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School of Civil Engineering, Cornell University tests on cold formed steel studs for the American Iron and Steel Institute

Cornell University School of Civil Engineering

SCHOOL OF CIVIL ENGINEERING, CORNELL UNIVERSITY

TESTS ON COLD FORMED STEEL STUDS

for The American Iron and Steel Institute

Fifth Progress Report

November 1940

I. Scope of this report

(a) Results of tests on single stud panels having collateral attached at various intervals along the studs and by means of several types of fastenings.

(b) Shear strength of Armstrong Linoleum Cement bonding Homosote to steel studs.

(c) Discussion of the theoretical factors in formulating test program.

(d) Summary.

II. Method of testing

Panels were made up using the 8 foot I (A studs) sections in the manner shown in Fig. 26. The collateral was attached to the stud only, being free at all four edges. The stud was supported at each end by a knife edge that was parallel to the web of the stud. In order to observe the deformation of the stud under load, a wire (see Fig. 26) was stretched tight over the ends of 3/8 inch rods rigidly attached to the web of the stud and of sufficient length to project outside the collateral. After the panel was on the testing machine the wire was adjusted to a plumb line and the ends of the intermediate rods brought tangent to the wire. The wire was fastened to the rods with tape. The movement of the wire was observed by two transits placed at some distance along lines AB and CD (Fig. 26).

III. Results

(a)

lat-al	Collateral Attachment Type	Spacing	Ultimate Load	Type of Failure
	Armstrong Linoleum Cement		12,950	Collateral loosened at e- Stud failed locally at e
			7,800	Collateral sheared off
	3/16" bolts with Washers	8"	15,200	Local end failure
		8"	13,750	" " "
Celotex	Metal Screws 7/16" Heads	41"	16,750	Minor axis failure(1/3 -
		41"	12,400	Screws pulled out of Celotex
	Metal Screws 3/8" Head with Washers	27"	15,350	" " " "
		8"	16,350	Local end failure

.e:
..ll studs were 8 foot I sections. See Fig. 27A for details
d connection of collatoral to stud and Fig. 27B for type of
end failure.

(b) Three tests were made to determine the shear strength
oped by Armstrong Linoleum cement between the Homosote col-
al and the steel studs. The average strength was 40 lbs. per
e inch. It is probable that strengths of this magnitude were
btained in the panels since the adhesion there was not as
ete.

(c) Theoretical discussion

These tests were made as pilot tests to formulate a program and procedure so as to determine the amount of lateral aint necessary to prevent failure of the individual studs ab inor axis and also to determine the maximum spacing of points pport.

To determine the next step in the program an analysis was to determine the conditions which must exist for specific ilure of the following types: (A) bending about the major axis;) instability of the flange; (C) collateral spacing great enough permit bending of the stud about the minor axis (assuming that e collateral material and its attachment will offer sufficient straint for the stud) and, (D) eccentricity sufficient to proce yield point stresses in the edge of the flange.

(A) The critical load required to produce failure about the jor axis was determined according to formula "d" page 184, .moshenko's Elastic Stability. (This formula applies to columns ere l/r is less than 110; for this case $l = 96$, $r = 1.47$, therere $l/r = 65.3$.)

$$\sigma_{cr} = 48{,}000 - 210\ l/r$$
$$= 48{,}000 - 210\ (65.3)$$
$$= 34{,}300 \text{ lbs/in}^2$$

nce the yield point is about this value it seems that a major is failure will be improbable.

(B) The critical stress for stability of the flange was termined by the formula "j" page 339, Timoshenko's Elastic ability

$$(\sigma_x)_{cr} = \frac{k\pi^2 D}{b^2 h} = \frac{k\pi^2}{b^2 h}\left(\frac{Eh^3}{12(1-r^2)}\right)$$

$$k = 0.5$$
$$E = 3(10)^7$$
$$h = 0.06$$
$$b = 1.0$$
$$r = 0.3$$

$$\sigma_{cr} = \frac{(0.5)\pi^2\ 3(10)^7(.06)^3}{1(.06)\ 12(1-0.3^2)} = 49{,}200 \text{ lbs/in}^2$$

This critical limit stress being above the yield point indicates that a failure due to instability of the flange is impossible.

