# Frame Response Considering Plastic Panel Hinges

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## INTRODUCTION

The AISC-LRFD Specification[1] designates connections as fully restrained (FR) or partially restrained (PR). Only a beam-to-column connection that has sufficient rigidity to hold the original angles between members unchanged may be considered as Type FR. The quantification of this requirement has been left to the judgment of the structural engineer. One major source of deformation that occurs in FR connections is shear deformation of the column panel zone when beams frame to the column flange(s). Connections that have traditionally been assumed to be fully restrained are, in fact, only partially restrained because they permit relative rotation between the beam and column. Figure 1 shows column ($\theta_c$), beam ($\theta_b$), and connection ($\theta_{conn}$) rotations. If the connection rotations are taken positive as shown, then:

$$\theta_{conn} = \theta_c - \theta_b \qquad (1)$$

For an FR connection, $\theta_{conn} = 0$ and $\theta_c = \theta_b$. This condition is nearly always assumed in conventional frame analysis; however, for most connections, this simplification is inaccurate because there are sources of relative movement between the connected elements and within the connection itself. A significant source of such deformation is the connection panel shear distortion shown in the inset in Figure 1. In order to minimize the impact of this source of deformation the 1993 AISC-LRFD provisions in Specification Section K1.7 limit the shear stress and deformation in panel zones to the elastic range. The new limits on the nominal web shear strength, $R_v$ are:

For $P_u \leq 0.4P_y$

$$R_v = 0.60F_y d_c t_w \qquad \text{(K1-9)}$$

For $P_u > 0.4P_y$

$$R_v = 0.60F_y d_c t_w [1.4 - P_u / P_y] \qquad \text{(K1-10)}$$

These equations are a linearization of the interaction equation for considering the effect of column axial load on shear yielding presented in References 2 and 3, namely:

$$R_v = 0.60F_y d_c t_w [1.0 - (P_u / P_y)^2]^{1/2} \qquad (2)$$

All three equations are shown in Figure 2 along with test data from Reference 2. The three tests were conducted on specimens with axial force on the columns in excess of $0.4P_y$. The tests indicated a significant reduction in the connection shear strength at yielding. (Other tests have been done but at lower axial loads.) These equations describe a condition of general yielding of the column web (including doubler plates) within the panel zone, but not the strength of the entire panel zone. Equation 2 is used in the AISC *Seismic Provisions for Structural Steel Buildings,*[4] Section 10.2.f.1 as one of the limitations on shear links in eccentrically braced frames

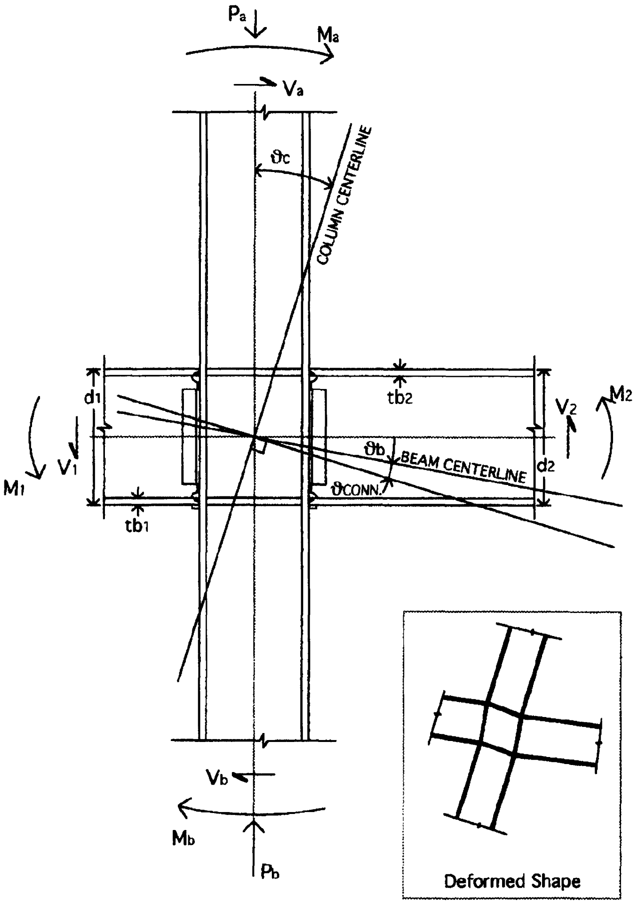

*Fig. 1. Joint forces and rotations.*

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(EBF). Equation 2 is equally valid for beam-to-column connection panels and shear links.

