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PERFORMANCE-BASED SEISMIC DESIGN AND BEHAVIOR OF A COMPOSITE BUCKLING RESTRAINED BRACED FRAME

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SUMMARY

The concept of Buckling Restrained Braced Frames is relatively new and recently their use has increased in the U.S., Japan and Taiwan. However, detailed design provisions for common practice are currently under development. Since the summer of 2002, researchers at the University of Michigan (UM) have been working cooperatively in a joint study with research team at the National Center for Research on Earthquake Engineering (NCREE), Taiwan, involving design, analysis and full scale testing of such a frame by pseudo-dynamic method.

The selected structure is a three story, three bay frame consisting of concrete-filled-tube (CFT) columns, steel beams, and composite buckling restrained braces. The frame was designed using an Energy-Based Plastic Design procedure recently developed by co-author Goel at UM. The method utilized selected target drifts (2.0% for 10% in 50 year and 2.5% for 2% in 50 year design spectra for this frame) and global yield mechanism. Because of the need for more precise control of design clearances between the end connections and steel casing of the braces, buckling restrained braced frames are excellent candidates for application of this newly developed design methodology.

The paper briefly presents the energy-based approach developed at UM as well as a modal displacement-based design procedure adopted by the research team at NCREE for calculation of design base shear for the frame. Results from inelastic response analyses of frames deigned by the two methods for a Taiwan earthquake are compared. The same frame was also designed for a U.S. location and analyzed under ground motions scaled for U.S. standards. The frames designed by the UM approach exhibited satisfactory dynamic responses for both Taiwan and U.S. ground motions.

INTRODUCTION

Excellent seismic behavior of buckling restrained braces (BRBs) (Tsai [1]) encouraged an experimental program at the National Center for Research on Earthquake Engineering (NCREE), Taiwan, in conjunction with analysis and design studies by researchers in the U.S. at the University of Michigan. In

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this program, the BRBs provide the primary seismic resistance mechanism to a 3-story 3-bay frame, tested under pseudo-dynamic loading at NCREE in October 2003. General layout of the prototype building is shown in Figure 1a, while a view of the test frame is shown in Figure 1b. For design purpose, two of such frames were assumed to resist the total seismic force for a 3-story prototype building. The seismic frames are indicated by thick lines in Figure 1a.

DESCRIPTION OF TEST FRAME

The frame was designed to resist the seismic loading through two separate mechanisms. The primary resistance is provided by buckling restrained braces in the central bay of the frame (Figure 1b). This bay is designed to act as a purely braced frame with all beam-to-column and brace-to-column connections made as simple (pinned) connections. The braces are designed to resist 80% of the total seismic force for each seismic frame, while 20% of the load is resisted by the two external bays, designed as moment frames with moment connections at the joints of exterior beams and columns. All columns are made of concrete filled tubes. Different sections are chosen for interior and exterior columns, while keeping the same size along the building height. Wide flange sections are used for beams. Different beam sizes are used at different floors, while keeping the same size in all the bays at each floor.

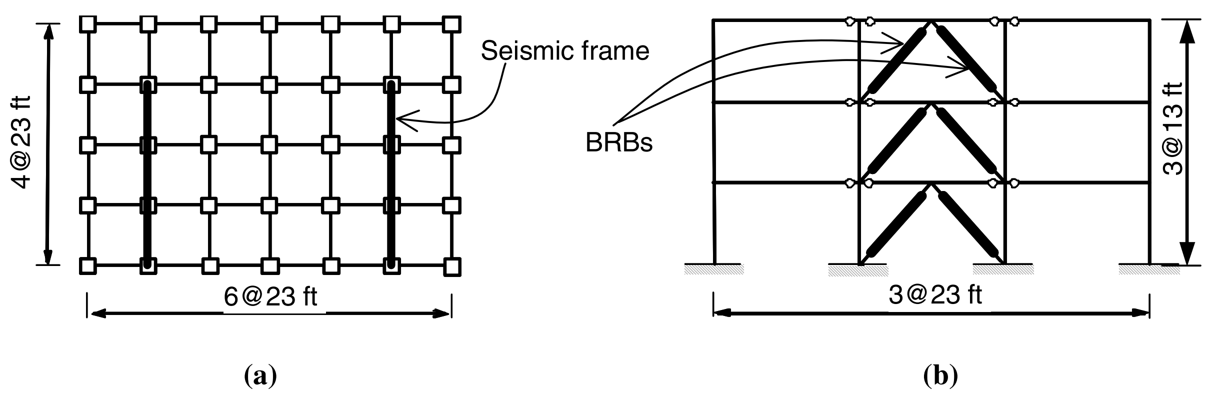
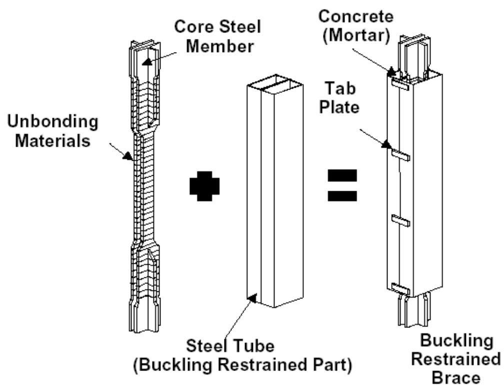


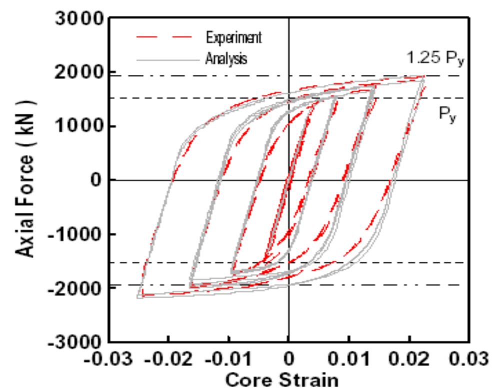
Figure 1: **(a)** Layout of the prototype building, **(b)** View of the test frame

BUCKLING RESTRAINED BRACE PROPERTIES

Buckling restrained braces are typically made by encasing a steel core member in a concrete filled steel tube (Figure 2a). The steel core is kept separated from the concrete filled tube by a layer of unbonding material applied on the surface of the steel core. The role of concrete encasing and steel tube is to prevent buckling of the steel core, so that a well formed load-displacement response of the brace is achieved under large displacement reversals. The unbonding material ensures that the force coming into the BRB is carried by the core only, without engaging the encasing material. Different configurations of BRBs were tested at NCREE under large reversed cyclic axial loading and an optimum configuration was selected for use in the test frame (Tsai [1]) (Figure 2a). A typical load-displacement response obtained from the selected BRB configuration is shown in Figure 2b. As can be seen, full hysteretic loops and excellent energy dissipation were achieved. However, it is to be noted that the yield load reached in compression was about 10% higher than that reached in tension. This needs to be accounted for while designing the frame.



