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SEISMIC DESIGN OF THE TALLEST BUILDING IN MEXICO

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Abstract

Growing urban sprawl in Mexico's largest metropolitan areas is beginning to have a negative impact on the way people work and live. The desire to be closer to work and public amenities is resulting in the need to build taller in city centers. Many cities in Mexico are seeing an increase in the number of tall buildings, including Monterrey, the second largest city in Mexico. Torre Koi, a mixed use high rise in Monterrey, at 279 meters tall and the tallest building in Mexico, is a recent example.

Until a minor earthquake was experienced in 2013, Monterrey was considered to be a non-seismic zone. The local building code in Monterrey does not cover seismic design of structures, not to mention the seismic design of tall buildings. The design team referred to the Federal Electricity Commission's *Manual de Diseño de Obras Civiles* (CFE Manual) for seismic design procedures as a prudent way to address potential seismicity. This paper will discuss the seismic analysis and design procedures employed on the high-strength concrete core and virtual outrigger structural system utilized on the tallest building in Mexico.

Keywords: Tall Buildings; Seismic Design; Response Spectrum Analysis, Virtual Outrigger System; High-Strength Concrete

1. Introduction

As urban areas become more and more crowded, the need for tall buildings is increasing due to land cost and the need to keep utilities and infrastructure condensed in a smaller area. The desire to minimize the use of automobiles inside cities to reduce pollution and hours of nonproductive activity is also driving vertical construction. The United States was the first country to start this movement, mainly in New York and Chicago during the late 19th and early 20th centuries. Recently, others like Brazil, Korea, Japan, and China are joining this trend while high-rise construction in countries such as United Arab Emirates and Saudi Arabia aim to create landmarks. In Mexico, the growing urban sprawl in many large cities, such as Mexico City and Monterrey is resulting in long commutes and lack of nearby services and infrastructure. The recession of 2009, combined with the increased urban sprawl has resulted in a trend to build taller and closer to the city centers where residents can be close to jobs and services. A recent example is Torre Koi, located in Valle Oriente, an exclusive area of San Pedro Garza Garcia, Nuevo Leon.

Much of southern Mexico and the Pacific Coast is seismically active, with more than ten earthquakes in the 21st century registering over 5.7 on the Richter scale and two registering over 8.0 since the famous 1985 Michoacán earthquake. Seismic design of building structures is not new, however, many local Mexican building codes do not provide guidance on seismic design and even fewer cover requirements for high-rise buildings. This paper presents the seismic analysis and design procedures utilized for Torre Koi.

2. Building Description

At 279 meters tall, the 69 story mixed-use tower, and centerpiece to the VAO complex, will be the tallest building in Mexico and third tallest building in Latin America when completed in late 2016. Torre Koi contains nine levels of below grade parking, a ground floor lobby, twenty levels of office space; the first fifteen floors above grade and five at the top of the tower, thirty-six levels of residential space (218 apartments and 18 penthouses), two mechanical floors at levels 21 and 62, and an amenities level with a swimming pool at level 22. Floor-to-floor heights are typically 2.85 meters at the parking levels, 4.2 meters at the lower office levels, and 4.0 meters at the residential levels. An architectural rendering is shown in Fig. 1.

Fig. 1 – Architectural Rendering courtesy of VFO Architects

3. Structural System Description

The structural system for Torre Koi responds to the architectural form of the building, while addressing the structural design requirements of a tall tower such as vertical gravity loads and large horizontal wind and seismic forces; as well as serviceability concerns of lateral wind and seismic load deflection, horizontal building acceleration due to wind and differential vertical shortening of columns and walls resulting from time dependent creep and shrinkage.

3.1 Gravity Force Resisting System

The tower floor framing is typically 25-centimeter post-tensioned cast-in-place concrete flat plate slabs with 10-centimeter thick slab bands between columns at the longer spans. The garage floor framing is typically post-tensioned cast-in-place concrete waffle slabs to match adjacent garage levels in the previously constructed phases of the VAO complex and to make efficient use of expensive materials relative to the cost of labor. The continuous slab is 7-centimeter thick with ribs having a total depth of 30 centimeters. Typical ribs are 15 centimeters wide and spaced at 1.53 meters on center. Beams 80 centimeters in width are located along the major column lines. The floor slab concrete strength varies from 50 megapascals at the lower levels to 35 megapascals at the upper levels in order to prevent the need to puddle concrete around the high strength walls and columns. Floor slabs are supported by mild steel reinforcement concrete columns that vary in size from 1-meter square to 1.6 meters x 2.0 meters. Three different concrete strengths (70 megapascals, 60 megapascals and 50 megapascals) were utilized to maximize the efficiency of the columns while minimizing their size and maximizing useable floor space.

