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Detailed Nonlinear FE Pushover Analysis of Alto Rio Building

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Abstract

The fifteen story Alto Rio building built between 2007 and 2009 in the city of Concepción, Chile, was the only modern building to experience catastrophic collapse during the 27 February 2010, Mw 8.8 Maule earthquake. This building was representative of those medium-rise buildings designed and built in Chile during the period 1996-2009 in which seismic resistance is provided by a dense array of thin wall bearing walls oriented in two orthogonal directions. A detailed nonlinear model of this building was developed in the computer program ABAQUS/Standard FE. This program was used in conjunction with the ANACAP-U concrete and reinforcing bar constitutive models developed by ANATECH Corporation. A parametric pushover analysis of the building was conducted considering several possible scenarios: with and without lap splice failure, and with and without the removal of some small gravity columns. This paper describes the development of the finite element model and discusses the results obtained from a parametric nonlinear static pushover analysis of the Alto Rio Building.

Keywords: Collapse, Lap splices, Maule earthquake, Nonlinear finite element model, Pushover analysis

1. Introduction

The fifteen story Alto Rio building built between 2007 and 2009 in the city of Concepción, Chile, was the only modern building to experience catastrophic collapse during the 27 February 2010, Mw 8.8 Maule earthquake, see Figure 1. Eight lives were lost as a result of the collapse of this building. This building was representative of many medium-rise buildings designed and built in Chile during the period 1996-2009. The lateral force resisting system of the Alto Rio building consisted of a dense array of predominantly 0.2 m thick transverse and longitudinal reinforced concrete bearing walls and a few thin and oblong columns. In a quest to understand the causes of collapse from the system behavior point of view, the entire Alto Rio building was modeled directly from its CAD files by using the Rhinoceros 3D computer graphics software followed by the FE mesh generation software FEMAP. The FE model consisted primarily of conventional 4-node shell elements of approximately 0.2 m x 0.2 m in size for the concrete walls and slabs and smeared rebar sub-elements for the steel reinforcement. The structure was modeled almost entirely using 4-node shell elements with embedded smeared reinforcing bars. The 4-node, 24 degree-of-freedom shell element used involved 2x2 Gaussian integration over the surface of the element and Simpson’s quadrature with three integration points (mid-surface and inner/top and outer/bottom surfaces of the shell) through the uniform thickness of the shell. The reinforcing bar sub-elements were modeled directly from the reinforcement detailing of the various structural components of Alto Rio defined in the CAD drawings. The analysis software of choice for performing a detailed nonlinear pushover analysis of the Alto Rio building was the ABAQUS/Standard FE computer program used in conjunction with the ANACAP-U concrete and reinforcing bar constitutive models developed by ANATECH and described below in this paper. A parametric pushover analysis of the building was conducted considering several possible scenarios: with and without lap splice failure, and with and without the removal of some small gravity columns. This paper discusses the development of the model and the results obtained from the parametric pushover analysis of the Alto Rio Building.

2. Building Description

2.1 General dimensions

Alto Rio was a fifteen story mixed-use building with an oblong floor plan with two underground parking levels, see Figures 1 and 2. This building was built at coordinates 36.83º S, 73.51º W with the longitudinal axis oriented N27ºW in the city of Conception, Chile. Construction took place between December 2007 and February 2009. The building had an L-shape elevation along the weak axis, with the two 21.80 m wide by 45 m long underground parking stories extending beyond the tower footprint towards Ave. Arturo Prat. The typical story was 11.80 m wide by 38.30 m with a story height of 2.52 m. At Level 1, the story height was 3.06 m. The upper three stories were stepped toward the southern end: 32.8 m x 11.80 m at Story 13 at an elevation 30.78 m from Floor 1; 25.55 m x 11.80 m at Floor 14 at elevation 33.30 m; and 19.40 m x 11.80 m at Floor 15 at elevation 35.32 m. Above Floor 15 there were two small floors, one used as a machine room and the other as the machine room rooftop. At 46.69 m elevation from level 1, the floor dimensions were 6.50 m x 11.60 m and at elevation 50.96 m from Floor 1 the rooftop dimensions were 6.10 m x 8.10 m. The building had a 0.8 m thick reinforced concrete mat foundation below the tower and 0.4 m thick foundation mat below the two underground parking stories that projected beyond the tower footprint.

The lateral force resisting system of the Alto Rio building consisted of a dense array of predominantly 0.2 m thick transverse and longitudinal reinforced concrete bearing walls. Figure 2 shows the plan view of Floor 1 and the layout of all the reinforced concrete elements at this level. A few transverse walls ending on Grids A, I and/or J had 0.2 or 0.25 m thick flanges and others had no flanges at all. Transverse walls on Grids 26 and 34 had curved flanges on Grid J. The plan view of Floor 1 highlights a number of important vertical discontinuities along

Courtesy of Constructora Socovil

(a) Building in the final stages of construction

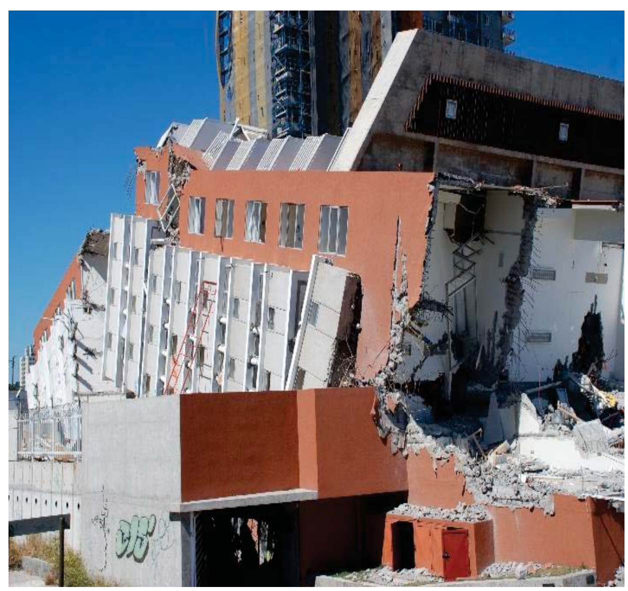


Photo by J Restrepo

(b) Building collapsed along the weak axis toward Ave. Arturo Prat

Figure 1 – View of Alto Rio building.

