Hybrid Moment-Resisting Steel Frames

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ABSTRACT

A new type of moment-resisting steel frame, called a hybrid moment-resisting frame, is described. Unlike a typical moment frame, where all member sizes and connection details fit a specific set of rules (e.g., for a special moment frame), the hybrid frame contains members and connections with a variety of detailing rules, including those typically associated with ordinary, intermediate and special moment frames. Elements that have special detailing are designed to yield at force levels well below the design basis earthquake and thereby provide some inelastic energy dissipation that helps to control dynamic amplification. Elements with ordinary detailing are designed to remain elastic during the design basis earthquake and to provide enough positive stiffness to counteract P-delta effects. The resulting system can be designed to perform better than the traditional special moment frame and to be more economical than the special moment frame because a limited number of elements and connections have special detailing. The behavior of the system is demonstrated through incremental nonlinear dynamic response history analysis.

Keywords: seismic design, moment-resisting frames, structural steel.

The current specifications for seismic resistant design (AISC, 2005a; AISC, 2005b; ASCE, 2010) require that special detailing be used in virtually all moment-resisting frame systems that are to be constructed in high seismic hazard regions. This detailing requires the use of designated flexural yielding regions with limited width-to-thickness ratios, highly ductile prequalified connection types, limited panel zone yielding and adherence to a strong-column weak-beam design philosophy. The structure must be designed such that first significant yield occurs at lateral force levels that are at or above the design basis earthquake (DBE) forces. The sequencing of plastic hinging is usually not explicitly designed, and hence, there is no guarantee that the slope of the structure's force-deformation response (pushover curve), including P-delta effects, is continuously positive up to the maximum expected drift. This a critical design issue, because it is much more likely that dynamic instability will occur when the post-yield stiffness is negative (Gupta and Krawinkler, 2000). This fact led to a significant revision in the 2003 NEHRP provisions (FEMA, 2004), where it is required that the pushover curve be continuously positive up to 1.5 times the target displacement if the stability ratio, based on initial elastic stiffness and on design level gravity loads, exceeds 0.10. This requirement was proposed for inclusion in ASCE 7-10, but was not adopted. Another consequence of not explicitly designing the hinging sequence is that the expected overstrength, which is implicitly included in the system's response modification coefficient, R, is not guaranteed. Indeed, there is nothing in the current design provisions that prevents a designer from developing a system for which a nonlinear static pushover analysis indicates that all of the hinges form nearly simultaneously.

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In a hybrid moment frame (HMF), the hinging sequence is explicitly designed to ensure a continuously positive post-yield pushover response. The HMF shares many of the features of the special moment frame (SMF), with the following exceptions:

1. The yielding sequence is set such that the first plastic hinges form at load levels well below the design basis earthquake, and the last hinges form at load levels consistent with the maximum considered earthquake. The inelastic energy dissipation provided through early yielding is expected to improve the performance of the structure subjected to earthquakes of intensity less than the design basis earthquake. The near-elastic response of the late-forming hinges is intended to guarantee a positive pushover response.

2. The detailing for the lateral load resisting components and their connections depends on the level of inelastic rotation that is expected in the various plastic hinges. The hinges that form first have the highest ductility demand and are detailed according to the rules for special moment frames. It is noted that these hinges may have ductility demands that exceed those expected from traditional SMF designs. The hinges that form last have the lowest ductility demand and are detailed

according to the rules for intermediate or ordinary moment frames.

Hewitt et al. (2009) compared the cost of an ordinary moment frame (OMF) with an SMF supposing that material and labor represent 30 and 70% of the total cost of the frame, respectively; that 50% of the labor cost is due to special connections; and that there are additional special inspection requirements for the connections. As a result of this scenario, the cost premium for an SMF over an OMF is about 22%. Even though this is a very rough estimate (because the foundation costs were ignored and moment frames were assumed as strength controlled), labor cost associated with fabrication and inspection of the connections is significant. Because HMF discussed in this paper limits the number of special connections and elements, it is expected to be more economical than a SMF.

The hybrid frame concept may be used for any structural system, such as concentrically braced frames or buckling restrained braced frames, as well as for moment-resisting frames. The concept of hybrid buckling restrained frames is particularly attractive because of the ability to tightly control the inelastic behavior of the yielding elements.

The advantages of hybrid frames will be demonstrated through two examples. The first example is of a simple hybrid braced frame and is used only to demonstrate the concepts and to introduce some of the features used in the analysis. The second example is of a nine-story steel moment resisting frame. Frames of this type are the main focus of this paper.

DEMONSTRATION OF CONCEPTS: A HYBRID BRACED FRAME

In this demonstration, a simple one-story braced frame is analyzed. This fictitious frame, shown in Figure 1, is intended to have the dynamic characteristics of a 15-story building, with a first mode period of vibration of 2.0 s. Two different versions of the frame are presented. The first frame, called the *normal frame,* has six identical diagonal braces; each with an axial strength of 141 kips. The second frame, called the *hybrid frame,* has bracing bars of the following strengths: bar 1 = 47 kips, bar 2 = 94 kips, bars 3 and 4 = 141 kips, bar 5 = 188 kips and bar 6 = 235 kips. The lateral strength of both structures, exclusive of P-delta

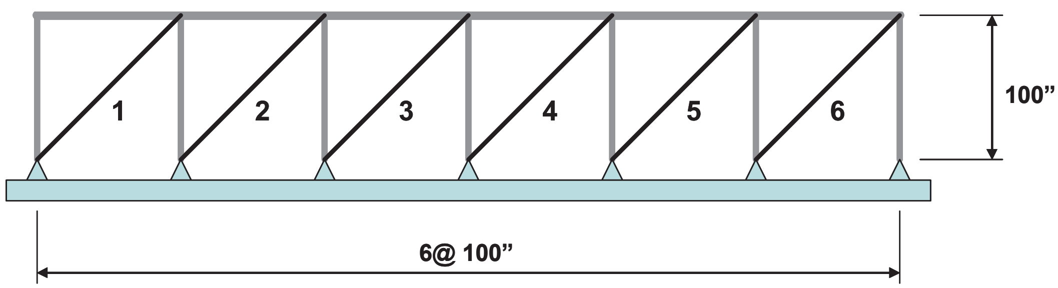

Fig. 1. A simple braced frame.

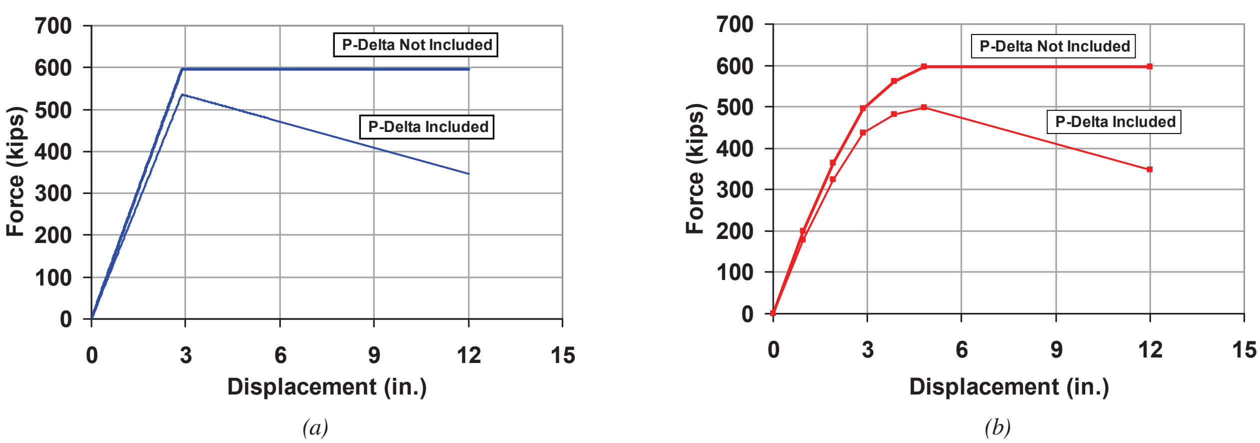

Fig. 2. Nonlinear static pushover curves for braced frame structure: (a) normal frame; (b) hybrid frame.

effects, is 600 kips. The axial stiffness of each of the bars, whether in the normal or hybrid frame, is 68.9 kips/in. The initial lateral stiffness of each frame is 207 kips/in. The force-deformation behavior of the bars was assumed to be elastic-plastic, without strain-hardening.

