Shear in Beam-Column Joints in Seismic Design of Steel Frames

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In unbraced steel frames, structural stability and resistance to lateral loads require the transfer of bending moments between beams and columns. Depending upon stiffness and strength requirements, this transfer of bending moments can be achieved by either semi-rigid or rigid beam-column connections. In both cases, the intersection between beams and columns (the beam-column joint) will be subjected to high shears whenever a significant unbalance of beam moments is present at the joint. A significant unbalance usually exists at exterior and corner joints, and at interior joints in the case of lateral load application such as wind or seismic effects. Somewhat simplified, the effect of an unbalance of beam moments on the moment and shear force diagram along the column is illustrated in Figs. 1a and 1b. Figure 1c shows the forces acting on a free body of an interior joint.

The effect of the shear forces in joints must be accounted for in the design of frames. In the design for strength, the joints must be capable of transmitting the high shear forces through the columns in accordance with the selected design procedure, which may be based on allowable stresses or ultimate strength. In the design for stiffness, it may be necessary to verify that the joint distortions caused by the shear forces do not excessively affect the story drift under lateral loads.

The shear design of beam-column joints is of particular importance in frames that may be subjected to severe seismic excitations. Such frames may experience dynamic actions which will cause stresses and deformations by far exceeding the service state values. This imposes ductility requirements on all elements in the structure which may have to undergo severe inelastic deformations. Specific ductility requirements have been incorporated in the design criteria for ductile moment-resisting space frames (Ref. 1, Sect. 2722). This type of frame is required by the Uniform Building Code for all buildings exceeding 160 ft in height (Ref. 1, Sect. 2312).

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This paper deals with the effects of shear in beam-column joints on the strength, stiffness, and ductility of moment-resisting frames under severe earthquake excitations. Emphasis is placed on an evaluation of presently used design criteria for joints in ductile moment-resisting frames. Suggestions are presented for modifications of these design criteria.

AISC DESIGN CRITERIA FOR JOINT SHEAR

For a joint with a web thickness t, the maximum shear force that can be transferred through the joint is given by the AISC Specification[3] as

$$V_{max} = 0.40 F_y d_c t \tag{1}$$

for working stress design, and

$$V_{max} = 0.55 F_y d_c t \tag{2}$$

for plastic design.

Equation (1) is obtained by multiplying the allowable shear stress ($0.40F_y$) with the effective shear area which is taken as the product of the column depth d_c times the web thickness t. Equation (2) is obtained by multiplying the yield stress in pure shear (equal to $F_y/\sqrt{3}$ according to von Mises yield criterion) with the effective shear area, which is taken as $0.95 d_c t$.

In both working stress and plastic design, V_{max} must be equal to or larger than the design shear force V, which is given by

$$V = \left(\frac{\Delta M}{0.95 d_b} - V_{col}\right) \tag{3a}$$

where $\Delta M = M_b + M_c$ (see Fig. 1) and V_{col} is the shear in the column outside the joint. When two beams of unequal depths d_1 and d_2 frame into the joint, V is given by

$$V = \left(\frac{M_b}{0.95 d_1} + \frac{M_c}{0.95 d_2} - V_{col}\right) \tag{3b}$$

When seismic effects contribute to the design shear force V, the allowable stresses may be increased by 33 percent;

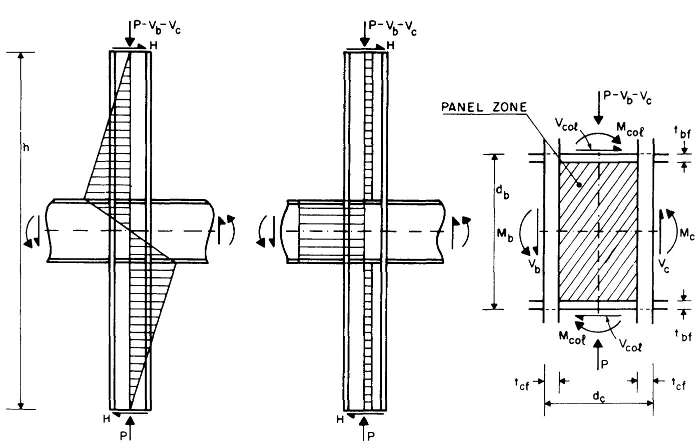


Fig. 1. Forces on column and joint

thus, V_{max} is given by

$$V_{max} = 0.53 F_y d_c t \tag{4}$$

It should be noted that this value is very close to that given for plastic design.

In seismic design, the shear design of joints is in most cases based on Eq. (4), and V is calculated from the internal forces (M_b, M_c, and V_{col}) produced by unfactored gravity and seismic loads. However, for ductile moment-resisting frames it is recommended in Ref. 2 that joints be designed for the maximum shear force that can be developed based on the strength capacity of the members framing into the joint. This may significantly modify the response characteristics of frames in severe earthquakes, as compared to frames with joints designed according to allowable stress criteria. The differences in the response characteristics are discussed later in this paper.

SHEAR BEHAVIOR OF JOINTS

The shear behavior of beam-column joints has been the subject of several experimental and analytical studies. References 4 to 10 are examples of more recent work. The observations reported herein are based primarily on the studies reported in more detail in Refs. 7 to 9. These studies were concerned with the monotonic and cyclic response characteristics of interior two-way joints with beams fully welded to the flanges of the column.

