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The performance of beams and columns continuously braced with diaphragms

George Pincus

Gordon P. Fisher

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CORNELL UNIVERSITY

SCHOOL OF CIVIL ENGINEERING

DEPARTMENT OF STRUCTURAL ENGINEERING

THE PERFORMANCE OF BEAMS AND COLUMNS CONTINUOUSLY BRACED WITH DIAPHRAGMS

Progress Report No. 2

by

George Pincus and Gordon P. Fisher

Sponsored by the American Institute of Steel Construction

February 1963

AISC-CORNELL Project

THE PERFORMANCE OF BEAMS AND COLUMNS CONTINUOUSLY BRACED WITH DIAPHRAGMS

Progress Report No. 2

Introduction:

In Progress Report No. 1, it was reported that the initial conception of the problem was that diaphragms act almost exclusively in shear when called upon to brace compression members, and that it had been decided to test single columns with finite widths of diaphragm attached, with width of the sheet being varied to provide a range of lateral support up to that which permitted full column strength, based on strong axis to be developed. However, the first test of this kind, Test CB-1, clearly indicated that the diaphragm acted essentially in flexure only and thus had the same effect as if it were regarded as a weak cover plate. It was already quite clear at the time of the First Progress Report that the shear contribution of the diaphragm was practically nil, based on observation of combined beam-sheet tests (refer to figure 1, First Progress Report). It was thought at that time that prevention of rotation of the ends of the diaphragm would produce shear-predominant action. While the sheet, thus restrained, provided far more support to the beam than in the previous tests and could be exactly calculated, the behavior was essentially flexural and not in shear.

A series of tests were proposed in Progress Report No. 1 to confirm the initial conclusion that supporting diaphragms act in flexure and function essentially as corrugated cover plates. As mentioned, this was exactly confirmed by the first test, making further tests of this character pointless. It was recognized furthermore, that any purely flexural contribution of the sheet was so small that nothing was to be gained in design, in terms of increased column strength, by counting on it. On the other hand, by experience and intuition, it seemed that the supporting ability of the sheet should be far greater than observed.

In rethinking the problem, it was concluded that the sheet could act in pure shear and provide the anticipated support only if all cross-sections were prevented from rotating. The simplest situation producing this condition is that of a diaphragm attached to two identical, (in all respects, including loading), columns or beams, as in figure 27. In fact, this is a rather realistic situation in that corrugated building siding is or could be attached continuously across two or more columns that are more or less identical.

Accordingly, it was decided that tests should be performed on pairs of columns, each loaded and supported separately and identically, but connected by a diaphragm, as in figure 14. To the investigators' knowledge, tests of this kind had never been performed, and were regarded as exceedingly difficult because of the necessity of centering with absolute certainty two interconnected and interacting columns. As a simpler and less tedious way of checking the notion that the diaphragm would act primarily in shear, beam sheet tests as in figure 4 were devised. These tests, performed with a variety of diaphragms, gave substantial assurance that the idea was correct and permitted the experimental determination of the effective shear rigidities. With the information thus obtained, several double column tests were performed.

Test on Single Column:

While this test is not of much interest, it is reported for the sake of completion. Test CB-1, see figures 1 and 2, consisted of a single centrally-loaded column with an L/r of 280 about its weak axis. An unspliced corrugated sheet, of. .024 X 1 1/4 aluminum, 28 inches wide, was used on both sides of the column and attached at every third valley with 1/4 inch Pow-R-Set pins.

The end sections were carefully milled, and end blocks were welded on using low hydrogen electrodes. The sheet was attached to the column and the assembled specimen was supported on knife-edges oriented in the direction of the web, in effect giving a hinged-hinged condition for buckling in the weak direction. Dial gages reading to 0.001 inch were placed at the quarter points and midpoint to read deflections in the weak direction.

The theoretical buckling load of the bare column (without sheet) is 8200 lbs. The predicted failure load of the column with the sheet attached in the manner described above is 9800 lbs. As can be noted in figure 2, the close agreement between the predicted theoretical and the actual failure loads gives clear evidence that the sheet acts as a cover-plate when attached in this manner to a single column.

In prediciting the theoretical failure load of a single member with a diaphragm attached as a coverplate, an effective modulus of elasticity of the diaphragm in the direction of the column axis must be used. This efeective modulus in this case perpendicular to the corrugations, has been found for the two types of diaphragm to be 40,000 psi and 24,000 psi respectively for the 0.024 inch aluminim sheet and the 26 page galvanized steel sheet (see Progress Report No. 1 and figure 3 herein).

<u>Tests on Double Beams with Diaphragms</u>:

In order to calculate the lateral supporting capacity of the attached sheet in double-column tests, one must know the effective shear modulus of the diaphragm. Since insufficient theoretical information is available on the shear rigidity of corrugated sheets, it has been necessary to determine the shear rigidity experimentally. The experimental arrangement, shown in figures 4 and 5, consists of a diaphragm attached to two identical 3 I 2.59 aluminum beams (selected for low flexural rigidity), in pure bending about their weak axes. The deflections of the bare beams without sheet can be calculated exactly or determined experimentally. When the sheet is attached, the deflection of the affected region is reduced, this reduction being due almost exclusively to shear action in the sheets. The net deflection is a measure of the effective rigidity of the sheet and may be used directly for the double-column tests in which the diaphragm acts essentially in the same manner. Loads are applied to the two individual beams by similar jacks on a common hydraulic system thus providing for identical loading.

It happens that the behavior of the sheet depends on a marked fashion on the spacing of the connectors which attach the sheet to the columns or beams, the shear rigidity increasing nonlinearly with decrease in spacing. The shear rigidity also decreases with width of sheet in a manner which is not clear because of insufficient experimental information. The tests reported here have been limited to the range of widths used for the double column tests and provide sufficient information to predict the results of those tests. However, it appears to be vital that early attention be given to the generalized behavior of diaphragm-connector combinations by appropriate theoretical and experimental study, so that the behavior can be predicted for any given situation. The determination of this general sheet behavior is considered to be not within the scope of the present investigation.

The results of one test using 0.024 X 1 1/4 aluminum corrugated sheet is shown in figure 6.

Results of tests using the Granco Plenum 26 gage corrugated galvanized steel sheets finally adopted for the column tests, are plotted in figures 7 through 13. For these latter tests, there was one 28 inch wide sheet on each side and the connector spacing varied from test to test. The effective shear modulus varied non-linearly from a low of 114,000 psi for one pin at every eighth corrugation to a high of 2,210,000 psi for a pin at every corrugation, as in figure 13. Similar tests were performed with sheets 17 3/4 inch wide, the results of which are also summarized in figure 13. Thus, within these limits, a specific effective shear modulus can be taken from figure 13 and used to predict the failure load of the columns which are attached to each other by the sheet.

The simple theory for determining the effective shear modulus from the net deflection is as follows. In figure 4, over the region L, a pure moment equal to P.l will exist, where P is the applied load per jack and l the cantilever arm of the load. A center line deflection with respect to points L/2 from the center is:

$$d_{CL} = \frac{P.l}{EI}\,\frac{L}{2}\,\frac{L}{4} = \frac{P.l.L}{8EI} \quad \text{(a)}$$

when there is no sheet.

If the sheet is subjected to a uniform shear, its radius of curvature can be related to a load q(x) acting throughout its length:

$$y''_{sheet} = \frac{\alpha}{AG}\,q(x) \quad \text{(b)}$$

where A is the cross-sectional area of the sheet, G its shear modulus and α a shear shape factor which equals 1.5 for a rectangular section.

However, since in this case the section over which the sheet is located is not subject to a distributed load but to a pure moment, an equivalent uniform distributed load q(x) is found as follows:

Deflection at center due to a pure moment:

$$d_{CL_{Bending}} = \frac{PIL^2}{8EI}$$

Deflection due to uniform distributed load q(x)

$$d_{CL_{q(x)}} = \frac{5\ q(x)\ L^4}{384\ EI}$$

the equivalent distributed load which will cause the same flexural center line deflection as the applied pure moment is:

$$d_{CL_{Bending}} = d_{CL_{q(x)}}$$

so that:

$$q(x) = \frac{48\ Pl}{5\ L^2} \quad \text{...(c)}$$

taking this value of q(x) into (b), integrating, and noting that

$$d_{CL_{sheet}} = \frac{1.2\alpha Pl}{A\ G_{eff}} \quad \text{...(d)}$$

from where the effective shear modulus for the type of sheet and the given connector pattern is found as:

$$G_{eff} = \frac{1.2\alpha l}{A} \left(\frac{P}{d_{CL}}\right)_{sheet} \quad \text{..............................(e)}$$

From the double beam sheet test the combined rigidity $\left(\frac{P}{d}\right)_{CL}$ test is found. The relation:

$$\left(\frac{P}{d}\right)_{CL}\text{test} = \left(\frac{P}{d}\right)_{CL}\text{beam} + \left(\frac{P}{d}\right)_{CL}\text{Sheets} \quad \ldots\ldots(f)$$

is then substitued above in (e) giving:

$$G_{eff} = \frac{1.2\ \text{[illegible]}\ 1}{A}\left[\left(\frac{P}{d}\right)_{CL}\text{test} - \left(\frac{P}{d}\right)_{CL}\text{beam}\right] \quad \ldots\ldots\ldots\ldots(g)$$

This relation gives the effective shear modulus of the sheets for the given connector spacing, type and sheet characteristics.

CENTREALLY-LOADED DOUBLE-COLUMN TESTS

General Test Setup:

The test setup for a double-column test is shown in figure 14. Two columns are placed parallel and adjacent to eachother and connected. The sheet is unspliced throughout the length of the columns and extends to within one inch of the end blocks. It is attached to the columns with 1/4 inch Pow-R-Set pins at the flange-web junction of each column in the valleys of the sheet.

The ends of the column are milled flat and end blocks are welded on using low hydrogen electrodes. The sheet is attached to the two columns and the assembly is placed in the testing machine. The columns are supported by knife-edges which for these tests were parallel to the web of the columns. The lower knife edges each rest on 100 kip jacks connected to a common hydraulic system. Thus the same load is applied to each column throughout the test, unaffected by minor variations in the individual lengths of the two columns.

Eight dial gages reading to 0.001 inch are used in each test. One gage at the top head measures movement of the upper knife-edge supports. Six gages, three on each column at mid-length and quarter points, read deflection in the weak direction. A dial gage at the center of the assembly reads deflection in the strong direction.

Centering

Centering progresses gradually from a low load to about 2/3 of the calculated failure load. When load is first applied to the double-column assembly, several out- of-line deflections due to eccentricity of loading can take place. If the assembly deflects in the strong direction, this indicated eccentricity is corrected for first.

If the load is eccentric in the weak direction, it is necessary to be able to detect whether one or both columns are eccentric and in which direction. This is done by oabserving the weak axis deflections and also the transverse stress in the diaphragm. Dial gages will indicate in which direction the assembly deflects. Electric strain gages paired back-to-back on each sheet and the readings averaged will determine whether the sheet is in tension, compression or unstressed. Appropriate corrections are then made according to the keh in figure 15. For example, in figure 15-a, the sheet is in tension and the assembly deflects to thh right, thus the right column must be shifted to the right. In (b), the same deflection takes place upon load application but because the sheet is in compression, the left column must be shifted to the right. Any combination of sheet stress and weak-direction deflection can be properly corrected as shown in (c) through (g) until perfect centering, (h), is obtained.

This centering procedure is then repeated at increasingly higher loads.

The SR-4 electrical strain gages at the center of each column on the inside edge of each flange, (four per column), give an indication of the stress distribution. These gages also afford a check on strong and weak axis eccentricity. Minor shimming of the end blocks will compensate for any slight non-parallelism of the end blocks.

<u>Test Results:</u>

Load deflection curves for the centrally-loaded double-column tests are plotted in figures 16 through 21. Selected photographs are presented in figures 22 through 25. The test results are summarized in Table I.

TABLE I

RESULTS OF PILOT SERIES OF TESTS
ON CENTRALLY-LOADED DOUBLE-COLUMN ASSEMBLIES,
DIAPHRAGM BOTH FACES, KNIFE EDGES PARALLEL TO WEBS

Test	Support	Pins	L/r_y	P_{yy} (Kips)	P_{xx} (kips)	Ptest(K)	Ptest/P_{xx}
CBB (formerly CB-2)	.024 Alum. width center to center of columns=14 in.	Every other groove	280	8.2	95.0	77.5	0.82
CFF (formerly CF)	.018 Grance Plehum, center to center of column=14 in.	First ten grooves, then every other groove	280	8.2	95.0	83.0	0.87
CNO	.018 Grance Plenum, center to center of columns=17 3/4"	First twelve grooves, then every other groove	160	26.1	106.0	86.0	0.81
CNN	"	"	220	13.3	101.0	98.8	0.98
COO	"	Every third groove first five pins, then every sixth groove	220	13.3	101.0	49.5	0.49
CPP	"	"	220	13.3	101.0	48.5	0.48

P_{yy} is the weak-axis failure load of the bare columns.
P_{xx} is the strong-axis failure load calculated from the CRC formula with an arbitrary residual stress level of $0.4f_y$ corresponding to an actual level of $0.28f_y$.
Ptest/P_{xx} x 100 is the percentage of full lateral bracing provided.

Test CBB was to determine the general behavior of a double-column assembly and to perfect the method of centering such assemblies. This was accomplished. A further purpose, of equal importance, was to establish beyond reasonable doubt that the supporting diaphragm spanning between two columns behaves in a manner radically different from that of a diaphragm applied to a single column, such as test column CB-1. In the latter, the diaphragm acts flexurally and contributes little to the carrying capacity of the column, whereas in the former, a substantially greater contribution is to be expected of the diaphragm due to its shear-predominant behavior. The following results are of interest:

(a)	Single column without sheet, $L/r_y = 280$	$P_{max} = 8.2$ kips
(b)	Single column, same except with two 7" wide diaphragms (calculated)	$P_{max} = 8.23$ kips
(c)	Single column, same except two 28" wide diaphragms (Test CB-1)	$P_{max} = 9.8$ kips
(d)	Double column, $L/r_y = 280$ with two 14" wide diaphragms (Test CBB)	$P_{max} = 77.5$ kips

It is clear from these results that the attachment of light diaphragms, even those of considerable width, did not increase the column capacity very much, whereas similar widths of diaphragm attached to double columns increased the capacity nearly tenfold, indicating an entirely different mode of behavior. It is also clear that the chief supposition of the project has been realized, namely that relatively light side-wall coverings, properly attached, can provide substantial lateral bracing.

For this test CBB, a convenient width of aluminum sheet was applied, without prior knowledge of how much lateral bracing it would provide. According to the CRC column formula, the strong axis load P_{xx} was calculated to be 95.0 kips; the macimum test load was 77.5 kips with failure in the weak direction. Thus the amount of bracing provided in this case was 77.5/95.0 or about 82% of that required to reach the full strong-axis load.

Test CFF was essentially like CBB except that a suitable steel diaphragm was substituted for the aluminum diaphragm, following the suggestion of the project advisory committee. It merely confirmed the results of Test CBB, without showing any clear advantage of steel over aluminum diaphragm. However, the steel corrugated sheet was used for the remaining tests. In designing this specimen, it was decided to increase the number of fasteners in the high shear regions near the ends of the columns to prevent premature connector failure. However, it was not known at this time how greatly the connector spacing affects the shear rigidity of the sheet. Information on this point, developed later, showed that the lateral shear support from the diaphragm in this test CFF was at least double or triple that which was anticipated, largely due to the increased number of fasteners. On the other hand, the column capacity was not increased very much over that of column CBB, evidently due to CFF being inelastic at failure whereas CBB was still elastic at failure. It is easily shown that the decrease of overall column rigidity due to onset of inelasticity at critical sections requires a corresponding increase of bracing in order to reach the expected column load.

