STRUCTURAL DESIGN AND PERFORMANCE BASED ENGINEERING FOR THE "BANCO DE LA NACIÓN" TALLEST TOWER IN LIMA, PERU

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Abstract

Completed in August 2015, 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. This is the new headquarter of the National Bank of Peru.

This paper presents a summary of the structural design, with emphasis in the seismic design by prescriptive code design and its assessment using performance-based seismic design (PBSD) approach. The main lateral resisting structural system 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. A summary of the key results is presented and discussed. In addition, based on results of this seismic assessment, the paper presents a discussion in regard with shortcomings in wall shear design as currently prescribed in codes as Peruvian code or ACI-318, some suggestions are included.

This project represents an example of the current and future development of high-rise construction in Peru.

Keywords: Performance-Based Seismic Design; Capacity Design; High-rise Building; Core Wall

1. Introduction

Located in San Borja district and completed in August 2015, 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 and was the venue for the World Bank Group-IMF Meetings in October 2015. The construction was awarded to "COSAPI engineering and construction”, Peru, with a design-build contract. COSAPI 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 and construction, 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 and their need to be revised.

2. 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 Fig. 1.

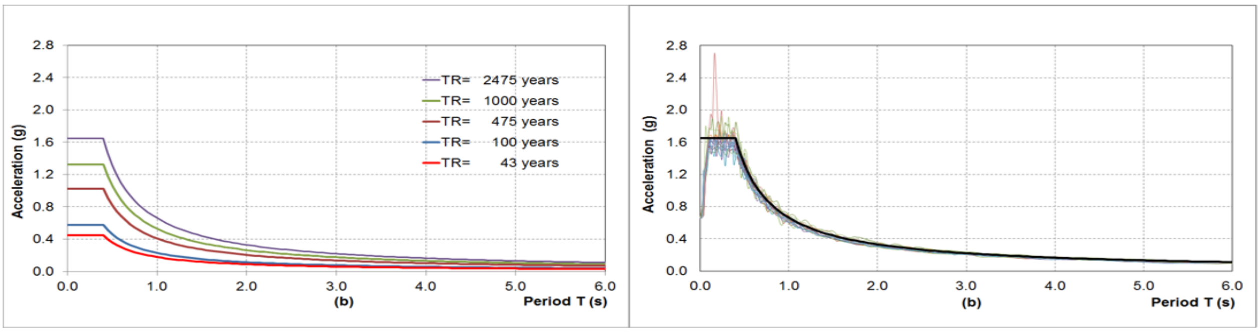

Fig. 1 – (a) Design spectrums. (b) Target spectrum and spectrums of 28 spectrum-matched ground motions for the MCE hazard level for site class S1 in Peruvian Code or C in ASCE 07-10 [3].

3. Structural System Description

The tower is a reinforced concrete building which has 30 office stories above grade and 4 basements levels, with a total building area of 66000 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 e.

3.1. 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/cm^2 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, it has 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.

3.2. 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.

3.3. 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 1 – Core walls and typical columns and beams dimensions and concrete strength

<table>
<tr><th rowspan="2">Floor Level</th><th colspan="2">Core Wall</th><th rowspan="2">Coupling Beams BxH (mm)</th><th>Tower Columns</th><th>Perimeter Beams</th><th>Concrete Strength</th></tr>
<tr><th>Flange thickness (mm)</th><th>Web thickness (mm)</th><th>BxH (mm)</th><th>BxH (mm)</th><th>f'c (kg/cm^2)</th></tr>
<tr><td>B4-B3</td><td>1150</td><td>700</td><td>1150x800</td><td rowspan="2">Variable</td><td rowspan="2">Variable</td><td rowspan="2">600</td></tr>
<tr><td>B1</td><td>1150</td><td>700</td><td>1150x2000</td></tr>
<tr><td>1F-4F</td><td>1150</td><td>600</td><td>1150x950</td><td rowspan="2">Variable</td><td rowspan="6">750x800
(Y direction)

600x800
(X direction)</td><td>600</td></tr>
<tr><td>5F-10F</td><td>1150</td><td>600</td><td>1150x950</td><td>500</td></tr>
<tr><td>11F-16F</td><td>1000</td><td>600</td><td>1000x950</td><td rowspan="4">1000x1000</td><td rowspan="2">420</td></tr>
<tr><td>17F-21F</td><td>850</td><td>600</td><td>850x800</td></tr>
<tr><td>22F-26F</td><td>700</td><td>600</td><td>700x800</td><td rowspan="2">350</td></tr>
<tr><td>27F-30F and Top</td><td>600</td><td>600</td><td>600x800</td></tr>
</table>

3.4. 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 improve 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 Fig. 2 (d) and (f).

4. 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], this code consists of various documents as NTP E060 for concrete design and NTP E030 for seismic design of buildings. Those codes were 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 in addition by ASCE 41-13 [7]. The structure was evaluated for two seismic levels: The Service Level (SLE) and the Maximum Considered Earthquake level (MCE).

