16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 3830

Registration Code: S-D1464418666

Structural Design of Seismic Isolation System Incorporating RC Core Walls and Precast Concrete Perimeter Frames

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Abstract

This paper presents the seismic isolation system incorporating high strength reinforced concrete (RC) core walls and precast concrete perimeter frames. As the seismic isolation devices, 42 seismic isolation rubber bearings and 10 oil dampers are installed on the isolation floor. The structural robustness is reflected into the seismic criteria which can be achieved by the tube structure combined seismic isolation system with notably rigid superstructure utilizing RC core walls.

Seismic criteria are categorized into two levels of earthquakes: Level 1 earthquakes and Level 2 earthquakes. Level 1 earthquakes are denoted as rarely occurred earthquakes with 50-year return period, which can occur several times in building service period. Level 2 earthquakes, on the other hand, are denoted as extremely rarely occurred earthquakes with 500-year return period, which can occur one time in building service period. To guarantee the high seismic resistant performances, all member stresses are supposed to be less than or equals to short-term allowable stresses and story drift is limited to less than 1/300 in the event of Level 2 earthquakes.

Seismic performances of the building are evaluated based on nonlinear time history analyses using Multi-Degree-of-Freedom (MDF) model with bending and shear springs in each story and sway-rocking springs on the isolation floor. The 3D frame model for nonlinear static pushover analyses is composed simultaneously to evaluate the validity of analyses results based upon MDF model.

Keywords: Seismic isolation system, Reinforced concrete core wall, Precast concrete perimeter frame

1. Introduction

Shimizu corporation Tokyo headquarter (Fig 1.) aims at providing highly anticipated office, which can contribute largely to sustainable societies by achieving both environmentally friendly, comfortable office and disaster control center. This office building is expected annual CO_2 emmission by 62% in the first year of its operation through various advanced technologies which include precast concrete parimeter frames incorporated with solar panels and radiant air conditioning system. This building achieved 70% reduction of CO_2 emmission by 2015 and finally aims to establish zero-carbon status in combination with carbon credits. In terms of disaster management facility, this building has enough functions and stocks to protect societies in time of disasters and act as a disaster control center to support people who cannot return home safely due to lacks of transportations and other infrastructures.

In order to achieve those purposes, seismic isolation system which is incorporated with reinforced concrete core walls and precast concrete perimeter frames is adopted as key lateral force resistant systems.

Fig 1. Overview

2. Structural Systems

2.1 Superstructure

The superstructure consists of a tube system optimized by both reinforced concrete core walls and precast concrete perimeter frames. (Fig 2. and Fig 3.) Reinforced concrete core walls are located at the center of each floor from B1 to 21th floor surrounding elevators, stairs and pipe shafts. The maximum thickness of the core walls is 700mm and the concrete strength is from 40 N/mm^2 to 60N/mm^2. Reinforced concrete core walls act as the "Central Pillar" to sustain up to 80% of the total shear forces of this structure. Precast concrete perimeter frames, on the other hand, act as both structural components and claddings. Precast concrete frames are normalized in the size of 3.2m x 4.2m, which windows, solar panels, gusskets and precast concrete girders and columns are all integrated into panel elements. Precast concrete framens are covered by aluminium casting which improve durability against weather and aging. Member sizes consisting perimeter frames and materials are determinded in terms of material qualities and construction workabilities. Concrete strength is from 48 N/mm^2 to 80N/mm^2, and rebar sizes are from D19 to D35 all in SD490 ($fy=490N/mm^2$). As lateral forces are basically loaded by core walls located at core zone and perimeter frames, building users can arrange office layouts freely without considering any columns and walls. (Fig 4. and Fig 5.)

