16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 3502

Registration Code: S-B1461854191

Structural Planning for 189-Meter-Tall Damped Building with Irregularly-Shaped Plan and Elevation

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Abstract

This building is a 189-meter-high commercial-office complex in Osaka, the second largest city of Japan, and has the characteristics of its irregular L shape both in plan and in elevation (combination of a commercial podium and an office tower). This paper introduces three distinctive means for use of a damped structure ensuring the safety against earthquake and rational structural planning to match the irregular building shape.

1. Belt/hat trusses and high-capacity oil dampers to match L-shaped elevation of tower

This building is L-shaped in elevation, with a 130-meter-tall tower standing out on the top of a 60-meter-high podium (mid-to-low rise section). Therefore prevention of the whipping of the tower and the torsional deformation excited by the planar shape is critical in structural planning. The bottom and top of the tower have belt truss and hat truss respectively to reduce the bending deformation. The floor equipped with MEP units serves as a truss floor, thereby matching the building purpose and the structural planning. To reduce the tower's torsional deformation, the span of the tower is reduced to about 60% of that of the podium with a changeover on the belt truss floor, which increases the tower's torsional stiffness. Furthermore, high-capacity oil dampers (damping force 6,000kN) are installed in the middle part of the tower to restrain the higher-mode deformation.

2. "Damping Tail" system to match L-shaped plan having eccentricity of loading

This building has an L-shaped configuration measuring approx. 150 by 150 meters in plan. A tower is architecturally planned only on one side of that shape, which yields a deviation between the centers of gravity and rigidity in the podium. Consequently the building is prone to an increase in torsional deformation if horizontal load is applied.

We devised the means of matching the centers of gravity and rigidity by adjusting planar rigidity to restrain such torsional deformation. Specifically, highly-rigid, buckling restrained braces are laid out in the tower's lower part, while the podium using a rigid-frame structure becomes relatively less rigid than the tower's lower part. Moreover, viscous damping devices are intensively located mainly along two edges of the L-shaped plan of the podium where torsional deformation tends to be amplified against the tower. We call this system "Damping Tail" system, aiming to restrain the podium deformation and absorb the seismic energy throughout the building at the "tail-shaped" portion attached to the tower.

3. Composite foundation to equalize deformations under different loading conditions

The foundation-supported loads of the tower are more than double those of the podium in this building. Furthermore, the podium and tower foundations, divided by the road and underground mall, may cause torsional deformation throughout the building if the horizontal deformations differ between the foundations in earthquake.

We studied vertical and horizontal deformations using sway-rocking and 3D FEM models including the ground, and applied multi-stage diameter-enlarged piles to the tower and a mat foundation to the podium to keep the foundations from torsional deformations and ensure structural safety.

Keywords: "Damping Tail" system; high-capacity oil damper; composite foundation

1. Introduction

In recent years, skyscrapers with bizarre planar/elevational shapes have started to emerge worldwide. Demands for large-scale, complicated shaped skyscrapers are on the increase in Japan which has been hit by large-scale earthquakes during recent years. On the other hand, these types of buildings are likely to have larger deformations and stresses caused by horizontal loads due to an earthquake or strong wind than regular-shaped buildings and therefore require a variety of considerations and cares in the designing process.

In this project in Osaka, we have focused especially on three planning processes: the elevational structure planning of a high-rise (tower) section, the planar structure planning of a low-rise (podium) section, and the foundation structure planning, and thus secured the safety of a complicated shaped skyscraper in an efficient way. This paper describes the details of the process.

2. Overview of Building

This is a project to rebuild a skyscraper complex which stands on the two sites across a road at the center of the Osaka-Umeda Terminal, the West Japan's largest terminal, serving an average of around 2.5 million people each day. We plan to build a 189-meter-skyscraper in the north site and a 54-meter-high tall building in the southeast site. Fig.-1 shows a concept rendering of a completed building. In terms of the building scale, the complex has 3 underground stories, 38 stories above ground and 2 penthouse levels, with a department store and a conference center in the low- and mid-rise section and offices in the high-rise section.The building configuration is featured by irregular shapes both in plan and in elevation: The building consists of an L-shaped plan vis-à-vis the urban center area and an L-shaped elevation in combination of a commercial podium and an office tower.

