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Performance-Based Seismic Design for High-Rise Buildings in Japan

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Abstract

This paper introduces the outlines of review and approval processes, general criteria and usual practices taken in Japan for the seismic design of high-rise buildings. The structural calculations are based on time-history analyses followed by performance evaluations. This paper also introduces structural design of two high-rise buildings: one is a 100 m high reinforced concrete residential building, and the other is a 300 m high steel building for mixed use.

Keywords: High-rise buildings, Performance-based seismic design, Input earthquake motion, Performance criteria, Time-history response analysis, Additional damping device, Robustness

1. Introduction

Building Standard Law of Japan requires structural calculations for buildings higher than 60 m to clarify forces and deformation produced continuously in each part of the building by loads and external forces, and to ensure conformity with certain criteria. For seismic design, this presumes performance evaluation based on time-history response analysis of the building. While the minimum criteria are effectively set by the law, the performance objectives are, as long as they do not fall below the minimum criteria, determined through the dialogues with the client and are not the code requirement.

In this paper, the outlines of general criteria and usual practices taken in Japan for seismic design of high-rise buildings are introduced together with the review and approval procedures. The outlines of seismic design of two high-rise buildings, which have recently been designed, are also introduced.

Note that the information contained in this paper is no more than general, simplified, and non-comprehensive outlines. Refer to the applicable laws, relevant authorities and organizations, etc. for detailed and accurate information. Note also that this paper has been written based on the situations before "The 2011 Off the Pacific Coast of Tohoku Earthquake (Tohoku Earthquake)" on 11 March 2011, and that the latest discussions on the re-evaluation of seismic safety are not reflected.

2. Code Requirements

Structural calculations for buildings higher than 60 m, based on time-history response analyses, are required to go through performance evaluation by organizations designated by the Ministry of Land, Infrastructure, Transport and Tourism (MLIT) prior to the approval by the Minister. These review and approval processes need to be completed before application to the "building confirmation", which is required to start construction.

The performance evaluation by the designated organization (e.g. BCJ, Building Center of Japan) is conducted based mostly on prescribed minimum requirements and criteria, an example of which can be found in the website of BCJ[2)] in Japanese. (It is effectively the same for all the designated organizations.) Among such requirements and criteria, some items deeply related to seismic design are introduced in the following sections, with the practices often taken by the structural engineers in Japan. Note that the performance evaluation also covers the effects by wind, snow, earth pressure, temperature change, etc., but they are not shown in this paper.

The structural calculation procedures as well as the minimum criteria for standard buildings (i.e. buildings less than or equal to 60 m in height) are well defined by the law based on the size, use, structural type, etc. The procedures are somewhat specification-based ones for small buildings (e.g. timber or RC-wall housing), and

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static-analysis-based ones for middle to large sized buildings.

Buildings not in excess of 60 m are also allowed to go through the performance evaluation if desired by the applicant or deemed necessary by the reviewer due to inconformity with the applicable range of the rather simple calculation procedures. Buildings with irregular shape, seismic isolation or new structural material/system often take this route.

3. Definition of Seismic Demand Levels

3.1. Design input motions for horizontal directions

See Table 1 for the summary of seismic demand levels for the design of high-rise buildings. Three or more time-history-based design input earthquake motions are required for each of the Rare Earthquakes (RE) and the Extremely Rare Earthquakes (ERE). The design input motions shall be generated by the structural engineer based on the acceleration response spectra provided by the Notification issued by the former Ministry of Construction and amended by MLIT. The spectra are defined at the exposed engineering bedrock with damping ratio of 5%. Here the exposed engineering bedrock is defined as the engineering bedrock located deep underground, not affected by the subsurface layers, having sufficient depth and stiffness, and with shear wave velocity of over 400 m/s. The spectrum for ERE is five times larger than RE. In generating the design input motions, appropriate considerations shall be given by the structural engineer to the amplification caused by the subsurface layers, phase differences, etc. The duration of the input motions shall be over 60 sec. The return periods are generally understood as 50 years for RE and 500 years for ERE, but are not clarified in the law.

Alternatively to one or more of the design input motions for ERE, the structural engineer may use simulated motions, prepared specifically for the site based on appropriate considerations to the distribution of active faults, the fault rupture model, past seismic activities, the ground structure, etc. State-of-the-art research achievements in seismology are often incorporated in simulating the site-specific motions.

In addition to the design input motions above, another set of three or more representative, recorded seismic motions shall be used for each of RE and ERE. Here the maximum velocities are normalized as 25 kine (cm/s) for RE and 50 kine for ERE, and the velocities above may be multiplied by the seismic zone factor, Z, shown in Table 1. The structural engineer shall make the selection of recorded seismic motions with appropriate considerations to the properties of the site and the building.

3.2. Design input motions for vertical directions

There are less provisions for vertical input motions compared to horizontal ones. In practice, structural engineers refer to relevant guidelines, use recorded vertical motions with appropriate factoring and/or use factored horizontal motions.

4. Performance Objectives

Performance objectives, or the minimum requirements, to each of the Rare Earthquakes (RE) and the Extremely Rare Earthquakes (ERE) are set as the "Damage Limit" and the "Collapse Limit", respectively, as detailed below and summarized in Table 2. These requirements shall be satisfied for all the adopted design input motions.

Table 1. Summary of seismic demand levels

	Rare Earthquakes (RE)	Extremely Rare Earthquakes (ERE)
	Reference return period*: 50 years	500 years
Notification motions	Three or more time-history-based input earthquake motions to be generated from the acceleration response spectra defined at the exposed engineering bedrock as below: Period (s) Acc. Resp. Spectrum (m/s^2) $T < 0.16$ $(0.61 + 6T)Z$ $0.16 \leq T < 0.64$ $1.6Z$ $0.64 \leq T$ $(1.024 / T)Z$, where T is the building's natural period and Z(= 1.0 to 0.7) is the seismic zone factor. In generating the input ground motions, amplification due to subsurface layers, phase differences, etc. shall be considered.	Three or more input ground motions based on five times the acceleration response spectrum shown left.
Simulated motions	N/A	Optional site-specific motions on replacement of one or more of the ERE-Notification Motions above.
Recorded motions	3 or more recorded motions normalized to have the maximum velocity of 25 kine.	3 or more recorded motions normalized to have the maximum velocity of 50 kine.

*The law does not mention return periods.

