Panel Zone Effect on the Strength and Stiffness of Steel Rigid Frames

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THIS PAPER IS a study of the effect of the panel zone on the strength and stiffness of steel rigid frames. The panel zone is defined as the portion of the frame whose boundaries are within the rigid connection of two or more members which have webs lying in a common plane.

Design criteria for calculating the shear capacity of the panel zone are given in the *Commentary to the AISC Specification** (Sect. 1.5.1.2 for allowable stress design and Sect. 2.5 for plastic design). However, the validity of these criteria as applied to steel rigid frames is subject to question, since they have not yet been adequately verified by research. In this paper the author has endeavored to provide answers to the following questions:

1. Can the panel zone be the most critical element in a steel rigid frame? That is, can the shear capacity of the panel zone determine the strength and stiffness of a steel rigid frame?
2. Can web doubler plates be used to increase the effective shear capacity of the panel zone?
3. Provided the web doubler plates are adequately welded, will the shear strains (and stresses) in the web doubler plate be compatible with those in the panel zone web of the reinforced member?
4. If the shear stress within the panel zone is known, can the panel zone's contribution to the drift of the frame be determined?
5. Are the design criteria for the panel zone given in Sect. 1.5.1.2 of the *Commentary to the AISC Specification* realistic or should they be modified in some instances?

In order to answer these questions, full scale tests were conducted at the School of Engineering of the University of Southern California.

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**Throughout this paper the term "AISC Specification" refers to the Specification adopted February, 1969.*

DESCRIPTION OF TEST SPECIMENS

Test specimens were fabricated in the form of a C-shaped rigid frame from W14×61 sections, as shown in Fig. 1. All test specimens were made from ASTM A36 material; ASTM A233 class E70 series electrodes were used for all welding. The AISC Specification was adhered to in all respects.

Although the test specimens did not resemble the usual configuration of a steel rigid frame, the desired information concerning the panel zone was provided because of the moment continuity at the corners. Since the moment created in the horizontal members by the applied load has to be transferred to the vertical member, the transfer mechanism must be provided by the panel zone. Consequently, the panel zone was subjected to large shear stresses and strains. This is exactly the same type of transfer mechanism that will exist in the panel zone of the columns of a steel rigid frame when it is subjected to lateral loads.

Table 1. Theoretical Capacities of Specimens

	Allowable Stress Design		Plastic Design	
Specimen	Allow. Capacity (kips)	Critical Element	Yield Capacity (kips)	Critical Element
1	33.7	Vertical member (combined bending and axial load)	53.5	Panel zone (shear stress)
2	16.8	Panel zone (shear stress)	23.0	Panel zone (shear stress)
3	32.9	Weld of web doubler plate to W14×61	50.3	Weld of web doubler plate to W14×61

