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3-D ANALYTICAL MODELING OF METAL BUILDING SYSTEMS WITH HARD WALLS

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Abstract

Metal Buildings with precast concrete or masonry walls (hard walls) have been identified by analytical modeling, shake table tests, and earthquake reconnaissance as being susceptible to collapse. While the steel frames have been shown to be resilient, the potential for wall failure and possible collapse is present. There exists a large stiffness differential between the walls and steel frames, which in turn generates high demands on brittle connections. Also, there is very little coordination between the metal building systems (MBS) engineer and the engineer-of-record who is responsible for the connections, which can result in improper connection design. When these connections fail in a non-ductile manner, the continuous load path is lost and the wall can fall away from the structure. The primary purpose of this research is to develop a new lateral force resisting system that relies on simple energy dissipating connections between the hard wall and the steel frame in order to enhance the global seismic performance and improve life safety. The energy dissipation will be geared in the longitudinal direction (parallel to the ridge) without losing out-of-plane strength in the transverse direction. This research included the development of 3-D models for nonlinear response history analyses. Various connection figurations for friction-slip devices were considered and optimized based on performance, efficiency, and practicality. A case study of one of the 3-D models was performed to assess the appropriateness of the modeling procedure. The 3-D models developed in this research will be used in an ongoing parametric study on the connections to quantify the energy dissipation capacities needed to improve the seismic performance of these structures.

Keywords: Metal Building Systems; Resilient Connections; Energy Dissipation

1. Introduction

1.1 Defining the Problem

The use of Metal Building Systems (MBS) is in high demand in the construction industry. As of 2014, Metal building systems accounted for over 50% of non-residential low-rise construction in the United States [1]. Reasons for the increase in the use of these systems include their cost-efficiency, durability, modularity, and speed of construction. Typically, these systems are one to two story structures and have a wide range of applications including use in commercial, industrial, recreational, religious, and educational structures. Metal Buildings with lightweight cladding have been shown to perform well in past earthquake events. This was demonstrated when the performance of metal buildings was examined by the MBMA after four earthquake events from 1983 through 1994, who found them to be resistant to seismic action [2].

It has been a recent trend to include precast concrete or masonry walls (hard walls) in these systems, generally for architectural aesthetics. Metal Buildings with hard walls have been shown in analytical studies, experimental tests, and post-earthquake reconnaissance to exhibit poor seismic performance. Specifically, the reconnaissance in New Zealand after the 2011 Christchurch earthquake showed that significant structural and non-structural damage occurred in metal buildings with non-load bearing concrete cladding or unreinforced masonry cladding [3]. This is particularly alarming considering that the modern building codes for New Zealand and the United States are comparable.

Although the steel frame of metal building systems has been shown to be resilient, the potential for hard wall failure and collapse exists in these structures during seismic events. Any non-ductile connections between these two different structural elements will experience significant force and deformation demands. The stiffness incompatibility between light, flexible steel framing and the heavy, stiff concrete or masonry walls (hard walls) can result in failure of these brittle connections. Additionally, there is often little or no coordination between the MBS engineer and the engineer-of-record who is responsible for the connections between the frame and hard walls. If these connections are improperly designed and not detailed for seismic resistance, it can result in the failure of the connections in a non-ductile manner. When this occurs, the continuous load path is lost and the hard wall can fall away from the structure, both of which occurred in MBS in Haiti and New Zealand [3]. The cost of damage in such failures is measured in lives lost, repair costs, and in business interruption expense.

1.2 Proposed Solution

In order to improve seismic performance of metal building systems with hard walls, it is necessary to develop a new seismic force resisting system with energy dissipating connections. Simple, reliable energy dissipating connections in the form of friction slip devices or yielding fuses will relieve the stiffness incompatibility that exists between stiff hard walls and flexible steel frames. These connections will be geared towards energy dissipation in the longitudinal direction (parallel to the ridgeline) while maintaining strength in the transverse (wall out-of-plane) direction. Previous research has focused only on the moment frame direction, whereas this research includes assessing the earthquake response of metal building systems in the longitudinal direction. Because both in-plane and out-of-plane demands must be known, three-dimensional models are necessary to analyze the demands along the continuous load path.

Before the connection designs can be developed, it is critical to understand how the new connections will impact the global seismic behavior of metal building systems with hard walls. To achieve this, 3-D models in SAP2000 need to be developed in order to predict the structural behavior of these systems during an earthquake. These models will be used to quantify the necessary connection strength, deformation demands, and energy dissipation capacities necessary to achieve both life safety and enhanced performance of metal buildings during seismic events.

