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Structural Design of High-Rise Concrete Condominium with Wall Dampers for Vibration Control

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Abstract

This paper presents a structural design of the "(Tentative Name) Toranomon Hills Residential Tower" which is currently under construction in Tokyo. The building is a reinforced concrete high-rise residential complex building with 54 stories above ground, 4 basement levels, and a building height of about 220 m. It is a requirement to provide the highest grade of residence in Japan, and in terms of the structural design, it is required to provide wide and comfortable spaces with high seismic performance. These requirements are satisfied by providing a total of 774 vibration control walls of two types. Also, to further improve the structural performance, steel fibers at the rate of 1.0vol% are provided in the ultra-high strength concrete used in the column members.

Keywords: Vibration control structure, Wall damper, Ultra-high strength concrete, Steel fiber reinforced concrete, Precast concrete

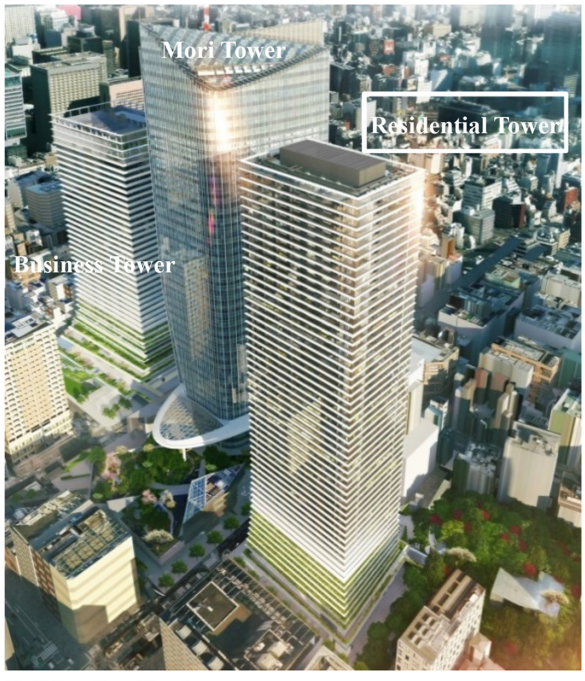
1. Introduction

The building "(Tentative Name) Toranomon Hills Residential Tower" that is the subject of this paper is a reinforced concrete high-rise residential complex with 54 stories above ground, 4 stories below ground, a building height of about 220 m, and a total floor area of 120,000 m², providing about 550 global level residences, which is the largest scale in the area.

Construction commenced in 2017, and is scheduled to be completed in early 2021. When completed it will be the highest reinforced concrete residential complex in Japan.

"Toranomon Hills Area" of Minato-ku, Tokyo is a major project on a site area of 75,000 m² with a total floor area of 800,000 m². This area includes the "Toranomon Hills Mori Tower" that was completed in June 2014, the "(Tentative Name) Toranomon Hills Residential Tower", "Toranomon Hills Business Tower" that are currently under construction, and the "(Tentative Name) Toranomon Hills Station Tower" that is currently being planned.

In addition a new station "Toranomon Hills Station" is scheduled to open partly in 2020 on the Tokyo Metro Hibiya Line that runs through this area, so development is in progress so that the "Toranomom Hills area" will evolve into an international new city and global business center equipped with various functions such as international



Building Specifications;

Name	: (Tentative Name) Toranomon Hills Residential Tower
Owner	: Mori Building Co., Ltd.
Designer & Contractor	: Takenaka Corporation
Exterior Designer	: Ingenhoven Architects
Interior Designer	: Tonychi And Associates
Location	: Atago 1-chome, Minato-ku, Tokyo
Uses	: Residential complex, retail, car parking, etc.

Figure 1. Toranomon Hills Area. (image)

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level offices, residences, hotels, commercial facilities, transport functions, etc.

2. Overview of the Building

Figure 2 shows a cross-section through the building. The cross-sectional composition of the building includes car parks, bicycle parks and equipment rooms in the basements, stories 1st to 3rd with provide the entrance, restaurants and retail shops, childcare support facilities, and a fitness club including a swimming pool as a common area. The residential units are from the 4th story upwards, the 4th to 11th stories are planned to be short-stay residential units. Dwelling units of the 12th story upward are condominiums. On the north side of the site is "Toranomon Hills Mori Tower", and on the south side is "Atago Jinja Shrine", it is planned that there will be pedestrian bridges connecting with these (Figure 3).

