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# Panel zone deformation demands in steel moment resisting frames

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ABSTRACT

Energy is dissipated by yielding at beam ends in steel moment resisting frames subjected to earthquakes. In addition, the column panel zone can be proportioned to participate in energy dissipation. AISC360, Eurocode 3, and FEMA-355D specifications have different philosophies for the design of panel zones which lead to different amounts of deformation demands. This paper presents a numerical study undertaken to quantify the deformation demands in panel zones proportioned according to different specifications. Pursuant to this goal, the panel zone design requirements of three specifications were compared first to identify differences and similarities. A parametric study was conducted by considering the beam depth, axial load level on columns, thickness of panel zone, and seismic hazard as the prime variables. Archetypes were analyzed by employing three-dimensional explicit nonlinear finite element time history analyses. The analysis results revealed that designs according to AISC360 and Eurocode 3 resulted in significant amounts of yielding of the panel zone while the designs according to FEMA 355D resulted in minimal amount of yielding. The panel zone deformation demands were quantified and an equation used to estimate the deformation levels was developed herein.


## 1. Introduction

Moment resisting frame (MRF) is among the types of lateral load resisting systems used in seismic design of steel buildings. A typical MRF is composed of columns and beams which are rigidly or semi-rigidly connected to each other as shown in Fig. 1. This system has architectural advantages over its counterparts because it provides an unobstructed space between columns. The AISC Seismic Provisions for Structural Steel Buildings (AISC341-10) [1], hereafter referred as AISC341-10, classifies MRFs into three broad categories, namely, Special Moment Resisting Frames (SMRFs), Intermediate Moment Resisting Frames (IMRFs), and Ordinary Moment Resisting Frames (OMRFs). Design requirements are the most stringent for SMRFs and less stringent for OMRFs. Ductility demands also vary among these categories being the highest for SMRFs and lowest for OMRFs.

A typical cruciform type beam–column sub-assemblage is shown in Fig. 1. It is assumed that the points of contraflexure occur at the column mid heights under the action of lateral loads. The forces and bending moments on the cruciform are given alongside the bending moment and shear force diagram for the column in Fig. 1. The part of the column which is surrounded by the beams and continuity plates is termed as the panel zone. Examination of the shear force diagram reveals that very high forces are produced in this region. These forces can potentially cause yielding of the panel zone and contribute to energy dissipation. Behavior of the panel zone is complex and various researchers have examined the stress patterns in detail. Krawinkler [2] and Tsai and Popov [3] reported typical shear stress distributions within the panel zone. These distributions indicate that the maximum amount of shear stress occurs at the center of the panel and the stresses reduce at locations close to the boundaries.

Energy is dissipated by yielding in the beams, columns, and panel zones of a MRF during a seismic event. In general, yielding in the beams and panel zones is preferred over yielding of the columns. Seismic provisions provide clauses which has a direct impact on the yield mechanism. SMRFs are expected to provide high ductility through forming plastic hinges at the beam ends. The panel zones can be designed to yield or remain elastic depending on the type of approach adopted. Beam hinging is stipulated by strong column weak beam concept where the sum of moment capacities of the beams that frame into the joint is smaller than the sum of moment capacities of the columns that frame into the joint. For IMRFs and OMRFs the requirements are less stringent and the strong column weak beam concept is not enforced according to AISC341-10. These systems, however, are expected to show low or moderately ductile behavior and are not allowed to be used in high seismic regions. For example, Minimum Design Loads for Buildings and Other Structures (ASCE7-10) [4], hereafter referred as ASCE7-10, does not permit the use of IMRFs and OMRFs in Seismic Design Categories D, E, and F.

In SMRFs, design and detailing of the panel zone have a significant impact on the behavior. If the panel zone is designed to remain elastic then the plastic rotation demands at the beam ends increase. Maintaining an elastic behavior may require thick doubler plates to be added to the panel zone region which adversely affects the economy. On the other hand, the panel zone can be designed to yield under the shear forces that develop in this region. Panel zone yielding participates in

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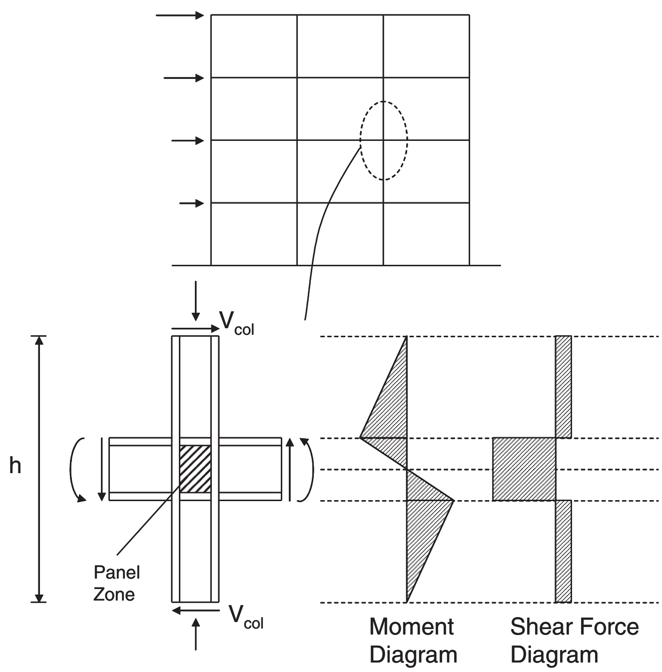


**Fig. 1.** A typical beam-to-column connection and internal force distribution for the column.

the plastic rotations and the rotation demands at beam ends reduce. Experiments conducted on beam–column sub-assemblages showed both behaviors [5–7]. There are several disadvantages of having yielding in the panel zones. One disadvantage is the formation of kinks at the beam flange which can lead to fractures in the connection. In addition, yielding may affect the integrity of the system if the plastic strains in the panel zone are too high. Furthermore, when compared with the retrofit of beams, retrofit of the panel zone after a seismic event would present difficulties in cases where this region experiences high plastic strains.

