Structural Design of 80-Story RC High-Rise Building Using 200 Mpa Ultra-High-Strength Concrete

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Abstract

This paper presents the design of an 80-story reinforced concrete (RC) high-rise building using 200 MPa ultra-high-strength concrete. Static nonlinear push over analyses, Level-1 and Level-2 nonlinear earthquake response analyses and nonlinear wind response analyses were carried out. Based on the three-dimensional static nonlinear analyses of the building subjected to design earthquake loading in two principal directions, the obtained maximum axial load ratio for the first story columns of 200 MPa compressive strength concrete were 0.53 and 0.48, respectively, at the ultimate limit state, which meets the design criterion based on the allowable compressive stress of concrete. The maximum story drift angle obtained under the synthetic wave motion at the construction site was smaller than the design limiting value of 1/100. While yield hinges developed only in some of the short beams, no yield hinge in columns was observed. The maximum ductility of 1.28 obtained in the beams is lower than the design limiting value of 4.0. The maximum story shear force for the level-2 wind load was almost half that of the level-2 earthquake load when using the lumped-mass model. The analyses confirmed that the use of 200 MPa concrete enables structural designers to provide the member sections with adequate sizes comparable to that of ordinary high-rise RC buildings. The analytical results showed that the performance of the building satisfies the design criteria for serviceability limits, design limits and ultimate limits.

1. Introduction

Recently, construction of high-rise apartment buildings has increased significantly, with the number of stories sometimes exceeding 50. High-strength materials are used in such buildings. The design concrete strength reaches 100 MPa and SD685 deformed bars of 685 MPa nominal yield strength are used as the main reinforcing bars.

To investigate the possibility of constructing taller buildings, this paper addresses the design of an 80-story high-rise apartment building using 200 MPa concrete and SD685 reinforcing bars.

2. Outline of building and structural design procedure

2.1 Building outline

The 80-story building is a frame structure with a total height (roof level above the ground) of 252m. The standard floor plan, shown in Fig. 1, has external dimensions of 33 m x 54 m with a void at the center and a story height of 3.1 m. Frames in the longitudinal direction are regularly spaced at 6.0 m intervals while those in the transverse direction are irregularly spaced with span lengths varying from 4.0 m to 9.5 m (see Fig. 1). Figure 2 shows...
the elevation of the frame along the transverse X direction (short side) and also the material and cross-section dimensions for all elements along the height.

The column concrete strength varied from 200MPa at the first stories of the building to 30MPa at the last ten stories.

Fig. 2 Elevation of 80-story building and material and cross-section characteristics.
stories. Similarly, beams with concrete strengths varying between 100 MPa and 30 MPa were considered. As for the reinforcement, the same type of steel was considered in all the columns and beams: SD685 steel was considered for main bars while higher strength deformed bars with nominal yield strength of 1275 MPa were provided as shear reinforcement. Tables 1 and 2 illustrate the reinforcement detailing of selected elements.

The building is assumed to be constructed in Tokyo, in a suburban residential area of flat terrain of category III (AIJ 1993), and founded on piles in type II soil, which corresponds to soft diluvial or firm alluvial layers. The pile tips are to be embedded in a layer strong enough to sustain the weight of the building.

### 2.2 Structural design concepts

The performance of the building was investigated for both seismic and wind loads, as shown in Fig. 3, according to the Japanese codes for the design and construction of buildings. Three limit conditions were set up, namely:

1. At least once during its service life, the building will experience a minor earthquake of level-1 (maximum velocity equivalent to 25 cm/s) and a standard wind of level-1 (maximum velocity at the top of the building equal to 5750 cm/s). Yield hinges should not develop on the main structural elements, while damage, cracks and deformations should be kept within the serviceability limits.

2. For the maximum probable earthquake of level-2 (maximum velocity equivalent to 50 cm/s) and a standard wind of level-2 (maximum velocity at the top of the building equal to 6680 cm/s), yield hinges should not develop on the main structural elements, while damage, cracks and deformations should be kept within design limits (see Table 3).