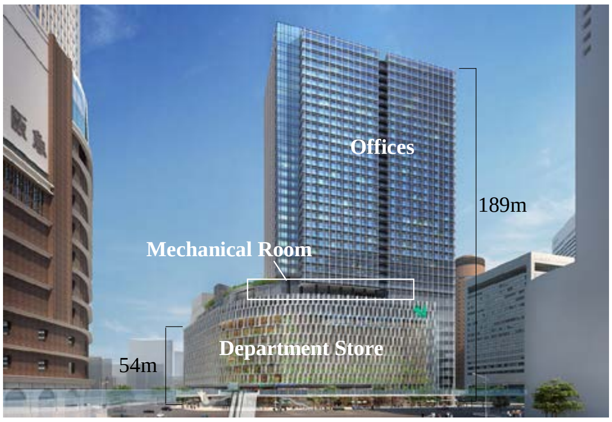


Fig.-1 Concept rendering of completed building

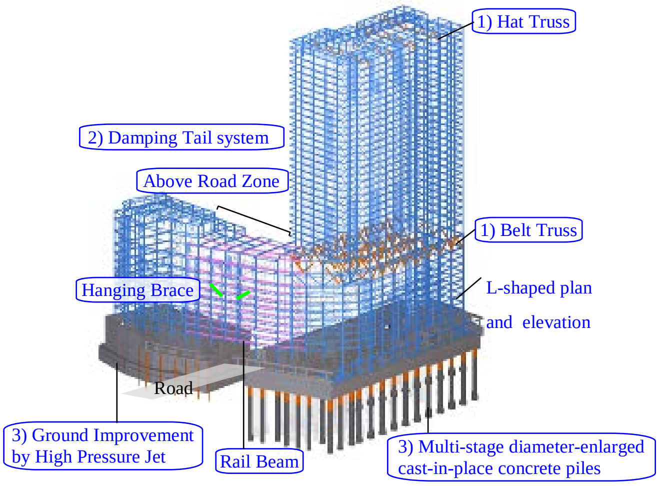


Fig.-2 Overview of structural planning

3. Overview of Structure

This building is steel structured above the ground with steel and reinforced concrete structures at the underground level. Additionally, it has a damped structure to reduce shaking and structure damages in an earthquake and serve as a countermeasure against an earthquake on a scale exceeding the statutory level of earthquakes. Two types of damping devices are employed in this building to provide adequate bearing force and stiffness by installing hysteretic dampers and ensure redundancy by using both hysteretic and viscous dampers. Fig.-2 shows an illustration overview of the structural planning.

This building uses the following three major distinctive means in the structural planning to realize the characteristic shapes in plan and elevation and ensure effective and reliable safety. More details are described in Section 4 below.

1) Use of belt/hat trusses and high-capacity oil dampers to match the L-shaped elevation;
2) Use of a "Damping Tail" system to match the L-shaped plan having eccentricity of loading; and
3) Composite foundation plan to equalize vertical and horizontal deformations under different loading conditions.

4. Seismic Design Criteria

There are more and more social demands for and interest in ensuring a margin of structural performance to prepare for an earthquake beyond the statutory level after the Great East Japan Earthquake in 2011. Table-1 shows the seismic design criteria for this building which have been established as the result of our negotiations with the Client. We have performed validation of the Safety Level 3 as a measure against an earthquake on a scale exceeding the statutory level of earthquakes. Besides, these criteria have a margin of about 10% over the story drift angle criteria established for normal high-rise buildings in Japan to improve the performance of the existing design criteria. Moreover, the member strengths specified in the members design criteria are under the elastic limit strength against the Level-2 earthquake motions in particularly critical areas: the belt truss, hanging braces above roads and overhead structures above roads.

Table-1 Seismic design criteria and safety decision criteria in designing superstructure members

Outline of earthquake motions		Level 1	Level 2	Level 3 (Seismic Safety Margin Analysis Level)
		- Rare. - Recurrence interval: Approx. 50 years	- Very rare. - Recurrence interval: Approx. 500 years	- 1.5 times stronger than the Notification Level-2.
Target building performance		Continuously usable	Repairable	Repairable (Reinforceable)
Super-structure	Story drift angle	4.55×10^{-3} rad. (1/220)	9.09×10^{-3} rad. (1/110)	13.4×10^{-3} rad. (1/75)
	Ductility factor of story	Allowable stress for short-term loading, or less	1.5 or less	3.0 or less
	Column	Allowable stress for short-term loading, or less	Less than ultimate strength*	Less than ultimate strength*
	Girder	Allowable stress for short-term loading, or less	Plastic hinges allowed	Plastic hinges allowed
	Brace	Allowable stress for short-term loading, or less	Plastic hinges allowed	Plastic hinges allowed
	Belt Truss Rail Beam Hanging-Brace	Allowable stress for short-term loading, or less	Less than elastic limit strength	Less than ultimate strength*
	Connection floor	$\tau < 0.38 \times \sigma_B^{0.5}$	$\tau<0.56 \times \sigma_B^{0.5}$	$\tau<0.75 \times \sigma_B^{0.5}$
Substructure	Underground frame	Allowable stress for short-term loading, or less	1.5 or less	Ductility factor of members < 4.0
	Foundation	Allowable stress for short-term loading, or less	Less than ultimate strength	Ultimate strength or less
	Pile bearing capacity	Allowable bearing capacity for short-term loading, or less	Ultimate bearing capacity, or less	Ultimate bearing capacity, or less

τ: Shearing unit stress od slab (N/mm^2)

σ_B: Specified design strength of concrete (N/mm^2)

* Formation of plastic hinges is allowed in some of the members.