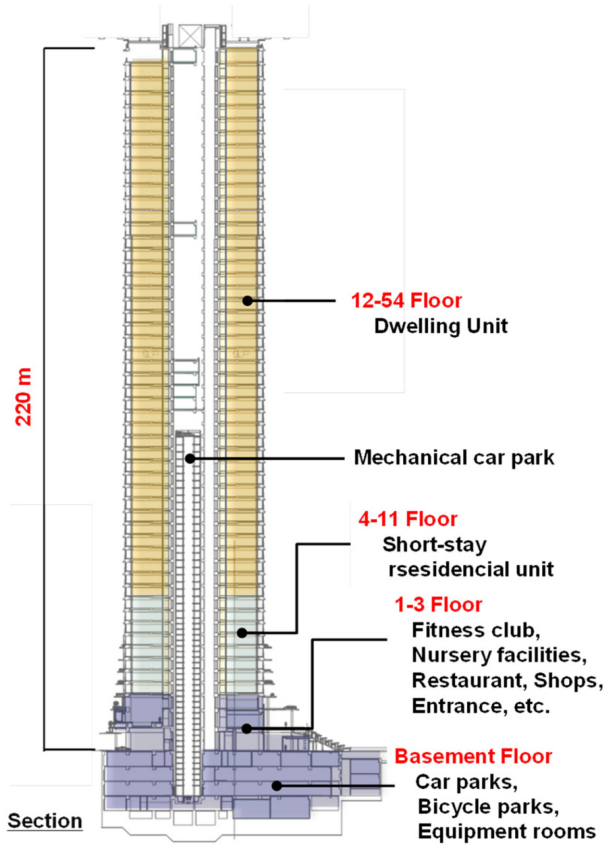


Figure 2. Vertical cross-section.



Figure 3. External view of lower stories. (image)

The plan of the standard stories has the core arranged in the center. The core includes elevators, various equipment rooms, and a mechanical car park. There are no columns in the corners of the standard stories, to provide views from the residential units with an open feeling.

3. Overview of the Structural Design

3.1. Concept of the structural design

It is a requirement that this building will provide the highest grade of residences within Japan. In order to provide wide and comfortable spaces, the structural design is required to provide a high story height, wide spans, and high seismic resistance.

The story heights have been set high and the spans have been set wide. Compared with the normal story height of 3,200 to 3,400 mm for a high-rise residential building in Japan, the story height is 3,700 to 4,400 mm, and compared with the normal spans of 6,000 to 6,500 mm, wide spans of 7,400 to 8,000 mm are being provided.

The requirements from the building owner in terms of safety during a major earthquake and dwelling comfort are higher than the criteria that are normally set. The inter-story drift during a major earthquake is normally 1/100 or less, but it will be 1/150 or less. During a major earthquake normally damage to the main structure is allowed (story plasticity ratio 2.0 or less), in contrast it is required that there be no damage to the main structural members (yielding hinges must not occur in the girders and columns).

Also, high design criteria have been set for the dwelling performance with respect to wind.

In order to satisfy the required performance as described above, a vibration control structure was adopted, in which a total of 774 vibration control walls of two types are provided, viscous wall dampers and friction wall dampers, with up to 16 on each story to achieve high seismic performance as well as wide spans and large story height, without columns and girders of excessively large cross-section.

Also, by providing an active mass damper on the rooftop, the wind oscillations during strong winds are reduced, to achieve dwelling comfort.

3.2. Overview of the structural planning

The above ground structure is a reinforced concrete (in part structural steel) moment resisting structure, is a vibration control structure with vibration control walls on each story.

