Column Web Compression Strength at End-Plate Connections

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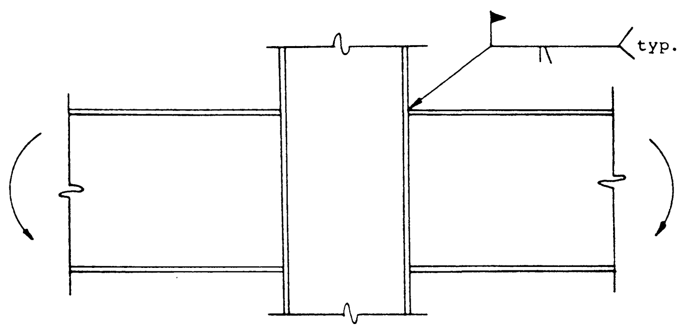
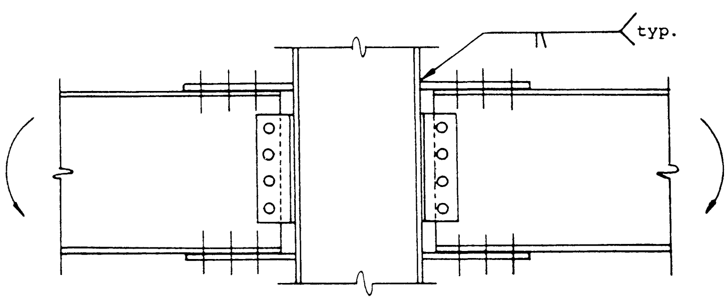
INTRODUCTION

Current North American specifications provide design criteria to prevent local failure of H-shaped columns when flanges or moment connection plates are welded to the column flange, as in Fig. 1. This design criterion was developed strictly for these types of connections. Application of these criteria when bolted end-plate connections are used, Fig. 2, may result in the unnecessary use of column stiffeners opposite beam flanges. Installation of column stiffeners is expensive and stiffeners can interfere with weak axis framing into the column, as in Fig. 3. If stiffeners between the flanges of H-shaped columns can be eliminated, the fabrication process is greatly simplified.



(a) All Welded Connection



(b) Welded and Bolted Connection

Fig. 1. Continuous beam-to-column connections

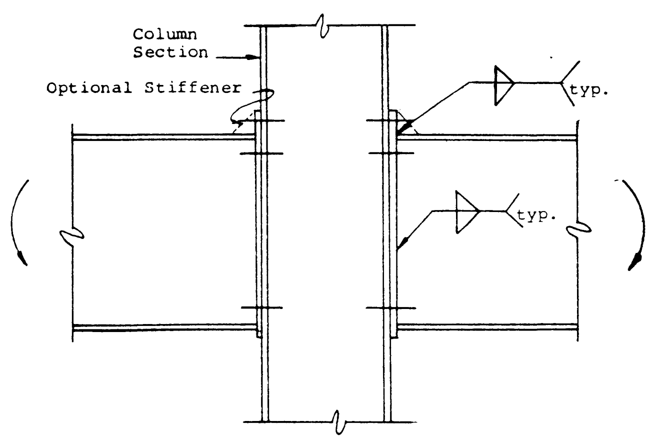


Fig. 2. Typical end-plate moment connection

A critical location in the column web of beam-to-column moment connections is at the toe of the column web fillet. For design of welded connections, the present (1978) AISC Specification[1] criterion is based on a load path assumed to vary linearly on a 2½:1 slope from the beam flange through the column flange and fillet, as in Fig. 4. If the stress at this critical section exceeds the yield stress of the column material, a column web stiffener is required opposite the beam compression flange.

In the case of end-plate moment connections, the width of the stress pattern at the critical section may be considerably wider due to insertion of the end-plate into the load path. The fillet weld connecting beam and end-plate may also influence the width, as well as end-plate stiffeners of the type in Fig. 2. To verify or discredit these

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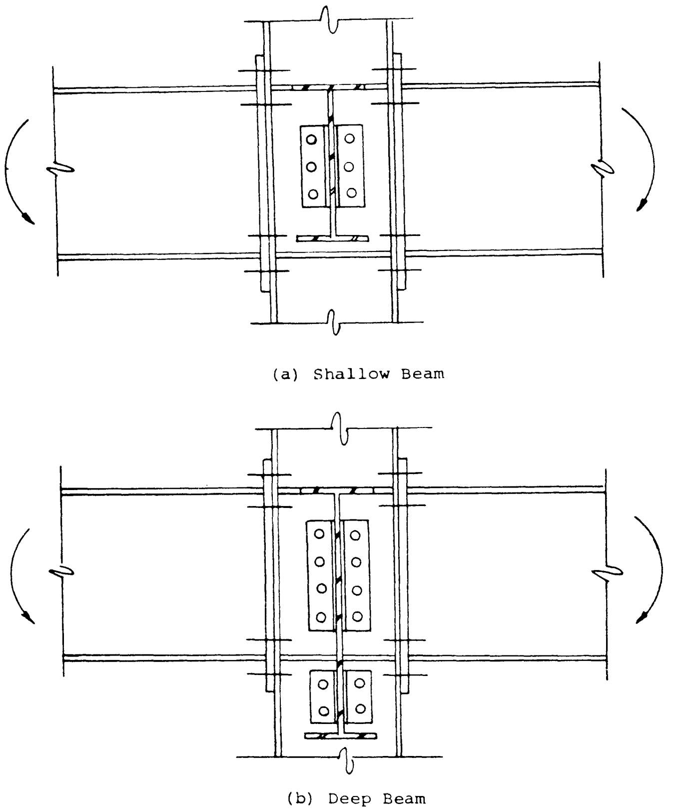