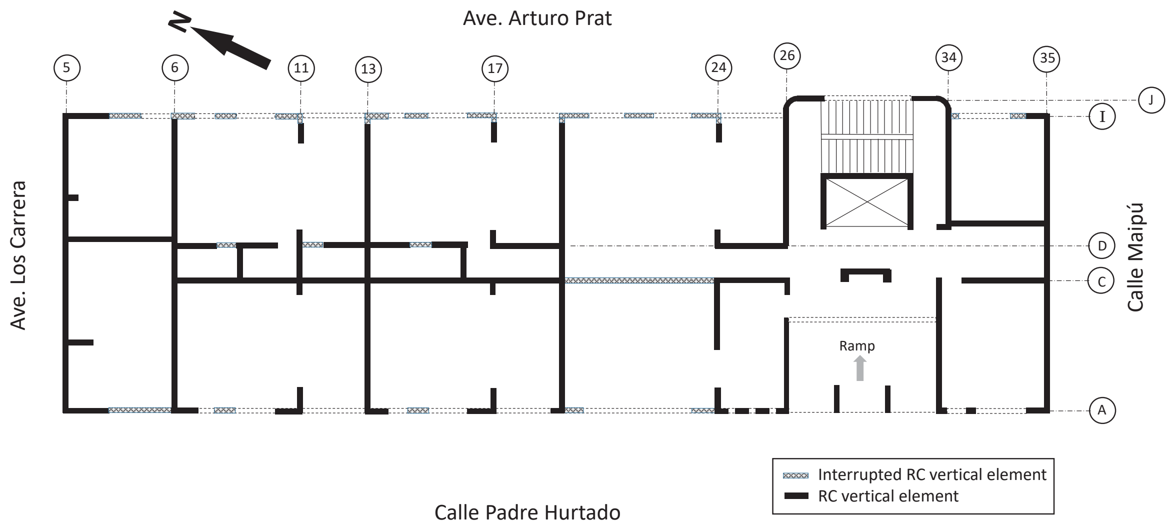


Figure 2 - Plan view of the first story structural elements (Piso 1) and key gridlines.

structural elements in longitudinal gridlines A, C, D and I. The ratio of the wall[1] area in Piso 1 to the typical floor area was approximately 2.5% in the longitudinal direction and 3.5% in the transverse direction. These ratios are within the ranges used in buildings of similar heights built in the 2000s in Chile.

2.2 Structural Design

The Alto Rio building was designed in accordance with the NCh433 [1] and NCh430 [2] loadings and reinforced concrete codes, respectively. The design response spectra for buildings contained in NCh433 was calibrated such that it has the average shape of the response spectra compatible with that obtained from historical records, but values are scaled so residential buildings with a fundamental period between 0.5 and 1.5 s have a base shear coefficient consistent with pre-1985 design standards [3]. Lateral forces obtained from design codes' ground response spectra are modified by the importance factor for the building, and by the inherent ductility and damping of the lateral system. Minimum and maximum base shear limits may control these forces. Limits are also considered to control maximum interstory drifts. The design of the reinforced concrete elements and detailing followed NCh430 which was largely based on ACI 318-05 [4]. A notable difference between ACI 318-05 and NCh430 is the lack of prescriptive requirements for boundary elements in the latter standard.

The specified concrete compressive strength was f'_c=25 MPa in the underground levels, Level 1 and Level 2, and f'_c=21 MPa in Level 3 and above. The specified yield strength for the reinforcement was f_y = 420 MPa. Walls had two curtains of uniform 8 mm or 10 mm diameter horizontal and vertical reinforcement with reinforcement ratio varying between 0.13% and 0.26%, which is considered light by the authors. Larger diameter bars (16, 18, 22 m diameter and, rarely, 25 mm diameter) were detailed at the wall ends and flanges. Like in many buildings built in Chile during this period, no detailing was made to ensure ductile response in this building. Figure 3 shows the detailing of the reinforcement at the intersection of walls on Grids 26 and J. The architectural firm detailed a curve at this intersection but no recognition was made in design for the need to provide cross-ties to resist the radial forces arising from the horizontal reinforcement curved around the concave side wall.

[1] To obtain this ratio, a wall was defined as an element with a length equal or greater than 2 m.

3. Field Observations of the Collapsed Building

3.1 General Observations

The tower of the Alto Rio building collapsed about the weak axis on Level 1 towards Ave. Arturo Prat, see Figures 1 (b) and 2. The first indications of visible failure were a number of lap-splice failures in a few elements along Grid A, see Figure 4 (a), reported also by others [5]. Lap-splice failures have also been observed in shake table tests of shear walls [6]. Crushing of the concrete was also evident in a transverse wall along Grid 35 between Grids D and I, see Figure 4 (b). This portion of the wall was dislodged towards Calle Maipú. In the vicinity of Grid J there was mainly rubble. IDIEM [7] indicates that visible parts of the stairwell presented significant damage and dislocation.

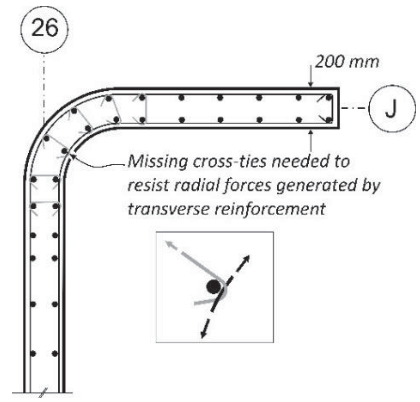


Figure 3- Reinforcing detailing curved wall on Grids 26 & J.

