THREE DIMENSIONAL INELASTIC MODELS TO ASSESS EARTHQUAKE DAMAGE OF REINFORCED CONCRETE WALL BUILDINGS

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ABSTRACT

A non-ductile failure has been observed in some reinforced concrete (RC) structural wall buildings during recent earthquakes in Chile and New Zealand. Although substantial experimental and analytical research has been conducted, several questions still remain about the behavior of structural walls and their 3D interaction with the rest of the structure. This article compares response results of two different inelastic models of one real building that was damaged during the 2010, Chile earthquake. The building suffered severe damage, concentrated at RC walls at the lower levels. The first model is a 3D Finite Element (FE) model developed in the software DIANA and uses 4-node shell elements. Concrete was modeled using the \textit{total strain rotating crack} model, and embedded reinforcement. The second model is a 3D-linear elastic FE model of the building combined with inelastic force-based fiber elements recently proposed for RC walls. Inelastic dynamic analyses were performed for both models showing an important increase in axial load in the walls, which seems to be critical in producing the same localized brittle bending-compression failure observed in the actual building. Consequently, these models could be used in practice to assess the condition of existing structural wall buildings and propose effective retrofit strategies.

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ABSTRACT

A non-ductile failure has been observed in some reinforced concrete (RC) structural wall buildings during recent earthquakes in Chile and New Zealand. Although substantial experimental and analytical research has been conducted, several questions still remain about the behavior of structural walls and their 3D interaction with the rest of the structure. This article compares response results of two different inelastic models of one real building that was damaged during the 2010, Chile earthquake. The building suffered severe damage, concentrated at RC walls at the lower levels. The first model is a 3D Finite Element (FE) model developed in the software DIANA and uses 4-node shell elements. Concrete was modeled using the total strain rotating crack model, and embedded reinforcement. The second model is a 3D-linear elastic FE model of the building combined with inelastic force-based fiber elements recently proposed for RC walls. Inelastic dynamic analyses were performed for both models showing an important increase in axial load in the walls, which seems to be critical in producing the same localized brittle bending-compression failure observed in the actual building. Consequently, these models could be used in practice to assess the condition of existing structural wall buildings and propose effective retrofit strategies.

Introduction

Reinforced concrete (RC) wall buildings are widely used in seismic countries such as Canada, Colombia, New Zealand and Chile [1],[2],[3],[4], among others, as a typical lateral force resisting system to provide an effective seismic load path for lateral motions in buildings, and simultaneously carry vertical loads to the ground. In Chile, approximately 78% of the RC building stock are shear wall buildings [5],[6],[7] typically intended for residential use and with a “fishbone” type floor plan configuration; this is a typical plan with a central longitudinal corridor with shear walls and transverse walls that separate building apartments and interior rooms [8]. Although these buildings showed, in statistical terms, an adequate seismic performance during the recent large magnitude 2010, Chile earthquake, a repetitive brittle failure was observed in many cases, characterized by concrete crushing and buckling of the vertical reinforcement in a zone that
extends from the wall boundary well into the web of the wall, typically observed in the lower building stories [9],[10],[11],[12],[13]. Buildings with similar damage in New Zealand after 2011 Canterbury earthquakes were also observed [4].

Significant experimental and analytical research has been conducted in order to explain the observed behavior. On the one hand, most of the experimental research on RC walls has been focused on studying isolated RC walls, reaching to the conclusion that confinement might not be enough to achieve an acceptable ductility for thin elements [14],[15]. Moreover, the higher the axial load, the more brittle the failure and the lower the drift capacity [16],[17],[18]. On the other hand, conventional simplified 2D models of isolated walls have been unable to represent the expected behavior of real buildings, since they generally neglect the interaction between the wall and the rest of the structure [19].

The objective of this research is to use two completely different inelastic models to understand the failure mechanism and reproduce the observed damage pattern of a RC wall building severely damaged during the 2010, Chile earthquake. The first is a Finite Element (FE) model developed in the software DIANA [20]; and second, a Fiber model for walls programmed in MATLAB [21] based on previous work [22].