2. Development of SAP2000 Model

This chapter discusses the components and the rationale behind the development of the SAP2000 model for metal building systems with hard walls. An isometric view of the model is shown in Fig.1.

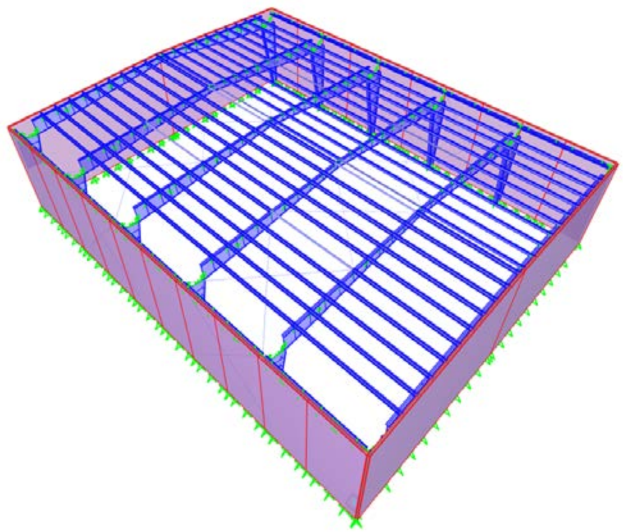


Fig. 1 - Extruded View of SAP2000 Model

2.1 Metal Building Frame Selection

192 metal building frame designs were available from the Approximate Fundamental Period Study for Metal Building Frames [4]. The objective was to select frames designed in high seismic zones (S_{DS}=1.0g) that would produce a sample with high variety in both building geometry and load conditions. Five frames were selected that, when combined, represent a wide spectrum of the metal building system population designed. The geometry and design parameters of each frame are listed in Table 1. For consistency, the frame numbers for this research will utilize the same model numbers as in the Approximate Fundamental Period Study [4].

Table 1 – Metal Building Design Sample Parameters

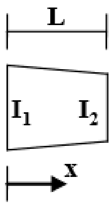
Model Number	**Building Type**	**Eave Height m (ft)**	**Length m (ft)**	**Snow Load KN/m^2 (psf)**	**Wind Speed ASCE 7-05 m/s (mph)**
16	Clear Span Symmetrical Gable	9.2 (30)	12.2 (40)	0	54 (120)
41	Clear Span Symmetrical Gable	9.2 (30)	30.5 (100)	0	38 (85)
42	Clear Span Monoslope	9.2 (30)	48.8 (160)	2 (42)	38 (85)
85	Clear Span Symmetrical Gable	4.6 (15)	12.2 (40)	0	54 (120)
138	Modular Symmetrical Gable	9.2 (30)	36.6 (120)	0	38 (85)

2.2 Elastic Metal Building Frame Model

2.2.1 Nonprismatic Element

Metal building moment frames have characteristics that are very different from conventional steel moment frames. Conventional moment frames are constructed using prismatic hot-rolled I shaped members. Metal building frames are constructed of built-up I shapes that have been optimized to reduce material weight [5]. Web-tapered sections are used to increase the flexural capacities where the moment demand is greater. The frame components utilized the nonprismatic beam-column element that is proprietary to SAP2000. Smith [4] has already shown that the nonprismatic beam-column element in SAP2000 provides sufficiently accurate results when the column or rafter segment is discretized into four elements. Fig.2 shows a single frame for Model 85.

To build the nonprismatic element, the two end cross-sections were generated. SAP2000 allows the user to set the variation in flexural stiffness for major axis bending and minor axis bending. The formula used in the moment variation for the nonprismatic element is shown in Fig.3. For linear web-tapered members, the variation in the major axis moment of inertia is predominantly a parabolic function (Fig.2). The axial, shear, torsional, mass, and weight properties all vary linearly over each segment [6]. The variation in the major axis was set to parabolic and for minor axis, a linear variation was set.



$$I(x) = \left[\left(I_1^{1/n}\right)\left(1 - \frac{x}{L}\right) \ + \ \left(I_2^{1/n}\right)\left(\frac{x}{L}\right) \right]^n$$

where,
n = 1 for linear variation,
n = 2 for parabolic variation, and
n = 3 for cubic variation,

Fig. 2 - SAP2000 Nonprismatic Moment of Inertia Variation [6]

At the location of segment transitions, the use of different flange sizes is common. When this occurs, there exists a discontinuity in the theoretical centroidal axes (Fig.3). In some instances, this separation can be as high as a several inches [7]. The SAP2000 frames in this research use a rigid link to connect the two nodes whenever this situation is encountered.

