Short steel columns, 1938 Proposal

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Preliminary Study of Test Program - It is the purpose of this preliminary report to present to the A.I.S.C. Technical Research Committee further details in regard to the Short Steel Column program, together with questions which have arisen as a consequence of further study and thought given to this investigation. It is desired either to obtain written comment on these matters from individual members of the committee or to hold another meeting of the committee in the near future for the purpose of their discussion. In the meanwhile pilot tests will be proceeding on the first part of the program.

At the October 5th meeting of the Technical Research Committee the program for the Short Steel Column investigation was discussed and the general objectives of the investigation were placed under two headings.

1. Flange Crippling of Short Columns.
2. Strength of Short Columns with Varying Eccentricities of End Thrust.

Each of these programs will now be discussed in greater detail than at the October 5th meeting and the questions on which comment are desired will be numbered for reference purposes in consecutive order.

I. Flange Crippling of Short Columns - Theoretical solutions* for the elastic buckling of a thin plate with one side unsupported are already available for the two extreme

*Timoshenko, ELASTIC STABILITY, pp. 339 and 342
conditions shown in Fig. la, (supported side simply supported) and Fig. 1b (supported side fixed). The outstanding leg of one-half of the flange of a rolled column section (Fig. 1c) is in a condition somewhere between that of Fig. la and lb. Timoshenko also gives solutions for this case under the heading "One side elastically built in" (loc. cit. p. 344). It is sufficient for the present discussion to point out that even for the weakest condition as shown in Fig. la all commonly rolled shapes and A.R.E.A. specifications for built up columns give proportions for which the critical stresses are above the yield point even in the case of structural nickel steel. The 6 x 8 x 7/16-in. angle, with 8-in. legs outstanding is in a doubtful region and is the only exception.

Question 1. Is it desirable to study the crippling strength of outstanding legs of columns in such cases where the critical loads are above the yield point?

For all standard sections this problem would be one of plastic buckling. For short columns with l/r ≤ 50 the critical buckling stress of the column as a whole is approximately equal to the yield-point stress of the material. Since the strength against local crippling insofar as axial loads are concerned is already above the yield point a detailed study of this problem is of doubtful practical importance.

Question 2. Would it not be desirable to limit elastic buckling tests to a relatively small program to check Timoshenko's theory?
Close correlation with Timoshenko's theory has already been found for angle struts.*

Whatever the answers to Questions No. 1 and No. 2 may be, a very important practical problem remains. If localized bending stresses are present in one or both flanges of a column section, what then will be the axial load which will produce flange buckling? Such localized stresses are induced by clip and seat angles in a beam-to-column connection. The local buckling strength of the column flange under this combined stress condition may possibly be much lower than for the case of axial load alone. Very nearly the same problem occurs at the cutoff points of cover plates of the compression flange of a plate girder, due in this case to the eccentricity of load transfer. Little attention appears to have been given to a theoretical analysis of this very practical problem. It is proposed therefore that a series of tests be made using the setup shown in Fig. 3, and that this problem be given particular attention.

Question 3. Is a program of tests as shown in Fig. 3 advisable?

Proposed Program for Flange Crippling Tests - Part Ia

A series of ten tests of short H-beam section with flanges planed to varying thickness. Axial load will be applied along the lines indicated in Fig. 4a and 4b. The local buckling of the flanges is not a function of l/r of the whole

column but depends rather on the ratio of $b/h$ of the individual flange. (See Fig. 1 for notation).

**Test Program Ia**

**Material:** Structural Silicon Steel

10 sections of 10 x 10 at 49 lb. WF beam sections each 4 ft. 4 in. long with ends milled at right angles with the longitudinal axis. Plane the outside of the flanges to five different thicknesses as follows:

10 columns in all

- 2 with flanges planed to 0.20 in. thickness
- 2 with flanges planed to 0.235 in. thickness
- 2 with flanges planed to 0.27 in. thickness
- 2 with flanges planed to 0.305 in. thickness
- 2 with flanges planed to 0.34 in. thickness

**Test Program Ib**

Flange crippling tests on columns with loaded beam connections.

Three column sizes, probably:

- 10 x 10 WF at 77 lb per foot
- 10 x 10 WF at 49 lb per foot
- 10 x 10 WF at 49 lb with flanges planed to 0.34 in. thickness

Three types of connection as shown in Fig. 5a, b, and c.