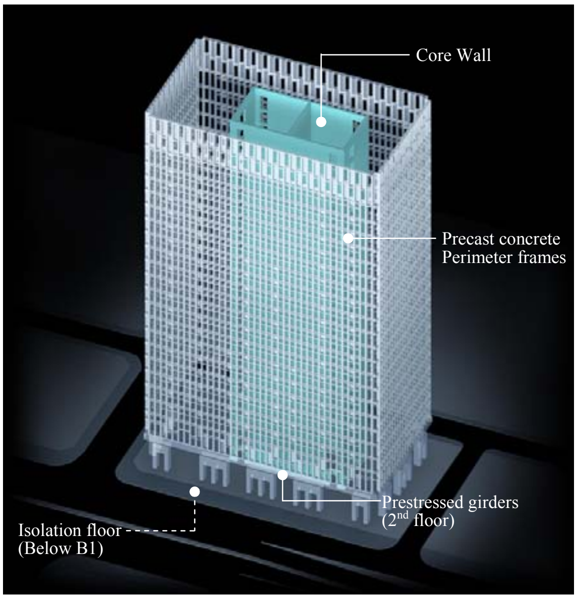


Fig 2. Structural Perspective

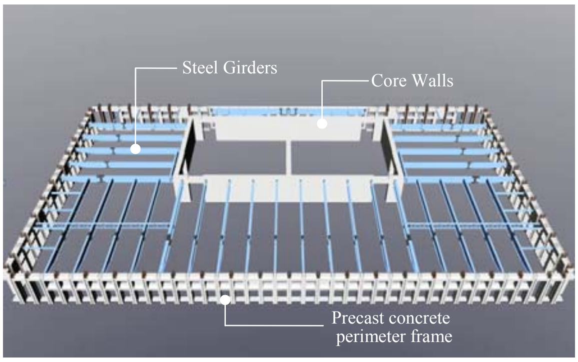


Fig 3. Frames on typical floor

Perimeter frames and reinforced concrete core walls are connected through slabs with the thickness of 150mm. Slabs are composed of metal decks with truss rebars in consideration with construction workabilities. Girders supporting composite slabs are all pin-connected to columns except for girders on the 2nd floor, which are prestressed to have the capacities to transfer vertical loads of perimeter columns into integrated columns on the 1st floor. Additionally the slabs on the 2nd floor are precast concrete slabs and ribs with the depth of 1170mm.

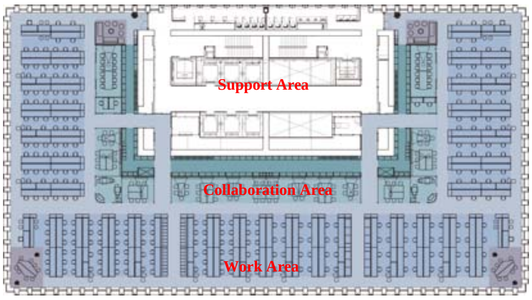


Fig 4. Office Layout

Fig 5. Office View

2.2 Seismic Isolation System

42 seismic isolators and 10 oil dampers are located on the isolation floor between B2 and B1 floor. (Fig 6.) Rubber bearings consists of 32 LRB (Lead Rubber Bearing) and 10 NRB (Natural Rubber Bearing). Most rubber bearings are installed at the same location as the column above, which support the axial load of one column. At the corner of core walls, on the other hand, two rubber bearings are installed to deal with high cyclic loadings in the event or large earthquakes. The average long-term compressive strength of rubber bearing is 13.8N/mm^2 which is below recommended axial capacity of rubber bearings. Providing that rubber bearings deform laterally by 400% in shear strain, the natural period of this structure is approximately 5.4sec. Oil dampers are simulated to absorb seismic energy effectively also in the event of earthquakes with the characteristics of long term period. The design clear length of seismic isolation floor is 600mm.

