Welded built-up columns, April 1966

L. Tall

Follow this and additional works at: https://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports

Recommended Citation
Tall, L., "Welded built-up columns, April 1966" (1966). Fritz Laboratory Reports. 65.
https://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports/65

This Technical Report is brought to you for free and open access by the Civil and Environmental Engineering at Lehigh Preserve. It has been accepted for inclusion in Fritz Laboratory Reports by an authorized administrator of Lehigh Preserve. For more information, please contact preserve@lehigh.edu.
Welded Built-Up Columns

WELDED BUILT-UP COLUMNS

by
Lambert Tall

Fritz Engineering Laboratory Report No. 249.29
Welded Built-Up Columns

WELDED BUILT-UP COLUMNS

by

Lambert Tall

Fritz Engineering Laboratory
Department of Civil Engineering
Lehigh University
Bethlehem, Pennsylvania

April 1966

Fritz Laboratory Report No. 249.29
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>1</td>
</tr>
<tr>
<td>1. INTRODUCTION</td>
<td>4</td>
</tr>
<tr>
<td>2. RESIDUAL STRESSES AND MECHANICAL PROPERTIES</td>
<td>11</td>
</tr>
<tr>
<td>2.1 FORMATION</td>
<td>15</td>
</tr>
<tr>
<td>2.2 WELDED PLATES AND WELDED SHAPES</td>
<td>21</td>
</tr>
<tr>
<td>2.3 THEORETICAL STUDIES</td>
<td>25</td>
</tr>
<tr>
<td>2.4 MECHANICAL PROPERTIES</td>
<td>27</td>
</tr>
<tr>
<td>3. COMPRESSION MEMBERS</td>
<td>30</td>
</tr>
<tr>
<td>3.1 RESIDUAL STRESSES AND COLUMN BUCKLING</td>
<td>32</td>
</tr>
<tr>
<td>3.2 WELDED COLUMNS</td>
<td>34</td>
</tr>
<tr>
<td>3.3 FACTORS IN COLUMN STRENGTH</td>
<td>36</td>
</tr>
<tr>
<td>3.4 REINFORCED COLUMNS AND HYBRID COLUMNS</td>
<td>42</td>
</tr>
<tr>
<td>3.5 HEAVY COLUMNS AND FLAME-CUT PLATES</td>
<td>43</td>
</tr>
<tr>
<td>3.6 WIDTH-THICKNESS RATIOS AND PLATE BUCKLING</td>
<td>45</td>
</tr>
<tr>
<td>3.7 DESIGN IMPLICATIONS</td>
<td>88</td>
</tr>
<tr>
<td>4. SUMMARY</td>
<td></td>
</tr>
<tr>
<td>5. ACKNOWLEDGEMENTS</td>
<td></td>
</tr>
<tr>
<td>6. NOMENCLATURE</td>
<td></td>
</tr>
<tr>
<td>7. TABLES AND FIGURES</td>
<td></td>
</tr>
<tr>
<td>8. REFERENCES</td>
<td></td>
</tr>
</tbody>
</table>
1. INTRODUCTION

A research project on the "Strength of Welded Built-Up Columns" has been in progress at Lehigh University under the guidance of Task Group 1 of the Column Research Council. This Task Group originally was assigned the task of determining the relationship between material properties and the strength of columns. The early studies were concerned with rolled columns, leading to a number of reports of which References 1 and 2 were the preliminary report and the summary report respectively.

The attention of the Task Group was directed towards welded columns in 1959, and this report represents a summary of the findings from that time through 1965, as well as a discussion of their significance. These welded columns were fabricated from UM plates and were from 6" x 6" to 10" x 10" in size with plates from 3/8" to 3/4" thickness. More recently, pilot investigations have commenced on welded shapes which are fabricated from flame-cut plates, or from thick plates. This recent work and its preliminary conclusions are described briefly.

Perhaps the most important findings of the previous studies were concerned with the significance of the influence of residual stresses on column strength. Based upon these ideas, and using the tangent modulus concept for the buckling of inelastic columns, a
column strength curve was prepared by the Column Research Council for rolled columns of ASTM A7 steel, and this column curve was adopted as the design curve for all columns by the American Institute of Steel Construction in 1961, with the inclusion of a suitable factor of safety.

Figure 1 shows diagrammatically how the column curve depends on the residual stress distribution. The stress-strain relationship obtained from the stub column test* reflects the presence of residual stresses. This is evident from Fig. 1a where, for any fiber, when the sum of the applied stress and the compressive residual stress acting on that fiber becomes equal to the yield stress, yielding will commence in that fiber. The beginning of yielding implies that the stress-strain relationship for the complete cross section is no longer linear, or elastic, as would be the case for a coupon, since a coupon contains no residual stresses.

The column curve in Fig. 1d results from the use of the stress-strain relationship (Fig. 1b) and the tangent modulus concept for buckling. The tangent modulus curve (Fig. 1c) may be used in the computation of column strength.

Reference is made throughout to the various progress reports that contain the detailed experimental and theoretical work.

* The stub column test is an important control test in experimental investigation of columns, and is described further in Ref. 3.
Although simple, pinned-end columns do not occur in practice, they must be regarded as the basic column, since all column specifications (including those of beam-columns) are in terms of such a column. Thus, the scope of this study is limited to centrally loaded columns, generally of ASTM A7 or A36 steel, but with some information for other grades also.
2. RESIDUAL STRESSES AND MECHANICAL PROPERTIES

Although residual stresses have been studied for many years, it is only in the past decade that it was realized that they are a major influence in the strength of compression members, both columns and plates. This had lead to a rather complete study of their formation, magnitude and distribution, and their effect on engineering structures. (1,2,4)

Residual stresses exist in rolled, welded, and cold-straightened structural shapes. Their removal by annealing is costly and sometimes impossible, but a control over their influence is possible to achieve.

The study of residual stresses and the strength of columns has necessitated a study of the mechanical properties of the material used. Some of the findings are reported here.

2.1 FORMATION

Residual stresses are formed in a structural member as a result of uneven relaxation of plastic deformations; they are stresses which exist in the cross section even before the application of an external load. This relaxation may be due to cooling after hot-rolling or welding, or due to fabrication operations such as cold-bending or cambering. Because of the localized heat input of welding, the relaxation of the plastic deformations in welded shapes always occurs during the process
of cooling from the welding temperature to air temperature; the plastic deformations result from the fact that some parts of the shape cool much more rapidly than others, causing inelastic deformations in the slower cooling portions.

2.2 WELDED PLATES AND WELDED SHAPES

The scope of this section, and of this report, is limited to "thin" plates no greater than 1" in thickness, and to "small" size shapes. Some preliminary data from a pilot study is presented for "thick" plates and for "heavy" shapes. Both the manual shielded-arc and the automatic submerged-arc welding processes were considered for plates and shapes.

The two shapes which may be regarded as basic are the H-shape and the box-shape, Fig. 2. These shapes are made up of welded plates which are either center-welded or edge-welded. It has been shown\(^{(5)}\) that the residual stress distribution in a welded shape is similar to the separate residual stress distributions of the component welded plates. The statement is true for shapes made up of thin plates similar in size. This means that the residual stress distribution can be estimated to within 10% for most welded shapes without recourse to actual measurements.\(^{(5)}\)

Plates

In both plates and shapes, the residual stresses of prime interest for structural purposes are the longitudinal residual stresses.
The "method of sectioning"\(^{(1)}\) was used to measure the stresses; the plates used were sufficiently long so that a uniform state of stress would exist in the central portion where the longitudinal residual stresses were measured.

The residual stress distributions typical in welded plates up to 10" wide and 1" thick are shown in Figs. 3 and 4, and in a welded plate 16" wide in Fig. 5. From these figures, it is seen that the distribution of residual stress in welded plates is smooth and approximately parabolic in shape, except at the weld.

At the weld, the residual stress is tension, with a magnitude equal to the yield point of the weld metal, generally about 50% higher than that of the parent material.\(^{(5,6,7)}\)

The magnitude and distribution of residual stress in both manually and automatically welded plates are essentially identical; the compressive residual stresses are slightly greater in plates welded automatically, and the tensile residual stresses at the weld are slightly smaller but spread over wider region.\(^{(8)}\)

It is the first pass of weld that causes the major build-up of the residual stress -- the effect of succeeding passes of weld is greater at the edge than at the weld. The second and third passes increased the edge stress by about 50%\(^{(6,8)}\). The experimental studies\(^{(6,8)}\) have shown that, where welding conditions are uniform along the plate, the residual stress is also uniform along the length. Plates smaller than about \(\frac{1}{2}\) inch in thickness show the same longi-
tudinal residual stress on both faces. The welding process changes the mechanical properties of the plate only in the vicinity of the weld; the most important change is that the yield stress is increased about 50%, which requires a proportionate increase in the compressive residual stresses elsewhere in the distribution. The experimental studies\(^{(6,8)}\) have made it possible to predict approximately the residual stress magnitude and distribution for plates of A7 and A36 steel, welded either manually or automatically. (No similar information is available for A441 steel; however, the residual stresses in welded flame-cut plates of A514 steel have been studied.\(^{(9)}\) Table 1 gives the estimated values for welded structural carbon steel plates; as an example, for a 10 in. wide plate with a weld along the centerline, the residual stress would be about 52 ksi tension at the center, and about 23 ksi at the edges. In this table, the residual stress is independent of weld size and plate thickness; actually, weld sizes and plate thickness are inherent in the table, since any particular plate width would correspond to a certain narrow range of weld size, and of plate thickness.

