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A new steel panel zone model including axial force for thin to thick column flanges

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Abstract. During an earthquake, steel frame columns can be subjected to high axial forces combined with inelastic rotation demand resulting from story drift. Generally, the whole beam or component can be represented with one element. In elasto-plastic analysis, subdivision is necessary if the plastic deformation occurs within two ends of beams. If effects of the joint panel are necessarily considered in the analysis, the joint panel should be represented with an independent element. It is a special element to represent the shear deformation of the joint panel in the beam–column connection zone. Several analytical models for panel zone (PZ) behavior exist, in terms of shear force-shear distortion relationships. Among these models, the Krawinkler PZ model is the most popular one which is used in the AISC code. Some studies have pointed out that Krawinkler's model gives good results for the range of thin to medium column flanges thickness. This paper, introduces a new model to estimate the response of shear force-shear distortion for the PZ including column axial force. The model is applicable to both thin and thick column flange. To achieve an appropriate PZ mathematical model first, the effects of PZ strength and stiffness on connection response are parametrically studied using finite element models. More than one thousand and four-hundred beam-column connections are included in the parametric study, with varied parameters; then based on analytical results a simple mathematical model is presented. A comparison between the results of proposed method herein with FE analyses shows the average error especially in thick column flange is significantly reduced which demonstrates the accuracy, efficiency, and simplicity of the proposed model.

Keywords: panel zone; shear strength; beam-column connection; axial force; FEM

1. Introduction

Steel special moment-resisting frames (SMRFs) are one of the most popularly used lateral load resisting structural systems. They are considered to be most effective for this function because of their high ductility and high energy-dissipation capacity due, in turn, to plastic hinge formation in the beams and the column bases, and joint panel zone (PZ) shear deformation. The capability of SMRFs to resist lateral load is provided by frame action: the development of bending moments, and shear forces in the frame members and joints. Because of their high ductility, U.S. building codes assign the largest force reduction factors to SMRFs, thus obtaining the lowest lateral design

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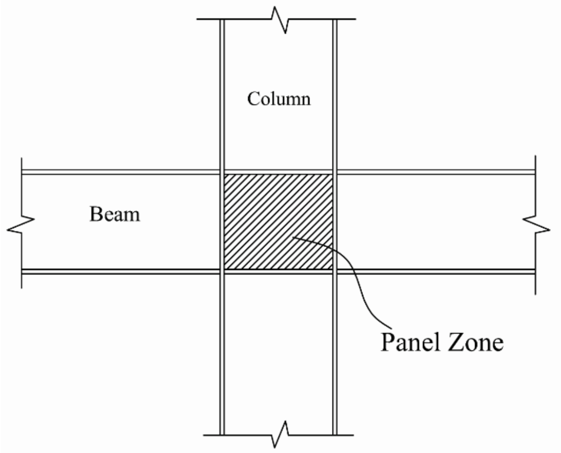

Fig. 1 Joint panel in steel frames

forces for an equivalent static analysis. From an architectural standpoint, SMRF systems allow a very effective use of space and maximum flexibility for openings layout, due to the absence of bracing elements or structural walls.

The PZ is the portion of the column within the depth of the connecting beams in a steel connection (Fig. 1). The transfer of moments between beams and columns causes a complicated state of stress and strain in the PZ. Under the action of forces, the PZ deforms in three modes: axial, shear, and bending. Usually only the shear deformation of the PZ has a significant effect on the behavior of steel frames and is of interest to designers. (Jin and El-Tawil 2005).

PZ design provisions have undergone large changes in the past four decades. The PZs 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 main reasons for this trend. First, then existing provisions (e.g., SEAOC 1975) ignored the contribution of column flanges to the shear strength of the PZ. Second, the moment demand on the connection was, in many cases, overestimated by: (1) assuming that framing beams could attain their full plastic capacity, and (2) disregarding gravity moments in computing connection demands. Since steel frame design is often 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 PZs. In addition, gravity moments on interior steel connections tend to counteract seismic moments; however, their effect is rather small especially in systems with a few perimeter moment resisting frames (El-Tawil 2000). When combined together, these two factors (underestimating strength and overestimating demand) often resulted in the need for PZ reinforcement, which was mainly provided through doubler plates (Jin and El-Tawil 2005).

Experimental investigations on PZ behavior initiated in the late 1960's and early 1970's, including Krawinkler *et al*. (1971), Bertero *et al*.(1973), and later on Popov (1987), showed that the PZ has high reserve strength after yielding, large ductility, stable hysteresis loops, and considerable cyclic strain hardening. In recognition of these observations, building codes increased the rated shear strength of the PZ by taking into attention 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 useful effects of gravity moments. The increased PZ strength and reduced shear demand meant that steel frames designed using these provisions could maintain larger inelastic PZ distortions during an earthquake compared to earlier frames (Jin and El-Tawil 2005).

Investigations by Tsai and Popov (1988) and El-Tawil *et al*. (1999) showed that PZs designed based on the abovementioned specifications could undergo large inelastic shear distortions before reaching their rated shear capacity. This causes problems at the connection welds since large PZ 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 lateral region, but also in the shear tab welds. Evidence indicates that weak PZ behavior may have played a role in the fractures that occurred during the Northridge earthquake (El-Tawil 2000).

PZ design provisions were made more accurate immediately after the Northridge earthquake. The design demand was increased to account for strain hardening and overstrength in beam steel (FEMA-267 1995). Two years later, FEMA-267A (1997) recommended that the design shear force should be calculated by supposing 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) mentioned that these demands could still be too low for interior connections and illogical and inappropriate for exterior connections.

FEMA-350 (2000), proposed design guidelines that are considerably different from previous provisions. The proposed rules are not a function of PZ strength (as defined in previous specifications) or beam plastic capacity. Rather, they are according to the premise that framing beams and the PZ 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 accepted into next seismic provisions published by AISC (2002). Rather, the new AISC provisions removed the 80% cap and specified demands based on full beam plastification, i.e., the provisions essentially reverted back to earlier PZ specifications, even though with some refinements. For example, 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 calculating demand.

Joint panel is the connection zone of beam and column members in steel frames. Subjected to reaction forces of the beam and column ends adjacent to a joint panel, three possible deformations can occur in the joint panel (Fig. 2): (a) stretch/contract, (b) bending and (c) shear deformations. Because of restraint of adjacent beams, stretch/contract and bending deformations of the joint panel are very small and can be ignored. Shear deformation is therefore dominant for the joint panel and an experimental deformation of the joint panel is shown in Fig. 3 (Li and Li 2007).