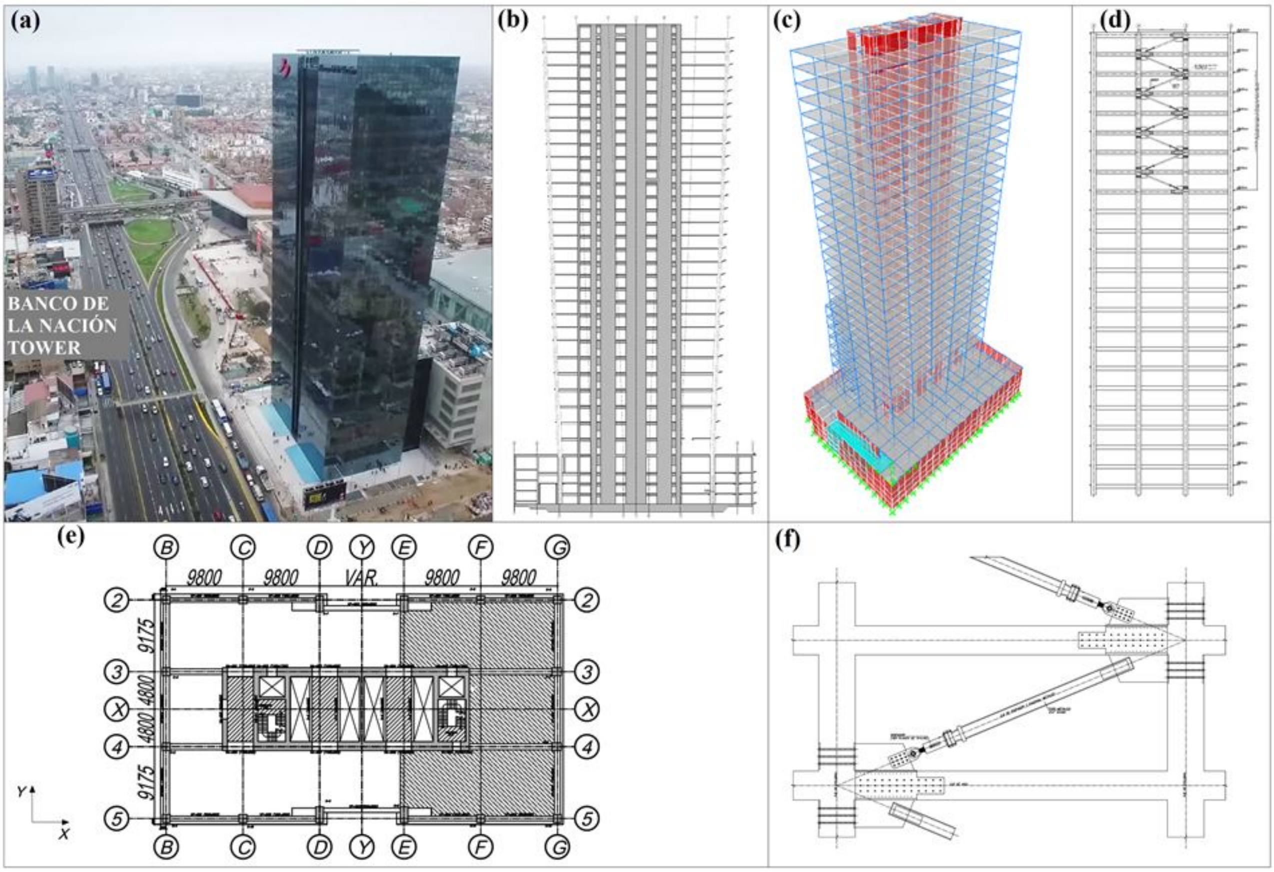

Fig. 2 – “Banco de la Nación”: (a) Finished tower view; (b) Elevation; (c) Structural 3-D ETABS model; (d) Damping system arrangement (e) Typical floor framing (f) Detail of dampers.

5. Code Design

The Code Design is based on a modal spectrum analysis as prescribed by NTP E030 in 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

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

5.1. 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 3 – Effective stiffness in DBE Code Design

Element	**Flexure**	**Element**	**Flexure**
Core Walls	0.50 E_c I_g	Columns	0.50 E_c I_g
Coupling Beams	0.20 E_c I_g	Perimeter Beams	0.35 E_c I_g

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

Fig. 3 shows drifts for the X and Y directions. 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 4 shows the first three vibrations modes.