Three tests on each of the above with different amounts of load on the cantilever arms.

The foregoing program lb calls for 27 tests in all, with exact details to be worked out later after comment by the committee.
II. Short Steel Column Program - At the October 5th meeting it was decided that Part II of the program would be a general study of short steel columns eccentrically loaded. The \( l/r \) ratio of these columns was to be in the 20-30-40 range.

The purpose of this work is primarily that of determining simple workable formulas for column design which are both safe and economical when used in the design of columns in building frames. It is assumed that the eccentricity of load acting on the end of the column is known. Methods for determining this eccentricity in any actual building column depend on many factors and are outside the scope of the present investigation. In laying out a program, however, it is necessary to give some thought to the structural behavior of a building frame. Considerable work along this line has already been accomplished by the British Steel Structures Research Committee* and their reports have been studied in detail. The last noted reference presents the final recommendations on the design of columns. The allowable stresses are presented in the form of a chart based on theoretical analyses which is presented here in Fig. 6, because of its usefulness in laying out a column program. If anything similar to Fig. 6 should result from the present investigation it seems likely that the results could be presented in the form of an empirical formula as suggested at the meeting of the committee. The chart in Fig. 6 is based on the worst

* First Report of the Steel Structures Research Committee pp. 211-224
Second Report of the Steel Structures Research Committee pp. 13-43
Final Report of the Steel Structures Research Committee pp. 436-545, pp. 559-565
probable condition which might arise in a building column regardless of whether the bonding is in the form of single or double curvature. (No reduction of \( l/b \) ratio of the compression flange has been made in this chart.)

The results of the British Steel Structures Research Committee show that the proportions of columns in buildings usually allow them to be designed with practically the same allowable stress as is used in beam design provided the columns are designed for known eccentricities of load. The \( l/r \) of building columns is large only in the upper stories of a building where the axial loads are small as compared with the bending moments. For such a condition, as for example, a ratio of direct stress to bending stress of 1:4, the allowable combined stress from Fig. 6 would be 17,500 p.s.i. for an \( l/r \) as high as 100, as compared with a basic allowable stress of 18,000 p.s.i. for short columns. When the axial load becomes the major factor, as in the lower stories of a building, the \( l/r \) of the columns decreases along with the critical value which allows design on the basis of such high working stresses. For 95 per cent axial stress the column may be designed for 17,000 p.s.i. maximum stress if the \( l/r \) is 30 or less.

It seems evident from Fig. 6 and from the foregoing discussion that for short columns in the 20-40 range of \( l/r \) as proposed for this investigation, failure will not result until the maximum stress in the column very nearly reaches the yield point of the material. The experimental check on
this question is one of the primary purposes of Part II of the short column program. It now seems desirable, in addition, to test columns of higher $l/r$—up to 100—for cases where the eccentricities of load are high. These tests would tie the program in with other work and permit much more general and complete conclusions to be reached. In general, the tests should cover the following range:

<table>
<thead>
<tr>
<th>Ratio of Average Axial Stress to Maximum Bending Stress</th>
<th>Range of $l/r$ to be Tested</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 1</td>
<td>100 - 140 (only 3 tests)</td>
</tr>
<tr>
<td>1 - 4</td>
<td>80 - 120</td>
</tr>
<tr>
<td>1 - 2</td>
<td>60 - 100 (above 100)</td>
</tr>
<tr>
<td>1 - 1</td>
<td>40 - 80</td>
</tr>
<tr>
<td>2 - 1</td>
<td>20 - 60</td>
</tr>
<tr>
<td>4 - 1</td>
<td>20 - 40</td>
</tr>
<tr>
<td>1 - 0</td>
<td>20 - 40</td>
</tr>
</tbody>
</table>

**Question 4.** Is this modification of the program desirable?

The method of testing is another problem concerned with Part II of this program. In a building, if the beams framing into a column are symmetrically loaded so as to introduce no eccentricity of load into the column, the beams will then provide a restraining action at the column ends. In this case a reduction of $l/r$ would be permissible in calculating critical loads. On the other hand, if the beams introduce bending into the column, the tendency for buckling will be increased.

Three different methods of testing columns with combined thrust and moment will now be discussed.
Method 1 - The column is loaded axially in the testing machine with varying eccentricities of load and the buckling or yielding strength determined. (See Fig. 7a.)

Advantages: 1. Simple fabrication of test columns.
2. Simple test procedure.