3.2 Materials

IDIEM [8] and DICTUC [9] report the main material mechanical properties of steel reinforcement bars and concrete cores extracted from the collapsed building. Steel reinforcement of up to a diameter of 12 mm had an average yield and tensile strengths of 509 and 746 MPa. Bars 16 to 25 mm in diameter had an average yield and tensile strengths of 472 and 772 MPa, respectively. Concrete core compressive strengths exhibited an average strength $f_c^{'}$ = 42.4 MPa for level 2 and lower, and $f_c^{'}$ = 48.8 MPa for the remaining levels. The average specific density of the concrete was 2.36.

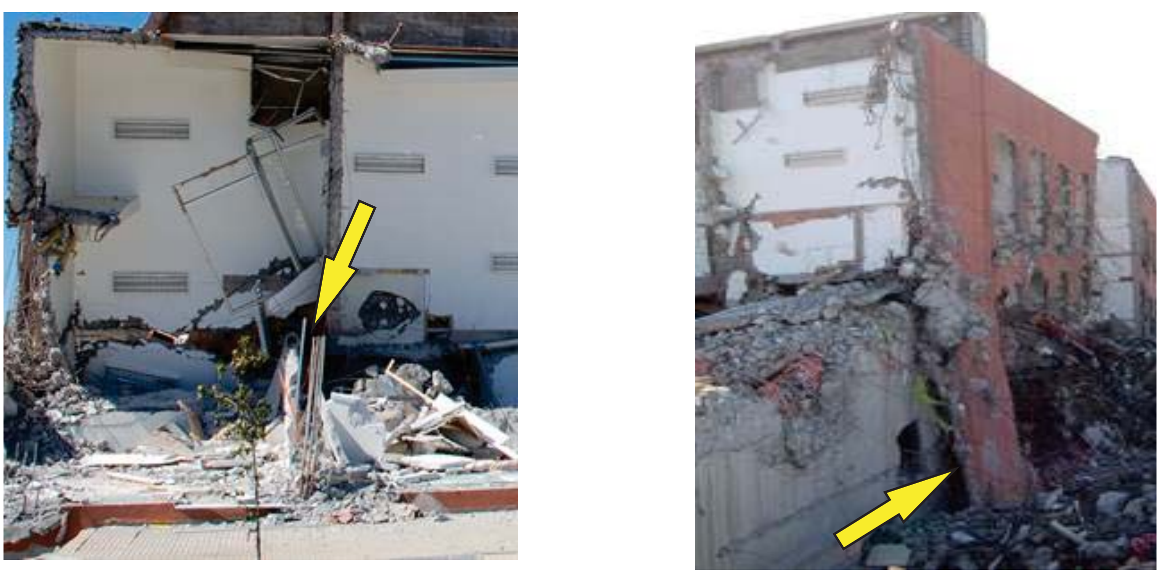


Photo by J Restrepo

(a) Lap splice failure in wall on Grids 8 & A

Photo by J Restrepo

(b) Crushed and laterally displaced wall along Grid 35

Figure 4- Structural failures visible in Level 1 as seen from Calle Padre Hurtado.

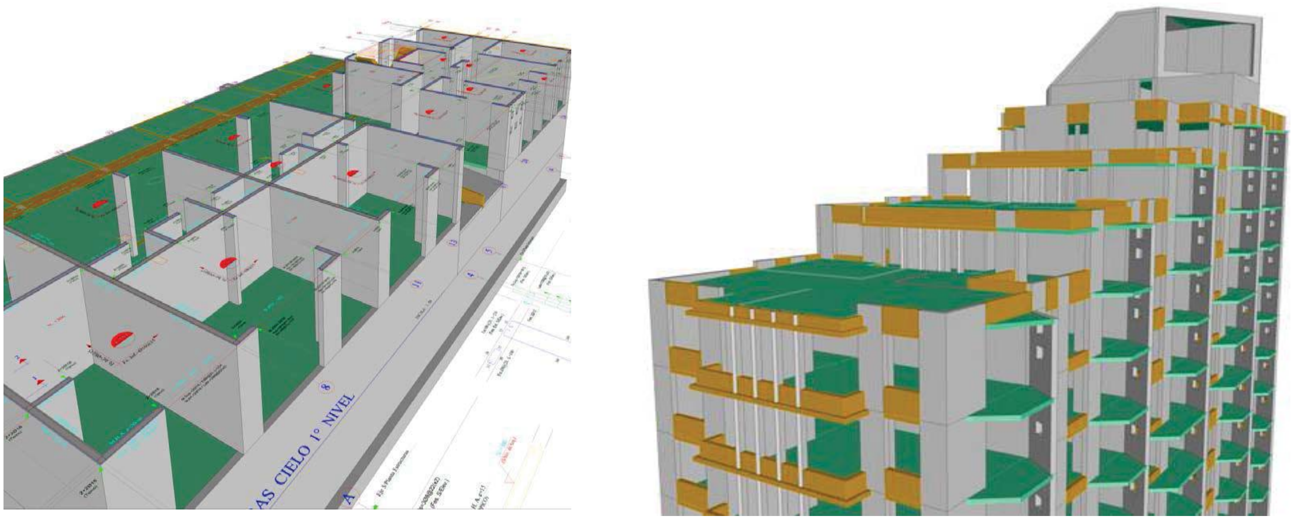
4. Model Development

4.1 Geometric Model

To perform a comprehensive seismic assessment of the Alto Rio building, a detailed three-dimensional geometric model was developed from the structural drawings. The goal of the modeling process was to generate a three-dimensional (3D) digital model that recreates the as-built Alto Rio structure with the highest fidelity as possible. This process began with the acquisition of structural drawings in CAD format. These drawings provided a comprehensive illustration of the structure that drove the modeling process. The drawings included the thickness and location of walls, columns, beams, slabs, mat foundation, stair cases and other various elements as well as the reinforcing scheme for each component. To maximize the precision between CAD drawings and the final FE mesh used for analysis, the drawings were used to directly construct the geometric model of the building. The CAD drawings were imported into Rhinoceros 3D as a collection of lines. The drawings were placed in a global 3D coordinate system to correspond with the appropriate location within the structure. By placing the CAD drawings in this manner, the geometric model was constructed by extruding the imported lines to generate 3D solids, see Figure 4(a).