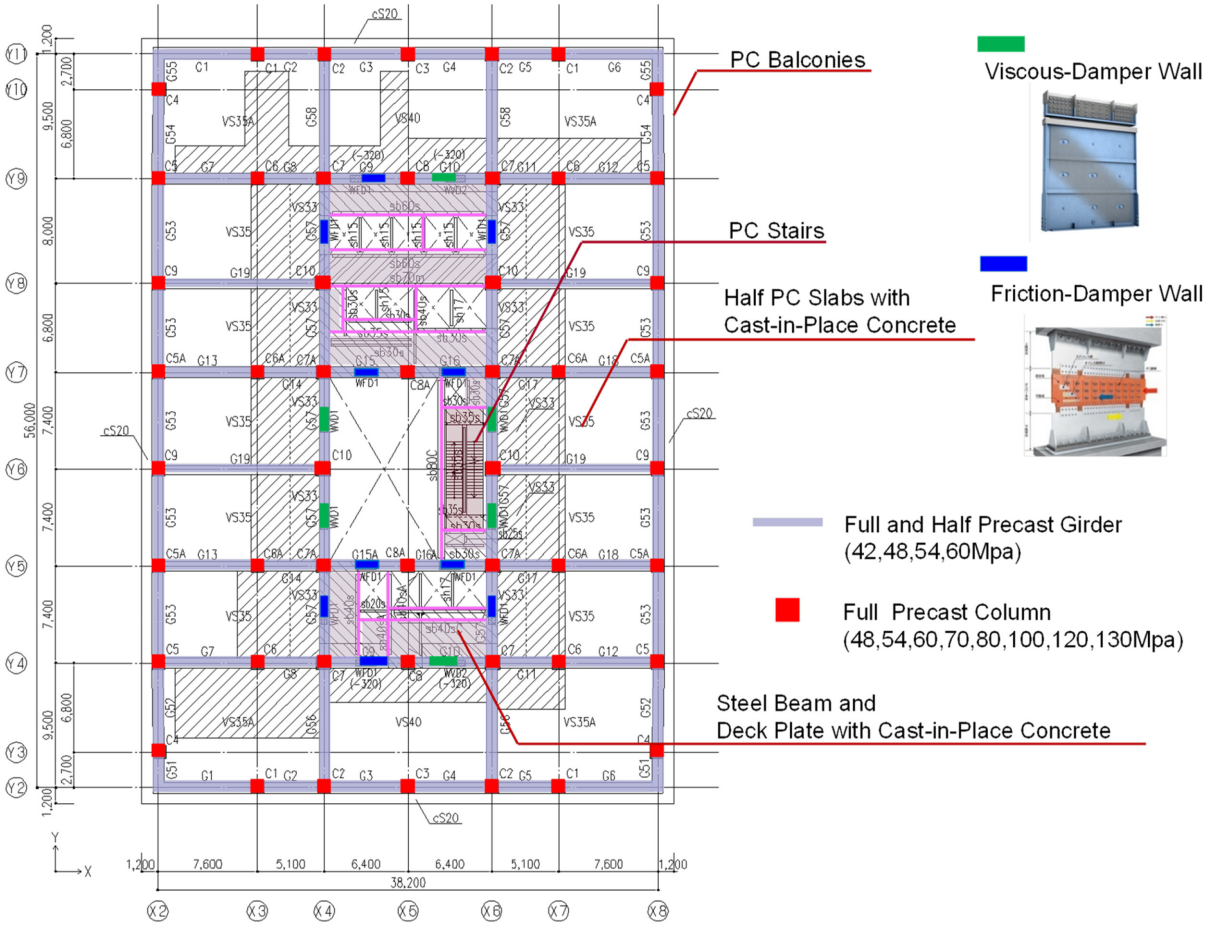


Figure 4. Structural planning for the standard story.

The underground structure has reinforced concrete walls including basement external walls around the periphery, as the main seismic structural elements. The foundation structure is a piled-raft foundation consisting of spread foundations on a strong sandy supporting stratum, and in-situ driven concrete piles.

Figure 4 shows the structure on a standard story. The framing on a standard story consists of reinforced concrete moment-resisting frames of 6 spans and 8 structure planes in the X-direction and 7 spans and 4 structure planes in the Y-direction. Vibration control walls are provided in the internal structure planes formed by the periphery of the core in the building, with 2 No. viscous wall dampers in the X-direction and 4 No. in the Y, and 6 No. friction wall dampers in the X-direction and 4 No. in the Y. Wall Dampers are located 16 on each story.

Taking into consideration vibration, noise, and deflection within the residential units, the floor slabs are composite floor slabs composed of prestressed void precast plates and concrete cast in-situ. In order to increase the degree of freedom of planning the residences, it is avoided providing beams other than the girders. So the slab area is large at 80 to 120 m^2, and the total thickness of the floor slabs is 350 to 400 mm in order to ensure soundproofing performance above and below the residential units.

Note that within the core the floor system consists of structural steel beams, and reinforced concrete slabs with steel deck plates.

The column concrete strength (compressive strength) is 48 to 130 Mpa taking the long-term axial load ratio into consideration, the girder concrete strength is 42 to 60 Mpa, and the floor slab and beam concrete strength is 36 Mpa. Note that 1.0vol% steel fibers were added to the 120 and 130 Mpa concrete (ultra-high strength concrete) used in the 1st to 13th stories, to increase the structural and fire resisting performance.

The main reinforcement of strength, in the columns and girders is 590, 490, and 390 Mpa, with maximum diameter 41 mm. The shear reinforcement of strength in the columns and girders is 785 Mpa for high strength shear reinforcement, and 295 MPa.

Note that mechanical anchorages are used to anchor the girder main reinforcement to the girder-column joints, and mechanical joints are used for the joints in the column and girder main reinforcement.

3.3. Overview of the vibration control walls

Two types of vibration control wall are used: viscous

wall dampers and friction wall dampers.

Viscous wall damper;

Viscous wall dampers as shown in Figure 5 are dampers that use the viscous shear resistance of a high viscosity material, in which an internal steel plate is placed between external steel plates filled with the viscous material. By supporting the viscous-damper walls between the girders on the floors above and below, inter-story vibrations of the building produce relative movement of the fixed plates and the movable plate of the damper, so energy is absorbed by the viscous shear resistance force which produces the damping effect.

Friction wall damper;

Friction wall dampers as shown in Figure 6 are dampers that use a frictional material with an extremely small difference between the static friction coefficient and the dynamic friction coefficient. They are configured with a structure in which a stainless steel plate and the friction material are arranged between a fixed plate on the upper side and a movable plate on the lower side so that they are stacked on each other, and then fixed by tightening with bolts from the outside. During an earthquake the inter-story deformation of the building causes relative movement between the fixed plate and the movable plate, thereby producing a friction force which causes the damping effect.

By using these two types of vibration control wall in combination, a wide range of damping effects can be obtained, ranging from reduction of damage in a major earthquake to reducing the oscillations remaining after the earthquake, so effective control can be achieved.

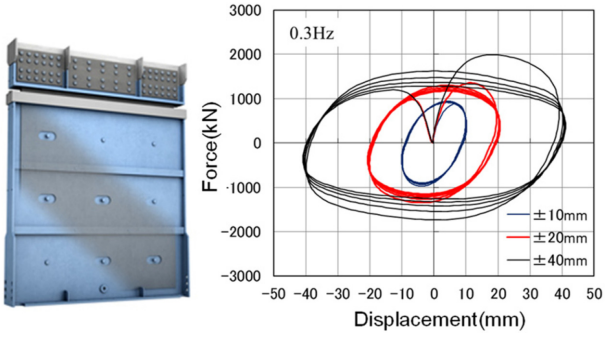


Figure 5. ViscousWall Damper.