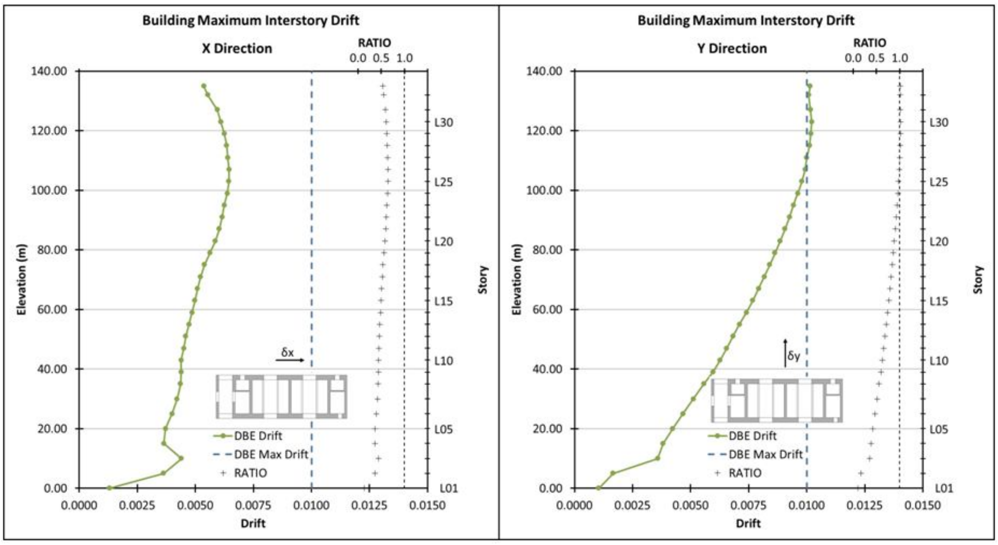

Fig. 3 – DBE Maximum Interstory Drifts

Table 4 – Period and mass participation summary

Vibration Mode	**Period (sec.)**	**Mass Participation (%)**		**Dominant Direction**
		X	**Y**	
1	5.9	90	0.01	Translation mode on X direction
2	4.0	0.02	95	Translation mode on Y direction
3	3.5	0.05	0.06	Torsional

Fig. 4 shows moments and shears along the height for the global (building) and for the core. They show that the core takes about 90% and 80% of the seismic action in terms of moments and shears, respectively. We conclude that the tower is an essentially core wall building.

5.2. Seismic Design Summary

The core shear walls were designed for shear and moment as prescribed in RNE [1] and ACI 318 [2], complemented with capacity design approach as delineated by Paulay and Priestley [10]. For this, shear design considered 2.2 times those from analysis in an attempt to take into account the shear amplifications reported by various authors like Adebar [14].

Shear design graphic is shown in Fig. 5a. Concrete confining is provided per ACI-318 considering special shear walls provisions. Fig. 5b shows the design graphic for coupling beams which use capacity also, where the Demand/Capacity ratio is permitted to go up to 1.25.

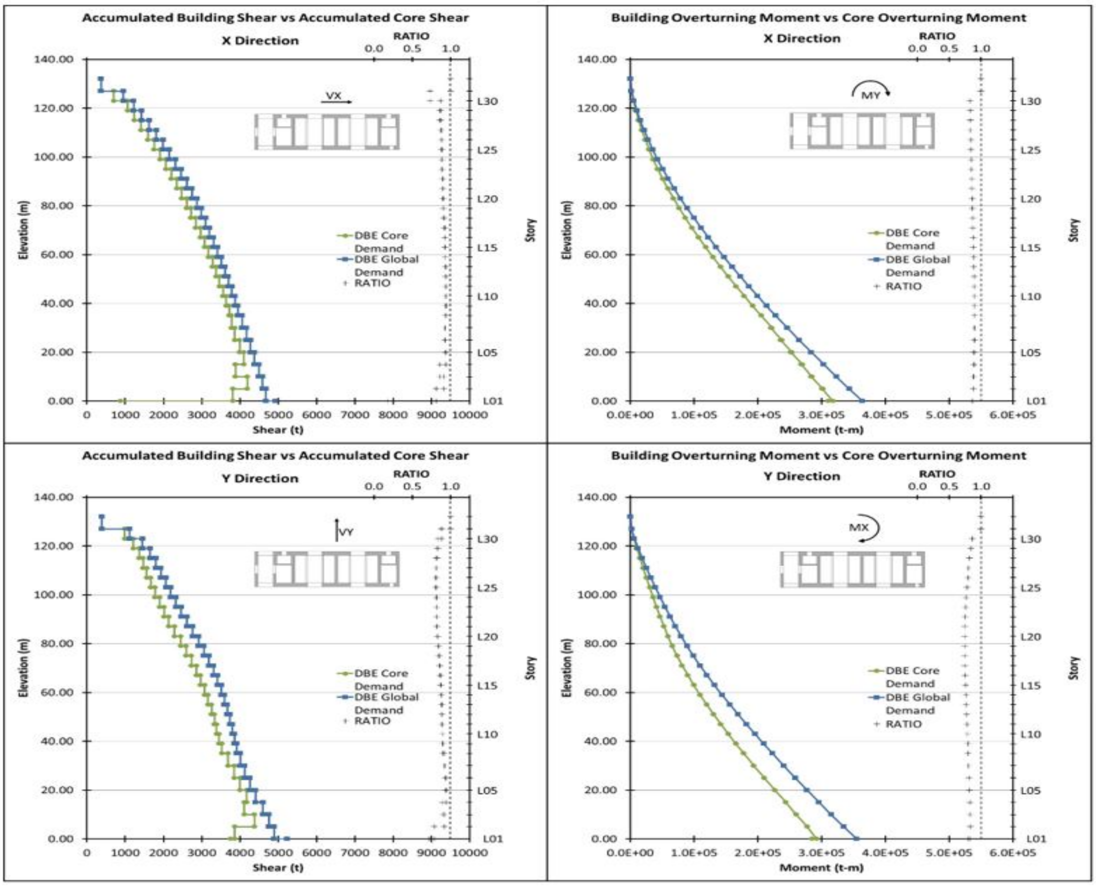

Fig. 4 – Shears (t) and Moments (t-m) for Building vs. Core.

