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# Steel columns of rolled wide-flange section progress report no. 1, AISC Research Report No. 190, Nov. 1942, Reprint No. 58 (42-2)

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# STEEL COLUMNS OF ROLLED WIDE FLANGE SECTION

## PROGRESS REPORT NO. 1

BY BRUCE JOHNSTON AND LLOYD CHENEY

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
COLUMN RESEARCH AT LEHIGH UNIVERSITY

COMMITTEE ON TECHNICAL RESEARCH
AMERICAN INSTITUTE OF STEEL CONSTRUCTION

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No list

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AMERICAN INSTITUTE OF STEEL CONSTRUCTION

NOVEMBER, 1942

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# FOREWORD

DURING the spring of 1938, the Committee on Technical Research of the American Institute of Steel Construction decided to initiate a program of tests in order to obtain answers to several moot points that existed in regard to the behavior of wide flange column sections with respect to (a) the compressive strength of flanges, and (b) the behavior of wide flange columns under eccentric loading.

A program of tests was accordingly established at the Fritz Engineering Laboratory of Lehigh University and work commenced in September, 1938.

The results of the investigation on the local compressive strength of wide flange columns are presented in the accompanying Progress Report No. 1, by Dr. Bruce Johnston, Associate Director, Fritz Engineering Laboratory, Lehigh University, and Mr. Lloyd Cheney, A. I. S. C. Research Fellow, Lehigh University.

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NEW YORK, N. Y.
NOVEMBER, 1942

# PROGRESS REPORT NO. 1

# STEEL COLUMNS OF ROLLED WIDE FLANGE SECTION

## AMERICAN INSTITUTE OF STEEL CONSTRUCTION COLUMN RESEARCH AT LEHIGH UNIVERSITY

BY BRUCE JOHNSTON* AND LLOYD CHENEY†

This progress report will serve as a general introduction to the program of column tests sponsored by the American Institute of Steel Construction at the Fritz Engineering Laboratory of Lehigh University between September 1938 and June 1942.

This report will also present test results on the local compressive strength of the column flanges. Theoretical analyses, in general, will not be made, as the literature provides extensive references on this subject. Moreover, the elastic buckling phenomena usually treated by mathematical analyses are primarily valid outside the range of usual application of the rolled structural steel wide flange column section.

The list of references appended to this Progress Report No. 1 will also be referred to in later progress reports. This list of references makes no pretense at completeness. Salmon, in his book on columns[11]**, published in 1920, lists 375 references to previous analytical and experimental works on this subject. More recently, in 1940, Moisseiff and Lienhard[18] review the subject of "Elastic Stability applied to Structural Design" and list 52 references, mostly from German sources, and mostly on work published since 1920. The references listed at the end of this report have been selected for their availability and are all in English, but do not necessarily represent the original work on any particular subject. "Theory of Elastic Stability" by S. Timoshenko[12], together with the references just cited, furnish a very adequate bibliography on the subject of columns.

The American Institute of Steel Construction program of tests at Lehigh University may be divided into the following parts:

(1) Local compressive strength tests of flanges of wide flange columns, reported herein.

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† Instructor of Applied Mechanics, Case School of Applied Science, Cleveland, Ohio,—Formerly A. I. S. C. Research Fellow, Lehigh University.

** Numerals refer to references listed at the end of this report.

(2) Tests of eccentrically loaded columns, reported in Progress Report No. 2.

(3) Tests of columns as part of frames, now in progress.

(4) Tests of stiffened plates in compression, now in progress.

Preliminary progress reports have been circulated previously in mimeographed form[1,2]. These reports have been studied by the American Society of Civil Engineers Committee on Design of Structural Members as well as by the Committee on Technical Research of the A.I.S.C., by whose permission the two committees are cooperating on the general subject of column research. All of the Fritz Laboratory Staff have contributed to the program. Professor Hale Sutherland is Director of the Laboratory and Mr. Howard Godfrey was Engineer of Tests when most of the tests were made. Mr. Robert Mains, present Engineer of Tests, and Mr. George Packer, A.I.S.C. Research Fellow, assisted in the preparation of this report.

# GENERAL REVIEW OF COLUMN TESTS AND SUMMARY OF FACTORS AFFECTING COLUMN STRENGTH

This section is in large part abstracted from the First Progress Report of the A.S.C.E. Committee on "Design of Structural Members"[6]*.

Extensive work on column tests was carried on by two A.S.C.E. Committees. The first of these was the Special Committee of the Board of Direction on Steel Columns and Struts, organized in January 1909, and whose work was terminated in 1918. Special reference is made to the "General Programme for Column Tests" in a closing discussion of this committee's final report[7]. This program, with minor modifications, might well serve today as a guide on the question of further column research. As a matter of fact, the work now in progress or recently completed fits into the program very well.

In 1923 another A.S.C.E. committee, the "Special Committee of the Board of Direction on Steel Column Research" was formed. It submitted three reports[8,9,10] and finished its work in 1933. These reports cover a very complete and detailed review of previous column tests together with results of many new tests made for the committee.

Other important sources of information[11,12,13] include references to most of the work done on columns during the past two hundred years.

A summary of the factors that affect the strength of a column will provide the basis for understanding the general problem. These factors may be defined by the way they affect the strength of an "idealized" column. An "idealized" column will be defined as one that is made of a perfectly elastic material, is loaded axially through frictionless pins at each end, is perfectly straight, and does not fail locally. This idealized column will buckle elastically at the "Euler" critical load.

$$P_{cr} = \frac{\pi^2 EI}{l^2} \tag{1}$$

in which: $E$ = Young's Modulus,
$I$ = Moment of Inertia,
$l$ = Length of Column.

The idealized column is usually quite different from the actual column as constructed and used in a structure. In the actual column the strength of the column is different from that given by the Euler formula because of: (1) the non-linear shape of the stress-strain relation, (2) accidental imperfections, (3) the known end eccentricity, (4) the shape of the cross section, (5) the torsional behavior, (6) shearing

* These numbers refer to references listed on pages 37 and 38, at the end of this report.

deformation, (7) local buckling or crippling of a part of the column, (8) method of fabrication, and (9) continuity of action in a frame.

(1) *Non-Linear Shape of Stress-Strain Relation*—Above the proportional limit the relation between stress and strain is no longer defined by Young's elastic modulus. The strength of the column is reduced, and may be determined approximately by using a "reduced modulus", $E_R$, in the "Euler" formula, Eq. (1)[14]. In the case of structural steels the yield point represents the practical upper limit of column strength for short columns which do not buckle elastically.

(2) *Accidental Imperfections* such as curvature, end eccentricity, non-homogeneity, etc., act to reduce the strength.

(3) *Known End Eccentricity*—When the material has an elastic stress-strain relationship the maximum stress may be calculated by the "secant" or "eccentricity" formula. In the case of materials with a well-defined yield point, such as structural steel, the load at which maximum stress reaches the yield point may be divided by an arbitrary factor of safety to indicate a safe design load. When the eccentricity is in the strong plane the possibility of lateral-torsional buckling should be investigated[3].

(4) *Shape of Cross Section*—The shape of the column cross section affects the strength when considered in conjunction with a material having a non-linear stress-strain relation[14].

(5) *Torsional Behavior*—Certain shapes of thin material may buckle by twisting, under either axial or eccentric load[3,15].

(6) *Shearing Deformation*—The theoretical strength of a column is reduced, especially in the case of the built-up column, when shearing deformation is considered[12].

(7) *Local Buckling or Crippling of a Part of the Column*—Many different cases are reviewed by S. Timoshenko[12].

(8) *Method of Fabrication*—A method of fabrication which introduces initial stresses or causes warping of the component parts may reduce the strength of the column.

(9) *Continuity of Action in a Frame*—Compression in a strut reduces its bending stiffness, whereas tension increases the bending stiffness. Buckling of a member in a frame ensues when the summation of bending stiffness becomes equal to zero at any joint of a frame[16].

It is also important to emphasize that much of the knowledge of column behavior has been based on laboratory experiments in which the ends of the column are either milled flat, or simulate a pin end by use of a knife edge or a roller nest. Actual columns usually have framed end connections which are not equivalent to the end conditions in the usual laboratory test. In a laboratory test of an eccentrically loaded column the eccentricity is usually maintained at a constant value up to failure, but the equivalent eccentricity of load in a framed column varies as the load varies. For these and other reasons the difference between laboratory tests and actual column behavior should always be kept in mind.

The multiplicity of factors affecting column strength has led to some confusion of thought in dealing with the problem. It is obviously impracticable to consider all factors at once in a design formula. Investigators frequently have considered only one or two of the factors and have then magnified the factors considered to include arbitrarily all of the others. For example, in the case of non-ferrous alloys and some of the high-strength steels, the non-linear stress-strain relationship[14] may well be the most important factor affecting the strength of an axially loaded column. Imperfections of shape, curvature, and accidental eccentricity may be covered approximately by modifying the assumed stress-strain relationship. In the case of structural steel, accidental eccentricities and curvature may be the more important factors, and the relatively small variation from a linear stress-strain relation up to the yield point may be taken care of by modification of the eccentricity or secant formula. Another investigator[17] has proposed to take account of all factors by assumed initial curvature of the column, which results in formulas for maximum stress similar to the secant formula.

Mention should also be made of the paper by Leon S. Moisseiff and Frederick Lienhard[18], which proposes rules of design for the plate elements in compression members. Another A.I.S.C. research project at Lehigh is devoted to questions raised by this paper. At the U. S. Bureau of Standards, still another A.I.S.C. research project is being conducted to determine the compression strength of plates with various shaped holes.