Shapes

Typical distributions of residual stresses in 6 in. and 10 in. welded H- and box-shapes are shown in Fig. 6. Figure 7 presents the residual stress distribution for a 10 in. T- and L-shape.

When the residual stress distributions are compared in Figs. 3 and 6 for welded plates and welded shapes respectively, it is seen that the distributions are similar in shape, and the same in magnitude to
within about 15%. Thus, the residual stress in a welded shape may be estimated approximately from the residual stress distribution of the separate welded plates which may be regarded as the component parts of the built-up shape. (The only reservation is that the free arm, or web, or T-shapes contain residual stresses of very small magnitude away from the weld.) Figure 8 shows the residual stress distribution predicted by the use of Table 1 for a 10 x 10 inch box shape; this estimate is superimposed on the actual experimentally measured distribution.

It was found that welded H-shapes have residual stress distributions different from those in rolled H-shapes, and with considerably larger magnitudes; compressive and tensile residual stresses occur in the same areas of the cross section. Welded box shapes have high tensile residual stress at the corners (that is, at the welds) and compressive residual stress distributed over a wide area of the mid-portion of the component plates. Edge preparations such as machining or flame-cutting have little effect on the final residual stress distribution of the welded box shapes, since welding is the final operation and has the greatest influence in the formation of residual stress.

The welding of cover plates to rolled H-shapes reverses the existing residual stress distribution in the shape; after welding, there are tensile residual stresses of high magnitude at the flange tips, that is, at the weld. See Fig. 9. A similar effect can be obtained merely by placing a line of weld along the flange tips.

It is quite common for welded shapes to be fabricated from plates which have been flame-cut to size before welding. The process
of flame-cutting is similar to that of welding in that it is a source of heat input. Thus, after cooling, the flame-cut edge will have residual stresses of tension. When such flame-cut plates are used in the fabrication of welded shapes, the process of welding induces a new residual stress distribution into the shape, one with tensile residual stresses at the weld, and with some modification to the residual stress distribution existing in the plates before welding. The change to the residual stress at the flame-cut edge will depend on the width of the plate, because of equilibrium requirements. Thus, a relatively narrow plate will have the high tensile residual stresses at the flame-cut edge reduced considerably by welding at the center, or even changed to small magnitudes of compressive residual stress. A wide plate, however, with the edges relatively far from the weld, will show little change at the edge due to the weld. The residual stress distribution in a 9 x 10 inch welded H-shape fabricated from flame-cut plates of A36 steel is shown in Fig. 10.

It was shown in Ref. 4 for rolled shapes that the effect of yield strength on the residual stress distribution is not as great as is the effect of geometry. This is shown for a 9 x 10 inch welded shape in Fig. 10 where the residual stresses are compared for A36 and A514 steels respectively.

**Thick Plates and Heavy Shapes. (Pilot Study)**

Welded thick plates are used in buildings, bridges, and in the hulls of ships and submarines. Unlike the "thin" plates considered
above, "thick" plates generally will not have a uniform distribution of residual stress through the thickness.

Little information is available on welded heavy shapes, yet they are used extensively, particularly as structural members in the lower stories of multi-story buildings, in major bridges, and in launching gantries for rockets and space vehicles. In addition to the difference in thickness, heavy shapes differ from the lighter ones also in that the ratio of area of weld to that of parent metal is smaller for the heavy shapes. Thus, proportionately, there should be a smaller heat input for heavy shapes than there is for lighter shapes, and so heavy shapes would be expected to contain residual stresses of a smaller magnitude.

The following preliminary results are those from a pilot study on heavy shapes.

Surface residual stresses for a 14 x 15 inch H-shape of A441 steel made up of UM plates for a groove weld and a fillet weld respectively, are shown in Figs. 11 and 12. The variation of residual stress through the thickness of the flange and web of Specimen C-1 is shown in Fig. 13. The magnitude of the residual stresses is comparatively high, similar for both the fillet and groove welds, and not less than in smaller shapes. This is not in line with what was expected and will be discussed later. Similar information is presented in Fig. 14 for a 14 x 15 inch H-shape of A441 steel made up of flame-cut plates. The effect of flame-cutting is shown by the high tensile residual stress at the flange tips.
It will be noted in comparing Figs. 11 and 12 that there are slightly higher stresses (about 15% average) in Column C3 than in Column C1. The former, with groove welds, has a higher heat input, and the difference, though small, is consistent with what would be expected.

2.3 THEORETICAL STUDIES

Even though a considerable amount of experimental work is involved in obtaining estimates of the magnitude and distribution of residual stress, it is only through theoretical study that the whole mechanism of the formation of residual stress can be understood. Also, it is hoped that even more precise correlation can be established in the future between theory and experiment.

There were two separate theoretical studies conducted during the period of this summary report. The first study\(^{7,11}\) considered the whole topic of welded plates and heat input and presented a step-by-step method to compute both thermal and residual stresses for any given welding condition. The step-by-step analysis represents the complete thermal stress history of the plate. The second study\(^ {12}\) presented a two-step method for determining residual stresses directly and comparatively rapidly. The two-step method presents a great simplification in the computation of residual stresses; however, the thermal stresses cannot be computed by this method.

The thermal stress is defined as that stress existing at the particular time interval in question after the onset of welding; the
residual stress is the thermal stress at a time interval of infinity -- that is, the residual stress is that existing when the plate or shape has completely cooled.

The temperature distribution in a plate heated by welding depends upon the thickness and width of the plate, and speed of welding, the position of the weld, the thermal properties of the weld, and the heat input due to the welding. Figure 15 shows, diagrammatically, the temperature distribution due to a moving source of heat -- this temperature distribution is called "quasi-stationary", since it is constant with respect to the moving heat source. The computed temperature distribution in an 8 x \( \frac{1}{2} \) inch center welded plate is shown in Fig. 16. The theoretical thermal and residual stress distributions for this same plate and heat input are shown in Fig. 17. The effect of multi-pass welds can be taken into consideration for the thermal stress history, provided the time difference is known.\(^{(7,12)}\)

The effect of heat input dominates all other effects considered in the computations -- such as geometry and thermal properties. Figure 18 shows theoretically that the magnitude of the residual stresses is a function of the magnitude of the heat input. The higher heat input (broken line) creates higher compressive residual stresses away from the weld; there is no direct relationship between heat input and residual stress as there is between heat input and temperature distribution. Increased heat inputs do not increase the tensile residual stress at the weld because this stress is limited by the yield point of the weld metal.
The computation for residual stresses developed in Ref. 7 included the effect of initial (cooling) residual stresses. Their effect is particularly important in those areas away from the weld where the material undergoes no plastic deformations.\(^{(11)}\)

The two-step method for the computation of residual stresses represents a significant reduction in the amount of work required. Since most structural engineering applications of residual stress are not concerned with the thermal stress history, this represents a significant step. The two-step method\(^{(12)}\) is based on the maximum temperature envelope set up in the plate by welding. The residual temperature strain which causes residual stress is the amount of cooling temperature strain in excess of the portion which remains elastic during heating. The elastic portion of the temperature strain at maximum temperature is reversible and is fully recovered on complete cooling. The residual stress set up depends on the elastic portion of the total cooling temperature strain for each element when it reaches its maximum temperature.

The residual stresses computed by the two-step method closely approximate those computed by the much more laborious step-by-step method.

The two-step method is of particular value when computing the residual stresses in thick plates, both the usual surface residual stresses, and the residual stresses through the thickness. The use of the step-by-step method would be so extremely long and tedious as to be impractical for such plates. The two-step method and its application...
is still under development, and only a preliminary pilot investigation has been conducted for the theoretical prediction of residual stresses in thick plates and heavy shapes.

Figure 19 shows the computed residual stresses for the top and bottom faces of a 6 x 1 inch A7 plate welded with a ½ inch center V-weld.\(^{(12)}\) The lack of correlation when compared to experimental values is mainly due to not including the initial (cooling) residual stresses in the computations.