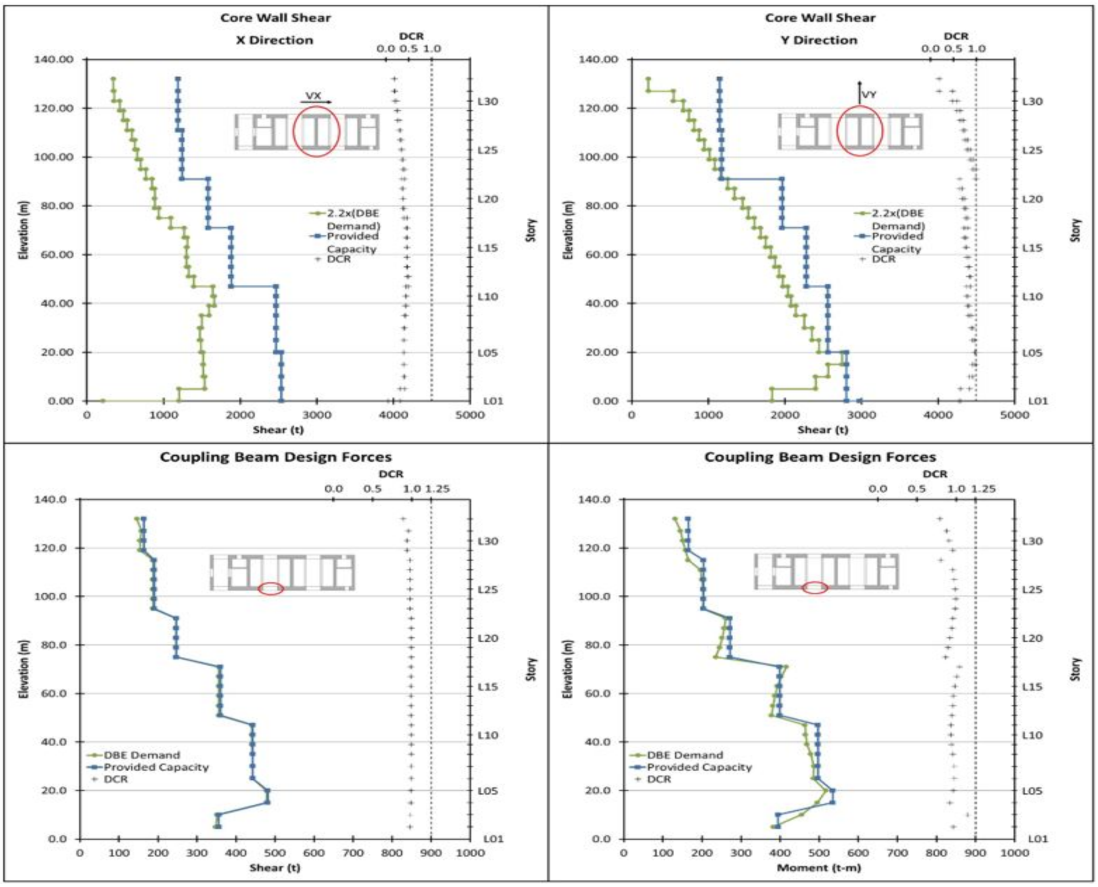

Fig. 5 – DBE Strength Design for (a) Shear Walls and (b) Coupling Beams.

6. Service Level Earthquake (SLE) Seismic Evaluation

This evaluation complies 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; the amplitude of the response spectrum 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 in order to obtain the service demands, this model has the same features as in the code design model. It uses effective stiffness as shown in Table 5, taken from ATC-72 [9] and ASCE 41-13 [7].

Table 5 – Effective Stiffness in SLE Evaluation

Element	Flexure	Element	Flexure
Core Walls	$0.75\ E_c\ I_g$	Columns	$0.50\ E_c\ I_g$
Coupling Beams	$0.30\ E_c\ I_g$	Perimeter Beams	$0.50\ E_c\ I_g$

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

The SLE Load Combinations (for strength and drift demands) are: $1.0D + 0.50L \pm 1.0E_x \pm 0.3E_y$ and $1.0D + 0.50L \pm 0.3E_x \pm 1.0E_y$. Where D is dead load; L is the unreduced live load (it is taken as 50% as per important structures); E_x and E_y are the serviceability response spectrum in X and Y direction, respectively.

6.1. SLE Acceptance Criteria

The obtained displacements and strength demands are checked against the acceptance criteria bellow:

- 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]).
- Story drift shall not exceed 0.005 of the story height in any story.
- The shear stress in the core walls shall be limited to $4 \cdot \sqrt{f'_c}$.

6.2. SLE Analysis Results and Seismic Evaluation

Fig. 6 shows drift demands in two directions; both are lower than the 0.005 limit. Fig. 7 shows the strength evaluation for shear walls and coupling beams, both acceptance criteria are satisfied.