Nonlinear static pushover plots of the normal and hybrid frames are shown in Figures 2a and 2b, respectively. Response curves with and without P-delta effects are shown in the figures. Where included, the P-delta analysis emulates a structure with an average story stability ratio of 0.10.

To investigate the dynamic behavior, the normal and hybrid structures, with and without P-delta effects included, were subjected to the 1940 Imperial Valley ground motion, with a peak ground acceleration of 0.35 g. For each case, the structure was repeatedly subjected to this ground motion, with each analysis using an incrementally larger ground-motion multiplier. The multipliers ranged from 0.2 to 2.0, in increments of 0.2. For this example, it was assumed that a multiplier of 1.0 corresponds to the design basis earthquake (DBE) and the factor of 1.5 corresponds to the maximum considered earthquake (MCE).

Analysis was run using NONLIN-Pro (Charney and Barngrover, 2006), which uses the Drain 2D-X (Prakash et al., 1993) analysis engine. All analyses were run with an inherent damping ratio of approximately 0.02. One set of analyses was run without P-delta effects and the other with P-delta effects. When P-delta effects were considered, both the normal and hybrid structures were dynamically unstable when the ground motion multiplier exceeded 1.5.

Plots of the results for the models without P-delta effects are shown in Figures 3a through 3d. Figure 3a plots the ground-motion multiplier on the vertical axis and the computed roof displacement on the horizontal axis. The displacements appear to be similar between the two systems, except that it is noted that the hybrid frame displacements are about 12 to 15% less than the normal frame displacements for the first two increments of loading. For all ground-motion levels less than or equal to the MCE, the residual inelastic

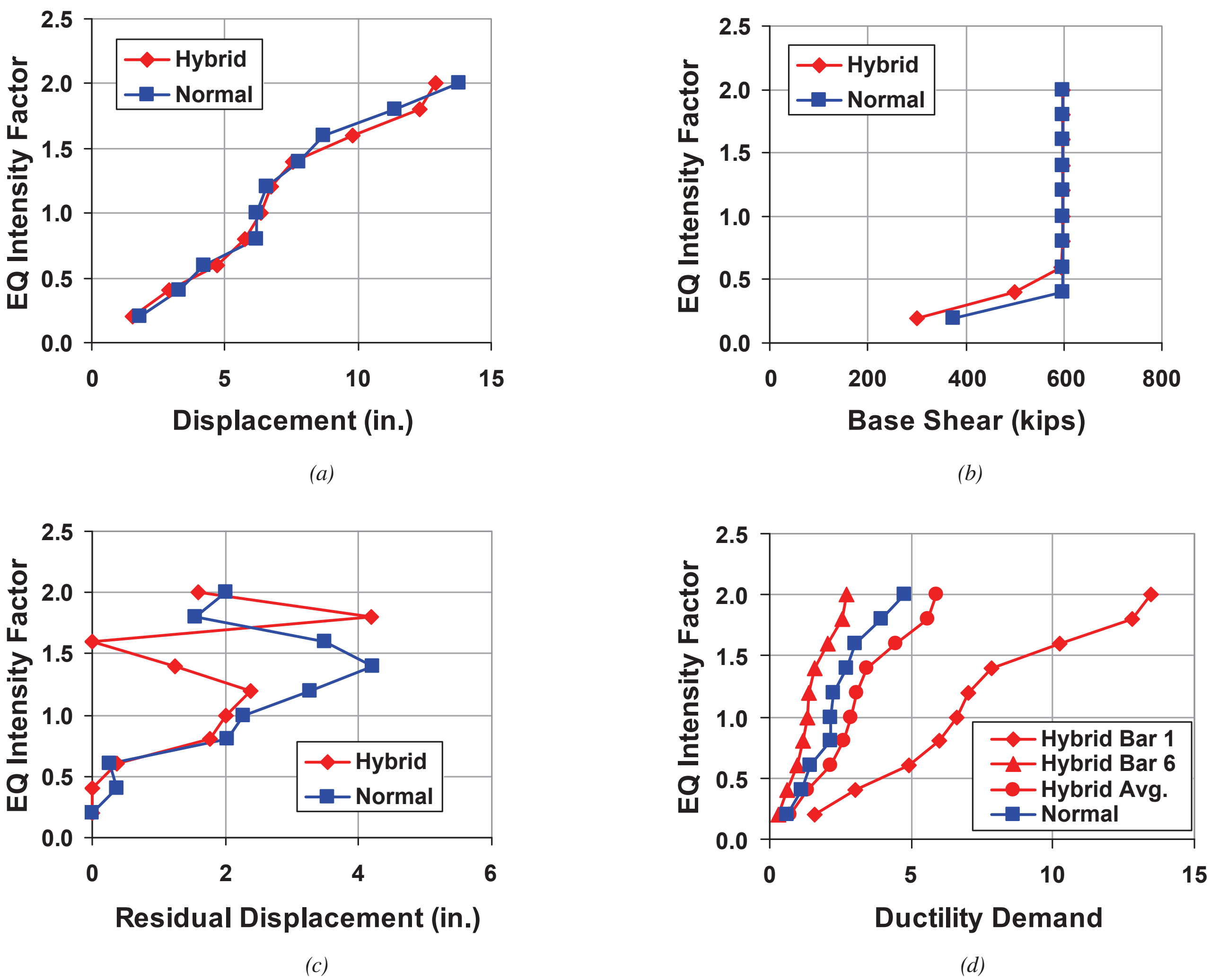

Fig. 3. Results of frame analysis without P-delta analysis.

deformations, presented in Figure 3c, are significantly lower for the hybrid frame when compared to the normal frame. (Residual deformations are the permanent lateral deformations that remain in the structure after ground shaking has ceased.) At the ground motion intensity level of 1.8, however, the residual deformations in the hybrid frame exceed those in the normal frame. The base shears for the hybrid frame, shown in Figure 3b, are also lower than those for the normal frame for the first two increments of ground motion intensity.

Displacement ductility demands for bar 1, bar 6 and for the average of all bars are presented in Figure 3d. For the hybrid frame, bar 1 is the weaker bar, and as expected, the ductility demand is the highest. At the DBE level (multiplier 1.0), the ductility demand for bar 1 is 6.61. At the same intensity, the ductility demand for bar 6 is only 1.32, and the average ductility demand for all hybrid bars is 2.88. For the normal frame, the ductility demand for all bars is the same at each intensity level and is 2.15 at the multiplier of 1.0.

It appears from the results that the hybrid frame is performing as expected. Displacements at low-level ground motions are reduced due to the early yielding and associated hysteretic behavior of bars 1 and 2. Delayed yielding of the stronger bars provides a component of elastic stiffness that controls residual deformations.

When P-delta effects are included, the performance of the hybrid frame is further improved when compared to the normal frame. This is illustrated in Figures 4a through 4d, where it may be seen that the total displacements, Figure 4a, are significantly less in the hybrid frame at all ground motion levels up to the DBE. This improved performance is due to the significant reduction in residual deformations, shown in Figure 4c. As mentioned earlier, both the hybrid and normal frames displayed dynamic instability when the ground motion multiplier exceeded 1.5. This is due to the negative stiffness of the pushover curves (see Figure 2) at larger displacements.

