CHAPTER 8 THE DESIGN AND ANALYSIS OF MULTI-STORY BUILDINGS WITH LARGE MOMENT END-PLATE CONNECTIONS

8.1 INTRODUCTION

According to the *AISC Seismic Provisions for Structural Steel Buildings* (1997), steel moment-resisting frames can be designed such that beam hinging or yielding of the connection components is the system's primary form of inelastic behavior. Fully restrained (FR) construction is limited to beam hinging for intermediate and special moment frames, and connection yielding or beam hinging is allowed for ordinary moment frames. The overall inelastic capacity of the system must be determined experimentally and is described by a measure of inelastic rotation in the proximity of the beam-to-column connection. The anticipated performance of the structure is then assumed to be directly related to the maximum plastic rotation provided by the system. However, this interpretation can be misleading, for it is well known that a structural component's seismic performance is best measured as the area contained inside its hysteretic response curve.

The hysteretic curves (applied moment vs. plastic rotation) for connection yielding are typically "pinched" in shape and enclose a lesser area than those developed for beam hinging. Thus, it can be argued that beam hinging dissipates a greater amount of energy than does connection yielding for the same considered plastic rotation capacity. In this chapter, known moment-rotation characteristics of large moment end-plate connections are used to examine multi-story frames under actual earthquake excitations to better understand the role of connection response in the overall behavior of a building's lateral force resisting system. Impacts of FR and PR design philosophies are discussed and practical cases are analyzed for different frame configurations. As an introduction, the impact of the *International Building Code* (2000) and the *AISC Seismic Provisions for Structural Steel Buildings* (1997) on metal building design is considered with a purpose of establishing guidelines for when the latter document should be considered in the design of such structures. Metal buildings are usually prefabricated structures that can be designed for different purposes (e.g. warehouses, churches, commercial buildings), but most have lightweight roofing and wall systems. Here, a specific type of metal building system that uses flat roof single-span rigid frames is analyzed under design earthquake, wind, and actual earthquake loading to show that elastic design of these systems is the most feasible design methodology for seismic loading.

8.2 SEISMIC DESIGN OF METAL BUILDING SYSTEMS

As a result of failures uncovered after the Northridge earthquake, the *AISC Seismic Provisions* has become more stringent in its design provisions for moment frame structures. Although the changes are justified, they are not necessary for every type of building system. Some structures can be safely designed to resist earthquake forces elastically without concern of structural collapse. Metal buildings are typically lightweight, thus small inertia forces from the design earthquake will not usually result in an inelastic response of a system that is properly designed to resist wind forces. According to the document itself, the *AISC Seismic Provisions* is applicable to buildings that are classified as Seismic Design Category D or higher (e.g., E or F) by the governing building code. Using the *International Building Code* (2000) along with the AISC document, it is shown in the following sections that adherence to the *AISC Seismic Provisions* is not required in most cases except for locations on the west coast and a few regions east of the Rocky Mountains.

8.3 ESTABLISHING SEISMIC DESIGN CATEGORIES

The Seismic Design Category of a structure is a function of the occupancy or use of the structure and the seismicity of the site. Since metal buildings are not usually considered structures that represent a substantial hazard to human life, it will be assumed in this analysis that the structures are classified as Seismic Use Group I. With this assumption, the short period (also called 0.2 second) spectral acceleration, S_s, and the one-second spectral acceleration, S_1, directly determine the structure's Seismic Design Category. To avoid the stringent requirements of the *AISC Seismic Provisions*, S_s and S_1 must both be below certain limiting values that result in a Seismic Design Category D classification. These limiting values are listed in Tables 8-1 and 8-2. For example, if a geotechnical investigation determines a location to be classified as Site Class A and the spectral accelerations for the site are S_s = 0.90 g and S_1 = 0.45 g, the Seismic Design

Category is D or higher (S_1 is greater than 0.375 g and controls the classification). However, if the same location is classified as Site Class C with S_s = 0.40 g (less than 0.6603 g) and S_1 =0.15 g (less than 0.1859 g), the Seismic Design Category is less than D and the *AISC Seismic Provisions* do not apply. Figures 8-1 and 8-2 show the information in Tables 8-1 and 8-2 graphically for locations east of the Rocky Mountains. Figure 8-1 shows locations (areas inside dark outlines) where adherence to the *AISC Seismic Provisions*, based on short period spectral accelerations, is required for different site classifications. For Site Classes A, B, C, and D, the area is limited primarily to the New Madrid fault region and most of South Carolina. However, for Site Class E, the area is greatly expanded to include much of the eastern United States. Figure 8-2 shows locations where adherence to the *AISC Seismic Provisions*, based on one second spectral accelerations, is required for different site classes. For Site Classes A, B, and C, the area is limited primarily to the New Madrid fault region and most of South Carolina. Unlike the short period map for Site Class D, the one-second area is much larger and includes a greater portion of the southeastern United States. For Site Class E, about one half of the eastern United States is included.

