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# Behavior of weak column strong beam steel frames

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ABSTRACT: Moment-resisting steel frames are highly regarded for their seismic performance. This regard is based on their ductility and inelastic performance, since inelastic deformation is used to dissipate energy during major earthquakes. Recent changes to seismic design provisions in the United States permit the use of steel frames which develop inelastic deformation in columns and panel zones of the steel frame. These changes have significant impact on the seismic performance of moment-resisting steel frames. This impact is examined through inelastic analysis and an experimental investigation. The results show that frames which dissipate energy in their columns may have poor seismic performance during major earthquakes if they do not have reserve strength beyond the minimum required by most seismic design provisions. Further, columns with slender flanges or high axial forces may not have as favorable seismic performance as other columns.

## INTRODUCTION

Ductile moment-resisting steel frames behave well during earthquakes, since they have good strength, stiffness and energy dissipative characteristics. This reputation is based on field performance and laboratory experiments on frames with good connection details and strong-column weak-beam (SCWB) joints. Recent changes to the Uniform Building Code (UBC) [ICBO (1988)] may have some impact upon the seismic performance of these frames. Many of these changes center around the weak-column strong-beam (WCSB) structural system.

Four major design issues are important in the seismic performance of these frames. First, the magnitude of forces used in seismic design are much smaller than the forces which can be expected during a major earthquake. The building is designed by the allowable stress method for a relatively small seismic force that may be expected during a minor seismic event. During a major earthquake the safety of the frame is assured by inelastic deformation, which results in dissipating some of the seismic energy to dampen the magnitude of dynamic response, thereby controlling deformations of the structure. The minimum base shear required by the 1988 UBC is

$$V = \frac{ZIC}{R_W} W \tag{1a}$$

$$C = \frac{1.25\ S}{T^{2/3}} \tag{1b}$$

$$T = .035\ h_n^{3/4} \tag{1c}$$

These design forces are similar to those used in earlier UBC provisions, but the fundamental period, T, may play a different role than in earlier provisions. The period, T, may be computed approximately (Eq. 1c) or by a dynamic analysis, but the minimum seismic design force cannot be less than 80% of that required with the approximate period. The $R_W$ values are assigned to various structural systems according to perceived ductility of the structural system. Ductile systems are assigned large values of $R_W$, and this results in small seismic design forces. The provisions make no distinction between SCWB and WCSB frames in the determination of $R_W$ and the seismic design forces. Special slenderness limits are required for the web, flanges and lateral support to help assure this ductility, but the UBC uses these limits for the beams only.

Second, SCWB joints are assured in the 1988 UBC by

$$\Sigma Z_c\ (Fy - f_a) / \Sigma Z_b\ Fy > 1.0 \tag{2}$$

SCWB joints are generally believed to be more ductile than WCSB joints, but some constraints may make it difficult to achieve SCWB joints in practice. The 1988 UBC permits limited usage of WCSB joint in recognition of these practical constraints. The strength of the joint need not satisfy Eq. 2 if the axial column load does not exceed 40% of the column yield force, if the shear resistance of the story is more than 50% greater than the story above it, or if the column is not part of the lateral load-resisting frame.

A third major issue is the seismic drift limits for the seismic design forces. The UBC limits drift to the smaller of $\frac{0.03}{R_W}$ or .004 times the story height. However, drift limits may be checked with the lateral forces associated with the computed dynamic period rather than the forces required for the minimum lateral resistance. The calculated stiffness is often smaller than the stiffness associated with the approximate code period (Eq. 1c), and this substitution may substantially ease the drift limit provisions.

