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DEVELOPMENT OF A SUPER-HIGH-RISE REINFORCED CONCRETE FLAT PLATE BUILDING

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ABSTRACT

A five-year National Project (simply referred to as "New RC") for development of structures such as super-high-rise buildings has been promoted by the Ministry of Construction of Japan since 1988. This paper reports the development in the last two years of a trial design of super-high-rise reinforced concrete flat plate type buildings and an experimental study on sub-assemblages of columns and flat plates and of shear walls and flat plates. Two types of structural system of the core wall type and the wall type were employed in the trial design, based on structural design guidelines proposed by the New RC project. The trial design and the experimental study confirmed that a super-high-rise reinforced-concrete flat-plate-type building can be realized in an intensive seismic area like Japan.

KEYWORDS

New RC project; trial design of super-high-rise RC structure; flat plate; core wall type; wall type; structural design guideline; experiments of flat plate subassemblages

SEISMIC DESIGN EARTHQUAKE GROUND MOTION AND DESIGN CRITERIA

In the seismic design, two kinds of earthquake ground motion, called simulated earthquake ground motions, were employed as shown in Table 1. The Level 1 earthquake ground motion is defined as the maximum possible ground motion and the Level 2 earthquake ground motion, as the maximum credible ground motions at the construction site. These earthquake ground motions were proposed by the New RC project.

Seismic design criteria for dynamic response analysis of flat plate type buildings were based on structural design guidelines proposed by the New RC project. The maximum responses for two levels of earthquake ground motion, Level 1 and Level 2, shall meet the conditions shown in Table 2.

The flat plate was designed to ensure residential functionality even after a large earthquake. The residential functionality was estimated based on the rank of floor vibration functionality according to the Guidelines for the evaluation of habitability to building vibration by AIJ. The rank of floor vibration functionality after Level 1 and Level 2 earthquakes were Rank 1 and Rank 2, respectively.

Table 1. Earthquake ground motion for seismic design

Name of earthquake ground motion	Level 1 Max. Acc.	Level 1 Max. Vel.	Level 2 Max. Acc.	Level 2 Max. Vel.	Phase characteristics
New RC 01	158cm/sec^2	24.6cm/sec	395 cm/sec^2	58.9cm/sec	Random
New RC 02	163cm/sec^2	25.8cm/sec	407 cm/sec^2	64.6cm/sec	Hachinohe

Table 2. Seismic Design Criteria

Earthquake ground motion	Maximum response value	Criteria
Level 1	a)Maximum interstroy deflection angle	≦1/200
	b)Member ductility factor for boundary beam	≦ 2.0
	c)Member ductility factor for other members	≦ 1.0
Level 2	a)Maximum deflection angle at the centroid of the seismic lateral forces	≦ 1/120
	b)Maximum interstroy deflection angle	≦ 1/100
	c)Member ductility factor for boundary beam	≦ 4.0
	d)Member ductility factor for flat plate	≦ 2.0
	e)Member ductility factor for other members	≦ 1.0

TRIAL DESIGN OF CORE WALL TYPE APARTMENT BUILDING

Outline of Structure

The trial design structure is a 50-story, 151.5-meters high reinforced concrete apartment building. The standard plan form is square with the core wall at the center of the plane, as shown in Fig. 1. It consists of a 41.8m×41.8m typical floor area and 16.2m×16.2m center core area. The center core area is enclosed by four L-type shear walls. Maximum member sizes are 95cm×95cm for interior columns, 80cm×80cm for exterior columns, 95cm for wall thickness, 95cm×80cm for boundary beams and 25cm for flat plate thickness. The maximum compressive strength of concrete employed in the structural design is 1000kgf/cm^2. The yield strength of longitudinal reinforcement in columns, boundary beams and shear walls; shear reinforcement in columns and boundary beams; and transverse reinforcement in shear walls are 7000kgf/cm^2, 13000kgf/cm^2 and 10000kgf/cm^2, respectively. To prevent punching failure of the flat plate at the column-plate connection, the yield strength of flat plate reinforcement is specified to be 3500kgf/cm^2.