# COMPRESSIVE STRENGTH TESTS OF COLUMN FLANGES OF WIDE FLANGE SECTIONS

A column is usually made up of component parts which may be considered as plate elements. These plate elements may buckle locally if their thickness is relatively small in comparison with the width between ribs or between component parts of the column which hold the plate elements in line. Structural sections are usually proportioned so that local buckling will not occur in the elastic range, in which case the plate elements will usually buckle "inelastically" or by "plastic buckling" at an average stress somewhere between the proportional limit and the yield point of the material. In very compact sections buckling may occur at stresses above the yield point, but the yield point usually represents the practical upper limit of strength. Theoretical solutions of elastic buckling have been made for various idealized edge or boundary conditions. Many of these are presented in Timoshenko's work on Elastic Stability[12] and recent work in Germany is listed by Moisseiff and Lienhard[18]. The results of these analyses give a value for the average critical stress at which buckling will take place, i.e.

$$\left.\begin{matrix}\sigma_{cr}\\ \text{or}\\ \tau_{cr}\end{matrix}\right\} = k\,\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{w}\right)^2 \qquad (2)$$

$\sigma_{cr}$, $\tau_{cr}$ = critical direct or shear stress respectively
$k$ = constant which depends on proportions of the plate and boundary conditions
$E$ = Young's Modulus
$\nu$ = Poisson's Ratio
$t$ = thickness of plate element
$w$ = width of plate element

An approximation may be made when the critical buckling stress is in the inelastic range by substituting a reduced modulus $E_r$ in place of $E$ in Eq. (2). A conservative estimate of the reduced modulus may be made by basing it on the slope of the tangent to the stress-strain diagram at any particular point. However, the stress beyond the proportional limit may not affect the plate in the same manner in every direction. Allowance is made for this hypothesis by some investigators. Moisseiff and Lienhard[18] propose, for example, values of $E_r$ for silicon structural steel, structural steel, and structural aluminum alloy 27 ST, based on records of column tests. Since all columns as well as plates are actually somewhat crooked, and rarely uniformly stressed at the ends, it follows that the $E_r$ proposed by Moisseiff and Lienhard includes allowances for such factors.

The sharply defined "yield point" in the case of structural steel defines the upper limit of stress at which a plate element will usually wrinkle into waves regardless of its proportions.

In considering the local buckling of the outstanding flange elements of a structural section, two extremes of edge condition are illustrated in Fig. 1(a) and 1(b). When the length, $L$, is very large Moisseiff and Lienhard give the value of "$k$" in Eq. (2) as follows:

1(a) One edge simply-supported, one edge free $k = 0.43$

1(b) One edge built in, one edge free $k = 1.28$

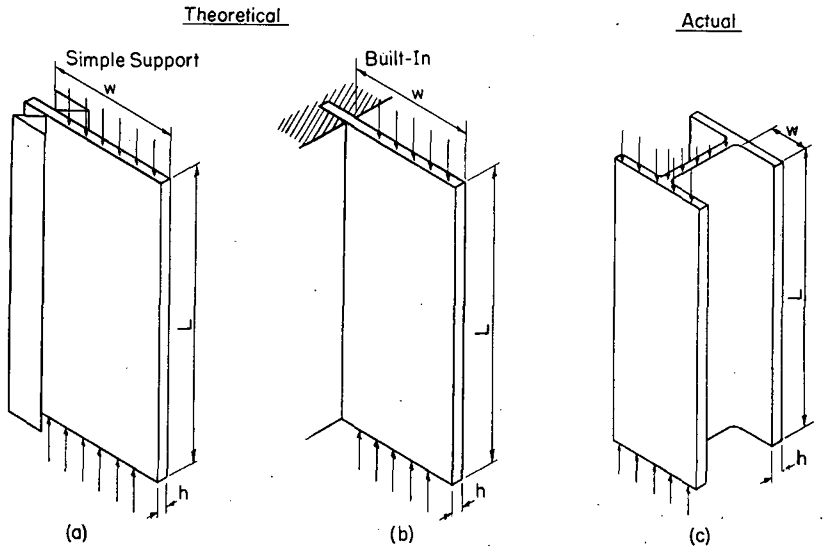


**Fig. 1.—Theoretical limiting edge conditions for outstanding parts of structural sections.**

Fig. 2 shows a plot of the critical elastic buckling stresses for these two limiting cases. The outstanding part of a column flange (see Fig. 1c) will be partially fixed along one edge and the critical stress will be somewhere between the two extremes, as indicated by the center portion in Fig. 2. An equal legged angle, however, satisfies condition 1(a) since both legs will buckle simultaneously. Some of the current specification limitations are also shown on Fig. 2.

The usual structural sections are proportioned so that elastic buckling will not take place and the material will therefore develop the yield-point stress, or nearly so, prior to buckling. In order to verify this fact and study the behavior of sections that are relatively thinner than those now rolled, a series of twenty tests was made in which the flanges of a 10 WF 49 column section were planed to different thicknesses. Ten tests (No. C1 to C10) were made with structural steel (ASTM A7-39) and ten similar tests (No. S1 to S10) with silicon structural steel (ASTM A94-39).

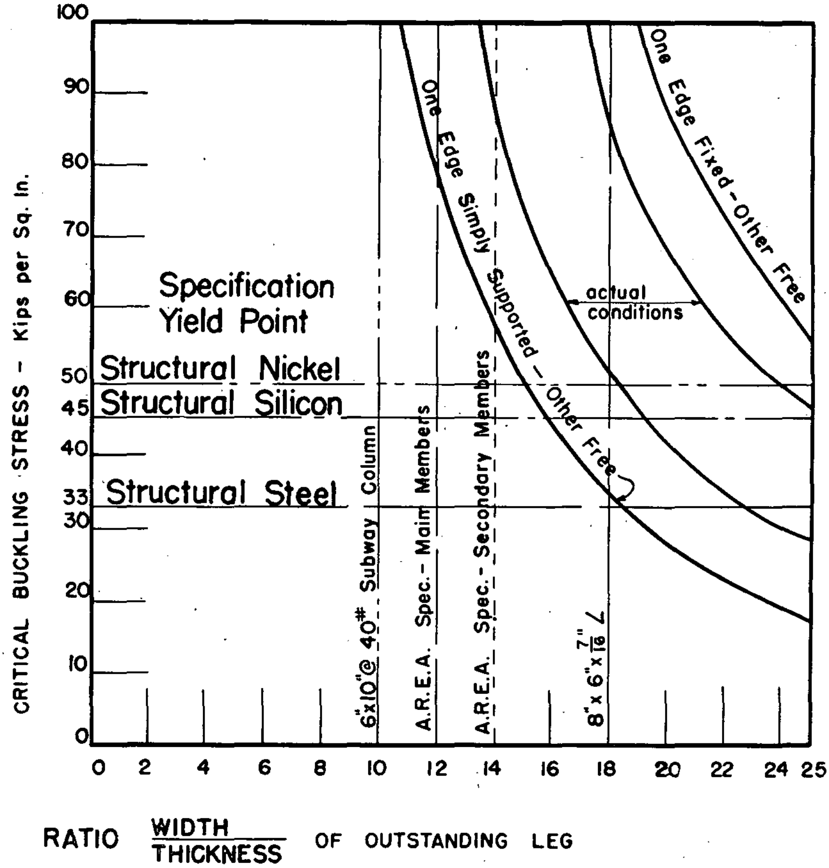


Fig. 2.—Relation between critical buckling stress and width-thickness ratio of the outstanding leg.

In each series of ten tests, two similar groups involving five different flange thicknesses were tested, one group axially loaded (No. 1 to 5) and the other (No. 6 to 10) loaded at the kern point to give zero stress in one flange and maximum in the other. The specimens were milled at each end. Bearing blocks fifteen inches square and five inches thick, having a four-inch length of 10 by 10 in. by 140 lb. WF section welded to them, were used as shown in Fig. 3 to obtain as near ideal stress conditions at the end as possible. A knife edge of heat-treated alloy steel fifteen inches in length was used to apply the load to the bearing blocks. In the preliminary load-centering tests, Huggenberger tensometers were attached to the outside corners of each column at mid-height and trial runs were made until the proper strain distribution was obtained, uniform in the case of the axially loaded columns and zero across one

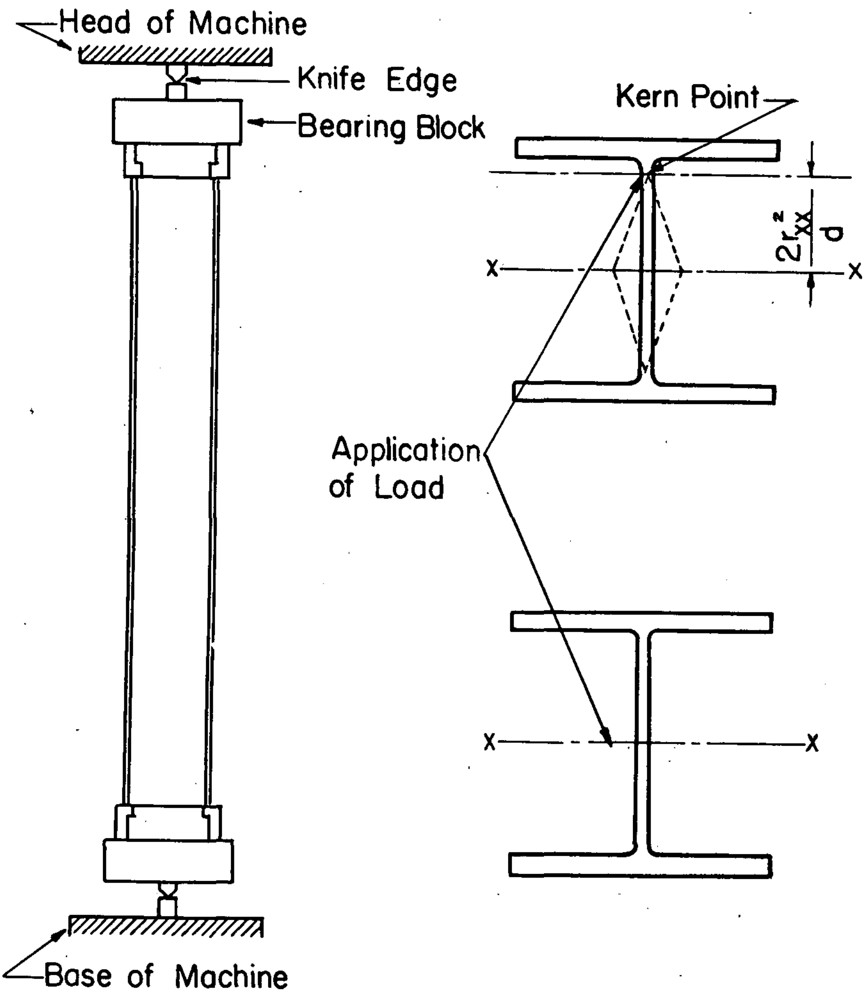


Fig. 3.—Set-up used for flange buckling tests.

flange in the case of the kern point loading. These load-centering runs were made in the elastic range and the desired results were obtained within a tolerance of a few per cent. During the actual tests to failure the tensometers were replaced by four "compressometers" which measured the deformation at each corner over a 46-in. gage length by means of 1/1000-in. dial gages attached to guided steel bars. This is shown in Fig. 4, which is a picture of one of the specimens ready for test.

