FEASIBILITY OF HYBRID MASONRY IN SEISMIC REGIONS

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ABSTRACT

Hybrid masonry is a new structural system that combines reinforced concrete masonry panels with typical steel framing. Current industry practice has limited the application of this system to low-rise buildings in low seismic zones; however, the relative capacities of the system components suggest that hybrid masonry may be a feasible option for more seismically active regions. Simple analytical models are developed to establish a fundamental understanding of seismic behaviour and to help predict the ultimate strength and expected system ductility. These models along with capacity design procedures are used to establish a suite of representative prototype building structures. The design of these prototype buildings focuses on adequately proportioning the critical structural components so that the total drift capacity can be maximized without allowing soft story failure mechanisms to develop. Ultimately the goal is to provide estimates, based on the evaluation of the prototype designs, of the feasible limits of application of hybrid masonry systems in moderate and high seismic zones. A summary of results and proposed design guidelines are presented.

KEYWORDS: hybrid masonry, seismic loading, inelastic analysis, capacity design

INTRODUCTION

Hybrid masonry is a new structural system that is currently being tested and analysed to gain a better understanding of the system's structural behaviour and performance and to study the feasibility of using it in moderate to high seismic regions [1]. Hybrid masonry's current use has been limited to low-rise construction in the eastern half of the United States; however, the system does show promise as a viable option for applications in taller buildings and higher seismic regions.

The hybrid masonry system combines reinforced masonry panels with a steel frame so that the beneficial qualities of masonry and steel combine to provide the necessary stiffness, strength, and ductility. Hybrid masonry is typically subdivided into three distinct groups, Type I, II, or III [2]. The main difference between these three types is the method in which load is transferred between the steel frame and masonry panels. Type I systems, seen in Figure 1, act as non-loadbearing shear walls that are only capable of transferring in-plane shear forces via the connector elements that tie the steel frame to the masonry along the top of the panel.

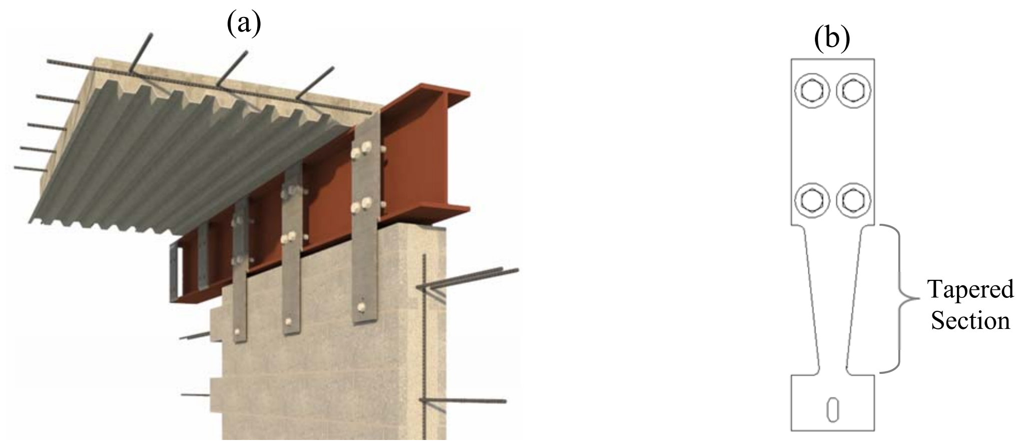

Figure 1: Type I Hybrid Detail (a) Link Plate Connectors from IMI (b) Fuse Connector

Type II and Type III systems are modified such that they become load bearing systems that transfer both in-plane shear and gravity forces. The focus of this paper will be the seismic design considerations for the Type I system.