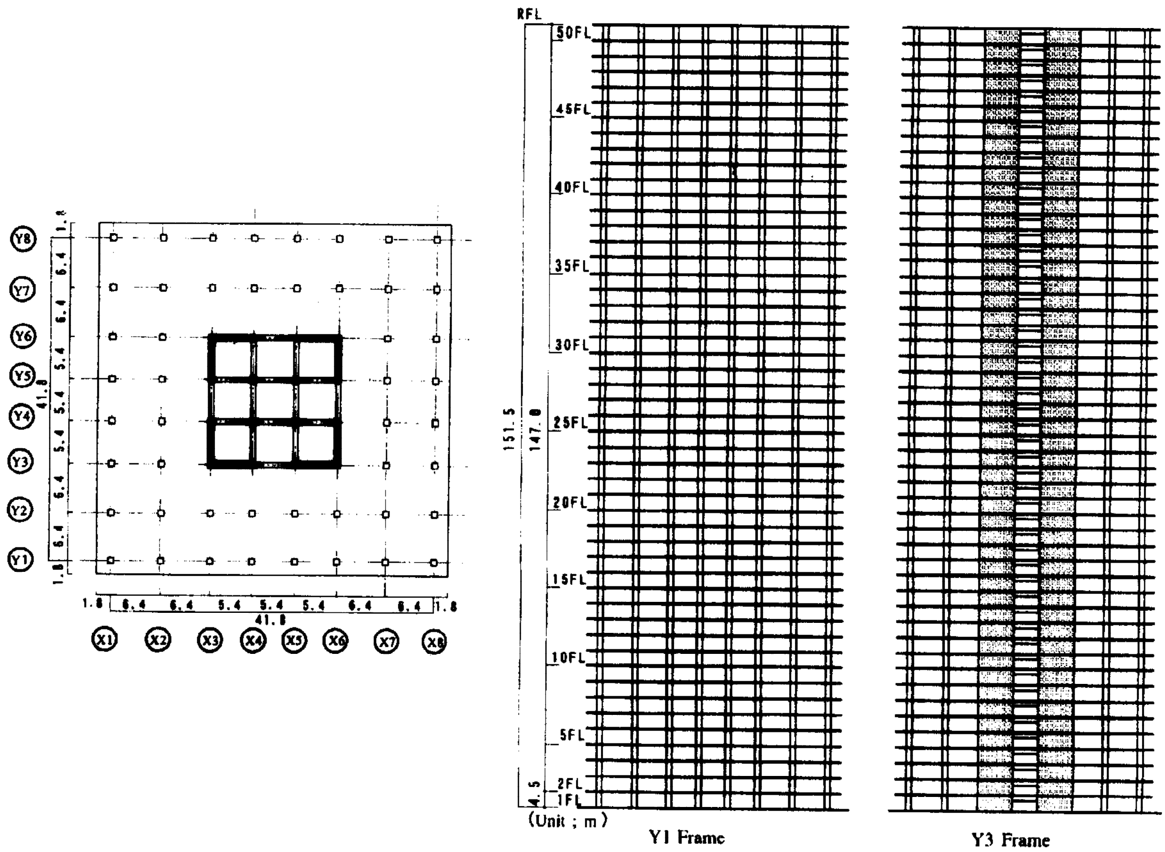


Fig. 1. A flat plate structure of core wall type

Nonlinear Dynamic Response Analysis

Nonlinear dynamic response analyses for Level 1 and Level 2 were carried out using the plane frame model. The effective width of the flat plate was evaluated by the finite element method. A degrading tri-linear type model of hysteresis loops obtained from the experimental study were used for the moment-rotation relation of the flat plate. For the force restoring characteristics of the wall, was employed the degrading tri-linear model evaluated by the increment analyses of the fiber model on plane maintenance assumption. The damping coefficient of 3% of the instantaneous stiffness of the structure was applied.

The distributions of the maximum story drifts obtained by the nonlinear dynamic response analyses due to the Level 2 are shown in Fig. 2. The maximum story drifts were 1/261 rad. due to Level 1 and 1/106 rad. due to Level 2. The maximum deflection angle at the centroid of the seismic lateral forces was 1/137 rad. The occurrence state of cracking and yield hinges due to the Level 2 are shown in Fig. 3. The yield hinges were generated at the end of the boundary beams and the flat plates located in the middle stories. The obtained maximum plastic ratio of flat plates and boundary beams were 1.09 and 2.02, respectively. The maximum response values obtained by non-linear dynamic response analyses satisfied the seismic design criteria.