The objective of this paper is to report on studies of the impact of panel zone yielding and elastic and inelastic panel deformations on frame strength and drift. Two theories that describe panel hinge behavior are considered and analytical results describing the effects of varying (1) axial load on the columns, (2) beam section, (3) column section, and (4) doubler plate thickness are presented. The results demonstrate that the new 1993 AISC-LRFD provisions are necessary.

## PANEL HINGE BEHAVIOR

Under the combination of gravity and lateral loads on a steel frame the beam-column panel zone will deform. In Figure 1 the forces that cause the shear deformation are shown. The usual calculation of the panel shear force in terms of the factored beam moments and depths shown in the figure is:

$$V = M_1 / 0.95d_1 + M_2 / 0.95d_2 - V_c \qquad (3)$$

where $V_c$ is the average of column shears above and below the joint. A somewhat more accurate calculation is:

$$V = M_1 / (d_1 - t_{b1}) + M_2 / (d_2 - t_{b2}) - V_c \qquad (4)$$

The column axial load contributes to the effective stress and strain within the connection and is a factor in the first yield of the panel zone. Figure 3 shows the response of a shear panel to the force $V$. At low shear force the response is elastic and the shear distortion of the panel is

$$\gamma = V / (A'G) \qquad (5)$$

where $A'$ is the column web area defined in the specification and $G$ is the elastic shear modulus. At $V = R_v$, as given by Equations K1-9 and K1-10 or Equation 2 the panel undergoes a large reduction in stiffness. The post-yield stiffness has been simplified to a bi-linear behavior in Reference 2 and to a tri-linear behavior in Reference 5. These post-yield stiffnesses result from distortion of the connection panel boundaries and both predictions are shown in Figure 3 for the combination of sections noted.

Testing reported in References 2, 6, and 7 has shown that properly designed panel zones have some reserve strength beyond shear yielding. The additional capacity over equations K1-9 and K1-10 is:

$$\Delta R_v = 0.60F_y d_c t_w [3b_{cf} t_{cf}^2 / d_b d_c t_w] \qquad (6)$$

and is included in the LRFD Specification with limitations. This additional shear capacity is implied to occur in the range of panel hinge shear strains from $\gamma_y$ to $4\gamma_y$. It therefore represents a post-yield stiffness of

$$dV_p / d\gamma = \Delta R_v / 3\gamma_y = G b_{cf} t_{cf}^2 / d_b \qquad (7)$$

instead of the elastic stiffness

$$dV_e / d\gamma = GA' \qquad (8)$$

This description of panel hinge behavior has been used in research on seismic loading of steel frames and the model assumes that the plastic moment capacity of the column flanges about their own plastic neutral axes is achieved between $\gamma_y$ and $4\gamma_y$.

Another description of panel hinge behavior in the inelastic

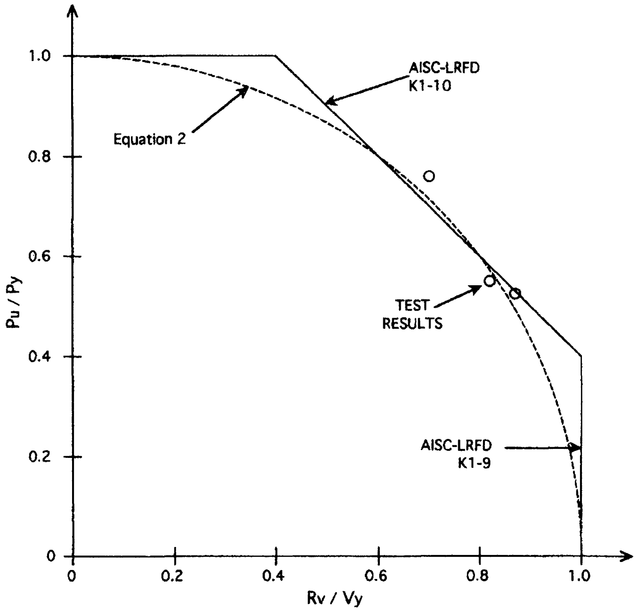


Fig. 2. Panel zone interaction diagram.

