International Journal of
High-Rise Buildings
www.ctbuh-korea.org/ijhrb/index.php

International Journal of High-Rise Buildings
September 2022, Vol 11, No 3, 157-170
https://doi.org/10.21022/IJHRB.2022.11.3.157

Structural Design of High-Rise Building in Toranomon-Azabudai Project (A Block)

Kazumasa Okabe[1], Kai Toyama[1], Takuya Furuta[1], Jyunichi Yamashita[2], Hiroki Mukai[2†], Takahiro Goseki[2], Shingo Masuda[2], Dai Shimazaki[3], Yusuke Miyagi[3], and Yuji Ozawa[3]

[1]*Mori Building Co., Ltd.*
[2]*Nihon Sekkei, Inc.*
[3]*Shimizu Corp.*

Abstract This paper explains about structural planning and structural design of the high-rise building in Toranomon-Azabudai Project (A Block) which is now under construction. The building is about 330 meters high, has 4.2 aspect ratio, and the outline of the building has shallow curve. We adopted seismic response control structure. The building is a steel rigid frame structure with braces, and it has enough stiffness to obtain its primary natural period to be less than about seven seconds, in consideration of wind response, seismic response and inhabitability for the wind shaking. In terms of business continuity plan, the building has a high seismic performance; value of story drift angle shall be 1/150 or less and members of the building remain almost undamaged while or after a large earthquake. Active mass dumper shall be installed at the top of the building to improve inhabitability while strong wind is blowing.

Keywords Structural design, Seismic response control structure, Seismic performance, Inhabitability, Active mass damper

[†]Corresponding author:
Hiroki Mukai
Tel: +81-50-3139-7143
Fax: +81-3-5325-8844
E-mail:
mukai-h@nihonsekkei.co.jp

1. Introduction

The building is the A block of the Toranomon-Azabudai Project which is in Toranomon-Azabudai District Type 1 Urban Redevelopment Project Area. The site is close to Kamiyacho Station on the Hibiya Subway Line and Roppongi 1-chome Station on the Nanboku Subway Line The location is highly convenient for transportation.

The project promotes large-scale land use conversion through urban redevelopments. It upgrades traffic node function by improving streets and pedestrian networks in the area. Also, it upgrades residential functions including hotel to meet diverse needs. One goal of this project is to create a town that can be a base for international business and exchange with an environment that helps foreigners to live comfortably.

The A block building is one of three high-rise buildings in this large project. The higher part of building serves as offices and residences and the lower part of building serves as commercial facilities and school.

2. Building Outline

Location:1-1000-1, Azabudai, Minato-ku, Tokyo

Building use: Offices, residences, retail stores, grocery stores, restaurants, cafes, bars, fitness gyms, medical facility, school, parking lots, assembly halls, museums.

Developer: Toranomon-Azabudai District Urban Redevelopment Association

Designer: Mori Building Co. Ltd., Nihon Sekkei, Inc., Shimizu Corp. (design of underground structure)

Main Contractor: Shimizu Corp.

Building area: Approx.15,300 m^2

Gross Floor Area: Approx.461,800 m^2

Number of Floors: 5 basement floors, 64 floors above ground, 2 floors of penthouse

Building Height: Approx.330 m

Structural Type:

Part above the ground: S structure (Columns: CFT and S structure, Beams: S and partially SC structure), Rigid-frame structure with braces (Seismic response control structure)

Part under the ground: SRC structure (partially S and RC structure), Rigid-frame structure with seismic bearing walls

Foundation structure: Raft foundation (partially Pile foundation)

Damping devices: Oil dampers, Viscous-wall dampers

Figure 1. Perspective.

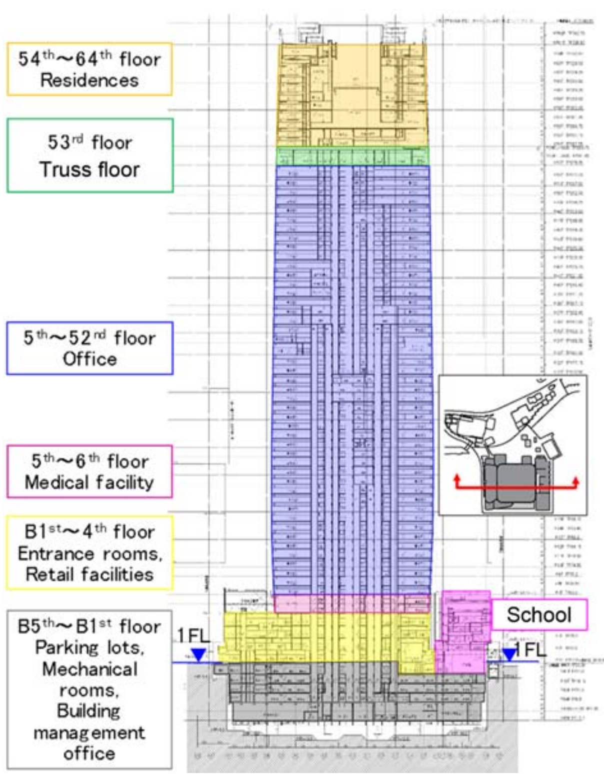

Figure 2. Building use components.

Others: Installing Active mass dampers (AMD) on the top of the building.

3. Structural Plan

3.1 Overall Structural Plan

This building is a multi-use complex building planned in A block of Toranomon-Azabudai district which is about 330 m high and has 64 floors and 5 floors underground. The building is composed mainly of a tower and school building and underground structure which covers the entire site. Building layout plan is shown in Fig. 3. Perspectives of structural frame are shown in Fig. 4 and 5.

The tower is in the middle of the site. It has 64 floors, and its maximum height is about 330 m. In south, north, and west side of the tower, there are one to four story buildings, and they are connected to the tower as one structure.

The school building has seven floors, and its height is about 35 m. It is on east side of the tower and planed structurally separate from the tower.

The underground part of the building covers entire site. Approach level of north part of site is lower and it is same as the level of 1st basement floor for the site is sloped in south-north direction. The north part of underground building lies under 1st basement floor level (partially under 3rd basement floor).

Other parts of the building which are one to two story and structurally separate from the tower are planned on west and north side of the tower. A chimney which is also structurally separate from other parts of the building is planned on northeast side of the site which stands from 3rd basement floor to about 50 m high.

The building use components are shown in Fig. 2. 5th basement floor serves as mechanical rooms, 4th to 1st basement

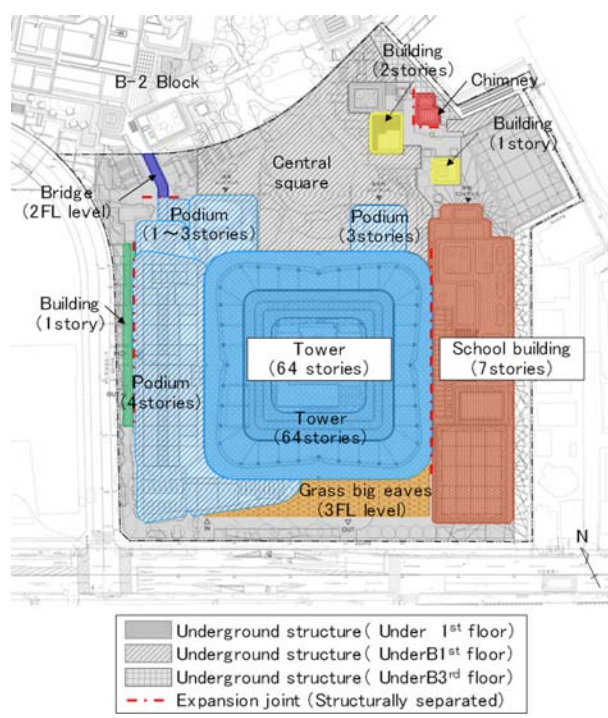

Figure 3. Building layout.