Panel zone strength, is the fourth significant influence to the seismic behavior of steel moment-resisting frames. Krawinkler (1971, 1978) has shown that bending moments due to seismic loading cause large shear stresses in the panel zone. The UBC limits this shear force to

$$V = 0.55\, F_y\, d_c\, t_w \left[ 1 + \frac{3\, b_c\, t_{cf}^2}{d_b\, d_c\, t} \right] \qquad (3)$$

The shear capacity permitted by this equation is somewhat larger than that permitted in earlier UBC provisions. Further, the panel zone shear force checked by this equation may be the smaller of 80% of the plastic capacity of the beams or 1.85 times the seismic design moments plus the dead load moment.

## PRIOR EXPERIMENTAL RESEARCH

A substantial body of knowledge underlies these design provisions. Popov (1969), Vann (1973), Mitani (1977), Suzuki (1977), and Takanashi (1973) performed experiments on cantilever beams and beam column elements. A smaller number of cyclic load tests on beam-column connections and frame subassemblages were performed by Bertero (1973), Popov (1969,1975), Krawinkler (1971), and Kato (1973). Overall frame behavior experiments were performed by Carpenter (1969), Clough (1975), Takanashi (1984), and Wakabahashi (1967, 1973, 1967). These frame experiments are limited to a few inelastic tests with arbitrary cyclic load or deformations. Only two experiments, performed by Clough (1975) and Takanashi (1984), directly correlate the inelastic frame behavior to seismic excitations.

Past research and experience relates primarily to SCWB behavior, where the inelastic behavior is concentrated in the beams with little or no axial force. The inelastic behavior under cyclic loading of SCWB components is very good if the sections are compact with adequate lateral support. When these requirements are satisfied, full hysteresis curves with large amounts of energy dissipation are expected. Panel zone yielding has been observed during these experiments, and it is regarded as a very good source of energy dissipation during earthquakes. Panel zone yielding is neither WCSB or SCWB behavior, and it can be an additional limitation on the lateral resistance of the frame. Krawinkler (1978) proposed Eq. 3 as an estimate of panel zone strength. This estimate is consistent with research results, but there are no reliable and accepted methods for calculating the deflection of frames with panel zone yielding when this resistance is used. Tsai (1990) and Wang (1988) examined deformations and deflections due to panel zone deformation, but the models used in these studies are empirical and are based on specific test results.

Suzuki (1977) performed a number of small scale flexural tests on columns reflecting the inelastic behavior of WCSB joints. These results suggest that the hysteretic behavior can be either good or poor. Poor behavior results if the axial force or slenderness of the flanges or web are too large. Popov (1975) performed 6 tests on WCSB subassemblages, and these tests provide similar conclusions to those from the column tests. Popov suggests that the behavior of WCSB frames will be adequate if the axial force is kept below 50% of the yield force. This observation is based on the assumption that the inelastic response and ductility demand of SCWB and WCSB frames are similar. Takanashi (1984) performed a shaking table test on a small scale 3 story WCSB frame, and it collapsed on the shaking table during the test.

## ANALYTICAL INVESTIGATION

Extensive inelastic analyses were performed on a wide range of SCWB and comparable WCSB structures. These frames were 3, 8 and 20 stories, and were designed to the minimum UBC seismic zone 4 requirements by an experienced structural engineer. The frames used WCSB and SCWB joints at all locations, but some frames had partial systems with WCSB joints at specific locations. The building employed perimeter framing for the lateral resistance, and each frame was symmetric. Figure 1 shows a plan view of this structure and Fig. 2 shows an elevation with typical member sizes for 20 story WCSB and SCWB frames. The DRAIN-2D computer program [Kananaan and Powell (1973)] was used to analyze the inelastic response for a wide range of acceleration records.