(a)



(b)

Figure 2: **(a)** Configuration of the BRB adopted, and **(b)** Typical load-displacement behavior of a BRB (From Tsai [1])

DESIGN CONSIDERATIONS

A comparative study is presented on the behavior of three prototype frames designed to meet common performance criteria through different design procedures. The first frame was designed by the research team at NCREE. For this frame, calculation of base shear was done following a multi-modal displacement-based seismic design (DSD) procedure and the guidelines stipulated in the 2002 Draft Taiwan Seismic Design Code. Design of that frame was done by elastic method. The second frame was designed by the team at University of Michigan. In this case, same base shear as calculated by the NCREE team was assumed. However, a plastic design procedure, recently developed by Goel, was adopted to design the frame (UM Frame 1). The third frame (UM Frame 2) was designed for a base shear calculated by following a simple energy-based procedure developed at UM (Leelataviwat [2], Lee [3]). Frame design was done by the plastic design method as used for UM Frame 1.

The basic design parameters were selected by the research team at NCREE following the 2002 Draft Taiwan Seismic Design code. Total seismic weight of each floor was divided equally between the two seismic frames. The seismic weights applied on each frame were as follows,

1st and 2nd Floor: 714 kips, and 3rd Floor: 564 kips

In order to calculate the design base shear for the prototype building, two performance criteria were considered and the one that resulted in higher design base shear was chosen for the design of the test frame. In the first performance criterion (*Life Safety*), maximum roof drift was set at 0.02 radian when the building is subjected to an earthquake that has a 10% probability of exceedance in 50 years (10/50). In the second performance criterion (*Collapse Prevention*), maximum roof drift was set at 0.025 radian for a 2/50 seismic event. A real ground motion time history was scaled by appropriate factors to represent the 10/50 and 2/50 events. Scaling was done by considering the 5% damped Pseudo Spectral Acceleration (PSA) of a SDOF system with period T=1 sec. The scaling factors were determined by equating this spectral acceleration to the corresponding values prescribed in the draft Taiwan seismic code for 10/50 and 2/50 events at a hard rock site. The two resulting time histories have PGA values of 0.461g and 0.622g, respectively.

For the purpose of frame design and for performing the push-over analysis, the total base shear needs to be distributed over the three floor levels. The force at i-th floor was calculated by using the following equation:

$$F_i = \frac{m_i \delta_i}{\sum_{i=1}^{N} m_i \delta_i} V_d \,, \tag{1}$$

where m_i and δ_i are the mass and target displacement, respectively, of the i-th floor, and V_d is the total design base shear. The relative floor forces obtained are as follows:

1st Floor: 0.11, 2nd Floor: 0.365, and 3rd Floor: 0.525

DESIGN BASE SHEAR

Taiwan Design

A brief description of the NCREE procedure to arrive at the design base shear is presented in this section. A detailed description of this procedure can be found elsewhere (Tsai [1]).

In the first step, the frame was idealized as a MDOF system with three degrees of freedom. Modal Contribution Factors (MCF) for the three modes, as well as their modal masses and modal story drifts, were then computed. Since, for this particular frame, the contributions from the 2nd and 3rd modes (MCF = 0.008 and 0.002, respectively) were insignificant compared to the contribution from the 1st mode (MCF = 0.99) (Tsai [1]), only the 1st mode was considered for design purposes. Thus, the three floor displacements of the 1st mode were used to obtain an effective system displacement δ_{eff} associated with the modal target roof drift.

In the next step, the ductility demand for the 1st mode of the frame was computed. Because 80% of the seismic force was to be carried by the braced frame, yield drift of the effective system was computed based on the drift at the point of brace yielding and increased by 25% to account for the contribution from the moment frame. From the target maximum story drifts, ductility demand for each story was calculated and a simple average was taken as the effective ductility demand for the system. Using this ductility demand, and from the effective target displacement δ_{eff}, the effective time period of the system was obtained from the inelastic displacement spectrum of the ground motion considered. Corresponding effective stiffness K_{eff} value of the system was computed from this time period.

Finally, the base shear required at the point of target drift was computed by simply multiplying K_{eff} by δ_{eff}. This ultimate base shear was reduced to the yield base shear by assuming a bi-linear load-displacement curve with a strain hardening of 5% and the ductility demand as computed earlier. This yield base shear served as the design base shear (V_d) of the frame. Of the two performance criteria, the base shear computed from the second criterion governed and was equal to 415 kips.

UM Design

The base shear was re-calculated by using a procedure developed at UM (Leelataviwat [2], Lee [3]), where a fraction of the peak elastic input energy of an earthquake to a structure is equated to the energy needed by the structure in getting pushed up to the maximum target displacement. The procedure is briefly described below.

In the first step, the base shear-roof displacement profile of the frame was modeled by an idealized tri-linear curve, as shown in Figure 3. This tri-linear curve was obtained by considering the base shear-roof displacement profiles of the braced frame and the moment frame separately. Both of these profiles were idealized by elastic-perfectly plastic responses. Roof drift of the braced frame at yield point can be easily calculated from the geometry of the frame. As mentioned earlier, the base shear carried by the braced frame at this point was assumed as 80% of the total design base shear V_d, which is an unknown at this stage. Based on past analysis results, roof drift of the moment frame at yield was assumed as 2%, carrying the remaining 20% of the total base shear. These two bi-linear curves were superimposed to obtain the tri-linear load-displacement curve of the whole frame (Figure 3). This tri-linear curve was further simplified to a bi-linear curve (Figure 3) by equating the areas under the two curves. The design ductility demand μ for the frame was calculated from this curve.

