PERFORMANCE-BASED SEISMIC EVALUATION AND RETROFITTING OF HISTORIC UNREINFORCED CONCRETE WALL BUILDINGS WITH INTERIOR STEEL FRAMES

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ABSTRACT

Performance-based seismic evaluation of historical constructions is a challenge due to the difficulty of their structural analysis that accurately captures their nonlinear behavior. This paper focuses on the use of 3-D nonlinear finite element method for static pushover analysis of unreinforced concrete (URC) wall buildings with interior steel frames for seismic evaluation and retrofit design. Three such historical school buildings with different levels of structural complexity were modeled in detail using advanced finite element programs to obtain their linear and nonlinear behavior under monotonically increasing lateral loads. The capacity curves obtained from pushover analyses were used for performance evaluation using the Capacity Spectrum Method. Retrofit design verification for one of the buildings was performed using the same approach. The results show that despite its involved modeling process and computational expense, pushover analysis of buildings with URC/URM elements using 3-D nonlinear finite element method can become a powerful practical tool for improved seismic performance evaluation and retrofit design.

Keywords: Performance-based design, Seismic evaluation, Pushover analysis, Retrofitting

1. INTRODUCTION

Performance-based seismic evaluation and design procedures are commonly used for buildings around the world to mitigate the increasing seismic exposure and risk in urban areas [1-3]. For low to mid-rise frame buildings, the behavior of which are controlled by their first mode shape vector, nonlinear static pushover analysis has proven itself as a fairly accurate and efficient tool for performance based design and evaluation [4, 5]. Buildings with URC or URM structural walls, however, continue to be a challenge for researchers and practitioners alike due to difficulties in their nonlinear pushover analysis. Historic structures present a special challenge due to the limited information regarding the material properties and the structural system. Research in this area is mainly concentrated on URM buildings due to relatively modest number of historic buildings with URC structural elements. However, the methods developed for nonlinear analysis of URM structures can be easily extended to URC structures due to similarities in their macro-scale behavior and failure modes. The research presented in this paper is a comprehensive 3-D nonlinear finite element modeling study performed for buildings with URC structural walls [6]. While the general approach is valid for most URC or URM structures or their components, this study focused on steel frame buildings with URC shear walls, classified as S4 type buildings by FEMA [7, 8]. Buildings with this type of structural system were commonly built in the late 19th and early 20th centuries in the U.S. and around the world. Those still standing today have historical and architectural significance and their seismic performance is a concern for the owners. Damage experienced by both concrete and masonry wall buildings during

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recent earthquakes and the lack of consistent and accurate methods and criteria for their structural evaluation, retrofit and repair is the main motivation behind this research. The main objective of this research was to investigate the use of 3-D nonlinear finite element modeling approach for static pushover analysis of S4 type buildings in order to establish a practical procedure for improved seismic evaluation and retrofit design/verification practices. Three S4 type buildings with different levels of geometric and structural complexity were selected as case studies for pushover analyses. Detailed finite element models of these buildings were constructed using two different finite element programs, SAP2000® [9] and ABAQUS® [10], to determine their linear and nonlinear behavior. Nonlinear material behavior of concrete was modeled using the Concrete Damage Plasticity (CDP) model. Post-cracking tension softening behavior of concrete was modeled through the stress-cracking displacement relationship approach, and an exponential softening curve was used for improved accuracy. The fracture energy value needed to construct the softening curve was obtained from the CEB-FIP Model Code [11] using the characteristics of the concrete samples obtained from case-study buildings. The Riks Method [12] was used as the iterative nonlinear solution procedure. A viscoplastic regularization method was used for stability and convergence of the solution process. The capacity curves obtained from pushover analyses were used to determine the performance level of buildings by implementing the Capacity Spectrum Method [13, 14] described in ATC-40 [1] and ATC-55 [2] reports. Finally, retrofit design verification studies for one of the buildings were performed using the same approach outlined above to investigate the effectiveness of the selected retrofit strategy.

2. DESCRIPTION AND FE MODELING OF THE CASE-STUDY BUILDINGS

The selected case-study buildings are located in Istanbul, Turkey, and are owned by an American high school. Designed and built by U.S. companies in the early 20th century (between 1913 and 1924), the buildings have historical and architectural significance. All three buildings are S4 type buildings with distinct structural characteristics despite their common type. Fig. 1 shows the FE models of the buildings constructed using SAP2000 and ABAQUS programs.