Shear demand and capacity of the panel zone must be determined at the design stage in order to ensure desired behavior. Historically there have been different approaches for both the demand and the capacity calculations. A joint shear versus joint distortion behavior according to

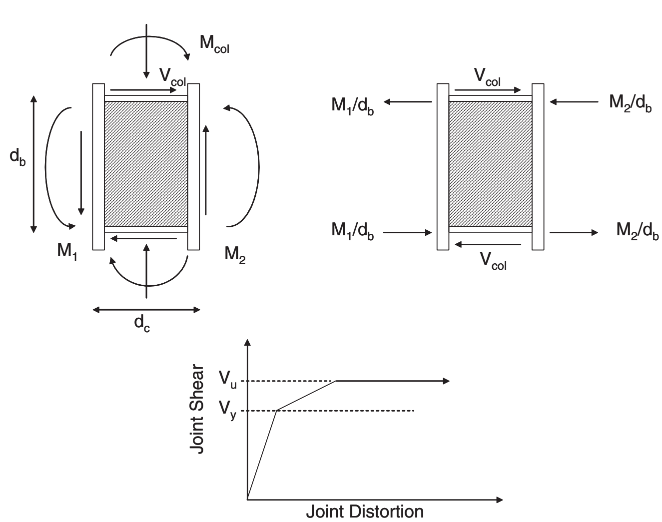


**Fig. 2.** Free body diagram of a panel zone and shear versus distortion response.

Krawinkler [2] is shown in Fig. 2. The behavior is elastic until the yield force in shear ($V_y$) which is expressed as follows:

$$V_y = 0.55F_y d_c t_{cw} \tag{1}$$

where $F_y$ is yield stress of the column, $d_c$ is the depth of column, and $t_{cw}$ is the thickness of column web. Eq. (1) is obtained by multiplying the yield stress in pure shear with the effective shear area. The yield stress is considered to be equal to $F_y/\sqrt{3}$ according to von Mises yield criterion and the effective shear area is taken as $0.95 \times d_c \times t_{cw}$. Experimental results [5–7] have revealed that the panel zone has excess capacity beyond the first yield capacity. Column flanges form a frame around the panel zone and this frame can provide additional shear resistance until plastic hinges form in its flanges. In addition, a significant amount of strain hardening occurs after the panel zone yields which contributes to the increase in resistance. The ultimate shear carrying capacity ($V_u$) can be estimated by taking into account additional increases in capacity due to the frame action and strain hardening. As expected, the amount of increase is directly related to the thickness of the column flange. A model was developed by Krawinkler [2] to arrive at an expression for ultimate shear carrying capacity ($V_u$). This expression was slightly modified in the past and forms the basis of the panel zone capacity expression given in AISC Specification for Structural Steel Buildings (AISC360-10) [8], hereafter referred as AISC360-10. According to this specification $V_u$ can be expressed as follows:

$$V_u = 0.6F_y d_c t_{cw}\left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_{cw}}\right] \tag{2}$$

where $b_{cf}$ is the width of column flange, $t_{cf}$ is the thickness of column flange, and $d_b$ is the depth of beam. The term outside of the square brackets is identical to Eq. (1) except that the coefficient of 0.55 is replaced with 0.6. The term in the square brackets takes into account the additional capacity provided by the frame action of the column flanges and strain hardening.

Eurocode 3 [9] has a different approach for calculating $V_u$. A basic expression in the form of Eq. (1) is provided for the yield resistance ($V_y$) and an increase beyond the yield resistance is provided which takes into account plastic hinge formation in the column flanges. The following expression is given for the design resistance of a panel zone in Eurocode 3:

$$V_u = \frac{0.9F_y A_{vc}}{\sqrt{3}} + \frac{4M_{pl,fc,Rd}}{d_s} \tag{3}$$

where $M_{pl,fc,Rd}$ is the plastic moment capacity of column flanges, and $d_s$ is the distance between the centerline of the beam flanges. In the above equation $A_{vc}$ is the shear area of column which in Eurocode 3 is given by $A - 2 \times b_{cf} \times t_{cf} + (t_{cw} + 2r) \times t_{cf}$ where $A$ is the gross area of column, and $r$ is the root radius of the column flange. The 0.9 factor is used to account for the reduced shear capacity of the panel under axial loads [10,11].

Forces and bending moments produced on a typical panel zone, given in Fig. 2, are considered for an interior joint in order to calculate the shear demand. The column faces are subjected to bending moments of magnitude $M_1$ and $M_2$ which are transferred from the beams. These moments can be represented as a force couple having a magnitude equal to the bending moment divided by the beam depth. The resultant shear force on the panel ($V_{PZ}$) can be represented as follows:

$$V_{PZ} = \frac{M_1 + M_2}{d_b} - V_{col} \tag{4}$$

where $V_{col}$ is the shear force resisted by the column. Specifications adopt Eq. (4) with modifications. Differences arise due to the differences in the values of bending moments, depth of the beam and consideration of