패널 영역의 단조거동에 대한 모델링

Modelling for Monotonic Behavior of the Panel Zone

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요 약 : 단조하중(monotonic loading)하의 강재 보-기둥 절점의 패널영역(panel zone) 거동을 정의하기 위해 해석요소를 개발하였다. 본 해석요소를 위한 하중-변위관계로 단조모델(monotonic model)을 제안했는데 이 모델은 실험결과와 좋은 상관관계를 보여주었다. 개발된 해석요소인 패널영역 요소(panel zone element)는 단순하면서도, 이를 이용한 구조해석결과가 실험결과와 잘 일치하는 것으로 나타났다. 한편 단조모델에 근거한 좀 더 현실적인 패널 영역의 설계전단강도를 제안하였다.

핵 심 용 어 : 패널 영역, 지진 거동, 단조 하중, 전단 강도, 해석 모델

1. Introduction

In this paper, a mathematical model is developed for describing the monotonic load-deformation response of the panel zone region at the bare steel beam-to-column joint of a steel moment frame. This study was performed as a supplement to a companion paper (Kim 1997) on the cyclic behavior of the panel zone of bare steel beam-to-column joints. The panel zone is the portion of the column contained within the beam-to-column joint. When a moment frame is subjected to lateral loads, high shear forces develop within the panel zone. The resulting deformations of the panel zone can have an important effect on the response of the frame in both the elastic and inelastic ranges of frame behavior (Tsai 1988).

Numerous tests have been performed in the past two decades to investigate the load-deformation behavior of the joint panel using connection subassemblies. Some significant

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observations from these tests are:

i) Joint panel zones often develop a maximum strength that is significantly greater than the strength at first yield. However, large inelastic panel zone deformations are typically required in order to develop maximum panel zone strength.

ii) Panel zone deformations can add significantly to the overall deformation of a subassembly, for both elastic and inelastic behavior of the panel zone.

iii) Panel zone stiffness and strength can be increased by the attachment of web doubler plates to the column within the joint region. The effectiveness of doubler plates is affected by the method used to connect them to the column.

iv) In the inelastic range, panel zones can exhibit very ductile behavior, both for monotonic and cyclic loading. Experimentally observed hysteresis loops are typically very stable, even at large inelastic deformations.

The Uniform Building Code since its 1988 Edition and LRFD manual since its 1991 Edition permit the formation of plastic hinges in the panel zones of steel moment frames under earthquake loading. Thus, rather than forming flexural hinges in the beams or columns, the primary source of energy dissipation in a steel moment frame can be the formation of plastic shear hinges in the panel zones. Consequently, an analytical model is needed to predict the monotonic and cyclic inelastic response of a panel zone.

To include panel zone deformation in frame analysis, the traditional center-to-center line representation of the frame must be modified. Figure 1 shows a comparison of experimental results and analytical results for Krawinkler Specimen A-1 (Krawinkler 1971). The experimental overall response of the specimen was dominated by the yielding of the panel zone. The analytical results are obtained by using center-to-center line dimensions of the specimen. From the figure it can be seen that the analysis using the center-to-center line dimension or rigid offset, which can not account for the yielding of a panel zone, may produce misleading results for a frame with weak panel zones in which the yielding is permitted.

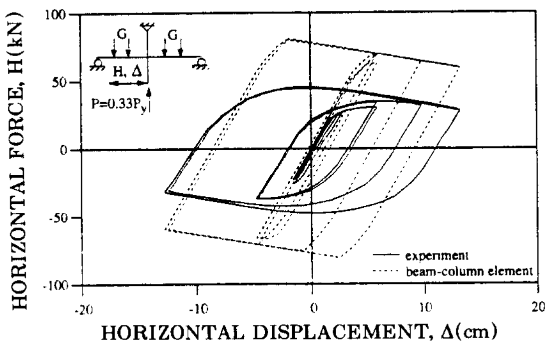


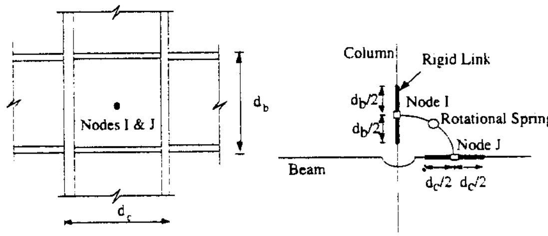
FIG. 1 Comparison of Test Results and Analytical Results Obtained by Using Center-to-Center Line Dimension Modeling(Krawinkler 1971)