Fig. 3. Weak axis framing details

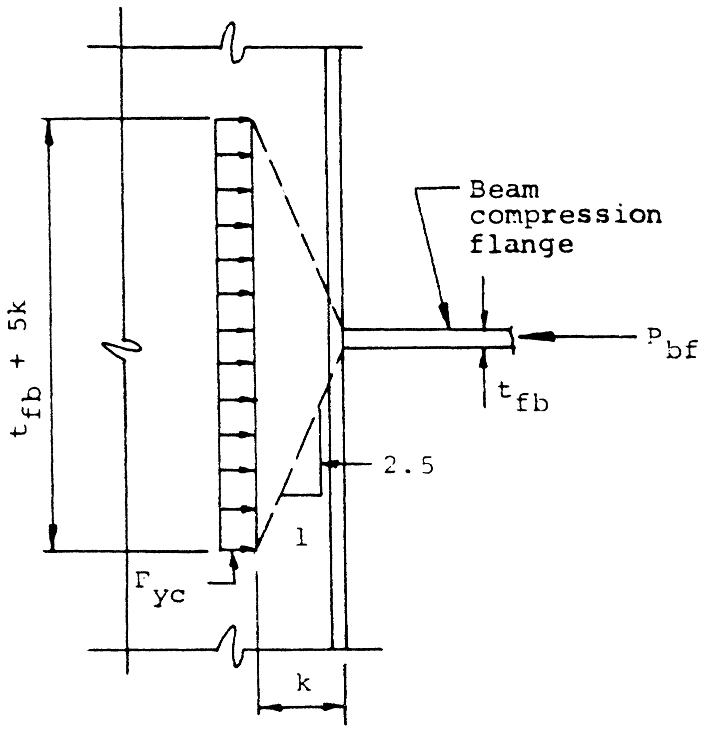


Fig. 4. Distribution of stresses at beam compression flange

assertions, an extensive literature survey was conducted,[2] followed by experimental and analytical studies. The result is a proposed design criterion for column web yield compression strength at end-plate connections.

LITERATURE REVIEW

Graham, Sherbourne, and Khabbaz[3] conducted a series of tests in which a bar was welded to the column flange to simulate the beam compression flange in a welded connection (Fig. 1a). This simulated beam-flange connection neglected possible effects of column axial load and the effect of the compression from the beam web on the column-web strength. Based on test results using the simulated beam flange, the authors conservatively suggest that compressive stress distributes through the column web on a 3½:1 slope. Using this relationship, results in the following equation for the maximum force which can be resisted by the column web,

$$P_{max} = F_{yc} t_{wc} (t_{fb} + 7k) \tag{1}$$

where P_{max} = maximum force the column web is capable of resisting (kips), F_{yc} = yield stress of column material (ksi), t_{wc} = column web thickness (in.), t_{fb} = beam flange thickness (in.), k = column "k" distance (in.). Because of additional compression supplied by the beam web, subsequent full connection tests gave lower results than obtained in the simulated beam flange test. According to the authors, if the stress is distributed on 2½:1 slope through the column, a conservative estimate for the full connection test is obtained. Hence, they recommend for design

$$P_{max} = F_{yc} t_{wc} (t_{fb} + 5k) \tag{2}$$

Newlin and Chen[4] attempted to develop a method of determining ultimate loads for the compression region of column sections having slender webs. Further, they attempted to develop a single formula for predicting the maximum web capacity of a column section regardless of the d_c/t_{wc} ratio rather than separate equations for strength and stability. Fifteen tests in several series were performed to investigate the effect of varying flange and loading conditions. In addition, results from tests conducted by Chen and Oppenheim[5] were also included in the study. One series of tests investigated the effect of opposing beams of unequal depth at an interior beam-to-column moment connection. This geometry results in a situation where the loads applied to the compression region are eccentric. A second series investigated the contribution of the column flange to the load carrying capacity of the column web. In this series, cover plates 1-in. thick, 20-in. long and slightly wider than the specimen flanges to permit fillet welding all around were used. All tests were performed with simulation of the beam flange by welding a bar to either the cover plate or directly to the column.

Test results show the ultimate load of a column web is "essentially unaffected by the eccentric load condition." It appears eccentric loading has the effect of adding a small amount of stiffening to the stiffness of the web. The conclusion was that Eq. 2 is conservative for eccentric loading conditions.

To investigate the contribution of the column flange on column web buckling, W10x29 and W12x27 A36 column sections were first tested as control specimens. Plates, 1 in. by 20 in., were then welded to both flanges of each section and the tests repeated. The resulting load vs. deflection curves for the W10x29 tests are shown in Fig. 5. The increase in ultimate load with the cover plates added was approximately 31% for the W10x29 section and 33% for the W12x27 section. However, a reserve strength of only 4.8% existed for both sections at ultimate load with very limited ductility. As a result, the authors state the "presence of a cover plate on a column flange should not be considered as part of the k dimension," and "these results further support the relative insignificance of the column flange thickness as compared with web dimensions."

