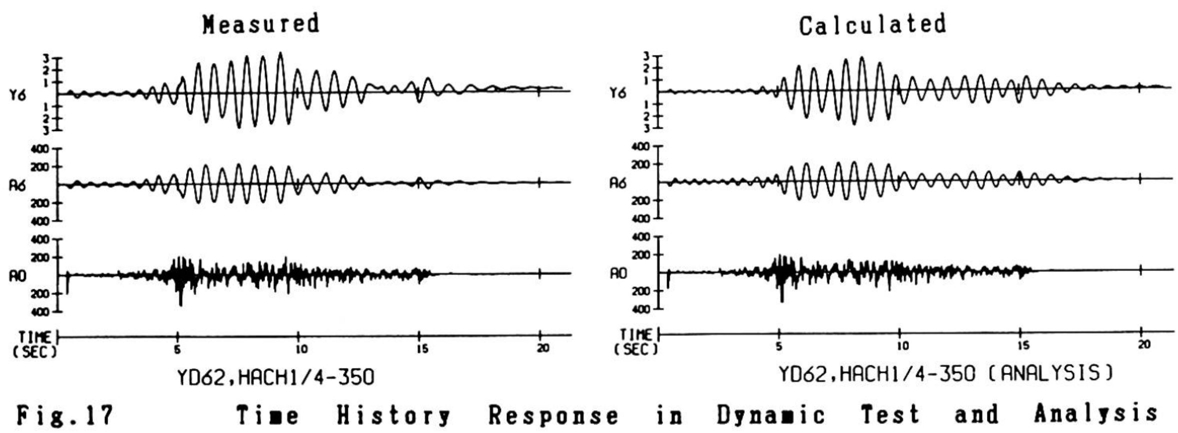

Fig.17 Time History Response in Dynamic Test and Analysis