PERFORMANCE BASED ENGINEERING
FOR THE “BANCO DE LA NACIÓN”
TALLEST TOWER IN LIMA, PERU

L. Romero¹, C. Casabonne² and E. Olivares³

ABSTRACT

Opened in June 2016, the 30-story, 135 m high “Banco de la Nación” Tower is the new tallest building in Lima, Peru; one of the highest seismic regions in the world with seismic events prone to overcome an 8 Mw earthquake. This building is the new headquarter of the National Bank of Peru.

This paper presents a summary of the seismic design by code and its assessment using performance based seismic design (PBSD) approach. The main lateral resisting structural system of the building for seismic actions is a cast-in place reinforced concrete (RC) central core; and the secondary lateral structural system consists of perimeter RC frames. The building has a raft foundation which extends to the entire basement footprint. Complementary damping system of fluid-viscous dampers is provided in the short direction in order to improve the comfort under earthquake actions and to get a similar seismic performance in both directions.

The PBSD approach follows the “Tall Buildings Initiative, Guidelines for Performance-Based Seismic Design of Tall Buildings, 2010”, document developed by Pacific Earthquake Engineering Research Center (PEER). The seismic assessment using PBSD approach is based on non-linear response history analysis (NLRHA), with 14 pair spectrum-matched ground motions, which are representative of subduction earthquakes. A summary of the key results is presented and discussed. After the completion of the project an additional evaluation was done following the procedures in the “An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region”. This project represents an example of the current and future development of high-rise construction in Peru, where PBSD approaches are growingly applied in current projects under developing.

¹Senior Structural Engineer, GCAQ Ingenieros Civiles S.A.C., Lima, Perú (email: luisromero@gcaq.com.pe)
²CEO, GCAQ Ingenieros Civiles S.A.C., Lima, Perú (email: carloscasabonne@gcaq.com.pe)
³Structural Engineer, GCAQ Ingenieros Civiles S.A.C., Lima, Perú (email: eduardoolivares@gcaq.com.pe)

Performance Based Engineering for the “Banco de la Nación” Tallest Tower in Lima, Peru

L. Romero¹, C. Casabonne² and E. Olivares³

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Introduction

Located in San Borja district and opened in June 2016, the 30-story, 135 m high “Banco de la Nación” (BN) Tower is the new tallest building in Lima, Peru; one of the highest seismic regions in the world. This is the new headquarter of the National Bank of Peru. The construction was awarded to “COSAPI engineering and construction”, Peru, with a design-build contract. COSAPI

¹Senior Structural Engineer, GCAQ Ingenieros Civiles S.A.C., Lima, Perú (email: luisromero@gcaq.com.pe)
²CEO, GCAQ Ingenieros Civiles S.A.C., Lima, Perú (email: carloscasabonne@gcaq.com.pe)
³Structural Engineer, GCAQ Ingenieros Civiles S.A.C., Lima, Perú (email: eduardoolivares@gcaq.com.pe)

lead the engineering team integrated among others by ARQUITECTONICA, in charge of the Architecture and GCAQ Civil Engineers from Lima-Peru as the structural engineering designer. The total project cost is approximately US$ 150 million. The construction team included BOUYGUES from France. This project represents an example of the current and future development of high-rise construction in Peru, where concrete is the preferred construction material for buildings.

The paper presents an overview of the design, with emphasis in seismic design of the central core of the tower. The paper describes the initial design by prescriptive Peruvian code RNE [1] and ACI 318-11 [2], and in a final stage, its seismic assessment using the current procedures in Performance Based Seismic Design (PBSD) applied to tall buildings. In addition, it discusses some of the shortcomings in the current codes procedures for shear design of structural walls.