![Building R](image1)
![Building L](image2)
![Building C](image3)

(a) SAP2000 Models

![SAP2000 Models](image4)
![ABAQUS Models](image5)

(b) ABAQUS Models

Fig. 1 Finite element models of the case-study buildings
Building-R (rectangular) is a four storey building with plan dimensions of 15.3 m × 30.5 m and floor heights of 3.5 m in the basement, 4.11 m in the ground floor and 3.9 m in the upper floors. Building-L (L-shaped) is a four storey L-shaped building. The dimensions of the building at the outer edges are 34.9 m by 23.7 m, and those at the inner edges are 20.1 m by 9.6 m, constituting a plan area of approximately 635 m². The floor heights are 3.5 m in the basement and 4.11 m in the remaining floors. Building-C (C-shaped) is a five storey symmetric building with plan dimensions of 53.23 m by 24.25 m at the outer edges, constituting a plan area of approximately 1112 m².

The structural systems of all three buildings are steel frames with concrete bearing/shear walls. The interior steel frames, shown in Fig. 1a, resist the gravity loads. All steel frame members are embedded in concrete with no transverse reinforcement for fire protection and for lateral support. The frame members are connected by riveted connections with little moment resisting capacity. The floor systems consist of reinforced concrete slabs which are simply supported by the exterior walls with the help of their reinforcement embedded in the walls in the form of ninety degree hooks. The lateral load resisting systems of all buildings primarily consist of exterior unreinforced concrete shear walls which also carry part of the floor loads. The fact that all exterior walls are perforated shear walls with rather large window and door openings raised concerns about their seismic performance in view of the complex in-plane load behavior of perforated shear walls.

FE modeling of the buildings was performed using two different finite element programs as shown in Fig. 1. All three buildings were first modeled using SAP2000, a program that enables pushover analysis of frame structures, however is unable to consider material nonlinearity for 2-D or 3-D elements used to model walls. Hence, the same models were also built in ABAQUS, an advanced finite element program with a more involved modeling process, and SAP2000 models were used to verify the ABAQUS models through comparison of building weights and dynamic characteristics (hence the initial lateral stiffness) obtained from linear analyses. ABAQUS was used for all the subsequent nonlinear pushover analyses.

The weight of all buildings and their fundamental periods in two principal axes obtained from linear analyses performed using both SAP2000 and ABAQUS are listed in Table 1. Comparison of the results obtained for both sets of models was used as a verification of the models built in ABAQUS which paved the way for subsequent nonlinear pushover analyses.

### Table 1 Comparison of models built using SAP2000 and ABAQUS

<table>
<thead>
<tr>
<th>Building</th>
<th>SAP2000 Models</th>
<th>ABAQUS Models</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Building Weight (tons)</td>
<td>Mode 1 Period (sec)</td>
</tr>
<tr>
<td>R</td>
<td>3037</td>
<td>0.14166</td>
</tr>
<tr>
<td>L</td>
<td>4530</td>
<td>0.10203</td>
</tr>
<tr>
<td>C</td>
<td>10183</td>
<td>0.20789</td>
</tr>
</tbody>
</table>

3. NONLINEAR MATERIAL MODELS

Proper modeling of the nonlinear behavior of concrete, however, is essential for accurate analysis of URC buildings. Behavior of concrete under tension, generally ignored in linear analyses, is the most important modeling aspect in nonlinear pushover analysis of URC buildings considering that the ultimate failure is most likely to take place through concrete cracking under tension. There are two commonly used models for nonlinear behavior of concrete: Concrete Smeared Cracking (CSC) Model and Concrete Damage Plasticity (CDP) Model. The CDP Model [15, 16] is more effective for modeling the nonlinear behavior of unreinforced concrete since it can represent the nonlinear behavior of concrete by focusing on the stiffness reduction due to damage and failure rather than tracing the crack propagation. The model requires three sets of parameters that describe the compressive behavior, the tensile behavior, and plasticity as explained below.

