NATIONAL ADVISORY COMMITTEE FOR AERONAUTICS

TECHNICAL NOTE 2930

STRENGTH ANALYSIS OF STIFFENED THICK BEAM WEBS WITH RATIOS OF WEB DEPTH TO WEB THICKNESS OF APPROXIMATELY 60

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SUMMARY

The results of an experimental investigation of the strength of plane diagonal-tension webs with ratios of web depth to web thickness of about 60 are presented. An analysis of the beams indicated that the methods of strength analysis presented in NACA TN 2661 are applicable to beams with the flanges symmetrically arranged with respect to the web if the portal-frame effect is taken into account.

INTRODUCTION

Strength-analysis methods for plane diagonal-tension webs are presented in reference 1 and are verified in reference 2 for beams with ratios of web depth to web thickness of approximately 115 and greater. Verification of the strength-analysis methods for lower values of the ratio of web depth to web thickness is desirable because the methods presented in reference 1 are semiempirical. The results of the strength tests on the 10 beams tested in the present investigation provide verification of the strength-analysis methods for beams with ratios of web depth to web thickness of approximately 60. The results of the tests and a discussion of the results are presented.

SYMBOLS

A_U cross-sectional area of upright, sq in.

A_{U_e} effective cross-sectional area of upright, sq in.

E Young's modulus, ksi

G_e effective shear modulus (includes effects of diagonal tension and plasticity), ksi

I moment of inertia, in.^{4}

L length between reinforced bays, in.

P force, kips

P_{ult} ultimate force on beam, kips

Q_F static moment of flange area about neutral axis of beam, in.^{3}

Q_W static moment about neutral axis of web area above neutral axis, in.^{3}

S transverse shear force on beam, kips

S_W transverse shear force on web, kips

d spacing of uprights, in.

d_c clear spacing between uprights, in.

h depth of beam, in.

h_c clear depth of web, in.

h_e effective depth of beam, in.

k diagonal-tension factor

t thickness of web, in.

τ_{cr} critical shear stress, ksi

$\tau_{cr_{calc}}$ calculated critical shear stress, ksi

τ_{ult} shear stress at ultimate load, ksi

ωd flange flexibility factor, $0.7d\sqrt[4]{\frac{t}{(I_C + I_T)h_e}}$, where I_C and I_T are moments of inertia, about their own axis perpendicular to web, of compression flange and tension flange, respectively

TEST SPECIMENS

Ten beams with 24S-T3 aluminum-alloy webs and 24S-T4 aluminum-alloy flanges were tested in this investigation. The uprights were on only one side of the web and were equal-leg angles of 24S-T4 extruded aluminum alloy except for beam 10 which had angles formed from 24S-T3 sheet. All beams had ratios of web depth to web thickness of approximately 60 and the uprights were arranged to give ratios of upright spacing to web depth d/h of approximately 0.25 or 0.50. Beams 1, 2, and 3 had flanges which were not symmetrical about the web but had the vertical legs of both flange angles on the same side of the web as shown in section A-A of figure 1. The other beams were symmetrical, one flange angle on each side of the web, as shown in sections B-B and C-C of figure 1. The uprights of the symmetrical beams were not attached to the flanges but were fitted tightly in between them. The end bays and one bay on either side of the center of the beam were reinforced with an additional sheet the same thickness as the web. One symmetrical beam was built without a web or uprights except in the end and center bays which were reinforced in the other beams. This beam was used to measure the portal-frame effect.

The nominal dimensions of the beams are shown in figure 1 and the actual properties of each beam are given in table 1.

Each beam was given a code designation which parallels the designations used in reference 2. For example, VI-11-2S has the following meaning:

VI	designates the present series of tests (series I, II, III, IV, and V were published in ref. 2)
11	is the approximate depth of the beam in inches
2	is the number of the beam within the series
S	indicates single uprights

TEST PROCEDURE

The specimens were tested as simply supported beams in the jig shown in figure 2, which supported the beam laterally but did not restrain the bending of the beam. Each of the vertical guide bars had rollers between it and the frame so that it could move freely. On several beams, the loads on the end supports were measured in addition to measuring the load applied at the center to determine the friction between the beam and the supporting structure. This friction was less than 2 percent of the applied load.

Buckling loads for the webs were determined by observing the webs during loading and from the measured strains in the uprights. The load at which the load-strain curve for the upright departed from a straight line was taken as the buckling load. The uprights are usually unstrained if the web is not buckled; but, in the present tests, bending of the uprights as a result of unsymmetrical construction or slight initial eccentricities caused some strain in the uprights as soon as any load was applied to the beam.

On beams 9 and 10, strains measured by pairs of resistance-type wire strain gages on opposite sides of the web were averaged and the average strains at 45° and 135° to the longitudinal center line of the beam were used to compute the shear stresses at 0° and 90°. These gages were located on the longitudinal center line and $2\frac{1}{4}$ inches above and below the center line.