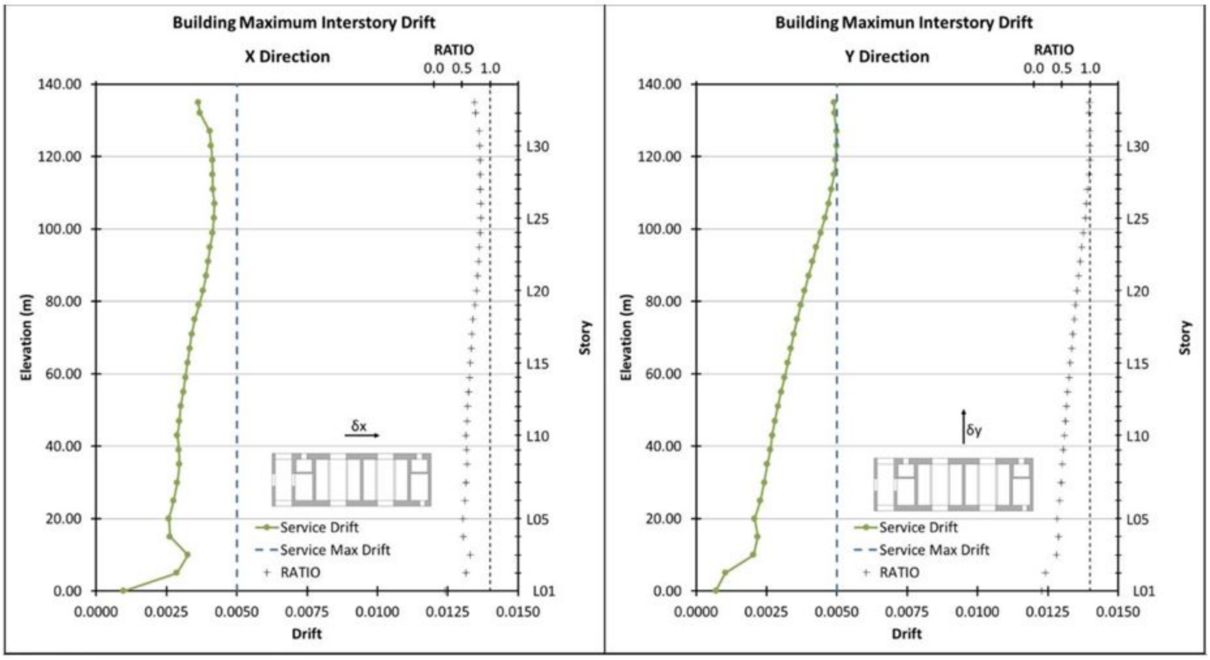

Fig. 6 – SLE Maximum Interstory Drifts.

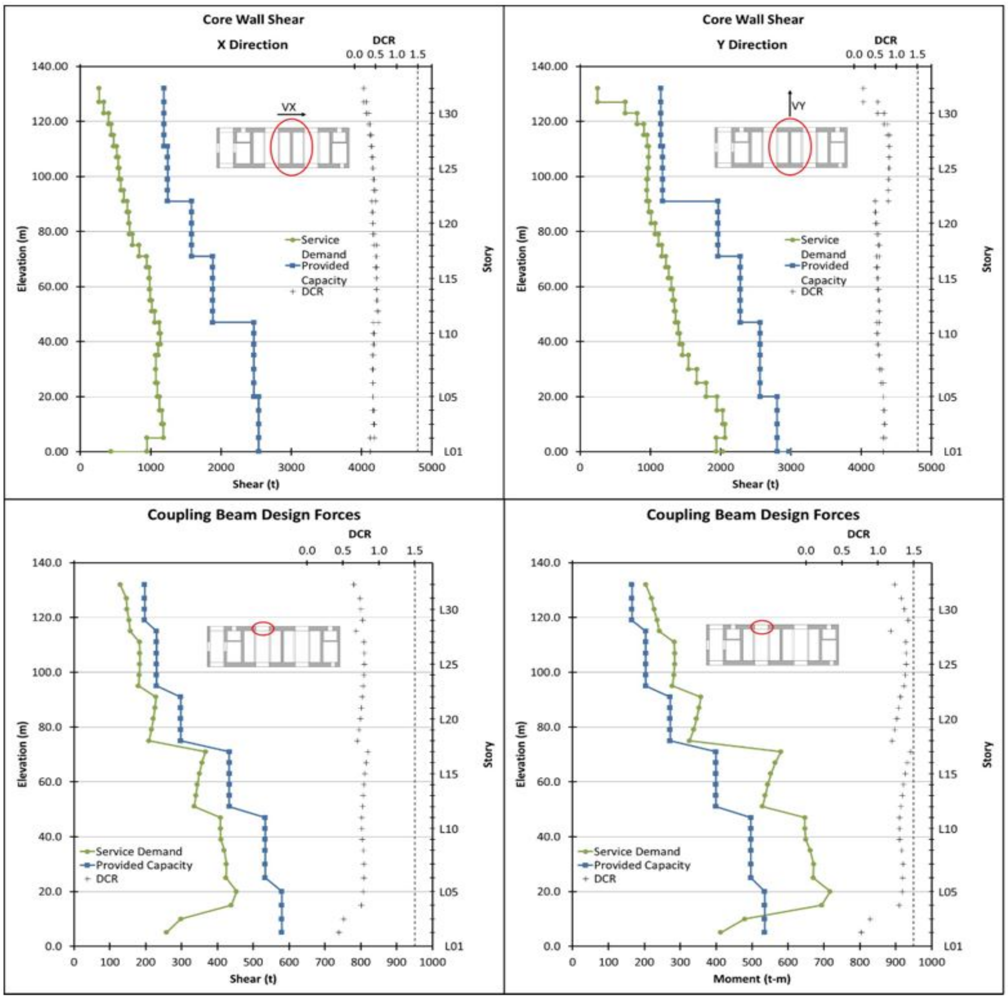

Fig. 7 – SLE Strength Evaluation.

7. 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 Fig. 1b). For this purpose, a non-linear model was 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, so it is modeled with elastic member properties. The inelastic modeling of core walls uses non-linear vertical fiber elements representing the expected behavior of the concrete and reinforcing steel; the concrete stress-strain relationship is based on Mander's model described in Paulay and Priestley [10] for confined concrete with confining ratios as required by ACI 318 [2]. Fig. 8 shows the summary of NLRHA results.