As seen in Figure 1, the connectors used in Type I hybrid masonry systems can be either link plate connectors or fuse connectors. The key distinction is the geometric modification made by tapering the central section of the fuse connector, which directly affects the elements' inelastic behaviour. The inelastic performance of hybrid masonry is dictated by the relative capacities of the system's individual components. Thus, there are two basic design approaches that depend on where yielding is concentrated. The first method, using fuse connectors, assumes that the yielding will be limited to the fuse connectors while the second approach, using link plate connectors, concentrates all of the yielding in the masonry panels. Both of these approaches are acceptable means of design; however, each method has unique concepts related to the inelastic capacity of the yielding elements which must be carefully considered throughout the seismic design process [3].

Ultimately the goal is to evaluate the limits of feasible application of the hybrid masonry structural system in moderate to high seismic regions by outlining the necessary design considerations that utilize non-linear response characteristics and a performance based capacity design approach. This paper will highlight some of the key differences between using fuse connectors and link plate connectors that need to be considered in a seismic design process to limit the formation of undesirable failure mechanism.

DESIGN APPROACH

The Type I hybrid masonry design procedure outlined here follows a basic capacity design approach. The first step is to determine an ideal failure mechanism and the corresponding locations where inelastic behaviour needs to develop for the desired mechanism to be achieved. Then, the other elements in the system are designed, based on the capacity of the yielding elements, to remain elastic. The standard approach for typical moment-resisting frames is to use a strong column weak beam (SCWB) approach where the system is designed such that the plastic hinges form within the beams. The beams are designed to provide ductility at the plastic hinges and the columns are designed to remain elastic throughout the full loading history [4]. Thus it is important for the designer to not merely consider how each element will resist the applied forces; but rather that the structure is proportioned so that the moment and shear strengths of non-yielding members exceed the forces that correspond to the yielding limit of all the critical elements.

If plastic hinges form in the columns, then the stability of the structure is severely reduced; this approach typically results in the formation of a soft story, and may also be referred to as a weak column strong beam (WCSB) case. Typically soft story mechanisms are considered undesirable modes of failure due to their increased potential for complete system failure, and therefore great effort should be taken during design to ensure they will be avoided.

These common design procedures lend themselves well to the design of Type I hybrid masonry systems. Each of the two Type I design approaches mentioned above are dependent on whether the designer selects to concentrate the yielding, or plastic hinge formation, in the fuse connectors or in the masonry panels. In either case, the design intent is to spread inelastic behaviour over the height of the system (yielding at multiple levels) and to prevent column hinging. The following sections provide an overview of key design considerations made for a set of prototype building structures. The prototypes were developed as a set of symmetrical building plans modelled after the SAC Model Buildings [5]. Buildings located in Los Angeles, Seattle, Salt Lake City and Boston were considered, and height variations included 3, 6, 9, and 12 stories. Fuse connectors and link plate connectors were both considered with regards to how the inelastic response of these system components contributed to the seismic design for the overall system.

INELASTIC SYSTEM RESPONSE

The inelastic response of the hybrid masonry system is a vital component of the performance-based capacity design approach that was used to perform a preliminary design and analysis of a suite of prototype buildings. The location of the inelastic response and the resulting overall system ductility is dictated by the relative strengths of the masonry and steel connectors (fuse connectors vs. link plate connectors), so key design considerations based on expected inelastic response of the critical structural components are highlighted here.

First, consider the case where fuse connectors are used to join the steel frame and concrete masonry panels. The overall design objective is to induce a relatively uniform, global failure mechanism, and to achieve this goal, lateral strength must be appropriately proportioned between each of the building's stories. Since the distribution of system strength is directly related to the distribution of fuse connectors, a preliminary design distribution of fuse connectors may be chosen using the force profile shown in Figure 2, which is the result of the equivalent lateral force procedure outlined in ASCE 7-10 [6].

