Evaluation of FEMA-350 Seismic Provisions for Steel Panel Zones

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Abstract: Steel panel zone design provisions have undergone large changes in the past four decades. The recently introduced FEMA-350 (2000) guidelines are substantially different from previous provisions, and are no longer a function of the plastic strengths of the panel zone and adjoining beams. Rather, they are based on the premise that the framing beams and the panel zone both yield simultaneously to promote controlled inelastic participation of both components. This paper evaluates and discusses the new provisions based on an examination of published test data and the results of transient analyses of buildings with four-, eight-, and sixteen-story moment resisting steel frames. The limited test results reviewed do not confirm the adequacy of the new panel zone provisions and furthermore suggest that panel zone deformation demands could be affected by connection detailing. The frame analysis results show that the FEMA-350 provisions lead to a rather low level of panel zone participation. A discussion of design implications and research needs is provided.

DOI: 10.1061/(ASCE)0733-9445(2005)131:2(250)

CE Database subject headings: Steel; Panels; Frames; Beam columns; Connections; Deformation; Design; Fractures.

Introduction

The panel zone is the portion of the column within the depth of the connecting beams in a moment resisting steel connection (Fig. 1). The transfer of moments between beams and columns causes a complicated state of stress and strain in the panel zone. Under the action of forces, the panel zone deforms in three modes: Axial, shear, and bending. Only the shear deformation of the panel zone has a significant effect on the behavior of steel frames and is of interest to designers.

Panel zone design provisions have undergone large changes in the past four decades. The panel zones of steel moment frame structures of the 1960’s and 1970’s were generally strong in shear. As discussed in El-Tawil (2000), there are two primary reasons for this trend. First, then existing provisions (e.g., SEAOC 1975) ignored the contribution of column flanges to the shear strength of the panel zone. Second, the moment demand on the connection was, in many cases, overestimated by: (1) Assuming that framing beams could reach their full plastic capacity, and (2) disregarding gravity moments in calculating connection demands. Since steel frame design is frequently governed by drift limitations, beams are deeper and have larger plastic strength than otherwise required by seismic strength provisions thus increasing the design demand on the panel zones. Furthermore, gravity moments on interior steel connections tend to counteract seismic moments; however, their effect is rather small particularly in systems with a few perimeter moment resisting frames (El-Tawil 2000). When combined together, these two factors (underestimating strength and overestimating demand) frequently resulted in the need for panel zone reinforcement, which was mainly provided through doubler plates.

Experimental studies on panel zone behavior initiated in the late 1960’s and early 1970’s, including Krawinkel et al. (1971), Bertero et al. (1973), and later on Popov (1987), showed that the panel zone has high reserve strength after yielding, large ductility, stable hysteresis loops, and significant cyclic strain hardening. In recognition of these observations, building codes increased the rated shear strength of the panel zone by taking into consideration the contribution of column flanges after yielding (e.g., ICBO 1988). The demand was also reduced by the 1987 SEAOC Commentary (SEAOC 1987) and the 1988 Uniform Building Code (ICBO 1988) and capped at 80% of the shear generated by the framing beams as they reached plastic capacity to take advantage of the beneficial effects of gravity moments. The increased panel zone strength and reduced shear demand meant that steel frames designed using these provisions could sustain larger inelastic panel zone distortions during an earthquake compared to earlier frames.

Studies by Tsai and Popov (1988) and El-Tawil et al. (1999) showed that panel zones designed according to the above-mentioned specifications could undergo large inelastic shear distortions prior to reaching their rated shear capacity. This creates problems at the connection welds since large panel zone shear distortions lead to local kinking of the column flanges at the corners of the joint where beam flanges are welded to the column flanges. These kinks result in high stress and strain demands not only in this immediate region, but also in the shear tab welds (Fig. 1). Evidence suggests that weak panel zone behavior may have played a role in the fractures that occurred during the Northridge earthquake (FEMA-267A 1997; El-Tawil 2000).

Panel zone design provisions were made more stringent immediately after the Northridge earthquake. The design demand was...
increased to account for strain hardening and overstrength in beam steel (FEMA-267 1995). A couple of years later, FEMA-267A (1997) recommended that the design shear force should be calculated by assuming that the framing beams reached 80% of their plastic capacity, which was previously the cap placed on shear demand calculations in FEMA-267 (1995). El-Tawil et al. (1999) and El-Tawil (2000) pointed out that these demands could still be too low for interior connections and illogical and unwarranted for exterior connections.