To model the behavior of panel zones in frame analysis, Lui (1985) developed a joint model based on the finite element method. The model consists of seven elements for interior beam-to-column joints: one web element, two flange elements (beam elements), and four connection elements. Although capable of representing a variety of deformation modes of panel zones, this model employed a simple hardening rule suitable for the monotonic loading and therefore cannot realistically model cyclic behavior. Another disadvantage of this model is its high computational cost. Other finite element models using more sophisticated hardening rules could be developed for the analysis of column panel zones. However, in this study it was decided to use nonlinear rotational springs as the basis for modeling the panel zone in the nonlinear dynamic analysis of moment resisting frames because of its simplicity and computational efficiency.

Several researchers (Fielding 1971; Krawinkler 1971; Wang 1988; etc.) proposed panel zone shear force V-panel zone deformation γ relations for monotonic loading. These relationships have been employed as a mathematical model required for nonlinear rotational springs and Krawinkler's V-γ relations were used to determine the design shear strength (LRFD Eq. 8-1) of the panel zone of LRFD manual. However, it was pointed out by Krawinkler that a new model might be needed for joints with thick column flanges since his V-γ relations were derived empirically from test and analytical results of thin to medium thick column flange joints. Wang also showed that Krawinkler's V-γ relations overestimated the panel zone shear strength of the joints with thick column flanges. In this study, existing panel zone shear force V-panel zone deformation γ relations will be examined by making comparisons with experimental data or with other analytical models. Where these comparisons show poor performance of the existing models, the model will be improved and a recommendation for the design shear strength of the panel zone will be made.

2. General Characteristics of Panel Zone Element

The panel zone element is essentially a rotational spring element, which transfers moment between the columns and beams framing into a joint (Kanaan and Powell 1973). The panel element has no dimension and connects two nodes with the same coordinates. One of these nodes is attached to the element(s) modeling the columns framing into the joint, as shown in Fig. 2, while the other node is attached to the element(s) modeling the beams. Therefore, the moment transferred by the panel element is related to relative rotation between the columns and beams framing into a joint. The vertical and horizontal translations of the two nodes are constrained to be identical so that the column and beam ends move together.



(a) Interior Joint (b) Idealization of Interior Joint

FIG. 2 Idealization of Beam-to-Column Joint

Therefore, one vertical, one horizontal, and two rotational degrees of freedom exist at each joint.

The relative rotation between the connected nodes is related to the node rotations as follows:

$$d\gamma = \{1 \ \ -1\}\begin{Bmatrix} d\theta_I \\ d\theta_J \end{Bmatrix} \tag{1}$$

$$d\gamma = \mathbf{a} \cdot \mathbf{dr} \tag{2}$$

or

where $d\gamma$ is the increment of relative rotation (panel element deformation), and $d\theta_I$ and $d\theta_J$ are the increments of rotation of the connected nodes.

Then the tangent stiffness relationship is

$$dM^{pa} = K_t\, d\gamma \tag{3}$$

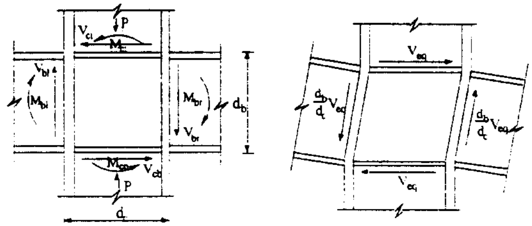
where dM^{pa} is the increment of moment applied on a joint and K_t is the rotational tangent stiffness of the joint. In terms of nodal rotations, the stiffness, $\mathbf{K}_T$, is given by

$$\mathbf{K}_T = \mathbf{a}^T K_t\, \mathbf{a} \tag{4}$$

The definable properties of the panel zone element are the rotational stiffnesses and yield moments for a monotonic loading, and hysteretic rules for a cyclic loading. In the following sections, existing models for a monotonic loading are examined and a new model is presented. This is followed by the development of hysteretic rules for a cyclic loading model in the companion paper (Kim 1997).