Qualitatively, the most important characteristics of the joint behavior can be summarized as follows: The shearing stresses in the panel zone caused by lateral loading are highest at the center of the panel, with a moderate but definite drop towards the four corners. When the joints were stressed beyond the elastic range, yielding in the panel propagated in most cases rather slowly from the center towards the level of the beam flanges. This is reflected in the load-deformational response of joints, which exhibits an elastic range, followed by a range of gradually decreasing stiffness, and then stabilizes to a small and almost constant stiffness for a long range of deformation. The latter stiffness can largely be attributed to strain-hardening in the material. The transition range between elastic stiffness and strain-hardening stiffness is primarily due to the fact that not only the panel zone in the joint resists the shear caused by an unbalance of beam moments; the elements surrounding the panel zone also contribute significantly to this resistance, in particular the bending resistance of the column flanges and the in-plane stiffness of the beam webs adjacent to the joint. The distribution of shear deformations throughout a joint can be studied from the deformed shape of the joint area of a W8x67 column shown in Fig. 2.

All tested joints exhibited a remarkable ductility and very stable and repetitive hysteresis loops under cyclic loading (see Fig. 3). In carefully detailed joints, no drop in strength was noticeable even at extremely large inelastic distortions, although in some specimens with thin panel zones diagonal buckling in the panel was observed. The only detrimental effect caused by excessive joint distortions was the formation of local kinks in beams and column flanges outside the joint, as illustrated in Fig. 4. These kinks caused high strain concentrations at the regions where the beam flanges were welded to the column, which in turn led to fracture of the material. However, this fracture occurred only after several load reversals at extremely large joint distortions. Thus, if joints are carefully detailed and if all welding in and

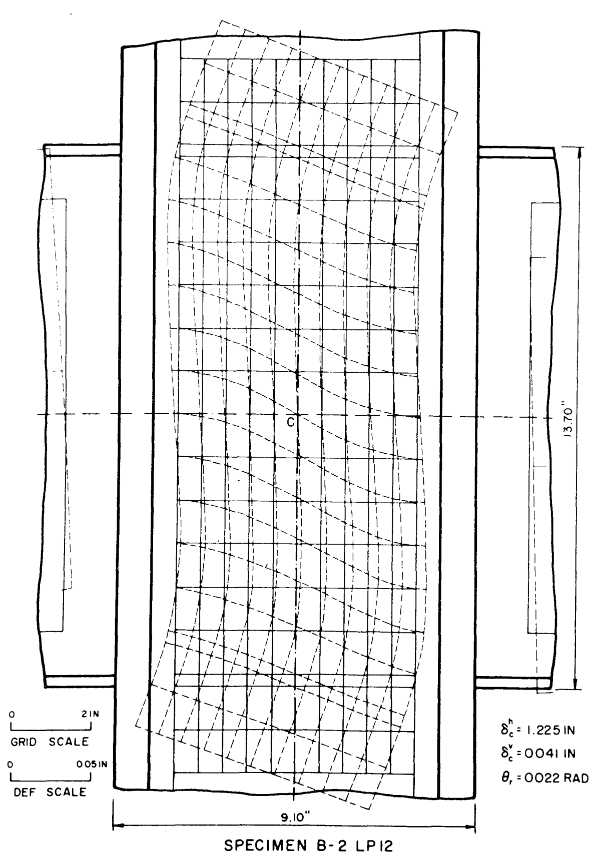
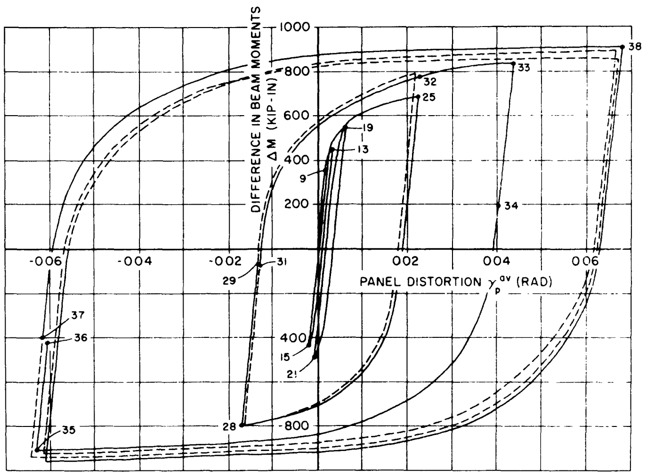


Fig. 2. Deformations in panel zone



SPECIMEN A-1

Fig. 3. Load-deformation diagram for joint

around the joint is done carefully, joints *per se* are elements with excellent energy dissipation characteristics.

A quantitative evaluation of the load-deformational response of joints can be made from the graphs shown in Fig. 5. In this figure are plotted the experimentally obtained V-γ_p^{av} diagrams of three test specimens whose properties are summarized in Table 1. The shear force V was calculated from Eq. (3a) and the average shear distortion γ_p^{av} was obtained from relative displacement measurements at the four corners of the joints. To permit a direct comparison between different joints, the graphs are normalized with respect to the AISC plastic design strength V_y, as given by Eq. (2), and the corresponding yield strain in shear, $\gamma_y = F_y/(\sqrt{3}\,G)$.

