EFFECTS OF BRICK VENEER WALL SYSTEMS LOADED IN-PLANE ON THE SEISMIC RESPONSE OF MEDIUM-RISE BUILDINGS

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ABSTRACT

The seismic performance of masonry veneer wall systems has come under increasing scrutiny in the past few years. As part of this scrutiny an investigation evaluating the interaction of masonry veneers and medium rise structural frames under seismic loads was conducted.

In this investigation, the in-plane interaction of structural frames and masonry veneer wall systems are evaluated. The responses of the medium rise building system to seismic loading are compared using a mass representation of the masonry veneer wall system (as is allowed by design codes) and by a more accurate strut and frame model. These building frame-wall system models were subjected to the design based earthquake (DBE) and maximum considered earthquake (MCE) levels of selected ground motion. A parametric study was conducted that encompassed the range of stiffness and strength of both the frames and the wall systems encountered in common construction practice in the United States.

The paper will present a summary of the effects of the veneer wall systems on the response of the building frames that support them, when loaded in their in-plane direction, during a seismic event. It will be shown that the current prescriptive code requirements are adequate for the buildings, but in some cases quite conservative.

KEYWORDS: brick veneer, medium-rise buildings, in-plane loading, seismic response

INTRODUCTION

Masonry veneer wall systems are commonly used in many parts of the United States. These wall systems are comprised of an exterior clay masonry layer connected to an interior backup wall system over an air cavity by ties. The air cavity functions primarily as a drainage system. The backup wall of wood or steel studs, reinforced concrete or concrete masonry units, supports the brick veneer and can support loads from the structure. The metal ties transfer lateral loads from the exterior veneer to the interior backup [1-4]. These wall systems provide an aesthetically pleasing, durable cladding. However, their performance under seismic loading has come under increasing scrutiny in the past few years. As part of this scrutiny an investigation evaluating the interaction of masonry veneers and medium rise structural frames under seismic loads was conducted.

The investigation described in this paper focuses on the interaction of masonry veneer wall systems and medium size structural frames under in-plane seismic loads. The premise of this study is that even though veneer wall systems designed according to prescriptive code requirements are unlikely to collapse under in-plane seismic loading, they do interact with the building frame and affect the distribution of the lateral loads during a seismic event. The current design provisions do not address this transfer mechanism, and veneer wall systems are typically modelled simply as masses [5, 6]. As this modelling approach may overestimate the lateral loads that act on structural frame, the study evaluated the response of the veneer wall systems in a variety of medium rise structural frames under seismic loading to determine whether refinement of these provisions is warranted.

INVESTIGATION METHODOLOGY

A medium rise ten story office building with a 22.86m x 38.1m (75 ft. x 125 ft.) plan dimension, a 7.62m (25 ft.) bay width in both directions, and 3.66m (12 ft) story height was considered in this investigation. Nonlinear 2-D finite element models of the veneer wall systems were developed and incorporated into nonlinear 2-D numerical models of the exterior bays along the short side of the ten story structural building frame, in order to simulate the most severe loading condition. The OpenSees analysis tool and library of elements were used to accomplish this task [7]. A detailed description of the building frame models and their development can be obtained in Desai's work [8]. All the models used in this investigation were calibrated using static and quazi-static experimental results.

A dynamic analysis was performed on the combined veneer wall – building frame models under Design Based Earthquake (DBE) and the Maximum Considered Earthquake (MCE) scaled ground motions [8]. The ground motion records used for the analyses were selected to create the worst effect on the veneer wall systems. Additionally, select veneer wall system and building frame parameters were varied in order to understand their influence on the response of systems [8]. In the parametric study, a stiff reinforced concrete (RC) shear wall system, and a flexible steel moment resisting frame system was considered. It was assumed that the response of most medium rise building frame systems will lie between these bounds [8].