Generally, the whole beam or column component can be represented with one element. In elasto-plastic analysis, subdivision is necessary if the plastic deformation occurs within two ends of beams. If effects of the joint panel are necessarily considered in the analysis, the joint panel should be represented with an independent element. It is a special element to represent the shear deformation of the joint panel in the beam–column connection zone.

This element, models nonlinear shear deformation in the area of the joint where the beams and columns intersect. The joint region includes a length of column within the depth of the connecting beams. The shear deformation is due primarily to opposing moments from the columns and beams at the joint caused by the frame being subjected to lateral loads (Fig. 2; Krishnan and Hall 2006).

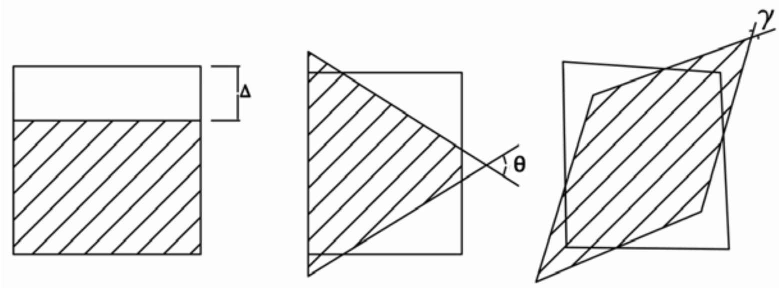

Fig. 2 Deformations of the joint panel (Li and Li 2007)

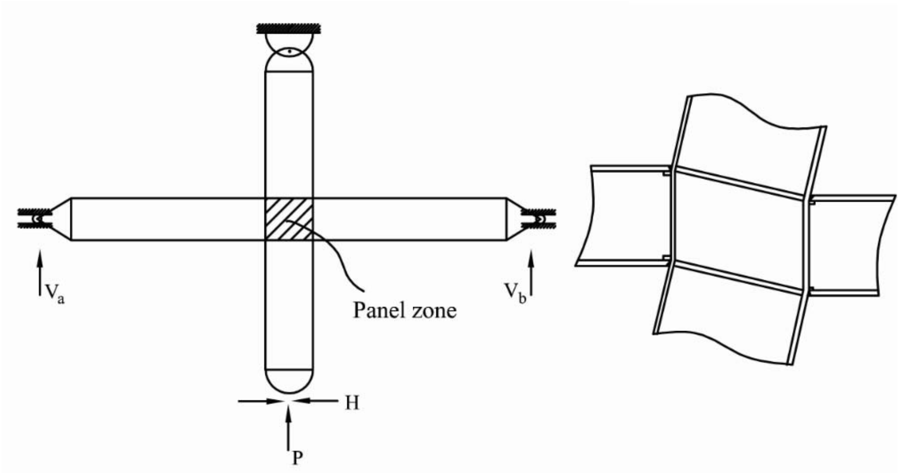

Fig. 3 Joint panel deformation (Li and Li 2007)

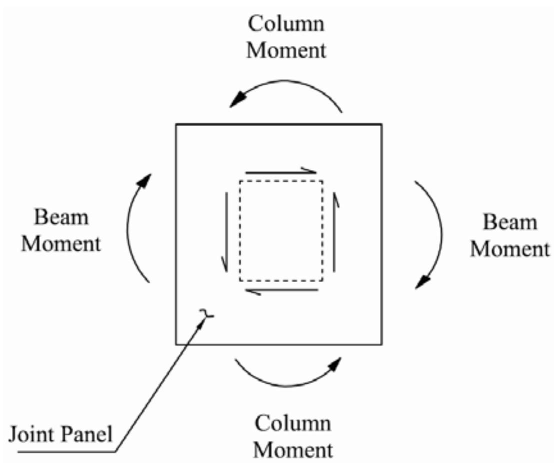

Fig. 4 Panel zone shear caused by beam and column moments at the joint (Krishnan and Hall 2006)

Well-proportioned moment-resisting connections supply large, stable, plastic rotational capacity. Certain modes of behavior, such as beam flange yielding and PZ yielding, are ductile whereas others may not be. The design aims at mobilizing yielding of at least one ductile element while precluding any undesirable failure modes.

Nonlinear time-history analysis of steel frames subjected to earthquake ground motions has showed that columns in the bottom stories are often subjected to combined high axial load and inelastic flexural demand resulting from story drift. Seismic loading usually results in double-curvature bending in these columns. Analysis results have revealed expected story drift ratios of approximately 2% (Sabelli 2001). This level of drift results in inelastic rotation demand combined with high axial force demand in the columns. The reliability of columns under this level of combined cyclic loading has not formerly been experimentally validated and little guidance is available in codes and standards of practice (Newell and Uang 2008).

The purpose of this paper is to discuss and evaluate the new PZ relationships. This is achieved by examining previously published test data and by analyzing more than one thousand and four-hundred finite element models.

2. Panel zone shear strength in the American code

The US design practices of MRFs structures has reflected the results of research activities performed starting from the 70s; especially up to the 1994 Northridge earthquake, a weak PZ–strong beam philosophy has been permitted. After the Northridge earthquake and the studies conducted in the contest of SAC Joint Venture, the American design codes (AISC 2005 and FEMA-350 2000) propose that, in the seismic design of MRFs, yielding may take the form of plastic hinging in the beams (strong PZ–weak beam philosophy), plastic shear deformation in the column PZs (weak PZ–strong beam philosophy) or, preferably, through a combination of these mechanisms (intermediate design philosophy; Brandonisio *et al*. 2012).

Seismic design provisions for the PZs have seen considerable changes in the past three decades as information regarding the cyclic behavior of the panel region has cumulated. As discussed in Popov (1987), there are principally three schools of thought for PZ design. The first approach, referred to hereinafter as the strong PZ approach, requires the PZ to remain elastic during seismic loading. Calculations made based on 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 (El-Tawil 2000).

According to test results that suggested that PZs are inherently ductile elements, an opposite design philosophy has been advocated for low-rise steel frames. In this approach, termed weak PZ design, the PZ is proportioned so that it absorbs most of the inelastic deformations in the structure during seismic loading (El-Tawil 2000). This philosophy may adversely affect connection ductility as is demonstrated by the information presented herein, and is counter to current thinking. There is growing consensus among structural engineers that excessive PZ deformation may be adverse to overall connection ductility (FEMA 1997).