In 2000, FEMA-350 (2000), which culminated from the SAC Joint Venture (http://www.sacsteel.org), proposed design guidelines that are substantially different from previous provisions. The proposed criteria are not a function of panel zone strength (as defined in previous specifications) or beam plastic capacity. Rather, they are based on the premise that framing beams and the panel zone should yield at the same time to promote balanced behavior (i.e., inelastic participation of both components) under earthquake loads. These provisions were, however, not adopted into subsequent seismic provisions published by AISC (AISC 2002). Rather, the new AISC provisions eliminated the 80% cap and specified demands based on full beam plastification, i.e., the provisions essentially reverted back to earlier panel zone specifications, albeit with some refinements. For instance, compared to the SEAOC (1975), the new AISC provisions account for the effect of column flanges in the capacity calculation and material overstrength in the beam is explicitly accounted for in computing demand.

The purpose of this paper is to discuss and evaluate the new FEMA-350 (2000) panel zone seismic provisions. This is achieved by examining previously published test data and by conducting comprehensive studies of frames with panel zones designed according to these provisions.


Consider the connection subassemblage shown in Fig. 2, where the bending moments in the beams and columns vary linearly. When flexural yielding develops in the beam at a distance $x$ from the column face, the bending moment at the column centerline, $M_{cc}$, is,

$$M_{cc} = \sum M_{yb} \left( \frac{L/2}{L/2 - d_c/2 - x} \right) = \sum M_{yb} \left( \frac{L}{L'} \right)$$  \hspace{1cm} (1)

where $L' = L - d_c - 2x$; and $M_{yb}$ = yield moment of the beam at the critical section. The shear in the column needed for static equilibrium of the subassemblage is $M_{cc}/h$. Referring to Fig. 2, the total shear in the panel zone at beam yielding ($V_{pc=M_y}$) is, therefore,

![Fig. 1. Weak panel zone behavior](image1)

![Fig. 2. Panel zone forces](image2)
In order to evaluate the FEMA-350 Survey of Selected Experimental Data and for the purpose of investigating the effect of panel zone yielding on connection behavior, the results of 21 large-scale tests conducted by Lee et al. (2000) and Ricles et al. (2000) are considered. The results of these two test programs were used along with all other SAC connection results to calibrate the FEMA-350 (2000) provisions. Emphasis is placed on these two test programs in particular because they addressed welded unreinforced flange moment connections that were explicitly proportioned to investigate panel zone participation. Furthermore, the behavior of this type of configuration is rather simple, which facilitates drawing conclusions about panel zone behavior.

The Lee et al. (2000) specimens (hereafter referred to as UM specimens) were single-sided connections with beam members ranging in size from W24×68 to W36×150 and column members ranging from W14×120 to W14×257. The Ricles et al. (2000) tests (hereafter referred to as Lehigh tests) were both single-sided and double-sided with only one beam size W36×150 and two column sizes of W14×311 and W14×398.

For each specimen, the required panel zone thickness is calculated as specified in FEMA-350 (2000), Eq. (4) is used instead of Eq. (5) in calculating the required thickness \( t_{\text{balance}} \), because the material properties are known precisely and there is no need for the \( C_c \) and \( R_{yc} \) factors. Connection details are described at length in the references and global response parameters are taken directly from the references and plotted versus \( t_{pc}/t_{\text{balance}} \) which is the ratio of the provided to required panel zone thickness. A ratio greater than 1.0 implies that the panel zone thickness is greater than required by FEMA-350 (2000), while a value less than 1.0 implies that the panel zone is weaker than required.

The test results of the cumulative connection plastic rotation and the maximum panel zone shear distortion are plotted as a function of \( t_{pc}/t_{\text{balance}} \) in Figs. 3(a) and 3(b), respectively. Readers are referred to FEMA-289 (1997), which lists equations for computing various deformation measures such as connection plastic rotation and other measures used herein. The results from each study are plotted separately in Fig. 3 to enable drawing conclusions about each test program. While bounding lines are drawn in Fig. 3(a), they are not drawn in Fig. 3(b) because the scatter is too great. The two test results highlighted with arrows are not included within the bounds in Fig. 3(a) because they failed prior to the development of significant panel zone distortion, i.e., panel zone deformation clearly did not play a role in their failure.

Although the UM study noted that increasing panel zone participation was beneficial to overall ductility, the Lehigh study came to a contradictory conclusion. In spite of considerable variation and scatter in the results, the bounding lines drawn on the aggregate data in Fig. 3(a) appear to support the Lehigh conclu-
sions and show that specimens with larger \( t_{pc}/I_{balance} \) (i.e., with stronger panel zones) are generally more ductile than specimens with lower \( t_{pc}/I_{balance} \) (i.e., with weaker panel zones). The data plotted in Figs. 3(a and b) show that although the Lehigh specimens attained panel zone plastic distortions that are comparable in value to the UM specimens, their cumulative plastic rotations are generally greater. This is attributed to the use of improved access hole details in the Lehigh specimens.