3.2 Lateral Force Resisting System

Resistance to horizontal wind and seismic loads is provided by a central reinforced concrete core coupled with the tower columns by an indirect outrigger system comprised of a perimeter reinforced concrete belt wall and stiff slab diaphragms. The aspect ratio of the structure is 8.7:1 about the East/West axis and 5.9:1 about the North/South axis. The aspect ratio of the structural core is 19.5:1 about the East/West axis and 10.8:1 about the North/South axis at the base. The concrete core stops two stories below the top level. Lateral force resistance of the top two floors is provided by slab-column and slab-beam moment frames. The two indirect outrigger system assemblies occur between levels 21 and 22, about 40% of the height above grade, and at level 62 which is the top story of the concrete core. By utilizing all of the tower columns and the reinforced concrete core walls with an indirect outrigger system of belt walls and stiff floor diaphragms, an efficient system is realized without the need for outriggers directly connecting perimeter columns to the central core; which can impact architectural requirements and result in complex structural connections and time-dependent load transfer between the columns and core walls. The virtual outrigger system results in a reduction of seismic building period of 20%, reduction of seismic core base moment of 25% and a reduction in seismic drift of 30% in the North/South direction over a core only lateral force resisting system. Reductions in the East/West direction were approximately half of those in the North/South direction. Reductions in building responses for wind loads due to the virtual outrigger system were slightly higher than those for seismic responses. Two columns east of the core walls, linked by coupling beams to the core walls throughout the middle third height of the building, provide additional resistance against lateral seismic forces.

The north and south walls work together by concrete link, or coupling beams over door openings varying in depth from 1 meter to 2.75 meters, and span/depth ratios typically ranging from 1.5 to 3. Link beams are typically reinforced with mild steel reinforcement, however, due to high shear demand loads, a few are reinforced with structural steel plate members. The east/west, or flange walls, range in thickness from 1.05 meters at the base to 0.9 meters at the top and are 1.2-meter thick at the belt levels. The north/south, or web walls, are 0.6 meters and 0.45 meters thick with the thickness remaining constant over the building height. Three different concrete strengths (70 megapascals, 60 megapascals and 50 megapascals) were utilized to maximize the efficiency of the walls. The concrete belt walls vary from 0.6 meters to 1.6 meters thick. Slabs at the top and bottom of the belt walls are 30 centimeters thick at levels 21 and 22. 40 centimeters thick slabs are required at levels 62 and 63 to accommodate concentrated shear stresses at large floor openings for new stairs and elevators outside of the core that service the levels above the core termination.

3.3 Foundations

The tower columns and core walls are supported on a pile supported mat foundation. The mat foundation is 36 meters x 52 meters x 4 meters thick and is supported by 77 1.5-meter diameter x 7-meter long piles. The concrete strength is 55 megapascals for the piles and 40 megapascals for the mat. The 7,500 cubic meters of concrete for the mat was placed in a continuous pour by 1,200 concrete trucks and 7 concrete pumps over 26 hours, making it the second largest mass concrete placement made in an urban area in Mexico. A 3-dimensional isometric of the building's structural system is shown in Fig 2. Fig. 3 shows typical framing plans of the tower.