NONLINEAR MODELING OF THE VENEER WALL SYSTEMS

The masonry veneer wall system configurations were selected to encompass the range encountered in construction practice. Three different types of wall systems were considered in this investigation [8]: A system having a high stiffness (a stiff concrete masonry backing wall and stiff ties) defined one extreme, one having a low stiffness (a flexible steel stud backing wall and flexible ties) defined the other extreme, and finally, a system having an intermediate stiffness (a stiff concrete masonry unit (CMU) backing wall and flexible ties).

A typical steel stud backing wall has attached sheathing and acts somewhat like a shear wall. The sheathing carries the lateral load and transmits it to the steel studs via the screw connectors. The steel stud backing wall was modeled as a braced frame as per Okail [9]. The in–plane behavior of the steel stud backing wall was represented by a 2-D braced frame (Figure 1). In this braced frame representation of the backing wall, the vertical elements of the frame, modeled using elastic beam column elements [8, 9], represented the steel studs. Axial truss elements were used for the cross braces, and modeled the sheathing and connection behavior. The horizontal axial truss elements were used to transfer lateral loads into the cross braces. The cross braces transferred the lateral loads to the base [9]. Thus, the combined system of the horizontal axial truss elements and the cross braces was used to model the lateral force carrying ability of the sheathing and screw connectors. To improve numerical efficiency and stability, only one backing wall frame was used in each bay to model the effect of the sheathed stud wall systems in these areas [8].

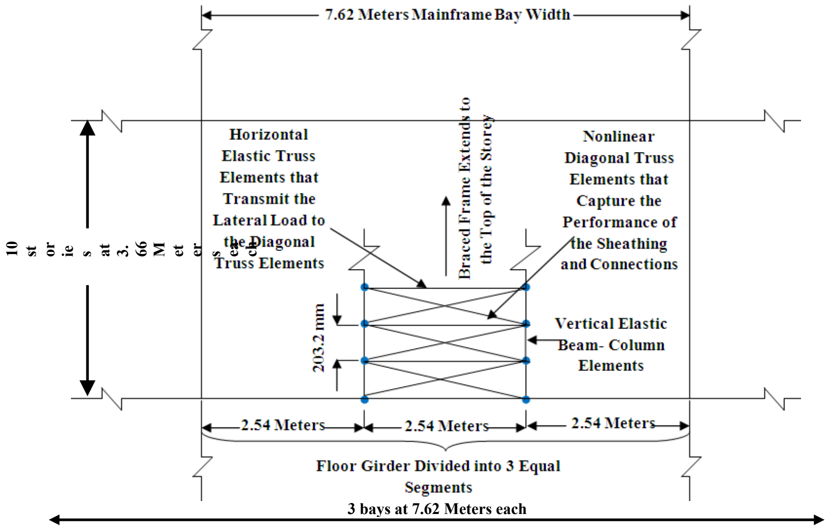

Figure 1: Equivalent Single Bay Braced Frame Representation of Stud Backing Wall

The equivalent backing wall frame model was developed in two steps. Firstly, a frame backing wall model with studs at 406 mm (16 in) on center was developed [8]. The response of this model was calibrated using Hikita's [10] quasi-static cyclic in-plane racking test results, adjusting the model properties to ensure that its response in a nonlinear pushover analysis closely matched that of the tested wall [8]. Hikita [10] tested a sheathed steel stud shear wall that was the same as the steel stud backing wall in the current investigation.

The vertical column elements of the frame were modeled using steel elastic beam–column elements, the horizontal beam elements were modeled using elastic truss elements, and the diagonal cross braces, using nonlinear truss elements [8]. The vertical elastic beam – column elements were assigned a standard elastic modulus of steel, 200 kN/mm^2 (29000 ksi) [8]. The horizontal truss elements were assigned a simple linear elastic material model, as their sole role was to transmit horizontal forces to the braces, and were assumed to have an elastic modulus of 10.3 kN/mm^2 (1500 ksi) [8]. The diagonal truss elements had a tri–linear material model associated with them (Figure 2) and the characteristics of this model were determined during calibration [8]. A comparison between the analytical model and experimental results is shown in Figure 3. The model showed an initial stiffness that was identical to the experimental stiffness. In the inelastic range the predicted behavior matches the test results reasonably well, especially for extreme values [8]. This established a stud sheathing wall model based on a 406 mm (16 in) stud spacing.

