Fritz Laboratory Reports

Civil and Environmental Engineering

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Short steel columns, 1938 Proposal

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SHORT STEEL COLUMNS

by
Cherry

Preliminary Study of Test Program - It is the purpose of
this preliminary report to present to the A.I.S.C. Tech-
nical 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 ob-
tain 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 proceed-
ing on the first part of the program.

At the October 5th meeting of the Technical Research
Committee the program for the Short Steel Column investiga-
tion was discussed and the general objectives of the inves-
tigation 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 solu-
tions* for the elastic buckling of a thin plate with one
side unsupported are already available for the two extreme

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* Timoshenko, ELASTIC STABILITY, pp. 339 and 342

conditions shown in Fig. 1a, (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. 1a and 1b. 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. 1a 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

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* SOME NEW EXPERIMENTS ON BUCKLING OF THIN WALL CONSTRUCTION by Bridget, Jerome, and Vosseller - Transactions, Am. Soc. Mech.Engr. Applied Mechanics Division, Vol.65, p.569, 1934

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:

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          (2 with flanges planed to 0.20  in. thickness
          (
          (2 with flanges planed to 0.235 in. thickness
   10     (
 columns  (2 with flanges planed to 0.27  in. thickness
 in  all  (
          (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 1b calls for 27 tests in all, with exact details to be worked out later after comment by the committee.

II. <u>Short Steel Column Program</u> - 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 determ-
ining 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 pre-
sent 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 recom-
mendations 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 sim-
ilar 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

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* 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 bending is in the form of single
or double curvature. (No reduction of l/b ratio of the com-
pression 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 ex-
ample, 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 allow-
able 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 crit-
ical value which allows design on the basis of such high work-
ing 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 addi-
tion, 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:

Ratio of Average Axial Stress to Maximum Bending Stress	Range of l/r to be Tested
0 - 1	100 - 140 -(only 3
1 - 4	80 - 120 (tests
1 - 2	60 - 100 (above 100
1 - 1	40 - 80
2 - 1	20 - 60
4 - 1	20 - 40
1 - 0	20 - 40

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 in-
troduce 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 cal-
culating 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 com-
bined 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 sec-

                 tion of column, unless special pro-

                 vision is provided to develop ten-

                 sion 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 connect-
ions 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 eccentri-