The MCE Load Combination is: 1.0D + 0.50L + E (The NLRHA uses one load combination for MCE earthquake.), where D is dead load; L is the unreduced live load (it is taken as 50% as per important structures) and E is the earthquake load (two components ground motions records).

7.1. MCE Acceptance Criteria

The acceptance criteria are summarized below:

Ductile Actions and Drift: Mean demands were used. Capacity was calculated using expected material properties and strength reduction factors set to 1.0.

Brittle Actions: 1.5 times the mean demands were used. Capacity was calculated using specified material strengths and strength reduction factors set to 1.00. This was applied for example to core wall shear.

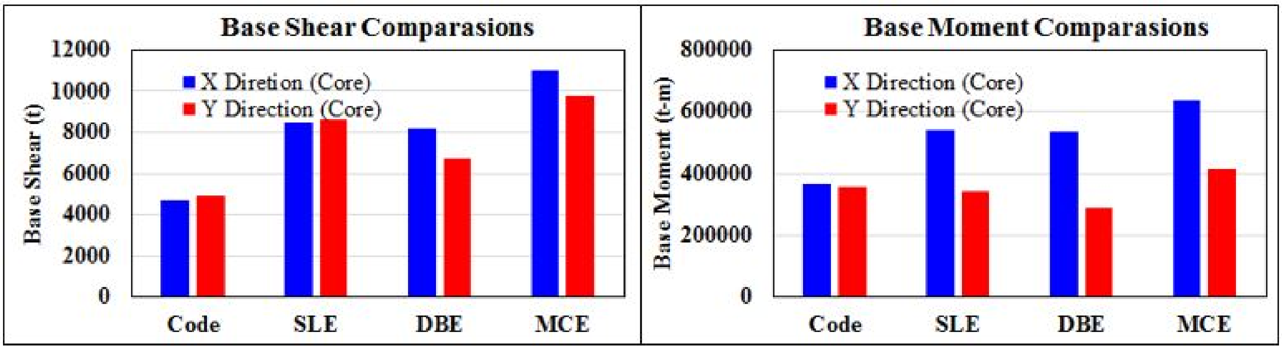

Fig. 8 – Summary of NLRHA results.

7.2. MCE Inter-Story Drift Evaluation

Maximum inter-story drifts along the building high on the X and Y directions are illustrated in Fig. 9. It can be seen that the demanded drifts are all lower than 0.03, which is the limit prescribed by PEER [6]. This is a consequence of the adopted 0.01 maximum drift in code design.

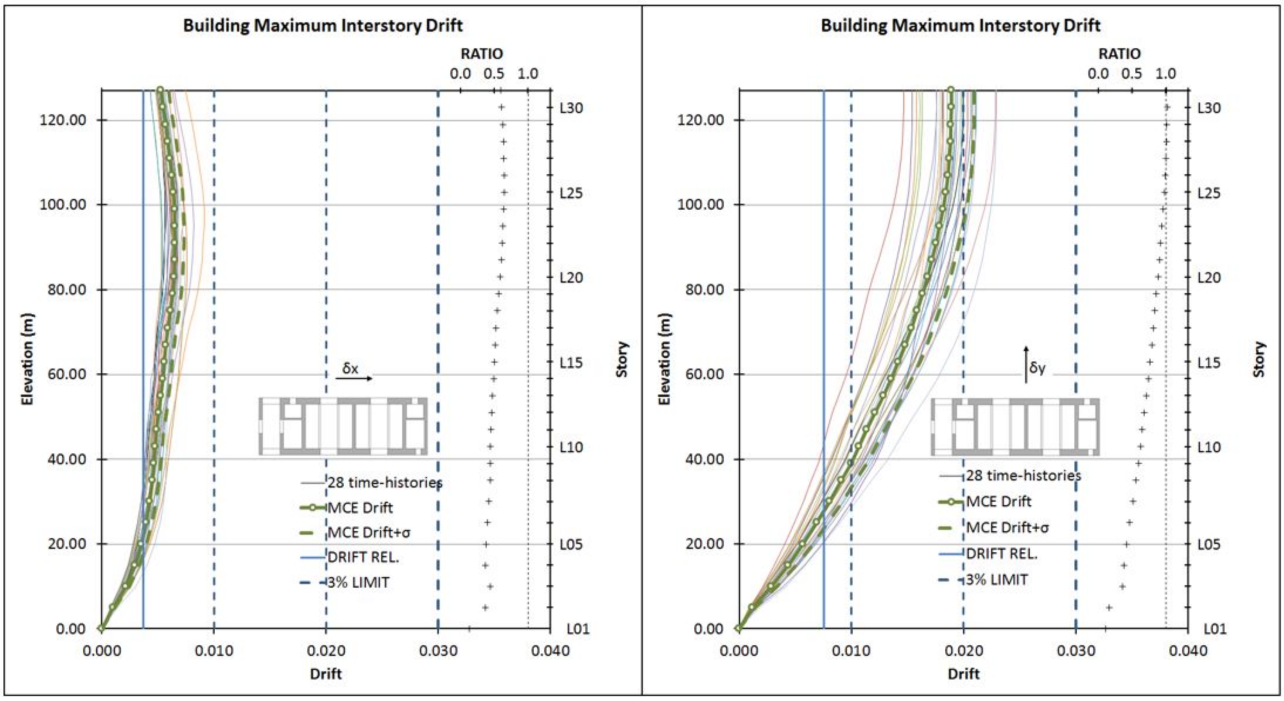

Fig. 9 – MCE Drift Evaluation.