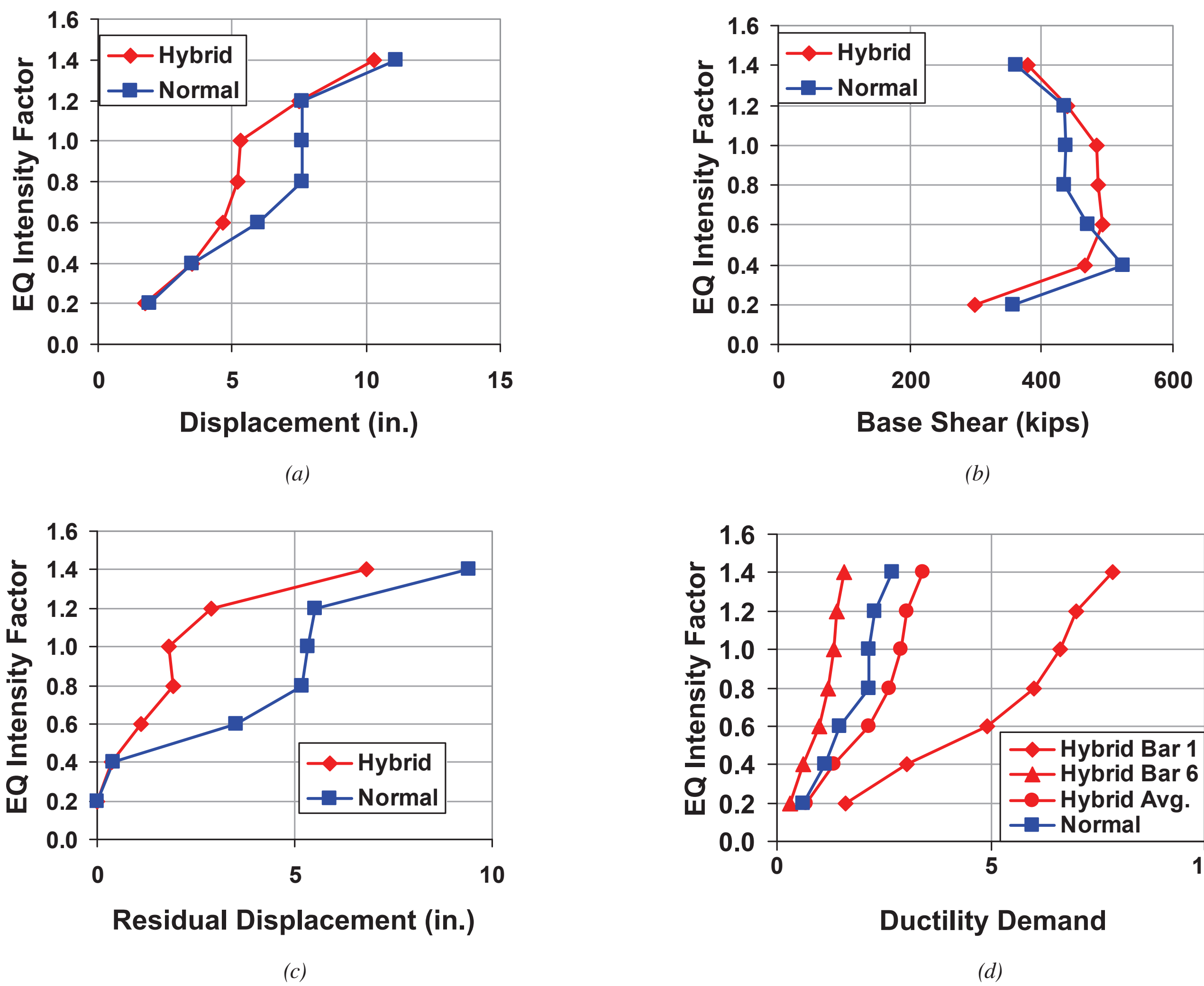

Fig. 4. Results of frame analysis with P-delta analysis.

Table 1.
ASCE 7-10 Design Parameters for Hybrid Frame

Design Parameter	Value
0.2-s spectral acceleration, S_S	1.25 g
1.0-s spectral acceleration, S_1	0.5 g
Site class	D
0.2-s design acceleration, S_{DS}	0.83 g
1.0-s design acceleration, S_{D1}	0.5 g
Seismic use group	II
Importance factor	1.0
Seismic design category	D
Effective seismic weight, W	10,500 kips

It is interesting to note from Figure 4b that for ground motion multipliers between 0.6 through 1.0, the base shears for the hybrid frame are somewhat greater than for the normal frame. This is not a disadvantage for the hybrid frame, because the lower base shears for the normal frame are associated with P-delta related strength loss.

ANALYSIS OF A HYBRID MOMENT-RESISTING FRAME

Models and Design Procedures

The hybrid moment frame concept is demonstrated by the analysis of a five-bay, nine-story frame building, located near Seattle, Washington. The geometry of this building is identical to that studied in the SAC Steel Project (FEMA, 2000). The ASCE 7 design parameters used for the design are summarized in Table 1. Four different frame configurations were used in this study. The first configuration, hybrid-0, is closest to the normal frame design, because the same girder sizes were used for each bay in a given story. The other three configurations are the real hybrid designs, referred to hybrid-1, hybrid-2, and hybrid-3 frames, because the girder sizes for these frames are different in different bays. The hybrid-0 frame is the least hybrid (closest in design to the traditional frame), and hybrid-3 frame is the most hybrid (furthest in design concept from the traditional frame). Figure 5 shows the member sizes used for the different frames. Member sizes for the girders are shown above each girder, with the hybrid-0 frame at the bottom and the hybrid-3 frame at the top. The column sizes were the same for all of the designs.

The two exterior girders of the hybrid frames (bays 1 and 5) were designed as special moment frames (SMF), the two interior girders (bays 2 and 4) were designed as intermediate moment frames (IMF) and the middle girder (bay 3) was designed as an ordinary moment frame (OMF). For this reason, a new response reduction factor, R, and deflection amplification factor, C_d, were assumed as 6 and 5, respectively, for hybrid frame design. Note that these values are close to the weighted average R and C_d values of the SMF (two bays), IMF (two bays) and OMF (one bay). After the sections of the hybrid-0 were found by using $R = 6$ and $C_d = 5$, the plastic capacities were changed throughout the story for the other real hybrid frames. The plastic capacities of the exterior girders were decreased by 25, 37.5 and 50% for the hybrid-1, hybrid-2 and hybrid-3 frames, respectively. Because the main idea of the hybrid frame concept is to keep the total strength of the story the same, the plastic capacity of the middle girder was increased by 50, 75 and 100% for the hybrid-1, hybrid-2 and hybrid-3 frames, respectively. Bay-2 and bay-4 girder capacities were kept the same for all the frames. In summary, as the frame identification number gets bigger, the frames become more hybrid, with a greater variation in beam sizes at each story.

The column sections were kept the same for all the designs, but the panel zone doubler plate thicknesses were changed as necessary to meet AISC panel zone rules. Reduced beam sections were used for all the girders except for the girder in the middle bay, which was designed according to the rules for an OMF. The strong column–weak beam requirement was satisfied at the joints of the columns on column lines 1, 2, 5 and 6. Material nonlinearity was considered through assigning a bilinear moment-rotation relationship to beams and columns. Two percent strain hardening was used in the development of moment-rotation relationships. See Atlayan (2008) for a much more detailed description of the step-by-step procedures of beam, column and panel zone design. Panel zones were explicitly represented by use of Krawinkler's model (Charney and Marshall, 2006). P-delta effects were included in all analysis, using a special linear "ghost frame," which captures the entire gravity load tributary to the leaning columns. The inherent damping was determined by setting the critical damping ratio to 2% at the natural period of the structure and at a period of 0.2 s, as it was done in the SAC Report (FEMA, 2000).

Two types of analysis were performed for each frame: nonlinear static pushover analysis (NSP) and incremental dynamic analysis (IDA). For both analyses, gravity loads were followed by the static pushover lateral load pattern or dynamic earthquake load case. All structural analyses were conducted using Perform-3D (CSI, 2006), using a planar frame that is parallel to the design ground motion.