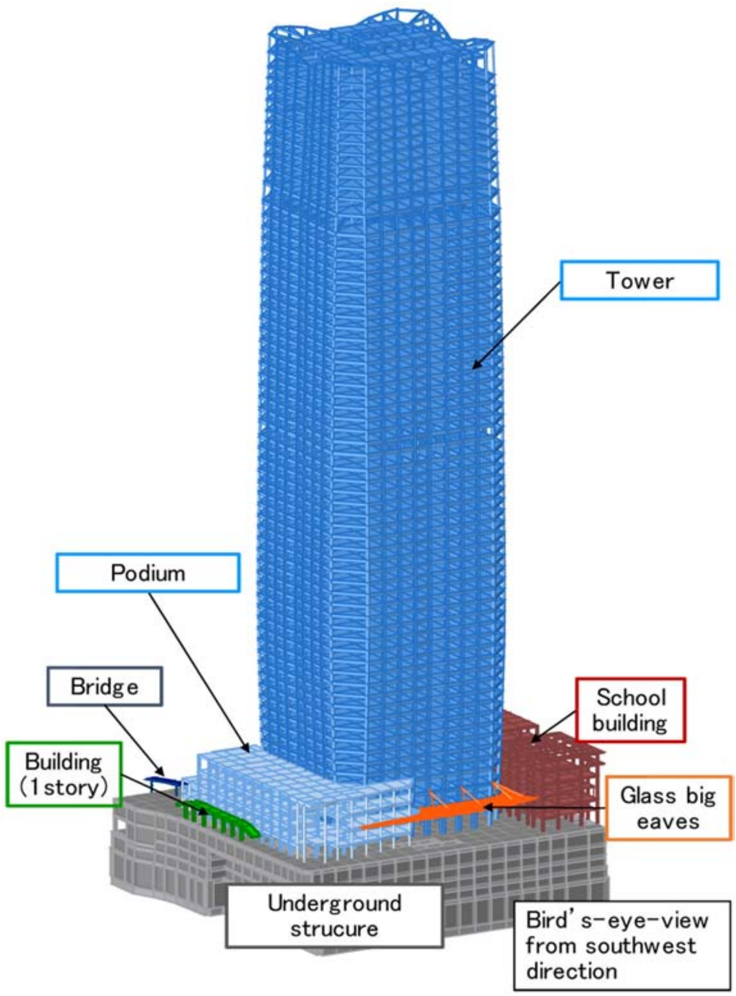

Figure 4. Perspective of structure frame. (bird's-eye-view from southwest direction)

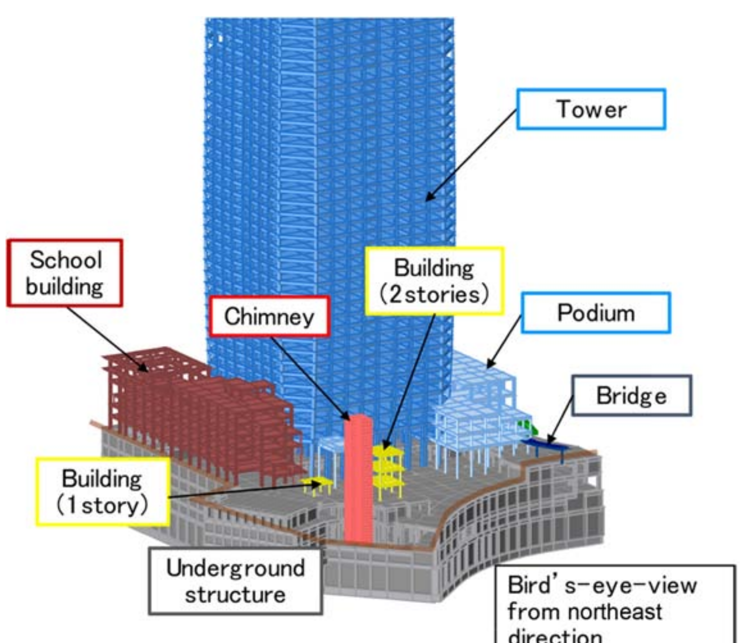

Figure 5. Perspective of structure frame. (bird's-eye-view from northeast direction)

floors as parking lots, mechanical rooms and building management office, 1st basement floor and 1st floor as entrance rooms, retail stores and parking lots, 2nd to 4th floor as retail stores, 5th and 6th floor as medical facility, 7th to 52nd as offices, 53rd as "truss floor" where columns are relocated and 54th to 64th as residences.

3.2 Structural Plan of the Tower

3.2.1 Frame Plan

Structure frame plan of the tower is shown on Fig. 6. The tower is a S structure and has rigid-frame structure with braces. It is a seismic response control structure which absorbs seismic and wind energy with damping devices installed in the building core.

Up to 52nd floor, column type is concrete-filled-tube (CFT) to reinforce bending rigidity and bearing forces. With buckling restriction braces (BRB) installed in the building core, the horizontal stiffness and bearing force are improved.

For ideal column span for residence and office are different, we set 53rd floor as "truss floor" where truss frames are formed in the whole floor to relocate the