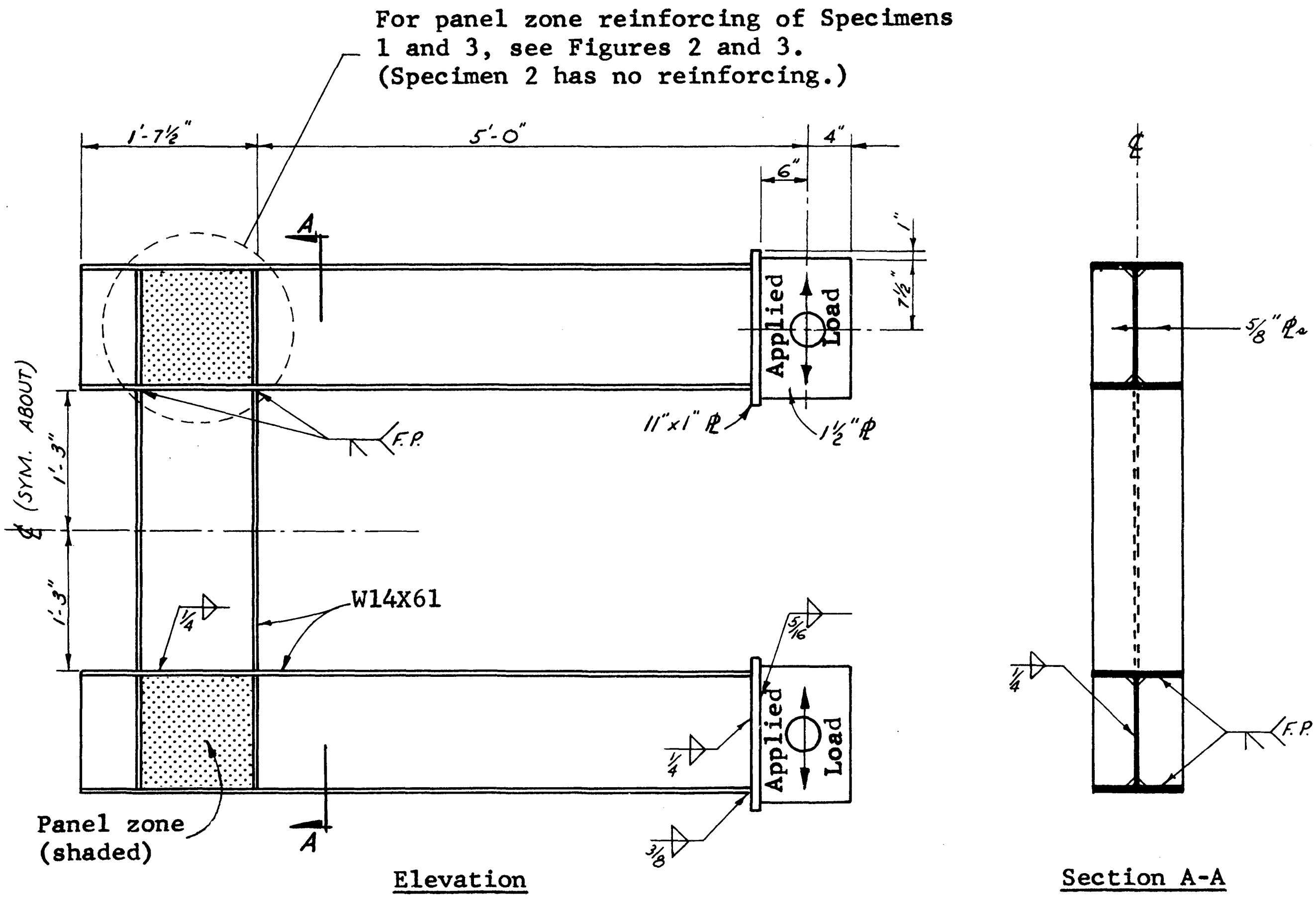


Fig. 1. Test specimen details

Three specimens were tested. The specimens were identical in every respect *except within the panel zone:* Specimen 1 had a ½-in. web doubler plate (see Fig. 2); Specimen 3 had a ⅝-in. web doubler plate in the panel zone (see Fig. 3). In Specimen 1 a relatively large weld was used to attach the web doubler plate to the W14 web, while in Specimen 3 a much smaller sized weld was used.

Based on the AISC Specification, the theoretical load capacities of the specimens are as indicated in Table 1. These load capacities are based on a yield stress of 36.0 ksi for A36 material. However, the actual yield stresses of the fabricated material were somewhat in excess of 36.0 ksi. Certified mill test reports gave these values for the yield points:

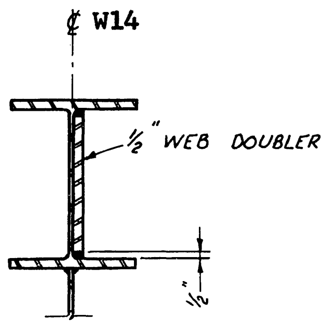
W14×61 .... F_y = 40.1 to 41.4 ksi
½" web doubler plate .... F_y = 37.5 to 38.1 ksi
⅝" web doubler plate .... F_y = 42.2 to 43.1 ksi

Table 1 indicates that the element controlling the load capacity of Specimen 1 was not the same for allowable stress design and plastic design. This is due to the fact that in allowable stress design the AISC Specification does not specify as high a safety factor for shear stress as it does for bending and axial stresses.

Since Specimen 2 did not have a web doubler plate, its load capacity was theoretically much less than those of Specimens 1 and 3. The study of this particular problem was an essential part of the investigation.

TESTING INSTALLATION

Load was applied to the specimen by means of a Riehle testing machine with a capacity of 160 kips. The opposite end of the specimen was supported on a calibrated spring which carried the tributary weight of the specimen. Lateral stability of the compression flanges of the specimen was provided by a timber bracing frame which had steel angles as guides. See Fig. 4.



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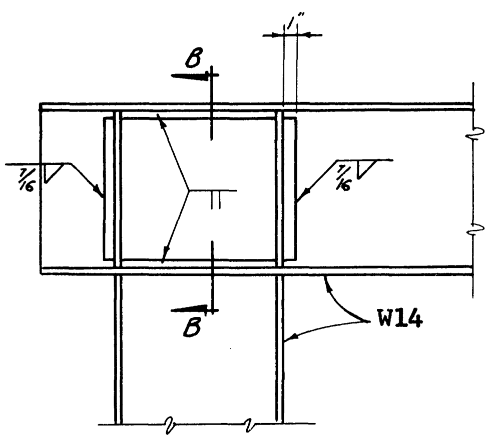
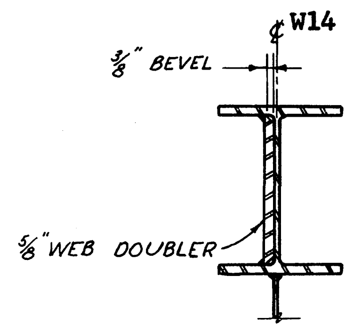


Fig. 2. Panel zone of Specimen 1



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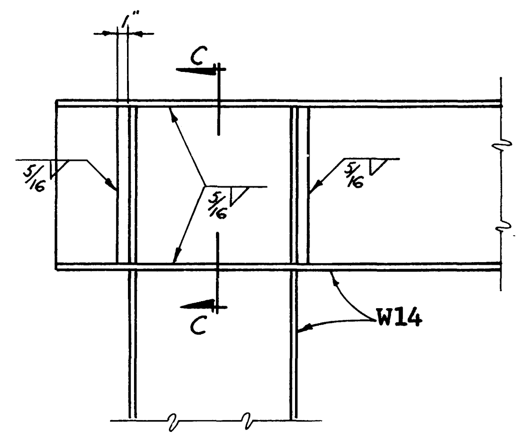


Fig. 3. Panel zone of Specimen 3

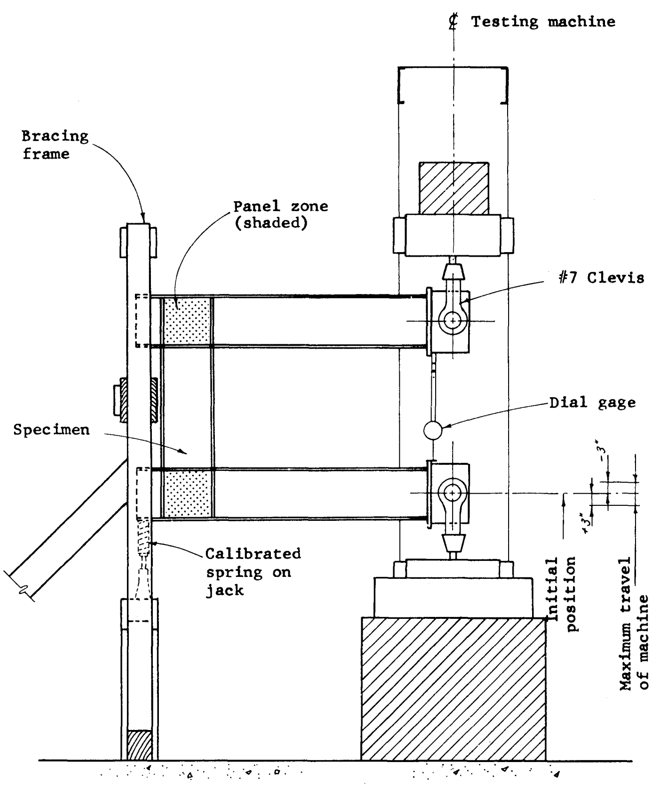


Fig. 4. Schematic of testing installation

Load-Deflection Measurements—Dial gages were used to measure the deflection of the specimen at two critical locations:

1. At the ends or tips of the specimen. (See Fig. 4.)
2. Within the panel zone a triangulation system consisting of three dial gages was attached to one inch round rods which were glued (epoxy) to the web of the W14×61. These gages measured the angular deformation of the panel zone as load was applied to the specimens. (See Fig. 5.)