The third design philosophy, which is a compromise between the above two approaches, requires the PZ to take part along with the beams in seismic energy dissipation. This methodology, is termed “balanced PZ design”.

The nominal shear strength R_n as well as the way to estimate the required shear strength R_u have been established by the specification and are related by the following expression $R_u = \varphi_v R_n$, where φ_v is the resistance factor for the PZ strength. The following expressions for PZ nominal shear strength design are specified by AISC 2010.

When the effect of PZ deformation on frame stability is not considered in the analysis:

For $P_r \leq 0.4P_c$

$$R_{ny} = 0.6F_y d_c t_w \tag{1}$$

For $P_r > 0.4P_c$

$$R_{ny} = 0.6F_y d_c t_w \left(1.4 - \frac{P_r}{P_c}\right) \tag{2}$$

when frame stability, including plastic PZ deformation is considered in the analysis:

For $P_r \leq 0.75P_c$

$$R_{np} = 0.6F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^{\ 2}}{d_b d_c t_w}\right) \tag{3}$$

For $P_r > 0.75P_c$

$$R_{np} = 0.6F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^{\ 2}}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2P_r}{P_c}\right) \tag{4}$$

where F_y is specified minimum yield stress of the column web, d_c is depth of column, t_w is thickness of column web, b_{cf} is width of column flange, t_{cf} is thickness of column flange, d_b is depth of beam, P_r is required axial strength, P_c is equal to P_y (axial yield strength of the column).

Eqs. (1) and (3) are based on Krawinkler (1978) proposal. The factor in Krawinkler model is $1/\sqrt{3} = 0.577$ while it is 0.6 in AISC code. Although slight modifications have occurred over time no significant modifications have been made. Some studies have pointed out that Krawinkler's model gives good results for the range of thin to medium column flanges thickness (Chen and Lui 1991).

3. Finite element model

To achieve an appropriate model, first an extensive parametric study regarding the effective factors on the behavior of PZ is carried out by ANSYS (2011) software. These parameters are: column flange thickness (t_{cf}), column web thickness (t_w) and beam flange thickness (t_{bf}). All parametric studies were done for SPE1 (Hedayat and Celikag 2009), SAC3 (Lee *et al*. 2000), SAC5 (Lee *et al*. 2000), SAC7 (Lee *et al*. 2000) specimens which represent a wide range of connections of different beam overall depths (from 450 mm to 912 mm). Details of these specimens are presented in Table 1.

The value of t_{cf} varies from 0.75, 1, 1.25, 1.5, 1.75 and 2 time of the original value in references. Similarly, t_w varies from 0.75, 1, 1.25, 1.5 and 1.75 time of the original value in references. Also, t_{bf} varies from 0.75 and 1, 1.5 2 time of the original value in references. The ratios P_r / P_c are 0.2, 0.4, 0.75 and 0.9. Therefore, the total number of made specimens are (4 specimens) × (6 column flange thicknesses) × (5 column web thicknesses) × (3 beam flange thicknesses) × (4axial load ratio) = 1440.

Table 1 Details of SPE and SAC group

Specimen	Type	Section	Yield stress (Mpa)
SAC7	Beam	W 36 × 150	250
	Column	W 14 × 257	345
SAC5	Beam	W30 × 99	250
	Column	W 14 × 176	345
SAC3	Beam	W24 × 68	250
	Column	W 14 × 120	345
SPE1	Beam	W18 × 46	250
	Column	W 14 × 82	345

Table 2 Geometric parameters of SAC and SPE specimens

specimen	Shear tab (mm)	No. of A325 SC Bolts (mm)	Continuity plate (mm)	Weld type and size (mm)	
				Beam flange	shear tab
SPE1	324 × 127 × 10.32	4Φ22	285 × 265 × 16	CJP, root opening = 9 mm, bevel angle = 30° and E70TG-K2	Fillet, 8 mm, E70T-7
SAC3	457 × 127 × 9.50	6Φ22	355 × 335 × 16		Fillet, 8 mm, E70T-7
SAC5	610 × 127 × 12.70	8Φ25	375 × 345 × 19		Fillet, 8 mm, E70T-8
SAC7	762 × 127 × 15.88	10Φ25	350 × 330 × 25		Fillet, 8 mm, E70T-7

For instance, Fig. 5 shows the details of the specimen SAC7. The length of the beam and the column for all these specimens were 3429 mm and 3658 mm respectively. Other geometric parameters of these specimens are summarized in Table 2. Both the shear tab and continuity plates were ASTM A36 (yield stress = 250 MPa) and all welds were E70TG-K2 electrode.

Version 14.0 of the general purpose nonlinear finite element program ANSYS was used to model 1440 fully restrained bolted web-welded flange beam-to-column moment connections. Shell-element models were prepared to study local and global instabilities in the connections because such models are computationally more efficient than solid-element models for this purpose (Kim 2000). A four-node shell element (Shell 181 element with six degrees of freedom at each node) has been used to model the specimens. Such elements were successfully employed by El-Tawil *et al*. (1998) for a related study funded by the SAC Joint Venture. The size of the finite element mesh varied over the length and height of the specimen. A fine mesh was used near the connection of the beam to the column. A coarser mesh was used elsewhere in order to reduce the computational efforts. Beam flanges were modeled using 5 layers of elements through the flange depth and 10 elements across the flange half-width. The distribution of geometric imperfections matched the first eigenvector of the loaded connection configuration. The maximum imperfection was chosen as one percent of the beam flange thickness.