                 city 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 mat-
ters of design which might cause him some consternation
if he were not aware that it would be possible to devi-
ate 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.
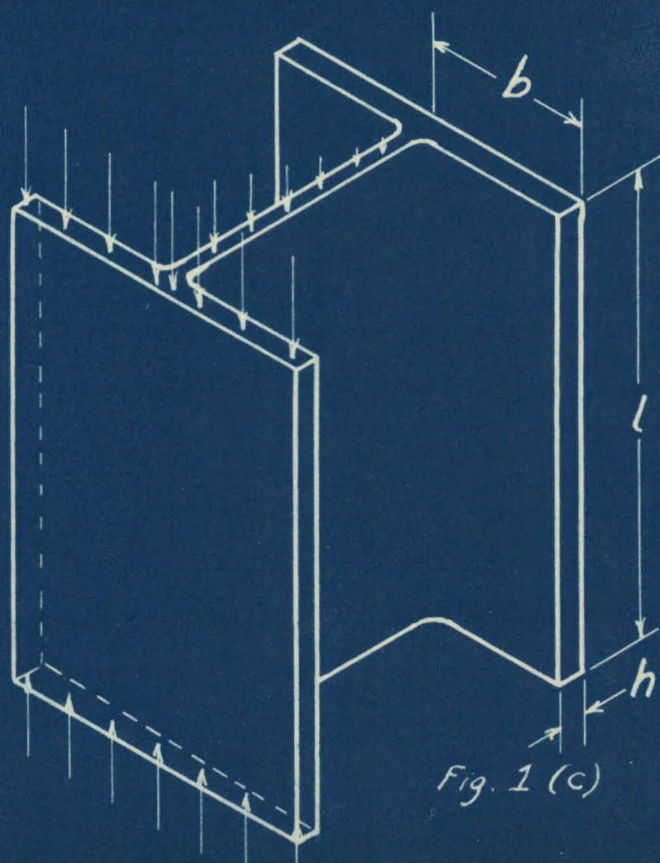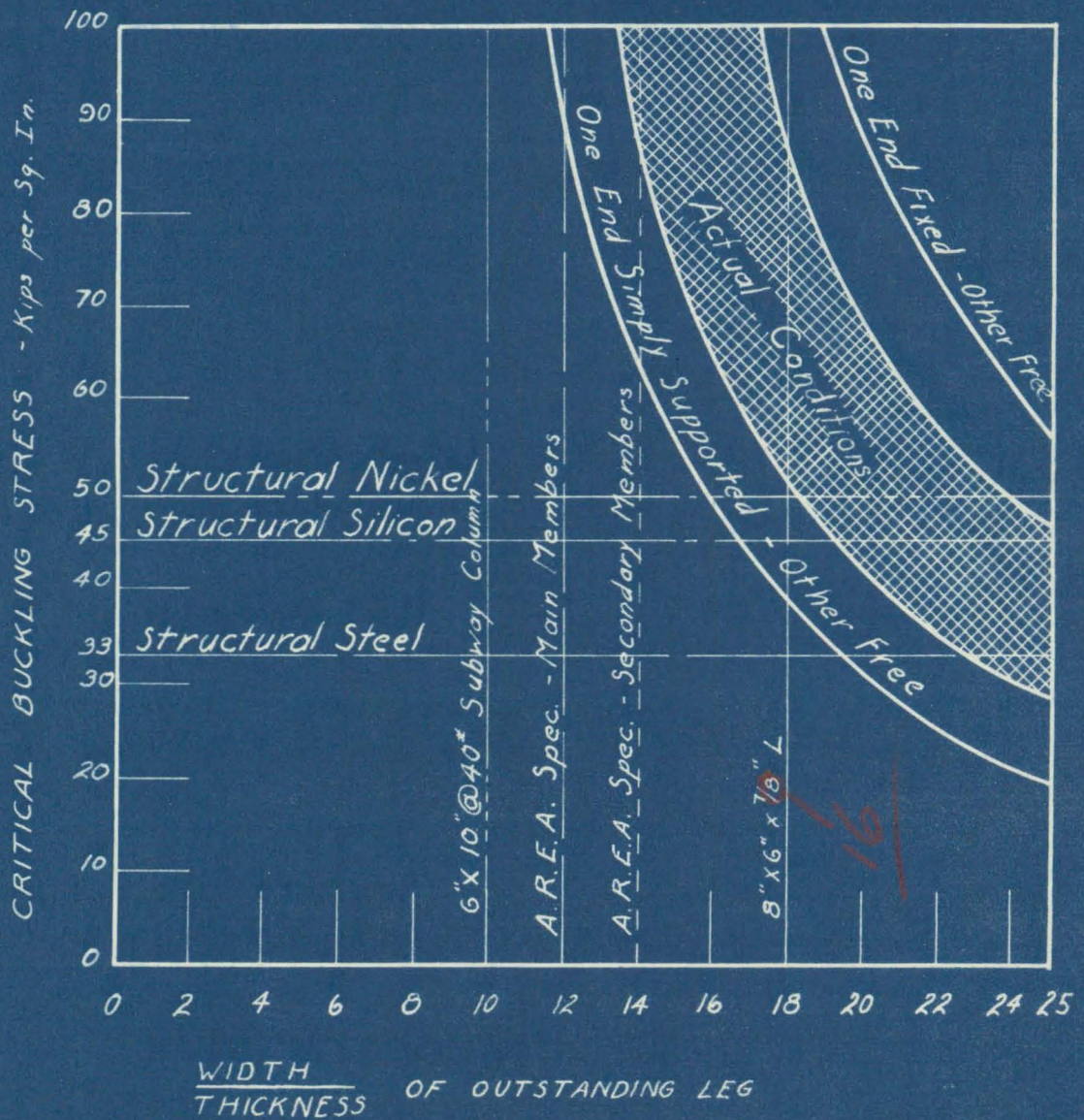
Fig. 1(a)

