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Tests on light beams of cold formed steel for the American Iron and Steel Institute

American Iron and Steel Institute

SCHOOL OF CIVIL ENGINEERING, CORNELL UNIVERSITY

TESTS ON LIGHT BEAMS OF COLD FORMED STEEL

FOR THE AMERICAN IRON AND STEEL INSTITUTE

Seventeenth Progress Report - March 25, 1941

I. SCOPE OF THIS REPORT

Failure tests on two specimens each of beams type A, B, C, D, E, F, G, have been carried out. The lengthy final evaluation of the results will be given in a later report. However, the results of the tests reported herein allow the drawing of rather definite conclusions.

II. FAILURE TESTS OF BEAMS A TO G INCL.

(a) Method of Testing

Two beams of each type were loaded by two equal loads symmetrically situated with respect to the center of the beam, each load being applied at a distance of 3 ft from the support. The beams were laterally supported at the load points and at the supports by means of the frames described in the Sixteenth Report, thereby, lateral instability was prevented and the beams were forced to fail either by local buckling or by simple yielding in the portion of the beam between the load points. Two Huggenberger gages were mounted on each half of the top flange between the load points and deflection observations were made in addition to the strain measurements.

(b) Results

Only the final evaluation of the strain readings will reveal the loads at which local buckling began. At the present time, therefore, only the ultimate load, the ultimate stress, and the particular form of failure of each of the beams can be reported on. This information is summarized in the following table.

TABLE

Failure tests on two specimens each of beams type A,B,C,D,E,F,G.

Beam	Gage	Ult. load	Nominal ult. stress	Type of failure	Design stress
A 1	14	13.000	29.500	welds	25.000
A 2	14	14.800	33.600	local buckling	25.000
B 1	14	14.900	43.900	yielding	yield stress
B 2	14	12.500	35.400	welds	yield stress
C 1	16	9.800	24.100	local buckling	10.000
C 2	16	9.475	23.300	local buckling	10.000
D 1	16	8.500	27.200	local buckling	25.000
D 2	16	8.935	29.500	local buckling	25.000
E 1	16	8.000	35.100	yielding	yield stress
E 2	16	8.090	35.500	yielding	yield stress
F 1	18	5.030	20.800	local buckling	10.000
F 2	18	4.990	20.600	local buckling	10.000
G 1	18	5.410	31.200	local buckling	25.000
G 2	18	5.475	31.600	local buckling	25.000

Special Observations:

(a) Beams A 1 and B 2 did not fail in the top flange, but failed in the welds joining the bottom flange to the web. In both cases 11 (beam A 1) and 15 (beam B 2) welds in a row on one side of the web between the load and the support broke almost simultaneously. In order to avoid a similar failure of beam A 2, the bottom flange of this beam was tack welded on both sides to the turned over parts of the web. In addition, four welds on the top flange broke on beam D 1 close the load point on the side toward the support. This fact may be responsible for the somewhat lower strength of D 1 as compared with D 2.

(b) Shear buckling of the web between the loads and the supports was observed on both specimens of types F and G. On F 2 the shear waves were first noticed at P = 3400 lb, on beam G 2 at P = 3500 lb. The waves formed rather suddenly at these loads. On beams F 1 and G 1 the first formation of these waves escaped observation and they were noticed only at higher loads.

(c) In no case was failure caused by premature buckling of the lips of the top flanges. These lips remained essentially straight until failure developed in the horizontal portion of the top flange

(d) Approximately quadratic waves were observed on the top flanges of all beams which failed by local buckling. These waves formed at loads considerably below the ultimate, but in all cases this distortion was on a very small scale until immediately before failure.

(e) No such waves were observed on those beams which failed by yielding; the shape of these beams remained practically undisturbed up to the failure load. At this load the top flanges of these beams were distorted in a rather irregular manner in no way resembling the distinctly local buckling waves observed on the other beams.

(c) Discussion

(1) It is seen from the tabulated results of the present tests that the nominal ultimate stresses range from about 20,000 p.s.i. up to the yield point. Consequently, the main purpose of this series of tests has been achieved, viz., information has been obtained on the relation of the ultimate stress to the dimensions of the top flange and on the necessary dimensions of the top flange in order that buckling be avoided and that the beam fail in yielding.

(2) The results of the tests of identical specimens of the same type are in excellent agreement, except for beams A 1 and B 2, which failed in the welds.