Table 2. Summary of performance objectives / minimum requirements

		Demand levels	
		Rare Earthquakes (RE)	Extremely Rare Earthquakes (ERE)
Performance objectives or minimum requirements	General	Damage Limit	Collapse Limit
	Drift story angle	1/200 or less	1/100 or less
	Story ductility factor	(See below)	2.0 or less
	Stresses in each structural elements	Short-term allowable stress or less	(See below)
	Ductility factor of each structural elements	(See above)	4.0 or less

*Some of limiting values may be surpassed if the equivalents are shown and accepted by the reviewers.

4.1. Damage limit

The methods a) and b) below shall be used to confirm that each part of the building will not be damaged by RE. (Other methods may be used for seismic isolation layers if certain conditions are satisfied.)

a) The response drift angle of each story shall not exceed 1/200. This shall not apply if it is confirmed that the deformation of the elements necessary for structural resistance will not cause significant damage to each part of the building.

b) Induced stresses in the elements necessary for structural resistance shall not exceed the short-term allowable stresses. Alternatively, it shall be confirmed that there will be no harmful residual cracks or strains after the earthquake. This shall not apply to the elements for seismic response control complying with a certain Notification.

4.2. Collapse limit

The methods a) through d) below shall be used to confirm that the building will not collapse due to ERE. (Other methods may be used instead of a) through c) for seismic isolation layers if certain conditions are satisfied.)

a) The response drift angle of each story shall not exceed 1/100.

b) The response ductility factor of each story shall not exceed 2.0.

c) The response ductility factor of individual members composing the elements necessary for structural resistance shall not exceed the lesser of 4.0 and the criteria determined based on the structure method and vibration properties of the building. This shall not apply to the elements for seismic response control complying with a certain Notification.

d) If the response exceeds the values indicated in a), b) or c), a certain items shall be confirmed according to the degree of the excess.

5. Modeling Procedures

5.1. Modeling

The vibration system model shall be established, based on the structure type and dynamic properties of the building, so as to make it possible to appropriately evaluate the forces and deformations in each part of the building. If it is more appropriate to evaluate the responses of specific members, then the vibration model should be established accordingly. Hysteresis characteristics and damping properties in the model shall be set so as to appropriately reflect the structure type and dynamic behavior of the building. Where the hysteresis characteristics are set story-by-story basis, the characteristics shall be set based on static elasto-plastic analysis using appropriately determined load distributions on each story.

In many cases in the design of high-rise buildings, the structural engineer establishes an equivalent shear or an equivalent bending and shear separated, multiple lumped-mass model with hysteresis characteristics of each story determined through the repetition of preliminary dynamic analyses and elasto-plastic pushover analysis. (Pushover analysis using the full 3-D model is conducted to determine the hysteresis characteristics of each story in the lumped mass model. Preliminary dynamic analyses using the lumped-mass model are conducted to determine the load distributions for the pushover analysis.)

With the advance of computers and software, larger and more detailed models, even full 3-dimensional ones consisting of individual members, are becoming to be used for response analyses. However, in most cases, such models are used to confirm the validity of the simplified lumped-mass models and not to extract design forces and moments for individual member sizing.

5.2. Response analysis

Responses of the building shall be obtained by appropriately solving the equations of motion for the vibration system model subjected to the input earthquake motion. Here the responses to each of the two orthogonal major directions of the building shall be obtained separately. In addition, responses to the earthquake motions in the two directions simultaneously, or alternatively at a 45 degree angle to the major direction, shall be evaluated by an appropriate method. Effects of vertical earthquake motion shall be evaluated with appropriate considerations to the simultaneity with the horizontal motions as well as the scale and the form of the building. The effects of phase differences shall be appropriately considered when the building is expected to be affected by such effects. Also, the effects of horizontal displacements to vertical loads (so-called P-Δ effect) shall be appropriately considered.

6. Foundation Interaction

Where the effect of soil-structure interaction is expected to be significant, the vibration system model shall be capable of incorporating such effect.

While springs and dampers in swaying and rocking are often used to represent the effects of the ground, an integrated model of the structure as multiple lumped-mass model and the soil as 2 or 3-dimensional FEM model is sometimes used.

7. Damping

Damping properties in the model shall be set so as to appropriately reflect the structure type and dynamic behavior of the building.

While there is no imposed methodology or figure for setting damping properties, overall damping is often incorporated in the form of frequency proportional, tangent stiffness proportional or Rayleigh type damping. The damping ratios are set by the structural engineer based on considerations to the structural type, dynamic properties, etc. In some cases, the damping ratio for the first mode is assumed to be 2% for steel structures, 3% for reinforced concrete or steel-encased concrete structure. Where the building is equipped with additional damping devices and/or components, the effects are also incorporated into the model.

8. Gravity load-resisting systems

It shall be confirmed that there will be no damage, or the induced stresses will be within long-term allowable stresses, in the elements necessary for structural resistance due to long-term loads such as dead load, imposed load, snow load in designated regions, earth pressure, load due to temperature change and loads due to shrinkage.

In most cases in high-rise buildings in Japan, the gravity load-resisting systems are integrated with horizontal ones and are designed integrally.

9. Non-structural Systems

The roofing materials, exterior materials and curtain walls facing outside (external cladding) shall be confirmed to be safe in terms of structural resistance with respect to wind pressure, earthquake or other vibration and impacts. As for earthquakes, the exterior cladding shall not be damaged by RE and shall not fall off due to ERE.

10. Project Example 1: Reinforced Concrete Residential Building with a Height of 100m[3)]

10.1. Structural configuration

First example is a high-rise reinforced concrete residential building as shown in Fig. 1. It is a new skeleton frame configuration (flexural- deformation control-type seismic-control structure using an oil damper) for absorbing energy and controlling flexural deformation with top-portion super beams and oil damper. This skeleton frame configuration enables realization of a skeleton infill (SI housing) within a super high-rise residence such that there are no columns or beams within the living space. As a further development of this skeleton frame configuration, the authors have developed a novel skeleton frame configuration for absorbing energy with high-toughness coupling beams that provide connections between core walls. As shown in Fig. 2, a super high-rise RC residence

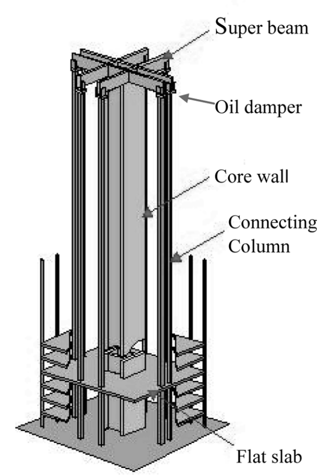


Figure 1. Super frame type seismic control structure.