4.2 Finite Element Model

The complex wall geometry of the Alto Rio building, makes it very difficult to use the more traditional design oriented nonlinear analysis The Rhino 3D geometry was exported into FEMAP [10], an advanced graphical pre- and post- processor with automated mesh generation capabilities. Since most of the critical elements in the structure were approximately planar and 0.2 m thick, the goal of the meshing process was to generate elements with a width and height as close to this dimension as possible so that the ratio of width or height to thickness of each element in general would not drop below a value of one. The structure was modeled almost entirely using 4-node shell elements for the concrete and smeared sub-elements for the reinforcing steel. The reinforcing steel sub-elements were modeled directly with ABAQUS input instructions based on an Excel spreadsheet file detailing the reinforcement which was created manually from the CAD drawings. The 4-node, 6 degree-of-freedom shell elements use 2x2 Gaussian integration points in the surface of the shell and 3



(a) Extrusion from CAD drawings

(b) Detailed view of geometric model

Figure 5 –Geometrical model of Alto Rio building developed in Rhinocerus 3D.

Table 1 –Key statistics of the FE model.

Nodes	463,860
Elements	466,085 (~200 N/element)
Degrees of freedom	2,767,608
Integration points	5,593,020

Simpson integration points through the uniform thickness of the shell (top, middle-depth, and bottom). These shell elements use the thick shell element formulation and have hourglass modes in either bending or membrane response. Since the 3D geometry was modeled by planar elements, every structural feature was represented by shell elements located at the mid-plane of each component. This was accomplished for all structural components except for the floor slabs. For the sake of model compatibility, the slabs were modeled with planar elements located directly at each floor elevation. Therefore, the middle surface of the finite elements slabs correspond to the top of the actual floor surface. The entire structure was modeled using planar shell elements except for a handful of small reinforced-concrete columns that were modeled using 2-node linearly interpolated Timoshenko beam elements. These beam elements have embedded reinforcing bar elements similar to that of the shell elements. Table 1 lists key statistics of the FE building model and Figure 6 depicts the FE mesh developed for the entire building and of a detail of Levels 1 - 3 by Grids 11 through 34 with Grids I and J.

4.3 Constitutive Material Models

The finite element analysis program of choice for the prediction of the nonlinear response of this building was ABAQUS/Standard FE [11], which was combined with the ANATECH developed ANACAP-U concrete and steel reinforcement constitutive models [12] [13]. The ANATECH ANACAP-U concrete constitutive (material) model was developed as a standalone finite element computer program and as a user constitutive model (UMAT) for the ABAQUS [11]. Enhancements to this concrete model have been made over the years through national research programs in the United States and into the ANACAP-U concrete and steel reinforcement model [14]. These research programs dealt with the failure analysis of reactor containment structures under core meltdown accidents, nuclear waste storage structure long term stability under elevated temperatures, spent fuel pool performance under combined earthquake and accident induced boiling, and the static and seismic stability of gravity dams. This vast amount of concrete modeling work resulted in significant advances in the state of the art of concrete structural analysis, including the modeling of concrete in the post critical regime considering spallation and progressive damage inducing strain softening and crushing, the modeling of rebar/concrete interaction, the modeling of concrete under long term sustained high temperatures, and the development of a rough crack model for the proper simulation of shear transfer across cracks for application to the shear failure of concrete containment structures.

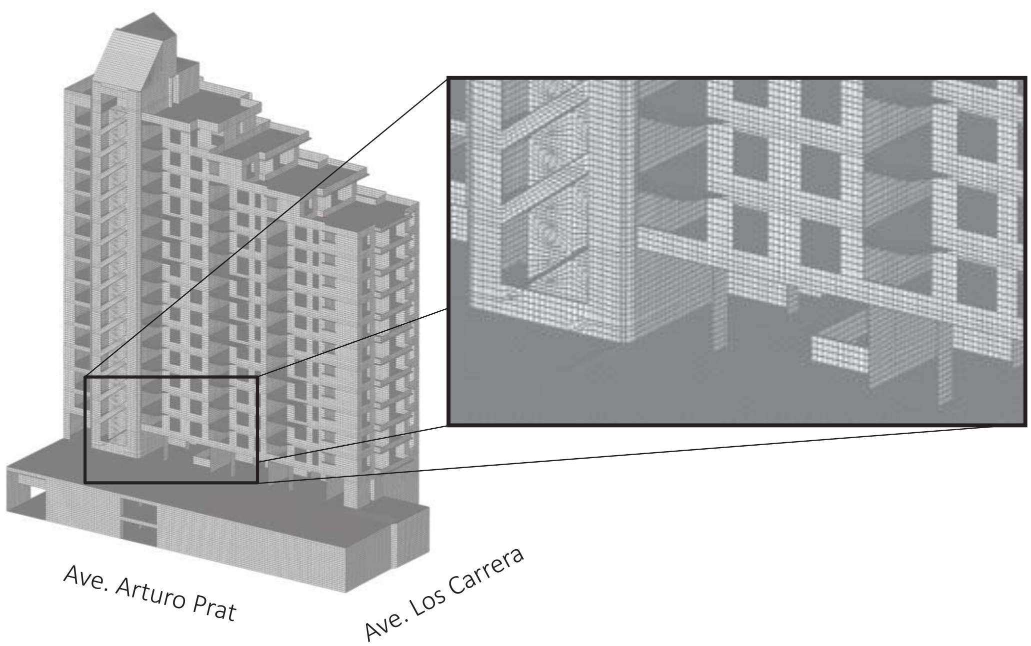


Figure 6: FE mesh of Alto Rio building developed in FEMAP [10].