For a particular project, if a geotechnical investigation is not performed, the design engineer must use the default of Site Class D (or higher if determined by the building official). Either way, the choice of avoiding a geotechnical investigation leads to the large controlling area shown in Fig. 8-2(d) where adherence to the *AISC Seismic Provisions* is required. The exemptions to following this document, as discussed above, are based only on attempting to show that the Seismic Design Category of a structure is A, B, or C. This leads to the disappointing conclusion that, for most of the United States, the only way to avoid the unnecessary stringent seismic detailing requirements is to perform a geotechnical investigation of the site to prove a site classification of A, B, or C. On the other hand, the problem can be avoided entirely if it can be shown that the structure will respond elastically to the design earthquake.

8.4 ELASTIC ANALYSIS OF METAL BUILDING SYSTEMS

In this section, flat roof single-span rigid frames that act as lateral force resisting systems are analyzed under design wind and earthquake loadings as specified in the International Building Code (2000). It is shown that for all practical frame configurations, wind loads far exceed those required to resist the design earthquake elastically. Based on recommendations from Newman (1997), frame widths between 60 and 120 ft, and eave heights between 10 and 24 ft, are considered. A standing-seam roof weighing 2.5 psf is modeled as flexible roofing and a frame tributary width, W, of 25 ft is used to calculate the roof dead load. Wall loads of 3.0 psf are assumed. Gravity loads from snow and rain are not considered in the analysis to increase the effect of inertia forces from the design earthquake. Based on probability analysis a small percentage of their design value is included in actual design.

Figure 8-3 shows the frame configuration and design earthquake loading for the structure. In Figs. 8-3(a) and 8-3(b), the base is shown fixed and pinned, respectively. The factored seismic loading terms w_{du}, w_{pu}, P_u, V_u, represent the beam load per unit length, the column load per unit length, column load (vertical contribution as required), and the equivalent lateral force, respectively. Current building codes such as the *International Building Code* (2000) require that the base be modeled as fixed. This causes concern when considering metal building systems. The column base of a metal building frame is much more flexible than those from typical moment frames that are part of multi-story buildings. On the other hand, it has not been shown in the literature that metal building systems can provide enough flexibility (in lieu of ductility) at the base for the elastic drift caused by earthquake forces. Without this information, current designs must include strong base plate connections that can develop the design seismic moment at the base. Otherwise, the system can collapse due to brittle failure at the base connection. However, in light of the results of this research, it is anticipated that testing on typical base plates will be performed to show that the pinned base assumption is adequate. Both fixed and pinned bases are considered here.

The simplified procedure (equivalent lateral force method) of section 1617.5 of the International Building Code (2000) is used to analyze the structure. However, the response modification factor, R, is not used to reduce the earthquake forces since the anticipated behavior of the structure is completely elastic. For reasons that will soon become apparent, this procedure is very conservative for metal building design. The design acceleration is assumed to be 120 percent of the design short period elastic response acceleration, S_{DS}. In other words, regardless of the structure's lowest natural period, the base shear is calculated using 1.2 times the peak of the design acceleration response spectrum. Two load cases are considered. They are 1.2D+1.0E and 0.9D-1.0E where D and E represent the dead load effect and seismic load effect, respectively. The seismic load effect is $E = \rho Q_E + 0.2S_{DS}D$ where ρ is the reliability factor (1.0, by definition, for this case), and Q_E is the effect of the horizontal seismic force. Load cases involving the system overstrength factor, Ω_O, are not considered due to the assumed elastic response of the structure. All assumptions and definitions above are in accordance with the *International Building Code* (2000).

For the load case 1.2D+1.0E, the design earthquake loads are found as

$$\begin{aligned}
V_u &= 1.0(\rho)\{1.2S_{DS}[(2.5W + w_B)L + 0.5(3.0)Wh(2) + 0.5w_C h(2)]\} \\
P_u &= 1.0\{0.2S_{DS}[0.5(3.0)Wh + 0.5w_C h]\} \\
w_{du} &= 1.2(2.5W + w_B) + 1.0[0.2S_{DS}(2.5W + w_B)] \\
w_{pu} &= 1.2(3.0W + w_C)
\end{aligned} \qquad (8-1)$$

where w_B and w_C are the column and beam weights per unit length, respectively.
For the load case 0.9D-1.0E, the design earthquake loads become

$$\begin{aligned}
V_u &= 1.0(\rho)\{1.2S_{DS}[(2.5W + w_B)L + 0.5(3.0)Wh(2) + 0.5w_C h(2)]\} \\
P_u &= -1.0\{0.2S_{DS}[0.5(3.0)Wh + 0.5w_C h]\} \\
w_{du} &= 0.9(2.5W + w_B) - 1.0[0.2S_{DS}(2.5W + w_B)] \\
w_{pu} &= 0.9(3.0W + w_C)
\end{aligned} \qquad (8-2)$$

Figure 8-4 shows the frame configuration and design wind loading for the structure. The base is pinned as typical for wind analysis of metal building systems. For simplicity, it will be assumed that hill or ridge effects need not be considered in the analysis. Although this is not always the case, these effects increase the design wind loading and thus defeat the purpose of this section, which is to compare minimum wind load effects to those from the design earthquake. Again, two load cases are considered. They are 1.2D+1.3$\underline{W}$ and 0.9D-1.3$\underline{W}$ where $\underline{W}$ represents the wind load effect. The underscore is used to separate the wind load effect from the transverse frame width W.

