The Evaluation and Retrofit of a Historic Unreinforced Masonry Building Using Nonlinear Adaptive Pushover And Dynamic Analysis Methods

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ABSTRACT :

Various nonlinear analysis techniques were used to analyze a historic unreinforced masonry landmark structure in San Francisco in its original and seismically retrofitted condition. The structure is a monumental 100-year old synagogue that survived the 1906 San Francisco Earthquake. Although the building escaped the Great 1906 earthquake with relatively little damage, the building was recently threatened with closure due to non-compliance with an Unreinforced Masonry Building Ordinance. The most appropriate structural solution consistent with preserving the historic fabric takes advantage of the dynamic separation between the modes predominated by in-plane and out-of-plane wall shaking. The solution consisted of a combination of intervention techniques, each developed to minimize disturbance to the nonstructural historic finishes and retain the original dynamic characteristics. The structure was subjected to linear and nonlinear static and dynamic analyses to benchmark its behavior during the 1906 earthquake. Adaptive pushover analyses were also performed using the first natural mode of vibration of each wall. To validate the full three-dimensional response of the building and to develop design forces for the new structural elements that were added to strengthen the system, a three-dimensional model was constructed in SAP2000 and subjected to static and dynamic analyses.

KEYWORDS: Historic Building; Seismic Retrofit; Nonlinear Analysis; Adaptive Pushover.

1. BACKGROUND

In general, unreinforced masonry bearing wall buildings (UMBs) of most vintages and types tend to pose significant risk of collapse in strong earthquakes. Such risk was acknowledged to be unacceptable legislatively in California when, in 1986, the State enacted legislation requiring local jurisdictions to identify and inventory their UMBs and develop programs for mitigating the seismic hazard these buildings imposed on the community. The subject of this paper, Sherith Israel, is one such property. In this paper, the building history and the retrofit project are briefly described; for more details, the reader is referred to other papers by the authors about this building (*Paret et al 2006, Freeman et al 2006, Paret et al 2007*). The remainder of the paper is devoted to describing the nonlinear analyses techniques used to evaluate the building and to design the retrofitting scheme.

2. HISTORY AND DESCRIPTION OF SHERITH ISRAEL

Founded in 1849, Congregation Sherith Israel was a pioneer synagogue of the American West. The congregation occupied two other buildings before commissioning the design of the present synagogue, located at the corner of California and Webster Streets in San Francisco. Designed by Albert Pissis, a prominent San Francisco architect trained at the Ecole de Beaux-Arts, Sherith Israel was constructed in 1904. A monumental structure with Classical and Romanesque design elements, the building has an ornately painted interior, has thick masonry bearing walls, is clad in Colusa sandstone, and is capped by a 60 foot dome (Figure 1). The thickness of the exterior brick walls vary from a maximum of roughly six wythes at the building base to four wythes above the sanctuary level. These walls are perforated by numerous window and door openings. As a result of articulation at archways in each elevation and at other locations, the exterior walls are locally substantially thicker than the nominal dimensions of the wall field.

With the exception of a roughly 20-foot (6 meter) wide strip around the perimeter of the building, interior loads above the entry level (which is supported directly on strip footings), including the weight of the dome and drum, are supported by riveted structural steel framing composed of heavy steel trusses and six built-up columns, providing an open and nearly column-free sanctuary space.

