



Structural Design of a Seismic Isolated Building with a Guyed Tower

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ABSTRACT: A large seismic isolated telecommunications building with a guyed mast tower was planned to be constructed upon liquefiable subsoil. The foundation and superstructure have to be designed taking soil condition into account. In order to penetrate to the firm layer and to establish rigidity to the soft soil layer, a continuous wall pile system was adopted. For seismic analysis, seven seismic waves were used to carry out the dynamic response. The results showed a prominent response of the structure on the higher modes for the site wave. For reducing this response, viscous wall dampers were installed in the building. Thus the safety of the structure and the realization of the project was achieved by the application of various structural means such as, continuous wall piles, seismic isolators, viscous dampers and by conducting thorough seismic analysis.

1 INTRODUCTION

This paper reports the structural design of a large seismic isolated telecommunications building with a unique cable-guyed telecommunication mast tower. Because the site is reclaimed land with soft soil, the foundation system and structural system need to be designed considering conditions of the soil. Also, design criteria for a building are established to be elastic even for the biggest earthquake expected in the construction site. To satisfy these criteria, the building has a seismic isolation system and damping devices are installed in its middle stories. As for the tower, the design criteria are similarly established to remain elastic against wind loads. The results estimated by wind tunnel test results are extreme because the site is near the tall buildings and the effect of wake is large. To introduce an improved structural performance, damping devices are also installed in the tower. Figure 1 shows CG perspective views of the building and the tower.



Figure 1. CG Perspective views

2 PROJECT OUTLINE

2.1 The building

This is a 12-story building with 54.25 m in eaves height. The building plan, with unit span of 6.4 m, is a rectangle measuring 116.8 m in the X-direction, by 43.2 m in the Y-direction, total floor area is 60993.42 m². A seismic isolation system is applied on this building in order to guarantee sufficient safety and to maintain building functions in the event of a disaster like a major earthquake. The framework has Concrete Filled Tube columns and steel beams, rigid frames with bracing. Also, viscous damping walls are arranged in the building's middle stories. Figure 2 shows the typical framing elevation and plan.

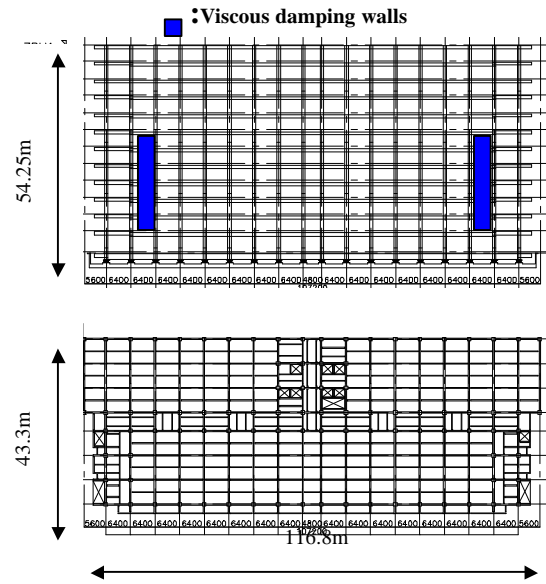


Figure 2. Framing elevation and plan

2.2 The site stratum and the foundation system

The site stratum consists of in-filled land for the first 20 m, another 20 m of alluvium to 44 m below, and further dense gravel and clay diluvium below it. Because the reclaimed layer is soft and likely to liquefy during a severe earthquake, a stiff continuous wall pile system, having a short natural period in foundation-ground system, was applied against liquefaction and for increasing seismic performance significantly. The pile's bottom level was set on a firm sand-gravel layer of diluvium encountered at 45 m below the surface. The stratum composition is shown in Figure 3; the arrangement of continuous wall piles is shown in Figure 4 below.

