ASSESSMENT OF PANEL ZONE DESIGN APPROACHES FOR STEEL MOMENT FRAMES

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ABSTRACT :

This paper deals with the design and behaviour of panel zones in steel moment-resisting frames. A detailed review of previous and existing European and North American panel zone design rules is carried out and the main differences between the various procedures are discussed. A detailed numerical investigation into the inelastic behaviour of moment frames incorporating panel zone response is then undertaken in order to assess the influence of a number of key parameters. The results obtained indicate the suitability of adopting 'balanced' design approaches, such as those proposed in recent North American provisions for the panel zone. In contrast, 'weak' panel zone designs often result in excessive distortional demands which can lead to unreliable performance of connection components. The findings of this study point out to some limitations in current European guidelines particularly in terms of the overestimation of panel zone capacity which unintentionally results in relatively weak panel zones. The numerical studies also illustrate the important role that can be played by the level of gravity loading applied on the beams. This effect, which is not addressed in any of the current provisions, is shown to have a significant influence on the lateral response, and is therefore necessary to account for in codified design guidance.

KEYWORDS: Panel Zone; Steel Joints; Steel Frames; Ductility

1. INTRODUCTION

The panel zone, which is the region of the column web delimited by the column flanges and continuity plates at a beam-to-column connection (Figure 1), is known to have ductile and stable hysteretic properties (Krawinkler *et al.*, 1971; Fielding and Huang, 1971). These features make the panel zone a very attractive component for energy dissipation in steel and composite moment-resisting frames under earthquake conditions. However, in such circumstances, the properties of the panel should be incorporated in the structural analysis and also accounted for when evaluating the lateral capacity of a structure.

The consideration of the panel zone contribution to the inelastic response has been examined in the last decade after the extensive damage that was observed in welded connections following the 1994 Northridge earthquake. This damage was widely attributed to excessive deformations in the panel zone region and lead to the introduction of more strict design rules in the US. In Europe, current seismic provisions (CEN, 2004) imply that the panel zone should be capacity designed such that it does not contribute significantly to the energy dissipation during an earthquake.

In this paper, an historical review of panel zone design in Europe and in the US is firstly provided. After discussing the main differences between the various design procedures, a detailed numerical investigation into the behaviour of moment frames, incorporating panel zone response, is presented. The study focuses on the influence of key parameters that affect the inelastic response of typical frame configurations. The results of this investigation illustrate significant limitations associated with current panel zone design provisions.

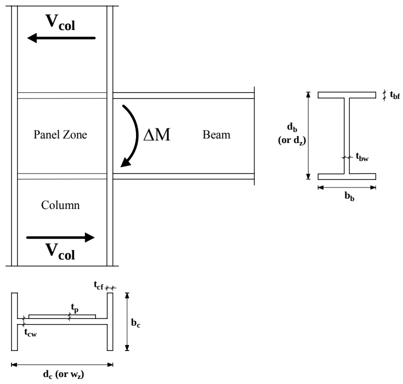


Figure 1 Schematic representation of a beam-to-column joint

2. PANEL ZONE DESIGN

A panel zone can be designed according to three different approaches which are directly related with the energy dissipating mode of the structure (Popov, 1987). Panel zones are classified as either strong, intermediate or weak in terms of strength and with respect to the flexural capacity of the connecting beams. There is no standard design practice for panel zone established at present, and there are notable differences between codes. Over the years, design criteria for panel zones have undergone significant changes. In the following sections, a review of North American and European practices for the seismic design of panel zones is presented.