Following the design procedure outlined in the International Building Code (2000) for low-rise buildings, the design wind loads for 1.2D+1.3W are found as

$$
\begin{aligned}
&p_{R1} = 1.2(2.5W + w_B) + 1.3q_h(-0.69 + 0.18)W \\
&p_{R2} = 1.2(2.5W + w_B) + 1.3q_h(-0.37 + 0.18)W \\
&p_w = 1.3q_h(0.40 + 0.18)W \\
&p_l = 1.3q_h(-0.29 + 0.18)W \\
&w_{pu} = 1.2(3.0W + w_C)
\end{aligned}
\qquad (8-3)
$$

where q_h is the wind pressure at height h.

For the load case 0.9D-1.0W, the design wind loads become

$$
\begin{aligned}
&p_{R1} = 0.9(2.5W + w_B) + 1.3q_h(-0.69 - 0.18)W \\
&p_{R2} = 0.9(2.5W + w_B) + 1.3q_h(-0.37 - 0.18)W \\
&p_w = 1.3q_h(0.40 - 0.18)W \\
&p_l = 1.3q_h(-0.29 - 0.18)W \\
&w_{pu} = 0.9(3.0W + w_C)
\end{aligned}
\qquad (8-4)
$$

Using the load cases discussed above for both seismic and wind loading, frames of various dimensions of L and h were analyzed to determine the maximum moments in the beam, columns, and connections. Hot-rolled, W21x44 and W21x68 beam sections are used in combination with W12x26 and W12x58 column sections as required using the aforementioned load combinations. However, typical built-up members for these frames will have thinner webs and weigh less. In effect, this will decrease the seismic forces, but have no effect on wind load combinations. A wind velocity of 90 mph is used, since it represents the lower bound design wind speed for most of the United States. Also, it is assumed that S_{DS} = 0.55g so that Fig. 8-1(d) can be used conservatively (actual area is smaller) to determine the regions where frame designs of specific configurations are controlled by wind loading. This value was chosen as an acceptable limit based on the map discussions of the previous section. To conclude that a frame designed for wind loading will respond elastically during the design earthquake, maximum moments in the beam, columns, and connections are compared.

Table 8-3 shows the maximum moment ratios (wind loading vs. earthquake loading) at the beam-to-column connection for various frame configurations and a fixed base. Since in all cases the ratio is greater than 1.00, wind loading controls the frame design and the *AISC Seismic Provisions* do not apply. Wind load requirements exceed seismic requiremens by 50% or more in all but four cases. However, the column base moment is 60% to 90% of the beam-to-column moment requiring very strong base plates.

Table 8-4 shows the maximum moment at the beam-to-column connection for various frame configurations and a pinned base. Although the ratios are reduced from the fixed based condition, wind loads still exceed seismic loads in all cases and no base moment is developed. These results are valid irrespective of the seismic design category (since they are based only on the level of ground shaking) and Fig 8-1(d), in lieu of Fig. 8-2(d), can be used to determine when the *AISC Seismic Provisions* apply. Also, it is interesting to note that for increased wind velocities, design wind forces increase and the ratios become even larger (this is significant for areas that are both near the coast and in higher seismic areas, like Charleston, SC).

To ensure the safety of the equivalent lateral force method used to generate design earthquake loadings on the structure, actual ground displacements from the Northridge earthquake are used to excite a metal building having L=80 ft and h=15 ft (building shown in Fig. 8-3). Using the equivalent lateral force method for this building (pinned base) in the Northridge area results in a design moment of about 250,000 lb-ft. The first three natural periods of the fixed base structure are 0.37 s, 0.26 s, and 0.12 s and the accompanying mode shapes are shown in Fig. 8-5. The first three natural periods of the pinned base structure are 0.53 s, 0.39 s, and 0.12 s and the corresponding mode shapes are shown in Fig. 8-6. It is interesting to note that the first mode for the fixed base structure primarily involves beam vibration. The second mode shape, which involves lateral sway, is usually the controlling (or first) mode shape for moment frames. This could make metal building structures more susceptible to vertical excitations than is typical for a standard lateral force resisting system. For the pinned base case, these first two modes are reversed as the sway mechanism has a higher natural period.

Horizontal ground displacements from the Northridge Earthquake, shown in Fig. 8-7, are used to excite the structure. For this analysis, it is assumed that the earthquake begins at a time of t=5 s and lasts approximately 60 s. During the prior five seconds the weight of the structure is increased linearly until its total weight is in place. Only the dead load of the structure is included in the analysis. Two-dimensional elastic beam elements are used to model the assembly. Five nodes along the columns and eleven nodes along the beam describe the model. Models composed of twice the number of nodes provide identical results to six significant figures. Horizontal and vertical displacements of the beam center node and the moment at the controlling connection are recorded with respect to time. The displacements may or may not be the maximum displacement of any node of the structure at a given time, but are measured given the primary mode shapes involved. For horizontal excitation, the moments at the two connections are different throughout the loading. The connection with the larger maximum moment is the controlling connection.

