Behavior and Design of Fuse-Based Hybrid Masonry Seismic Structural Systems

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SUMMARY:
Hybrid masonry is a new structural system that combines the strength characteristics of reinforced concrete masonry walls with conventional steel framing to produce an innovative hybrid system which is capable of resisting both gravity and lateral loads. This paper discusses the development of models to predict the feasibility of applying the hybrid masonry structural system in areas of moderate to high seismic activity. The models are dependent on a fuse-based approach which focuses yielding within either the masonry panels or connector elements. Full-scale tests of steel connector plates are used to determine the necessary design provisions for the connectors to remain elastic and act as the primary energy dissipating fuse elements. Design guidelines are suggested for implementing hybrid masonry lateral force resisting systems in buildings.

Keywords: hybrid masonry, seismic design, concrete masonry, steel frame

1. INTRODUCTION

Hybrid masonry is a new and developing structural system that combines reinforced masonry panels with a steel frame. Initially, the idea was proposed to simplify construction of steel-framed buildings that utilize masonry infill. The structural frame is constructed of structural steel members, and the infill panels are built using reinforced concrete masonry blocks. The main structural concept of hybrid masonry is that beneficial qualities of both steel and masonry aid in providing adequate stiffness, strength and ductility within the structural system. The system is unique in that the masonry panels serve as a primary structural component to brace the frame, unlike typical cavity wall construction where the concrete masonry serves only as a backup to the exterior façade.

Structurally, hybrid masonry is categorized into one of three distinct groups, Type I, II, or III. The key difference between the three types is the method in which load is transferred between the frame and the masonry wall. Type I walls, seen in Fig. 1, only transfer in-plane shear-forces via connector elements that tie the steel frame to the masonry along the top of the wall. Type II walls act as loadbearing shear walls because they transfer both in-plane shear and axial forces, and Type III is a modification of Type II in that the masonry wall is fully confined along each side by the steel frame [NCMA 2009].

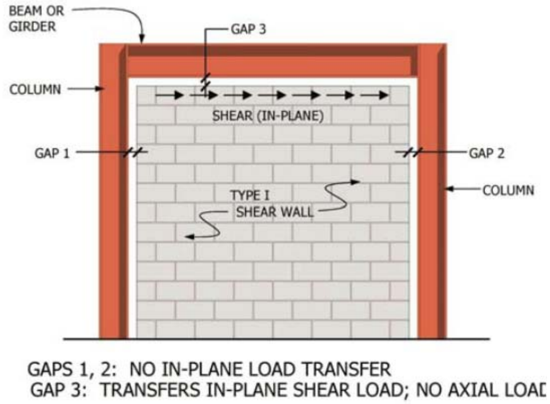


Figure 1. Type I Hybrid Wall [NCMA 2009].

Currently, hybrid masonry application is limited to low-rise structures, generally in the range of one to five stories, located within low-seismic regions. However, hybrid masonry systems would be a practical choice for projects that require lateral bracing around the building core, elevator shafts, or in structures that have an exterior masonry cavity wall coupled with a steel frame [Biggs 2011]. The discussion within this paper focuses on defining the range of application for the Type I hybrid masonry structural system in moderate to high seismic regions by recommending a maximum practical building height within each seismic region. A prior study [Eidini et al. 2012] considered a high seismic region case study, and the work reported in this paper covers a wider range of scenarios.

There are two basic design approaches for Type I systems, both of which require a predictable inelastic response of different components within the structural system. The first method assumes that the connector plates that link the masonry and steel frame are strong enough to remain within their elastic range during the full duration of any ground shaking, and thus the masonry panels serve as the yielding elements. In the second approach the assumptions made in the previous method are reversed such that the masonry panel is now designed to remain elastic, while the connectors serve as the energy dissipating fuse elements within the system. The second design approach will be highlighted in this discussion by first outlining the design approach for the ductile fuse plates.