Nonlinear Static Pushover Analysis

Nonlinear static pushover curves for the four different hybrid frames are illustrated in Figure 6. Note that the point of the first significant yield and the point at which the post-yield curve becomes negative are shown on the figure. As expected, the hybrid-3 frame starts yielding first, and the hybrid-0 frame yields last. The more reduction in the plastic capacity of the exterior bays, the earlier the structure starts yielding. In addition, the negative post-yield stiffness of the pushover curves is reached later, as the frames become more hybrid. It is foreseen that the early yielding of the pushover curve will provide hysteretic energy dissipation to the frame, which will result in a better dynamic behavior under less severe ground motions. Furthermore, negative post-yield stiffness has a significant effect on structures and is a significant contributor to dynamic instability. Although the frames were pushed until they reached 4% roof drift, it is predicted that the hybrid-0 (normal frame) will reach a steeper negative stiffness than the real hybrid frames if the frames are pushed more than 4% reference drift. This behavior may be observed in the last portion of Figure 6.

Having control of plastic hinge sequence is a key concept in hybrid frame design. For this design, the plastic hinges at the exterior bays formed first, and the ones at the middle bay formed last. As a result of pushover analyses, the hinges at the right ends of the exterior bays formed first. This is because the gravity loads were applied initially, and the lateral loads were acting toward the east direction, causing the moments with the same signs to accumulate at the right ends. In other words, if a particular girder had such a preload (due to gravity loads) that the positive moment hinges were near

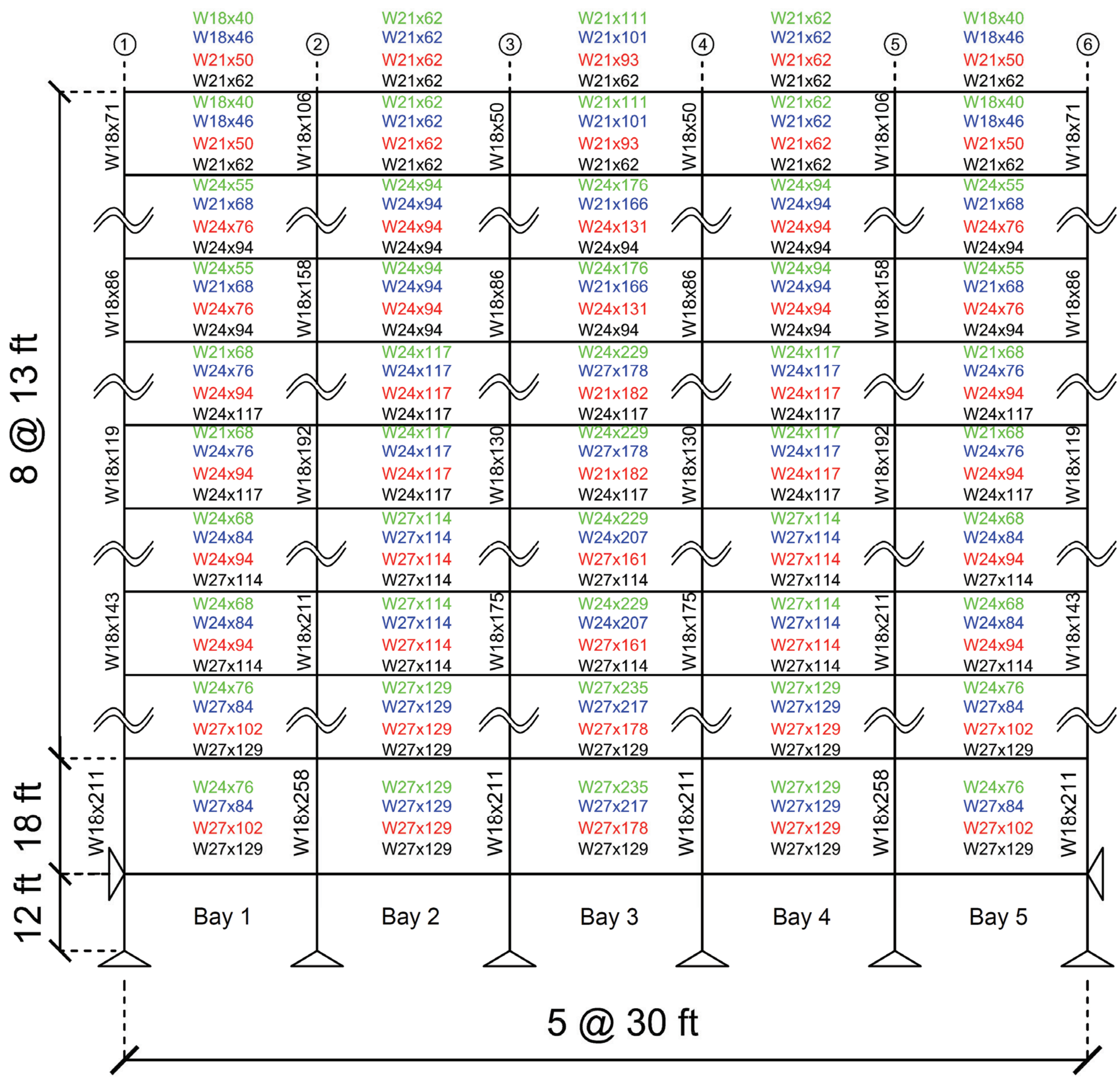

Fig. 5. Member sizes used for hybrid-0 to hybrid-3 frame (bottom to top).

Table 2. Ground-Motion Records Used in Analysis						
EQ No.	SAC Name	EQ Name	Time Step (s)	Newmark Integration Time Step	Scale Factor	Scaled PGA
EQ00	SE 21	Mendocino, 1992	0.020	0.005	0.403	0.311
EQ01	SE 23	Erzincan, 1992	0.005	0.005	0.657	0.313
EQ02	SE 25	Olympia, 1949	0.020	0.005	2.111	0.435
EQ03	SE 27	Seattle, 1965	0.020	0.001	6.214	1.087
EQ04	SE 29	Valparaiso 1, 1985	0.025	0.0025	2.088	1.178
EQ05	SE 31	Valparaiso 2, 1985	0.025	0.001	3.934	1.262
EQ06	SE 33	Deep Interplate	0.020	0.001	4.281	0.888
EQ07	SE 36	Miyagi-Oki, 1978	0.020	0.001	1.189	0.523
EQ08	SE 37	Shallow Interplate 1	0.020	0.005	1.054	0.632
EQ09	SE 40	Shallow Interplate 2	0.020	0.001	1.747	0.879

yield, it would take only a small incremental lateral load to cause yielding in this girder (whereas stronger girders would not have yielded until more lateral load was applied). This is exactly what is happening in the hybrid frames (i.e., the gravity preload influenced the sequence of yielding).

Incremental Dynamic Analysis

Incremental dynamic analysis, sometimes called dynamic pushover analysis, consists of a sequence of nonlinear response history analyses of the structure, with each analysis in the sequence subjecting the structure to the same basic ground motion, but at a higher intensity than the previous analysis in the sequence (Vamvatsikos, 2002). In this study, IDA analysis was conducted for the structure subjected to 10 different earthquake records at intensities of 0.2 to 2.0 times the ground motion scaled to match the design basis earthquake. The ground motions were scaled to match the ASCE 7 design basis spectrum at the structure's fundamental period of vibration. This scaling procedure is recommended for IDA analysis by Shome et al. (1998). The ground

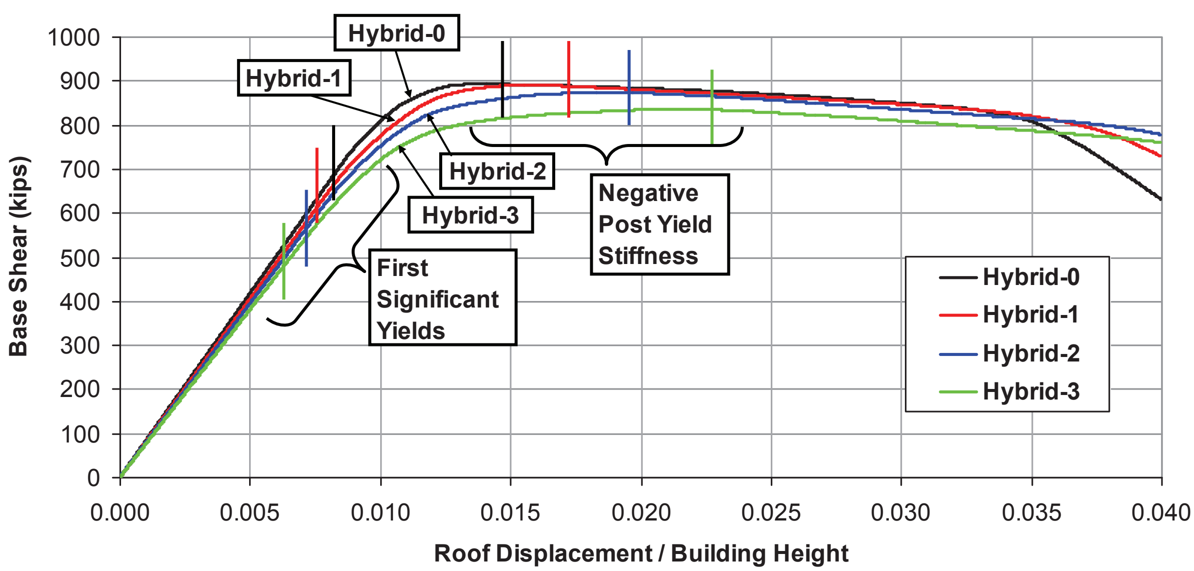

Fig. 6. Static pushover curves for hybrid frames.