The large scatter in Fig. 3(b) implies that either other test variables unduly influenced panel zone participation or that \( t_{pc}/I_{balance} \) [which is based on Eqs. (4) and (5)] is not a precise indicator of panel zone deformation. Both programs did indeed simultaneously consider other test variables, including weld procedures and various local details, in conjunction with panel zone strength. Some of these variables had the potential to adversely influence connection ductility, and hence the maximum panel zone plastic distortion achieved. In an attempt to isolate the effect of the panel zone on connection deformation, the relative contribution of the panel zone to the overall peak connection rotation is examined in Fig. 4. Regardless of how much connection plastic rotation actually occurred (a parameter affected by many test variables), a very weak panel zone should contribute almost 100%, and conversely, a very strong panel zone should contribute almost 0% to overall connection plastic rotation. The idea is that if \( t_{pc}/I_{balance} \) is a reasonable indicator of panel zone participation, then a trend should be more evident in Fig. 4 than in Fig. 3.

In Fig. 4, the contribution of panel zone plastic and total distortions to overall connection plastic and total rotations, respectively, are plotted versus \( t_{pc}/I_{balance} \). The method for computing panel zone contribution is described in Jin (2002). In spite of a discernable trend, the scatter in test results is still large. Moreover, a closer examination of the figure shows that substantial variations in panel zone contribution (especially plastic) can occur for the same \( t_{pc}/I_{balance} \). This is especially evident around \( t_{pc}/I_{balance} = 1 \), where there is a concentration of test results. To put this scatter in perspective, for \( t_{pc}/I_{balance} = 1 \), the panel zones of tested connections can contribute anywhere from 15 to 54% of the total plastic rotation [Fig. 4(a)].

The specimens that contribute to this large scatter come from both test programs. Focusing on the Lehigh specimens as an example, the four circles along the vertical line representing \( t_{pc}/I_{balance} = 1 \) in Fig. 4(a) are specimens T1, T2, T3, and T4. These are identical specimens in all respects (including coupon tested material properties) except for the shear tab detail. It appears from the test results that premature fractures in the shear tab welds changed the relative stiffness of critical areas in the connection and substantially influenced panel zone participation. The effect of connection detailing on inelastic panel zone participation can also be observed in 2 other Lehigh sets: C1, C2, and C5 as well as T5 and T6. The sensitivity of observed panel zone deformation to connection detailing, coupled with the fact that there is large vertical scatter in data in Figs. 3 and 4, make it difficult to draw positive conclusions about the accuracy and reliability of Eqs. (4) and (5).

Frame Studies and Hazard Level

To further investigate existing panel zone design provisions, three steel moment-resisting frame buildings that are 4, 8, and 16 stories high are designed, modeled, and subjected to a suite of earthquake records. Perimeter moment-resisting frames are selected as the lateral load structural system for all three buildings. Pertinent provisions in AISC (1993), FEMA 302 (1997), AISC (1997), and FEMA-350 (2000) are used to proportion the buildings, which are designed as special moment resisting frames with reduced beam sections (RBS). The frames were designed with RBS to comply with the FEMA-350 provisions in an economical manner. Frame design loads are obtained by assuming the buildings to be standard office buildings located in a region near Los Angeles such that \( S_g = 2.48g \) and \( S_s = 1.02g \), and site class = C. All buildings have identical floor plans. The floor plan, elevations, and design details for the buildings are shown in Fig. 5. The figure shows member selections for interior and exterior columns and for beams. The RBS connection design required 50% beam flange reduction for the majority of the connections. Some connections required panel zone reinforcement, which was achieved through doubler plates. Dimensions of the RBS connections and panel zone reinforcement are not provided here because of space limitations, but are documented in Jin (2002).

FEMA-350 (2000) defines two structural performance levels, Immediate Occupancy and Collapse Prevention. According to the seismic design philosophy stated in FEMA-350 (2000), a building...
structural system must be able to deliver Collapse Prevention performance for a seismic event with 2% probability of occurrence in 50 years (designated as 2/50 hereafter) and Immediate Occupancy for a seismic event with 50% probability of occurrence in 50 years (designated as 50/50 hereafter). These two hazard levels (i.e., 2/50 and 50/50) are targeted in the frame analysis exercise.