A similar comparison is shown in Fig. 20 for an automatically butt-welded 6 x 1 inch A36 plate -- the weld being deposited in four passes in a double-V groove. The effect of pass sequence plays only a limited role in the final residual stress magnitude, and thus only the maximum temperature envelope is needed to be used in the computation.\(^{(12)}\)

Some preliminary considerations have been given to the prediction of residual stresses in heavy shapes, including both the sequence of fabrication and the use of flame-cut plates. The analysis requires a knowledge of the heat flow from the welding, that is, the heat input proportions to the web and flange. Based on an analogy of the flow of electric current, it was shown\(^{(12)}\) that for thin plates (plates up to approximately 1 in. thickness) the heat is divided proportionately to the relative plate thicknesses, and that for very thick plates the heat is divided equally among the heat sinks (that is, the plates) irrespective of plate thickness. For intermediate thickness (1 in. to 2 in. approximately), a more complicated relationship exists for the distribution of heat.
The preliminary theoretical study indicated that one reason for the large magnitudes of residual stress measured in the heavy welded shapes is due to the original residual stress distribution in the component thick plates -- the actual joining process of welding contributes only very small magnitudes of residual stress, in the order of 5 to 10 ksi. Further statements in this direction will depend on further studies on residual stresses in thick plates. The only information on residual stresses for a heavy plate or shape is given in Fig. 21 where the residual stress distribution due to cooling from rolling is shown for a 14W426 shape. The high magnitudes of residual stress in this shape are due entirely to the cooling process.

2.4 MECHANICAL PROPERTIES

Many of the mechanical properties of steel play an important role in the formation of residual stresses and in the strength of compression members. Those concerned affecting residual stresses have been mentioned above. Probably one mechanical property more than any other plays a dominant role in the overall picture, and that is the yield value of steel.

The "yield point" and the "yield strength" have been defined by ASTM(13) and are given in the Nomenclature below. The "yield stress level" is a useful term, and may be defined as "the average stress during actual yielding in the plastic range. It is the stress determined in a tension test corresponding to a strain of 0.005 in/in."

It remains fairly constant for structural steel provided the strain
rate remains constant. The "yield stress" is a general term which encompasses all of these definitions for the yield value.

The relationship between these values for yield have been discussed in Ref. 2, which also considered the influence of residual stress on the stress-strain relationship for the complete cross section. The influence of strain rate on the yield stress level was demonstrated. The "static yield stress level", \( \sigma_{ys} \), was defined as the yield stress level for a zero strain rate.\(^{14}\) It was shown to be a basic value for the yield stress independent of machine, human, or strain rate effects. The static yield stress level is of direct application to the testing of structures and structural components and to research into their strength. It is important to note that most loads on building structures are static.

In contrast to the static value, the "dynamic yield stress level", \( \sigma_{yd} \), is the value of yield at the strain rate of testing, that is, at a strain rate of other than zero.

During the study covered by this report, many routine tests for the yield value were conducted, and the values obtained are recorded in the relevant reports referred to elsewhere in this paper. One aspect of the yield value was studied in detail both for this study and in collaboration with other studies; this was the effect of strain rate on the yield stress of structural steels.\(^{14}\)

Three structural steels were considered, A36, A441, and A514. Relationships were determined between the ratio \( \sigma_{yd}/\sigma_{ys} \) and the strain
rate. These are not simple relationships and were determined through regression analyses with confidence limits applied. Figure 22 shows the variation of the dynamic yield stress ratio ($\sigma_{yd}/\sigma_{ys}$) with respect to the strain rate; the center line shows the mean relationship while 95% of all test results lie between the two outside lines.

The dynamic yield stress ratio increases rapidly at low strain rates and very slowly at the higher strain rates included in this study; in addition, as shown in Fig. 23, it decreases with increase in static yield stress level.

When the difference ($\sigma_{yd} - \sigma_{ys}$) for all three steels of the study is plotted with respect to the strain rate, Fig. 24, the mean curves all lie in a narrow band over a wide range of strain rates. Thus, it is possible to use an average curve to define them:

$$\sigma_{yd} - \sigma_{ys} = 3.2 + 0.001 \dot{\varepsilon}$$

This expression may then be used to predict the static yield stress level of a specimen from a standard tensile coupon test; this equation is valid for the range of strain rate $200 < \dot{\varepsilon} < 1600$. Since the strain rate does not greatly affect the difference ($\sigma_{yd} - \sigma_{ys}$) for practical values of strain rate, the crosshead speed per inch of gage length per second may be used in place of strain rate in the equation.
3. COMPRESSION MEMBERS

The strength of a compression member depends to a great extent on its slenderness ratio. Only very slender columns will buckle elastically, and their buckling strength is defined by the Euler equation. \(^{(15)}\)

The reduced modulus concept \(^{(15,16)}\) had been regarded as the correct buckling theory for columns in the inelastic range until 1947 when Shanley published a paper \(^{(17)}\) giving the buckling load of a centrally loaded column as the tangent modulus load.

Through the efforts of the Committee on Research of the Column Research Council \(^{(18)}\) a decade and a half ago, it was shown that the key to the application of the tangent modulus concept to the steel column lay in the inclusion of the effect of residual stresses which existed in the cross section of the column even before the application of external load.

The tangent modulus load is the lower bound for column strength; it is the load at which an initially straight column will start to bend. The upper bound is the reduced modulus load since it is the maximum load a column will sustain if it is temporarily supported up to that load. The ultimate load of a column will lie between these two limits. Generally, test results will tend to approximate the tangent modulus load. Further, since it is a lower bound, the tangent modulus load has been used as the basis for a column strength formula. \(^{(19)}\) The behavior of an ideal centrally loaded column is typi-
fied by the load deflection curve in Fig. 25.

For a pinned-end column, the "buckling load" is defined as the bifurcation load, that is, the load at which the theoretically straight column is indifferent to assuming a deflected position. The column will deflect and will then continue to deflect laterally and to take further load. (15,17) The "ultimate load" is the maximum load a column can carry; it is reached gradually unlike the buckling load which is an instantaneous phenomenon.

To take into account the transition in the column curve from the Euler curve to the yield value, it was the practice in the past to develop complicated correction factors using estimated eccentricities or initial deflections, such as, for instance, were applied to the secant formula. It has been shown (2) that, for the hypothetical case of straight centrally loaded pinned-end columns, the transition curve is due entirely to the presence of residual stresses in the cross section.

3.1 RESIDUAL STRESSES AND COLUMN BUCKLING

For column cross sections containing residual stresses the tangent modulus and reduced modulus theories for column buckling define buckling loads differing from those for the same column free of residual stresses. (20) A column cross section containing residual stresses will have certain fibers yield before others when the column is loaded. Compressive residual stresses exist in these fibers even before the load is applied. The material of the cross section is no
longer homogeneous and the general equations for tangent modulus and reduced modulus no longer apply. (4) However, a comparatively simple solution for column buckling strength may be obtained with the tangent modulus concept when it is assumed that every fiber in the cross-section has an idealized elastic-plastic stress-strain relationship, which does, in fact, exist for most structural steels. (15) Thus, it was shown (21) that the buckling strength is a function of the moment of inertia of the elastic part of the cross section at that load.

**Rolled Columns**

When the column curves for rolled H-shapes of structural carbon steel are prepared, it will be seen that the straight-line and parabolic curves give satisfactory predictions for column strength in the weak and strong axes respectively. (2) The ultimate loads carried by such columns do not exceed the tangent modulus load enough to warrant other prediction methods.

Test results (2) for rolled H-shapes are given in Fig. 26 to illustrate the efficacy of the straight-line and parabolic assumptions. The column curves are cut off at L/r = 20 because of the effect of strain-hardening. The CRC Basic Column Curve (18) is an average (parabolic) curve which is used for bending about both axes; the curve is a compromise, being the average of test results for bending about both axes. It is the first column curve based on a theoretical study reflecting actual behavior. The CRC curve was adopted by the AISC (22) as the ultimate strength curve for columns of
structural carbon and high strength steels (yield points from 33 ksi to 50 ksi).

It was shown in Ref. 4 for rolled shapes and in Fig. 10 for welded shapes that the magnitude and distribution of residual stresses depend mainly on the geometry, and very little on the yield point of the material. Hence, it may be concluded that, as the yield strength increases, the effect of residual stress on column strength decreases. Except for the very slender columns, higher column strengths are obtained most simply by using steel with a higher yield point. Hence, the use of high strength steel gives higher column strengths by virtue both of the higher yield point and of the relatively decreased effect of residual stresses. Figure 27 presents some experimental results on weak axis column tests of the same cross-sectional shape for three different steels; it is seen that the influence of residual stresses on the strength of the columns of higher yield strength is not as pronounced as it is on the columns of lower yield strength. (The information in Fig. 27 has been non-dimensionalized to facilitate comparisons.) Currently, no allowance for this effect is made in column formulas.