It can be seen from the graphs that the elastic stiffness of joints is rather accurately defined by the ratio V_y/γ_y as given by the AISC equation. Nonlinear behavior, caused by yielding in the panel zone, starts at approximately equal

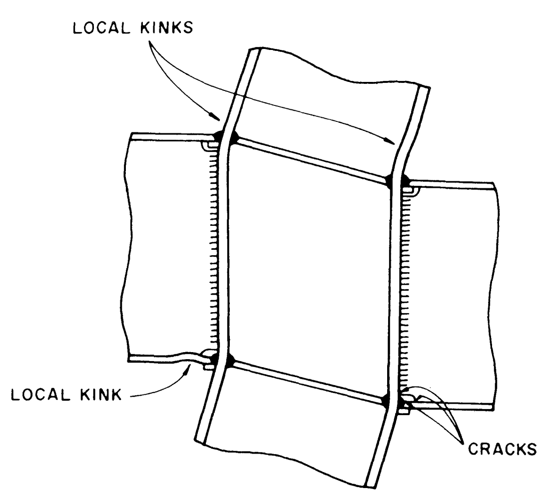


Fig. 4. Effects of excessive joint distortions

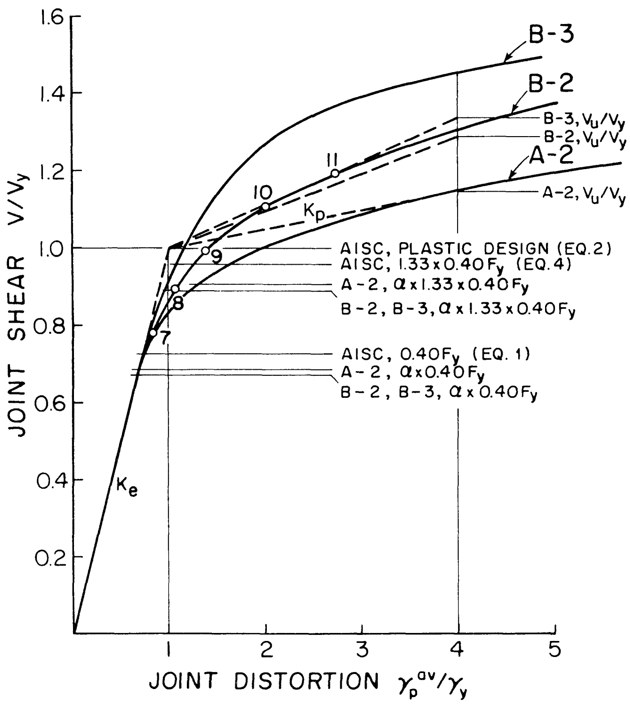


Fig. 5. Shear force-distortion diagrams for joints

shear levels (around 75 percent of V_y) for all three specimens. However, the post-yield stiffness and strength differ remarkably from specimen to specimen. This has also been observed by other investigators[4,5,6] and has led to several attempts to model more accurately the load-deformation characteristics of joints.

From experimental evidence and analytical studies it can be concluded that the post-yield strength and stiffness of joints depend on the stiffness of the elements surrounding the panel zone, primarily the flexural stiffness of the column flanges, and the aspect ratio d_b/d_c. These factors, as well as the stiffness of the beams and column outside the joint area, will strongly affect the extent of yielding in the panel zone. The propagation of yielding in the panel of specimen B-2 is illustrated in Fig. 6, which shows the yield boundaries for one-quarter of the panel corresponding to load levels indicated on the V-γ_p^{av} diagram in Fig. 5.

The mathematical modeling of joints is further complicated by the presence of normal stresses due to axial load and bending effects in the column and the bending moments in the beams. Also, the joint area is not subjected to concentrated shear forces at the beam levels, but to shear forces varying according to the distribution of bending stresses in the beams. Needless to say, design criteria for joints must be based on very simplified mathematical models, which nevertheless should incorporate the most important parameters that contribute to the force transfer within the joint. Based on this general discussion, several comments and suggestions regarding design criteria for joints are made in the following section.

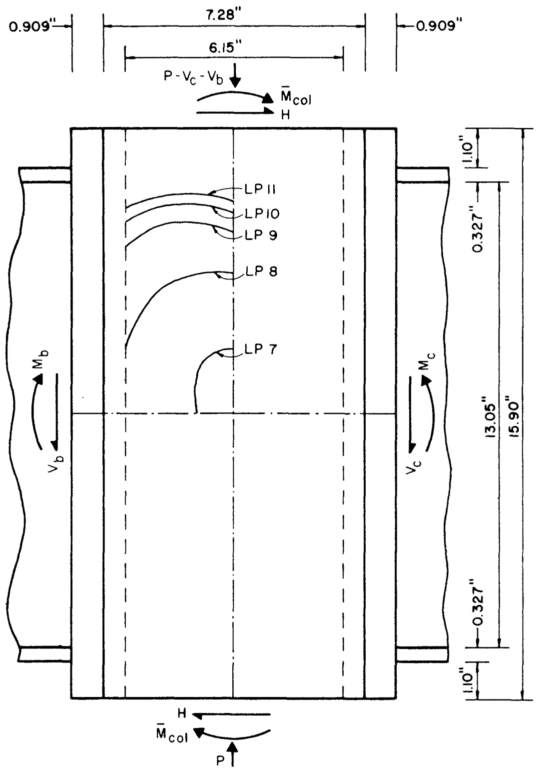
Table 1. Properties of Test Specimens

Spec.	d_c	t	t_{cf}	b_c	d_b	F_y^{web}	F_y^{fl}	$\frac{P}{P_y^{web}}$	$\frac{P}{P_y^{fl}}$	Web Reinf.	Horiz. Stiff.
	in.	in.	in.	in.	in.	ksi	ksi				
A-2*	8.03	0.255	0.396	5.80	10.03	41.0	40.5	0.32	0.325	no	yes
B-2**	9.09	0.627	0.908	8.16	13.72	47.0	42.5	0.37	0.41	no	no
B-3**	9.10	0.626	0.908	8.15	11.98	47.0		0.37		no	no

*Column is W8x24 section with flanges milled to simulate W14x68 prototype.
**Column is W8x67 section simulating W14x228 prototype.