It is noted the P_{max} values shown in Fig. 5 are based on measured dimensions and measured yield stress. The W10x29 control test k value was reported as 0.73 in. compared to a value of 1¹⁄₁₆ in. from the 7th Edition AISC *Manual of Steel Construction*.[6] No explanation was found for this rather large difference. The k value

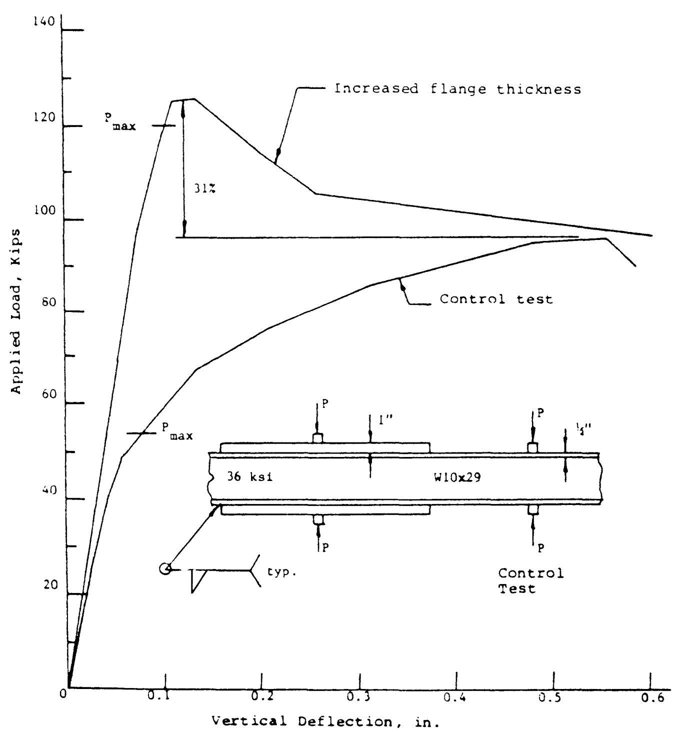


Fig. 5. Effect of cover plate on web capacity (from Ref. 3).

used to compute P_{max} for the cover plate test was taken as 1.73 in., the sum of the beam k and the 1 in. cover plate thickness.

Newlin and Chen[4] recommend that Eq. 2 not be used for design and that an interaction equation of the form

$$P_{max} = \frac{F_{yc}^{3/2}\, d_c}{180}\left(\frac{125 t_{wc}}{\sqrt[4]{F_{yc}}} - d_c\right) \qquad (4)$$

be used to check both web strength and stability. Or, in lieu of Eq. 4, a strength check be made using Eq. 2 and a stability check using

$$P_{max} = \frac{4100 t_{wc}^3 \sqrt{F_{yc}}}{d_c} \qquad (5)$$

Mann and Morris[7] reviewed the results of several research programs pertaining to column webs at end-plate connections and proposed design criteria. They stated the 1977 *European Convention of Constructional Steelwork Recommendations for Steel Construction* gives the following expression for the maximum load carrying capacity of the column web in the presence of an end-plate.

$$P_{max} = F_{yc} t_{wc} \,(t_{fb} + 5k + t_p + d) \qquad (6)$$

where t_p = end-plate thickness (in.), and d = projection of the end-plate beyond the compression flange of the beam but not greater than t_p (in.). The expression is based on the assumption that the stress is distributed on a 1:1 slope through the end-plate and on a 2½:1 slope through the column.

Witteveen, Start, Bijlaard and Zoetemeijer[8] conducted tests in the Netherlands in an attempt to develop design rules to compute the moment capacity of unstiffened welded (no end plate) and bolted (end plate) connections. For beams welded directly to the column flange, it is recommended that the column web strength be calculated from:

$$P_{max} = F_{yc} t_{wc} \,[t_{fb} + 5(t_{fc} + r_c)] \qquad (7)$$

where t_{fc} = column flange thickness (in.), and r_c = fillet between the flange and the web of the column (in.). For bolted end-plate connections, it is recommended that the end-plate and weld be considered in determining the ultimate load carrying capacity of the web according to:

$$P_{max} = F_{yc} t_{wc} \,[t_{fb} + 2\sqrt{2}\, a + 2t_p + 5(t_{fc} + r_c)] \qquad (8)$$

where a = weld dimension (in.). In this expression the stress distribution is assumed to be 1:1 through both the weld and end-plate.

Aribert, Lachal and Nawawy[9] tested European column sections with end-plate connections.* The compression

*This paper is in French. A translation of a pertinent section is found in Ref. 2.

beam flange was simulated by welding a bar to the end-plate. In addition to the testing, a numerical model was developed and maximum elastic, plastic and ultimate load expressions were obtained. After comparison with test results the following expressions were proposed.
Maximum elastic load:

$$P_{e_{max}} = F_{yc} t_{wc} (t_p + 2.3k) \tag{9}$$

Maximum plastic load:

$$P_{p_{max}} = F_{yc} t_{wc} (2t_p + 5k) \tag{10}$$

Ultimate load:

$$P_{u_{max}} = F_{yc} t_{wc} (6t_p + 7k) \tag{11}$$

Section 1.15.5.2 of the 1978 AISC Specification[1] specifies the required stiffener area to prevent column web crippling when flanges or moment connection plates for end connections of beams and girders are welded to the flange of H-shaped columns as

$$A_{st} = \frac{P_{bf} - F_{yc} t_{wc} (t_{fb} + 5k)}{F_{yst}} \tag{12}$$

where A_{st} = stiffener area (in.^{2}), P_{bf} = the computed force delivered by the flange or moment connection plate multiplied by 5⁄3, when the computed force is due to live and dead load only, or by 4⁄3, when the computed force is due to live and dead load in conjunction with wind or earthquake forces (kips), and F_{yst} = stiffener yield stress (ksi). Stiffeners are not required if A_{st} is negative. In the commentary on Sect. 1.15.5, it is stated that the actual force times the load factor, i.e. P_{bf}, need not exceed the area of the flange or connection plate delivering the force times the yield strength of the material.