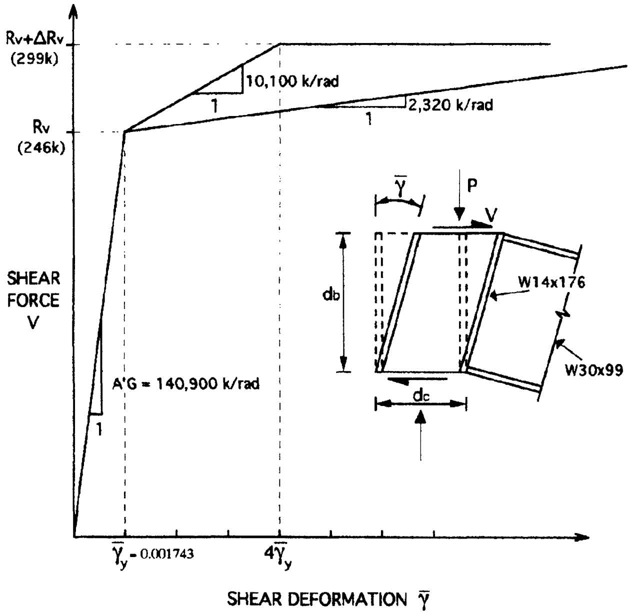


Fig. 3. Shear behavior of column connection.

range from Reference 2 is based on the elastic stiffness of the column flanges following shear yielding of the panel zone.

$$dV_p \, / \, d\gamma = 2Eb_{cf}\, t_{cf}^3 \, / \, d_b^2 \qquad (9)$$

This model assumes that the elastic stiffness of column flanges is available even at high shear strains and column axial loads. It has been used to successfully predict panel zone behavior in monotonically loaded and seismically loaded steel frames. Each of the models has limitations. The first model assumes that (1) the entire plastic bending strength of the flanges is available regardless of the axial load on the column and (2) this strength is reached at a shear strain of $4\gamma_y$. The second model assumes that (1) the elastic flange stiffness is not reduced by axial load on the column and (2) the post-yield stiffness is available at any magnitude of shear strain. The predictions from these two models are plotted for the column and beam noted in Figure 3. When compared with test data, the first model is more accurate initially at panel strain near $\gamma_y$ but the second model is more accurate if a bi-linear description of the behavior is to be used for shear strains at and beyond $4\gamma_y$. Both of these models are used in the following analyses.

## FRAME BEHAVIOR—STRENGTH

The responses of thirty-six frames were studied using a program (Shear-93) based on the methodology detailed in References 3 and 8. The program considers elastic and inelastic member and connection deformations shown in Figure 4, clear lengths of members, and second-order effects on equilibrium and member stiffness in the frame. The inelastic analysis is an incremental elastic analysis which includes member stiffness reduction as each plastic hinge forms in the beams and columns and panel hinges form in the connections. The clear member lengths are measured from face to face of connection panels as opposed to the center to center lengths. The second-order effects include (1) consideration of equilibrium in the deformed frame ($P$–$\Delta$ effect) and (2) reduction of member stiffness from axial load. The column axial loads were assumed to remain constant while the lateral load increased.

The geometry of the frames that were studied is shown in Figure 5. The beam sections used in the study ranged from W16×26 to W30×116. The columns ranged from W8×31 to W14×550 and A36 steel was assumed. All of the frames would generally be classified as "strong column/weak beam" designs although the results of the analyses would be applicable regardless of the design philosophy.