2.1. US Practice

Panel zone design rules in the US evolved over a number of years. Up until the mid-1980's, the design objective was to achieve a weak beam-strong column mechanism, as implied in the Commentary to the 1980 version of the Bluebook. The fulfilment of this objective required the consideration of strong panel zones which, in most cases, could only be accomplished through the addition of thick doubler plates and heavy welding. This design approach was later modified after extensive research carried out on steel joints revealed that panel zones may have a ductile and stable hysteretic behaviour that could enhance the energy dissipation capacity and also contribute to a reduction of the inelastic demands imposed on the beams (Fielding and Huang, 1971; Krawinkler *et al.*, 1975). Test results also indicated significant work hardening after shear yielding of the panel zones. This behaviour was attributed not only to the material strain-hardening but also to the contribution of the column flanges and continuity plates surrounding the panel zone, which led Krawinkler and his colleagues (Krawinkler *et al.*, 1975) to suggest a tri-linear behaviour for panel zones, as illustrated in Figure 2. Based on these findings and in order to reduce the costs related with extensive joint welding, the 1988 version of the Bluebook (SEAOC, 1988) introduced a new design criterion for panel zones which clearly allowed for energy dissipation of these components. The shear capacity of the panel zone was also modified in order to take into account the additional strength provided by the framing components surrounding the panel as described before. The expression for determining the panel shear strength resulted from a slight modification to that proposed by Krawinkler (1978), such that:

$$V_y = 0.55 f_y d_c t \left(1 + \frac{3 b_c t_{cf}^2}{d_b d_c t}\right) \tag{1}$$

In the expression, f_y is the yield strength of steel, t is the thickness of the panel zone and the all the remaining parameters refer to the geometrical properties of the beam and column, as shown in Figure 1.

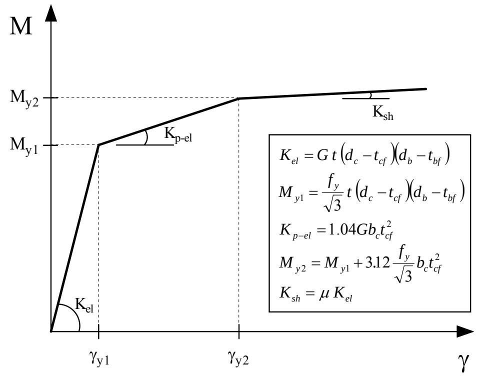


Figure 2 Moment-distortion response for panel zones (Krawinkler *et al.*, 1975)

The widespread connection failures observed after the 1994 Northridge earthquake revealed the limitations of the design provisions described above. An extensive research survey conducted after the event identified several factors that contributed to the widely observed fracture of welds in beam-to-column flange connections. Among these were the excessive panel zone distortions that resulted in local kinking of the column flanges with a consequent increase in the stress and strain demands in the beam-to-column interface (FEMA, 2000). This explanation has also been confirmed from experimental observations as well as from numerical simulations (El-Tawil *et al.*, 1999).

An extensive survey of test data obtained from welded flange-bolted web connections (FEMA, 2000) revealed that a number of factors influence the plastic rotation capacity of a steel sub-assemblage. These factors include the beam size and geometry, panel zone yielding, steel properties, welding types and procedures, weld access hole geometry, among others. Despite the significant scatter of the results, it was possible to conclude that the most ductile performance was achieved in joints designed for combined flexural and panel zone shear yielding, more precisely in those where flexural beam yielding occurred first. Those specimens exhibited not only the largest plastic rotation capacity, but panel zone shears were also well above those implied in codified expressions. These observations led to the formulation of new design criteria for panel zones which have been incorporated in FEMA 350 (FEMA, 2000). The new approach, which is termed *balanced design*, targets flexural yielding in the beam followed by yielding of the panel zone. This design objective is achieved by defining a minimum thickness (t) for the panel zone:

$$t \geq \frac{C_y M_c \left(\frac{h - d_b}{h}\right)}{(0.9)0.6\, f_{yc} R_{yc} d_c \left(d_b - t_{bf}\right)} \tag{2}$$

where the product $C_y M_c$ denotes the total moment in the panel zone produced by flexural yielding of the connecting beams and accounting for the expected steel yield strength of the column and beam. In the expression, f_{yc} represents the minimum steel yield strength of the beam whereas R_{yc} denotes the ratio between the expected and the minimum steel yield strength for the beam. Notwithstanding the consistency and rationale of this new design approach and the significant amount of test data that supports it, Jin and El-Tawil (2005) conducted a survey on additional sets of data and concluded that specimens satisfying the FEMA 350 condition did not confirm the accuracy and reliability of the design method.