3. Review of Existing Monotonic Models

Three researchers (Fielding 1971; Krawinkler 1971; Wang 1988) have proposed their mathematical models to predict panel zone behavior under monotonic loading. Their mathematical models are based on the equivalent shear force as explained below.



a) Panel Boundary Forces b) Equivalent Panel Shear Forces

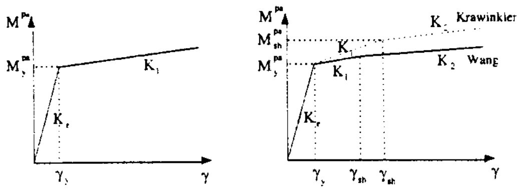
FIG. 3 Boundary Forces and Equivalent Shear Forces on Panel Zone

The boundary forces on a joint panel shown in Fig.3 can be transformed into an approximate equivalent shear force from equilibrium as follows;

$$\begin{aligned} V_{eq} &= \frac{M_{bl}+M_{br}}{d_b-t_{bf}} - \frac{(V_{ct}+V_{cb})}{2} \\ &\approx \frac{M_{bl}+M_{br}}{d_b-t_{bf}} \cdot (1-\rho) \\ &= \frac{M^{pa}}{d_b-t_{bf}} \cdot (1-\rho) \end{aligned} \tag{5}$$

where $\rho=(d_b-t_{bf})/H_c$, $M^{pa}=M_{bl}+M_{br}$ = panel zone moment, t_{bf} = thickness of beam flange, and H_c = column height. A key simplification in this analysis is that the beam moments are replaced by an equivalent couple, with the forces acting at mid-depth of the beam flanges. These forces produce a large shear in the panel zone. The shear in the column segments outside of the panel zone are then subtracted to obtain the net shear force, V_{eq}, acting on the panel zone. In obtaining the shear forces in the column segments outside of the panel zone, it is assumed that points of inflection in the column occur at a distance $H_c/2$ above and below the panel zone.

Existing panel zone shear force V-panel zone deformation γ relations for monotonic loading, which are based on the equivalent shear force V_{eq}, can be transformed into panel zone moment M^{pa}-panel zone deformation γ relations by Eq. 5. Fielding proposed a bilinear relationship as shown in Fig. 4a. Krawinkler and Wang each proposed different trilinear M^{pa}-γ relations as shown in Fig. 4b.



(a) Fielding's Bilinear Model

(b) Krawinkler and Wang's Trilinear Models

FIG. 4 Existing Panel Zone Moment-Panel Zone Deformation Models

In the elastic range, these three researchers all assumed that the in-plane stiffness of the beams framing into the column is sufficiently large to justify the assumption of rigid boundaries around the panel zone. Therefore, the shearing stress is uniformly distributed throughout the panel, and the elastic stiffness of the panel zone is determined by considering the area of the panel zone web. Fielding and Krawinkler considered the effective shear area A_{eff} of $(d_c - t_{cf}) \cdot t_{cw}$, and Wang considered the effective shear area A_{eff} of $(d_c - 2t_{cf}) \cdot t_{cw}$, where the subscripts 'c', 'f', and 'w' stand for column, flange, and web, respectively. They suggested the yield moment and elastic stiffness of panel zones as follows:

$$M_y^{pa} = \frac{V_y \cdot d_b}{(1-\rho)} = \frac{\tau_y \cdot A_{eff} \cdot d_b}{(1-\rho)} \tag{6a}$$

$$K_e = \frac{M_y^{pa}}{\gamma_y} = \frac{G \cdot A_{eff} \cdot d_b}{(1-\rho)} \tag{6b}$$

where V_y is the yield shear force of the panel zone, $\gamma_y = \tau_y / G$, and τ_y is the Von Mises yield shear stress of the column web, based on shear and axial force interaction. The Von Mises yield shear stress τ_y is taken as:

$$\tau_y = \frac{\sigma_y}{\sqrt{3}} \cdot \sqrt{1-(P/P_y)^2} \tag{7}$$

where P and P_y are axial force and axial yield force on the column, respectively, and σ_y is the yield stress of the column web.

For the inelastic range, Fielding considered a bilinear model with the following post-elastic stiffness K_1:

$$K_1 = \frac{5.2 \cdot G \cdot b_{cf} \cdot t_{cf}^3}{d_b \cdot (1-\rho)} \tag{8}$$

where b_{cf} and t_{cf} are width and thickness of column flange, respectively. Krawinkler proposed empirical formulas for the post-elastic stiffness K_1 and the second yield moment M_{sh}^{pa}, which is essentially associated with the de sign shear strength (LRFD Eq. 8-1), as follows:

$$K_1 = \frac{1.04 \cdot G \cdot b_{cf} \cdot t_{cf}^2}{(1-\rho)} \tag{9}$$

$$M_{sh}^{pa} = M_y^{pa} + \frac{3.12 \cdot \tau_y \cdot b_{cf} \cdot t_{cf}^2}{(1-\rho)} \tag{10}$$