Fig. 2 – 3-D Isometric of the Building Structure

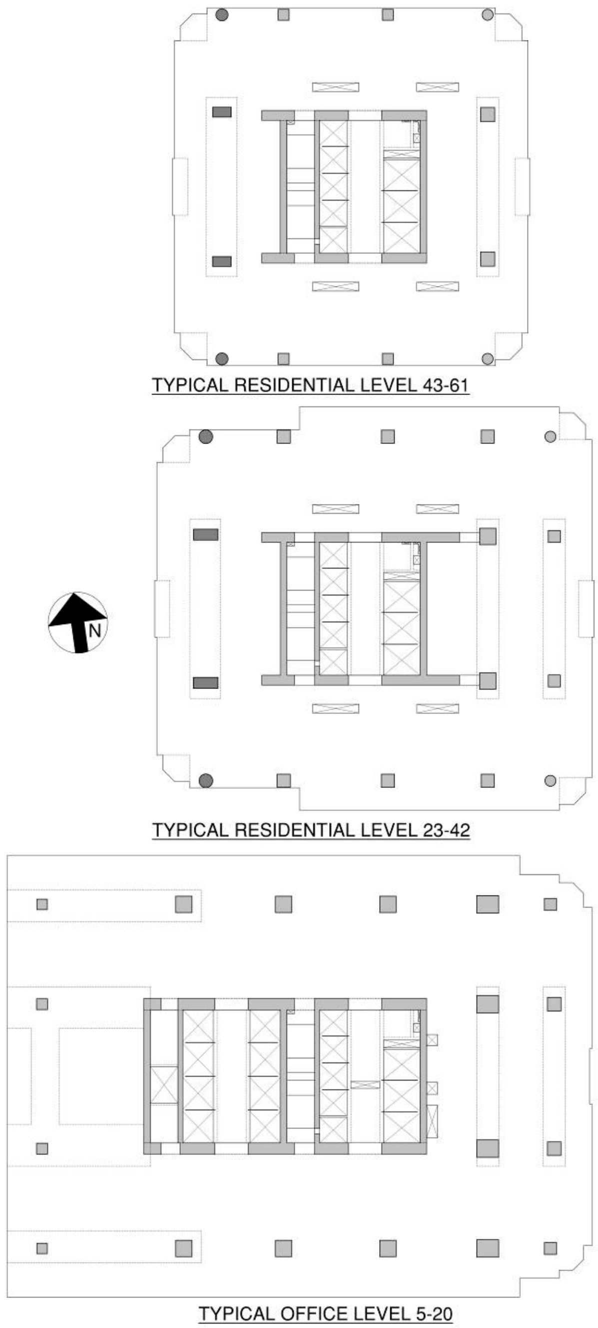

Fig. 3 – Typical framing plans

4. Seismology of Monterrey

Throughout history Mexico has experienced a considerable number of substantial earthquakes, mostly localized on the Pacific Coast, but until recently Monterrey was considered a non-seismic region. The metropolitan area of Monterrey is located in northeast Mexico, bounded by the Sierra Madre Oriental mountain range to the west and the Gulf Coastal Plains to the east. "Northeast Mexico is generally regarded to as a tectonically stable region, characterized by low seismicity and a lack of strong ground motion records" [1]. Using a prediction equation by Toro *et al*. [2], Galván-Ramírez and Montalvo-Arrieta [3] predicted the expected Peak Ground Acceleration (PGA) values for a rock site in Monterrey would be between 30 to 70 cm/s^2, or 0.03g to 0.07g [3]. Subsurface conditions at the project site consist of carbonated clay to depths of 3 meters and rock shales to depths of 30 meters where the building piles are founded, all underlain by limestone. Fig. 4 is a seismic hazard map of Mexico showing peak ground accelerations for a 2000 year return period earthquake. Fig. 5 is a similar map for the Mexican state of Nuevo Leon and surrounding areas.