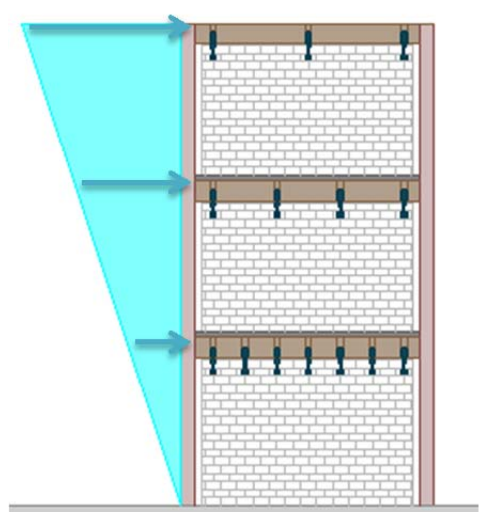

Figure 2: Sample Distribution of Fuse Connectors Using ASCE 7 Lateral Force Profile

A rational preliminary distribution of fuses is depicted in Figure 2 where the number of fuse connectors' increases towards the base to match the increasing story shear force. This preliminary design distribution served as a starting point for the present study, and further analysis was performed to verify that each building story was contributing to the overall roof drift and that no soft story mechanisms formed. Pushover analyses were used to analyse the relationship between local ductility demand on the fuse connectors and global deflection capacity. The intention of this part of the study was to verify that the fuse connectors were capable of providing the necessary ductility to ensure that all of the connectors reach their yield point before any single pair of fuse connectors reaches its ultimate deflection capacity. Using cyclic force-deflection data from tests done at the University of Hawaii on a single pair of fuse connectors a simple bi-linear force-deflection curve was developed, shown in Figure 3 [7]. This approach was considered to be conservative because the actual fuse connector response exhibited greater strength before degrading to behavior that is similar to the bi-linear approximation after many loading cycles.

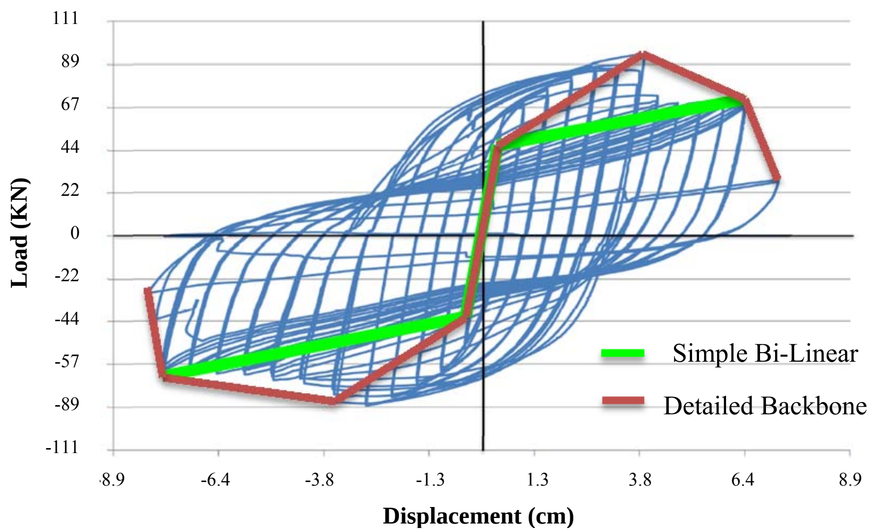

Figure 3: Hysteretic Response for Single Pair of Tapered Fuses with Load Deflection Curves

Pushover curves for the three-story prototype buildings under the ASCE 7 load profile are shown in Figure 4. Despite contributions of masonry panel deformation and axial steel column deformation, the lateral force-deflection behaviour is clearly dominated by the fuse connector response because the connectors reach their yield and ultimate capacities well before the

