Panel Zone Yielding in Steel Moment Connections

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ABSTRACT

The effect of panel zone yielding on the behavior of fully-restrained steel moment connections is investigated. The evolution of seismic design provisions for steel panel zones is discussed and experimental evidence is presented that suggests that an excessively weak panel zone could be detrimental to connection behavior. The observed behavior is explained through detailed inelastic finite element analyses. Based on the experimental and analytical evidence, modifications to existing seismic provisions for steel panel zones are proposed that make the provisions conceptually more transparent.

INTRODUCTION

As a result of extensive research conducted in the aftermath of the Northridge earthquake there appears to be consensus among structural engineers that a large number of factors contributed to the observed connection fractures (FEMA, 1997). These factors can be broadly classified into three categories related to detailing, welding and design practices that were prevalent prior to the earthquake.

Poor detailing practices include details that resulted in the development of large stress concentrations, excessive local ductility demands and high tri-axial restraint at the beam-to-column interface. Welding practices contributing to poor performance include the common use of low toughness weld metal and insufficient quality control. Design practices that appear to have played a role in the Northridge fractures include:

1. The practice whereby seismic resistance is concentrated in a few frame bays, which can result in significantly larger members and connections than had previously been tested.
2. Incorrect assumptions by designers regarding the yield strength of steel, for example, the use of dual-certified steel or steel with an actual yield strength that is higher than the specified nominal yield strength.
3. The development and use of design provisions which can result in excessively weak panel zones.

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The second and third design-related factors are closely associated, and together they could have played an important role in the Northridge failures. While limited panel zone yielding in itself is thought to be a benign effect that can be beneficial for dissipating seismic energy, excessive panel zone distortion can lead to local kinks in the column flanges, which can contribute to premature fracture at the beam-to-column interface (Krawinkler, 1978 and Popov, 1987). The fact that the actual yield strength of beams is greater than the specified nominal yield strength further contributes to weak panel zone behavior. Over strength beams create greater shear demands in the panel zone region.

The overall goal of this paper is to examine the effect of panel zone yielding on the behavior of fully-restrained steel connections. Specific objectives are:

1. Discuss evolution of seismic design provisions for steel panel zones.
2. Survey currently existing test results for evidence that panel zone yielding could be detrimental to connection ductility.
3. Quantify the effect of panel zone yielding on the potential for fracture of connections.
4. Use developed information to critique currently recommended seismic provisions for panel zone design and suggest modifications that make the provisions conceptually more transparent.

SEISMIC DESIGN PROVISIONS FOR PANEL ZONE

Seismic design provisions for the panel zones have seen significant changes in the past three decades as information regarding the cyclic behavior of the panel region has accumulated. As discussed in Popov (1987), there are essentially three schools of thought for panel zone design. The first approach, referred to hereafter as the strong panel zone approach, requires the panel zone to remain elastic during seismic loading. Calculations made according to this approach usually result in the specification of doubler plates. In addition to being uneconomical, doubler plates may require heavy welding that can result in distortion and residual stresses. Large welds also create a large heat-affected zone that increases the risk for brittle behavior in the connection region.

Based on test results that suggested that panel zones are inherently ductile elements, an opposite design philosophy has been advocated in Kawano (1984) for low-rise steel frames. In this approach, termed weak panel zone design, the panel zone is proportioned so that it absorbs most of the inelastic deformations in the structure during seismic loading. This philosophy may adversely affect connection ductility as is evidenced by the information presented herein, and is counter to current thinking. There is growing consensus among structural engineers that excessive panel zone deformation may be detrimental to overall connection ductility (FEMA, 1997).

The third design philosophy, which is a compromise between the above two approaches, requires the panel zone to participate along with the beams in seismic energy dissipation. This methodology, termed "balanced panel zone design," forms the conceptual basis of current seismic provisions. Following is a discussion of the different design philosophies adopted by various seismic codes.

Early Design Specifications

The basic equation for determining the design shear demand in the panel zone can be expressed as follows (SEAOC, 1976):

$$V_{des} = \frac{M_l}{d_l - t_{fl}} + \frac{M_r}{d_r - t_{fr}} - V_{col} \tag{1}$$

where M_l, M_r are the moments, d_l, d_r are the beam depths, and t_{fl} and t_{fr} are the flange thickness on the left and right of the connection respectively. V_{col} is the shear in the column, such that $V_{col} = (M_l + M_r) / H$ where H is the story height.

The 1978 *AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings* (AISC, 1978) specifies that the allowable shear force that can be transferred through the joint is given by:

$$V_{max} = 0.4F_y d_c t \tag{2}$$

for working stress design. In Equation 2, F_y is the yield strength, d_c is the column depth, and t is the panel zone thickness. When seismic effects contribute to the design shear forces, the allowable shear stresses may be increased by 33 percent. The panel zone shear strength in this case is:

$$V_{max} = 0.53F_y d_c t \tag{3}$$

When plastic design is considered, the capacity of the panel zone is:

$$V_{max} = 0.55F_y d_c t \tag{4}$$

The 1976 SEAOC Commentary (SEAOC, 1976) recommends that the panel zone should be designed to resist the shear forces generated by the beams framing into the joint when the beams reach their plastic strength. The shear demand can therefore be calculated from Equation 1 with the plastic moment capacities substituted for M_l and M_r. Based on a simple calculation, Popov (Popov, 1987) concluded that the SEAOC provisions can result in a panel zone thickness that is 23 percent in excess of that calculated using the 1978 AISC allowable stress design specifications. The assumption involved in this calculation is that seismic design is based on the beams reaching their plastic strength. However, in many instances the design is governed by drift limitations, resulting in deeper beams with larger plastic strength than specified by the seismic strength requirements. Hence the SEAOC (SEAOC, 1976) procedure could potentially result in larger panel zone thicknesses and is considered to be a strong panel zone design approach.