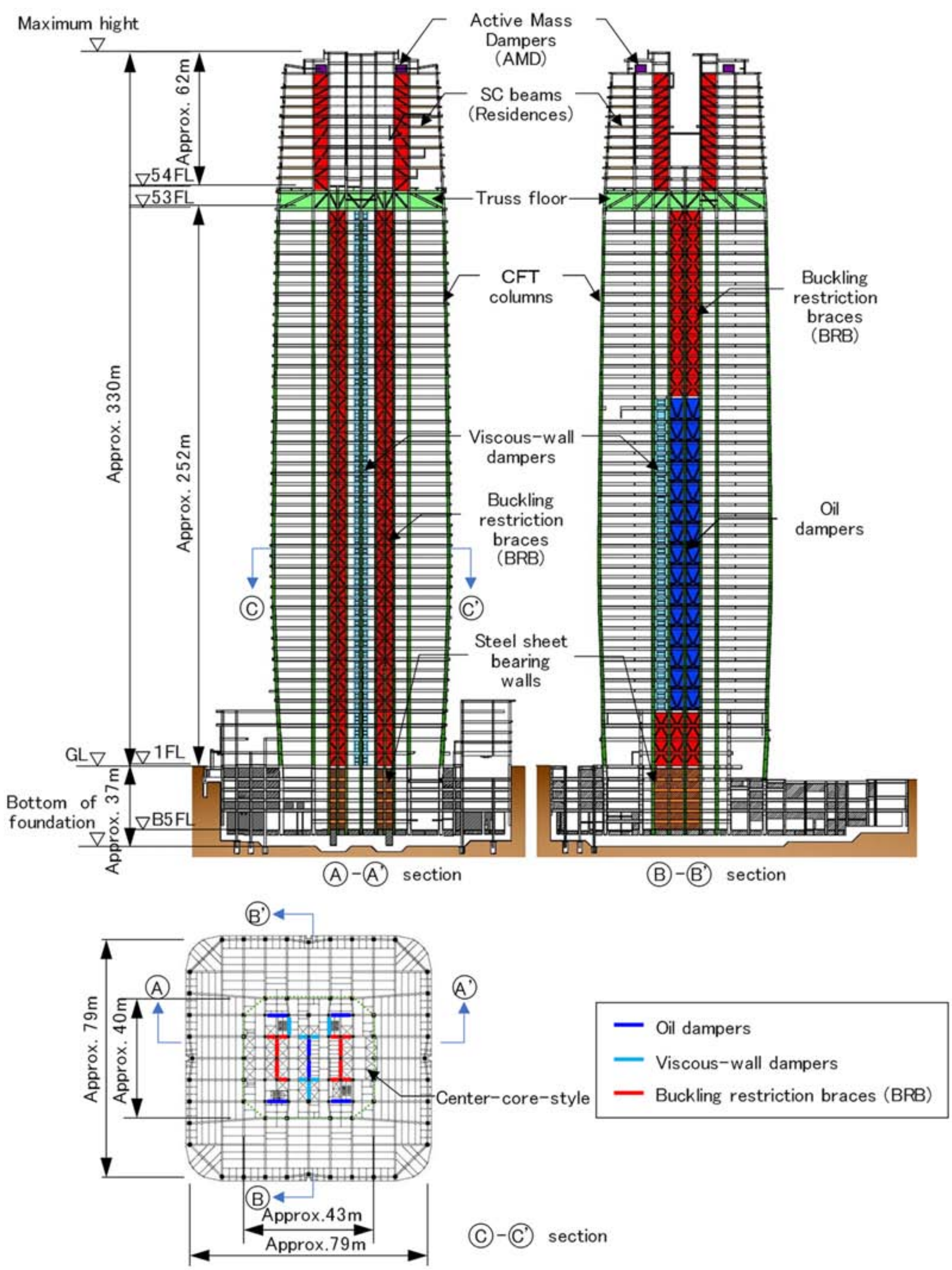

Figure 6. Structure frame plan of the tower.

columns. In the residences above 54th floor, concrete covered steel (SC) beams are adopted to reduce the heavy-weight floor impact sound. Also, BRB are placed mainly in the core to stiffen the frame. In the roof part of 64th floor residence, active mass dumpers (AMD) are installed to improve inhabitability against wind and to quickly reduce vibration after earthquake.

As core frame plan it has center-core-style, the size of core is about 43 by 40 m. The outline of the building has narrow curved lines. The planning of each floor slightly deforms as the outline is curved. In the lowest layer which is 1st basement floor, the plan is about 72 by 72 m (beams span about 14.4 m), in the middle layer where the outline grows outward it is about 79.2 by 79.2 m (beams span about 18.0 m) and at the top layer it is about 70.5 by 70.5 m (beams span about 13.7 m). Each perimeter column is slightly bended at 8 places of column-beam joint to fit the building outline.

3.2.2 Damping Device Plan

We adopted velocity-dependent oil dumpers and viscous-wall dumpers as damping devices, for those are effective for small amplitudes such as wind vibration and middle-to-small scale earthquakes to large amplitudes such as large-scale earthquakes. The devices are installed in the core of 5th to 52nd floor and it is mainly installed in 5th to 34th floor to absorb energy efficiently where story drifts are relatively large. As shown in Fig. 7, the oil dumpers are placed across two stories (skipping a floor) to double the velocity of the oil dumpers and to enhance energy absorbing efficiency.

3.2.3 Plan of Lower Part of the Tower (B1st to 4th floor) and Podium

For entrance rooms and retail stores in 1st basement floor to 4th floor have high story height which are 6.0 to 6.2 m and for there is a large atrium, as shown in Fig. 8, the perimeter columns are designed as 'long columns' which have more than 8 length/diameter ratio. For more than half of the columns of the tower are long columns, braces are placed in the core to keep horizontal stiffness.

On south, north, and west sides of the tower, there are one to four stories buildings, those low-rise buildings and the tower are planned as a single structure. Although the podium has unbalance placement which causes eccentricity ratio of the tower to be larger, but it contributes to gain horizontal stiffness and torsional stiffness.

3.3 Plan of Underground and Foundation Structure

The entrance of the building is at 1st floor level when it is accessed from Gaien-Higashi Street on the south side, and it is at 1st basement floor level when it is accessed from central square on the north side.

The underground part of building is planned as a single structure, and it covers almost entire site. The plan has about 140 by 160 meters pentagonal shape. It has basically five underground stories, but on the northeast part of the site, where swimming pool is located, it has three underground stories. In the general part, the ground level is at 1st floor level, but for the site is sloped, the ground level is at 1st basement floor level on the north part of the site.

At 2nd to 3rd basement floor level, there is a driveway, which access to other parts of the district, running through the building from east to west. And at 3rd and 4th basement floor level, there is an opening which have access to B-2 block building.

Underground structure is planned stiff enough to support the tower which is more than 300 meters high. The type of structure is mainly steel reinforced concrete (SRC) structure and reinforced concrete (RC) structure. The type of structure frame is rigid-frame structure with seismic bearing walls. But at the core of the tower, columns and beams are S structure (CFT for column) to

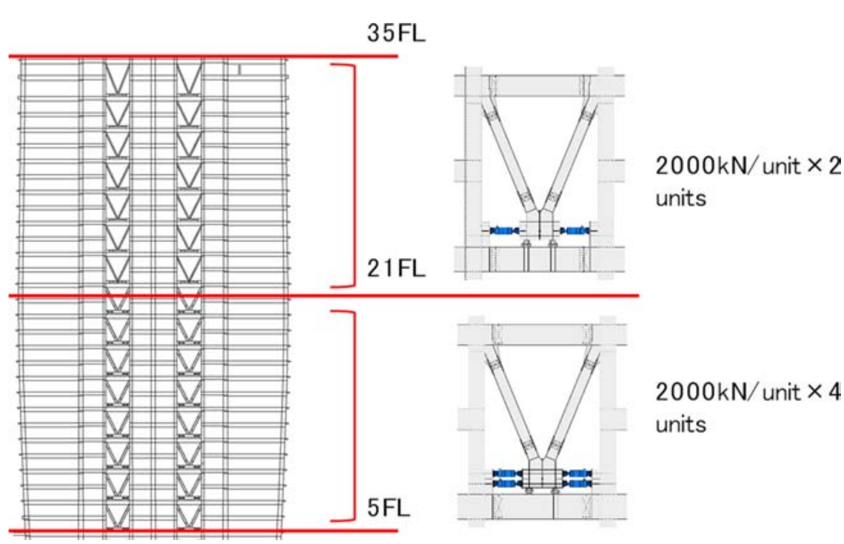

Figure 7. Oil dumpers which placed across two stories.

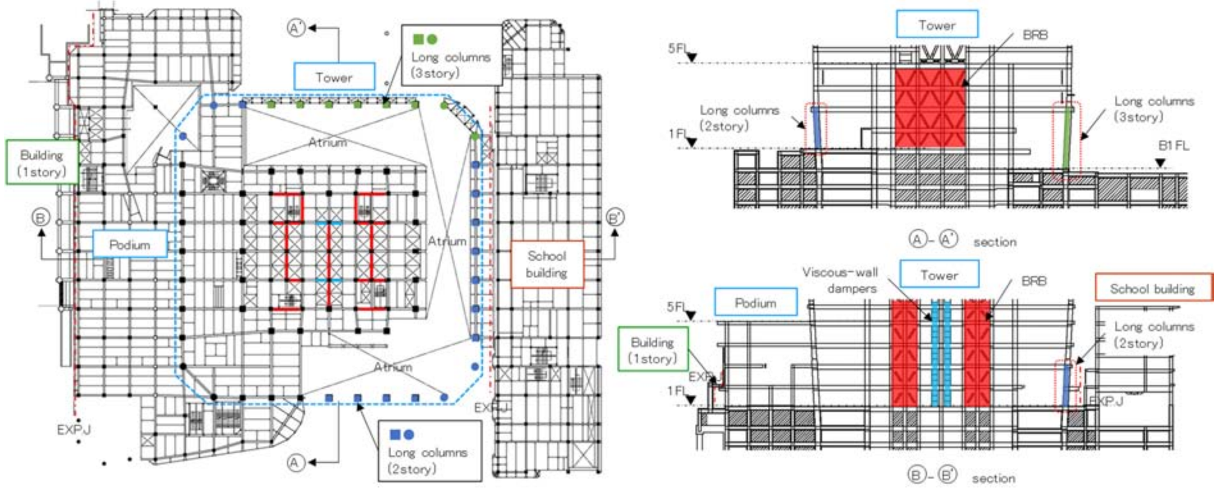

Figure 8. Structure frame plan of podium.

make the dimension of the members to be same as upper ground members for EVs and staircases to fit.