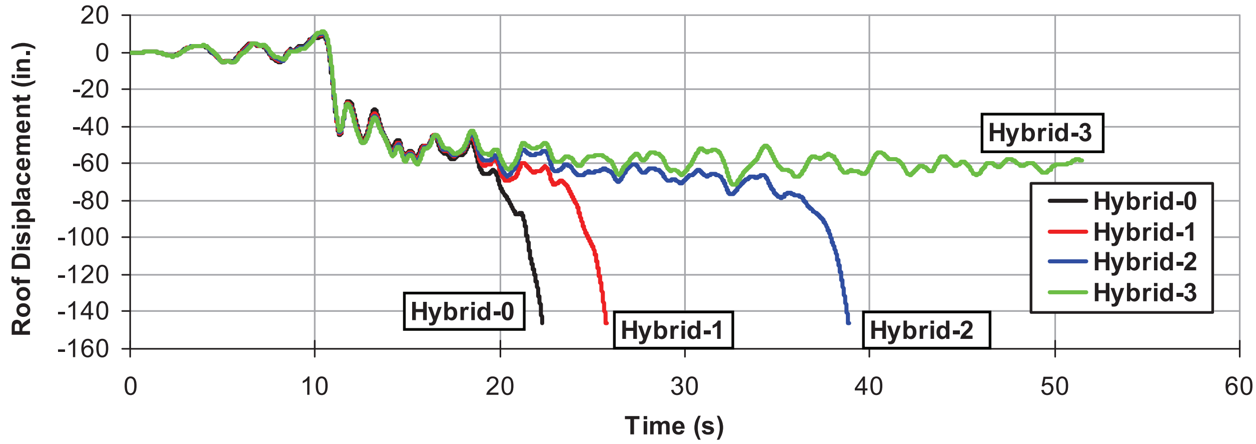

Fig. 7. Roof displacement response history of hybrid frames subject to EQ07 with scale of 2.0 times the anchored design spectrum scale.

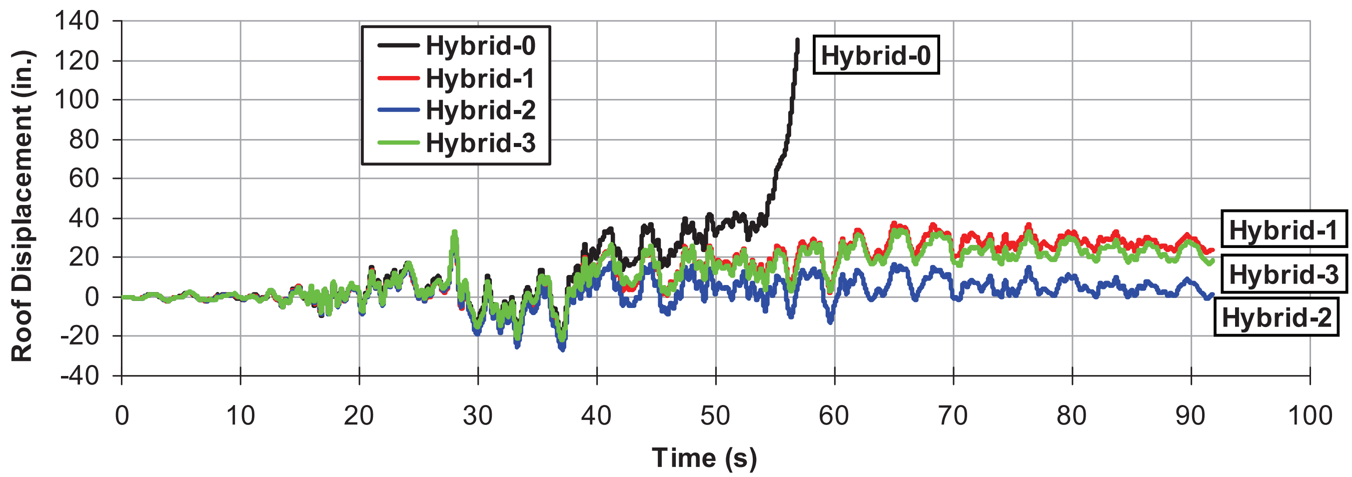

Fig. 8. Roof displacement response history of hybrid frames subject to EQ05 with scale of 1.6 times the anchored design spectrum scale.

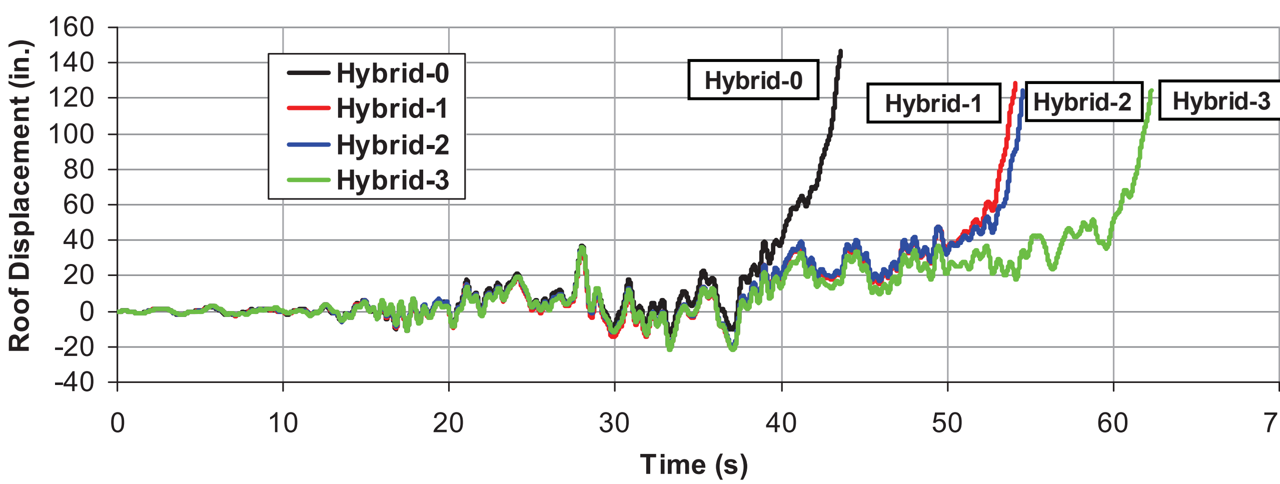

Fig. 9. Roof displacement response history of hybrid frames subject to EQ05 with scale of 1.8 times the anchored design spectrum scale.

motions used in the analysis are summarized in Table 2. It is noted that these ground motions, developed by Somerville (1996), are the same as those used in the original SAC research (FEMA, 2000).