The fixed base structure is analyzed first with no damping. Figure 8-8 shows the horizontal displacement of the beam center node during the horizontal excitation. As expected, the stiff structure follows the applied displacements very closely. Since the structure is not damped, it begins to vibrate at its second natural frequency. This is expected since it is excited in the direction of its second mode. Figure 8-9 shows the vertical displacement of the center beam node for the undamped fixed base structure. It is clear that the only motion of the beam is caused by the first five seconds of gravity loading prior to the earthquake. The beam center node vibrates as if it were in free vibration. Figure 8-10 shows the moment at the controlling connection node (left node in this case) for the undamped fixed base structure. A maximum moment of about 85,000 lb-ft is recorded. Since there is no damping, the connection vibrates continuously between 65,000 and 15,000 lb-ft near the end of the excitation. Note that a full reversal of loading does not occur and little opposite moment is developed throughout the loading.

Although the correct amount of damping to be used when modeling metal building systems has not been established, some damping does exist as in any structural system. In accord with code recommendations, it is now assumed that 5% damping is inherent in the system. Figure 8-11 shows the horizontal displacement of the center beam node for the damped structure. The response is basically the same as in Fig. 8-8 with the exception that the damping reduces the vibration of the structure at its second natural frequency. This is also shown in Fig. 8-12 where the initial vertical free vibration of the beam center node is almost completely dissipated in the first few seconds after the gravity load is in place. As shown in Fig. 8-13, the maximum moment at the connection is reduced to about 67,000 lb-ft, and after the first 15 seconds of earthquake excitation, earthquake-induced moments are insignificant.

As previously discussed, the fact that the first mode involves beam vibration, may cause significant forces to develop in the frame as a result of vertical earthquake excitations. Therefore, vertical ground displacements from the Northridge earthquake, as shown in Fig. 8-14, were used to excite the damped fixed base structure. Horizontal displacements of the beam center node are theoretically zero since the structure and loading are both symmetric. Vertical displacements of the beam center node are shown in Fig. 8-15. Although the displacement pattern is similar to the loading, note that the magnitudes of the displacement are almost double. Figure 8-16 shows the moment at the connection for the vertical loading. A maximum moment of about 73,000 lb-ft is obtained at an isolated spike. This value is larger than that due to the horizontal excitation. This is interesting since typical seismic design practice considers vertical effects to be only a fraction of those caused by horizontal excitations. Finally, the moment at the connection for the combined (horizontal and vertical) excitation is shown in Fig. 8-17. Since the maximum moments for the horizontal and vertical excitations do not occur at the same time, the maximum moment for the combined excitation remains about 73,000 lb-ft.

Now the response of the damped pinned base metal building is discussed. Figure 8-18 shows the horizontal displacement of the beam center node for the horizontal excitation. The results are very similar to those for the fixed base structure in Fig. 8-11. Figure 8-19 shows the moment at the connection for the horizontal excitation. As expected, compared to Fig. 8-13, the moments are greatly increased due to the change in load path. The increased natural period associated with the sway mode impacts the results as well. A maximum moment of about 140,000 lb-ft is obtained. This is over twice that obtained for the fixed base structure. The vertical displacement of the beam center node for the vertical excitation is shown Fig. 8-20. Again the results mimic the fixed base results (Fig. 8-15) quite well. Figure 8-21 shows the moment at the connection for the vertical excitation. A maximum of about 64,000 lb-ft is obtained. This is less than the 72,000 lb-ft for the fixed base structure (Fig. 8-16), but is close to this value since the natural periods for beam vibration are very close in the two cases. Also, the load path for vertical excitations is not altered very much since no horizontal inertial forces are developed. Finally, Fig. 8-22 shows the moment at the connection for the pinned base under the combined excitation. The maximum moment is reduced, by chance, to about 130,000 lb-ft. Since the analysis is elastic, the vertical and horizontal components of the earthquake can be used separately to excite the structure and then superimposed to get a combined solution. In this case, the maximum moment at the connection due to the horizontal effect is decreased due to a simultaneous negative earthquake induced moment caused by the vertical effect.

Although only two cases of linear time history analysis are considered here, these results suggest the conservative nature of the static design procedure for seismic loading. For the pinned base case, the maximum moment of 130,000 lb-ft as obtained for the combined excitation is about 50% of the design moment found for the Northridge area using the simplified procedure of the *International Building Code* (2000). It is also slightly less than the 132,000 lb-ft design moment found for wind loading. This shows clearly that elastic design of metal building systems is appropriate in areas of high seismicity. On the other hand, it has been shown that vertical excitations can cause significant moments to these structures, and accounting for a slightly larger vertical effect than is currently established in the codes may be required.

8.5 ELASTIC DESIGN OF METAL BUILDING SYSTEMS

The purpose of this part of the study is to show that elastic design of metal building systems for seismic forces is a feasible, economical, and safe approach to design. The maps and tables of the previous section are by no means inclusive, yet they do provide significant insight to when and why wind forces can control the design of metal buildings in most areas of the United States.