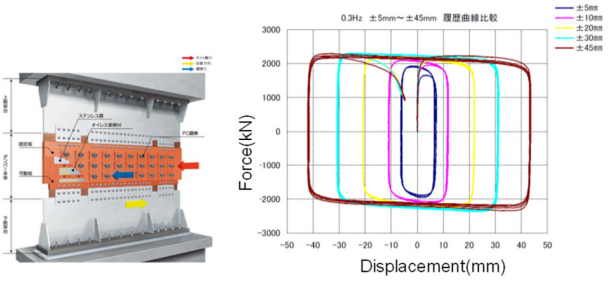


Figure 6. Friction wall damper.

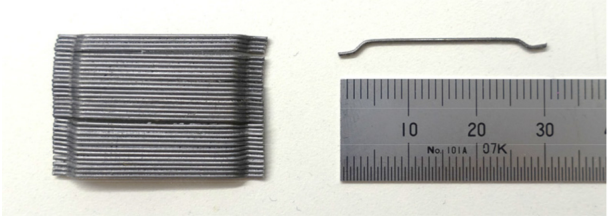


Photo1. Steel fiber.

3.4. Seismic Performance of Ultra-High strength Steel Fiber Reinforced Concrete Columns

3.4.1. Characteristics of ultra-high strength reinforced concrete columns

Reinforced concrete columns using ultra-high strength concrete can resist greater external forces compared with normal concrete, during an earthquake they exhibit very brittle behavior. Ultra-high strength concrete has a smaller air content compared with normal concrete, so during a fire steam pressure increases within the concrete, and spalling of the concrete can occur.

Therefore, polypropylene (PP) fibers are added to the concrete with compressive strength of 70 Mpa or more, and for concrete with compressive strength more than 100 Mpa in addition to the PP fibers, steel fibers are added in the proportion 0.5vol%, as shown in Photo 1.

In this building, 120 and 130 MPa (concrete compressive strength was 120 and 130 MPa) ultra-high strength concrete was used as the column material on stories 1st to 13th. In order to realize a high grade structural performance during an earthquake and during a fire, ultra-high strength reinforced concrete columns with steel fibers in the proportion 1.0vol% were adopted.

This section describes the seismic performance of ultra-high strength steel fiber reinforced concrete columns, as shown in the experimental results below.

3.4.2 Outline of experiment

The experimental parameters of column specimens and their corresponding values are given in Table 1. The specimens were about 1/4 scale of the full-size parts, and the experi-

Table 1. Characteristics of specimen

Specimen	F100-0.55	F105-0.55	F110-0.55
Amount of SF mixed	0.0Vol.%	0.5Vol.% (40kg/m^3)	1.0Vol.% (80kg/m^3)
Section : $B\times D$		250×250mm	
Length : L		1000mm	
Concrete strength f'_c[N/mm^2]	122.4	117.9	114.8
Longitudinal bars	12-D13 (SD685), f_y = 698N/mm^2		
Hoops	4-D6@65 (SD785), f_y = 919N/mm^2		
Axial force	$-0.9a_gf_y \sim 0.55BDf'_c$		

mental parameter was the amount of steel fiber mixed (0.0vol%, 0.5vol%, 1.0vol%).

The concrete strength was 120 MPa, the steel fibers had a fiber diameter of 0.62 mm, fiber length 30 mm, and the hook at the end, as shown in Photo 1.

The axial force was a varying axial force controlled so that when the calculated value of the column flexural ultimate strength was reached the axial load on the compression side was $0.55BDf'_c$ (B: column width, D: column depth, f'_c: concrete compressive strength), and on the tensile side was $-0.9a_g f_y$ (a_g: total cross-sectional area of the main reinforcement, f_y: yield stress).

3.4.3 Experimental results

Photo 2 shows the damage of specimen after the test.

In the case of the specimen F100-0.55 that did not contain the steel fibers, the area of spalling of cover concrete was large. On the other hand, in the specimens with steel fibers the area of spalling of the concrete was small, and the larger the amount of steel fibers the smaller the extent of damage.

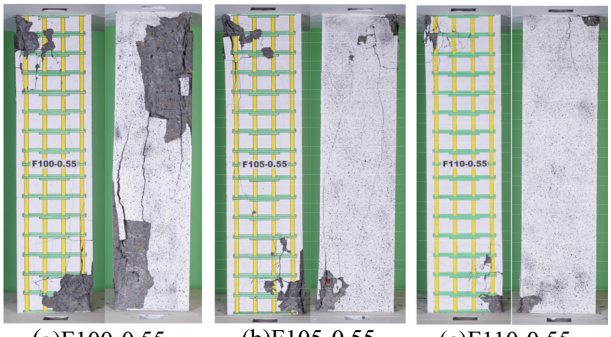
Figure 7 shows the relationships of shear force to the deformation angle and the distribution of maximum principal strain on the back side of the specimen when the deformation angle R=1/100.