3.2 WELDED COLUMNS

The strength of centrally loaded welded columns can be predicted by the same techniques as for rolled shapes with cooling residual stresses. However, it has been shown\(^{(11,25,26)}\) that the use of the tangent modulus concept gives too conservative a result and therefore is not realistic for the prediction of the strength of welded
columns -- an ultimate strength analysis is necessary. (25) Further, for welded columns, there is a greater effect due to out-of-straightness than for rolled shapes; (11,27) the out-of-straightness together with the high magnitudes of compressive residual stress produce the lower strengths.

Hence, the study of the strength of centrally loaded welded columns differs essentially from that of centrally loaded rolled columns -- welded columns need an ultimate strength study whereas the tangent modulus buckling load presents a realistic figure for rolled columns.

For centrally loaded columns, the ultimate strength is a load (in excess of the tangent modulus load) for the column in the deflected position, and hence is a post buckling problem for the column in the inelastic range. It is difficult to obtain a perfectly general solution for the ultimate load except for very simple column shapes which do not contain residual stresses and where the stress-strain relationship of the material may be expressed in simple form.

Columns which are eccentrically loaded either due to initial imperfections, eccentricity of application of load, initial deflection or curvature, or non-symmetry of residual stresses, show a pronounced reduction of column strength. (11,28,29) Eccentrically loaded columns may be analyzed theoretically in exactly the same manner and with the same approximations as those used to determine the ultimate strength of centrally loaded columns; in this case, however,
account must be taken of the fact that the eccentrically loaded column will start deflecting immediately upon application of load.

The theoretical analyses for ultimate strength are based on a number of simplifying assumptions to take into account such variables as geometric shape, residual stresses, stress-strain relationship, deflected shape of column, and have been described and summarized in Ref. 4, and are given in detail in Refs. 11, 25, 26, 30 through 35.

Theoretical and experimental test results for the strength of welded columns are given below.

The strength of welded built-up columns of A7 steel is shown in Figs. 28 and 29 for H- and box-columns of small size, (6" x 6" to 10" x 10"). (27) The theoretical ultimate loads are compared with the experimental values in Fig. 29.

The same experimental results for A7 columns are shown in Fig. 30, in comparison with the CRC curve adopted for design by the AISC. The reason for the somewhat lower strengths is twofold: the effect of residual stresses due to welding, and the effect of initial out-of-straightness.

The column with an initial out-of-straightness deflects from the beginning of application of load. The out-of-straightness creates early yielding in the region of high compressive residual stress -- this increases the column deflection, which in turn creates further yielding. The process continues until the maximum load is
reached; the process is more accelerated for the welded column than for the rolled column -- for the welded column, the maximum load is comparatively small and reached at a comparatively large deflection. This is illustrated for rolled and welded column test results in Fig. 21.\(^{(1,27)}\)

The maximum effect of out-of-straightness occurs for the longer columns, \((L/r\) from 60 to 120) and this effect is discussed below. For shorter columns, \((L/r\) from 30 to 60) the box shape tends to be stronger than the H-shape bent about the weak axis, since box-shapes retain the corners in the elastic condition throughout the life of the columns due to the favorable tensile residual stresses there as a result of the weld. (For the same reason, box shapes are able to sustain the maximum load for much larger deflections than the H-shapes\(^{(11)}\).) H-shapes, with the compressive residual stresses at the flange tips, lose a major part of their rigidity very early under load, since the flange tips yield first.

A "favorable" distribution of residual stress is one that leads to improved column strength. A favorable distribution has tensile residual stresses furthest from the axis of bending. Such a benefit may be seen from Fig. 32 which presents experimental results. Rolled box and H-shapes are compared with their welded counterparts. In every case, the rolled column displays a somewhat superior strength. Figure 32 shows that a reversal of the normal pattern of residual stress in the flange tips of an H-shape improves column strength markedly -- compare the rolled 8W31 shape before and after reinforcing by welding cover plates\(^{(10)}\).
(Fig. 32c), and compare the Japanese welded H-shape before and after the deposition of a bead of weld along the flange tips, Fig. 32c.\(^{(36)}\)

The great improvement in the strength of a welded column after removal of residual stress by annealing is also shown for the Japanese shape\(^{(36)}\) and for the 8W31 rolled shape,\(^{(1)}\) Fig. 32d.

Reversal of the residual stress in the flange due to welding of the flange tips or due to the use of flame-cut plates leads to higher column strengths for H-shapes. However, the strength of welded box-columns made from either machined or flame-cut plates is the same, since the process of welding does not change the favorable residual stress distribution in the flame-cut plate. This is discussed further below.

### 3.3 Factors in Column Strength

Aside from the effect of residual stress, a number of other factors should be considered in any general study of columns, both rolled and welded. These factors may play an important part in design. Those to be considered here are the shape of the cross section, higher yield strengths, out-of-straightness, and cold-straightening.

No particular shape can be regarded as being best for column use. Box-shapes, however, are somewhat stronger than corresponding H-shapes because of their favorable residual stress distribution; a cost-strength study would need to be made for any final decisions. For the low slenderness ratios (L/r up to about 50) when out of straightness is not as important a factor as for high slenderness ratios,
columns with a favorable residual stress distribution will be stronger than columns with an unfavorable distribution. Whether the residual stress is due to welding or to cooling after rolling, if the material furthest from the axis of bending is in a state of residual compression, then this material will yield first under load, leading to column failure at a lower load than would otherwise be expected.

Except for slender columns, higher column strengths are obtained most simply by using steel of higher yield strength. This was considered for rolled shapes above, (Fig. 27), and the same general comments apply for welded shapes, as may be seen in Figs. 33 and 34.

Out-of-straightness is a significant factor involved in the strength of columns. Out-of-straightness is used here to refer to all deviations which result in an eccentrically loaded column: initial curvature, eccentric application of load, and unsymmetrical residual stress distribution. In general, the maximum out-of-straightness allowed by specifications for columns is of such a magnitude that the corresponding welded-column strength will lie on the lower boundary of the test results shown in Fig. 30. The expected maximum out-of-straightness (for example, \( \frac{1}{4} \) in. in a 20 ft. column, AISC Specifications (22)) will reduce the column strength about 25 percent below that indicated by the CRC curve in the medium slenderness ratios (L/r about 60 to 120). The effect of out-of-straightness on shorter columns is not as great.

The usual structural columns, rolled or welded, will be cold-straightened to the specified tolerances. For rolled shapes, the pro-
cess of cold-straightening induces residual stresses which are of a similar magnitude, although different distribution, to the cooling residual stresses.\footnote{37} This means that findings based on rolled members with cooling residual stress patterns will be conservative when applied to straight rolled members whose cooling residual stress patterns have been modified by cold bending. At present, there are no test data on the effect of cold-straightening on welded columns. The effect of cold-straightening depends on the manner by which it is carried out, whether by "gagging" or by "rotorizing". "Gagging" concentrates the straightening at a few sections, leaving most of the column with its initial state of residual stress. "Rotorizing" is a continuous straightening process and changes the residual stress pattern completely. In any case, column strength based on cooling or welded residual stresses will normally be considered, as there is no assurance that the column will be cold-straightened so that these residual stresses will be changed to a more favorable distribution.

3.4 \textbf{REINFORCED COLUMNS AND HYBRID COLUMNS}

Hybrid construction is quite common today because of the multitude of steels available. Thus, structures can be designed economically by combining these different steels in hybrid construction -- the higher grades of steel can be placed where the most strength is needed.

Either the structure itself may be hybrid, or else a structural member may be hybrid. For instance, a hybrid structure such as
a multi-story frame would use high-strength steel columns for the lower stories, and mild-steel columns for the upper stories. On the other hand, the hybrid member uses different grades of steel within the member itself -- a column section would use the high strength material in the flanges. Design information has been available for hybrid beams and girders. (38).

The results of a theoretical and experimental study of the strength of hybrid columns (35) have shown that, except for very short columns, the behavior of the hybrid member is essentially identical with that of the homogeneous member when it is assumed that the shape is homogeneous high-strength steel throughout. Two principal types of shape were investigated, one with A514 steel flanges, flame-cut, and either A36 or A441 steel web, and the other with either A441 UM or A441 flame-cut steel flanges, and A36 steel web. The residual stress distribution for these hybrid shapes are the same as those of homogeneous shapes. (35) Thus, the test results shown in Fig. 35 may be expected from a knowledge of the strength of similar homogeneous columns. Figure 35 shows the computed tangent modulus curves for each test column, together with experimental results.