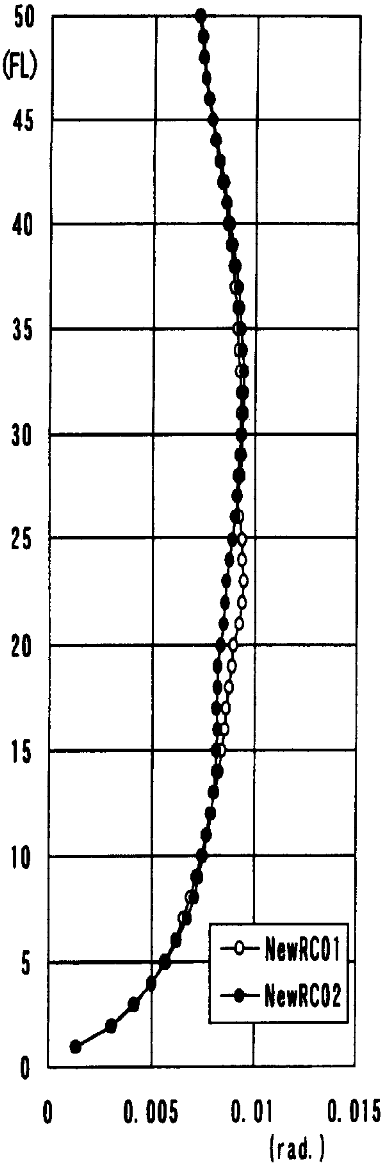


Fig. 2. Distribution of the maximum story drift

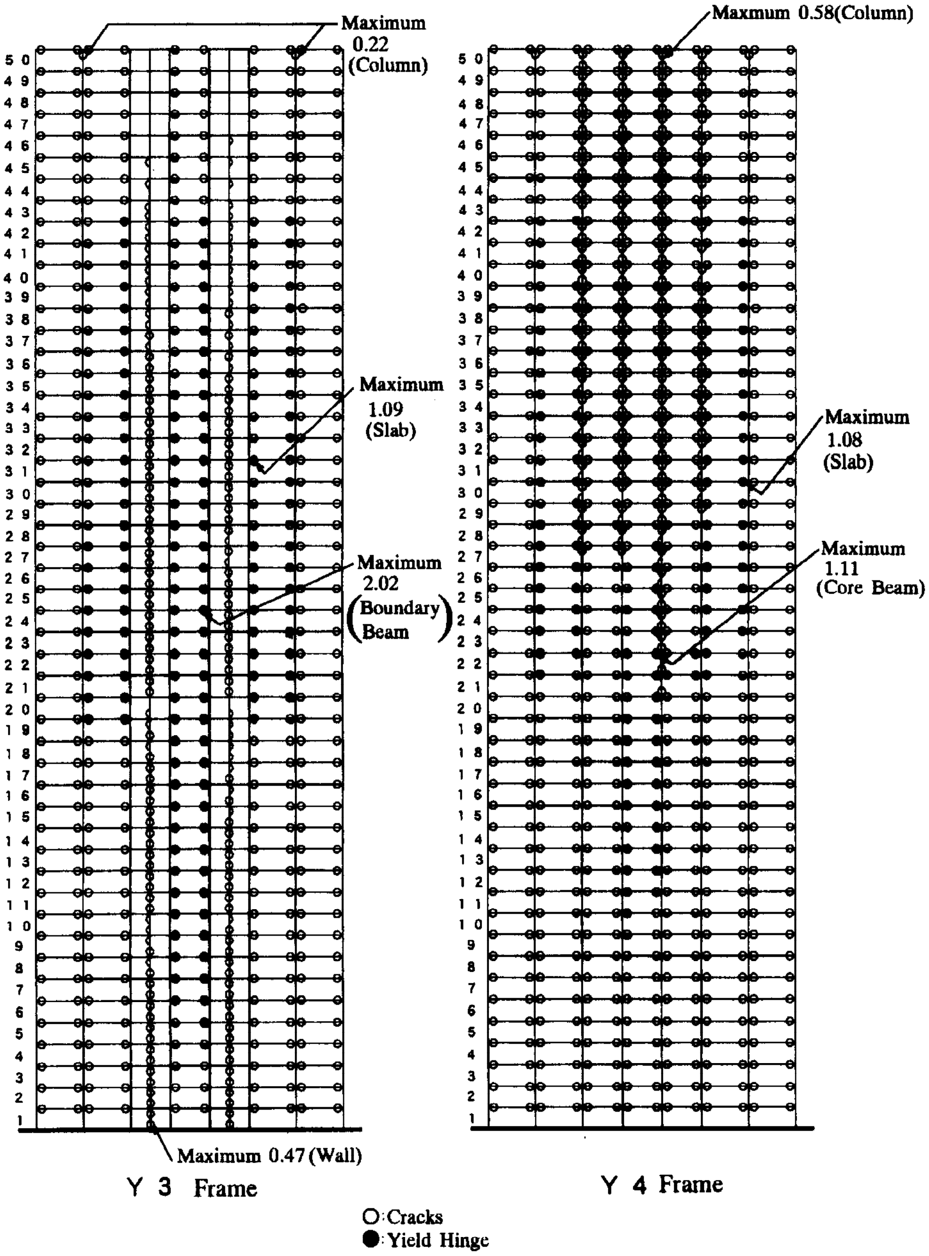


Fig. 3. Occurrence state of cracking and yield hinges

TRIAL DESIGN OF THE WALL TYPE FOR RESORT CONDOMINIUM

Outline of Structure

The trial design structure is a 40-story, 123-meter high reinforced concrete resort condominium. The standard plan form is a line-symmetric plane with curves as shown in Fig. 4. The earthquake resistant elements are the division walls between residential quarters in the Y direction, the curved walls at the core section, and two large walls in the X direction. The angle between the two large walls was determined so as to minimize planar torsion. Superbeams connecting the division walls and the curved walls are configured on the 14th, 27th and 40th floors in order to produce a bending relaxation effect. Maximum member sizes are 40cm × 300cm for the superbeams, 60cm for wall thickness and 25cm for flat plate thickness. The maximum compressive strength of concrete employed in the structural design is $1000 kgf/cm^2$. The yield strength of longitudinal reinforcement in the superbeams and both direction reinforcement in the shear walls, shear reinforcement in the superbeams, and flat plate reinforcement are $10000 kgf/cm^2$, $13000 kgf/cm^2$ and $5000 kgf/cm^2$, respectively.

