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Cornell University School of Civil Engineering Tests on light beams of cold-formed steel

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SCHOOL OF CIVIL ENGINEERING, CORNELL UNIVERSITY

TESTS ON LIGHT BEAMS OF COLD FORMED STEEL

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I. SCOPE OF THIS REPORT

This report contains the complete evaluation of test results of the I-beam tests reported on in the 33rd and 34th Beam Reports. In considering the results of this evaluation, two factors should be kept in mind which adversely affect the accuracy of results: (1) These beams were designed primarily to serve as control specimens for future channel tests. Their usefulness in verifying the equivalent width approach is somewhat impaired by the fact that the compression flanges represent only a rather small part of the total cross-sectional area. Thus, a moderate change of equivalent width of the top flange will affect the properties of the entire cross-section (moment of inertia, section modulus, position of neutral axis) only to a very small degree. Since it is these properties which govern the quantities measured in the tests (strains, deflections, ultimate loads), the accuracy of determining equivalent widths is much smaller than the experimental measuring accuracy. (2) As pointed out on p. 2 of the 34th Report, the values of the yield points of different specimens cut from the same steel showed very considerable variations. Therefore, evaluations based on the yield point are adversely affected by this variation of mechanical properties.

Despite these inevitable **limitations** the evaluation of the results of these tests is of considerable value. Previous investigations of the equivalent width were confined to the range of b/t from about 50 up to about 200. The present investigation covers the range from b/t = 14 to 56 and therefore fills in a gap in the low range. To be sure, exact values in this range are not as important as in the high range, since for such small b/t the reduction of the actual to the equivalent width is rather small and therefore the design moments of inertia and section moduli are not appreciably different from those computed for the full width.

II. DIMENSION OF BEAM SPECIMENS

The general shape of the cross-section of all beams is shown on drawing No. 323. Dimensions of individual types of beams (average of two specimens) are given in table 1 below.

Table I

Average Dimensions of Specimens

Type	t	b_1	b_2	a_1	a_2	h
4 - 2 - 16	.0599	3.95	3.93	0.56	0.53	3.97
4-2 1/2-16	.0603	5.11	5.09	0.74	0.78	3.98
4-2 1/2-12	.1070	4.97	4.97	0.75	0.74	3.94
4-2 1/2-10	.1342	4.89	4.92	0.72	0.71	3.96
8 - 3 - 15	.0647	6.04	6.04	0.80	0.80	7.98
6 - 3 - 12	.1082	6.03	6.00	0.80	0.77	5.92
8 - 4 - 15	.0664	8.03	8.05	0.84	0.83	7.99
6 - 3 - 9	.1478	6.01	6.00	0.68	0.68	5.95
8 - 3 - 12	.1091	5.98	6.02	0.80	0.75	7.94
8 - 3 - 9	.1473	6.05	6.01	0.66	0.65	7.97
8 - 4 - 12	.1109	7.95	8.01	0.79	0.77	7.95
8 - 4 - 9	.1471	7.96	7.88	0.74	0.76	7.91
6 - 3 - ~~12~~ 16	.0609	5.96	5.99	0.79	0.80	6.10

All beams were tested on a 138 in. span with two equal loads placed each at a distance of 56 in. from the respective supports.

III. EXPERIMENTAL DETERMINATION OF EQUIVALENT WIDTH

The equivalent widths of the top flanges were determined from the strain measurements in the same manner as explained in the 3rd Summary Report on Beams, p. 4. Strain readings obtained at approximately 75% of the ultimate load were used for this purpose in order to stay within the straight line portion of the stress-strain curve. In the following table 2 the ratios of the equivalent to the actual width obtained experimentally are compared with those obtained from the design chart A of the "Addendum to the 3rd Summary Report on Beams and 2nd Summary Report on Studs", Sept. 1943. Table 2 contains average values for the two beams tested of each type.