The physical properties of the materials were checked by means of tensile and compressive tests (1-in. gage length Huggenberger tensometers). The following Table No. 1 presents the average of test results of the flange material.

TABLE No. I

| No. of Tests | Location | Material | Type of Test | Stress in kips per sq. in. | | | Per Cent Elongation in 2 inches |
|---|---|---|---|---|---|---|---|
| | | | | Upper Yield | Lower Yield | Ultimate | |
| 6 | Flange | Structural Steel | Tension | 40.5 | 38.4 | 61.8 | 44.1 |
| 1 | Root of Flange | Structural Steel | Tension | 40.7 | 37.7 | 59.5 | .... |
| 2 | Root of Flange | Structural Steel | Compression | 41.2 | 38.9 | .... | .... |
| 5 | Flange | Silicon Steel | Tension | 46.0 | 45.1 | 77.5 | 42.9 |
| 2 | Root of Flange | Silicon Steel | Tension | 43.4 | 42.4 | 74.3 | 36.8 |
| 2 | Root of Flange | Silicon Steel | Compression | 40.9 | 40.5 | .... | .... |

Typical stress-strain diagrams of both the structural and silicon structural steels are shown in Fig. 5 for both the tension and compression tests. A close similarity is noted between the compression and tension characteristics in the case of structural steel. For structural silicon steel, the yield point is somewhat lower in compression than in tension. Tests made on a large number of samplings of structural, silicon structural, and low-alloy structural steels[4] indicate that both the yield point and shape of the stress-strain curves are usually similar in compression and tension for these steels. The modulus of elasticity, as determined by the tension and compression tests, varied between 29,200 k. s. i. and 30,600 k. s. i. with an average value of 29,800. The average value of $E$ of the column specimens, based on the compressometer readings, was 29,900 k. s. i., with somewhat more scatter of results, probably due to the difference between a 1-in. and a 46-in. gage length.

Two typical test results are shown in Fig. 6, which presents the load plotted against deformation per inch. The individual compressometer

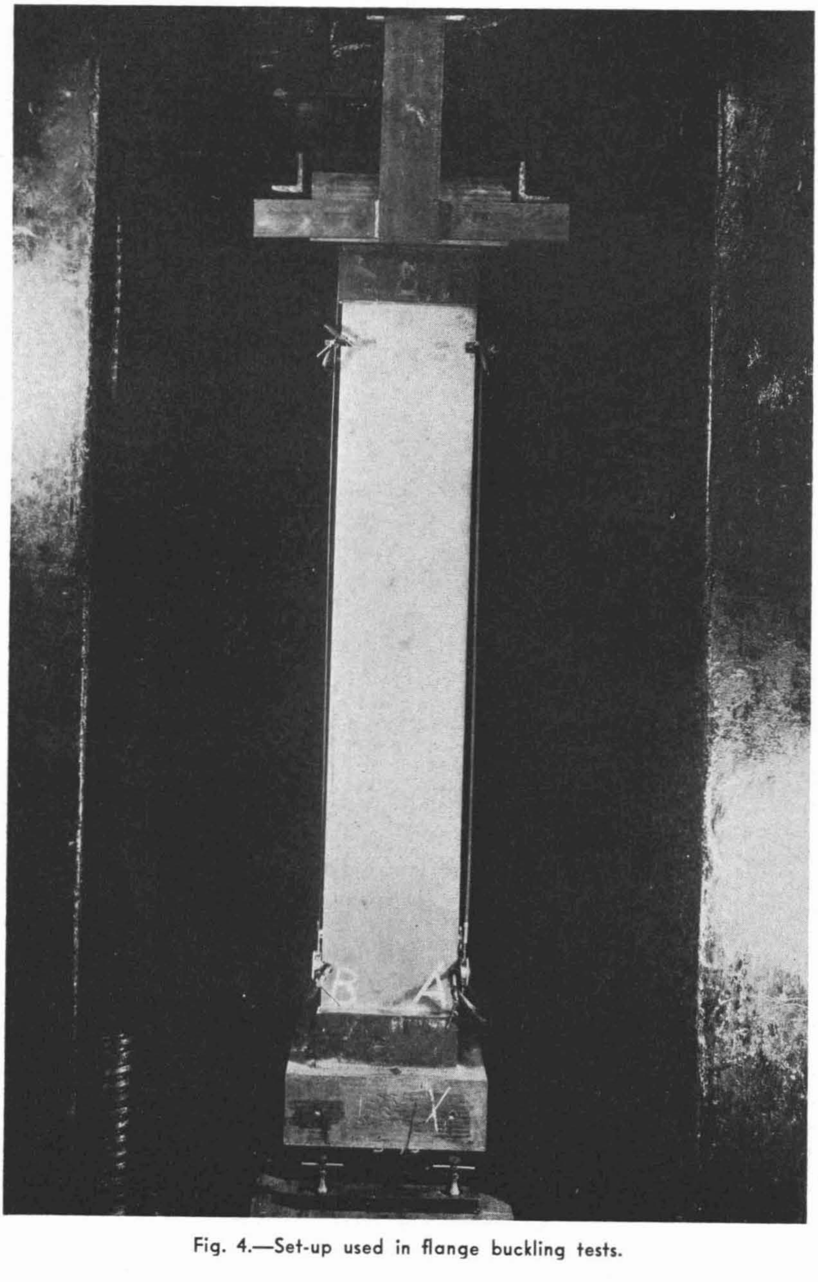

Fig. 4.—Set-up used in flange buckling tests.

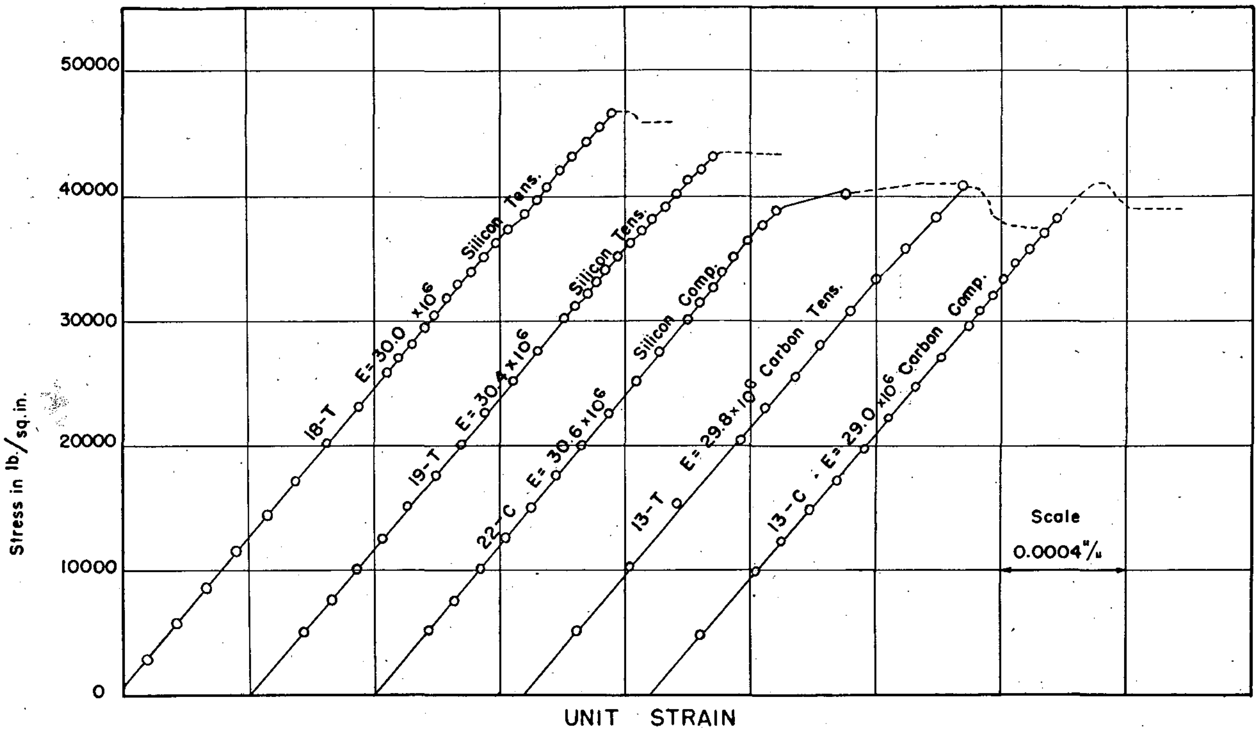


Fig. 5.—Typical stress-strain diagrams for structural carbon and structural silicon steels in tension and compression.

readings on the four corners remained nearly the same up to maximum load in the case of the axially loaded specimens. The average of these four readings is shown in the case of a typical axially loaded specimen in Fig. 6. The average of the pair of compressometers on the loaded flange is also shown in Fig. 6 for a specimen loaded at the kern. The well defined break in the load-deformation curve near maximum load is typical of these test results.