2. DUCTILE FUSE PLATES

A series of experiments were performed at the University of Hawaii at Manoa (UHM) to develop ductile fuse plates for use in Type I hybrid masonry (Goodnight et al, 2011; Ozaki-Train and Robertson, 2011). The design of the hybrid masonry system allows fuses to be easily replaced after a seismic event such that the structure may be restored to its original condition. This result is best achieved with a tapered fuse plate as shown in Fig. 2a. This particular fuse was manufactured from a 6” (153 mm) wide by ½” (12.7 mm) thick A36 flat bar using a plasma cutter. Fuse plates were placed on opposite sides of a masonry test panel at each fuse location. Each fuse plate was bolted to a side plate which was welded to the flanges of the steel beam. Slip critical bolts were used to ensure elastic response during yielding of the fuse. The bottom of the fuse plate was bolted through the bond beam in the course second from the top. A vertical slotted hole was used in the fuse plate at the through-bolt location to prevent vertical load transfer from the steel beam to the masonry panel. The steel beam above the masonry wall was displaced horizontally through a series of increasing displacement cycles. The tapered fuse at the start of the test and fully deformed after 1.5 in. (38 mm) of displacement may be seen in Fig. 2b and Fig. 2c respectively (Mitsuyuki and Robertson, 2012).

(a) (b) (c)

Figure 2. Tapered Fuse Test Specimen: (a) test setup; (b) fuse detail pre-test; (c) fuse detail post-test.

The fuse plates were developed using a test frame as shown in Fig. 3. A pair of fuses was tested in the same configuration as with the masonry wall. A number of different fuse plates were considered during this testing, including the constant width link connector shown in the test setup in Fig. 3. All connectors were subjected to the modified ATC-24 cyclic loading routine shown in Fig. 4. This routine includes three displacement cycles pre-yielding, and three cycles at each displacement after yielding of the connector. The best performing fuse plates were able to survive almost all cycles shown in Fig.4 prior to low-cycle fatigue failure.

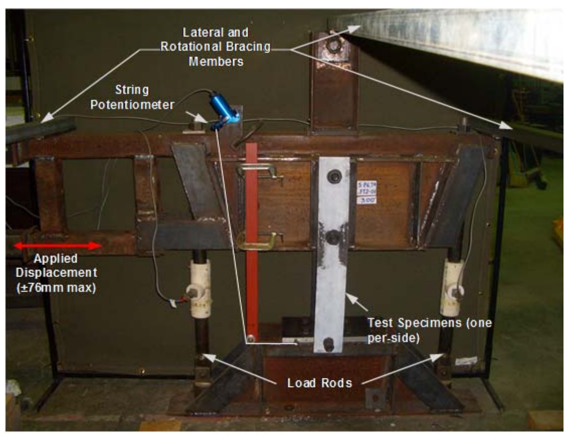


Figure 3. Connector and Fuse Plate Development Testing.

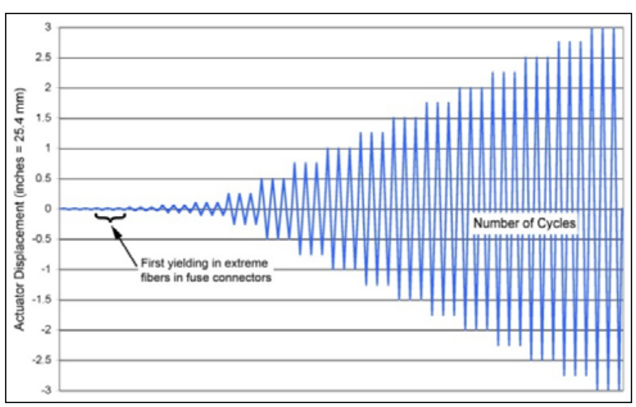
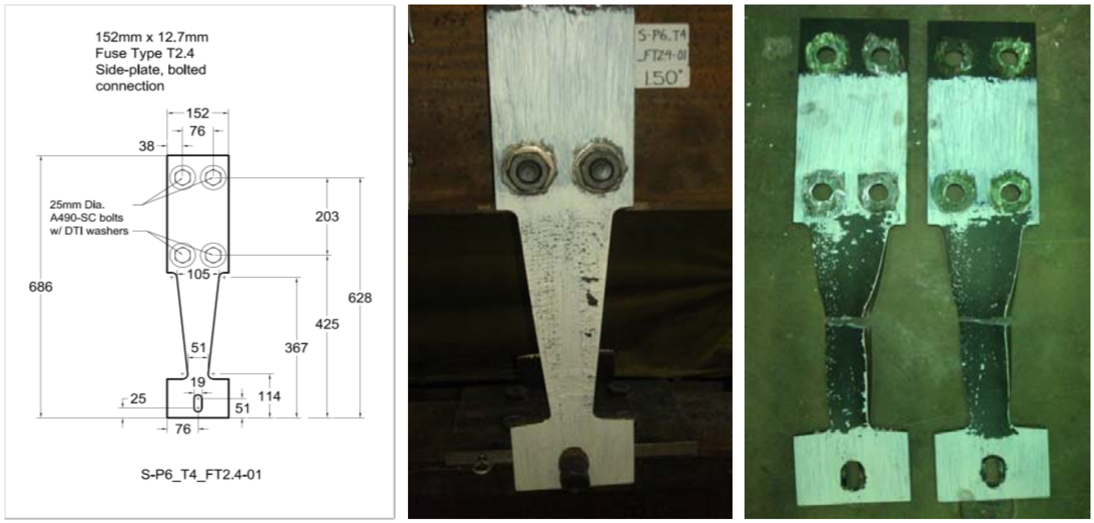