**Building description**

Different RC wall buildings located in Santiago, Viña del Mar and Concepción were reported with significant damage during 2010, Chile earthquake [9],[10],[13]. One of them was selected as a test-bed building to reproduce the expected damage pattern. It is a residential building located in Santiago, Chile, as shown in Fig. 1a) and b). The building has 18 stories and 2 basements, and has a typical “fish-bone” like configuration with central longitudinal walls in the East-West direction, and orthogonal walls in the North-South direction. The floor plan of the first basement is shown in Fig. 1c). The building is composed of 15 cm thick floor slabs and typical 20 cm thick RC walls with an average total wall density of 6.5% (total area of walls in both directions divided by tower area, calculated at the first level above grade). The building was designed with $f'_{c} = 25$MPa concrete compressive strength and steel A630-420H ($f_{y} = 420$ MPa). The basements are destined to car parking and present an enlarged floor plan area. The NEHRP site class is C, according to the office that designed the building.

Field investigations conducted after the earthquake showed that most severe damage was localized in the transverse RC walls of the north side of the building in the first basement level at axis U, Q and N1, as shown in (Fig. 1c)). The damage pattern of these walls is shown in Figs. 2a), b) and c) respectively. Damage was characterized by concrete crushing and reinforcement buckling, which affected almost the entire length of the wall, compromising the vertical stability of the elements. These walls present a similar configuration in height, with an irregularity in the wall length in the transition between the basements and the first story were the damage was concentrated, as is schematically shown in Fig. 1d) for wall Q. Additionally, walls in axis O and X and columns in axis 10 (see Fig. 1c)) suffered moderate damage at first basement. No significant damage was reported in RC slabs. Concrete crushing and reinforcement buckling was observed at the opposite side in wall N2 (Fig. 1c)), but in this case, the damage was concentrated at the base of first story level, instead of the first basement.
Analytical models

First, a linear elastic model of the building was developed in SAP2000 [23], called SAP-L hereon, and it is used as a reference. Gross section properties were used, with a concrete Young’s modulus $E_c = 22.6$ GPA. Thin shell elements were used for walls and slabs, and frame elements for beams and columns. The mesh was compatible and no rigid diaphragms were included at floor levels.
The model was restrained at the base of the lowest basement, where only displacements were restricted, and soil-structure interaction was not considered. The mass of the model was estimated as the sum of the mass coming from dead loads plus 25% the mass coming from live loads. Dead loads are represented by a load per unit area of 980 $N/m^2$ while typical live loads were 1960 $N/m^2$. The fundamental period of the building is 0.89 s and corresponds to a translational mode in the north-south direction of the building.

Based on the SAP-L model of the building, a Finite Element model in DIANA (FE-NL) and a Fiber model in MATLAB (Fiber-NL) were built to account for nonlinearities on walls of the first three lowest levels of the building. All vertical elements from second story and up are modeled with linear-elastic behavior, as well as slabs at all levels. This assumption is justified since most of the damage was localized at the first basement (level -1) as it was commonly observed in damaged buildings during the 2010, Chile earthquake [12],[13],[24].

Finite element model in DIANA (FE-NL)

This is a finite element model with inelastic material behavior using the software DIANA [20], Fig. 3a)). All vertical elements at the first three levels of the building (i.e. the two basements and the first level above grade) were modeled with inelastic behavior, with the exception of perimeter walls and columns at axis 12 (Fig. 3b)).

Figure 3. Finite element model in DIANA: a) 3D view of the building; and b) inelastic elements at first three levels.

To model concrete behavior, the smeared cracking approach is considered, following total strain rotating crack model. In this approach, cracked concrete is treated like a continuum material with its own stress-strain constitutive model, and the principal axes rotate with the principal strains during crack propagation. Compressive stress-strain constitutive for concrete is modeled using the Parabolic curve implemented in the software, where the softening branch is modified according to the element size in order to keep the compressive fracture energy constant, which reduces the mesh dependency effects. This model has been previously used showing good results for 2D RC walls [19],[25]. The walls presented almost no boundary confinement, thus only non-confined concrete was considered. For tensile behavior, Hordijk constitutive model is used. Fig. 4a) shows the theoretical stress-strain constitutive relation used for concrete in this case. Reinforcement is modeled using an embedded formulation, which assumes perfect bonding. The stress-strain
constitutive model for steel is the same for tension as it is for compression, and it considers exponential hardening (Fig. 4b)). This model does not include the effect of bar buckling or fracture.

Figure 4. Material uniaxial stress-strain behavior: a) concrete; and b) steel.

Gravity loads are applied as a first load case and equilibrium is achieved through regular iterative Newton-Rapshon method. Rayleigh damping was considered in the model, with a damping ratio of $\xi=3\%$ for $1.5T_0$ and $0.2T_0$, following NEHRP recommendations [26], where $T_0=0.89s$ is the fundamental period of the building model. For nonlinear dynamic analysis, the seismic record is applied as base excitation after application of the gravity loads. The equations of motion in time are integrated through Newmark’s method with a maximum step size of 0.01 s, while the solution of the nonlinear dynamic equilibrium equations are solved through Newton-Raphson method with tangent stiffness matrix with a convergence criterion based on energy norm with a relative tolerance of $10^{-4}$.