Simple
Support

b

l

h

Built
In

b

l

h

Fig. 1(b)

b

l

h
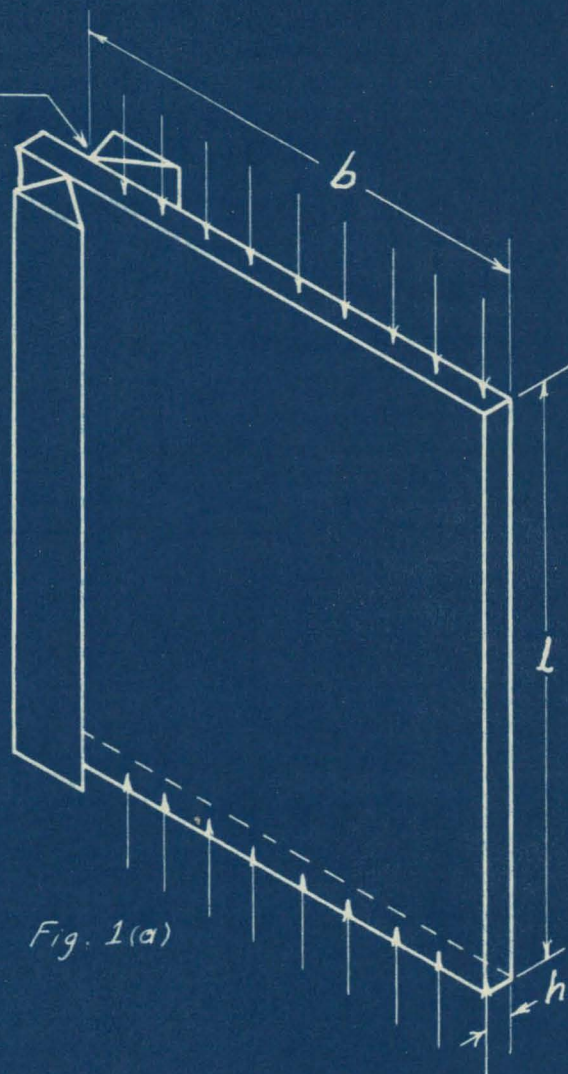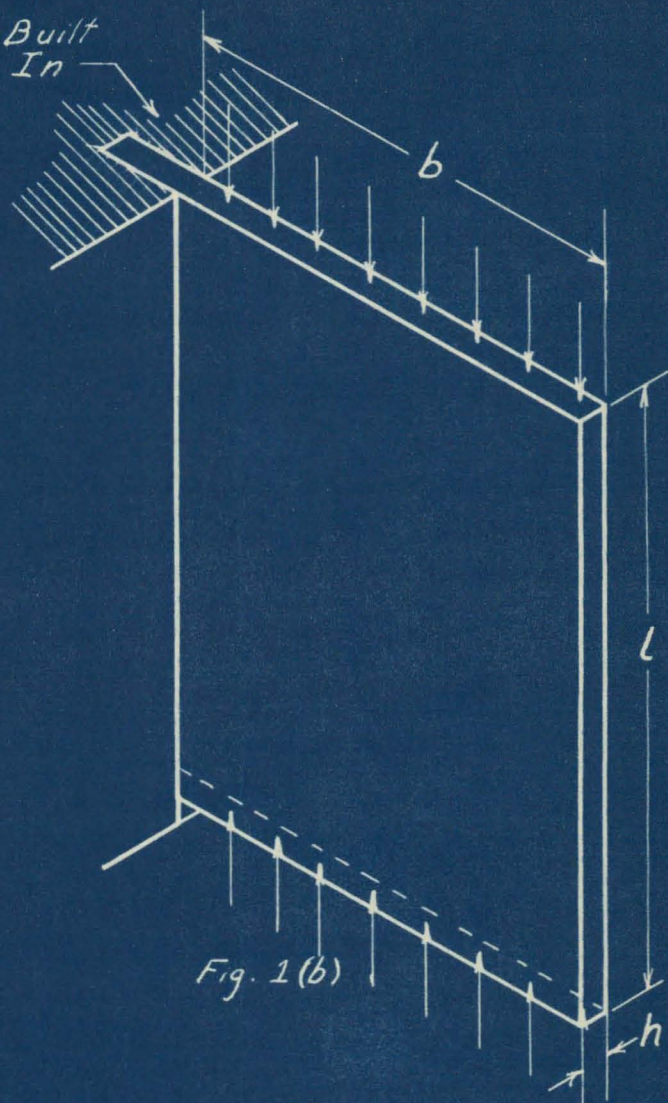
Fig. 1 (c)