Pre-Northridge Design Specifications

In an attempt to encourage balanced panel zone behavior, the 1987 SEAOC Commentary (SEAOC, 1987) and the 1988 UBC (UBC, 1988) significantly reduced the demand on the panel zone compared to the 1976 SEAOC (SEAOC, 1976). Instead of using the plastic strength of the framing beams to determine the shear demand, the design shear force is calculated from the gravity moments plus 1.85 times the specified seismic moments. Furthermore, the capacity of the panel zone is increased such that:

$$V_n = 0.55F_y t_c d_c \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_c}\right) \tag{5}$$

where, F_y, t_c, and d_c, are as defined previously; t_{cf} is the column flange thickness, b_{cf} is the column flange width, and d_b is the depth of the beam.

The term in brackets accounts for the contribution of the column flanges. The equation was derived assuming that the sides of the panel zone remain straight after panel zone deformation, and that design strength is reached at a panel zone plastic distortion of $4\delta_y$, where δ_y is the yield panel zone distortion (Krawinkler, 1978). As a result of the panel zone sides remaining straight, plastic hinges or "kinks" form in the column flanges (Figure 1). El-Tawil, Mikesell, Vidarsson, and Kunnath (1999) presents finite element analysis results that show that Equation 5 reasonably predicts the strength of panel zones with different beam depths, but slightly overestimates the strength of connections with very thick column flanges.

Although the moments caused by an earthquake on either side of an interior joint have the same sense, the moments caused by gravity loading have opposite senses, and tend to decrease the total moment acting on a connection. Referring to Figure 2, $M_l = M_{sl} + M_{gl}$ and $M_r = M_{sr} - M_{gr}$, where M_{sl} and M_{sr} are the seismic moments and M_{gl} and M_{gr} are the gravity moments on the left and right of the joint respectively. Assuming that M_l reaches the plastic moment capacity, M_{pl}, then $M_{pl} = M_{sl} + M_{gl}$. Furthermore, assuming that the plastic moment strength on the right side is equal to the right gravity moment plus the right seismic moment, then $M_{pr} = M_{sr} + M_{gr}$. The moment on the right side can then be expressed as:

$$M_r = M_{pr} - 2M_{gr} \tag{6}$$

Popov (1987) suggests that in most situations it is reasonably accurate to estimate the gravity moments using the fixed end moments, $wL^2/12$. Pre-Northridge and current seismic provisions (AISC, 1997) estimate the gravity moment reduction expressed by Equation 6 to be 20 percent of plastic moment capacities of the framing beams. In other words, the shear demand is capped at the force resulting from 80 percent of the plastic moment capacities of the framing beams where the 80 percent factor accounts for the presence of gravity moments (AISC, 1997).

The 20 percent reduction in the plastic moment capacity can significantly overestimate the reduction due to gravity moments in pre-Northridge frame designs in which seismic resistance is provided by a few perimeter moment frames (Figure 3). In a study reported in Chi, El-Tawil, Deierlein, and Abel (1998) (Figure 3), the gravity moment reduction is 2.7 percent in the first floor and 5.4 percent in the top floor of a 17-story building that was damaged in Northridge. System studies reported in Tsai and Popov (1988) and Wang (1988), in which steel moment frames are designed according to the 1987 SEAOC (SEAOC, 1987), indicate that panel zones could be subjected to demands that significantly exceed a plastic distortion of $4\delta_y$ as assumed in the derivation of Equation 5. Therefore, both the 1987 SEAOC (SEAOC, 1987) and the 1988 UBC (UBC, 1988) could create conditions that allow weak panel zone behavior to occur.

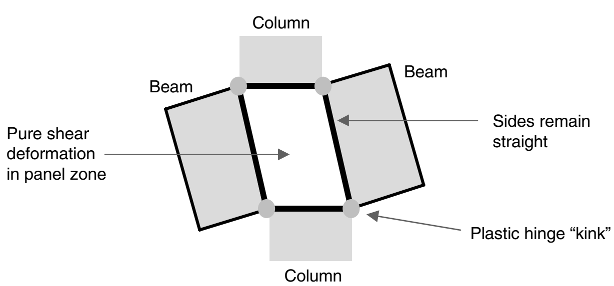


Fig. 1. Assumptions for derivation of Equation 5.

Current Design Specifications

In the aftermath of the Northridge earthquake, excessive panel zone yielding was widely discussed as a possible reason for the observed connection failures. Not only did pre-Northridge codes specify provisions that resulted in weaker panel zones than before, but also new trends in the production of ASTM A36/A36M steel resulted in actual yield strengths that exceeded the nominal capacity (Frank, 1997). Therefore, ASTM A36/A36M beams designed using nominal steel properties could be stronger than assumed, and could deliver larger shear forces to the panel zones. Overloading the panel zones in this manner could have resulted in significantly greater inelastic deformations than intended by design specifications, and as discussed in this paper, may have played a role in the observed failures.

To more accurately estimate the maximum shear forces supplied to the panel zone by overstrength beams, FEMA 267 Recommended Provisions (FEMA, 1995) increased the maximum design shear demands. This was achieved by multiplying the plastic moment capacity of the beams by a factor, β, that accounts for strain hardening and overstrength. The factor, β, is not utilized when calculating the capacity of the panel zone. Other than this change, the

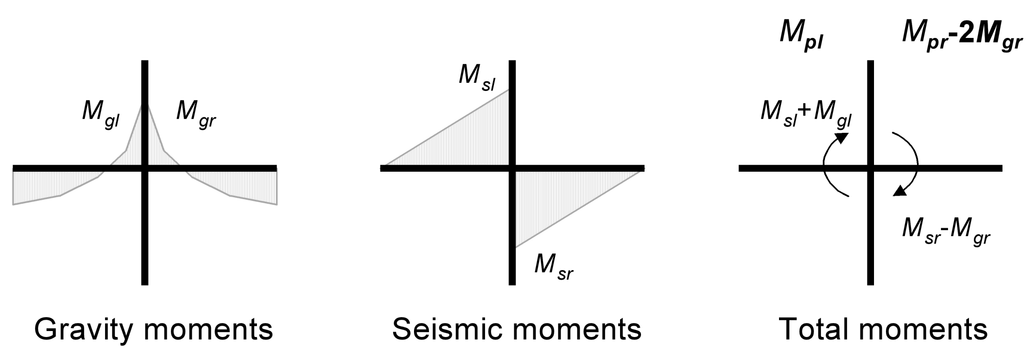


Fig. 2. Gravity and seismic moments on interior connection.

remainder of the panel zone provisions are identical to those in the 1988 UBC (UBC, 1988).