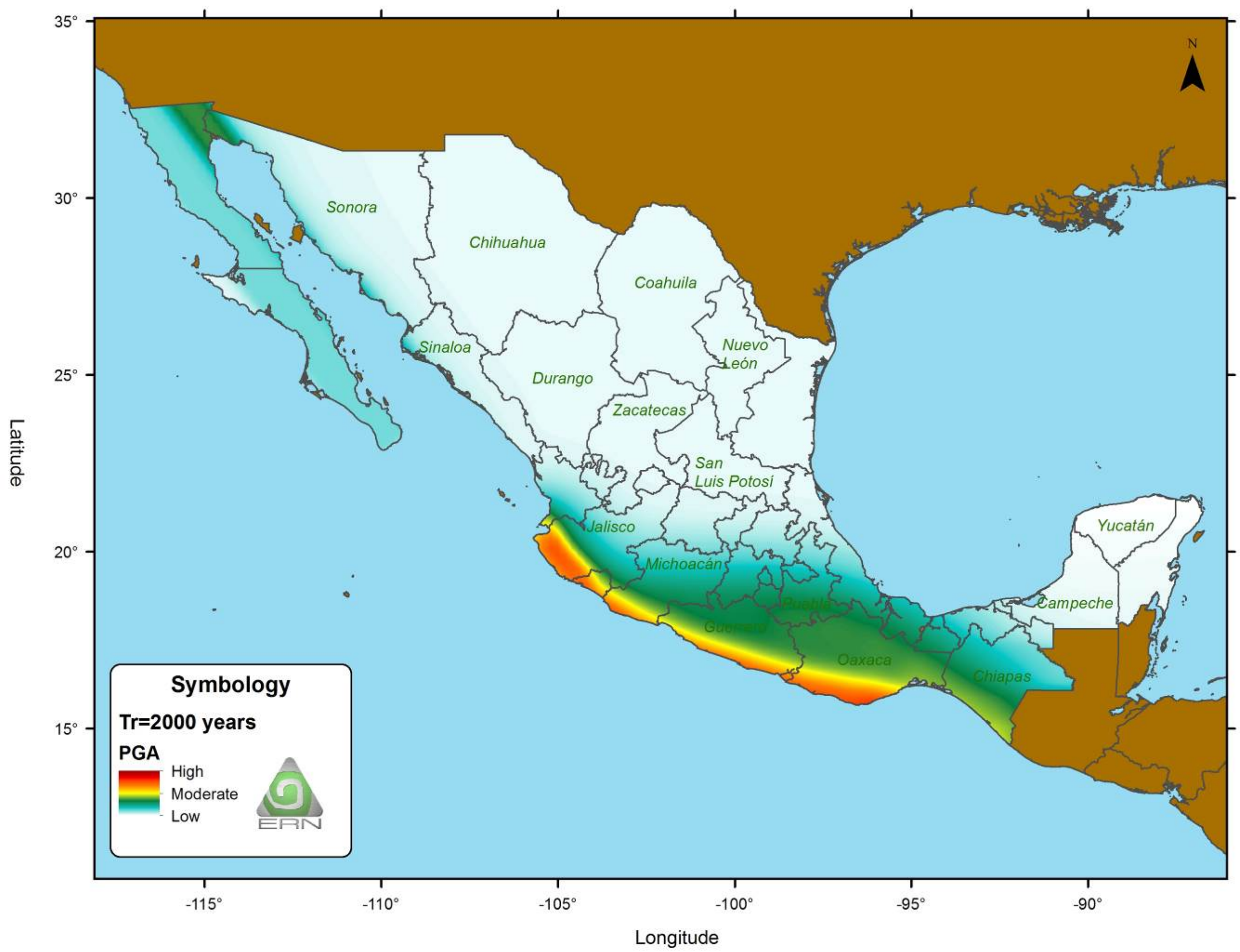

Figure 4 – Seismic Hazzard Map of Mexico

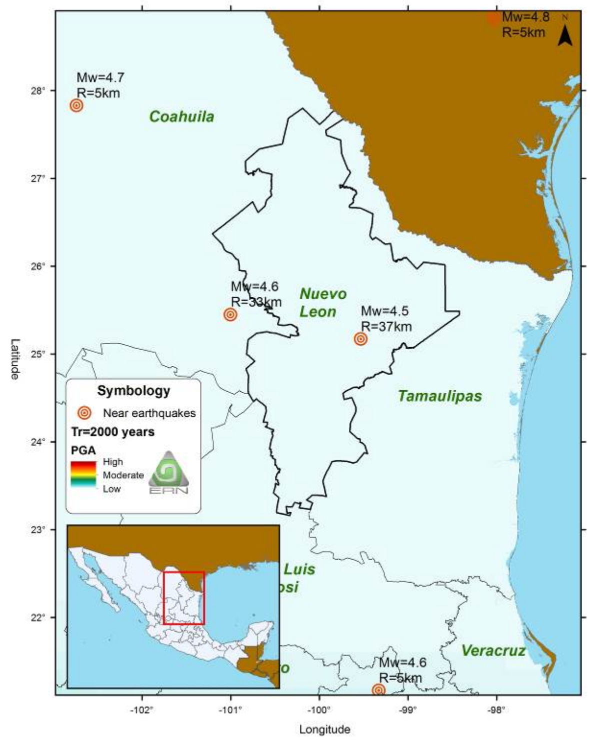

Figure 5 - Seismic Hazzard Map of Nuevo Leon, Mexico

5. Structural Analysis and Response

The structural analysis and design of Torre Koi was based various building codes and documents. The Monterrey Building Code, *Reglamento para las Construcciones en el Municipio de Monterrey* [4], was referenced for design loading requirements. The Monterrey code requires that seismic effects be considered, but does not give guidance on seismic loading, analysis, or design requirements therefore, the design team utilized the Federal Electricity Commission's *Manual de Diseño de Obras Civiles* (CFE Manual) [5] for seismic design procedures, which is standard practice in Mexico when the local building code does not provide seismic requirements. The *Normas Técnicas Complementarias para Diseño por Sismo* [6] of the Mexico City Building Code was also referenced. An analysis utilizing the *2012 International Building Code* (IBC) [7] and the American Society of Civil Engineer's *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-10) [8] was also performed for comparison purposes. IBC and ASCE seismic ground accelerations maps were prepared by the United States Geological Survey (USGS) and include seismic ground accelerations for northern Mexico. The seismic site class assumed for IBC calculations was based on research by Montalvo-Arrieta *et al* [1]. The final structural design was based on the CFE Manual as it more familiar to the local authorities as well as resulting in higher structural demand requirements. Design wind loads and wind accelerations were based on a wind tunnel analysis performed by Rowan Williams Davies & Irwin Inc. (RWDI).