<table>
<thead>
<tr>
<th>Story</th>
<th>Column A1</th>
<th>Column A2</th>
<th>Column C1</th>
<th>Column C2</th>
<th>Column E1</th>
<th>Column E2</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td><img src="image1" alt="Diagram" /></td>
<td><img src="image2" alt="Diagram" /></td>
<td><img src="image3" alt="Diagram" /></td>
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<td><img src="image8" alt="Diagram" /></td>
<td><img src="image9" alt="Diagram" /></td>
<td><img src="image10" alt="Diagram" /></td>
<td><img src="image11" alt="Diagram" /></td>
<td><img src="image12" alt="Diagram" /></td>
</tr>
</tbody>
</table>

### Table 1 Reinforcement feature for selected columns.

<table>
<thead>
<tr>
<th>Story</th>
<th>Girder C1-C2</th>
<th>Girder C2-C3</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
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<td><img src="image14" alt="Diagram" /></td>
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<tr>
<td>2</td>
<td><img src="image15" alt="Diagram" /></td>
<td><img src="image16" alt="Diagram" /></td>
</tr>
</tbody>
</table>

### Table 2 Reinforcement feature for selected beams.
3. For the ultimate earthquake exceeding level-2, collapse or brittle failure of structural elements should be prevented to satisfy ultimate limits (see Table 3).

Static and dynamic analyses were done to meet each limit condition. Table 3 shows the static design criteria. Regarding dynamic design criteria relative to seismic and wind loads, the maximum story drift angle should be less than 1/200 and 1/100, respectively, for level-1 and level-2 earthquake loads and wind loads.

For static design, three-dimensional elastic frame analyses using lateral load distribution (Ai distribution) along the height based on a preliminary elastic seismic response analysis of a multi-degree-of-freedom model for level-1 earthquake and wind response analysis for level-1 wind were performed. For both loads, the highest stress should satisfy the allowable stress design level.

The next step in the static design was to evaluate the ultimate strength based on three-dimensional nonlinear pushover analysis. The ultimate strength design was performed for the member stress using the lateral deformation at the building height gravity center as the largest of the following:
a. Displacement when a value for the area under the curve for the base shear force-lateral displacement at the building height gravity center twice that for the maximum response for level-2 earthquake is used, or
b. Maximum displacement at the building height gravity center relative to the building height gravity center not exceeding 1/100

Regarding the seismic load level for the allowable stress design, the load based on the $A_i$ distribution method was used, because the base shear force is larger than that of a preliminary elastic response analysis using level-1 earthquake records (El-Centro NS and Taft EW). For the wind load level for the allowable stress design, the loads acquired from level-1 wind tunnel test results were used.

The shear force distributions in the X direction for both wind and seismic loads are shown in Fig. 4. The seismic forces are greater than wind forces in the upper fourteen stories (i.e. 66-80), while the opposite holds for the remaining stories (corresponding to the base shear force coefficient $C_b = 0.037$). In the Y direction, the distributions of seismic and wind forces are nearly the same in the lower stories similar to the $A_i$ distribution (corresponding to the base shear force coefficient $C_b = 0.03$).

### Table 3 Criteria of static design.

<table>
<thead>
<tr>
<th>Limit Condition</th>
<th>Design Load</th>
<th>Design Terms</th>
<th>Design Condition and Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service Limit State</td>
<td>/ Lateral load distribution along the height is determined by elastic response analyses using level-1 earthquake and level-1 wind load. / Design elastic shear force is calculated according to Building Standard Law. / Member force used in allowable stress design is based on elastic analysis.</td>
<td>/ Design of members</td>
<td>/ Allowable stress design</td>
</tr>
<tr>
<td></td>
<td></td>
<td>/ Deformation of structure</td>
<td>/ Maximum story drift angle is less than 1/200.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>/ State of structure</td>
<td>/ No yield hinge is formed.</td>
</tr>
<tr>
<td>Design Limit State</td>
<td>/ Story shear force calculated by nonlinear pushover analysis is used for member design. A story drift angle of each story envelope curve that is larger than the results of nonlinear response analyses using level-2 earthquake and level-2 wind load is selected.</td>
<td>/ Design of columns</td>
<td>/ Stresses of column main bars are less than nominal yield strength.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>/ Deformation of structure</td>
<td>/ Maximum story drift angle is less than 1/100.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>/ State of structure</td>
<td>/ Yield hinges are designed to be formed at only beam ends, bottom of columns in the lowest story and top of columns in the upper most story.</td>
</tr>
<tr>
<td>Ultimate State</td>
<td>/ Ultimate strength design is performed for the member stress at the story drift level where the work-energy of the maximum response for level-2 earthquake has to be double or the story drift level of 1/100, whichever larger, in the relationship between base shear force and its corresponding lateral deformation at the building height gravity center.</td>
<td>/ Design of members</td>
<td>/ Ultimate strength design</td>
</tr>
<tr>
<td></td>
<td></td>
<td>/ Axial compression stress and tension stress in columns are, respectively, less than 0.65 of concrete strength and 0.7 of main bars' yield strength.</td>
<td>/ Deformation angle of beams are less than 1/20.</td>
</tr>
</tbody>
</table>