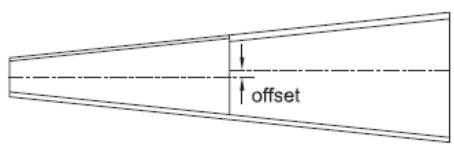


Fig. 3 - Centroidal axis offset at a plate change [7]

2.2.2 Secondary Framing Systems

Purlins are used as a secondary system to transfer the loading from the roof, along the purlin, into the primary frame. The purlins are generally cold-formed steel Z-shaped members that run continuously in the longitudinal direction of the building. For this research, a standard Z-section was used in all the models with a material assignment of a cold-formed steel. The nodal locations were set to the true location of the purlin in relation to the primary frame. A rigid link was used to connect the purlin node to the moment frame node (Fig.4).

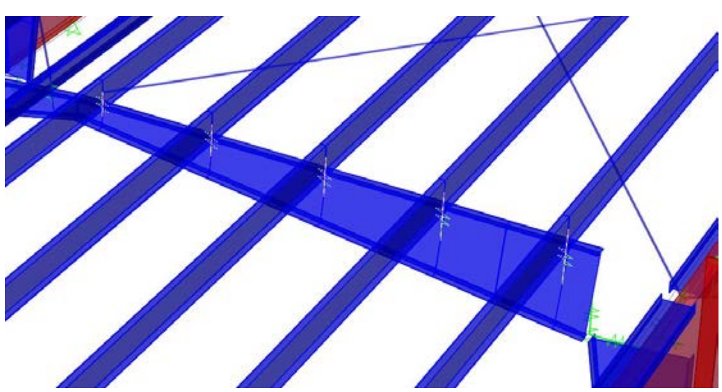


Fig. 4 - Rigid Links connecting Metal Building Frame nodes to Purlin Nodes

The diaphragm was not explicitly modeled. It has been shown experimentally and analytically that the moment frames in metal building systems act independently from one another. Additionally, the roof sheeting appears to provide negligible in-plane shear stiffness [8].

The purlins were modeled as a frame element spanning between each moment frame line. There was no shell element to brace and restrain motion in the purlin along the length. It is apparent that the mass distribution of the roof diaphragm could not be lumped along the length of the purlin, as this would generate superfluous modes of vibration in the purlins. Therefore, the mass of the roof is lumped at the nodes on the frame line based on the tributary area (Fig.5). The mass was assigned in all three spatial directions because horizontal ground excitation can induce vertical vibrations in the rafter segments.

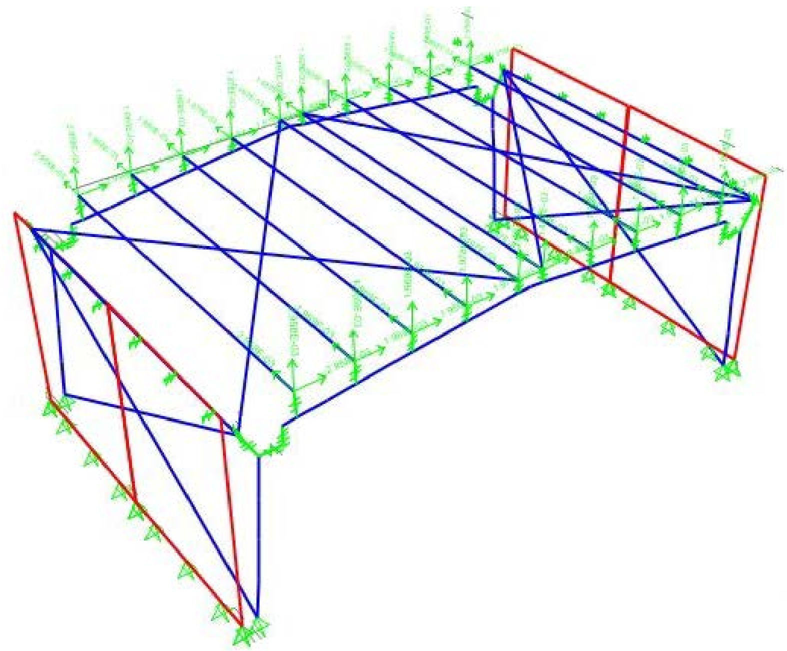


Fig. 5 - Partial SAP2000 Model showing nodal mass at Purlin Nodes

The superimposed dead load, collateral load, live load, and snow load were also applied at the purlin nodes along the primary frame based on tributary area. The self-weight and self-mass of the nonprismatic elements were calculated and included in the models using SAP2000's automatic features. The self-mass property modifiers for the purlins was set to zero as it was already included in the tributary area calculation.