5.1 Modeling

Three-dimensional mathematical models, based on the finite element method, of the building structure were created in the structural analysis software ETABS and MIDAS Gen. All primary and secondary structural elements were included in the analysis model to properly account for load distribution and P-Δ effects. Elements were sub-meshed between floors to adequately capture P-δ effects. Columns were modeled as frame elements while walls and slabs were modeled as 2-dimensional thin shell elements. All elements were modeled with linear elastic concrete properties that varied with the concrete strength. Stiffness modifiers were applied to the shell and frame elements based on the level of tension stress in the member to account for the effects of cracking on the building stiffness at both service and ultimate load levels. Rigid diaphragm constraints were utilized at typical floors, however, a semi-rigid diaphragm constraint was used for the floors at the tops and bottoms of belt walls as well as 2 floors above and below each wall to account for the true stiffness of the floors transferring load between the building core and belt walls.

5.2 Seismic Loading

The building's size, height, and complexity required that a dynamic modal response spectrum analysis be performed by the CFE Manual, however the simpler equivalent lateral force analysis procedure is acceptable by IBC due to the low level of seismicity. For comparison purposes, a modal response spectrum analysis per IBC was also performed. See Fig. 6 for a comparison of the CFE and IBC response spectrum. The CFE response spectrum is divided by a reduction factor, Q' of 1.6 per Eq. (1) and (2), to account for the ductility and irregularity of the building's structural system to obtain the design response spectrum. IBC procedure requires results, not the input design response spectrum, to be divided by a ductility factor, R which is similar to Q'.

$$Q' = Q \times F_r \qquad \text{for } T \geq T_a \tag{1}$$

$$Q' = [1 + (T/T_a)(Q-1)] \times F_r \qquad \text{for } T < T_a \tag{2}$$

Q = 2 based on the building's lateral force resisting system

F_r = 0.8 based on irregularity of the structure

Determination of the fundamental frequencies was an iterative process due to varying levels of concrete cracking resulting from the forces generated by the dynamic modal response spectrum analysis. The first 42 modes were considered to obtain a 90% modal mass participation in each of the structure's orthogonal horizontal directions to consider high mode effects and to capture irregularities in building mass and stiffness. Modal responses were combined by the Complete Quadratic Combination (CQC) method with a 5% modal damping. Directional effects where considered by combining response in the two analyzed orthogonal directions by the Square Root Sum of Squares (SRSS) method. Accidental torsional effects were considered by applying a 10% mass offset at each diaphragm. The design response spectrum in each direction was scaled for the acceleration due to gravity and as required by Eq. (3) to obtain force results. Due to the manner in which individual modes are combined in the response spectrum analysis, all response results were positive. In order to determine the negative responses on the foundations to design for uplift, equivalent lateral load cases were developed from the response spectrum analysis matching base shears and overturning moments.

$$V_{o\,MIN} = 0.8a\frac{W_o}{Q'} \geq 0.018W_o \tag{3}$$

W_o = Building Seismic Weight
$V_{o\,MIN}$ = Minimum Base Shear

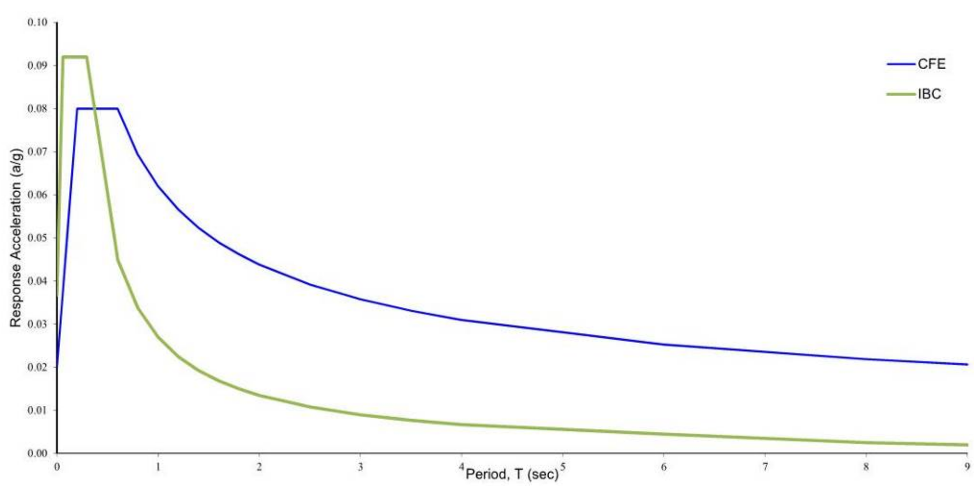

Figure 6 – Response Spectrum

5.3 Analysis

Initial structural responses for preliminary design were based on a 3-D linear analysis with non-iterative mass-based P-delta analysis in ETABS. This analysis was used to determine baseline responses and force distribution, preliminary member sizes and reinforcement quantities, and level of cracking in each structural element. Fig. 7 shows the deflected shape of the structure for the first three modes of vibration.

