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Design of Seismic Isolated Tall Building with High Aspect-Ratio

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Abstract

When seismic isolation system is applied to high aspect-ratio (height/wide-ratio) steel structures, there are several problems to be taken into consideration. One is lifting up tensile force on the isolation bearing by overturning moment caused by earthquake. Another is securing building stiffness to produce seismic isolation effects. Under these conditions, this paper reports the structural design of high-rise research building in the campus of Tokyo Institute of Technology. With the stepping-up system for the corner bearings, the narrow sides of single span framework are designed to concentrate the dead load as counter-weight for the tensile reaction under earthquake. Also we adopted concrete in-filled steel column and Mega-Bracing system covering four layers on north & south framework to secure the horizontal stiffness of the building.

Keywords: Structural design, Interlayer isolation system, High rise building, High aspect ratio

1. Introduction

The reported building is a general research building in Suzukakedai campus of Tokyo Institute of Technology, Japan. This project had two stages of construction. At the first stage, the single high-rise building as Fig. 1 was constructed. In the second stage, another high-rise building as Fig. 2 with the same scale with the first one was constructed beside and connected structurally. The first building was completed in June 2005, and the second building was extended 7 years later, in March 2012.

The first high-rise building was designed as a single building with high-aspect-ratio until the completion of the second stage. We also considered the effects of future extended building in the second stage. In this paper, the design statement of the first high-aspect-ratio building is reported.

2. Structure Outline and Interlayer Isolated System

The first stage building has 90.4 m height with 20 stories as in Fig. 3. The height of the standard story is 4.0 m and the standard floor plan has 6.6 m × 15.8 m. Since the building is used for the sensitive research facilities, the seismic isolation system was employed for securing high resilience against earthquakes.

Because of the sloped grand level, more than half of the first floor should be covered by the surrounding soil. Accordingly, the huge retaining wall supporting more than 10 m height of the ground pressure was required. After the design studies, we selected the interlayer isolated system between first and second floor. The first floor framework is designed with concrete structure integrated with the retaining wall, and the external soil pressure is dealt with by the whole first floor framework with non-seismic isolated structure. The framework of second floor and above is designed with steel structure with concrete in-filled steel columns. The aspect ratio of the isolated framework reaches approximately 5.4. The

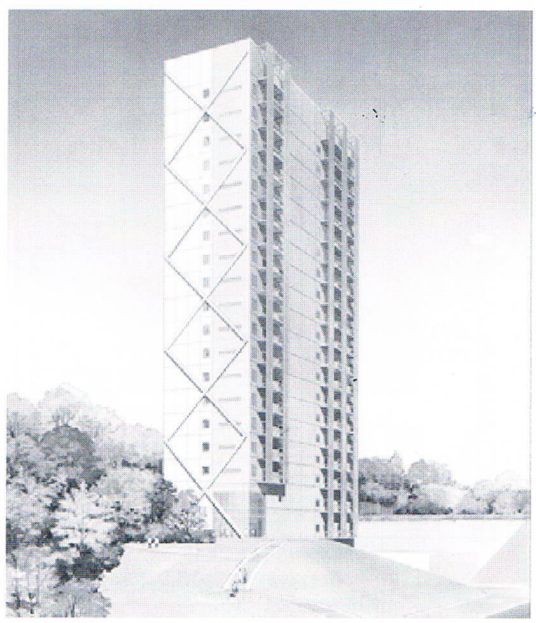

Figure 1. Single high-rise building at 1st stage.

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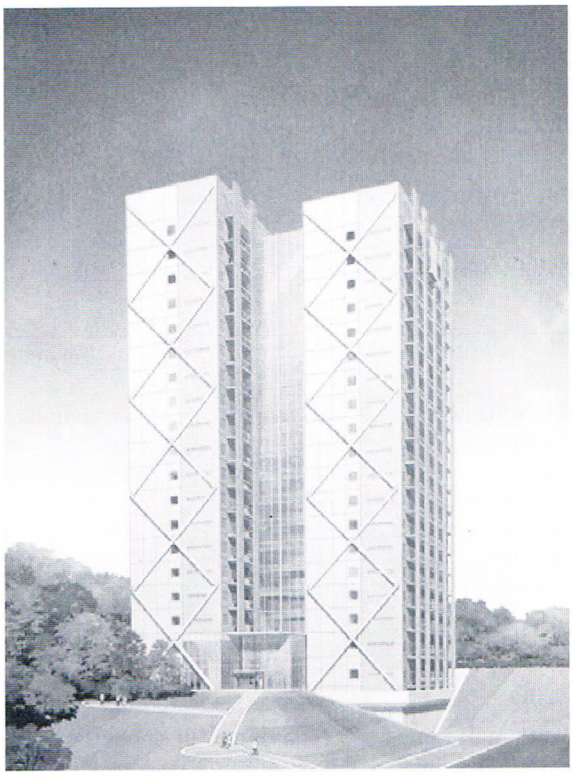


Figure 2. Twin high-rise buildings at 2nd stage.