One hundred twenty transient analyses are conducted to investigate the seismic behavior of the three systems. Scaled versions of the SAC (Somerville et al. 1997) suite of earthquakes are utilized in the time-history analyses. Only the Los Angeles records (where the buildings are located) are selected for the two hazard levels considered. The records are scaled further according to a method proposed by Shome et al. (1997) to reduce the required number of nonlinear analyses without introducing bias. Readers are referred to Jin (2002) for additional details on record scaling and figures showing the response spectra of the scaled and unscaled earthquakes. The structural behavior of the systems is examined at the two specified levels of seismic hazard by evaluating a number of performance parameters including interstory drift ratios, panel zone shear distortions, and column axial loads.

### Analysis Model

The frames are analyzed using the computer program DYNAMIX (El-Tawil and Deierlein 2001a, b). The program features inelastic beam column, rotational spring, and panel zone elements that can be used to represent steel frames. A common theme that ties together all the formulations supporting the program is the use of bounding surface stress-resultant plasticity models (El-Tawil and Deierlein 1998).

The rotational spring (representing the RBS) and panel zone elements are governed by one-dimensional bounding surface models. The panel zone model implemented in DYNAMIX is
shown in Fig. 6. The model is comprised of a number of rigid bar elements connected by pins. The model allows the intended panel zone distortion to occur and enforces associated constraints on the framing members. The stiffness of the model is concentrated in a central spring and is based on the relationship shown in Fig. 6.

The stiffness at any stage of loading is a function of the distances $d$ and $d_m$ shown in Fig. 6(b), i.e., it is based on a bounding surface model. The ultimate strength and elastic stiffness of the joint panel in the model are based on the data in Krawinkler (1978). The panel zone bounding surface model has three parameters that are calibrated to test results as discussed in Jin (2002) and El-Tawil and Deierlein (1996).

The frame model is depicted in Fig. 7, which shows how the beam column, panel zone, and RBS springs are used together to represent a subassemblage. Closed-form expressions are used to calculate the elastic stiffness of the RBS spring. $P-\Delta$ effects associated with the gravity system are considered in the analysis through the use of a leaning column carrying the weight of the interior gravity system. The leaning column is assumed to be axially stiff, but flexurally flexible and is attached to the building through stiff truss members. 5% viscous damping in first two modes is used for the four-story and eight-story buildings. For the sixteen-story building, 2% damping is used. Nominal material properties are used throughout.

**Validation of Analysis Model**

The models used in this work were extensively verified by comparing simulated responses to experimental results for moment resisting subassemblages in Krawinkler et al. (1971), Whittaker et al. (1996), Uang and Bondad (1996), Popov et al. (1996) and Engelhardt et al. (1996, 1998, 2000). Reasonable comparisons were achieved and the reader is referred to Jin (2002) for details of the entire validation exercise. The Engelhardt et al. (2000) verifications are most pertinent to this paper because they focused on RBS connections. Comparisons between computed and measured results for specimen DBBWC are shown in Fig. 8. Reasonable correlation between analysis and results is evident in the figure, especially for panel zone distortion [Fig. 8(b)]. The ratio $t_{pz}/t_{balance}$ is 0.89, and the specimen therefore almost conforms to
FEMA-350 (2000) design philosophy for panel zones. The sub-assemblage is modeled as described in the previous section and measured material properties are used to calculate member parameters.

As shown in Fig. 8, the model is valid up to about 0.04 radians total connection rotation (see arrows on Fig. 8). Beyond this rotation, lateral torsional buckling coupled with flange fracture precipitated severe degradation in structural behavior, which is not accounted for in the model. Although the connection model could have been calibrated to account for degradation in structural behavior (as described, for example, in El-Tawil and Deierlein 2001a, b), this was deemed not necessary because the response of the frame systems did not generate such extreme demands. For example, as shown later on in the paper, the greatest interstory drift did not exceed 0.05 radians, and such high demands occurred in only two of the twenty 2/50 earthquakes, and at only a few stories of the 16-story building at that.

Analysis Results

Pertinent performance measures are calculated from the parametric analyses. These are compared to corresponding pushover results and examined for evidence of behavioral trends. Voluminous data were generated, but only a small representative portion of it is shown in Fig. 9 for the 16-story frame. The figure shows three plots: Interstory drift, maximum panel zone plastic distortion at an interior node, and drift contributed by panel zone. The contribution of panel zone distortion to interstory drift is evaluated to identify the effect of panel zone distortion on the global response of the buildings. The contribution is calculated at each joint and averaged at each story level.