In addition to Eq. 12, a column web stability check is required. Compression flange stiffeners are required if the column web depth clear of fillets, d_c, is greater than

$$\frac{4{,}100 t_{wc}^3 \sqrt{F_{yc}}}{P_{bf}} \tag{13}$$

which is the same expression as recommended by Newlin and Chen[4] (Eq. 5 of this paper).

No mention of end-plate connections is made in the 1978 AISC Specification. In an end-plate design example in the 8th Edition AISC *Manual of Steel Construction*,[10] page 4-115, it is suggested that end-plate effects can be "conservatively" accounted for by assuming a stress distribution on a 1:1 slope through the end-plate. This would result in the following equation for stiffener area

$$A_{st} = \frac{P_{bf} - F_{yc} t_{wc} (t_{fb} + 5k + 2t_p)}{F_{yst}} \tag{14}$$

No mention is made of possible weld effects and it is not known if the assumed distribution is based on test results.

In the literature survey,[2] only two tests were found where end-plate effects were considered with American sections.[4] From these tests it was concluded that cover plates (end-plates) were not effective because of minimal reserve strength above first yield (<5%) and lack of ductility in the connection.

Literature concerning European testing and design practices are consistent in recommending an assumed stress distribution through the end plate of 1:1. In addition, one paper recommends use of the beam-flange to end-plate weld dimension when calculating the length of the critical section for determining the column-web compressive strength. To verify the European practices with American sections, a limited analytical and experimental research program was conducted.

ANALYTICAL STUDY

To determine analytically stress distributions and yield patterns in the compression region of the column web at end-plate connections, an inelastic, two-dimensional, finite element program developed by Iranmanesh[11] was used. To reduce computational costs and to more closely model the test set up used in the experimental phase of the study, only a portion of the beam consisting of the flange and web was used. Load was applied directly to the beam flange. Figure 6a shows a typical mesh, support conditions and loading. Smaller elements were used in the region on the web at approximately the k distance from the edge of the column flange.

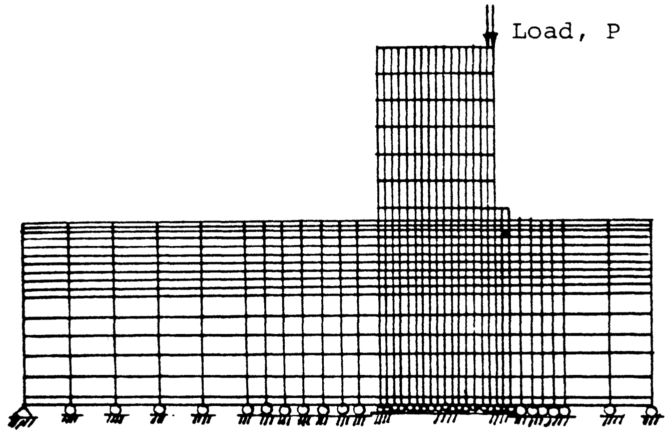
Since the computer program used was limited to two-dimensional elements, variation in thickness through the depth of the model, i.e. web, flange, weld and fillet thicknesses, was modeled by increasing the element stiffness based on the ratio of the element thickness to the thickness of the column web elements. A modulus of elasticity of 29,000 ksi in the elastic range and an assumed yield stress of 36 ksi were used.

For purposes of defining load levels, first yield was defined as the load at which the first element reached the yield strain, ϵ_y. Second yield was defined as when any element reached $3\epsilon_y$, third yield at $5\epsilon_y$, fourth yield at $7\epsilon_y$, fifth yield at $9\epsilon_y$ with an upper limit of $12\epsilon_y$ when the analysis was terminated. Typical progression of yielding through the web is demonstrated in Fig. 6.

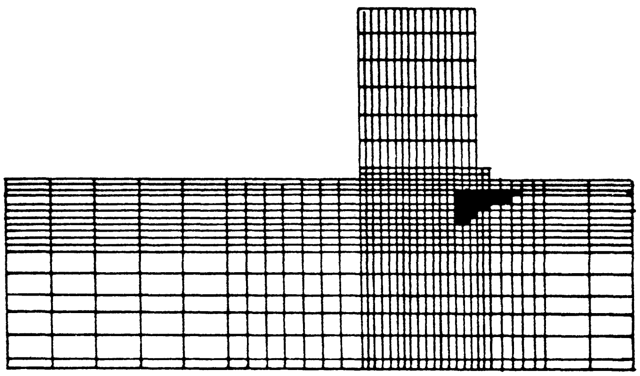
EXPERIMENTAL STUDY

Test Setup

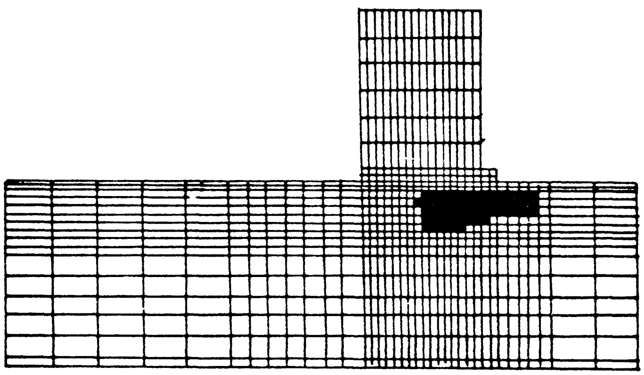
In the literature review, it was found a number of research projects have been conducted to determine the column-web strength at beam-to-column moment connections. However, except for one test, all of the re-