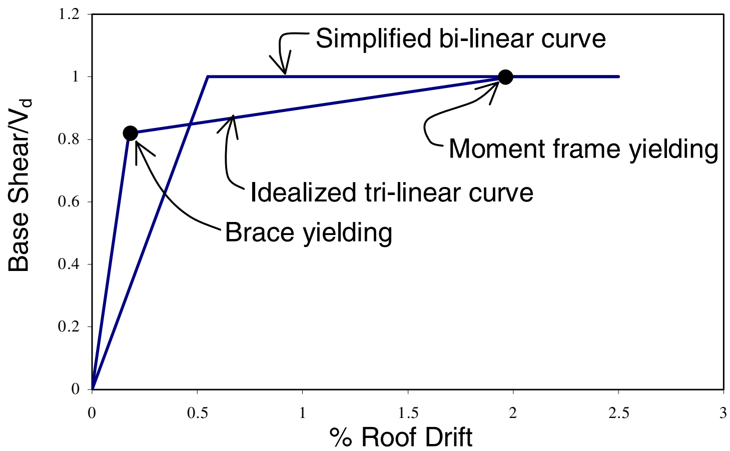


Figure 3: Idealized frame responses for *Collapse Prevention* criterion

In the next step, the peak input energy was calculated by considering an elastic SDOF system and by using the equation given by Housner [4], as shown below,

$$E = \frac{1}{2} M S_v^2 , \tag{2}$$

where M and S_v are the total mass and the pseudo spectral velocity of the system, respectively. However, for an inelastic system, this equation needs to be modified (Figure 4a). Thus, a modification factor γ was applied to Eqn. (2) to estimate the energy needed to push the idealized elastic-perfectly plastic system to the selected target displacement, as shown in Figure 4a. Applying this modification factor and converting S_v to spectral acceleration $C_e g$, the modified required energy, E_m , can be re-written as,

$$E_m = \frac{1}{2} \gamma W g \left[\frac{T}{2\pi} C_e \right]^2 , \tag{3}$$

where W and T are the total weight and the fundamental period of the system, respectively. C_e is the maximum base shear coefficient. Following the seismic provisions of IBC 2000 [5], the value of T for the 3-story frame was estimated as 0.37 sec. Using this period, C_e was obtained from the design response spectra given in the Draft Taiwan Seismic Code (2002) for the two considered hazard levels. The value of γ was obtained from the $\gamma - \mu - T$ relationship (Figure 4b) proposed by Leelataviwat [2].

The modified input energy, E_m is then equated to the total work done by the seismic forces applied to the frame as it is pushed to the target drift as shown in Figure 4a. For this purpose, a bi-linear load-displacement behavior (Figure 4a) and a linear distribution of the floor displacements along the height of the frame were assumed. A distribution of floor forces, as mentioned earlier, was also assumed. From this energy balance equation, design base shear V_d was obtained. As with the Taiwan method, the base shear computed from the second criterion (2.5% drift for 2/50 earthquake) governed and was equal to 340 kips, which is significantly smaller than that obtained from the Taiwan method.

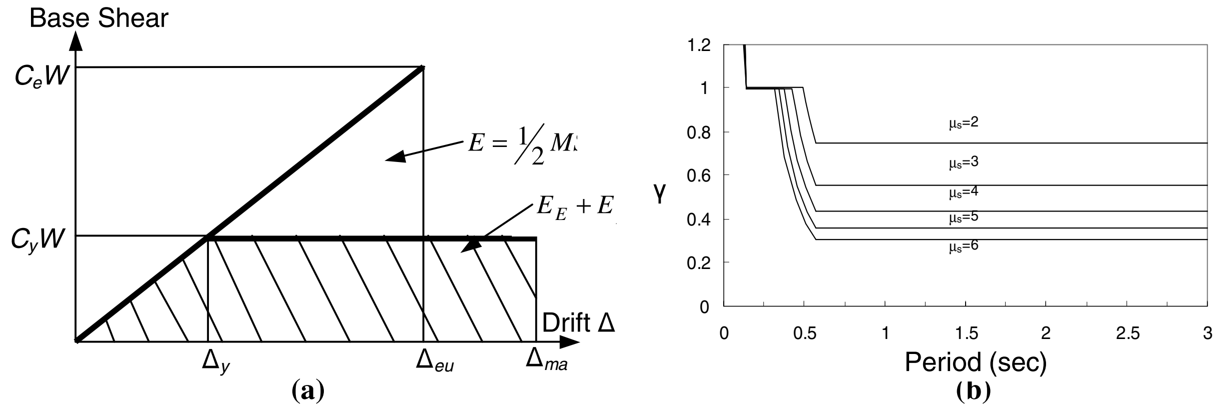


Figure 4: **(a)** Energy input in elastic and inelastic systems, **(b)** Energy modification factors against period

UM PLASTIC DESIGN METHODOLOGY

The research team at the University of Michigan adopted a simple plastic design procedure to design the test frame. Designs of the bracing part and the moment frame were done independently from each other with an 80-20% sharing of total base shear between the two components.

Material Properties

Material properties used for the design and analysis of the frame were kept the same as those used in the Taiwan design. All steel sections: core member of the braces, wide flange beams and steel tubes of the CFT columns, had nominal yield strength of 50 ksi with bi-linear stress-strain curves. Strain hardening ratios of 4% for beams and columns, and 2% for the brace members were considered. Additionally, a 10% overstrength was considered for the expected yield strength of steel for the beams and columns. However, no overstrength was considered for the brace members, because the material for the braces was to be tested before their fabrication in order to get an accurate measure of its strength, and its effect was to be incorporated by adjusting the cross-sectional areas of the braces. The concrete used for the CFT columns was assumed to have a strength of 5000 psi.