Load-Strain Measurement—At critical locations throughout the specimens electrical resistance type strain gages were used to determine actual strains. Many strain gages were attached to the panel zone, while other gages were attached to the flanges of the horizontal and vertical members at points of maximum moment.

Since shear strain cannot be measured with strain gages oriented parallel to the direction of the applied shear force, most of the gages within the panel zone were oriented along a diagonal across the panel zone. These

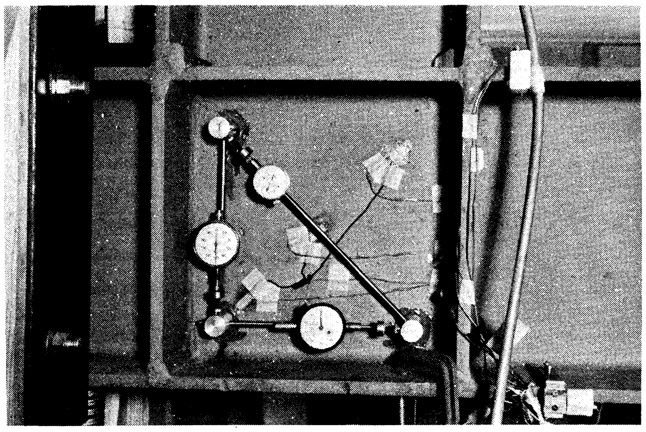


Fig. 5. Panel zone dial sags

gages therefore measured the principal normal strains in the panel zone (provided that it is assumed that only shear stresses occur). Based on the von Mises criterion that the average shear stress which will produce yielding is $F_y/\sqrt{3}$, it can be shown that for A36 material a principal normal strain of 0.000930 indicates the beginning of yielding due to shear stress.

TESTING PROCEDURES, MEASUREMENTS AND OBSERVATIONS

The three test specimens were subjected to cyclic loading which initially was set at a relatively small predetermined load, and then increased in predetermined increments (load control). Generally, the predetermined loads were applied to the specimen for three complete cycles of loading. However, when large inelastic deformations began to take place, the tip deflection of the specimen was controlled rather than the load (deflection control). As an example, the complete cycling program for Specimen 1 is shown in Fig. 6.

In this test the maximum load applied to each test specimen was not governed by the ultimate strength of the specimen, but rather by the maximum travel capacity of the crosshead of the testing machine. The maximum travel capacity was approximately 3 in. up or down from the original position of the crosshead of the testing machine.

None of the specimens "failed," if "failure" of a specimen is defined as the condition that an increase of tip deflection is accompanied by a decrease in applied load.