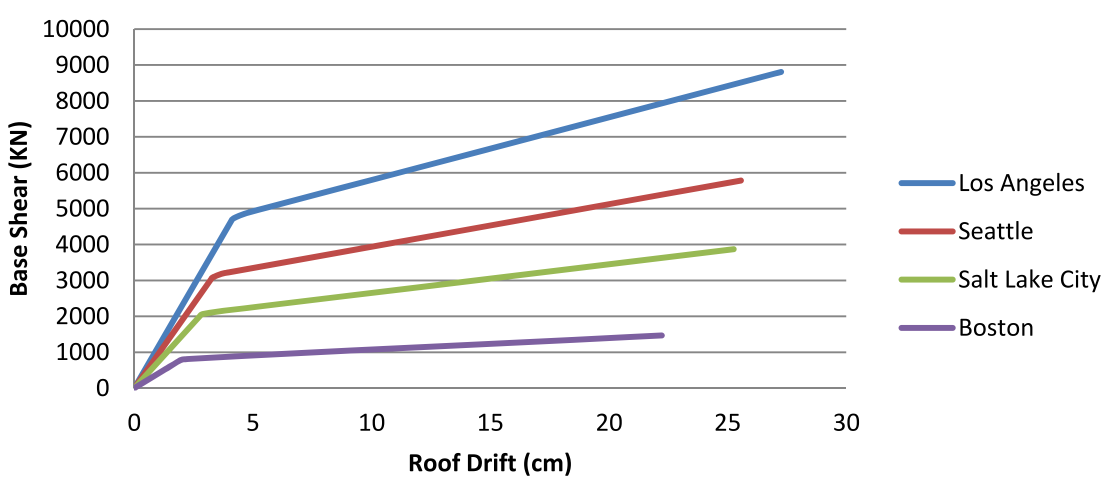

Figure 4: Simplified 3-Story Pushover Curves

masonry panels or steel frame become inelastic. Thus, each of the pushover curves resembles the simplified bi-linear load deflection curve that was developed using the tapered fuse test data shown in Figure 3.

A more detailed nonlinear pushover analysis was also conducted using the OpenSees software platform [8]. This model more accurately captures the elastic steel frame behaviour and the inelastic fuse connector behaviour, which is represented by the more detailed load-deflection backbone curve shown in Figure 3. Figure 5 shows the simplified and detailed pushover analysis results for the Los Angeles 3-story prototype structure. The two analyses converge to the same ultimate strength, although the path to that point is different. The OpenSees analysis has a lower initial stiffness since it includes flexibilities that are not considered in the simplified model.