Figures 7 and 8 illustrate the roof displacement response histories of hybrid frames subjected to EQ07 and EQ05 with scale factors of 2.0 and 1.6 times the anchored design spectrum scaling, respectively. These two earthquakes are the most severe ones out of all the earthquakes used in this study. As can be seen from Figure 7, hybrid-0, hybrid-1 and hybrid-2 frames reach dynamic instability, whereas the hybrid-3 frame, the most hybrid frame, resists the collapse with 60 in. residual displacement at the roof level. Similarly, all the real hybrid frames (except the normal frame, hybrid-0) resist the collapse under 1.6 times DBE scaled EQ05 motion (see Figure 8). Note that the hybrid-2 frame results in less residual displacement in Figure 8. When the scale factor of the same ground motion is increased from 1.6 to 1.8 (see Figure 9), all of the hybrid frames collapse; however, as the frames become more hybrid, they resist the collapse more—i.e., collapses occur at a later time.

Figure 10 shows the roof displacement response histories when the frames are subjected to EQ09 with IDA scaling of 2.0. Although none of the frames collapse, the residual

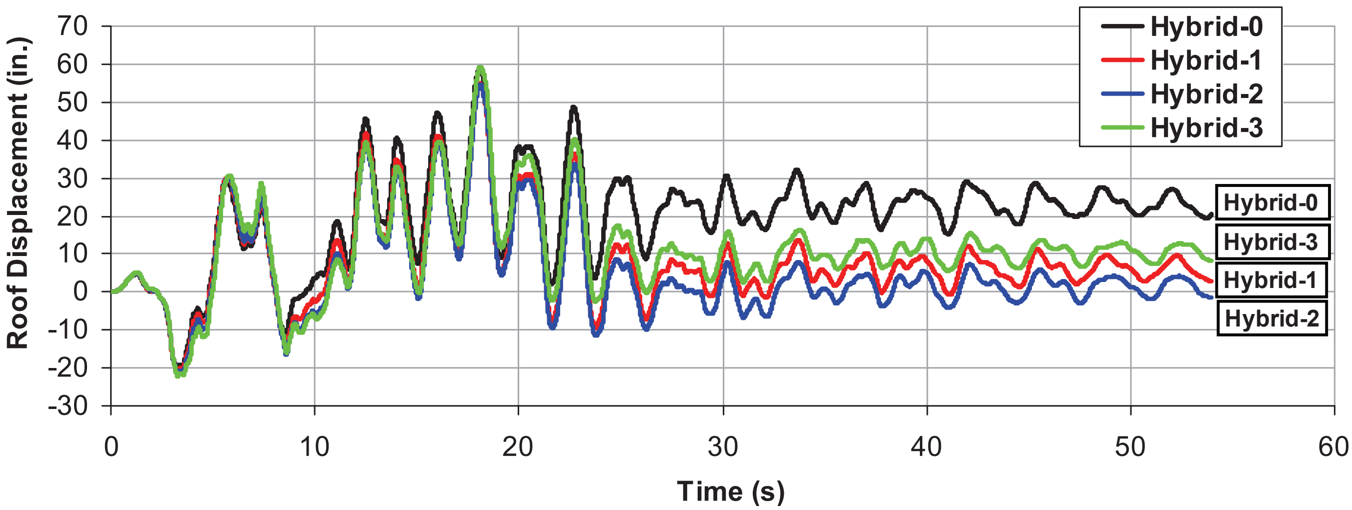

Fig. 10. Roof displacement response history of hybrid frames subject to Shallow Interpolate 2 (EQ09) with scale of 2.0 times the anchored design spectrum scale.

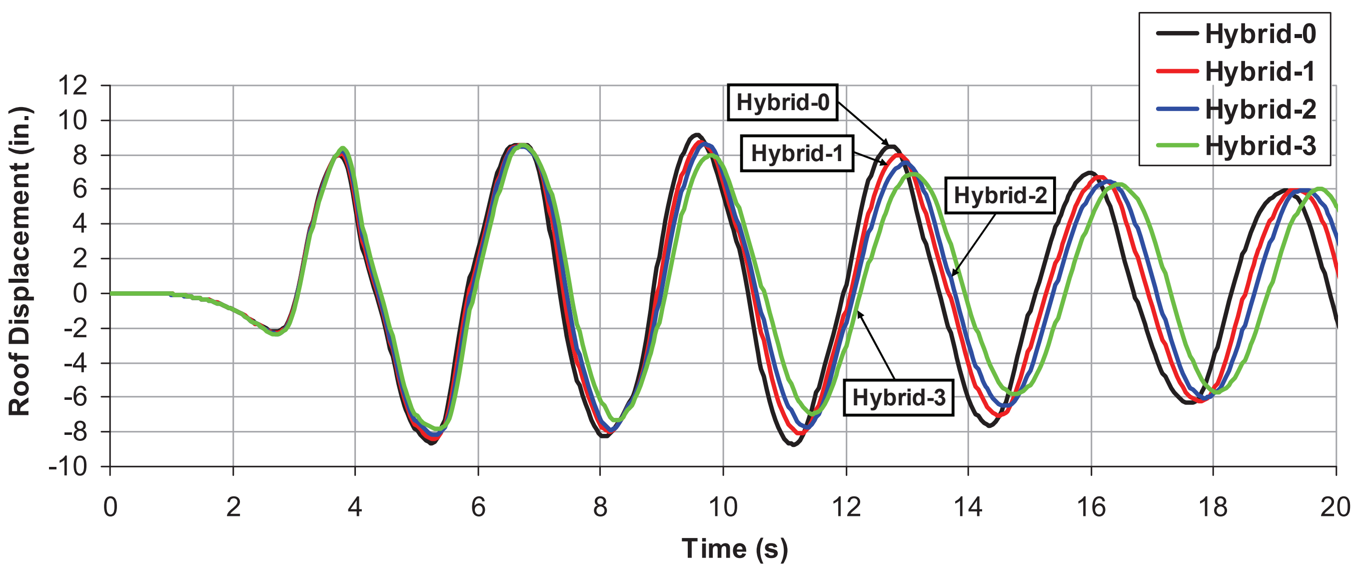

Fig. 11. Roof displacement response history of hybrid frames subject to Mendocino (EQ00) with scale of 0.4 times the anchored design spectrum scale.

displacement is the most for the hybrid-0 frame, which is actually the normal frame. Similar to the behavior in Figure 8, the hybrid-2, instead of the hybrid-3 frame, gives better results in terms of residual displacements. (See Figure 10.)

The effect of early yielding of hybrid frames on pushover curves is observed at low-scaled small magnitude earthquakes. Figure 11 shows an example of this behavior when the frames are subjected to EQ00. As the frames become more hybrid, the maximum displacements decrease due to hysteretic energy dissipation, which is a predicted result of early yielding. Similar results are obtained from EQ01, which is also a small magnitude earthquake.

Figures 12a and 12b illustrate the residual displacement IDA plots when the hybrid frames are subjected to EQ09 and EQ04, respectively. The real hybrid frames (hybrid-1, 2 and 3) result in better results (less residual displacements) for EQ09. The results of EQ04 are close; however, hybrid frames (especially hybrid-2 and 3) result in more residual displacements than the normal frame.