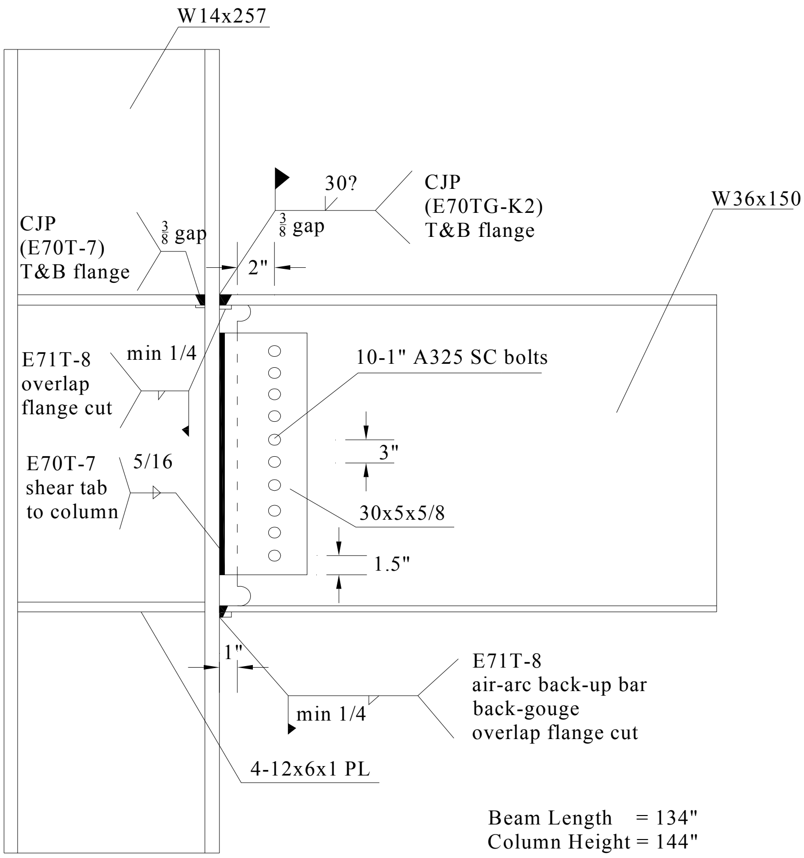

Fig. 5 Specimen SAC7 utilized by Lee *et al*. (2000)

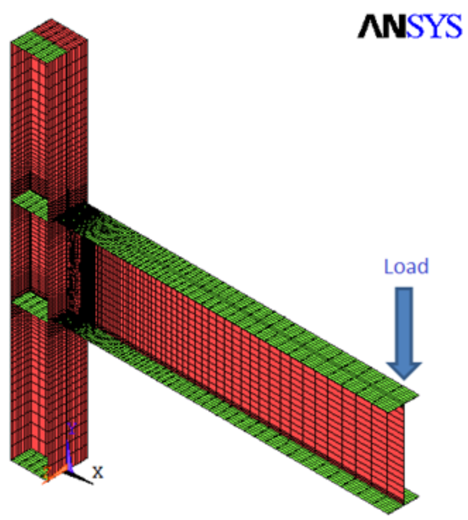

Fig. 6 Finite element model of specimens

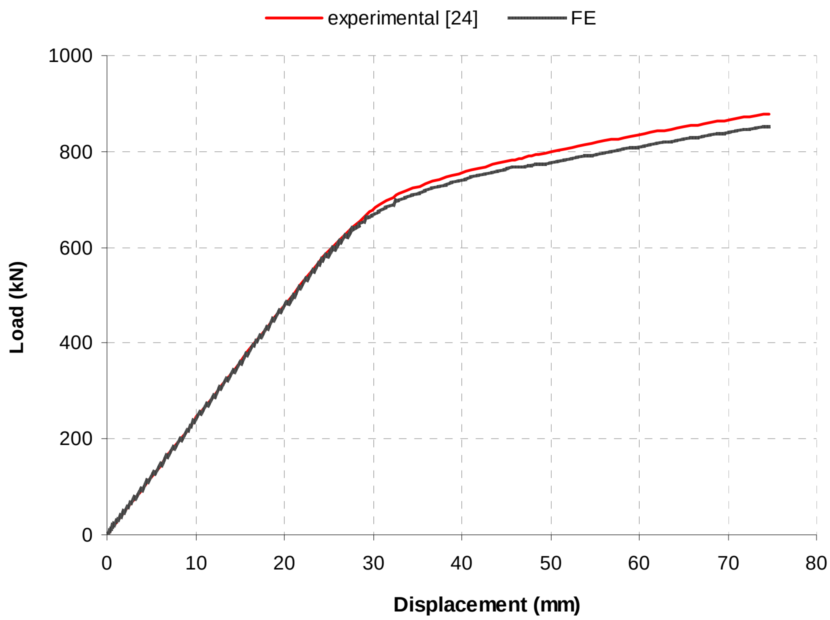

Fig. 7 Comparison between the predicted response in FE verification study and experimental results of SAC7 specimen

Two lines of nodes at each end of the column were restrained against translation only (i.e., a pinned connection) to approximately replicate the support conditions used for the laboratory tests. A vertical displacement history was imposed at the free end of the beam using the displacement control feature in ANSYS.

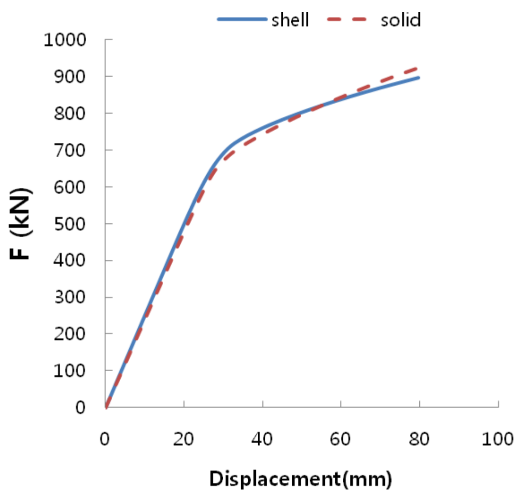

Fig. 8 Comparison between shell and solid elements in modeling of SAC7 specimen

Since verification is necessary for numerical models, before performing the parametric study some well-known experimental programs were considered to verify the finite element modeling methodology and general assumptions on the nonlinear analysis.

3.1 Verification study

To verify the accuracy of finite element modeling, specimens SAC 7 (Fig. 6) and specimen SPE1 were remodeled using finite element method. Shown in Fig. 7 is a comparison between analytical and experimental results. As this figure shows, the analytical result is in good agreement with experimental result.

In order to compare the results of shell and solid elements, the specimen SAC7 which has a thick column flange (45 mm) is modeled again using solid element. A comparison between shell and solid elements can be seen in Fig. 8.