Figure 3 shows the computed response of the first story of these frames due to the 1940 El Centro ground acceleration record. This record has a peak acceleration of approximately 0.33 g, but it is significantly less than an equivalent acceleration assumed for seismic zone 4. The dynamic response for the WCSB frame is somewhat larger than that of the SCWB frame, but both frames are well within acceptable drift limits. However, WCSB frames are very sensitive to the acceleration record used in the analysis and typically have much larger story drift than comparable SCWB frames. This is illustrated in a comparison between Figures 3 and 4. Figure 4 shows the computed response for the same frames with the unscaled Pacoima Dam acceleration record. The WCSB frame had story drifts which were well beyond

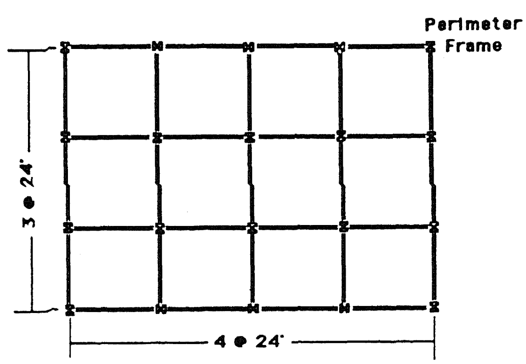


Fig. 1. Plan of Building

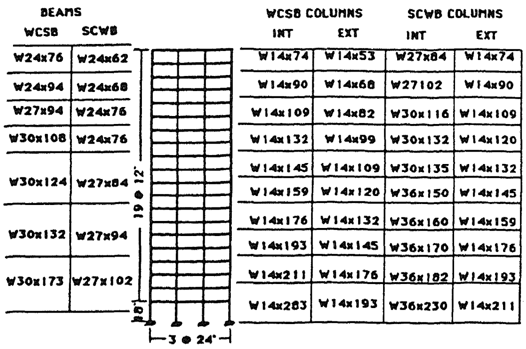


Fig. 2. Elevation of Lateral Load Frame and Member Sizes

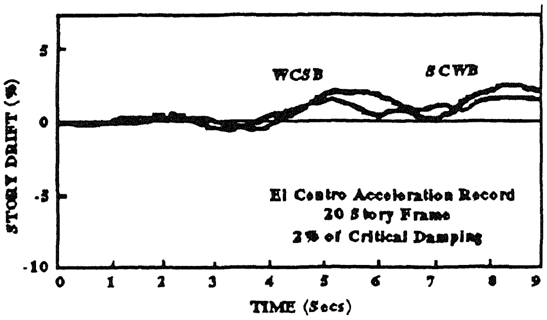


Fig. 3. Computed First Story Drift with the 1940 El Centro Acceleration Record

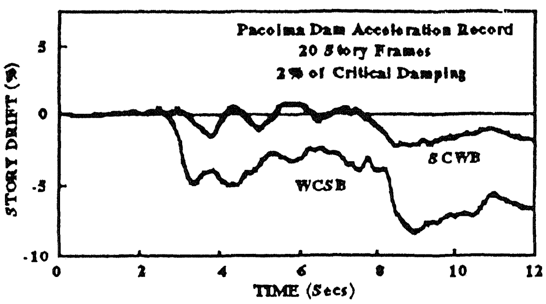


Fig. 4. Computed First Story Drift with 1971 Poicoma Dam Acceleration Record

acceptable limits and were approximately 3 times those noted for the SCWB frame. The WCSB and SCWB frames were designed for identical seismic loading, but WCSB frames were invariably stiffer and stronger than comparable SCWB frames because of the linear elastic design procedure. The small seismic design forces obtained by Equation 1, are applied to the frame with an allowable stress design method. The linear elastic column bending moments are invariably much larger at the base of the column than they are at the top of the first story column. Thus, the column has considerable reserve bending capacity at the top of the column in the WCSB frames when both ends develop plastic hinges during the seismic response. The reserve strength predicted in the WCSB frames resulted in less yielding in smaller earthquakes, and the SCWB frames sometimes had larger story drift and inelastic deformation under similar conditions.

The inelastic analyses considered flexural yielding only. The panel zones were assumed to be stiffened and strengthened to avoid deformation and yielding. If panel zone yielding had been included in the analysis, the performance of the WCSB frames may have been worse even for modest size earthquakes, because of the panel zone provisions noted earlier. However, there are no accurate theoretical models for predicting panel zone deformation.