The panel zone design shear forces were further increased in a supplement to FEMA 267 published in 1997 (FEMA, 1997). In these provisions, the clause that specified that the design shear force should be calculated from the gravity moments plus 1.85 times the specified seismic moments was removed. Instead, the seismic demands are computed from 80 percent of the plastic moments of the beams framing into the connection. This was previously the limiting condition for computing seismic demands.

The most recent specifications dealing with panel zone design are the AISC Seismic Provisions (AISC, 1997) and the NEHRP Recommended Provisions (FEMA, 1997). These specifications, which are identical and LRFD-based, compute the design shear strength of the panel zone from an equation similar to Equation 5, in which the 0.55 is rounded to 0.6. The shear demands are calculated from the following LRFD equation: $1.2D + 0.5L + \Omega_o Q_E$, where Ω_o is the system overstrength factor and is taken as 3 for special moment resisting frames and Q_E are the horizontal earthquake effects. As with other specifications, the shear demands are not required to exceed those calculated from 80 percent of the plastic moment capacity of the framing beams taking into account material overstrength and strain hardening. The resistance factor, ϕ, is taken as 0.75.

A system overstrength factor of 3 is usually reserved for designing elements that can fail in an undesirable manner when subjected to large demands (especially force demands), while $\phi = 0.75$ is generally applied to elements failing by fracture or leading to fracture failure. Nevertheless, the commentaries for both documents do not specifically indicate that the panel zone is considered to be a fracture prone element with a brittle mode of failure. However, it is instructive to note the codes' added conservatism regarding panel zone design in adopting these factors.

EFFECT OF PANEL ZONE YIELDING ON POTENTIAL FOR FRACTURE — SURVEY OF EXPERIMENTAL DATA

Roeder and Foutch (Roeder and Foutch, 1996) examined existing test results with the objective of investigating the effect of a number of key parameters on connection ductility. They concluded that beam depth, beam flange thickness, beam length to depth ratio and panel zone yielding strongly affected beam flexural ductility. In particular, it was shown that beams connected to joints with panel zone yielding had significantly lower flexural ductility compared to beams in joints with strong panel zones. Since beam flexural ductility was used in the investigation, it was not clear what effect panel zone yielding had on the overall connection ductility and whether panel zone yielding is truly detrimental to overall connection behavior. In a subsequent report, Roeder (Roeder, 1996) suggested that the maximum connection plastic rotation is indeed reduced in connections with panel zone yielding. However, the database from which the conclusion was drawn included connection failure modes that could not have been influenced by panel zone distortion.

To further investigate the role of panel zone yielding in connection failures, the database of SAC test results (FEMA, 1997) is examined. The SAC database is a com-

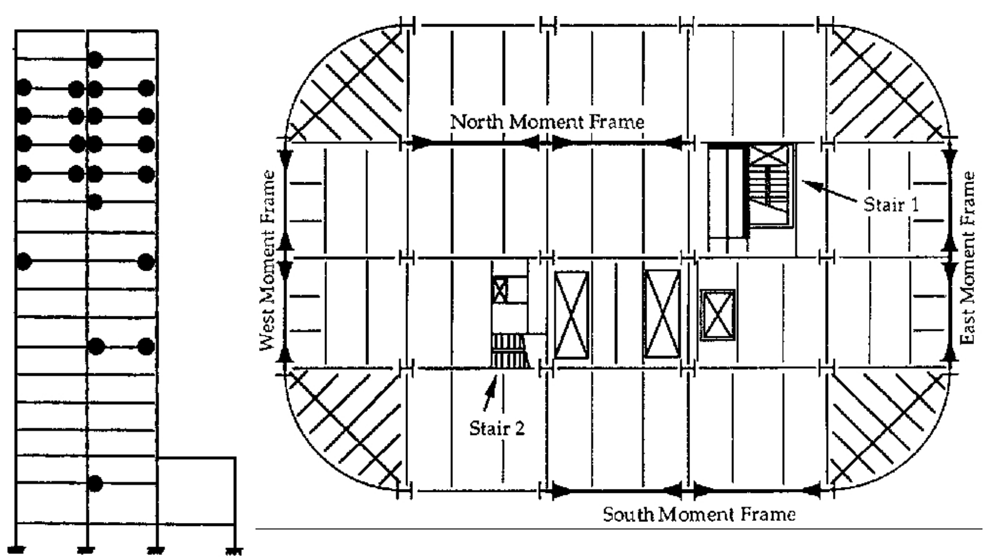


Fig. 3. Structural system for 17-story building damaged in Northridge (Chi et al. 1997). Note the large percentage of exterior (one-sided) connections.

pendium of tests conducted during Phase I of the SAC Steel Project and contains data pertaining to pre-Northridge type connections and repaired or strengthened connections. The advantage of looking at the SAC database (FEMA, 1997) alone is that all the connection experiments have more or less similar geometric and material properties. This eliminates the variability introduced by these factors. To further ensure the relevance of the test data, only the connections in which failure occurred at the weld-column interface are considered. Connections with failures at the weld-beam interface, the beam heat-affected zone and the beam flange were excluded from the database since these locations were considered to be sufficiently distant from the column so that they were not affected by panel zone distortion. Discussions in the following section suggest that the stress conditions at the weld-column interface were not significantly affected by panel zone distortions up to 0.005 rad. For the geometries being considered, this corresponds to a plastic panel zone distortion of approximately 0.0025 rad. Hence test results in which connection failure occurred before a panel zone plastic distortion of 0.0025 rad are not included in the database. These specimens failed before the panel zone distortions became large enough to contribute to the failure. Repaired connections in which welds or fractured regions were removed and replaced and which satisfied the above criteria are included in the database. However, connections with other types of repairs or strengthening strategies are excluded because they introduce additional parameters related to the modified geometry.