In the case of specimen F100-0.55 that did not contain steel fibers, spalling of the cover concrete occurred at column's top and column bases at R=1/166, and at that time the shear force suddenly decreased. The ultimate deformation capacity (the point at which the shear force was 80% of the maximum shear force) was R_{80}=1/67.

In specimen F105-0.55, concrete compression failure occurred at about R=1/166, and at that time the shear force was decrease, but spalling of the cover concrete did not occur. The maximum force was 1.03 times that of F100-0.55, and the ultimate deformation capacity was R_{80}=1/52, or 1.30 times that of F100-0.55.

In specimen F110-0.55, concrete compression failure occurred at about R=1/100, but spalling of the cover concrete did not occur, and thereafter the shear force increased and at R=1/67 the flexural strength was reached. The maximum force was 1.13 times that of F100-0.55, the ultimate deformation capacity was R_{80}=1/37, or 1.82 times that of F100-0.55.

From the above results it can be seen that by providing steel fibers in the concrete at the rate 1.0vol%, spalling of the cover concrete did not occur, and the flexural strength and the ultimate deformation capacity were improved, so it was possible to achieve an ultra-high strength reinforced concrete column with excellent seismic performance.



(a)F100-0.55 (b)F105-0.55 (c)F110-0.55

Photo 2. Damge of specimen after the test.

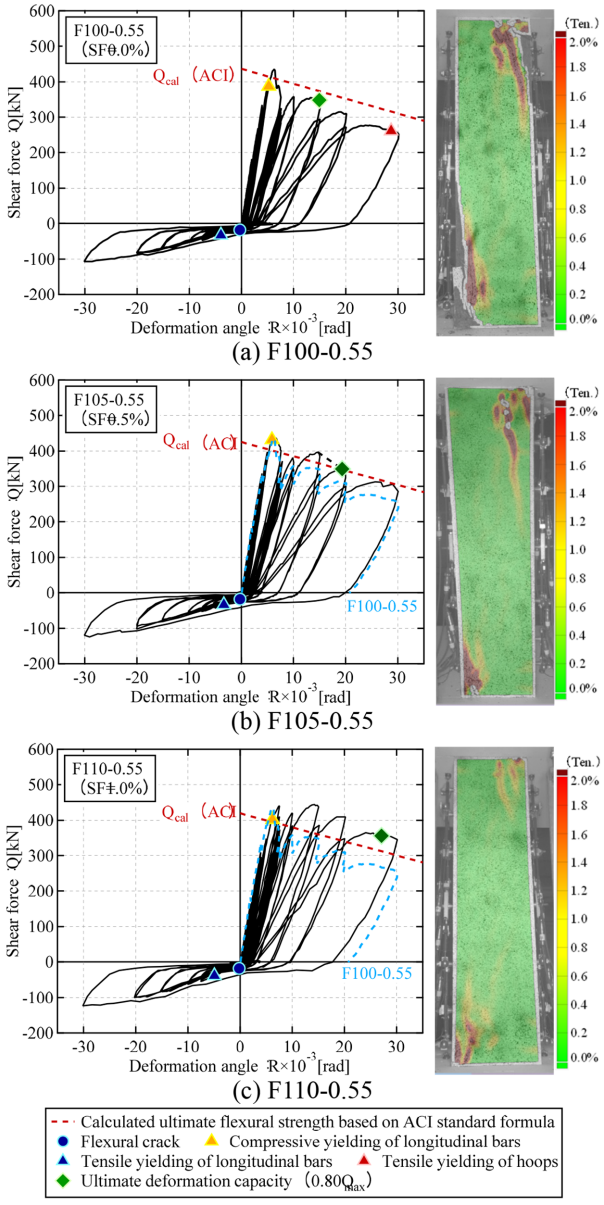


Figure 7. Shear force-deformation anlge response and distribution of maximum principal strain. (R=1/100)

4. Overview of Seismic Design

4.1. Design criteria

Design Criteria is set for inter-story drift, story plasticity ratio and condition of the girders and columns. Table 2 shows the seismic design criteria of the building. Seismic

Table 2. Seismic design criteria

Size of seismic motion	Maximum inter-story drift	Story plasticity ratio	Condition of the main members
Level 1	1/300	1.0 or less	Short-term allowable stress or less
Level 2	1/150	1.0 or less	Less than the yield stress

motions of Level 1, 2 are minor and major earthquakes, as it is explained later. Even under major earthquakes, yielding of the main structural members, namely the girders and columns, is not allowed.

To evaluate the behavior of the building during earthquakes, three-dimensional static elastic plastic analysis using the load increment method and seismic response analysis using a lumped mass model were carried out.

4.2. Seismic response analysis model

The analysis model used in the seismic response analysis was an equivalent bending shear model with 58 nodes at which the mass of each story from the B3 level to the R level was concentrated as shown in Figure 8, and the nodes had degrees of freedom in the excitation direction only.