Table 2

Ratios R of Equivalent to Actual Width

Type	P	s	b/t	R_{act}	R_{des}	R_{act}/R_{des}
4 - 2 - 16	1300	27,400	28.9	.95	.92	1.03
4-2 1/2-16	1500	25,600	38.3	.97	.86	1.13
4-2 1/2-12	2700	27,600	19.2	1.00	.98	1.02
4-2 1/2-10	3100	26,300	14.3	.98	1.00	.98
8 - 3 - 15	4750	27,400	42.6	.93	.83	1.12
6 - 3 - 12	5500	29,200	24.0	.94	.94	1.00
8 - 4 - 15	5750	28,000	56.0	.87	.74	1.17
6 - 3 - 9	7000	27,200	16.3	1.01	1.00	1.01
8 - 3 - 12	7500	26,300	23.6	.99	.96	1.03
8 - 4 - 9	10000	26,600	16.4	1.01	1.00	1.01
8 - 4 - 12	9500	27,200	32.0	.99	.90	1.10
8 - 3 - 9	12000	27,200	22.9	.95	.95	1.00
6 - 3 - 16	2600	23,500	45.0	.91	.83	1.10

In this table

P = Load at which strain observation was made

s = corresponding compression stress computed from reduced moment of inertia

R_{act} = ratio of equivalent to actual width as determined from strain measurements

R_{des} = design ratio of equivalent to actual width as determined from the quoted design chart A for stress s.

From the last column in table 2 it is seen that the agreement between computed and measured values is fairly good. The average ratio of R_{act}/R_{des} as computed from this column is 1.05, with a deviation from the mean of + 11%, - 7%. All experimental values but one are seen to be on the conservative side, i.e. the experimental equivalent widths exceed or are equal to the design equivalent widths.

In judging the accuracy of these results, point (1) of Section I of this Report should be kept in mind. In fact, the geometry of the sections of these beams is such that a deviation of a fraction of one percent in the calibration constants of the strain gages would account for a very considerable change in R_{act}/R_{des}, up to 5% and more. Although the instruments were calibrated as carefully as possible, an accuracy of a fraction of a percent cannot be guaranteed with the type of gage used.

Despite this inevitable limitation of the accuracy of these test results it is believed that the following conclusions can be drawn from table 2:

(a) In drawing design chart A the point for which R = 1 was rather arbitrarily chosen at b/t = 15, since no tests in that low range were then available. From the test results it is seen that for flanges with b/t up to 19.2 R = 1. Therefore the chosen limiting ratio b/t = 15 is obviously safe and conservative.

(b) For flanges with b/t larger than about 20, R_{act} was found to be smaller than 1 (one) as expected, i.e. a decrease in equivalent width was clearly established experimentally. This decrease, however, in all cases but two (6-3-12 and 8-4-9) was smaller than expected. Since the amount of deviation of the experimental from the design values is irregular and always on the conservative side, and the experimental accuracy is limited as explained before, it is believed that the design chart A should not be changed on the basis of these tests and that it can be regarded as sufficiently confirmed in the range of b/t = 14 to 55 to be recommended for safe use. It will be recalled that this is the range which heretofore had not been investigated experimentally.

IV. DEFLECTIONS

Table 3 below contains the measured deflections, the deflections computed from the full, unreduced moments of inertia and the deflections computed from the moments of inertia determined from the equivalent widths.