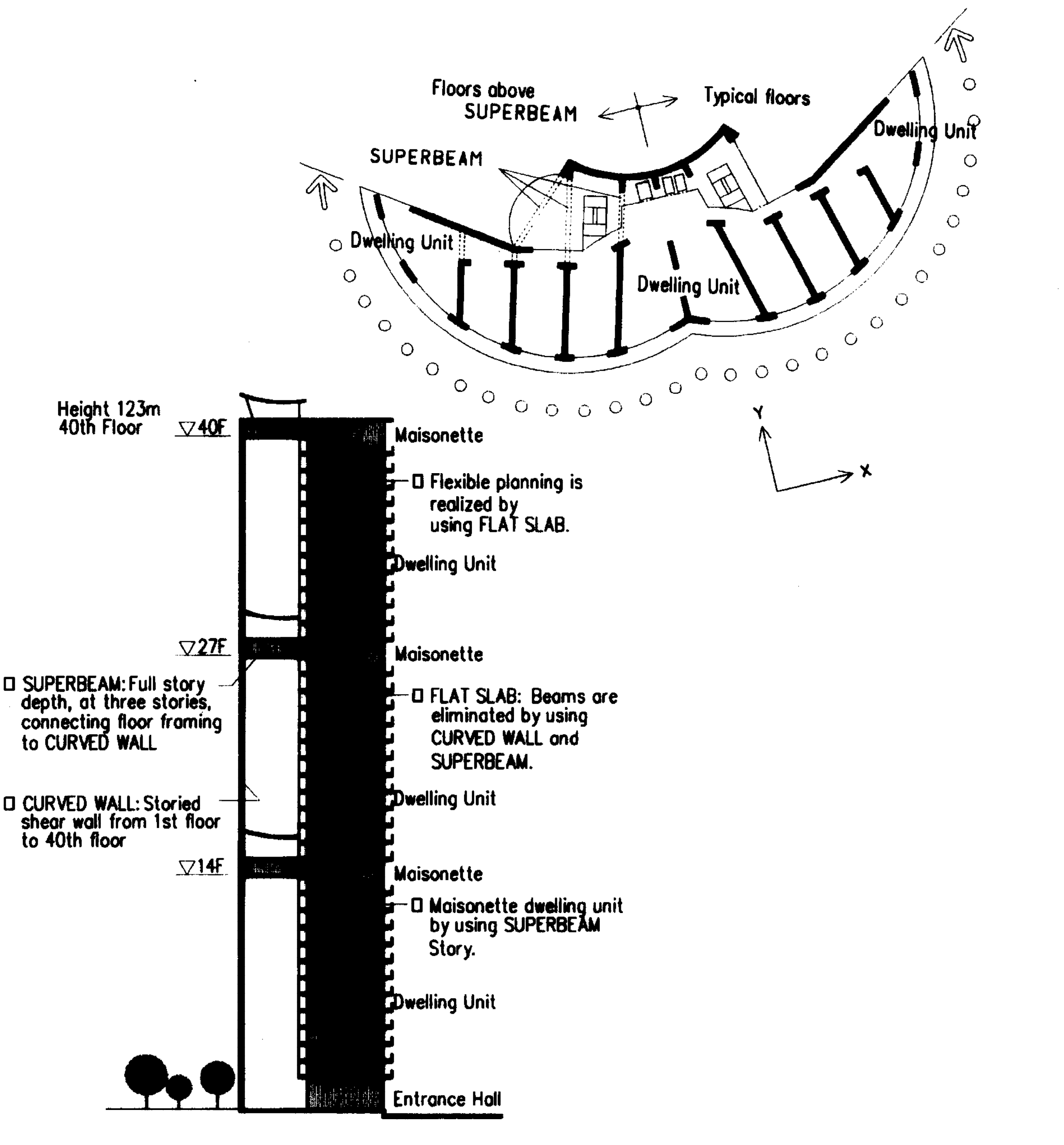


Fig. 4. A flat plate structure of wall type

Nonlinear Dynamic Response Analysis

Nonlinear dynamic response analyses for Level 1 and Level 2 were carried out using the lumped mass model. To model the building for these analysis, the stiffness of the entire main frame comprising shear walls, flat plates and superbeams was replaced by equivalent shear springs and equivalent torsion springs in the X and Y directions. For floors which have superbeams, two floors were concentrated to a single mass point and a quasi 3-dimensional model comprising a total of 37 mass points was implemented. The force restoring characteristics of the equivalent shear springs were the bi-linear type. The damping coefficient was estimated as Rayleigh damping, in which the numerical values of the coefficient were 0.03 for the primary frequency and 0.04 for the secondary frequency. To evaluate the safety of the randomly configured shear walls and the superbeams, the directions parallel to the division wall for residential quarters and the large walls were taken as input directions in addition to the X and Y directions.

The distributions of maximum story drift and maximum shear force obtained by the nonlinear dynamic response analyses due to the Level 2 in the X direction are shown in Fig. 5. Under Level 2, the maximum story drift was 1/250 rad. and the maximum deflection angle at the centroid of the seismic lateral forces was 1/347 rad. The results of nonlinear dynamic response analyses for other directions were similar to those in the X direction. The maximum response values obtained by non-linear dynamic response analyses satisfied the seismic design criteria.

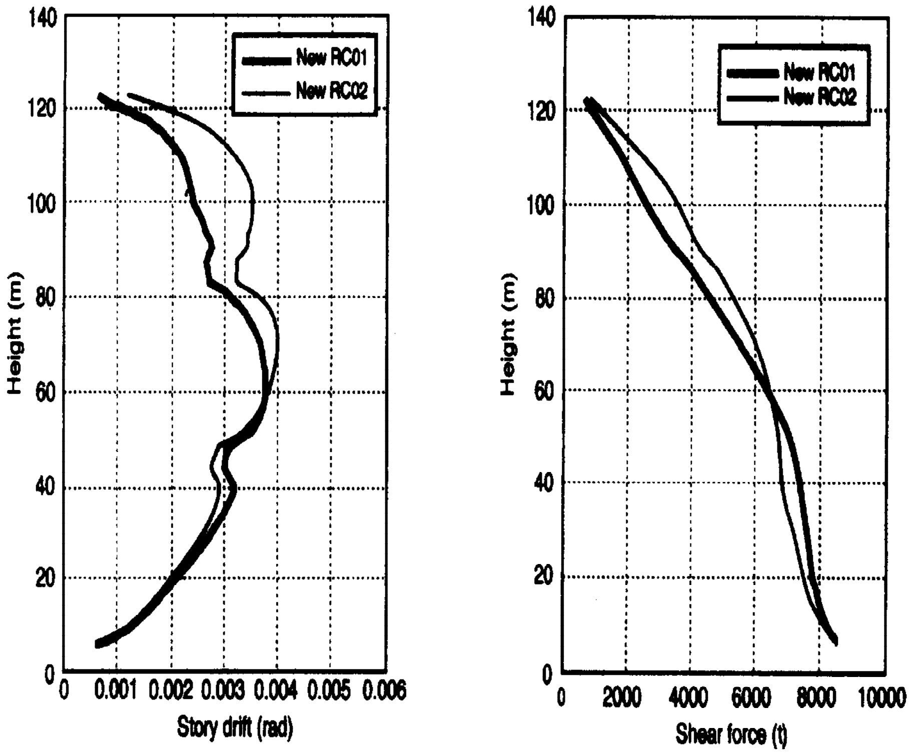