Figures 13a and 13b show the base shear IDA plots for EQ04 and EQ08, respectively. As the frames become more hybrid, the base shear decreases slightly under all the

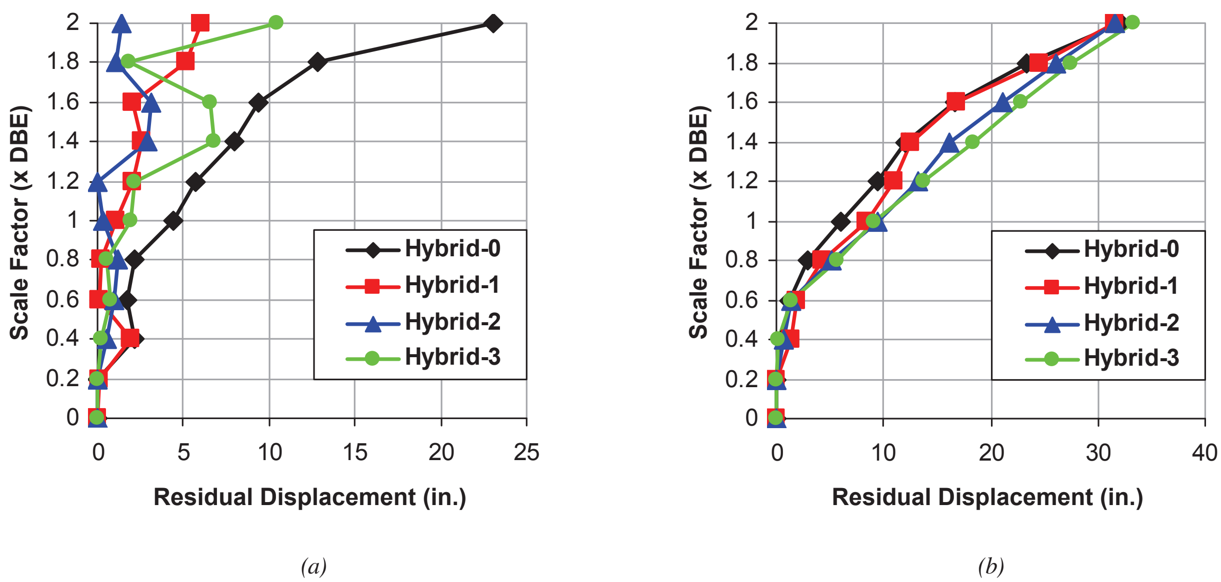

Fig. 12. IDA plots for residual roof displacement using (a) Shallow Interplate 2 (EQ09) and (b) Valparaiso (EQ04) ground motions.

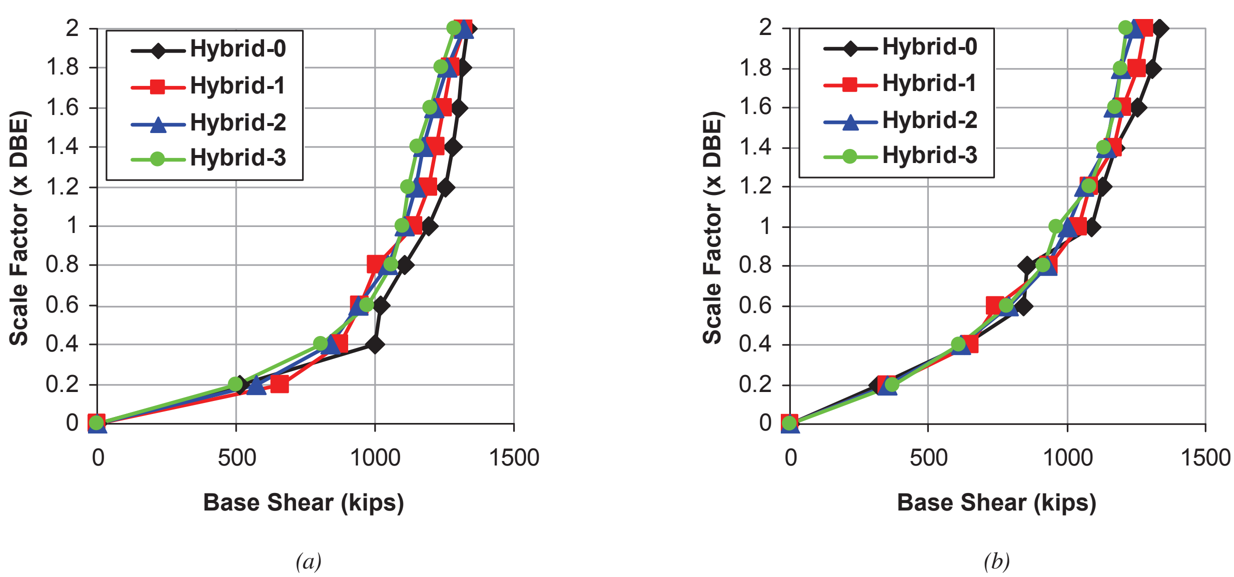

Fig. 13. Base shear IDA plots for (a) Valparaiso 1 (EQ04) and (b) Shallow Interplate1 (EQ08) earthquakes.

earthquakes. In the elastic part of the base shear IDA plots, base shears of different hybrid designs are almost identical. However, in the inelastic part, the normal frame results in slightly more base shear.

In hybrid frames, there is an increase in ductility demand for the elements that are expected to yield early. Figure 14 illustrates the ductility demand IDA plots for the entire hybrid frames subjected to EQ03. Plastic hinge rotations of the first-story bays were used to calculate the ductility demands.

As may be seen in Figure 14, bay-1, the weakest bay referring to SMF, has higher ductility than bay-2 and bay-3, which correspond to IMF and OMF systems, respectively. While bay-1 has the highest ductility demand, bay-3 has the lowest ductility demand for all of the hybrid designs. As the frames become more hybrid (from hybrid-0 to hybrid-3), the ductility demand difference between the bays increases at the same level of ground motion intensity. Because the hybrid-0 frame has the same girder sizes across the same level

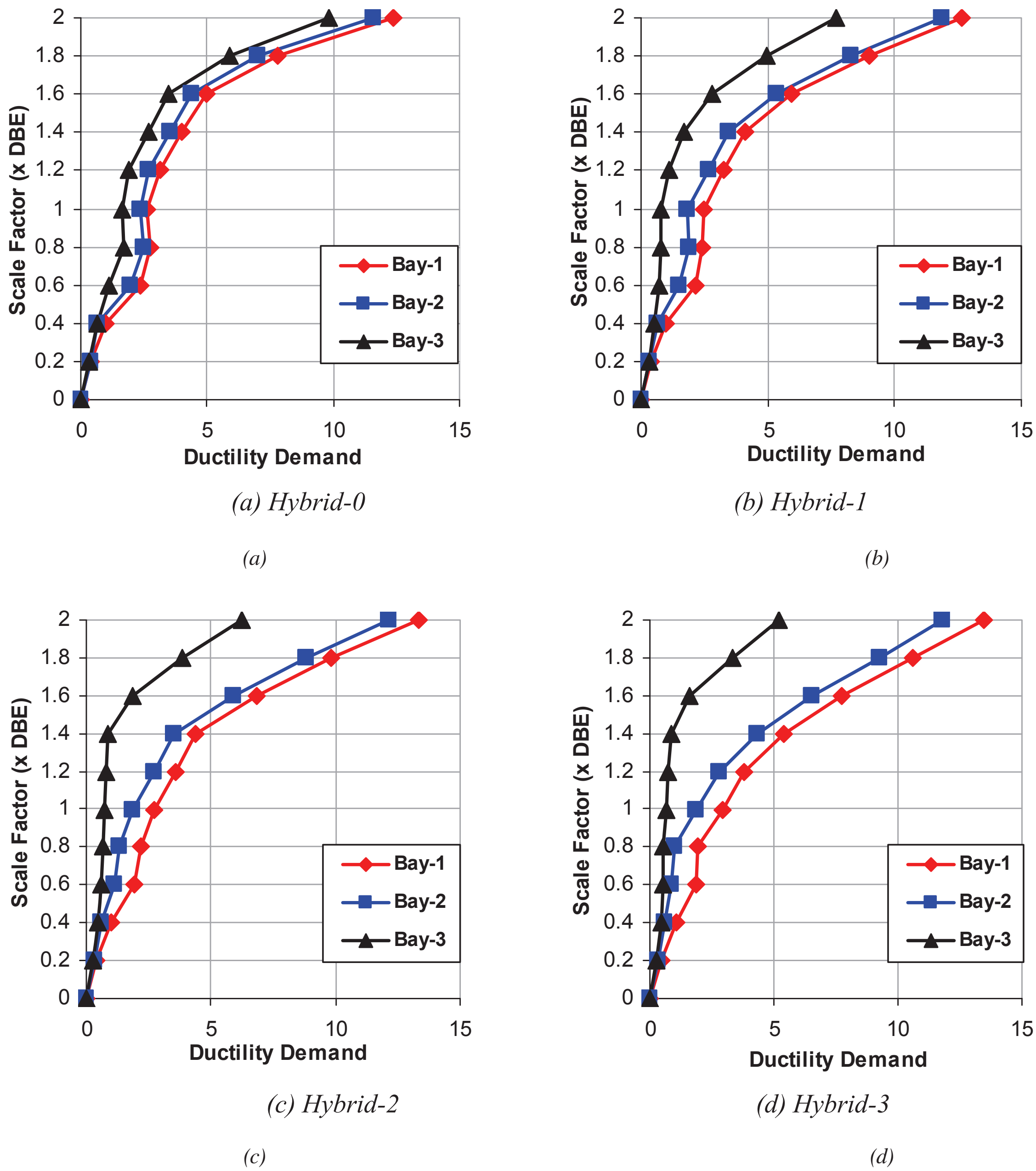

Fig. 14. Rotational ductility demand IDA plots for the first-story bays of hybrid frames (ground motion EQ03): (a) hybrid-0; (b) hybrid-1; (c) hybrid-2; (d) hybrid-3.