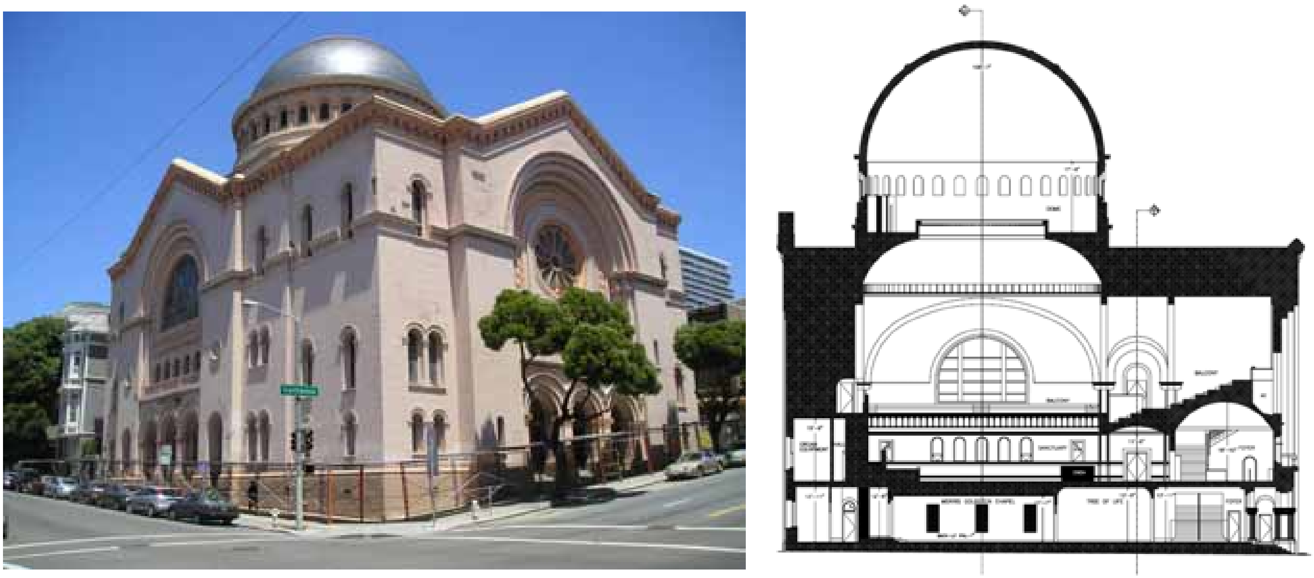

Figure 1 Exterior view of the southwest corner, and a north-south cross-section of Sherith Israel

3. STRENGTHENING TECHNIQUES

The dynamic behavior of the building is dominated by the response of the perimeter masonry walls, which contribute roughly 85% of the mass of the entire structure; thus, appropriate treatment of these walls is critical. Due to the presence of only very flexible diaphragms above the sanctuary floor level, significant dynamic separation between the in-plane and out-of-plane modes of the walls exists that strongly influences the development of inertial forces. This dynamic separation is structurally advantageous in that a significant proportion of the global mass of the building responds essentially as a long period structure (out-of-plane) subject only to modest spectral accelerations while a significant proportion of the global mass of the building acts effectively as a very short period structure (in-plane) with little or no amplification. The development of strengthening techniques that preserved this natural separation (as opposed to eliminating it as prior conceptual strengthening measures had proposed) was made a priority, second only to the goal of preserving the historic fabric of the building.

The structural solution selected to meet the intent of the UMB upgrade ordinance consists of a blend of traditional and non-traditional components, including installation of center-cored reinforcement in the masonry walls anchored into a reinforced concrete bond beam at the parapet coping; positive floor-to-wall anchorage details at all diaphragm levels; localized fiber-composite wrap at three critical brick masonry columns; and a system of Nitinol tension ties in the attic that interconnects the four perimeter walls at the gable end walls and is intended primarily to resist out-of-phase response of walls on opposing sides of the sanctuary.

4. NONLINEAR PUSOVER ANALYSIS

Nonlinear pushover computer analyses were performed using the nonlinear finite element program ADINA (*ADINA R&D 2005*) to evaluate the seismic behavior of the perimeter masonry walls of the building and to investigate the efficacy of the proposed retrofit schemes. The building has a generally rectangular plan with four primary perimeter masonry walls. The seismic behavior of each of the walls was evaluated. Each wall was modeled and analyzed under both in-plane and out-of-plane seismic forces in addition to gravity loads. Since the east and west walls are nearly identical, only three different primary wall models were built: north, south and west.

4.1. Model Geometry

Each wall was modeled in ADINA as a three-dimensional solid body with all the windows, doors, arches and articulations of the wall carefully represented in the model; this resulted in large and complex three-dimensional wall models with thicknesses that vary along the height and length of each wall as well as with openings of varying sizes and shapes. Several models were constructed for each wall to represent different loading conditions and strengthening options discussed above. The final analysis models included all the strengthening options recommended for each wall, with the exception of the attic Nitinol tension tie system, in order to understand the behavior of the walls if the response of the walls on opposite sides of the sanctuary were in-phase.