2.2.3 Panel Zone Modeling

It has been recognized that the panel zone is not a rigid element in metal building systems. This research uses the same modeling technique presented by Smith [4] due to its kinematic accuracy. Fig.6 shows the panel zone region as it was modeled in SAP2000. A rigid link was extended to the location of the spandrel beam near the backside of the panel zone.

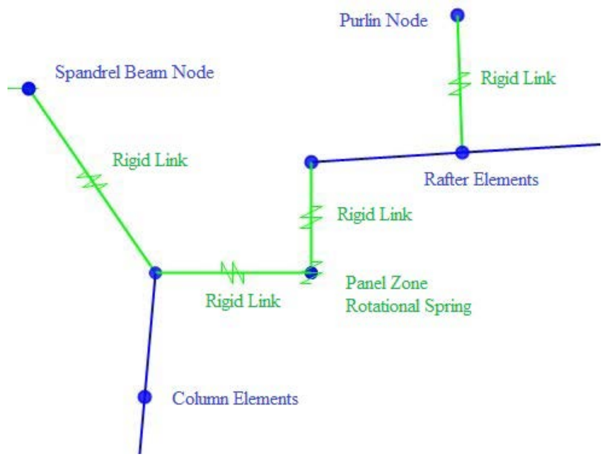


Fig. 6 - Panel Zone Modeling Scheme

The rotational spring stiffness was derived using the same procedure in Abaqus [9] described by Smith [4]. The spring stiffness values calculated in each model are displayed in Table 2. Because Frame 42 has a monoslope roof, the panel zones for the right side and the left side were different.

Table 2 - Rotational Stiffness for Panel Zone Spring

Model Number	Rotational Stiffness KN-m/radian (kip-inches/radian)
16	315000 (2790000)
41	691000 (6120000)
42 Left	504000 (4464000)
42 Right	700000 (6197000)
85	120000 (1060000)
138	531000 (4700000)

2.2.4 Column-to-Base Connections

The true column-base connection is partially rigid. A refined finite element model of the connection would handle this complex behavior, but it was decided not to include it in the SAP2000 models as it would have significantly increased the complexity of the model. It is unlikely that a discrete rotational spring model could be used to capture the changing rotational stiffness due to the changing axial load that exists during a dynamic earthquake analysis. Therefore, the columns in this research utilized an ideal pin condition.

2.3 Hard Wall Modeling

In the development of the SAP2000 models, two hard wall types were considered. The first was precast tilt-up wall panels and the other was a continuous masonry wall. This paper focuses on the tilt-up walls. The wall elements were modeled using a thin shell element with an assigned thickness equal to the nominal wall thickness. The shell elements in these models make up the vast majority of the total degrees of freedom in the model. A fine mesh density would increase the time required for a dynamic analysis. Also, one of the performance goals of the new seismic force resisting system is to move the inelastic behavior away from the wall elements and into the resilient connections. A convergence test was performed to find the minimum mesh density required. This study used a maximum element size of 915 mm x 915 mm (36 in. x 36 in) for the wall elements (Fig.7).