The ANATECH ANACAP-U concrete constitutive model [14] is based on the smeared-cracking methodology developed by Rashid [12] and a Drucker-Prager (J2) plasticity theory [15]. The most important aspect of the Alto Rio building is that the entire structural support system consisted of thin reinforced concrete walls. Thus, the entire structural system relied on unconfined concrete. The uniaxial compressive stress-strain curves of unconfined concrete exhibit abrupt strain-softening in the post-peak stress regime. The smeared cracking model allows cracks to form in three orthogonal directions, and cracks can open and close, but their directions are fixed and they don't heal. When cracks form, energy loss initiates locally at the crack locations. Cracking of biaxial and triaxial stress states are treated consistently with uniaxial cracking, but they occur at a slightly higher stress and slightly lower strain. Split cracking occurs at approximately zero stress and a tensile strain approximately twice that of the uniaxial tensile strength. This agrees well with the observed behavior of concrete test specimens.

The rough crack and damping models focus on two major areas of concrete behavior that have remained elusive to the state of the art of finite element analysis of concrete structures. The ANACAP-U concrete constitutive model treats the shear transfer mechanism with a shear retention factor in which the shear stiffness across an open crack is inversely proportional to the crack opening. In addition, ANACAP-U includes a rough crack model that introduces a representation of shear transfer that accounts for the crack roughness (aggregate size), crack slip, crack opening and the reinforcement stiffness across the crack. For damping, ANACAP-U uses a cracking consistent damping model that removes the usually assumed Rayleigh damping by modeling the damping within the mathematical framework of the smeared cracking model as a local anisotropic effect that depends on the crack status and the damage state of the material.

An improved constitutive model for the steel reinforcement that treats bar plasticity, strain-hardening/softening, Bauschinger effect and bond-slip behavior was also developed for ANACAP-U. A standard J2 plasticity model was modified to include the effects of bond-slip (or confinement) and anchorage-loss behavior. The modified model was empirically calibrated to observed material data. The model also introduces slip-friction behavior into the unloading/reloading path which improves the shape of predicted hysteresis loops in the presence of bond slip action. The ANATECH bond-slip model incorporates both the loop-pinching effect and the ultimate anchorage-loss effect. These effects are critically important to predicting the actual behavior of cyclic structural concrete test.

4.4 Lap-splice Model

A strength degrading model for lap-splices was developed in the finite element model and validated against the experimental results reported by Bimschas and Dazio [16] a reinforced concrete wall with transverse and longitudinal reinforcement similar to those in the structural walls of Alto Rio. In order to see the sensitivity of the building response to the degradation of the lap-splices, two models were developed: *Model a* and *Model b*, whose stress-strain curves under monotonic loading are given in Figure 7. *Model a* allows the lap-splice steel to reach the yield stress and maintain it up to a strain of 0.5% when the lap-splice starts to degrade whereas *Model b* starts to degrade immediately upon reaching the yield strength.

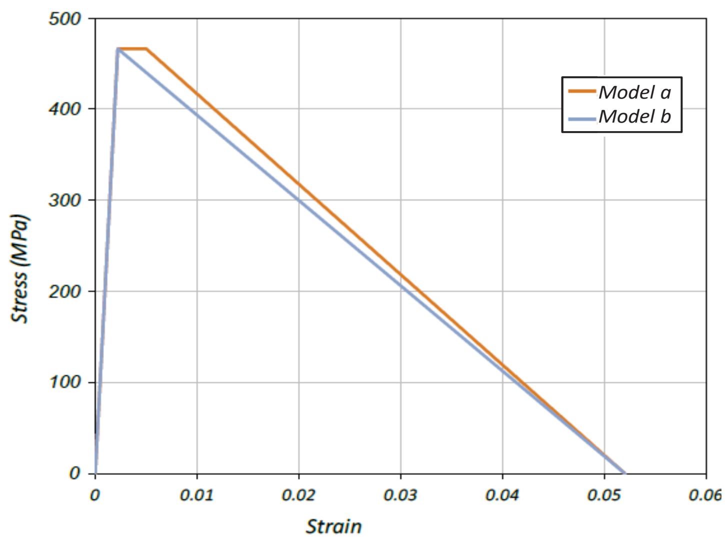


Figure 7- Strength degrading lap-splice models.

5. Nonlinear Static Pushover Analysis

5.1 Load Combination

Nonlinear static analyses were conducted, wherein the model was subjected to monotonically increasing lateral loads. The gravity loads were those corresponding to one of the load combinations recommended by ASCE 41-06 [17] - dead load plus superimposed dead load plus 25% of the design live load all multiplied by 1.1. The dead plus superimposed dead load was calculated equal to 93 MN for the entire building and 69 MN for the tower above Floor 1.

5.2 Modal Analysis

A modal analysis was performed for the building assuming full fixity at the foundation and using the modulus of elasticity of the concrete recommended in ACI 318-05 [4], calculated with the average concrete compressive strengths reported in Section 3.2. The first translational period was 0.49 s and corresponded to a combined lateral-torsional mode, see Figure 8. The second translational period was 0.36 s and corresponded to a longitudinal mode.