Of the 30 tests summarized in the SAC database (FEMA, 1997), only 15 connections met the criteria set above. The data for these connections were processed to calculate six quantities: maximum connection plastic rotation, beam plastic rotation, and panel zone plastic distortion, as well as cumulative connection plastic rotation, beam plastic rotation, and panel zone plastic distortion. By assuming that the contribution of column plastic flexural deformations to the beam tip deflections is negligible, information not reported in FEMA (1997) could be calculated. Test observations confirm that although columns in the tested subassemblages deformed measurably, they did not yield significantly in flexure. Hence, the beam plastic rotation is related to the connection plastic rotation and panel zone plastic shear distortion through the following equation:

$$\theta_{t,p} = \theta_{b,p}\left(\frac{l_b}{l_t}\right) + \gamma_p\left(\frac{l_b}{l_t}\right) - \gamma_p\left(\frac{d_b}{l_c}\right) \tag{7}$$

where:

$\theta_{t,p}$ = connection plastic rotation defined as the plastic tip deflection divided by l_t.

$\theta_{b,p}$ = beam plastic rotation, defined as the plastic tip deflection divided by the beam length.

γ_p = average plastic panel zone shear distortion.

l_b = beam length.

d_b = beam depth.

l_c = column length.

l_t = beam length plus one half of the column depth.

Table 1 shows a summary of all compiled results. The ratios of panel zone strength to beam flexural strength (P_z/P_b) are also shown in the table. Component strengths P_z and P_b are defined and discussed in the following section. The compiled data mostly covers connections with relatively weak panel zones in which the ratio of panel zone strength to beam strength ranges from 0.68 to 1.01. The cumulative beam plastic rotation (BPR) and the cumulative connection plastic rotation (CPR) are plotted versus the P_z/P_b ratio in Figures 4 and 5 respectively. Data for specimens EERC and UCSD (W30×99 beams and W14×176 columns) are plotted using solid triangles while data for specimens UTA and UCB (W36×150 beams and W14×257 columns) are plotted using solid circles. The distinction is made so that trends related to connection size could be investigated. A hollow circle around a data point indicates repaired connections.

In spite of significant scatter in the results, the lower beam flexural ductility for connections with weak panel zones is clearly evident in Figure 4. This is to be expected since weak panel zones absorb most of the inelastic deformations relieving the beams from large ductility demands. Scatter in the overall connection plastic rotation in Figure 5 is also large. The large scatter in the figure suggests that other factors besides panel zone yielding are contributing to the connection failures. Nevertheless, there is a definite trend indicating that connections with weak panel zones have lower cumulative connection plastic rotation compared to connections with strong panel zones. The trend appears to be similar in both connections sizes considered, and does not seem to be much affected by the repair procedures. To explore the reasons behind this trend, three-dimensional nonlinear finite element analyses are conducted.

ANALYTICAL STUDY

In order to investigate the effects of panel zone yielding on the potential for fracture of welded-bolted steel connections, two subassemblages from Series CWT analyzed in a previous investigation (El-Tawil et al., 1999) are chosen for additional analyses. The geometric characteristics of Series CWT configurations in El-Tawil et al. (1999) are derived from Specimen PN3 (Popov, Blondet, Stepanov, and Stojadinovic, 1996) tested during Phase I of the SAC Steel Project. Subassemblage PN3 consists of a W36×150 beam connected to a W14×257 column. The web is connected through a bolted shear tab with supplementary welding, while the flanges are joined using field groove welds. The

Table 1.
Summary of Test Results for Connection Specimens

	Specimen	P_z/P_b	$\theta_{t,max}$*	$\theta_{t,cum}$*	$\theta_{b,max}$*	$\theta_{b,cum}$*	γ_{max}*	γ_{cum}*
W30×99 beams W14×76 columns	EERC – PN1	0.80	0.85	9.44	0.40	4.40	0.65	7.25
	EERC – PN2	0.90	0.89	11.44	0.40	4.70	0.70	9.60
	EERC – PN3	0.97	1.78	24.21	1.00	12.50	1.15	17.00
	UCSD – 1	0.90	0.79	3.55	0.38	1.37	0.60	3.10
	UCSD - 2	0.90	2.05	19.70	1.40	9.35	1.00	14.90
	UCSD - 3	0.90	2.01	19.70	1.20	9.43	1.20	14.80
	EERC - RN1	0.80	0.86	5.11	0.38	1.63	0.70	4.90
	EERC - RN1A	0.80	1.08	10.53	0.60	4.20	0.70	9.00
W36×150 beams W14×257 columns	UTA - 2	0.99	0.60	2.50	0.38	1.50	0.35	1.20
	UCB - PN1	0.68	0.88	6.70	0.33	2.11	0.91	6.90
	UCB - PN2	0.68	0.34	1.75	0.10	0.84	0.42	1.40
	UCB - PN3	1.01	1.07	11.25	0.81	8.57	0.49	4.65
	UTA - 2R	0.99	0.50	3.50	0.22	2.00	0.43	2.80
	UCB - RN1	0.68	1.55	17.70	0.41	4.51	1.70	19.70
	UCB - RN3	1.01	1.61	23.63	0.98	14.43	1.00	14.70

$\theta_{t,max}$ = maximum total plastic rotation (TPR)
$\theta_{t,cum}$ = cumulative TPR
$\theta_{b,max}$ = maximum beam plastic rotation (BPR)
$\theta_{b,cum}$ = cumulative BPR
γ_{max} = maximum panel zone plastic distortion (PZPD)
γ_{cum} = cumulative PZPD
* Note: θ and γ in rad × 100

beam flanges and groove welds are 24 mm thick. Continuity plates (13 × 152 mm) are located at both sides of the column, and the backing bar was left in place during testing (Figures 6 and 7).