In the lower stories of multi-story buildings where the slenderness ratio of columns is low, columns of A514 steel might be considered. In the stories immediately above these, hybrid columns might be more economical because the lower-strength steel in the web can be overstressed without adverse effect. (35) This latter statement is important and reflects the strength of a yielded web when the
proper b/t ratios have been chosen; this is discussed below, Section 3.6.

A variation of the hybrid column is the column which is reinforced, often under load. A column is reinforced usually by welding cover plates to the flanges -- often the structure is in use and so the reinforcement is carried out under load. One series of such columns of A7 steel\(^{(10)}\) was welded under loads as high as 75% of the yield stress without causing any buckling or other changes. This is because the influence of welding was confined to a very small area in the vicinity of the weld. The mechanical properties in the major portion of the section are not affected enough to reduce the strength of the section. This holds true only when the weld is deposited along the length of the column -- if the weld is deposited transversely as with a splice, then the area affected by the weld becomes very large and critical, and the above statements no longer hold true.

Reinforcing improves the strength of columns, both because of the additional material and also because of the creation of a more favorable residual stress distribution. This is demonstrated above in Fig. 9; the welding changed the compressive residual stresses to tensile residual stresses. This is reflected in Fig. 36 -- the differences between the theoretical tangent modulus predictions for unreinforced and reinforced columns reflects the difference in residual stresses. The limited experimental results available bear this out. Little difference was observed between the strength of columns welded under load or under no load.
Reinforcement may also be accomplished merely by laying a bead of weld along the flange tips. The additional strength is achieved through the reversal of the residual stress there -- this additional strength can be quite substantial, as shown for the Japanese results in Fig. 32.

3.5 HEAVY COLUMNS AND FLAME-CUT PLATES

Although the results presented thus far show that some welded columns are not as strong as rolled columns, it should not be concluded that this is necessarily true for all welded columns. Those results were obtained from studies of welded columns built up of universal mill plates. They were H-shapes whose maximum size was 9" x 10" with 3/4" plates. These are relatively small members.

An important observation already made is the "favorable" residual stress distribution in flame-cut plates. Under some circumstances it has also been noted that the high tensile stresses at the edges of such plates are not decreased substantially when they are welded into shapes (see Fig. 10). Thus the shape retains the favorable pattern which has tensile stresses at the flange tips of an H-shape, and consequently one would expect it to be a higher strength than would otherwise be found in a welded column.

In addition to this, columns that are welded together from thick plates may be joined with welds that are considerably smaller with respect to total shape area than is the case with thinner plates. Thus, proportionately, there is smaller heat input for heavy shapes,
and so they would be expected to contain compressive residual stresses of smaller magnitude.

On the basis of these trends, therefore, preliminary pilot studies were undertaken into welded columns with flame-cut plates and into heavy columns welded from thick plates.

A comparison of the preliminary test results is shown in Fig. 37 for 6" x 7" and 9" x 10" welded H-shapes, both flame-cut plates and universal mill plates; the favorable residual stress distribution of the flame-cut plates does result in an increase in column strength. This pilot study indicates that welded shapes fabricated from flame-cut plates are stronger than their UM counterparts and may even approach a strength comparable to that of rolled W sections.

The residual stress distributions in the heavy welded shape of the test program is shown above in Figs. 11, 12, and 13; based on preliminary conclusions from these tests, it would appear that the comparatively large magnitudes of residual stress may be due more to cooling after rolling than due to welding. If this is true, then a reduction of weld size will not lead to improved column strength. This must be explored further.

No column tests have yet been conducted on heavy welded shapes; preliminary theoretical strength predictions for one heavy shape are available. (12) This is shown in Fig. 38 for a 14" x 15" H-shape, A36 steel, for a fillet weld. There is a strength in excess of that of a small 9" x 10" welded column. When the varia-
tion of residual stress is considered for the strength of column C4, the tangent modulus load is quite high in the lower slenderness ratios, but not for the higher slenderness ratios. That is, in the range of practical values of column slenderness ratios found in buildings, 30 to 60, the strength of a welded heavy shape compares favorably with the strength of rolled $W$ shapes. For higher strength steels, the influence of residual stress decreases.

It is important, therefore, to complete a program which will specifically delineate the influence of geometry and edge preparation in order that consistent design recommendations may be prepared.

3.6 WIDTH-THICKNESS RATIOS AND PLATE BUCKLING

Studies of plate buckling as influenced by residual stress have been underway simultaneously with other phases of the overall investigation of residual stresses and the strength of compression members.\(^{(34,39,40,41,42)}\) It was felt at the start of the research program that lower width-thickness ratios may need to be specified for the component plates of welded shapes. The studies have been both theoretical and experimental. Generally, the behavior of plates of high strength steel have been of more interest than that of structural carbon steel, since the use of high strength steel leads to thinner plates.

Reasonable correlation was obtained between test results and predictions for plates containing residual stresses of welding
or cooling types. (34,39,40,41,42) This is shown in Figs. 39 and 40, for A36 and A514 steels, respectively. Theoretical plate buckling curves have been obtained for a wide variety of residual stress distributions, both welding and cooling, for plates under various edge conditions. The edge conditions were chosen to resemble web plates in H-shapes as well as component plates for box shapes.

The theoretical studies considered elastic, elastic-plastic, and plastic buckling. For elastic buckling of a plate with residual stresses, the influence of the residual stresses on the buckling strength of the plate is independent of the critical stress, and can be evaluated from the residual stress distribution; Figs. 39 and 40. The possibility that a plate with residual stresses may buckle without any external load is shown in Figs. 39 and 40. This fact explains why a plate can distort due only to welding. Tests showed, (Figs. 39 and 40) that although considerable post buckling strength occurred for elastic buckling of plates, this was not the case for elastic-plastic buckling. AISC specifications (23) for width-thickness ratios are based on the assumption that no plate buckling occurs until the yield load of the section is reached and that the plate buckling curve intersects the yield line (line AB in Fig. 39) at 70% of the b/t ratio for the elastic plate buckling curve free of residual stress (point C in Fig. 39). This assumption is somewhat conservative for rolled A7 shapes and slightly on the unconservative side for welded A7 shapes. (42) Nevertheless, it is a good representative assumption. Based on this
point on the non-dimensionalized curve, (point C in Fig. 39) the specification gives the b/t ratio for steels of different yield strengths, non-dimensionalized by a factor of \(1/\sigma_y\). Thus, if the specification is conservative for A7 steel, it is certainly conservative for A514 steel, since residual stresses play a much smaller role with the higher strength steels. (34) Although the pilot tests indicated that the AISC b/t ratios may be slightly optimistic, there seems to be no reason for not using them for A514 steel. Thus, the AISC width-thickness ratios could be used for steels of yield strengths up to that of A514, for both welded and rolled shapes, Table 2.

3.7 DESIGN IMPLICATIONS

If all welded columns were not as strong as rolled columns then there would be no way of avoiding the application of some kind of reduction factor to the column formula for welded members. In all likelihood this factor would be variable with respect to the slenderness ratio.

The fact that some classes of welded columns are stronger than others suggests that correction factors would be needed only for certain classes of welded columns in order to have a consistent factor of safety.

Another approach would be to decide what minimum factor of safety is needed and to select one formula to apply to all columns, selecting as the base the average strength of that class of columns.
that consistently give the minimum strength. Although this procedure would be simple, it would be wasteful of material, since many classes of columns would have strengths in excess of the minimum.

A more rational approach -- providing for more economical structures with uniform safety -- is to delineate those conditions in which a column would fall into one category or another. Such categories would be established to take into account the demonstrative differences due to: method of fabrication, yield point of the material, edge preparation, geometrical configurations, and so on. Then through appropriate correction factors or formulas, consistent designs could be achieved. These formulas or factors would be minimal in number -- two, or at most three. Of course, the designer always has the option of using the most conservative factor if he wishes to do so.

The most critical need, however, is to complete what has been started to determine the influence of edge preparation (flame-cut edges), the influence of thickness, and other geometrical and welding effects. Upon these results will depend the specific design recommendations.
4. SUMMARY

It is the purpose of this paper to summarize the experimental and theoretical studies conducted into the strength of welded columns of structural carbon steel. These studies also included investigations of residual stresses and mechanical properties. The column sizes of the investigation are limited to 10" x 10"; some preliminary results are presented for one heavy shape.

Column Strength
1. The presence of residual stress is the reason for the transition curve in the column curve for initially straight axially loaded columns. Residual stresses reduce the buckling strength because of the early localized yielding that occurs at certain portions of the cross section. (Section 1, Fig. 1.)

2. The strength of rolled columns may be expressed simply in terms of the tangent modulus. (Section 1.)

Residual Stress
3. The residual stress distribution in welded plates differs from the rolled counterparts in that the shape of the pattern tends to be more unfavorable with respect to column behavior. (Figure 3.)