A single bay braced frame stud wall model was then developed and calibrated to produce the same behavior as a stud wall that used a 406 mm (16 in) stud spacing over the entire 7.62 m (25 ft) bay width [8]. This was done using a pushover analysis on both the models and the single bay frame properties were adjusted until its base shear versus roof displacement response curve showed a good match, as shown in Figure 4 [8]. Good agreement resulted when the diagonal truss elements were assumed to have the material response described in Figure 5.

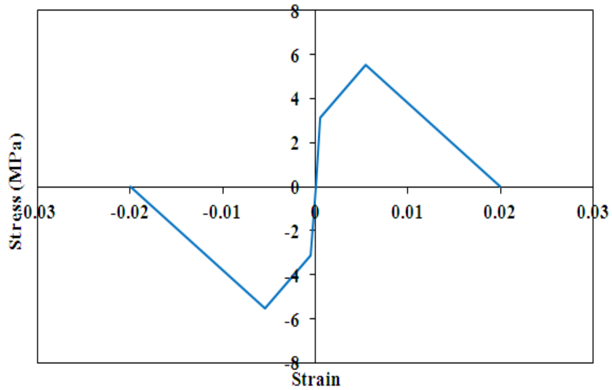

Figure 2: Diagonal Truss Element Model

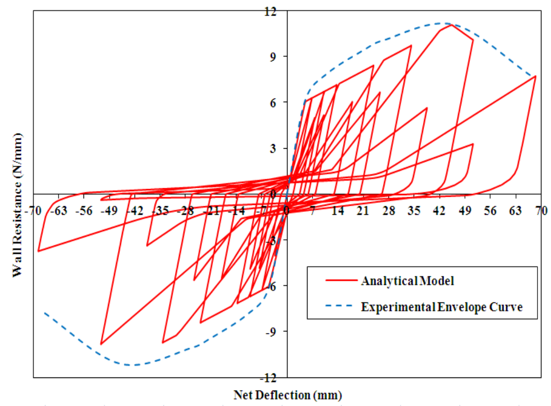

Figure 3: Experimental and Analytical Model Response of Stud Backing Wall

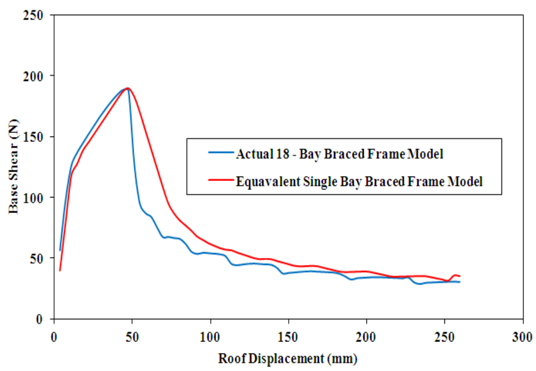

Figure 4: Pushover Analysis Comparison for Stud Backed Wall

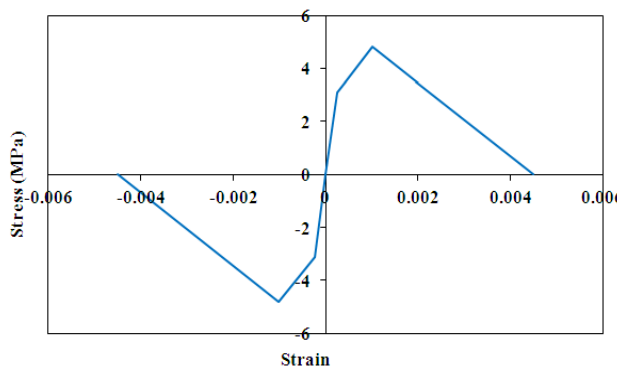

Figure 5: Diagonal Truss Element Material Model

Elastic beam-column elements were used to model the veneer in the in – plane direction. The veneer was assumed to remain in its uncracked state, under in – plane loads [10]. Inelastic behavior was not expected for the masonry veneer, so it was assumed to remain elastic under in-plane loads [10]. Thus, two vertical columns of elastic beam-column elements were used to represent the veneer in each bay of the 2-D frame as shown in Figure 6. These columns coincided with the vertical elements of the backing wall model. At each story level of the single bay braced frame backing wall, the beam-column veneer elements and the backup were connected to each other at their nodes by zero length elements representing the in - plane ties [8]. The double eye and pintle tie system of the stud backed wall was modeled using zero length elements [10]. Based upon Jo's work [11], the in-plane shear load-defection behavior of these ties was modeled using zero length tie elements in using the relationship shown in Figure 7.