#### 3.1 Concrete Model in Compression

Modeling of the compressive behavior of concrete in ABAQUS consists of defining a stress-strain curve with a linearly elastic portion until an initial yield stress is reached, followed by a plastic zone with strain hardening prior to the ultimate compressive stress and a post-peak strain softening portion. The initial yield of concrete is assumed to occur at 60% of the ultimate stress. The nonlinear portion of
the stress-strain curve in compression was obtained from Hognestad’s concrete model [17], using the characteristic compressive strength, \( f'_c = 14 \) MPa, obtained from testing of concrete cores taken from the case-study buildings, and the elastic modulus, \( E_c = 3250\sqrt{f'_c} + 14000 \) (MPa), calculated from the concrete strength in accordance with the Turkish Concrete Code [18].

3.2 Concrete Model in Compression Tension

Concrete behavior in tension is linearly elastic until cracking is initiated, after which a tensile strain softening response is observed. Proper modeling of both the tensile strength and the post-cracking tensile softening response is essential for predicting the lateral deformation and load capacity of URC buildings. For the tensile strength, the modulus of rupture calculated as 2.55 MPa according to the Turkish Concrete Code [18] was used since the observed failure mechanism for the analyzed buildings was through flexural cracking of concrete piers near the corners of openings. Post-cracking softening behavior was modeled using the cohesive crack model by Hillerborg et al. [19] in which:

\[
\sigma = f(w), \quad f(0) = f'_c, \quad G_f = \int_0^w \sigma dw = \int_0^w f(w)dw
\]

where, \( \sigma \) is the stress on the unit area of the specimen, \( w \) represents the width of crack opening, \( f'_c \) is the tensile strength of concrete at which cracking is initiated, and \( G_f \) is the cohesive fracture energy that is necessary to create and completely break a unit surface area of cohesive crack. The tension softening curve described by Eq. (1) has an initial value of \( f'_c \) and the area under the curve is equal to \( G_f \).

Several general expressions were proposed to approximate the softening behavior of concrete for use in analytical and numerical studies. The exponential softening curve proposed and experimentally validated by Cornelissen et al. [20] was used in this research for improved accuracy. Once the shape of the tension softening curve was selected, the next step was to determine the value of fracture energy which is ideally determined experimentally by fracture tests such as the work of fracture method proposed in RILEM [21] and by Hillerborg [22]. In cases where it was not possible to extract the specimens needed for fracture tests from buildings, the fracture energy can be obtained from the CEB-FIP Model Code [11] expression:

\[
G_f = (0.0469d_a^2 - 0.5d_a + 26) \times 10^{-3} \left( \frac{f'_c}{10} \right)^{0.7} \quad \text{(SI)}
\]

where \( G_f \) is in N/mm, \( f'_c \) is compressive strength, and \( d_a \) is the maximum aggregate size in mm. For the purposes of this research, three different \( G_f \) values of 125 N/m, 250 N/m, and 375 N/m were used in the pushover analyses to investigate the influence of \( G_f \) values on the analysis results.

4. ANALYSIS PROCEDURE

Nonlinear pushover analyses of the buildings were performed using the arc-length (Riks) method as the iterative solution procedure for the nonlinear algebraic equations [23]. The Riks Method is based on the Newton-Raphson Iteration Method with an improved convergence strategy by use of an arc instead of a line as the constraint on the solution path. Hence, the Riks procedure is less affected by the limit points, in the vicinity of which the tangent stiffness matrix becomes singular and causes divergence in the iteration procedure, improving the probability of obtaining a representative capacity curve for a given structure.

In the first step of each analysis, the gravity loads were applied on the structure instantaneously and were kept constant throughout the analysis. In the second step, lateral loads defined at each floor level in proportion with the equivalent lateral seismic loads [24] shown in Table 2 were ramped from zero. Proportioning the loads according to equivalent lateral seismic loads was preferred to using the first mode shape since the former produced more conservative results. The load increments were determined as part of the Riks procedure based on the stability of convergence and the solution was continued until termination occurred due to computational instability, or when the load increments were reduced to infinitesimal levels. The minimum and maximum arc length increments were set sufficiently small and large values, respectively, and the maximum number of iterations, which is the general termination criterion, was set to a high value to increase the probability of convergence.
Material models that display softening behavior and stiffness degradation may lead to computational problems in implicit programs such as ABAQUS. Introduction of viscoplasticity, also called viscoplastic regularization, is commonly used to overcome some of these difficulties [25-27]. Defining a small viscosity parameter in the material model causes the consistent tangent stiffness of the softening material become positive for sufficiently small time increments, improving the convergence rate in the softening regime.