The uncertainty in establishing an effective design method for panel zones is reflected in the criteria suggested in the current AISC seismic provisions (AISC, 2005). Despite the recommendations provided in FEMA 350 and the significant amount of research carried out on the subject within the SAC project (FEMA, 2000), the AISC provisions stipulate that any panel zone should be able to resist, as a minimum, the shear resulting from the projection of the expected beam plastic moments at the plastic hinge points, to the column flanges. This design approach aims to establish a balance between the flexural plastic capacity of the beams and the yield/plastic resistance of the panel zone. In the Commentary to the provisions, it is recognized that this design approach may be less conservative in some situations in comparison to that proposed in FEMA 350.

2.2. European Guidance

The design criteria for panel zone prescribed in the European seismic provisions have been different from those adopted in North America and have also been changing over the years. An early pre-standard version of the European code stipulated that a panel zone should simply be able to resist the shear induced by the seismic design forces. This criterion resembled that proposed in the 1988 version of the Bluebook and therefore did not provide any limit or control on the panel zone distortion and could hence lead to undesirable joint response.

The most recent version of Eurocode 8 (CEN, 2004) addresses the above-noted limitation by proposing a new design approach whereby the panel zone is proportioned to resist the shear induced from the formation of adjacent plastic mechanisms in beams or connections. Additionally, Eurocode 8 limits the contribution from this component to the plastic rotation of the plastic hinge region to 30%. On the other hand, the new version of Eurocode 3 (CEN, 2005) proposes a different relationship for the shear resistance of the web panel which accounts for the additional contribution of the column flanges to the resistance, an effect that was not considered in the pre-standard version of the code. The shear capacity of the panel zone is given by:

$$V_{wp,Rd} = \frac{0.9 f_y A_v}{\sqrt{3}\gamma_{M0}} + \frac{4M_{pl,fc,Rd}}{d_s} \tag{3}$$

In this equation, $M_{pl,fc,Rd}$ represents the plastic moment capacity of a column flange, d_s is the distance between the centreline of the beam flanges (i.e. d_b-t_{bf}) and the 0.9 factor is used to account for the reduced shear capacity of the panel under axial loads despite Eurocode 8 stating that this effect can be neglected. It is worth noting that Eurocode 3 only considers the contribution of a single doubler plate to the shear resistance of the panel zone. The code also limits the additional shear area from this supplementary plate to $b_s t_{cw}$ where b_s is the width of the plate and t_{cw} is the thickness of the column web.

Changes in the design objective for panel zones are evident from the description above. In its new version, Eurocode 8 shifts the intended design approach from a potentially weak to a stronger panel zone. This conservatism in the current seismic provisions reflects the uncertainty in estimating the actual contribution of the web panel to the inelastic joint response.

The review of European and US design provisions presented in this section exposes different interpretations regarding the panel zone contribution to the joint seismic response in moment frames. There are significant inconsistencies particularly in relation to the definition of the panel zone strength, the evaluation of the panel zone demand and the contribution of the axial and shear force in the column. Subsequent sections of this paper focus on clarifying some of these important issues with the aim of suggesting possible improvements and harmonisation of current guidance.

3. NUMERICAL INVESTIGATION

3.1. Structural Configurations

A numerical study into the panel zone behaviour in steel moment frames is carried out through the analysis of idealised structural systems of the types shown in Figure 3. These arrangements are representative of the global behaviour of moment-resisting frames under lateral loading conditions even though a certain level of simplification is associated with each type. While for the multi-bay structure the contra-flexure points are assumed to form at mid-height of the columns, in the cruciform arrangement contra-flexure points are also considered at mid-span of the beams. For all the cases the beam span (*L*) is assumed as 8 m whereas the column height (h) is taken as 3.5 m. European steel sections are adopted for both beams and columns. The sections considered are IPE 400 for beams and HEA 340 for columns. These sizes are chosen to control serviceability deformations in the beams and to fulfil capacity design requirements in the columns. The yield strength of the steel is taken as 275 N/mm^2.