The typical load response from the panel was characterized by three phases. First, elastic shear response followed by yielding, according to the von Mises criterion. Second, reserve in strength

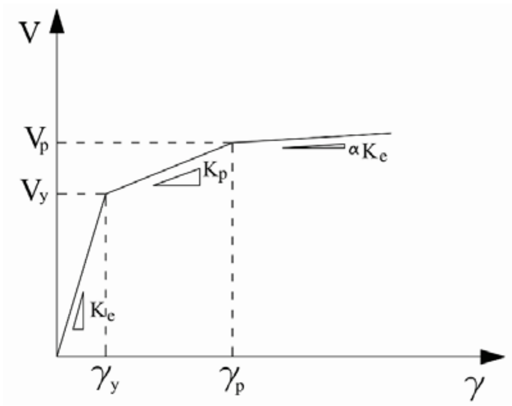

Fig. 9 Shear force-shear strain relationship for panel zone

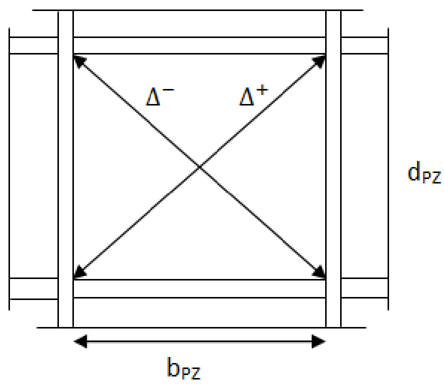

Fig. 10 Geometry of panel zone

corresponded to the surrounding elements of the panel. Finally, a post yield strength characterized by strain hardening of the steel. The elements that determine the stiffness and strength of a PZ are the web and the flange of a column. The sum of these two elements determines the shear-force shear-distortion ($V-\gamma$) curve of a PZ, and shows the trilinear behavior (Fig. 9).

PZ shear force and PZ shear distortion are computed based on the following relations, respectively (Ricles *et al*. 2004)

$$V_{PZ} = \frac{M_{b1} + M_{b2}}{h_t}(1-\rho) \tag{5}$$

$$\gamma = \frac{\Delta^{+} - \Delta^{-}}{2} \frac{\sqrt{d_{PZ}^2 + b_{PZ}^2}}{d_{PZ} b_{PZ}} \tag{6}$$

where M_{b1} and M_{b2} are, respectively, the beam end moments at the column faces; h_t is distance between center to center of beam flanges and ρ is defined as $\rho = h_t / H - d_b$ in which H is the story height and d_b is beam depth. Δ^{+}, Δ^{-} are the displacements of diagonal panel zone, and d_{PZ}, b_{PZ} are the vertical and horizontal distances of the panel zone (see Fig. 10), respectively.

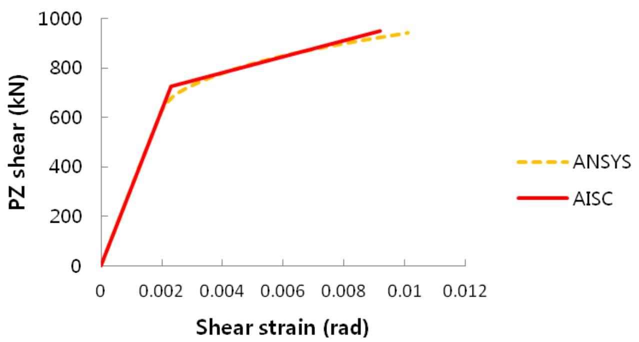

Fig. 11 PZ shear force (V_{PZ}) – PZ shear distortion (γ) for a thin column flange (SPE1) for axial load ratio 0.5

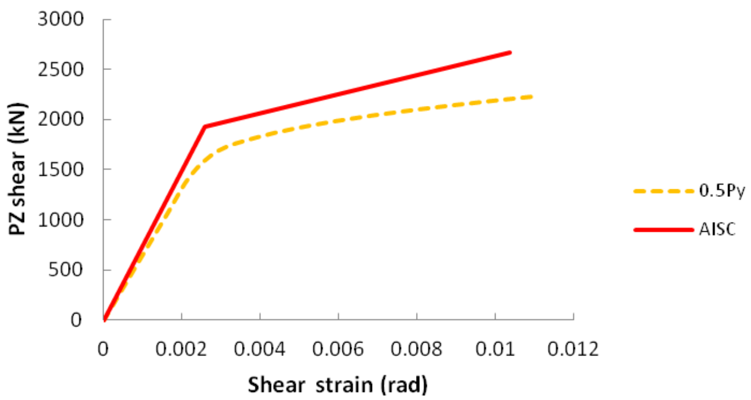

Fig. 12 PZ shear force (V_{PZ}) – PZ shear distortion (γ) for a thick column flange (SAC7) for axial load ratio 0.5

The AISC 2010 PZ model proposes relationships between PZ shear force and deformation for monotonic loading. These relationships have been used as a basis of mathematical models for nonlinear rotational springs representing the PZ. As it can be seen in Fig. 11, AISC model gives good results for joints with thin to medium thickness column flanges.

However, it is pointed out by Krawinkler that a new model might be needed for joints with thick column flanges. This issue can be observed in Fig. 12.

A common technique when a curve consistently has an elastic-plastic shape but with a gradual transition from elastic to plastic is to draw a tangent for the elastic stiffness and the plastic stiffness and to set the yield point at their intersection.

The typical load response from the panel was characterized by three phases. First, elastic shear response followed by yielding. Second, reserve in strength corresponded to the surrounding elements of the panel. Finally, a post yield strength characterized by strain hardening of the steel. An appropriate value of the strain-hardening can be assumed to fully define the tri-linear shear force-shear deformation relationship of the panel zones; in this study 4% strain-hardening is ascribed to the joint shear-shear distortion.

4. Proposed analytical model

Column flange thickness effects PZ yield shear and elastic stiffness (Kim and Engelhardt 2002). The finite element results indicate that there is a reduction of the yield and ultimate shear strengths which can be obtained using the following relations.

- When the effect of PZ deformation on frame stability is not considered in the analysis:

The portion of the shear force resisted by the web for an I-shaped section can be determined as follows

$$\beta = 1 - \frac{1}{2\alpha}\left(\alpha + \frac{(1-2\alpha)^3 - 1}{6}\right) \qquad \text{where} \quad \alpha = \frac{t_{cf}}{d_c} \tag{7}$$

On the other hand, based on von Mises criterion

$$\left(\frac{R_{ny}}{\beta R_{ny0}}\right)^2 + \left(\frac{P_r / A_c}{F_y}\right)^2 = 1 \tag{8}$$

in which A_c is cross-sectional area of the column and

$$R_{ny0} = 0.6F_y d_c t_w$$

therefore the nominal shear strength is given by

$$R_{ny} = 0.6F_y d_c t_w \left(\beta\sqrt{1 - \left(\frac{P_r / A_c}{F_y}\right)^2}\right) \tag{9}$$