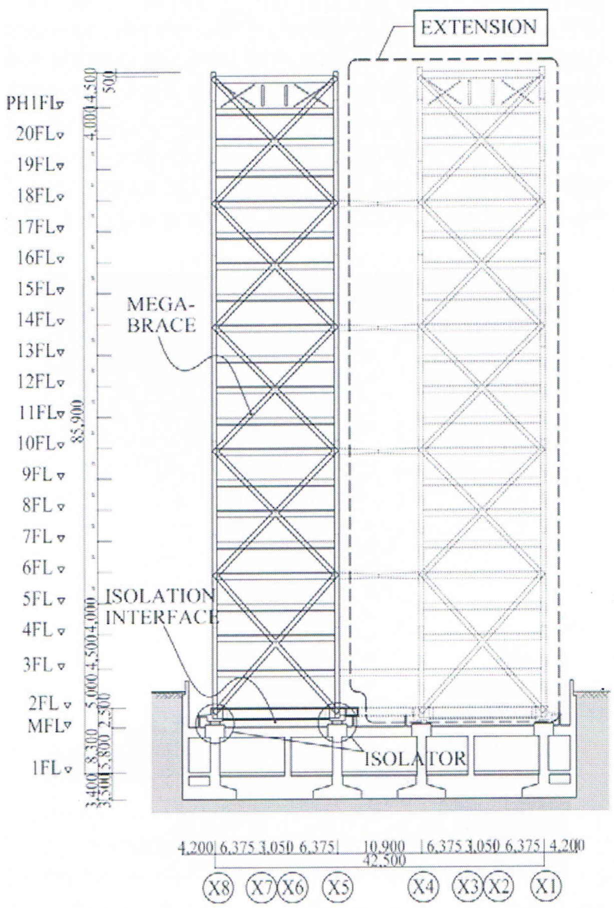


Figure 3. Framing elevation.

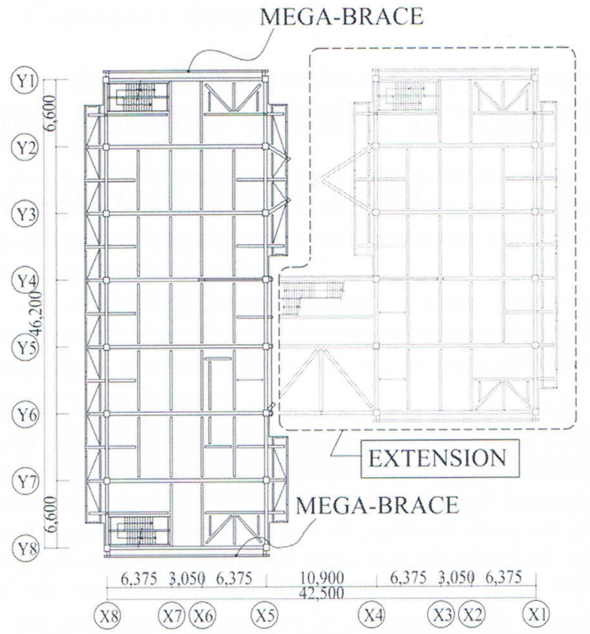


Figure 4. Framing plan.

direct foundation was employed since the ground is Red Soil which is hard enough to support the whole building structure.

The layout plan of the isolation devices is shown in Fig. 5. The devices are consisted from 16 isolator bearings, 56 steel dampers and 2 viscous dampers. The isolator is a natural rubber bearing composed of laminated rubber layers. 4 numbers of isolators with 1200 mm diameter (Detail A) are placed at each corner which has vertical loose detail at anchor bolts to accept the uplift deformation. Other isolators with 1100 mm diameter are integrated with the steel dampers (Detail B). And normal steel damper (Detail C) and viscous damper (Detail D) are placed at the end framework of narrow side.

We adopted two kinds of dampers at the isolated layer. The steel dampers are distributed as the main energy dissipater against earthquake. However, these steel dampers are designed to be kept in elastic against fluctuant wind loads of 50 years return period, to avoid cyclic fatigue failure under wind vibration. Therefore, the steel dampers will not be efficient to reduce vibrations under wind forces and small earthquakes where the steel dampers are in elastic. For vibratory motion are expected especially in the narrower direction of the building under wind load in the 1st stage, the viscous dampers were added on the shorter sides of the plan to control these small amplitudes.