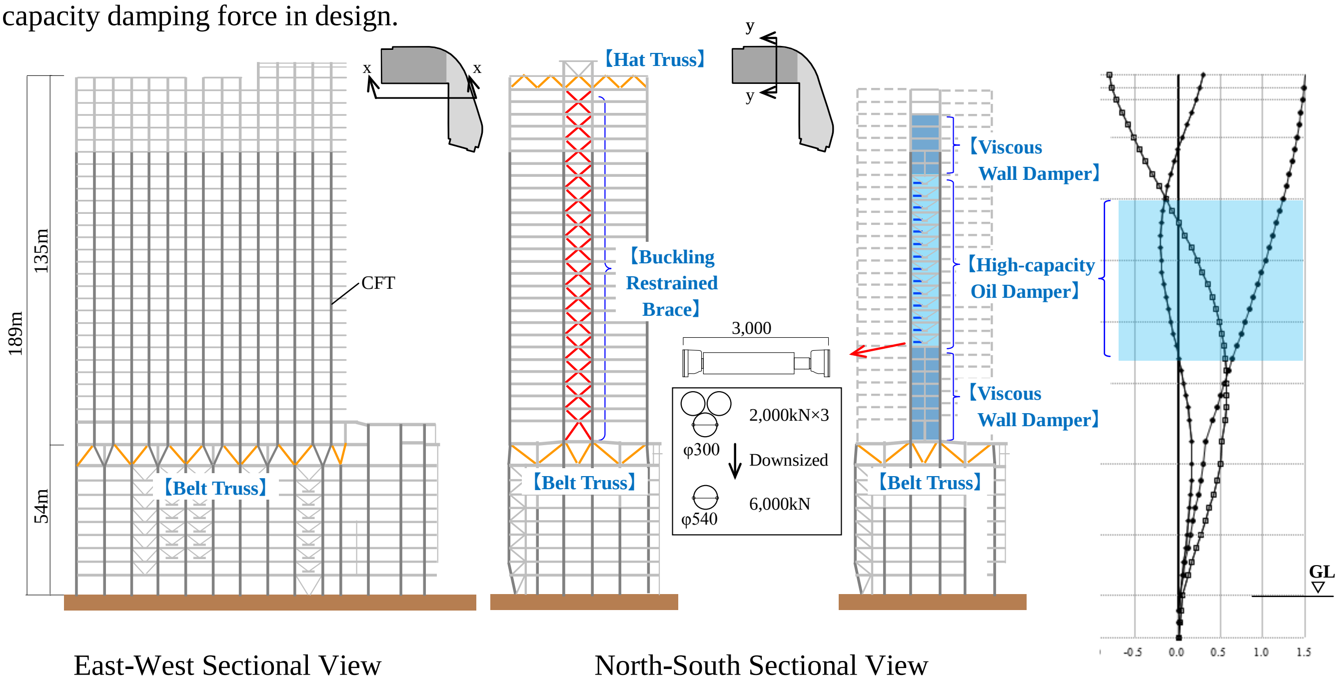
5. Structural Planning

5.1 Realization of L-shaped elevational structure planning of tower (application of belt truss serving as transfer truss, hat truss and new high-capacity oil dampers)

This building has an L-shaped elevation, with an approximately 135-meter-tall tower (offices) erected on the top of a nearly 60-meter-high podium accommodating a department store section. It is critical in structural planning to prevention of the whipping and the torsional deformation of the tower.

In this project, the floor equipped with MEP machines, located between the podium and the tower, serves as an 8-meter-high belt truss. Besides, the top floor is provided with a hat truss in the shorter-side north-south direction (Y) (Fig.-3). And moreover, the columns connected to those outriggers are made of CFT (concrete filled tube, max. Fc=80N/mm^2) to improve the vertical stiffness and thereby restrain the tower section's bending deformation. The belt truss functions as a transfer truss that transforms the east-west span (in the X-direction) which is 9.6 meters for the podium into a 6.4 meter span for the tower. The colonnade along the outer perimeter of the tower increases the torsional stiffness of the tower and reduces the torsional deformation (Fig.-3, 4).

Furthermore, high-capacity oil dampers with a damping force of 6,000kN [1] are installed to restrain the higher-order mode deformations in the middle part of the tower (Fig.-5). They are high-capacity slim dampers with smaller diameters achieved by a new mechanism while being arranged in the same way as the conventional parallel mechanism of three 2,000kN dampers. We have developed the system because we often encounter difficulties in securing a space for installing dampers despite our desire to ensure high-capacity damping force in design.



East-West Sectional View (X-direction)   North-South Sectional View (Y-direction)   Stimulus functions in Y-direction

Fig.-3 Structural sectional views

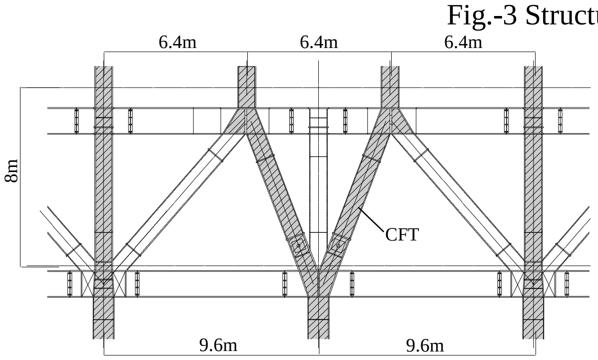
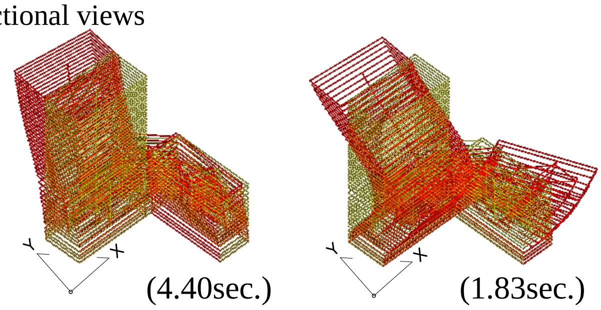


Fig.-4 Detail of belt truss



First, Y-direction   Second, Y-direction

Fig.-5 Natural modes of deformation

5.2 Realization of L-shaped planar structure planning of podium (application of “Damping Tail” system to match L-shaped plan having eccentricity of loading)

The podium of this building is an L-shaped configuration measuring approximately 150 by 150 meters in plan. Besides, there is a tower only on the north side, one of the sides of the L shape, which yields a deviation between the centers of rigidity and gravity in the podium. Consequently the building is prone to an increase in the torsional deformation of the whole podium when horizontal load is applied. It is critical to restrain the deformation.