Tests CNO and CNN along with CFF were regarded as covering a range of high L/r ratio. A convenient width of steel diaphragm was used which, on the basis of the present elastic theory, was thought to provide lateral bracing in excess of that required for full support. Thus it was expected that the full strong-axis buckling load would be reached. In fact, far more bracing was provided than was realized at this time, due again to the connector effects. For all practical purposes, test CNN was fully braced and very nearly reached the full strong-axis load. By all observations, it was an excellent test.

Test CNO, however, having the smallest L/r should have reached a higher test load than CNN, but it did not, due from all indications to imperfect centering. Test CNO is considered not to be a valid test and is to be disregarded.

Test COO was identical to CNN except that the connector spacing was tripled on the presumption that the number of connectors used in previous tests was more than enough to develop the full shear modulus of the diaphragm. It was expected that the macimum load reached would be the same as that for CNN. However, the maximum load reached was only about half of that of test CNN. It was at this point that serious attention was given to determining the effect on diaphragm shear rigidity of various combinations of width and connector spacing. Tests to this end have been described under double-beam shear tests.

Test CPP was identical to COO for the purpose of checking reproducibility of results and to provide additional assurance that the centering procedure was satisfactory. This was accomplished. It is seen that excellent reproducibility was obtained.

Failure in all cases was gradual as can be seen from the plots in figures 16 through 21. Once the column had buckled by a small amount a characteristic violent secondary failure at the connectors near the ends took place with the pins popping out and the sheets tearing.

CONCLUSIONS

1) Diaphragms spanning between columns, at least within the limits of these tests, provide substantial lateral support. The column failure load can be greatly increased up to the strong-axis load, by such diaphragm.

2) Further information on shear rigidity of sheets as affected by width and connector spacing is required in order to predict theoretically the lateral bracing contribution of any given diaphragm arrangement.

3) The present elastic theory must be extended to the inelastic case, in order to be able to predict more closely the diaphragm contribution to the capacity of non-slender columns. More lateral bracing must be provided when column inelasticity occurs than when the column remains merely elastic at failure. This is easily shown by the simple analysis of an idealized laterally supported column model. The member shown in figure 28 represents a perfect column with centrally applied end loads P. The column is assumed to consist of two infinitely rigid parts connected by a central coil spring, of specific rotational resistance B. This spring is the lumped flexural rigidty of the true column. A linear spring of specific resistance K representing some lateral support, (in this case the lumped shear resistance of the diaphragm) is attached to the center of the member. Assuming small deflections, the strain energy stored in the coil spring as loading progresses is:

$$Us = (M/2)\ (2\theta) = (B\theta)\ (2\theta) = 2B\theta^2$$

but $\theta = 2d/L$

thus, $Us = 8Bd^2/L^2$

The external work of the loads is:

$$W_P = 2P\Delta/2 = P\Delta$$

but $$/2 = a/2 - L/2 = L(d/L)^2$$

and $$W_P = P.2L(d/L)^2 = 2Pd^2/L$$

The strain energy stored in the linear spring is:

$$U_L = Kd^2/2$$

Equating the strain energy stored in the system to the external work:

$$U_S + U_L = W_P$$

whence $$P = 4B/L + KL/4$$

It is clear that in order to maintain a given load P, the lateral support K must be increased in proportion as B decreases due to the onset of inelas ticity.

FUTURE PLANS

It is expected that the experimental program will proceed as follows:

1) Additional double-beam tests to establish sufficient experimental information on the effect of sheet width and spacing of connectors on diaphragm rigidity. This information is necessary in order to predict more closely the maximum loads of the double-column assemblies.

2) Additional double-column tests with sheet on two faces of the assembly in which both P_{xx} and P_{yy} are within the elastic range. This condition possibly can be realized by placing the knife-edges parallel to the flanges. The present theoretical development appears adequate to predict closely the load increase of the assembly due to shear action of the diaphragms if the columns are elastic when failure develops.

3) Double-columns with diaphragm on one face only, corresponding to the more realistic situation, are to be performed. The elastic range will be investigated first; the present elastic hteory has been extended to this

case. Since in the single sheet assembly there is no longer symmetry, as in the case of sheet on both sides, twist will generally accompany the lateral buckling of the member at failure. The effect of this twisting action appears to be minor incomparison with the lateral deflection at failure. Thus, the predicted load, based on symmetric conditions, seems to give an approximate value for the true failure load. Work is currently in progress to verify this supposition which is to be substantiated by actual test results.

In addition to the experimental work outlined above, an attempt will be made to extend the present theory to the inelastic range.

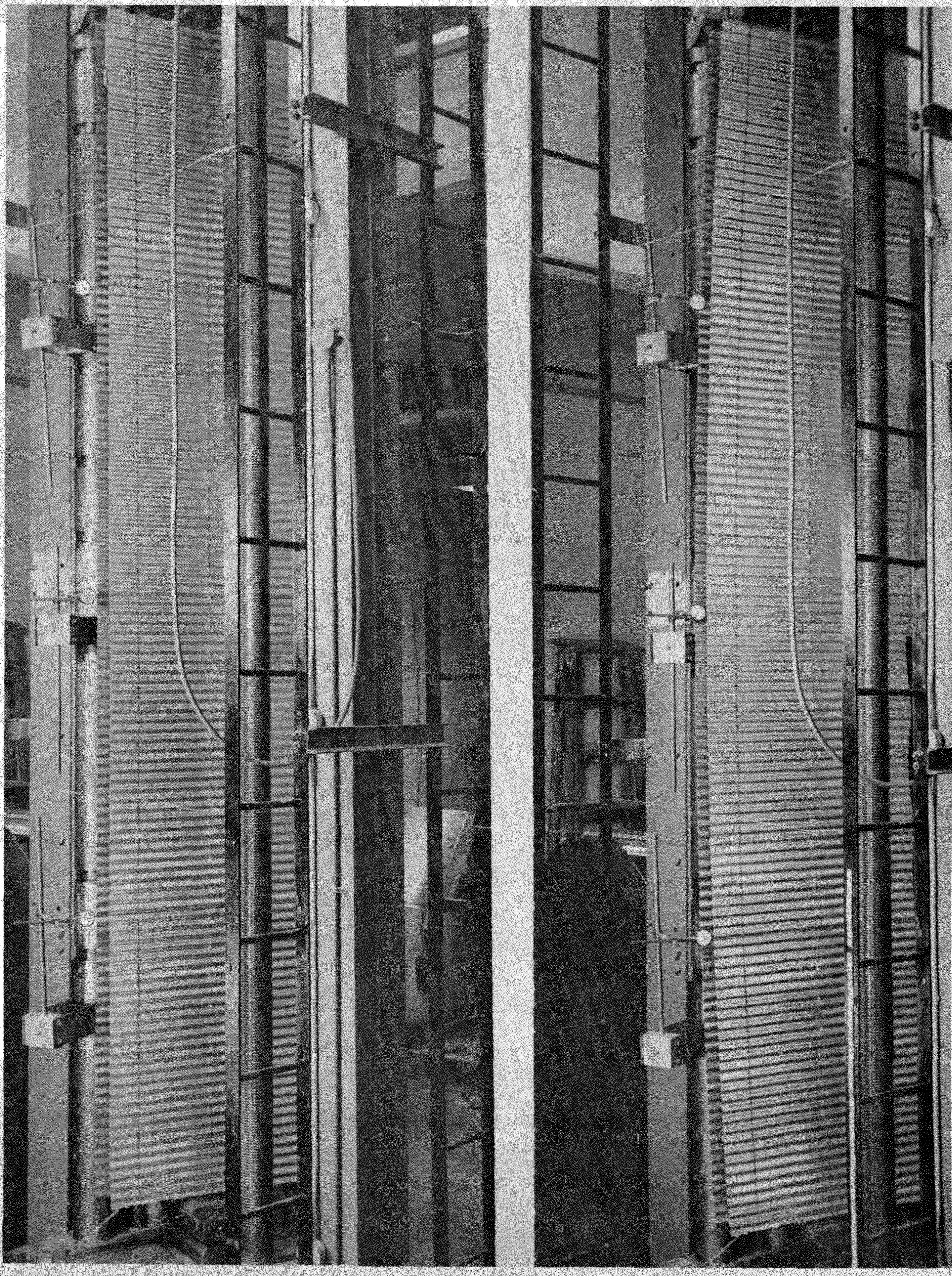


FIGURE 1

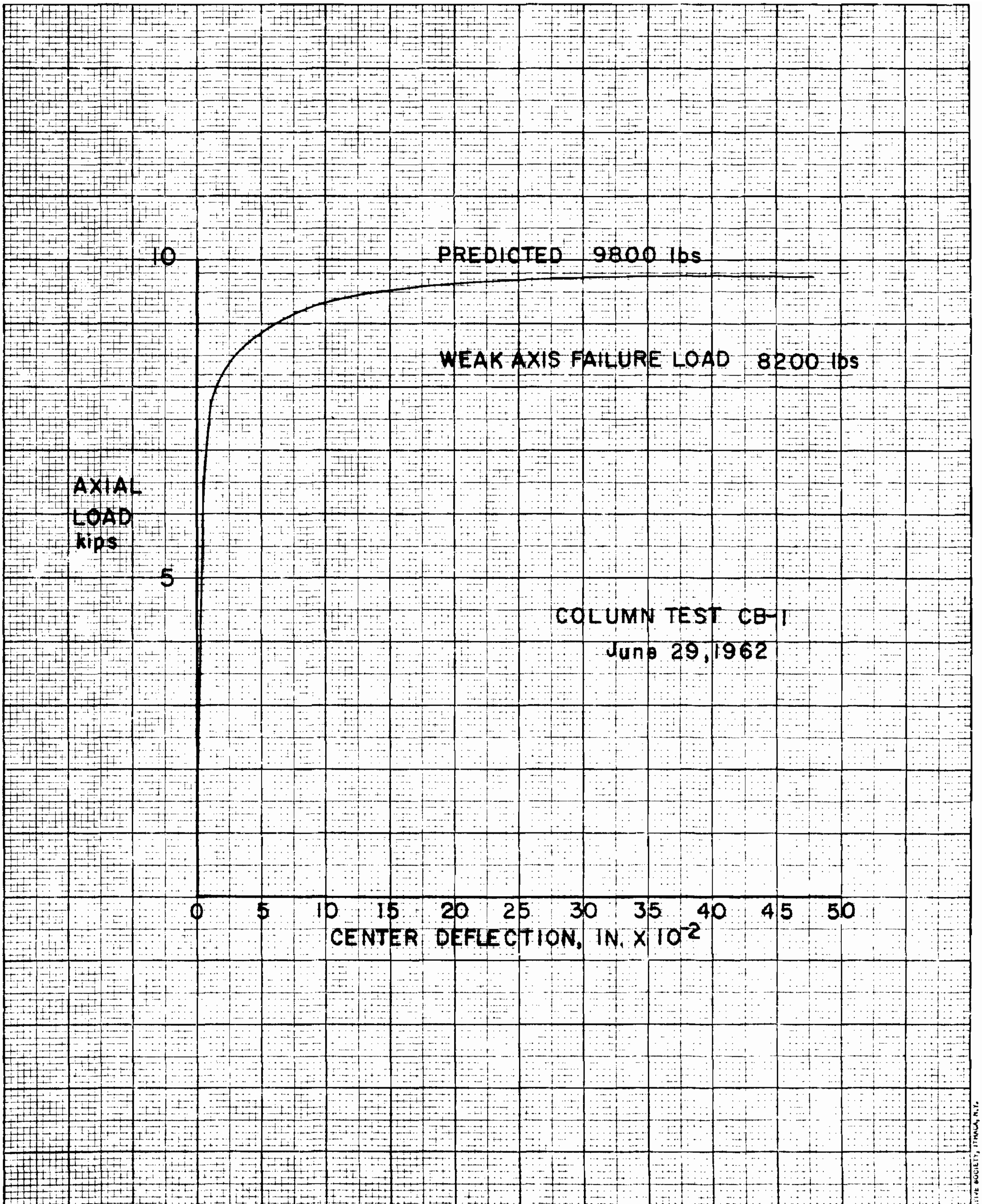
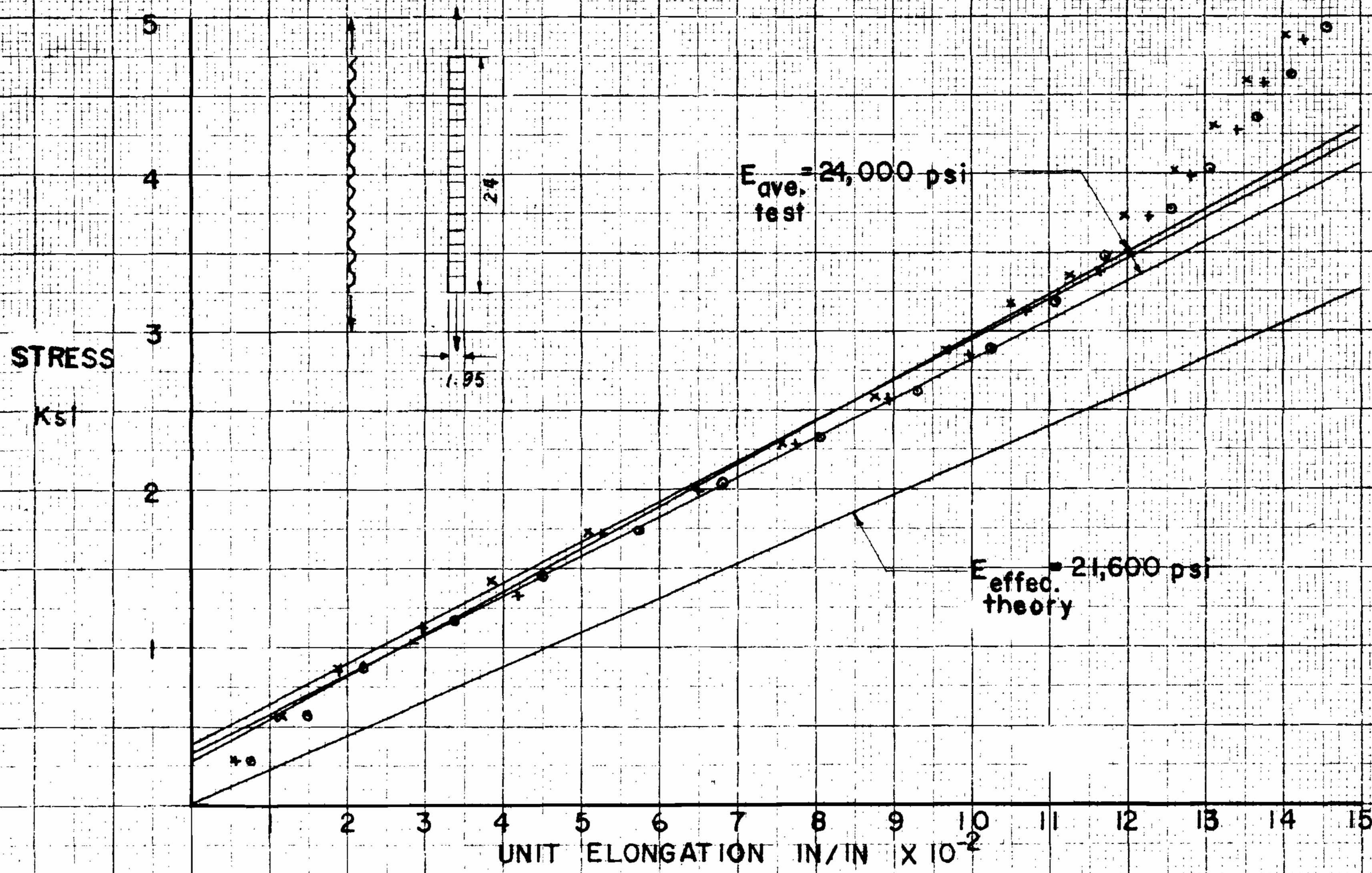