3. Details of Structural System

There are two structural features in this building except for the ordinary isolation system, these are Mega-Brace and bearing mounted with vertical loose detail. The Mega-Brace increases the building stiffness, and the bear-

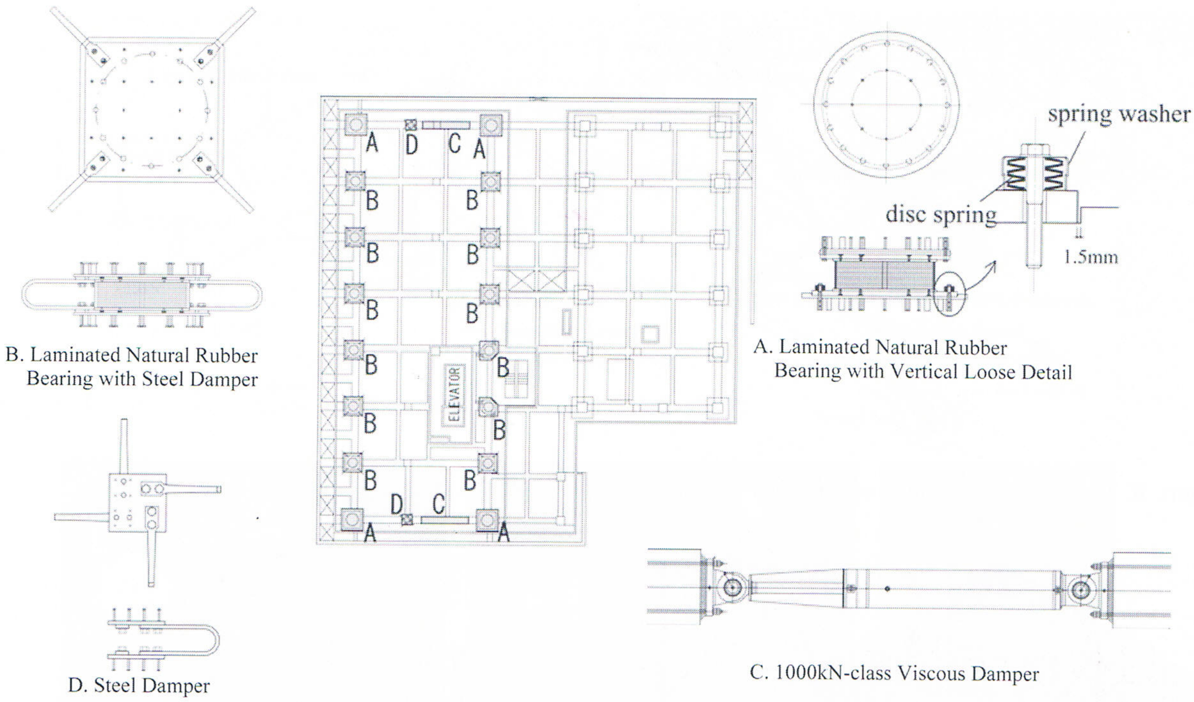


Figure 5. Device layout of isolation interface.

ing mounted with vertical loose detail reduces the tensile force of rubber bearings under lift-up action.

3.1. Design of mega-brace

The Mega-Brace, an architectural feature of the building facade, is placed on the PC wall by the steel casting bracket joint (Fig. 7), in other words, the Mega-Brace consists eccentric brace out of the main framework plane. This is effective to control the excessive axial force working on brace under earthquake, and reduce the section of the brace. In this way, the brace receives about half of the horizontal load.

The section of the Mega-Brace is rectangular steel hollow section of 500 mm × 160 mm, and the thickness are changing from 19 mm at the top layer to 32 mm at the bottom layer. The brace covers 4 stories by single unit, and is connected to the main framework through pin joints of steel casing brackets. To avoid buckling under compression, it is restrained in perpendicular direction to the main framework at the intermediate floors as shown in Fig. 8. These Mega-Brace sizes are decided to maximize the horizontal stiffness of the superstructure as far as the limiting conditions of lift-up mechanism at the corner bearings as described later.

To confirm the strength and stiffness of the steel casting bracket, the eccentric joint for the brace was analyzed by FEM load-deformation analysis simulating earthquake loads.