Underground structure bears against all sorts of forces constantly and while earthquakes and winds. For the depth of the structure, which is from GL to B5FL, is about 30 m, exterior wall bears against large forces such as soil pressure, water pressure and ground loading load. Unbalanced soil pressure shall cause horizonal force, for the site is sloped. Also, large horizontal force shall be applied from the tower which is caused by earthquake or wind. To bear the forces, the thickness of underground exterior walls shall be ranged from 600 mm (at 1st basement floor) to 3000 mm (at 5th basement floor). Thickness of floor slabs are designed thick enough to transfer the force between seismic bearing walls and underground exterior walls. The seismic bearing walls shall be placed in a well-balanced manner horizontally and vertically. For the thickness of slabs and seismic bearing walls placement are planned appropriately, the underground structure can transfer the large horizontal force from the tower to underground exterior walls and foundation.

The foundation is a raft foundation using a mat slab. The bearing ground shall be sand or gravel soil of Tokyo layer or Kazusa layer which is about 32.8 m below GL and which N-value is more than 60. Although maximum ground pressure of the area beneath the tower is very large and it is more than 1000 kN/m^2 in long-term, short-term ground pressure does not largely exceed the long-term ground pressure, for the placement of the tower which is at middle of the whole structure and the podium

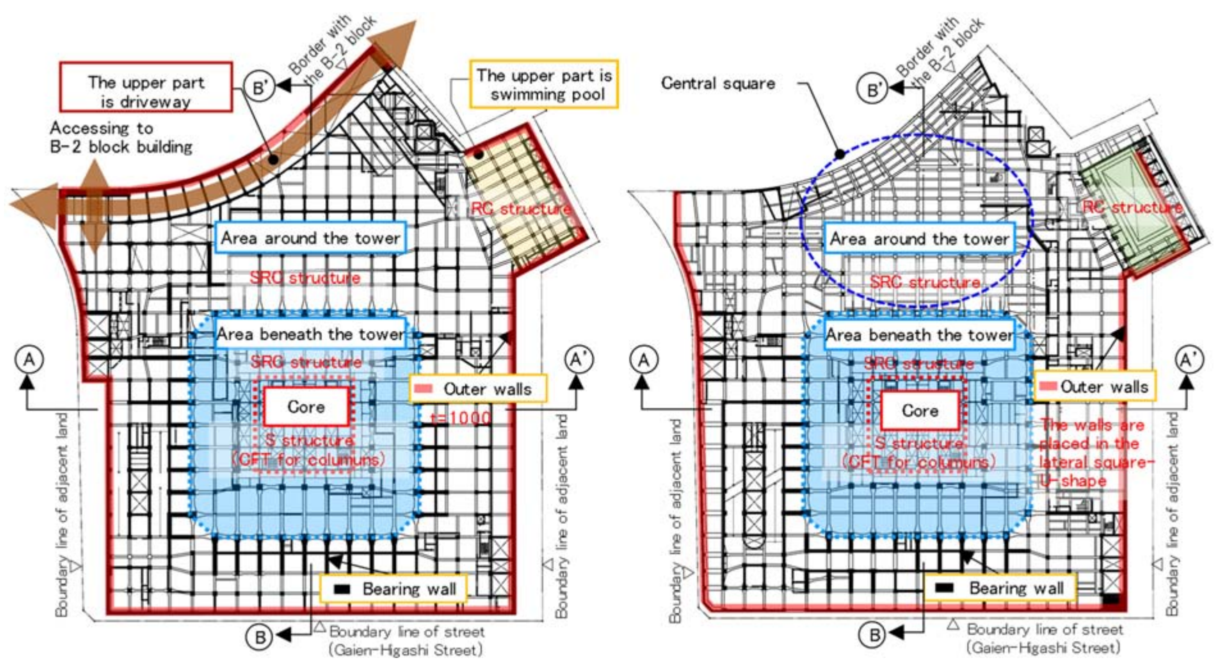

Figure 9. Plan of underground structure. (Left: 3rd basement floor, right: 1st basement floor)

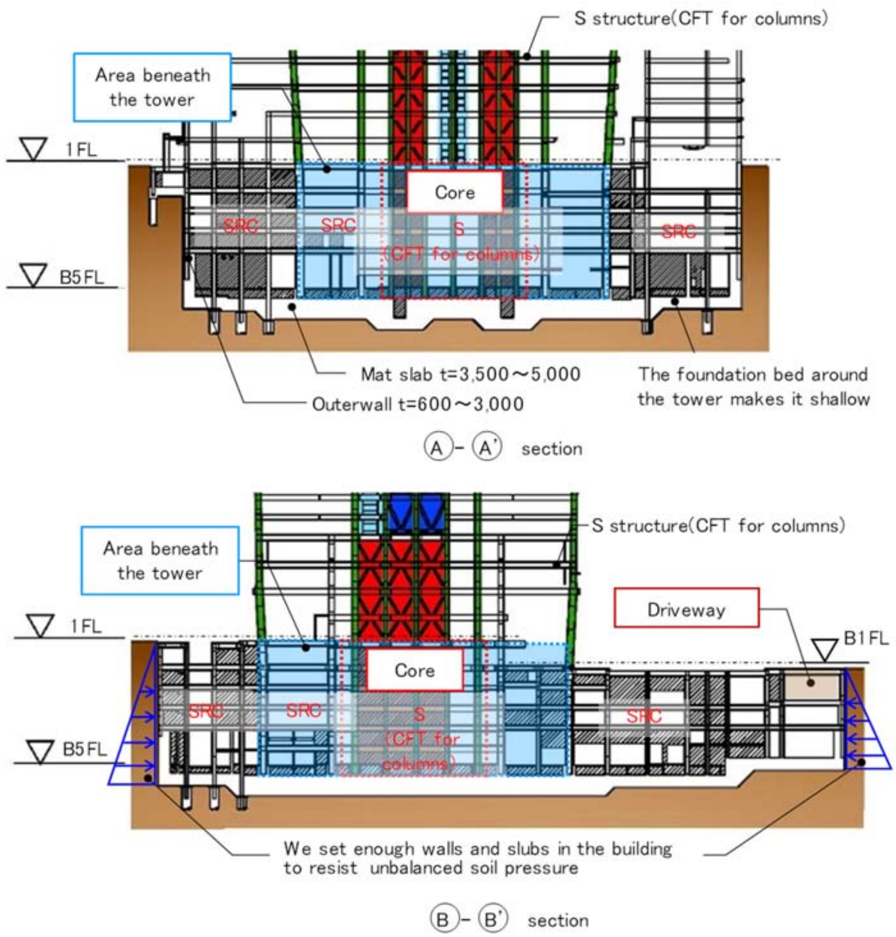

Figure 10. Elevated plan of underground structure.

which spreads around the tower is very efficient to prevent the building to fall over. We have calculated the bearing capacity of the ground with information from geotechnical report following Japanese Building Standards and set it as 1100 kN/m^2 for long-term and 2200 kN/m^2 for short-term.

The dimension of the mat slab is 5.0 m thick in the area beneath the tower and 3.0 m thick in the podium area. For it has enough stiffness and bearing capacity and for ground sinking is well considered, the mat slab can securely transfer the large force from upper ground and underground to the bearing ground.