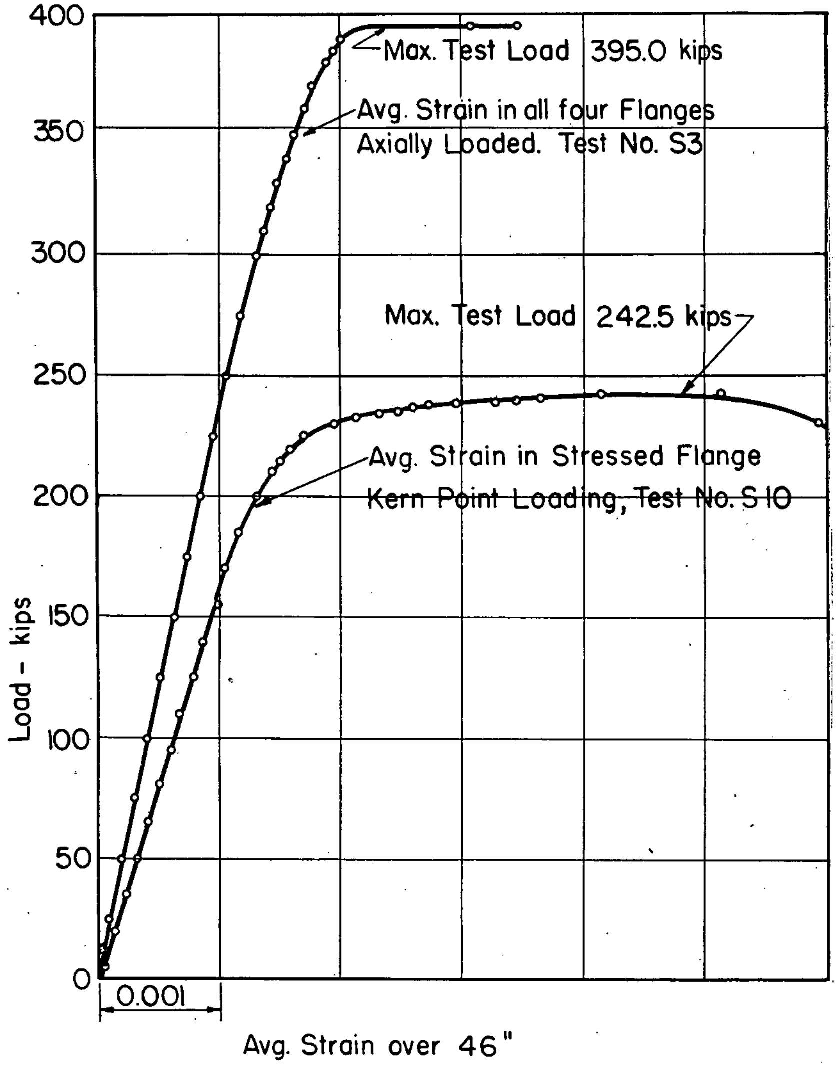


Fig. 6.—Typical test results, flange buckling tests.

The maximum test load was taken as a criterion of failure. Because of the sharp break in the load deformation curves near maximum load, the load at a "general yield" determined by an average strain offset of 0.002 was in all cases within a few per cent of (or identical with) the maximum load. The lack of reserve strength between yielding and ultimate in the case of these relatively short columns would seem to be typical of the unusually thin flanges of the test specimens.

Fig. 7 shows the typical condition of the test specimens after removal from testing machine.

The stress in the buckled flanges at maximum load is shown in Fig. 8 and in the following Table No. II, which also gives other test information. As shown in Fig. 8, the width of the outstanding part of the flange (w) was taken as the distance from the edge of the fillet to the edge of the flange. All dimensions were measured at a number of points and averaged on each specimen. Thicknesses were measured to the nearest 1/1000-in. and overall widths and depths to the nearest 1/100-in. The

## TABLE No. II

| Test No. | Average Thickness "t" of Buckled Flange | Average Width of Buckled Flange | Average "w" Deducting Fillets and Web | Average w/t Buckled Flange | Maximum Load in kips | Measured Area | Approximate Stress in Buckled Flange at Maximum Load k. s. i. |
|---|---|---|---|---|---|---|---|
| C 1 | 0.209 | 9.89 | 4.49 | 21.5 | 282.3 | 7.63 | 37.1 |
| C 2 | 0.244 | 10.00 | 4.55 | 18.6 | 310.0 | 8.43 | 36.8 |
| C 3 | 0.273 | 10.03 | 4.56 | 16.7 | 326.0 | 8.72 | 37.4 |
| C 4 | 0.317 | 9.99 | 4.55 | 14.3 | 354.0 | 9.74 | 36.4 |
| C 5 | 0.345 | 9.89 | 4.49 | 13.0 | 393.0 | 10.27 | 38.2 |
| C 6 | 0.205 | 9.88 | 4.49 | 21.8 | 146.0 | 7.59 | 38.5 |
| C 7 | 0.248 | 10.02 | 4.56 | 18.4 | 160.0 | 8.40 | 38.1 |
| C 8 | 0.272 | 10.02 | 4.56 | 16.8 | 175.0 | 8.69 | 40.3 |
| C 9 | 0.306 | 10.02 | 4.56 | 14.9 | 199.3 | 9.51 | 41.9 |
| C10 | 0.345 | 10.02 | 4.56 | 13.2 | 216.5 | 10.21 | 42.3 |
| S 1 | 0.202 | 10.03 | 4.41 | 21.8 | 284.0 | 7.46 | 38.0 |
| S 2 | 0.243 | 10.09 | 4.45 | 18.3 | 347.0 | 8.33 | 41.7 |
| S 3 | 0.283 | 10.02 | 4.41 | 15.6 | 395.0 | 9.04 | 43.7 |
| S 4 | 0.318 | 10.03 | 4.41 | 13.9 | 420.0 | 9.52 | 44.1 |
| S 5 | 0.348 | 10.04 | 4.42 | 12.7 | 455.0 | 10.47 | 43.5 |
| S 6 | 0.209 | 10.07 | 4.43 | 21.2 | 148.5 | 7.56 | 39.2 |
| S 7 | 0.237 | 10.03 | 4.41 | 18.6 | 156.7 | 8.34 | 37.6 |
| S 8 | 0.278 | 10.10 | 4.45 | 16.0 | 205.0 | 8.93 | 45.9 |
| S 9 | 0.315 | 10.08 | 4.44 | 14.1 | 230.0 | 9.51 | 48.4 |
| S10 | 0.353 | 10.07 | 4.44 | 12.6 | 242.5 | 10.44 | 46.4 |

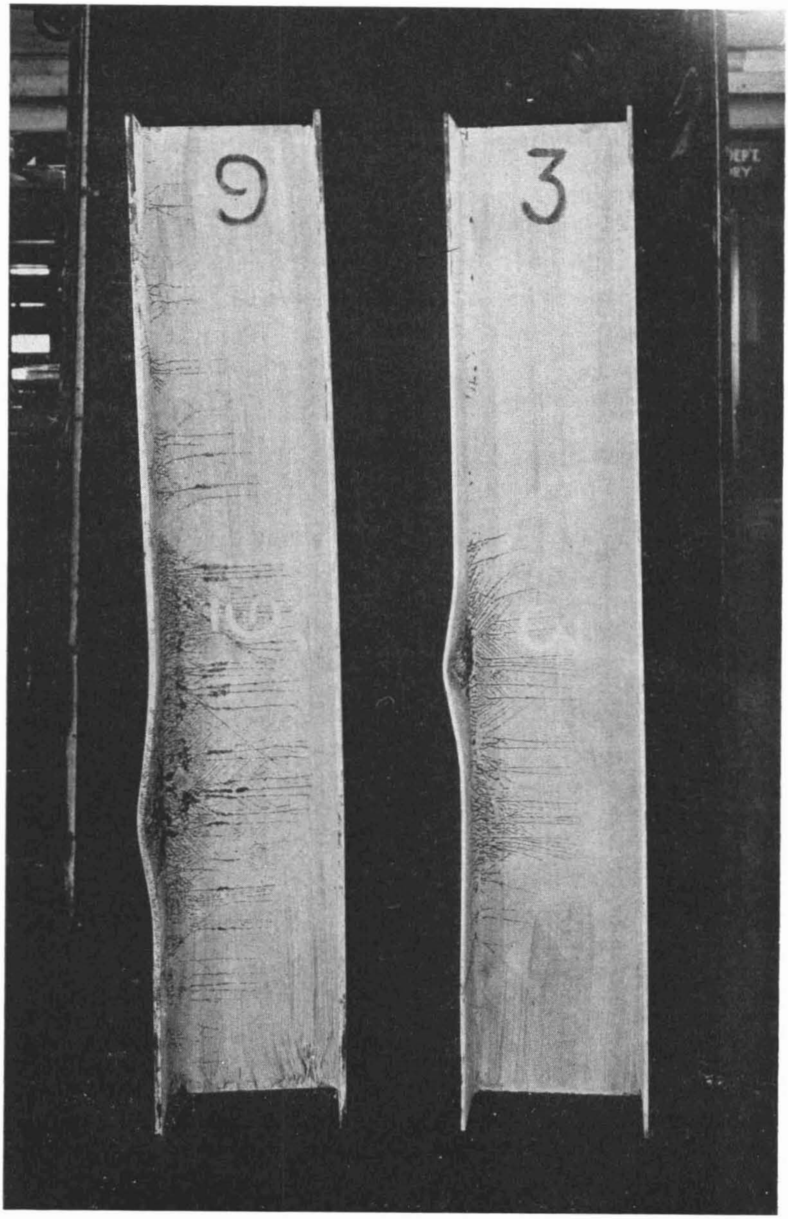


Fig. 7.—Typical condition of test specimens after test (kern point loading, test S-10 and S-7).

stress in the buckled flange was taken as $P/A$ in the case of the axially loaded specimens and $2P/A$ in the case of those loaded at the kern. In the latter case the stress would be strictly correct only in the elastic range, but gives an indication of the stress at failure in the same terms that would be used in calculating the working load by the designer.

Fig. 8 also shows the critical stresses used as a design basis by Moisseiff and Lienhard[18]. These are obtained by multiplying the allowable values by the factor of safety of 2.00 which they propose. All of the test values are above the Moisseiff and Lienhard curves, increasingly so as the ratio of $w/t$ increases. This is due to the partial restraint offered by the unbuckled web along one edge of the outstanding flange leg.