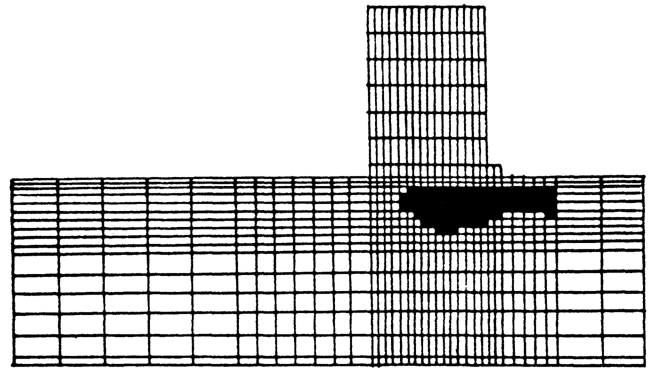
(a) 1st Yield, 145.1 kips



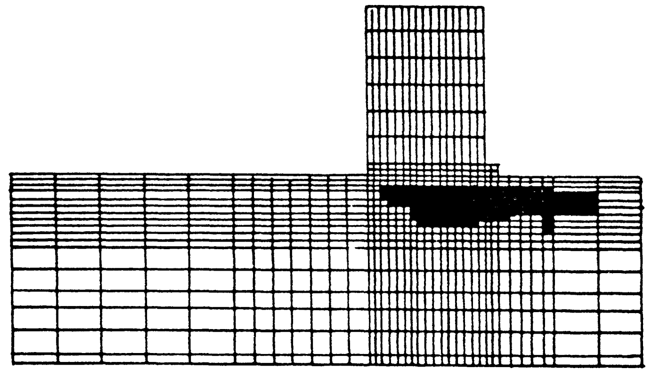
(b) 2nd Yield, 227.8 kips



(c) 3rd Yield, 246.8 kips



(d) 4th Yield, 283.1 kips



(e) 5th Yield, 298.7 kips

Fig. 6. Typical yield pattern progression from finite element analysis

search conducted in the U.S. has been limited to welded moment connections, and results of that test are considered inconclusive. European studies reviewed involved only their own sections. Thus, a limited number of tests were conducted to substantiate that load in the compression region of beam-to-column moment end-plate connections is distributed over a greater length of the column web than for welded connections.

Six tests were conducted with combinations of beam and column sections, shown in Table 1. The test set up is shown schematically in Fig. 7. Figure 8 is a photograph of the set up. Each column section was placed in a horizontal position with the load applied through the flange of the WT section using a hydraulic ram. Load was monitored with a load cell located between the ram and specimen. Lateral movement of the upper column flange was restricted by a lateral brace mechanism attached to the test frame.

Instrumentation consisted of strain gages and displacement transducers. For all six tests, strain was measured on each side of the column web at the toe of the fillet. Strain gages were located along the web to cover the expected load distribution length. In addition to the gages on the column web, strain gages were placed on the flange and stem of the WT beam section. Figure 7 shows typical locations. Two displacement transducers were used to measure vertical displacement of the column flanges. An additional transducer was used to monitor lateral displacement of the top flange of the column section.

Each specimen was loaded in approximately 10-kip increments, with readings of all instrumentation recorded at each increment. A load-deflection curve was plotted to monitor any nonlinearity. The loading was continued

Table 1. Column-Web Strength Tests

Test	Beam	Column	End-Plate Stiffener	End-Plate Thickness (in.)	Maximum Applied Load (kips)	Col. Yield Stress (ksi)	Calculated Yield Load		
							5k* (kips)	6k (kips)	7k (kips)
1	WT9x25	W14x90	No	7/8	178	38.6	174	197	221
2	WT9x25	W14x90	Yes	7/8	225	38.6			
3	WT9x48.5	W14x111	No	1½	280	33.3	221	251	280
4	WT9x48.5	W14x111	Yes	1½	330	33.3			
5	WT16.5x70.5	W14x99	No	1¼	290	38.8	235	263	291
6	WT16.5x70.5	W14x99	Yes	1¼	300	38.8			