4. Structure Design

4.1 Seismic Design

4.1.1 Seismic Performance Criteria

In terms of the business continuity plan, the building shall be able to use after a large earthquake without large repairment. Seismic performance criteria are shown in Table 1. For “rare seismic ground motion (level 1)” the building shall remain undamaged, for “extremely rare seismic ground motion (level 2)” the building shall remain almost undamaged. For further performance “seismic ground motion for margin verification (level 3)” is applied to evaluate the marginal strength of the building with 1.5 times multiplied notification waves of level 2, regional waves anticipating epicentral inland earthquakes (earthquakes occurring directory under the region), South-Kanto earthquake and simulated long-period waves assuming Nankai Trough earthquake. We consulted Reference 1) for the creation method of the regional waves anticipating epicentral inland earthquakes (earthquakes occurring directory under the region) and South-Kanto Earthquake. Specifications of input ground motions are shown in Table 2. Pseudo-velocity response spectrum diagrams of the design input seismic ground motions are shown in Fig.11.

In consideration of influence on wind response, seismic response and inhabitability for the wind shaking, the primal natural period of the building shall be less than seven seconds.

4.1.2 Sections and Materials of Main Members

Sections of main members and quality of materials are shown in Table 3. The columns of the tower mainly have built-up box shaped sections which are 1600 by 1600 mm at maximum, the maximum thickness of the steel plate is 80 mm and material classes are 490 to 590 N/mm^2 (Steel applicable to large heat input). For columns in the core of

Table 1. Seismic performance criteria

Seismic ground motion level		Rare ground motion	Extremely Rare ground motion	Ground motion for margin verification		
		Level 1	Level 2	Level 3		
Type of ground motion		・Observed waves (Vmax = 0.25 m/s) ・Notification waves	・Observed waves (Vmax = 0.50 m/s) ・Notification waves ・Long-period wave	・Regional waves (South-Kanto earthquake, Epicentral inland earthquake) ・BCJ-L2	・Notification waves (Level 2 ×1.5)	・Long-period waves (Nankai trough earthquake)
Building drift angle		1/360 or less	1/180 or less	1/120 or less		
Story drift angle	4th floor or above	1/300 or less	1/150 or less	1/100 or less		1/90 or less
	below the 4th floor	1/400 or less	1/200 or less	1/150 or less		
Member ductility factor		less than 1.0	1.5 or less	3.0 or less		
Member cumulative damage factor		-	-	less than 1.0		

Table 2. Specifications of input seismic ground motions (level 2, level 3)

Seismic ground motion level		Level 2			Level 3		
Type	Ground motion name	Max. acceleration (m/s^2)	Max. velocity (m/s)	Duration time (s)	Max. acceleration (m/s^2)	Max. velocity (m/s)	Duration time (s)
Observed wave	El-Centro 1940 NS	5.11	0.50	53.8	-	-	-
	Taft 1952 EW	4.97	0.50	54.4	-	-	-
	Hachinohe 1968 EW	2.43	0.50	234	-	-	-
Notification wave	Notification Wave1 (Hachinohe EW phase)	3.56	0.52	120	5.35	0.78	120
	Notification Wave2 (Kanto NS phase)	4.09	0.40	200	6.14	0.61	200
	Notification Wave3 (Kobe NS phase)	3.81	0.54	60.0	5.72	0.82	60.0
Long-period wave	KA1	0.46	0.22	655	-	-	-
Simulated wave	BCJ-L2	-	-	-	3.56	0.57	120
Regional wave	South-Kanto2	-	-	-	3.24	0.48	328
	Directly under the ground1	-	-	-	4.74	0.66	328
Long-period wave	Nankai trough1 (Mw8.7)	-	-	-	3.45	0.35	740
	Nankai trough2 (Mw9.0)	-	-	-	1.85	0.27	660

the tower and columns of the podium, cold-formed square steel tubes are applied, which are 1000 by 1000 mm at maximum, the maximum thickness is 40 mm and material classes are 490 to 550 N/mm^2. The columns at corner of the tower are round steel pipes which are 1600Φ at maximum, the maximum thickness is 100 mm and material classes are 490 to 550 N/mm^2. For the main beams built-up H-shaped sections are applied, which height are 900 to 1500 mm high, maximum thickness is 55 mm and material classes are 490 to 550N/mm^2. For the braces of the truss floor, built-up H-shaped sections which height are 600 to 900 mm, and built-up box shaped sections which are 900 by 800 mm are applied, maximum thickness is 45 mm and material classes are 490 to 550 N/mm^2.

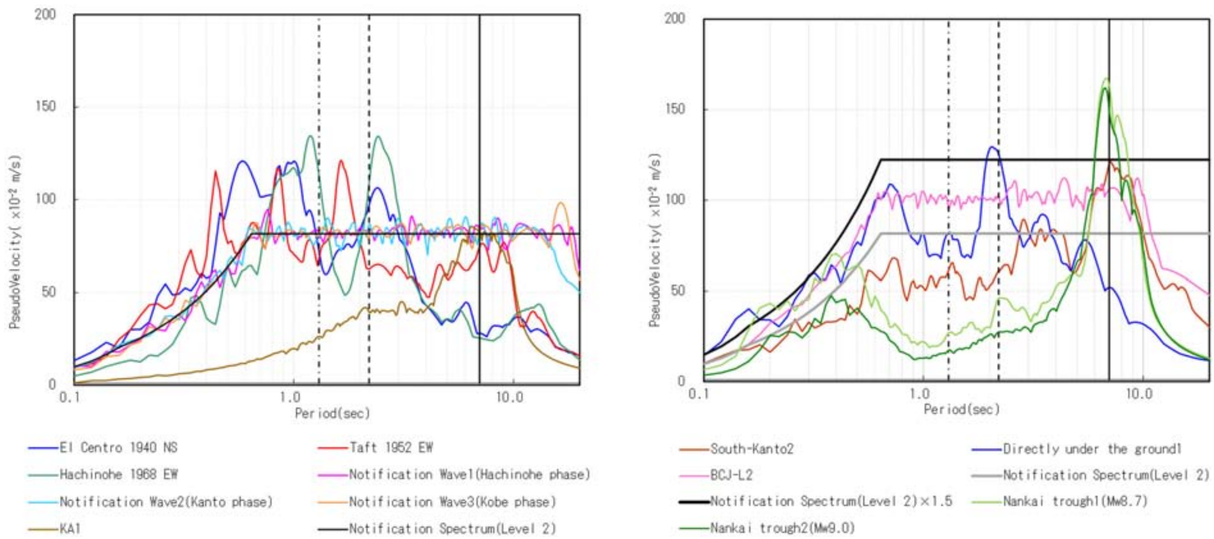

Figure 11. Pseudo-velocity response spectrum diagram of the design input seismic waves. (Left: level 2, right: level 3)

Table 3. Sections and materials of main members

Area	Structural Type, Type of section		Section (mm)	Max.plate thickness (mm)	Type of steel	Notes
Columns	CFT	Built-up box-shaped	□-800×800~ □-1600×1600	80	490~590 N/mm² C Class (Applicable to large heat input)	Max. strength of the filled concrete 120 Nmm²
		Cold-formed square steel tube	□-800×800~ □-1000×1000	40	490~550 N/mm² T,TF Class	
		Round steel tube	○ -1000~ ○ -1600	100	490~590 N/mm² C Class	
	S	Built-up box-shaped	□-700×700~ □-900×900	55	490~550 N/mm² C Class (Applicable to large heat input)	-
		Cold-formed square steel tube	□-700×700~ □-900×900	40	490~550 N/mm² T,TF Class	-
		Round steel tube	○ -850~ ○ -1000	55	490~550 N/mm² C Class	-
Beams	SC	Built-up H-shape, Ready-made H-shape	H-550×250~ H-900×450	40	490~550 N/mm² B Class	55th~ PH2nd floor
	S	Built-up H-shape, Ready-made H-shape	H-750×250~ H-1050×400	40		
		Built-up H-shape	H-1200×450~ H-1400×800	55		53rd~54th floor
			H-1200×300~ H-1500×700	55		2nd~52nd floor
Braces	S	Built-up H-shape	H-600×450~ H-800×650	45	490~550 N/mm² B Class	53rd~54th floor
		Built-up box-shaped	□-900×800	45	490~550 N/mm² B Class	

Specification of BRB and Damping devices are shown in Table 4. 1,134 units of BRB are installed, which yield strength are from 3,000 to 8,000 kN. 586 units of damping devices are installed. 304 units of Oil dampers are installed, which maximum damping is 2,000 kN. And 282 units of Viscous-wall dampers are installed, which maximum damping is from 2,000 to 3,870 kN.