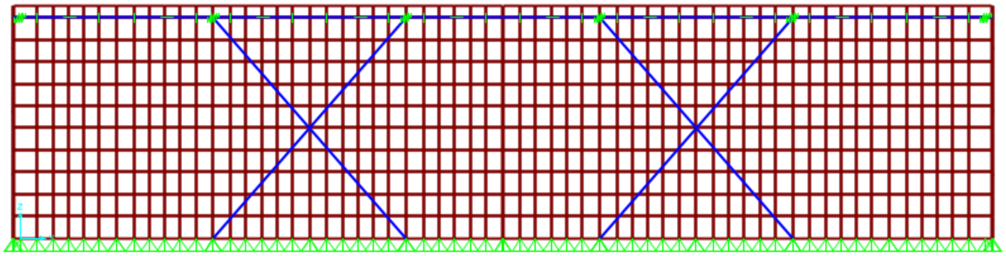


Fig. 7 - Mesh Refinement of Tilt-up Wall Panels

The design of precast tilt-up wall panels are often controlled by the stripping process, transportation, and construction load. The thickness of the panel is sized so that during the removal of the panel from the mold, the wall segment does not crack. Because the seismic loads are not expected to exceed the cracking moment of the tilt-up wall panel, the wall elements were modeled using an elastic concrete material with 28 MPa (4000 psi) concrete. For the nonlinear dynamic analyses, it is assumed that the panels will remain uncracked during the earthquake. Precast tilt-up wall panels that are built at a fabrication plant and have to be transported to a work site, must be dimensioned in a manner that is transportable. The distance between frame lines is 7.6 m (25 feet) in all the models. For the sidewalls, two discrete tilt-up panels are used in each bay. These panels are not connected to each other and a gap of 13 mm (0.5 inch) separates them. The segmented wall panels are modeled so that they will not interact with each other in any way during the dynamic analysis. Openings in the end wall segments were not explicitly modeled. To account for openings, the end wall shell element mass and weight properties were reduced by 20%.

2.4 Longitudinal Bracing and Diaphragm Bracing

For metal building systems, it is typical for the lateral force resisting system in the longitudinal direction of the building to be composed of diagonal tension-only rod braces. Table 3 displays the longitudinal dimension of each metal building frame, as well as the number of bays that contain the bracing system. It is not common to place rod bracing in adjacent bays due to constructability issues.

Table 3 - Longitudinal Geometry of Metal Building Sample

Model Number	Transverse Dimension m (ft)	Longitudinal Dimension m (ft)	Number of Bays	No. of Bays with Rod Bracing
16	12.2 (40)	22.9 (75)	3	1
41	30.5 (100)	38.1 (125)	5	2
42	48.8 (160)	53.3 (175)	7	3
85	12.2 (40)	22.9 (75)	3	1
138	36.6 (120)	38.1 (125)	5	3

The sidewall bracing for this research was designed as an ordinary concentrically braced frame [10]. The equivalent lateral force method was used to size the bracing. Due to the large mass of the end walls, the seismic load combinations controlled the design. The roof diaphragm bracing was designed using the overstrength factor, Ω, to ensure the braces remain elastic for the full strength of the energy dissipating fuse elements. The 3-D SAP2000 models were used to perform the structural analysis of the bracing system. This is a departure from common practice, as these systems are designed using 2-D models of the longitudinal frame and roof diaphragm. This was done here for convenience since the 3-D models had already been generated.

The roof diaphragm bracing was located at purlin nodes. An effort was made to maintain an aspect ratio close to 1:1 for the x-bracing. The purlins act as struts in the roof diaphragm. If a purlin was found to have inadequate axial capacity, an adequate pipe strut was used to replace the purlin (Fig.8).

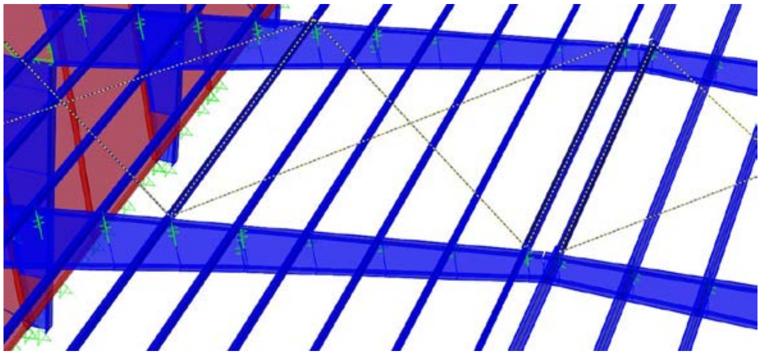


Fig. 8 - Roof Bracing and Pipe Struts in Diaphragm

To model the nonlinear brace behavior, a tension only axial hinge was utilized. The backbone curve assigned to the rod braces was based on the FEMA hinge definitions for a brace in tension [11]. It is assumed that the brace can reach an ultimate capacity of 1.27 times the yield stress.

2.5 Connection Configurations

The location of the new resilient connections must be constructible, practical, and effective. To provide relief for the stiffness incompatibility between the hard wall and metal building frame, two connection configurations were considered in the SAP2000 models. The spandrel beam transfers the out-of-plane seismic load from the walls into the metal building moment frame. The spandrel beams were designed using ASCE 7 [12] and AISC 360 [13]. The 4.6 m (15 ft) wall required a W310x32.7 (W12x22) and the 9.2 m (30 ft) wall required a W310x38.7 (W12x26).