(C) After the first tests were made an analysis was made to termine the spacing of collateral fastenings such that the ritical load would be reached for failure about the minor axis. his analysis would indicate whether the collateral attachments ere sufficient. The values recorded below were computed for alues of l/r over 110 by Euler's formula $P_{cr} = \frac{\pi^2 EI}{l^2}$ which educes to $l = 4880/\sqrt{P_{cr}}$. For values of l/r less than 110 the alues of l were computed from the formula referred to previously

$$\sigma_{cr} = 48,000 - 210\ l/r \quad \text{which reduces to}$$

$$l = 77.3 - \frac{P_{cr}}{438}$$

P	15	16	17	18	19	20	21	22	23	Yield Poir
l	40	38.7	37.4	36.3	34	31.7	29.5	27.1	24.9	21.1

P = load in kips
l = length of hinged end column in inches

Test P-5 shows that the end is partially restrained; see Fig. 27A for details of end connection.)

(D) The effect of eccentricity was next considered. Using the well known formula for combined direct stress and cross bendin

$$= \frac{P}{A} \pm \frac{Mc}{I}$$

Which reduces to $e = (\frac{\sigma}{P} - \frac{1}{A}) \frac{I}{c}$ where e is the eccentricity of the direct load producing the bending stress. Substituting in the values

$$A = 0.705; \quad I = .0806; \quad c = 1$$

and using σ equal to the yield point stress, say 35,000 lbs/sq. we obtain the eccentricity required to produce a yield point stre in the outer fibers of the flange. The formula becomes

$$e = \frac{2820}{P} - 0.1146$$

Values here have been assigned to the load P and the value of "e" computed. The results have been plotted and are shown as Fig. 28 attached.

This curve shows that failure of the column at the end may due to small eccentricities that cause the flange stress to ach the yield point of the material. (It should be noted that e formula used does not apply for values of "e" greater than .115 since a reversal of stress would occur.)

To illustrate the use of the curve, consider the test P-8 th an ultimate load of 16,350 lbs., the curve shows that an centricity of less than 0.06 inches would cause flange stress ual to the yield point.

Since the collateral carries part of the load the maximum ombined stress will probably occur between the end of the stud nd the first collateral attachment, thus producing the local rushing of the flange near the end.

It is to be noted also that the values of "e" are the maxi- um values and that smaller value might produce yield point stres f the loads were eccentric along both axes.

The eccentricity of the load may be caused by a number of actors and it would seem doubtful if greater loads can be carried y the studs, than have been already obtained.

Summary

(1) This report covers pilot tests on 8 foot I studs (Type) which were made to assist in the development of a test pro- edure.

(2) Collateral used (even thin material) was so strong hat with reasonable spacing of attachments failure about the inor axis did not occur.

(3) If the free length of stud between collateral attach- ents was made great enough (35 to 45 inches) failure occured bout the minor axis.

(4) For rather large spacing of the collateral attachments (but less than referred to in # 3) failure by pulling the screws through collateral occured.

(5) For tests where the collateral was attached as in # 2, the stud failed by local crushing between the first collateral attachments and the end support of the stud (see Fig. 27B).

(6) Computations show that for loads applied an eccentricity of the order of magnitude of the sheet metal thickness will produce stress in the edge of the flange equal to the yield point. This fact may contribute greatly to local failure referred to in # 5 above.