Structural members around the steel casting bracket required the further detailed studies. As shown in Fig. 8, a bracket is attached on the panel zone of column. The major force worked on the bracket appears in the vertical direction under earthquake, for the horizontal reaction forces from

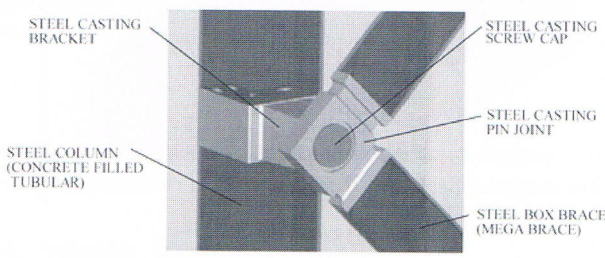


Figure 6. Element of Mega-Brace end.

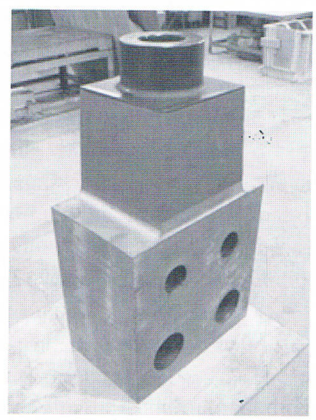


Figure 7. Steel casting bracket.

upper and lower brace is canceled each other. As shown in Fig. 9, which indicates the stress state of steel casting bracket by finite element analysis, the stress concentration appears on the base of the bracket portion. However, from the stress-strain curve on Fig. 10, the ultimate strength of the bracket is approximately 1.5 times bigger than the expected maximum design force. Therefore, it is clarified

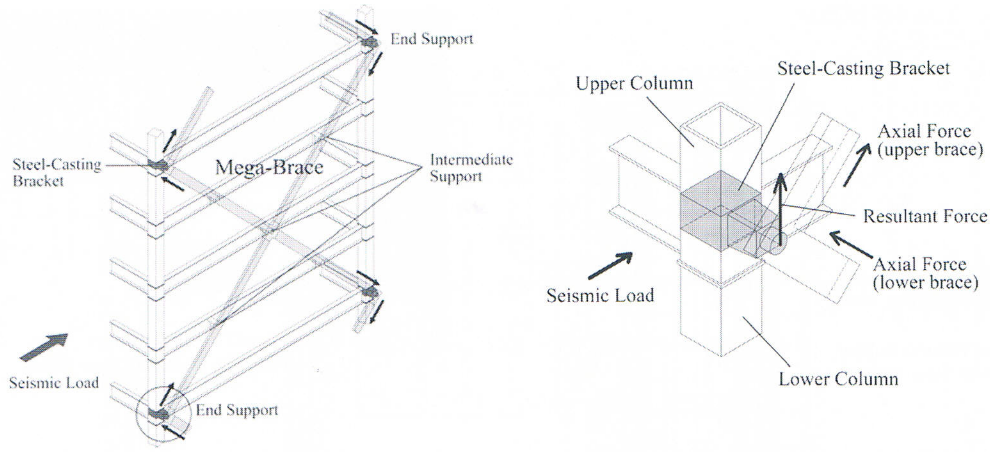


Figure 8. Seismic load and load on the steel casting bracket.

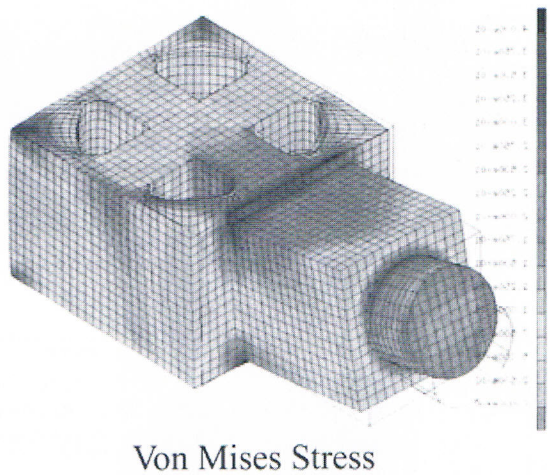


Figure 9. Result of FEM analysis.

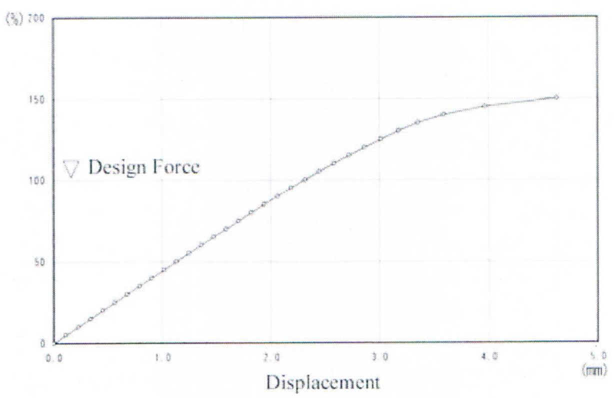


Figure 10. Load-displacement curve of steel casting.