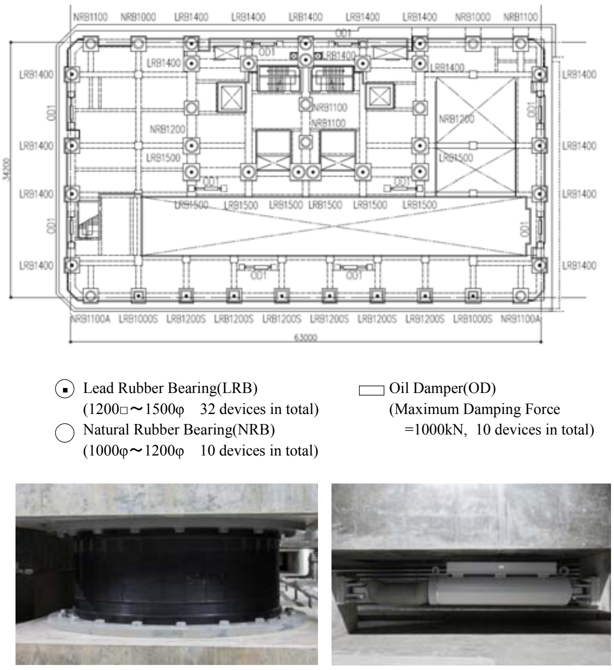


Fig 6. Location of Isolation Devices

2.3 Basement and Foundation

The basement structure contains moment frames with reinforced concrete shear walls. The columns on the B3 and B2 floors are precast concrete members and were installed soon after completing reinforced concrete mat slab foundation. The construction work of girders on the isolation floor follows the installments of precast concrete columns, and, as a result, optimize the construction schedule by constructing the structure above the isolation floor and the structure on the B3 and B2 floor simultaneously. The size of precast concrete columns are minimized by adopting high strength concrete with the strength of 80N/mm^2. Retaining walls surrounding the basement from the level of isolation floor to the level of the ground are incorporated with soil cement pillar walls and thus make it possible to minimize the thickness of the retaining walls and construct in the narrow spaces between the building and the property line.

The foundation consists of spread foundation of reinforced concrete mat slab. The level of foundation bottom edge matches the upper layer level of silty sand which secures the enough stiffness with the velocity of 400m/s in secondary waves. The velocity of the soil was measured in soil investigation. The long-term allowable stress of the soil is 800N/mm^2 based upon the plate bearing test on the soil. The maximum thickness of the reinforced concrete mat slab is 3250mm.

3. Seismic Criteria

3.1 Seismic Criteria and Seismic Input

Table 1. shows the seismic criteria of the structural responses for rarely occurred earthquakes (Level 1) and extremely rarely occurred earthquakes (Level 2) respectively. To guarantee the high seismic resistant performances as a disaster management facility, building functions are expected to be maintained after Level 2 earthquakes. To take into consideration the stable performances after Level 2 earthquakes, it is also determined not only to limit all member stresses less than or equals to short-term allowable stresses for Level 2 earthquakes, but limit the maximum response story drift to 1/500 for Level 1 and 1/300 for Level 2 earthquakes respectively. Furthermore any damages on claddings are intended not to be occurred.

As seismic imputs for design, 3 seismic waves (Kokuji waves) stipulated in Building Standard Law in Japan, 3 seismic waves based upon the observed earthquakes in the past (El Centro 1940, Taft 1962, Hachinohe 1968) and 4 seismic waves reproduced based upon earth fault models were adopted. Sesmic waves stipulated in Building Standard Law include earthquakes with the characteristics of far-field phases (Kanto Earthquake East-West Direction in 1923 reproduced by Japan Meteorogical Agency), earthquakes with the characteristics of near-field phase (JMA Kobe earthquakes North-South Direction in 1995) and seismic waves with the characteristics of random phase.