- when frame stability, including plastic PZ deformation is considered in the analysis

$$R_{np} = R_{ny} + 3\gamma_y K_p$$
$$= 0.6F_y d_c t_w \left(\beta \sqrt{1 - \left(\frac{P_r / A_c}{F_y} \right)^2} \right) + 3 \left(\frac{\sqrt{F_y^2 - (P_r / A_c)^2}}{G\sqrt{3}} \right) \left(\frac{1.095 b_c t_{cf}{}^2 G}{d_b} \right) \tag{10}$$

after simplifying and using the curve fitting technique on data bank of analyses

$$R_{np} = 0.6F_y d_c t_w \lambda \times \mu^{\frac{1}{3}} \tag{11}$$

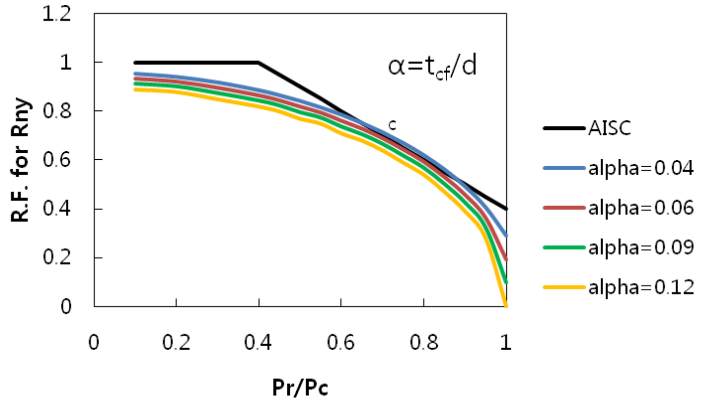

Fig. 13 Comparison between reduction factor (R.F.) for Rny suggested by AISC and Present study

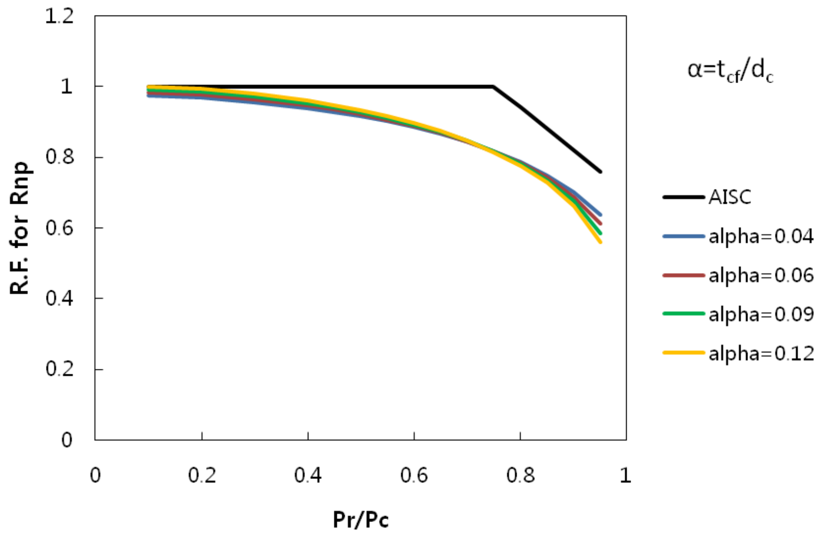

Fig. 14 Comparison between reduction factor (R.F.) for Rnp suggested by AISC and Present study

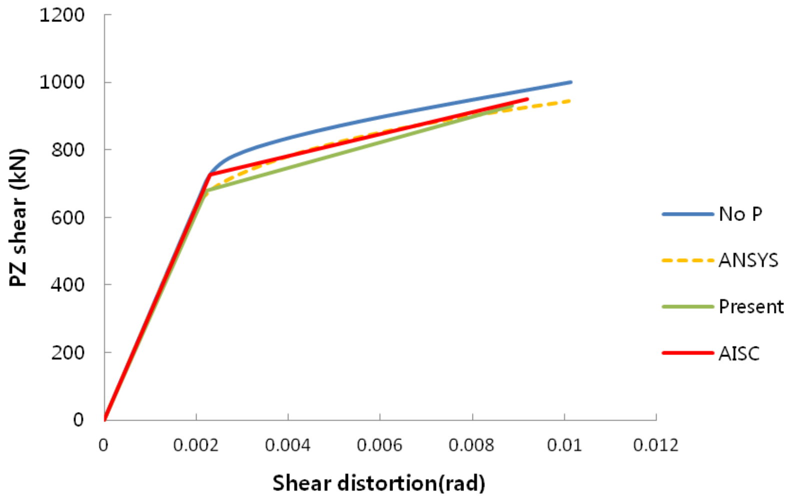

Fig. 15 Comparison of present model for a thin column flange (SPE1) for axial load ratio 0.5

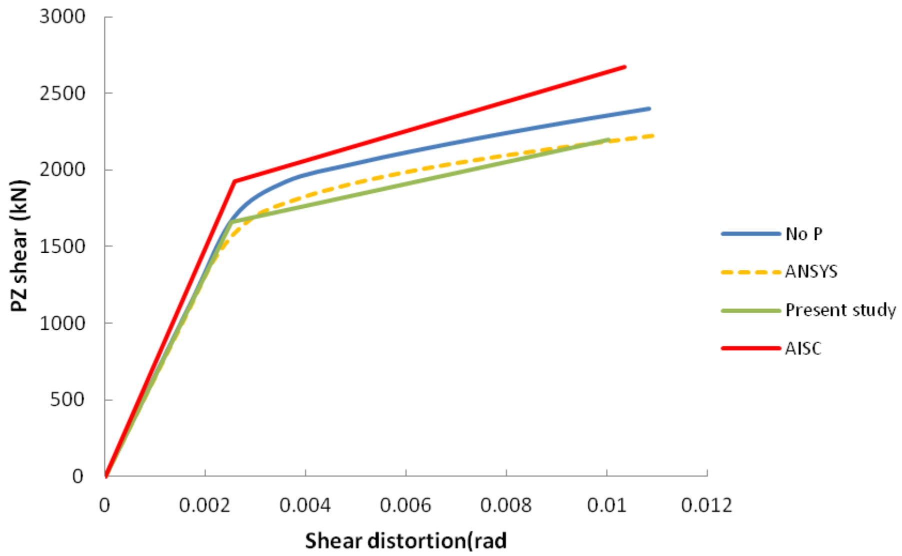

Fig. 16 Comparison of present model for a thick column flange (SAC7) for axial load ratio 0.5

where $\lambda = \left(1 + \dfrac{3b_{cf}t_{cf}^{\ 2}}{d_b d_c t_w}\right)$ and $\mu = \dfrac{1}{\lambda}\left[\beta\sqrt{1-\left(\dfrac{P_r / A_c}{F_y}\right)^2} + \dfrac{3.17 b_{cf}t_{cf}^{\ 2}}{d_b d_c t_w F_y}\sqrt{F_y^2 - (\dfrac{P_r}{A_c})^2}\right]$.