During the pushover analyses of the case-study buildings, in cases where the capacity curve obtained from an analysis indicated that the analysis was terminated prematurely due to a computational instability or a false detection of unloading, both of which became evident from the load-displacement curve and the deformed shape, then viscoplastic regularization of the constitutive equations was performed by introducing a small viscosity parameter in the CDP model. The influence of viscoplastic regularization on the analysis results is shown in the next section.

| Table 2 Story and base shears calculated using the equivalent lateral seismic load procedure [24]| Story Shears (tons) |
|---|---|---|
| Floor | Building-R | Building-L | Building-C |
| 1 | 128.97 | 183.1 | 293.3 |
| 2 | 247.95 | 373.1 | 580.0 |
| 3 | 355.26 | 532.7 | 669.0 |
| 4 | 543.36 | 814.0 | 909.0 |
| 5 | - | - | 1825.6 |
| Base Shear (tons) | 1276 | 1903 | 4277 |

5. ANALYSIS RESULTS AND DISCUSSION

The capacity curves obtained from nonlinear pushover analyses of the case-study buildings are shown in Fig. 2. Due to the high stiffness and vibration frequencies of the buildings, the seismic design loads are the same in both principal directions. Hence, the pushover analyses were performed only in the weaker directions, i.e. along the shorter plan dimensions of the buildings.

Fig. 2a shows the three capacity curves obtained from analyses of Building-R using three different values of the concrete fracture energy, \( G_F \). While the capacity curves that correspond to \( G_F = 125 \text{ N/m} \) and \( G_F = 250 \text{ N/m} \) appear to be similar, \( G_F = 375 \text{ N/m} \) produces a curve that shows considerably higher lateral load and deformation capacity. The results show that (for the same tensile strength) the value of concrete fracture energy may affect both the load capacity and ductility of buildings with URC components. In view of the drastically different behavior produced by the conservative values of \( G_F \), there is need for further research focusing on the influence of the concrete fracture energy value on building behavior and the suitability of the CEB-FIP Model Code expression as a conservative design expression.

The plan irregularity of Building-L due to its L-shape geometry and the complexity of its finite element model resulted in premature termination of the pushover analyses performed using concrete fracture energy values of 250 N/m and 375 N/m as shown in Fig. 2b. In order to improve computational stability, the viscoplastic regularization procedure explained in Section 3.2 was applied by introducing a viscosity parameter within the plasticity parameters in the CDP Model. Fig. 2b also shows the improvement achieved by introducing a viscosity parameter of \( 10^{-6} \) sec. in the CDP Model. The capacity curves obtained after viscoplastic regularization are consistent with those obtained before the regularization, and provide a more complete representation of the building’s nonlinear behavior that is supported by the evolution of stresses and deformations in the building. The capacity curves obtained for Building-C using two different values of \( G_F \) are shown in Fig. 2c. The figure also shows the effect of viscoplastic regularization for two different values of the viscosity parameter. As can be seen from the figure, the magnitude of the viscosity parameter has little influence on the capacity curve provided that it is sufficiently small compared to the time increments used in the analysis.
Performance evaluation of the case-study buildings were performed using the Capacity Spectrum Method [13, 14] in the acceleration-displacement response spectrum (ADSR) format [28] as initially described in the ATC-40 report [1] and later improved in ATC-55 [2]. This simplified nonlinear procedure provides a clear graphical representation of a building’s performance level and the impact of various retrofit strategies by comparing the capacity spectrum with spectral acceleration response spectra representations of earthquake demands [1]. A performance level is associated with the intersection of the capacity and demand curves, the so-called performance point.

As an initial assessment of the performance level, the capacity curves of the case-study buildings shown in Fig. 3a were converted to ADSR format and were compared with the elastic response spectrum with 5% damping as shown in Fig. 3b. As can be seen from the figure, the capacity spectra of Buildings L and C intersect the elastic response spectrum, which represents the highest level of design seismic demand, at the constant acceleration region. The capacity spectrum for Building-L intersects the elastic response spectrum at $S_d = 0.0035$ m which corresponds to a roof displacement of $\Delta_{\text{roof}} = 0.0044$ m and a drift ratio of 0.03%. Similarly, the capacity spectrum for Building-C intersects the elastic spectrum at $S_d = 0.0148$ m which corresponds to a roof displacement of $\Delta_{\text{roof}} = 0.0188$ m and a drift ratio of 0.09%. The drift ratios for both buildings are significantly lower than the 0.3% limit given in FEMA-356 [29] for immediate occupancy (S-1) performance level for nonfill URM walls. Considering that a nonlinear performance evaluation will reduce the demand spectrum due to increased damping, one can conclude without further analysis that the expected performance level for both Building-L and Building-C are immediate occupancy.
The capacity spectrum of Building-R in Fig. 3b is significantly lower than the elastic spectrum which calls for a detailed nonlinear performance evaluation and possible retrofitting. Results of performance evaluation for Building-R using ATC-40 and ATC-55 capacity spectrum solutions are shown in Fig. 4c. The figure shows that the capacity spectrum of Building-R falls below both ATC-40 and ATC-55 curves with no performance (intersection) point. Hence, it was concluded that the building has collapse potential under design seismic loads and needs retrofitting to improve its performance.