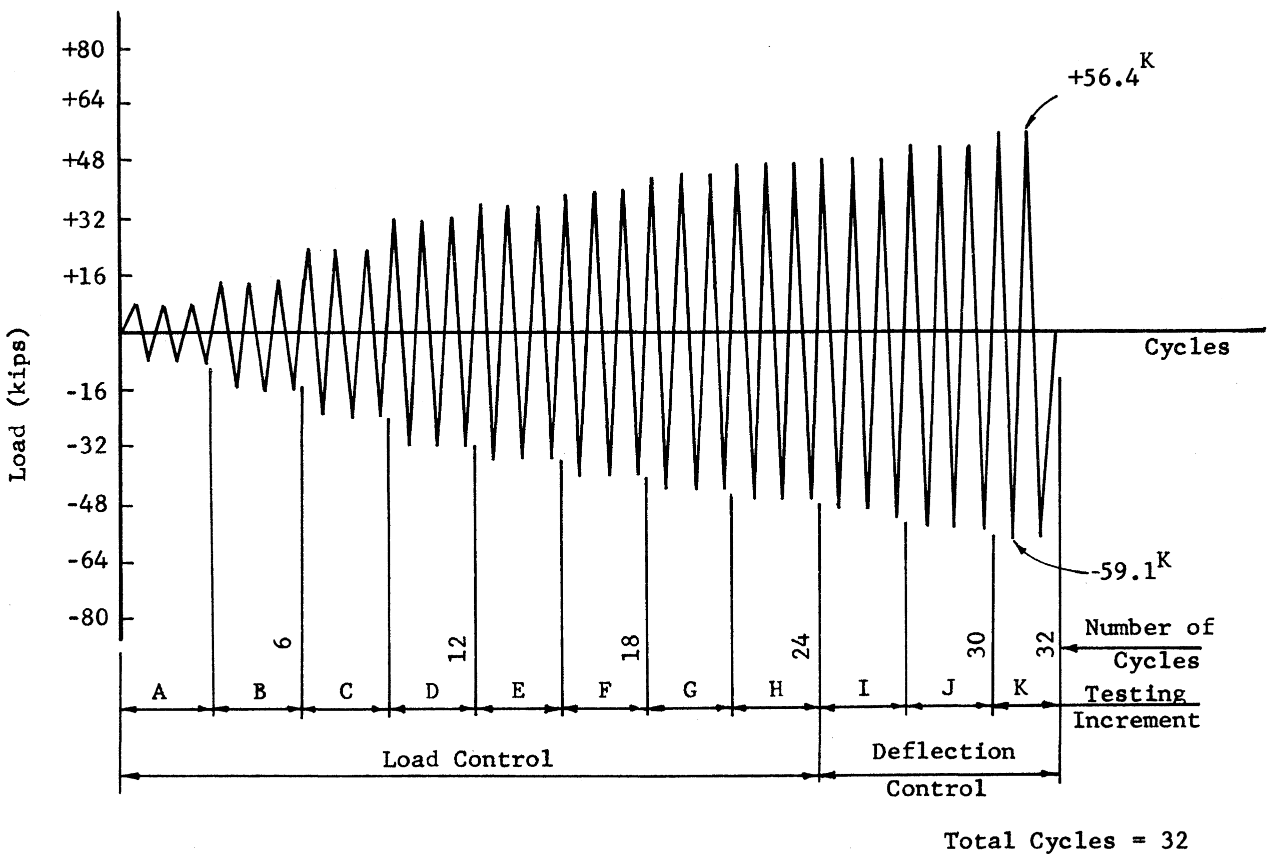


Fig. 6. Cycling program for Specimen 1

Specimen 1—This specimen was subjected to 32 cycles of loading. The maximum applied load was 59.1 kips, which occurred at the maximum travel of the crosshead of the testing machine. Although the specimen did not fail nor did cracks appear, relatively large inelastic tip deflections occurred for loads in excess of 43 kips. In addition, minor local buckling of the flanges of the horizontal members occurred, and the vertical member was permanently bent about its weak axis.

Specimen 2—This specimen was subjected to 36 cycles of loading. The maximum applied load was 40.2 kips. Although the specimen did not fail nor did cracks appear, relatively large inelastic tip deflections occurred for loads in excess of 24 kips. Almost all of the inelastic deformation of the specimen was due to yielding of the panel zone. At the maximum applied load, the average strains within the panel zone were approximately 15 times greater than the yield strain, and the panel zone had buckled about ⅛-in. per 12-in. run. There was virtually no bending of the vertical member about its weak axis.

Specimen 3—This specimen was subjected to 32 cycles of loading. The maximum applied load was 58.6 kips. Although the specimen did not fail nor did cracks appear, relatively large inelastic deformations occurred for loads in excess of 47 kips. This specimen behaved much like Specimen 1, and for the larger loadings the vertical member was permanently bent about its weak axis; however, no local buckling of the flanges of the horizontal members was observed.

ANALYSIS AND DISCUSSION OF TEST RESULTS

Load-Tip Deflection Comparison—The test results obtained by measuring the tip deflection of the specimens indicated that there was a large difference in both the strength and stiffness of the three specimens. Specimens 1 and 3 performed almost identically. However, Specimen 2 demonstrated that its strength and stiffness were very much less than those of the other two specimens. A comparison of some of the results is shown in Fig. 7, where the average absolute value of the maximum amount of tip deflection occurring during the first cycle of each testing increment has been plotted as a function of the corresponding average absolute value of the maximum tension and compression loads for that cycle. Points on the load-deflection curves indicate the allowable stress capacity and the plastic design capacity of each specimen as calculated per the AISC Specification.

The following observations can be made from Fig. 7:

1. The strength and stiffness of Specimen 2 are much less than those of the other two specimens. The difference is *entirely* due to the omission of a web doubler plate on Specimen 2.
2. For loads greater than 33 kips, Specimen 1 is not as stiff as Specimen 3. This can be accounted for since the web doubler plate for Specimen 3 is ⅛-in. thicker than the web doubler plate for Specimen 1. On the other hand, at maximum load the stiffness of Specimen 1 exceeds that of Specimen 3; this may be due to the fact that in Specimen 3 an undersized weld was used to attach the web doubler plate to the W14 web.

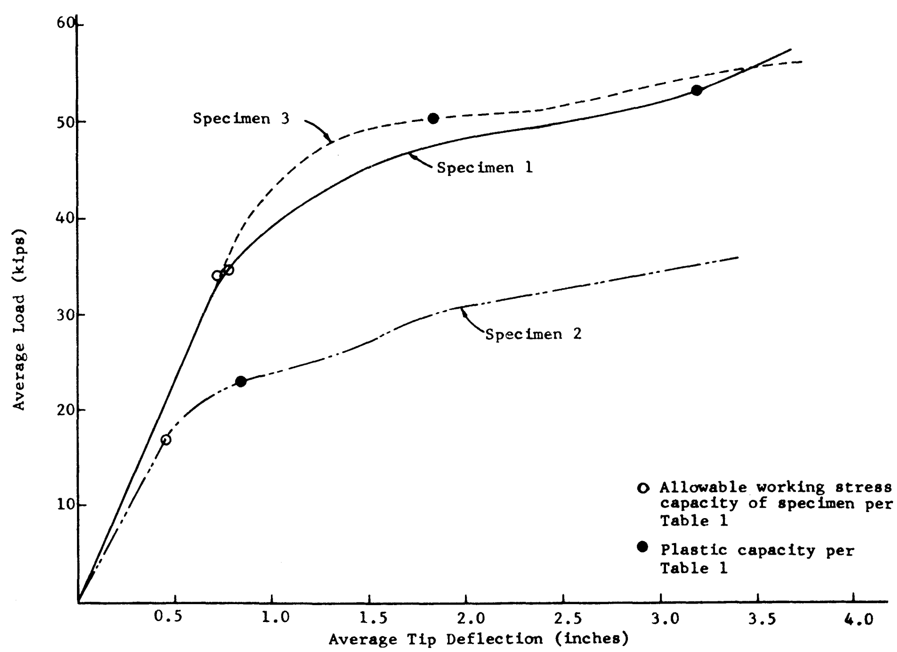


Fig. 7. Load versus tip deflection of specimens

If an engineer had actually designed frames identical to the test specimens and the panel zone had been neglected, the load capacity of the frame would have been calculated as approximately 33.7 kips. Figure 7 shows that for a load of 33.7 kips, a frame designed in accordance with the AISC Specification, such as Specimen 1, would undergo a deflection of 0.75 in., while a frame in which the panel zone had not been properly designed, such as Specimen 2, would undergo a deflection of 2.9 in.

Panel Zone Strains—In Figs. 8, 9, and 10 the average strain in the panel zone is plotted as a function of the applied load. This strain was obtained by taking the average of the absolute value of the maximum readings of the five gages located at the four corners and the center of the panel zones. The strains recorded by the five gages were found to be relatively uniform, which indicates that the usual assumption of uniform shear stress in the panel zone is valid.