Reduction factor (R.F.) is the factor of $0.6F_y d_c t_w$ in the case of yield strength (R_{ny}) and it is the factor of $0.6F_y d_c t_w\left(1+\dfrac{3b_{cf}t_{cf}^{\ 2}}{d_b d_c t_w}\right)$ in the case of ultimate strength (R_{np}).

The applicable form of Eqs. (9) and (11) are plotted in Figs. 13 and 14, respectively.

It is assumed that strain hardening begins at $\gamma = 4\gamma_y$. The axial force of column is constant during analysis.

A comparison between this model and all of 1440 FE models shows the average and maximum error among are equal to 1.05% and 10.06%, respectively. For instance, Figs. 15 and 16 show samples that using these corrections (Eqs. (9) and (1)1), the proposed trilinear model is compatible with FE results, especially in the case of thick column flanges. In these figures "No P" indicates case of without axial force.

Figs. 17 through 24 show the variation of column flanges thickness for the nominal shear strength when the effect of PZ deformation on frame stability is not considered in the analysis. In these figures "V ANSYS" means the shear panel zone obtained from ANSYS software at the

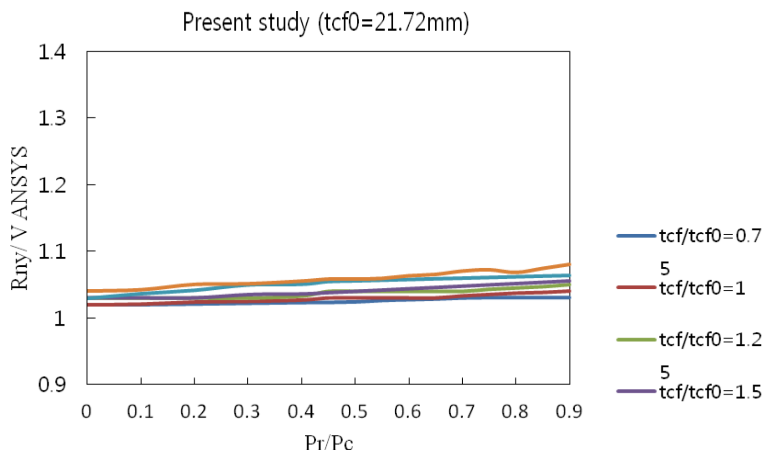

Fig. 17 Present study: Variations of column flange thickness in non-dimensional shear yield strength of panel zone in generated specimens from SPE1

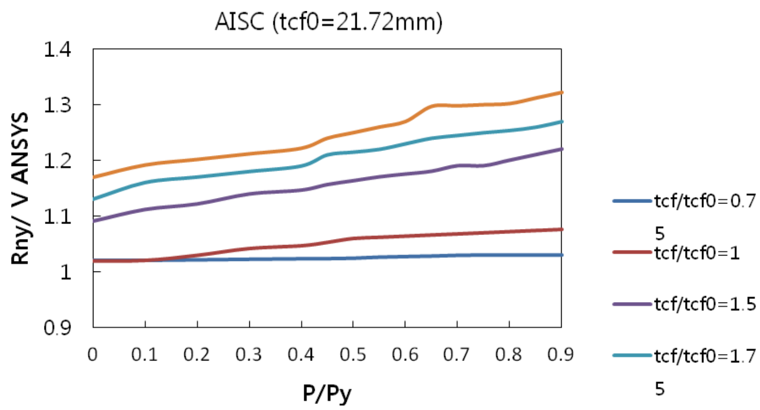

Fig. 18 AISC: Variations of column flange thickness in non-dimensional shear yield strength of panel zone in generated specimens from SPE1

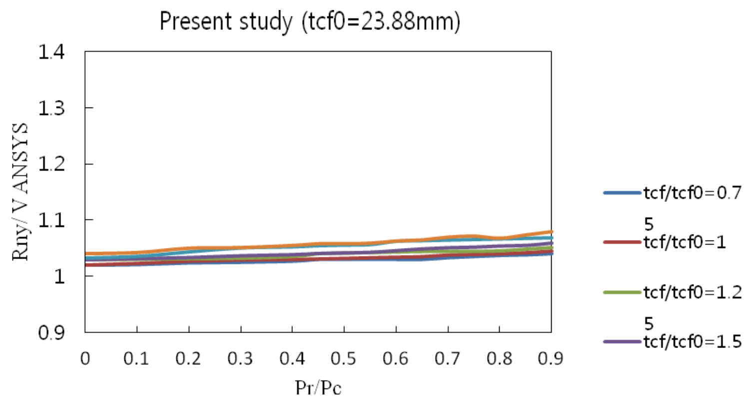

Fig. 19 Present study: Variations of column flange thickness in non-dimensional shear yield strength of panel zone in generated specimens from SAC3

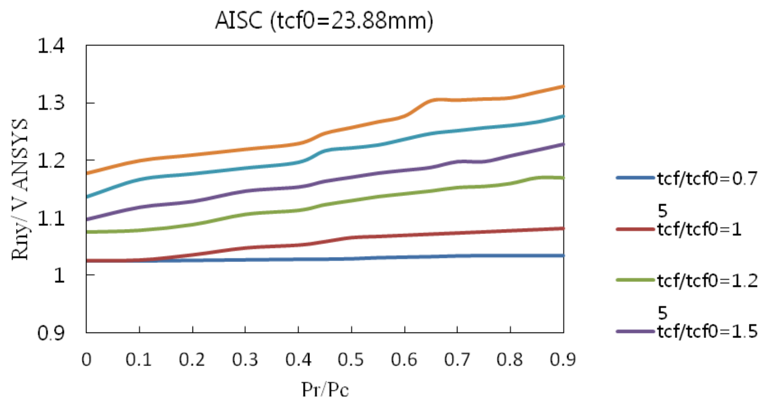

Fig. 20 AISC: Variations of column flange thickness in non-dimensional shear yield strength of panel zone in generated specimens from SAC3