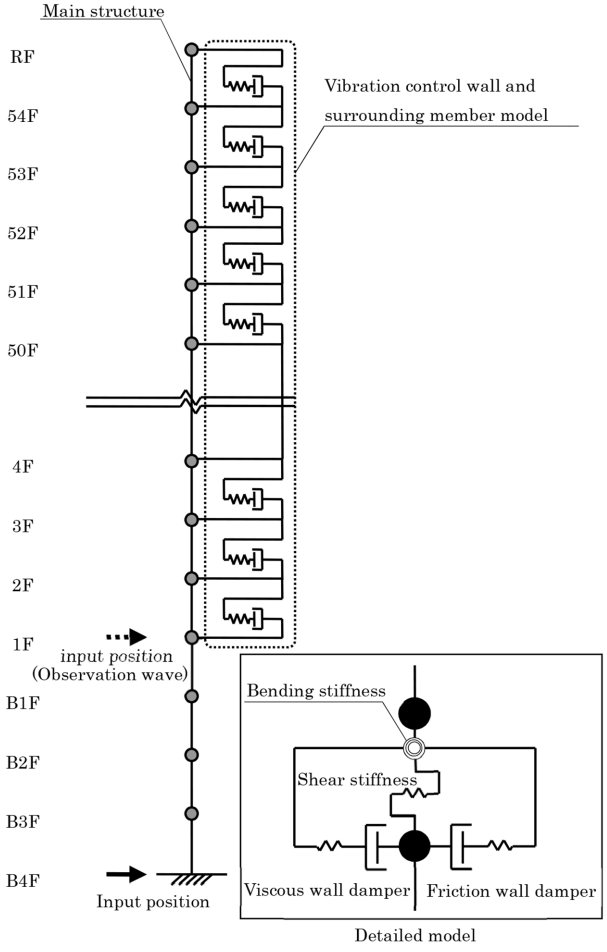


Figure 8. Seismic response analysis model.

The building has a piled-raft foundation, and both the spread foundations and the piles are supported on a strong sandy stratum, so it was considered that the dynamic soil structure interaction of the building would not affect the vibration properties of the building. Therefore the model was given fixed foundation support conditions, fixed at the B4 floor level position. However taking into consideration that the observation waves are wave forms that were measured at the ground surface, the model was fixed at the 1st floor level position for these cases.

The stiffness and restoring force characteristic of each story of the main structure were set to be degrading quadrilinear, based on the load- deformation curve obtained from static elastic plastic analysis using the load increment method. The bending stiffness was set to be elastic using the initial stiffness properties.

The damping in the analysis model was set to be proportional to the instantaneous stiffness for each story element, taking into consideration internal frictional damping. The damping factor for the main structure element was 3.0% with respect to the primary angular frequency, and was assumed to be 0.0% for the members around the vibration control walls.

The vibration control walls were modeled as a Maxwell model in which the damping of the vibration control walls and the stiffness of the surrounding members were connected in series, which were connected in parallel with the main structure. Note that there is variation in the damping force of the vibration control walls due to differences in the product at the time of shipment and temperature variations. Therefore, the seismic response analysis was carried out taking this variation into consideration, and a study was implemented regarding the seismic design criteria.

4.3. Eigenvalue analysis results

Table 3 shows the natural periods obtained from the eigenvalue analysis. The primary natural period in the building short direction or X-direction is 5.526 seconds, and in the long direction or Y-direction is 5.095 seconds.

4.4. Input ground motions

The input ground motions used for design were simulated seismic motions produced based on the spectrum prescribed in Ministry of Construction Notification No. 1461 of the year 2000 (hereafter: Notification wave), and representative seismic waves that were measured in the past (hereafter: Observation wave).

Two levels were set for the input level of the Notification wave, the “rarely occurring seismic motion” (Level 1 seismic

Table 3. Natural periods (in seconds)

Direction	1st	2nd	3rd
X	5.526	1.650	0.877
Y	5.095	1.691	0.955

motion) and the "extremely rarely occurring seismic motion" (Level 2 seismic motion), and at each level three waves were adopted with different phases.

The input level of the observation wave was set so that the maximum velocity amplitude for the "rarely occurring seismic motion" (Level 1 seismic motion) was 0.25 m/s, and for the "extremely rarely occurring seismic motion" (Level 2 seismic motion) was 0.50 m/s.

Table 4 shows summary of the input ground motions used in the seismic response analysis, and Figure 9 shows the pseudo velocity response spectra of the "extremely rarely occurring seismic motions" (Level 2 seismic motion).

Table 4. Summary of input ground motions

Type	Ground motion name	Max. Accelerations (cm/s^2)		Duration (sec.)
		Level 1	Level 2	
Observation	EL CENTRO 1940NS	255.0	510.0	53.76
	TAFT 1952EW	248.4	496.9	54.40
	HACHINOHE 1968NS	174.5	348.9	234.0
Notification	Notification wave 1 (random phase)	73.2	338.7	120.0
	Notification wave 2 (Hachinohe phase)	72.8	363.8	120.0
	Notification wave 3 (Kobe phase)	71.1	355.7	120.0

4.5. Results of seismic response analysis

Figure 10 shows the maximum response analysis results (inter-story drift, story plasticity ratio, response acceleration, story shear force coefficient) for the Level 2 seismic motions. Note that the analysis results take into