The beam spans in Figure 5 were made equal at 20′-0

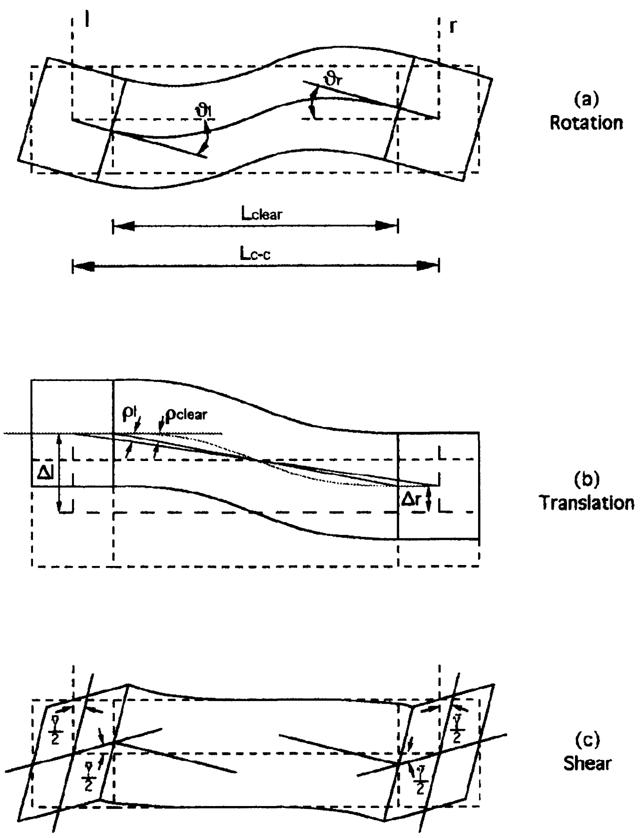
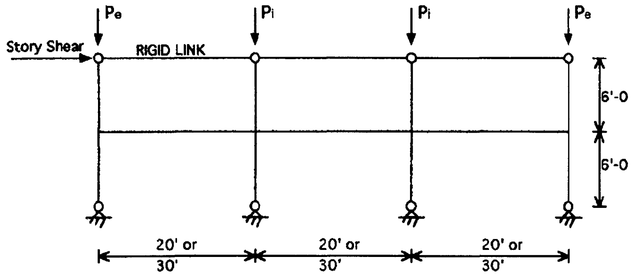


*Fig. 4. Member and connection deformations.*



| Frame | P/Py | | 1 | 2 |
|---|---|---|---|---|
| A | 0.25 | Beams | W18x35 | W30x99 |
| | | Int. Cols. | W12x53 | W14x211 |
| | | Ext. Cols | W12x53 | W14x109 |
| B | 0.5 | Beams | W18x35 | W30x99 |
| | | Int. Cols. | W14x90 | W14x311 |
| | | Ext. Cols | W10x49 | W14x176 |
| C | 0.75 | Beams | W18x35 | W30x99 |
| | | Int. Cols. | W14x132 | W14x500 |
| | | Ext. Cols | W14x90 | W14x257 |

*Fig. 5. Frame description.*

except for the beam sections W30×99 and heavier where the spans were 30′-0. All floor-to-floor heights were taken as 12′-0 with inflection points assumed at mid-height of the columns. No gravity loads were placed on the girders so that the effects of column axial load and panel stiffness could be observed on the frame behavior. (The program allows for girder loads which would cause the leeward panel hinges or plastic hinges to form earlier in the response histories.) Of the thirty-six frames investigated, the results for two designs at three $P/P_y$ ratios were selected as representative for discussion. The frames are designated as A, B, or C for the three $P/P_y$ ratios of 0.25, 0.5, and 0.75 respectively and as 1 or 2 for the lighter and heavier beam sections tabulated in Figure 5.