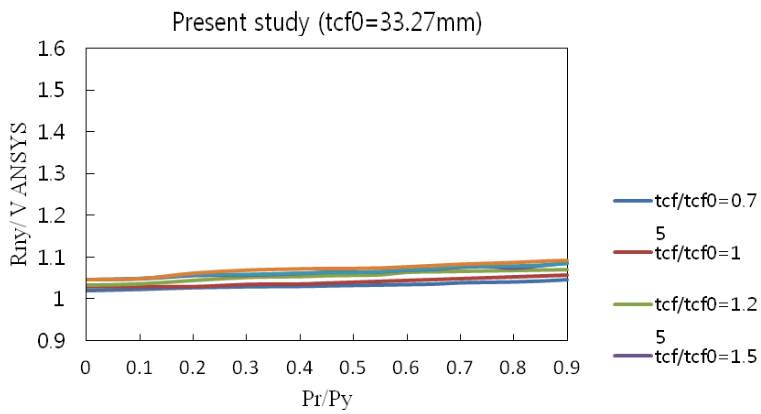

Fig. 21 Present study: Variations of column flange thickness in non-dimensional shear yield strength of panel zone in generated specimens from SAC5

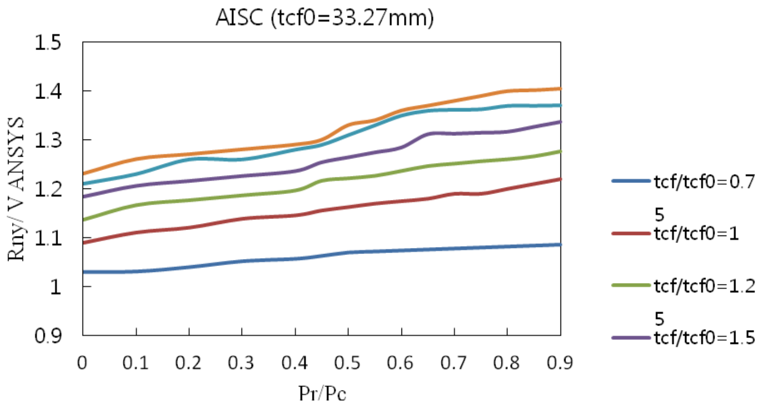

Fig. 22 AISC: Variations of column flange thickness in non-dimensional shear yield strength of panel zone in generated specimens from SAC5

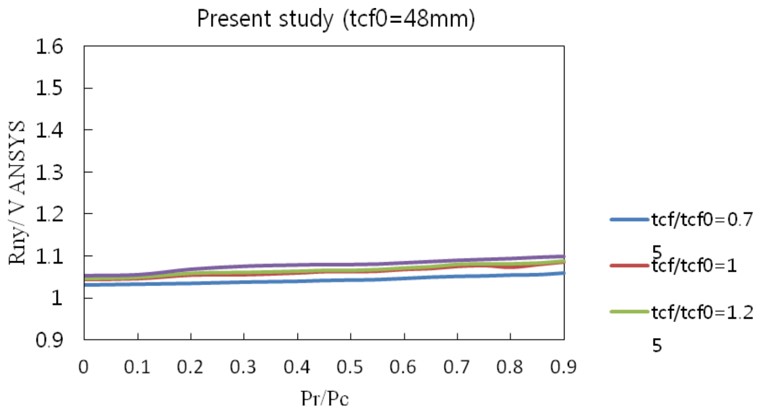

Fig. 23 Present study: Variations of column flange thickness in non-dimensional shear yield strength of panel zone in generated specimens from SAC7

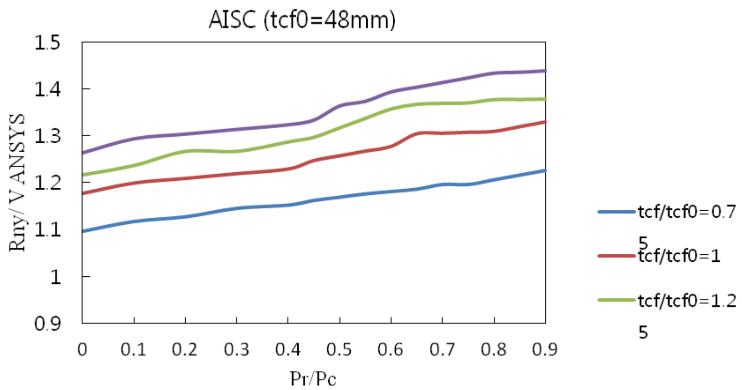

Fig. 24 AISC: Variations of column flange thickness in non-dimensional shear yield strength of panel zone in generated specimens from SAC7

Table 3 Errors in PZ shear strength in comparison with 1440 FE models if PZ deformation is not considered in the analysis

PZ deformation is not considered in the analysis	R_{ny}	
Model	AISC	Proposed model
Average error (%)	19.05	1.05
Max error (%)	38.24	8.62

Table 4 Errors in PZ shear strength in comparison with 1440 FE models if Plastic PZ deformation is considered in the analysis

Plastic PZ deformation is considered in the analysis	R_{np}	
Model	AISC	Proposed model
Average error (%)	11.11	1.08
Max error (%)	16.10	10.06

center of PZ. As can be seen in these figures, when proposed mathematical model is used errors are reduced compared with those taken by AISC relations.

Also, the obtained errors of the proposed mathematical model and other mathematical models in comparison with 1440 FE models are listed in Tables 3 and 4 to show the accuracy of the present model.

Tables 3 and 4 show that the model introduced in the present work has better performance as compared to the AISC code.

5. Conclusions

The purpose of thin panel zone yielding during an earthquake motion is to absorbing energy at the panel zone instead of undesirable column locations. A good structural design is for strong columns and weak beams or panel zones. If a structure is designed with thick panel zones so a plastic hinge can be formed in the beam. This is a desirable ductile failure mode so that people can escape without structural collapse. A mathematical model was represented using a combination of rigid and flexible components by means of stiffness and resistance values obtained from empirical relationships. The nonlinearity of the response is obtained by means of inelastic constitutive laws used for the spring elements. Extensive finite-element analyses were conducted to study the effect of axial force of column on PZ yielding. A new approach for representing the PZ component in steel moment-resisting frames is proposed in this paper including the axial force of column. Validation of the proposed approach is carried out by comparison against available experimental results coupled with detailed numerical simulations. The comparisons illustrate the accuracy, simplicity and reliability of the approach developed, and its general applicability to both thin and thick column flanges.

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