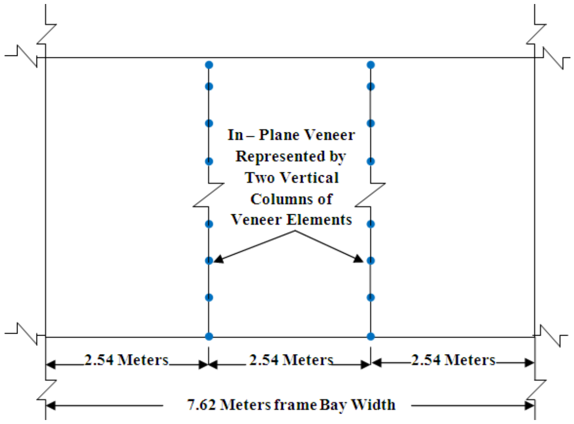

Figure 6: Veneer Elements in Stud Backed Wall

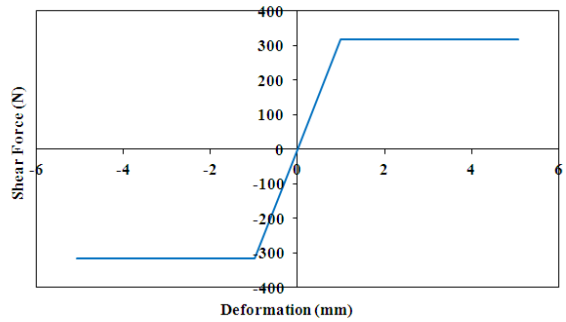

Figure 7: Shear Behavior of the Double Eye and Pintle Tie System

To simulate vertical support of the masonry wall system, a vertical, 69 mm (2.73 in) long, elastic beam-column element with very high flexural stiffness, was attached to the third points of the floor girders at every bay of each story of the building frame [8]. The bases of the two vertical column lines of the braced frame representation of the steel stud backing wall were attached to the free ends of these vertical beam column elements via zero length elements as shown in Figure 8. These zero length elements used material models with three degrees of freedom (two translational and one rotational) and were designed to permit the transfer of vertical and horizontal forces between the backing wall and the supporting elements, but not to permit moment transfer, thereby simulating a pin support [8].

The backing wall braced frame models were attached to the corresponding one third points of the floor girder at the next higher story level, once again, through zero length elements [8]. A simple elastic material model was assigned to these zero length elements with a high stiffness for horizontal translational degrees of freedom, and a low stiffness for the vertical translational and rotational degrees of freedom [8]. This idealization permitted a transfer of horizontal lateral seismic forces from the girder into the steel stud backing wall, while preventing the transfer of vertical loads and moments into the backing wall [8].