*$F_{yc}\,t_{wc}\,(t_{fb} + 5k + 2t_p + 2\,t_w)$

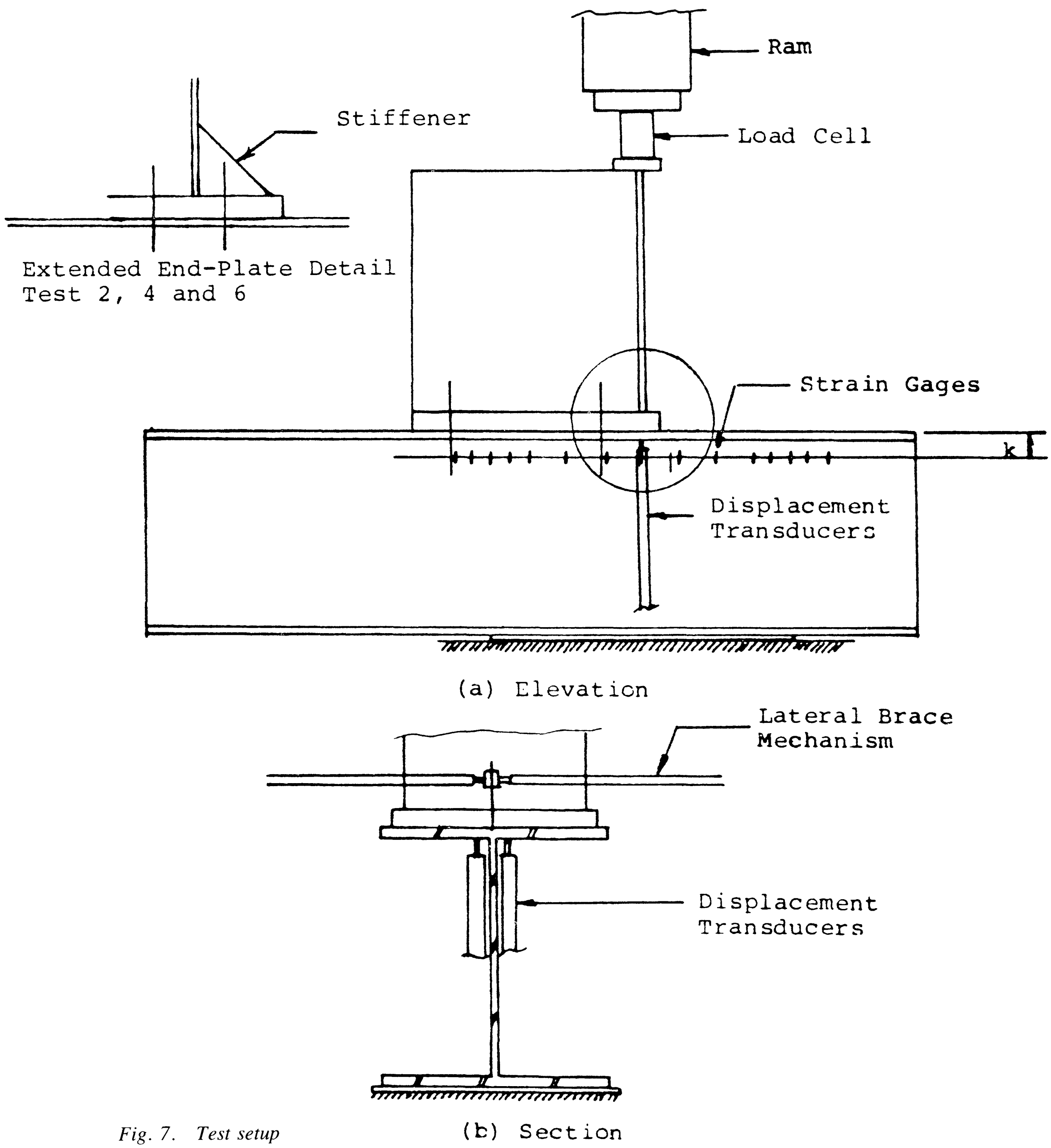


Fig. 7. Test setup

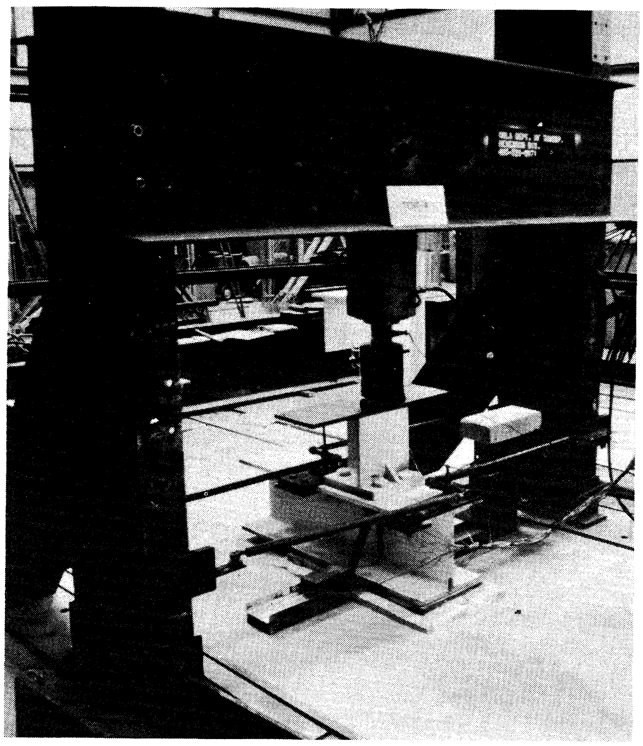


Fig. 8. Photograph of test set up

until failure of the specimen occurred. Standard tensile column tests were made from samples cut from the column webs. Results are shown in Table 1. The measured yield stresses varied from 33.3 ksi to 38.8 ksi.

The web stress distributions are compared through distribution lengths with corresponding load levels referred to as the $5k$, $6k$ or $7k$ level based on the equation

$$P = F_{yc}t_{wc}(t_{fb} + 5k + 2t_p + 2t_w) \qquad (15)$$

where t_w = beam flange to end-plate weld leg dimension and with $5k$ replaced with $6k$ and $7k$ for the higher levels. All terms are defined as previous and measured yield stress and cross-section dimensions were used for the calculations. The corresponding yielded length along the critical column web section will be referred to as the $5k$, $6k$ or $7k$ length in the following discussion of the test results.