Design of Moment Frame

The moment frame was designed for 20% of the total base shear, to be carried by the exterior columns and the exterior beams. The design of the frame was done following the guidelines provided in a related research by Leelataviwat [2] and Lee [3], and using the proposed plastic design methodology. Designs of the columns and beams were done simultaneously to balance the member capacities, so that a desired yield mechanism is achieved. The design steps are described below.

As a first step, a desired failure mechanism was selected. The assumed failure mechanism consisted of beam-column junctions developing plastic hinges only in the beams at all three floors, and a plastic hinge at the base of each column (Figure 5a). Thus, strong column-weak beam philosophy was followed. It should be mentioned that the design strength of the columns was calculated only from the steel tube section, neglecting the contribution from the concrete core.

An initial required strength of the column section was estimated from the consideration of avoiding the possibility of a *story mechanism* in the first story. This was done by considering the base shear to be resisted by a story mechanism in the first story, *i.e.*, the columns in the first story forming plastic hinges at both ends (Figure 5b). Minimum required flexural capacity of the columns was obtained by equating the overturning moment due to the shear carried by the columns to the combined design capacity of the plastic hinges at the ends of the columns. A 10% margin of safety was also considered. A square tubular section with strength greater than the required strength was then assigned to the columns. Since the same column section was continued along the full height, story mechanisms in the upper stories were automatically avoided.

Required flexural strengths for the beams were calculated in the next step. Beam strengths were determined by assuming a scenario where the moment frame is acted upon by its full share of base shear, distributed at the three floors as described earlier, and having developed plastic hinges in all three beams and at the base of the columns (Figure 5c). Thus, the total resisting moment provided by the capacity of the plastic hinges in the beams and column bases was equated to the total overturning moment of the applied floor forces. This, however, produced only one equation, whereas three beam strengths needed to be determined. This was dealt with by assuming the flexural capacities of the beams at the three floors to be proportional to the applied story shears (Figure 5c). The required flexural strengths of the beams were then calculated and available wide flange sections with strengths closest to the required strengths were selected.

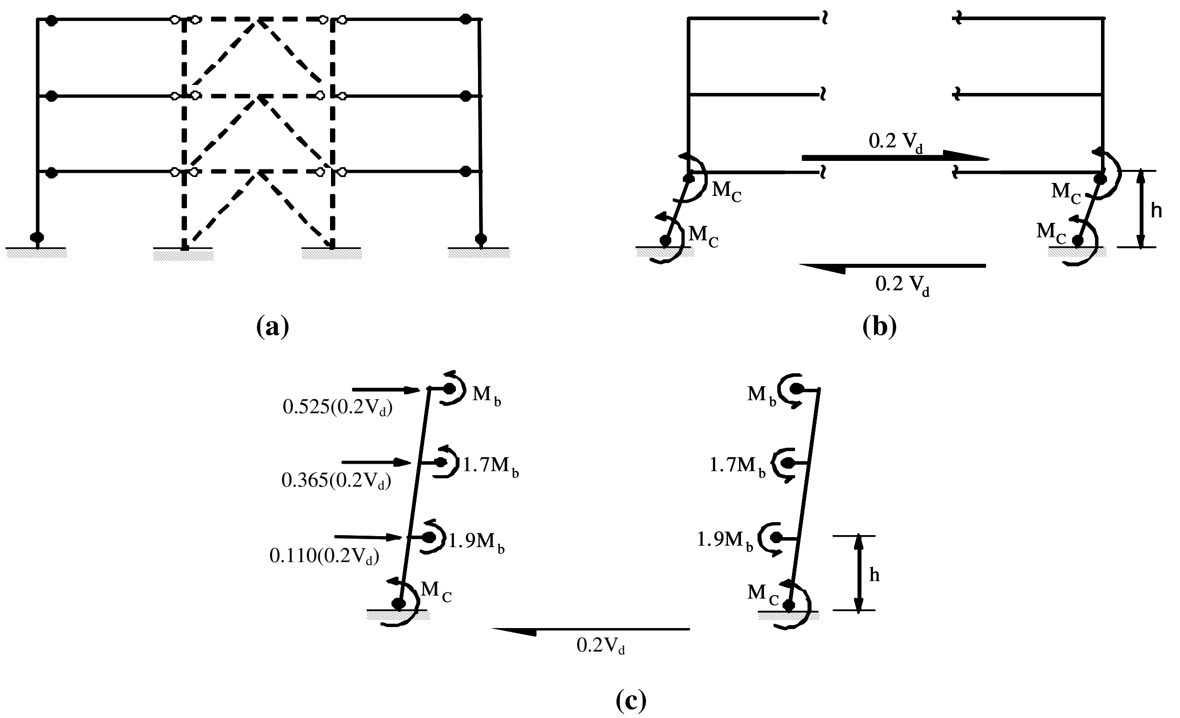


Figure 5: **(a)** Moment frame yield mechanism, **(b)** Minimum column strength from story mechanism, and **(c)** Moment demands on beams

As a last step, the expected beam strengths with strain hardening and the applied floor forces were used to calculate the moment demands on the column at the beam-column joints to ensure that no plastic hinge formed in the column at any location.

Design of Braced Frame
As mentioned earlier, the braced bay of the frame was designed to resist 80% of the total lateral forces. Design of the braced frame involved design of the braces, the interior beams and the interior columns.

Since all connections in the braced frame were assumed pinned, the applied shear force in each story was resisted by the pair of braces only. Because the bases of the interior columns were assumed to be fixed, these columns act as cantilevers and carry some amount of shear as the frame undergoes lateral drift. However, for simplicity and to be conservative, this effect was neglected. Thus, cross-section area of the braces in each story was obtained by simply considering the horizontal component of the design yield forces of the two braces and equating that to the corresponding story shear, as shown in Figure 6.