The base of the veneer elements were connected to the building frame girders through zero length elements that were configured to transmit the vertical force between the veneer and the floor girder, but to transmit no moment [8]. This connection was also configured to account for the sliding frictional resistance at the base of the veneer by assigning an elastic-plastic spring in the horizontal translational degree of freedom direction at the base of the veneer [8]. This spring was configured to have high initial stiffness until a breakpoint load corresponding to the frictional force, $f = \mu R$, was reached, thereby ensuring little to no movement of the base of the veneer until the sliding frictional resistance was overcome [8]. The top of the veneer was unsupported [8].

The CMU backed wall system was modeled based upon work done by Jo [11]. A single vertical column of elastic beam-column elements was used to model the CMU backing wall system under in-plane loading. It was assumed that the CMU backing wall does not experience inelastic behavior under in – plane loads. Thus, an elastic model was used to describe its response under lateral in – plane loading [11]. Figure 9 shows the locations along the floor girder at which the veneer and backup elements of the wall system were attached.

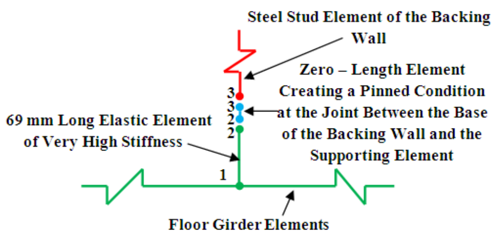

Figure 8: Attachment of Base of Stud Backed Wall to Frame

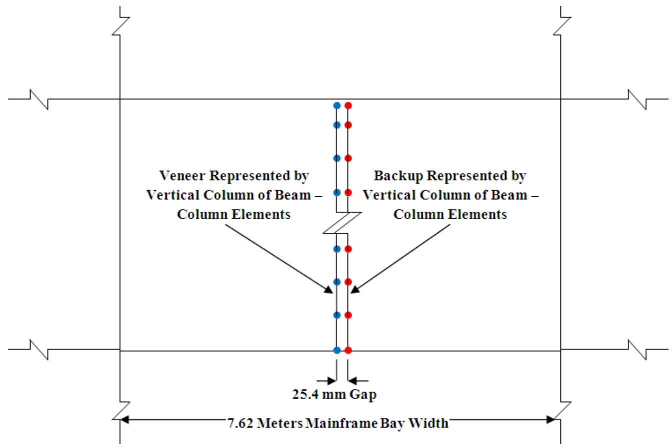

Figure 9: Veneer and Backup Elements in CMU Backed Wall System

The veneer was also idealized using a single vertical column of elastic beam-column elements under in – plane loads [11]. The rocking behavior of the veneer was found to be negligible for a 3.66 m (12 feet) long wall (Jo, 2010). Consequently, rocking is likely to be insignificant for a 7.62 m (25 feet) long wall and was ignored in this analysis [8].

Both the double eye and pintle (described earlier) and triwire tie systems (Figure 10) were studied in the CMU backed wall, and were modeled using axial truss elements [11].

As shown in Figure 11, the attachment of the CMU backed veneer wall system to the main building frame girder was similar to that used with the stud backed wall [8]. However, the veneer and backup were each represented by a single vertical column of elastic beam-column elements. The floor girder was divided into three segments, with the first and third segment equal in length. The central girder segment was 25 mm (1 in) in length, corresponding to the gap between the veneer and backup. The vertical column representing the CMU backing wall was attached to one end of the central floor girder element, and the vertical column representing the veneer was attached to the other end. The veneer was attached at its base in the same manner as described for the stud backed system. The base element of the CMU backing wall was attached to the floor girder using a zero length element that fully transferred the horizontal, vertical forces and bending moments from the backing wall to the floor girder. The top of the backing wall was unsupported [8].

As the in-plane CMU backing wall and veneer model developed by Jo [11] showed good agreement between predicted and measured behavior, it was used in this analysis but was adjusted to account for differences in wall dimensions and material properties [8].