Tests 1, 3 and 5 consisted of a WT beam section welded to the end plate, which was then bolted to the column with four bolts between the edge of the WT stem and the flange, as shown in Fig. 7. Tests 2, 4 and 6 were conducted with the same sections as Tests 1, 3 and 5, respectively, but with an extended end plate and two additional bolts on the outside of the beam flange as in Fig. 7. A triangular stiffener plate was welded in the plane of the WT stem between the WT flange and the extended portion of the end plate for these tests.

Test Results

Complete test results, found in Ref. 12, consist of load vs. deflection data and stress distribution data. The load vs. deflection data for each test includes a theoretical plot obtained from a finite element analysis, as well as measured vertical displacement of the column flange on each side of the web. In addition, stress distributions on the column web, calculated from measured strains assuming elastic-perfectly plastic material behavior, were plotted at various load levels. Figures 9 and 10 show typical results.

Test 1. Test 1 consisted of a WT9x25 beam with a W14x90 column. The beam was attached to the column with 1⅛-in. dia. A325 bolts through a ⅞-in. thick end plate. The column web yield stress obtained from a coupon test was 38.6 ksi. The maximum applied load for this test was 178 kips. However, instrumentation problems occurred, and it is not believed the maximum capacity of the col-

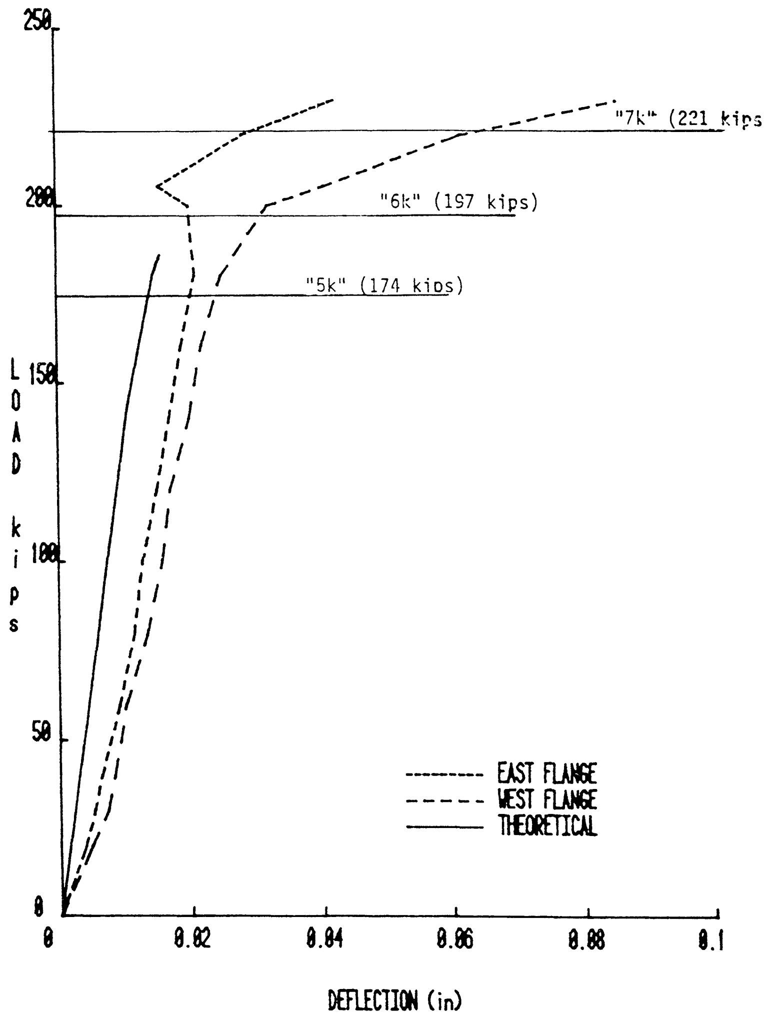


Fig. 9. Typical load vs. deflection plot (Test 2)

umn web was reached in the test. The $5k$, $6k$ and $7k$ loads were respectively 174 kips, 197 kips and 221 kips. At the maximum load level the yielded portion of the column web, as determined from strain measurements, exceeded the $6k$ distance.

Test 2. Test 2 was conducted with the same beam and column section as well as bolt size and end-plate thickness as in Test 1, but with an extended end plate and triangular stiffener as described previously. The maximum applied load was 225 kips and the $5k$, $6k$ and $7k$ loads were 174 kips, 197 kips and 221 kips. The column web yield stress was 38.6 ksi.

From the load vs. deflection plot shown in Fig. 9, it is evident the column web deflected sharply slightly above the $6k$ load level. From the measured stress distributions shown in Fig. 10, the column web at the critical section was yielded along a length of approximately $6k$ at the maximum load level. Note that the extended end plate and triangular stiffener had little effect on the yield strength.