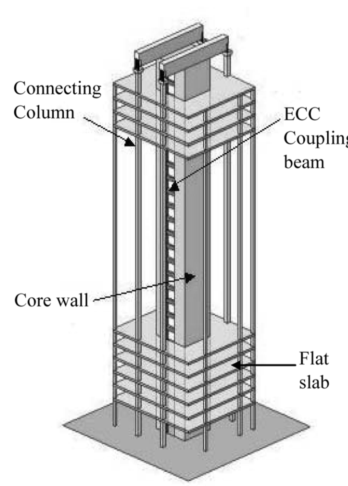


Figure 2. New combination type seismic control structure.

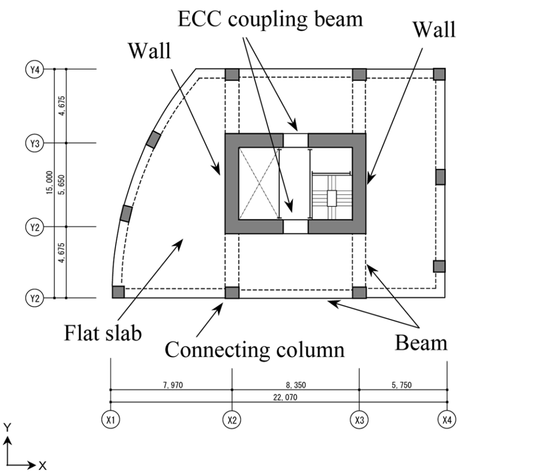


Figure 3. Typical floor plan.

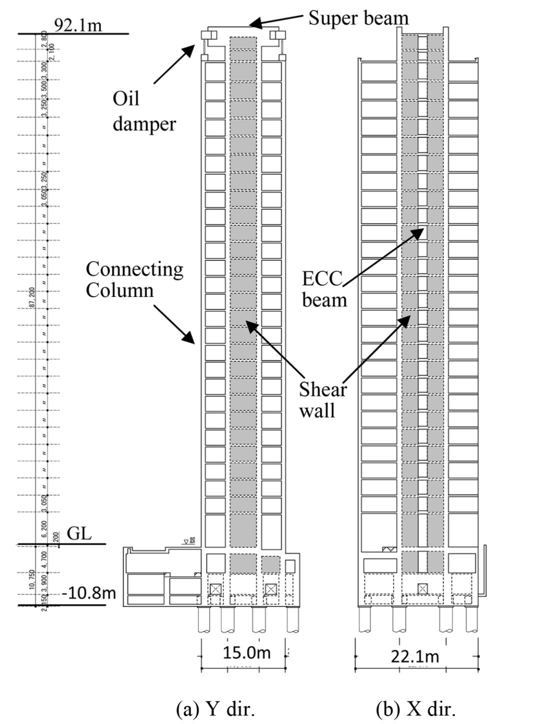


Figure 4. Section.

has been realized having high anti-seismic characteristics together with superior living comfort, through the combination of an oil damper type in one direction with coupling beam type parallel shear walls in the other direction. Here, for the high-toughness coupling beams, engineered cementitious composites (ECC) are employed, as these show superior tensile characteristics and, even when used for the short span, show sufficient energy absorption capabilities even for major deformations.

10.2. Structural configuration of the high-rise RC seismic-control building structure

The subject building is a 27-floor building, constructed in Tokyo, that is a 93 m tall RC high-rise residence whose standard floor plan is shown in Fig. 3. At the outer periphery, 10 columns are arranged, and in the center, there is arranged a symmetrical configuration comprised of C-shaped core walls set in opposition to each other; here, the seismic force from any direction is mostly concentrated at the center core walls.

Figure 4 shows the framing elevation for each direction. In the transverse (Y) direction, super beams are provided at the top portions of the core walls. An oil damper is provided between top portions of the outer periphery columns (connecting columns) and the apexes of the super beams; support of oil damper reactions is provided by a total of 4 connecting columns. The other columns are placed with the purpose of supporting the weight of each floor. In the longitudinal (X) direction, the parallel shear walls of the core are connected, at each floor, with ECC coupling beams. The ECC used here are materials having superior tensile capabilities compared with ordinary concrete, and are expected to have effects as dampers at times of deformation due to their large energy absorption capabilities.

10.3. Analytical model

In the seismic response analysis, the below-listed procedures were followed, and analysis was performed with the setting of a 3D elasto-plasticity vibration model (Fig. 5).

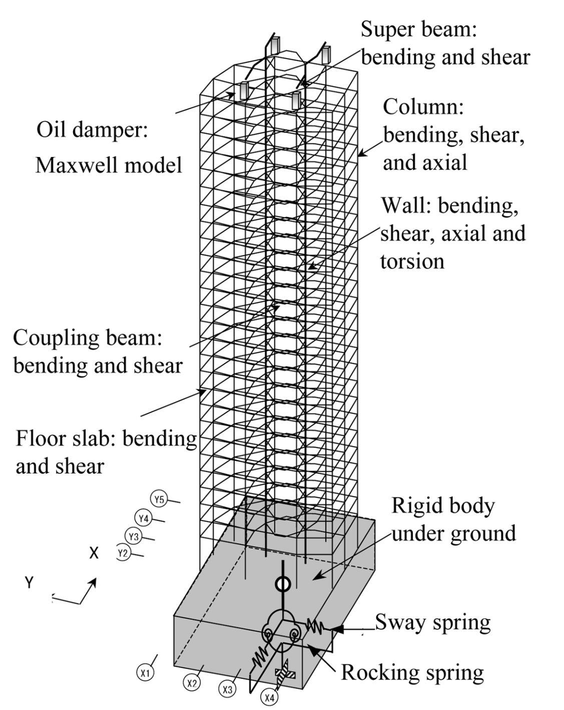


Figure 5. Analysis model.