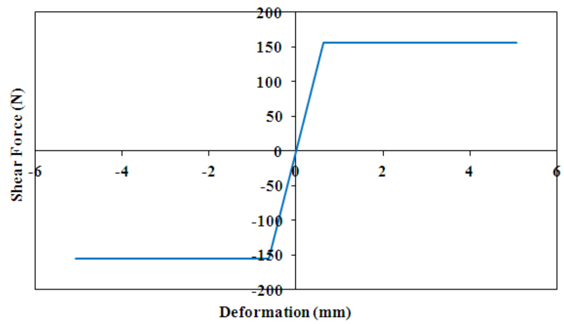

Figure 10: Shear Behavior of the Tri-wire Tie System

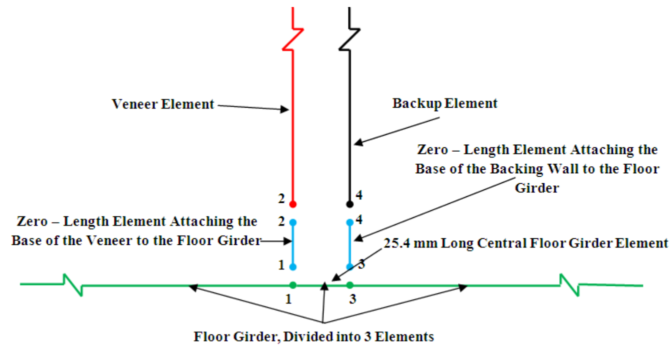

Figure 11: Base Attachment of CMU Backed Wall System to Frame

EFFECTS OF CMU BACKED WALLS ON STEEL MOMENT FRAME RESPONSE

The profiles of the peak structural building frame acceleration, with both mass and system representations of the attached veneer wall system, loaded in its in-plane direction, are shown in Figure 12. This figure shows the response acceleration profile of the frame at the instant of time where acceleration peaked. This peak acceleration was obtained by considering the acceleration response of every node of the frame over the entire duration of the ground motion, and selecting the highest of these values. The peak accelerations always occurred on the top storey of the frame. Both the stiffest and most flexible masonry veneer wall system and building frame combinations are shown.

The steel moment resisting frame was the most flexible structural building frame considered in this investigation. For both wall system representations (mass and detailed model), the acceleration profile along the frame height reflected a first mode dominated response of the frame (see Figure 12). This is consistent with the expected response of a medium rise building. However, the acceleration magnitudes are higher than those seen in the RC shear wall system, due to the flexibility of the steel framed system. The top story of the steel moment frame showed significantly higher acceleration compared to the storey below it due to the “whipping effect” seen in most flexible moment frames. The whipping effect became less pronounced when the veneer wall system is represented by a detailed the wall system model. In addition, the difference between the peak frame acceleration profiles along the frame height for the different representations of the wall system is more pronounced for the steel frame than for the RC shear wall system, once again, due to the flexibility of the steel moment resisting frame.

Figure 12 also shows that system models that used mass representations of the veneer walls showed higher values of peak response accelerations that those used detailed veneer wall systems models, for both flexible and stiff tie systems. Furthermore, the accelerations for systems incorporating the walls with the flexible ties were slightly higher from those with the walls with the stiff tie systems. Table 1 shows the peak structural frame acceleration values for the model representations of the wall system. This table also lists the percentage differences in the peak acceleration responses of the frame for the mass and model representations of the wall system, for the different tie types, ground motions, and ground motion intensities.

The corresponding displacement profiles followed patterns similar to those described for the acceleration profiles.