Load-deflection measurements were made on several beams as well as on the beam with a web in the end and center bays only. These measurements were used in determining the portion of the total beam load which was carried by the portal frame.

RESULTS AND DISCUSSION

On beams 1, 2, and 3 the vertical legs of both angles in the flange were on the same side of the web because this arrangement allowed the single uprights to be attached to the flange without joggling the uprights. It was apparent during the tests of these unsymmetrical beams that the flanges rolled and bent the web along the edge of the flange. An estimate of the detrimental effect of this bending of the web in the unsymmetrical beams may be obtained by comparing the results of tests on unsymmetrical beams 1 and 3 with the results of tests on similar symmetrical beams 5 and 8. At failure, the nominal shear stress in the web of beam 1 was 18 percent less than in beam 5 and in beam 3 the stress was 14 percent less than in beam 8. This detrimental effect resulting from the unsymmetrical arrangement of the web and flanges has not been noted in beams with $\frac{h}{t} = 115$ or greater.

In a built-up beam with deep heavy flanges and reinforced end bays, the portal frame formed by the heavy flange and the reinforced web sections connecting them may carry a significant part of the total shear on the beam. This portal-frame effect was discussed in the appendix of reference 1 and a formula for obtaining an approximation of this effect was presented. The formula which gives the ratio of shear carried by the web to the total shear on the beam is

$$\frac{S_W}{S} = \frac{1}{1 + \frac{24EI}{L^2htG_e}} \tag{1}$$

where I is the moment of inertia of one flange about an axis through the center of gravity of the flange perpendicular to the web plane and L is the length between reinforced bays.

In the present investigation, formula (1) was checked experimentally by two approximate methods. The first method attempted to duplicate experimentally the analytical manipulations used in deriving formula (1). The shear carried by the web was obtained by subtracting from the total shear on the beam the shear carried by the portal frame when the tip deflection of the frame was the same as the deflection of the beam. By using this experimental procedure, the ratio of the web shear to the total shear for beams 7 and 8 was 0.75 just before the beams failed. The ratio obtained from formula (1) was 0.77.

The other method of obtaining the portal-frame effect which was used on beams 9 and 10 was to compute the shear in the web from the measured strains in the web. The disadvantage of this method was that the individual strains in the web became too large to measure at approximately 60 percent of the load required to fail the beams. For low loads, this method indicated higher ratios of shear in the web to the total shear than the other methods; but, it appeared from extrapolating the curves showing the ratio of shear carried by the web to the total shear that near failure the ratios obtained by this method would be about the same as the ratios obtained by measuring the deflections.

The beams were analyzed by the method presented in reference 1 with the value of 0.77, computed from formula (1), being used for the ratio of shear carried by the web to the total shear on the beam. The shear stresses in the webs at failure were computed by the formula

$$\tau_{ult} = \frac{S_W Q_F}{It}\left(1 + \frac{2}{3}\frac{Q_W}{Q_F}\right) \tag{2}$$

where $S_W = 0.77 \frac{P_{ult}}{2}$. This formula gives the average shear stress in the web according to the engineering theory of bending.

The critical shear stress for the web was determined experimentally for several beams and was within 10 percent of the calculated critical shear stress.

The ratio of the actual failing load to the predicted failing load for web rupture P_{ult}/P_W is given in the last column of table 2. In calculating the strength of the webs the basic allowable shear stress was corrected to the actual properties of the material and it was also assumed to be increased 5 percent because of the friction in the web-to-flange connection when the bolts were drawn tight. The ratio of the actual to the predicted failing load for each of the symmetrical beams predicted to fail by web rupture was between 1.11 and 1.12. Strength predictions for beams predicted to fail by web rupture would be expected to average 10 percent conservative because the pure shear tests from which the allowable stresses were determined showed a scatter of ± 10 percent and the basic allowable shear stress used in the strength predictions represents the lower edge of the scatter band of the pure shear tests. The results of the present tests are therefore approximately what is to be expected.

The average ratio of actual to predicted failing load for the beams predicted to fail by forced crippling of the uprights was 1.08. All the symmetrical beams tested exceeded the predicted strength for upright failure. Thin-web beams with uprights not connected to the flange sometimes fail before the full strength of the web or uprights is developed because the web wrinkles at the ends of the uprights and the ends of the uprights cut through the web. There was no indication of this type of failure in the thick-web beams used in the present investigation.