In this project, we adjusted planar rigidity to achieve balance between the tower and the podium sections and match the centers of rigidity and gravity. Specifically, highly-rigid, buckling restrained braces are laid out in the tower’s lower part, while the podium using a rigid-frame structure becomes relatively less rigid than the tower’s lower part (Fig.-6). Moreover, viscous damping devices are intensively located mainly along two edges of the L-shaped plan of the podium section where torsional deformation tends to be amplified. We call this system “Damping Tail” system, aiming to restrain the podium section’s deformation and absorb the seismic energy throughout the building at the “tail-shaped” portion attached to the tower (Fig.-7). Table-2 shows a list of the damping devices applied to this building.

In this building, it is important in structural planning to firmly integrate the tower section and the podium section. On the two ends of their connection installed are steel large-box-section beams called Rail Beams with the maximum thickness of 80 mm and yield strength of 385 N/mm^2 each to bear the bending moment and axial force applied to the connection (Fig.-8). The shear force applied to the connection is borne by the slab (the maximum strength is Fc=36 N/mm^2). The shearing unit stress of the slab is set so as to be lower than its cracking unit stress, thereby ensuring the transmission of shear force through the slab.

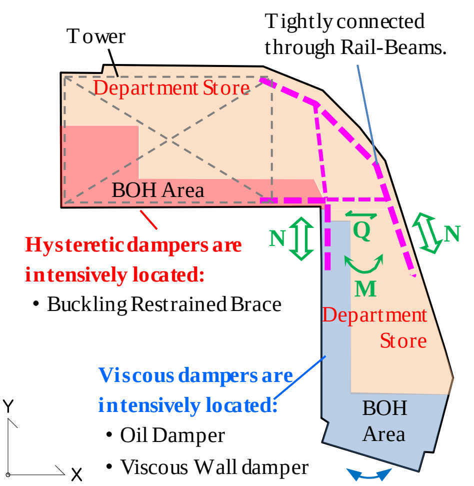
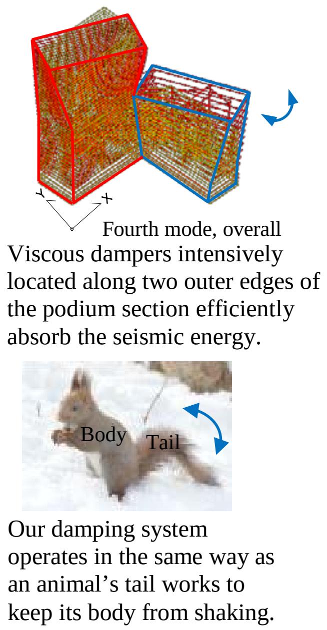


Fig.-6 Layout of damping members



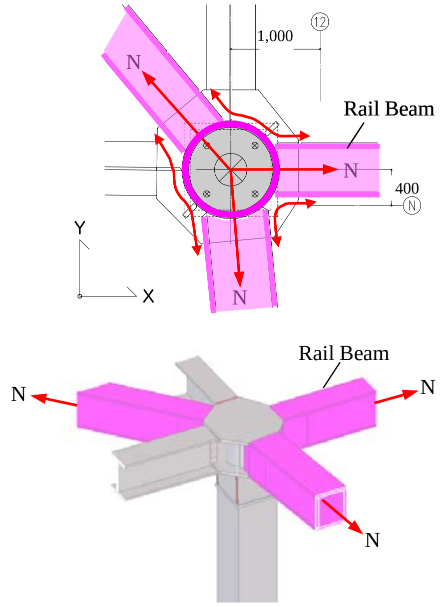


Fig.-7 Overview of Damping Tail system Fig.-8 Detail of Rail Beam joint

Table-2 Damping devices applied to this building

Hysteretic Dampers	Viscous Dampers	
Buckling Restrained Brace	Oil Damper	Viscous Wall damper
  728 pcs. Low yield steel braces having excellent deformation performance absorb seismic energy.	 310 pcs. Oil in cylinders absorbs the shaking and impact on the building.	 230 pcs. Viscous wall between steel plates absorbs the shaking and impact on the building.
Rigidity matching the weight of the tower area ensured before seismic energy is absorbed.	Seismic energy fexibly absorbed without giving rigidity.	

5.3 Composite foundation structure planning to equalize vertical and horizontal deformations under different loading conditions

The foundation structure mostly serves the tower section. The cast-in-place concrete piles are driven with the tip load bearing layer 50 meters below the ground level in the north area where the support reactions reach 40,000 to 50,000 kN. Our originally developed multi-stage diameter-enlarged cast-in-place concrete piles with their middle enlarged-diameter parts additionally placed in the gravel layer near 27 meters below the ground level [2] are used where the load is especially heavy or extraction force is applied to the piles (Fig.-9).

A mat foundation is applied to the podium in the southeast site, where the bearing layer is a gravel layer near 27 meters below the ground level. Under the foundation bottom 18.5 meters below the ground level, the existing frame, approximately 3.5 meters thick, remains down to 22 meters below the ground level. In order to ensure the safety during dismantling of the exisitng frame, the approximately 5 meter thick ground between the foundation bottom and bearing layer was improved by high-pressure jet, which was also used for ground improvement during new building construction.