The geometry of each wall was initially modeled in AutoCAD and then imported into the ADINA User Interface (AUI). The ADINA mesher was then used to discretize each model into three-dimensional finite elements which can be analyzed by ADINA (Figure 2). The size of the mesh was constrained by the maximum problem size that ADINA can run, and hence the element size was generally about 18 to 24 inches on average. High-order 20-node three-dimensional solid elements were typically used in the analysis to model both masonry and concrete. The 20-node elements are superior to the 8-node elements more typically used in the modeling of solids, and are especially effective in modeling structures and components that behave predominantly in bending such as walls (out-of-plane) and slabs. The number of nodes and elements in a typical model is shown in Table 1 for the north, south and west walls. Figure 2 shows the three-dimensional ADINA models for the different walls.

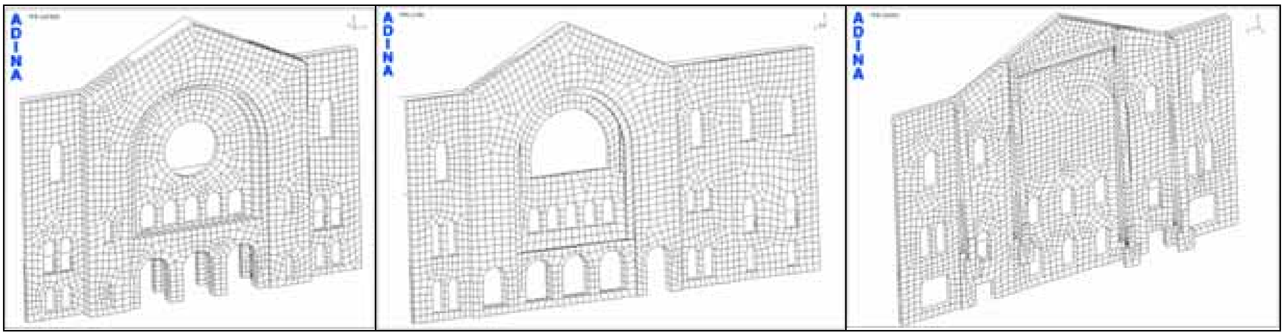

Figure 2 ADINA meshes for the south, west and north walls, respectively.

Table 1: Mesh properties for the north, west and south wall models

	Element Size (ft)	Number of Nodes	Solid Elements (Wall)	Truss Elements (Steel Bars)	Beam Elements (Diaphragm)
North Wall	1.5	72735	30839	10849	196
West Wall	2	45892	18679	8317	187
South Wall	1.9	48984	21331	7200	148

4.1. Application of Seismic Loading

The analysis of each wall started by applying the gravity weight of the wall. This was followed by a pushover analysis in which seismic forces were applied laterally to the wall. The walls were assumed to be fixed at their bases and laterally supported at their edges by the perpendicular walls. The seismic forces were applied either perpendicular or parallel to the wall surfaces in order to simulate seismic loading of the wall out-of-plane or in-plane, respectively. The purpose of each of these analyses was to obtain a relationship between the applied seismic force and the resulting displacement of the top of each wall, i.e. a pushover curve. This was done using two different approaches: a static pushover analysis procedure, and an adaptive dynamic pushover analysis procedure.

4.1.1 Walls Loaded Out-of-Plane

The out-of-plane analyses were developed to simulate seismic behavior resulting from forces that the walls experience during an earthquake due to their out-of-plane mode of vibration. In this type of analysis, since the mass of the diaphragms tributary to each wall is only a very small fraction of the mass of each wall, it was assumed that each wall only needs to resist out-of-plane lateral forces arising from its own inertial mass. The perpendicular walls were assumed to provide out-of-plane translational restraint but little rotational restraint at the edges of the walls being studied. This restraint was idealized by using translational supports at the wall vertical boundaries. These analyses also considered the effect of the lateral support provided by the retrofitted diaphragms at the sanctuary, balcony, and roof levels.

4.1.2 Walls Loaded In-Plane

The in-plane wall analyses were developed to simulate seismic behavior resulting from in-plane seismic forces resisted by each wall along its length. In this mode of behavior (in-plane), the walls represent the building's primary lateral force resisting elements and resist seismic forces due to their own inertial mass in addition to tributary inertial loads from the connecting orthogonal walls and lateral loads from the floor and roof diaphragms.