Figure 4. Modified ATC-24 Cyclic Loading Routine.

A typical fuse plate is shown in Fig. 5a. The fuse was tapered so that yielding would initiate at the top and bottom of the fuse simultaneously. Fig. 5b shows one of the fuses during testing; the flaking of the white paint coating indicates yielding of the steel along the full fuse length. Fig. 5c shows the pair of fuses after failure during cycling to 3 in. (76 mm) displacement. The fuse plates sustained over 45 cycles of this displacement routine, with a peak displacement of 2.75 in. (70 mm), representing a lateral drift ratio of 1.75 percent for a 13 foot (4 m) story height. This was considered adequate to serve the purpose as a seismic energy dissipating fuse. Larger drift levels could be achieved by increasing the length of the fuse.



(a) (b) (c)

Figure 5. Typical 6" Fuse Plate: (a) schematic; (b) during test; (c) after failure.

The hysteretic response of the pair of fuse plates, as seen in Fig. 6, represented the relationship between the applied load and the in-plane lateral displacement. The displacement represented the in-plane deformation of the fuse plates. Initiation of yielding occurred at around 10 kips (44.5 kN) at a displacement of approximately 0.2 in. (5 mm). The peak lateral load of 20 kips (89 kN) was reached at a displacement of 1 to 1.5 in. (25.4 to 38 mm). Soon after the peak load was reached, lateral torsional buckling of the fuses led to a 25 percent drop in the load. This load was then maintained to the maximum displacement of 3 in. (76 mm) when the fuses ruptured.

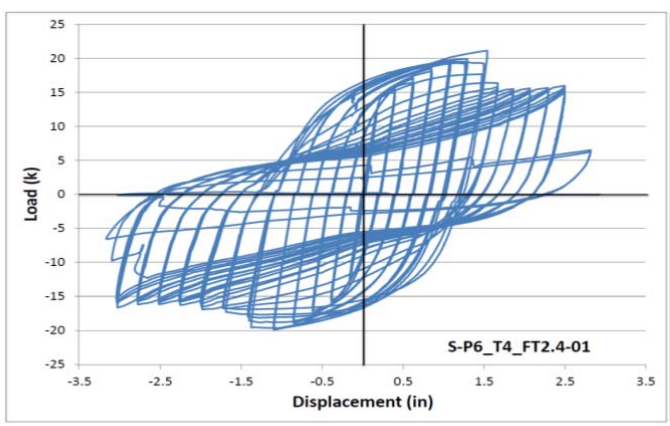


Figure 6. Hysteretic Response for Single Pair of 6" Tapered Fuses.

The force-deflection curves for the testing of multiple pairs of fuses were found to be scalable from the response of a single pair of fuses. Based on these results further extrapolation was done to estimate the total fuse capacity for an entire story in a building structure.

3. PROTOTYPE BUILDINGS

A set of typical building layouts was defined as a basis for the system-level study. Since current practice is limited to buildings of one to five stories, it was critical to develop a set of standard prototype buildings which further explored the practical height limitations of this system. Three different building plans from the SAC Steel Project were used as the basis for the present exploratory study [FEMA 335C]. Some slight alterations were made which resulted in three prototype buildings: the three, six and nine-story structures given in Fig. 7.