Figures 8 and 10 contain two curves: one curve shows the average strain in the web doubler plate, and the other the average strain in the panel zone web of the W14×61. It was originally thought that the average strains in the web doubler plate and the web of the W14 would be equal, but this was not the case. In general, the strain in the web doubler plate was significantly less

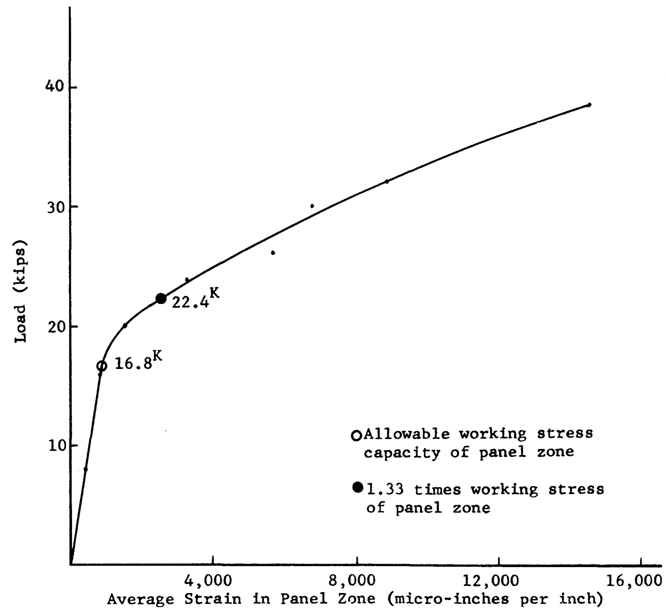


Fig. 9. Average panel zone strain for Specimen 2

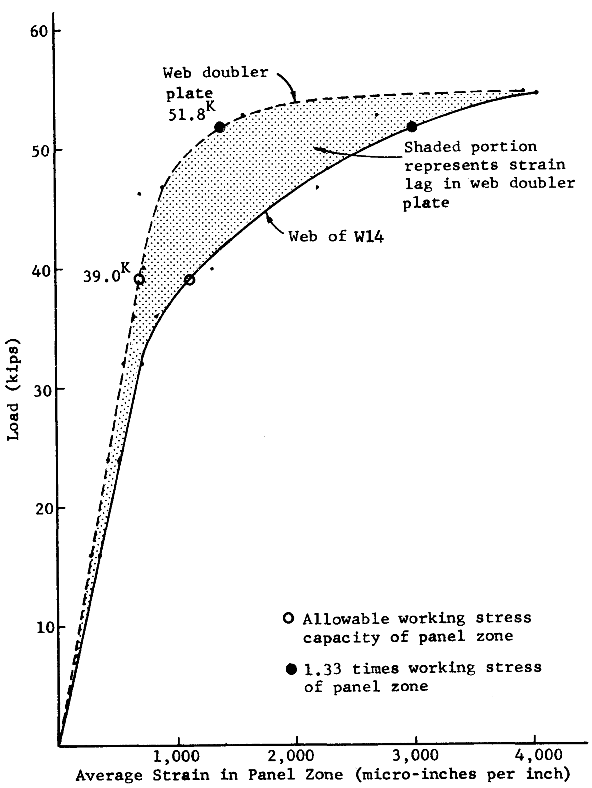


Fig. 8. Average panel zone strain for Specimen 1

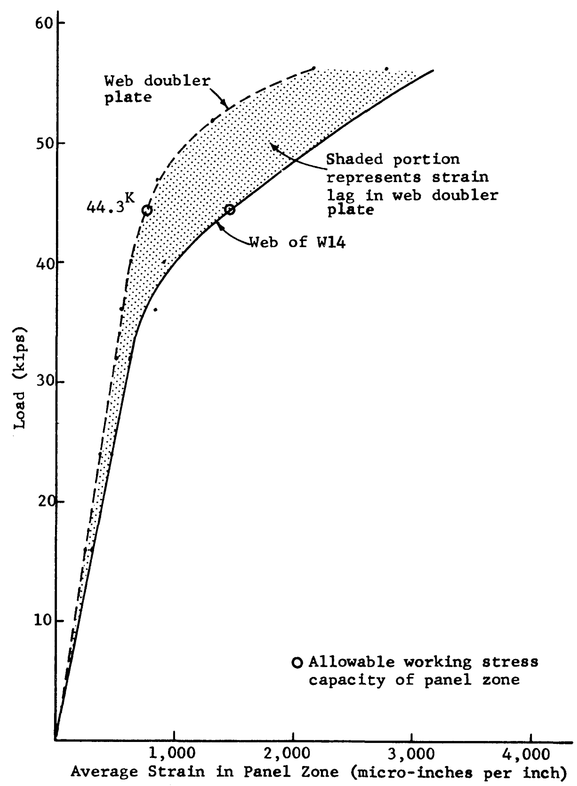


Fig. 10. Average panel zone strain for Specimen 3

than that in the panel zone web of the W14. For instance, at a load of 47.0 kips for Specimen 1 the average strain in the doubler plate was 878 micro-inches per inch, while that in the panel zone web of the W14 was 2188 micro-inches per inch. Thus, based on its cross-sectional area, the web doubler plate does not usually carry its proportionate share of the shear force that has to be transferred across the panel zone. On the other hand, at maximum load it can be seen that for Specimen 1 the strains are approaching compatibility. This means that at the ultimate capacity of the panel zone, a web doubler plate which is adequately welded to the member to be reinforced will probably carry its proportionate share of the shear force.

Figures 8 and 10 show that small inelastic deformations occurred at the allowable stress capacity (14.5 ksi) of the panel zone. These figures also indicate that significant inelastic deformations occurred at 1.33 times the allowable stress capacity (19.3 ksi), which is the stress level permitted by the code for wind and seismic forces.

Figure 9 indicates that exceptionally large strains occur in the specimen without a web doubler plate (a strain of 930 indicates yielding).