Even if the structural system is excited by an earthquake of a magnitude greater than the design earthquake, the results suggest that only a small amount of inelastic deformation, if any, will occur. As a final note, some design recommendations should be established for elastic seismic design. First, it should be realized that some locations of the United States (e.g., California, New Madrid, Charleston, etc.) are prone to high spectral accelerations, and the static lateral force procedure for seismic loading will result in frames that must resist moments two to three times those developed by wind loads. In these cases, it may prove more economical to design metal buildings to respond inelastically with proper seismic detailing. On the other hand, the nonlinear time history analysis of this chapter suggests that a less conservative simplified design procedure may be beneficial for metal building systems. Further study is needed to determine if this is true. For areas of low to moderate seismicity, stringent detailing requirements are not necessary since structures can be designed to behave elastically under the design earthquake. It should be noted, however, that a structure may respond inelastically in the event of severe earthquake loading. Therefore, the primary lateral force resisting system should be designed such that all brittle limit states are avoided. For example, for rigid single-span frames, this can be achieved by designing the moment end-plate connections such that the bolt rupture strength is 10 to 20 percent stronger than the end-plate bending strength.

8.6 NONLINEAR TIME HISTORY ANALYSIS OF MULTI-STORY FRAMES

The second part of this chapter deals with the inelastic response of multi-story buildings (not lightweight metal buildings) composed of steel moment frames under seismic excitations. Specifically, the aim here is to determine exactly what effect the shape of typical hysteretic curves (applied moment vs. plastic rotation) has on energy dissipation and response characteristics of frames composed of moment end-plate connections. Hysteretic curves that are based on connection yielding and those developed for beam hinging are very different in appearance. Connection yielding causes a “pinched” response that encloses relatively little area inside the backbone hysteresis loop. As a result, less energy is dissipated by this mechanism. On the other hand, curves developed from beam hinging represent a much more ductile structural response and these large loops can dissipate a significant amount of energy during an earthquake.

In seismic design practice, design earthquake forces are reduced by a seismic force reduction factor R that accounts for the inelastic behavior of a structural system. The particular value for R depends on the type of lateral force resisting system being considered, and it represents a summation of the effects of global damping, hysteretic damping, and period shift due to stiffness degradation. Since global damping is usually considered as some percent of critical damping (e.g., 5%) and used when performing an elastic time history analysis, it is not included in this nonlinear time history study. Instead, this study of two-story frames will focus on determining what role hysteretic damping, period shift, and moment redistribution play in a structure's overall dynamic response.

Two frames, as shown in Fig. 8-23, are considered under cyclic loading. The beam-to-column connections are labeled in this figure and have the experimentally determined (Sumner et al., 2000) moment vs. plastic rotation characteristics shown in Fig. 8-24. Figure 8-24(a) shows the moment/plastic rotation characteristics of a four-bolt extended connection and Fig. 8-24(b) shows the measured response curves for an eight-bolt extended stiffened connection. In each of the two figures, (a) and (b), both thin and thick end-plate response curves are given. The thin plate and thick plate curves represent connection yielding and beam hinging mechanisms, respectively. However, the curves do include all forms of inelastic behavior as incurred during testing. Hence, panel zone deformations are included in these "total plastic rotation" plots. Transition values for the curves are given in Tables 8-5 and 8-6.

Ten frame configurations are considered. Five use four-bolt extended end-plates and the other five use eight-bolt extended stiffened end-plate connections. For the two-story one-bay frame of Fig. 8-23(a), the connections C1 through C4 are four-bolt extended end-plates that are modeled three different ways. First, they are modeled as fully restrained elastic springs of infinite stiffness to determine the elastic response of the structure in the absence of any energy dissipating or force reducing mechanism. Next they are modeled using both the thin and thick plate curves shown in Fig. 8-24(a), but as partially restrained connections that unload along the loading curves given (i.e., nonlinear elastic). In these cases, only period shift and moment redistribution affect the dynamic response. Finally, the connections are modeled using the same curves, but hysteretic behavior is included for both the thick and thin plate connections. Here, it is assumed that the connections unload along the initial slope of the response curve when loaded beyond the first segment. This complete model accounts for period shift, moment redistribution, and hysteretic damping.

For the two-story two-bay frame of Fig. 8-23(b), the connections C1 through C8 are eight-bolt extended stiffened end-plates. These connections are modeled as discussed in the previous paragraph, but using the curves shown in Fig. 8-24(b).

8.7 PROCEDURE

The frames of Fig. 8-23 were modeled using the ANSYS finite element program with a sufficient number of two node beam elements for the beams and columns and nonlinear spring elements to represent beam-to-column connections. A convergence study showed that five nodes along each story's column and six nodes along each bay's beam provide accurate results. The frames were designed using an equivalent lateral force procedure to insure that the elastic structural response would exceed the maximum strength of the connections. However, since no R value is assumed, and it has been shown in the previous sections that this simplified method can be quite conservative and/or earthquake specific, it is anticipated that the actual structural response due to a moderate earthquake may result in an elastic response of the structure. In other words, the structure is temporarily over-designed for reasons to be discussed in Section 8.9. For this section, the cyclic loading will have an amplitude large enough to cause inelastic behavior of the system.