The main variable in Series CWT in El-Tawil et al. (1999) is the column web thickness, which is varied to study the effect of panel zone yielding on connection behavior. The column web thickness is 21 mm and 56 mm in specimens CWT2 and CWT5 respectively. The two configurations chosen for additional analyses are denoted CWT2-W and CWT5-W. The specimens are identical to CWT2 and CWT5 in El-Tawil et al. (1999) except for the addition of a 10 mm reinforcing fillet weld as recommended by FEMA 267 (FEMA, 1995). Specimen CWT2-W is intended to represent connections with weak panel zones while CWT5-W is intended to represent the behavior of specimens with relatively strong panel zones.

The beams of both configurations are assumed to have a yield strength of 485 MPa (70 ksi), which represents an upper limit for the yield strength of commercially available

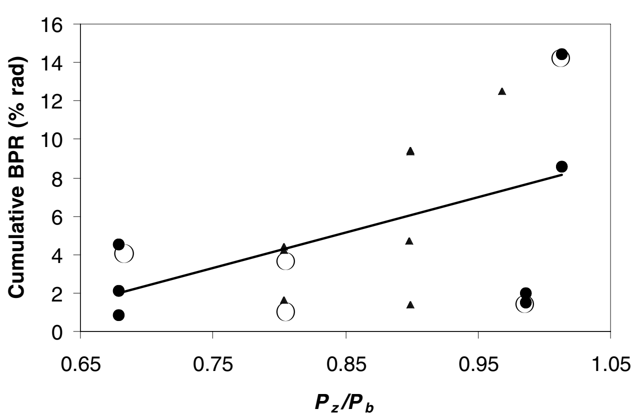


Fig. 4. Cumulative beam plastic roations vs. panel zone strength for SAC connections.

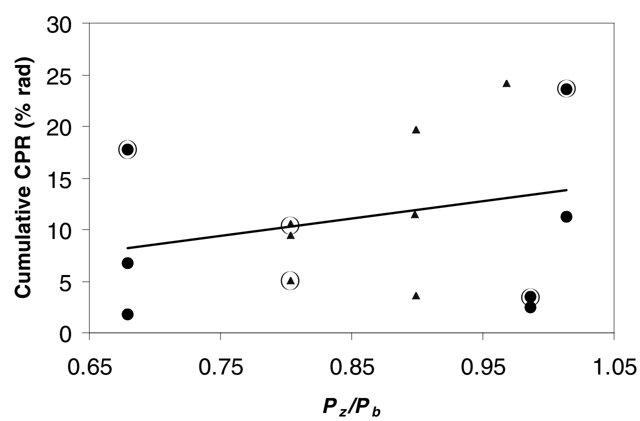


Fig. 5. Cumulative connection plastic rotation vs. panel zone strength for SAC connections.

ASTM A36/A36M steel (Frank, 1997). The welds are assumed to have the same properties as the beam material. The columns have a yield strength of 310 MPa (45 ksi) which is a lower limit for commercially available ASTM A572/A572M Gr. 50 (345) steel (Frank, 1997). These material properties represent a plausible situation that would result in severe demands on the panel zone region of the configurations being studied.

Shown in Table 2 are the tip loads required to cause the analysis configurations to reach their panel zone strength (P_z), beam strength (P_b), and column strength for pure bending (P_c). The panel zone strength is calculated from Equation 4 and hence does not recognize the additional strength reflected in Equation 5. These loads, as well as the ratios P_z/P_b and P_c/P_b give an indication of where inelastic deformation is expected to occur, and are useful in categorizing and evaluating results. It is clear from Table 2 that most of the inelastic deformation is expected to occur in the panel zone in specimen CWT2-W. Specimen CWT5-W, on the other hand, has a much stronger panel zone, and is expected to have significant beam plastification.

Finite Element Model

The finite element model utilized in this study is comprised of a mixture of 4-node shell and 8-node brick reduced integration elements. The part comprised of brick elements is localized around the intersection between the beam bottom flange and the column, as shown in Figure 8. A graded shell element mesh is used to model the remainder of the subassemblage. Multipoint constraints are used to enforce compatibility between the different types of finite elements. The shell elements employed have 5 degrees of freedom (DOFs) per node, of which 3 are displacement DOFs and 2 are out-of-plane rotational DOFs. Brick elements, on the other hand, have 3 DOFs per node. The multipoint constraints employed ensure that the rotation of a line of brick element nodes to which a shell element node is connected corresponds to the rotation of the shell node. The portion of the shear tab in contact with the beam web is considered to be completely monolithic with the beam web. The analyses account for material nonlinearities through classical metal plasticity theory based on the Von Mises yield criterion and account for the effects of strain hardening. Geometric nonlinearities are accounted for through a small strain, large displacement formulation. Details of how material and geometric nonlinearities were implemented, as well as elastic and inelastic convergence studies and verification exercises are reported in El-Tawil, Mikesell, Vidarsson and Kunnath (1998). The analyses are conducted using the computer program ABAQUS (Hibbit, Karlsson and Sorenson, 1996).