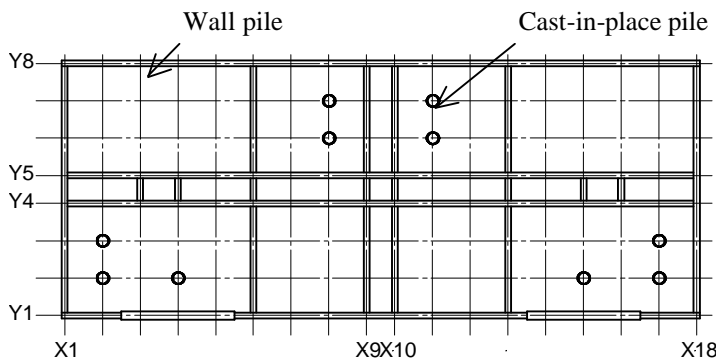


Figure 4. Arrangement of continuous wall piles

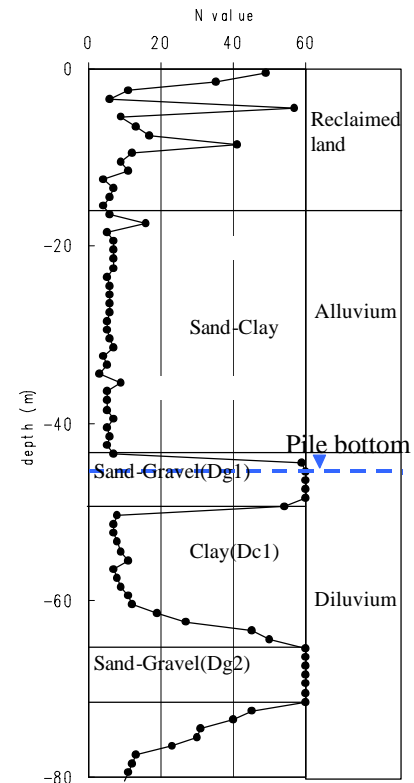


Figure 3. Stratum composition

2.3 Damping device and isolator device

Viscous damping walls are used for damping devices, of which 24 and 22 are arranged in the X- and Y-directions, respectively, between 3 to 8 story. The damping walls are proportionally and symmetrically arranged around the building core and periphery to expect effective performance even when torsional vibration occurs to the superstructure and not to become obstacles in planning the building. The device arrangement is shown in Figure 5.

The isolator system comprises 46 lead rubber bearings (LRB), 42 rubber bearings (RB), and 4 cross linear bearings (CLB) which comes to 92 isolators in all, LRB and RB are 1000.-1500.in diameter and 280 mm in total thickness of the rubber portion. The LRBs are arranged in the building periphery,

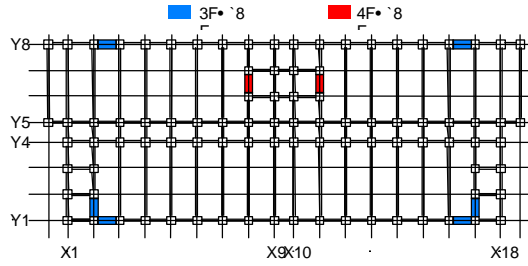


Figure 5. Arrangement of viscous damping walls

with RBs in the central part to reduce eccentricity of the seismic isolated story as much as possible and secure torsional stiffness necessary for the isolated story. To prevent the building from up-lift, CLBs are arranged right beneath the columns, where lateral force causes high axial compression and tensile force. The device arrangement is shown in Figure 6.

2.4 Tower

Simply explained, the structure is a pinned column held upright by four pairs of stay cables, a scheme that is straightforward and appropriate for this particular location. Weighing approximately 1650 tonnes, the steel tower rises to a height of 145 meters above the building (see Figs. 7 and 8 for typical platform layout and the elevation view respectively). The spinal mast element is a 1.7 meter diameter hollow pipe with a wall thickness that varies between 50 and 80 mm. The cables in the tower are 200 mm parallel wire strand (PWS) cables, consisting of 499 galvanized steel wires wrapped within a polyethylene coating. The remaining frame elements are comprised of built-up boxes, pipes, and H-sections. Steel cast pieces are utilized for the cable connections.