Figures 6 through 11 show the results of the analyses using Shear-93 in the form of story shear versus story drift histories for the frames A1, A2, B1, B2, C1, and C2. In Figure 6 for frame A1, the lower curves show the second-order frame behavior with unstiffened panel zones; each of these two curves uses one of the plastic panel hinge models described above. Model 1 in the legend is based on Equation 7 and Model 2 is based on Equation 9. For comparison, three other curves are shown. The third curve shows the second-order analysis for the condition when the column panel zones are stiffened in accordance with the following design criterion:

$$t_p = t_{cw}\,(V/R_v - 1) \qquad (10)$$

where $t_{cw}$ is the thickness of the column web, $t_p$ is the doubler plate thickness, and $V$ is calculated from Equation 3 or 4. With the panel zones stiffened to this degree, panel hinges cannot form and only plastic hinges form in beams or columns. The fourth curve represents a conventional second-order analysis using center-to-center column heights and girder clear spans. These member lengths are used to attempt to allow for the stiffening effect of connection size without using offsets while, at the same time, allowing for flexibility in the panel zone. Commercial analysis programs permit these assumptions. The final analysis in Figure 6 is a conventional first-order elastic analysis using centerline dimensions. Only the initial slope is shown for this analysis.

In Figure 6, with unstiffened column panel zones, the structure forms sufficient panel hinges to permit excessive story drift to the extent that the plastic strength of the frame is not achieved. For this reason, AISC-LRFD (1993) equations K1-9 and K1-10 are necessary to avoid premature collapse behavior. All four panel hinges achieve a strain of $4\gamma_y$ at the point shown in Figure 6. The reserve strength given by Equation 6 is achieved at or before this point in all four connections. Figure 7 shows similar behavior for the heavier beam section in frame A2.

In Figures 6 and 7 the behaviors shown are for examples where $P/P_y = 0.25$. In Figures 8 and 9 for frames B1 and B2

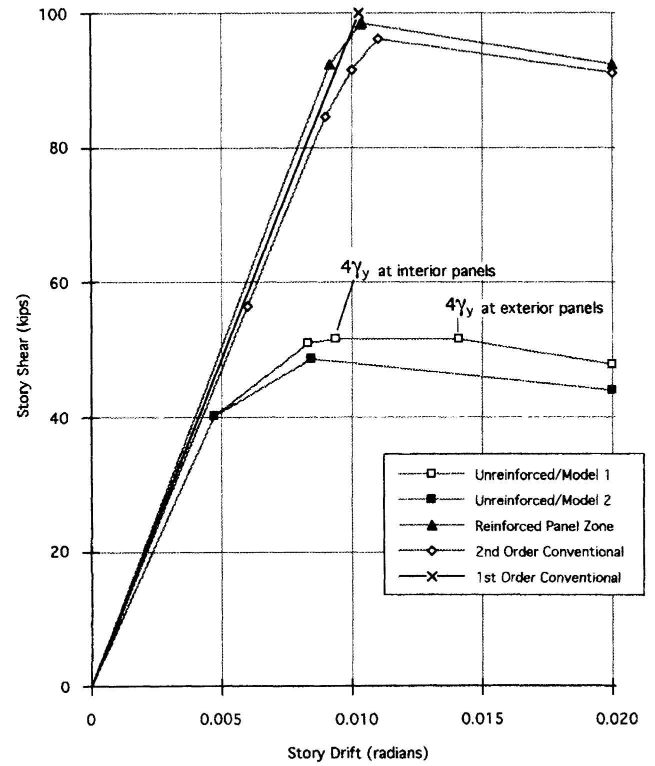


Fig. 6. Frame A1.