RELATION BETWEEN CRITICAL BUCKLING STRESS
AND WIDTH-THICKNESS RATIO OF THE
OUTSTANDING LEG

Figure - 2

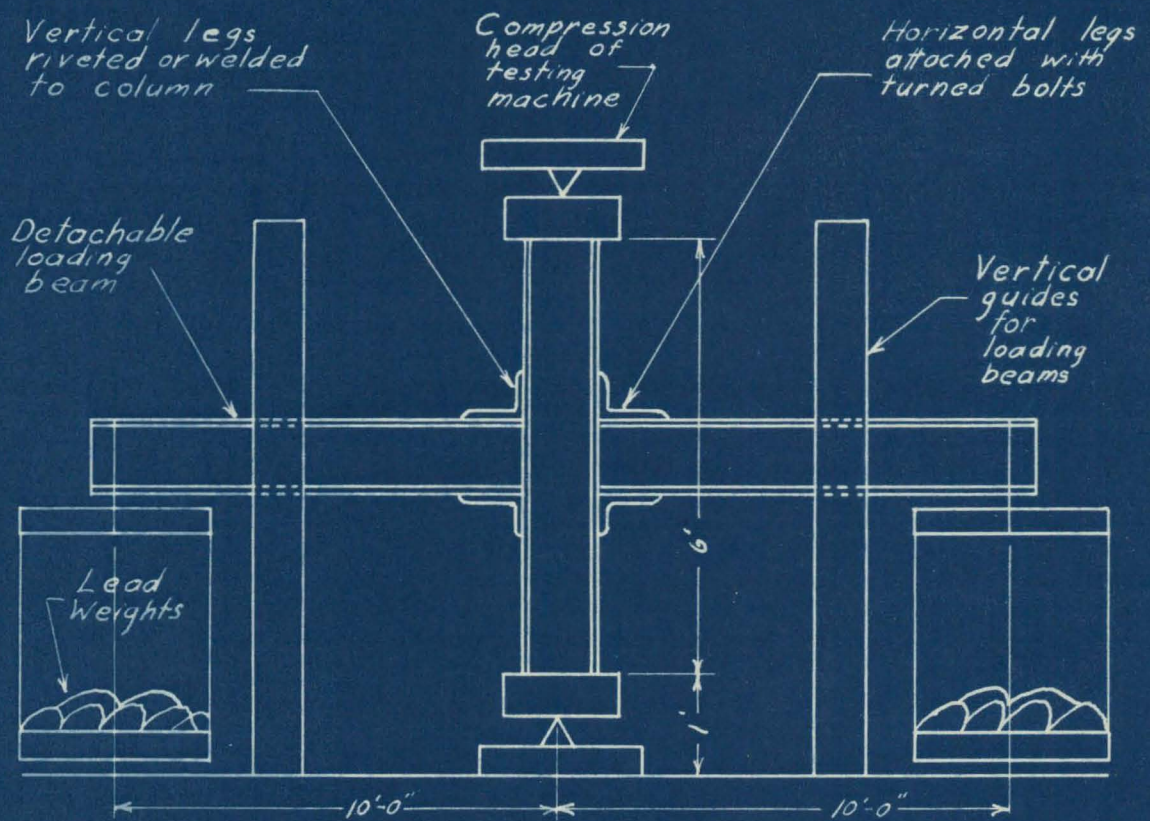Vertical legs riveted or welded to column