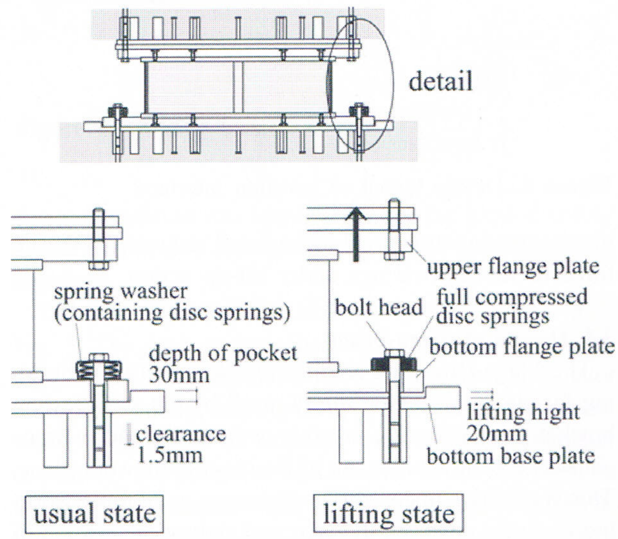


Figure 11. Detail of vertical loose anchor.

Figure 12. Setting of spring washer.

that the steel casting bracket is strong enough.

3.2. Design of seismic isolator with vertical loose anchor

The seismic isolators placed at each corner are mounted with vertical loose detail and are recessed into the pocket of base plate. Under strong earthquakes, these bearings will receive the tensile forces from the Mega-Braces larger than the dead load which cause lift-up. However, the bearing allowing to lift-up without reaction forces can transfer the extra tensile reaction forces to the next framework in the perpendicular direction (Fig. 13). The column

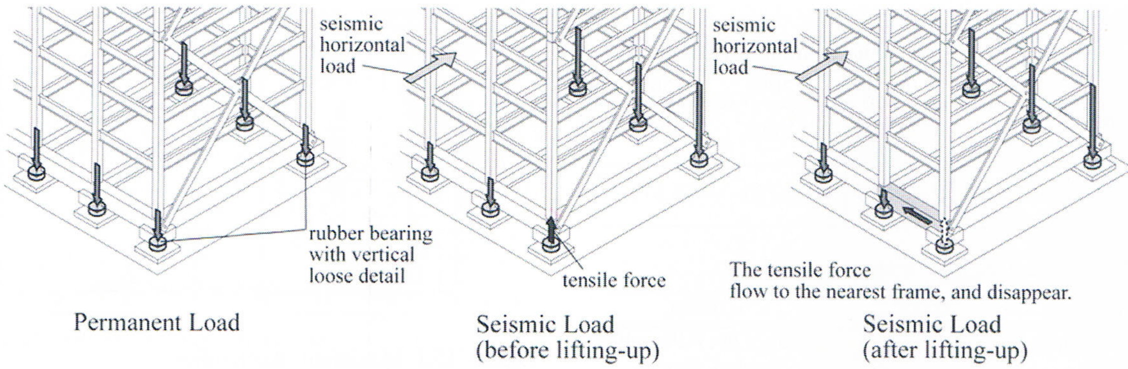
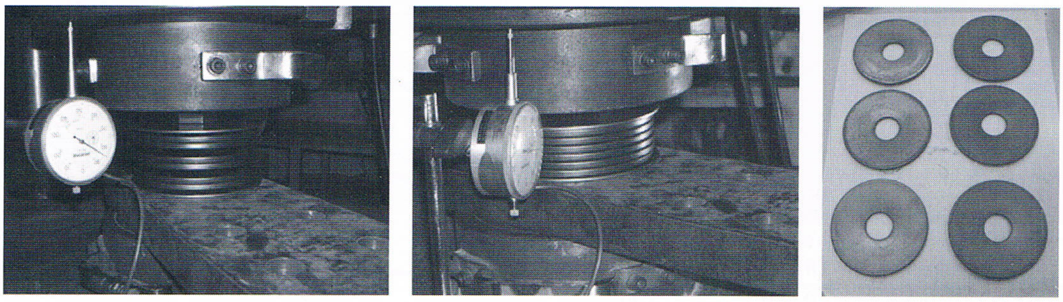


Figure 13. Transform mechanism of lift-up forces at corner bearings.



a) Monotonic loading test – Before loading
b) Monotonic loading test – After loading
c) Disc springs – After cyclic loading

Figure 14. Loading test of disc springs.

next to the corner framework has larger dead load, which is enough to suppress the transferred reaction.