The stiffening ratio A_U/dt required to give simultaneous failure of the web and uprights may be estimated from the curves of figure 23(a) in reference 1. The beams used in the present investigation provide some checks on the reliability of these curves for estimating stiffening ratios of heavily loaded beams. For a structural index of approximately 25 and $\frac{d}{h} \approx 0.50$, the stiffening ratio for a balanced design would be between the ratios for beam 6, which failed by forced crippling of the uprights, and beam 7, which failed by web rupture. When the load on beam 6 was increased slightly above that necessary to cause forced crippling of the uprights, the web ruptured. The stiffening ratio was 0.0748 for beam 6 and 0.1112 for beam 7. The curves in reference 1 show that a stiffening ratio of 0.08 is required in a balanced design. Beam 10, which had a structural index of approximately 25 and $\frac{d}{h} \approx 0.25$, should have a stiffening ratio of 0.1 for a balanced design. The actual stiffening ratio was 0.0927 and simultaneous failure of the web and uprights occurred.

CONCLUDING REMARKS

The methods of predicting the critical shear stresses, forced-crippling failure of uprights, and rupture of webs presented in NACA TN 2661 are applicable to stiffened webs with ratios of web depth to web thickness of 60 provided the flanges are arranged symmetrically about the web and the portal-frame effect is accounted for before analyzing the web. The accuracy of the strength predictions for beams with ratios of web depth to web thickness of 60 is about the same as for the beams discussed in NACA TN 2662 which had ratios of web depth to web thickness of 115 or greater.

Langley Aeronautical Laboratory,
National Advisory Committee for Aeronautics,
Langley Field, Va., February 17, 1953.

REFERENCES

1. Kuhn, Paul, Peterson, James P., and Levin, L. Ross: A Summary of Diagonal Tension. Part I - Methods of Analysis. NACA TN 2661, 1952.

2. Kuhn, Paul, Peterson, James P., and Levin, L. Ross: A Summary of Diagonal Tension. Part II - Experimental Evidence. NACA TN 2662, 1952.

TABLE 1

PROPERTIES OF TEST BEAMS

Beam	h_e, in.	h_c, in.	t, in.	d, in.	Uprights, in.	A_U, sq in.	A_{U_e}, sq in.	$\frac{A_U}{dt}$	$\frac{A_{U_e}}{dt}$	ωd
VI-11-1S	11.62	8.5	0.2053	6.0	$\frac{5}{8} \times \frac{5}{8} \times 0.0621$	0.0763	0.0240	0.0620	0.0195	0.96
VI-11-2S	11.62	8.5	.2078	6.0	$\frac{5}{8} \times \frac{5}{8} \times 0.0968$	.1170	.0332	.0938	.0267	.96
VI-11-3S	11.62	8.5	.2006	6.0	$\frac{3}{4} \times \frac{3}{4} \times 0.1286$	.1755	.0544	.1460	.0452	.96
VI-11-4S	11.12	7.0	.2080	6.0	$\frac{1}{2} \times \frac{1}{2} \times 0.0683$	.0543	.0144	.0435	.0115	.96
VI-11-5S	11.12	7.0	.2059	6.0	$\frac{5}{8} \times \frac{5}{8} \times 0.0620$	.0780	.0245	.0632	.0198	.96
VI-11-6S	11.12	7.0	.2019	6.0	$\frac{3}{4} \times \frac{3}{4} \times 0.0610$	.0906	.0317	.0748	.0262	.96
VI-11-7S	11.12	7.0	.2089	6.0	$\frac{5}{8} \times \frac{5}{8} \times 0.1252$	.1394	.0371	.1112	.0295	.96
VI-11-8S	11.12	7.0	.2073	6.0	$\frac{3}{4} \times \frac{3}{4} \times 0.1277$	.1758	.0536	.1412	.0430	.96
VI-11-9S	11.12	7.0	.2059	3.0	$\frac{1}{2} \times \frac{1}{2} \times 0.0316$	.0316	.0093	.0512	.0151	.48
VI-11-10S	11.12	7.0	.2034	3.0	$\frac{1}{2} \times \frac{1}{2} \times 0.0612$	.0565	.0149	.0927	.0245	.48

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TABLE 2

TEST DATA AND RESULTS

Beam	$\tau_{cr_{calc}}$, ksi	P_{ult}, kips	τ_{ult}, ksi	$\frac{\tau_{ult}}{\tau_{cr_{calc}}}$	k	Predicted P_{ult} (a)		Observed failure	$\frac{P_{ult}}{P_{FC}}$	$\frac{P_{ult}}{P_W}$
						P_{FC}, kips	P_W, kips			
VI-11-1S	27.0	165	26.6	----	-----	186	190	Web	0.89	0.87
VI-11-2S	27.0	175	28.0	1.04	0.008	187	193	Web	.94	.91
VI-11-3S	27.0	180	29.8	1.10	.020	197	186	Web	----	.97
VI-11-4S	28.0	192	31.9	1.14	.028	180	182	Forced crippling	1.07	----
VI-11-5S	28.0	192	32.3	1.15	.030	176	184	Forced crippling	1.09	----
VI-11-6S	28.0	186	32.0	1.14	.028	175	177	Forced crippling	1.06	----
VI-11-7S	28.0	203	33.6	1.20	.039	190	183	Web	----	1.11
VI-11-8S	28.0	208	34.7	1.24	.047	191	186	Web	----	1.12
VI-11-9S	28.0	184	31.0	1.11	.022	177	181	Forced crippling	1.04	----
VI-11-10S	28.0	198	33.7	1.20	.039	177	179	Web and forced crippling	1.12	1.11
								Average[b]	1.08	1.11

[a]Subscripts have the following meaning:
FC forced-crippling failure
W web rupture

[b]Average does not include beams 1, 2, and 3 with flanges not symmetrical about web.