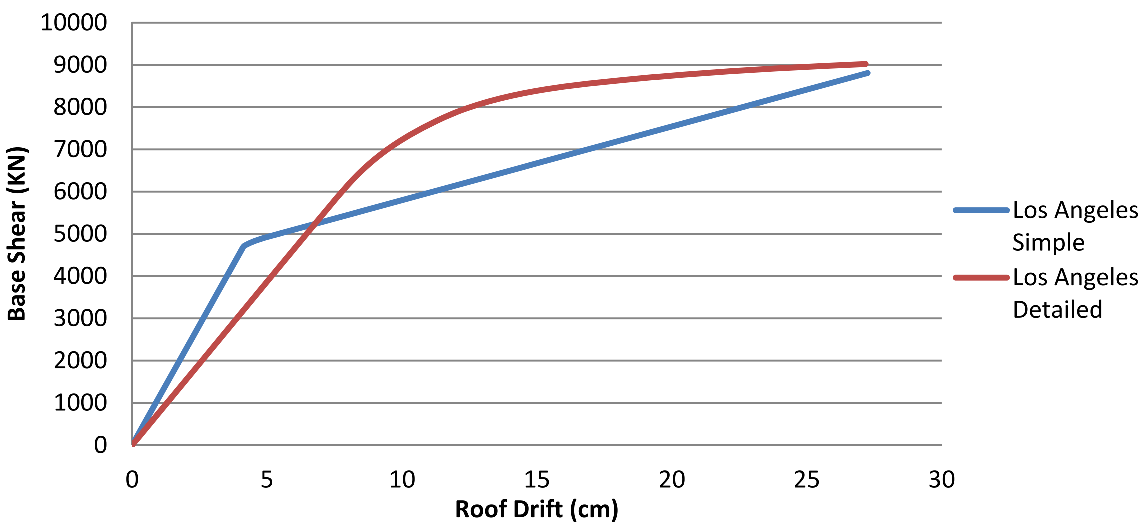

Figure 5: Los Angeles 3-Story Pushover Analysis Comparison

The OpenSees model was used to examine the possible formation of soft story behaviour, which occurs when lateral drift is concentrated within a single story of the structure while the remaining stories experience minimal drift. If this was observed, then the distribution of fuse connectors was altered to improve the distribution of ductility and overall inelastic response of the building. This process was repeated for each of the prototype buildings until the relative drift distribution fell within an allowable drift envelope which was defined as being within five percent from an idealized uniform drift distribution. Final drift distributions for each of the 3-story structures are shown in Figure 6 along with the allowable drift envelope that was used as the target range. Thus, the chosen fuse connector distributions appear to provide the preferred distribution of yielding for the prescribed design lateral force profile that is based on first mode response which is expected to govern for low and moderate rise structures where hybrid masonry is currently being utilized. However, concentration of yielding in one story is still possible since the inertial force profile that develops due to an input ground motion will not necessarily match the assumed design force profile. Further investigation using nonlinear dynamic analysis is needed.

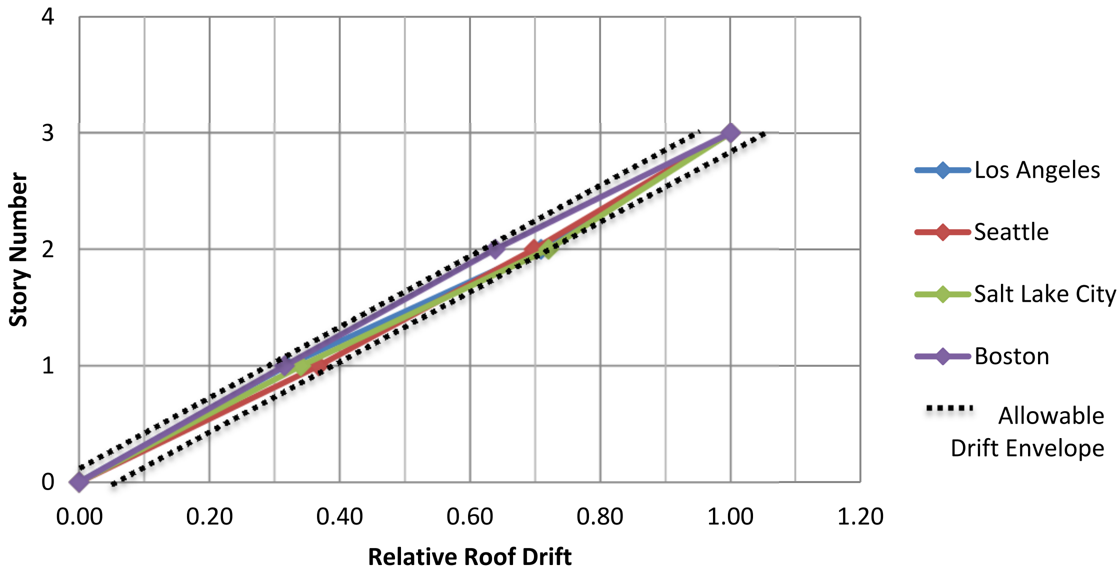

Figure 6: Relative Distribution of Drift for 3-Story Structures

Using a similar design philosophy, link plate connectors can be used between the steel frame and masonry panels. The major difference between these two approaches is the location of inelastic response. The link plate connectors are stiffer and stronger than their tapered fuse counterparts, and thus the yielding is focused in the concrete masonry panels. The overall design goal remains the same; to achieve a uniform failure mechanism and an even distribution of system ductility. The distribution of system stiffness, strength and ductility depends on the design of each individual masonry panel. The higher stiffness and strength of hybrid masonry designed with link plate connectors may be desirable for performance limit states of continued operation or immediate occupancy, particularly in buildings with high risk categories. For earthquake intensities larger than the design basis, some masonry damage may be acceptable provided that life safety concerns are met. To study the feasibility of this design approach, current code requirements for reinforced masonry walls were used to design the hybrid masonry panels.