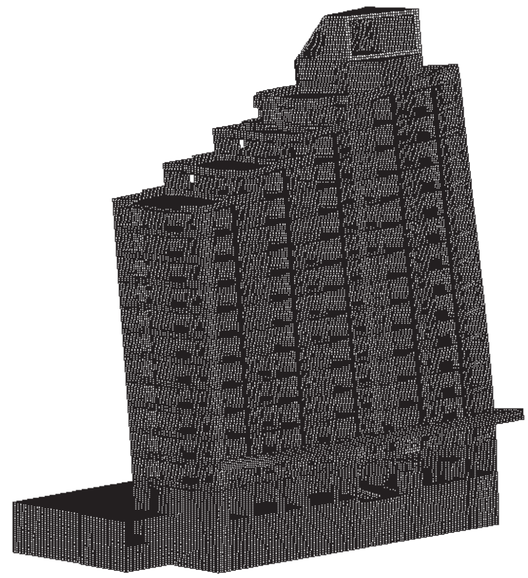


Figure 8- First translational (lateral-torsional) mode identified for the building.

5.3 Pushover Analysis Procedure

The pushover analysis was performed in two phases. In phase 1, lateral forces were applied incrementally with a load matrix that had the shape of the first transversal mode. The lateral forces were applied in many locations in the slabs, and pushing the building towards Ave. Arturo Prat. The lateral force resultant for this load matrix was at 23.78 m from Floor 1. These forces were applied until near maximum lateral load when a near singular matrix hampered the analysis, where ABAQUS can exhibit lack of convergence. In phase 2, loading was re-started from a lateral load level in phase 1 prior to stopping the analysis. Incremental lateral displacements were applied at nine points distributed throughout the slab of the tenth story at an elevation of 23.30 m from Floor 1. The displacement based phase of the pushover was used to obtain the peak lateral force and to also obtain the softening response. Non-linear geometry was considered in both phases of the analysis.

5.4 Cases Investigated

Pushover analyses were conducted for four cases. Case 1 investigated the response of the building built ensuring no lap splice would occur. Case 2 investigated the response of the building assuming all longitudinal bars in Level 1 had lap splices with a response given by *Model a*, see Figure 7. Case 3 investigated the response of the building assuming all longitudinal bars in Level 1 had lap splices with a response given by *Model b*, see Figure 7. Case 4 was identical to Case 3, except that first level columns on Grids 11, 17 and 24 x Grid I. Case 4 studied the collapse hypothesis postulated by IDIEM [7] that the collapse of the building was triggered by failure of the upper end of this columns where there was an abrupt transition.

5.5 Results

Figure 9 shows the pushover curves calculated for the four cases studied and presented as overturning moment at Floor 1 versus relative Floor 13 to Floor 1 lateral displacements. Drift ratios can be calculated dividing the displacements by 30.8 m. Figure 9 also plots the overturning moments about Floor 1 calculated from the lateral forces above Floor 1 derived using NCh433 [1] and factored by 1.4, as required by the standard for soil class II, which was the soil class determined via the standard penetration index and used to design the building.