PREDICTED 9800 lbs
WEAK AXIS FAILURE LOAD 8200 lbs
AXIAL
LOAD
kips
10
5
COLUMN TEST CB-1
June 29, 1962
0
5
10
15
20
25
30
35
40
45
50
CENTER DEFLECTION, IN. X 10^-2

FIGURE 2



GRANCO PLENUM COUPONS

WEAK DIRECTION

26 GAGE 0.5 DEEP

August 1962

FIGURE 3

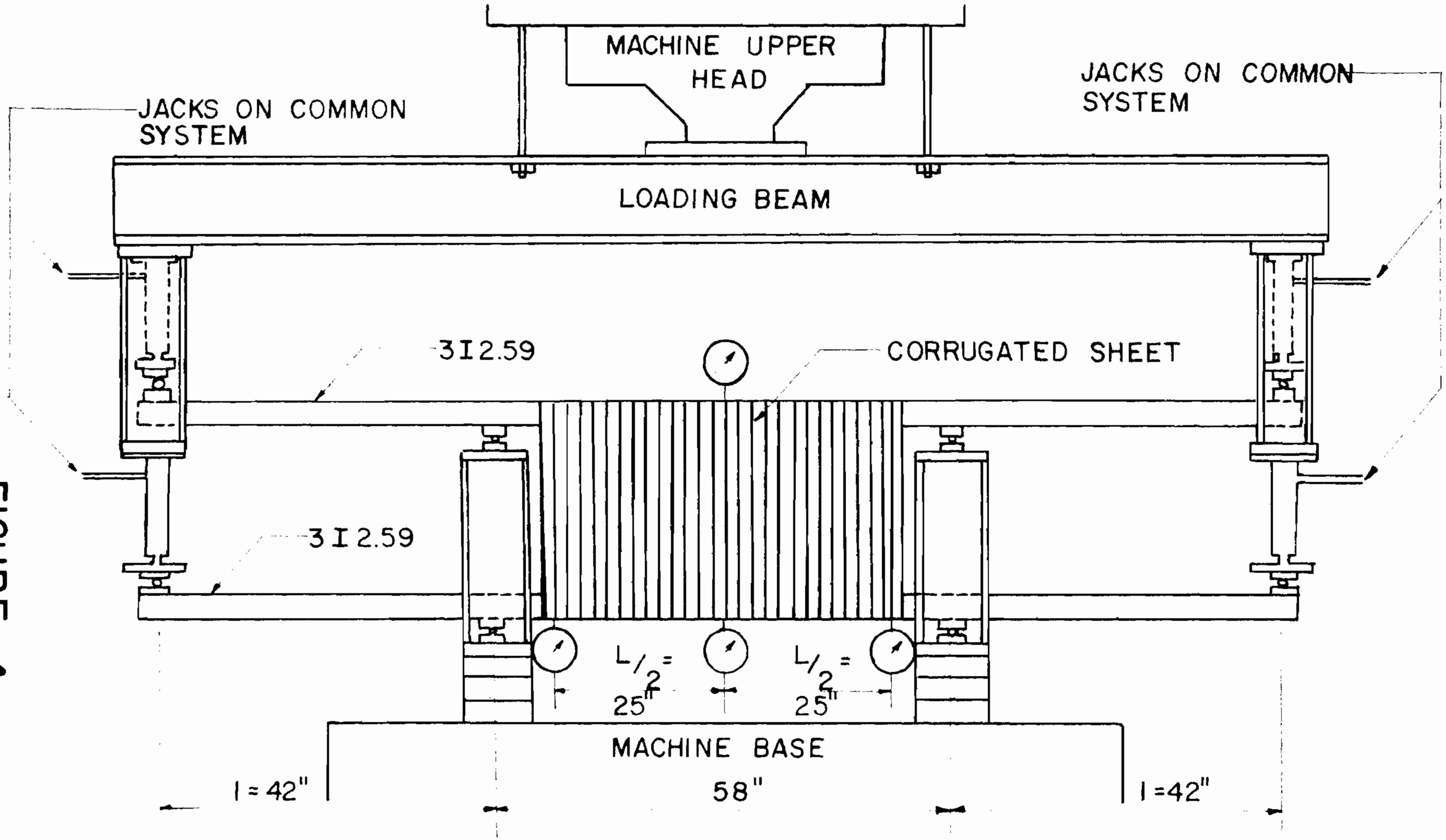


FIGURE 4

DOUBLE BEAM SHEAR TEST SETUP

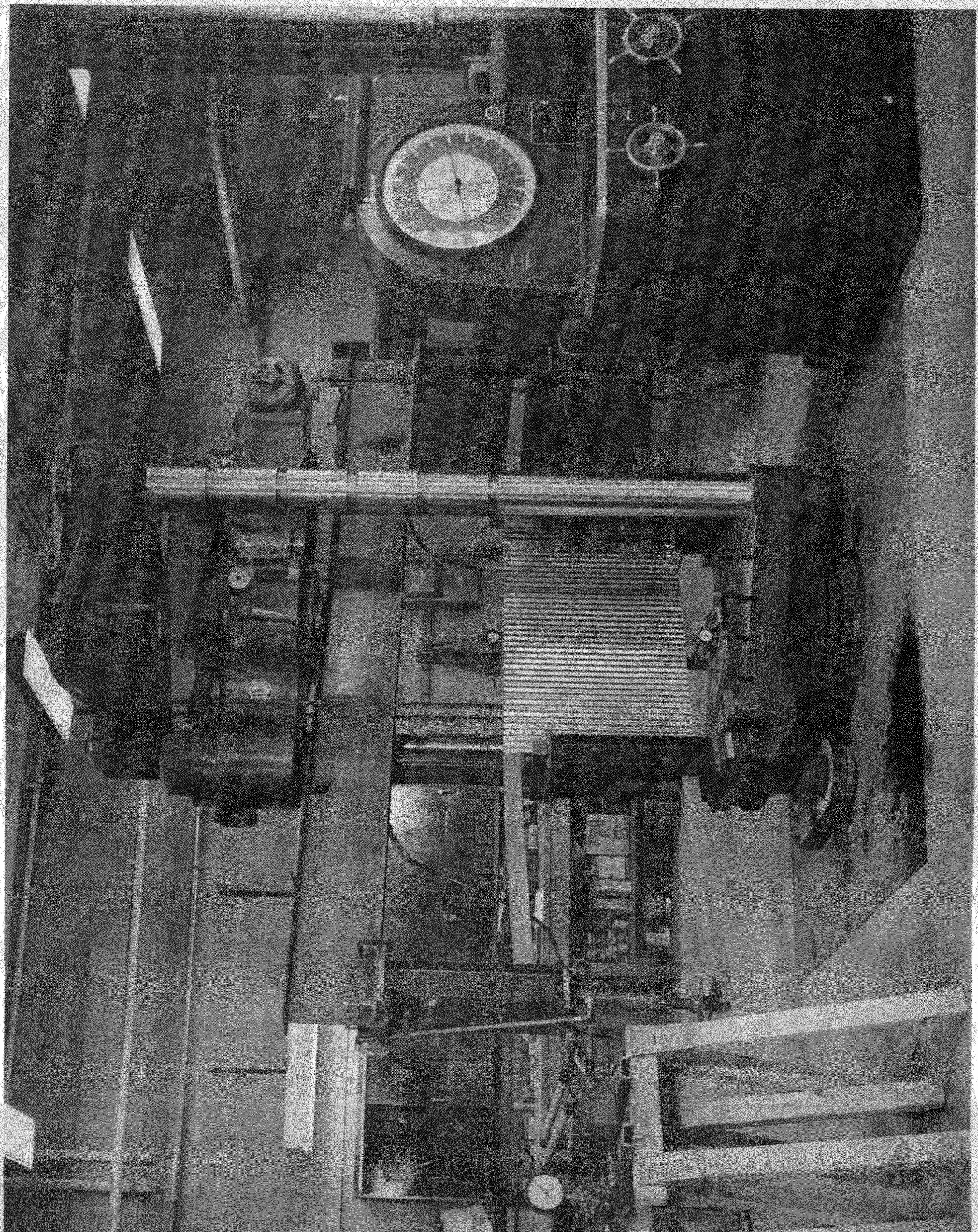


FIGURE 5

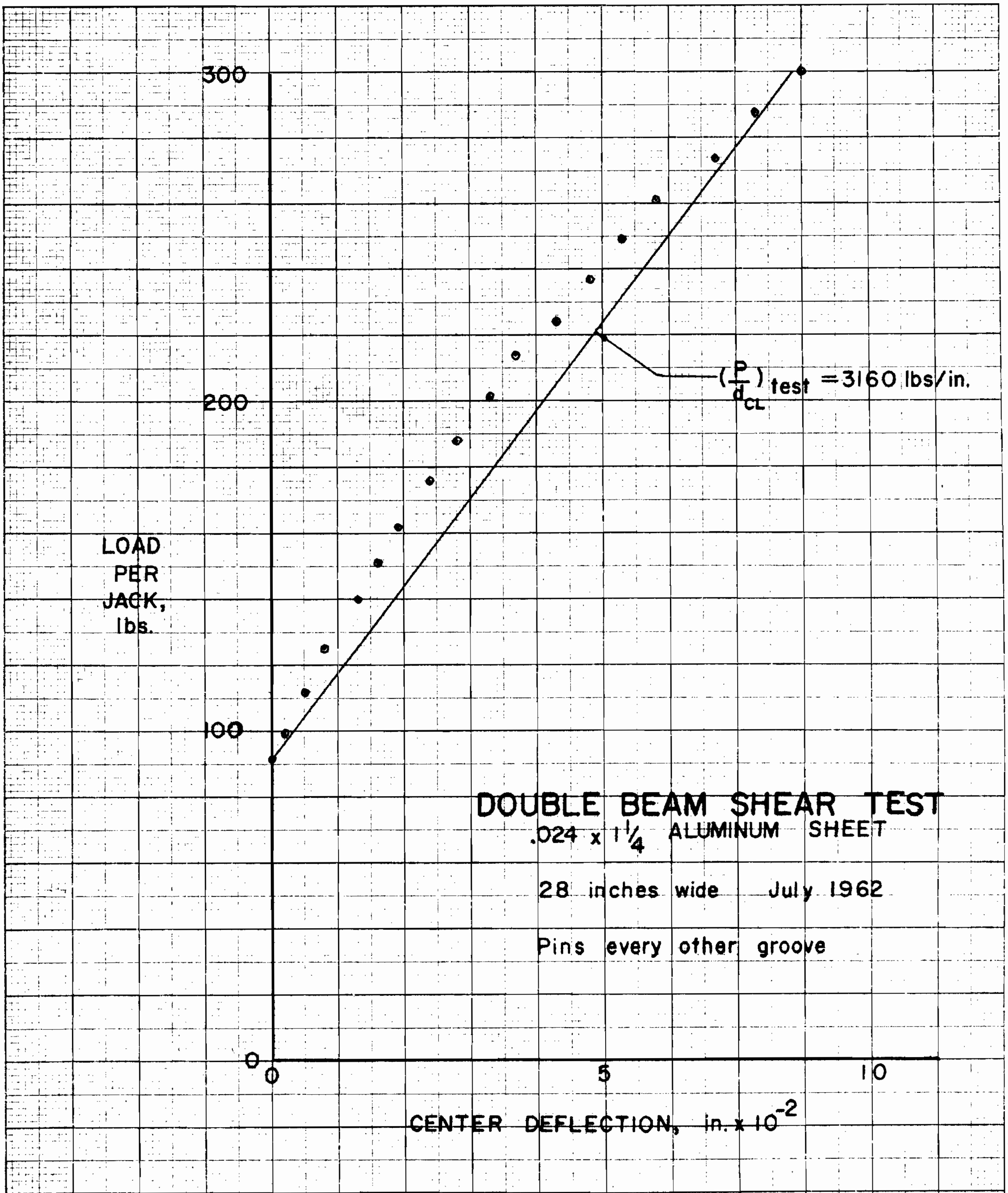


FIGURE 6

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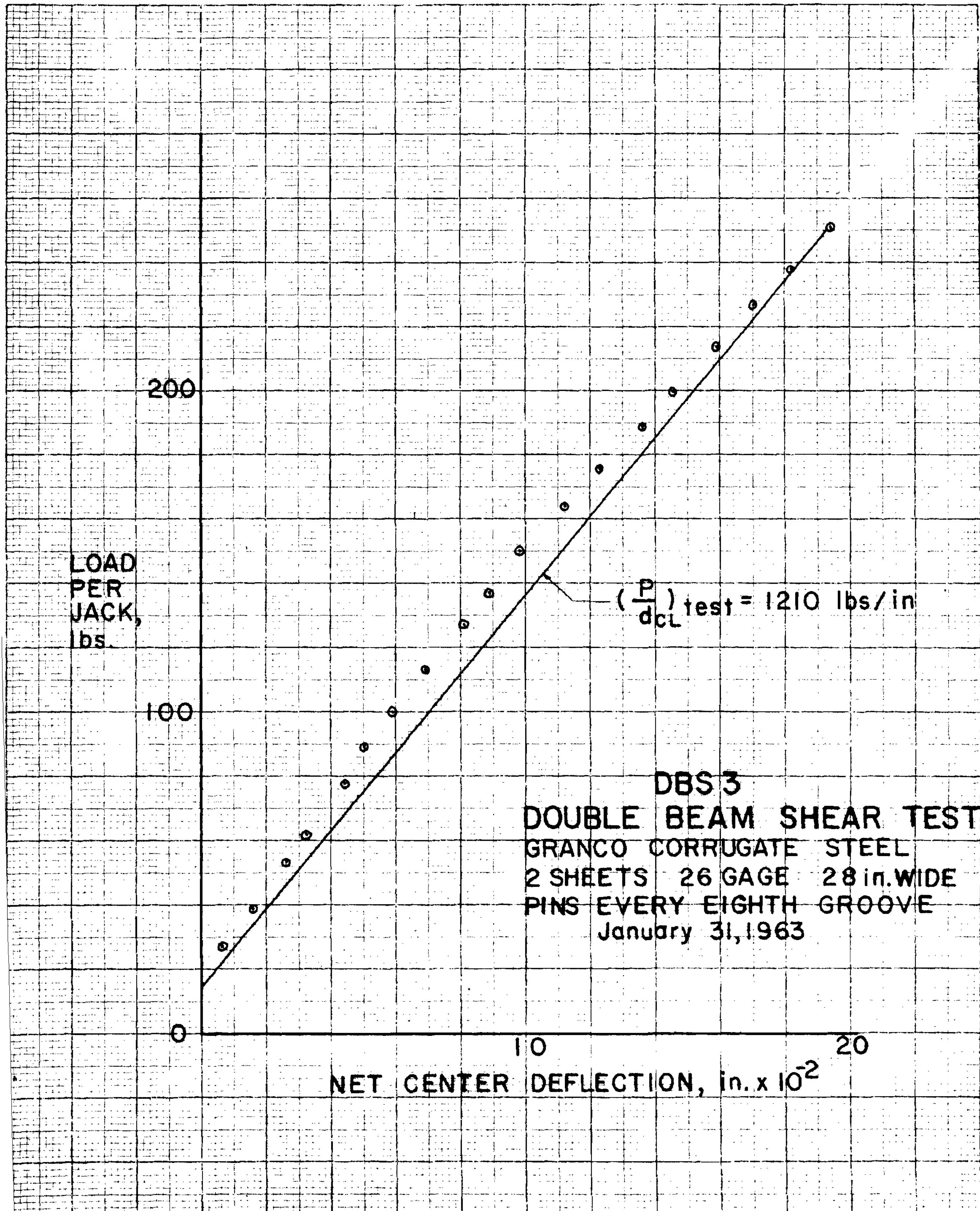
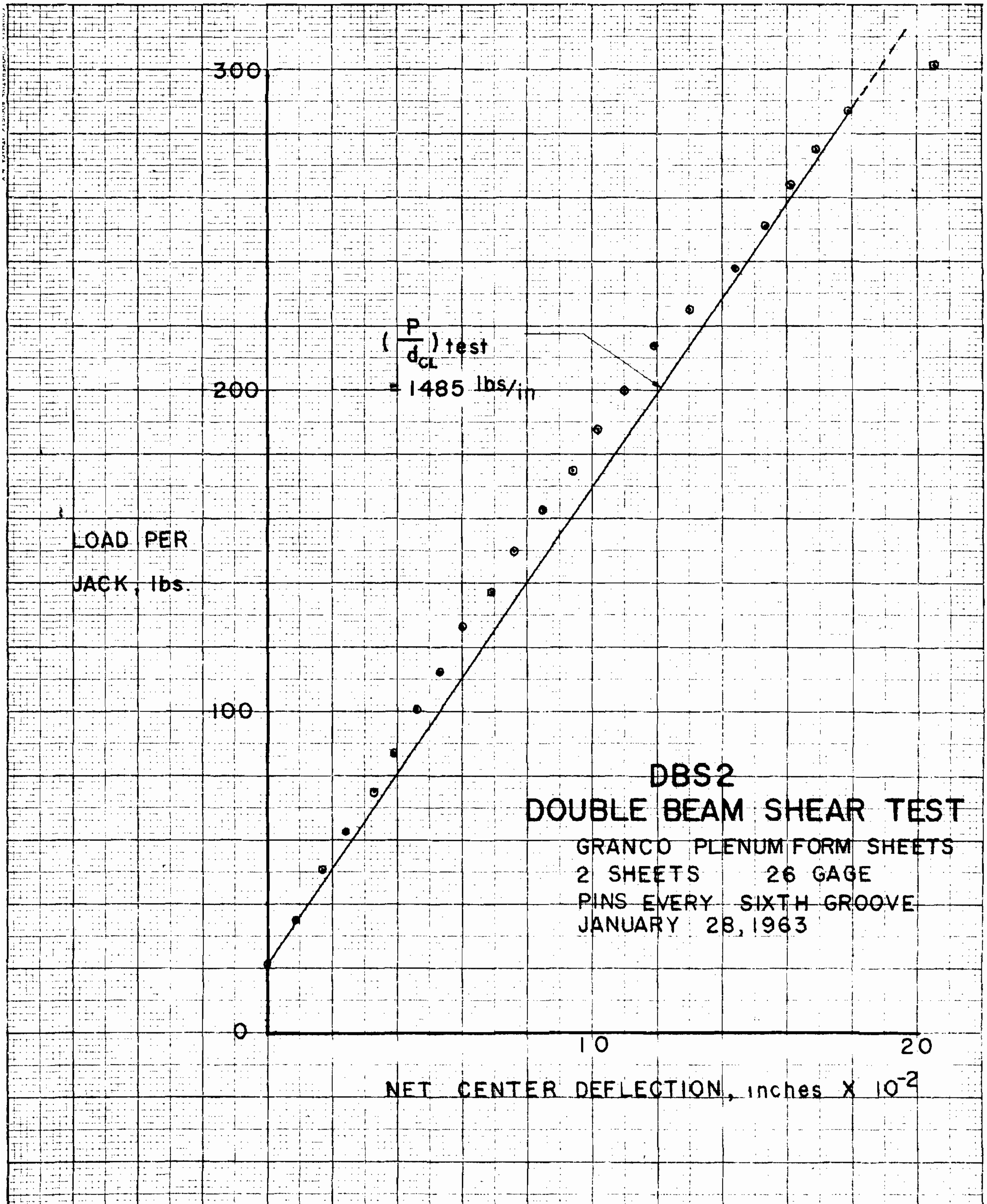