DESIGN LEVELS FOR JOINT SHEAR

AISC Plastic Design (Eq. 2)—As can be seen from Fig. 5, the shear force corresponding to Eq. (2) causes controlled inelastic deformations which decrease with an increase in column flange thickness (A-2 vs. B-2) and a decrease in the aspect ratio d_b/d_c (B-2 vs. B-3). It can be expected that for heavier columns (thicker flanges) the extent of inelastic deformations will be even smaller. In all cases the shear capacity exceeded this force level; for thick column flanges and small aspect ratios by a large amount. It appears that Eq. (2) is very conservative for plastic design, which is usually concerned with the ultimate strength of elements. A model for ultimate strength is proposed later in this section.



SPECIMEN B-2 – YIELD BOUNDARIES

Fig. 6. Propagation of yield boundary in panel

Equation (2) is often taken as a measure of general yielding in the panel zone. As such it gives good results for joints with thin column flanges, but may be rather conservative in the case of very thick column flanges and small aspect ratios. Further research may be necessary to derive a design equation which pays more attention to the stiffness of the elements surrounding the panel.

AISC Allowable Stress Design (Eqs. 1 and 4)—Figure 4 indicates that Eq. (1) defines a force level which will cause essentially elastic response in the joint, although yielding commenced at the center of the panel at about the same level. The shear force defined by Eq. (4) (gravity plus seismic forces) did cause some inelastic response in the joints. Similar observations were made by other investigators.[5,6] The consequence of these inelastic joint distortions will be a slight increase in story drift at the allowable stress design level. Again, the amount of inelastic distortions decreases significantly for joints with thick column flanges and small aspect ratios.

One reason for the early yielding in the panel zone is the effect of the normal stresses due to axial column loads on the yield stress in shear. This effect can be included in Eqs. (1) and (4) by multiplying the allowable stress values by a factor α, which is given by von Mises yield criterion as

$$\alpha = \sqrt{1 - \left(\frac{P}{P_y}\right)^2} \qquad (5)$$

where P is the axial column load at the design level and P_y is the yield axial load. The shear forces obtained by including the factor α in Eqs. (1) and (4) are shown in Fig. 5.

It should be noted that in most cases it is not necessary to include the factor α in Eq. (2) for plastic design. This equation gives a measure of ultimate strength or, at least, general yielding of the panel zone. Experimental evidence[7] has shown that almost all of the axial force in the column is transferred to the column flanges in the joint once the panel zone has yielded in shear. Clearly, this only holds true for columns in which the flanges have the necessary capacity to resist the full axial load plus eventual bending stresses in the yielded joint region.

Ultimate Shear Strength of Joints—All experimental evidence has shown that the actual ultimate shear strength of joints is much higher than that given by Eq. (2). However, this ultimate strength usually is associated with unacceptable large inelastic joint distortions. Nevertheless, in plastic design philosophy which is based on the formation of plastic hinges, there appears to be no *a priori* need to design joints such that they behave essentially elastically under the actions of "ultimate" or factored loads. It is reasonably simple to incorporate in mathematical models of frame structures the joints as individual elements and account for their inelastic actions.[11,12,13] If these inelastic actions do not adversely affect the strength and stiffness requirements for frames, it may be overly conservative to design joints according to Eq. (2).

The load-deformational behavior of joints is peculiar insofar that it does not exhibit an elastic-almost perfectly plastic response, but gradually decreasing stiffness characteristics. It is appropriate, therefore, to associate ultimate strength with that level of shear force that can safely be transferred through the joint with *controlled* inelastic deformations, rather than with essentially elastic behavior. A total angle of distortion equal to four times the angle of distortion γ_y should be acceptable and is used in the following proposed design equation for ultimate shear strength in joints. If this criterion is accepted, it can be seen from Fig. 5 that the corresponding shear strength usually exceeds the AISC plastic design value by a large amount, for instance, by 46 percent for specimen B-3.

The mathematical model for strength and stiffness calculations is shown in Fig. 7. It consists of an elastic-perfectly plastic shear panel surrounded by rigid boundaries with springs at the four corners. These springs simulate the resistance of the elements surrounding the panel zone, in particular the bending resistance of the column flanges. The shear panel is active until general yielding of the panel zone occurs. Equation (2) is used to define general yielding although, as discussed previously, this equation may be rather conservative for joints with thick column flanges and small aspect ratios. Thus, the elastic stiffness is given by

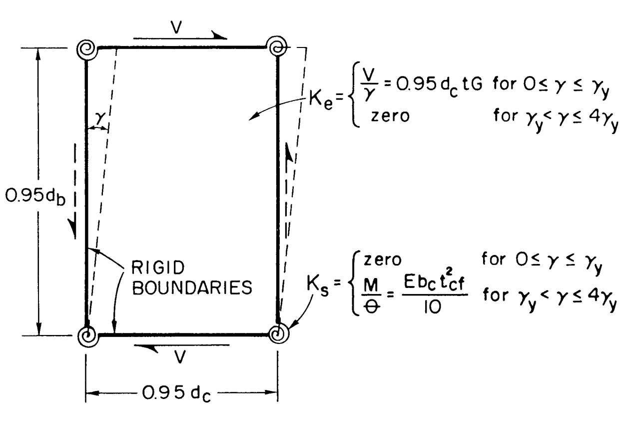


Fig. 7. Mathematical model for joint

$$K_e = \frac{V}{\gamma} = 0.95 d_c t G \tag{6}$$

This equation is valid until $\gamma = \gamma_y = F_y/(\sqrt{3}\,G)$. When this value is substituted in Eq. (6), the shear force at general yielding is obtained as

$$V_y = 0.55 F_y d_c t$$

which is identical to Eq. (2). It should be noted that Eqs. (2) and (6) do account, to some degree, for the beneficial effect of the elements surrounding the panel zone, since the effective shear area is taken as $0.95 d_c t$, which is usually larger than the actual shear area. If it is assumed that the shear stress distribution is uniform across the depth of the web and decreases linearly to zero through the column flanges, then the actual shear area would be $(d_c - t_{cf})t$.