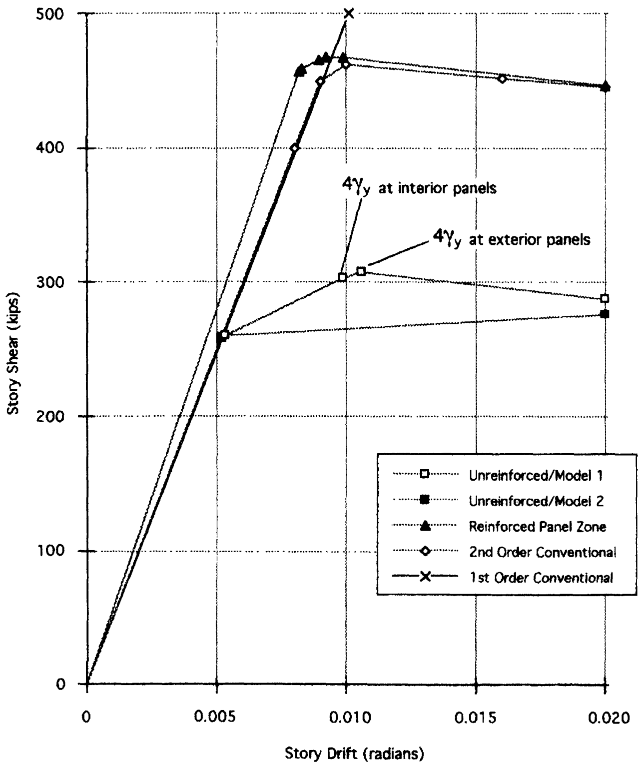


Fig. 7. Frame A2.

the axial load is higher, $P / P_y = 0.50$. In Figure 8, the analysis for unstiffened column webs shows that a gross shear strain of $4\gamma_y$ is reached only after the ultimate strength of the frame is achieved and unloading has begun. This demonstrates that the post-yield strength given by Equation 6 should not be used without an analysis that proves that it can be achieved. In this example, $\Delta R_v$ is only partially realized at the ultimate load. In the load-deformation history in Figure 9 where $P / P_y = 0.50$ for frame B2 the margin between shear yielding and ultimate strength is about 20 percent. In this case the inelastic shear strain of $4\gamma_y$ is not reached and the reserve post-yield strength is not realized. It would be unsafe to assume that $\Delta R_v$ from Equation 6 is available in the panel zones in this frame.

For $P / P_y = 0.75$, an extremely high level of factored column load, a shear strain level of $4\gamma_y$ is reached in the exterior connections in frame C1 but only on the unloading part of the curve after the ultimate strength is reached. This example also shows that it is not adequately safe to assume $\Delta R_v$ is available without analysis.

Frame C2 behavior in Figure 11 illustrates a frame that requires no doubler plates for strength. Panel hinges formed in the exterior connections but plastic hinges formed at the girder ends almost immediately thereafter.

In these examples, except for C2 where strengthening was not required, the sizes of the doubler plates were then calculated from Equations K1-9, K1-10, 3, and 10. The story shear versus story drift responses were recalculated and these are noted in Figures 6 through 10 as "Reinforced Panel Zone." In all cases, the full plastic strength of the frames was achieved.

## FRAME BEHAVIOR—STIFFNESS

A notable difference among the analyses is in the initial slope of the story shear versus drift behavior. The deformation of the column web, stiffened or not, contributes to the frame drift. In the discussion above, doubler plates were used as strengthening devices to avoid premature collapse (flat or unloading portions of the shear-drift curves). When the behavior of frames with the strengthened connections is compared with a conventional second-order analysis which does not account for the panel zone deformation, the stiffening effect of the doubler plates can be seen. In every example the drift of the frame with doubler plated connections is less than that obtained from a conventional second-order analysis done assuming clear spans for the beams and center-to-center lengths for the columns.

Figures 6 through 11 include also the results of first-order frame analyses. These expectedly underestimate the story drift in all cases. In comparing the stiffnesses of the reinforced frames with the first-order analyses, there are no trends. For frames A1 and A2 the stiffeners result in less drift than predicted by the first-order analysis. For frame B2 the use of stiffeners results in the same drift as the first-order analysis