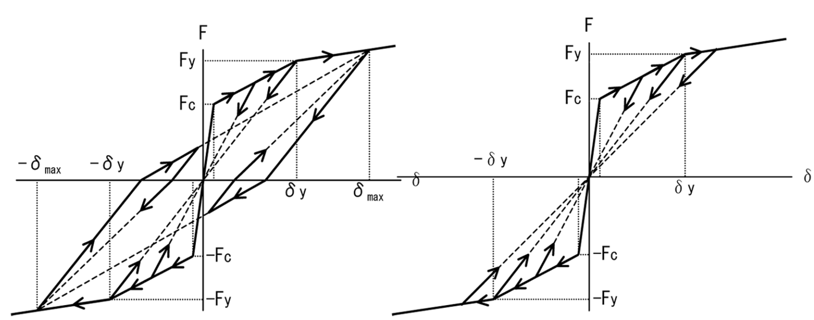
1) The materials (walls, columns, coupling beams, top beams, and slabs) comprising the building were modeled as beam elements.

2) For the respective components modeled as beam elements, the elasto-plasticity characteristics noted below were considered.
   • Walls: bending and shear elasto-plasticity (tri-linear type), axial elasto-plasticity
   • Columns: bending elasto-plasticity (Tri-Linear type), shear elasticity, and for the columns equipped with the seismic control apparatus, axial elasto-plasticity
   • Coupling beams, top beams, slab-equivalence beams: bending elasto-plasticity (tri-linear type), shear elasticity
3) In this configuration, a seismic control device (oil damper) is installed between the top beams and column heads of perimeter columns. Here, the seismic control device is taken as a Maxwell-type model of dashpots, connected in series, having an equivalence spring and a damping coefficient. The dashpots are given bi-linear type nonlinear characteristics.
4) For the tri-linear type skeleton curves regarding the bending and shear of the wall elements and the bending of columns and top beams, the degrading tri-linear type hysteresis rule of Fig. 6(a) (Muto loop) proposed by Dr. Muto was introduced. For the bending of slabs, an origin-oriented type hysteresis rule of Fig. 6(b) was introduced. Also, for the coupling beams, to enable simulation of test results, given was a hysteresis rule having half each of the respective characteristics of a Muto loop and an origin-oriented type loop.
5) Within each floor, walls and columns were connected with a rigid floor; further, top beams and the seismic control device were attached.
6) The underground portion was taken as one rigid body, and the effects of piles and ground soil were considered with sway springs in the X and Y direction, and rocking springs within each respective vertical surface, introduced at the bottom edge of the foundation beams.
7) For damping of the building structure, a Rayleigh-type damping system is supposed. As for the damping constant, at the time of elasticity when the bottom of columns at first floor is fixed, damping factors of 0.03 are set respectively at the overall primary natural period (Y-direction primary) and overall sixth natural period (Y-direction secondary). The damping matrix is set proportionally to the initial rigidity matrix.
8) The ground spring used was an equivalence ground constant obtained via response analysis of the ground, and evaluation was performed considering burial effects and group pile effects. The analysis model considers buried portions of the underground portion as a mass-less and rigid foundation, and this is a model with an added lateral dynamic spring at the buried portion side walls and a group pile dynamic spring at the bottom end of the foundation.

10.4. Seismic response analysis results

10.4.1. Input earthquakes and design criteria

Input earthquakes used in the seismic response analysis



(a) Muto loop (b) Origin-oriented loop

Figure 6. Hysteresis loop.

Table 3. Input earthquake for design

		Rare Earthquakes (RE)		Extremely Rare Earthquakes (ERE)	
		Vmax (mm/s)	Amax (mm/s^2)	Vmax (mm/s)	Amax (mm/s^2)
Standard motions	El Centro 1940 NS	250	-	500	-
	Taft 1952 EW	250	-	500	-
	Sndai038 1978 EW	250	-	500	-
Notification motions*	Building code motion A	-	640	-	3,200
	Building code motion B	-	640	-	3,200
	Building code motion C	-	640	-	3,200

*Notification motions are defined at the engineering bedrock.

Table 4. Design criteria

	Rare Earthquakes (RE)	Extremely Rare Earthquakes (ERE)
Story drift angle	1/200 rad.	1/100 rad.
Ductility factor of structural member	Allowable stress for short-term loading, or less	4.0 or less
Margin of core wall to ultimate strength	-	1.5
Rotation angle of edge of coupling beams	-	1/30 rad.
Response maximum speed of control devices	-	400 mm/s
Response maximum stroke of control devices	-	100 mm

Table 5. Natural period

Direction	X			Y		
Mode	1st	2nd	3rd	1st	2nd	3rd
Natural period (s)	2.34	0.60	0.28	2.50	0.52	0.22

were of 2 levels: the Rare Earthquakes (RE), and the Extremely Rare Earthquakes (ERE); for each of these levels, standard motions that are actual waveforms of recorded ground motions in the past (recorded motions) and notification motions defined in Japan building code were used as summarized in Table 3.

Design criteria for RE and ERE input earthquakes are shown in Table 4.

10.4.2. Analysis results

The natural periods are shown in Table 5. Also, representative examples of response analysis results are shown in Fig. 7.

In regards to the response maximum inter-story drift angles for ERE seismic ground motion, at building code motions (RINKAI-92 phase) for both directions alike, at maximum, these were 1/113 in X direction, and 1/106 in Y direction; these were thus less than the criteria of inter-story drift angle of 1/100. It was also confirmed that, for flexure and shear responses of shear walls, respective ultimate strengths had a margin of 1.5 or higher. The maximum responses of rotation angle of edge portion of coupling beams were 1/41; it was thus confirmed that these were within the criteria of 1/30. The responses of the seismic control devices were confirmed, at a response maximum speed of 249 mm/sec, to be less than the tolerance value of 400 mm/sec, and the response maximum stroke was 43 mm, and thus less than the tolerance value of 100 mm.

Figure 8 shows the energy absorption characteristics of each portion at the time of an ERE earthquake. In Y direction, the oil dampers bear 38.5%, and in X direction, the ECC coupling beams bear 32.3%; it was thus understood that both of these make major contributions to reduction of building responses.

10.4.3. Dynamical investigation of piles

We performed seismic response analysis of piles with input motion of ERE-Notification motions, and investigated the anti-seismic safety characteristics of the pile bodies. In the analysis, as the pile stress at the time of an earthquake, consideration was made of pile stress due to inertia force from the upper portion of the structure, and pile stress due to ground vibration. The analysis model is shown in Fig. 9.