Hybrid masonry panels must be designed with adequate flexural and shear strength to resist the force demands at each story. The masonry panel strength should be reduced at each successive story moving from the first story up to the third story to allow for a more even distribution of ductility demand and relative lateral displacement. An effort should be made to over-reinforce the walls to resist shear forces using horizontal reinforcement; this will help to prevent less desirable shear type failures from controlling the system response. Figure 7 is a schematic diagram that displays how the vertical and horizontal reinforcement must be sequentially altered either by reducing the size of rebar or by increasing the spacing between bars, thus reducing the number of bars required.

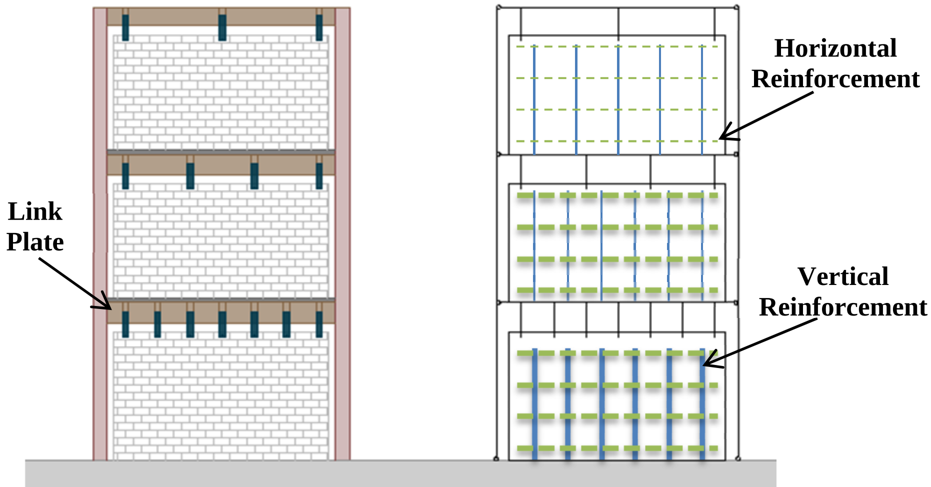

Figure 7: Masonry Panel Reinforcement Schematic

Figure 7 does not represent a specific case, but rather the general configuration that arises due to the design lateral force distribution. Each building design was checked to confirm that the proposed reinforcement plan would offer adequate flexural strength and shear strength, and an iterative process was used to determine the most efficient reinforcement size and spacing that was capable of providing the necessary strength while minimizing total overstrength. An abbreviated summary of the panel designs for the 3-story Los Angeles prototype is shown in Table 1.

Table 1: Los Angeles 3-Story Panel Reinforcement with Corresponding Panel Strength

Story	Vert. Bar Size (mm^2)	Vert. Bar Spacing (cm)	Flexural Capacity (KN)	Horiz. Bar Size (mm^2)	Horiz. Bar Spacing (cm)	Shear Capacity (KN)
First	387	81.28	**1930**	387	60.96	2131
Second	284	81.28	**1499**	200	60.96	1672
Third	200	101.6	**965**	129	81.28	1428

The panels were designed such that the capacity was controlled by the flexural strength of the vertical reinforcement, represented by the bolded values in Table 1. Notice that the ratio between each of the controlling force values for adjacent stories was not perfectly linear; this occurs because the Masonry Standards Joint Committee (MSJC) has specific design requirements for minimum reinforcement size and maximum spacing [9]. These code requirements were somewhat restrictive in developing the design for the masonry walls; however, based on the computed shear capacities it seems that the link plate connector hybrid masonry frames may also be capable of providing adequate strength and ductility to ensure uniform story displacement and the formation of a preferred global failure mechanism.