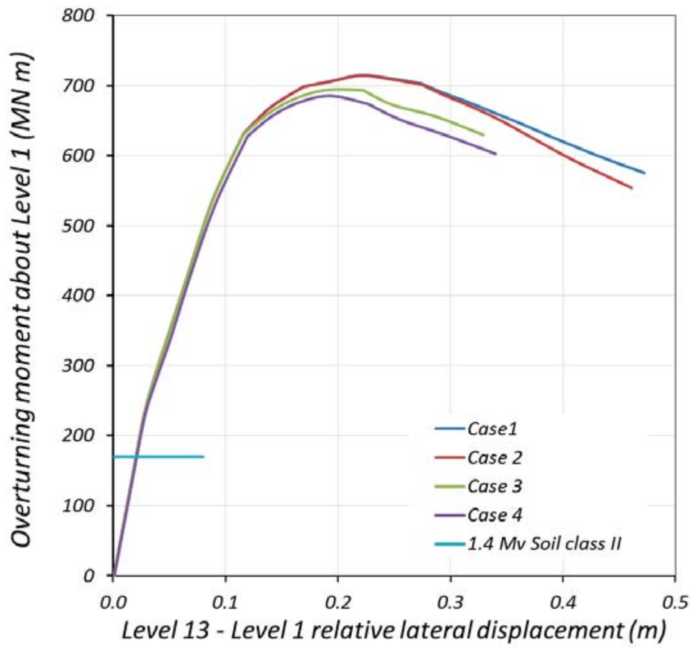


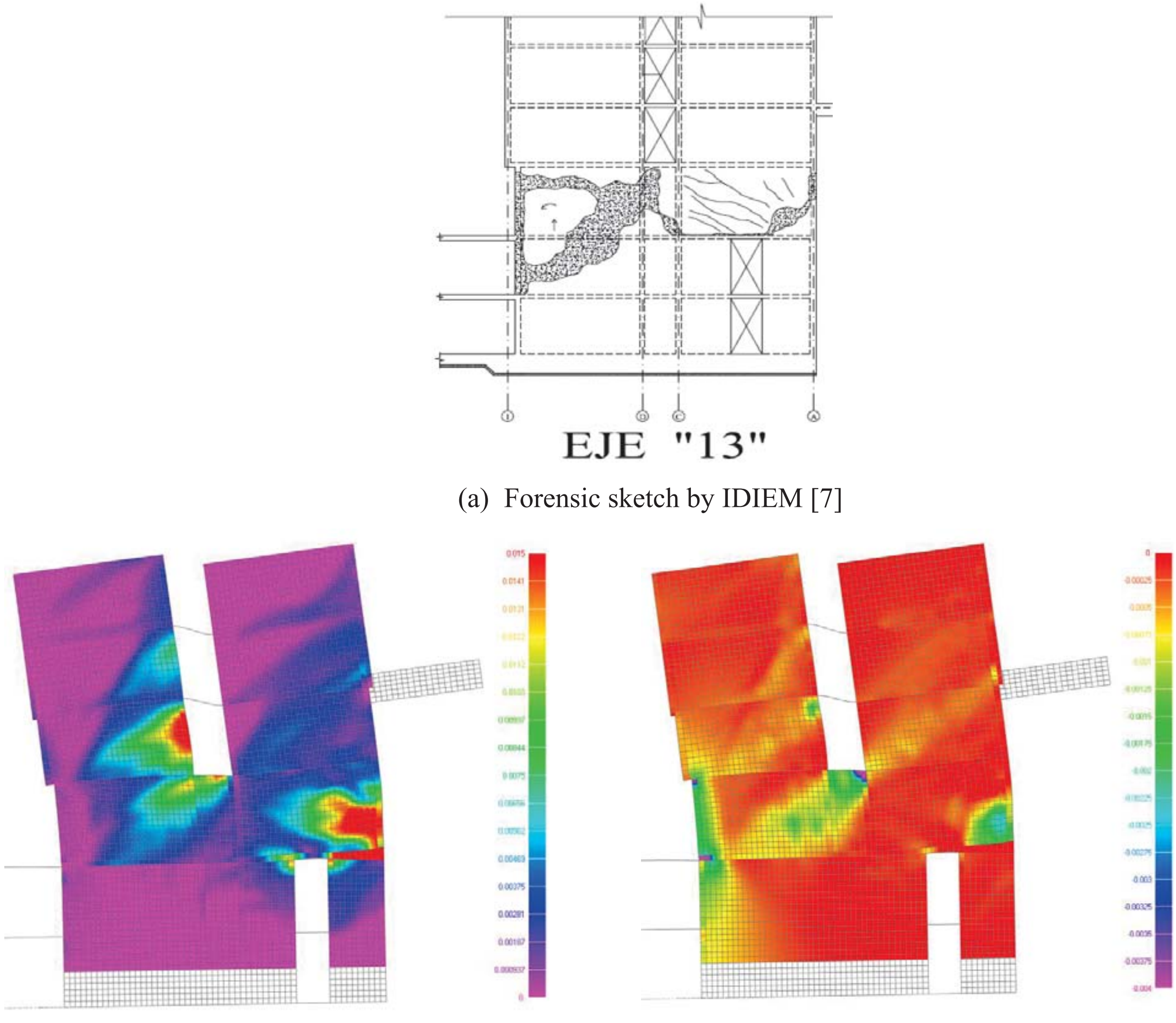
Figure 9- Pushover curves computed for Cases 1 -4.

For all the four cases investigated, the overturning moment calculated at Floor 1 with the design lateral forces above this level and factored by 1.4, as required by NCh433, is significantly smaller than the capacities calculated with the pushover analyses. This is due to the inherent overstrength in the building, which according the pushover analysis is about 4. Parallel effort by Zhang et al. [18] has revealed similar trends, albeit the overstrength computed is 3.5, which is slightly smaller than that computed here. The overstrength is largely due to the effects of unintended slab to wall coupling, to the contribution of wall flanges in tension that were often ignored in design in Chile, and to a lesser extent to the material overstrength and additional longitudinal reinforcement provided in design. In all cases studied, the peak response is attained at about 0.2 m of relative lateral displacement between Floors 13 and 1 measured at the middle of the slab in Floor 13. The drift ratio corresponding to this displacement is 0.65%, which is very similar to that reported by Zhang et al. In all cases softening occurs gradually but in Zhang et al., which reports the results for Case 1, softening is very abrupt. In this study, all four cases reach similar peak loads, indicating little sensitivity to lap-splice failure or lap-splice failure combined with gravity column failure. The main reason why the model is not so sensitive to the lap-splice model is because a number of local failures are observed to occur when degradation of the lap-splices also occurs.

Figure 10 compares the state of damage of the transverse wall on Grid 13 reported in the forensic study of IDIEM [7] with the principal compressive and tensile strains computed for Case 2 when Floor 13 is displaced 0.46 m relative to Floor 1. Although perfect agreement is not expected between an actual structural element and that computed, the concentration of large strains and damage predicted by the model compare favorably with that reported for this wall. Figure 11 shows the principal tensile and compressive strain contours computed for the entire building when the relative lateral displacement at Floor 13 is 0.46 m. The FE model clearly identifies Level 1 as critical and where there is significant concentration of strains in the walls, as well as indicates the development of yield lines in the slabs and beams crossing Grids C and D in all floors. Figure 12 shows the principal tensile strain contours in the structural elements in Level 1 along Grids I and J. The slab of the parking structure on floor 1 has been made invisible for clarity. The presence of a door opening below the flange of the curve wall at Grid J & 24 is found to initiate crushing in the building and this is followed very closely by crushing of the other walls in Grids J&36 Grids I&35 and Grids 5&I. From the perspective of a static pushover analysis, it can be concluded that crushing of the poor detailed curved walls surrounding the stair case could have triggered the collapse of the alto Rio building. It is noted that Zhang et al. (2017), using nonlinear dynamic time history analyses also pointed out that crushing of the curved walls seem to be have triggered the initiation of collapse of the Alto Rio building.

6. Conclusion

A detailed Finite Element nonlinear model was developed for Alto Rio building, which was the only modern building that had a catastrophic collapse during the Mw 8.8 27 February 2010 Maule earthquake. Pushover analyses were conducted for four different cases, including cases where lap splices degraded and a case where in addition to lap splice degradation, three gravity columns that had an abrupt transition at Level 1 were removed. The analyses revealed very little sensitivity to lap splice degradation and to column removal. The pushover analyses indicate that the peak lateral strength is attained at a (Floor 13 to Floor 1) drift ratio of about 0.6% and that the building possessed significant overstrength. The overstrength was assessed to be largely due to unintended slab to wall coupling. The onset of softening, and ultimately of collapse, in the pushover analysis was triggered by crushing of the concrete in walls on Grid J surrounding the staircase of the Alto Rio building. These walls possessed round corners and had been poorly detailed.