7. RETROFIT DESIGN VERIFICATION FOR BUILDING R

Fig. 4 Retrofitting of Building-R and comparison of capacity curves and building performance before and after retrofitting
Retrofitting of URM and URC buildings can be performed using several methods such as base isolation, bonding FRP polymer sheets or strips, grouted reinforcement in drilled cores, or reinforced shotcrete overlays [30]. Considering the historical and architectural character of Building-R, several feasible retrofit alternatives were evaluated from engineering, economics, completion time, and aesthetics viewpoints. In view of the collapse mechanism observed from nonlinear pushover analyses, installation of steel channel sections around the openings in the exterior walls was found to be the optimum retrofit strategy. Window and door openings in the exterior walls along the width of the building were framed with 2 cm-thick channel sections with 17 cm flange width. Fig. 4a shows one of the exterior walls of Building-R after retrofit application.

Retrofit design and verification for Building-R was performed using the same approach described in the previous sections. The finite element model of Building-R was modified to include the retrofit application as shown in Fig. 4a. Capacity curves obtained from nonlinear pushover analyses of Building-R before and after retrofitting are compared in Fig. 4b, Fig. 4c and 4d show the capacity spectrum solutions for before and after retrofitting, respectively, using the exact procedures described in ATC-40 and ATC-55. Both procedures essentially gave the same results, except for the jump in the ATC-40 curve in Fig. 4c, which was caused by the difference in the acceleration and velocity response reduction factors as the secant period of the structure shifts from constant acceleration region to constant velocity region on the 5% damped elastic spectrum. This problem does not appear in the improved procedure described in ATC-55.

Examination of Fig. 4 shows that while the capacity spectrum solutions show no performance point for Building-R before retrofitting, indicating potential collapse of the building under design seismic demand, the expected performance level after retrofitting is immediate occupancy with hardly any nonlinear deformation demand on the building. This shows the effectiveness of the retrofit strategy through an objective performance evaluation.

8. CONCLUSIONS

As the first comprehensive 3-D nonlinear finite element study performed for buildings with URC walls, this study demonstrates the feasibility of the method for use in practical applications as an improved pushover analysis tool compared to currently used and actively investigated equivalent frame methods. While the value of such approximate methods to the practicing engineer cannot be denied, there is also a need for comprehensive nonlinear finite or discrete element analysis studies to build a collective body of knowledge and experience that will serve as a practical or research tool that can replace, complement, or verify the results obtained from simplified methods. This is especially necessary in view of the fact that the finite or discrete element solutions that are sometimes used to validate the simplified methods are not problem-free themselves when it comes to analysis of buildings with URC/URM structural components. Fracture mechanics concepts can be used to describe the post cracking tensile behavior of concrete, with the fracture energy value obtained from the CEB-FIP Model Code through an inherently conservative design expression. Topics such as experimental verification of this approach for a simple wall configuration, potential mesh-dependency of analysis results, optimum meshing strategies and mesh sizes etc. that were beyond the scope of this research are among the topics of additional research need to build the aforementioned body of knowledge and experience. Despite the current popularity of simplified nonlinear analysis methods, future demand for improved performance-based design and evaluation tools for URC/URM buildings is likely to necessitate use of advanced nonlinear analysis methods at least for those buildings that have functional, historical, or architectural significance. Hence, the authors view parallel research on advanced and simplified methods that can be used in a complimentary fashion as the optimum strategy, and consider the research presented in this paper as an intermediate step in an ongoing research that aims to perform 3-D nonlinear time-history analysis of URC buildings to achieve improved accuracy and objectivity in performance-based design and evaluation.

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