SYSTEM FEASIBILTY IN SEISMIC REGIONS

The equivalent lateral force procedure as outlined by the American Society of Civil Engineers (ASCE) [6] was used to compute seismic base shear force demands for each of the prototype building layouts. The base shear force was found by calculating the product of each buildings' weight (W) and a seismic response coefficient (C_s), as seen in Equation 1 below.

$$V_{base} = C_s \times W \quad (1)$$

In each case a number of assumptions were made in the base shear computation. The building site classification category was considered to be site class B which served as a default condition because all of the corresponding site coefficient factors, F_a and F_v, were 1.0 for the spectral response acceleration parameters that were used. Without any previous hybrid masonry system test results to use as a reference, the response modification coefficient (R-factor) was assumed to be in the range of 5 to 7 based on coefficients currently employed for ductile masonry and steel seismic force resisting systems. A summary of selected parameters from the equivalent lateral force procedure are found in Table 2.

Table 2: Summary of Fuse Connector Requirements

Location	Stories	Building Weight (KN)	Pairs of Fuse Connectors (R = 5)	Pairs of Fuse Connectors (R = 6)	Pairs of Fuse Connectors (R = 7)	Number of Bays (R = 5)	Number of Bays (R = 6)	Number of Bays (R = 7)
Los Angeles	3	28930	147	123	105	7	6	5
	6	59296	204	170	146	10	8	7
Seattle	3	28930	98	82	70	5	4	4
	6	59296	136	113	93	7	6	5
	9	88515	150	125	107	7	6	5
Salt Lake City	3	28930	63	52	46	3	3	3
	6	59296	68	57	49	4	3	3
	9	88515	75	63	54	4	3	3
Boston	3	28930	26	21	18	2	1	1
	6	59296	28	23	20	2	2	1
	9	88515	30	25	22	2	2	1
	12	117734	40	34	29	2	2	2

The estimation for the number of pairs of fuse connectors required was based on the shear capacity of the fuse connectors tested at the University of Hawaii at Manoa. The tests showed that a single pair of fuse connectors has a yield strength of 45 KN [7]. The number of pairs of connectors listed in Table 2 corresponds to the number of fuse connectors required at the first story, where the shear force demands were the largest. Each of the subsequent upper stories has fewer connectors because the shear force carried by each story decreases as the story number increases. The estimate for the number of bays indicates the number of structural bays that would need to be filled with a reinforced concrete masonry panel. It was assumed that the minimum spacing between pairs of fuse connectors would be limited to 406 millimetres or the nominal length of a standard concrete block. Therefore the maximum number of pairs of fuses that would fit within a typical 9.1 meter bay was 22.

Schematic diagrams were then created and used to evaluate the overall feasibility of utilizing a hybrid masonry system in each of the different building layouts [10]. The diagrams offer a visual representation of possible panel locations for each of the prototype buildings, and in each example the hybrid panels were placed throughout the building footprint in an attempt to maximize the available functional interior space in a realistic manner. Figure 8 shows the schematics for a sample of Los Angeles prototypes.

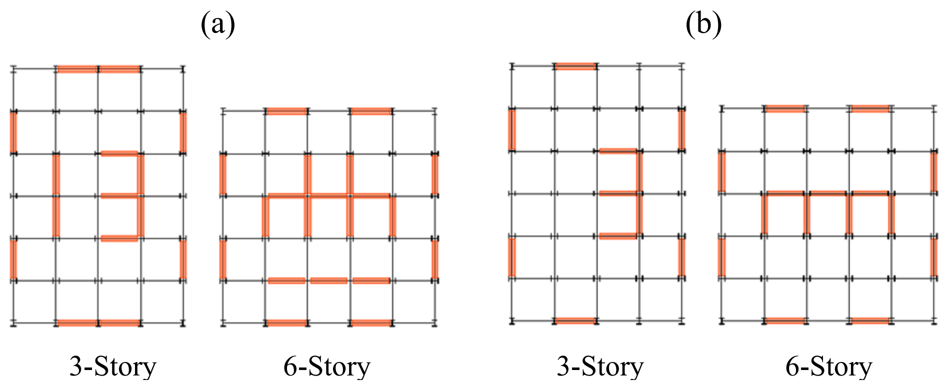

Figure 8: Los Angeles Prototype Plans with Hybrid Masonry Panel Locations (a) R=5 (b) R=7