The underground and foundation areas of the podium and tower are divided by the road and underground mall. Therefore, we studied vertical and horizontal displacements caused by sustained loading and earthquakes by using static and dynamic methods to keep the foundations from remarkable differences in deformations [3,4]. The dynamic study uses different equivalent shear models, each equipped with sway springs and rocking springs, according to the foundation type, as elaborated in Chapter 6.

Application of the foundation types that firmly support the podium and the tower respectively is effective to keep the planar deformation angles of the podium's and the tower's underground portions in an earthquake as extremely small as up to 1/6,400 (Fig.-10) even against Level-2 earthquake motions. Besides, the difference in deformation under vertical loading between the foundations of the podium and the tower is as slight as 1/1,100. The behaviors of the underground portions in an earthquake have been separtely validated by using a 3D FEM model (Fig.-11) [3,4].

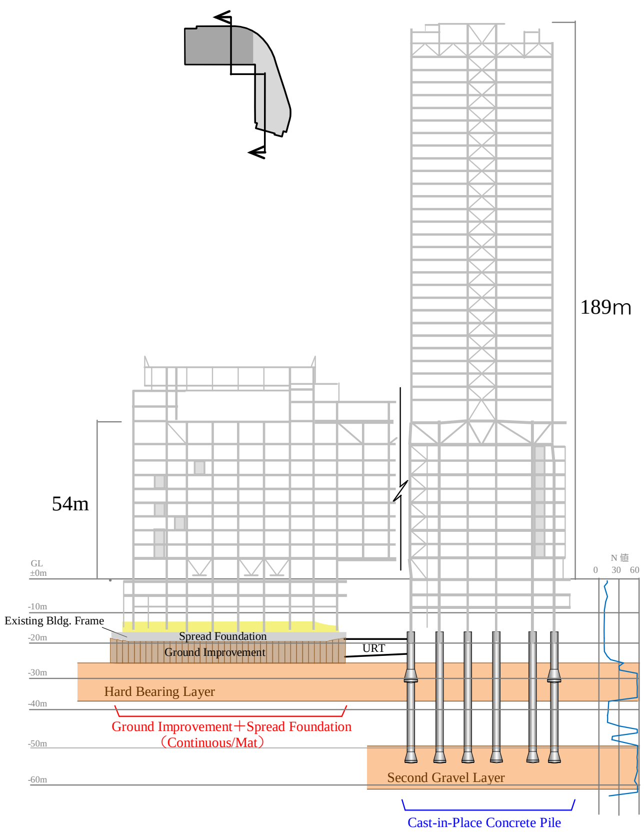


Fig.-9 Illustration overview of foundations

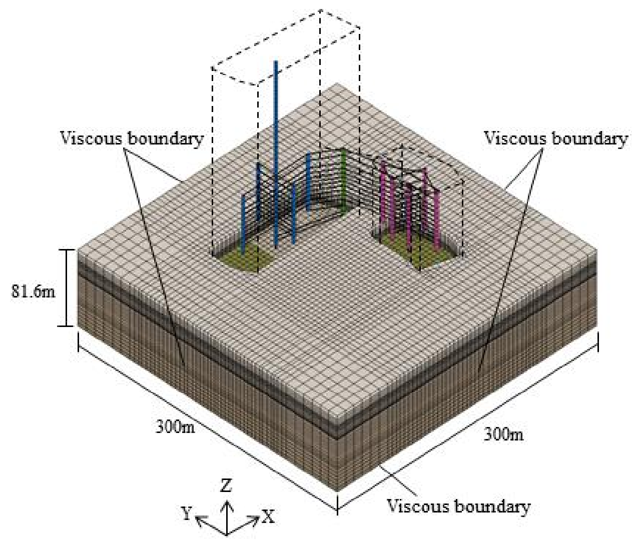


Fig.-10 Three-dimensional FEM model

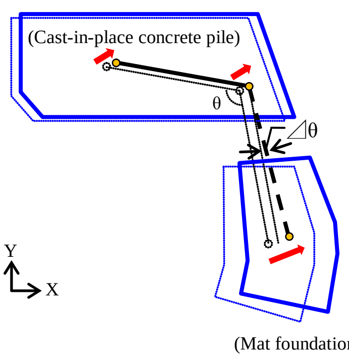


Fig.-11 Torsional deformation of underground portions in earthquake

6. Study of Seismic Response Analysis Results

6.1 Seismic response analysis model

This building consists of a podium L-shaped in plan and a regular-shaped tower located at a biased position above the podium. Therefore, it is critical to grasp the building behaviors, considering the effects of the podium's eccentricity and torsion. We have conducted a 3D seismic response analysis using a full model to evaluate them. The full model has mass points as tremendous as approximately 53,000, which requires a lot of calculation time. Thus we used an equivalent shear model on the 11 blocks, into which the whole building is split, in the design development phase after confirming the compatibility of the two models. Moreover, the data of the 11 blocks were tallied in 3 areas according to the building configuration, the results of which were compiled. Fig.-12 shows the equivalent shear model used for the seismic response analysis.