Table 3

Measured and Computed Deflections

Beam	P	P'	d_{act}	d_{full}	d_{eq}	d_{full}/d_{act}	d_{eq}/d_{act}
4 - 2 - 16	1100	900	.630	.582	.609	.925	.966
	1100	900	.660	.582	.609	.882	.923
4-2 1/2-16	1400	1200	.705	.636	.673	.903	.955
	1400	1200	.705	.636	.673	.903	.955
4-2 1/2-12	3300	3000	1.020	.974	.983	.954	.964
	3300	3000	1.010	.974	.983	.964	.974
4-2 1/2-10	3500	3200	.935	.857	.857	.916	.916
	3500	3200	.945	.857	.857	.907	.907
8 - 3 - 15	4750	4250	.420	.388	.410	.925	.975
	5250	4750	.455	.433	.457	.952	1.000
6 - 3 - 12	6000	5500	.665	.612	.622	.921	.935
	5500	5000	.650	.556	.566	.855	.870
8 - 4 - 15	6000	5500	.435	.407	.447	.935	1.030
	6000	5500	.435	.407	.447	.935	1.030
6 - 3 - 9	7500	7000	.665	.582	.583	.875	.877
	7500	7000	.705	.582	.583	.825	.827
8 - 3 - 12	9000	8500	.490	.484	.496	.987	1.010
	9000	8500	.485	.484	.496	1.000	1.020
8 - 3 - 9	11000	10000	.470	.431	.432	.917	.919
	11000	10000	.460	.431	.432	.936	.939
8 - 4 - 12	10000	9500	.470	.440	.455	.936	.968
	10500	10000	.485	.464	.479	.955	.988
[illegible] - 4 - 9	13000	12000	.470	.434	.440	.923	.938
	13000	12000	.460	.434	.440	.944	.958
6 - 3 - 16	2800	2600	.495	.468	.496	.945	1.000
	2800	2600	.510	.468	.496	.917	.974

In this table

P = load, in lb., at which deflection was recorded. This load was chosen from the load deflection curves given in preceding reports just below that load at which the curve begins to curve off markedly from the approximate straight line.

P' =load increment, i.e. P minus the initial load.

d_{act}= measured deflection for load increment P'

d_{full}= computed deflection for load increment P' determined from the full, unreduced moment of inertia deflection for

d_{eq} = computed load increment P' determined from the moment of inertia calculated from the equivalent width.

In computing d_{full} and d_{eq} the deflection due to shear was added to that due to bending, the latter being the only one usually considered. In these beams the deflection due to shear amounted to from 1% to 3% of the total deflection. This shows that for ordinary design computations of members of this type the shear deflection is negligible just as in most standard heavy steel structures.

In computing d_{eq} the equivalent width was determined from design chart A for the stress corresponding to P rather than from the strain readings. This was done because from a practical point of view it is of primary interest to ascertain whether the deflections a designer would compute by means of this chart do agree within reasonable limits with the observed deflections.

From table 3 it is seen that, with one exception, all deflections computed on the basis of the full moment of inertia are smaller than the observed ones. That is, the use of the full, unreduced width results in design computations which err on the dangerous side. Of the deflections computed from the equivalent width three coincide exactly with the observed ones, three are slightly larger and the rest smaller. In all cases where the computed deflections are smaller than the observed ones, the difference is smaller for d_{eq} than for d_{full}.

The average of d_{full}/d_{act} is .924 with a deviation from the mean of +8%, - 12%, the average of d_{eq}/d_{full} is .953 with a deviation from the mean of +8%, - 15%. Three types of beams showed particularly large irregularities in forming, viz. 6-3-9, 8-3-9, and 8-4-9. In all these beams the two component channels were shifted with respect to one another, thus producing beams which are unsymmetrical about the web. It is well known that deflections are particularly sensitive with regard to deviations of the loading plane from the principal plane of the cross section, any slight such deviation producing a marked increase in deflection if the two principal moments of inertia are greatly different from each other, as they are in these beams. If, for this reason, these beams are eliminated from the averages, the average value of d_{full}/d_{act} is .930 with a deviation from the mean of + 8%, -7%, the average value of d_{eq}/d_{act} is .968 with a deviation of +6%, -10% from the mean. It is thus seen that, in the average, deflections computed from the equivalent width are about 3% amaller than observed ones, whereas deflections determined from the full width are about 7% too small. There seems to be no conclusive explanation for this 3% deviation, except that the regularity of shape of these specimens was not too good, as noted in previous reports. This made exact loading as well as exact computation rather difficult.

One may conclude, then, that for beams in this range of b/t the influence of the reduction of the equivalent width is not very pronounced, about 4% in the average, but that the agreement of computed with observed values is decidedly better if the equivalent width from design chart A is used.

V. FAILURE MOMENTS

Table 4 below contains the observed bending moments at the ultimate loads, the computed failure moments determined on the basis of the full width, and the computed failure moments determined on the basis of the equivalent width.