It should be emphasized that all of the frames used in this study were minimum designs to the UBC provisions. They were relatively flexible frames with the minimum reserve strength. It is probable that the performance of both the WCSB and SCWB frames would have been improved, if they had additional strength and stiffness.

Analytical results suggest that the inelastic energy dissipation of WCSB frames are consistently concentrated in a critical single story rather than distributed over the height of the structure as with the SCWB system. This is because of the large story drifts noted in the first-story of the WCSB frame. While WCSB frames may have large story drift and concentration of inelastic deformation, there are ways of improving their performance. For instance performance is significantly improved if SCWB joints are used at some connections on each floor level of the WCSB frame. Improved performance was also noted when the seismic design forces were increased or the allowable design stresses were decreased for WCSB frames. Much more information regarding the results of the analysis are available from Schneider, Roeder, and Carpenter (1991).

## EXPERIMENTAL INVESTIGATION

Comparison of the computed behavior for some elements in the WCSB frame to results obtained in past experiments causes some concern. Analysis results indicate that WCSB elements may have ductility requirements of two to three times the demand predicted for SCWB frames because of the concentration of yielding. However, results from previous experiments on inelastic deformations in columns support no evidence that actual wide-flange steel sections have the necessary available ductility capacity to resist this predicted demand. Furthermore, these inelastic analyses do not include deterioration or failure of the element, and so additional experiments were performed to further evaluate this behavior. The experiments had to employ a realistic simulation of the true seismic response, and a pseudo-dynamic subassemblage test method was developed to achieve this objective. The test procedure is economical in that only a critical element of the structure was tested. It is realistic in that deformations applied to the test specimen are based on the dynamic properties of the structure and a specific ground acceleration record. The measured behavior of the test specimen is used to update dynamic properties of the structure for each time step of loading. Five subassemblages were tested by this method.

SC-B was a basic SCWB frame test specimen. The panel zone was stiffened and strengthened to prevent the web from yielding. Due to the factor of safety in allowable stress design, the true yield stress of the material, and the oversize of members in the design process, the predicted plastic frame strength had approximately 2.30 times the UBC design load. WC-B was the basic WCSB frame test, and it had a plastic capacity of 2.95 times the UBC load. WC-B had relatively stocky column webs and flanges compared to SC-B.

WC-PZ was a weak-column frame with the same member sizes as in the WC-B frame, but the panel zone strength was designed to satisfy the 1988 UBC minimum requirements. Its capacity was also 2.90 times the UBC design load. WC-ASP was a weak-column frame designed to study the influences of increased aspect ratios of the column flanges and web and minimum panel zone web requirements. It had the same column size as SC-B, and had a plastic strength capacity of 2.90 times the UBC design force. The above WCSB frames all had considerable reserve strength beyond the minimum required in a UBC design, because the true yield stress of the steel was considerably larger than the nominal yield stress. WC-$R_W$ was a WCSB designed to eliminate this reserve strength. The plastic capacity of this frame was 2.26 times the minimum UBC design load. This plastic capacity is nearly identically to that noted for SW-B.

All five frames were first tested with a pseudo-dynamic application of the 1979 Imperial Valley College acceleration record, since the dynamic analysis showed that this was a damaging acceleration record for both SCWB and WCSB frames. SC-B, WC-B, WC-ASP, and WC-PZ all survived this acceleration record very well. These three WCSB frames all had approximately 22% reserve strength above the minimum required by the UBC. They had larger story drifts and greater concentration of damage than SC-B, but they survived the earthquakes well. However, there was greater deterioration and visible deterioration to the columns with thinner webs and flanges as illustrated in Fig. 5. The UBC minimum panel zone provisions resulted in greater story drift than the comparable stiffened panel zone.