Panel Zone Deformation—The shear or angular deformation of the panel zone had a significant effect on the stiffness of the specimens. Figure 11 shows the shear deformation in a given specimen at a specified load. The average absolute value of the maximum deformation occurring during the first cycle of each testing increment has been plotted as a function of the corresponding average absolute value of the maximum tension and compression loads for that cycle. The curves were obtained in the following manner:

The components of tip deflection which could be accurately calculated for a given load were subtracted from the total tip deflection. These components were those caused by bending and shear stress in the horizontal members, and those due to bending and axial stress in the vertical member. (These components of tip deflection remain elastic for the loadings indicated in Fig. 11.) The difference between the measured (total) tip deflection and the sum of the calculable components was attributed entirely to the deformation of the panel zone, since all other sources of deflection had been taken into account. The deformation determined in this manner has been termed "computed panel zone deformation."

The curves shown in Fig. 11 were substantiated by two other experimentally measured quantities. One of these was the change in length measured by a dial gage across the diagonal of the panel zone web of the W14. Based on this change in length, the angular deformation of the panel zone was determined. A graphical comparison between the "computed panel zone deformation," shown in Fig. 11, and the dial gage measured deformation is shown in Fig. 12.

The other measured quantity used for verification of Fig. 11 was the average reading of the strain gages located in the panel zone. Based on these readings, the angular deformation of the panel zone was determined. A graphical comparison between the "computed panel zone deformation," shown in Fig. 11, and the strain gage measured deformation is shown in Fig. 13.

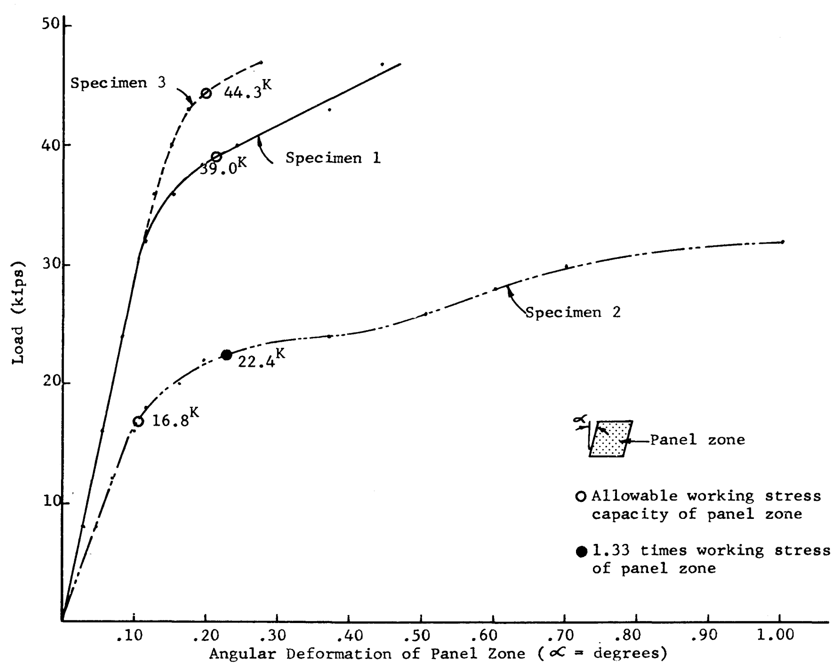


Fig. 11. Load versus deformation of panel zone

Figures 12 and 13 illustrate good correlation between the computed and measured panel zone deformations for Specimen 2. However, since Figs. 12 and 13 indicate varying degrees of correlation between the computed and measured panel zone deformations for Specimen 3, it can be concluded that the curve for this specimen in Fig. 11 is only moderately accurate.

Figure 14 is similar to Fig. 11, except that the average shear stress in the panel zone has been plotted instead of the load. The shear stress has been computed using the assumptions stated in Sect. 1.5.1.2 of the *Commentary to the AISC Specification*. Figure 14 may be used to determine the amount of angular deformation of the panel zone once the shear stress in that zone is calculated.

Figure 14 shows that for a given average shear stress in the panel zone, a specimen with a web doubler plate will have a greater panel zone deformation than a specimen without such a plate. This is due to the fact that the web doubler plate does not carry its proportionate share of the shear force that has to be transferred by the panel zone. The web doubler plate, therefore, is not as effective as it is assumed to be. When the average shear stress in the panel zone is equal to the allowable shear stress, the deformation of the test specimen without web doubler plates (Specimen 2) is 0.11 degrees, and the corresponding deformation of test specimens with web doubler plates (Specimens 1 and 3) is almost 0.2 degrees.