4.2. Static Pushover Analysis Procedure

The static pushover analysis is a force-controlled procedure in which the distribution of the seismic forces along the surface of the wall is typically assumed constant throughout the pushover analysis. The distribution can be either uniform, or proportional to the deformed shape of the wall under the predominant mode of response (in-plane or out-of-plane). The modal loading conditions due to the in-plane and out-of-plane modal excitations were represented by a distribution of the applied forces along each wall proportional to the primary mode shape in each direction as determined from a global linear dynamic analysis - performed in SAP2000 (*Computers and Structures 2004*) - of the whole structure. The in-plane seismic force distributions typically corresponded to a parabolic distribution of seismic forces with height. In contrast, the out-of-plane seismic force distributions corresponded to the primary out-of-plane mode shape and generally had a more complex two-dimensional shape that varied across the wall surface as a result of the translational supports along the three edges of each wall in the out-of-plane direction. The modal mass participation ratio was typically about 0.45 for out-of-plane behavior, and about 0.85 for in-plane behavior.

The analyses were performed by applying a small increment of the seismic lateral force, which was then gradually amplified in subsequent steps while monitoring the displacement of the roof. The resulting pushover relation between the total applied force and the roof displacement typically exhibited a softening effect due to cracks that develop in the masonry structure as it was pushed beyond its elastic limit, in addition to the yielding of the diaphragm and steel reinforcement in the retrofitted configuration of the wall (Figure 3). The analysis was terminated when the target applied force was reached or when the finite element model failed to converge to a solution after a certain number of iterations. The latter typically governed as the wall became severely cracked, which caused numerical convergence difficulties in the finite element model as more and more iterations were required to make the analysis converge to a solution with an acceptable error tolerance. Generally, this behavior can be somewhat improved by using smaller increments of the applied forces in each step, but this does not always work predictably, and can be very computationally expensive and time consuming. However, as long as the wall can be demonstrated to have enough strength and displacement capacity to withstand the design earthquake event, the performance can be deemed satisfactory and the analysis need not be continued beyond that point. Another shortcoming of the static pushover method is that the distribution of the lateral forces is typically assumed to stay constant throughout the analysis, and hence is independent of the level of damage and cracking in the wall and independent of the resulting evolution of the mode shapes. In order to overcome some of these difficulties, a second method of analysis --- as described below --- was developed.

4.3. Adaptive Pushover Analysis Procedure

In order to overcome some of the shortcomings of the static pushover analysis procedure and provide a more realistic representation of the force-displacement response of each of the structural masonry walls, a displacement-based adaptive pushover procedure was developed that results in a better approximation of the nonlinear wall response while avoiding some of the problems of the static pushover procedure. This procedure consists of the application of small successive displacement increments that are proportional to the wall's predominant mode shape in the direction of interest, and computation of the wall's base shear resisting force at every step. The procedure takes into account the nonlinear behavior of the wall, including cracking of the masonry and yielding of the steel reinforcement and diaphragm during the analysis. Since the mode shape used in the analysis is re-computed at the end of each step in reflection of the instantaneous properties of the structure after the application of the displacement increment, the method results in an adaptive pushover procedure in which the mode shape changes continuously during the analysis. One caveat of this approach is that not only will the pushover envelope be different from that obtained from a static pushover analysis, but it may also depend on the selected size of the displacement increment. Ideally, very small displacement increments should be used, and modal properties should be computed after each increment. Since this is usually infeasible (i.e. time- and cost-prohibitive), larger displacement increments are usually used. Care needs to be taken to select displacement increments that are small enough to capture the progressive change in the wall's stiffness with increasing deformation.