**Site Seismicity**

Over the past 500 years, the Peruvian coast has been hit by numerous destructive earthquakes historically documented. The main source of seismic events affecting this region is the subduction of the Nazca plate beneath the South American plate, which generates large-scale events that can overcome a magnitude of 8 on the moment scale Mw. A specific Seismic Hazard Study [4] was developed for this project, with peak ground accelerations (PGA) of 0.18g, 0.23g, 0.41g, 0.53g and 0.66g, for earthquakes with return periods of 43 years (SLE), 100 years, 475 years (DBE), 1000 years and 2475 years (MCE), respectively. Also, design spectrums were determined for different return periods for rock and soil site conditions, as shown in Figure 1.

![Figure 1](image.jpg)

Figure 1. (a) Design spectrums. (b) Target spectrum, spectrums of 28 spectrum-matched ground motions for the MCE hazard level for site class S1 in Peruvian Code or C in ASCE 07-10 [3].

**Structural System Description**

The tower has 30 office stories above grade and 4 basements levels, with a total building area of 66 000 square meters. The typical floor-to-floor height is 4.00 m in the top 23 stories and 5.00 m in the first 7 stories. It was designed and built in reinforced concrete.

**Geotechnical Conditions and Foundation System**

The soil is a dense conglomerate of alluvial deposits that underlies most of the city down to the base rock around a depth of 200 m to 300 m, and increases its density with depth. The geotechnical report [5] indicates a soil allowable capacity of 8.0 kg/cm2 at the foundation depth and a 700 m/s
shear wave velocity. This soil is classified as Class “S1” in the RNE [1] and a soil factor S=1.00 or Class “C” in ASCE 07-10 [3]. The bearing stratum is the dense conglomerate. The water table is found at around 60 m depth. The foundation is a mat of variable thickness that covers the whole basement footprint. Thickness varies from 2.5 m under the tower footprint and 1.0 m in the perimeter. The lower basement level is -15.50 m; the bottom of excavation is approximately at -18.5 m. There is a 500 mm thick wall in the basement perimeter.

**Gravity System**

Above grade, the building has perimeter frames whose columns have an outward inclination in the East-West direction on each side of the tower. (See dimensions in Table 1.). The tower floor systems are post-tensioned slabs without beams to connect perimeter columns to the core. Slab thickness is 225 mm at typical average spans up to 11.00 m, and for longer spans thickness range from 250 mm to 400 mm. The basement floors are 200 mm thick slabs also post-tensioned.

**Primary Lateral Load Resisting System**

The primary lateral load resisting system is the central core with coupled shear walls plus the perimeter special moment frames. As the building stiffness is provided basically by the core, as shown in item 4, it could be considered as an essentially core wall building. The thickness of core walls in the X direction varies along the height and in the Y direction have a constant 600 mm thickness. Table 1 shows the structure dimensions and concrete strength for all members in the different floor levels.

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>Core Wall</th>
<th>Coupling Beams BxH (mm)</th>
<th>Tower Columns BxH (mm)</th>
<th>Perimeter Beams BxH (mm)</th>
<th>Concrete Strength f’c (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flange thickness (mm)</td>
<td>Web thickness (mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B4-B3</td>
<td>1150</td>
<td>700</td>
<td>1150x800</td>
<td></td>
<td>Variable</td>
</tr>
<tr>
<td>B1</td>
<td>1150</td>
<td>700</td>
<td>1150x2000</td>
<td>Variable</td>
<td>600</td>
</tr>
<tr>
<td>1F-4F</td>
<td>1150</td>
<td>600</td>
<td>1150x950</td>
<td>Variable</td>
<td>500</td>
</tr>
<tr>
<td>5F-10F</td>
<td>1150</td>
<td>600</td>
<td>1150x950</td>
<td>750x800 (Y direction)</td>
<td>420</td>
</tr>
<tr>
<td>11F-16F</td>
<td>1000</td>
<td>600</td>
<td>1000x950</td>
<td>600x800 (X direction)</td>
<td>350</td>
</tr>
<tr>
<td>17F-21F</td>
<td>850</td>
<td>600</td>
<td>850x800</td>
<td></td>
<td></td>
</tr>
<tr>
<td>22F-26F</td>
<td>700</td>
<td>600</td>
<td>700x800</td>
<td></td>
<td></td>
</tr>
<tr>
<td>27F-30F and Top</td>
<td>600</td>
<td>600</td>
<td>600x800</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Supplementary Resisting System**