Wang suggested the post-elastic stiffness K_1 as follows:

$$K_1 = 0.7 \cdot G \cdot b_{cf} \cdot t_{cf}^2 \tag{11}$$

Krawinkler and Wang assumed that strain hardening begins at $\gamma_{sh} = 4\gamma_y$ and $3.5\gamma_y$, respectively. The strain-hardening branch stiffness K_2 was suggested as follows:

$$K_2 = \frac{G_{st} \cdot A_{eff} \cdot d_b}{(1-\rho)} \tag{12}$$

where G_{st} is the strain hardening shear modulus.

The existing models have been applied to four specimens tested by Krawinkler (1971), Fielding (1971), and Slutter (1981). Figures 5, 7, 8, and 10 show test results compared with the existing monotonic models. Additional comparison is shown in Figs. 6, 9, 11, and 12. In these figures, finite element analysis predictions are provided for Slutter's Specimen 1. This specimen was analyzed a number of times, varying the column flange thickness. The finite element analyses provide an indication of the expected response of panel zones in columns with thick flanges because only limited experimental data is available for cases with thick column flanges. These analyses were also reported by Wang (1988). The specimen

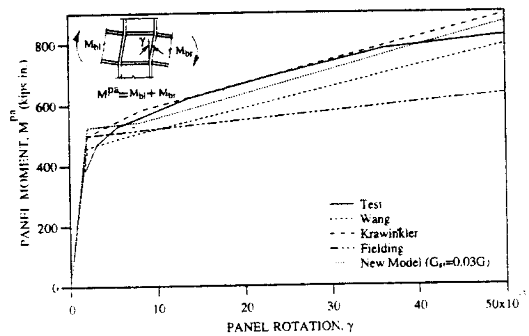


FIG. 5 Comparison of the Monotonic Model and Test Data for Krawinkler Specimen A2 with t_{cf}=1cm (Krawinkler 1971)

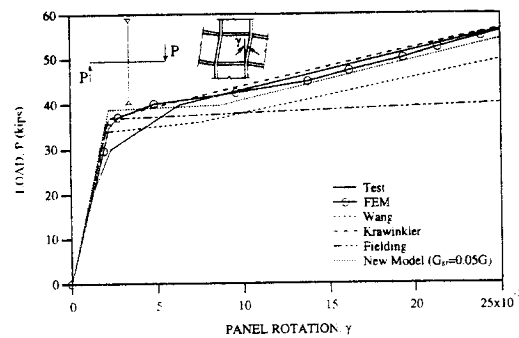


FIG. 7 Comparison of the Monotonic Model and Test Data for Slutter Specimen 1 with t_{cf}=1.8cm(Slutter 1981)

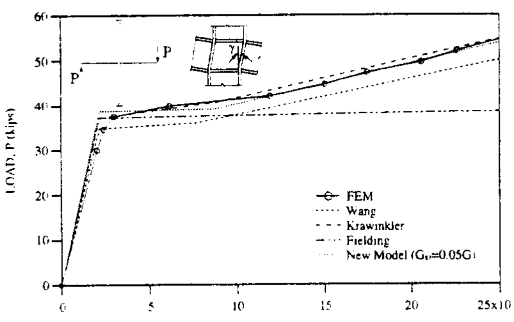


FIG. 6 Comparison of the Monotonic Model and FEM Results for Slutter Specimen 1 with t_{cf}=1.35cm (Wang 1981)

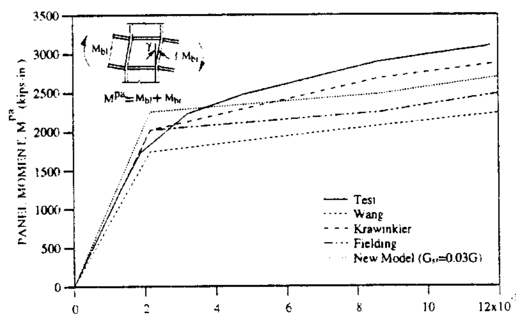


FIG. 8 Comparison of the Monotonic Model and Test Data for Krawinkler Specimen B2 with t_{cf}=2.37cm (Krawinkler 1971)

tested by Slutter is analyzed by using two-dimensional finite element model, in which flanges and webs are represented by beam elements and plain stress elements, respectively. The analytical results are compared with the corresponding test data in Fig. 7 and the correlation is reasonable. This comparison shows that the finite element analysis can properly predict the panel zone behavior under monotonic loading.