WC-$R_W$ collapsed under the applied loading during this first acceleration record. This collapse occurred despite the fact that it had nearly identical reserve strength to SC-B, because of the greater story drift and greater concentration of damage developed in WCSB frames. Figure 6 shows a comparison of the measured response of the first story of WC-$R_W$ and SC-B. The failure of WC-$R_W$ was ultimately precipitated by tearing of the column flange near the beam-column connection as illustrated in Fig. 7, because of the large drift and deformation.

The four surviving frames, SC-B, WC-B, WC-PZ, WC-ASP, were then subjected to 1.85 times 1940 El Centro. All four frames performed well during this second acceleration record, and they were subsequently subjected to a series of severe cyclic deformations. They all exhibited great ductility, but some deterioration was noted in later cycles.

The story drifts achieved with the WCSB frames

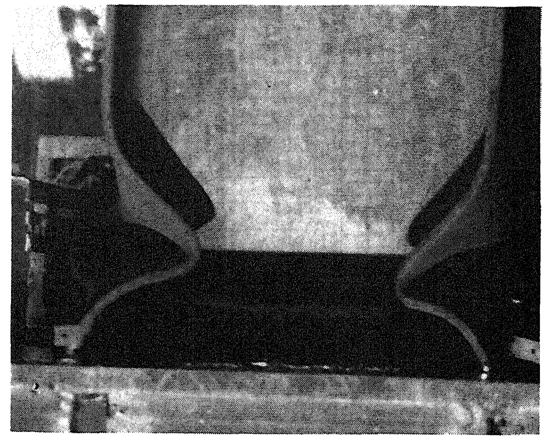

Fig. 5. Photo of Buckling with a Slender Web and Flange

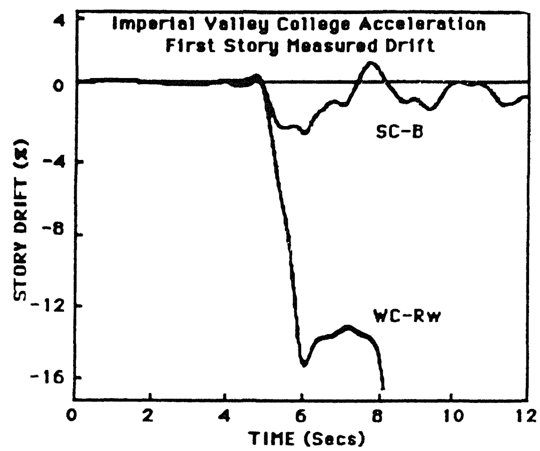


Fig. 6. Measured Story Drift for SC-B and WC-$R_W$

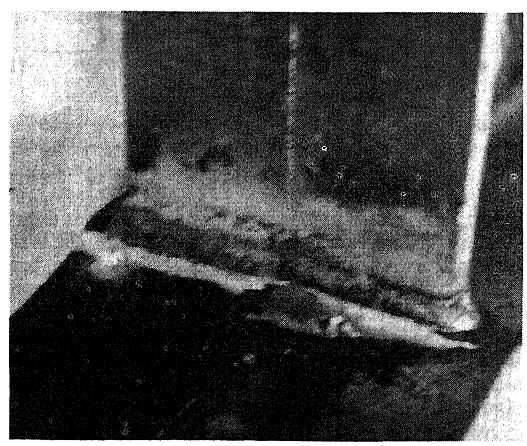

Fig. 7. Photo of Torn Column Flange for WC-$R_W$

were considerably larger than comparable SCWB frames as predicted in the analysis. The WCSB frames appear to have adequate ductility to sustain these larger ductility demands if the axial force in the column is kept relatively small, the webs and flanges are relatively stocky, and there is adequate reserve strength in the structure. The ability of the WCSB system to survive the required inelastic deformation is less clear if the axial force becomes large or if the slenderness of the web and flange is too large. This is a matter of some concern, since there are presently no limits on web and flange slenderness in UBC or other similar seismic design provisions. Additional information regarding these tests is available elsewhere [Schneider, Roeder and Carpenter (1991)].