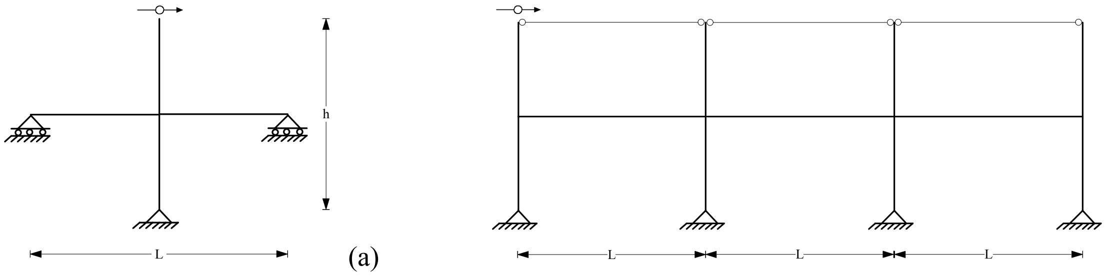


Figure 3 Sub-structure configurations: (a) Cruciform and (b) Multi-bay

The sub-structures are modelled in the finite element program OpenSees v1.7.3 (PEER, 2006). Nonlinear static pushover analysis is performed on each structure by controlling the displacement of the top node of each sub-structure up to a target drift of 4% of the storey height. Additional details regarding the numerical models and procedures adopted in this study can be found elsewhere (Castro *et al.*, 2008).

3.2. Parametric Considerations

A detailed parametric study is performed on the two types of sub-structure. A wide range of parameters was investigated but in this paper only those thought to be of notable influence on the panel zone behaviour will be presented, namely the panel zone strength and the level of gravity loading in the sub-structure. A complete description and discussion of more extensive parametric studies is provided elsewhere (Dávila-Arbona, 2007).

The study of the panel zone strength comprises the analysis of seven cruciform sub-structures (Figure 3a) covering a range of weak to strong panel zones. The design criterion adopted for these cases consisted of the definition of a strength ratio between the yield capacity of the panel zone and the plastic moment capacity of the adjacent beams which can be expressed in mathematical terms as follows:

$$\frac{t\left(d_c - t_{cf}\right)f_y}{\sqrt{3}} = \alpha \sum M_{pl,b} \left(\frac{1}{d_b - t_{bf}} - \frac{1}{L - d_c + t_{cf}} \frac{L}{h} \right) \tag{4}$$

The term ‘α’ in Eqn. 4 is a strength factor that can be used to define different capacity ratios between the beams and the panel zone. This design criterion is very similar to that proposed in FEMA 350 except that the shear demand is evaluated based on the beam plastic moment capacity and not on the beam yield capacity. Additionally, the shear demand in this criterion is based on the moment developing at the column face and not on a projection of that moment to the centreline of the column. As shown in Table 1, seven cases are defined in this study with panel zone strengths ranging from 70% to 110% of the total plastic moment capacity of the connecting beams. In order to support the discussion of the results, the panel zone thicknesses required by the different European and US design provisions are also provided in the table.

Table 3.1 Cases considered for the panel zone strength investigation

Case	**Panel zone thickness (*t*)**
PZU	9.5 mm
PZ70	21.9 mm
PZ80	25.1 mm
PZ90	28.2 mm
PZ100	31.3 mm
PZ110	34.5 mm
PZRIG	∞

Case	**Panel zone thickness (*t*)**
Bluebook 1988	23.1 mm
Eurocode 8 (2004)	23.0 mm
AISC 341-05	26.8 mm
FEMA 350	29.3 mm