Each of the blocks was provided with a mass point where the mass of the corresponding area was concentrated. Besides, the horizontal rigidity of the frame in each block was assessed using an equivalent shear plastic spring. The blocks were connected to one another by the elements having the horizontal bending rigidity and shear rigidity in consideration of the thickness and minimum width of the slab connecting one block to another. The earthquake motions were input on the position of the third basement floor. The sway springs and rocking springs in consideration of the dynamic interaction between the structure and the ground were set to the position of the third basement floor. Concerning the internal viscous damping of the building, the damping constant of each building in the first mode was set to 2%, and the ground's damping was considered only for the rocking vibrations. Table-3 shows the natural periods of a mass system model.

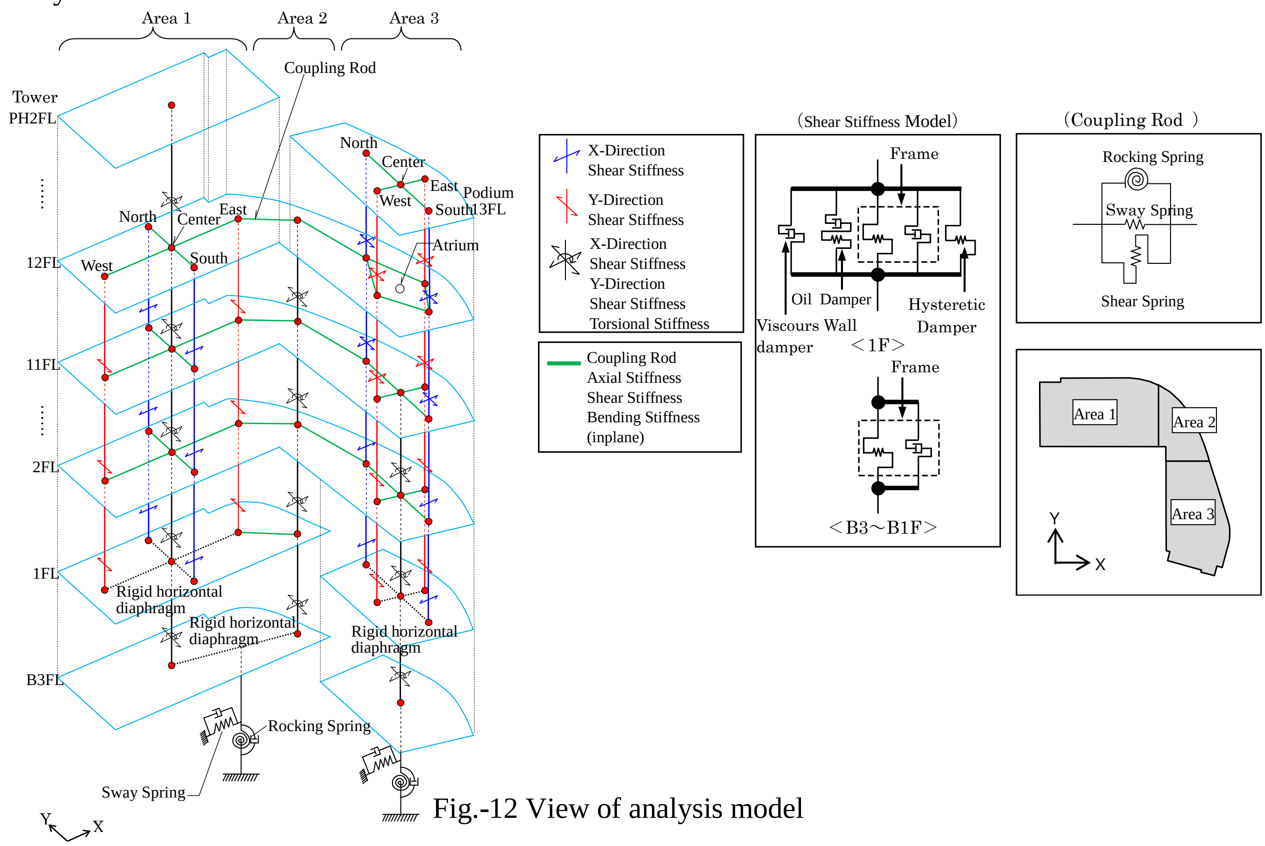


Fig.-12 View of analysis model

Table-3 Natural periods (in seconds)

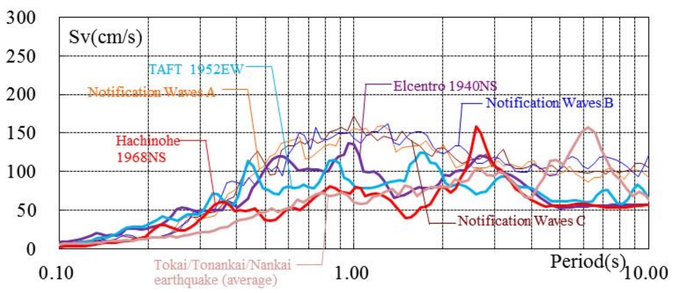
Entire mode order		1st	2nd	3rd	4th	5th	6th
Natural period (sec.)	Equivalent shear model	4.40	3.72	3.38	2.17	1.83	1.56
	Full model	4.33	3.60	3.38	2.08	1.65	1.47
Predominant mode		Tower Y: First	Tower X: First	Tower Torsion: First	Podium XY: First	Tower Y: 2nd	Tower X: 2nd

