PERFORMANCE BASED SEISMIC EVALUATION OF A 62 STORY RC TOWER IN ISTANBUL

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ABSTRACT

A 62-story tall reinforced concrete office building with a free-standing height of 250 m has been designed in Istanbul in accordance with the new Turkish Earthquake Code (TEC, 2017). According to TEC 2017, buildings taller than 70 m and located in the first and second seismic zones are classified as tall buildings and these building shall be designed to meet the special rules based on performance-based seismic design approach. The tall building in Istanbul, having a parallelogram shaped floor plan and reinforced concrete core wall-peripheral frame system will be one of the tallest buildings in Europe. In this study, a comprehensive description of the structural system of the building is given. Then the design criteria for seismic and wind actions and the analytical models prepared for different performance levels are explained. Finally, the results obtained for the Immediate Occupancy performance under 43-year earthquake and Life Safety performance level under the 2475-year earthquake are presented and discussed. In this sense, this study presents the first real case implementation of the new Turkish Earthquake Code for the design of tall buildings in severe seismic zones.

Keywords: Tall buildings; Seismic performance of tall buildings; Performance based seismic design; Nonlinear dynamic analysis

1. INTRODUCTION

The number of tall buildings are steadily increasing in the world and a large portion of these buildings are located in the regions of high seismicity. The number of tall buildings constructed in Istanbul which is located in one of the most severe seismic zones in the world is reaching 200. Unlike regular buildings, tall buildings are peculiar due to their specific architectural properties and building configurations. In addition, seismic response of tall buildings under the effect of seismic and wind loading is different since the contribution of higher mode effects is significant on their dynamic behavior. When considering all these factors, high strength materials and innovative structural systems are generally employed in order to resist the unique challenges introduced by these structures in severe seismic zones (Moehle, 2005). Due to these differences, standard seismic design provisions that are developed for regular buildings cannot be utilized directly for seismic design of tall buildings. Moreover, the studies show that traditional analysis and assessment methods from the prescriptive force-based design methods based on linear elastic analysis are also inadequate for design of tall buildings (Budak et al., 2017). Accordingly, performance based seismic design (PBSD) approach has been increasingly carried out in order to assess the seismic behavior of tall buildings. According to the new Turkish Earthquake Code (TEC, 2017), buildings taller than 70 m and located in the high seismic zones are classified as tall buildings and these buildings shall be designed to meet special rules based on performance-based seismic design approach. In this study, after summarizing the fundamental design steps of the building, a comprehensive description of the structural system and seismic and wind design criteria are given. Finally, selected some results on the seismic performance assessment of the building are presented.

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2. FUNDAMENTAL DESIGN STEPS AND PERFORMANCE OBJECTIVES OF THE BUILDING

The Istanbul tower building whose architectural render with its construction view in January 2018 is shown in Figure 1 is under construction in Istanbul. The building has 62 stories which totals to 250 meters in height. In this section, the main design steps from the first conceptual architectural drawings to final design are summarized.

The design process starts with the initial architectural drawings (design concept or main theme) which are not final drawings but they represent a basis for the rest of the work. After structural engineer receives preliminary architectural drawings, the first step is to justify whether it is possible to design and construct what the architecture has in mind. Structural engineer and architect work together at this stage and collaborate with wind engineer, fire engineer and the other stakeholders in order to achieve a successful preliminary design, leading to a preliminary structural configuration and structural dimensions.