Fiber model in MATLAB (Fiber-NL)

The Fiber-NL model combines an elastic finite element model extracted from SAP-L, with inelastic fiber elements for walls in the first three levels. From the Sap2000 software, mass and stiffness matrixes were exported and loaded in MATLAB, then, the selected inelastic walls in the first three levels of the building were replaced with force-based fiber elements. Fig. 5a) shows a wall of shell elements connected to the force-based fiber element used (Fig. 5b) and c)) [22]. The fiber element maintain the cross section given by the shell elements, but incorporates several nonlinear behaviors for each type of fiber. All the concrete fibers used unconfined concrete properties, with a cyclic stress-strain relationship based on the Kent and Park model [27], while the relationship for steel includes the monotonic curve as a bound of the cyclic behavior, the Bauschinger effect, and the bar buckling. The latter was introduced through a softening curve in the region where $\varepsilon < 0$, with a reduced stiffness when reloading from this zone, as can be seen in Fig. 5d. The softening slope representing bar buckling depends on the slenderness ratio of the reinforcement defined as $L/d$, where $L$ and $d$ are the vertical reinforcement length and diameter respectively. Therefore, the bigger the slenderness ratio, the steeper the slope of the softening curve [22]. For simplicity, the length $L$ was taken as the spacing of the transverse reinforcement. All the fibers are regularized taking the length associated with each integration point, as explained elsewhere [22], where a detailed description of constitutive models can be found. Then, as in the FE-NL model, the Newmark method with Newton-Raphson iterations were used.
In this section, results from linear and nonlinear history analyses using the FE-NL and Fiber-NL models are compared. The focus is placed on the critical walls U, Q, N1 (Fig. 1c)). Results are shown for the PuenteAlto record, which was registered during the 2010 Chile earthquake close to the building (11.37 km) and with similar soil conditions (NEHRP site class C). The North-South (NS) component of the seismic record presented a PGA of 0.27 g (Fig. 6) and was applied to the Y direction of the building.

The Fiber-NL and FE-NL models predict similar inelastic roof displacements along Y direction over the entire response as shown in Fig. 7, where the history of the lateral roof displacement at the center of mass of the top floor are plotted for the SAP-L, Fiber-NL, and FE-NL models. Wall failures are marked with circles and are defined as the instant when the wall reaches a peak in strength before an evident loss in the load carrying capacity. It is evident that both the instant of failure and the amplitude of displacement at failure match between the Fiber-NL and FE-NL models. Moreover, the instant of failure is the same for the three studied walls.

Fig. 7 also shows that for the first four seconds, responses match almost exactly between the three models. This is due to the limited nonlinear behavior in the Fiber-NL and FE-NL models. However, around \( t \approx 55 \) s, after a cycle of amplitude around 7 cm, inelastic responses part away from the elastic one.
Table 1 shows the roof displacements at failure for each critical wall and for each model. The predicted roof drifts at failure vary between 0.29% and 0.35%, smaller than typical values of around 1% encountered in the literature for Chilean wall buildings [28],[29]. However, small drift ratios at failure of 0.55% have also been observed by some authors [30],[31]. In our case, the limited roof drift ratio predicted is attributed to the large wall density and due to early wall failures triggered by the increased brittleness as a result of high axial loads.

Table 1. Roof displacement and roof drifts at failure for critical walls.