When the panel has yielded uniformly, an additional increase in shear strength ΔV can only be attributed to the resistance of the elements surrounding the panel. This resistance can be approximated by springs at the four corners whose stiffness is that corresponding to concentrated rotations of the column flanges at each corner. When the boundaries of the panel zone are assumed to be rigid, this spring stiffness can be approximated by[7]

$$K_s = \frac{M}{\theta} = \frac{E b_c t_{cf}^2}{10} \tag{7}$$

The post-elastic stiffness of the joint, attributable to the four springs, is then computed as

$$K_p = \frac{\Delta V}{\Delta \gamma} = \frac{1.095 b_c t_{cf}^2 G}{d_b} \tag{8}$$

This equation is obtained from the work equation $0.95 d_b \Delta V \Delta\gamma = 4M\theta$, with $\theta = \Delta\gamma$ and $E = 2.6G$.

From Eqs. (6) and (8), the ratio of elastic to post-elastic stiffness is obtained as

$$\frac{K_p}{K_e} = \frac{1.15 b_c t_{cf}^2}{d_b d_c t} \tag{9}$$

It is evident that the post-elastic stiffness as given by Eq. (8) is mathematically correct only as long as the moments in the column flanges remain elastic. However, experimental studies[7] have shown that shear yielding spreads to the corners of the panel zone usually only at large angles of distortion, γ, and not only the column flanges, but also parts of the panel zone, are effective in resisting shear beyond the value defined by Eq. (2). Consequently, the actual post-elastic tangent stiffness of joints is usually higher than K_p for a significant range of inelastic distortions.

If it is assumed that the post-elastic stiffness of the joint K_p is valid for a range of $\Delta\gamma = 3\gamma_y$, the ultimate strength V_u of joints (at an angle of distortion equal to $4\gamma_y$) is then given by

$$V_u = K_e \gamma_y + 3 K_p \gamma_y$$

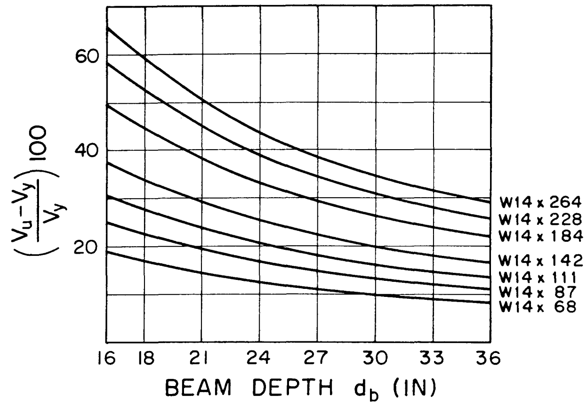


Fig. 8. Ultimate shear strength vs. AISC plastic design strength—increase in percent

Since $K_e\gamma_y$ is equal to V_y, this equation can be rewritten as

$$V_u = V_y\left(1 + \frac{3K_p}{K_e}\right) = 0.55F_yd_ct\left(1 + \frac{3.45b_ct_{cf}^2}{d_bd_ct}\right) \tag{10}$$

The second term in the brackets represents the increase in strength beyond V_y which is given by the AISC plastic design equation. In percent, this increase is illustrated in Fig. 8 for several W14 columns with unreinforced webs.

Since for each column section with an unreinforced panel zone the ultimate shear strength depends only on the beam depth d_b, design charts can easily be constructed to facilitate design calculations. Such a design chart, with V_u plotted against d_b, is presented in Fig. 9 for W14x43 to W14x264 sections and F_y = 36 ksi.

The ultimate strength values V_u and the bilinear response characteristics of the mathematical model described by K_e and K_p are compared to experimental results in Fig. 5. As can be seen, in all three specimens V_u was attained at distortions equal to or smaller than $4\gamma_y$ and an appreciable reserve strength beyond V_u is evident. It is expected that this model will give good results for interior joints when the axial column load ratio P/P_y is less than 0.50 and when the combined action of axial load and bending moment in the column will not cause yielding outside the joint, since early yielding of the column will decrease the resistance of the elements surrounding the panel zone. The model should not be applied to corner joints which are bounded by framing elements only on two faces of the panel zone. When two beams of different depth frame into the column in interior joints, it is conservative to use the larger value of d_b in Eq. (10).