The second parameter investigated is the level of gravity loading. While the cruciform sub-structure (Figure 3a) is adequate for scenarios with no vertical loads, it cannot capture the influence of gravity loading. Accordingly, for this study the multi-bay sub-structure (Figure 3b) is utilised. This arrangement is able to provide a more realistic representation of a moment-resisting frame under a combination of vertical and lateral loads. The level of gravity loads is established based on the ratio between the moment developed at the beam end and its plastic moment capacity assuming beam fixity at both ends. Three different cases (GL25, GL50 and GL75) are considered in which the gravity loads develop approximately 25%, 50% and 75% of the plastic moment capacity of the idealised beams. An additional case (GL0), which does not incorporate any gravity loading, is also analysed for cross-comparison with other parametric studies. In all four cases, the panel zone thickness is assumed as 31.3 mm, hence representing the same thickness adopted in the PZ100 case, as indicated in Table 1. Key results and observations obtained from the two parametric studies are discussed in the following sections.

4. DISCUSSION OF RESULTS

4.1. Influence of Panel Zone Strength

The overall response obtained for all seven cases indicated in Table 1 is depicted in Figure 4 in the form of pushover curves (in terms of base shear against storey drift as a percentage of storey height). The plot clearly shows the influence of the panel zone on the stiffness and capacity of the system. The need to account for the column web panel in the analysis, particularly in cases with weak and intermediate panel zones, is evident from the figure. However, for relatively strong panels (i.e. PZ100 and PZ110), the global response is not notably different from that provided by the idealised PZRIG case. From Figure 4, it is also interesting to observe the expected response of a sub-structure designed according to the Eurocodes. As indicated in Table 1, this would correspond to a pushover curve somewhere in between the PZ70 and PZ80 curves. Despite the stated design objective of Eurocode 8 of achieving a relatively strong panel zone, it is clear from Figure 5 that the behaviour is in fact characteristic of a weaker panel zone design. The lower resistance of components designed to the Eurocodes is a result of the expressions for evaluating the panel zone capacity in Eurocode 3, and not with the approach adopted in Eurocode 8.

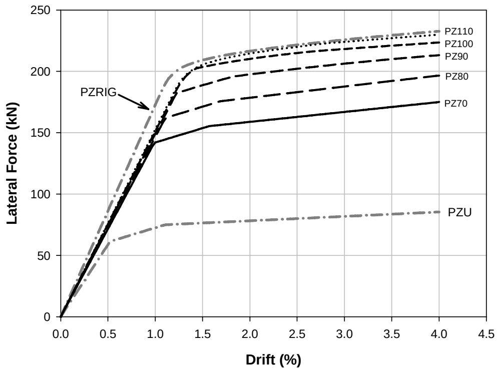
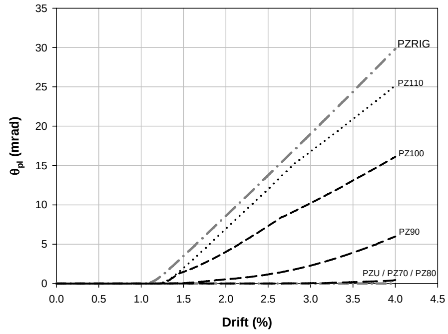
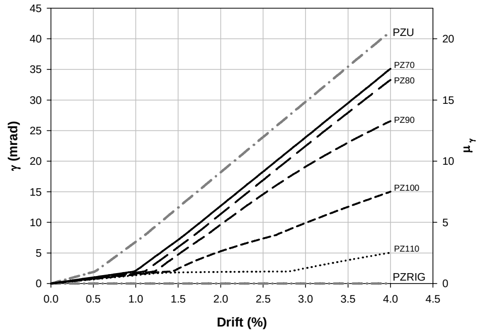


Figure 4 Pushover curves for different panel zone designs

The local response of the sub-structure components also provides significant insight into the behaviour. The plastic hinge rotations recorded for increasing drift and for all seven cases are provided in Figure 5a. The graph clearly shows the reduction in the inelastic demand on beams when the panel zone strength is decreased. This behaviour illustrates the benefits of involving the panel zone in the inelastic response of the system. On the other hand, as expected, a reduction of the panel zone strength implies larger inelastic distortional demands on this component. This is illustrated in Figure 5b which indicates the extremely high ductility demands imposed on the panel zone in the weak and intermediate designs (i.e. PZU, PZ70, PZ80 and PZ90). More importantly, it is worth noting the favourable performance of the balanced case (PZ100). For an inter-storey drift of about 3%, a value typically adopted in performance based design to define the life safety limit state, the distortion demand imposed on the panel zone is around 10 mrad which is relatively low in comparison with other cases.