(3) The beams which failed by local buckling, did so at a stress considerably above their design stress and also at a load considerably higher than that at which distinct buckling waves were observed. This result is in agreement with observations reported in the Twelfth Report. It is further seen that the ultimate stresses are by no means proportional to $(t/f)^2$ as should be expected from the Timoshenko theory. A still more striking fact is the following: It is seen from the table that, except for A 1 and B 2 where the welds failed, the ultimate load of all beams of the same gage is about the same, regardless of the dimensions of the top flange. This behavior suggests the thought that for each particular thickness there is a definite optimum width which is that maximum width at which yielding without buckling occurs. Any increase in width beyond this limit will but little affect the ultimate load of the beam. In addition it is seen that the ultimate loads are approximately proportional

to the squares of the flange thickness, regardless of the widths. It seem, therefore, at the present writing that the Timoshenko theory of plate buckling will have to be discarded as a basis for design specifications for thin wall beams.

(4) A study of the literature revealed that this general behavior of thin sheets in compression is well known and deliberately made use of in the airplane industry. In particular it is well known that the ultimate load of thin sheets is considerably and, for very thin sheets, many times larger than that load at which the first local buckling occurs. A sheet with considerable buckling waves will still safely carry its load and return to its flat shape when unloaded (so called oil-canning on airplane wings, etc., in service). The explanation of the discrepancy between the buckling theory and results of thin sheet tests is to be seen in the fact, that the buckling theory assumes that deflections, which are small as compared with the thickness of the sheet, are sufficient to cause failure. This is true for fairly thick plates, but does not hold for thin sheets, in which a buckling wave, the amplitude of which is many times larger than the sheet thickness, will not necessarily cause yielding of the outer fibers. This is exactly what happens in thin flanges. This phenomenon of "large buckling" to date has resisted all attempts of rigid mathematical treatment and the chances for a practicable exact mathematical solution are practically nil. V. Karman has given a semiempirical analysis of this behavior (Trans. A.S.M.E., vol. 54,

p. 53, 1932) which is well confirmed experimentally and widely used in airplane design. The results of the present tests are in general in agreement with this approach; in particular, the constancy of the ultimate load for each particular thickness, regardless of the width, is exactly predictable from V. Karman's treatment. It is, therefore, to be hoped that on the basis of this theory and of the present tests, supplemented by results published in the airplane literature, a simple design formula will be evolved which will be different from formulae based on Timoshenko's theory.

(5) As mentioned before, the lips in no case were the reason for premature failure of the top flanges. The present tests, therefore, indicate that lips proportioned according to the principles stated in the Thirteenth Report, provide sufficient support for the flange without failing themselves. The present tests, however, do not give any indication whether lips of smaller or larger depth than the ones tested in this program would give the same results. In other words, a safe way of designing the lip has been established; it is, however, not proved that a wider range of allowable lip widths is not possible.

(6) Shear buckling in the webs of the 18 ga beams has again been observed. As in previous tests (see Twelfth Report) these waves did not impair the strength of the beam, i.e., the beam ultimately failed in the top flange at a load far greater than that at which the first web buckling was observed. This behavior, too, is well known in other structural fields and its evaluation in code form should not make much difficulty. Additional

tests on the cut-off outer portions of the present beams should shed more light on this question. These tests are also designed to give information on the necessity of using web stiffeners under concentrated loads. It is planned to carry out this investigation after the main tests on the present series have been finished.

(7) The welds of beams A 1 and B 2 failed in simple shear. The fact that no such failure occurred in B 1 shows that considerable non-uniformity is encountered in the present spot welds. This behavior points to the necessity for obtaining additional information on the strength of spot welds in order to make possible a safe design of composite beams of the present or similar types.

(8) It will be noticed that in the present beams, as well as in those discussed in the Sixteenth Report, yield point stresses have been obtained on all specimens which were supposed to develop such strength. No such stresses have been obtained on the beams reported in the Twelfth Report. The main difference between the present set-up and the one formerly used consists, as pointed out before, in the use of laterally restraining frames. These frames serve the same purpose as bracing would do in a structure. Therefore, the considerably greater strength developed in laterally supported beams points to the great importance of proper lateral bracing in thin wall structures.

III. SUMMARY

(1) Tests to investigate local buckling of the top flanges have been carried out on two specimens each of beams A to G.

(2) The ultimate stresses of these beams ranged from about 20.000 p.s.i. up to the yield point.

(3) It is believed that the detailed evaluation of these test data will yield sufficient information for design specifications with respect to the strength of compression flanges.

(4) A preliminary analysis of the results seems to show that such design formulae will be based on an approach to this question widely used in the airplane industry rather than on Timoshenko's theory of plate buckling.

(5) Shear buckling in the webs was observed in all 18 ga specimens.

(6) Weld failure occurred in two of the 14 ga beams. This occurrence again points to the necessity of obtaining additional information on the strength of spot welds.

(7) The present tests as well as those discussed in the Sixteenth Report point to the great practical importance of proper bracing of thin wall structures.