Selection of the structural system and initial proportioning structural members is the most crucial stage in design since decisions to be taken by architecture and structural engineer at this stage affects all the forthcoming decisions related to project. Previous studies on tall buildings indicate that core wall slenderness ratio (the ratio of building height to the small dimension of the core wall) should be between 6 to 8 if a shear wall system is employed as the lateral load resisting core wall system, but this ratio may be increased to 10-15 by using alternative innovative structural systems (Arup, 2013; Tomasetti, 2009). Other studies also show that for tall buildings up to 30 stories, a concrete core shear wall with peripheral columns and flat slab floors (shear-wall frame interaction systems) are adequate to resist lateral loads adequately (Colaco, 2005). Reinforced concrete rigid frame-shear wall interacting systems resists effectively lateral loads in the range up to 50 or 60 stories (Ali et al., 2007). Considering that the slenderness ratio of the core wall system as imposed by the architectural plan is 13-15 and the building
consists of 62 stories, it is not appropriate to construct the lateral load resisting system only from a core shear wall frame interaction system. For achieving a consensus, the outriggered reinforced concrete structural system with core shear walls and perimeters columns (some of them are composite) is selected for the building. Core shear walls dimensions, column dimension and the other dimension of the structural members are estimated by considering the preliminary architectural drawings, engineer’s experience and intuition, preliminary simplified analysis results etc. Shear wall dimensions are gradually reduced toward the top floors without any sudden loss of rigidity between floors. Dimensions of columns are determined by considering the preliminary architectural drawings and the axial load level on columns. The dimensions of coupling beams are determined from the thickness of the shear wall (width of the coupling beam), door openings (length and depth of the coupling beam) and considering the shear and moment actions on them. The tower perimeter beam dimension is selected by considering the flexural action on the beams. Gravity floor systems are selected by taking into account the architectural needs and gravity loads. After the initial structural member dimensions are estimated by joint work of structural engineer and architect, the preliminary structural model of the building is developed in order to obtain the input data of wind tunnel tests. The necessary data in order to carry out wind tunnel tests are the dimension of the building, the building height, story height, the mass and the stiffness distribution of the building, the first six eigenvalue analysis results, damping, base shear and overturning moment etc. Then, 1:400 scale model of the building with and without the proximity (including existing and proposed buildings) is developed and wind tunnel tests are carried out in accordance with the standard procedure for wind tunnel testing in civil engineering as described in ASCE-2012, WtG 1986 and EN 1991-1-4-2005 in order to determine wind loads for structural design and cladding design. The detailed information about wind tunnel tests is given in the ongoing chapter for building. After the wind tunnel tests results satisfy the pre-determine wind safety criteria, the initial proportioning of structural members is completed. On the other hand, geotechnical survey and seismic hazard analysis is utilized for the project site. All of this information is necessary for detailed final design. Finally, the building is designed according to capacity design principles under selected design basis earthquake and design wind loads, followed by two performance levels: serviceability evaluation and collapse prevention evaluation according to the provisions of both the new Turkish Earthquake Code draft version (TBEC, 2017) and the current Turkish Earthquake Code (TEC, 2007) which must be performed officially.

According to TEC-2017, two performance level are defined under seismic excitation. The first performance level is service level evaluation (immediate occupancy) to check the structure under high probability of occurrence (frequent) earthquakes with return periods of 43 years, (50% probability of exceedance in 30 years, SLE). At this stage, it is desirable that the structural members shall remain essentially elastic. A small post-yield deformation may be allowed for ductile members but a permanent damage must not be appreciated. The second performance level is collapse prevention (CP) evaluation stage to check the structure under the low probability of occurrence earthquakes with return periods of 2475 years (2% probability of exceedance in 50 years, MCE). It is desired to maintain stability under expected strong earthquakes, namely collapse of the structures is undesirable. Instead of these, limited damage in specified locations and a specific stress value is permitted for a reasonable design.

Two performance levels are also defined under wind loads. The first one is human comfort criterion which is associated with the human perception of acceleration. The top horizontal peak acceleration of the building under wind load of 10 years from wind tunnel test results should remain below a specified acceleration limit to guarantee to its occupant comfort. The second performance level is service level evaluation stage to check structure in accordance with displacements under wind load of 50 years. The design process is concluded when the pre-determined target performance objectives are satisfied.

3. DESCRIPTION OF THE BUILDING, MATERIAL PROPERTIES AND DESIGN LOADS

3.1 The Structural System

The building will be one of the most exclusive tall building projects in Istanbul, which is under construction. The building possesses a parallelogram shaped flor plan, with a reinforced concrete core
wall and a composite peripheral column system and an outrigger system. Typical podium floor plan with the basic dimensions is presented in Figure 2-a. It is 62 stories tall, with 7 basement levels, B7-B1 (5x3.4 m + 2x5.0 m), a ground level, GL, (5.0 m) and 54 upper ground levels, L1-L54, (8.5 m + 53x4.0 m), which is 252.5 meters in height. The building has a typical tower floor area (44.0 by 44.0 m) of approximately 1780 square meters for each floor above grade.