To stiffen the steel central mast, four sets of prestressed steel cables and outriggers span the entire height of the tower. In profile they form a bowstring truss, a system that works in a similar manner as the rigging mechanism of a yacht's mast. During construction, the cables are slid over the saddle pieces of each outrigger, then prestressed at the ends that are located at the center and base of the tower. The upper pair of cables (Cable 1) is prestressed at 5500 kN (11,000 kN per pair), and the lower set of cables (Cable 2) has 5000 kN of prestress.

Stay cables are arranged in pairs extending in four directions rather than the minimum of three to match the symmetry of the building. The layout of the cables in plan stabilizes the twisting of the tower; they also give a margin of safety in the unlikely event that a cable should be severed. Stay cables 3 and 4 are prestressed 6000 kN and 7500 kN respectively. With a combination of prestress from the stays and tension truss cables, the mast element experiences approximately 85,000 kN of axial force.

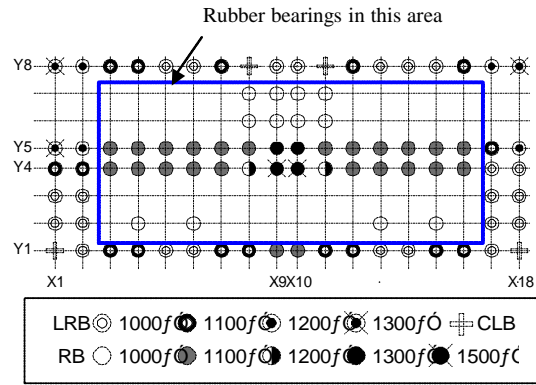


Figure 6. Arrangement of isolators

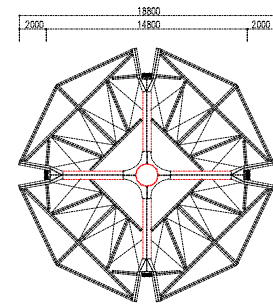


Figure 7. Typical platform

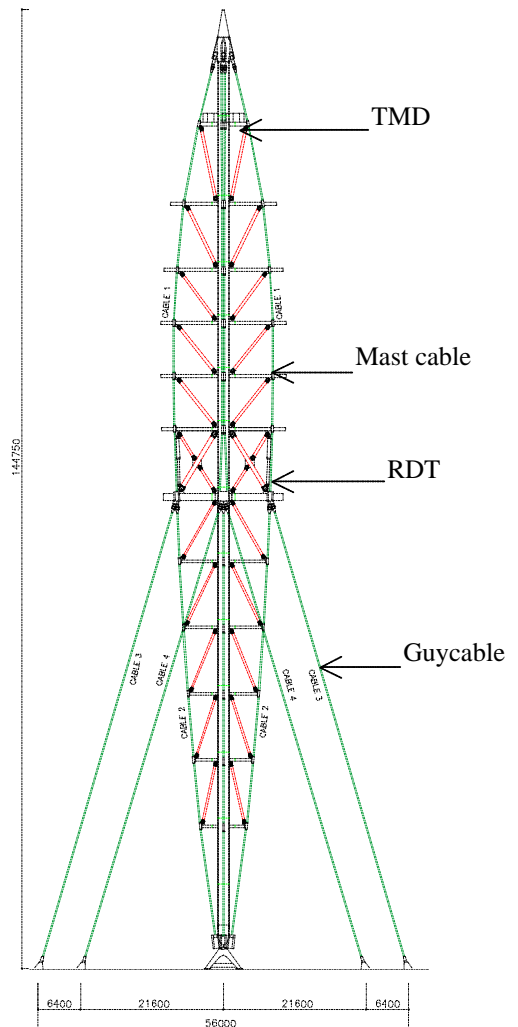


Figure 8. Elevation View

Damping devices were also installed. Two 7.5 tonne tuned mass dampers (TMD) were set on the top platform of the tower(see Fig.9). Consisting of layered rubber pads and steel plates with a mass at the top, the TMD was tuned accordingly to reduce the response. Four pairs of rotary damping tubes (RDT) were installed in parallel with the cable truss near the center platforms to dampen the bending movement of the tower. Figure 10 illustrates RDT. It is important to note that the effects of the dampers were not considered in the structural design of the members, but to introduce an improved structural performance.