4.1.3 Results of Seismic Response Analysis

Abstract of seismic response analysis model is shown in Fig. 12. The seismic response analysis model is a three-dimensional skeleton analysis model, 1st floor, and

Table 4. Specification of BRB and Damping devices

Area	Type		Yield strength (x 10^3N)		Unit number
Braces	Buckling restriction braces(BRB)		3,000		70
			4,000		552
			6,000		394
			8,000		188
				Total amount	1,134
Area	**Type**	**Max. damping force (x10^3 N)**	**Damping coefficient (x10^3 Ns/mm)**	**Max. velocity (mm/s)**	**Unit number**
Damping devices	Oil dumpers	2,000	75.0~1.44	300	304
	Viscous-wall dampers	3,870	119~19.8	90	90
		2,000	37.0~6.2	90	192
				Total amount	586

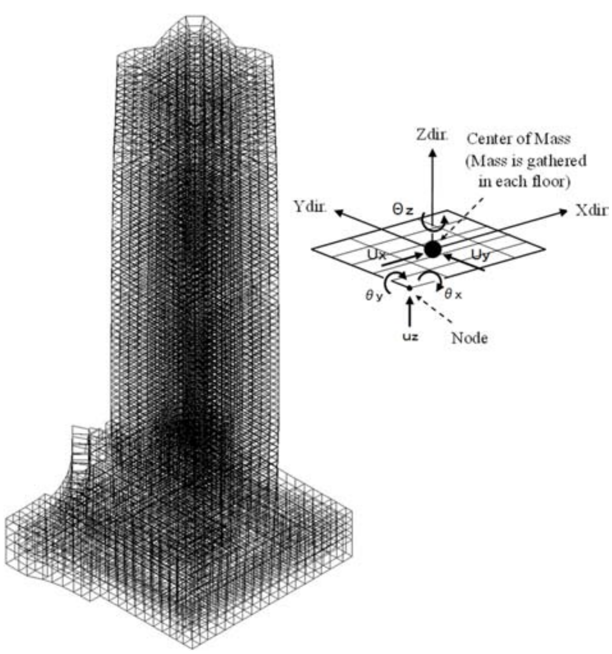

Figure 12. Abstract of seismic response analysis model.

Table 5. Result of natural period analysis (unit: second)

Direction	1st	2nd	3rd
X	7.06	2.23	1.38
Y	7.04	2.20	1.30

underground frames are fixed in horizontal direction and seismic ground motion is input to the bottom of the foundation. The internal viscous damping is h = 2.0%, and the instantaneous rigidity is proportional.

Result of natural period analysis is shown in Table 5. Primal natural periods are 7.06 seconds for X-direction and 7.04 seconds for Y-direction.

Results of seismic response analysis are shown in Fig. 13. Maximum story drift angles are 1/158 (Notification wave2) for level 2 and 1/108 (Notification wave2) for level 3 (1/96 (Nankai Though 2 (Mw 9.0)), which are within criteria.

Fig. 14 shows breakdown of absorbed amount of seismic input energy. For level 2, the absorbed energy of the main frame is less than 3% of seismic input energy, and the damage of the main frame is extremely small. Also, damping devises absorbed about 30% of seismic input energy. For level 3 (for Nankai Though1, and Nankai Though2), main frame absorbed about 20% of seismic input energy, and main frame is damaged. Fig. 15 shows cumulative damage factor of beams[2)] for level 3 (for Nankai Though1, and Nankai Though2). Because maximum value of cumulative damage factor is 0.52, which is within criteria (1.0), beams do not fracture.

4.2 Wind-resistant Design

4.2.1 Wind-resistant Design Criteria

Wind-resistant design criteria are shown in Table 6. For wind pressure of 'extremely rare storm (level 2)', the building shall remain undamaged.

Wind pressure is set based on wind tunnel experiment. Equivalent static wind loads in wind direction, perpendicular to wind direction and torsional direction are calculated by modal wind response analysis. Static analysis is performed to confirm that the structure frame is safe against the wind loads.

Result of Wind Response Analysis and Static Analysis

As design wind velocity, larger wind velocity of those two is adopted: the wind velocity calculated following Order for Enforcement of the Building Standards Act, the wind velocity calculated following AIJ Recommendations for Loads on Buildings (which return period is 100 years and 500 years) With the design wind velocity and wind

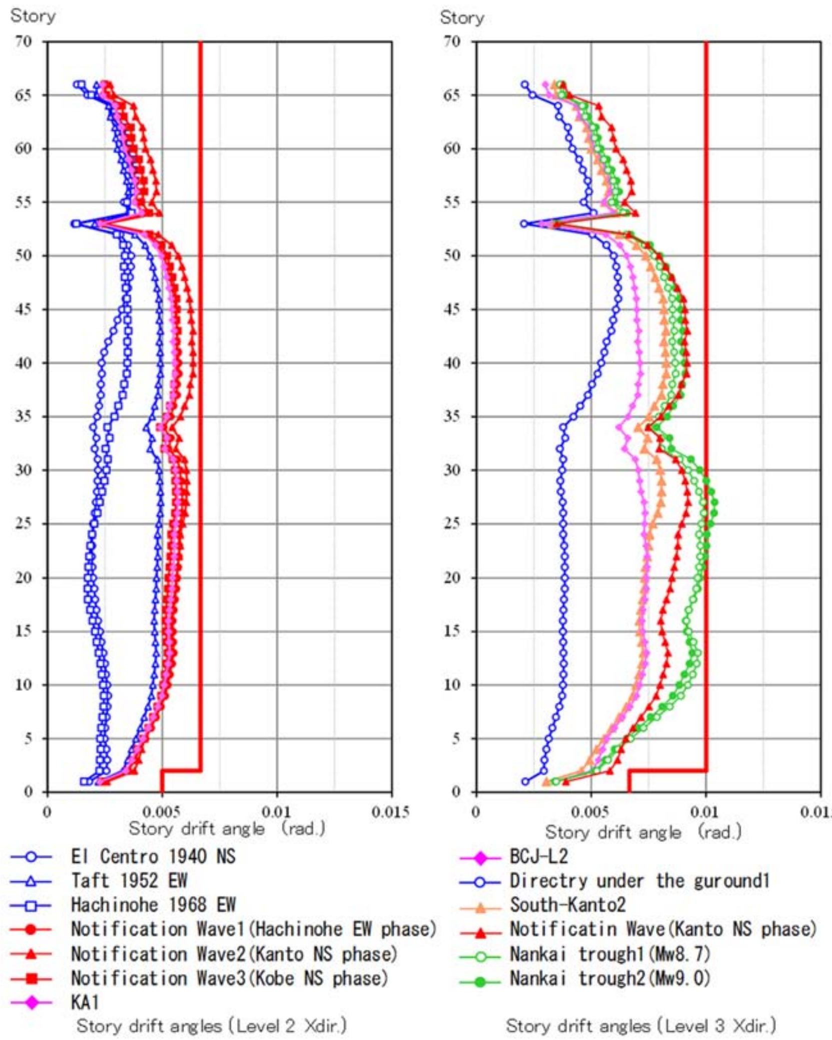

Figure 13. Results of seismic response analysis. (Left: level 2, right: level 3)