The analysis results confirmed that the pile flexural response values were within the ultimate flexural strengths, and that the pile shear response values were within the

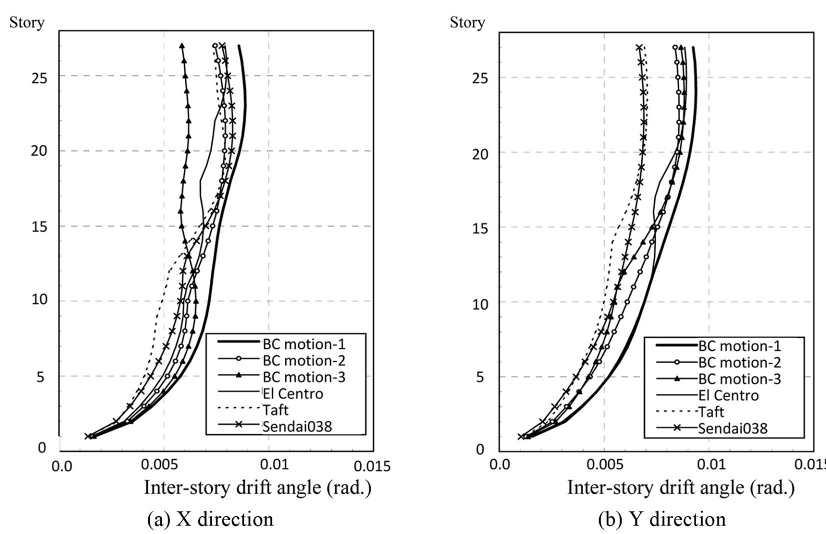
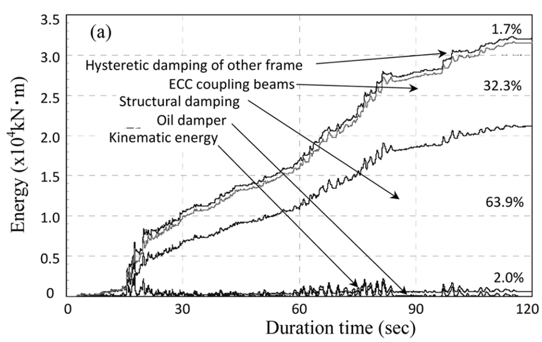


Figure 7. Maximum inter-story drift angle, (a) X direction, (b) Y direction.

ultimate shear strengths.

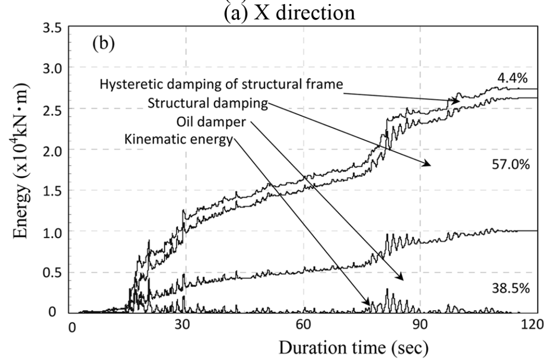
10.5. Conclusion on the performance-based seismic design

In designing a 100-meter high R/C residential building, various studies and designing efforts were conducted in order to ensure sufficient safety against earthquakes as follows: (1) developing a new skeleton frame configuration for absorbing energy and controlling deformation with oil dampers; (2) developing and employing the engineered cementitious composites (ECC) material for high-toughness coupling short-span beams, that show sufficient energy absorption capabilities even for major deformations; (3) performing seismic response analysis with a 3D elasto-plasticity vibration model considering pile and soil interaction; (4) setting the design criteria against very severe earthquakes considering enough robustness; (5) considering the pile stress due to inertia force from the upper portion of the structure and pile stress due to ground vibration in pile design.



(a) X direction

(a) X direction



(b) Y direction

Figure 8. Energy share ratio of each element, (a) X direction, (b) Y direction.

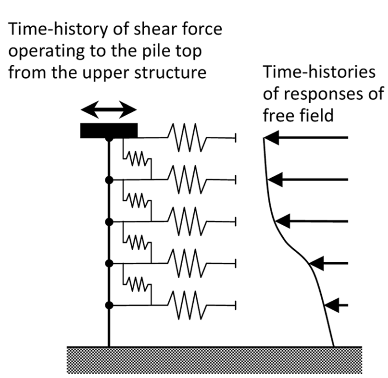


Figure 9. Soil-pile analysis model.

11. Project Example 2: Abeno Harukas, the Highest Building In Japan [4)]

11.1. Outline of building and structure

The "Abeno Harukas" is to be the first building that reaches as high as 300 m in the seismic-prone country, Japan. The building is currently under construction, and scheduled to open in 2014. Situated in Abeno, Osaka, the

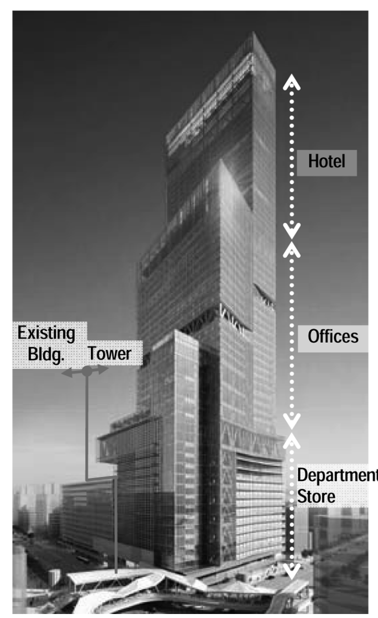


Figure 10. Northwest view rendering.

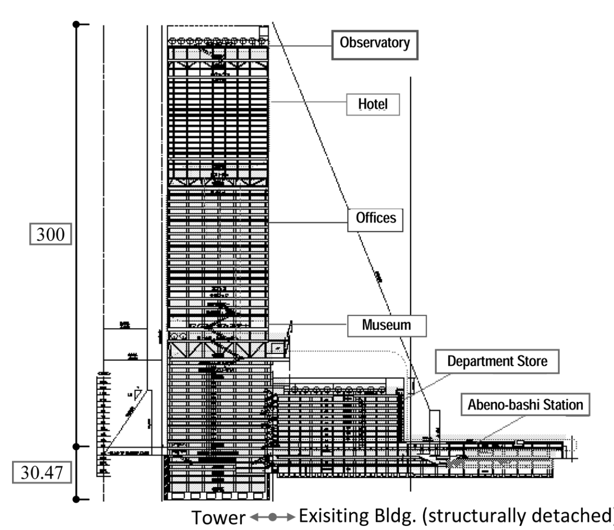


Figure 11. East-west sectional view.