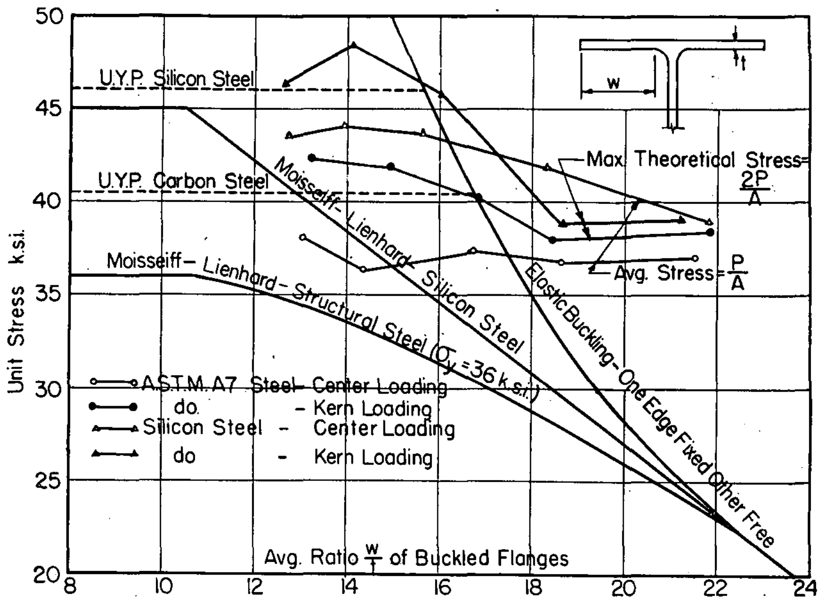


**Fig. 8.—Stress in buckled flange at maximum load.**

It should be noted that in the axially loaded specimens the average critical stresses in all cases are somewhat below the upper yield point noted in the coupon tests. In the case of the silicon structural steel the material had only a slightly higher upper yield point than the specification minimum of 45 k. s. i., which is also the value assumed by Moisseiff and Lienhard. The carbon structural steel in the test specimens had an average upper yield point of 41.3 k. s. i. whereas Moisseiff and Lienhard assume 36 k. s. i. and the A.S.T.M. specification minimum is 33 k. s. i. Structural steels tested at the Fritz Laboratory have occasionally had upper yield points below 33 k. s. i. and lower yield points in the neighborhood of 30 k. s. i. Outstanding parts of structural steel sections made of such steels may be expected to buckle plastically at stresses somewhat below the Moisseiff-Lienhard curve in Fig. 8.

A record of the final buckled or bent shape along the outer edge of each flange in each test is shown in Fig. 9, 10, 11, and 12. The offsets noted in these figures represent the buckled shape after considerable plastic deformation and after removal from the testing machine. The shape of these waves, with sharp peaks, is typical of plastic buckling, in contrast to the less peaked wave frequently encountered in elastic buckling.

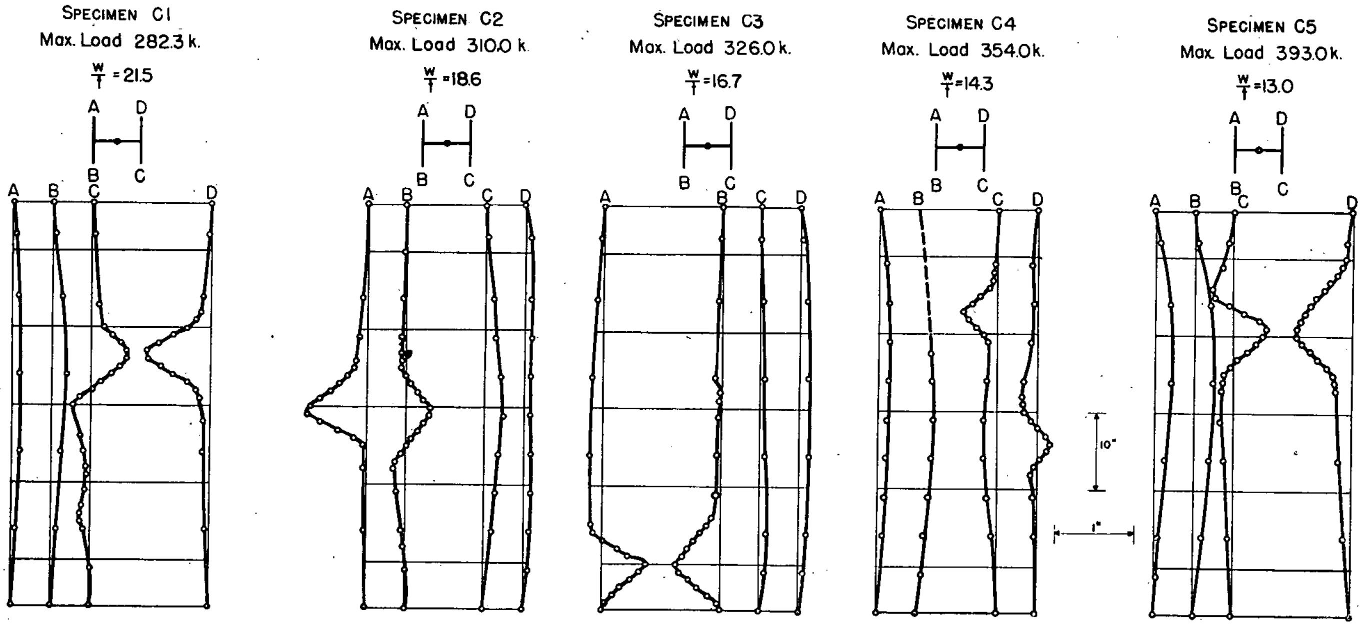


Fig. 9.—Carbon steel specimens loaded at centroid. Profiles of flanges of columns after failure.

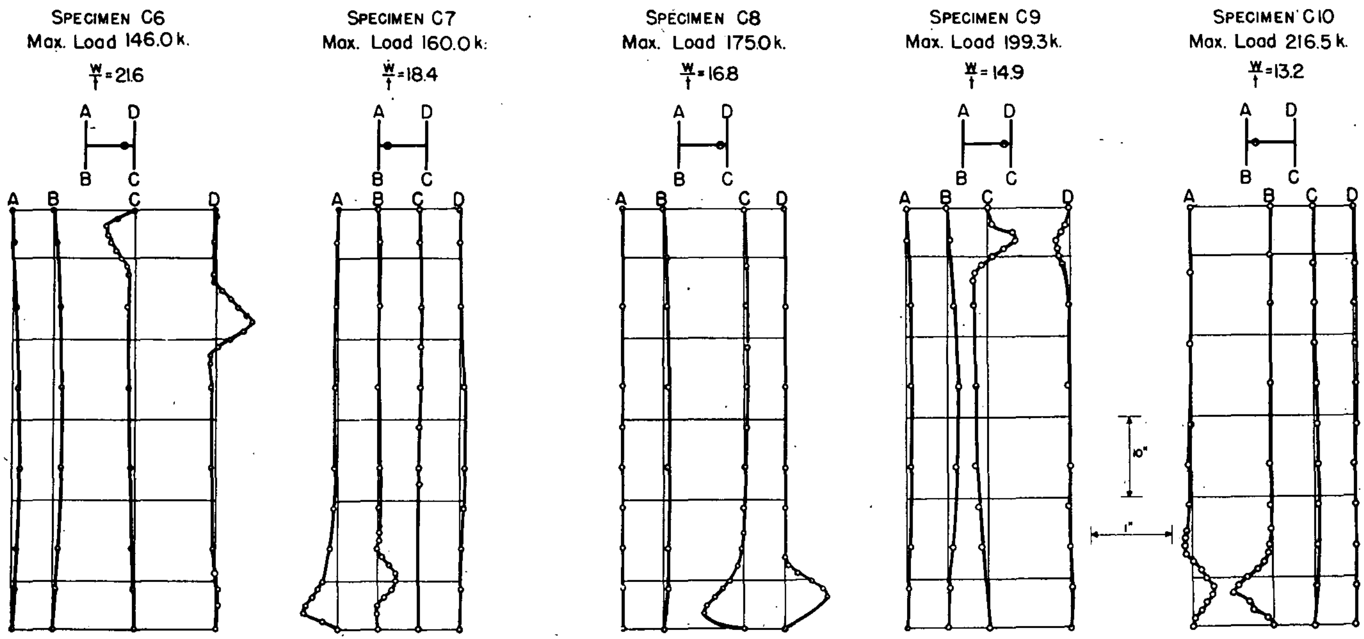


Fig. 10.—Carbon steel specimens loaded at kern point. Profiles of flanges of columns after failure.