Table 1. Seismic Criteria

Contents	Seismic Inputs	
	Level 1	Level 2
Definition	・Seismic Inputs which can be occurred several times in building service period	・Seismic Inputs which can be occurred one time in building service period
	・Return period of 50 years	・Return period of 500 years
Seismic Waves	・3 waves defined as rarely occurred earthequakes which are stipulated in Building Standard Law in Japan (Kokuji Wave)	・3 waves defined as extremely rarely occurred earthequakes which are stipulated in Building Standard Law in Japan (Kokuji Wave)
	・3 waves based upon the observed seismic waves and are standarlized in the response velocity sperctum of 0.25m/s (El Centro 1940, Taft 1962, Hachinohe 1968)	・3 waves based upon the observed seismic waves and are standarlized in the response velocity sperctum of 0.50m/s (El Centro 1940, Taft 1962, Hachinohe 1968)
		・4 site specific waves reproduced based upon earth fault models
Buiding Condition	Maitaining building function	Maitaining building function
Maximum Resposnse Story Drift	1/500	1/300
Member Stress	Less than or equal to Short-Term Allowable Stress	Less than or equal to Short-Term Allowable Stress
Isolation Devices	Stable lateral deformation	Stable lateral deformation
	No tensile stress	Tensile stress is less than or equals to $1N/mm^2$

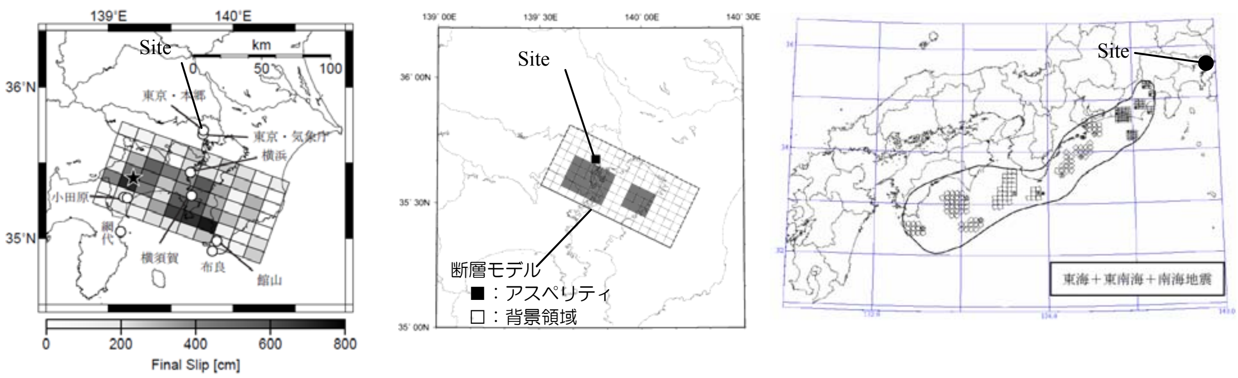
Seismic waves reproduced based upon earth fault models are considered as site specific ground motions which should be calculated to be adopted in design.

1. Reproduced seismic waves based upon Kanto Earthquake in 1923 (Magnitute 7.9) :

   The seismic wave is produced through the calculation combining 3 dimension finite difference method and broad-band hybrid method by statistical Green's function.

2. Seismic waves on plate boundary region in the northern part of Tokyo Bay (Magnitute 7.3) :

   The seismic wave is produced by combining statistical Green's function method and earth failure model produced by Central Disaster Management Council. (2004)



1. Kanto Earthquake 1923   2. Plate boundary earthquake In the northern part of Toyo Bay   3. Tokai, to Nankai and Nankai earthquake

Fig 7. Earth failure models considerd in design

3. Continuous earthquakes considering the series of Tokai, To nankai and Nankai earthquakes (magnitude 8.7) :

   The seismic waves is produced by combining statistical Green's function method and earth failure model by Central Disaster Management Council. (2003)

4. Crustal seismic waves occurred in the region of unspecified earth failures (Magnitute 6.9) :

   The seismic wave is produced assuming that ground motions are occurred randomly in the region of earth failures near the construction site.

Seismic waves stated above are considered as the ground motions which are equivalent to Level 2 earthquakes in seismic intensity. The building performances are examined to meet the seismic criteria for these site specific ground motions.