An energy dissipation system of fluid-viscous dampers by Taylor Inc. was incorporated in the 8 upper floors of the Y direction perimeter frames. Its primary function is to improved serviceability and to get similar seismic drifts in both directions. It is not for issues of strength, since all the seismic resistance has been relied upon the concrete structure and thus, we have not considered it in the seismic assessment by PBSD methodology discussed below. Sixteen dampers were provided in the short direction (eight on each lateral side) shown in Figure 2 (d) and (f).
Design Methodology

The structural design is based on a prescriptive methodology according to current codes. The reference code is the Peruvian Code “Reglamento Nacional de Edificaciones” RNE [1] and this was complemented with US Codes: ACI 318 [2] and ASCE 7-10 [3]. This methodology will be referred to as Code Design, and has been the main procedure to determine the strength and stiffness requirements.

In the final stage of the design, the tower designed by Code Design procedures is evaluated following the Performance Based Seismic Design (PBSD) approach, in accordance with guidelines of the PEER Report 2010 [6] and ASCE 41-13 [7]. The structure was evaluated for two seismic levels: The Service Level (SLE) and the Maximum Considered Earthquake level (MCE).

![Figure 2. “Banco de la Nación”: (a) Finished tower view; (b) Elevation; (c) Structural 3-D ETABS model; (d) Damping system arrangement (e) Typical floor framing (f) Detail of dampers.](image)

Code Design

The code design is based on a modal spectrum analysis as prescribed by RNE [1]. Where the design earthquake, is defined as a 475-year return period earthquake, which is characterized with a 5.0% critical damping design spectrum. The seismic parameters calculated and seismic loads are shown in Table 2. For the spectrum analysis we modified the 5% design spectrum using the B1 and B2 factors from ASCE 41-13 [7] to get a 2.5% critical damping design spectrum.
### Table 2. Seismic Design Criteria by Code and Seismic Analysis Summary

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Importance Factor</td>
<td>U = 1.3</td>
</tr>
<tr>
<td>Zone Factor</td>
<td>Z = 0.41</td>
</tr>
<tr>
<td>Site Class, Site Class Coefficient</td>
<td>S1, S = 1.0</td>
</tr>
<tr>
<td>Lateral System</td>
<td>Building Frame, Special Reinforced Concrete Shear Walls</td>
</tr>
<tr>
<td>Response Modification Coefficient</td>
<td>R = 6.0</td>
</tr>
<tr>
<td>Building Period</td>
<td>$T_x = 3.9 \text{ s}; T_y = 6.0 \text{ s}$</td>
</tr>
<tr>
<td>$C_x = ZUCS/R$</td>
<td>$C_{xx} = 0.023; C_{sy} = 0.015$</td>
</tr>
<tr>
<td>$C = 2.5T_p/T$</td>
<td>$C_x = 0.258; C_y = 0.167$</td>
</tr>
<tr>
<td>$C_{\text{min}} = 0.125R$</td>
<td>$C_{\text{min}} = 0.75 \leftarrow \text{governs}$</td>
</tr>
<tr>
<td>Seismic Response Coefficient</td>
<td>$C_{xx} = 0.067; C_{sy} = 0.067$</td>
</tr>
<tr>
<td>Seismic Weight</td>
<td>W = 90500 t</td>
</tr>
<tr>
<td>Design Base Shear</td>
<td>$V_x = 0.8C_{xx}W = 4825 \text{ t}; V_y = 0.8C_{sy}W = 4825 \text{ t}$</td>
</tr>
</tbody>
</table>