Table 1: Percentage Differences in Peak Steel Moment Frame Accelerations for Mass and Model Representations of CMU Backed Wall System

Ground Motion	Intensity	Tie Type	Difference (%)	Peak Frame Acceleration (g)	Tie Type	Difference (%)	Peak Frame Acceleration (g)
Sylmar	DBE	Flexible	40	0.91	Stiff	57	0.66
Sylmar	MCE	Flexible	45	1.26	Stiff	52	1.11
Tarzana	DBE	Flexible	11	2.75	Stiff	11	2.74
Tarzana	MCE	Flexible	15.7	4.19	Stiff	22	3.9

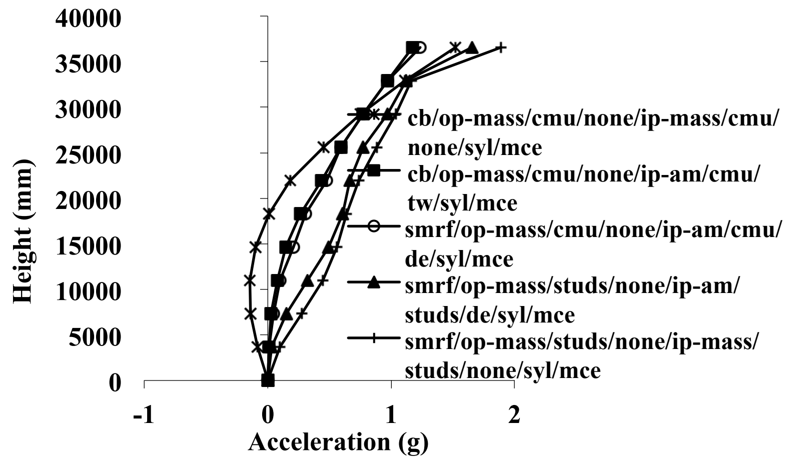

Figure 12: Maximum Response Acceleration Profiles up the Height of the Stiffest and Most Flexible Frames (Note: Table 2 contains an explanation of symbols used in the legend)

Table 2: Symbol Legend

Symbol	Description	Symbol	Description
cb	reinforced concrete braced frame (shear wall)	syl	sylmar ground motion
smrf	steel moment resisting frame	studs	steel stud backing wall
op-mass	out-of-plane wall mass representation	mce	maximum considered earthquake
none	ties not modeled in mass representation of wall	ip-mass	in-plane wall mass representation
de	Double eye and pintle tie	cmu	concrete masonry unit backing wall
tw	tirwire	ip-mass	in-plane wall model representation

EFFECTS OF STUD BACKED WALLS ON STEEL MOMENT FRAME RESPONSE

The peak building structural frame accelerations along the height of the steel moment resisting frame with masonry veneer wall systems backed by steel stud walls, are lower when a detailed wall system model is used than when a mass representation is used. The shape of the profile of the accelerations along the frame height was similar to that described for the CMU back systems in the steel moment frame, but the magnitudes differed. Table 3 shows the difference in peak frame accelerations using the two masonry veneer modeling techniques.

Table 3: Percentage Differences in Peak Steel Moment Frame Accelerations for Mass and Model Representations of Stud Backed Wall System

Ground Motion	Intensity	Difference (%)	Peak Frame Acceleration (g)	Intensity	Difference (%)	Peak Frame Acceleration (g))
Sylmar	DBE	16	1.25	MCE	12	1.65
Tarzana	DBE	25	1.91	MCE	21	3.1

EFFECTS OF CMU BACKED WALLS ON RC BRACED FRAME RESPONSE

The profiles of the peak building structural frame accelerations for the RC shear wall system and CMU backed masonry veneer wall systems, for both the mass and model representations of the wall systems, were similar to those described for the steel moment resisting frame. The model representation of the walls produced slight differences in the response profiles along the frame height, compared to their mass representations. At both the DBE and MCE levels of both ground motions, the peak acceleration of the RC shear wall system (at the 9^{th} and 10^{th} stories) was always higher for the mass representation versus the detailed veneer wall model. However, for the remaining storeys, the converse was true. Table 4 shows the percentage differences in the acceleration responses of the frame for the different parameters considered in the investigation. The peak building structural frame acceleration profiles for both the flexible and stiff tie systems in the walls were nearly identical. Additionally, the pronounced whipping effect observed in the case of the steel moment frame was not seen for the RC shear wall system. Finally, the magnitudes of the peak frame response accelerations were lower than those recorded for the steel moment frame system.