The energy dissipating connections were located at the interface between the walls and the spandrel beam (Fig.10). For Model 85, one friction connection was placed in each tilt-up panel. These connections are designed to slip at a prescribed longitudinal load and are the only connections that have a longitudinal resistance (See Fig.9 for slip direction). The other two connections provide out-of-plane resistance and no longitudinal resistance. The metal building framing system will be able to move as one unit in the longitudinal direction, allowing multiple energy dissipating connections to participate. This connection configuration requires communication between the hard wall engineer and the engineer-of-record so that connection forces are adequate.

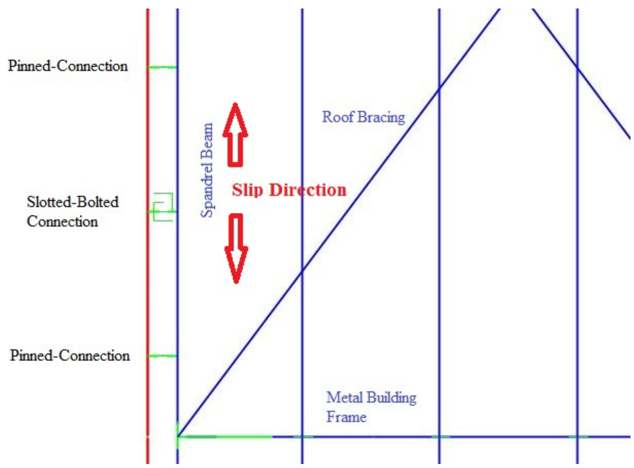


Fig. 9 - Wall-Spandrel Connection Configuration

2.6 Connection Parameters

For the case study, 3-D Model using Frame 85 will use slotted-bolted friction connections between the hard wall and spandrel beam. The resilient friction connection was modeled in SAP2000 using a multi-linear plastic link element. The rotational degrees of freedom of the link element were set as fixed. SAP defines the local axis U1, U2, U3, R1, R2, and R3 respectively as axial deformation, vertical deformation, horizontal deformation, torsional deformation, lateral bending rotation, and vertical bending rotation. For the wall-spandrel resilient connection configuration, the link local U3 direction (horizontal displacement) corresponds to the longitudinal direction of the metal building system. The nonlinear properties of an elastic-perfectly plastic behavior were assigned with an initial stiffness of 5.3 KN/mm (30 kips/in) and a slip force of 13.3 KN (3 kips). The axial stiffness (U1) and vertical shear stiffness (U2) was set to remain linear. The axial stiffness assigned was 53 KN/mm (300 kips/in) and the vertical shear stiffness was 5.3 KN/mm (30 kips/in).

2.7 Nonlinear Dynamic Analyses

The earthquake ground motions that were used in the nonlinear dynamic analyses are listed in Table 4. The suite of ground motions was downloaded from the PEER NGA database [14]. The earthquakes were scaled so that the spectra of the individual ground motions matched the MCE spectra for Riverside, California at three frequencies of importance to the analysis. The earthquake records were selected due to the fact that the shape of the spectra were consistent with the MCE spectrum.

Table 4 - Earthquake Suite for Nonlinear Dynamic Analyses

	NGA#	**Event**	**Year**	**Station**	**Timestep (seconds)**	**Scale Factor**	**Duration (seconds)**	**Frame Direction**	**Long. Direction**
1	57	San Fernando	1971	Castaic Old Ridge Route	0.01	3.1	30	ORR021	ORR291
2	125	Friuli, Italy	1976	Tolmezzo	0.005	3.028	36.345	TMZ270	TMZ000
3	126	Gazli, USSR	1976	Karakyr	0.005	1.073	16.265	GAZ000	GAZ090
4	184	Imperial Valley-06	1979	El Centro Differential Array	0.005	1.456	38.96	EDA360	EDA270
5	725	Superstition Hills	1987	POE	0.01	2.366	22.3	POE270	POE360
6	752	Loma Prieta	1989	Capitola	0.005	1.697	39.955	CAP090	CAP000
7	960	Northridge	1994	Canyon Country - W Lost Canyon	0.01	1.272	19.99	LOS000	LOS270
8	1084	Northridge	1994	Sylmar Converter Station	0.005	0.876	40	SCS142	SCS052
9	1107	Kobe, Japan	1995	Kakogawa	0.01	2.147	40.96	KAK000	KAK090
10	1158	Kocaeli	1999	Duzce	0.005	1.872	27.185	DZC270	DZC180
11	1513	Chi-Chi, Taiwan	1999	TCU079	0.005	1.434	90	TCU079-N	TCU079-E

The inherent damping used in all analyses was represented using Rayleigh Damping. Metal building systems have been shown experimentally to have lower inherent damping than conventional moment frames due to fewer connections [4]. The damping ratios were set to 2% critical damping at periods that corresponded to the natural period of vibration and 20% of the natural period ($0.2T_n$).