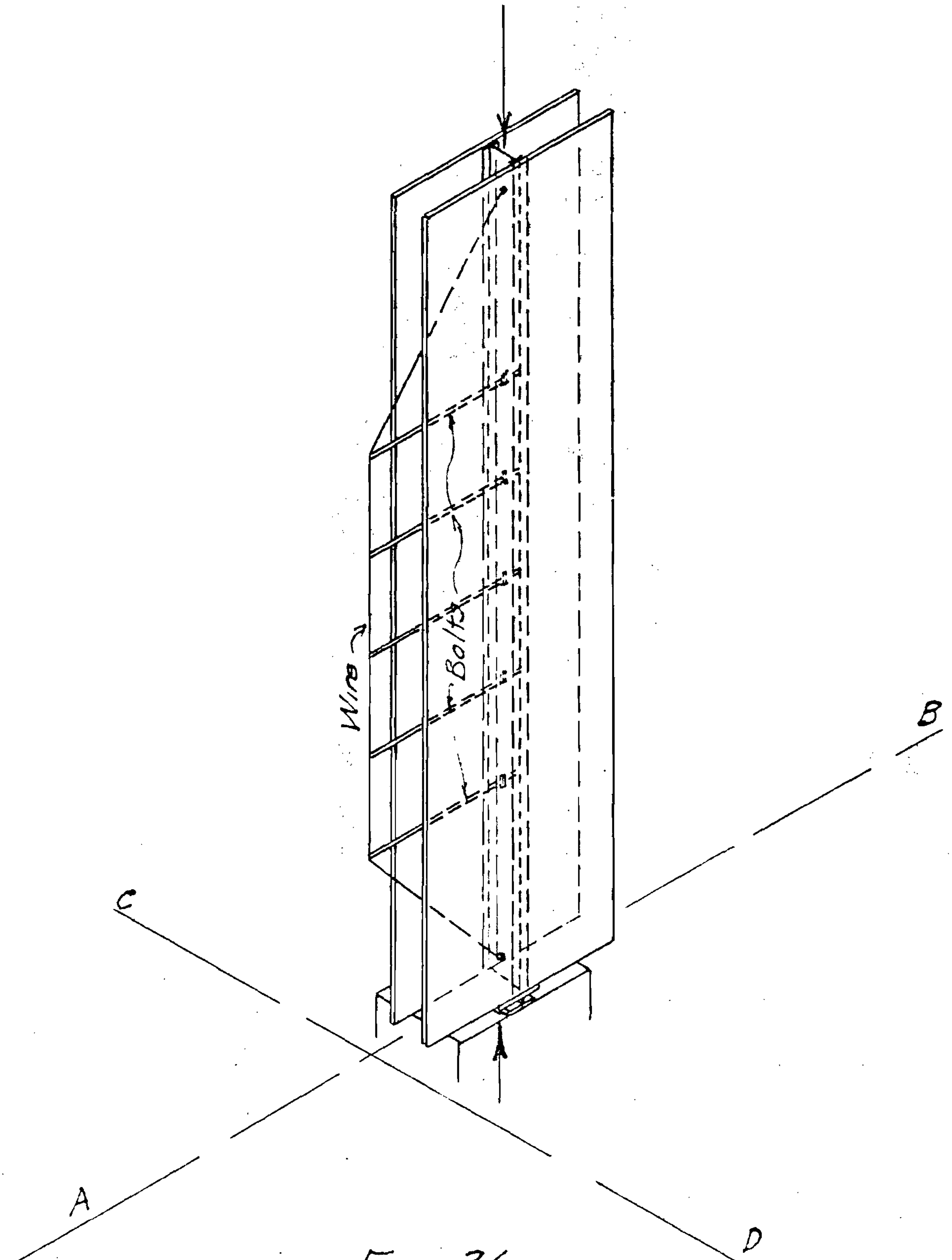


FIG 26

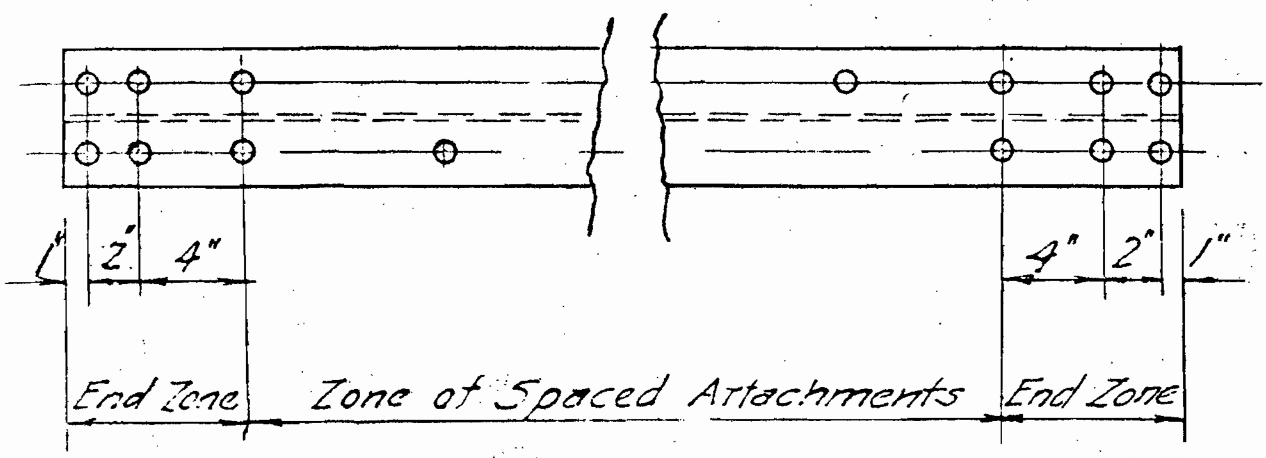


FIG 27A

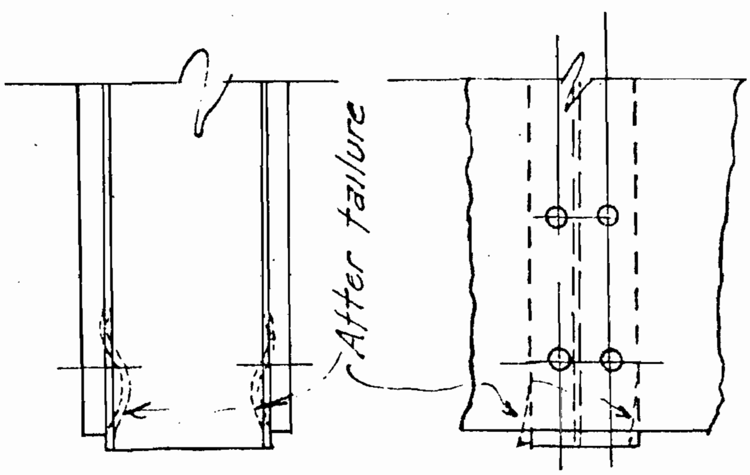
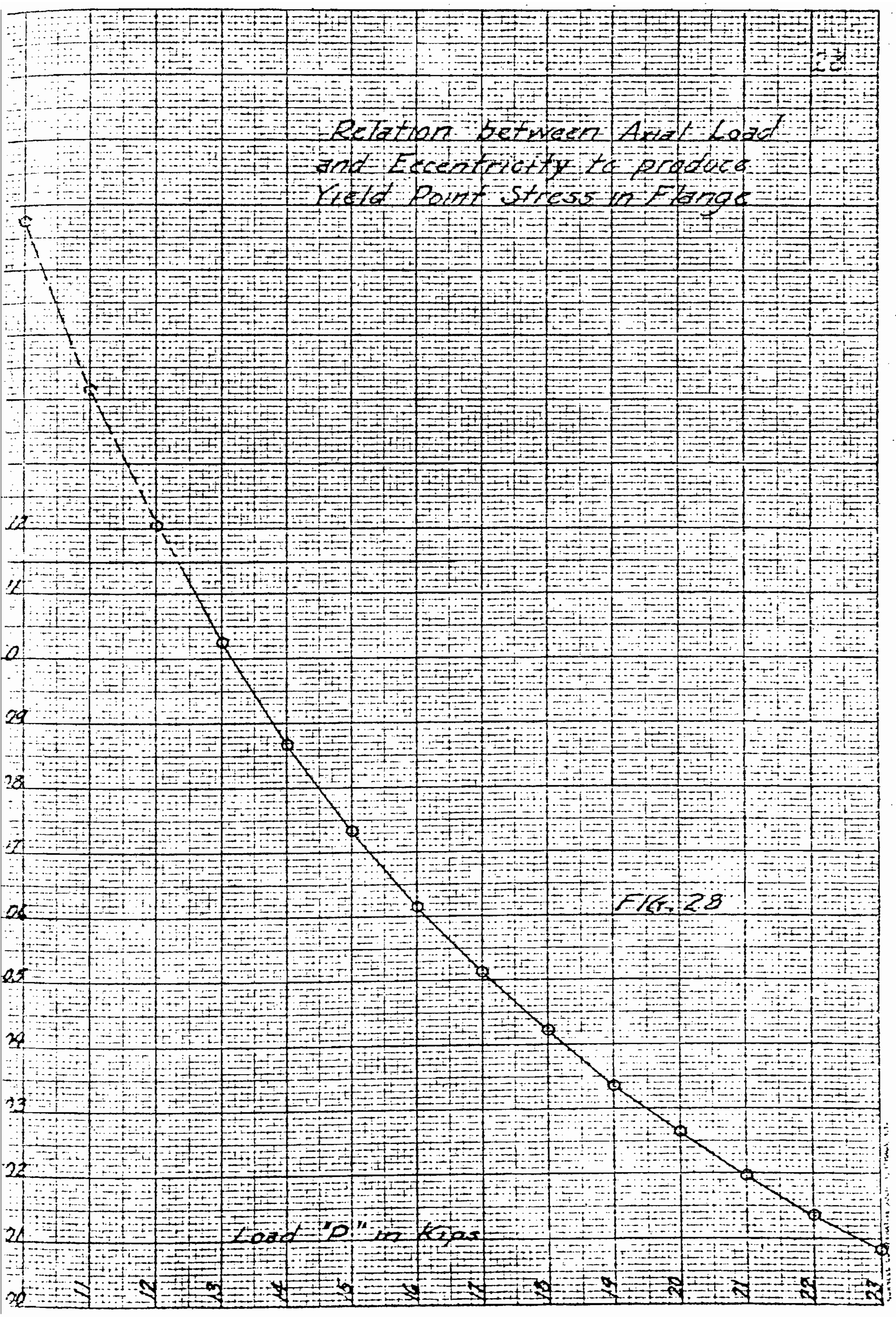


FIG 27 B.



Relation between Axial Load
and Eccentricity to produce
Yield Point Stress in Flange
FIG. 28
Load "P" in Kips
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14
15
16
17
18
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21
22
23