For this purpose, the four-bolt extended (4E) frame is considered to be one of four identical frames resisting an equal portion of the earthquake induced lateral loads. The building is assumed to be 100 ft by 100 ft in plan with frame dimensions of L=30 ft and h=15 ft as identified in Fig. 8-23(a). A dead load of 75 psf is assumed for each floor. The beams are W24x68 hot-rolled sections with tributary widths of 7.5 ft for dead load calculations. The column sections are W14x120. The effective mass of the slab (one-fourth of the total slab mass) is modeled as a mass per unit length along the beam and the acceleration of gravity is not included in the analysis. Hence the dead loads actually carried by the frame are entered as applied loads prior to the dynamic analysis. For this structure, the first three natural periods are 0.857 s, 0.280 s, and 0.253 s. The corresponding mode shapes are shown in Fig. 8-25. The first and third modes involve primarily sway, whereas the second mode is primarily beam vibration.

The eight-bolt extended stiffened (8ES) frame of Fig. 8-23(b) is also considered to be one of four identical frames resisting an equal portion of the earthquake induced lateral loads. Here, the building is 180 ft by 180 ft in plan with frame dimensions of L=30 ft and h=15 ft. A dead load of 75 psf is assumed for each floor. The beams are W36x150 hot-rolled sections that are assumed to have tributary widths of 7.5 ft. The column sections are W14x257. For this two-story two-bay frame, the first three natural periods are 0.667 s, 0.225 s, and 0.191 s. The corresponding mode shapes are shown in Fig. 8-26. Again the first mode is a sway mechanism, but the second and third modes are in reverse order for this structure.

To more easily compare the response characteristics of these frames under dynamic loading, the frames are excited with the ground displacements shown in Fig. 8-27. The dynamic loading does not begin until t=5 s to allow the frames' tributary loads enough time to develop without creating a significant dynamic effect. These static loads are applied with linearly increasing magnitudes until the total tributary dead load is applied at t = 5 s. For t > 5 s, the loading is sinusoidal with an amplitude of 0.1 ft. The periods of the loadings are 120% of the frame's first natural period. The loading periods are 1.028 s and 0.800 s for the two frames, respectively.

8.8 4E FRAME RESULTS

The 4E frame of Fig. 8-23(a) is now analyzed using the finite element method for the loading of Fig. 8-27(a). Five different connection types are considered. First the connections are assumed to be linear elastic springs of infinite stiffness. This is done to determine the elastic response of the structure in the absence of any energy dissipating or force reducing mechanism. The response will be called fully restrained/elastic. Next the applied moment vs. plastic rotation curves of Fig. 8-24(a) are used. Partially restrained response characteristics for the thick and thin end-plates are assumed. In other words, it is assumed that the connections unload along the loading curves shown. These responses will be called the partially restrained/thick plate and partially restrained/thin plate responses in accord with the separate curves given in Fig. 8-24(a). Finally it will be assumed that the connections unload parallel to the slope of the initial portion of the moment-plastic rotation curves and dissipate energy. These responses will be called the hysteretic behavior/thick plate and hysteretic behavior/thin plate responses as appropriate. The loading is applied until the maximum plastic rotation, as shown in Fig. 8-24, is obtained or the earthquake ends. If this maximum rotation is achieved, it is concluded that the connection fails in the ultimate limit state determined experimentally.

Figure 8-28 shows the fully restrained/elastic moments at each of the four connections identified in Fig. 8-23(a) vs. time. The abscissa begins at time t=5 s. A maximum moment of about 1350 k-ft is seen. This exceeds the maximum moment capacities of 917 k-ft and 875 k-ft as shown in Fig. 8-24(a) for the thick and thin plates, respectively. The first story connections, C1 and C2, carry significantly higher moments than the second story connections. This is expected for linear elastic analysis. Also note that as expected, two connections carry positive moment while the other two connections take negative moment, and vice-versa, throughout the loading.

Figure 8-29 shows the partially restrained/thick plate response of the connections. The time scale is greatly reduced due to the failure of a bottom story connection at approximately t = 9.50 s. During the initial portion of the loading, little moment is reduced compared to the elastic response. This is because very little period shift occurs for beam hinging during the initial loading. At about t = 7 s, moment redistribution begins to occur (e.g., the C_4 curve approaches the C_2 curve). This increases the moments taken by the top story so that the entire structure becomes more effective in resisting the applied loading. At about t = 8 s, the curves begin to appear "cut off". This is a result of the stiffness degradation shown in Fig. 8-24(a) as the thick plate response curve flattens out.

Figure 8-30 shows the partially restrained/thin plate response of the connections. The time scale is reduced to only about 7.5 s since failure occurs during the first 3 s of sinusoidal loading. It is clear that complete moment redistribution does not occur prior to connection failure. Also, note that the moments at the connections are smaller than the fully restrained/elastic moments during the initial loading due to a larger period shift. However, for this connection, with a relatively steep initial slope of the moment vs. plastic rotation curve, the decrease in moment magnitudes are not significant.