However, this design causes further two problems, one is the impact force of reaction, and another is the detail of the isolator allowing lift-up.

For the first issue, we adopt the disc spring washer on the anchor bolt of the isolator to absorb the impact. There are 16 numbers of anchor bolts with spring washer at bottom flange of isolator (Fig. 12). The detail of the washer is shown in Fig. 11. The spring set is consists of 6 numbers of 6.9 mm thickness disc springs of 145 mm diameter, and the springs had curved surface as dish are alternately piled on the anchor bolts. When the lift-up action work on the isolator, the disc springs are compressed by the anchor bolt head and the bottom flange of the isolator can lift-up without tensile stress. The spring stiffness was designed to have the compression capacity as same as the tension capacity of the isolator, that is the load of the rubber allowable tensile stress 1.0 N/mm^2.

For the second issue, the flange plates of the rubber bearing isolators are designed to recess into the pocket with 30 mm depth since the expected maximum lift-up deformation is around 20 mm. The horizontal deformation is dealt with the lap between the flange plate and the pocket while transferring the horizontal force.

To confirm the performances of the springs under cyclic deformations, experiments on disc spring washer were carried out (Fig. 14). As results the performance of disc spring washer under monotonic and cyclic loading test was confirmed to be appropriate to the design estimation.

4. Seismic Response Analysis

As the result of seismic response analysis of the building, the dynamic properties, the design criteria for response analysis, and the seismic wave list for dynamic analyse are shown at Table 1, 2 and 3 respectively. Further to the seismic waves for dynamic analysis, not only recorded seismic waves as El Centro NS, but also the simulated artificial waves considering the ground amplification at site simulating expected Minami-Kanto Earthquake. The maximum acceleration of these seismic waves are scaled in Level-2 which is about 500 years return period seismic input in Japan.

The results of seismic response analysis under Level-2 seismic wave are shown in Fig. 15. The deformation of the isolated layer in the longitudinal direction is about 0.30 m against design clearance of 0.60 m. The superstructures were designed with ASD criteria keeping the members in allowable stress against seismic loads. The clarified seismic response on 2nd floor is less than the base-share coefficient of C_B=0.10.

The maximum and minimum vertical stress on rubber bearings are satisfactory settled in the capacity in all load cases, including seismic force along diagonal direction, vertical direction and taking the performance variation of

Table 1. Natural periods (sec.)

	Super-structure	Share Strain of Isolator		
		minute	50%	150%
Span Direction	2.179	2.869	3.608	4.231
Longitudinal Direction	2.507	3.099	3.783	4.379

Table 2. Structural design criteria

Part	Subject	Criteria
Superstructure and Understructure	Stress in Member	Within the Allowable Stress Capacity
	Story Drift Angle	1/200 or less
Isolated Layer and Isolator	Deformation	0.54 m or less
	Shear Strain	250%* or less
	Pressure on Rubber	between-1.0N/mm^2 and 30.0 N/mm^2

*Limit Deformation for Performance Guarantee of Isolator.

Table 3. Seismic waves for dynamic analysis

Seismic Wave	Maximum Velocity (m/sec)	Maximum Acceleration (m/sec^2)
A. EL CENTRO 1940 NS	0.500	5.11
B. TAFT 1952 EW	0.500	4.97
C. HACHINOHE 1968 NS	0.500	3.33
D. Simulated Wave-SITE*	0.330	2.63
E. Simulated Wave-GNH**	0.658	3.55
F. Simulated Wave-GNE***	0.500	4.23

*Simulated wave following the site condition (MINAMI KANTO).
**Simulated wave on governmental notification (Phase HACHINOHE 1968 NS).
***Simulated wave on governmental notification (Phase EL CENTRO 1940 NS).

Table 4. Maximum values at Level-2 seismic response

Subject (unit)	Span Direction	Longitudinal Direction
Acceleration of Top Story (m/sec^2)	1.998	2.245
Base Shear Coefficient of 2nd floor (-)	0.079	0.084
Story Drift Angle (rad.)	1/337	1/249
Deformation of Isolated Layer (m)	0.220	0.286
Compress Pressure of Isolator (N/mm^2)	24.05	19.78
Tensile Pressure of Isolator (N/mm^2)	0.11	0.09

isolator (hard and soft case) into account.

5. Construction of Mega-Brace

The most important items for the construction of Mega-Brace are tolerances. We paid attention to keep high-accuracy at shop fabrication and alignment at the site.