Table 4

Observed and Computed Failure Moments

Type	b/t	s_{yp}	M_{act}	M_{full}	M_{eq}	M_{full}/M_{act}	M_{eq}/M_{act}
4 - 2 - 16	28.9	30,200	43,100 43.12	41,700	39,000	.97	.9[illegible]
			42,600 42.56			.98	.9[illegible]
4-2 1/2-16	38.3	30,200	49,200 49.28	50,700	45,900	1.03	.9[illegible]
			49,800 49.84			1.02	.92
4-2 1/2-12	19.2	35,100	108,200 108.2	96,200	95,500	.89	.88
			106,400 106.4			.91	.90
4-2 1/2-10	14.3	35,700	120,200 120.4	120,000	120,000	1.00	1.00
			124,500			.97	.97
8 - 3 - 15	42.6	37,300	153,000 152.9	185,000	165,000	1.21	1.08
			171,000 171.64			1.08	.97
6 - 3 - 12	24.0	35,100	199,000 199.08	189,000	183,000	.95	.92
			202,000 202.16			.94	.90
8 - 4 - 15	56.0	37,300	193,000 193.2	230,000	190,000	1.19	0.99
			200,000 200.2			1.15	0.95
6 - 3 - 9	16.3	33,100	254,700 254.8	240,000	239,000	.94	.94
			254,000 254.0			.95	.94
8 - 3 - 12	23.6	36.200	291,700 291.2	289,000	277,000	.99	.95
			319,400 319.2			.91	.87
8 - 3 - 9	16.4	33,100	375,000 375.2 [illegible]	349,000	349,000	.93	.93
			378,000 378.2 [illegible]			.92	.92
8 - 4 - 12	32.0	36,200	322,000 322	355,000	332,000	1.09	1.02
			350,000 350			1.02	.95
8 - 4 - 9	22.9	33,100	440,000 439.4 [illegible]	415,700	407,000	.94	.93
			434,000 434.0 [illegible]			.96	.94
6 - 3 - 16	45.0	30,300	92,600	99,400	87,500	1.07	.95
			98,000			1.01	.89

In this table

s_{yp} = lower yield point from tension ~~tests~~

M_{act} = ultimate moment from observed ultimate load

M_{full} = computed ultimate moment from full, unreduced cross-section, i.e. $M_{full} = s_{yp} \times S_{full}$

M_{eq} = computed ultimate moment determined from equivalent width, i.e. $M_{eq} = s_{yp} \times S_{eq}$

The equivalent width for S_{eq} was determined from design chart A for a stress equal to s_{yp}.

The analysis of the results given in table 4 should give more accurate information on the validity of the equivalent width approach than tables 2 and 3 for the following reasons: (a) Whereas tables 2 and 3 hold for stresses corresponding to about 75% of the ultimate loads, table 4 refers to failure loads, that is, essentially to yield point stresses. Since, for a given b/t the equivalent width decreases with increasing stress, the difference in behavior of the specimens computed for full and for reduced width is bound to be larger in table 4 than in tables 2 and 3. (b) The section modulus and with it the ultimate moment is more sensitive to a change in equivalent width than the moment of inertia and the corresponding deflection. On the other hand, factors adversely affecting the accuracy of table 4 are (a) the irregularities in shape of the specimens noted before and (b) the rather large variations in yield point for a given steel mentioned in the 34th Report.

In order to visualize the influence of the equivalent width on the ultimate moment, the ratios given in the last two columns of table 4 will be averaged separately for the low range of b/t, up to 30, and for the higher range, from 30 to 56. In the low range the difference between equivalent and actual width, according to chart A, is comparatively insignificant, whereas in the higher range it is considerable.

The average ratio M_{full}/M_{act} in the low range is .948 with a deviation from the mean of +5%, - 8%. For the high range the average ratio is 1.091 with a deviation of +11%, - 8%. The average ratio of M_{eq}/M_{act} for the low range is .927 with a deviation from the mean of +7%, -6%, for the high range .967 with a deviation of +11%, -5%.