\[
\begin{array}{|c|c|c|c|}
\hline
\text{Story number} & \text{Wind load} & \text{Seismic load} \\
\hline
\text{Story shear force (kN)} & \text{Story shear force (kN)} \\
\hline
\end{array}
\]

Fig. 4 Seismic and wind load distributions in X direction.

Therefore, these base shear coefficients are used for the design of the building based on the allowable stress method.
3. Static nonlinear analysis

Three-dimensional analysis was performed for the frame structure. Columns and main beams were modeled by a three-component bending-shear element composed of a stiffened plastic rotation spring at both ends and a shear elastic spring in the central part. Furthermore, an elastic-plastic spring was considered along the longitudinal axis for each column. The beam-column joints and floors were assumed rigid in their planes. A trilinear moment-rotation idealization was assumed for the flexural behavior of columns. A degrading trilinear idealization is used for the restoring force characteristics.

4. Earthquake response analysis

4.1 Modeling assumptions
An equivalent lumped-mass shear-building model and a three-dimensional frame model were considered in this study.

The analysis of the equivalent 80-lumped-mass shear-building model and the three-dimensional frame model is based on the story shear force vs. inter-story displacement relationships obtained from the static nonlinear analysis presented previously, where a degrading trilinear idealization is used for the restoring force characteristics.

The equivalent shear model is considered to be fixed at the bottom of its first story. Structural damping, corresponding to the first natural period, is assumed proportional to the instantaneous stiffness assuming a constant value of the internal viscous damping equal to 3%. The first and second natural vibration periods of the three-dimensional model are 7.04 s and 2.00 s in the X direction and 5.50 s and 1.73 s in the Y direction.

4.2 Applied earthquake records
The selected ground motion records are based on the vibration periods’ range of the building. For the level-1 earthquake, El-Centro NS and Taft EW ground motion records were selected for assessing the response of the building, while for the level-2 earthquake, synthetic ground motion records were generated based on the Japanese code’s type-II soil spectrum considering a random phase, the phase of the 1993 Kushiro-Oki earthquake and another synthetic wave record (Site wave) generated from the same fault model reflecting the ground characteristics corresponding to the 1923 Kanto earthquake. The velocity response spectra corresponding to level-2 earthquake synthetic waves are shown in Fig. 6.

4.3 Results of analyses
Figure 7 shows the distribution along the height of the building of the maximum story drift angle corresponding to the shear-building model subjected to level-1 earthquake input motions. Maximum story drift angle values were obtained in both the X and Y directions under Taft ground motion and are equal to 1/305 and 1/370, respectively. Also, maximum base shear coefficients were obtained under the same wave record in both directions.
X and Y and are equal to 0.017 and 0.020, respectively. The maximum story drift angles in the lower stories for El Centro ground motion are half those for the Taft ground motion.

Figure 8 shows the distribution along the height of the building of the maximum story drift angle, the maximum story shear force and the maximum overturning moment corresponding to the three-dimensional model and subjected to level-2 earthquake input motions. The maximum values of the story drift angle, obtained under the Site wave motion for both the X and Y directions, are equal to 1/146 and 1/172, respectively, which are both values below the design value of 1/100. The maximum values of the story shear force obtained under the same input motion in both the X and Y directions are equal to 0.053 and 0.050, respectively. To allow a comprehensive comparison, the figure also contains the distributions corresponding to the equivalent shear model and shows that its response, in terms of maximum story drift angle, maximum story shear force and maximum overturning moment, is in all cases higher compared to the three-dimensional model. The reason for this difference is understood to be due to the excessive bending deformability of columns and vibration modes of the lumped-mass model.