Compression head of testing machine

Horizontal legs attached with turned bolts

Detachable loading beam

Vertical guides for loading beams

Lead Weights

6'

1'

10'-0"

10'-0"

FIGURE - 3   TESTING ARRANGEMENT FOR PROGRAM - I (b)

$\dfrac{2\,r_{xx}^2}{d}$

X

Application of Load

X

d

Fig. 4 (a)

$\dfrac{2\,r_{yy}^2}{w}$

Application of Load

Y

Y

W
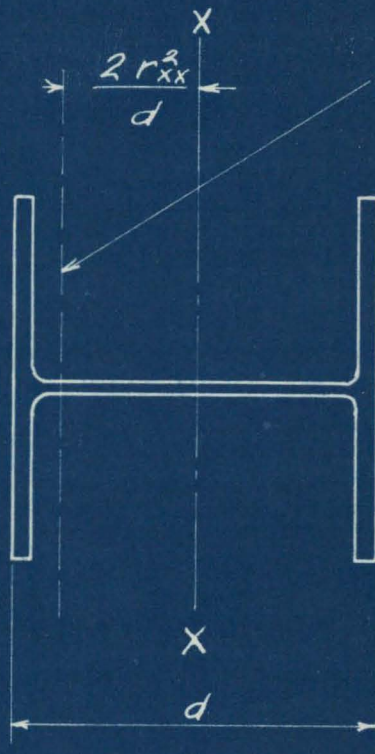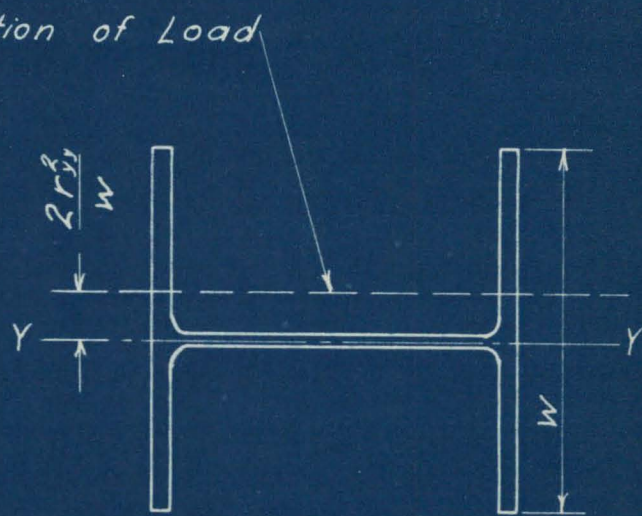
Fig. 4 (b)
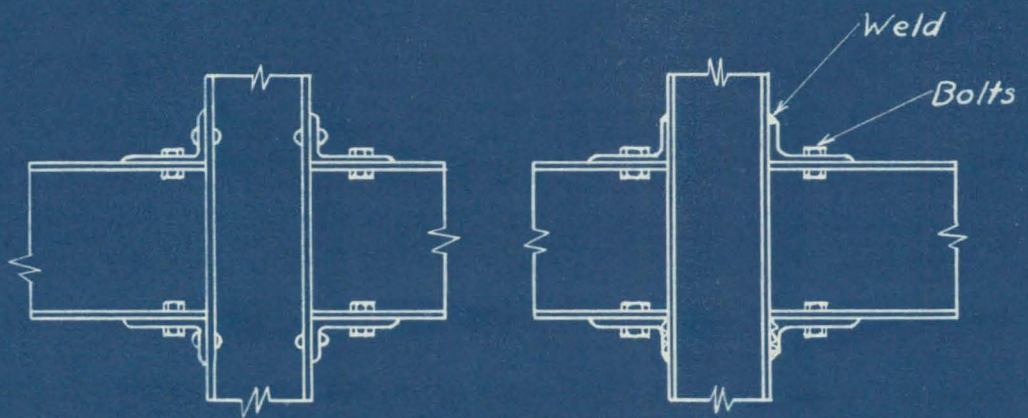
Fig. 5 (a)   Fig. 5 (b)
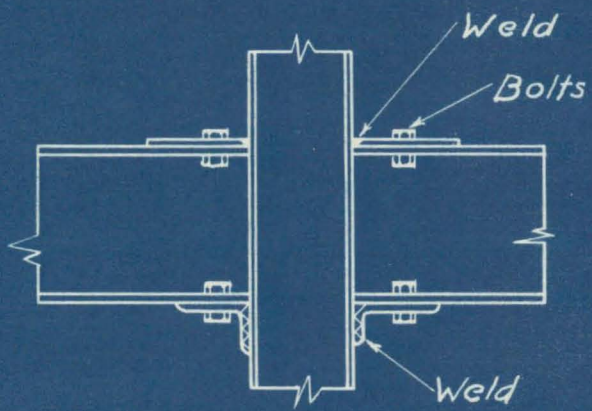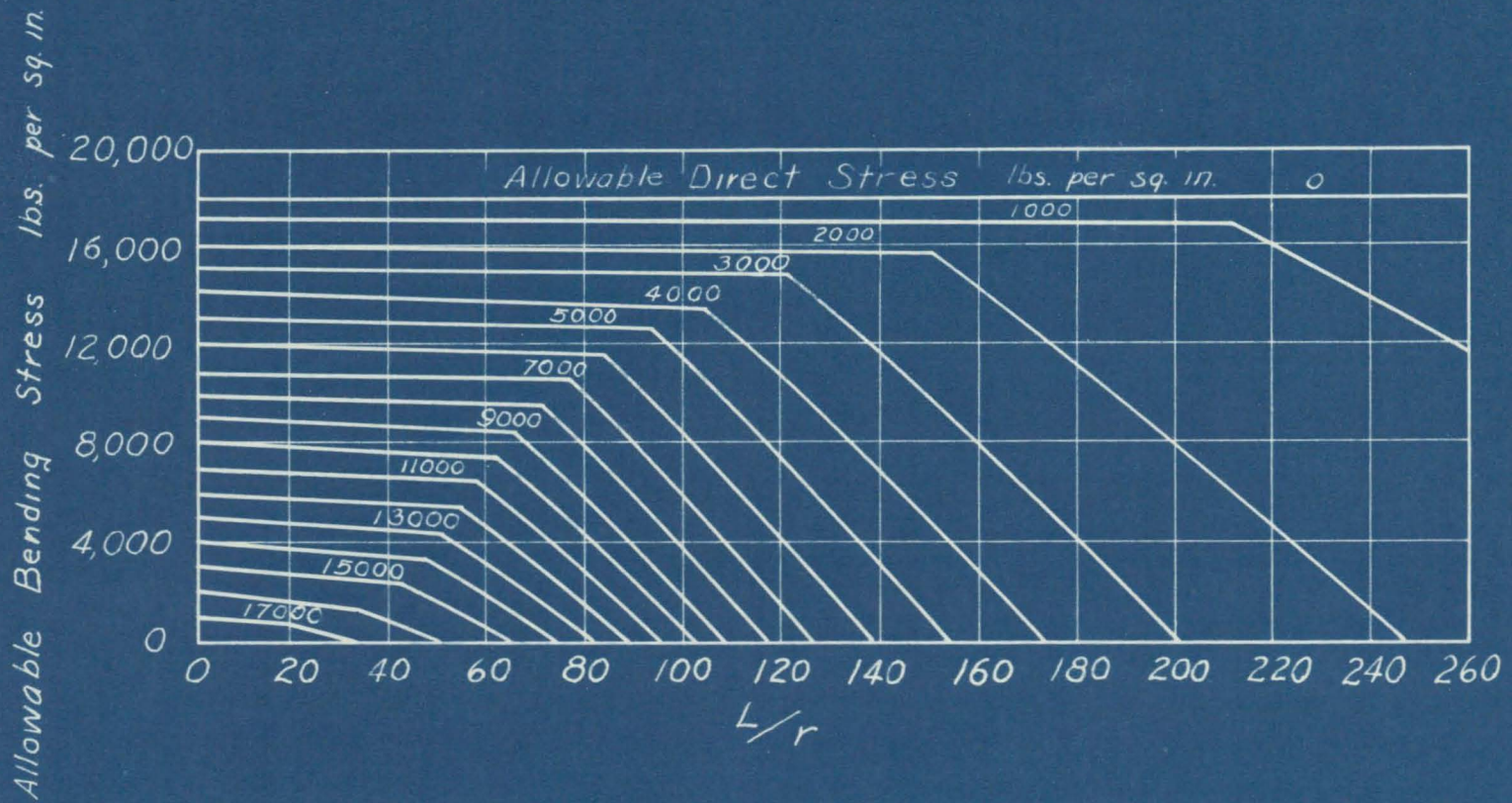
Fig. 5 (c)