A comparison of the first two of these average ratios, for unreduced sections, shows clearly the influence of the equivalent width. In the low range, where the equivalent and the actual widths are not significantly different, the ultimate moment computed from the full section is in the average about 5% smaller than the observed ones. For the high range however, where the influence of the equivalent width is much more significant, computed ultimate moments are in the average about 9% above the observed ones with a maximum discrepancy,, on the dangerous side, of about 20% in two cases.

A comparison of the average ratios for reduced cross sections shows that in both ranges the computed ultimate moments are slightly smaller than the observed ones, about 3% and 7% respectively, and the maximum deviation on the unsafe side, in but one case, is 8%. The fact that the average ratios computed from the reduced sections, are slightly below 1(one) is easily explained by the well established fact that steel beams do not fail when the outermost fibers reach yield point stress, but continue to carry increasing loads until yielding spreads at least over a significant part of the section. (See Volterra: J. Inst. Civ. Eng. (London), Mar. 1943, Winter: ibid., Oct. 1943 and Winter: Trans. A.S.C.E.vol. 105 pp. 673-679. Also Timoshenko: Strength of Materials and our Summary Reports on Roof Decks). The fact that the full strength computed by the method explained in these references and used in evaluating the roof deck tests was not reached in these tests is due to the following: Theoretical full strength is obtained when yielding spreads over the entire section. This can happen, however, only if the sections remain stable up to that load. This was the case in roof decks where ribs provided full support for the flanges up to failure. In the present tests, once the entire area of the compression lips begins to yield, the lip fails (by yielding, not by elastic buckling) and therefore no longer affords support to the compression flanges, with consequent failure of the entire beam. For this reason no use should be made of any excess strength beyond M_{eq} of table 4 for sections with stiffening lips, since the full, theoretical yield point strength can never be developed in such sections. Thus, the usual approach $M_{ult} = s_{yp} \times S_{eq}$, based on straight line stress distribution, gives a wholly adequate and only very slightly conservative criterion for the strength of beams of this type.

It can therefore be concluded that the results of table 4 represent a very satisfactory verification of the equivalent width approach and the corresponding design chart A with regard to the most important phase of design practically, namely that of determining the strength of the members.

VI. ADEQUACY OF LIPS

The lips for the specimens of this series were designed according to the requirements of the Tentative Specifications. In no case was there any sign that premature failure was caused by breakdown of the lip. On the contrary, the results, particularly of table 4, indicate that in all cases the compression flanges reached the full maximum strength of doubly supported (fully stiffened) elements. Therefore, although no systematic investigation of lip dimensions has yet been undertaken, the present test, within their range of dimensions, confirm the safety of the quoted design requirements. ?

VII. CONCLUSIONS

(1) This investigations covers the low range of b/t of compression flanges, from 14 to 56, and thereby supplements previous investigations which were concerned with the range of b/t from 50 up to about 200.

(2) An analysis of the observed ultimate loads reveals very satisfactory agreement with ultimate loads predicted by means of the proposed equivalent width approach, whereas predicted ultimate loads computed from the full, unreduced cross sections erred considerably on the unsafe side.

(3) Agreement with observed deflections was better for deflections computed from the equivalent width than when computed from the full, unreduced width, although the difference between the two approaches with regard to deflections, as expected, was not as pronounced as with regard to ultimate loads.

(4) Equivalent widths determined directly from strain readings were generally somewhat larger than predicted from design chart A. However, because of the inevitably unsatisfactory experimental accuracy of this part of the investigation the only conclusion to be drawn is that the assumed value of $b/t = 15$ for $R = 1$ on design chart A appears to be justified experimentally as a conservative end point of all the curves.

(5) On the basis of these conclusions it is believed that the information in design chart A in the low range of b/t (not hitherto investigated experimentally) can be recommended for practical use without change.

(6) Lips were designed according to the Tentative Specifications and proved wholly satisfactory. This does not represent, however, a systematic experimental confirmation of these design requirements over the entire allowed range of lip dimensions.

TYPICAL
CROSS SECTION
OF
BEAM