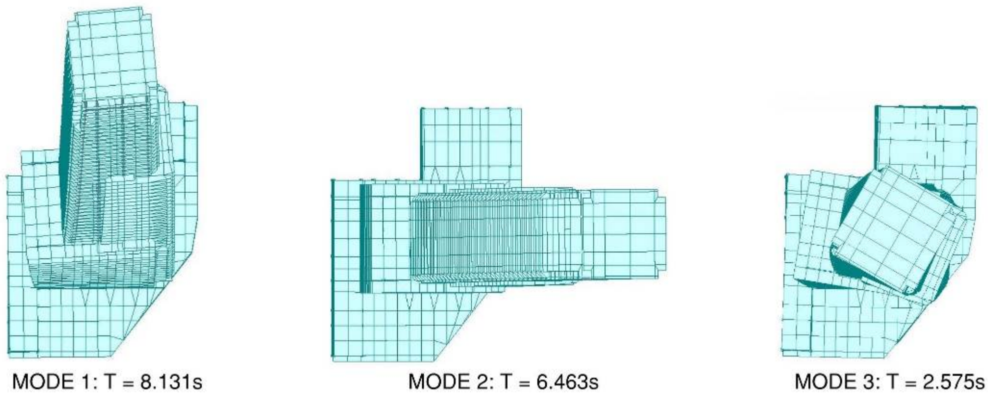

Figure 7 – Building Periods

Once preliminary member sizes and reinforcement quantities were determined, a non-linear staged construction analysis was performed to account for load distribution resulting from the actual construction sequence. The tower floor plates step non-symmetrically moving upwards while the building core is connected to columns on the east side by link beams at floors 23 through 42, which results in non-uniform gravity load distribution between columns and the core walls. Structural members were optimized to minimize non-uniform gravity load stress, however, the structure is still anticipated to experience some permanent tilt or out-of-plumb geometry upon completion of construction. The design team recognized that tilt reported by applying gravity loads to a completed model will be greatly exaggerated. As construction progresses, building elements will be built to theoretical location, shifting formwork as needed, therefore a non-linear staged construction analysis will more accurately predict the gravity load distribution and shape of the permanently deflected structure at the completion of construction. Member sizes and reinforcement quantities were further refined from results of the non-linear staged-construction analysis.

A final 3-D non-linear analysis with staged construction loading and time dependent concrete properties capturing the long term effects of concrete creep and shrinkage on the load distribution and deflected shape of the building was performed using the structural analysis software MIDAS Gen. Concrete material has time-dependent properties, such as creep, shrinkage, and modulus of elasticity, which must to be considered in the structural analysis of tall buildings. These time-dependent material properties, in conjunction with the proposed sequencing of construction for the tower, will affect the overall vertical (gravity) load stress distribution within the structural system during construction, in the final/constructed condition, and over time in service. Time dependent material properties were modeled in MIDAS for modulus of elasticity, creep, and shrinkage. The development of the concrete time dependent material property curves was based on Gardner & Lockman *Design Provisions for Drying Shrinkage and Creep of Normal Strength Concrete* [9], which reflects the effect of size and surface/volume ratio on the rate at which shortening occurs and because it does not require some of the extensive concrete mix information required by other methods that was not available during design. These properties were also modified to account for the presence of concrete reinforcement. Final member sizes and reinforcement quantities were refined as necessary. Fig. 8 & 9 shows expected permanent lateral deformation resulting from dead load at the end of construction and at 30 years from substantial completion of the building structure. During construction, the concrete sub-contractor instrumented numerous columns to measure the actual vertical shortening of the concrete columns in order to compare with, and calibrate the structural analysis model. Actual column shortening results are slightly less than the results predicted.