For each graph, 20 points are plotted at each floor level; one for each earthquake in the suite. Each point represents the maximum calculated response for that particular earthquake. To bound the data, the maximum and minimum points at each level are connected yielding the “min” and “max” lines in Fig. 9. The “median” lines are also plotted to give a sense for the statistical distribution of the data. Following Shome et al. (1997), the term “median” refers to the geometric mean, which is the exponential of the average of the natural logarithms of data.

Several observations are evident from the analyses. Maximum interstory drift demands are not excessively high for the 4- and 8-story buildings, slightly exceeding 0.03 radians for both buildings. For the 16-story building, several earthquakes in the suite resulted in interstory drifts well in excess of 0.03 radians, reaching a maximum close to 0.05 radians [Fig. 9(a)]. However, it is noteworthy that the median drift demand for all three buildings is less than 0.03 radians.

Maximum panel zone distortions are quite small in all three frames; the maximum demand occurred in the lowest floor of the 16-story building and was 0.006 radians [Fig. 9(b)]. The median panel zone plastic distortion demands in all three buildings are also rather small, generally less than 0.0025 radians. The maximum contribution of the panel zone to total drift is limited to less than 26%, with median contributions generally lower than 18%. This level of panel zone contribution is rather low and is partly attributed to rounding off in the design process and minimum thickness limits for doubler plates, both of which lead to somewhat overdesigned panel zones.

This effect can be better seen in Fig. 10, where the median panel zone contributions to total drift calculated from the frame analyses (for all three buildings) and the corresponding ratios from the test results are plotted together versus $t_{pc}/t_{balance}$. Fig. 10
Design Implications

Two issues with design implications are raised in this paper. The first is that the limited experimental results surveyed do not confirm the accuracy and reliability of the FEMA-350 (2000) panel zone provisions [Eq. (5)]. The large vertical scatter in Fig. 3(b) and the scatter around \( t_{pz}/t_{balance} = 1 \) as marked on Fig. 10. Analyses for the 50/50 runs showed panel zone plastic distortions are almost negligible, making their contribution to energy dissipation minimal for this hazard level.

The second issue is that the FEMA-350 (2000) specifications appear to lead to underutilization of the panel zone in the three frames studied. For example, for the 2/50 hazard level, the contribution of panel zone distortion to drift (median values) is less than 18% for all three frames, while the contribution observed in tests is 17–36% for specimens with balanced panel zones. For the 50/50 runs, panel zone contribution is negligible. It is perhaps fortunate that panel zone participation is limited, given the doubts about adequacy of the FEMA-350 design criteria and lack of consensus within the structural engineering community regarding the level of beneficial inelastic panel zone distortion.

Summary and Conclusions

The panel zone seismic design provisions in FEMA-350 (2000) were evaluated and discussed in light of two sets of carefully selected test programs and results from a series of transient frame analyses. The limited test results reviewed do not confirm the accuracy and reliability of the FEMA-350 (2000) panel zone provisions and furthermore suggest that panel zone deformation demands could also be affected by connection detailing. Four-, eight-, and sixteen-story frames with reduced beam sections were proportioned according to current seismic design specifications and subjected to a suite of earthquakes representing 50/50 and 2/50 hazard levels. The analyses accounted for inelastic member and panel zone behavior and included P-delta effects. The models used in the analyses were validated through comparisons to measured test data and were then used to investigate panel zone demands. The analyses show that existing provisions lead to a rather low level of panel zone participation for the three buildings studied. A limitation of the research presented here is that the frame analysis results were obtained without considering statistical variations in material properties, therefore additional research is recommended to further investigate this issue.

The large changes that have occurred in the approximately 40 year history of panel zone provisions attest to the fact that the effect of panel zone deformation on connection response have never really been well understood. Based on the evidence presented, the writers feel that the issue is not yet fully resolved, and that additional large-scale testing is still needed to clarify the advantages and disadvantages of inelastic panel zone deformation. It is likely that the dependence of inelastic panel zone distortion on connection detailing will complicate the move away from prescriptive criteria such as the FEMA-350 (2000) and current AISC provisions (AISC 2002) toward performance-based specifications that precisely quantify demand and capacity parameters for the panel zone region.

Acknowledgments

Financial support for this research was provided in part by the U.S. National Science Foundation (Grants No. CMS 9870927 and 0296210) and the Departments of Civil and Environmental Engineering at the Univ. of Michigan and Univ. of Central Florida. The opinions stated here are those of the writers and do not necessarily reflect the views of the sponsors.

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