Fig. 5. Distributions of maximum story drift and maximum shear force in the X direction

EXPERIMENTAL STUDY OF SUBASSEMBLAGES

Outline of Experimental Study

Flat plate buildings haven't been used for major seismic structural system in Japan, so few experimental studies have been carried out relating to them, especially with high strength concrete and reinforcing bars. The objectives of this experimental study were to investigate the seismic capacity and the behavior of column-flat plate connections. The purpose of the experimental study are as follows;

i) to evaluate the influence of a balcony at the free edge of the flat plate on shear strength and deformation of the connection

ii) to determine the skeleton curve of the load-deflection relationship

iii) to estimate the effective width of the flat plate

iv) to compare the strength of specimens with the values calculated by the current AIJ equations

v) to survey the structural performance of corner column-flat plate connections under 45 degree loading

The experimental study consisted of four series of tests: on inner column-flat plate connections, on edge column-flat plate connections, on corner column-flat plate connections and on wall column-flat plate connections. The portions of specimens in the proposed building are shown schematically in Fig. 6. The specimens were 1/2.5 scale models of the proposed building. The column was 250mm × 250mm and the flat plate was 100mm thick. The design strength of concrete was 600 kgf/cm^2 and the reinforcing bars in the flat plate were SD295 (normal yield strength of 3000 kgf/cm^2). The shape and reinforcing bar arrangement of edge column-flat plate connection specimen are shown in Fig. 7.