Figure 9. Tuned mass damper

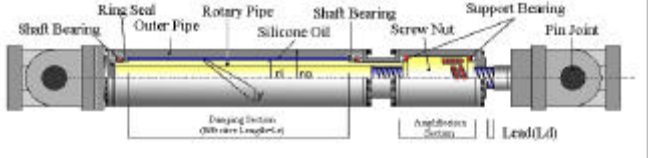


Figure 10. Rotary damping tube

3 STRUCTURAL EXAMINATIONS OF THE BUILDING

3.1 Examination of the effect of the foundation system over the superstructure

This examination shows differences in response properties and seismic effectiveness of the foundation system and the superstructure arising from use of continuous wall piles (hereafter, “wall pile”) and use of cast-in-place concrete piles (hereafter, “cast-in-place pile”). The Penzien model, continuous series of subsoil-pile-superstructure, is used as an analytical model. Each cast-in-place pile is arranged directly beneath each isolator. For the superstructure, the building is modeled as an equivalent shear model with 13 masses; the pile structure is modeled as a flexural shear model. Wall piles are constructed on Dg1 as the pile bottom level; cast-in-place piles are constructed on Dg2. Earthquake waves for analysis are KOKUJI 202 and Site wave. (Refer to 3.3)

As the analytical results of both models, maximum response drift in the X-direction is shown in Figure 11 and maximum response of interstory shear force is shown in Figure 12. Because of wall pile’s high stiffness, the drift of wall pile is one-fifth of that of cast-in-place pile. This may be constraining effects on surrounding soil’s deformation and soil behavior is similar to the wall piles. Regarding to the shear force of the superstructure, difference due to foundation forms is significant, but difference due to earthquake waves is negligible and the response on continuous wall pile is small. Both models show the similar results that the response on the site wave is largest in the middle stories, which is obviously seen in continuous wall pile. This may depend on earthquake ground motion characteristics and building vibration characteristics.