In addition to these examination, seismic performances of the building are investigated for large-scaled earthquake. The seismic intensity of the large-scaled earthquake is 1.5 times as large as that of Level 2 earthquakes. This seismic wave is adopted in design to clarify the additional robustness of the building in the event of unpredictably large earthquakes. It is determined that the maximum response story drift for this earthquake should be less than 1/200 and the lateral deformation of isolation floor should be less than the clear length of the isolation floor. (600mm) The pseudo velocity response spectrum of seismic waves adopted in design are plotted in Fig 8.

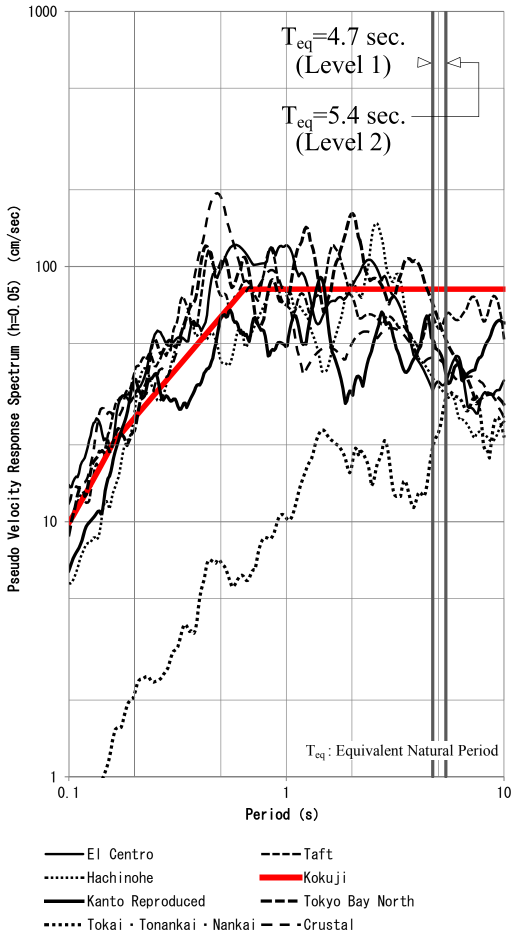


Fig 7. Site Specific Seismic Waves

3.2 Nonlinear Time History Analysis

Multi-degree-of-freedom (MDF) model with 27 masses and stories modeling from B3 floor to roof floor is adopted to carry out nonlinear time history analysis. MDF model includes bending and shear springs in each story and sway-rocking spring considering seismic isolation devices on the isolation floor. Support constraint is modeled as fixed through the analysis. Internal damping of MDF model is expressed as the function that is in proportion to the initial stiffness of the structure. Damping coefficients are 2% for both X and Y directions respectively.

The 3D frame model for nonlinear static push over analyses and the time history analyses is composed simultaneously to evaluate the analysis results of MDF model and investigate torsional behaviors of the structure and structural response for diagonally introduced seismic imputs. (Fig 9.) Although the superstructure originally shows in-plane eccentricity due to the location of reinforced concrete core walls, in-plane eccentricity on the isolation floor is dismissed through the method of arranging the postion of seismic isolation devices. As a result, the primary vibration modes of

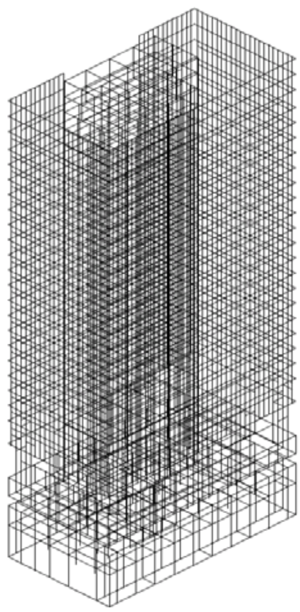


Fig 9. 3D model for nonlinear analysis

Table 2. Modal analysis results

	1st	2nd	3rd	note
Fixed	2.05sec	1.41sec	—	Isolation floor to be fixed.
Level1	4.84sec	4.68sec	4.57sec	Deformation of isolators to be 200mm.
Level2	5.36sec	5.22sec	5.08sec	Deformation of isolators to be 400mm.
	(Y-dir)	(X-dir)	(Torsion)	