Fielding's bilinear model shows the poorest performance at large rotations regardless of column flange thickness because this model neglects strain-hardening. The performance of Krawinkler's model appears acceptable for panel zone joints with column flange thickness less than about one inch. However, for thicker column flanges, this model significantly overestimates panel zone strength. It appears that Krawinkler's model significantly overestimates the contribution of the column flanges and LRFD Eq. 8-1 significantly overestimates the design shear strength for thicker column flanges. Wang's model generally underestimates panel zone strength regardless of column flange thickness apparently because in this model, the effective shear area of the panel zones is calculated as $(d_c-2t_{cf}) \cdot t_{cw}$ instead of the other models' effective shear area $(d_c-t_{cf}) \cdot t_{cw}$.

4. Proposed Monotonic Model and Comparison with Test Data

In this study, to account for the fact that Fielding's model performed well except at large panel zone rotations regardless of the column flange thickness, simple modifications were applied to Fielding's model to provide better correlation with test and FEM results. Modifications to the model are accomplished as follows. i) The effective shear area of the panel zone is calculated as the column web area of $d_c \cdot t_{cw}$ because, as the ratio of column flange thickness to column depth increases, the influence of column flange thickness on panel zone yield moment and elastic stiffness increases. ii) Assuming that strain-hardening starts at $4 \cdot \gamma_y$ (Krawinkler 1971), strain-hardening effects are added after the post-elastic stiffness K_1 which is the second branch of the M^{pa}-γ relations. iii)

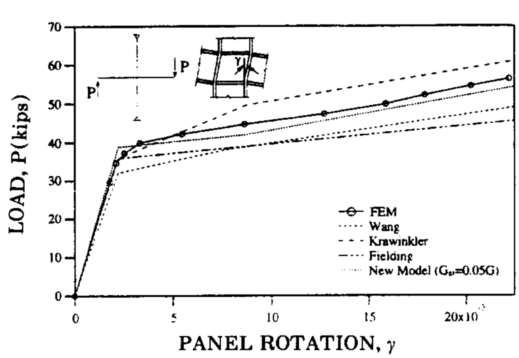


FIG. 9 Comparison of the Monotonic Model and FEM Results for Slutter Specimen 1 with t_{cf}=2.7cm (Wang 1988)

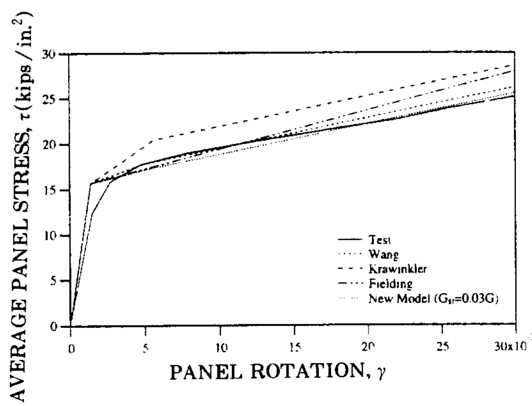


FIG. 10 Comparison of the Monotonic Model and Test Data for Fielding Specimen with t_{cf}=3.5cm (Fielding 1971)

To take into account the fact that a doubler plate is not fully effective in resisting panel zone shear (Becker 1971), the effective shear area of the panel zone with a doubler plate, A_{eff} is calculated as $(d_c \cdot t_{cw} + R_f \cdot t_{dp} \cdot W_{dp})$, where t_{dp} is the thickness of a doubler plate, w_{dp} is the width of a doubler plate between column flanges, and R_f is the reduction factor to account for the strain incompatibility between a doubler plate and column web.