<table>
<thead>
<tr>
<th>Wall - model</th>
<th>U</th>
<th></th>
<th>Q</th>
<th></th>
<th>N1</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fiber-NL</td>
<td>FE-NL</td>
<td>Fiber-NL</td>
<td>FE-NL</td>
<td>Fiber-NL</td>
<td>FE-NL</td>
</tr>
<tr>
<td>Roof Displacement (cm)</td>
<td>14.76</td>
<td>13.19</td>
<td>13.95</td>
<td>13.35</td>
<td>15.09</td>
<td>15.82</td>
</tr>
<tr>
<td>Roof Drift (%)</td>
<td>0.32</td>
<td>0.29</td>
<td>0.31</td>
<td>0.29</td>
<td>0.33</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Fig. 8 shows local results for walls U, Q and N1 in terms of axial load ratio (ALR), versus the roof lateral displacement of each critical wall. The ALR is defined as $P / (A_w f'_c)$ where $P$ is the axial load in the wall, $A_w$ is the cross section area of the wall, and $f'_c$ its nominal concrete strength. As conventionally used, ALR is defined positive for compression. Three distinctive phases are identified in the wall behavior. Theses phases, marked in Fig. 8a) are: i) a rather linear-elastic zone with some inelasticity in the tension side due to concrete cracking, followed by ii) a brittle failure characterized by large axial loads (larger than 50%), and finally, iii) residual cycles. Fig. 8a), b), and c), show the responses for walls U, Q, and N1, respectively.
The ALR at failure for each critical wall and model are shown in Table 2. The ALR at failure predicted with the Fiber-NL model are in general larger than the ones predicted by the FE-NL model with an average difference of 10.5% in ALR between all walls. The difference can be explained due to the difference in the strain distribution. Since the FE-NL does not predict that plane sections remain plane after deformation while the Fiber-NL does, the strain profile is not linear along the wall length, which results in different stresses and ultimately axial loads and bending moments.

Table 2. Axial load ratio (ALR) at failure for critical walls.

<table>
<thead>
<tr>
<th>Wall - model</th>
<th>U</th>
<th>Q</th>
<th>N1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fiber-NL</td>
<td>FE-NL</td>
<td>Fiber-NL</td>
</tr>
<tr>
<td>ALR at failure (%)</td>
<td>63</td>
<td>51</td>
<td>62</td>
</tr>
</tbody>
</table>

Table 3 shows maximum strains before failure at the extreme point A (see Fig. 1d)) of the critical section of walls U, Q and N1 for both Fiber-NL and FE-NL models. Maximum and minimum strains predicted by both models range between -0.21%≤ε≤0.21% at point A, which is the point with most damage predicted by the model and it is also where damage was reported in the actual building. This result shows that no inelastic excursion occurs before failure other than concrete cracking. It evidences that in these cases, the failure of the wall was initiated by concrete crushing rather than buckling of longitudinal reinforcing bars due to cyclic tensile-compressive strains as has been reported in other studies [29]. Additionally, in the opposite side of the wall strains remain below 0.1%, i.e., the wall is almost kinematically restrained by the aisle transverse wall.

Table 3. Maximum strains at point A for critical walls.

<table>
<thead>
<tr>
<th>Wall - model</th>
<th>U</th>
<th>Q</th>
<th>N1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fiber-NL</td>
<td>FE-NL</td>
<td>Fiber-NL</td>
</tr>
<tr>
<td>Maximum strain ε_max (%)</td>
<td>0.12</td>
<td>0.15</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Conclusions

Inelastic dynamic analyses of a reinforced concrete wall building that suffered severe damage during the 2010, Chile earthquake were performed using two completely different 3D inelastic models, namely, a Fiber model in MATLAB (Fiber-NL), and a Finite Element model in DIANA (FE-NL). The Fiber-NL model combines an elastic linear model from SAP, with force based fiber elements to model inelastic walls concentrated in the first three stories, since observed damage was concentrated there. The material behavior considered in this case accounts for the Bauschinger effect, bar buckling, and fracture for high tensile strains. On the other hand, the FE-NL model is a 3D finite element model in which nonlinear constitutive behavior of concrete has been considered in elements located at the first three stories. The reinforcement was also considered, but the constitutive model in this case does not consider bar buckling nor fracture. Nonlinear response history analyses were carried out using a real seismic record registered during 2010 earthquake and measured in the same city and in a site with similar soil conditions than the studied building.
Although with very different formulations, both models predict that the failure of walls was non-ductile, with a sudden loss of load carrying capacity after reaching ALR larger than 50% and associated to roof drift ratios around 0.3% approximately. The cyclic response of the walls shows that they behaved essentially elastic in compression until they reached a peak ALR and bending moment, which suggest that a bending-compression failure occurred. The main source of inelasticity before failure was due to concrete cracking in tension, without significant steel yielding. Additionally to the brittleness of each wall, in several cases, more than one wall failed in the same cycle, threatening the behavior of the entire building.

Finally, it is remarkable that the two inelastic models, with completely different formulations predict a comparable brittle behavior, with similar forces and displacements. This undoubtedly make the models more robust and the predicted behavior more credible, especially when these results match the reported damage of the building after the 2010 Chile earthquake. It is expected that the models show good match for other buildings with brittle wall failure, while larger differences between models are expected for buildings with other failure modes. In the future these results should be validated for other buildings undergoing similar damage

Acknowledgments

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