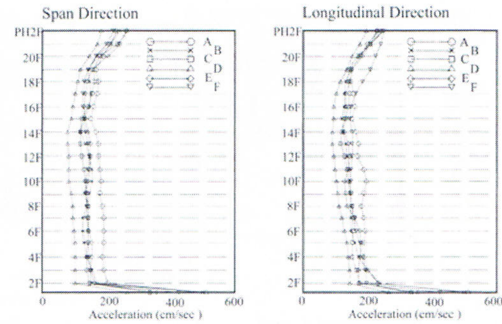


Figure 15.1 Maximum acceleration.

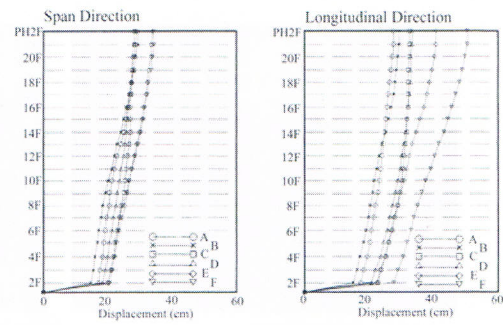


Figure 15.2 Maximum displacement.

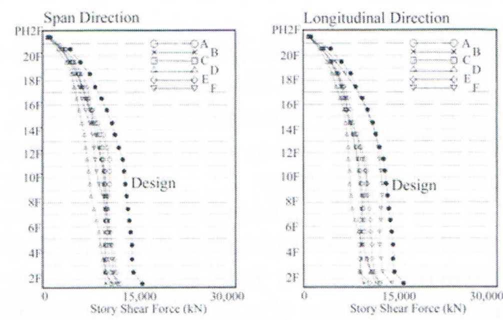


Figure 15.3 Maximum story shear force.

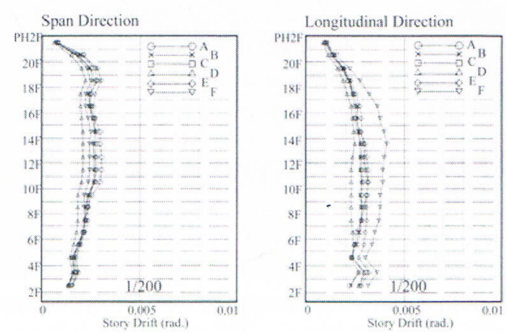


Figure 15.4 Maximum story drift angle.

The supporting points of Mega-Brace bracket were surveyed before the attachment at the center and connec-

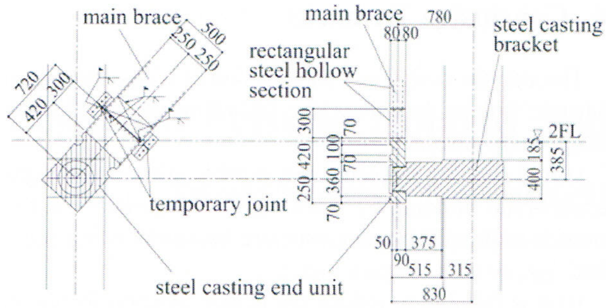


Figure 16. Detail of Mega-Brace end.

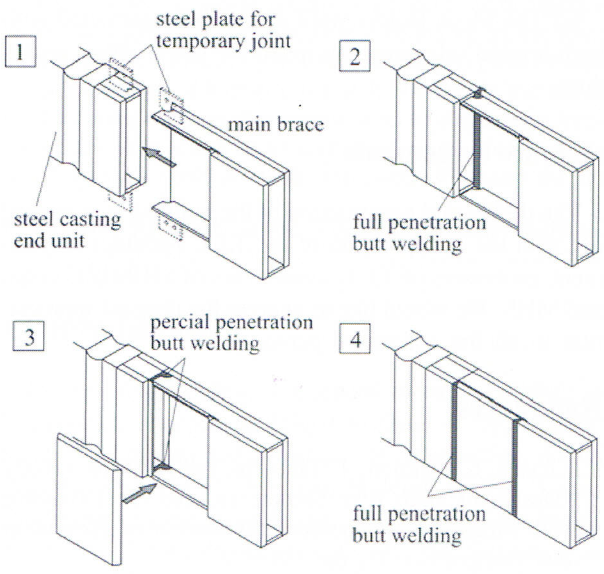


Figure 17. Welding works of Mega-Brace end.