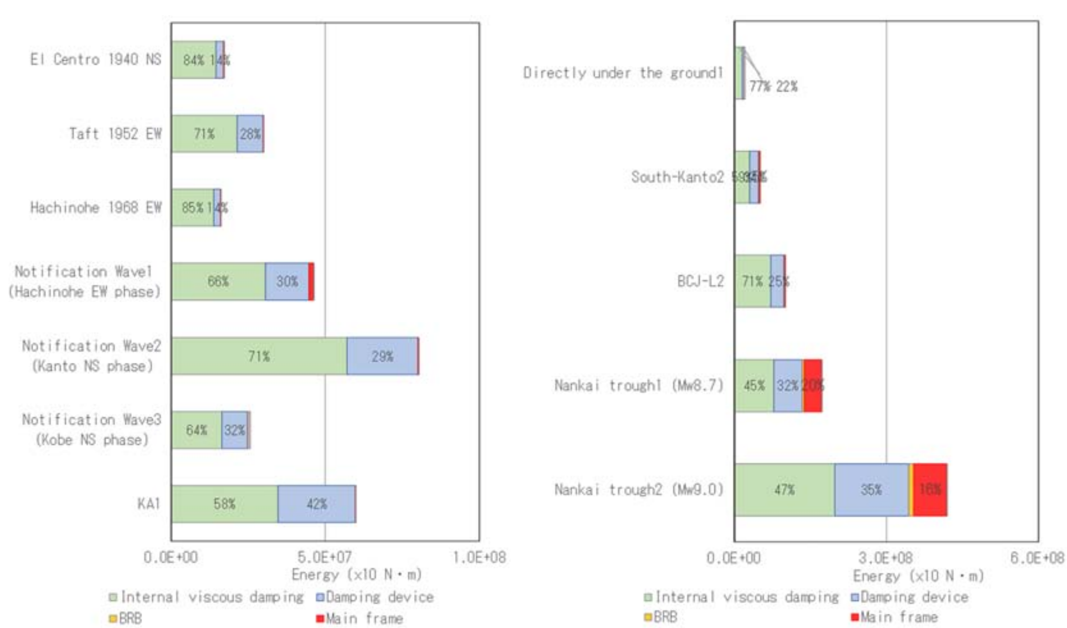

Figure 14. Breakdown of absorbed amount of seismic input energy. (Left: level 2, right: level 3)

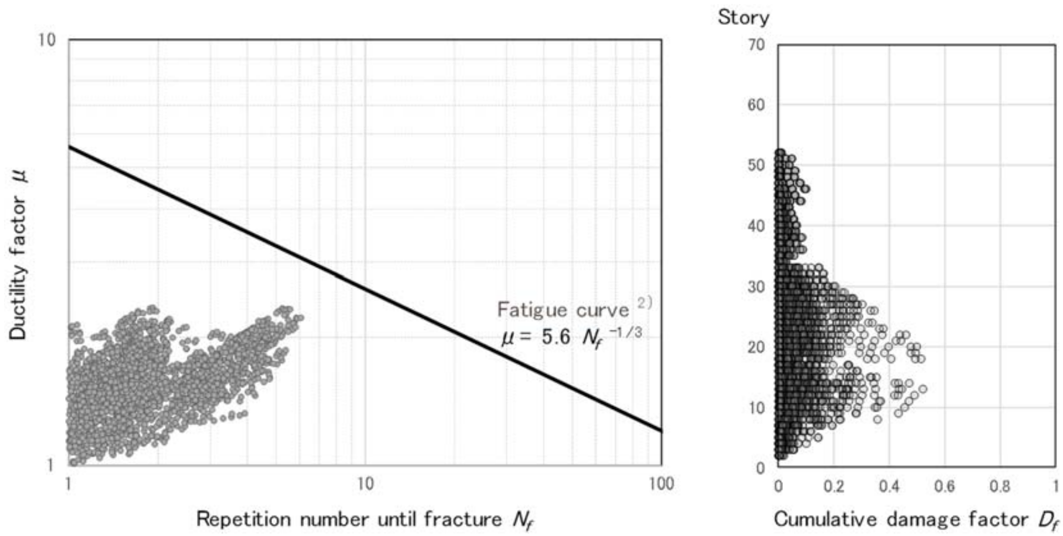

Figure 15. Damage degree of beams. (Level3: Nankai trough1, Nankai trough2)

Table 6. Wind-resistant design criteria

Wind presser level	Extremely rare storm
	Level 2
Return periods	500 years
Story drift angle	1/200 or less
Member stress	No more than the allowable strength for short-term loading

Figure 16. Wind tunnel experiment model.

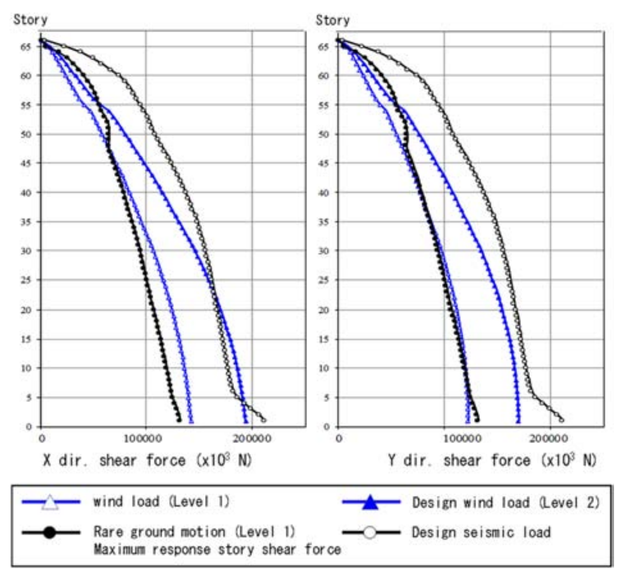

Figure 17. Design seismic load and design wind load.

load acquired from wind-hall experiment, wind pressure (equivalent to level 2) is calculated by modal analysis. In Fig. 17, seismic design load and wind design load for allowable stress design are shown. For this building, the design seismic load exceeds design wind load except from 5th to 22nd floors. In terms of designing strength of the members, seismic load is dominant.

Fig. 18 shows result of static analysis which design wind load (equivalent to level 2) of wind direction, perpendicular to wind direction and torsional direction is applied for each wind direction. Maximum story drift angle is 1/228 (at 26th floor) and it is below 1/200. Stress of every member of the building is below allowable strength for short-term loading.

4.2.3 Study about Aerodynamic Instability of Structure

For this building, design wind velocity is close to the resonance wind velocity of vortex induced vibration which is calculated from peak frequency of power spectrum density obtained by wind tunnel experiment. Therefore, we performed rocking vibration experiment to make sure that there is no probability of vortex induced vibration and aerodynamic instability vibration. Fig. 19 shows result of rocking vibration experiment. Because the experimental value does not exceed analysis value in the range of design wind velocity, there is no probability of vortex induced vibration and aerodynamic instability vibration.