LOAD PER JACK, lbs.
200
100
0
10
20
NET CENTER DEFLECTION, in. x 10^-2
(P / d_CL) test = 1210 lbs/in
DBS 3
DOUBLE BEAM SHEAR TEST
GRANCO CORRUGATE STEEL
2 SHEETS 26 GAGE 28 in. WIDE
PINS EVERY EIGHTH GROOVE
January 31, 1963

FIGURE 7



300
200
100
0
10
20
LOAD PER
JACK, lbs.
NET CENTER DEFLECTION, inches X 10^{-2}
$\left(\frac{P}{d_{CL}}\right)_{test}$
= 1485 lbs/in
DBS2
DOUBLE BEAM SHEAR TEST
GRANCO PLENUM FORM SHEETS
2 SHEETS 26 GAGE
PINS EVERY SIXTH GROOVE
JANUARY 28, 1963

FIGURE 8

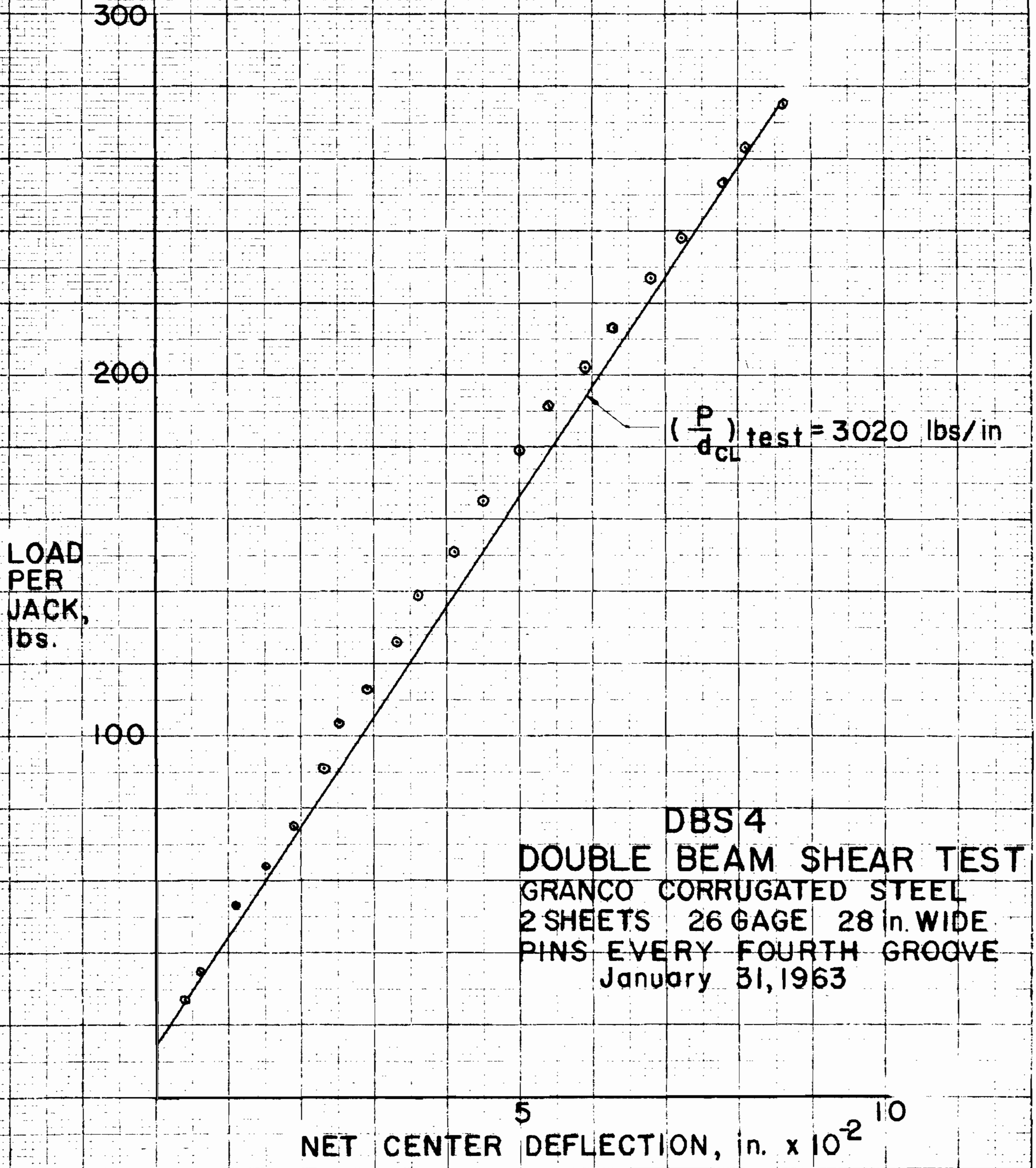

300
200
100
LOAD PER JACK, lbs.
$\left(\frac{P}{d_{CL}}\right)_{test}$ = 3020 lbs/in
DBS 4
DOUBLE BEAM SHEAR TEST
GRANCO CORRUGATED STEEL
2 SHEETS 26 GAGE 28 in. WIDE
PINS EVERY FOURTH GROOVE
January 31, 1963
5
10
NET CENTER DEFLECTION, in. x 10^{-2}

FIGURE 9

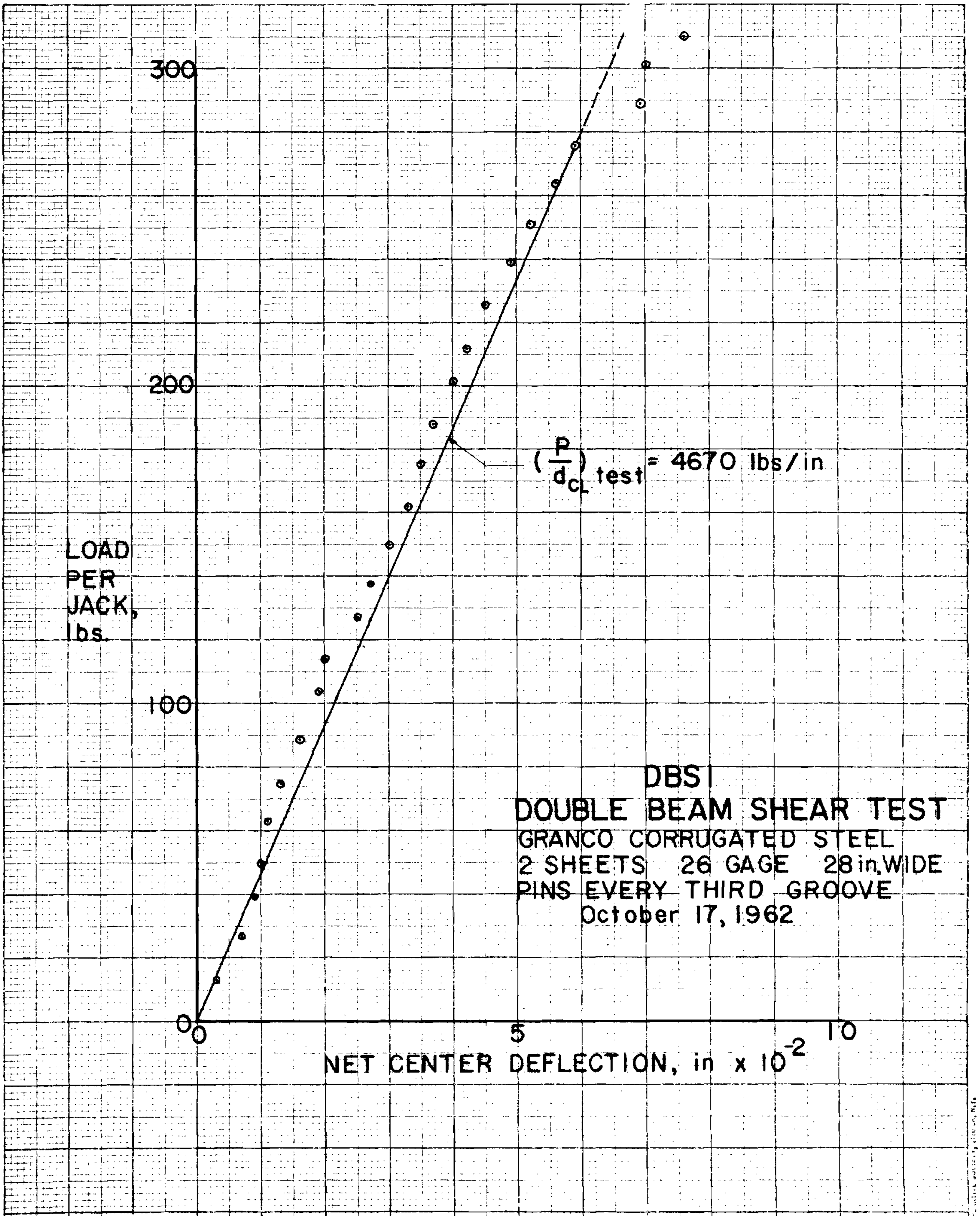

300
200
100
0
LOAD PER JACK, lbs.
0
5
10
NET CENTER DEFLECTION, in x 10^{-2}
$(\frac{P}{d_{CL}})_{test}$ = 4670 lbs/in
DBS1
DOUBLE BEAM SHEAR TEST
GRANCO CORRUGATED STEEL
2 SHEETS 26 GAGE 28 in. WIDE
PINS EVERY THIRD GROOVE
October 17, 1962

FIGURE 10

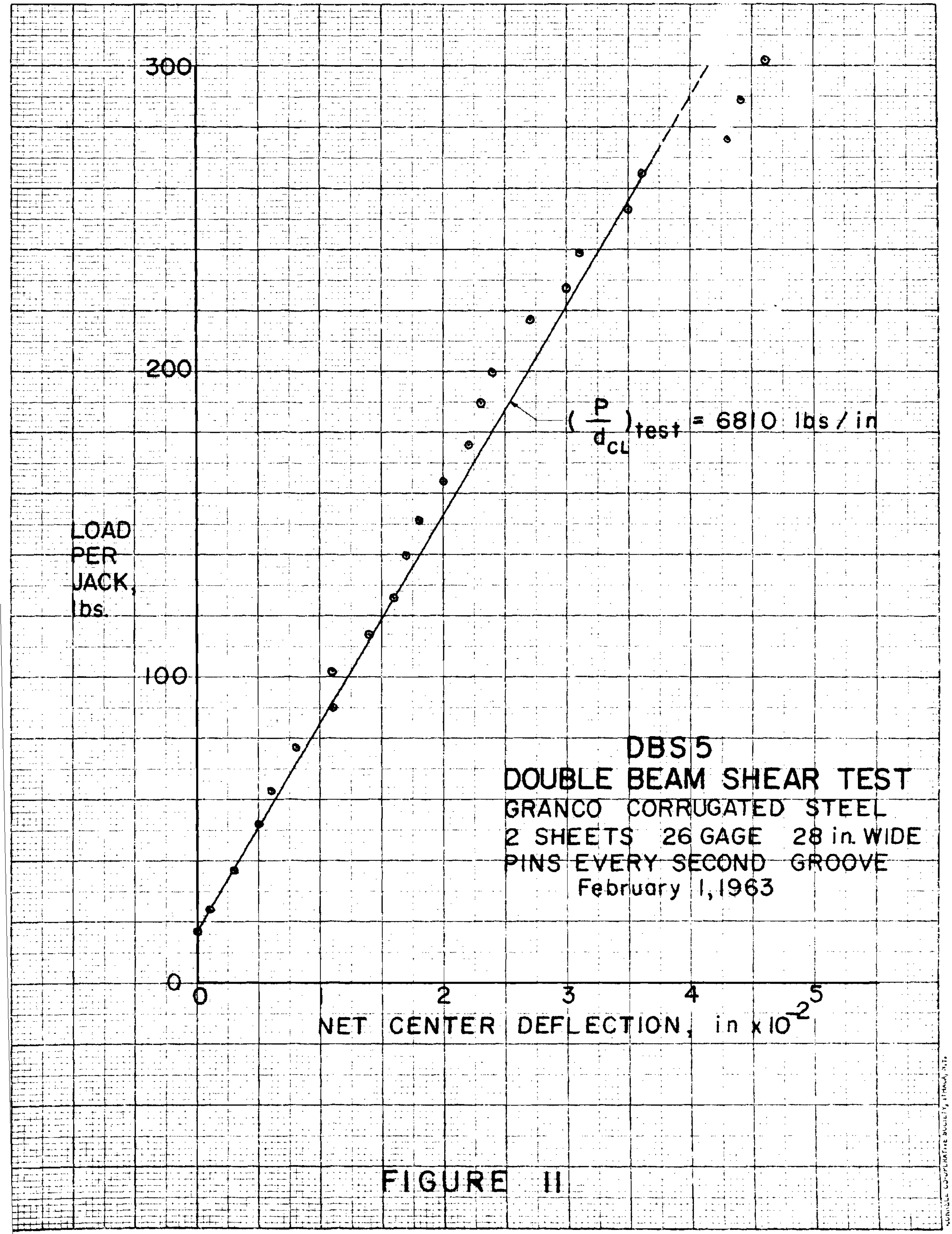

300
200
100
0
LOAD
PER
JACK,
lbs.
0
1
2
3
4
5
NET CENTER DEFLECTION, in x 10^{-2}
$\left(\frac{P}{d_{CL}}\right)_{test}$ = 6810 lbs / in
DBS 5
DOUBLE BEAM SHEAR TEST
GRANCO CORRUGATED STEEL
2 SHEETS 26 GAGE 28 in. WIDE
PINS EVERY SECOND GROOVE
February 1, 1963