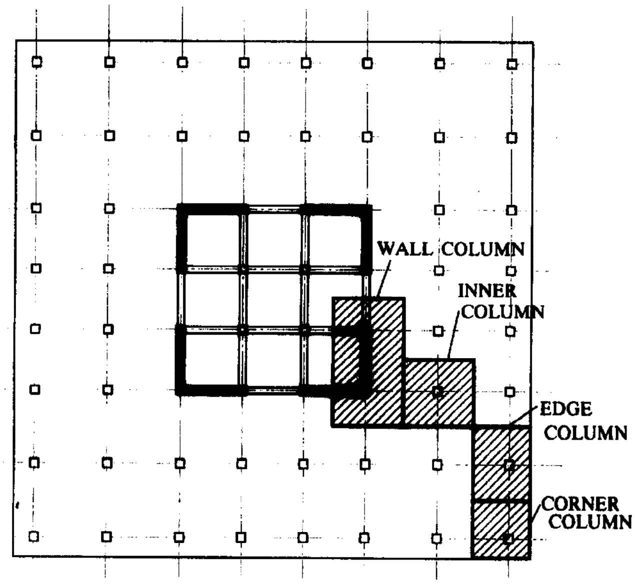


Fig. 6 Portions of specimens in the proposed building

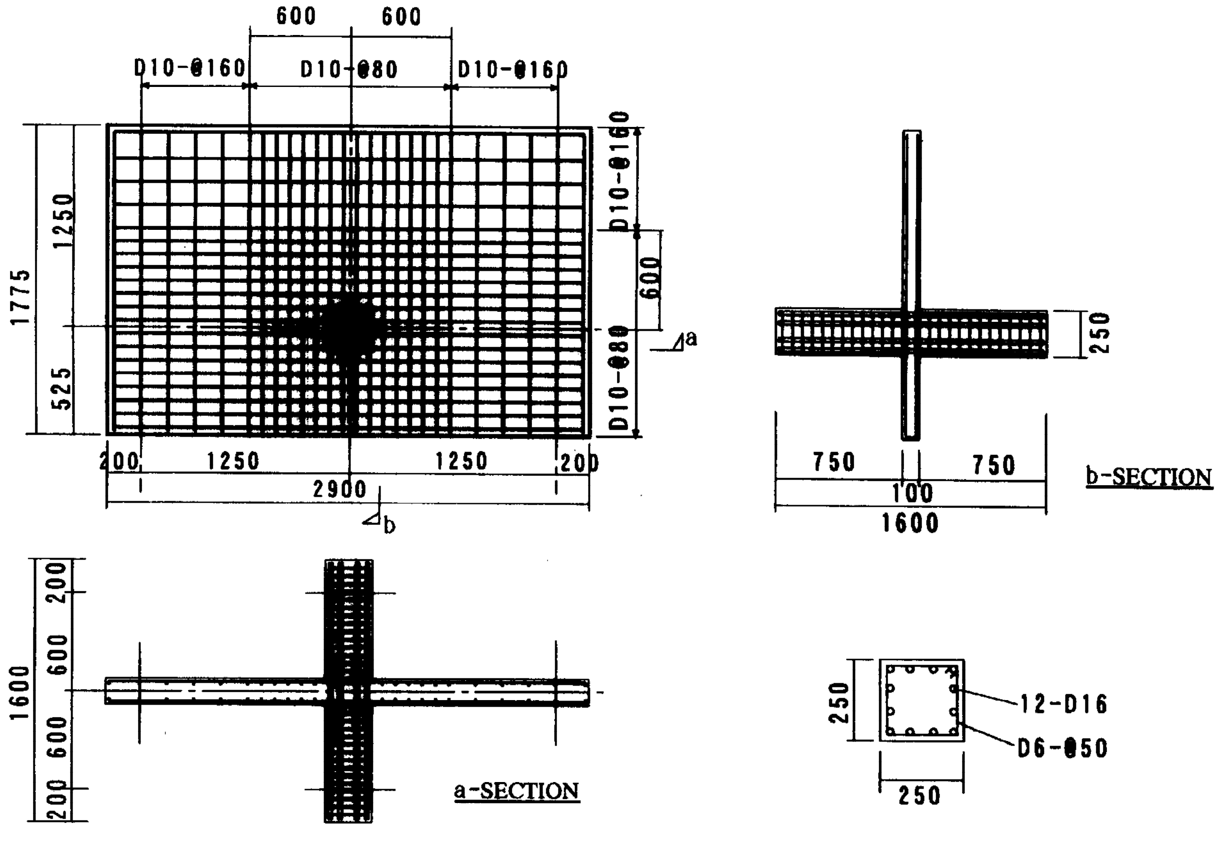


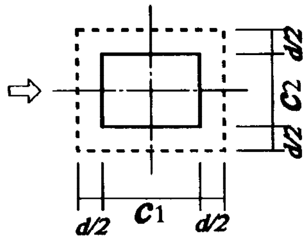
Fig. 7 Shape and reinforcing bar arrangement of edge column-flat plate connection series

Results

The load-deflection relationships of the edge column-flat plate connections are shown in Fig. 8. The difference between these specimens is the existence or not of a balcony at the free edge of flat plate. The force restoring characteristics of each specimens were stable up to 2% in drift angle. The ultimate strength of the specimen with a balcony was larger then that without a balcony.

The relationship between Vtest/V0 and Mtest/M0 according to AIJ current equations is shown in Fig. 9. The relationship of total shear strength and moment transferred from the flat plate are obtained from former experiments as shown bellow;

$$\frac{Vu}{V0} + \frac{Mu}{M0} = 1$$



where, Vu : ultimate shear strength by vertical load
Mu : ultimate bending moment
V0 : ultimate shear strength when only vertical load is applied
$V0 = \tau u\ Ac$
$Ac = 2d(C1+C2+2d)$
M0 : ultimate bending moment when only moment is applied
$M0 = Mf+Ms+Mt$
Mf : moment transferred by bending of flat plate at peripheral section
Ms : moment by shear force at peripheral section
Mt : moment by torsion at peripheral section

When (Vtest/V0)+(Mtest/M0) is less than 1, the failure mode of specimens was flexural yield of the flat plate. The specimens whose failure mode was punching failure also had enough ductility. The ultimate torsional moments of specimens of the wall column-flat plate series are very large.

The effective width ratio necessary to model the flat plate structure as the equivalent frame structure is shown in Fig.10. The drift angle increased, as the effective width ratio decreased. The reinforcement ratio of the inner column-flat plate series was smaller than that of the other series, and the effective width of the inner column - flat plate series tended to be smaller than that of other series was obtained.

The skeleton curve of the load-deflection relationship was represented by a tri-linear model as shown in Fig. 11. The initial and second stiffness were determined by multiplying the elastic stiffness EI by $\alpha 1$ and $\alpha 2$, respectively, where EI is calculated assuming that the entire width of flat plate was effective, and the reduction factors $\alpha 1$ and $\alpha 2$ were derived from former experimental data. The first break point of the skeleton curve was at 0.5% of the drift angle. This modeling the by tri-linear curve adequately represents the experimental result.

The equivalent damping factors obtained from the experimental study were 5~10% in the range of the supposed design drift angle of 5~10%.