Table 6. Input earthquake motions

		Rare Earthquakes (RE)		Extremely Rare Earthquakes (ERE)		Presumed Maximum Earthquakes (PME)	
		Vmax (mm/s)	Amax (mm/s^2)	Vmax (mm/s)	Amax (mm/s^2)	Vmax (mm/s)	Amax (mm/s^2)
Standard motions	Elcentro 1940 NS	(1) 250	2,555	500	5,110	-	-
	TAFT 1952 FW	250	2,485	500	4,970	-	-
	Hachinohe 1968 NS	250	1,669	500	3,338	-	-
Notification motions	Notification motion A (N.M.-A)	76	552	379	2,517	(2) 528	3,244
	Notification motion B (N.M.-B)	109	471	613	1,929	817	2,919
	Notification motion C (N.M.-C)	83	497	383	2,098	560	2,604
Regional motions	Nankai earthquake NS	(3) -	-	210	863	-	-
	Nankai earthquake EW	-	-	275	987	-	-
	Tonankai/Nankai erathquake NS	-	-	-	-	268	1,216
	Tonankai/Nankai erathquake EW	-	-	-	-	419	1,067
	Uemachi Fault Zone (UFZ) Case 1 NS	-	-	-	-	329	3,485
	UFZ Case 1 EW	-	-	-	-	813	4,107
	UFZ Case 2 NS	-	-	-	-	291	2,317
	UFZ Case 2 EW	-	-	-	-	456	3,269

(1), (2) and (3) means the category.

building will accommodate 60 stories above ground and 5 basement floors, and will become a new landmark. See Figs. 10 and 11 for the external view and the east-west section, respectively.

The superstructure is composed of three "blocks" having setbacks on the north side. The lower block is for the Kintetsu Department Store, the middle one for offices and the upper one for a hotel. The upper block has a large atrium in the center. Located between the blocks and at the top of the upper one are the transfer-truss floors. Outrigger trusses are placed in the transfer-truss floors and some other locations. Most of the superstructure is moment resistant steel frames. Concrete-filled steel tubes (CFT) with high strength steel (tensile strength: 590 N/mm^2) and concrete (cylinder compressive strength: 150 N/mm^2) are used for the columns. A total of four types of dampers, both viscous ones and hysteresis ones, are placed mainly at the four corners in the lower block, around the central core in the middle block and around the atrium in the upper block in order to absorb energies input by earthquakes or wind. The substructure is a steel-encased reinforced concrete construction. The structures are supported by piled raft foundation.

11.2. Input earthquake motions

Three kinds of input earthquake motions are used, that are, the Standard Motions based on recorded earthquakes, the Notification Motions generated from given acceleration spectra and the Regional Motions simulated with consideration to the seismicity around the site. The levels of earthquakes are categorized as the Rare Earthquakes (RE), the Extremely Rare Earthquakes (ERE) and "the Presumed Maximum" Earthquakes. (PME) The PME, which are not in the category defined by the Law, have been generated by factoring by 1.5 the ERE-Notification Motions. See Table 6 for the maximum velocities and accelerations of all the earthquakes used. Here, the earthquakes in box (1) are used to check the conformity to the legal requirements. The ones in box (2) indicate the earthquakes for grasping the margins of seismic safety. The ones in box (3) are the Regional Motions. The velocity response spectra are shown in Figs. 12 and 13 together with natural periods of the building described in the following section.

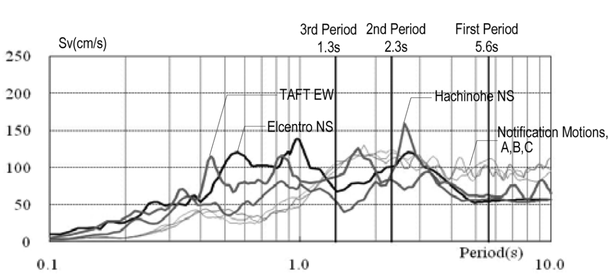


Figure 12. Velocity response spectra (Extremely Rare Earthquakes, damping ratio = 0.05).

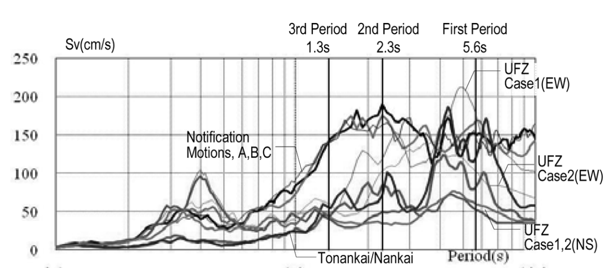


Figure 13. Velocity response spectra (Regional waves, damping ratio = 0.05).

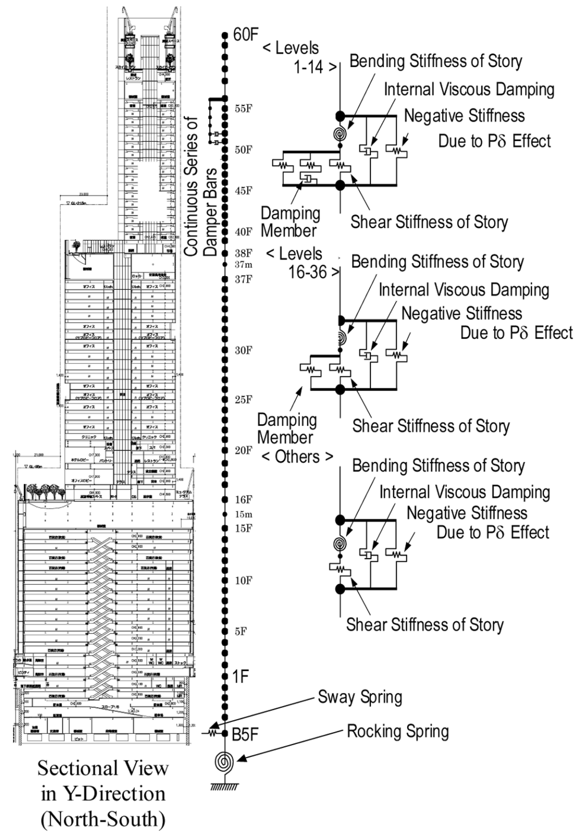


Figure 14. View of analysis model.