Figure 9 shows all the steps of the response of the three-dimensional model under the Site wave record relating the first-floor shear force to its corresponding overturning moment. While the figure shows a linear relationship in the results obtained from the static analysis, it also shows that despite the continuous increase in the shear force, the increase in the overturning moment, obtained from dynamic analysis, converges towards a limit value of approximately ±10 GN·m. This fact means that the assumed $A_i$ distribution and the external force distribution are different beyond a certain level of lateral displacement due to the change in the stiffness of the elements.
response analysis is based on the existing wind tunnel test results and met the design criteria.

Meanwhile, the structural design of this building includes 200 MPa concrete columns. All maximum values are well below the design criterion of 0.65. The use of 200 MPa concrete results in the same cross section as in the columns of an ordinary high-rise RC building. Such smaller sections reduce dead load, save building materials and enable longer beams. However, due to the slenderness of the members, deflection is generally expected to become a dominant aspect for the total design. Meanwhile, the structural design of this building achieved both the deflection control and the member design forces, and met the design criteria.

5. Wind response analyses

5.1 Wind resistant design concepts
Apart from the design procedure presented previously on earthquakes, the performance of the building should also satisfy rank III habitability to wind vibration (response below estimation curve H3 shown in Fig. 14) as specified in the provisions on comfortability (AIJ 1991). Comfortability was examined by using the first natural frequency of the building and the maximum response corresponding to the wind velocity level for a one-year return period.

5.2 Horizontal wind load
The horizontal wind load used for the present wind response analysis is based on the existing wind tunnel test results of a similarly shaped building whose load time history data was obtained in 72 directions with 5-degree pitches. Furthermore, while the effect of the cut-edge of the building corner on the response was taken into account, the influence of surrounding buildings was not included.

The design wind velocities at the top of the building (252 m high) for a level-1 wind (100-year return period) and level-2 wind (500-year return period) are 57.5 m/s and 66.8 m/s, respectively. These values were calculated based on the load codes (AIJ 1993) where the basic wind
velocity is 38 m/s assuming the area to be a flat terrain of category III. As for the estimation of the habitability to wind vibration, a wind velocity of 29.3 m/s was used according to the provisions for comfortability (AIJ 1991).

5.3 Results of analyses
Modal response analyses of generalized one-degree-of-freedom elastic systems were first carried out in order to assess the critical direction of the design wind load (in terms of the maximum base shear force). By normalizing the vibration shape modes to unity, the first generalized masses in the X and Y directions were assessed and found to be equal to 64,100 tons and 74,300 tons, respectively. The first natural frequencies in the X and Y directions are 0.13 Hz and 0.17 Hz, respectively, and the damping ratio is 1.5% (AIJ 2000). The variation in the base shear force, related to the wind direction angle for level-1 and level-2 wind velocities, are presented in Fig. 11. The maximum base shear forces were obtained in the orthogonal direction of the wind direction when applied along the X and Y axes. When the wind direction angle is set to 90° and 180° from the Y axis, the resulting maximum base shear force is obtained along the perpendicular axis of each direction with an angle of 0° and 90°, respectively. The latter maximum shear force directions were properly selected for the shear-building response investigation.

The same story characteristics (degrading trilinear form of the story restoring forces) of the 80-lumped-mass model used for the static nonlinear analysis presented in section 3 were used for the wind response investigation. The first story was considered fixed at the story bottom and the internal viscous damping was assumed proportional to the instantaneous stiffness and equal to a constant value of 1.5% (AIJ 2000).

While the time history response analysis was carried out for 650 seconds, the first 50 seconds were excluded because the focus was on the steady state response. The results of the non-linear analysis corresponding to level-1 wind velocity are presented in Fig. 12, where the distributions of the story shear force, the story drift angle and the story overturning moment along the height of the building are greater in the X direction than in the Y direction. It is noticed that contrary to the elastic response analysis (see Fig. 11), Fig. 12 shows that the building response in the wind direction is larger than the response in the across-wind direction when the wind action is in the X direction. In such case, it is understood that the average component, which is not affected by the non-linearity effect, is included in the response.

The maximum base shear force coefficients in the X and Y directions are 0.029 and 0.021, respectively. The maximum story drift angles in the X and Y directions satisfy the level-1 design criteria (1/200) and are equal to 1/201 and 1/378, respectively.