Table 4: Percentage Differences in Peak RC Shear Wall System Accelerations for Mass and Model Representations of CMU Backed Wall System

Ground Motion	Intensity	Tie Type	Difference (%)	Peak Frame Acceleration (g)	Tie Type	Difference (%)	Peak Frame Acceleration (g)
Sylmar	DBE	Flexible	22	0.79	Stiff	21	0.81
Sylmar	MCE	Flexible	19	1.23	Stiff	23	1.17
Tarzana	DBE	Flexible	13	0.95	Stiff	15	0.91
Tarzana	MCE	Flexible	13	1.48	Stiff	15	1.47

EFFECTS OF STUD BACKED WALLS ON RC BRACED FRAME RESPONSE

The shapes of the response acceleration profiles for the RC shear wall system and steel stud backed masonry veneer wall systems were similar to those seen for the systems discussed in previously. However, the acceleration profiles were slightly different than the trend described previously. The magnitude of the peak building structural frame accelerations were higher when the veneer wall system was represented by a detailed model than when the wall was represented by its mass.

However, the RC shear wall system was the stiffest system considered, and the peak response accelerations of this system with the walls represented by their masses were much lower than those and this difference was small. These differences are tabulated in Table 5.

Table 5: Percentage Differences in Peak RC Shear Wall System Accelerations for Mass and Model Representations of Stud Backed Wall System

Ground Motion	Intensity	Difference (%)	Peak Frame Acceleration (g)	Intensity	Difference (%)	Peak Frame Acceleration (g)
Sylmar	DBE	6	0.64	MCE	6	0.97
Tarzana	DBE	3	1.25	MCE	9	1.65

Examination of Tables 1 through 5, shows that in all the frame and wall system configurations, the peak frame response accelerations under the Tarzana ground motion were higher than those under the Sylmar ground motion, at both the DBE and MCE levels, for both modelling approaches. This is most likely because the Tarzana ground motion incorporates a wider range of frequencies at higher acceleration amplitudes than the Sylmar ground motion. This will excite a greater number of fundamental modes of response in the frame.

CONCLUSIONS

As a result of this investigation, the following conclusions were made:

1. The in-plane peak accelerations of the flexible steel moment resisting structural building frames with the masonry veneer wall systems are likely significantly lower than those predicted using only a mass representation (as is typically done in practice). However, for the stiff reinforced concrete shear wall system, the structural frame accelerations were slightly higher when a detailed veneer wall system model is used. This suggests that, overall, the representation of the veneer wall systems by their masses is a conservative approach for the design of the frames, although this approach can significantly over estimate the effect of the wall systems when these are incorporated into flexible structural frames. Furthermore, a mass model for the veneer wall system can produce slightly unconservative results when addressing masonry veneer walls in stiff reinforced concrete shear wall systems.

2. The peak structural frame acceleration profiles indicated that the system was responding primarily in its first vibratory mode. However, the steel moment resisting frame responses did show some traces of higher mode response in the form of the "whipping effect" created at the top story of the frames. The topmost story of the moment frames showed peak response accelerations much higher than that expected for first mode response alone. The use of a detail masonry veneer wall model reduced the intensity of this whipping effect and lowered peak frame accelerations.

3. In general, the magnitudes of the peak structural frame accelerations evaluated in this investigation were higher at both the DBE and the MCE levels of the Tarzana ground motion as compared to the corresponding levels of the Sylmar ground motion. This is most likely

because the Tarzana ground motion incorporates a wider range of frequencies at higher acceleration amplitudes than the Sylmar ground motion. This excites a greater number of fundamental modes of response in the frames.

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