It should be noted that the computed ultimate shear strength V_u is based on a simplified mathematical model which is in good agreement with experimental results for joints with thin to medium thick column flanges. For joints

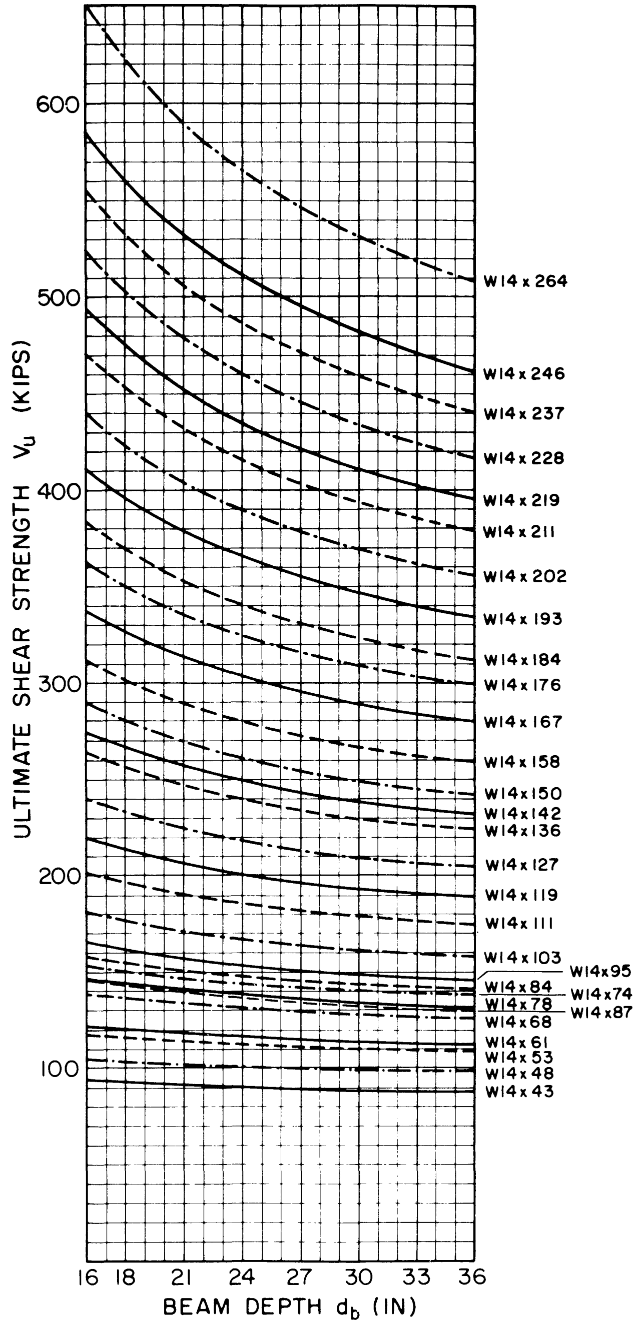


Fig. 9. Ultimate shear strength for unreinforced joints in W14 columns of A36 steel

in columns with very thick flanges, further experimental evidence is needed to verify the predicted shear strength.

Effectiveness of Web Reinforcement—In seismic regions it is customary to use web doubler plates when the column section alone is inadequate to resist the design shear force given by Eqs. (3). Experimental studies[6,9] have shown that in reinforced webs larger distortions are caused in the column web than in the doubler plates. The difference in distortions is relatively small for doubler plates in contact with the column web, but is significant when plate stiffeners are welded at a distance away from the column web. Thus, in the latter case the web stiffeners cannot be considered fully effective.

Plate stiffeners are shear elements which, in case they are in contact with the column web, can be treated similarly to column webs. When welded to the column flanges, the effective shear area of a stiffener of thickness t_s and depth $(d_c - 2t_{cf})$ can be taken as $(d_c - t_{cf})t_s$ and the shear force V_s that can be transferred through the stiffener can be computed as

$$V_s = F_{des}(d_c - t_{cf})t_s \tag{11}$$

where F_{des} is the design shear stress, which may be $0.40F_y$, $1.33 \times 0.40F_y$, or $F_y/\sqrt{3}$. Thus, if the design shear force V is larger, by an amount $\overline{V}$, than the shear force that can be resisted by the unreinforced joint, the required thickness of a web doubler plate is given by

$$t_{s(reqd)} = \frac{\overline{V}}{F_{des}(d_c - t_{cf})} \tag{12}$$

It should be noted that the presence of a web stiffener does not affect the post-elastic stiffness of the previously discussed mathematical model, since K_p depends primarily on the stiffness of the elements surrounding the panel zone. Therefore, the ultimate shear strength of joints with doubler plates is given by

$$V_u = V_{u(col)} + \frac{F_y}{\sqrt{3}}(d_c - t_{cf})t_s \tag{13}$$

where $V_{u(col)}$ is the ultimate shear strength of the unreinforced joint as given by Eq. (10).

EFFECT OF JOINT STRENGTH AND DEFORMATION ON THE SEISMIC RESPONSE OF FRAMES

In severe earthquakes it must be expected that frames will have to undergo deformations several times larger than those computed under service loads. The amount and distribution of deformations, which may be highly inelastic, depend on the relative strength and stiffness of the individual elements in the frame. Ideally, frames should be designed such that inelastic actions in severe earthquakes be concentrated in those elements which can provide high ductility. At the same time, much attention must be paid to stiffness requirements at all levels of deformation to limit the story drift for damage control and stability considerations.

This points out a problem in the design of joints which usually are very ductile elements, but exhibit a rather small stiffness when stressed significantly beyond the allowable stress value (see Fig. 5). Thus, the stiffness of a frame whose joints are designed just for the beam moments due to code seismic forces will decrease soon after the design force level, since the ultimate strength of such joints will often be too small to permit the attainment of plastic moments in the beams. The low post-elastic stiffness of the frame will cause an increase in story drift which in turn will magnify the P-δ effect. The question whether or not this story drift is acceptable from the standpoint of damage and P-δ control may have to be answered through an inelastic analysis of the structure.