Performance Indicators

Since cracks are not explicitly modeled in the finite element model, the principal stresses are used as an indicator of the potential for brittle fracture. If a crack or some other flaw exists, high principal stresses will result in large stress intensity factors at the crack tips, which increase the potential for brittle fracture. Brittle fracture occurs abruptly and is not accompanied by significant global plastic deformations. Alternatively, a high triaxial tensile stress can lead to a reduction in ductility as described by LeMaitre (1996) and

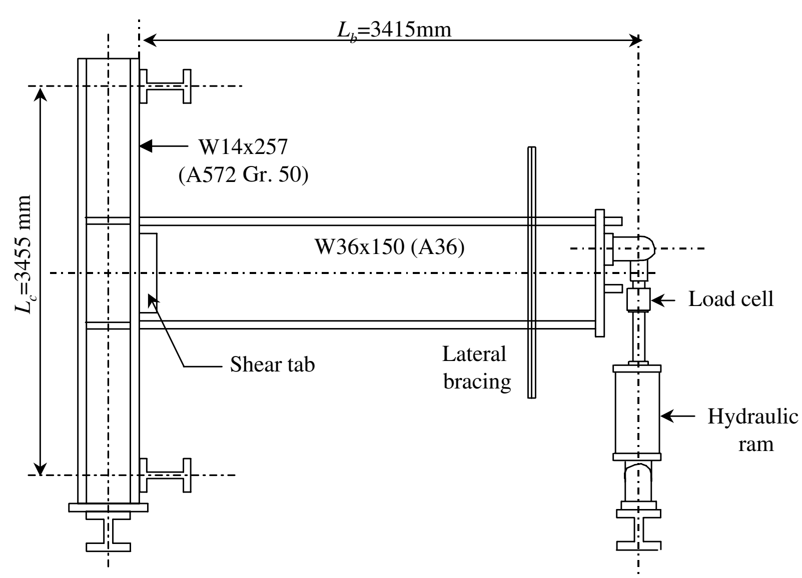


Fig. 6. Test setup for specimen PN3.

Table 2. Properties of Subassemblages CWT2-W and CWT5-W									
Member	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	P_b (kN)	P_z (kN)	P_c (kN)	P_z/P_b	P_c/P_b
CWT2-W (Column)	416	21	406	48	–	–	–	–	–
CWT2-W (Beam)	911	16	304	24	1330	498	1797	0.38	1.35
CWT5-W (Column)	416	56	406	48	–	–	–	–	–
CWT5-W (Beam)	911	16	304	24	1330	1372	2010	1.04	1.51

others. Under such conditions, high tensile triaxial stresses cause rapid damage accumulation in metals through micro-void nucleation and coalescence. Such a process is known as ductile fracture initiation, and is usually accompanied by plastic deformations prior to fracture. A quantity termed the rupture index is computed to determine the potential for ductile fracture.

The ratio between the hydrostatic stress and the Mises stress ($\sigma_m / \bar{\sigma}$) is known as the stress triaxiality ratio. This ratio is an important quantity when considering ductile rupture of metals. High triaxiality ($0.75 < \sigma_m / \bar{\sigma} < 1.5$) can cause a large reduction in the rupture strain of metals. Very high triaxiality ($\sigma_m / \bar{\sigma} > 1.5$) can result in brittle behavior (Lemaitre, 1996 and Barsom and Rolfe, 1987). A crude, albeit effective, criterion for calculating the strain at ductile fracture initiation is given in Hancock and Mackenzie (1976):

$$\varepsilon_f = a \exp(-1.5 \frac{\sigma_m}{\bar{\sigma}}) \tag{8}$$

where:

σ_m = hydrostatic stress

$\bar{\sigma}$ = Mises stress

ε_f = failure strain

a = a material constant

The rupture index is defined in this work as the ratio between the plastic equivalent strain normalized by the yield strain and the ductile fracture strain (calculated using Equation 8) multiplied by the material constant a, such that:

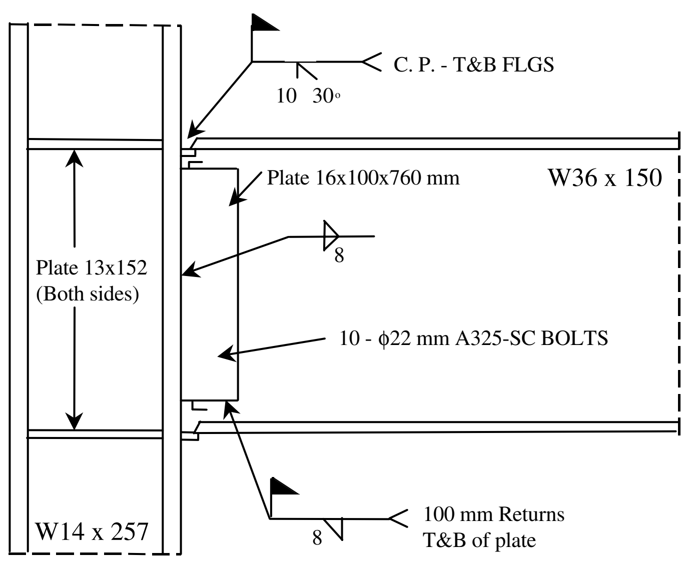
$$\text{Rupture Index (RI)} = a \frac{\varepsilon_p / \varepsilon_y}{\varepsilon_f} = \frac{\varepsilon_p / \varepsilon_y}{\exp\left(-1.5 \frac{\sigma_m}{\bar{\sigma}}\right)} \tag{9}$$

where:

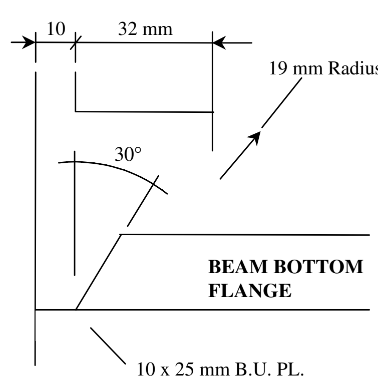
ε_p = the plastic equivalent strain

ε_y = the yield strain of steel

Equation 8 can be used to compare between the potential for ductile fracture of two configurations by comparing between the rupture index at critical points. This exercise has a number of limitations that must be appreciated (Hancock and Mackenzie, 1976). A ductile fracture criterion should involve a certain minimum characteristic length that represents the fracture process, which is not represent-



(a) specimen PN3



(b) details of lower access hole

Fig. 7. Details of connection specimen PN3.

ed in this methodology. Further, the failure criterion depends on the direction of rolling, initial imperfections in the steel, and the strain at which ductile damage starts to accumulate, all of which are not accounted for in the above equation. However, this method presents a convenient way to compare between different analyses, keeping in mind the above limitations. Research reported in Hancock and Mackenzie (1976) shows that Equation 8 is accurate for three different types of steel.