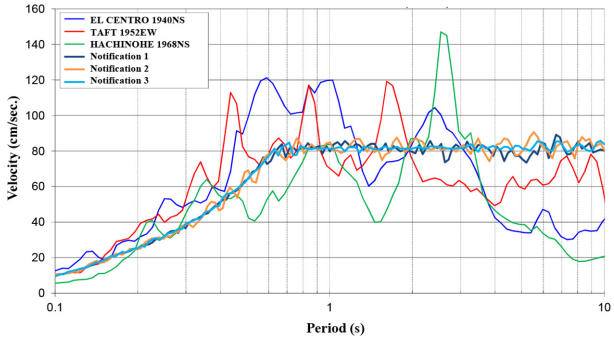
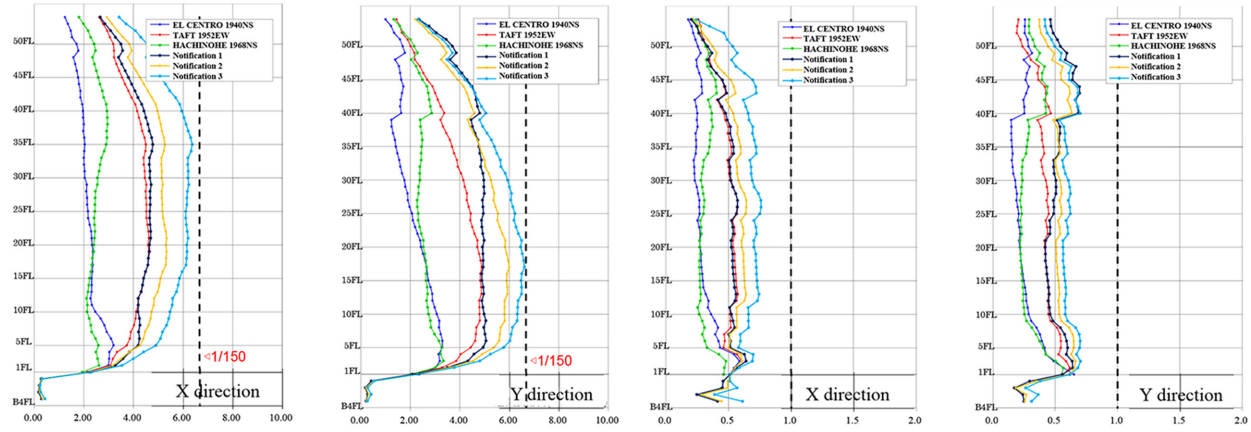
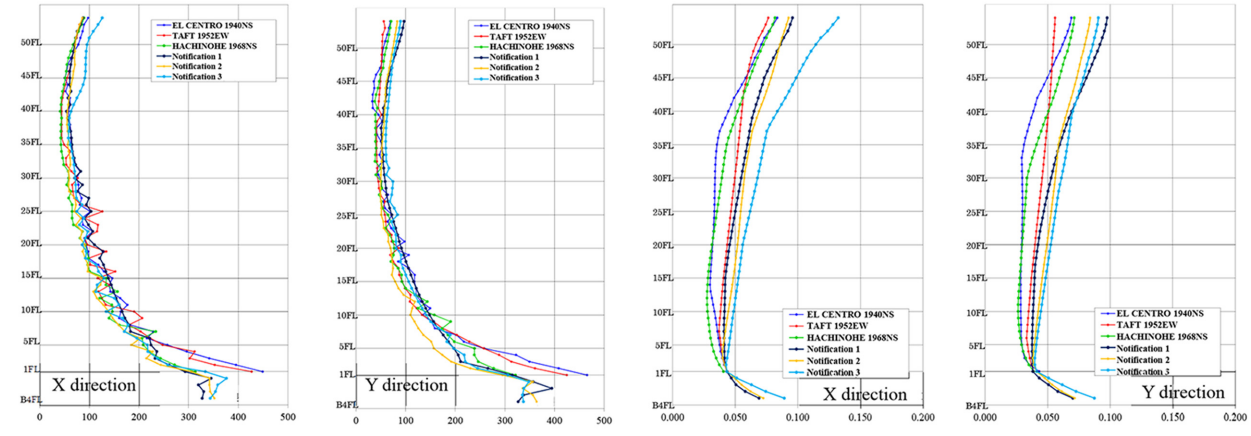


Figure 9. Pseudo velocity response spectra. (h=5%)



(a) Inter-story drift ($x10^{-3}$ rad.) (b) Story plasticity ratio



(c)Acceleration (cm/s^2) (d)Story shear force coefficient

Figure 10. Maximum response analysis results.

consideration variation in the vibration control walls, so the analysis results show the case where the damping has been reduced.

The maximum inter-story drift in the X-direction was 1/170, and the maximum inter-story drift in the Y-direction was 1/165, so the seismic design criteria of 1/150 is satisfied. Also, it was confirmed that the maximum response story plasticity ratio in both the X and Y directions were the criteria of 1.0 or less.

The maximum response acceleration on each story tended to increase towards the lower stories and the results for the 20th story and higher stories were about 100 cm/s^2 or less.