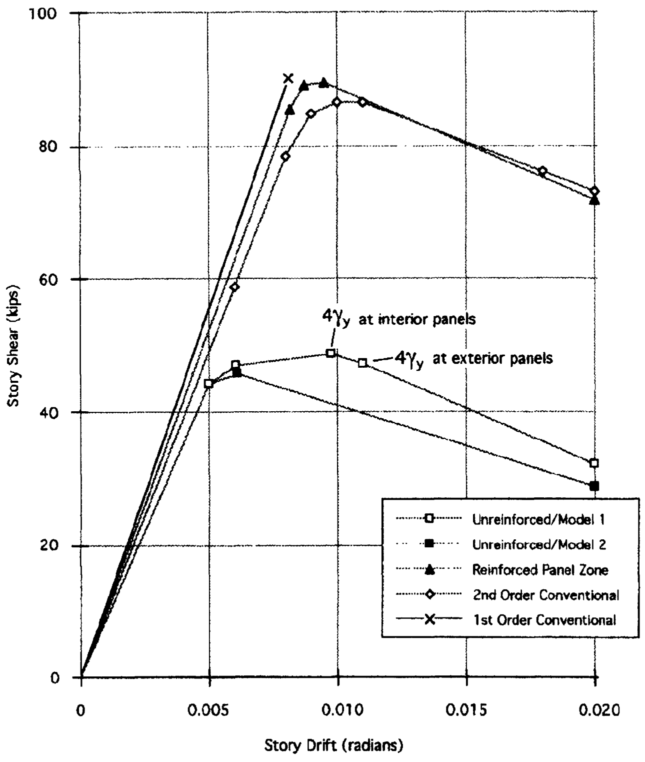


*Fig. 8. Frame B1.*

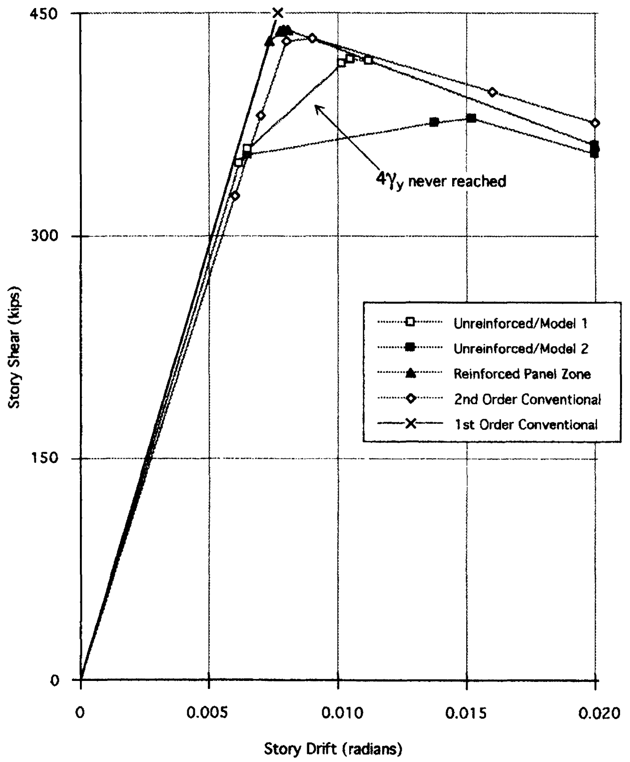


*Fig. 9. Frame B2.*

and for frames B1, C1, and C2 the doubler plates designed on the basis of strength do not reduce the drift to that of the first-order analysis but, rather, to a value that is intermediate to the first- and second-order analyses. Increases in doubler plate thickness over that required for strength will reduce the story drift; however, there is a limit on the effectiveness of such a measure.[3]

For the frames with unstiffened panels, any correlation between the actual frame stiffness and that obtained with first- and second-order analyses that did not explicitly include panel deformation and panel hinge formation is purely coincidental. Using member offsets and rotational springs can only approximately adjust for connection shear deformation. Similarly, using the clear spans for girders and center-to-center lengths for columns in the examples is an erratic and unreliable adjustment. As a rule the flexural deformation of an artificially lengthened beam or column will be different than the connection panel shear deformation. The connection panel deformation shown in Figure 4(c) affects joint rotations, member end rotations and displacements, and relative member end translations. It is necessary to model the connection as a member in light of these complexities. The formation of panel hinges and subsequent inelastic behavior of the panel zones must be included in analysis if the limitations in Equations K1-9 and K1-10 must be exceeded.