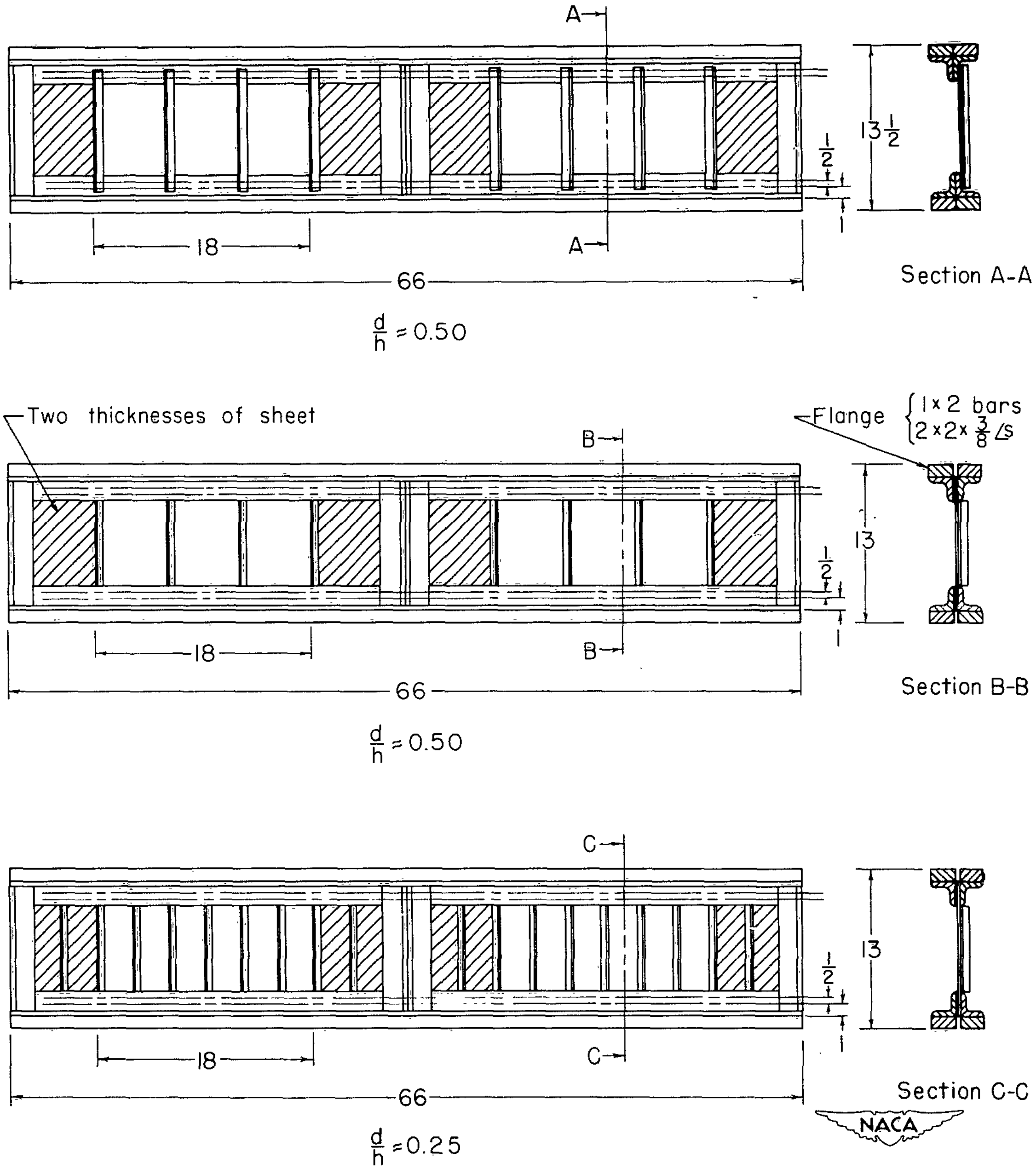

Figure 1.- Test beams.

Figure 2.- Test jig.

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Copies obtainable from NACA, Washington

1. Beams, Diagonal-Tension (4.3.4.2)
2. Loads and Stresses, Structural - Shear (4.3.7.5)

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