Disadvantage: 1. Eccentricities limited to core section of column, unless special provision is provided to develop tension between one side of column and testing machine head.

Method 2 - The axial load would in each test be applied in line with the centroidal axes of the column. End moments would be introduced through riveted beam-to-column connections with detachable loading beams as shown in Fig. 7b.

During the test definite end moments could be applied to the column after which axial load would be applied until the column yielded and failed.

Advantages: 1. Applied end moments accurately known and column always located in center of testing machine.
2. Large range of equivalent eccentricity easily obtained.
3. Actual conditions closely simulated.

Disadvantages: 1. More expensive and complicated test set-up than Method 1.
2. Equivalent eccentricity of axial load decreases during test, but can be calculated readily for any given load.
Method 3 - Column tested as part of a frame as shown in Fig. 7c. End moments developed in column by tightening turnbuckle and loading spring.

During test definite end moments would be developed after which axial load would be applied until the column yielded and failed.

Advantages: 1. A still closer simulation of actual conditions in a building frame.
2. Large range of eccentricity easily obtained.
3. Set-up is simpler than Method 2.

Disadvantages: 1. Introduces a new variable.
2. Calculation of end moments is a statically indeterminate problem.

SUMMARY OF PROPOSED PROGRAM

Part I - Flange Crippling.

a. Pilot tests on 10 x 10-in. WF short columns at 49 lb. to try out test methods.

b. Test series Ia - 10 short columns with flanges planed to variable thickness.

c. Test series Ib - 27 short columns with loaded beam connections. Three flange thicknesses, three types of connections, three different beam loads.

d. Theoretical analysis of flange crippling problem, correlation with test results, and design recommendations.
Part II - Columns With End Moments.

a. About 6 columns with zero eccentricity of load and variable $l/r$ (20-40) tested by Method 1.

b. Between 20 and 30 columns tested by Method 2 with varying $l/r$ and varying ratio of end moment to axial load. Details to be decided on after tests in Part I are finished.

c. Study of results and presentation of design methods.

The preceding program may exceed slightly the limitation of time and money available but is proposed in this form so that alterations may be made in line with recommendations of the committee.
APPENDIX A

The material contained in this appendix consists of the comment advanced by the members of the Structural Research Committee of the American Institute of Steel Construction after their review of the foregoing report.

It does not seem desirable to go farther with a study of the crippling strength of outstanding legs of columns except through a relatively small program to check Timoshenko's theory. Test Program Ia is the part of the program offered for this topic. It apparently studies width to thickness ratios of flange from 15 to 25 within which range it should be possible to make the flange cripple before the yield point of the column is reached. It seems this is worth doing. It not only would exhibit the sufficiency of the A.R.E.A. rules as plotted on Fig. 2, but it would also give a designer not governed by standard specifications some leeway in matters of design which might cause him some consternation if he were not aware that it would be possible to deviate from the usual flange thickness rules in case of necessity.

It seems very important to test columns not only under idealized conditions covered by Fig. 2, where the only load on the flange is the axial load, but also for actual conditions in a building, where portions of the
flange are under transverse moments large in comparison with axial stresses. Program Ib is directed toward this topic and additional suggestions and criticism concerning the types of connections are needed.

The investigation of combined axial load and bending in the range indicated by the table on page 7 appears to be justified in view of the evidence presented. This will dispose directly of Section 6(a) of the A.I.S.C. specification which is an admitted rule of thumb and frequently criticized.

As to the testing methods indicated on Fig. 7a, 7b, 7c, it seems the most desirable would be 7c cutting off the two beams to the right of the spring connection and omitting the rest of the beams and auxiliary columns. This would make the end moments determinate instead of indeterminate. In this arrangement, however, a problem arises. As the axial load is increased the column will gradually bend prior to buckling and the load in the spring will decrease as a result of the bending. The applied moment, therefore, will likewise be decreased. Just how serious a matter this would be in an actual test is a problem which will require further study. This is a detail which may be settled later following additional and more widespread comment.
RELATION BETWEEN CRITICAL BUCKLING STRESS AND WIDTH-THICKNESS RATIO OF THE OUTSTANDING LEG

Figure - 2
Figure - 3  Testing Arrangement for Program - I (b)
Figure -5 BEAM TO COLUMN CONNECTIONS - PROGRAM - I (b)
Figure 6. Allowable bending stress in building columns (British Steel Structures Research Committee.)