The adaptive pushover analysis was achieved by performing a dynamic analysis using ADINA's "Modal Superposition" feature. Modal superposition is a nonlinear analysis technique that allows the use of a subset -- usually a small number -- of the structure's mode shapes to approximate the dynamic response. The method is usually used to perform nonlinear time history analysis of large problems that are difficult to analyze with traditional implicit integration methods. The adaptive pushover response of the structure can be viewed as the dynamic response of the structure due to a single, but varying, mode. Hence, the modal superposition method was used, with a single mode, to obtain the pushover envelopes for the various walls. Different types of excitations (pulse, sine wave, constant acceleration etc…) were used to load the wall mode, but since only one mode shape was used in the modal superposition analysis, the pushover envelope was generally independent of the loading being used. The only effect of the loading is that the different rates of movements (velocities) resulting from the different loadings can result in a difference in the frequency of re-calculation of the modal properties. Because of this, the pulse loading was generally preferred since it resulted in a semi-uniform velocity during most of the analysis and caused the modal properties to be re-computed at fairly regular displacement increments. In some of the analyses, a more frequent rate of modal updating was specified during the initial portion of the pushover curve, during which the structure experiences a significant reduction in its stiffness.

4.4. ADINA Analysis Results And Interpretation

The results from the ADINA pushover analyses were evaluated and interpreted to derive important performance information at various displacement levels. The pushover curves were also compared to spectral demand curves that were developed as the design basis for the seismic retrofit using the Capacity Spectrum Method (*Freeman 1998*). Figure 3 and Figure 4 show the pushover curves for the three analyzed walls both out-of-plane (left) and in-plane (right). In Figure 4, the pushover curves were converted to the ADRS format (S_d versus S_a) to allow easy comparison with demand spectra modified for different ductility levels and their equivalent effective damping (*Freeman 2006*). The spectral displacement S_d was obtained by dividing the roof displacement by the roof participation factor which ranges between 1.15 and 1.54 (Table 2). The spectral acceleration S_a was obtained by dividing the total base shear by the effective modal weight in the principal mode of vibration.

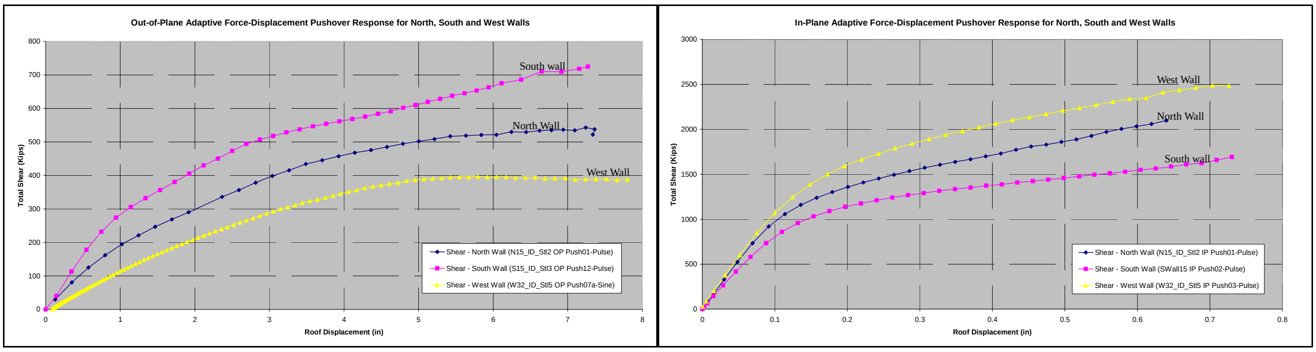

Figure 3 Adaptive pushover (Shear vs Displacement) of north, south and west walls out-of-plane & in-plane

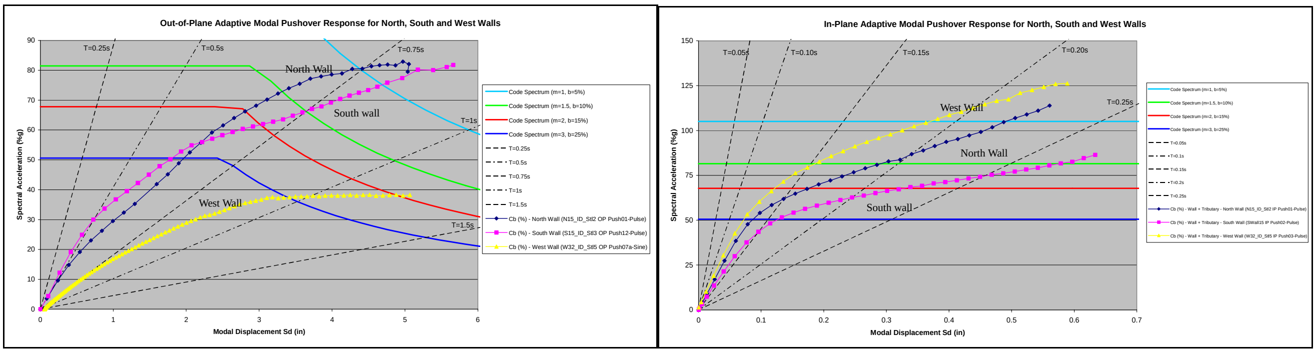

Figure 4 Adaptive pushover response (Sa vs Sd) of north, south and west walls out-of-plane & in-plane