It must be emphasized that, in frames whose joints are close to the shear level given by Eq. (4) under seismic design moments, plastic hinges in the beams often cannot develop, due to the limited shear strength of the joints. In these cases ductility of beams is of less concern, but much attention must be paid to careful detailing of joints, which may have to undergo severe inelastic strain reversals during major earthquakes. Experimental evidence has shown that very large inelastic distortions can be tolerated in carefully detailed joints.

Maximum strength and stiffness of moment-resisting frames is achieved when all joints are designed for the maximum shear force that can possibly be developed, based on the strength capacity of the members framing into the joint. Such a design criterion is recommended for ductile frames in the SEAOC *Recommended Lateral Force Requirements*,[2] which are widely used in areas of high seismicity. When the AISC plastic design equation [Eq. (2)] is used to fulfill this design criterion, the joints will remain essentially elastic throughout a severe earthquake and inelastic deformations will be concentrated in beams and possibly in several columns. This may impose severe ductility requirements on these elements while the joints, which by nature are ductile elements, will not participate in energy dissipation. Thus, the use of Eq. (2) may be too conservative and may even be detrimental in cases where the framing elements cannot provide the necessary ductility demands. Here it would be advantageous to let joints participate to a larger degree in energy dissipation.

Therefore, whenever it is deemed necessary to design joints for the capacity of the connected members, it is appropriate to use an ultimate strength value for the shear design of joints. Such an ultimate shear strength, which is associated with controlled inelastic distortions, was defined in the previous section by Eqs. (10) and (13). When joints are designed according to these equations, the strength capacity of the connected elements can still be developed and the overall frame stiffness will not be affected significantly. In this case the joints will participate in dissipating energy, which will reduce the ductility requirements for inelastic regions in beams and columns. Also, this will severely reduce the use of heavy doubler plates, whose performance depends strongly on the quality of welding.

The improved behavior of a frame assembly with the joint undergoing larger inelastic distortions versus that with a more rigid joint was verified experimentally on two otherwise identical beam-column subassemblages.[9] In both cases the stiffness and maximum strength of the subassemblages were almost identical; however, under severe inelastic load reversals, local instabilities in the beams and

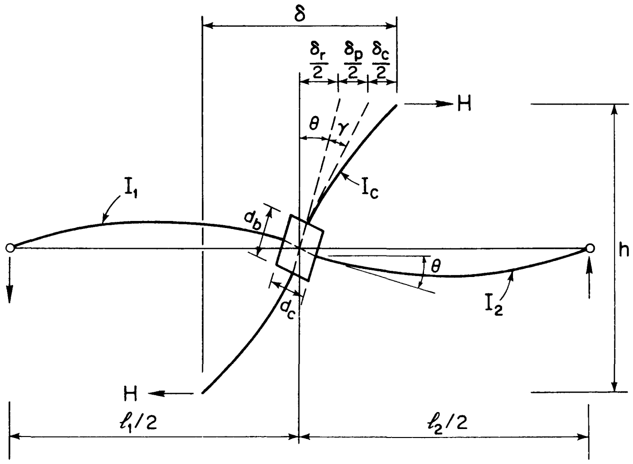


Fig. 10. Components of lateral deflection in beam-column assembly

a decrease in strength and stiffness did occur much earlier in the subassemblage with the more rigid joint.

EFFECT OF JOINT DISTORTION ON THE ELASTIC STIFFNESS OF FRAMES

Methods for incorporating joint strength and stiffness in the analysis of frames have been developed[11,12,13] and are considered in at least one general purpose frame analysis program.[11] The joint response can be represented by a tri-linear model with stiffnesses K_e and K_p [see Eqs. (6) and (8)], followed by a strain-hardening stiffness or perfectly plastic behavior. Whenever the strength of joints is less than that required to develop the capacity of the connected members, an analysis including this tri-linear model will give important information on the actual distribution of inelastic deformations in frames subjected to severe earthquakes.

In the elastic range the frame stiffness is of primary interest for story drift calculations. It is common practice to account for the effect of joint distortions by basing drift calculations on center line dimensions of beams and columns rather than clear span dimensions. Presented below is an approximate method which explicitly accounts for joint distortions in the computation of lateral deflections and permits a direct comparison with deflections based on center line dimensions.

The method is based on portal method assumptions, which implies that a frame can be resolved into simple beam-column assemblies with points of inflections at midspans of beams and midheights of columns. The deflected shape of such a subassembly and its dimensions and properties are shown in Fig. 10. Neglecting second order effects and lateral deflection due to axial column deformations, the story drift δ can be computed as the sum of the three deflection components shown in Fig. 10, where

δ_c = lateral deflection caused by flexural deformations in the column

δ_r = lateral deflection caused by flexural deformations in the beams and eventual deformations in the connecting media (in semi-rigid connections)

δ_p = lateral deflection caused by shear distortions in the joint

When the small vertical deflection of the beams at the column faces is neglected and the beam-to-column connections are rigid, these deflection components can be computed as follows:

$$\delta_c = \frac{(h - d_b)^3}{12EI_c} H \tag{14}$$

$$\delta_r = \frac{h^2\left(1 - \dfrac{2d_c}{l_1 + l_2}\right)}{6E\left(\dfrac{I_1}{l_1 - d_c} + \dfrac{I_2}{l_2 - d_c}\right)} H \tag{15}$$

$$\delta_p = \gamma(h - d_b) = \frac{h - d_b}{d_c t G} V \tag{16}$$

The joint shear force V is given by

$$V = \left[\frac{h}{0.95 d_b}\left(1 - \frac{2d_c}{l_1 + l_2}\right) - 1\right] H \tag{16a}$$

but conservatively may be taken as

$$V = \frac{h}{d_b} H \tag{16b}$$

Using Eq. (16b), δ_p is given by

$$\delta_p = \frac{h(h - d_b)}{d_b d_c t G} H \tag{16c}$$

Equation (15) can be simplified if $l_1 = l_2 = l$ and $I_1 = I_2 = I_b$ and becomes

$$\delta_r = \frac{h^2(l - d_c)^2}{12EI_b l} H \tag{15a}$$

When joint distortions are neglected and deflection computations are based on center line dimensions, δ_p from Eqs. (16) and d_b and d_c in Eqs. (14) and (15) become equal to zero.