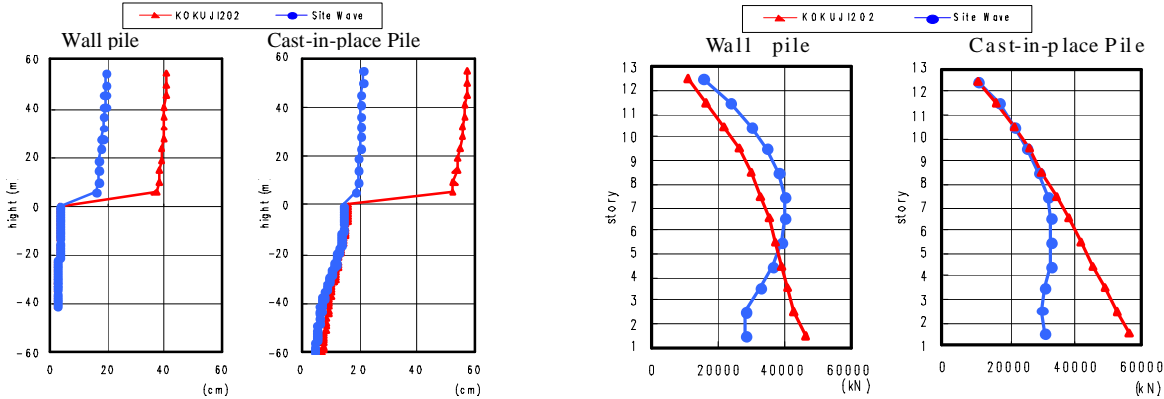


Figure 11. Maximum response drift

Figure 12. Maximum response shear force

3.2 Analysis model

The dynamic analytical model for the building is the equivalent shear model with 13 masses, the isolated story's base fixed, while restoring force characteristics are supposed to be elastic. Because the weight ratio of the tower to the building is only 2%, the tower may be considered a part of the roof. This modeling is verified compared with response results of the dynamic analysis of tower. Damping is the stiffness proportional type of damping ratio $\eta=2\%$ for the first natural mode of the building neglecting the isolated story, damping walls are modeled as a dashpot depending on interstory velocity. As restoring force characteristics of each isolator, LRB is Bi-Linear, RB is Linear and CLB is Bi-Linear. The dynamic analytical model for examining the building is shown in Figure 13 and the dynamic model for examining the tower is shown in Figure 14. Also, Table 1 shows natural period of building model in each direction.

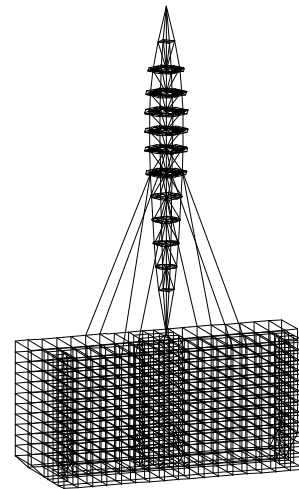
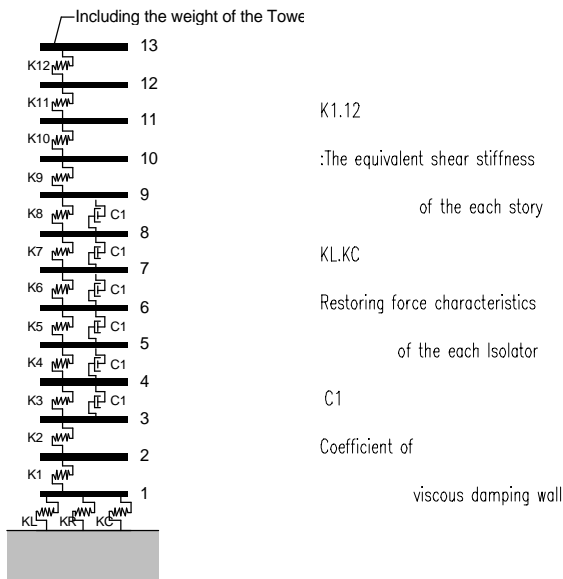


Figure 13. Dynamic analytical model for the building

Figure 14. Dynamic analytical model for the tower (Seismic model)

Table 1. The natural period of building model (seconds)

| strain of Isolator | X-direction | | | Y-direction | | |
|--------------------|-------------|-------|-------|-------------|-------|-------|
| | 1st | 2nd | 3rd | 1st | 2nd | 3rd |
| 0% | 1.489 | 0.537 | 0.320 | 1.441 | 0.528 | 0.320 |
| 0.5% | 2.216 | 0.720 | 0.404 | 2.184 | 0.717 | 0.400 |
| 10% | 2.808 | 0.769 | 0.414 | 2.783 | 0.764 | 0.408 |
| 100% | 4.540 | 0.805 | 0.420 | 4.525 | 0.804 | 0.416 |
| 200% | 4.896 | 0.816 | 0.422 | 4.883 | 0.808 | 0.415 |

3.3 Earthquake ground motions for analysis

The following waves are used as earthquake ground motion for analysis. Three earthquake waves are conventionally observed (hereafter, “observed waves”), three simulated waves in consideration of soil conditions at the designated site (“KOKUJI waves”), and one simulated wave in consideration of the fault and soil conditions which is expected to affect the site the most (“site wave”). KOKUJI waves are

input in the base of the isolated story. KOKUJI waves are simulated earthquake ground motions based on the spectra officially announced by the Ministry of Land(see — line in Fig.15 and Fig.16), Infrastructure and Transport and calculated by non-linear earthquake response analysis in consideration of surface soil conditions. The maximum velocity of the observed waves on the ground level is set to be 70 cm/s. Figure 15 and 16 show acceleration and velocity response spectra of KOKUJI waves and site wave.