(a) Forensic sketch by IDIEM [7]

(b) Principal tensile strains at 0.46 m

(c) Principal compressive strains

Figure 10 – Damage reported in transverse wall on Grid 13 and principal strain contours computed for Case 2 at a relative lateral displacement of 0.46 m in Floor 13.

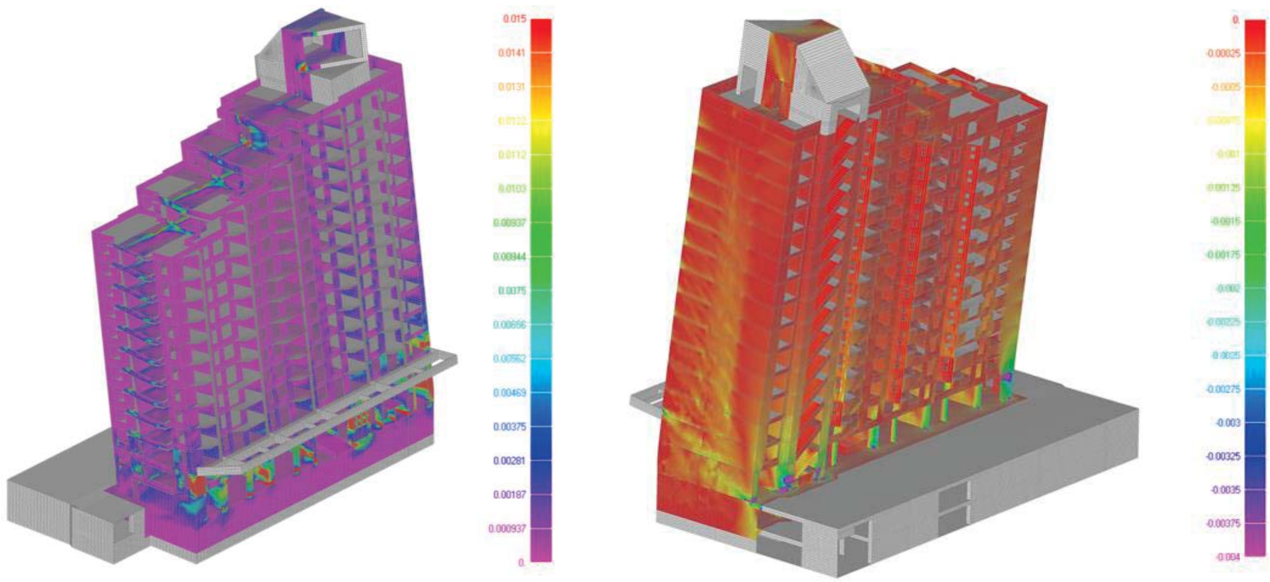


Figure 11- Views of the building when pushed towards Ave. A. Prat 0.46m at Floor 13 relative to Floor 1 (Case 1).

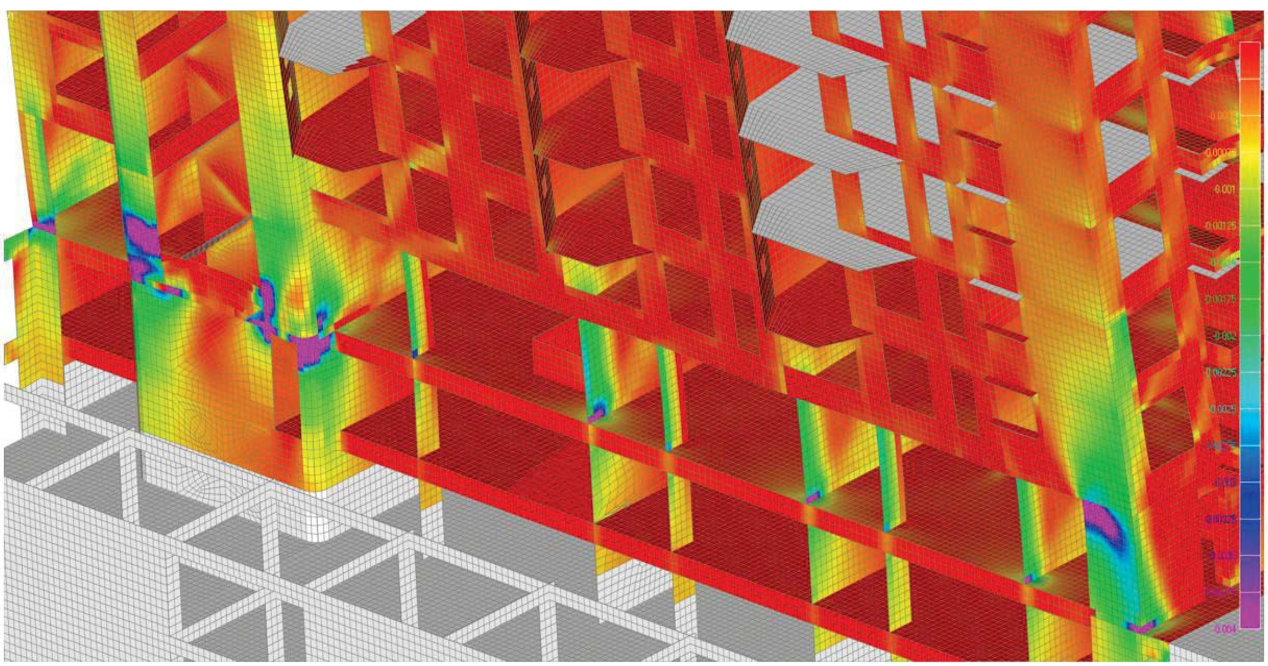


Figure 12- Principal compressive strains at 0.46 m relative displacement of Floor 13 (Case 1).

7. Copyright

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