7.3. MCE Shear Evaluation of the Core Wall

MCE core wall shear demands on the X and Y directions are shown in Fig. 10, where the DBE shears (obtained from an additional NLRHA for DBE level) are presented also. Shear capacity (ϕVn) was calculated according to ACI-318 [2] using a factor ϕ=1.00. Shear demands were calculated as a mean plus one standard deviation, $(1 + 1.0 \cdot \sigma) \cdot \overline{V_u} \leq \emptyset \cdot V_n$, this is in line with the philosophy in PEER [6]. Fig. 10 shows that all demand/capacity ratios (DCR) are lower than 1, this is due to the 2.2 factor to amplify design shears to be use in Code Design procedures.

At this point we discuss issues related to the shear design in ACI 318 [2] and RNE [1]. These codes use shear demands taken directly from analysis without any amplification, and calculate the shear capacity using a strength factor ϕ. However, many authors as Klemencic [12] have reported that, due to complex dynamic behavior of tall buildings, the MCE shear demands can be three to four times those anticipated by a typical code design. In the same line Adebar [13] and Dezhdar & Adebar [14] reported that shear amplification have a maximum value of 2.0 for DBE level. Rutenberg [15] and Adebar et al. [13] presented some procedures to take into account shear amplification and estimated more realistic shear demands to use in design. For this project, it can be observed that shear demands are in average 2.4 and 3.2 times those calculated from Code Design analysis for DBE and MCE levels, respectively; these are in line with that described before. Therefore, those shear forces calculated from analysis by code and used directly in design are on the unsafe side and may result in non-conservative design. Those findings suggest the need to incorporate in those codes some method for amplifying/modifying the shears from analysis in order to get realistic shear demands and safe design, especially since a shear failure is an undesirable failure mode.

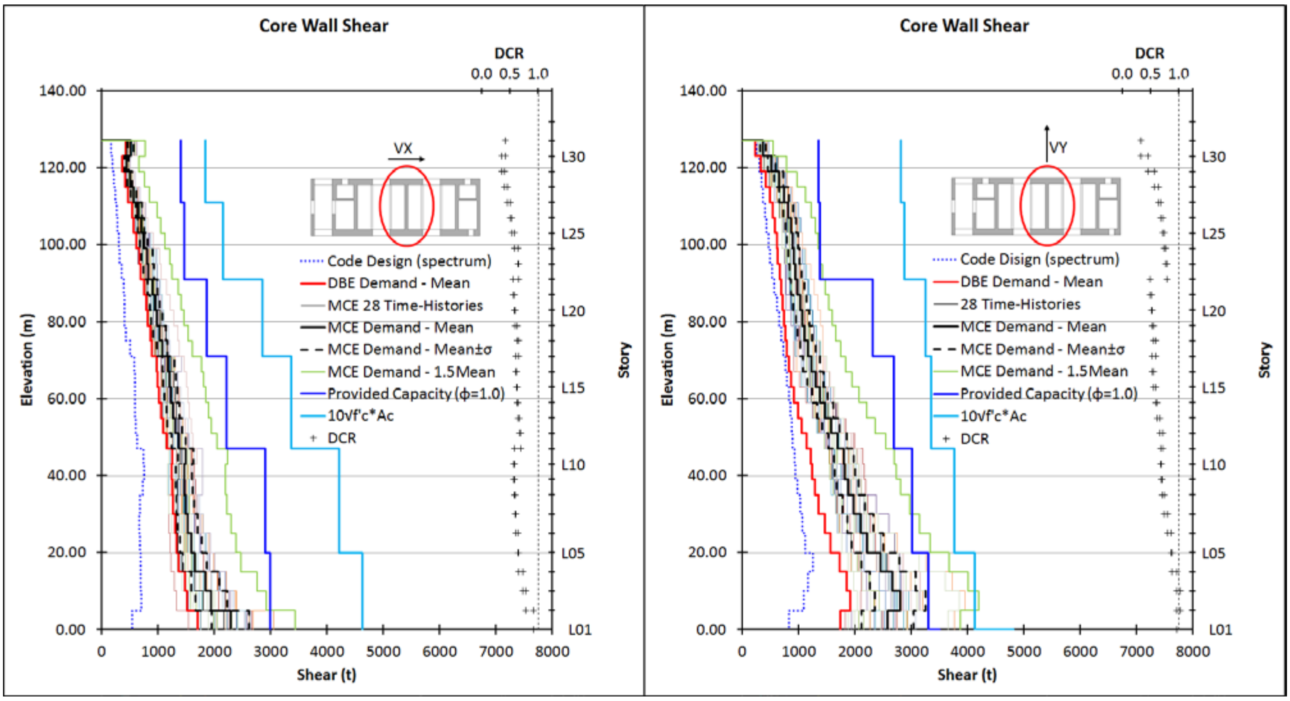

Fig. 10 – MCE core center wall shear in X and Y direction.

7.4. MCE Strain Evaluation of the Core Wall

Fig. 11 shows that all 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.