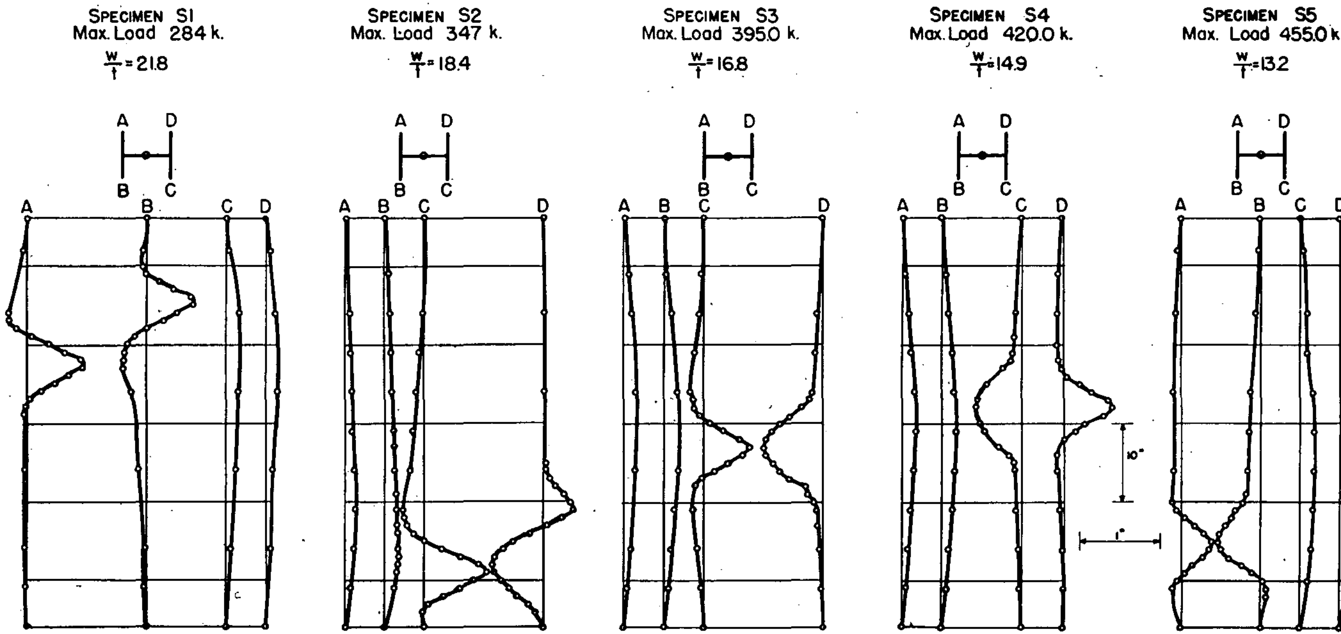


Fig. 11.—Silicon steel specimens loaded at centroid. Profiles of flanges of columns after failure.

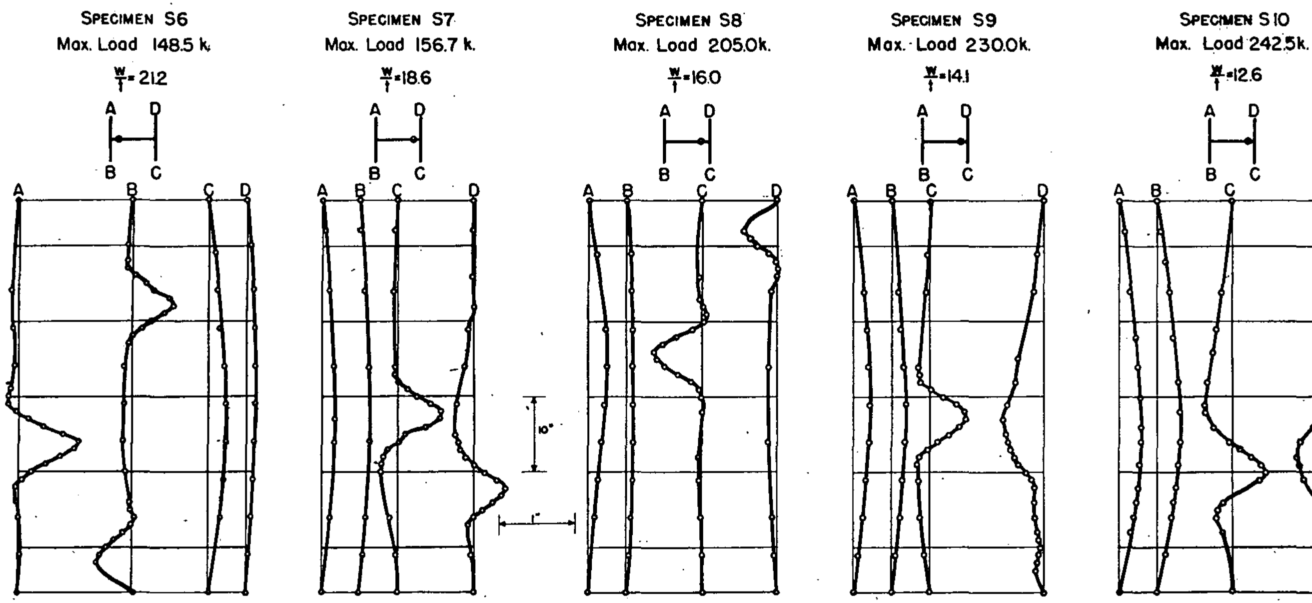


Fig. 12.—Silicon steel specimens loaded at kern point. Profiles of flanges of column after failure.

## EFFECT OF BEAM CONNECTIONS ON LOCAL COMPRESSIVE STRENGTH OF COLUMN FLANGES

The purpose of this part of the program was to compare the effect of various types of building connections on the local buckling of column flanges. Three specimens as shown in Fig. 13, 14, and 15 were ordered from a structural steel fabricator. A study of the effect of welded top angle connections on the bending of column flanges had been made by Lyse and Mount[19]. On the basis of one of the most critical cases indicated in this report, the riveted connection and welded tie plate connection were designed in such a manner as to apply essentially the same load, per inch of connection, to the column flange. By so designing the specimens it was thought that they were put on an equal basis since in each case the same line load was applied to the column.

The same bearing block and knife edge as previously described were used to apply load to the column but the lower bearing block was used without a knife edge to insure stability of the set-up. The specimens were set up in the testing machine and a trial column load applied.

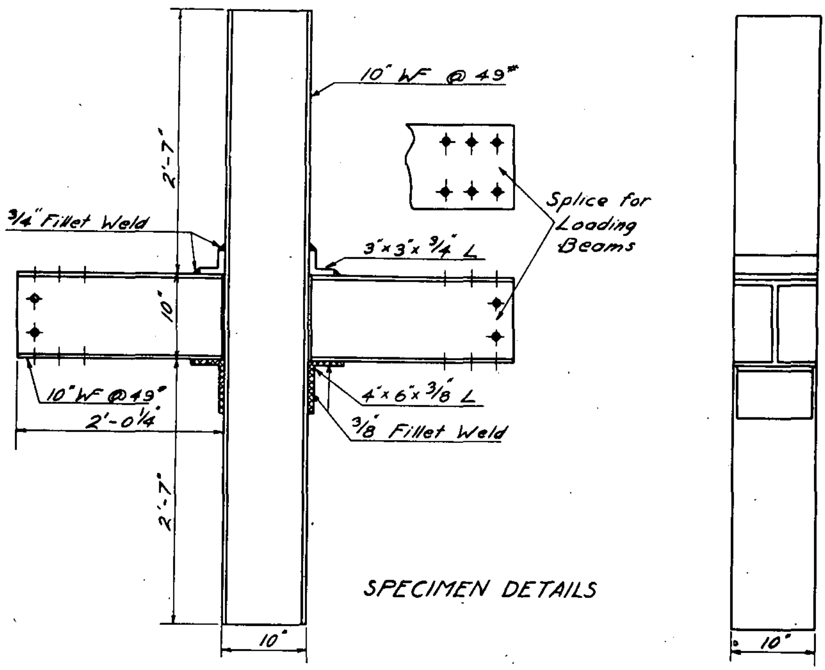


Fig. 13.—Welded top angle and seat connection.

Huggenberger tensometers were attached to the flanges midway between the top of the specimen and the beam connection. This position was selected in order to eliminate the interference of local stress concentrations near the ends of the columns, near rivet holes, adjacent to welds, etc. Increments of load within the elastic range were applied and the strains on all four flanges noted. Adjustments of the knife edge with shims were made until the strains in all four flanges were nearly equal for the load increments, as in the case of Part I. When the column was thus properly centered the cantilever arms were attached by means of the splices. The test set-up is shown diagrammatically in Fig. 16 and photographed in Fig. 17. The dead weight was applied to the loading beams in increments up to the design load of the connection. Displacements of the column flange, deflections of the loading beams, and rotations of the loading beams at the connections were measured as the bending load was applied. An axial load of fifty thousand pounds had previously been applied to the column to insure against any movement of the whole set-up due to the process of applying the bending load.

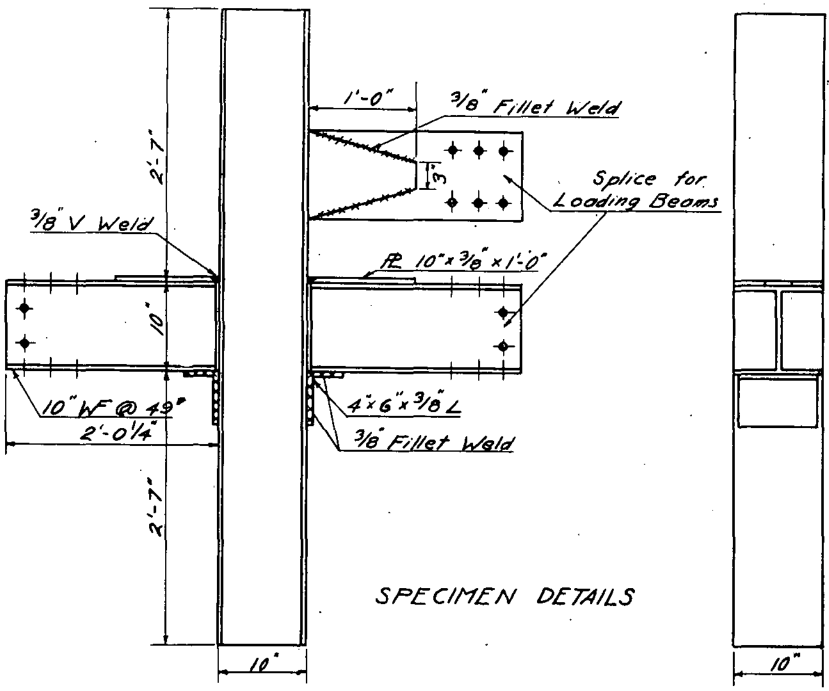


Fig. 14.—Welded top plate and seat connection.