The results of the non-linear analysis corresponding to the level-2 wind velocity are presented in Fig. 13, which shows the distributions of the story shear force, the story drift angle and the story overturning moment in the X and Y directions along the height of the building, as well as the nonlinear response of the shear-building model to the level-2 earthquake Site wave record. It appears that the distribution trend is similar to the trend of level-1. The maximum base shear force coefficients in the X and Y directions are 0.033 and 0.026, respectively. The maximum story drift angles in the X and Y directions satisfy the level-2 design criteria (1/100) and are equal to 1/179 and 1/264, respectively. They are smaller than the response values obtained under the Site wave record.

5.4 Estimation of habitability to wind vibration
Modal analysis was carried out to evaluate the building response. Structural secondary elements were considered in the evaluation of the first natural vibration frequencies, which were taken 1.2 times (AIJ 2000) the values mentioned previously. The first natural vibration frequencies in the X and Y directions are equal to 0.17 Hz and 0.22 Hz, respectively. The damping ratio is considered equal to 1% (AIJ 2000). This value is smaller than the damping ratio in the previous seismic and wind response analyses. This reduction is due to the low vibration amplitude.

![Fig. 11 Base shear force related to wind direction angle (one-degree-of-freedom response analysis).](image-url)
under wind loads (AIJ 2000). The maximum response is obtained in the across-wind direction and the maximum accelerations at the upper floor in the X and Y directions are 6.3 cm/s² and 4.4 cm/s², respectively. These values are plotted along with the habitability to wind vibration estimation curves in Fig. 14. Although the value in the X direction does not satisfy the rank III requirement, providing shaking control equipment brings the results to a sufficient level to satisfy the required design criteria.

6. Concluding remarks

A trial design of an 80-story RC building using a 200 MPa ultra-high-strength concrete was performed. The following conclusions of practical significance can be drawn.

1. The first natural periods of the three-dimensional model for the X (short side) and Y (long side) directions are 7.04 s and 5.5 s, respectively. The natural periods of the lumped-mass model are greater than those of the three-dimensional model.

2. Based on the linear static analysis using Ai distribution and the seismic and wind response analyses, the base shear coefficients of 0.037 and 0.03 were used for the

allowable stress design in the X (short side) and Y (long side) direction, respectively.

3. In the X (short side) direction, the seismic forces in the upper stories are greater than wind forces while the opposite holds for the remaining stories. In the Y (long side) direction, the distribution of seismic force and wind force
is nearly equal in the lower stories similar to the $A_i$ distribution.

4. Based on the three-dimensional nonlinear static analysis, at the ultimate limit stage, the obtained maximum axial load ratio for 200 MPa concrete strength of the first story columns in the X and Y directions are 0.53 and 0.48, respectively, and meet the design criterion of a value smaller than 0.65.

5. The maximum values of the story drift angle obtained under the Site wave motion for both the X and Y directions are equal to 1/146 and 1/172, respectively, which are below the design value of 1/100.

6. The assumed $A_i$ distribution and the actual external force distribution differ. The first-floor shear force is linearly related to its overturning moment for the static analysis of the three-dimensional model. However, for the dynamic analysis, the increase in the overturning moment converges towards a limit value.

7. For the level-2 seismic response analysis, while yield hinges develop only in some of the short beams of the first 15 stories, no yield hinge in columns is observed in any of the frames. The maximum ductility of 1.28 is obtained in some beams in the second story, which is lower than the design value of 4.0.

8. For the level-2 seismic response analysis, the maximum axial load ratios of 0.45 and 0.37 are reached in columns with concrete strength of 100 MPa and 200 MPa, respectively, and are far below the design criterion corresponding to 0.65.

9. The lumped-mass model was considered for the nonlinear wind response. The across-wind vibrations produced the maximum effect on the building.

10. The maximum story shear force for the level-2 wind vibration is almost half for the level-2 earthquake response when using the lumped-mass model.

11. The maximum story shear force and drift angle distribution trend under level-2 wind load is almost similar to under level-1 wind load. All values are below the design criteria.

12. For habitability to wind vibration, the maximum acceleration in the X direction in the upper floor (equal to 6.3 cm/s²) is 43% higher than in the Y direction. The value in the X direction does not satisfy the rank III requirement of AIJ regulations. However, providing shaking control equipment brings the acceleration level to the required level.

References