11.3. Response analysis model and eigenvalue analysis

The whole structural elements are contracted to a lumped mass model with 68 masses. See Fig. 14. The stiffness of each story is represented by equivalent bending and shear separated springs. Here, the bending component is elastic and the shear component is elasto-plastic. The P-Δ effect is considered through springs with negative stiffness. Additional damping devices are modeled on a story-by-story basis. Overall structural damping is also considered through the Rayleigh's method. Here the damping ratio is set as 2% for each of the first and the second natural modes. Ground sway and rocking springs are set at the bottom of basement. The damping of the ground is considered only in the rocking vibration. Earthquake motions are to be input at the basement 5th floor.

The natural periods of the building obtained by the eigenvalue analysis are shown in Table 7.

11.4. Seismic design criteria – target seismic performance

Through the dialogues with the client on the desired performance of the building during and after earthquakes, it has been determined as the design policy that the building shall generally have one-grade higher performance than usual high-rise buildings. This section introduces the adopted criteria, or the target performance, for the overall building as well as individual members.

11.4.1. Design criteria for overall building

See Table 8 for the criteria/target adopted for the overall building. The building is intended to remain "repairable" even after ERE, while the corresponding legal requirement is to avoid collapse. Further safety is pursued by mostly applying the usual criteria at ERE to the responses to PME.

11.4.2. Design criteria for individual members

The design criteria for individual members are shown in Table 9. Here, the forces and stresses are calculated by pushover analysis using a full 3-D model, subjected to the maximum story shears observed in the dynamic response

Table 7. Natural periods of the building (in seconds)

Direction	1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th
X	5.64	2.31	1.38	0.92	0.76	0.67	0.55	0.47	0.43	0.37
Y	2.77	2.39	1.31	0.85	0.69	0.60	0.49	0.45	0.40	0.38

Table 8. Design criteria/target for overall behavior

Level of earthquake		Rare Earthquakes (RE)	Extremely Rare Earthquakes (ERE)	Presumed Maximum Earthquakes (PME)
Approximate return period		50 years	500 years	(1.5 times stronger than the Extremely Rare, Notification Motions)
Target building performance		Serviceable	Repairable	Repairable/reinforceable
Super-Structure	Story drift angle	1/200	1/100	1/75
	Ductility factor of story	(Short-term allowable stress)	1.0	2.0
Substructure	Underground frames/ foundation	Short-term allowable stress	Ultimate strength	Underground frame: Ductility factor of members < 4.0 Foundation/piles: Ultimate strength
	Bearing capacity of piles	Short-term allowable bearing capacity	Short-term allowable bearing capacity	Ultimate bearing capacity

Table 9. Design criteria for individual members

Area	Rare Earthquakes (RE)	Extremely Rare Earthquakes (ERE)	Presumed Maximum Earthquakes (PME)
	Functions maintained. Hardly damaged. Repair hardly required.	Major functions secured. Minor damaged. Minor repair required.	Specified functions secured. Moderately damaged. Limit of repair.
	Short-term allowable stress	Elastic limit or less than ultimate strength.	Less than ultimate strength or plastic hinges allowed.
Column	Short-term allowable stress	Less than ultimate strength	Less than ultimate strength (except some)
Girder	Short-term allowable stress	Less than ultimate strength	Plastic hinges allowed.
Brace	Short-term allowable stress	Less than ultimate strength	Plastic hinges allowed.
Truss	Short-term allowable stress	Less than elastic limit	Less than ultimate strength
Steel plate wall	No shear yielding	Shear yielding allowed	Shear yielding allowed
Friction damper	Sliding rotation allowed	Sliding rotation allowed	Sliding rotation allowed
Earthquake resisting wall	Short-term allowable stress	Less than ultimate strength	Less than ultimate strength

analysis using the lumped-mass model. In the columns or trusses, plastic hinges are not allowed even under PME. For the beams and braces, plastic hinges are allowed under PME but not under the ERE.

11.5. Results of response analysis

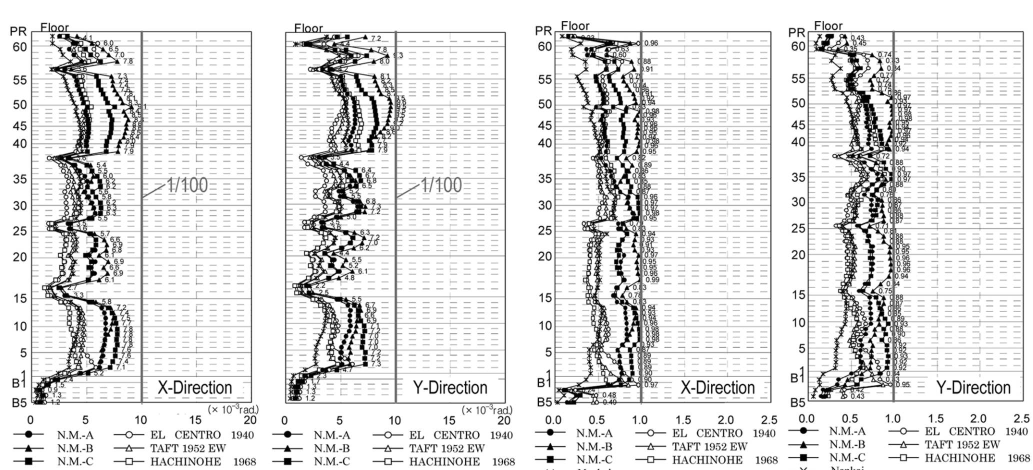
Figure 15 shows the results of response analyses under ERE in each of X- and Y- directions. It is seen that the maximum story drift angles are well below the criteria of 1/100, while the maximum ductility factors of each stories are just below the criteria of 1.0 for a number of stories, implying that the ductility factors are in general the dominant design conditions. Note that the criteria here, or the target performance, are generally one-rank higher than usual high-rise buildings.

Figure 16 shows the results of response analyses under PME generated by factoring ERE – Notification Motions. The responses are well below the criteria. Figs. 17 and 18 show the results under some of the Regional Motions. The responses are also well below the criteria.

Besides the verifications above, a number of additional studies have been conducted to confirm the robustness of the adopted design. The studies include the response analyses under the following earthquake motions: (1) Uemachi Fault Zone (UFZ) Motions evaluated at different locations along the fault line; (2) UFZ Motions, originally having the predominant period close to the 1st natural period of the building, manipulated to match the 2nd or the 3rd natural period; (3) Tonankai/Nankai Motions manipulated to match the 1st natural period of the building. The studies also include the response analyses using the analysis model (4) with/without ground springs; (5) with reduced (by half) properties of damping devices. The variations of the responses were found to be minor or reasonable, implying that the design is not oversensitive to the above parameters regarding the earthquake input or the analysis model.