The interior beams were assumed as pin connected to the interior columns at both ends, while the braces connect at the beam midspan. The beams were designed for a combination of axial force and bending moment. As the frame drifts, one brace goes into tension and the other into compression. Horizontal components of these forces represent the axial load applied at the midspan of the interior beam. Vertical components of these forces tend to cancel each other. However, since the yield strength of the compression brace is assumed to be 10% higher than that of the tension brace, an unbalanced upward force remains once both braces have yielded, causing bending moment in the beam. Thus, the interior beams were designed as simply supported beams with an axial load and a transverse load applied at the center, as shown in Figure 6. The wide flange sections used in the exterior beams were checked against this combined loading following the provisions of the AISC steel design manual [6], and were found adequate for use in the braced frame.

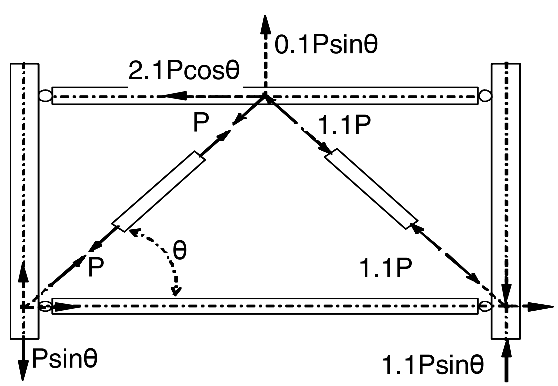


Figure 6: Forces in braced frame after yielding

Design of the interior columns was done primarily from axial load consideration (Figure 6). Columns in any story carry the vertical components of the brace forces from the stories above. Thus, interior columns in the 1st story were designed for an axial force which is the sum of the vertical components of the brace forces from the 2nd and 3rd stories. Brace forces were calculated at the point when the frame is at its assumed target drift of 2.5%. Also, the higher yield load of the compression brace compared to that of the tensile brace, as mentioned earlier, was considered. However, there are some other factors that affect the load on the interior column. These are:

1. Shear force on the exterior beams is transferred through the pinned connection and acts as axial load on the interior columns. From the deflected shape of the frame, it is seen that this shear force always acts in the opposite direction to the force coming from the braces, thus reducing the net load on the column.

2. Shear force on the interior beams adds a small tensile load on both interior columns.

3. Interior columns carry some shear and bending moment by virtue of their bent shape.

For simplicity of design, the above factors were neglected.

COMPARISON OF UM FRAMES WITH TAIWAN FRAME

The cross-section areas of various members obtained from the two UM designs and from the Taiwan design are shown in Table 1. It can be seen that the UM Frame 1 had almost identical member sizes as those in the Taiwan frame. Some differences can be seen in the sizes of the beams at all three floors - those in the UM frame being lighter than those in the Taiwan frame. While the difference is small at the 1st and 2nd levels, the beams at the 3rd (roof) level in the UM frame were significantly lighter than those in the Taiwan frame. The UM Frame 2, being designed for a significantly smaller base shear, was much lighter than the other two frames.

Table 1: Member cross-section areas (in^2) of frames for Taiwan site

		Taiwan Frame	UM Frame 1	UM Frame 2
Braces	1st Floor	5.12	5.34	4.60
	2nd Floor	4.50	4.75	4.10
	3rd Floor	2.64	2.80	2.41
Exterior Columns	1st Floor	20.0	20.0	20.9
	2nd Floor	20.0	20.0	20.9
	3rd Floor	20.0	20.0	20.9
Interior Columns	1st Floor	18.4	18.4	15.0
	2nd Floor	18.4	18.4	15.0
	3rd Floor	18.4	18.4	15.0
Beams	1st Floor	17.1	16.2	13.3
	2nd Floor	14.6	13.5	11.8
	3rd Floor	12.7	9.13	7.68

FRAME DESIGNED FOR U.S. EARTHQUAKES

To further the study on applicability of the energy-based approach to calculate design base shear, the same frame was re-designed for a U.S. location (UM-US). Seattle was chosen as the site for the frame. For design purpose, an increased triangular profile of lateral forces, as specified in IBC 2000 [5], was used. The frame was designed to meet the performance criterion of 2.5% target roof drift in a 2/50 seismic event. Using a spectral acceleration of 1.98g for a 2/50 event, the design base shear V_d was computed as 680 kips. Table 2 shows the member sizes obtained for this frame.

Table 2: Member cross-section areas (in^2) of frame for Seattle site

	Braces	Exterior Columns	Interior Columns	Beams
1st Floor	9.18	30.4	15.0	29.1
2nd Floor	7.46	30.4	15.0	24.0
3rd Floor	4.04	30.4	15.0	12.6

PUSH-OVER ANALYSIS

Static push-over analyses were performed on the first three frames as a first step in evaluating and comparing their behavior. SNAP-2DX program, developed at the University of Michigan for non-linear static and dynamic analysis of 2D frames (Rai [7]), was used for this purpose. The push-over analysis was done under *displacement-control* mode and the frames were pushed to about 3% roof drift with the lateral loads being applied at three floors in the same ratio as the design distribution of the base shear. This ratio was maintained while the controlling roof displacement applied to the frame was increased. The displacement of the roof was increased in steps of 0.05 inch. Lateral load at any floor was distributed equally at all five nodes. *i.e.*, four beam-column joints and one brace-beam joint. BRBs were modeled using a simple truss element with bi-linear load-displacement relationship. The truss element had a compression capacity 10% higher than its tension capacity, and 2% strain hardening. The beams and the CFT columns were modeled using a beam-column element. Bi-linear moment-rotation curves with 4% strain hardening were assumed for these members. Appropriate P-M interaction relations for the beam-column elements were also used. The resulting push-over curves obtained for the three frames were compared with the load-displacement curves originally assumed in the design stage. Locations and sequence of yielding were noted and compared for the three frames.

DYNAMIC ANALYSIS

Dynamic analyses were carried out on all the frames by subjecting them to scaled real ground motions using the SNAP-2DX program. All the ground motions used represented seismic events with 2% probability of occurrence in 50 year intervals. The ground motion time history used to analyze the frames designed for the Taiwan site was obtained by appropriate scaling of a record from the Chi-Chi earthquake. UM-US frame was subjected to three ground motions selected from those recommended by SAC for a Seattle site. Some characteristics of the selected ground motions are given in Table 3. Frame members were modeled as in the push-over analysis. A 5% viscous damping ratio was considered. Floor masses were distributed equally at the five joints at each level.