Table 2. Wall expected displacement demands using the Capacity Spectrum Method

Masonry Wall	Direction of Loading	Spectral Displ., Sd (inches)	Roof Participation Factor	Roof Displacement (inches)	Effective Ductility
South Wall	Out-Of-Plane	2.77	1.29	3.57	2.2
North Wall	Out-Of-Plane	3.22	1.43	4.61	1.6
West Wall	Out-Of-Plane	4.74	1.54	7.29	2.1
South Wall	In-Plane	0.21	1.16	0.24	2.4
North Wall	In-Plane	0.16	1.15	0.18	2.1
West Wall	In-Plane	0.13	1.28	0.16	1.9

Figure 5 shows the pushover curves for the south wall both parallel and orthogonal to its plane (in-plane and out-of-plane). As described earlier, in the out-of-plane direction, the wall was assumed to resist inertial forces due to its own self weight only, while in the in-plane direction, the wall resists inertial forces due to self weight in addition to tributary inertia loads from the connecting orthogonal walls as well as the interior elements of the building. The modal weight for the in-plane pushover cases were adjusted to account for this additional inertia, which results in an apparent, but fictitious, reduction of capacity when compared to the pushover curve for the wall only, and is represented by the second curve Figure 5 (right).

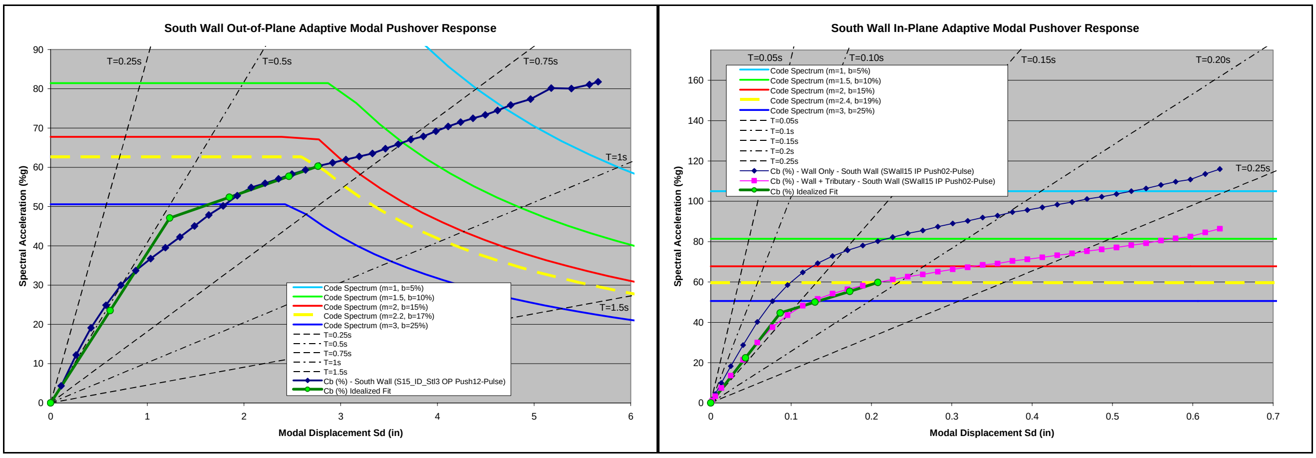

Figure 5 Capacity Spectrum demand of south wall in the out-of-plane (left) & in-plane (right) directions

The displacement demand for each of the walls was computed using the Capacity Spectrum Method. In order to define the ductility associated with a given displacement level, a bilinear idealization of the pushover curve, providing a best fit to the actual curve, was used to estimate the ductility demand (Figure 5) . The maximum expected spectral displacement is then estimated by interpolating between the different demand curves until convergence is obtained. The bilinear idealizations for the south wall are shown Figure 5, and the expected peak displacements and ductilities are summarized in Table 2 for all three walls. The expected displacement demands range from 3.6 inches to 7.3 inches (9 cm to 19 cm) in the out-of-plane direction (corresponding to a displacement ductility of 1.6 to 2.2), and from 0.16 inches to 0.24 inches (0.4 cm to 0.6 cm) in the in-plane direction (corresponding to a displacement ductility of 1.9 to 2.4).