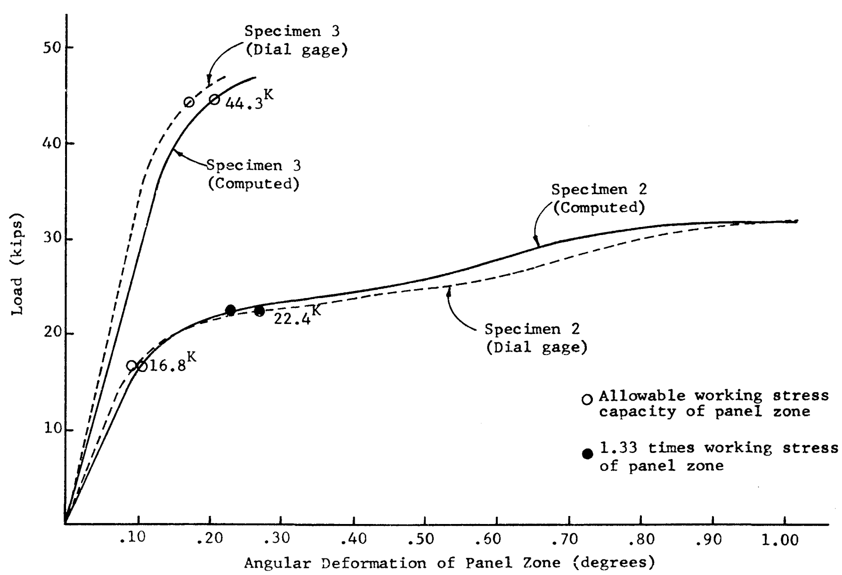


Fig. 12. Verification of deformation of panel zone

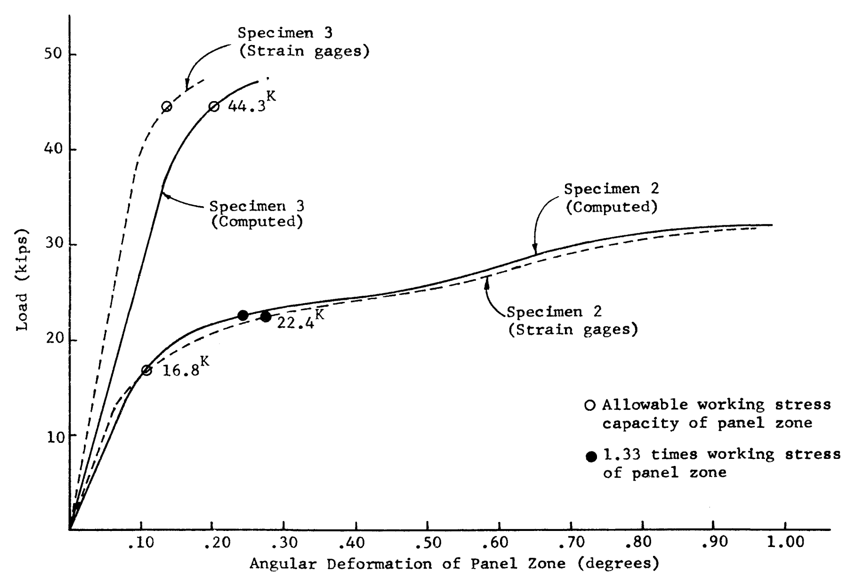


Fig. 13. Verification of deformation of panel zone

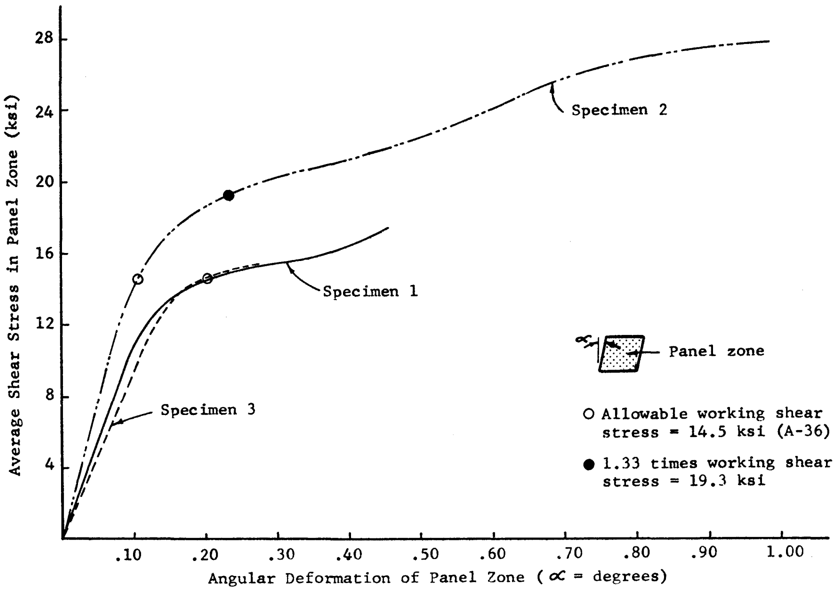


Fig. 14. Shear stress versus deformation of panel zone

Figure 14 clearly indicates that yielding of the panel zone starts at an average shear stress level not generally associated with yielding. For Specimen 2, shear yielding begins at an average stress of about 13 ksi, which is due to local yielding within the panel zone. For Specimens 1 and 3, shear yielding starts at an average stress of approximately 10 ksi, which primarily is caused by the strain lag shown in Figs. 8 and 10.

It should be noted that Fig. 14 is based entirely on the results of testing a W14×61 shape of A36 material. However, since the major portion of the deformation of the panel zone is due to shear and not bending, Fig. 14 can probably be applied to other light column shapes of A36 material with a reasonable degree of accuracy. The curve for Specimen 2 can be used for light column shapes that are not reinforced with web doubler plates, whereas the curves for Specimens 1 and 3 can be used for light column shapes reinforced with web doubler plates provided that they have about the same ratios of column web thickness to doubler plate thickness as those of the test specimens.

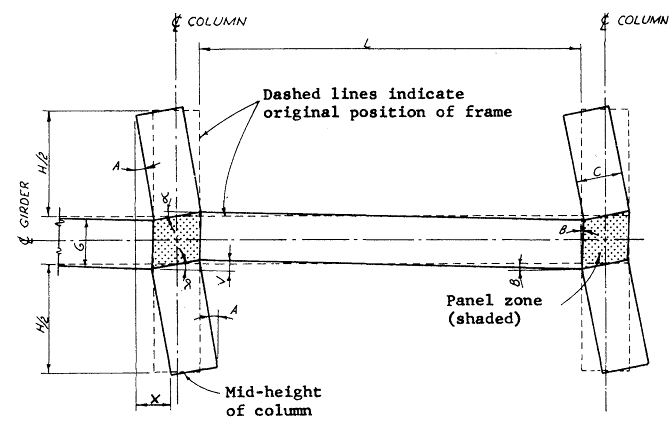
PANEL ZONE DEFORMATION AND FRAME DRIFT

The relationship between panel zone deformation and drift of a rigid frame is shown in Fig. 15. The columns and girders outside the panel zone are assumed to be infinitely rigid, and the panel zone is the only element undergoing deformation. Of course, in a real frame the columns, girders and panel zones all will contribute to the drift; however, the purpose of Fig. 15 is only to determine the part of the drift that is contributed by the panel zone deformation.

Based on Fig. 15, the effect of the panel zone deformation on the frame drift is determined as follows:

$$A = \alpha - B$$

where α can be determined from Fig. 14 when the shear stress in the panel zone is determined and B can be determined from the geometry of Fig. 15.