6.2 Input earthquake motions

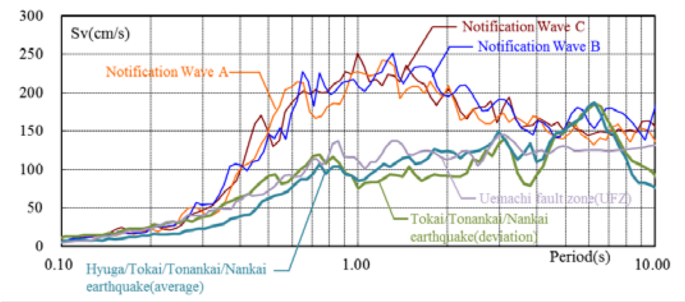
The input earthquake motions can be classified into three types as shown below in Table-4: (1) The earthquake motions provided for in the Japanese laws and the standard earthquake motions, (2) the earthquake motions beyond an expected level which are 1.5 times higher than the Level-2 earthquake motion accelerations on engineering bedrock and (3) the earthquake motions in consideration of regionality such as long-period earthquake motions (subduction zone) [5, 6] and epicentral earthquake motions. Fig.-13 shows the velocity response spectra of the input earthquake motions. We discussed the earthquake motions (2) and (3) above with the Client after March 11, 2011 (the date of the Great East Japan Earthquake) and decided to apply them to our analysis.

Table-4 Input earthquake motions for design purpose and seismic safety margin analysis

		Level 1		Level 2		Level 3 (Seismic Safety Margin Analysis Level)	
		Vmax (mm/s)	Amax (mm/s^2)	Vmax (mm/s)	Amax (mm/s^2)	Vmax (mm/s)	Amax (mm/s^2)
Standard Waves	Elcentro 1940 NS	(1) 250	2,555	500	5,110	–	–
	TAFT 1952 EW	250	2,485	500	4,970	–	–
	Hachinohe 1968 NS	250	1,669	500	3,338	–	–
Notification Waves	Notification Wave A	159	1,107	744	3,995	(2) 1,050	5,250
	Notification Wave B	132	996	714	4,278	1,065	6,075
	Notification Wave C	102	1,128	485	3,993	700	5,230
Regional Waves	Tokai/Tonankai/Nankai earthquake (average)	(3) –	–	304	1,916	–	–
	Tokai/Tonankai/Nankai earthquake (deviation)	–	–	–	–	494	2,956
	Hyuga/Tokai/Tonankai/Nankai earthquake (average)	–	–	–	–	430	2,313
	Uemachi fault zone (UFZ)	–	–	–	–	1,155	5,322



level 2

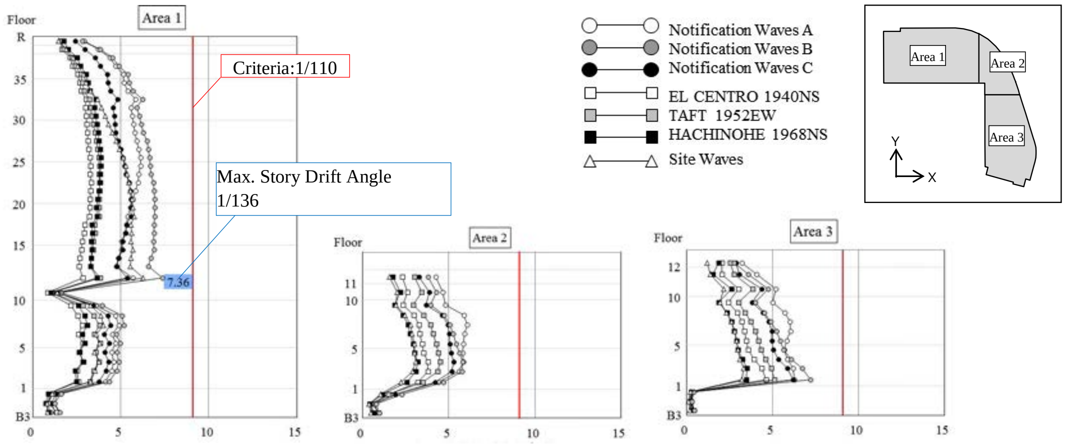


level 3

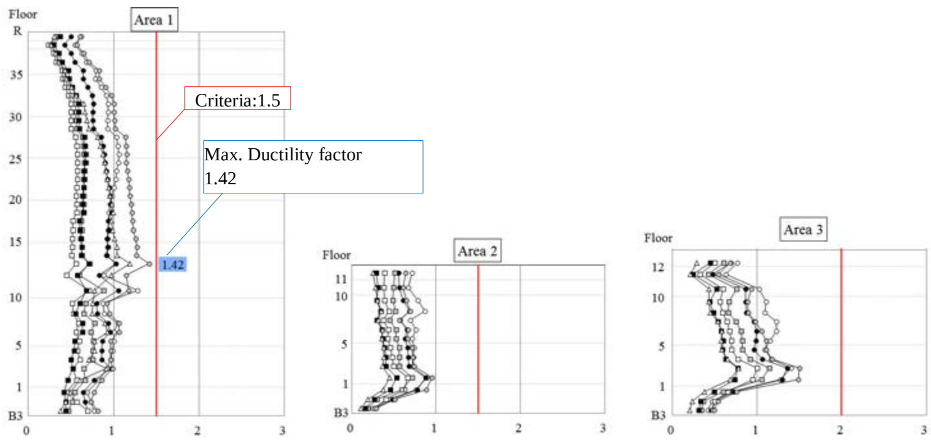
Fig.-13 Velocity response spectra

6.3 Results of seismic response analysis of Level 2

Fig.-14 shows the results of the analysis of the responses to the Level-2 earthquake motions, all of which meet the design criteria established in Section 4. These results indicate that the belt and hat trusses exhibited effects in the Area -1, showing a reduction of the story drift angles at the 10th and penthouse floor levels compared with those at the other floor levels. Furthermore, installation of high-capacity dampers has caused a mitigation of the higher-order mode effects as demonstrated by the story drift angle values at the 18th to 31st floor levels which are not increased so much as those at the other floor levels. We have also confirmed that the responses to the Level-1 earthquake motions meet the design criteria.