Table 3: Characteristics of 2/50 ground motions

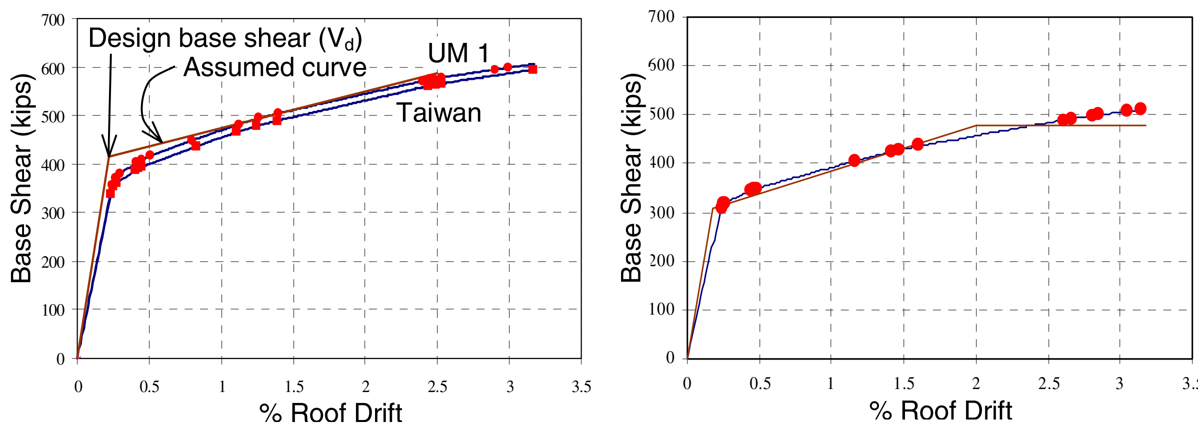
	Magnitude	Distance (miles)	Scale factor	Duration (sec)	PGA (g)	Spectral Intensity (in.)
1999 Chi Chi	7.3	-	2.87	45.0	0.62	129
1992 Erzincan	6.7	1.24	1.27	20.78	0.61	130
1949 Olympia	6.5	34.8	4.35	79.98	0.82	115
1985 Valpariso	8	26	2.9	99.98	1.57	179

DISCUSSION OF RESULTS

Push Over Analysis

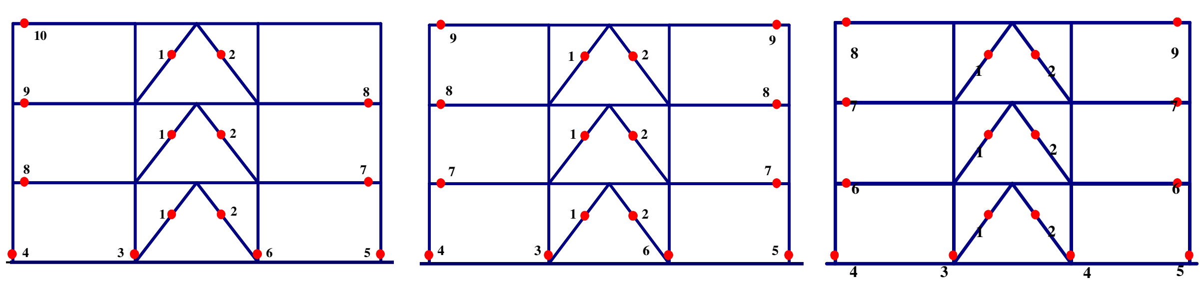
Figure 7a shows the base shear vs. roof drift push-over curves for Taiwan Frame and UM Frame 1, which were designed for the same base shear. Displacements at which yielding occurred at various locations of the frames are also indicated on the curves. These curves are superimposed on the basic load-displacement curve constructed from the design base shear (415 kips) and ductility demand (11.4) as mentioned earlier, and by using a 4% strain hardening. As it can be seen from the figure, static load-displacement behaviors of the two frames are almost identical, with the UM frame carrying slightly higher force. Also, push-over curves are in excellent agreement with the assumed bi-linear curve, as can be seen from Figure 7a. Figure 7b shows a similar push-over curve for UM Frame 2, which was designed for a lower base shear as calculated by the UM method. The pushover curve is superimposed on the idealized tri-linear curve, which was obtained by adjusting the design tri-linear curve (shown in Figure 3) for overstrength and strain hardening. The two curves are seen to be in good agreement.

The locations and sequence of yielding are seen to be similar in all three frames (Figure 8). Yielding occurred at the intended locations only. However, at 2.5% roof drift, none of the frames achieved the complete mechanism. Also, for all three frames, the ratio of base shear carried by the moment frame and the braced frame were very close to the design ratio of 1:4.



(a) (b)

Figure 7: **(a)** Push-over curves for Taiwan Frame and UM Frame 1, and **(b)** Push-over curve for UM Frame 2



(a) (b) (c)

Figure 8: Locations and sequence of yielding - **(a)** Taiwan Frame, **(b)** UM Frame 1, and **(c)** UM Frame 2

Dynamic Analysis

Various aspects of the dynamic behavior of the three frames for the Taiwan site are shown in Figures 9a through 9d. Maximum values at three floors, however, did not necessarily occur at the same time instant. The three frames behaved in a very similar fashion. From the plots of maximum floor displacements (Figure 9a), it is clear that the lateral displacement profile of all three frames followed a linear pattern. However, the maximum roof drifts for Taiwan Frame and UM Frame 1 were below the target roof drift of 2.5%, and were about 2.1%. The third frame (UM Frame 2) performed more closely to the intended performance level and reached a roof drift of 2.6%. From the plot of maximum story displacements (Figure 9a) and maximum inter-story drifts (Figure 9b), it is evident that the three floors of all three frames underwent a fairly synchronized motion, indicating a predominantly first mode of vibration. UM Frame 2, did show some irregularity as the maximum drift of the second story exceeded the 2.5% limit to reach to almost 2.8% drift (Figure 9b). However, time history of the floor displacements showed that there was only one time instant when this exceedance occurred. At all other time instants, the third frame also showed uniform story drifts as the first two frames.