Figure 8-31 plots the hysteretic behavior/thick plate response of the connections. The complete time scale is utilized, since failure of a connection does not occur. Note that very little moment redistribution occurs for this configuration. The energy dissipated by histeretic damping is significant, and only about 0.009 of the available 0.033 radians of plastic rotation capacity is utilized. This is important, since the most ductile segment of the thick plate response curve is not reached. In other words, a significant increase in loading can be resisted by this structure without failure. Figure 8-32 shows the hysteretic behavior/thin plate response of the connections. Although the system performs well, more moment redistribution is required since less energy is dissipated for the thin plate response. Here, 0.013 of the available 0.021 radians of plastic rotation are utilized. Although much of the available plastic rotation capacity of the connections remains, it is clear that not nearly as much additional load can be sustained by the structure as compared to its beam hinging counterpart. Also note that the required strength of the connections is not reduced much throughout the loading. This is significant when compared to the elastic response of Fig. 8-28.

It is clear that for the 4E frame, both the thick and thin end-plate hysteretic models can resist the given excitation safely. However, as PR connections, both the thick and thin end-plates perform poorly, and it is shown that period shift has relatively little effect on the overall seismic performance of these frames. The results show that hysteretic damping is the primary force reducing mechanism of moment resisting frames and that the shape of the hysteretic curves plays a large role in determining the overall response of the structure. Although both the thin and thick end-plates with hysteretic behavior resist the applied loading safely, it is clear that more inelastic rotation is required of the thin end-plate due to its smaller enclosed hysteretic area. This will become more apparent for the 8ES frame of the next section.

8.9 8ES FRAME RESULTS

The 8ES frame of Fig. 8-23(b) is now analyzed using the finite element method for the loading of Fig. 8-27(b). Figure 8-33 shows the fully restrained/elastic moment at each of the eight connections identified in Fig. 8-23(b) vs. time. Figures 8-33(a) and (b) show the first and second story connection responses, respectively. The abscissa begins at time t = 5 s, and a maximum moment of about 4600 k-ft is seen on the bottom story. This exceeds the maximum moment capacities of 3333 k-ft and 2792 k-ft as shown in Fig. 8-24(b) for the thick and thin plates, respectively. Again, the first story connections, C1 through C4, carry significantly higher moments than do the second story connections C5 through C8.

Figure 8-34 shows the partially restrained/thick plate response of the connections for the 8ES frame. The time scale is greatly reduced due to the failure of a bottom story connection at approximately t=7.65 s. During the initial portion of the loading, little moment is reduced compared to the elastic response. This is because very little period shift occurs for beam hinging during the initial loading. Moment redistribution seems to begin for the bottom story almost immediately after the initiation of cyclic loading. At about t = 6.5 s, the peaks of the curves become flatter. This is a result of the stiffness degradation shown in Fig. 8-24(b) as the thick plate response curve flattens. However, the flattening is not as prominent as for the previous case shown in Fig. 8-29. Figure 8-35 shows the partially restrained/thin plate response of the connections. The time scale is reduced to only about 6.9 s due to the almost immediate connection failure. Note that the moments are significantly smaller than the fully restrained/elastic moments during the initial loading due to a considerable period shift. However, connection failure still occurs and the overall performance is poor.

Figure 8-36 plots the hysteretic behavior/thick plate response of the 8ES connections. The complete time scale is utilized, since failure of a connection does not occur. As in the 4E case, very little moment redistribution occurs for this configuration. The energy dissipated by hysteretic damping is significant, and about 0.017 of the available 0.025 radians of plastic rotation capacity are utilized. Here, the most ductile segment of the response curve is partially used and the connection performs very well. Figure 8-37 shows the hysteretic behavior/thin plate response of the connections. For this connection and the 8ES frame, the system performs poorly since very little energy is dissipated by the hysteretic behavior. The hysteretic thin plate connection fails at just about the same time as its partially restrained counterpart.

From these cyclic loading results, several response characteristics can be concluded. First, for very low seismicity, where the connections respond at only about 2/3 of their capacity, the thin plate connections actually outperform their thick plate counterparts. This is not significant though, since period shift seems to only account for a maximum of about 30% reduction in moments for the given excitation. Under the design earthquake, an elastic system is expected to see seismic forces several times larger than a properly detailed inelastic system. For both the 4E and 8ES frames, it is clear that significant hysteretic behavior is required to successfully reduce elastic forces caused by an earthquake. Hysteretic curves developed for connection yielding provide much less energy dissipation than do curves developed for beam hinging. Therefore, frames designed so that beam hinging is the controlling mechanism can be expected to significantly outperform frames detailed for connection yielding under design earthquake loading.

8.10 4E FRAME WITH EARTHQUAKE EXCITATION

To test the conclusions drawn from the previous section's results, the 4E frame is now excited by horizontal ground displacements of the 1979 Imperial Valley earthquake as shown in Fig. 8-38. Again, the actual seismic excitation begins at time t = 5 s. The earthquake has a duration of approximately 38 s. Again, it should be mentioned that the frame was purposely over-designed to consider its response both in the elastic and inelastic range. The seismic loading of Fig. 8-38 was magnified, as is typical, for nonlinear time history analysis to cause an inelastic response of the structure.

Figure 8-39 shows the response of connection C1 vs. time for the fully restrained/elastic case. A maximum moment of about 675 k-ft is obtained which is less than the maximum strength of the 4E connections. Since most of the significant moments are between t = 15 s and t = 25 s, this range is shown in Fig. 8-40 for all four connections. As for the case of cyclic loading, the bottom story always carry the significant portion of the loading, and opposing connections are always undergoing moments in opposite directions.