(a)



(b)

Figure 5 (a) Plastic hinge rotations in beams and (b) Panel zone distortions for different designs

4.2. Influence of Gravity Loads

The global response obtained from the multi-bay frames covering the four cases (GL0, GL25, GL50 and GL75) considered in this study (all assuming balanced panel zone design) is presented in Figure 6. As shown in the plot, this parameter can have a significant influence on the inelastic response of a moment-resisting frame, particularly for cases associated with high levels of gravity load. The distinct formation of two yield points and the reduction in lateral capacity for increasing values of vertical load are evident. This behaviour is directly related to the difference in beam moments on each side of the joints which is further illustrated in Figure 6 by the earlier initiation of nonlinearity for the cases with higher gravity loads. This difference in response compared to the case with no vertical loads has important consequences in terms of the inelastic response of the beams, as discussed below.

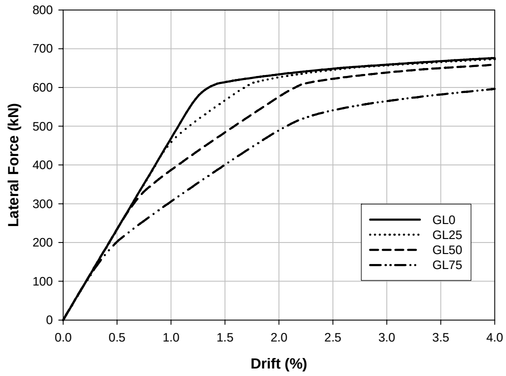


Figure 6 Pushover curves for different levels of gravity loading

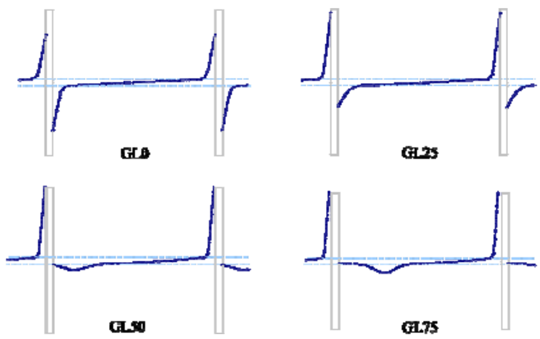


Figure 7 Internal beam curvatures for 4% drift

The curvature distribution in the internal span of the multi-bay sub-structure is depicted in Figure 7 for the target drift considered in the analysis (4%). Examination of the plots shows that there are significant concentrations of inelasticity in the beam ends under negative moment. This effect is even more pronounced for cases GL50 and GL75, where it is observed that a shift in the location of the plastic hinges under positive moment away from the column face occurs due to the high level of gravity loads.

4. CONCLUSIONS

In this paper, a review of the various approaches for panel zone design available in Europe and in the US was carried out. A numerical study was undertaken in order to investigate the influence of a number of key parameters on the inelastic response of this type of structure. The main parameters shown to have a salient influence on the response are the panel zone strength and the level of gravity loading.

The results obtained highlighted the merits of adopting a balanced design for the panel zone as this can lead to significant reductions in terms of plastic hinge rotations in the beams. However, the findings also indicate that weak panel zone designs can result in very high distortional demands which are known to cause unreliable behaviour of the other components of the beam-to-column connection, particularly in welds. The results also show that a balanced design approach results in a favourable behaviour, especially for cases with relatively low levels of gravity load. However, the situation may be significantly different in cases where gravity loading constitutes a significant fraction of the moment applied to the beams.

Overall, the findings of this study highlight a number of significant differences between codified design approaches for panel zones. There is clearly a need for further numerical studies, supported by experimental validation to develop more reliable procedures for the seismic assessment and design of panel zones components.

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