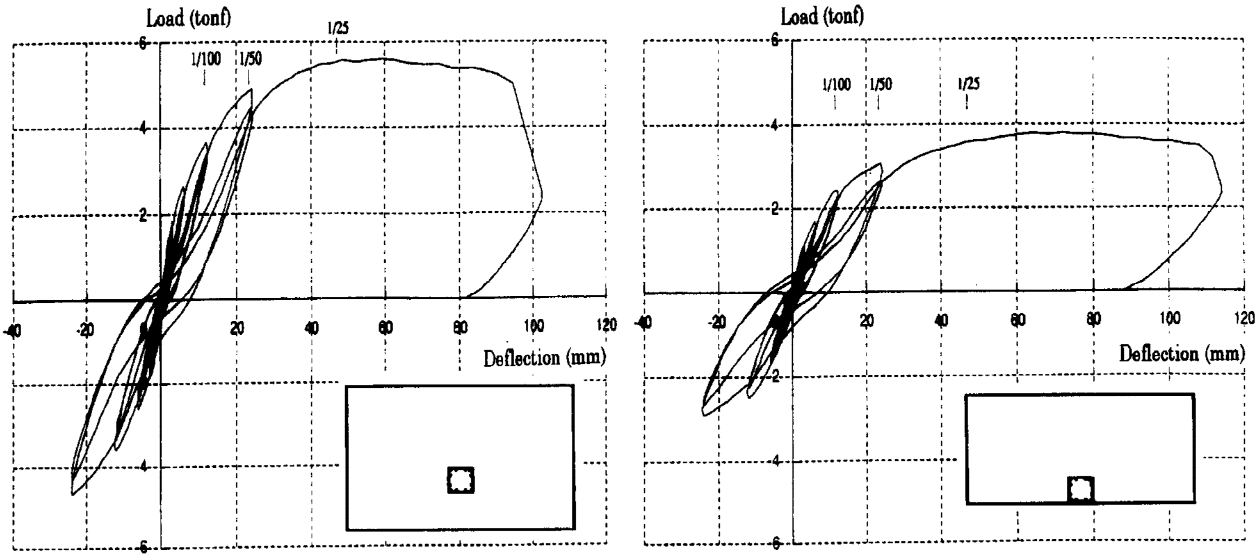


Fig. 8 Load-deflection relationships of the edge column-flat plate connection

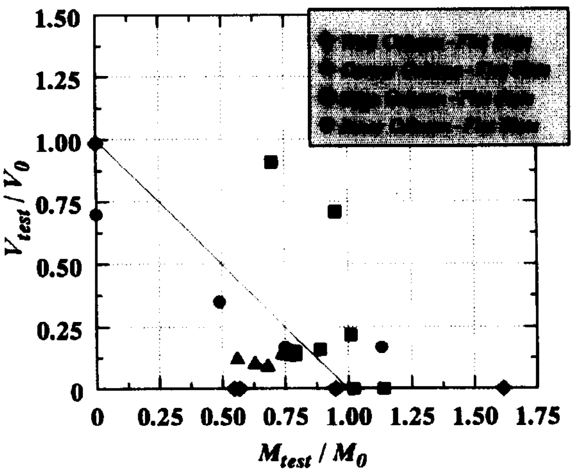


Fig. 9 Relationship between Vtest/V0 and Mtest/M0

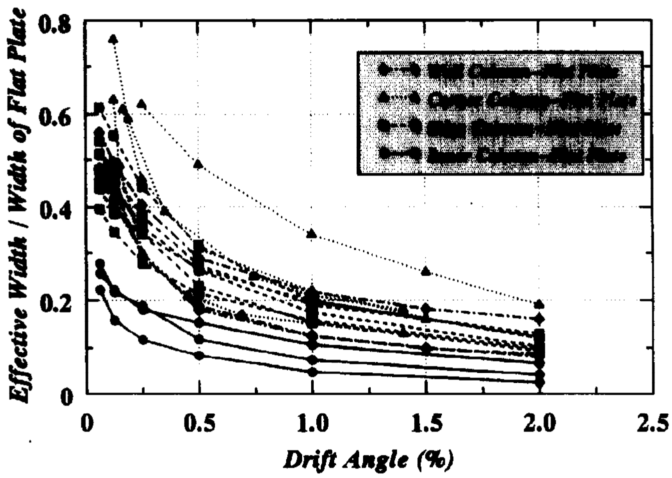


Fig. 10 Effective width ratio

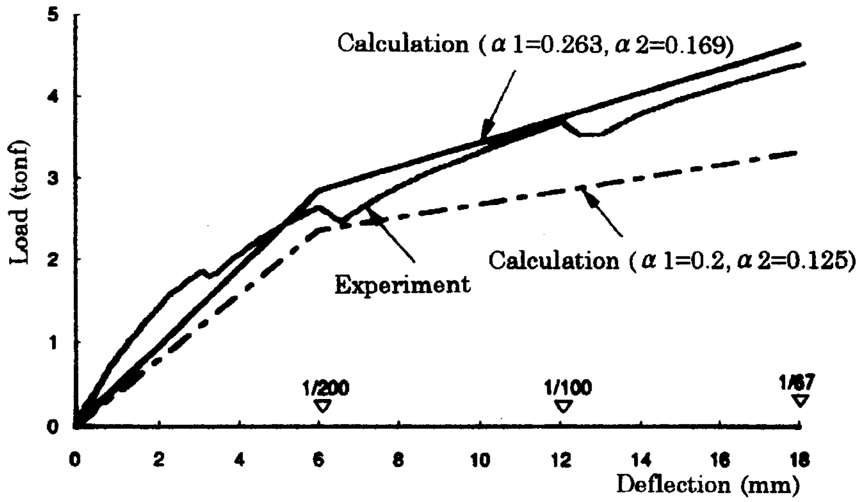


Fig. 11 Skeleton curve of load-deflection relationship

CONCLUSIONS

The following conclusions were obtained from the trial design and the experimental study;

(1) Both super-high-rise flat-plate-type buildings can be designed in an intensive seismic area like Japan.

(2) The flat-plate systems employed in these trial designs were proven to have enough strength and ductility by the experimental study.

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