the structure are parallel modes in both X and Y direction (Table 2.). The natural periods in the first order are 5.2sec for X direction and 5.4sec for Y direction respectively in the event of Level 2 earthquakes, 4.7sec for X direction and 4.8sec for Y direction respectively in the event of Level 1 earthquakes. The natural period in the first order assuming that the isolation floor is infinitely stiff inlateral directions are 1.4sec for X direction and 2.1sec for Y direction respectively. This means that high amount of lateral stiffness is added in proportion to reinforced concrete core walls.

Fig 10. shows the analysis results of nonlinear time history analysis for level 2 earthquakes. The maximum response accerelation is less than 200gal even in the case that quality dispersion of isolation devices are considered. The phenomenon that response accelerations of the superstructure are mostly the same in each floor clarify that seismic behaviors of the superstructure are almost the same as those of single-degree-of-freedum (SDF) model, which the structure deform rigidly in the lateral direction due to the tube system incorporated with reinforced concrete core walls and perimeter frames. The maximum lateral deformation of the isolation floor is 378mm and the story shear coefficient of B1 floor above the isolation floor is 0.066. The design shear coefficient of B1 floor is determined to 0.075. The maximum response story drifts are 1/828 for Level 1 earthquakes and 1/448 for Level 2 earthquakes respectively. Both values are below the seismic criteria shown in Table 1.

In regard to structural safety in terms of the recent researches and investigations on seismology, the seismic waves which has the characteristics of long-period ground motions are produced. The seismic wave which has twice as much intensity as the site specific ground motion based upon Kanto Earthquake in 1923 (1. in section 3.1) was adopted in design.

One of the characteristics which long-period earthquakes can cause to LRB is the deterioration of shear yield stiffnesses of lead material due to continuous cyclic loading. To take this phenomenon into consideration, the shear yield strength of LRB is assumed to be 50% of the maiximum capacity in analysis. The capacities of oil dampers are determined to meet the seismic criteria that the lateral deformation of the isolation floor is less than 600mm under the analysis condition stated above.

In the event of Level 2 earthquakes, 75% of the total absorbed energy of the isolation floor is attained by plastic deformation of lead material, and 20% of the absorbed energy is by the damping characteristics of oil dampers (Fig 11.).