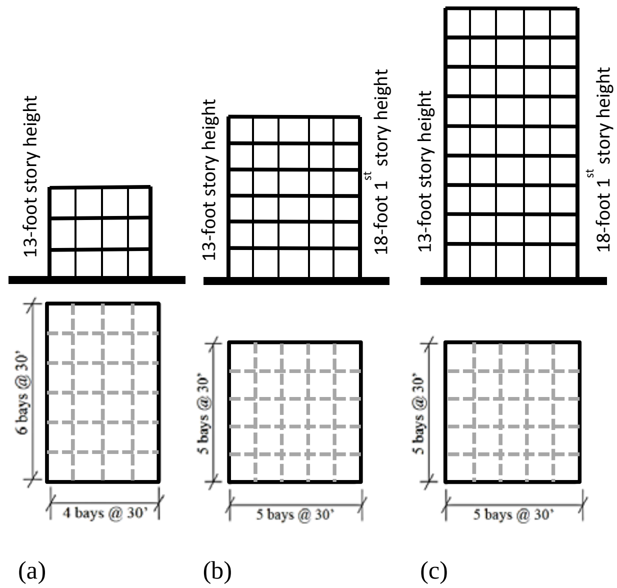


Figure 7. Prototype Buildings: (a) 3-story; (b) 6-story; (c) 9-story.

Each of these three prototype buildings was considered for four cities representing a range of seismic hazard. Los Angeles represented a high seismic region, while Seattle, Salt Lake City and Boston represented a range of moderate seismic regions, from high to low. Therefore the study included twelve different permutations, one for each of the three prototypes in each of the four specified cities.

4. LATERAL SYSTEM DESIGN

The equivalent lateral force procedure as outlined by the American Society of Civil Engineers (ASCE) was used to compute the seismic base shear force for each of the twelve permutations [ASCE 7-10]. The base shear was found by taking the product of the weight (W) and the seismic response coefficient (C_s) for each building, seen in Eqn. 4.1.

$$V = C_s W \tag{4.1}$$

In each case, the building site classification category was considered to be site class B. Site class B served as a simple default condition since both site coefficients, F_a and F_v, are 1.0 for all mapped spectral response acceleration parameters. Site class B represents a high quality soil profile, so sites with poorer soils may have further restrictions. The approximate period for each prototype was estimated based on the total height of the structure, where the period (T_a) was found by multiplying a coefficient (C_t), taken as 0.02, with the height of the building (h_n) raised to the power of x which was taken as 0.75, seen in Eqn. 4.2.

$$T_a = C_t h_n^x \tag{4.2}$$

The approximate periods for all three buildings are plotted along with the design response spectra for all four sites in Fig. 8. It is evident that as the height of the building increases, the estimate for the approximate period also increases; therefore, the spectral response acceleration may be greatly reduced, depending on the site location.

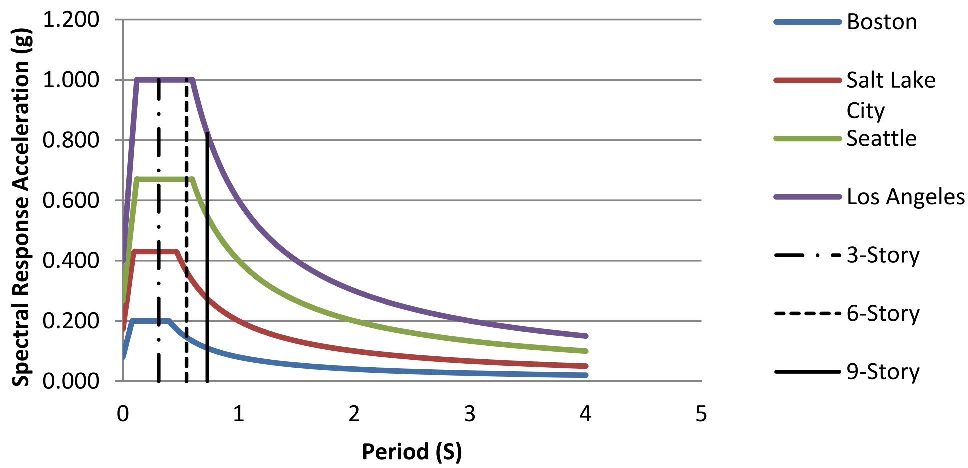


Figure 8. Design Response Spectra with Approximate Periods.