Discussion of Analysis Results

Analyses of both configurations are conducted up to a total plastic rotation (TPR) $\theta_{t,p}$ = 0.03 rad. The TPR is split up into three components: column plastic rotation, panel zone plastic distortion (PZPD), and beam plastic rotation (BPR). These represent the contributions of the column, panel zone, and beam components to the connection plastic rotation (El-Tawil et al., 1998). The rupture index and the maximum principal stress are plotted versus the panel zone distortion in Figures 9 and 10 respectively. Both quantities are sampled at two different locations at the weld column interface. The first position is located at an integration point at the middle of the weld-column interface, 6 mm into the column and 6 mm above the lower surface of the beam bottom flange. This location is termed "column location" hereafter. The second position is located at an integration point at the middle of the weld-column interface, 6 mm into the weld, and 6 mm above the weld bottom skin. This location is termed "weld location." The position of the sampling points can be roughly seen in Figures 9 and 10.

A number of observations can be made from Figures 9 and 10:

- At small panel zone distortions (< 0.005 rad), the principal stresses are lower in CWT2-W (weak panel zone) than in CWT5-W (strong panel zone).
- The principal stresses appear to level off at the yield stress level in the strong panel zone case. However, the principal stress keeps getting larger with increasing panel zone distortion in the weak panel zone case, reaching 160 percent of yield at the column location at a TPR of 0.03 (Figure 9a).
- At a TPR of 0.03, the rupture index at the column location (Figure 10a) is significantly higher in CWT2-W compared to CWT5-W. The index in CWT2-W is about 5 times greater than in CWT5-W at a TPR of 0.03.
- At the weld location (Figure 10b), the rupture index rises more rapidly in the strong panel zone case compared to the weak panel zone case. However, the rupture index ends up higher in CWT2-W at a TPR of 0.03.
- At both weld and column locations, the rupture index increases with increasing panel zone distortion. The rate of increase becomes considerably larger at a panel zone distortion of about 2.2 percent.

DESIGN IMPLICATIONS OF EXPERIMENTAL SURVEY AND ANALYSIS RESULTS

In spite of the limited number of connection tests examined and considerable scatter in the test results, the survey

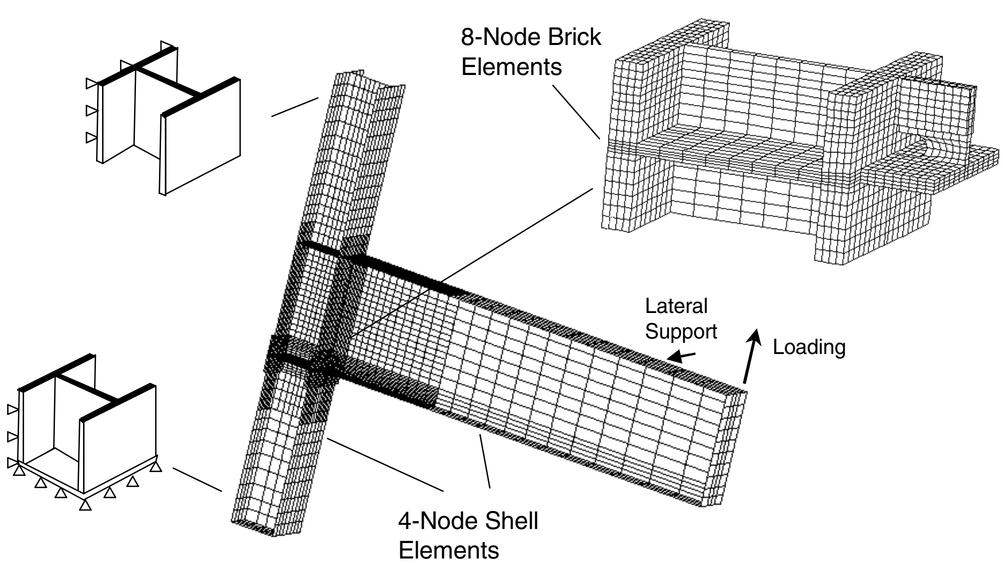
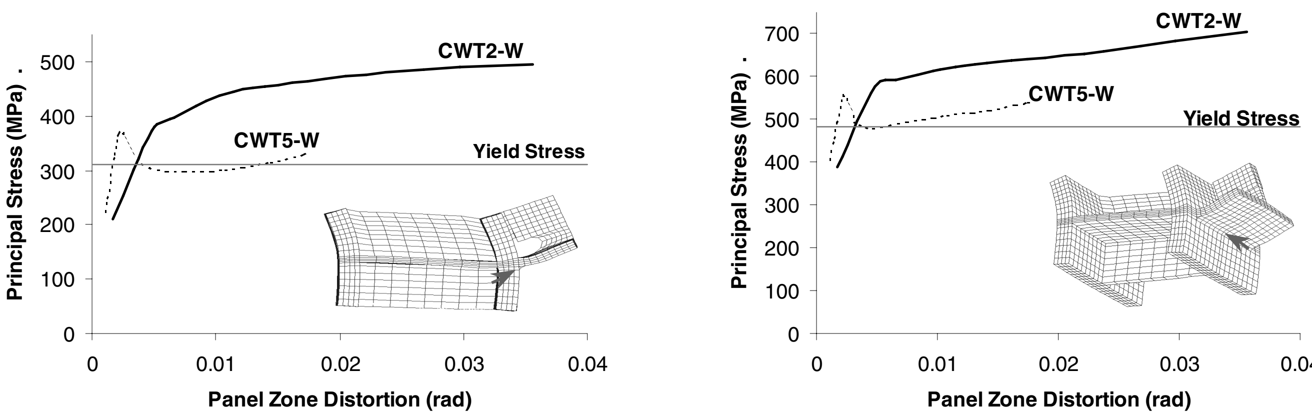