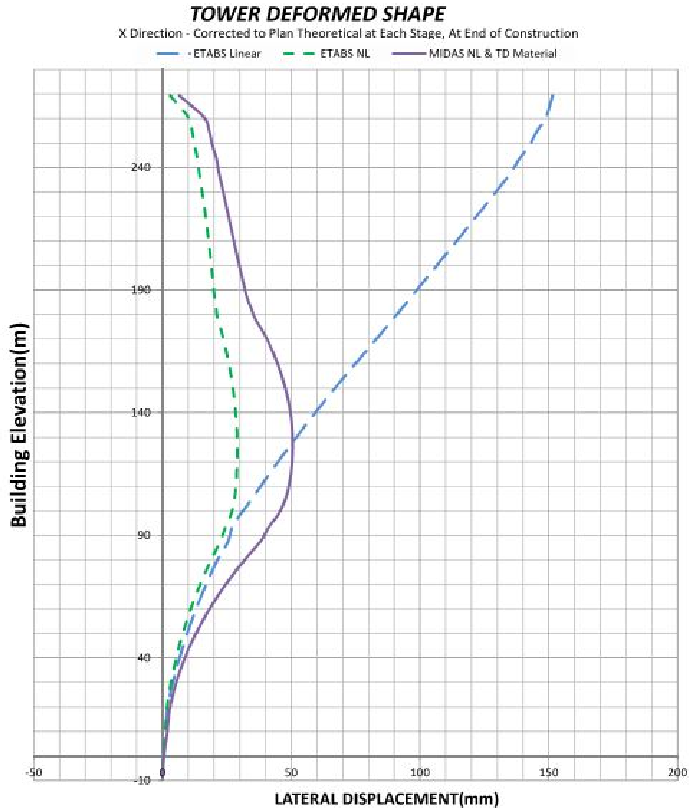

Figure 8 – Lateral Deformation Due to Each Analysis Type

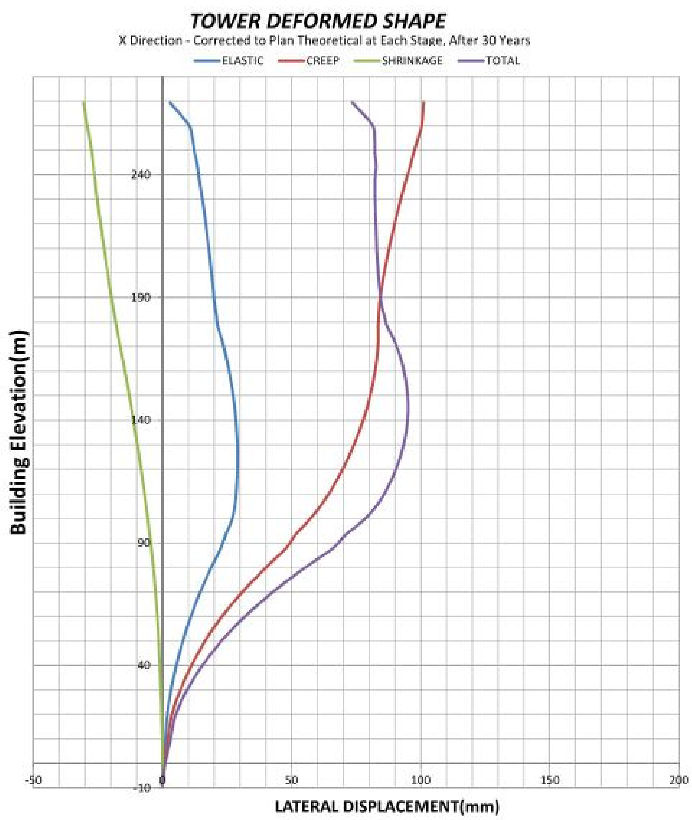

Figure 9 – Lateral Deformation at 30 Years

5.4 Story Drift

Controlling story drift is one of the most important structural seismic design considerations to ensure satisfactory performance of the entire building under a seismic event. Story drift is the difference in lateral deflection of vertically aligned points on floors between any two adjacent stories. Excessive story drift can result in damage to building elements, such as infill walls and cladding, and secondary structural members not included in the structure's lateral force resisting system. The CFE requires that story drifts shall be limited to 0.006 when the structural framing consists of flat slab systems. In order to obtain drift results from the analysis, the design response spectrum was scaled by a factor equal to Q' to account for the inelastic deformations the structure will experience in a real seismic event. An additional scale factor to achieve a minimum base shear is not required when checking drift. Fig. 10 shows the maximum lateral story drifts under a load combination that included dead load, superimposed dead load, 50% of the live load, and the seismic response spectrum. The negative peaks occur at the belt wall floors where load is being transferred between the concrete core and perimeter columns and at the ground level where additional concrete shear walls exist below. Story drifts are within code required design limits.