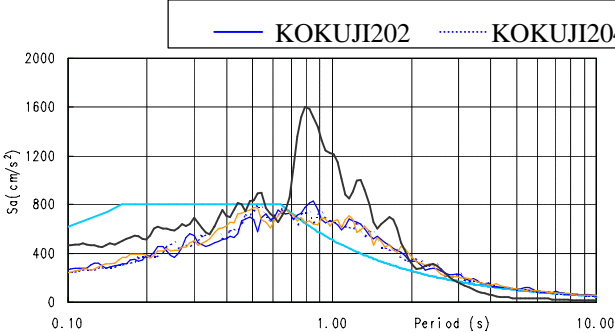


Figure 15. Acceleration response spectra

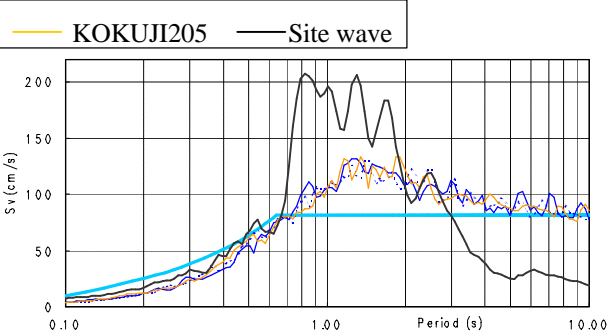


Figure 16. Velocity response spectra

3.4 Dynamic analysis results

The original project did not have the installation of viscous damping walls to this building. Figure 17 shows the maximum response interstory shear force of typical earthquake ground motion in the X-direction on the case not considering fluctuation of isolators’ performance values (hereafter, the “standard case”) without viscous damping walls.

According to analytical results, while all shear force is a less than the designed shear force, site wave response differs from those of other earthquake waves. This shows the fact that the second mode including the isolated story causes resonant vibration near the site wave’s powerful period, as shown in Table 1 and Figure 15, 16. Even if it could satisfy design criteria without installing damping walls in the standard case, it was difficult to satisfy criteria in considering fluctuation of isolators and other factors, so installation of damping devices was examined and determined to use. The building is seismic isolated structure and it’s story drift is relatively small. Therefore, Viscous damping walls which were suitable for slight deformation were adopted.

Figure 18 show a comparison of analytical results in the X-direction with and without installing damping walls (maximum response shear force). We used KOKUJI202, which was prominent response among KOKUJI waves, and the site wave for seismic response analysis. Installation of damping walls shows reduced responses, by 15% in shear force and satisfies the design criteria sufficiently even considering the fluctuation of each device’s performance value.