Fig. 8. Details of finite element model.

of experimental data provides some support for the growing consensus that a weak panel zone strategy can be detrimental to connection performance and can decrease cumulative connection ductility. The finite element analyses provide an explanation for this trend and suggest that a weak panel zone design strategy puts the connection region at higher risk for both brittle and ductile fractures compared to a connection with a stronger panel zone. On the other hand, a strong panel zone design generally requires strengthening of the panel zone region. This may entail the use of heavy welding, which in addition to being expensive, results in residual stresses and a large heat affected zone. Residual stresses create distortion in the connection region, while heat affected zones are generally more brittle than the surrounding material and can therefore adversely affect connection ductility. Hence it is important for economic as well as structural reasons to ensure balanced panel zone behavior under severe seismic loading.

RECOMMENDATIONS FOR PRESCRIPTIVE SEISMIC DESIGN PROVISIONS

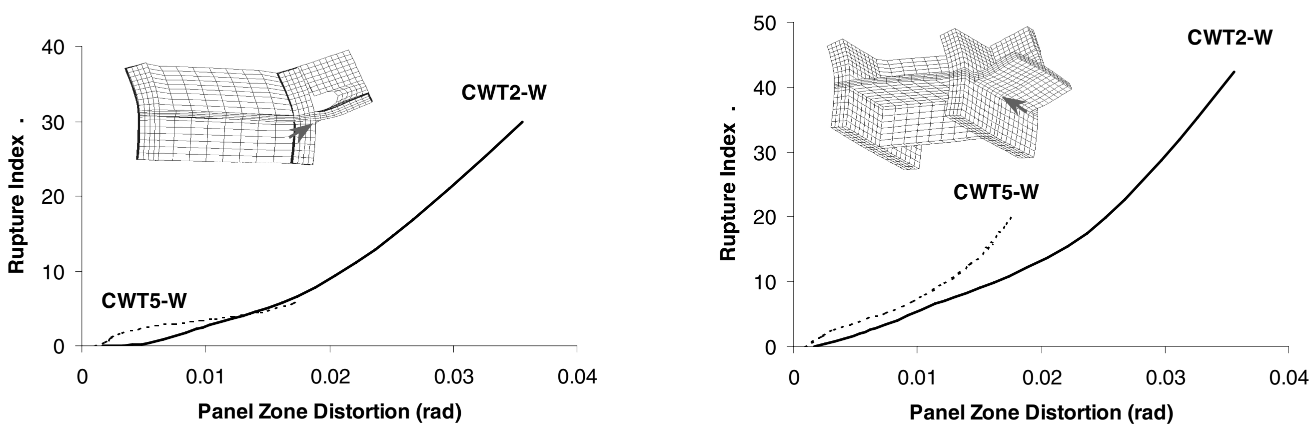
The intent of the panel zone design provisions of the 1997 AISC Seismic Provisions (AISC, 1997) is difficult to interpret; that is, it is not clear to the designer whether such a design procedure will result in strong, balanced, or weak panel zone behavior. The strength equation includes the bracketed term in Equation 5, which implies that inelastic panel zone distortion is expected when the panel zone reaches its predicted strength. However, the design strength is reduced considerably through the use of a resistance fac-



(a) sampled inside column skin

(b) sampled in weld region

Fig. 9. Principal stress vs. panel zone distortion.



(a) sampled inside column skin

(b) sampled in weld region

Fig. 10. Rupture index vs. panel zone distortion.

tor of 0.75 and so it is not apparent if the intent is to allow such panel zone distortions to occur at all. Furthermore, when the 80 percent criterion is applied, the demand provisions overestimate the true demand (see Figures 2 and discussion in section on Seismic Codes). The degree by which the demand is overestimated depends on how high the gravity moments are.

The experimental and analytical information presented in this paper support the growing consensus that excessive panel zone distortion can be detrimental to connection behavior. It is therefore important to ensure that the panel zone undergoes controlled distortions under severe seismic loading. The use of several factors including resistance factors in the current AISC Seismic Specifications (AISC, 1997) masks the conceptual basis of the provisions and makes it difficult to determine if balanced panel zone behavior can be expected. Current panel zone provisions can be made more transparent by specifying design criteria that are similar to the strength provisions that enforce strong column, weak beam behavior.

It is recommended that the demand be calculated using the expected strength of the beams at the face of the column adjusted to reflect the effect of the gravity moments. In other words, if it is assumed that the moment on one side (e.g. left side) of the connection reaches the expected plastic capacity of the framing beam, the moment on the other side can then be estimated using Equation 6:

$$M_r = M_{pr} - 2M_{gr} \tag{6}$$

$$V_n = 0.55F_y t_c d_c \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_c}\right) \tag{5}$$

The strength of the panel zone should be calculated according to Equation 5 without the use of a resistance factor:

The use of Equation 5 implies that some inelastic panel zone distortion is expected at the predicted strength, and therefore the panel zone is expected to contribute to energy dissipation in a controlled manner during severe seismic loading. The advantage of the proposed modification is that it makes the provisions conceptually more transparent.

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