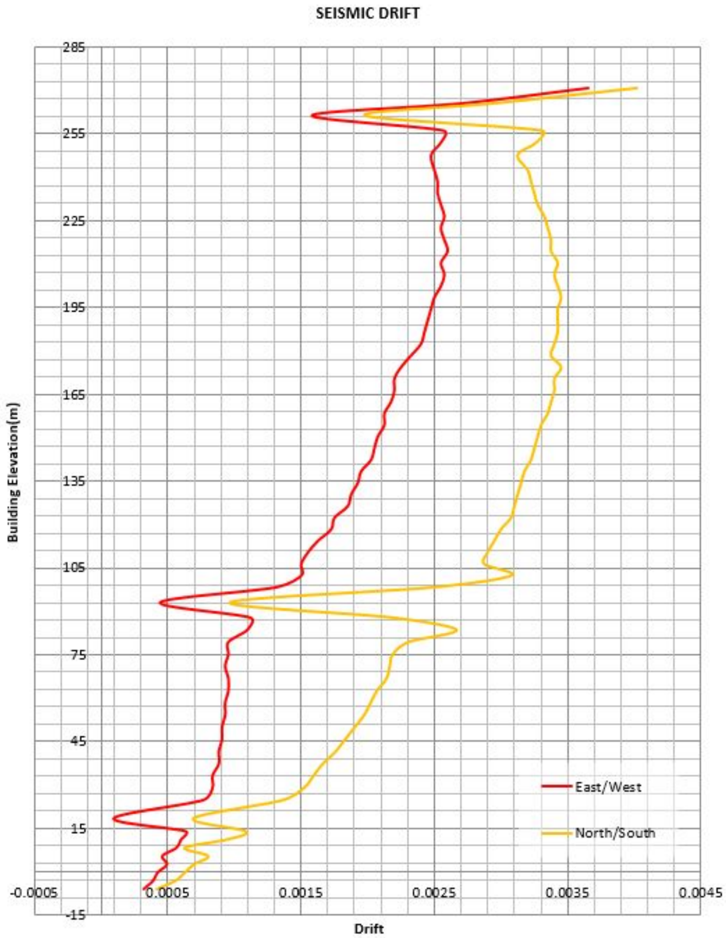

Figure 10 – Seismic Drift

6. Design and Detailing

Preliminary member sizes and reinforcement designs were based on responses from the initial ETABS 3-D linear analysis. Preliminary structural designs were checked against responses from subsequent ETABS and MIDAS Gen 3-D non-linear analysis with modifications made as necessary. Design of the concrete structural elements was in accordance with the 2011 American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318M-11)* [10].

6.1 Critical Design Elements

The elements most critical to the performance of the structure were the coupling beams, belt walls, and belt wall diaphragm slabs. These elements are the links between the structural building core and columns that allow the entire structure to work together in resisting lateral forces on the building. Coupling beam pan to depth (L/D) ratios varied from 0.8 to 5, but were typically in the range of 1.5 to 3. Coupling beams with L/D greater than 4 were designed per standard flexural and shear provisions of ACI 318. When L/D was less than 4, the beams were considered “deep beams” and were designed utilizing strut-and-tie models. At a number of very highly loaded coupling beams, concrete alone was not sufficient to resist the forces and expected rotations. At these locations, structural steel plates were encased within the concrete coupling beams. The steel plates were designed for the full factored shear force while the concrete and reinforcement resist the factored design moment. Steel plate embedment into the shear wall piers were designed in accordance with the American Institute of Steel Construction’s *Seismic Provisions for Structural Steel Buildings (AISC 341-10)* [11].

The concrete belt walls and diaphragm slabs work together as virtual, or indirect outriggers to transfer a portion of overturning moment from the core to perimeter columns. In this system there is no direct connection between the core walls and columns by direct outriggers such as trusses or walls. The overturning moment creates a horizontal couple of forces in the stiff floor diaphragms, which cause the belt walls to tilt and follow the core’s rotation. The perimeter columns resist the belt wall tilt with a vertical force couple by varying axial forces. The belt walls and diaphragm slabs were designed for shears, moments, and axial loads extracted at critical locations via section cuts from the ETABS model. Where necessary, strut-and-tie models were employed. In addition to mild steel reinforcement, and like the typical floors slabs, the diaphragm slabs were reinforced with Post-Tensioning tendons to help resist the vertical gravity loads and to increase the stiffness of the virtual outrigger system, making it more efficient in resisting wind and seismic loads.

6.2 Detailing

Due to the relatively low level of seismicity in Monterrey, the structure was not required to meet any special prescriptive seismic detailing requirements, however, a number of best practices were utilized to enhance the ductility of the structural system. Mechanical couplers were used at highly loaded columns and shearwall segments and columns with net tensile stresses to more effectively transmit tension forces. Headed reinforcement was utilized to anchor reinforcement in the belt walls and diaphragm slabs at heavily reinforced section to ease congestion and ensure the reinforcement was developed.

7. Conclusions

Given that the local building codes in Monterrey do not cover seismic design, the structural design team was required to look elsewhere for guidance on seismic design of a high rise structure in Mexico. Various documents and codes on seismic design in Mexico, as well as the United States, were consulted.

The final structural design of Torre Koi was based on a combination of wind load and seismic results. The stiffness of the structure was governed by horizontal wind acceleration limits for occupant comfort of 18 milli-g at the top residential floor and 25 milli-g at top floor for office use under a 10-year wind. The difference in the manner in which wind loads and seismic loads act on the building structure resulted in strength design of certain members controlled by seismic forces while others were driven by wind forces. Due to the dynamic and cyclic nature of seismic loads, select seismic detailing was incorporated into the structure, even where seismic loads do not control member size and reinforcement quantities, to increase the structure’s ductility. Torre Koi will be an icon for Mexico and the city of Monterrey for years to come.

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