Max. story drift angles ($\times 10^{-3}$rad)



Max. ductility factors

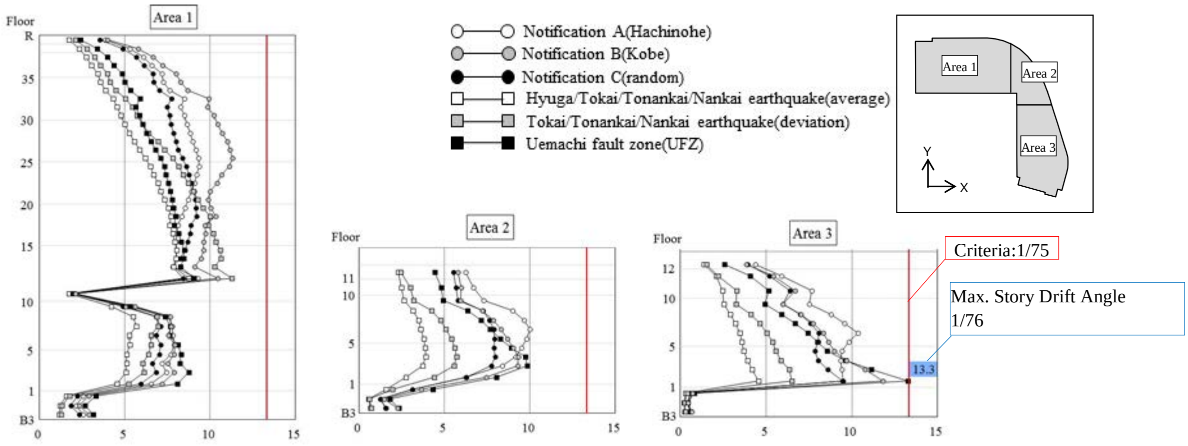
Fig.-14 Results of response analysis of Level-2 earthquake motions (Y Direction)

6.4 Results of seismic response analysis of Level 3 (Seismic safety margin analysis level)

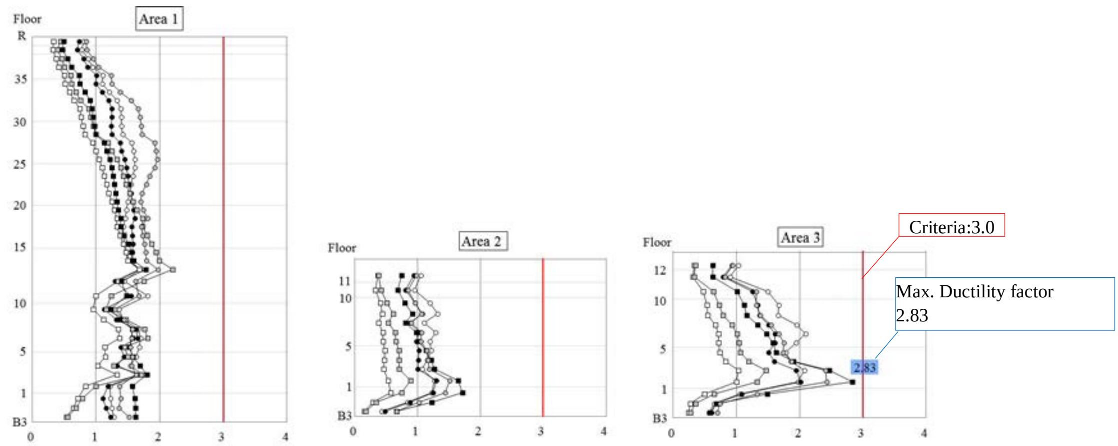
The Great Hanshin-Awaji Earthquake (epicentral earthquake) in 1995 and the Great East Japan Earthquake (subduction-zone earthquake) in 2011 have led to an increased need for study of and preparedness for various types and scales of earthquakes in Japan.

This building is situated in the heart of a city and expected to be used by tens of thousands of people every day. The designers and Client have shared the importance of ensuring the safety of such many and unspecified users and of the building which was incorporated into the structural planning and design of the building.

Specifically, we have performed validations against an Uemachi Fault Zone Earthquake (epicentral) and a Nankai Trough Earthquake (subduction-zone) which are highly likely to affect this building, and furthermore against earthquakes on a scale which is 1.5 times larger than that of the statutory level of earthquake motions. Fig.-15 shows the results of these validations.



Max. story drift angles ($\times 10^{-3}$rad)



Max. ductility factors

Fig.-15 Results of response analysis of Level-3 earthquake motions (Y Direction)

7. Conclusion

The skyscraper irregularly-shaped both in plan and in elevation has achieved the seismic performance beyond the conventional design criteria by using the following three distinctive means to ensure safety in a reliable and effective way: 1) Framing system, 2) installation of damping devices and 3) foundation system.

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