## CONCLUSIONS

A number of conclusions can be drawn from the DRAIN-2D inelastic analysis:

1) Some acceleration records cause more yielding and plastic deformation in both WCSB and SCWB frames than others. WCSB frames invariably have much larger story drift and greater concentration of plastic deformation than comparable SCWB frames when subjected to these acceleration records.

2) All of the frames were minimum UBC designs. The 80% limitation on minimum seismic design forces controlled the seismic design forces, and the forces associated with the true dynamic period of the bare steel frame were used for drift limits. Both WCSB and SCWB frames had larger story drifts than the 2% commonly postulated with code design.

3) Increased design forces and tighter drift limits resulted in improved performance of WCSB frames. WCSB frames that remain elastic throughout the earthquake sometimes had smaller deflections than comparable SCWB frames, since they are inherently stronger in resisting lateral loads than comparable SCWB frames. However, the inelastic performance of WCSB frames was clearly inferior to SCWB frames unless the WCSB frame was designed for seismic loads which were 33% to 100% larger than that used for comparable SCWB frames.

The pseudo-dynamic test procedure worked well. Five frames were tested, and several conclusions may be drawn from this series of tests:

1) Four of the five frames survived the intense seismic ground motion. This reflects the advantages of ductile moment-resisting steel frames in seismically active regions.

2) Inelastic displacement levels for both frame types were larger than the 2% drift level generally postulated for frames designed by the minimum UBC provisions. Further, the ductility demand was distributed in the two structural systems as suggested in the inelastic analysis.

3) Axial loads prevented initial buckles from recovering upon load reversal and accentuated flange and web buckling in the column. This resulted in an incremental increase in plastic axial deformations upon subsequent cyclic rotational demands. Column shortening was also sensitive to the slenderness of the flanges and web of the cross-section. Stable hysteretic behavior was exhibited at an axial load ratio of 0.20 $P/P_y$. However, frame strength began to deteriorate rapidly upon an axial load increased to 0.30 $P/P_y$.

4) Hysteretic behavior of the panel zone was very stable. The strength increased without deterioration, well into the inelastic range, but the minimum 1988 UBC panel zone requirement had mixed results on frame performance. Panel zone deformation frequently increased story drift, interacted with WCSB behavior, and reduced the reserve strength of the frame. Panel zone shear distortions may adversely effect the performance of the moment connection of the beam element.

5) Slenderness of the flanges and web had significant influence on the hysteretic behavior of the frame. Slender flanges and web led to more rapid deterioration in the hysteretic behavior than for stockier sections, particularly in the presence of axial load. Current compact section requirements in the 1988 UBC apply only to the beam elements in a WCSB frame. Results of this research suggest that the requirement should be extended to include any column which may yield during a seismic event.

6) Reserve strength of the frame is important to seismic performance. WCSB frames have exhibited satisfactory seismic performance under some specific conditions. Predicted inelastic floor displacements must be kept under 5% story drift and the load ratio under 0.20 $P/P_y$. Weak-column frames must have about a 25% extra strength to achieve a 5% story drift for some acceleration records. If local damage to the wide-flange sections is critical, the aspect ratio of the column cross-section should satisfy $b_f/2t_f \leq 5.7$ and $d/t_w \leq 35$.

These results are based on 1/2 scale tests. Smaller members may offer more ductility and better performance than the full-scale counterpart. Welded connections were probably of much higher quality than obtained in most standard construction.

Additional research is needed to precisely establish the design limits for acceptable WCSB frame behavior. This study should include parameters such as aspect ratios of the flange and web, axial load ratios, and number of deformation cycles. Inadvertent WCSB action due to composite floor slabs should also be considered. The influence of panel zone rigidity and deformation on overall frame performance should be investigated more thoroughly.

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