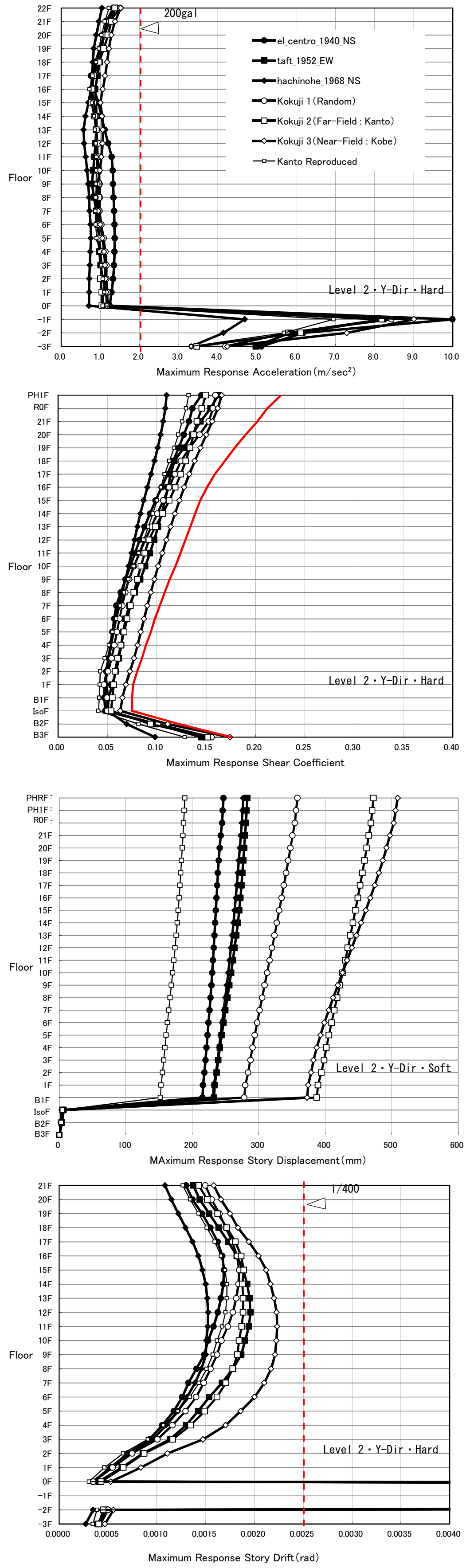


Fig 10. Analysis results of nonlinear time history analysis

As a pioneering technology, “Shimizu Original Self Monitoring System” is introduced in the building. This technology makes it possible to tansmit the information on the instant evaluation of structural damages to building users. Structural damages which are monitored through the devices attached to the structure are assembled through the network and show the results instantly on the monitor of disaster controlling room (Fig 11.).

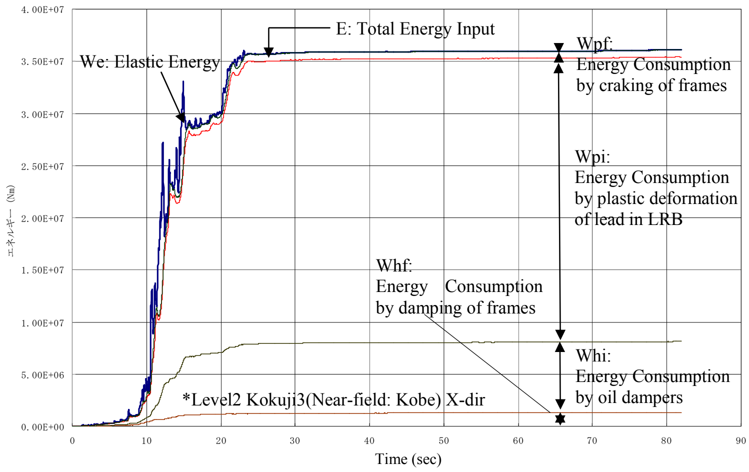
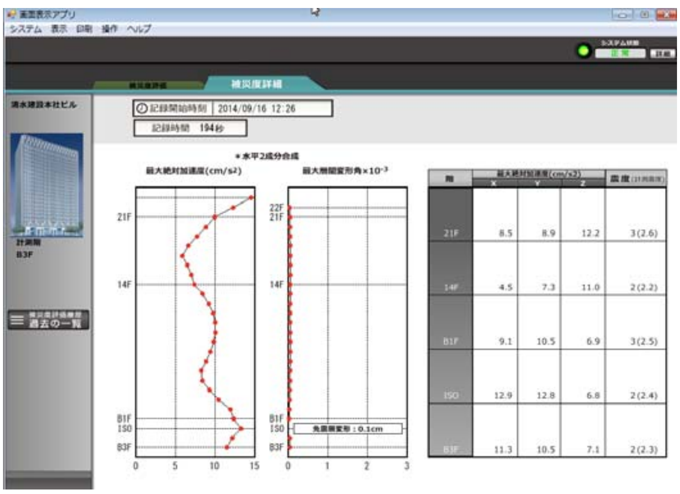


Fig 11. Time history of energy consumption

Simplified display for controll room staffs



Detailed display for structural engineers

Fig 11. Example of monitoring results display

4. Perimeter Precast Frames

4.1. Aluminium Casting Finish

Aluminium casting finish provide durability of the claddings. The difference of linear expansion coefficient between aluminium and concrete , on the other hand, should be considered to avoid cracks on the surface of the claddings due to thermal expansions. In this building, urethane foam is sprayed between aluminium and concrete. This details is modeled as finite element and thermal stress analysis is conducted to investigate the occurrence of cracks due to thermal expansion. Figure 12. shows the results of he analysis. Results show that the maximum deformation due to thermal variation from -10 degree to 80 degree is 1mm, which won’t affect the function of claddings.