4. Welded H-shapes have residual stress distributions different in shape from those in rolled H-shapes, and considerably larger in
magnitude. Compressive and tensile residual stresses are distributed
over approximately the same areas of the cross section. (Figure 6.)

5. Box shapes, fabricated from four plates, have high tensile
residual stresses at the corner welds and compressive residual stresses
elsewhere, (Fig. 6.)

6. The residual stress distribution in a welded shape is similar
in shape and to within about 15% in magnitude to the separate residual
stress distributions of the component welded plates. (Section 2.2.)

7. The residual stress distributions in welded plates and shapes
may be estimated from tables of stress values prepared from this study.
(Table 1, Fig. 8.)

8. The compressive residual stresses in the flange tips of
H-shapes may be reversed to tension by reinforcement, either by the
welding of cover plates to the flanges, or by the placing of a line
of weld along the flange tips. (Figure 9.)

9. Flame-cutting, being a source of heat, leaves residual
stresses of tension at the flame-cut edge after cooling. (Section
2.2, Fig. 10.)

10. Welded H-shapes fabricated from flame-cut plates will retain
tensile residual stresses at the flame-cut edge. (Figure 10.)

11. The magnitude of both compressive and tensile residual
stresses measured in a 14" x 15" welded heavy H-shape was comparatively
high, and was not less than that measured in smaller welded shapes.
It is suggested that the large magnitudes of residual stress in heavy welded shapes may be due to the original cooling residual stresses in the thick rolled component plates. (Section 2.3, Figs. 11 and 12.)

12. The effect of heat input dominates all other effects in the creation of residual stresses. (Section 2.3.)

13. It is possible to determine theoretically the residual stresses in welded plates and in certain welded shapes. Two methods were prepared in this study: the step-by-step method which determines also the thermal stresses, and the two-step method which presents a quick and accurate method of computation. (Section 2.3, Figs. 17 and 19.)

**Mechanical Properties**

14. It was possible to develop a simple relationship between the static yield stress of steel and the strain rate of testing. (Section 2.4, Figs. 23 and 24.)

15. Yield stress has a small but negligible effect on the magnitude of residual stress; hence the influence of residual stress on the strength of columns of high strength steel is not as pronounced as for columns of structural carbon steel. (Section 3.1, Figs. 27 and 33.)

16. Welding changes mechanical properties only in the vicinity of the weld. The yield stress of the weld metal is about 50% higher than that of the parent material for structural carbon steel. (Section 2.2.) (On the other hand, weld metal in A514 steel will have a lower yield stress than the parent material when a lower strength elec-
Welded Columns

17. Welded columns contain residual stresses of a larger magnitude than do comparable rolled columns -- this implies that column strength characteristics are different for such welded columns from those for rolled columns. (Section 3.2.)

18. Welded H-shaped columns fabricated from UM plate tend to have lower strengths than corresponding rolled columns. This is due to two effects: a) the greater compressive residual stress, and b) the initial out-of-straightness which has a greater effect for larger magnitudes of compressive residual stress. (Section 3.2, Fig. 31.)

19. Welded box columns tend to be stronger than welded H-columns bent about the weak axis because of a more favorable residual stress distribution. (Section 3.2, Fig. 30.)

20. Tensile residual stresses at the flange tips are favorable for the strength of H-shaped columns. (Fig. 32.)

21. Maximum column strength in H-shapes is obtained from "favorable" residual stress distributions -- such as due to the use of flame-cut plates. Other favorable distributions are those due to reinforcing, either by welding cover plates or else by laying a bead of weld down the flange tips. (Figs. 32 and 37.)

22. Annealing will remove most of the residual stresses, and this improves column strength. (Fig. 32.)
23. The strength of welded columns may be affected by the welding process, since the welding process has a direct effect on the formation of residual stress. In theory, welded columns of higher strength could be fabricated if welding techniques could be developed such that lower heat inputs are generated resulting in lower magnitudes of residual stress. (Section 2.3.)

24. Hybrid columns, with low-strength web and high-strength flanges, may be designed on the assumption that the shape is homogeneous high-strength steel throughout, except for very short columns, because the lower-strength web can be overstressed without adverse effect. (Section 3.4.)

25. The flame-cutting of structural plates used in the fabrication of columns by welding improves the strength of H-shaped columns in comparison to rolled columns. Preliminary studies on two small shapes have indicated that the strength of welded shapes furnished from flame-cut plates may approach that of comparable rolled shapes, for structural carbon steel. (Section 3.5, Fig. 37.)

26. Preliminary theoretical studies, taking into account the variation of residual stress throughout the thickness of the flanges, have indicated that heavy welded columns may have strengths greater than small welded columns for slenderness ratios less than about 50. (Section 3.5, Fig. 38.)

27. In summary, the present status of studies of the strength of welded columns is that there are certain categories of welded columns
whose strength does not match that of their rolled counterparts. On the other hand there is evidence that other classes of welded columns are stronger than the minimum observed up to the present time. The present critical need is to carry out studies which will enable one to categorize the differences and to prepare appropriate design recommendations.
5. ACKNOWLEDGEMENTS

This paper presents a summary of the results of an experimental and theoretical study into the strength of welded columns.

The investigation was conducted at Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University, Bethlehem, Pennsylvania. The Pennsylvania Department of Highways, U. S. Department of Commerce - Bureau of Public Roads, sponsored the major part of this study. Associate studies, referred to in this paper, were sponsored by the American Institute of Steel Construction, the United States Steel Corporation, and the Engineering Foundation through the Column Research Council. Column Research Council Task Group 1 under the chairmanship of John A. Gilligan provided valuable guidance.

Special thanks are due to Lynn S. Beedle, Director of Fritz Engineering Laboratory and Acting Head, Department of Civil Engineering, for his helpful criticism and advice in the preparation of this paper, and throughout the course of the investigation. Fiorello R. Estuar reviewed the paper and made a number of excellent suggestions which were incorporated. William C. Cranston assisted in the preparation of the figures. Appreciation is due also to the author's colleagues who assisted in various parts of the study and to whose work reference is made throughout the paper.

-42-
6. NOMENCLATURE

b width of cross section of shape; width of plate
b/t width-to-thickness ratio in plate
c distance from neutral axis to extreme fiber
e eccentricity of load; out-of-straightness of column
e/b eccentricity ratio in column
E modulus of elasticity
E_t tangent modulus
I moment of inertia
L column length between pin ends
L/r slenderness ratio
P load; subscript y refers to load at yield
r radius of gyration
t thickness of plate
ε strain
ε˙ strain rate = dε/dt
δ deflection of column at mid-height
σ stress
σ_r residual stress
σ_y yield stress, yield point, yield strength, or yield stress level
σ_yd dynamic yield stress (strain rate is not zero)
σ_vs static yield stress (corresponds to zero strain rate)
Ultimate load: The ultimate load is the maximum load a column will carry. It is not coincident with the buckling load for an axially loaded column.

Stub column: A stub column is a short compression specimen, sufficiently long for use in measuring the stress-strain relationship for the complete cross section, but short enough to avoid buckling in the elastic and plastic ranges.

Yield point: The yield point is the first stress in the material, less than the maximum attainable stress, at which an increase in strain occurs without an increase in stress. (ASTM A370-61T)

Yield strength: The yield strength is the stress corresponding to the load which produces in a material, under the specified conditions of the test, a specified limiting strain. (ASTM A370-61T)

Yield stress level: The yield stress level is the average stress during actual yielding in the plastic range; this stress remains fairly constant. When the stress is not constant it is taken as the stress corresponding to a strain of 0.5 percent.
7. TABLES AND FIGURES
Table 1  Average of Experimental Values of Residual Stress Distribution in Welded Plates

**CENTER WELDED PLATES**

<table>
<thead>
<tr>
<th>Plate</th>
<th>Nos. of Measurements</th>
<th>$\sigma_{ro}$ (ksi)</th>
<th>$\sigma_{rc}$ (ksi)</th>
<th>$Z_1 + Z_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A M</td>
<td>A M</td>
<td>A M</td>
<td>A M</td>
</tr>
<tr>
<td>Narrow</td>
<td>3 22</td>
<td>35 44</td>
<td>22 22</td>
<td>0.21 0.19</td>
</tr>
<tr>
<td>Medium</td>
<td>1 22</td>
<td>52 52</td>
<td>24 23</td>
<td>0.19 0.18</td>
</tr>
<tr>
<td>Wide</td>
<td>6 7</td>
<td>52 58</td>
<td>11 7</td>
<td>0.17 0.13</td>
</tr>
</tbody>
</table>