![Figure 2](image)

The selected structural system responds to the architectural form and usage of the building, while ensuring the structural design requirement of tall buildings when exposed to vertical gravity loads and varied lateral wind and seismic forces scenario. In other words, the selected structural system is capable of providing adequate strength, stiffness and energy dissipation capacity to withstand the design objectives such as the serviceability issues of the building due to lateral wind loads deflection and acceleration and the serviceability concerns due to lateral seismic loads deflection; as well as, the collapse concerns of the building under maximum expected earthquake shakings and differential vertical shortening of shear walls and columns stemming from time dependent deformations.

The gravity force resisting system of the building consists of slabs, columns and shear walls. Depending on architectural usage and span between structural members, the basement levels floor system is selected as cast-in place reinforced concrete flat plate slab for the tower part and beam and slab floor or cast-in place reinforced concrete flat plate slab for podium part. The slab thickness of basement levels is selected as 20-30 cm according to vertical load analysis results. The tower floor framing system is typically 20-26 cm cast-in place reinforced concrete flat plate slabs sitting on surrounding RC beams, (1.0 m x 0.6 m), supported by perimeter columns and shear walls. Reinforced concrete columns are used at basement levels and between L28-L54 upper ground levels. The dimension of the RC columns are 1.6 m x 1.6 m at the basement levels under the tower part and decrease to 1.4 m to 1.4 m at the upper basement level. The dimension of columns are 1.1 m x 1.1 m at the L28 upper ground level and gradually reduce to 0.7 m x 0.7 m at the top story. The dimension of RC columns are 0.9 m x 0.9 m at the basement level under the podium part. However, composite columns with dimensions 1.1 m x 1.1 m is selected at the ground level and between L1-L27 at the upper ground level in order to satisfy design objectives such as the axial force ratio on columns and architectural reasons.

The lateral load resisting system of the building consists of slabs, beams, columns, shear walls, and one two story level outrigger system. The two-story height outrigger system, reinforced concrete trusses with a group of strong beams are located between levels L29-L31 at the upper ground level, which is presented in Figure 2-b. Totally 10 outrigger trusses, 3x2 of them in the North-South direction and the others in the East-West direction are constructed in order to withstand lateral loads by integrating effectively peripheral columns with core shear walls. The dimension of outrigger system is 1.2 m x 0.75 m for diagonal and at the upper part of beams (pink color members) and 1.0 m x 0.75 m at the lower part of beams (black color members), respectively. The thickness of shear walls at the core is 1.1 m at the lower stories and it is gradually reduced to 0.6 m at the top stories. Core shear walls are connected with coupling beams distributed in floor plan as required by architectural need and design objectives. A
varied number of reinforced concrete and composite coupling beam section is utilized in order to provide sufficient resistance under lateral loads. Thickness of perimeter shear walls embedded in the soil in the North, South and East side of the basement levels are selected as 40-85 cm in order to withstand the external soil pressure. A mat foundation system having a thickness 1.8 m under podium footprint and 4.8 m under tower footprint is arranged in order to withstand the design forces.

3.2 Material Properties and Gravity Loading

Selected concrete classes at the tower columns and core wall shear walls and coupling beams is 60 MPa and 40 MPa for all other members. Structural steel is S460 with yield strength $f_{y}=460$ MPa for composite columns and coupling beams, and reinforcing steel is S420 with yield strength of $f_{y}=420$ for all other members. Gravity loads acting on the building are selected according to TS-498 and ASCE 7-10.

3.3 Wind Tunnel tests

Wind tunnel tests have been conducted in order to determine wind loads by constructing a 1/400 scale model of the building with and without the proximity. According to the wind tunnel test results, the critical wind speed for the building is predicted as 76 m/s for the most critical wind directions. This wind velocity is a mean wind velocity (10-minute average). It is more than 2 times higher than the 50-years mean wind velocity in tower height which is clearly higher than the safety criterion of 1.25 according to EN 1991-1-4 (2005). Accordingly, the building is not endangered by vortex shedding effects.