Figure -5   BEAM TO COLUMN
CONNECTIONS - PROGRAM - I (b)

FIGURE 6   ALLOWABLE BENDING STRESS
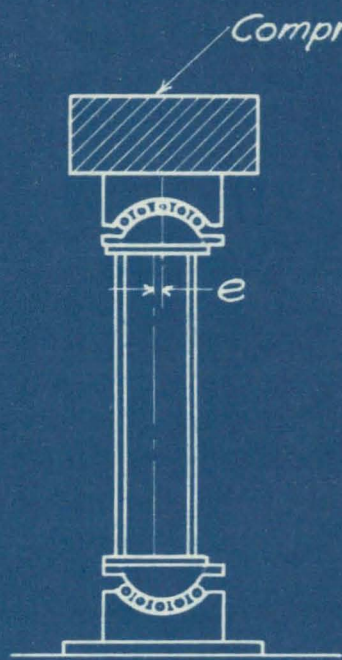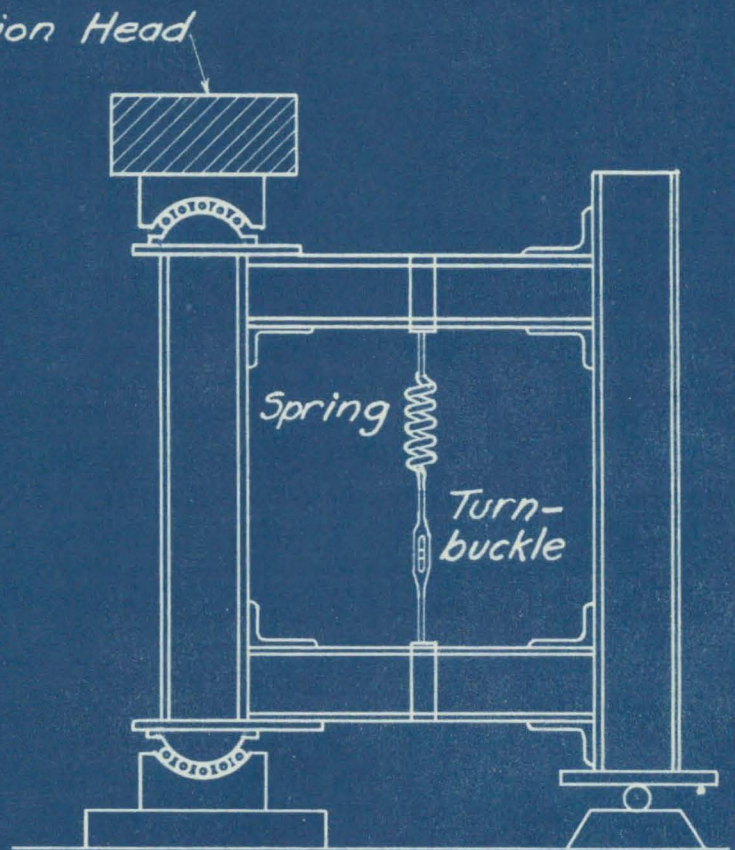IN BUILDING COLUMNS (BRITISH
STEEL STRUCTURES RESEARCH COMMITTEE.)

Compression Head

e

Fig. 7a.

Compression Head

Spring

Turn-buckle

Fig. 7c.

Compression Head

Detachable
Loading
Beams

Guides

Lead
Weights

Fig. 7b