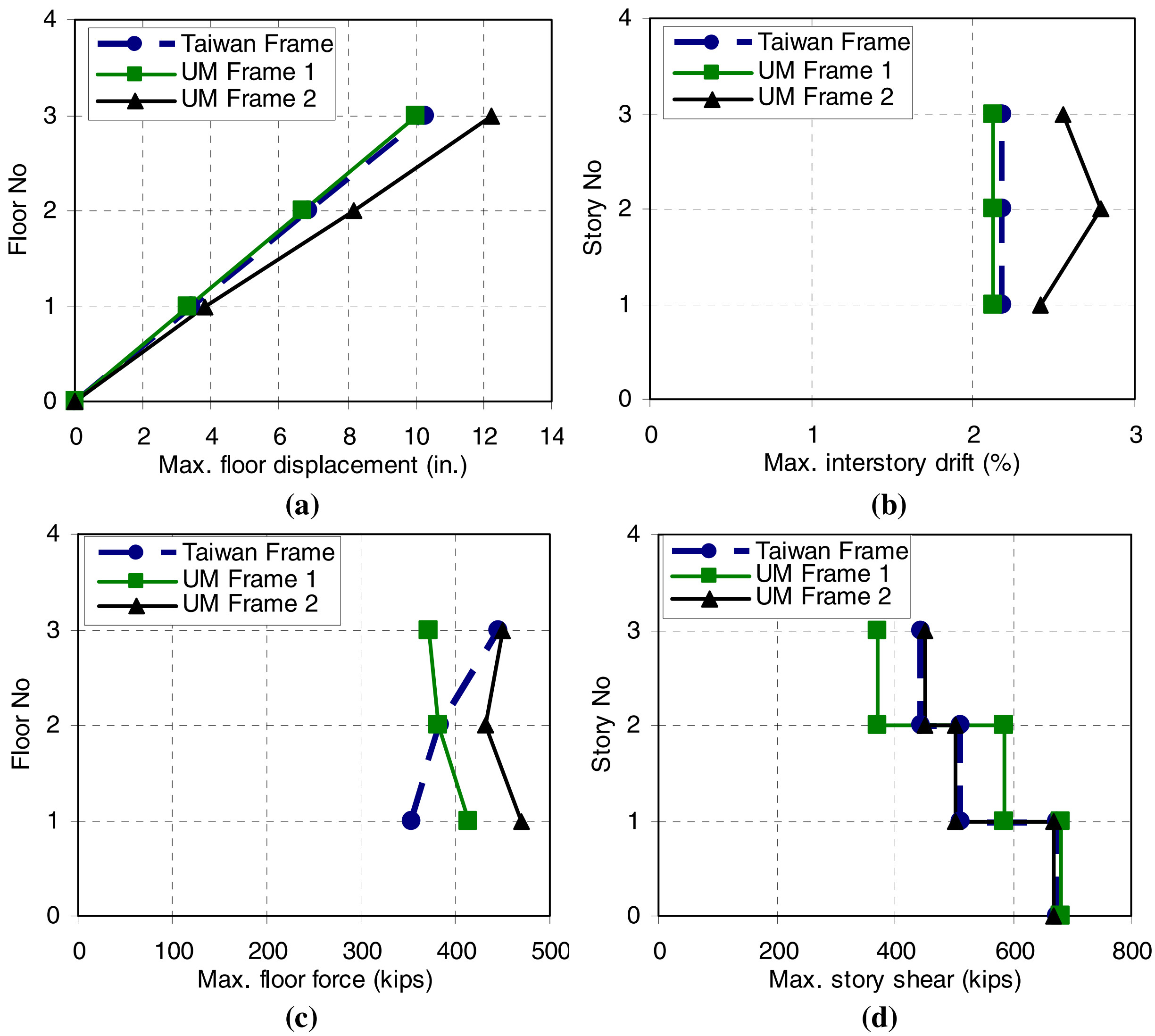


Figure 9: Dynamic response parameters of three frames for Taiwan site

From the plot of story shears (Figure 9d), it is seen that the maximum base shear is significantly higher than that obtained from the push-over analysis for all three frames. This may be attributed to the significantly different floor force distribution obtained from the dynamic analysis (Figure 9c) compared to what was assumed for the design of the frames and for the push-over analysis. This was verified by using the floor force distribution at the time instant when the maximum base shear was reached, and performing a push-over analysis with that force distribution.

Figures 10a through 10d show selected response parameters from the dynamic behavior of the frame designed for a Seattle site (UM-US) when subjected to three different ground motions representing 2/50 seismic events. The frame remained well within the target roof drift of 2.5%, reaching 1.56%, 1.48%, and 1.51% (Figure 10a) for the Erzincan, Olympia and Valpariso ground motions records, respectively. As observed in the previous three frames, this frame also exhibited approximately uniform drift over the three stories for the three ground motions, as seen from Figures 10a and 10b. Further, a very different floor force distribution from that assumed for the design of the frame was seen for three three ground motions (Figure 10c), with lower floors carrying larger forces. Again, this can be the reason for larger base shears (Figure 10d) than that calculated for design. As can be observed from Figures 10c and 10d, the more the deviation of the floor force distribution from the design distribution, larger the base shear carried by the frame.