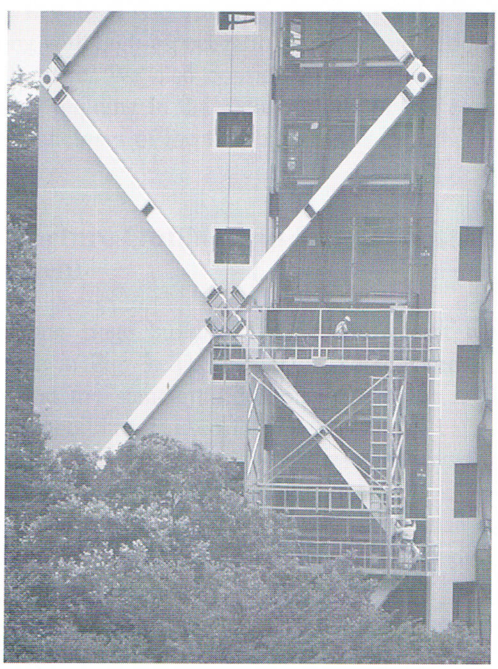


Figure 18. Construction of Mega-Brace.

Figure 19. Construction of 1st stage building.

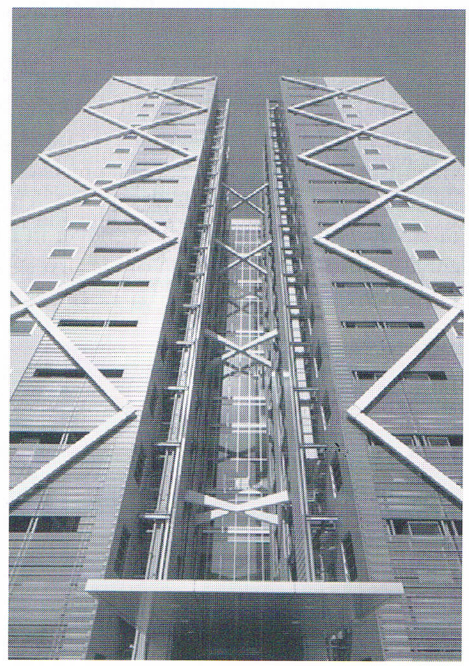


Figure 20. Mega-Brace elevation from south side.

tion to the intermediate floor, and adjusted by grinding the joint plates to keep the Mega-Brace in right position.

In the construction sequence, the braces were mounted after the installation of PC curtain walls. At first, the brace end unit of steel casting was installed as a connection to the steel casting bracket (Fig. 16), and then, the main brace was temporally connected to the end unit by erection pieces. In consideration of contraction by thermal strain,

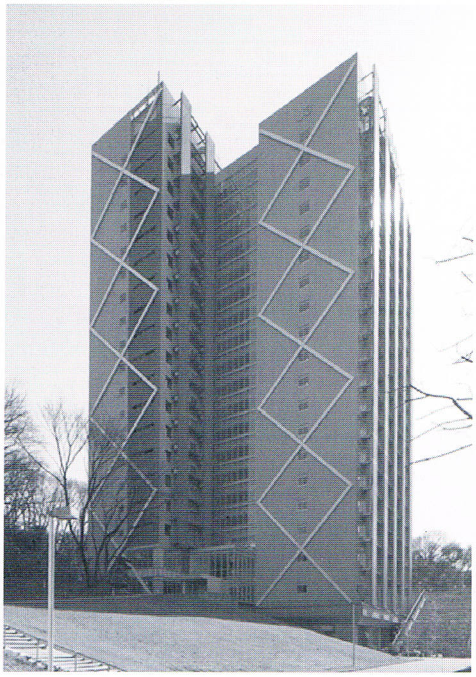


Figure 21. Overall view of the building.

welding between end unit and main portion were carried out after the installation of all Mega-Braces.

As shown in Fig. 18, the welding works were carried out from external side following the procedure shown in Fig. 17. As a result, the inclination of the supporting point for the Mega-Brace can be achieved with precision within 1/1200 through these construction procedures. The completion of structure of the first stage is shown in Fig. 19, and seven years after the completion of the first stage, the second stage construction in March 2012 (Figs. 20, 21).

6. Conclusive Remarks

Through the design and construction of high-aspect-ratio seismic isolated buildings, the following valuable knowledge were obtained.

i) It is feasible to secure the suitable stiffness of high-aspect-ratio building by installing the Mega-Brace at the outside ends of the main structure by steel casting bracket.

ii) It is feasible to reduce the tensile reaction forces on the seismic isolator at corners under the lift-up actions by adopting the recessed isolator allowing the vertical deformation up to 20 mm.

iii) The Mega-Braces were able to be constructed with high-leveled tolerances by inventive construction procedure.

Acknowledgements

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