## CONCLUSIONS

1. The AISC-LRFD 1993 interaction equations K1-9 and K1-10 accurately describe general shear yielding of the connection panel hinge in the presence of axial load.
2. The AISC-LRFD 1993 requirement of limiting the use of post-yield strength of panel hinges by requiring analysis of the impact of plastic panel zone deformations on frame behavior is demonstrated to be necessary.
3. The reserve strength given by AISC-LRFD equation K1-11 is not achievable in some frames such as those described in Figures 8, 9, and 10.
4. The post-yield strength part of Equation K1-11 with its corresponding panel deformation of $4\gamma_y$ provides a good estimate of post-yield panel hinge stiffness for small inelastic shear strains between $\gamma_y$ and $4\gamma_y$.
5. A connection panel hinge model (Equation 9) which is based on post-yield stiffness of the panel boundaries does not have the limitations of the strength model and may be sufficiently accurate for static and seismic analysis.

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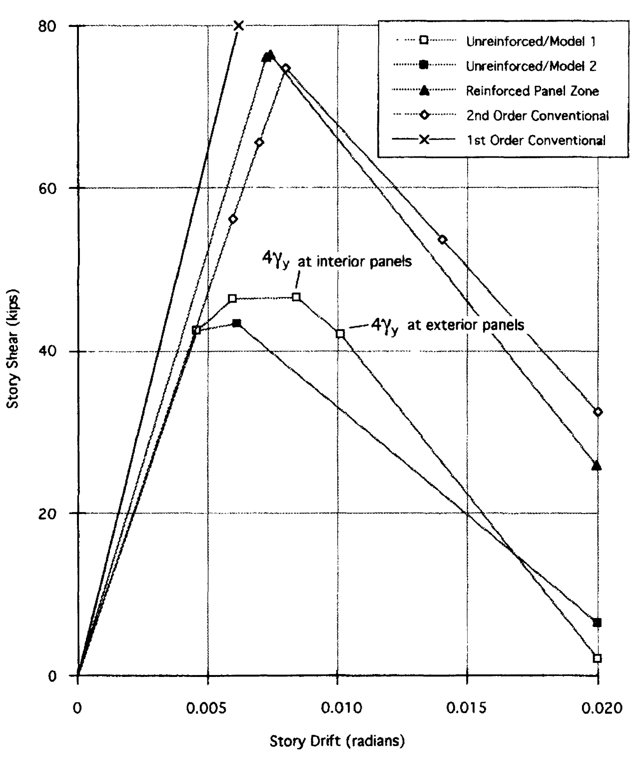

*Fig. 10. Frame C1.*

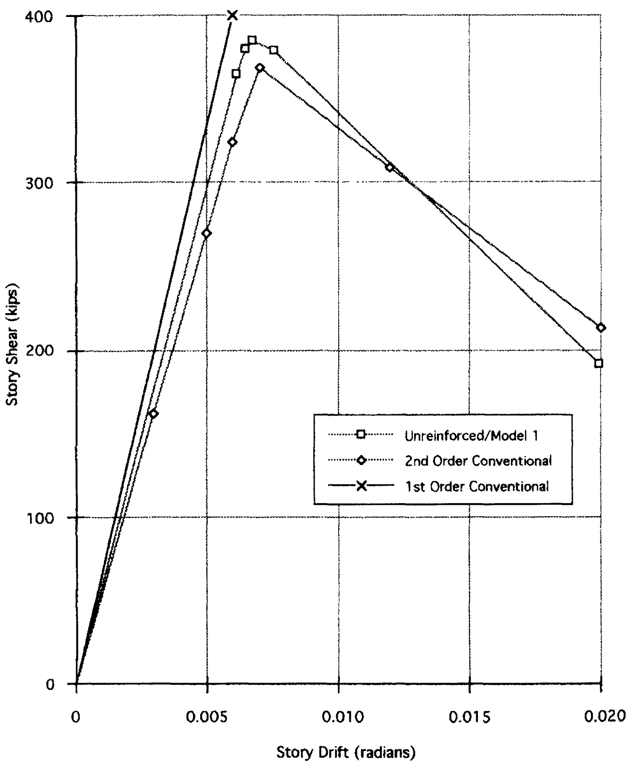

*Fig. 11. Frame C2.*

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