(story), the ductility demands of different bays are very close to each other (see Figure 14a).

In addition, the plastic hinges of bay-3 do not yield until the scale factor of 1.4 times the DBE for hybrid-2 frame and until 1.6 times the DBE for hybrid-3 frame. However, the plastic hinges of the same bay yield at a scale factor of 1.0 times the DBE for the hybrid-0 and hybrid-1 frames. As may be seen in Figure 15, as the frames become more hybrid, the ductility demand of bay-3 decreases, and the ductility demand of bay-1 increases (except for the scale factors of 0.6 to 0.8). As a result, the hinges that form first have the highest ductility demand and are detailed according to the rules for SMF systems, and the hinges that form last have the lowest ductility demand and are detailed according to the rules for OMF.

Note that only the first-story IDA ductility demands of Seattle earthquake (EQ03) are displayed in this paper. The Seattle earthquake resulted in about 30 in. residual displacement for all hybrid frame designs when the scale factor of 2.0 times the DBE was used. Different ground motions will result in different ductility demands; however, the general trend in the ductility demand of the different bays (corresponding to different moment frame systems) will be similar.

As a result of this preliminary moment frame study, real hybrid frames (hybrid-1, 2 and 3) always gave better results than hybrid-0 (normal) frame when the structures were subjected to severe earthquakes that caused collapses or significant residual displacements; i.e., hybrid frames are useful in terms of collapse prevention. This structural behavior can be explained with the effect of the relatively late occurrence of negative post-yield stiffness in hybrid frames (see Figure 6). Although hybrid frames could not improve the structural performance when the frames are subjected to EQ01, EQ02 and EQ04, if the overall performance is considered, hybrid frames resulted in better dynamic responses of the system. It is authors' opinion that hybrid frames especially perform better under pulse-type earthquakes where incremental velocities occur and give rise to damage or collapses. Figure 7 displays a nice example of this behavior. As may be seen, the residual displacements begin at the 11th second, and this is where the highest incremental velocity occurs in the Miyagi-Oki (EQ07) earthquake.

This preliminary hybrid moment frame study shows that this new approach may be considered at the design stage of new structures; however, further research is necessary, including an optimization study (where different R and C_d factors can be used with different new design configurations) as well as new strategies for hybrid frame development.

AN ALTERNATIVE STRATEGY FOR DEVELOPING HYBRID BEHAVIOR

In the moment frames studied in this paper, hybrid behavior was obtained by varying the moment capacities of the girders across the bays and by use of gravity preload. Another approach for achieving hybrid behavior would be the use of steels with varying yield strength. The use of low-strength steels and stainless steels might be particularly attractive for the early-yielding components of hybrid frames.

Among carbon steel alloys, two grades have been identified that have a low yield stress and strain and that have excellent ductility. These materials, called LYP steels (for

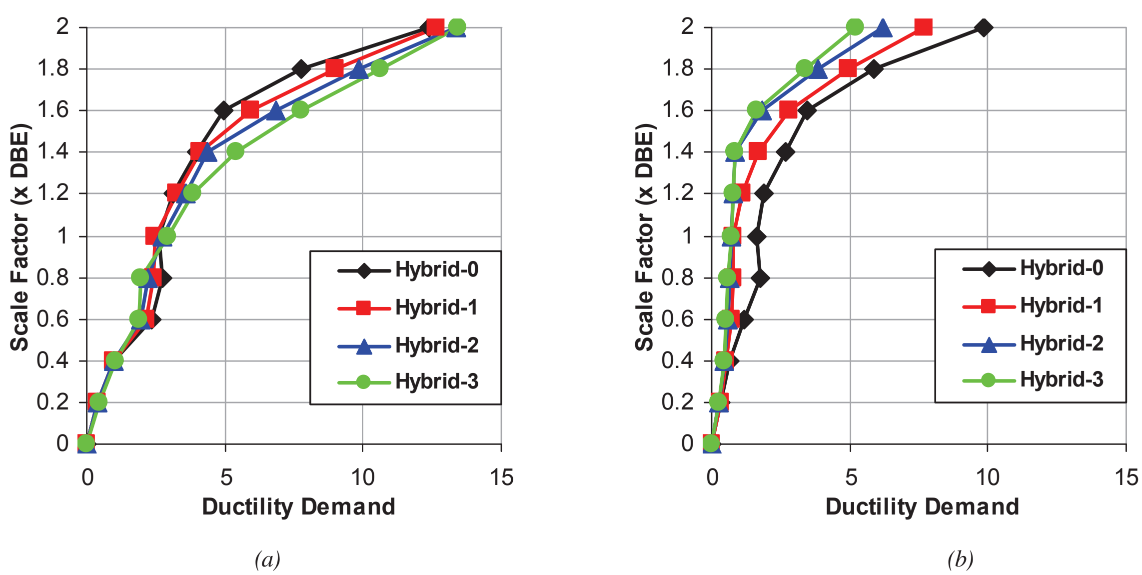

Fig. 15. Rotational ductility demand IDA plots for (a) bay-1 and (b) bay-3 of the first story (ground-motion EQ03).

low yield point), have yield stresses as low as 14.5 ksi (Saeki et al., 1998). The modulus of elasticity of the 14.5 ksi steel is approximately 22,500 ksi. While this modulus is lower than the modulus of structural steels (29,000 ksi), the yield strain of the 14.5 ksi steel (14.5/22,500 = 0.00064 in./in.) is significantly less than that of structural steels (50.0/29,000 = 0.00172 in./in.). Chen et al. (2001) tested four buckling restrained brace specimens using LYP and found them to be particularly effective for systems in which early yielding was desirable. Stainless steels have a relatively low yield stress when compared to structural steels and have excellent energy dissipation capacity. DiSarno et al. (2008) explored the use of stainless steels in both concentrically braced frames and eccentrically braced frames and found that the strain hardening characteristics of the stainless steels delayed inelastic buckling, which contributed to enhanced overstrength in the systems studied.

SUMMARY AND CONCLUSIONS

While the work reported in this paper is preliminary, it appears that there are significant benefits associated with the concept of hybrid frames. By carefully controlling the sequence of yielding, there is a clear indication of improvement in response at all levels of ground shaking, particularly at higher levels where dynamic instability may be more prevalent. At lower levels of shaking, the improvement is less significant, although there is a trend toward reduced displacements and base shears. This behavior is associated with the energy dissipation provided by early yielding of the low-strength plastic hinges.

For the frames studied, there is a significant increase in ductility demand, compared to traditional special moment frames, for those elements and connections that are expected to yield early. Although it is expected that traditional special moment frame detailing will suffice for these locations, additional research needs to be done to determine how much ductility can actually be provided by such connections. It may be necessary to develop special connection details for these areas. The use of special low-strength steels should also be investigated.

Additionally, the hybrid frames described herein were designed on an ad-hoc basis, because no specific rules have been established for assigning the sequence of yielding. It is expected that improved performance can be obtained if the sequence of hinging is more formally optimized. The use of an energy-based procedure is being explored for use in the development of an optimum hinging sequence.

Finally, additional work needs to be done to determine if significant economy is obtained by the hybrid frames. Such economy would be expected even if the performance of the hybrid frames was equivalent to the normal frames. This advantage in economy is due to the reduction in the number of special moment connections in the structure.

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