Before each nonlinear dynamic analysis, a nonlinear static analysis with P-delta effects was performed with the dead load and collateral load. The nonlinear dynamic analyses started from the last step of that nonlinear static analysis. The time-integration method used was the Hilber-Hughes-Taylor alpha method. The default alpha value used in the all analyses was -0.05.

3. Case Study Results

3.1 Modal Analysis Results

The fundamental periods of vibration provide insight into the dynamic characteristics of a structure. A modal analysis was performed for each model.

Table 5 - Periods of Vibration and Mass Participation Ratios

Frame Number	Mode Number	Period (Seconds)	Ux (Moment Frame Direction)	Uy (Longitudinal Direction)	Uz (Vertical Direction)
16	1	0.625	0.496	0	0
	3	0.368	0.068	0	0
	7	0.339	0	0.279	0
41	1	0.761	0.463	0	0
	3	0.436	0	0.412	0
	4	0.379	0	0	0.000467
42	1	0.857	0.487	0	0
	3	0.751	0.0147	0	4.22E-05
	4	0.631	2.7E-05	0.448	0.000211
85	1	0.417	0.469	0	0
	2	0.218	0	0.307	0
	5	0.186	0	0.0017	0
138	1	0.742	0.422	7.5E-06	0
	3	0.566	3.74E-06	0.370	1.75E-05
	4	0.380	0.000304	1.28E-05	0

As illustrated in Table 5, there exists very low mass participation ratios for each structure. In Smith's [4] fundamental period study, metal building frames with a high aspect ratio exhibited lower mass participation ratios. When looking at the first longitudinal mode of vibration, the only mass that participates in that mode of vibration is mass that is attached to the flexible steel frame. The rest of the mass is locked away in the stiff side wall panels. Shear wall vibration can only be excited at very high frequency vibrations due to its high stiffness. This demonstrates that metal building systems with hard walls are essentially two different systems tied together, each with very different stiffness and dynamic characteristics. This further emphasizes the importance of the connections between these two elements.

The fact that the mass participation ratios in this research are so low for metal building systems with hard walls, the applicability of the Equivalent Lateral Force (ELF) method to design these structures comes into question. The ELF method is used to determine the seismic load for design. It assumes that the first mode of vibration has a mass participation of 100%. Another finding of this research was that hundreds of modes were required to reach 90% mass participation. A linear response history analysis procedure may need to be used for determining seismic demands for these structures.

3.2 Slotted-Bolted Friction Connection Behavior

The 3-D model utilized 2-node link elements to model a friction connection between the side wall panels and the spandrel beam. The slip force assigned to the link element was 13.3 KN (3 kips) with an elastic stiffness of 53 KN/mm (300 kips/in) in out-of-plane direction, and 5.3 KN/mm (30 kips/in) in the in-plane direction. One slotted-bolted friction connection was placed in each side wall panel. For each bay, there were a total of 6 wall connections with two being slotted-bolted friction connections.

The response history of the two tension-only side wall braces and one of the friction connections during the Imperial Valley excitation for Model 85 is shown in Fig.11. The axial force of the braces is plotted along with the horizontal shear force of the friction connection. The results show that the friction device reached its prescribed slip force and successfully relieved the large stresses that would have developed in the wall panel. The cyclic response of the friction device demonstrates favorable behavior in regard to energy dissipation. Fig.12 displays the hysteresis loop of one of the friction connections. The maximum positive displacement was 34 mm (1.34 inches) and the maximum negative displacement was -37 mm (-1.46 inches).

The tension-rod braces provided self-centering characteristics to the friction connections throughout the earthquake. In order to achieve a low damage self-centering behavior, the tension-rod braces must be designed with enough strength to be able to restore the resilient connection to the neutral position or a residual drift could result (Fig. 11). By moving the inelastic mechanism to the resilient connections, the braces can be designed to remain elastic. The fact that this new connection can activate several times and self-center is a promising means of adding energy dissipation and improving the global performance of the system.