FIGURE 11

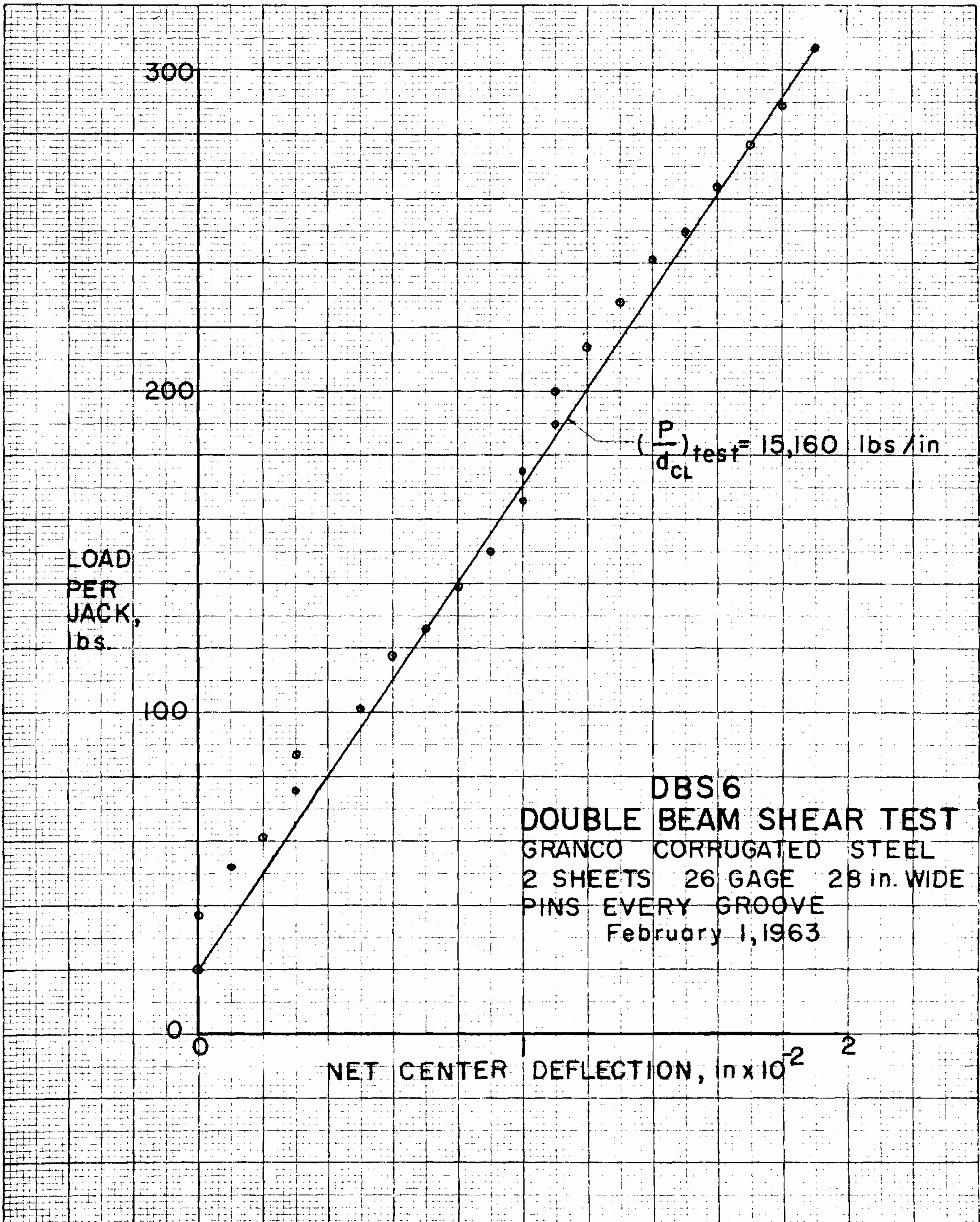
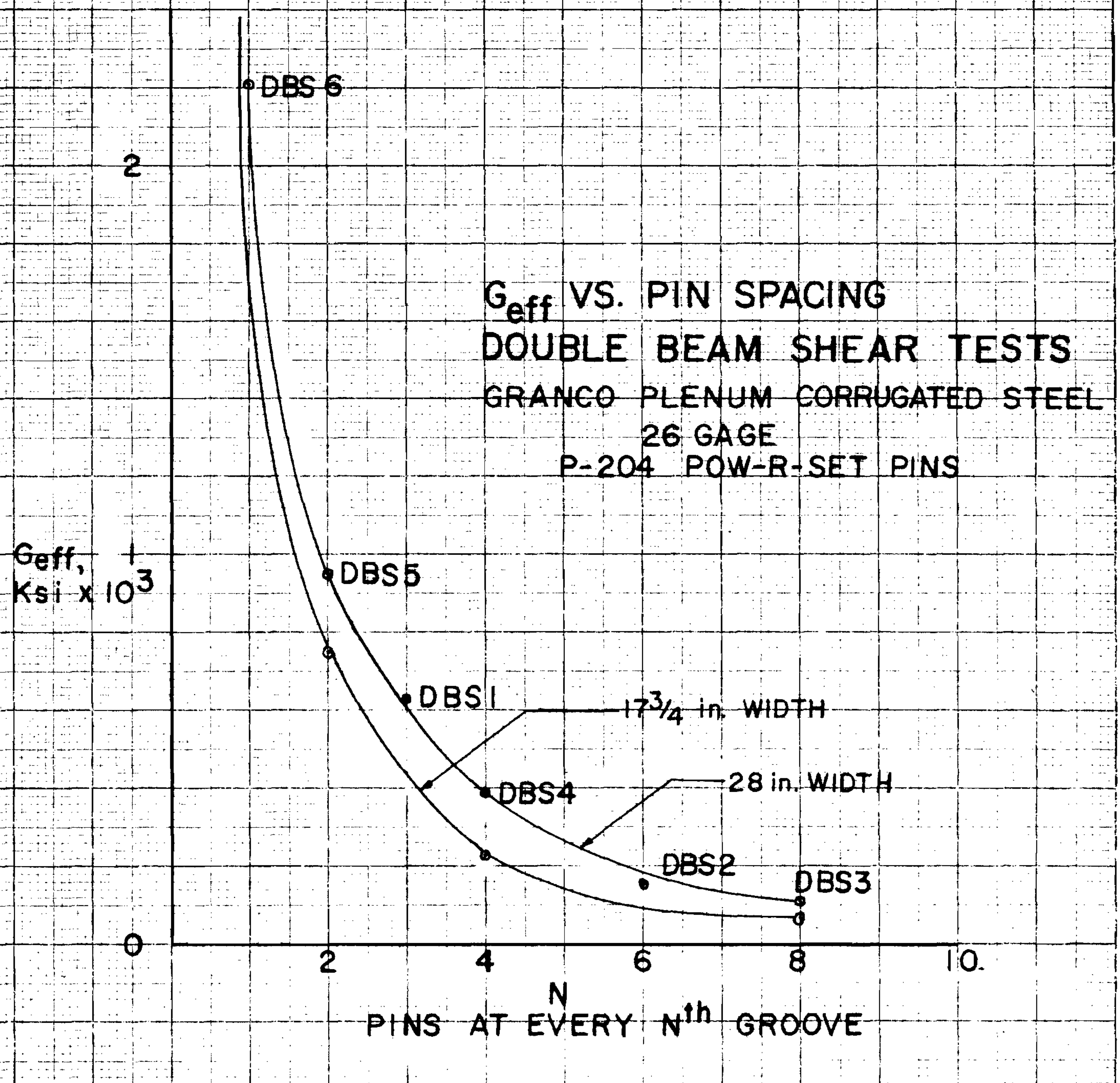
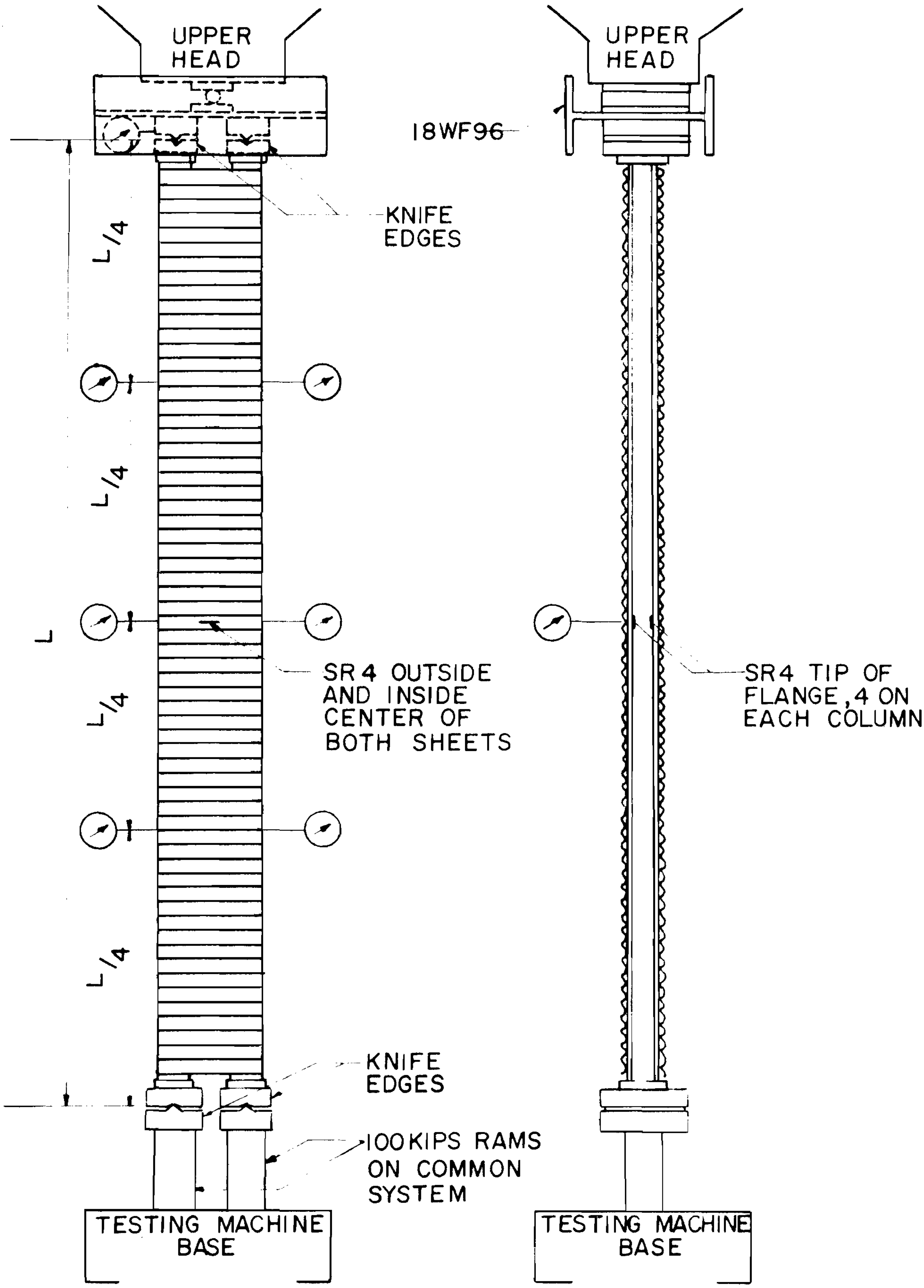

300
200
100
0
LOAD PER JACK, lbs.
$(\frac{P}{d_{CL}})_{test}$ = 15,160 lbs/in
DBS 6
DOUBLE BEAM SHEAR TEST
GRANCO CORRUGATED STEEL
2 SHEETS 26 GAGE 28 in. WIDE
PINS EVERY GROOVE
February 1, 1963
0
1
2
NET CENTER DEFLECTION, in x 10^{-2}

FIGURE 12



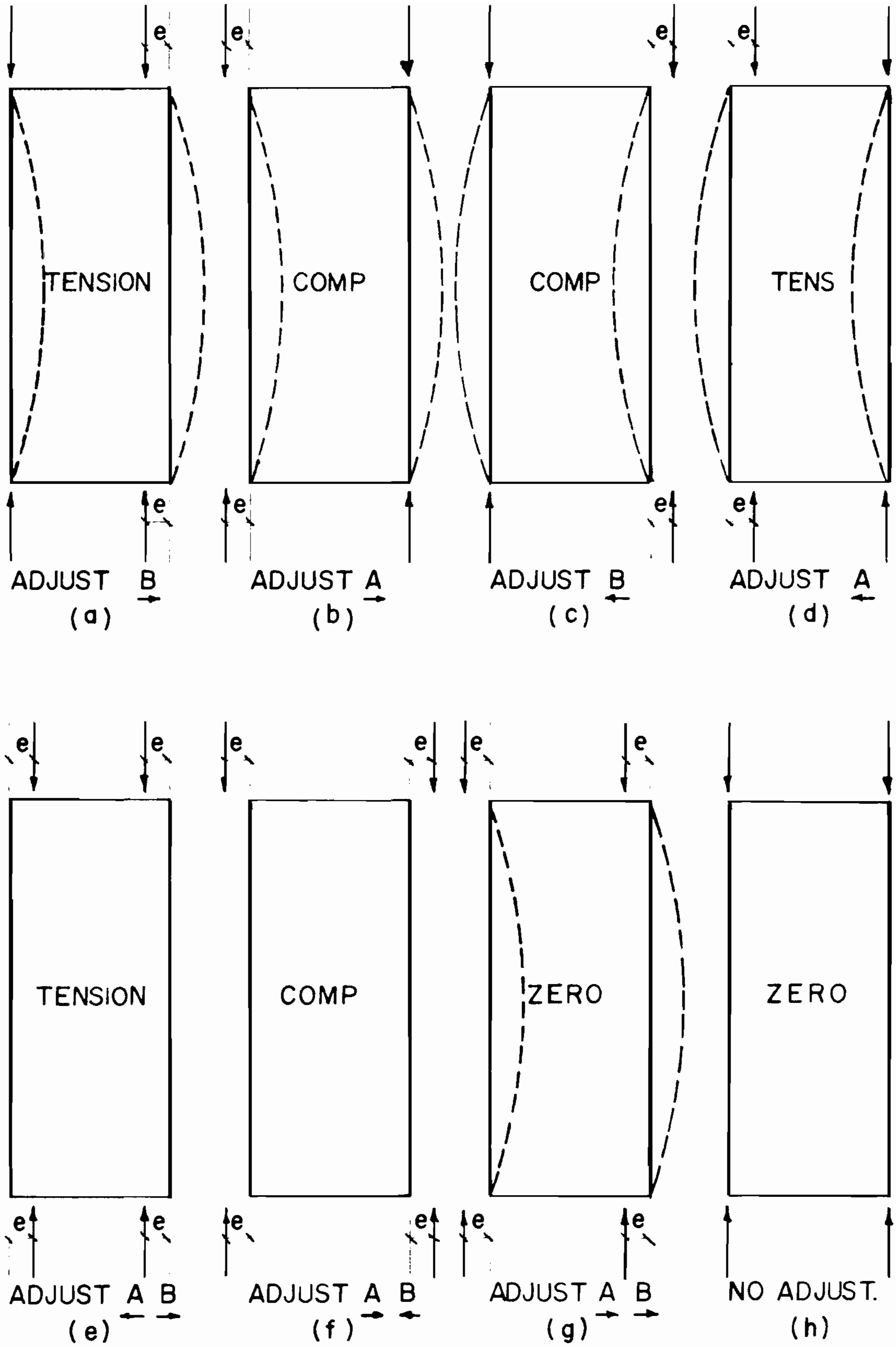
G_{eff} VS. PIN SPACING
DOUBLE BEAM SHEAR TESTS
GRANCO PLENUM CORRUGATED STEEL
26 GAGE
P-204 POW-R-SET PINS
G_{eff}, Ksi x 10^3
2
1
0
DBS 6
DBS5
DBS1
DBS4
DBS2
DBS3
$17\frac{3}{4}$ in. WIDTH
28 in. WIDTH
2
4
6
8
10.
N
PINS AT EVERY N^{th} GROOVE