Clearly, the overall feasibility of the hybrid masonry system is dependent on the chosen R-factor, which ultimately must be validated through large-scale testing and extensive analyses. To gain preliminary insight into the ductility capacity of Type I hybrid masonry systems with fuse connectors, the OpenSees models that were introduced previously were used to estimate a maximum achievable roof drift ductility ratio (μ_{roof}) based on the yield and ultimate deflections of a fuse connector. These deflections were based on small scale connector testing done at the University of Hawaii [11], and assuming a system R-factor of 6 as a median value of the range previously examined. The roof drift ductility was computed as the ratio of maximum inelastic roof drift (δ_i), which is defined by the point when the fuses in the most critical story reach their ultimate deflection capacity, to elastic roof drift (δ_e) when the first pair of fuses reach their yield drift capacity, as shown in Equation 2. Tabulated results from this computation are shown in Table 3.

$$\mu_{roof} = \frac{\delta_i}{\delta_e} \qquad (2)$$

Table 3: 3-Story Prototype Drift Ductility

Location	Roof Disp. @ Yield (cm)	Elastic Drift (δ_e)	Roof Disp. @ Ultimate (cm)	Inelastic Drift (δ_i)	Roof Drift Ductility (μ_{roof})
Los Angeles	4.93	0.0041	27.56	0.0190	4.59
Seattle	4.24	0.0036	25.78	0.0181	5.08
Salt Lake City	3.56	0.0030	25.15	0.0182	6.07
Boston	3.05	0.0026	21.59	0.0156	6.08
				AVG.	**5.5**

The average roof drift ductility of the hybrid masonry system was computed to be 5.5, which based on an equal-displacement assumption suggests that the system R-factor would fall at the lower end of the range that was previously used to check feasibility. This is only an initial evaluation of R-factors for Type I hybrid masonry with fuse connectors, and more detailed analysis and further study is required.

CONCLUSION

Based on the results from these analyses, Type I hybrid masonry systems appear to offer a feasible structural option for both moderate and high seismic regions, with even greater height potential seeming to be practical for lower seismic areas where it is currently being used. The results from the equivalent lateral force seismic design procedure provide feasible limitations to building height in various seismic regions. In high seismic zones, like Los Angeles, hybrid masonry systems that use fuse connectors seem to only be practical up to five or six stories. In moderate seismic regions, like Seattle or Salt Lake City, the feasible height restrictions could be increased to eight or nine stories, while the height of buildings in the lowest seismic regions, like Boston, appear to be reasonable up to twelve stories. These heights correspond to limitations based on possible arrangements of masonry panels that would still allow for practical building occupancy.

The Type I hybrid masonry structural systems that utilize link plate connectors also hold promise; however, additional detailed feasibility studies need to be completed along with, parametric nonlinear dynamic analyses to verify that idealized system performance objectives may be achieved. The results of the preliminary code compliance check does however suggest that the reinforced concrete masonry panels may be designed to provide the necessary strength proportioning without creating undesirable soft story failures.

Critical aspects of the assumptions made throughout these analyses are being verified through large-scale testing at the University of Illinois at Urbana-Champaign. In general, it seems that the Type I hybrid masonry system may be developed to provide the necessary strength and ductility to serve as an efficient structural system with possible applications ranging from high rise in low seismic regions to lower rise buildings in high seismic regions.

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