Figure 8-41 shows the hysteretic behavior/thick plate response of the connections under this seismic excitation. The maximum moment of 520 k-ft is still in the first segment of the response curve shown in Fig. 8-24(a) for the thick plate. Therefore, the reduction in maximum moment is due solely to period shift (i.e., no hysteretic behavior takes place). Since the initial slope of the thin plate curve is less than the thick plate slope, it is expected that the thin plate hysteretic moments will be even smaller. This is a result of a larger period shift. Figure 8-42 plots the hysteretic/thin plate response of the connections for the t=15 s to t=25 s time scale, and the maximum moment is reduced to 400 k-ft. Period shift causes the thin plate connection to respond more favorably than the thick plate connection. However, this will not be the case for significant ground shaking where an inelastic response of the structure is required.

To cause the structure to respond inelastically to the given excitation while still allowing for direct comparison of results, the 4E frame is now excited by the same earthquake excitation but magnified by a factor of four as shown in Fig. 8-43. Figure 8-44 shows that the fully-restrained/elastic response of the structure is exactly four times that obtained in Fig. 8-40, as expected for linear elastic analysis. A maximum moment of 2700 k-ft is obtained which is well beyond the capacity of the connections. Figure 8-45 shows the hysteretic behavior/thick plate response of the connections. Here, 0.013 radians of a total plastic rotation capacity of 0.033 radians are utilized. Significant ductility still exists. However, for the hysteretic behavior/thin plate case shown in Fig. 8-46, 0.016 radians of a total plastic rotation capacity of 0.021 radians are utilized. Hence, significant ductility does not remain for this connection.

8.11 CONCLUSIONS ON THE HYSTERETIC BEHAVIOR OF MOMENT END-PLATE CONNECTIONS

Frames can be designed with strong or weak moment end-plate connections (thick or thin end-plates) so that elastic forces caused by the design earthquake can be reduced by the structure's nonlinear response. However, the shape of the connection's hysteretic curves (applied moment vs. plastic rotation) greatly impacts the response characteristics and overall success of the structure. For low (not design) earthquake forces, a frame composed of weak connections can be expected to resist seismic forces just as well as frames designed with connections stronger than the adjoining beam. In fact, lower moments will occur in the weak connection frame due to a more significant period shift accompanying the more flexible structure. This fact is not significant, however, should the actual design earthquake occur.

Under design (or larger) earthquake loading, the fully restrained/elastic structure should be expected to withstand forces up to or larger than eight times those that the properly detailed inelastic system will actually resist. In such cases, hysteretic damping becomes the primary force reducing mechanism, and if not enough ductility is available, the structure will fail. Although it is clear that hysteretic curves developed from connection yielding are "pinched" and enclose a much smaller area than curves developed for beam hinging, current codes do not regulate the source of inelastic rotation in all cases. For example, ordinary moment frames are allowed to be designed for connection yielding with the expectation that the structure's seismic performance will be comparable to that system designed for beam hinging of the same plastic rotation capacity. These results show that under the design earthquake, this is simply not the case.

TABLE 8-1. Limiting short-period spectral response accelerations for different site classifications.

Site Class	S_s (g)
A	0.9375
B	0.7500
C	0.6603
D	0.5522
E	0.3382

TABLE 8-2. Limiting one-second period spectral response accelerations for different site classifications.

Site Class	S_1 (g)
A	0.3750
B	0.3000
C	0.1859
D	0.1320
E	0.0587

TABLE 8-3. Maximum moment ratio for wind loading vs. elastic earthquake loading (fixed base).

	L = 60 ft.	L = 80 ft.	L = 100 ft.	L = 120 ft.
h = 10 ft.	1.82	1.76	1.73	1.87
h = 15 ft.	1.89	1.79	1.70	1.79
h = 20 ft.	1.62	1.57	1.42	1.48
h = 24 ft.	1.64	1.59	1.44	1.50

TABLE 8-4. Maximum moment ratio for wind loading vs. elastic earthquake loading (pinned base).

	L = 60 ft.	L = 80 ft.	L = 100 ft.	L = 120 ft.
h = 10 ft.	1.39	1.22	1.17	1.20
h = 15 ft.	1.53	1.33	1.26	1.26
h = 20 ft.	1.26	1.12	1.08	1.08
h = 24 ft.	1.32	1.18	1.15	1.14

TABLE 8-5. Transition values for four-bolt extended thick and thin end-plates.

Thick Plate Plastic Rotation (rad), Moment (k-ft)	Thin Plate Plastic Rotation (rad), Moment (k-ft)
0.0005, 650	0.001, 542
0.0025, 767	0.012, 833
0.01, 913	0.021, 875
0.033, 917	

TABLE 8-6. Transition values for eight-bolt extended stiffened thick and thin end-plates.

Thick Plate Plastic Rotation (rad), Moment (k-ft)	Thin Plate Plastic Rotation (rad), Moment (k-ft)
0.001, 2500	0.003, 1667
0.006, 3000	0.01, 2583
0.025, 3333	0.019, 2792