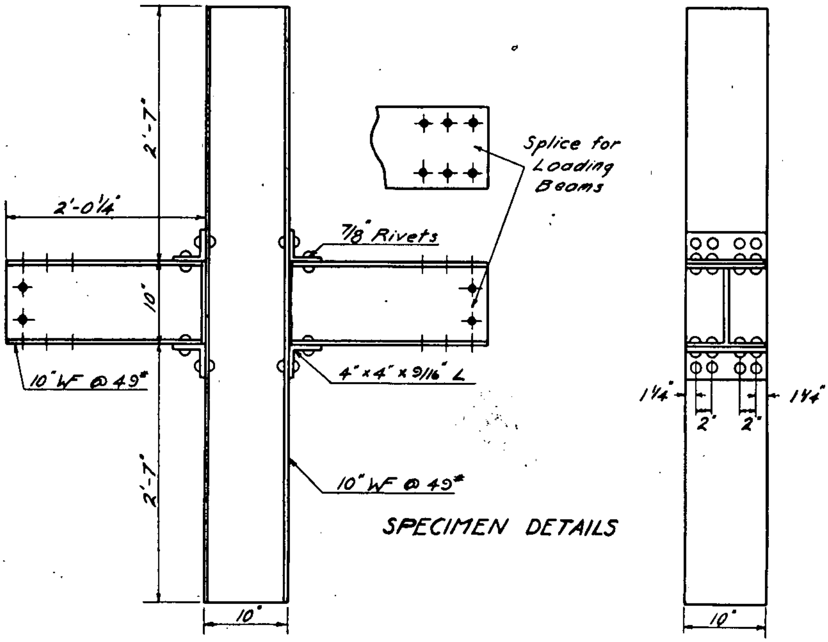


Fig. 15.—Riveted top angle and seat connection.

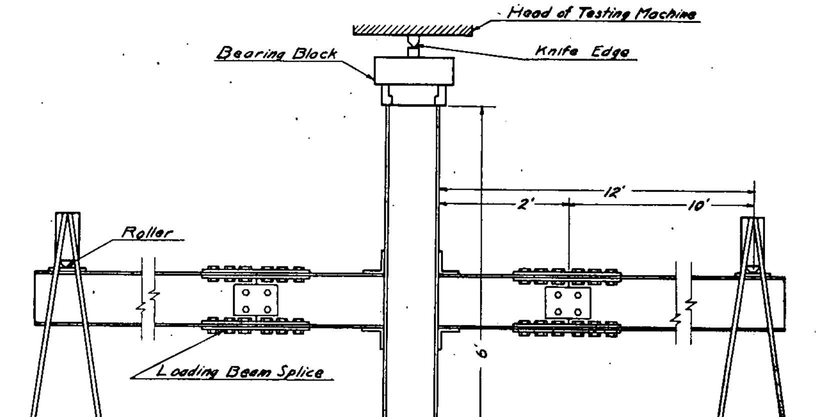


Fig. 16.—Testing arrangement.

With the full load on the cantilever arms, increments of axial load were applied to the column. Strains in the flanges were measured with a Whittemore strain gage having a twenty-inch gage length. Gage points were selected so that the gage length covered the portion of the flange most affected by the connection. Axial load was applied until failure of the columns occurred. Column strains, beam deflections, and column flange displacements were measured at each load increment. Throughout the test to destruction the full design load of the connection remained on the cantilever loading beam.

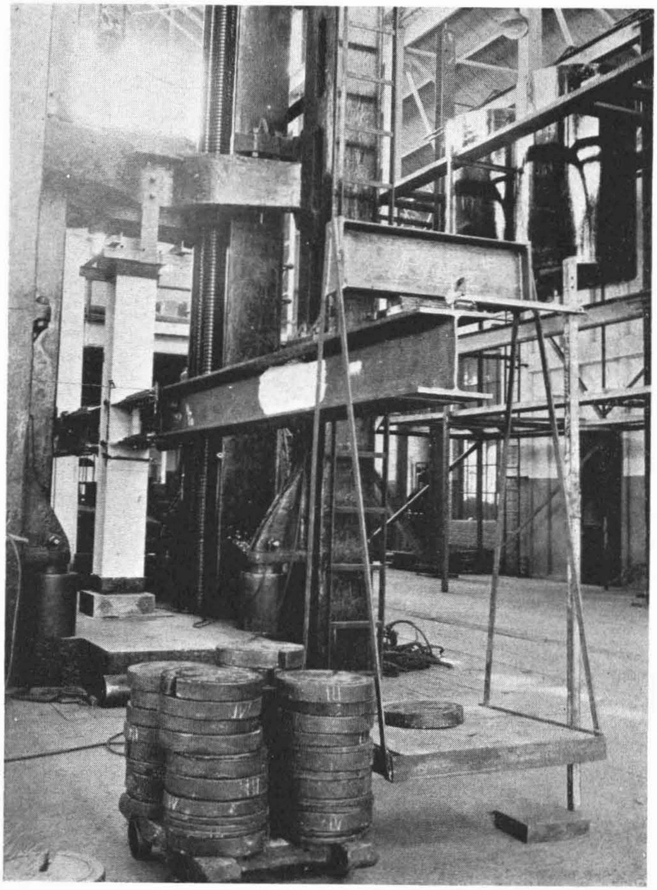

Fig. 17.—Test set-up.

The average of tensile test results of material in the column flange and web, weighted in proportion to respective areas, were:

| | |
|---|---|
| Yield point........................ | 37.6 k. s. i. |
| Ultimate........................ | 61.7 k. s. i. |
| Per Cent Elongation in 2 in........... | 44.4 |

A summary of the test results is given in the following Table No. III

**TABLE No. III**

| Type of Connection | Average Stress in k. s. i. in Column at Maximum Load | Efficiency Based on Yield Point |
|---|---|---|
| Riveted Angle Connection.................. | 33.6 | 0.89 |
| Welded Angle Connection.................. | 34.6 | 0.92 |
| Welded Plate Connection.................. | 35.3 | 0.94 |

Table III indicates that the welded top plate connection was least harmful in lowering the maximum capacity of the column whereas the riveted angle was the most harmful. The welded top plate stiffens the flange and inhibits bending of the outstanding parts. On the other hand, the welded top plate in a different design might introduce local concentration of stress into the column web, but this did not appear to be harmful in the present instance at design loads.

The initial and general yielding of these column connection assemblages was very gradual and it is difficult to assign any definite "limit of structural usefulness". Fig. 18 shows average displacements of the column flanges at an average compressive stress of 17 and 28 k. s. i., these being approximately in the same proportion as 20 and 33 k. s. i., the tensile allowable and tensile yield specification stresses, respectively. The displacements were measured between plates bolted along the center of the web and a line about one inch in from the outside edge of the flange. Each point on Fig. 18 represents the average of two readings measured at symmetrically opposite points on the two outstanding legs of one of the column flanges. The measurements were made on only one of the two flanges and it so happened that in Fig. 18a the least bent flange was measured whereas in Fig. 18b the most bent flange was measured. Fig. 19 shows the relation between average stress and average longitudinal strain measured across the connection by means of the 20-in. Whittemore gages. This measurement was not taken on the riveted connection specimen. Fig. 20 shows the relation between the

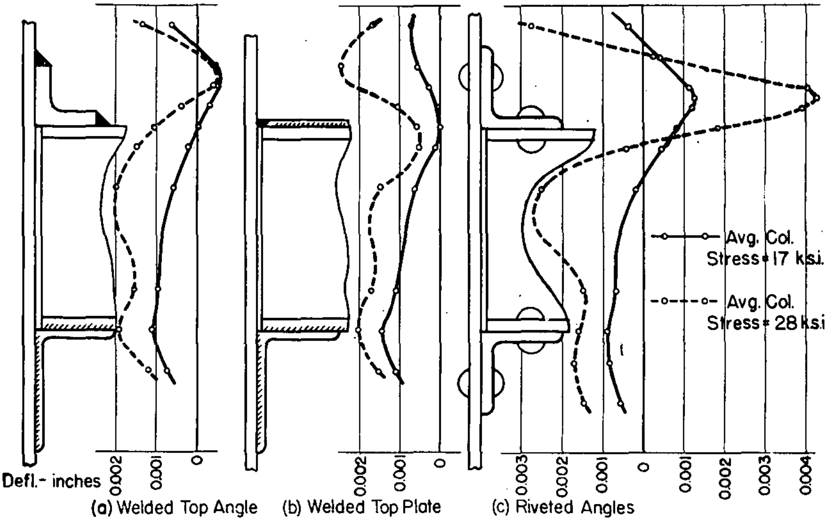


Fig. 18.—Average displacement along one face of column one inch in from outer corner.

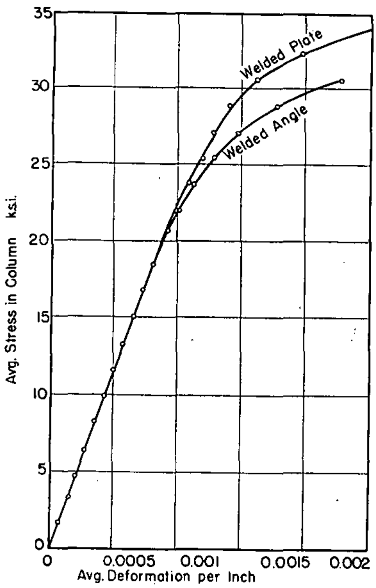


Fig. 19.—Average longitudinal deformation across connection.

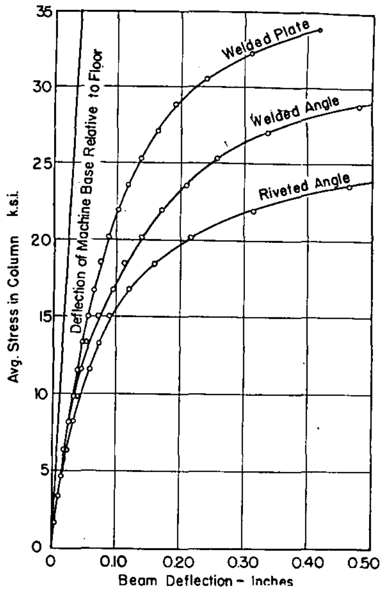


Fig. 20.—Average deflection at ends of beam.

average stress in the columns and the average deflection at the two ends of the cantilever beams. These deflections were in reference to the laboratory floor, hence are not quantitatively correct in reference to the column, but indicate the difference in behavior of the three types. The dashed line shows the deflection of the testing machine base relative to the laboratory floor. Fig. 20 shows that column stress caused increased connection rotation even at low loads and the increase of rotation became greater as column stress increased. The bending moment in the connections was constant, hence the change of deflection in Fig. 20 was a function of column stress only. Fig. 20 indicates rapidly increasing connection rotation at about the following average stresses:

20 k. s. i. in the case of the riveted connection,

25 k. s. i. in the case of the welded top angle connection,

30 k. s. i. in the case of the welded top plate connection.