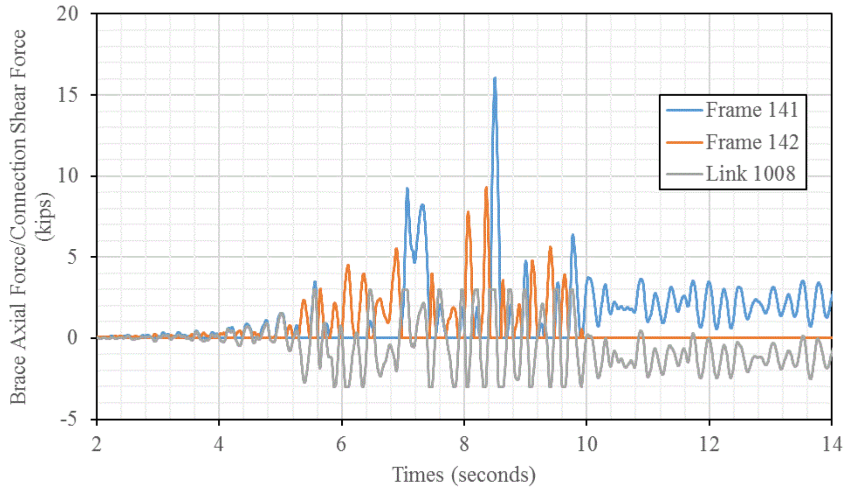


Fig. 11 - Response History of Tension-Only Side Wall Braces and Friction Connection

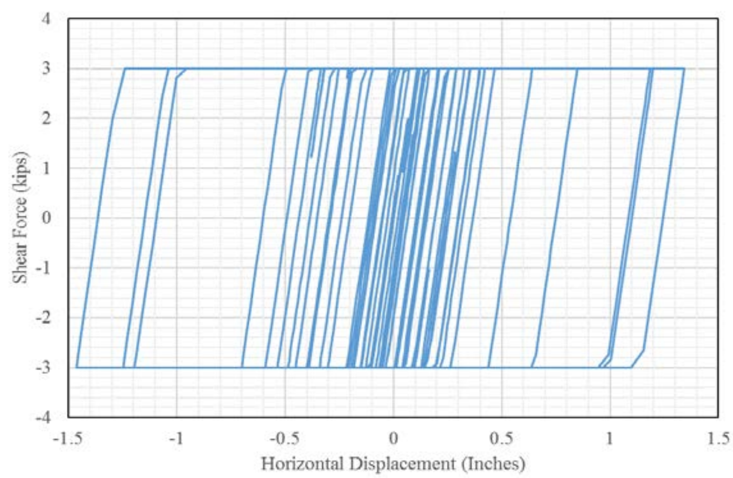


Fig. 12 - Hysteresis Loop of Friction Connection 1008 during Loma Prieta Excitation

4. Conclusion

Post-earthquake reconnaissance [3] revealed that seismic performance of metal building systems in the longitudinal direction is unacceptable. Poorly designed connections between the hard walls and steel frame can fail in a brittle manner and the wall can fall away from the structure. A new seismic force resisting system that relies on resilient connections can relieve the stiffness incompatibility that exists between the hard wall and steel frame and dissipate energy. Before work could proceed, a 3-D computer model for metal building systems with hard walls needed to be developed. The interaction between the steel frame and hard wall is a 3-D problem as the connections experience both in-plane and out-of-plane force and deformation demands.

The research presented in this paper lays out a modeling procedure for metal building systems with hard walls in SAP2000 that can be used for nonlinear dynamic analyses. A case study was performed for one of the metal building systems in 3-D to assess component behavior. Based on the success of the case study, this 3-D model can effectively be used to quantify connection strength, post-yield deformation, and energy dissipation capacities required to achieve enhanced performance of metal building systems with hard walls.

There are several findings from this research regarding the 3-D modeling procedure. A two-node multilinear plastic link element is appropriate for modeling the friction connection between the hard walls and steel frame. Friction connections successfully relieve the stiffness incompatibility between the stiff hard walls and flexible steel frames during earthquake excitation. Tension-only x-bracing provides self-centering characteristics to the friction connections. Energy dissipation capacities of the friction connections show promising behavior due to self-centering characteristics and potential for multiple activation cycles. 3-D models of metal building systems that have explicitly modeled hard walls exhibit very low mass participation ratios in both horizontal directions. It is likely that the mass within the hard walls is not participating in the fundamental modes of vibration.

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