**EDGE WELDED PLATES**

<table>
<thead>
<tr>
<th>Plates</th>
<th>Nos. of Measurements</th>
<th>$\sigma_{rt}$ (ksi)</th>
<th>$\sigma_{rm}$ (ksi)</th>
<th>$\sigma_{re}$ (ksi)</th>
<th>Positions of Zero &amp; Max. Res. Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A M</td>
<td>A M</td>
<td>A M</td>
<td>A M</td>
<td>Z_1 A M</td>
</tr>
<tr>
<td>Narrow</td>
<td>2 6</td>
<td>40 46</td>
<td>24 19</td>
<td>12 5</td>
<td>0.18 0.12</td>
</tr>
<tr>
<td>Medium</td>
<td>1 5</td>
<td>38 55</td>
<td>21 23</td>
<td>7 7</td>
<td>0.16 0.11</td>
</tr>
<tr>
<td>Wide</td>
<td>3 8</td>
<td>45 41</td>
<td>22 14</td>
<td>4 7</td>
<td>0.13 0.07</td>
</tr>
</tbody>
</table>

A - Automatically welded
M - Manually welded
Narrow: to 8" width
Medium: 8" to 14"
Wide: 14" to 20"
Table 2  Width-Thickness Ratios (After AISC)

<table>
<thead>
<tr>
<th>Projecting Elements</th>
<th>Formula</th>
<th>Width-Thickness Ratio</th>
<th>Plate Coefft.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>33</td>
<td>36</td>
</tr>
<tr>
<td>Single Angle</td>
<td>$\frac{2400}{\sqrt{\sigma_y}}$</td>
<td>13</td>
<td>13</td>
</tr>
<tr>
<td>Flange</td>
<td>$\frac{3000}{\sqrt{\sigma_y}}$</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>Stem of Tee</td>
<td>$\frac{4000}{\sqrt{\sigma_y}}$</td>
<td>22</td>
<td>21</td>
</tr>
<tr>
<td>Web</td>
<td>$\frac{8000}{\sqrt{\sigma_y}}$</td>
<td>44</td>
<td>42</td>
</tr>
<tr>
<td>Cover Plate</td>
<td>$\frac{10,000}{\sqrt{\sigma_y}}$</td>
<td>55</td>
<td>53</td>
</tr>
</tbody>
</table>
Fig. 1  Residual Stress and the Column Curve
Fig. 2  Basic Welded Shapes
Fig. 3  Residual Stresses and Welded Plates
Fig. 4 Residual Stresses and Weld Passes

3/8 Single-V Weld
A7 Steel

3 Passes
Fig. 5  Residual Stresses in Wide Plates
Fig. 6  Residual Stresses in Welded Shapes
Fig. 7 Residual Stresses in T and L Shapes
Fig. 8  Predicted and Measured Residual Stresses in a Box Shape
Fig. 9  Reversal of Residual Stresses
Fig. 10  Residual Stresses for Flame-Cut 9" x 10" H Sections
Fig. 11  Experimental Residual Stress Distribution for Shape C3
Fig. 12  Experimental Residual Stress Distribution for Shape C1
COLUMN C1
14 x 15 H, (U.M.)
A 36, Fillet Weld

Variation Across Thickness:

Surface Residual Stress

Stress Scales

Outside Face

Inside Face

Thickness:

Surface:

Fig. 13 Residual Stress Through Thickness of Heavy Shape Fabricated from UM Plates
Variation Across Thickness:

Surface Residual Stress

Outside Face

Inside Face

Stress Scales

Thickness

Surface

0 50 ksi

Fig. 14 Residual Stress Through Thickness of Heavy Shape Fabricated from Flame-Cut Plates
Temperature distribution at C, rO$^\infty$ section shown.

Isotherms Showing Temperature Distribution

Fig. 15 Temperature Distribution Due to a Moving Source of Heat
Fig. 16 Computed Temperature Distribution in Center Welded Plates
Fig. 17 Computed Thermal Stresses During Welding
Edge Welded 12" x $\frac{3}{4}$" Plate

Distance from edge in inches

- 3/8" Weld
  - Heat Input = 5.9 Kw
- 3/4" Weld
  - Heat Input = 11.8 Kw

THEORETICAL DISTRIBUTION

Fig. 18  Heat Input and Residual Stress
6" x 1" PLATE
A7 STEEL
\( \frac{1}{2} " \) CENTER V-WELD

Theory:
- Top face
- Bottom face

Experimental:
- Top face
- Bottom face

Fig. 19 Residual Stress Computed in Thick Plate
Fig. 20  Residual Stresses in Butt-Welded Thick Plate
Fig. 21 Residual Stresses in Rolled 14W426
Fig. 22 Relationship Between Dynamic Yield Stress Level, Static Yield Stress Level and Strain-Rate - A36 Steel
Fig. 23 Estimated Curves Relating Dynamic Yield Level, Static Yield Stress Level and Strain-Rate for A36, A441, A514 Steels, in terms of $\frac{\sigma_{yd}}{\sigma_{ys}}$
Fig. 24  Estimated Curves Relating Dynamic Yield Level, Static Yield Stress Level and Strain-Rate for A36, A441, A514 Steels, in terms of $(\sigma_{yd} - \sigma_{ys})$
Fig. 25 Behavior of Centrally Loaded Columns
Fig. 26  Column Curves and Test Results for Rolled H-Shapes (A7 Steel)
STEEL

- A7 ($\sigma_y = 36$ ksi)
- A242 ($\sigma_y = 56$ ksi)
- "T-I" ($\sigma_y = 110$ ksi)

$$\lambda = \frac{1}{\pi} \sqrt{\frac{\sigma_y}{E}} \cdot \frac{KL}{r}$$

Fig. 27  Yield Point and Strength of Rolled Columns
Fig. 28  Welded H-Columns (6" x 7" and 9" x 10")
Fig. 29  Welded Box Columns (6" x 6" and 10" x 10")
Fig. 30  Welded Column Strength
Fig. 31 Effect of Out-of-Straightness and Residual Stress

\[ \frac{\sigma}{\sigma_y} \]

\(8W F 31\) Rolled \( (\frac{ec}{r^2} = 0.038 ; \frac{L}{r} = 58 )\)

\(6\times7H\) Welded \( (\frac{ec}{r^2} = 0.014 ; \frac{L}{r} = 53 )\)

\(\delta\), deflection at mid height in inches
Rolled           Welded           Rolled           Welded  
° 12WF65-A7   H  9x10H-A7   ²  A36 (10x10)   □  A7  
¾ 8WF31-A7    H  6x7H-A7   ²  A36 (6x6)    □  A7  

\[
\frac{\sigma_{cr}}{\sigma_y} = \frac{\gamma}{\pi} \sqrt{\frac{\sigma_y}{E}} \cdot \frac{KL}{r}
\]

(a) H-Shapes

(b) Box Shapes

Original          Reinforced

¾ (8WF31-A7)   X
△ (Japanese H)  V

Original          Annealed

¾ (8WF31-A7)   ●
△ (Japanese H)  ▲

\[
\frac{\sigma_{cr}}{\sigma_y} = \frac{\gamma}{\pi} \sqrt{\frac{\sigma_y}{E}} \cdot \frac{KL}{r}
\]

(c) Reinforcement

(d) Annealing

Fig. 32 Welded, Rolled and Annealed Columns
Fig. 33  Yield Point and Strength of Welded H-Columns

\[ \lambda = \frac{1}{2\pi} \sqrt{\frac{\sigma_y}{E}} \cdot \frac{K}{r} \]
Fig. 34 Yield Point and Strength of Welded Box Columns

\[ \lambda = \frac{1}{\pi} \sqrt{\frac{\sigma_y}{E}} \cdot \frac{KL}{r} \]
<table>
<thead>
<tr>
<th>Shape</th>
<th>Flange</th>
<th>Web</th>
<th>Curve</th>
<th>Test Point</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A 514</td>
<td>A 36</td>
<td>------</td>
<td>o</td>
</tr>
<tr>
<td>2</td>
<td>A 441</td>
<td>A 36</td>
<td>------</td>
<td>x</td>
</tr>
<tr>
<td>3</td>
<td>A 441</td>
<td>A 36</td>
<td>------</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td>(Flame-cut)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>A 514</td>
<td>A 441</td>
<td>------</td>
<td>□</td>
</tr>
<tr>
<td>5</td>
<td>A 514</td>
<td>A 441</td>
<td>------</td>
<td>△</td>
</tr>
</tbody>
</table>

\[ \lambda = \frac{1}{\pi} \sqrt{\frac{\sigma_y}{E}} \cdot \frac{L}{r} \]

**Fig. 35** Strength of Hybrid Columns
Fig. 36  Strength of Reinforced Columns
Fig. 37 The Strength of Welded Shapes Fabricated from Flame-Cut Plates
Fig. 38 Comparison of Theoretical TM Load Curves for C4
Fig. 39 Plate Buckling Curve with Test Points (A36 Steel)