Wind loads are calculated in two horizontal and rotational directions for 50 years return period. In addition, the horizontal peak acceleration which is at the outer corner of the top floor level and include along wind, crosswind and radial accelerations, is obtained as 2.1, 2.0 and 1.9 % g for winds of 10 years, 5 years and 2 years of return periods, respectively. The calculated horizontal peak acceleration to be taken in consideration for 10 years of return period is in tolerable range (2-2.5 % g) for office use (CTBUH,2008).

3.4 Site Specific Seismic Hazard Analysis

A site-specific probabilistic seismic hazard analysis (PSHA) has been utilized in order to develop the SLE, and MCE response spectrum and spectrally matched time-history pairs. The PSHA methodology employed in this study relies on the historical and contemporary seismicity, the neotectonic faulting structure as well as the ground-motion modelling. Site specific response spectra, the design spectra according to TEC-2007 and generated ground motion components with acceleration response spectra of 2475 years are presented in Figures 3 a, b. The scaled representative ground motion pairs are selected according to ASCE 7-10.

![Figure 3](image_url)

Figure 3. (a) Comparison of smoothed site-specific response spectra (43 and 2475 years) with the design spectrum. (b) Acceleration response spectra of generated ground motions.
4. ANALYTICAL MODELS AND ACCEPTANCE CRITERIA

A three-stage design approach has been employed for the building after the initial proportioning of structural members. In the first stage, the building has been designed according to the TEC-2017 and TEC-2007 based on capacity design principles by utilizing linear response spectrum analysis under selected design base earthquake. Linear elastic model has been improved by employing effective stiffnesses presented in Table 1 under service level earthquake (SLE) loads and minimum base shear force under design earthquake is selected as 0.03W, where W is the structure seismic weight under the combination G+0.3Q. According to TEC-2017 provisions, the axial load level on shear walls and columns obtained from the specified load combination (G+0.25Q±E) must be limited by 0.35 and 0.40, presented in equation 1 and 2, respectively. Although the provision for limiting axial load demand on columns is the same, the provision for limiting axial load demand on shear walls is reduced to 0.25 by the peer review team members. On the other hand, the diagonal outrigger members are designed under only lateral loads since the construction of these members are performed after the construction of building is completed.

Table 1. Stiffness modifiers for different design objectives

<table>
<thead>
<tr>
<th>Element Type</th>
<th>SLE (TR=43)</th>
<th>Wind (TR=50)</th>
<th>MCE(TR=2475)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC columns</td>
<td>Nd/AscIc≤0.10</td>
<td>0.30EIk</td>
<td>0.30EIk</td>
</tr>
<tr>
<td></td>
<td>Nd/AscIc≥0.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear walls</td>
<td>-</td>
<td>0.75EIk</td>
<td>0.75EIk</td>
</tr>
<tr>
<td>Beams</td>
<td>-</td>
<td>0.70EIk</td>
<td>0.70EIk</td>
</tr>
<tr>
<td>Slabs</td>
<td>-</td>
<td>0.50EIk</td>
<td>0.50EIk</td>
</tr>
<tr>
<td>Coupling Beams</td>
<td>Conventional</td>
<td>0.30EIk</td>
<td>0.30EIk</td>
</tr>
<tr>
<td>Slabs</td>
<td>Diagonal</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[
\frac{N_d}{A_{cf}f_{ck}} \leq 0.35 \quad \text{(for shear walls)} \quad (1)
\]

\[
\frac{N_d}{A_{cf}f_{ck}} \leq 0.4 \quad \text{(for columns)} \quad (2)
\]

Linear response spectrum analysis is carried out by using effective stiffnesses specified in Table 1 under generated site-specific response spectrum, shown in Figure 3, for service level evaluation (SLE, 43 years EQ) stage. P-delta effects are considered in the analysis. The considered earthquake load combinations in performance analysis are given in equation 3 and 4. Performance of the building under the 43-year earthquake is accepted as satisfactory if the flexural demand-capacity ratios of all structural members are less than 1.5 and the shear demand-capacity ratios are less than 0.7. In addition, the interstory drift ratio should be less than 0.5% in both directions but this value is specified as 0.6% in the TEC-2017. Expected material strengths are utilized for this set of analysis. Expected material strengths are 1.3fck, 1.17f_y and 1.1f_y for concrete, reinforcement steel and structural steel, respectively. Here fck and fy are the characteristic strength of concrete and yield strength of steel, respectively.