### Analysis Model and Seismic Analysis Results

A 3-D elastic finite element model that conforms to RNE, was used to perform a linear modal response spectrum analysis using ETABS [8] software. The structure was modeled from the foundation to the roof levels. The model considers the columns fixed at the bottom and the core walls and perimeter walls pinned at their base. The foundation is not included in the model. The lateral resisting elements were modeled considering the modified stiffness shown in Table 3, taken from ATC-72 [9] and ASCE 41-13[7]. The ground motion is applied at the top foundation level. Table 4 shows the first three vibrations modes. The structure was dimensioned to meet the 0.01 drift limit by ASCE 07-10 [3] in both directions; this limit is more restrictive than the 0.007 drift limit by RNE [1], which uses gross stiffness.

#### Table 3. Effective stiffness in DBE Code Design

<table>
<thead>
<tr>
<th>Element</th>
<th>Flexure</th>
<th>Element</th>
<th>Flexure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Walls</td>
<td>0.50 $E_c I_g$</td>
<td>Columns</td>
<td>0.50 $E_c I_g$</td>
</tr>
<tr>
<td>Coupling Beams</td>
<td>0.20 $E_c I_g$</td>
<td>Perimeter Beams</td>
<td>0.35 $E_c I_g$</td>
</tr>
</tbody>
</table>

Note: Gross sections were used for axial and shear stiffness.

#### Table 4. Period and mass participation summary

<table>
<thead>
<tr>
<th>Vibration Mode</th>
<th>Period (sec.)</th>
<th>Mass Participation (%)</th>
<th>Dominant Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>1</td>
<td>5.9</td>
<td>90</td>
<td>0.01</td>
</tr>
<tr>
<td>2</td>
<td>4.0</td>
<td>0.02</td>
<td>95</td>
</tr>
<tr>
<td>3</td>
<td>3.5</td>
<td>0.05</td>
<td>0.06</td>
</tr>
</tbody>
</table>

### Code Design Summary

The core walls were designed for shear and moment as prescribed in RNE [1] and ACI 318 [2], complemented with the capacity design approach as delineated by Paulay and Priestley [10]. Shear design forces are 2.2 times those from analysis to consider the shear amplifications reported by authors as Adebar [14]. Concrete confining is provided per ACI-318 for special structural walls.
Service Level Earthquake (SLE) Seismic Evaluation

The service level evaluation was performed to comply with that required by PEER [6] in order to demonstrate that for moderate earthquake the structure remains essentially elastic with only some minor yielding of ductile elements. The SLE earthquake was defined as a 43 year return period and it was represented in the form of a site-specific 2.5% damped acceleration response spectrum; it was increased by a factor of 1.30 in order to achieve compatibility with Code Design criteria for Important Structures. A 3-D elastic finite element model with ETABS [8], was used to perform a linear modal response spectrum analysis to obtain the service demands, which has the same features as in the code design model; which uses effective stiffness, see Table 5, from ATC-72 [9] and ASCE 41-13 [7]. The SLE Load Combinations (for strength and drift demands) are: 1.0D + 0.50L ± 1.0Ex ± 0.3Ey and 1.0D + 0.50L ± 0.3Ex ± 1.0Ey. Where D is dead load; L is the unreduced live load (it is taken as 50% as per important structures); Ex and Ey are the serviceability response spectrum in X and Y direction, respectively.

Table 5. Effective Stiffness in SLE Evaluation

<table>
<thead>
<tr>
<th>Element</th>
<th>Flexure</th>
<th>Element</th>
<th>Flexure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Walls</td>
<td>0.75 E_c I_g</td>
<td>Columns</td>
<td>0.50 E_c I_g</td>
</tr>
<tr>
<td>Coupling Beams</td>
<td>0.30 E_c I_g</td>
<td>Perimeter Beams</td>
<td>0.50 E_c I_g</td>
</tr>
</tbody>
</table>

Note: Gross sections were used for axial and shear stiffness.