The proposed monotonic panel moment-rotation relationships are described as follows. The yield moment M_y^{pa} and elastic stiffness K_e are

$$M_y^{pa} = \frac{V_y \cdot (d_b - t_{bf})}{(1-\rho)} \tag{13a}$$

$$V_y = \tau_y \cdot A_{eff} = \tau_y \cdot (d_c \cdot t_{cw} + R_f \cdot t_{dp} \cdot W_{dp}) \tag{13b}$$

$$K_e = \frac{M_y^{pa}}{\gamma_y} = \frac{G \cdot A_{eff} \cdot (d_b - t_{bf})}{(1-\rho)} \tag{13c}$$

The post-elastic stiffness K_1 and the second yield moment M_{sh}^{pa}, at which the strain hardening starts, are

$$K_1 = \frac{5.2 \cdot G \cdot b_{cf} \cdot t_{cf}^3}{d_b \cdot (1-\rho)} \tag{14a}$$

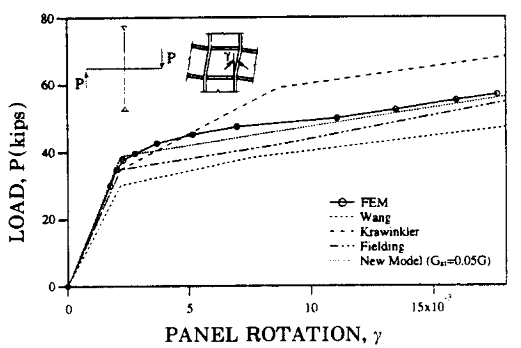


FIG. 11 Comparison of the Monotonic Model and FEM Results for Slutter Specimen 1 with t_{cf}=3.6cm (Wang 1988)

$$M_{sh}^{pa} = M_y^{pa} + \frac{15.6 \cdot \tau_y \cdot b_{cf} \cdot t_{cf}^3}{d_b \cdot (1-\rho)} \tag{14b}$$

The strain hardening stiffness K_2 is

$$K_2 = \frac{G_{st} \cdot A_{eff} \cdot (d_b - t_{bf})}{(1-\rho)} \tag{15}$$

The developed monotonic model has also been applied to the same four specimens as in the previous section. Figures 5 to 12 show comparison of the developed model with test results and FEM results for various column flange thicknesses. From these figures it can be seen that the simple modifications have improved the performance of the developed model except for Krawinkler Specimen B2 (Fig. 8). The suggested model underestimates the panel zone strength of Krawinkler Specimen B2 by about 13%. This can be attributed to the fact that the developed model cannot properly model unusual premature strain hardening effects due to a very short yield plateau ($\varepsilon_s = 4.4\varepsilon_y$) of the stress-strain relations of the column web. The correlation between the model predictions and the responses obtained by the test and FEM analysis is reasonable regardless of column flange thickness. From the above discussion, it can be found that when the design shear

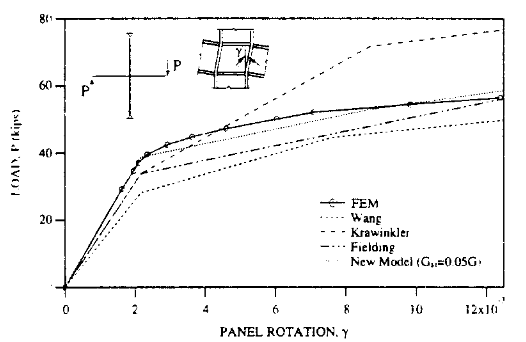


FIG. 12 Comparison of the Monotonic Model and FEM Results for Slutter Specimen 1 with t_{cf}=4.51cm (Wang 1988)

strength is based on Eq. 14b instead of Eq. 10, it can be more realistic. The design shear strength of the panel zone, which is based on Eq. 14b, is suggested as follows:

$$V_n = 0.6\sigma_y A_{eff}\left(1+\frac{15b_{cf}\,t_{cf}^{3}}{d_b(d_b-t_{bf})A_{eff}}\right) \qquad (16)$$

It should be noted that in the effective shear area A_{eff} a reduction factor is used to account for the fact that a doubler plate is not fully effective in resisting paneffel zone shear forces.

In this study the thickest column flange was t_{cf}=4.51cm. In actual design practice, even thicker column flanges may be used, perhaps on the order of 8 to 13 cm. Additional test predictions for such column sections are needed to further verify this simple monotonic model and the proposed design shear strength. No such data was found in the literature.

5. Conclusions

The objective of the study in this paper was to develop a model to describe monotonic behavior of the bare steel beam-to-column joints. Existing models for monotonic loading were reviewed and Fielding's model was improved to obtain better correlation with experimental results. In spite of the simplicity of the model, reasonable agreement was established between model predictions and test results for panel zones. The new design shear strength of the panel zone, which was based on the proposed monotonic model and was more realistic, was suggested. More tests are needed to further study the panel zone with thicker column flanges (order of 8 to 13 cm). The hysteretic rules for the panel zone behavior under cyclic loadings and the effectiveness of doubler plates are also presented in the companion paper.

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