Fig. 21, 22, and 23, illustrate the condition of the test specimens after removal from the testing machine. It will be noted that the failure is very similar in the cases of the welded angle and riveted angle connections. In the case of the riveted angle, the flange buckled the greatest amount slightly below the rivets, (Fig. 21a) while the welded angle caused the flange to buckle the greatest just below the top weld (Fig. 22a). In Fig. 23(a) and (b) it may be seen that the buckled wave occurs above the top of the beam, the tie plate connection apparently having little or no effect upon the failure of the member as a short column.

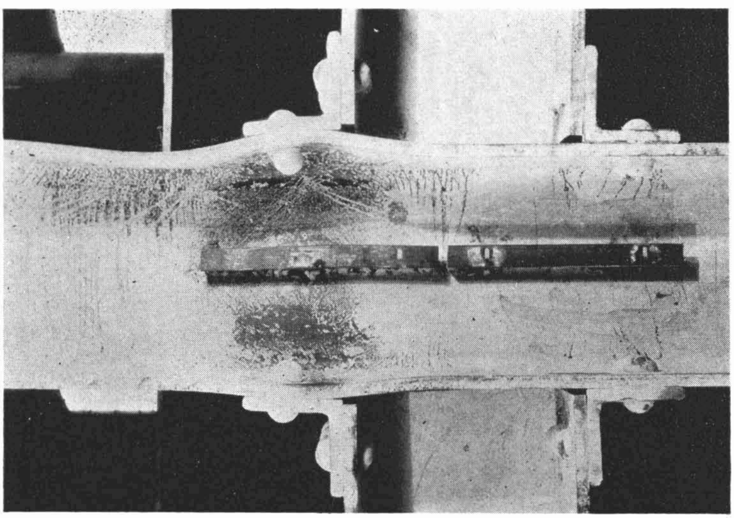

Fig. 21(a)

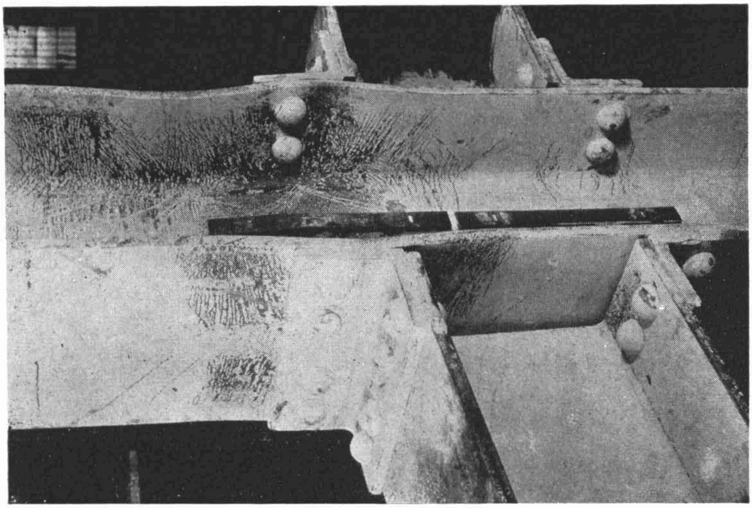

Fig. 21(b)

Riveted top angle and seat angle connection after column test.

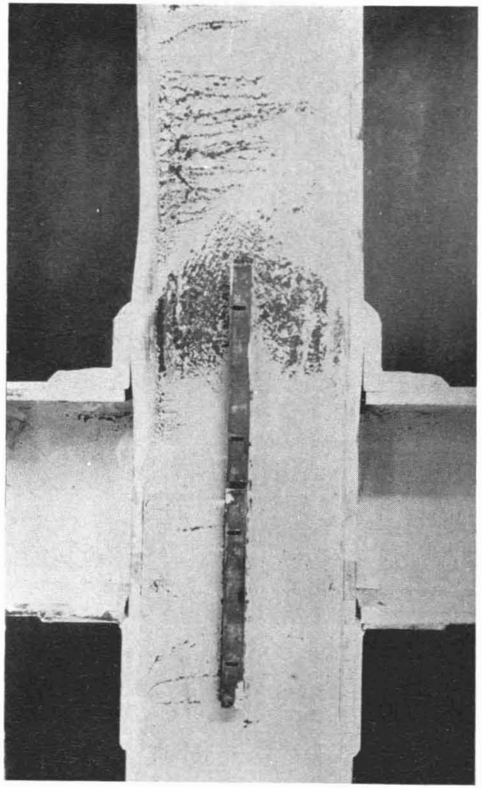

Fig. 22(a)

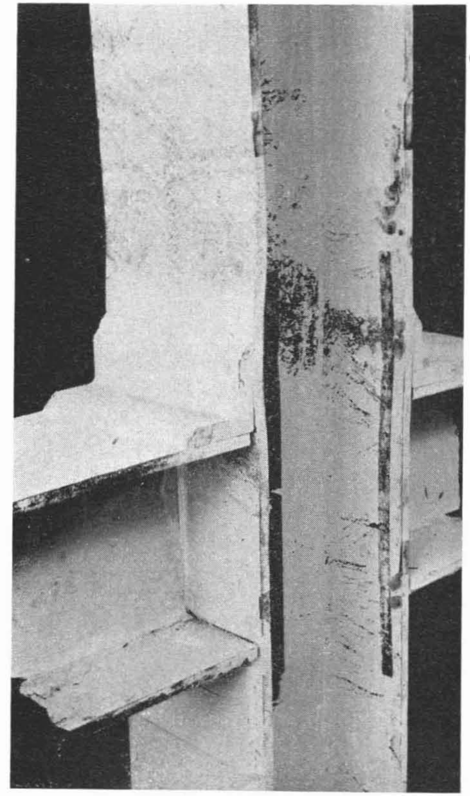

Fig. 22(b)

Welded top angle and seat angle connection after column test.

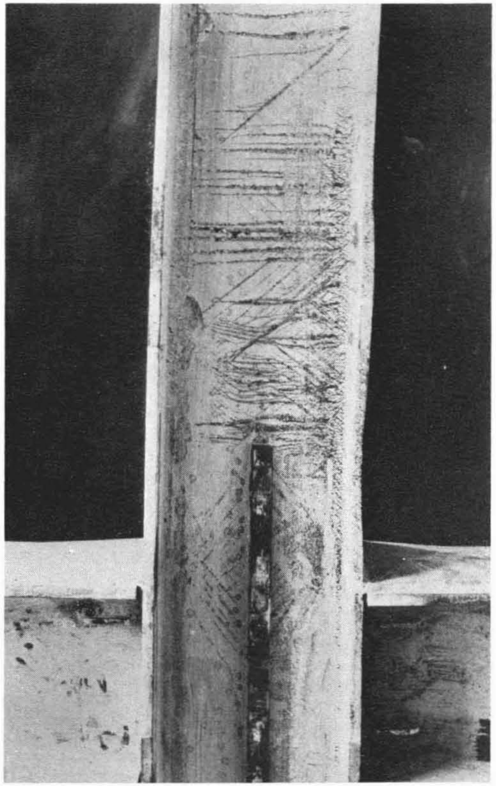

Fig. 23(a)

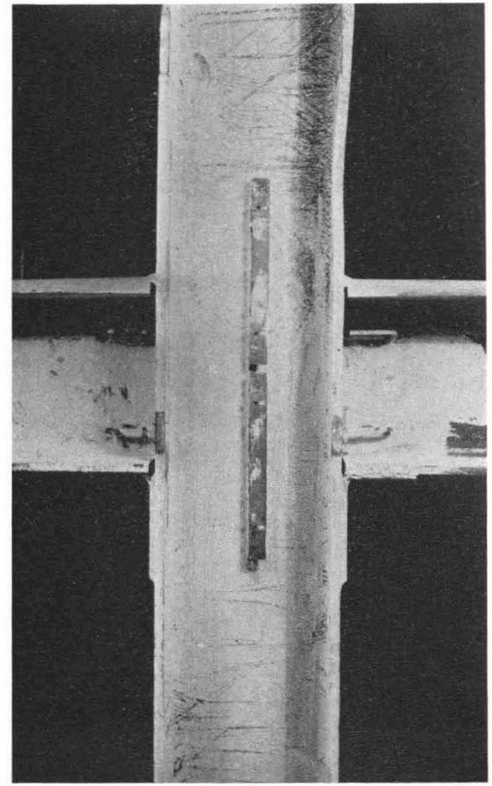

Fig. 23(b)

Welded top plate and seat angle connection after column test.

## SUMMARY AND CONCLUSIONS

(1) This report presents local flange buckling test results of twenty tests of 10 WF sections with flanges planed to varying thicknesses. Both carbon structural and silicon structural steels were tested.

(2) Both the carbon steel and silicon steel specimens, when loaded at the kern point, with knife edge parallel with flange, developed strengths corresponding to a maximum computed flange stress equivalent to the upper yield point, for $w/t$ ratios of 16 or less.

(3) Axially loaded specimens developed flange stresses between 90 and 95 per cent of the upper yield point, in both silicon structural and carbon structural steel, for $w/t$ ratios of 18 or less.

(4) Results of three tests are presented in which short columns are compressed while local moments are applied to the columns by beam connections.

(5) For the particular proportions of columns and connections tested, the welded top plate and seat connection had the least harmful effect and the riveted top and seat angle connection had the most harmful effect on the load carrying capacity of the column. The welded top and seat angle connection had an effect intermediate between the other two.

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