4.6. Energy absorption percentage

In order to determine the effects of the vibration control walls, the energy absorbed during the earthquake was calculated, and the percentage of the input energy was obtained as shown in Table 5. The energy absorption percentage for each of the Level 2 seismic motions in the X and Y directions varied slightly, but the vibration control devices (viscous wall dampers and friction wall dampers) accounted for 40 to 50%.

5. Overview of the Construction Scheme

The main structure of the columns and girders consist of the following three types of precast member.

(1) Column members

These are members that extend from the level of the top of the girder on the story below to the bottom of the girder on the story above, and on the upper side the main reinforcement extends so as to connect to the upper story. Also at the bottom of the member, mechanical joints are embedded in order to form a joint with the main reinforcement of the story below.

(2) Girder members with integral joint parts

Sheath tubes are provided so that the main column reinforcement from the lower story can pass through the

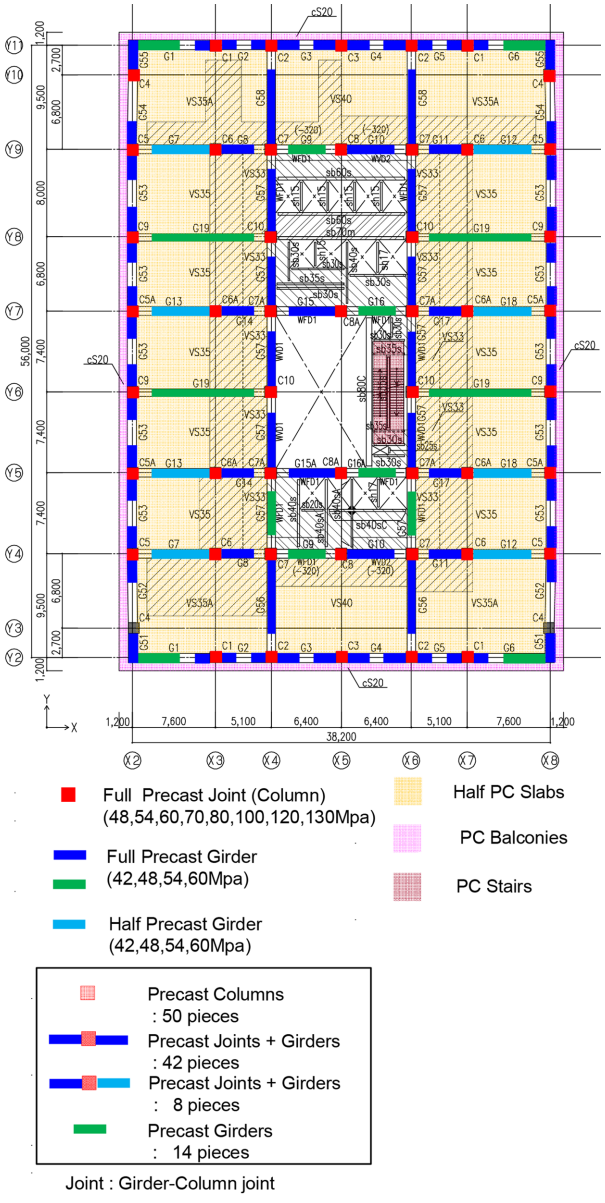


Figure 11. Precasting plan for the standard stories.

Table 5. Energy absorption percentages

Input ground motion name	direction	Total energy input (kNm)	Vibration control wall		Main structure hysteretic damping	Internal viscous damping
			VWD	FWD		
EL CENTRO 1940NS	X	74,842	8%	39%	20%	33%
	Y	66,064,	18%	30%	23%	29%
TAFT 1952EW	X	110,188	8%	39%	28%	25%
	Y	105,699	17%	31%	30%	22%
HACHINOHE 1968NS	X	76,601	9%	42%	19%	30%
	Y	77,921	20%	33%	23%	24%
Notification 1 (random phase)	X	264,574	8%	40%	20%	32%
	Y	266,380	17%	31%	28%	24%
Notification 2 (Hachinohe phase)	X	252,812	8%	40%	20%	32%
	Y	217,725	16%	33%	26%	25%
Notification 2 (Kobe phase)	X	123,035	8%	43%	23%	26%
	Y	124,395	16%	33%	29%	22%

girder to column joint. By filling the sheath tubes with grout material the joint with the main reinforcement from the lower story is made integral. Also, the main column reinforcement that has passed through is connected to the mechanical joints in the bottom of the precast column on the story above.

(3) Girder members

These are precast members with the girder reinforcement extending from both ends, the main reinforcement is connected with mechanical joints to the adjacent precast girders, and in-situ concrete is cast to integrate the members. Two types are used depending on the situation. One is a full precast girder in which the whole cross-section of the girder is precast, and the other is a half precast girder in which the upper part of the girder cross-section is cast in-situ.

In the construction of the structure above ground in this building, constructability is being improved by precasting the columns, main girders, slabs, and stairs as shown in Figure 11, and it is planned to construct one floor in five days at the shortest.

6. Conclusions

Construction of this building is scheduled to be completed in early 2021, and upon completion it will become the highest reinforced concrete residential building in Japan.

We could realize severe design criteria by applying vibration control structure using wall dampers.

We would like to appreciate for the opportunity to participate in design of this project as structural engineers.