\[
1.0D + 0.25L \div E_x \div 0.3E_y \quad (3)
\]

\[
1.0D + 0.25L \div E_y \div 0.3E_x \quad (4)
\]

Nonlinear dynamic analysis is carried out for collapse prevention level under a suite of seven ground acceleration pairs which is consistent with improved site-specific maximum consider earthquake response spectrum (Perform 3D V5.0, 2011). Both material and geometric nonlinearity are taken into account in the nonlinear 3D analytical model in order to improve a more realistic model. Fiber model and lumped plasticity model is employed for shear wall members and frame type members, respectively [Budak et al., 2017]. Slab members are taken into account in nonlinear model by idealizing as equivalent beam type elements with respect to ASCE41-13. Effective stiffnesses for MCE level presented in Table 1 are used. Performance of the building under the 2475-year is accepted as satisfactory if the obtained results satisfy the strain limits given in Table 2. In addition, shear failure of any member is undesirable.
Accordingly, the maximum average shear forces obtained under the selected ground motion pairs are less than the shear capacity of the members by using expected material properties. Performance limits for maximum average transient interstory drift ratios and the maximum drift ratio in each story for each ground motion is taken as 0.03 and 0.045, respectively.

Table 2. Compressive and tensile strain limits for concrete and reinforcement steel

<table>
<thead>
<tr>
<th>Performance level</th>
<th>Shear walls, columns, beams and conventional coupling beams</th>
<th>Diagonal coupling beams</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \varepsilon_c )</td>
<td>( \varepsilon_t )</td>
</tr>
<tr>
<td>Operational Level</td>
<td>0.0025</td>
<td>0.006</td>
</tr>
<tr>
<td>Immediate Occupancy</td>
<td>0.0035</td>
<td>0.01</td>
</tr>
<tr>
<td>Life Safety</td>
<td>0.01</td>
<td>0.03/0.02*</td>
</tr>
<tr>
<td>Collapse Prevention</td>
<td>0.015</td>
<td>0.045/0.03*</td>
</tr>
</tbody>
</table>

* for shear wall members

In Table 2, \( \varepsilon_c \) is the compressive strain of concrete and \( \varepsilon_t \) is the tensile strain of reinforcement steel.

Linear static analysis is carried out for service level evaluation under wind loads for 50 years return period obtained from wind tunnel test results. Effective stiffness values specified in Table 1 is performed in order to generate the 3D analytical model. In this performance level, interstory drift ratio at each story is controlled. It is taken as \( \frac{h}{500} \), where \( h \) is the story height.

5. EVALUATION OF PERFORMANCE RESULTS

Free vibration properties are calculated by employing effective stiffness values for SLE level. Eigenvalue analysis and the modal mass participation results are presented in Table 3. Due to having a parallelogram shaped floor plan, torsional effects are observed in the analysis. Some of selected performance based design results are shown below to check their compliance with pre-defined target performance levels.

Table 3. Free vibration properties

<table>
<thead>
<tr>
<th>Mode</th>
<th>Period(s)</th>
<th>( \omega ) (rad/s)</th>
<th>Mass Participation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>( U_x )</td>
</tr>
<tr>
<td>1</td>
<td>6.01</td>
<td>0.30</td>
<td>30.38</td>
</tr>
<tr>
<td>2</td>
<td>4.15</td>
<td>0.23</td>
<td>22.52</td>
</tr>
<tr>
<td>3</td>
<td>3.33</td>
<td>0.00</td>
<td>0.06</td>
</tr>
<tr>
<td>4</td>
<td>1.74</td>
<td>0.07</td>
<td>7.36</td>
</tr>
<tr>
<td>5</td>
<td>1.26</td>
<td>0.00</td>
<td>0.36</td>
</tr>
<tr>
<td>6</td>
<td>0.95</td>
<td>0.03</td>
<td>3.05</td>
</tr>
</tbody>
</table>