FIGURE 13



DOUBLE COLUMN TEST SETUP

FIGURE 14



DOUBLE COLUMN CENTERING

FIGURE 15

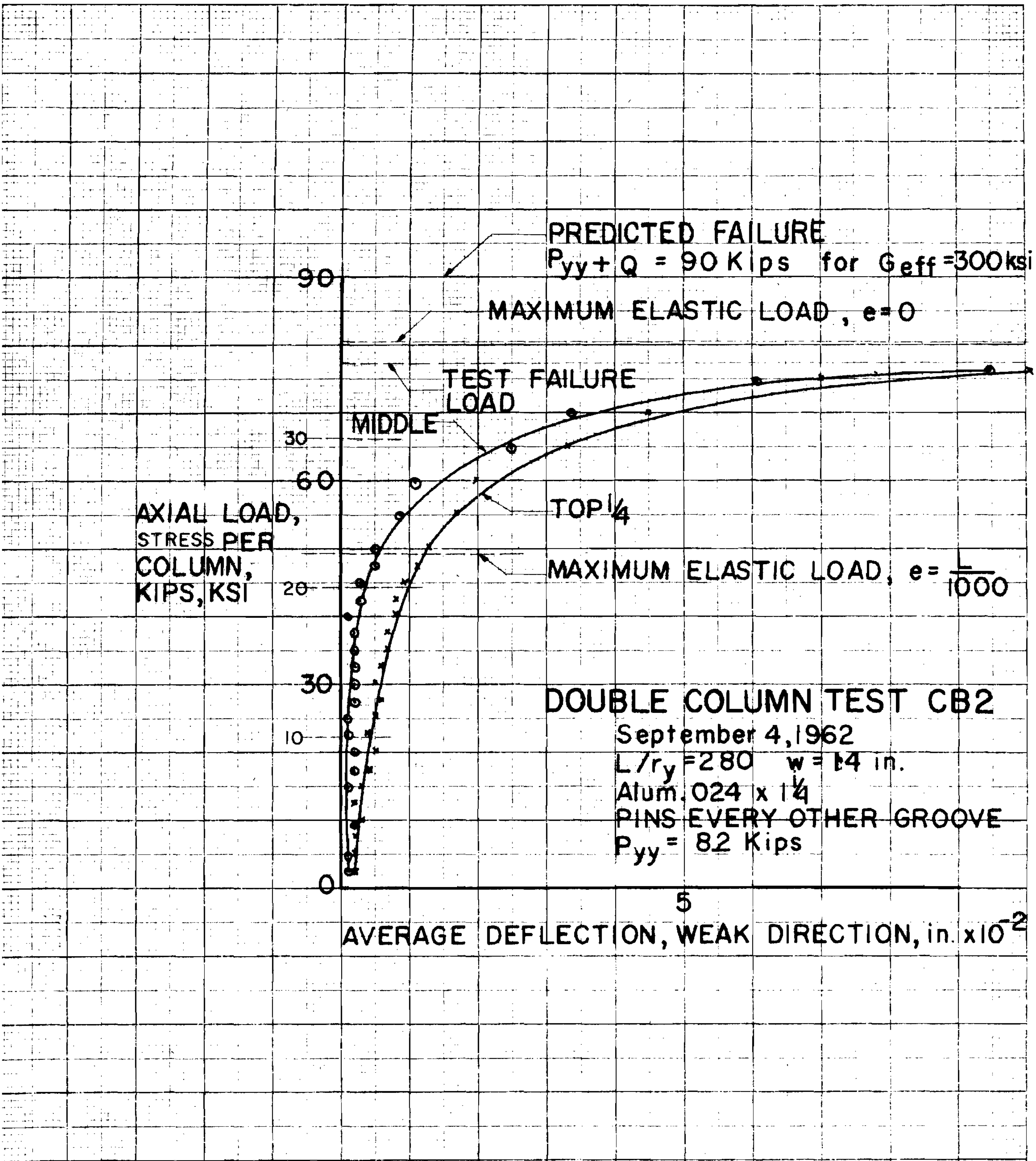

PREDICTED FAILURE
Pyy + Q = 90 Kips for Geff = 300 ksi
MAXIMUM ELASTIC LOAD, e = 0
TEST FAILURE LOAD
MIDDLE
TOP 1/4
MAXIMUM ELASTIC LOAD, e = L/1000
AXIAL LOAD, STRESS PER COLUMN, KIPS, KSI
90
60
30
0
30
20
10
5
AVERAGE DEFLECTION, WEAK DIRECTION, in. x10^-2
DOUBLE COLUMN TEST CB2
September 4, 1962
L/ry = 280 w = 14 in.
Alum. .024 x 1/4
PINS EVERY OTHER GROOVE
Pyy = 82 Kips

FIGURE 16

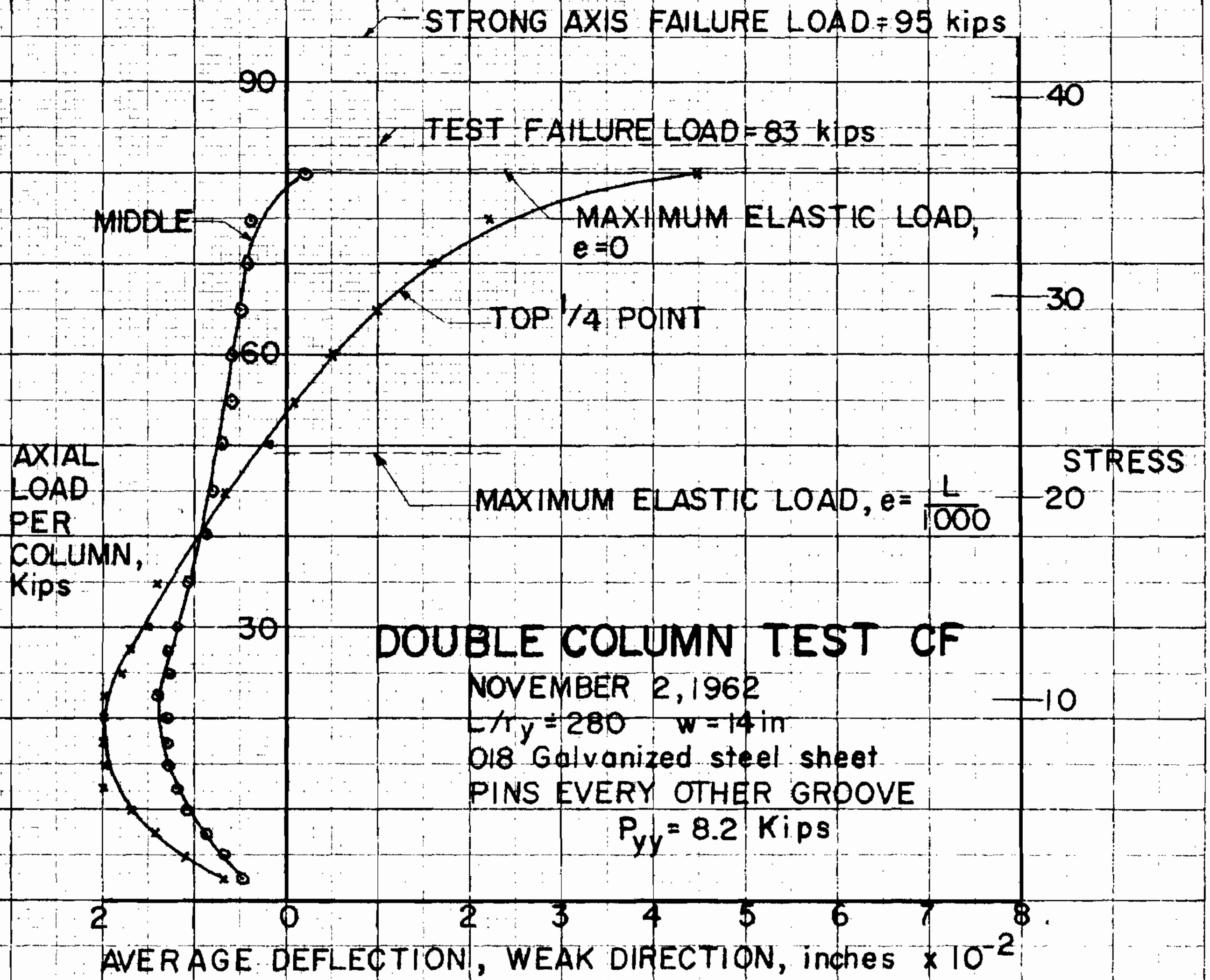

STRONG AXIS FAILURE LOAD = 95 kips
TEST FAILURE LOAD = 83 kips
MIDDLE
MAXIMUM ELASTIC LOAD, e = 0
TOP 1/4 POINT
AXIAL LOAD PER COLUMN, Kips
STRESS
MAXIMUM ELASTIC LOAD, e = L/1000
DOUBLE COLUMN TEST CF
NOVEMBER 2, 1962
L/ry = 280 w = 14 in
.018 Galvanized steel sheet
PINS EVERY OTHER GROOVE
Pyy = 8.2 Kips
90
60
30
40
30
20
10
2
0
2
3
4
5
6
7
8
AVERAGE DEFLECTION, WEAK DIRECTION, inches x 10^-2

FIGURE 17

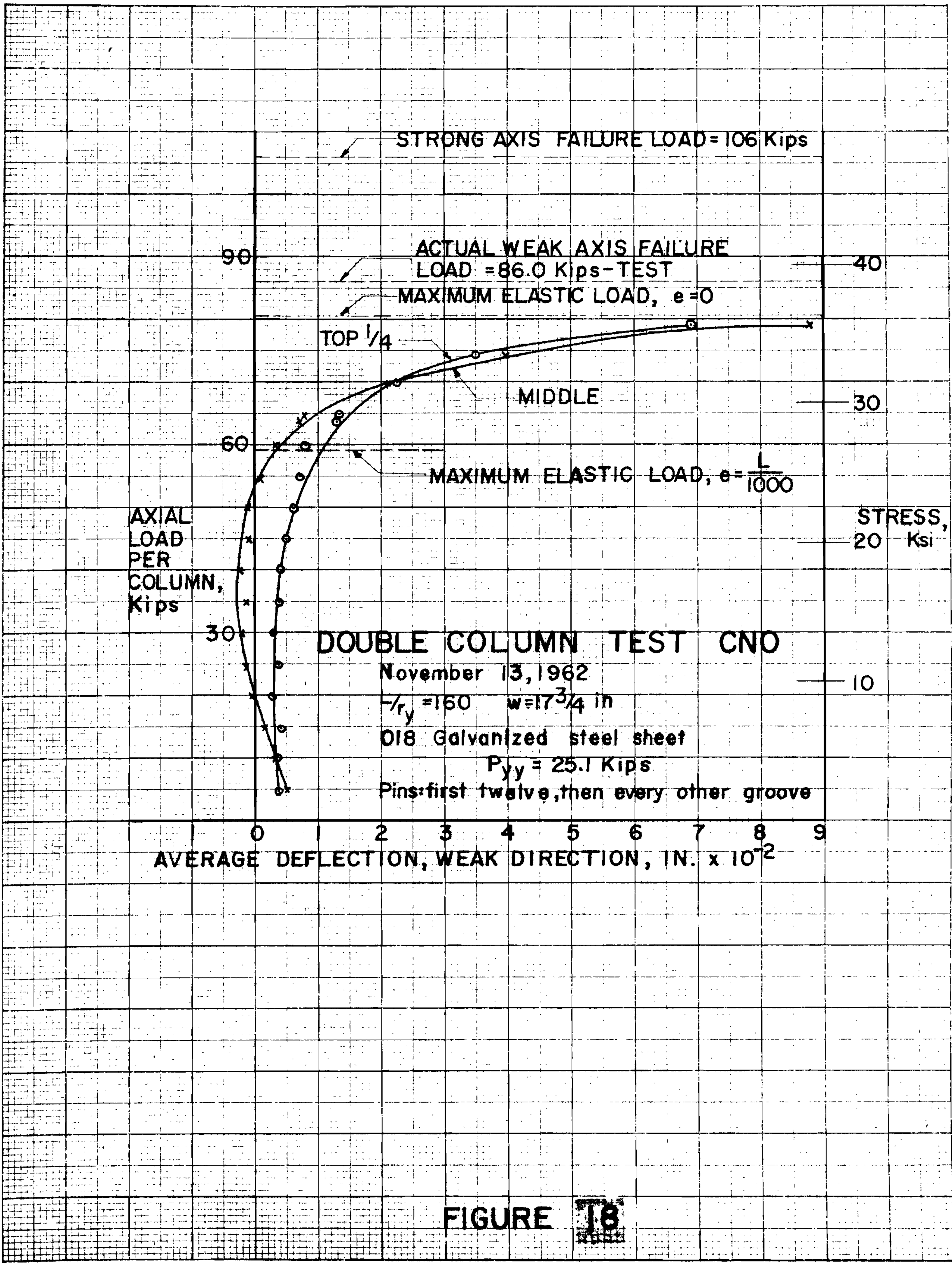

STRONG AXIS FAILURE LOAD = 106 Kips
ACTUAL WEAK AXIS FAILURE
LOAD =86.0 Kips-TEST
MAXIMUM ELASTIC LOAD, e=0
TOP 1/4
MIDDLE
MAXIMUM ELASTIC LOAD, e= L/1000
AXIAL LOAD PER COLUMN, Kips
STRESS, Ksi
90
60
30
40
30
20
10
DOUBLE COLUMN TEST CNO
November 13, 1962
L/ry =160 w=17 3/4 in
018 Galvanized steel sheet
Pyy = 25.1 Kips
Pins: first twelve, then every other groove
1
0
1
2
3
4
5
6
7
8
9
AVERAGE DEFLECTION, WEAK DIRECTION, IN. x 10^-2