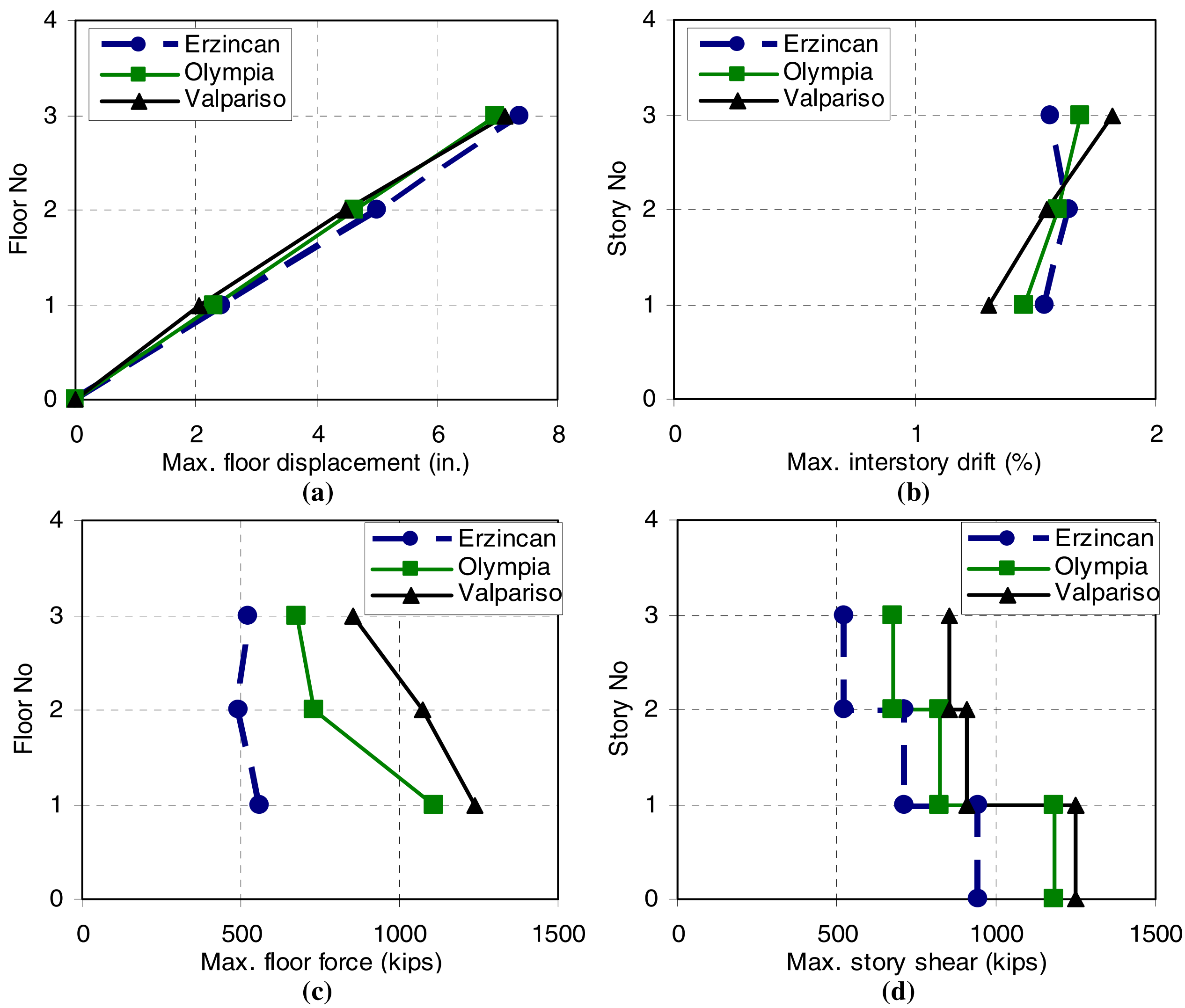


Figure 10: Dynamic response parameters of frame UM-US for three Seattle ground motions

SUMMARY AND CONCLUSION

Results from the seismic analysis of frames with buckling restrained braces are presented in this paper. Two frames were designed by using a plastic design procedure proposed at the University of Michigan (UM) by the co-author Goel. The third frame was designed by researchers at NCREE, Taiwan. These three frames were designed for a site in Taiwan. The first UM frame, as well as the Taiwan frame were designed for a base shear calculated by using a modal displacement-based procedure, while the second UM frame was designed for a base shear calculated through an energy-based method developed at UM. These designs and analyses formed the foundation of the testing program performed at NCREE to evaluate the seismic performance of buckling restrained braced frames at full scale. A fourth frame was then designed for a U.S. site to validate the applicability of the energy-based design method for this type of frames.

The 3-story 3-bay full scale frame was a combination of braced frame and moment frame, where the braced frame carried most of the lateral force. Two performance criteria were considered for the design of the frames in the Taiwan site: 2% roof drift for a 10/50 earthquake, and 2.5% roof drift for a 2/50 earthquake. A real ground motion time history, scaled by appropriate factors, was used to represent the two seismic events. Design base shear was calculated for both performance criteria and the governing base shear and corresponding performance level were chosen for design. Push-over and dynamic analyses were carried out on these three frames. The frame at the U.S. site was designed only for the criterion of 2.5% drift in a 2/50 event. Three ground motions representing 2/50 earthquakes were applied to this frame to study its dynamic response.

The Taiwan Frame and UM Frame 1 had almost identical frame member sizes, and as a result, similar pushover and dynamic responses were observed. In the dynamic analysis, inter-story drifts in the three stories were quite uniform. Maximum roof drifts were below the design roof drift of 2.5%. The UM Frame 2 was significantly lighter, and marginally exceeded the target roof drift. Nearly uniform inter-story drifts were observed in this frame also, even though a slight non-uniformity was encountered at the peak roof drift.

The Taiwan Frame and UM Frame 1 were designed for the same seismic force, 80% of which was carried by the braced part of the frame. Different design approaches followed by the Taiwan team (elastic) and the UM team (plastic) made little difference in the sizes of the braces. Main difference was in the member sizes of the moment part of the frame. However, while designed to resist only 20% of the base shear, the moment frame did not have any significant influence on the overall frame response. As a result, very similar inelastic response under push-over as well as dynamic analyses was observed.

UM Frame 2 was designed for a significantly smaller base shear, and as a result, was much lighter than the previous two frames. This frame reached a maximum roof drift of 2.6% in the dynamic analysis, very closely meeting the target roof drift.

UM-US frame also performed satisfactorily under all three selected ground motions. Roof drift remained well within the 2.5% limit and the three floors exhibited nearly uniform drifts.

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