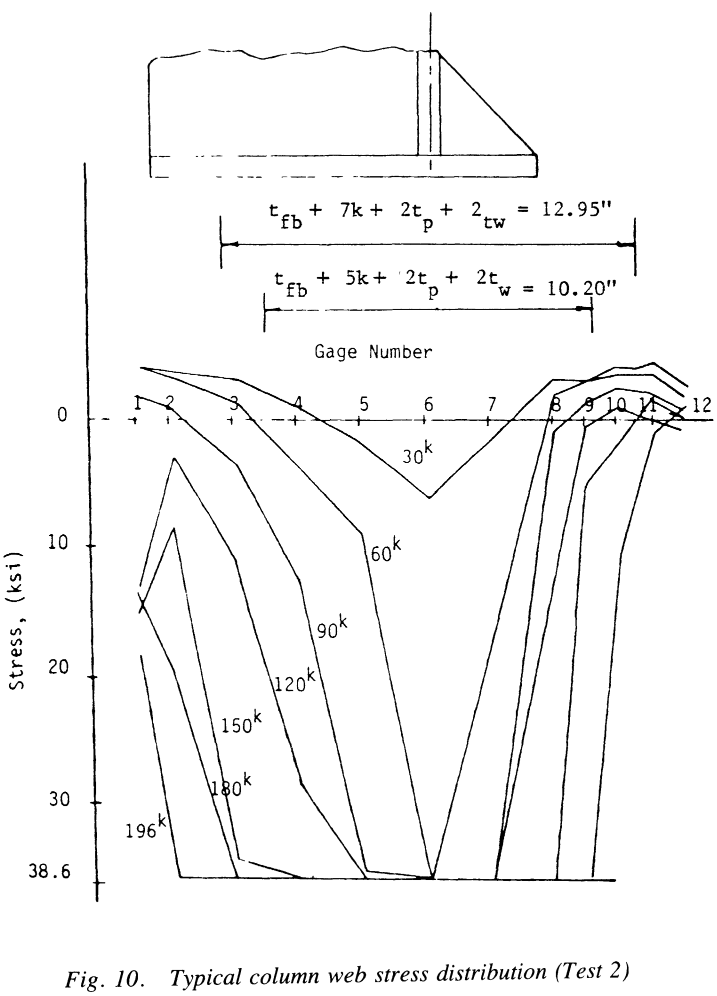


Fig. 10. Typical column web stress distribution (Test 2)

Test 3. A WT9x48.5 beam was welded to a 1½-in. thick end plate for Test Specimen 3. Four 1⅜-in. dia. A325 bolts were used to attach the end plate to a W14x111 column. The column-web yield stress was found to be 33.3 ksi.

The maximum load applied of 280 kips was at the $7k$ load levels, and the yield length along the column web at this load level was approximately $6k$.

Test 4. The test configuration for Test 4 was identical to Test 3, except for the extended end plate and triangular stiffener. The measured column web yield stress was 33.3 ksi. Maximum applied load exceeded the $7k$ level, and the yielded portion of the web exceeded the $7k$ distance.

Test 5. A WT16.5x70.5, the largest WT used in the testing program, was bolted to a W14x99 column section for this test. Four 1½-in. diameter A325 bolts were used to connect the 1½-in. thick end plate to the column flange. The yield stress of the column web was found to be 38.8 ksi. Maximum applied load was slightly less than the $7k$ level, and the yield length along the column web was approximately the $7k$ length. Distribution was centered toward the WT web.

Test 6. Test 6 was conducted with identical sections to those in Test 5. An extended 1½-in. thick end plate with triangular stiffener was used together with 1½-in. dia. A325 bolts. Comparison of results from Tests 5 and 6 shows little increase in load carrying capacity with introduction of the stiffener. The $7k$ load was exceeded in Test 6, but only slightly. However, the width of the yielded portion of the web is greater in Test 6 than Test 5.

Summary

With the exception of Test 1, the $6k$ load level was exceeded in all tests. Further, the length of the yielded portion of the web generally agreed with the maximum load reached, and the measured yield patterns were in general agreement with those obtained in the finite element analyses. The existence of a triangular stiffener plate between a beam flange and an extended end plate was not found to be effective as a method of increasing the yielded length of column web. Thus, results of this investigation indicate the $6k$ load level is an acceptable and slightly conservative web strength estimate for beam-to-column, moment, end-plate connections.

For Tests 2 through 6, failure occurred by excessive lateral movement of the column top flanges, in some instances breaking the lateral brace mechanism. This instability was caused by lack of restraint from the column web once the material had yielded below the applied load. Thus, particular attention must be paid to the local lateral stability of columns without weak axis framing. It also should be noted that no axial load was applied to the column section during testing. However, Graham, Sherbourne and Khabbaz[3] state that axial load has a negligible effect on similar test results.

DESIGN RECOMMENDATION

Based on the results presented here, it is recommended the column web yield strength at the compression region of beam-to-column, moment, end plate connections be estimated from

$$P_{max} = F_{yc}t_{wc}(t_{fb} + 6k + 2t_p + 2t_w) \quad (16)$$

where all terms have been defined previously, provided sufficient lateral bracing exists to prevent out-of-plane buckling of the column flanges. In the format of the current AISC Specification,[1] the required stiffener area is then

$$A_{st} = \frac{P_{bf} - F_{yc}\, t_{wc}(t_{fb} + 6k + 2t_p + 2t_w)}{F_{yst}} \quad (17)$$

whenever the value of A_{st} is positive. No stiffeners are required if A_{st} is negative.

This recommendation is a significant liberalization of the current recommendations for welded, beam-to-column, moment connections, but appears to be justified from previous European studies and results of this current study. Use of this recommendation may result in limited ductility in the column portion of the connection. Hence, ductility must be provided by beam yielding prior to development of the full strength of the column web.

Although not directly a part of the study reported here, the AISC criterion for web stability (Eq. 13 of this paper) is recommended for use at the column compression region at beam-to-column, moment, end-plate connections.

ACKNOWLEDGMENTS

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