In addition to thermal stress analysis, the full-scaled infrared light exposure tests are carried out. (Figure 13.) Surface temperature of alminium is changed from air temperature to 85 degree for 5 days. Strain measurements of aluminium and concrete through the experiment shows that there are no clear clacks produced by thermal changes.

4.2 Precast Concrete Panels

The exterior panels consist of precast concrete girders and columns. Each panels are connected to each other by casting concrete onto half precast concrete zones on panel edges. One of the issues to adopt this construction method relate to rebar joints in half precast concrete zones. As rebar joints , which are mechanical joints, should be installed in half precast concrete zones, it is essential to investigate the details of joints in terms of structural safety and construction workabilities. Regarding to construction workability, grouting to laterally installed joints is the key issue. Through many construction work experiments in full scaled specimens, effective grouting methods are studied. In terms of structural safety, full-scaled beam-column joints experiments are conducted. (Figure 14.) The experiments show that major damages are not observed until the lateral drift to 1/300. Furthermore major strength reduction is not observed until the story drift to 1/100. Through the experiments, it is concluded that the joint details adopted in this precast concrete panels won't affect overall performances as structural members.

The exterior precast concrete panels are made of high strength concrete with design standard strength of 80N/mm2 to minimize the size of structural members. In order to prevent each panels with frames from various cracks, which include thermal cracks caused by hydrogen heat, cracks due to autogenous shrinkage during production processes and other cracks resulted from drying shrinkage or ambient temperature changes after erection, limestone which has minor shrinkage strains is used as concrete aggregates. Additionally, Advanced Fire Resistant (AFR) high strength concrete containing polypropylene (PP) fibers are used to prevent panels from exploding in the event of fire accidents. In order to verify fire resistances of the panels, columns axial loading tests with fire exposures are conducted. (Figure 15.) Columns specimens are 1/2-scaled of the actural member sizes and manufactured in the precast concrete fabricator. The speciments without PP fibers show explosions of cover concrete in 30 minutes after fire exposures and finally lost axial loading capacities after 128 minutes. The specimens with PP fibers, on the other hand, didn't show explosions due to fire exposures and could sustain axial loading capacity for 4 hours.

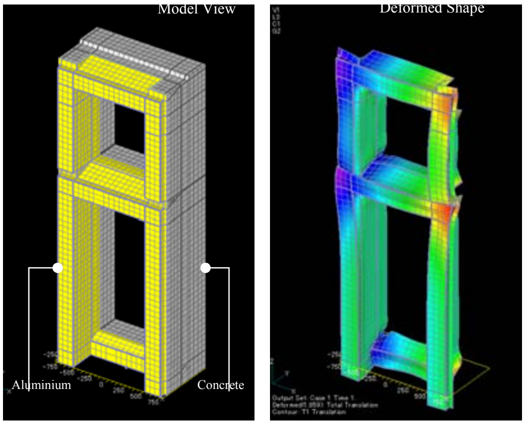


Figure 12. Thermal stress analysis

Figure 13. Thermal test

Figure 14. Beam-column joint experiment

Figure 15. Fire resistance experiment

5. Conclusions

Shimizu Corporation Tokyo Headquarters play a pivotal role in the architectural field in terms of structural engineering, sustainable technologies and construction methodologies. As a disaster management facility, the structural performances and seismic criteria are fully investigated based upon nonlinear time history analyses. In order to contribute more to societies, this building should be the prototype office building for future generations.

6. References

[1] Yamazaki, Y., Tabuchi, T., Kataoka, M., and Shimazaki, D. (2014): 3D/BIM Applications to Large-scale Complex Building Projects in Japan. *International Journal of High Rise Buildings*, 13(4), pp. 311- 323.