- Elastic buckling
- Elastic-plastic

\[
\frac{\sigma_{cr}}{\sigma_y} = 0.23
\]

\[
\frac{\sigma_{rl}}{\sigma_y} = 0.16
\]

- Critical Stress
- Ultimate Stress

- \( b/t = 45 \)
- \( b/t = 64 \)

- \( \sigma_y \) for \( \frac{b}{t} \)

\[
\frac{\sigma_y}{E} = \frac{36}{30,000}
\]

\[
\frac{\sigma_{rl}}{\sigma_y} = \begin{cases} 
0.75 & \text{for } \frac{b}{t} = 20 \\
0.50 & \text{for } \frac{b}{t} = 40 \\
0.25 & \text{for } \frac{b}{t} = 60 \\
0.12 & \text{for } \frac{b}{t} = 80 \\
0 & \text{for } \frac{b}{t} = 100 \\
0 & \text{for } \frac{b}{t} = 120 \\
0 & \text{for } \frac{b}{t} = 140 \\
\end{cases}
\]
Fig. 40 Plate Buckling Curve with Test Points (A514 Steel)
8. REFERENCES

1. A. W. Huber, L. S. Beedle
   RESIDUAL STRESS AND THE COMPRESSION STRENGTH OF STEEL

2. L. S. Beedle, L. Tall
   BASIC COLUMN STRENGTH
   ASCE Proc., ST7, Vol. 86, July 1960

3. L. Tall
   STUB COLUMN TEST PROEDURE
   Also Document X-282-61, International Institute of
   Welding, Oslo, July 1962.
   Also, Appendix of Ref. 18, ("CRC Guide")

4. L. Tall
   RECENT DEVELOPMENTS IN THE STUDY OF COLUMN BEHAVIOR
   Journal, Institution of Engineers, Australia, Vol. 36,
   Dec. 1954

5. N. R. NagarajaRao, F. R. Estuar, L. Tall
   RESIDUAL STRESSES IN WELDED SHAPES
   Welding Journal, Vol. 43, July 1964

6. N. R. NagarajaRao, L. Tall
   RESIDUAL STRESSES IN WELDED PLATES

7. L. Tall
   RESIDUAL STRESSES IN WELDED PLATES -- A THEORETICAL STUDY
   Welding Journal, Vol. 43, Jan. 1964

8. N. R. NagarajaRao, L. Tall
   RESIDUAL STRESSES IN AUTOMATICALLY WELDED PLATES
   Fritz Laboratory Report No. 249.22, in preparation

9. E. Odar, F. Nishino, L. Tall
   RESIDUAL STRESSES IN T-1 CONSTRUCTIONAL ALLOY
   STEEL PLATES
   Fritz Laboratory Report No. 290.4, Jan. 1965
   (To be published in Welding Journal)

10. N. R. NagarajaRao, L. Tall
    COLUMNS REINFORCED UNDER LOAD
    Welding Journal, Vol. 42, April 1963

-88-
11. L. Tall
THE STRENGTH OF WELDED BUILT-UP COLUMNS
Ph.D. Dissertation, Lehigh University, May 1961

12. F. R. Estuar
WELDING RESIDUAL STRESSES AND THE STRENGTH OF HEAVY COLUMN SHAPES
Ph.D. Dissertation, Lehigh University, Sept. 1965

13. ASTM
METHODS AND DEFINITIONS FOR MECHANICAL TESTING OF STEEL PRODUCTS
A370-65

EFFECT OF STRAIN RATE ON THE YIELD STRESS OF STRUCTURAL STEEL

15. L. Tall, Editor-in-Chief
STRUCTURAL STEEL DESIGN
Ronald Press, 1964

16. F. Bleich
BUCKLING STRENGTH OF METAL STRUCTURES
McGraw-Hill, 1952

17. F. R. Shanley
INELASTIC COLUMN THEORY

18. B. G. Johnston, Editor
GUIDE TO DESIGN CRITERIA FOR METAL COMPRESSION MEMBERS,
COLUMN RESEARCH COUNCIL, 2nd Edition
Wiley, 1966

19. Column Research Council
THE BASIC COLUMN FORMULA
CRC Tech. Memo. No. 1, May 1952

20. L. Tall, A. W. Huber, L. S. Beedle
RESIDUAL STRESS AND THE INSTABILITY OF AXIALLY LOADED COLUMNS
Also, International Institute of Welding Colloquium,
Liege, Belgium, June 1960, Commission X Document.

21. C. H. Yang, L. S. Beedle, B. G. Johnston
RESIDUAL STRESS AND THE YIELD STRENGTH OF STEEL BEAMS
Welding Journal, Vol. 31, April 1952
22. AISC
   SPECIFICATION FOR THE DESIGN, FABRICATION AND ERECTION
   OF STRUCTURAL STEEL FOR BUILDINGS
   AISC, 1961, revised 1963

23. D. Feder, G. C. Lee
   RESIDUAL STRESS AND THE STRENGTH OF MEMBERS OF HIGH-
   STRENGTH STEEL
   Fritz Laboratory Report 269.2, March 1959

24. L. S. Beedle, T. V. Galambos, L. Tall
   COLUMN STRENGTH OF CONSTRUCTIONAL STEELS
   Steel Design and Engineering Seminar, U. S. Steel
   Corp., Pittsburgh, May 1961

25. Y. Fujita
   BUILT-UP COLUMN STRENGTH
   Ph.D. Dissertation, Lehigh University, Aug. 1956

26. F. R. Estuar, L. Tall
   THE STRENGTH OF WELDED BUILT-UP COLUMNS
   Fritz Laboratory Report 249.16, in preparation

27. F. R. Estuar, L. Tall
   EXPERIMENTAL INVESTIGATION OF WELDED BUILT-UP COLUMNS
   Welding Journal, Vol. 42, April 1963

28. T. W. Wilder, W. A. Brooks, E. E. Mathauser
   THE EFFECT OF INITIAL CURVATURE ON THE STRENGTH OF
   AN INELASTIC COLUMN
   NACA Tech. Note No. 2872, Jan. 1953

29. A. W. Huber, R. L. Ketter
   THE INFLUENCE OF RESIDUAL STRESS ON THE CARRYING
   CAPACITY OF ECCENTRICALLY LOADED COLUMNS
   IABSE Publications, Zurich, Dec. 1958

30. A. Nitta
   ULTIMATE STRENGTH OF HIGH-STRENGTH STEEL CIRCULAR
   COLUMNS
   Ph.D. Dissertation, Lehigh University, Oct. 1960

31. J. H. Pielert
   THEORETICAL LOAD-DEFLECTION RELATIONSHIPS FOR WELDED
   BOX COLUMNS WITH INITIAL DEFORMATION
   M.S. Thesis, Lehigh University, Sept. 1965

32. B. G. Johnston
   BUCKLING BEHAVIOR ABOVE TANGENT MODULUS LOAD
33. L. Tall, F. R. Estuar  
DISCUSSION TO REF. 32  

34. F. Nishino  
BUCKLING STRENGTH OF COLUMNS AND THEIR COMPONENT PLATES  
Ph.D. Dissertation, Lehigh University, Oct. 1964

35. N. R. Nagaraja Rao  
THE STRENGTH OF HYBRID STEEL COLUMNS  
Ph.D. Dissertation, Lehigh University, May 1965

36. Y. Fujita  
ULTIMATE STRENGTH OF COLUMNS WITH RESIDUAL STRESSES  
Journal, Society of Naval Architects of Japan, Jan. 1960

37. A. W. Huber  
THE INFLUENCE OF RESIDUAL STRESS ON THE INSTABILITY OF COLUMNS  
Ph.D. Dissertation, Lehigh University, Aug. 1956

38. R. W. Frost, C. G. Schilling  
BEHAVIOR OF HYBRID BEAMS SUBJECTED TO STATIC LOADS  

39. Y. Ueda  
ELASTIC, ELASTIC-PLASTIC, AND PLASTIC BUCKLING OF PLATES WITH RESIDUAL STRESSES  
Ph.D. Dissertation, Lehigh University, Aug. 1962

40. Y. Ueda, L. Tall  
BUCKLING OF PLATES WITH RESIDUAL STRESSES  

41. F. Nishino, Y. Ueda, L. Tall  
EXPERIMENTAL INVESTIGATION OF THE BUCKLING OF PLATES WITH RESIDUAL STRESSES  
Fritz Laboratory Report 290.3, April 1966. To be published by ASTM.

42. F. Nishino, L. Tall  
RESIDUAL STRESSES AND THE LOCAL BUCKLING STRENGTH OF STEEL COLUMNS  
Fritz Laboratory Report 290.11, in preparation.