Without any full system test results to use as reference, the response modification coefficient (R) was assumed based on existing ductile steel seismic force resisting systems. A Type I hybrid masonry system with ductile steel fuse plates was assumed to offer a similar level of inelastic response to that of an eccentrically braced steel frame, which has R = 7. A summary of the basic parameters from the equivalent lateral force procedure are found in Table 1.

Table 1. Summary of Equivalent Lateral Force Results.

Location	Floors	Building Weight (kips)	Base Shear (kips)	Pairs of First Floor Fuse Connectors	Number of Bays
Boston	3	6504	157	16	1
	6	13331	187	19	1
	9	19900	200	20	1
Salt-Lake City	3	6504	398	40	2
	6	13331	455	46	3
	9	19900	499	50	3
Seattle	3	6504	626	63	3
	6	13331	936	94	5
	9	19900	1058	106	5
Los Angeles	3	6504	933	94	5
	6	13331	1391	140	7
	9	19900	1557	156	8

The estimates for the number of pairs of fuse plates required were based on results from the University of Hawaii connector tests, as reported above. The tests show that a pair of fuse plates has a yield strength of 10 kips. The number of pairs of fuse plates listed corresponds to the first floor requirements, where the building shear was equal to the full base shear force. Each of the subsequent upper levels was only required to transfer the shear force from the remaining floors above; therefore, the number of required fuses decreases from the first floor to the top floor. The estimate for number of bays corresponds to the number of reinforced masonry panels that would be required within the structure. The assumption was that the minimum spacing between pairs of fuses was 16 inches, which equals the nominal length of a standard concrete block. Thus, the maximum number of pairs of fuses that fits within a typical 30 foot bay was 22. The number of bays corresponded to the first floor requirements, but unlike the fuse plate requirement, the number of bays remained constant at each level and instead the number of fuse plates per bay was reduced as illustrated schematically in Fig. 9.

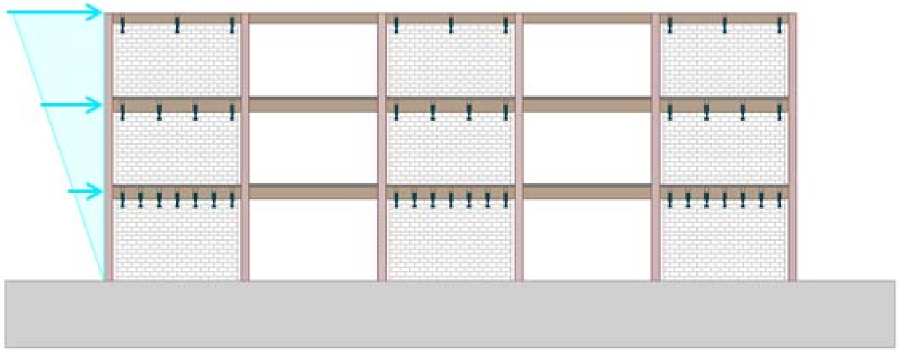


Figure 9. Sample Distribution of Fuse Plates within Single Building Frame.

Schematic diagrams were created to evaluate the feasibility of employing a hybrid masonry system in each of the prototype buildings, with focus on practical plan layouts. The diagrams offer a visual representation of possible masonry panel locations for each of the prototypes. In each case an attempt was made to space the masonry panels throughout the building to maximize useable interior area. Then each building was evaluated based on apparent practicality and interior usability. The schematics for the three Los Angeles prototypes are shown in Fig. 10.

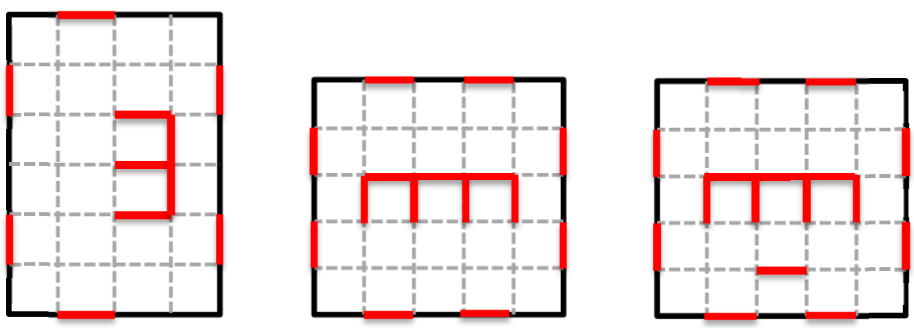


Figure 10. Los Angeles Prototype Plans with Hybrid Masonry Panel Locations.