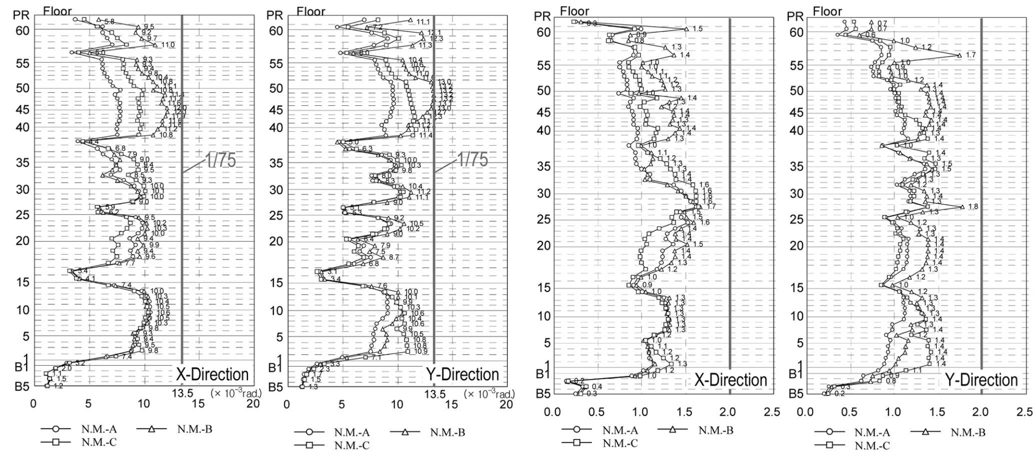
11.6. Conclusion on the performance-based seismic design

In this section, the performance-based seismic design of the first 300 m high building in Japan was introduced.



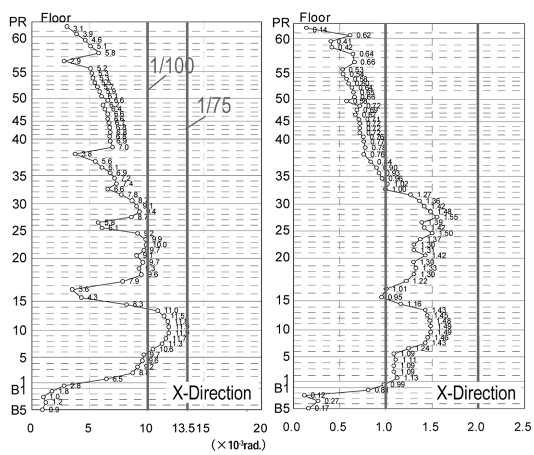
(a) Max. story drift angle (b) Max. ductility factor

Figure 15. Results of response analysis under the Extremely Rare Earthquakes.



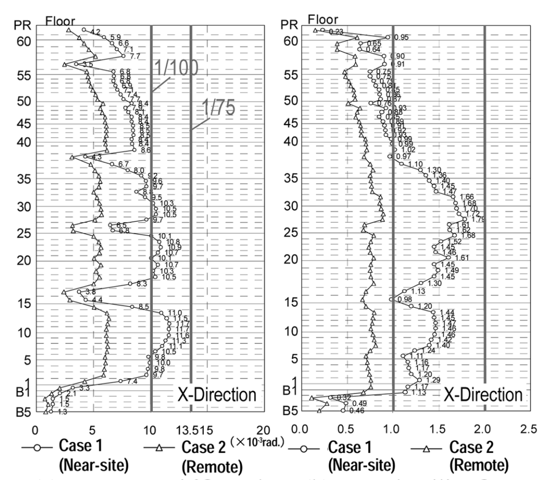
(a) Max. story drift angle (b) Max. ductility factor

Figure 16. Results of response analysis under the Presumed Maximum Earthquakes generated by factoring the Extremely Rare, Notification Motions.



(a)Max. story drift angle (b)Max. ductility factor

Figure 17. Results of response analysis under Tonankai/ Nankai Earthquake.



(a)Max. story drift angle (b)Max. ductility factor

Figure 18. Results of response analysis under UFZ Earthquake.

To ensure higher levels of safety, the seismic design criteria were generally set as one-rank higher ones than usual high-rise buildings or the minimum legal requirement. The studies included the response analyses to the earthquakes 1.5 times larger than the legally defined Extremely Rare Earthquakes as well as the earthquakes simulated with considerations to the seismicity around the building site. In addition, the robustness of the design was studied by checking the sensitivities to the variation of parameters of the input earthquakes or the analysis model.

12. Other Items of Interest

In this paper, the outlines of review and approval processes, general criteria and usual practices taken in Japan for seismic design of high-rise buildings were introduced. The outlines of seismic design of two high-rise buildings were also introduced.

While the above are the typical procedure and the latest examples of seismic design in Japan, it should be noted that there have recently been intensive research and discussions on how seismic design for high-rise buildings should be, in response to "The 2011 Off the Pacific Coast of Tohoku Earthquake (Tohoku Earthquake)", which struck Japan on March 11 2011.

One example would be the growing awareness of long-period ground motions. In the Tohoku Earthquake, some high-rise buildings in Tokyo, around 200 km from the epicenter, continued to vibrate for nearly 10 minutes. Even a high-rise building in Osaka, 800 km from the epicenter, suffered significant long-period vibrations.

Experts point out that the probability of major earthquakes, whether off-coast plate borderline type or directly above the epicenter, is still high or has become even higher. While high-rise buildings to the current seismic codes are expected not to be severely damaged under the major earthquakes with somewhat shorter predominant period, there is a concern on those having longer predominant period and lasting a longer duration of time due in part to the characteristics of subsurface ground. The government is currently in a direction to set forth new requirements on the seismic design of high-rise buildings regarding the effect of such ground motions.

Natural disasters, especially major earthquakes, often bring us renewed awareness of the power of nature as well as newly realized issues to resolve. Structural engineers should have keen awareness of such latest information and knowledge, and incorporate them into the actual performance-based design of high-rise buildings.

13. Review Procedures

As stated before, structural calculations for buildings higher than 60 m, based on time-history response analyses, are required to go through performance evaluation by organizations designated by MLIT. The performance evaluation by the designated organizations is generally conducted in a form of committees held as required, where the structural engineer explains the structural design for the building based on the drawings and calculations, and answers to the queries raised by the reviewers. The answers often include additional calculations and/or testing. Most of the reviewers are academic experts in the relevant fields.

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