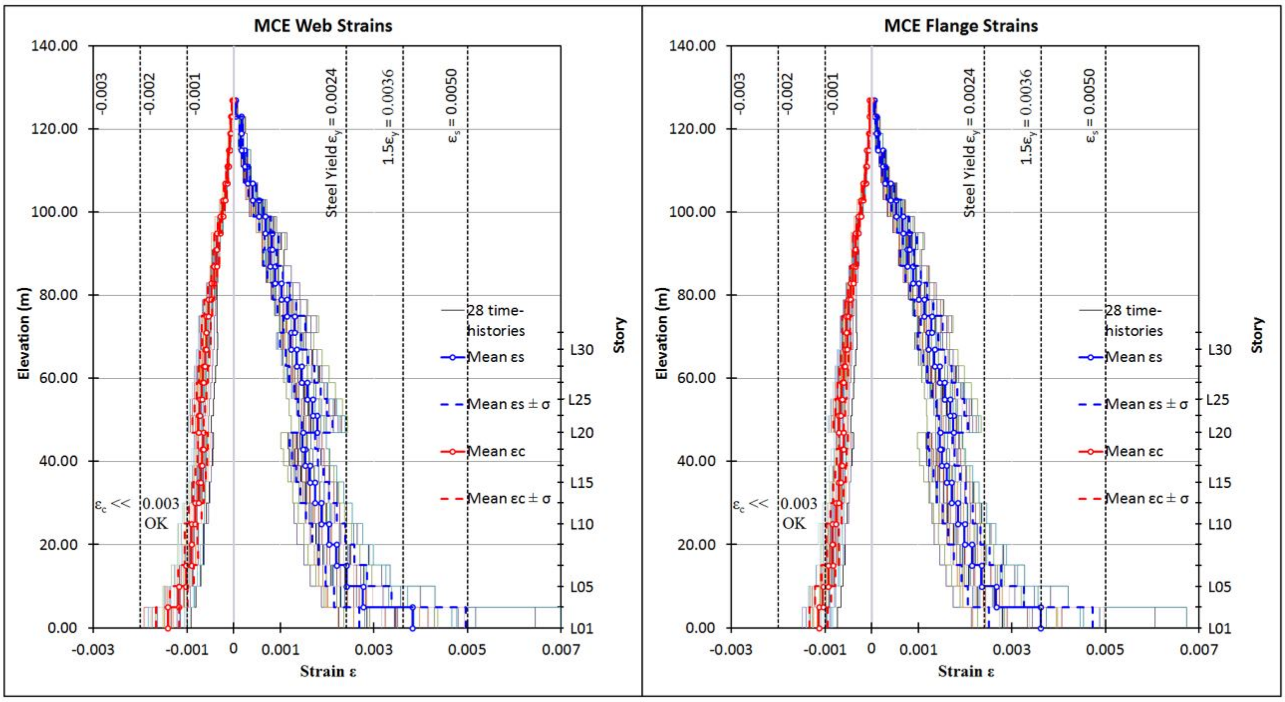

Fig. 11 – MCE core web and flange strains.

8. Construction

The building construction was completed in 15 months, complying with the clients´ requirements. It represented a new record in the Peruvian construction practice. This tower is the first post-tensioned slab building to use a self-climbing ahead formwork system for the core wall. See Fig. 12.

Type II mechanical splices (ACI-318 [2]) were widely used at all main reinforcing of the key elements as core wall, columns, perimeter beams and for the delayed slab-core wall connections. Mechanical Splices, were required to comply with the "bar breaking" criteria as a way to maintain the intrinsic bar ductility.

Another milestone of this project was in the construction of the raft foundation. This is the first big foundation pouring in one time in Peru -approximately 5600 cubic meters of self-consolidating concrete -, completed in 32-hour continuous pour. In order to avoid harmful high temperatures in the concrete mass and to limit concrete temperature in its setting process to 75^0 C, the maximum delivery temperature was limited to 23^0 C. For this, ice and cement of low hydration temperature was specified in the concrete mix.

Fig. 12 – Construction of the "Banco de la Nación" Tower.

9. 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.

Design based on sound engineering principles using a logic structural system, capacity-design approach, good drift control and good detailing, in general may have a good seismic performance under big earthquakes larger than design earthquake as has been demonstrated for this project in the evaluation by the PBSD approach.

Shear design was the dominant criteria to determine the walls' web thickness of core walls; this was addressed in the design by using a shear equal to 2.2 times the shear from spectrum analysis, and thus it was successfully verified in the evaluation using non-linear response history analysis.

The evaluation by PBSD approach shows that shear demands are in average 2.4 and 3.2 times those calculated from Code Design using spectrum analysis for DBE and MCE levels, respectively. This observation is in line with what was reported previously for various authors and reflect a serious shortcoming in the design by codes like Peruvian Codes or 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 incorporate in those codes some procedure to modify shears from analysis to arrive a safe shear design.

The application of the PBSD approach permits to identify with some confidence issues not addressed by codes. One of these is the amount of provided confining reinforcement, it could be only as much as necessary and not in strict compliance with ACI 318 in order to obtain some economy. And its application will help to make possible that many currently on-hold projects become a reality.

10. Acknowledgements

The authors wish to thank and acknowledge to COSAPI engineering and construction from Peru and Bouygues from France for continuing support during the design development, as well as our design team colleagues at GCAQ.

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