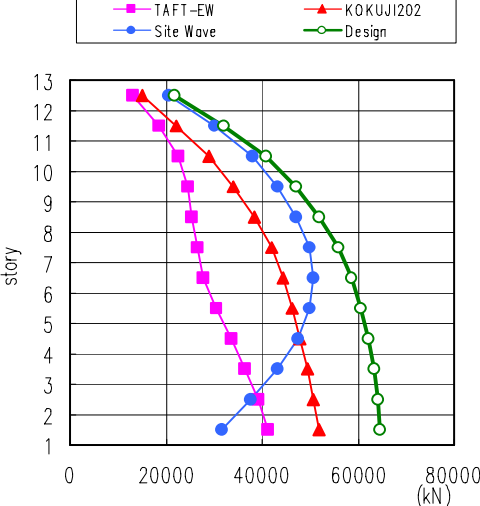


Figure 17. Maximum response shear force

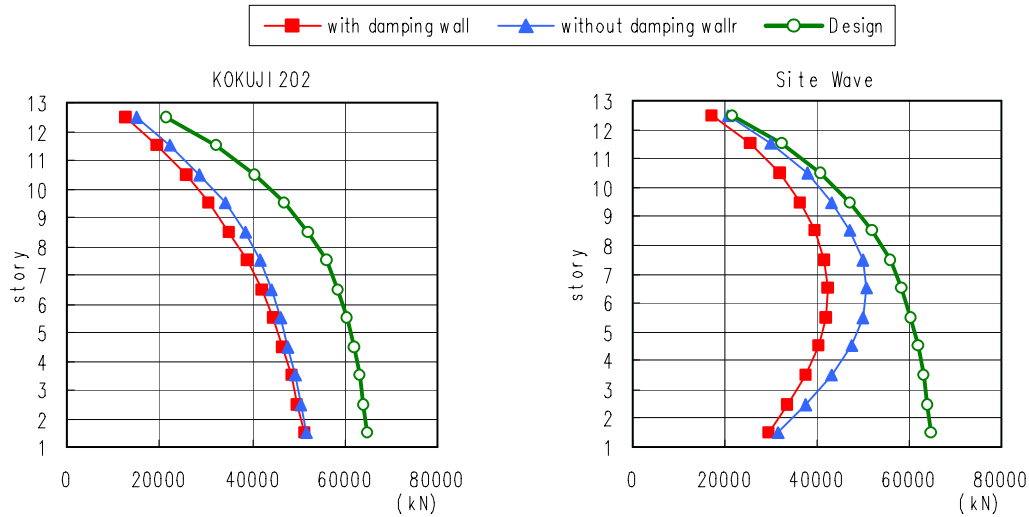





Figure 18. Maximum response shear force

4 STRUCTURAL EXAMINATIONS OF THE TOWER

4.1 Wind analysis

Deemed to be the governing case in this tower design, in order to assess the performance of the tower, non-linear wind time history analyses were conducted. The 500-year return period wind time history inputs were produced by extracting data from the wind tunnel testing. Three wind types were applied to the model for three conditions depending on the layout of the antennas and the direction of wind. Similar to the seismic design criteria, all members were designed to remain elastic. Under the assumption that the wind forces would have very little effect on the isolators of the building, the three-dimensional model for the wind analyses did not include the building. Again the damping coefficient for the analyses was set to 1%. The modal properties of the wind model are shown in Table 2. Compared with the results from the seismic analyses, it can be observed that wind loads were indeed, the governing case for this tower. The maximum tilting angle of the platforms hit a peak of approximately 0.95 degrees. In order to further reduce the tilting angles, TMDs and RDTs were installed in the tower. The axial forces of the stay cables are confirmed to be within allowable limits.

Table 2. Modal properties of the wind model(Tower Model)

| Mode 1, X Direction | Mode 2, X Direction | Mode 3, X Direction |
|---|---|---|
| 1.9434 seconds | 1.0480 seconds | 0.4669 seconds |
|  |  |  |

4.2 Examination of the damping device

In regards to the damping device, Time history wind force analysis was carried out about the following cases of wind loads to examine the most effective installation method (see Fig.19). Under the assumption that the wind forces would have very little effect on the isolators of the building, the three-dimensional model for the wind analyses did not include the building.

Case A The case of installing only TMD

Case B The case of installing four RDT on TR1 line of PL6

Case C The case of installing four RDT on a cable line of PL6

Case D The case of installing 16 RDT on a cable line of PL6

Case E The case of combining Case A and Case C

Analysis results for the wind load of the above five cases are shown in Figure 20.

We found that added damping ratio is from 1 to 2% approximately by installing TMD. Also, when RDT is installed on a cable line of the upper PL6 where guycables show largest axial displacement when wind load is applied, it is the most effective place as the installing location; added damping is from 1 to 2%, similarly to TMD. Moreover, even if the number of installed RDT increases, the effect isn't so large. Also, with an installed the damping device, results of response analysis on effects in the seismic load are presented in Figure 21. For the seismic analyses, three-dimensional model including the building was used(see Fig.14). It can be understood that effects of the damping device in earthquakes are larger than in wind load. This is because repetition components of the tower deformation in the earthquakes are larger in comparison with those in wind load.