Based on the panel distributions within each of the three prototype buildings located in Los Angeles, the interior space is segmented for the six-story and nine-story buildings. Whether this panel density is admissible for architectural constraints is a subjective question. The large base shear forces computed for these buildings required a large number of the structural bays to be filled with reinforced masonry panels, represented by the heavy red line segments in Fig. 10, to provide adequate space for the necessary number of fuses. Thus, it appears that in the highest seismic regions, like Los Angeles, the use of hybrid masonry may be constrained to low-rise buildings. Similar diagrams were created for the remaining prototypes and the structural layouts were less constrained for regions where the seismic hazard was less severe. Thus, the feasible height for these more moderate seismic regions, such as Seattle, Salt Lake City and Boston, is expected to exceed the tallest case considered (nine stories).

5. DEFLECTION CAPACITY BASED ON FUSE BEHAVIOR

Static pushover analyses were conducted to analyze the relationship between local ductility demand on the fuse plates and the global deflection capacity. The intention was to show that the fuse plates were capable of providing the necessary ductility such that all of the fuse plates could reach their yield point prior to any single fuse reaching its ultimate deflection capacity. Earthquake demands on yielding structural systems may exceed this definition of ductility demand, and need to be studied using nonlinear dynamic analyses, which are beyond the scope of this study. Using cyclic load and deflection data from the single pair of fuse plate tests at the University of Hawaii, as seen in Fig. 6, resulted in the formulation of a simple bi-linear load deflection plot.

A summative force-deflection curve for each story was determined by multiplying the shear force at yield and at ultimate by the number of pairs of fuses located at each corresponding level. It was assumed that the full structural system acted as the collective sum of its individual parts, so the total shear capacity for any given floor was equal to the shear capacity of one pair of fuses times the number of pairs at that level. The stiffness of the masonry wall was also modeled and included in the analysis. However, the fuses reach both their yield and ultimate capacity well before panel reinforcement reached yield. Note that in this design approach the fuse plates serve as the primary energy dissipation components while the reinforced masonry panels remain linear, with high stiffness, and add little to the total lateral displacement of the system.

A variety of lateral-force distributions were used to incrementally increase the lateral load on each prototype building. As the load was increased, the total displacement of each floor level was estimated based on the load-deflection curve for each story. An abbreviated summary of the results for the Los Angeles 3-story structure is shown in Table 2, and the four load case scenarios are depicted in Fig. 11.

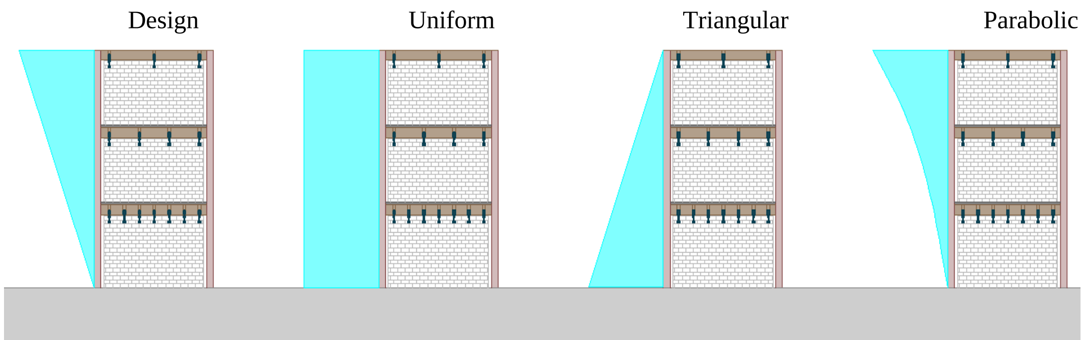


Figure 11. Pushover Analysis Load Profiles.

Table 2. Los Angeles 3-Story Pushover Analysis Summary.