Partial Elevation of Rigid Frame

Notation
α = Angular deformation of panel zone, radians
A = Rotation of column from vertical position, radians
B = Rotation of panel zone to maintain compatibility, radians
C = Column depth
G = Girder depth
H = Clear height of column
L = Clear span of girder
V = Difference in elevation of panel zone corners
X = Drift or horizontal deflection of frame per story

Fig. 15. Relationship between panel zone deformation and frame drift

$$B = \frac{V}{L} \quad \text{and} \quad V = (\sin A)(C) \cong AC$$

$$B = \frac{AC}{L}$$

Therefore,

$$A = \alpha - \frac{AC}{L} = \frac{\alpha L}{L + C}$$

The drift of the frame is the sum of the two half-lengths of the columns rotating counterclockwise through the angle A, less the panel zone rotating clockwise through the angle B. Thus,

$$\begin{aligned} X &= 2\left(\frac{H}{2}\right)(A) - (B)(G) \\ &= \frac{H\alpha L}{L + C} - \frac{ACG}{L} \\ &= \left(\frac{\alpha}{L + C}\right)(HL - CG) \qquad (1) \end{aligned}$$

Equation (1) gives the drift or horizontal deflection of the frame per story. Frequently the drift of a frame is expressed as the ratio of horizontal deflection per story to story height. Therefore,

$$\begin{aligned} \text{Drift} &= \frac{\left(\frac{\alpha}{L + C}\right)(HL - CG)}{G + H} \\ &= \frac{(\alpha)(HL - CG)}{(L + C)(G + H)} \qquad (2) \end{aligned}$$

The maximum allowable drift for most multi-story building frames is usually specified as somewhere between the limits of 0.0025 and 0.0050.

The shear in the panel zone of a rigid frame can be determined by several methods (for example, see Ref. 2, page 5.7–29). It should be noted that for an interior column of a rigid frame subjected to lateral loads, the girder moments acting on the column will be in the *same direction concurrently*. Therefore, an interior column can be subjected to very large shears from these lateral load girder moments.

The following examples illustrate the influence of panel zone deformation on the drift of a rigid frame.

Example 1

Given:
A rigid frame with W14×61 columns of A36 material is subjected to a lateral load which produces a shear stress of 14.5 ksi in the unreinforced panel zones of the columns. The dimensions of the frame are: $C = 1.5$ ft, $G = 2.0$ ft, $H = 10.0$ ft and $L = 28.0$ ft. Determine the drift of the frame considering *only* panel zone deformation.

Solution:
From Fig. 14 and the curve for Specimen 2, it is found that at the allowable shear stress of 14.5 ksi the deformation of the panel zone = 0.11 degrees. Converting from degrees to radians and substituting into Eq. (2), the drift becomes:

$$\text{Drift} = \frac{(0.11)(10.0 \times 28.0 - 1.5 \times 2.0)}{57.3(28.0 + 1.5)(2.0 + 10.0)} = 0.0015$$

This drift produced by deformation of the panel zone probably would be acceptable, providing the columns and girders possessed adequate stiffness.

Example 2

Given:
Data the same as Example 1, except that the shear stress in the panel zone is 1.33 times the allowable shear stress = 19.3 ksi (stress permitted by code for wind and seismic forces).

Solution:
Using Fig. 14, the deformation of the panel zone = 0.23 degrees. With a linear extrapolation from the data given for Example 1:

$$\text{Drift} = \frac{0.23}{0.11}(0.0015) = 0.0031$$

This drift produced by deformation of the panel zone may not be acceptable, since the additional components of drift due to the columns and girders outside the panel zone might create a total drift greater than the allowable.

Example 3

Given:
Data the same as Example 1, except that the shear in the panel zone has been neglected in the design, and the applied lateral load on the frame produces a shear stress of 24.0 ksi in the panel zone.

Solution:
Using Fig. 14, the deformation of the panel zone = 0.60 degrees. By linear extrapolation from Example 1:

$$\text{Drift} = \frac{0.60}{0.11}(0.0015) = 0.0082$$

This drift produced by deformation of the panel zone is probably greater than many structures could tolerate.

These examples illustrate that the panel zone must be considered in the design of rigid frames subjected to lateral loads.

It can be observed from Fig. 14 that, if the columns in these examples had their panel zones reinforced with web doubler plates and the panel shear stresses remained constant, the drift produced by the panel zone deformation would be greater than that calculated.

CONCLUSIONS

The results of this study provide answers to the five questions posed in the introduction of the paper. However, it must be emphasized that the conclusions are based entirely on the results of tests using a W14×61 shape of A36 material, and that extrapolation to other sizes of members must be done with caution. Thus, the conclusions primarily apply to steel rigid frames which have light wide-flange shapes as columns.

1. The panel zone can be the weakest element in a steel rigid frame, and the strength and stiffness of the entire frame therefore may depend on the shear capacity of the panel zone. The panel zone is especially important for rigid frames subjected to lateral loads.

2. Web doubler plates can be used effectively to increase the shear capacity of the panel zone. When high shear stresses are present in the panel zones of a steel rigid frame, the strength and stiffness of the frame can be significantly increased by using web doubler plates.

3. The shear strains (and stresses) in a web doubler plate are generally less than those in the panel zone web of the reinforced member. This means that a web doubler plate does not usually carry its proportionate share of the shear force being transferred by the panel zone. However, when the shear strains in the panel zone reach values of three to four times yield, the strains in the doubler plate and the web of the reinforced member become equivalent.

4. When the shear stress in the panel zone of a steel rigid frame is known, the panel zone's contribution to the drift of the frame can be determined by using Fig. 14 and Eq. (2).

5. The design criteria for the panel zone given in Sect. 1.5.1.2 of the *Commentary to the AISC Specification* appear to be realistic. However, when the drift of a rigid frame is critical and/or web doubler plates are used, the allowable shear stress in the panel zone should be reduced in some cases. The amount of reduction should be based on the desired structural performance of the rigid frame. Figure 14 can be advantageously used in determining the reduced allowable shear stress in the panel zone, since it indicates the stress at which yielding will start and the amount of panel zone deformation that can be anticipated.

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