SLE Acceptance Criteria

i) The demand to capacity ratios for all elements of the lateral resisting system shall no exceed 1.5. (Where strength capacities were calculated in accordance with ACI 318 [2]); ii) Story drift shall not exceed 0.005 of the story height in any story; and iii) The shear stress in the core walls shall be limited to $4 \cdot \sqrt{f'_{c}}$.

SLE Analysis Results and Seismic Evaluation

Figure 3 shows drift demands in two directions; both are lower than the 0.005 limit. Figure 4 shows the strength evaluation for shear walls and coupling beams, both acceptance criteria are satisfied.

Figure 3. SLE Maximum Interstory Drifts.
Maximum Considered Earthquake (MCE) Seismic Evaluation

This evaluation is to verify that the structure has an adequate safety against collapse under a severe 2475-year mean return period earthquake called Maximum Considered Earthquake or MCE. The evaluation is based on non-linear response history analysis (NLRHA), with 14 pair spectrum-matched ground motions (see Figure 1b), this uses a non-linear model created in Perform-3D [11]. The non-linear model includes inelastic member properties for core wall flexural behavior and coupling beams. Core wall shear behavior is assumed to remain elastic. The inelastic modeling of core walls uses non-linear vertical fiber elements representing the expected behavior of the concrete and reinforcing steel. The MCE Load Combination is: 1.0D + 0.50L + E; where D is dead load; L is the unreduced live load (50% as per important structures) and E is the earthquake load (two components ground motions records).

MCE Acceptance Criteria

For Ductile Actions and Drift: mean demands were used; capacity was calculated using expected material properties and strength reduction factors set to 1.0. For Brittle Actions: 1.5 times the mean demands were used; capacity is based on specified material strengths and no reduction factors.

Interstory drift

Maximum interstory drifts are illustrated in Figure 5, the demanded drifts are all lower than 0.03, which is the limit prescribed by PEER [6].
Core Wall Shear

MCE core wall shear demands on the X and Y directions are shown in Figure 6. Shear capacity ($\phi V_n$) was calculated according to ACI-318 [2] using a factor $\phi=1.00$. Shear demands were calculated as a mean plus one standard deviation, $(1 + 1.0 \cdot \sigma) \cdot V_u \leq \phi \cdot V_n$, this is in line with the philosophy in PEER [6]. Figure 6 shows that all demand/capacity ratios (DCR) are lower than 1. At this point we present some discussion related to the shear design in ACI 318 [2] and RNE [1], which use shear demands taken directly from analysis without any modification nor shear amplification. However, many authors as Klemencic [12] have reported that, due to complex dynamic behavior of tall buildings, the shear demands can be three to four times those anticipated by a typical code design; Adebar [13] and Dezhdar & Adebar [14] reported that shear amplification has a maximum value of 2.0. For this project, it can be observed that MCE shear demands are on average 3.2 times those calculated from Code Design, these are in line with that described previously. Therefore, those shear forces calculated from analysis by code and used directly in code design may result in dangerous design.

Core Wall Strain MCE Evaluation

Tension and compression strain demands in the core walls are less than limit strains prescribed in PEER [6]. Reinforcing yielding is mainly developed at the base and a little in the middle high.
Conclusions and Recommendations

A resume of the key points of the seismic design and seismic assessment of the Banco de la Nación Tower was presented. The seismic design followed a prescriptive code design based on Peruvian Codes and ACI 318-14. The seismic assessment followed the TBI Guidelines given in PEER.

Shear design was the dominant criteria to determine the thickness of core walls. The evaluation by PBSD approach shows that MCE shear demands are in average 3.2 times those calculated from Code Design using spectrum analysis, this is in line with the reported previously for others and reflect a serious shortcoming in the design by codes as ACI 318, which prescribe to use shear forces take directly from analysis without any modification. This may turn out in an unsafe design. Therefore, the authors suggest to revise the shear design procedure of codes.

Acknowledgments

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References