Load Case	**Total Roof Disp./Drift @ Yielding (in/%)**	**Location of Initial Yielding**	**Total Roof Disp./Drift @ Ultimate (in/%)**	**Unyielded Story**	**Displacement Ratio of Ultimate to Yielding**
Design Load	(0.77/0.17)	All (F1, F2, F3)	(8.71/1.86)	-	11.31
Uniform Load	(0.63/0.14)	F1	(5.09/1.09)	-	8.08
Triangular Load	(0.50/0.11)	F1	(3.31/0.71)	F2, F3	6.62
Parabolic Loading	(0.70/0.15)	F3	(6.85/1.46)	-	9.84

As expected, yielding was initiated at the first floor for a majority of the load cases tested. The only exception was the parabolic load case which portrayed initial yielding at the third floor. Therefore, each of the first three load cases displayed in Table 3 was controlled by the ultimate deflection capacity of the first story, while the third story ultimate deflection capacity limited the parabolic load case. The pushover curves for each of the load cases may be seen in Fig. 12.

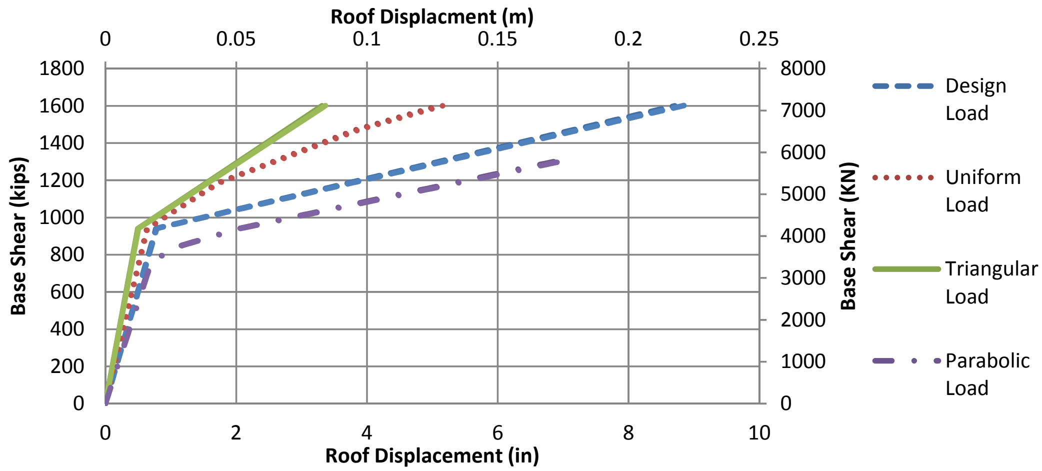


Figure 12. Los Angeles 3-Story Pushover Curves.

In general the hybrid masonry system offered enough ductility to provide between one and two percent ultimate roof drift where the percentage drift was found by dividing the displacement values by the total structure height, which for the three-story structure was 39 feet (11.9 m). Nonlinear dynamic analyses need to be done to evaluate whether displacement demands are within these drift limits.

6. CONCLUSIONS AND FUTURE WORK

Based on exploratory studies to date, one of which is described in this paper, the current limits in application of the hybrid masonry structural system may be expanded beyond contemporary practice, which only deals with low-rise structures in low seismic zones. Based on the results of the simple feasibility study, hybrid masonry systems appear to provide a practical alternative for both moderate and high seismic regions with modest height limitations in each region. In the most active seismic regions, Type I hybrid masonry structures may be limited to shorter buildings (less than nine stories), while buildings exceeding nine stories could feasibly be used in moderate seismic regions. These heights correspond to practical arrangements of hybrid panels that are admissible for typical architectural layouts and occupancy types. The present study is dependent on the assumed condition for site class B, along with the other assumptions made during the analysis procedure, such as R = 7. Thus, the height limitations should be scaled accordingly to account for differences in local site conditions and the actual response modification coefficient for the system when it is determined.

Now that there is a better understanding of the expected capabilities of hybrid masonry, the next phase of research will focus on creating a full set of design guidelines. The full system design approach will address the design of each major component of the system from steel frame members and masonry panels to the crucial steel-masonry interface created by the connector elements. Critical aspects of the predictions and assumptions made throughout this analysis process will be verified by large-scale testing, which will be completed at the University of Illinois at Urbana-Champaign. The hybrid masonry structural system shows promise as an innovative structural configuration, and it is expected that further experimental and numerical research results will validate this outlook.

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