FIGURE 18

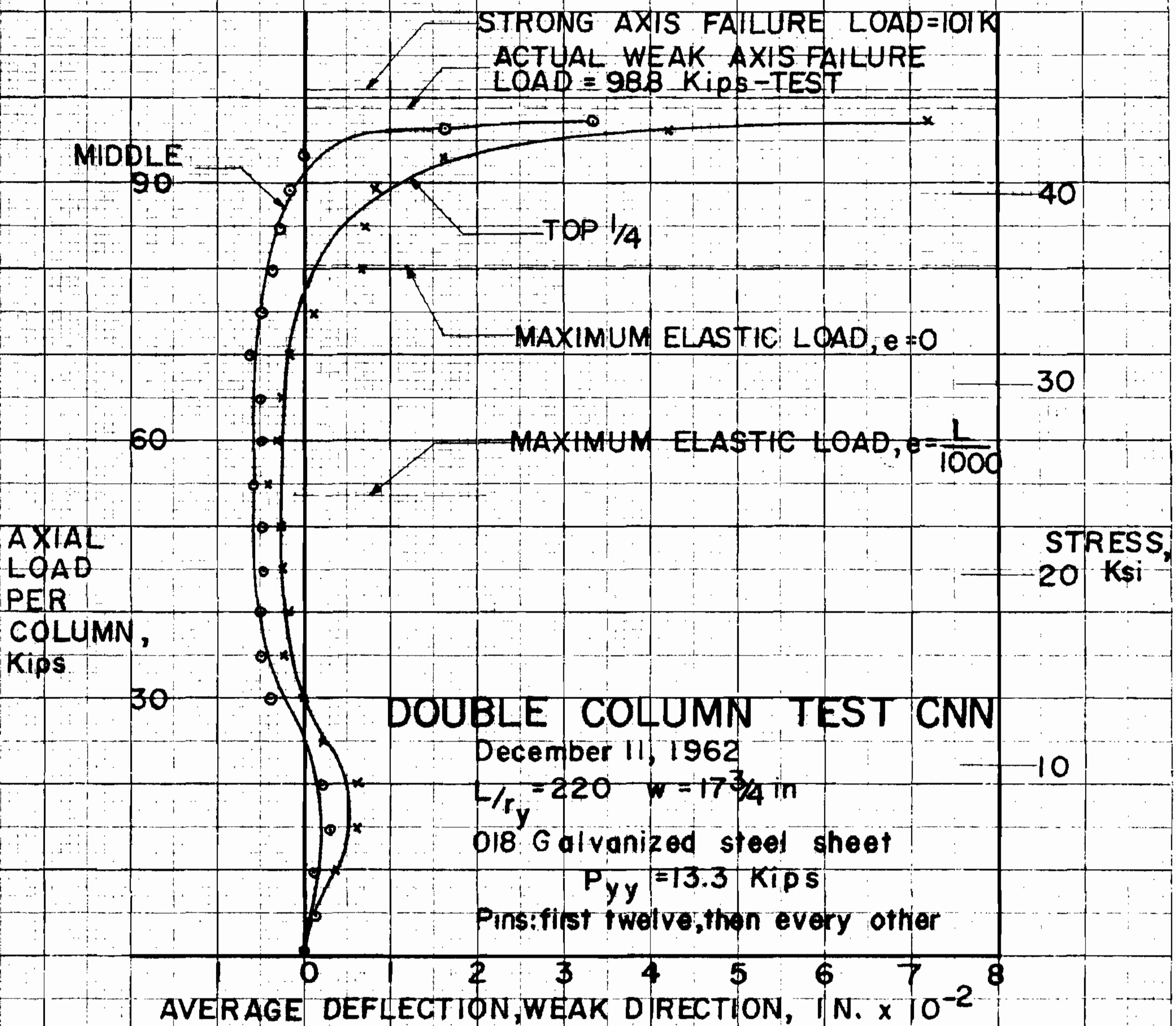
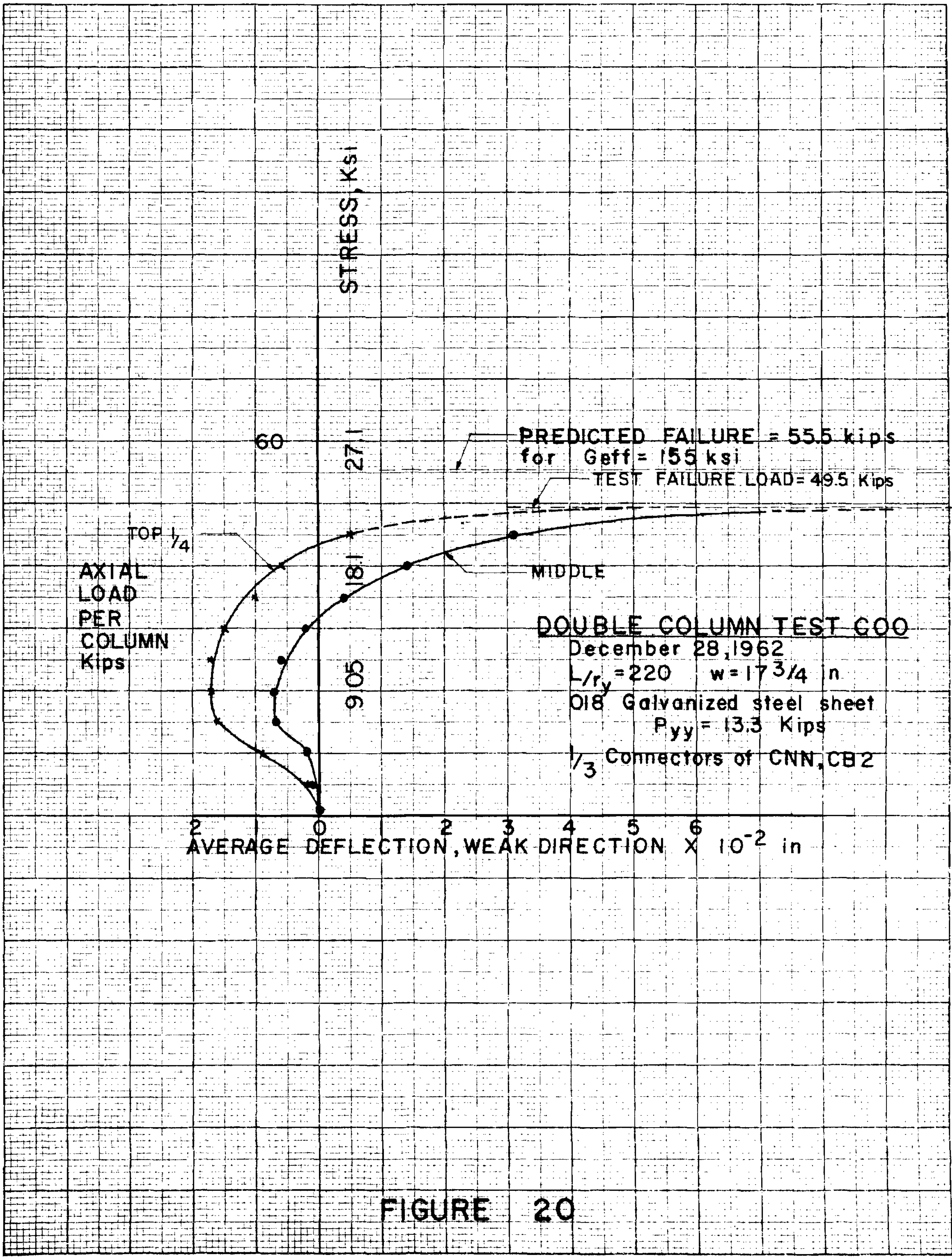
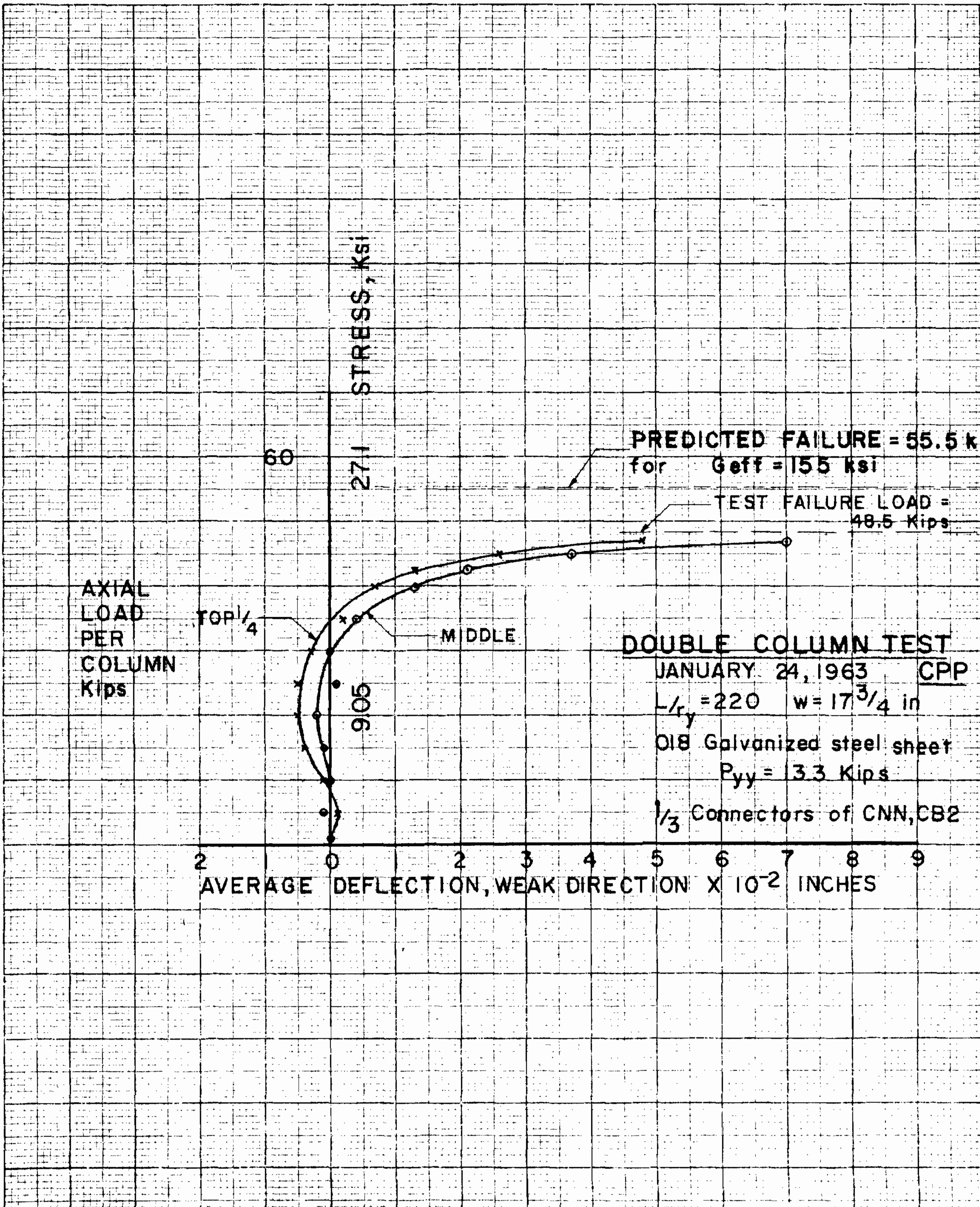

STRONG AXIS FAILURE LOAD=101K
ACTUAL WEAK AXIS FAILURE LOAD = 98.8 Kips-TEST
MIDDLE
TOP 1/4
MAXIMUM ELASTIC LOAD, e=0
MAXIMUM ELASTIC LOAD, e=L/1000
AXIAL LOAD PER COLUMN, Kips
STRESS, Ksi
90
60
30
40
30
20
10
DOUBLE COLUMN TEST CNN
December 11, 1962
L/ry=220 w=17 3/4 in
018 Galvanized steel sheet
Pyy =13.3 Kips
Pins: first twelve, then every other
1
0
1
2
3
4
5
6
7
8
AVERAGE DEFLECTION, WEAK DIRECTION, IN. x 10-2

FIGURE 19



STRESS, Ksi
60
27.1
18.1
9.05
PREDICTED FAILURE = 55.5 kips
for Geff = 155 ksi
TEST FAILURE LOAD = 49.5 Kips
TOP 1/4
MIDDLE
AXIAL LOAD PER COLUMN Kips
DOUBLE COLUMN TEST COO
December 28, 1962
L/ry = 220 w = 17 3/4 in
018 Galvanized steel sheet
Pyy = 13.3 Kips
1/3 Connectors of CNN, CB2
2 1 0 1 2 3 4 5 6
AVERAGE DEFLECTION, WEAK DIRECTION X 10^-2 in

FIGURE 20



PREDICTED FAILURE = 55.5 k
for Geff = 155 ksi
TEST FAILURE LOAD = 48.5 Kips
STRESS, Ksi
60
271
905
AXIAL LOAD PER COLUMN Kips
TOP 1/4
MIDDLE
DOUBLE COLUMN TEST
JANUARY 24, 1963 CPP
L/ry = 220 w = 17 3/4 in
018 Galvanized steel sheet
Pyy = 13.3 Kips
1/3 Connectors of CNN, CB2
2
1
0
1
2
3
4
5
6
7
8
9
AVERAGE DEFLECTION, WEAK DIRECTION X 10-2 INCHES