Clearly, Eqs. (14) to (16) give only an estimate of the story drift, since deflection compatibility between adjacent beam-column assemblies is disregarded. As such, these equations are most useful in the preliminary design phase to evaluate the relative importance of the three drift components and the effect of joint distortions on the story drift.

Numerical results of samples of deflection calculations are shown in Table 2. The two beam-column assemblies A and B were taken from the 17th and 5th story, respec-

Table 2. Lateral Deflections of Beam-Column Assemblies

	Column	Beams	Center Line Dimensions			Clear Span Dimensions				$\frac{\delta\ (\text{C.L.})}{\delta\ (\text{C.S.})}$
			$\delta = \delta_c + \delta_r$	δ_c/δ	δ_r/δ	$\delta = \delta_c + \delta_r + \delta_p$	δ_c/δ	δ_r/δ	δ_p/δ	
			(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
A	W14x68	W18x45	$0.0361H$	0.33	0.67	$0.0436H$	0.18	0.51	0.31	1.21
B	W14x228	W24x68	$0.0124H$	0.24	0.76	$0.0134H$	0.15	0.63	0.24	1.08

tively, of a 20-story steel frame[7] with a bay width of 24 ft and a story height of 12 ft. In both subassemblies the joints were unreinforced. As can be seen from column (7) of the table, joint distortions did contribute significantly to the story drift in both cases. This contribution is only partially offset by basing deflection calculations on center line dimensions [see column (8)]. The effect of joint distortions on the story drift depends on the stiffness of joints relative to that of beams and columns. This effect may be significant when the shear in joints under design forces is close to the allowable stress value given by Eq. (4), since in this case the joints will be relatively flexible and may experience some inelastic distortion.

SUMMARY AND CONCLUSIONS

This paper discusses the importance of joint shear in the response of frame structures to severe earthquakes. Presently used AISC design criteria for joint shear are reviewed in the light of limited experimental evidence. The effects of high shear in joints on the strength, stiffness, and energy-dissipation characteristics of frames are discussed. The most important conclusions can be briefly summarized:

1. Joints usually are very ductile elements capable of undergoing severe inelastic strain reversal without a decrease in strength.

2. The shear force defined by the AISC design equation for combined gravity and lateral loads [Eq. (4)] usually causes some inelastic deformation in joints. This inelastic deformation is reduced when the allowable shear stress is modified by the factor α [Eq. (5)], which accounts for the effect of the axial force in the column on the yield stress in shear. The significant difference in inelastic deformations in joints with thin versus thick column flanges indicates that the AISC equations do not account fully for the effects of the elements surrounding the panel zone.

3. Experimental evidence shows that joints exhibit a significant reserve strength beyond the AISC plastic design level [Eq. (2)]. Solutions for the ultimate shear strength, associated with controlled inelastic distortions, are given in Eqs. (10) and (13) for joints with unreinforced and reinforced webs, respectively.

4. The response of frames to severe earthquakes depends strongly on the strength and stiffness of joints. When joints are designed according to allowable stress criteria [Eq. (4)], inelastic deformations may be concentrated primarily in the joints and to a lesser degree in plastic hinge regions of beams and columns.

5. Maximum strength and stiffness of frames is attained when all joints are designed for the maximum shear force that can be developed based on the strength capacity of the members framing into the joint. The need for this design criterion has not been fully established, although it is widely used in areas of high seismicity. If this criterion is used, the joints should be permitted to participate in energy dissipation through inelastic deformations. This can be accomplished by basing the design of joints on the ultimate shear strength value given by Eq. (10) or (13).

6. Joint distortions contribute significantly to the elastic story drift in frames. Equations (14) to (16) permit an estimate of the effect of these distortions on the lateral deflections.

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NOTATION

b_c = width of column
d_b = depth of beam
d_c = depth of column
E = modulus of elasticity of steel
F_y = yield stress of steel in tension
G = shear modulus of steel
h = story height
H = horizontal force
I_b, I_1, I_2 = moment of inertia of beam
I_c = moment of inertia of column
K_e = elastic stiffness of joint
K_p = post-elastic stiffness of joint
l, l_1, l_2 = bay width, center-to-center of columns
M_b, M_c = moment in beam at face of column
t = thickness of web
t_{cf} = thickness of column flange
t_s = thickness of web stiffener
V = design shear force in joint
V_{col} = shear force in column outside the joint
V_s = shear strength of web stiffener
V_u = ultimate shear strength of joint

V_y = shear force causing general yielding in joint
ΔM = difference in beam moments at faces of column
δ_c, δ_p, δ_r = components of lateral deflection
γ = angle of shear distortion
γ_p^{av} = average angle of shear distortion in joint
γ_y = angle of shear distortion at general yielding

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