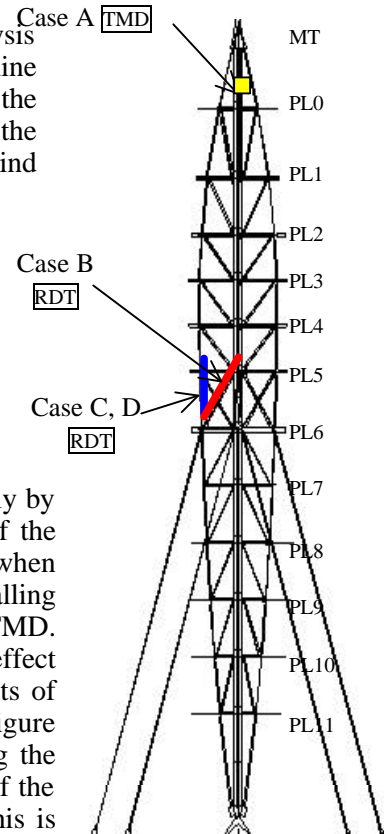


Figure 19. The layout cases of damping device

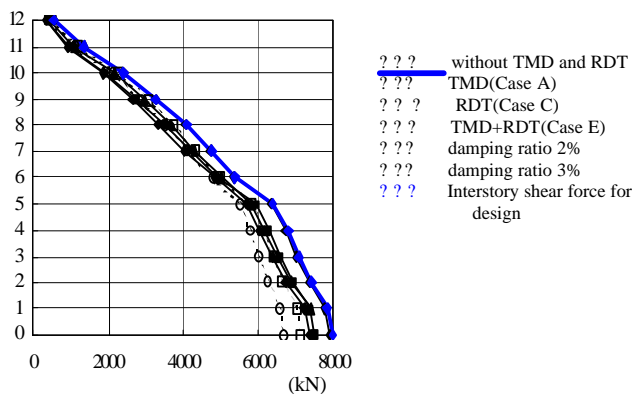


Figure 20. Shear force in wind load

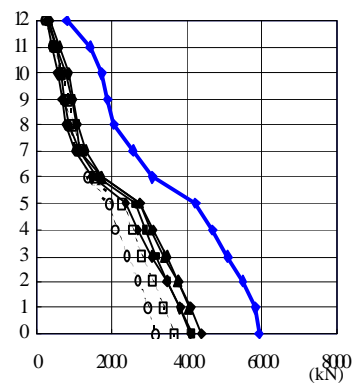


Figure 21. Shear force in earthquake

5 CONCLUSIONS

The results from the response analysis of the lumped mass model(Also including Penzien model)and the three-dimensional frame model, done on the seismic isolated building with a guyed tower which was planned to be constructed upon liquefiable subsoil, help to draw the following conclusions:

(1) As for the foundation system, a continuous wall pile foundation is chosen for the large-scale seismic isolated building to be constructed on the reclaimed ground. Even if shear force caused by ground drift during earthquake becomes larger compared to a cast-in-place concrete pile foundation, it can be expected to reduce the seismic force acting on the long period structure like this seismic isolated building.

(2) As for the superstructure, the response was greatly reduced by applying viscous damping walls on the isolated structure considering the site's soil conditions and the building's vibration characteristics. If earthquake ground motion characteristics coincide with the response of the building higher modes like this building, the reduction of response by damping devices is also effective.

(3) In consideration of results of the wind tunnel test, that the wind load was predominant, and the fact that the tower had a unique shape, various examinations were conducted for the wind in particular; special means such as installing the damping device were taken for vibration control. As for the effects of the damping device, response analysis was carried out to confirm its effects for wind and for earthquake. As a result, since repetition components of tower deformation are larger in the earthquake, damping effects were shown distinctly. Although the effects for wind were not as clear as those for the earthquake, it was confirmed that installation of the damping device was effective for wind because it damped approximately 1 to 2%.

This project is due for completion in June of 2004.

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