Figure 4-a presents the axial load level on shear wall P3 labelled in Figure 2-a under design earthquake level. The thickness of the shear wall P3 like other shear walls at the base is controlled by axial load level on the shear wall \( (N_d/f_c^*A_c<0.25) \). Shear wall thicknesses are gradually reduced up to top floors by satisfying axial load level without any sudden loss of rigidity between floors, resulting in construction problems etc. This is one of the good starting point in order to determine the thickness of shear walls. Columns dimensions are also defined in such a way. On the other hand, as it is mentioned above for
wind tunnel test, the calculated horizontal peak acceleration to be taken in consideration in 10 years of return period is almost reached at the upper limit value which is in tolerable range (2-2.5 % g) for office usage. Accordingly, it can be inferred from these results that the limit state of axial load level on shear walls at the base under the design earthquake level with wind load of 10 years is controlling the stiffness of the structure.

Figure 4. (a) Axial load level on shear wall P3 (b) Moment D/C ratio of shear wall P3 under SLE level

Figure 4-b and 5-a present the moment and shear demand capacity (D/C) ratio of shear wall P3 under SLE shaking. Performance of the building under the 43-year earthquake is accepted as satisfactory if the flexural demand-capacity ratios of all structural members are less than 1.5 and the shear demand-capacity ratios are less than 0.7. The results show that moment and shear D/C ratio shear wall P3 under the 43-year earthquake is satisfied. Figure 5-b shows that the maximum average shear forces obtained under the selected ground motion pairs are less than the shear capacity of the members by using the expected material properties.

Figure 5. (a) Shear force D/C ratios of shear wall P3 under SLE level (b) Shear force D/C ratios of shear wall P3 under MCE level

Figure 6 (a) and 6 (b) show the maximum average axial compressive and tensile strain with the maximum axial compressive and tensile strain in each story at each edge of the shear wall (edge I and J) for each ground motion. The results obtained from nonlinear dynamic analysis under MCE level show that the obtained maximum average axial compressive and tensile strain which is -0.0015 and +0.0035, respectively, satisfies easily pre-defined performance levels which are defined in Table 3.
Figure 6. Axial compressive ($\varepsilon_{cs}$) and tensile strain ($\varepsilon_{ts}$) at (a) edge I and (b) edge J of shear wall P3 under MCE shaking.

Figure 7 (a) and 7 (b) show the calculated interstory drift ratio under the wind loads of 50 years, SLE and MCE shakings. The results show that all of these results satisfies the pre-defined performance level specified in Section 4. Although the base shear obtained under SLE shakings is higher than the base shear obtained under wind loads of 50 years, the inter story drift ratio in X direction under wind loads is higher due to torsional wind loads.

As a result of dynamic analysis under MCE level, the most nonlinear deformations mainly occur in the diagonal outrigger members. Outrigger members are subjected to both high tensile and compressive forces under MCE shaking. According to analysis result, an average tensile strength of 21340 kN and an average compressive force of 59020 kN develop in the most critical outrigger members. The outrigger members reached their ultimate tensile strength capacity and yielded. The occurred tensile strain is 0.06, which is less than the ultimate strain capacity of 0.1 for reinforcement steel grades. Moreover, the compressive forces remain below the buckling capacity of the outrigger members (88710 kN).

Figure 7. (a) Comparison of inter story drift ratio in X direction under SLE level ($\Delta x_{EQ43Y}$) with wind loads of 50 years ($\Delta x_{W50Y}$), (b) maximum transient inter story drift ratios for each ground motion with maximum mean transient interstory drift ratios ($\Delta x_{\text{mean}}$) obtained from a set of MCE shakings in X direction.
6. CONCLUSIONS

In this study, the seismic design of a tall building in Istanbul is explained according to the performance based design approach. The analysis results show that the building satisfies the pre-defined target performance levels in accordance with the provisions of TEC-2017. In this sense, this study presents the first real case implementation of the new Turkish Earthquake Code for the design of tall buildings in severe seismic zones in Turkey.

7. ACKNOWLEDGEMENTS

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