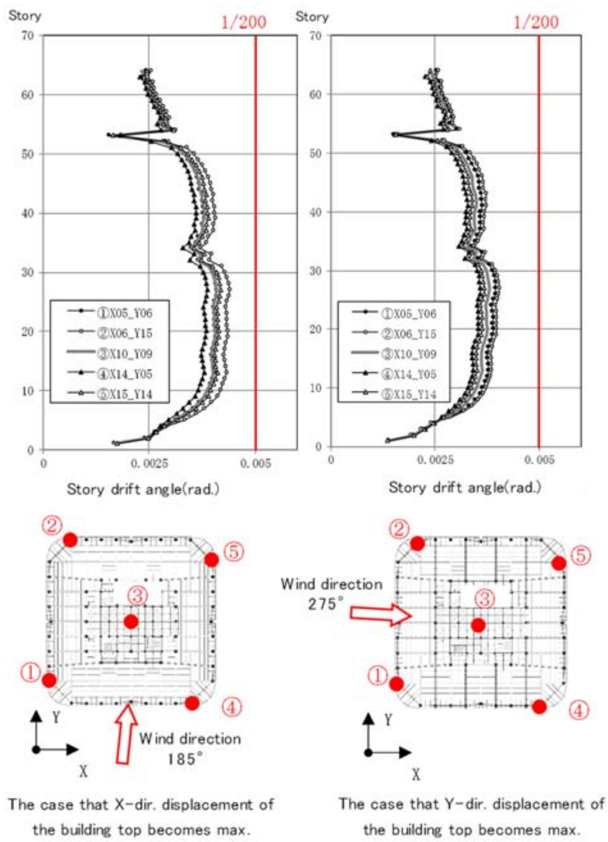

Figure 18. Result of static analysis for design wind load. (equivalent to level 2)

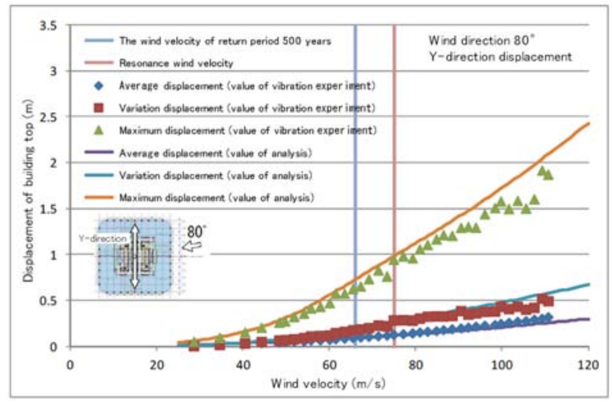

Figure 19. Result of rocking vibration experiment.

4.2.4 Study about Inhabitability

Active mass dumpers (AMD) are installed in PH floor for mainly improving inhabitability while strong wind is blowing. Each AMD mass is about 30 ton. Two devises are installed for each X-direction and Y-direction to control vibration of each direction, there are four devices in total. Specification of AMD is set aiming the response acceleration to be less than inhabitability evaluation curve for horizontal vibration "H-30" of the "Guidelines for the Evaluation of Habitability to Building Vibration (AIJ)" for wind of one year-return-period and less than "H-50" for wind of five year-return-period. It is possible to reduce about 30% of response acceleration by installing AMD.

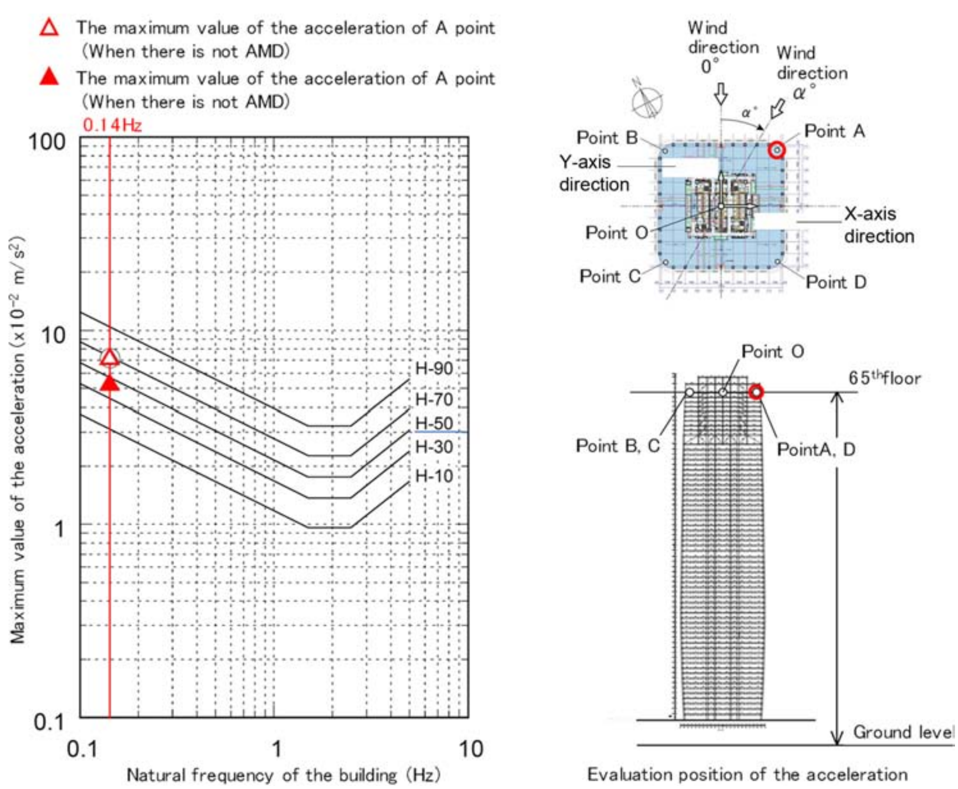

Figure 20. Inhabitability evaluation for the horizontal vibration for wind. (5 year-return-period)

Figure 21. The current construction situation.

5. Conclusion

This paper generally explains about structural plan and structure design of the tower of A block of Toranomon-Azabudai project.

When designing a 300 m class steel-framed skyscraper like this building, natural period of building will be long for example 6 to 8 seconds. It is an important issue to decide the stiffness of the building considering the effects of wind response, long-period ground motion, and inhabitability against wind.

For this building we decided the natural period to be less than about 7 seconds. We designed the building to prevent increase of wind response. For long-period ground motion, we considered not only the ground motion regulated by law but also largest scale ground motion that is assumed. For inhabitability, AMD are installed at the top of the building for improving inhabitability.

Following those policies, we adopted steel rigid frame with braces and BRB in the building core, and adopted seismic response control structure with damping devices which are effectively placed to absorb energy. Therefore, we succeeded to design economical building with high seismic performance, considered with inhabitability.

Acknowledgments

The framework construction of the tower is completed in April this year, the building is scheduled to be completed in 2023.

Finally, Authors would like to express their sincere gratitude to all personals involved in this project for giving them the opportunity to design this project.

References

TAKASHI Yamane, SUMIO Nagahashi (2005). A STUDY ON A GENERATION OF SIMULATED EARTHQUAKE GROUND MOTION CONSIDERING PHASE DIFFERENCE CHARACTERISTICS Part4 Generation method of simulated earthquake ground motion with the Fourier amplitude prescribed by the w-square model and the Fourier phase selected based on the standard deviation of the phase difference. Architectural Institute of Japan’s Journal of Structural and Construction Engineering, No.590, pp.45-53.

NARIHARA Hiroyuki, YASUDA Satoshi, HASEGAWA Takashi, and YAMADA Satoshi (2013). Study on Safety Assessment Methods for Super-high-rise Steel Buildings Against Long-period Earthquake Ground Motions (Part 20 Estimation of Low Cycle Fatigue Damage for Steel Beam at Welded Ends). Architectural Institute of Japan’s Summaries of technical papers of annual meeting, pp.1087-1088.