Sample ADINA analysis results are presented in Figure 6. The figure shows deflected shapes and masonry cracking patterns for the south wall loaded both in-plane and out-of-plane. For visualization purposes, the deflected shapes shown are amplified by a factor of 50 for loading out-of-plane and by a factor of 100 for walls loading in-plane. The plots provided are captured at the peak displacements that are expected for each wall, and represent the level of deformation and cracking expected under the design event.

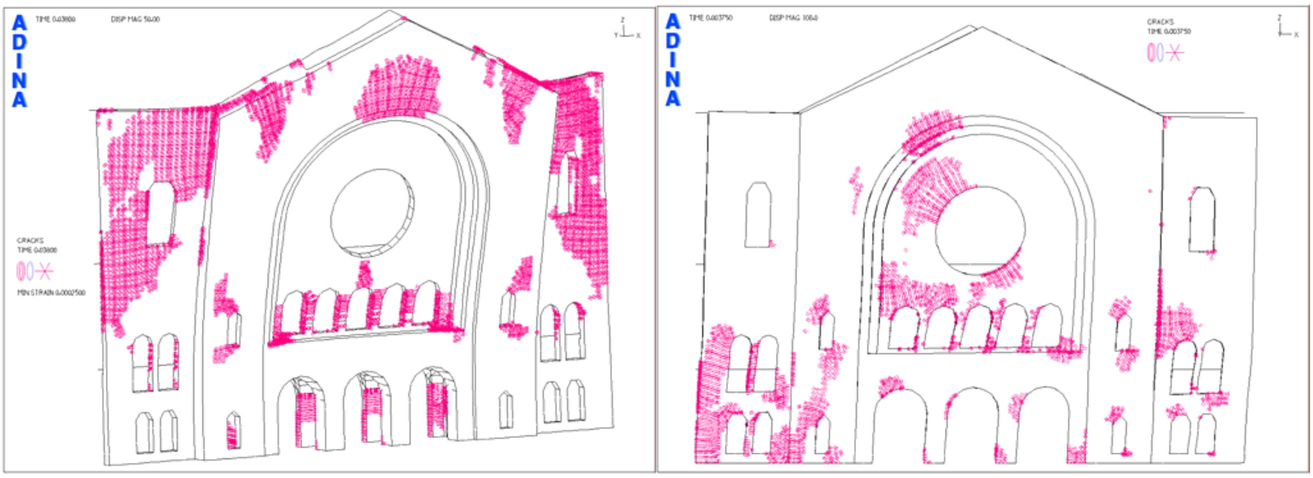

Figure 6 Deflected shape and cracking pattern of south wall due to out-of-plane (50X), and in-plane (100X) loading

The displacements in the in-plane direction of the wall are very small. This reflects the inherent strength and stiffness of the existing wall and its ability to resist large seismic forces. In the out-of-plane direction, the walls are significantly more flexible, which results in higher peak displacements under design seismic forces. A significant portion of the proposed seismic retrofit is intended to strength the walls in their out-of-plane direction and provide a positive connection from the walls to the roof and floor diaphragms.

5. CONCLUSION

Nonlinear pushover analysis techniques were used, in combination with the Capacity Spectrum Method, in order to justify the use of an unconventional performance based strengthening scheme of a historic unreinforced masonry building. The nonlinear analyses performed using the finite element software ADINA helped show the effectiveness of the repair scheme and guided the design by providing a method for testing the effect of various reinforcement alternatives on the local and global behavior of the building's four perimeter masonry walls. The analyses also validated to general philosophy of the strengthening scheme, which was to retain the flexibility inherent in the original building, which permitted the out-of-plane inertial response of the walls to be essentially independent from the in-plane response of the walls.

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