FIGURE 21


DOUBLE
COLUMN TEST
C.F.
NOV. 2, 1962
LOAD=0
DOUBLE
COLUMN TEST
C.F.
NOV. 2, 1962

FIGURE 22

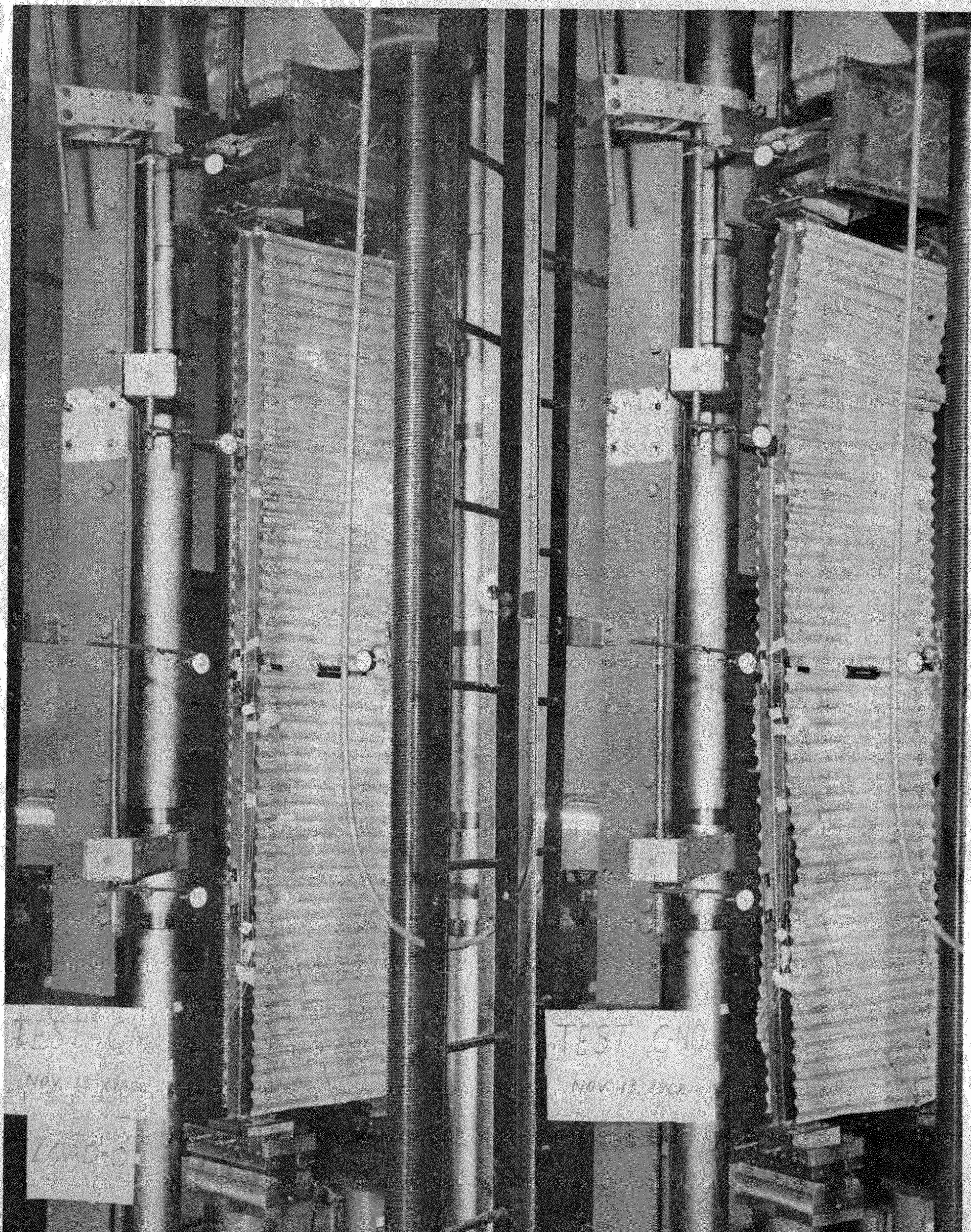


FIGURE 23

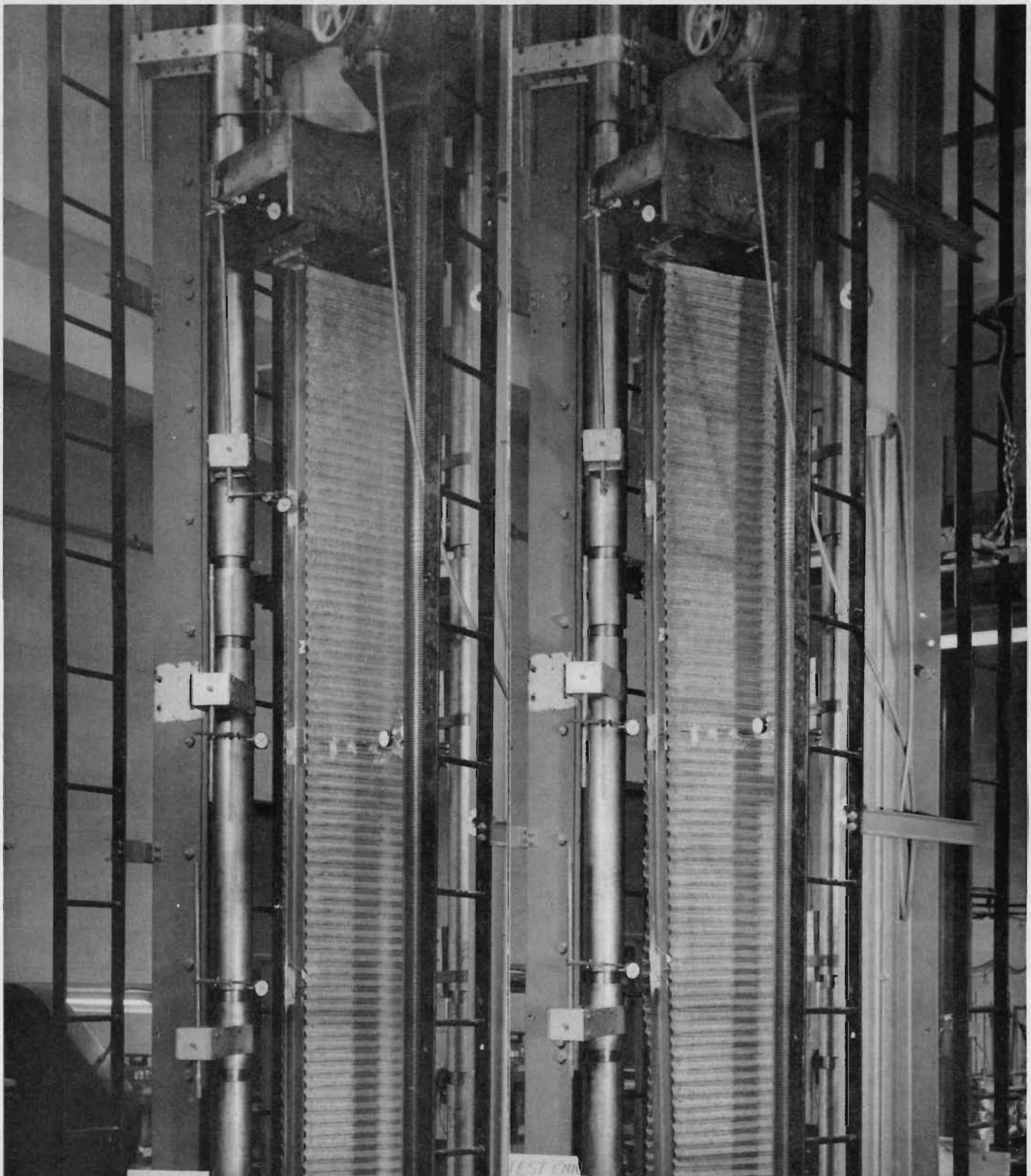


FIGURE 24

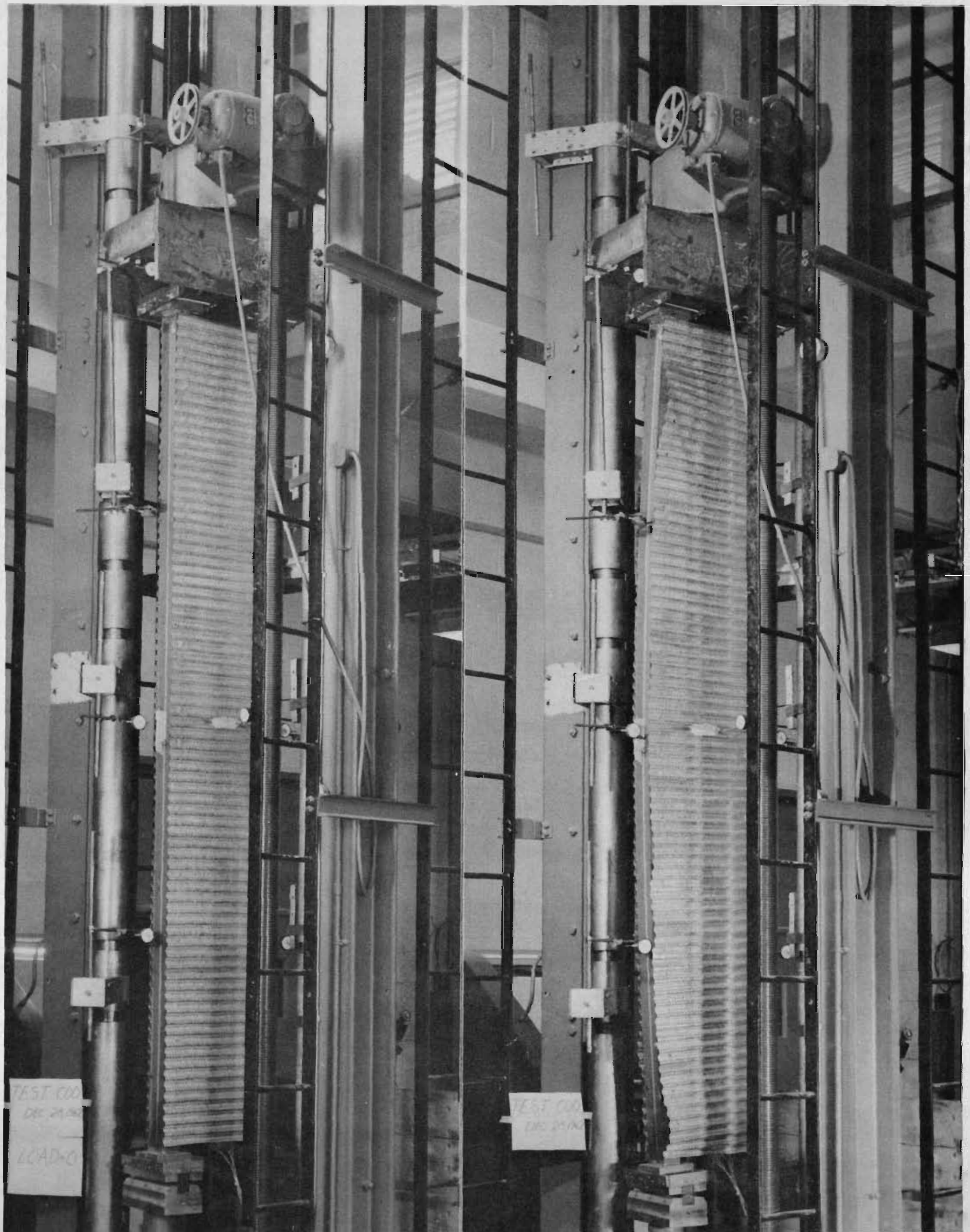
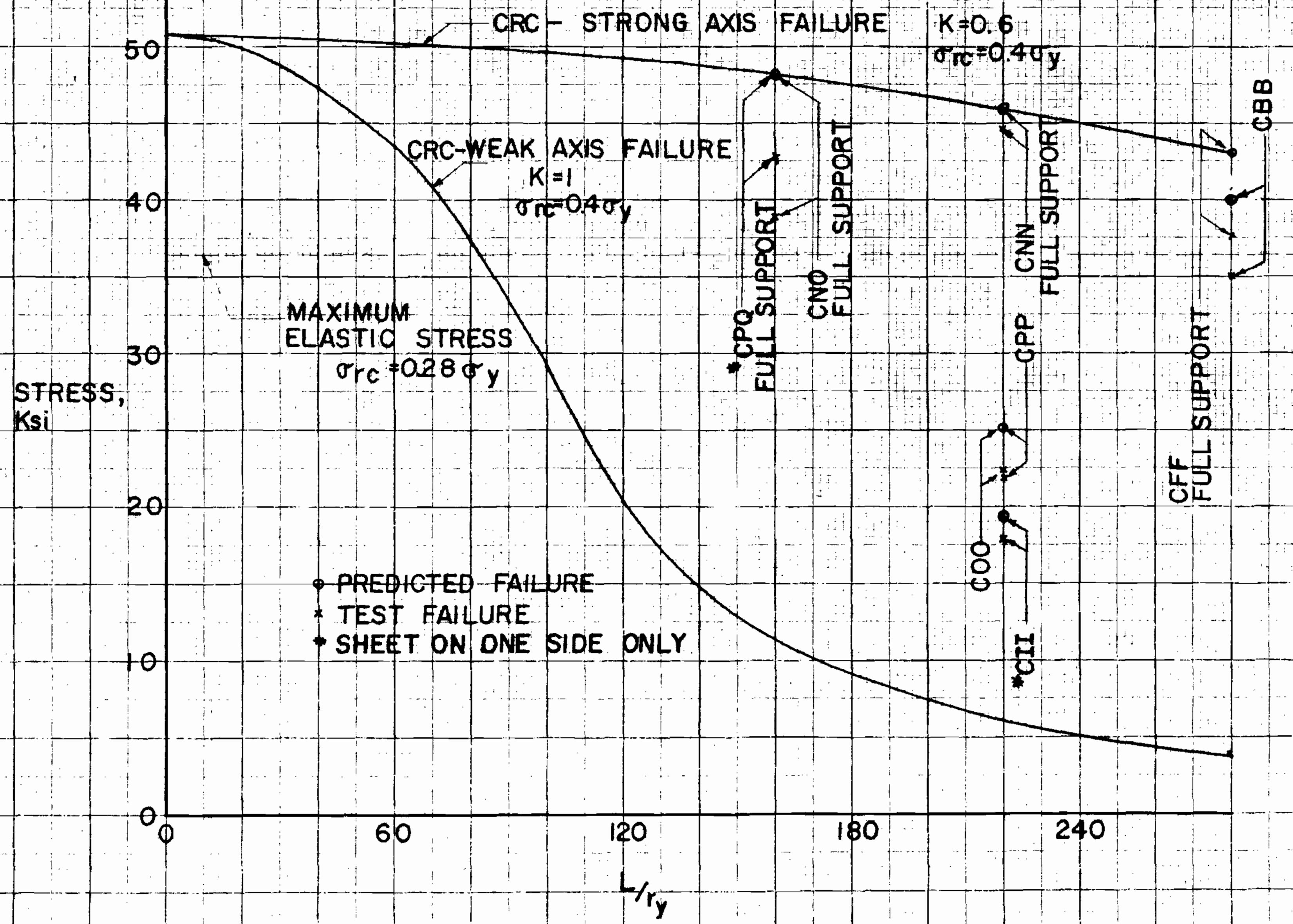


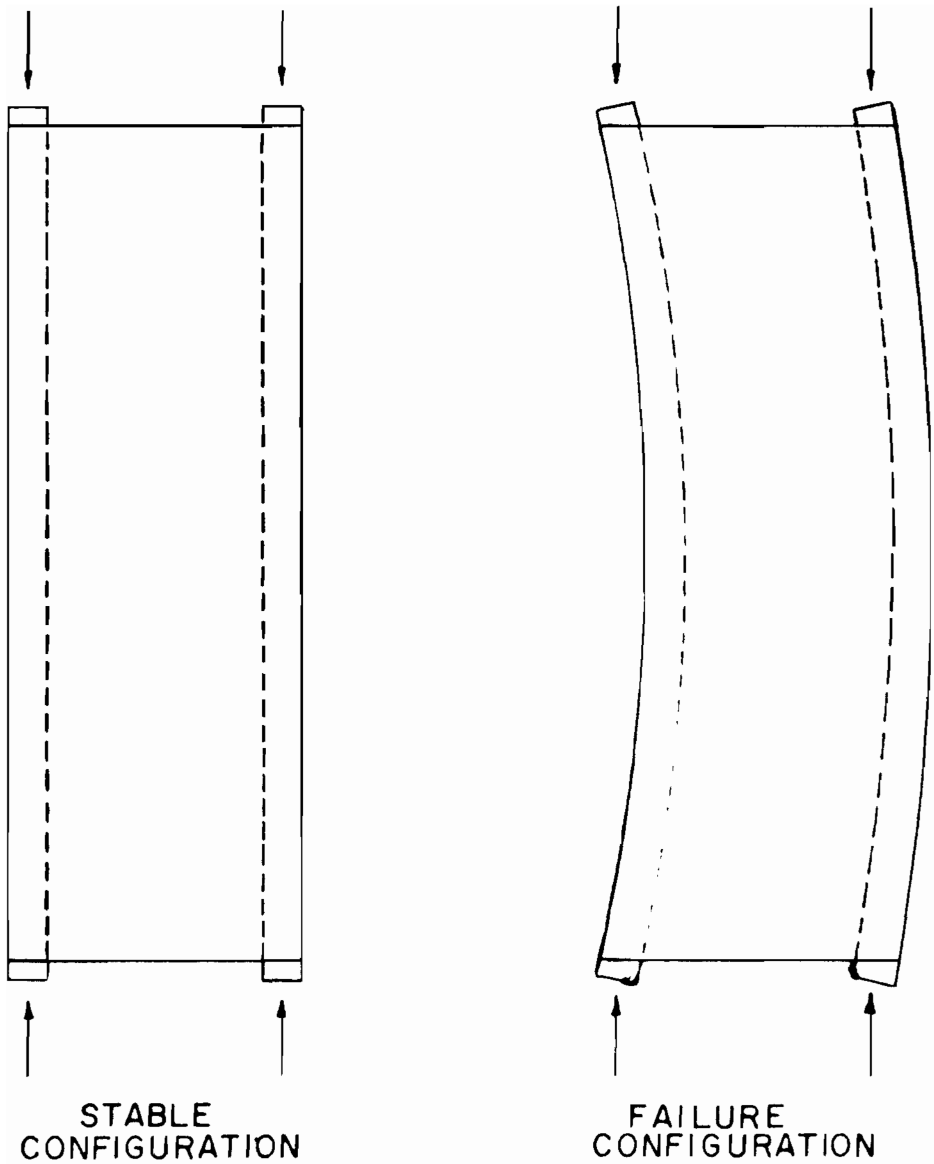
FIGURE 25



DOUBLE COLUMNS - 2 SHEETS TESTS

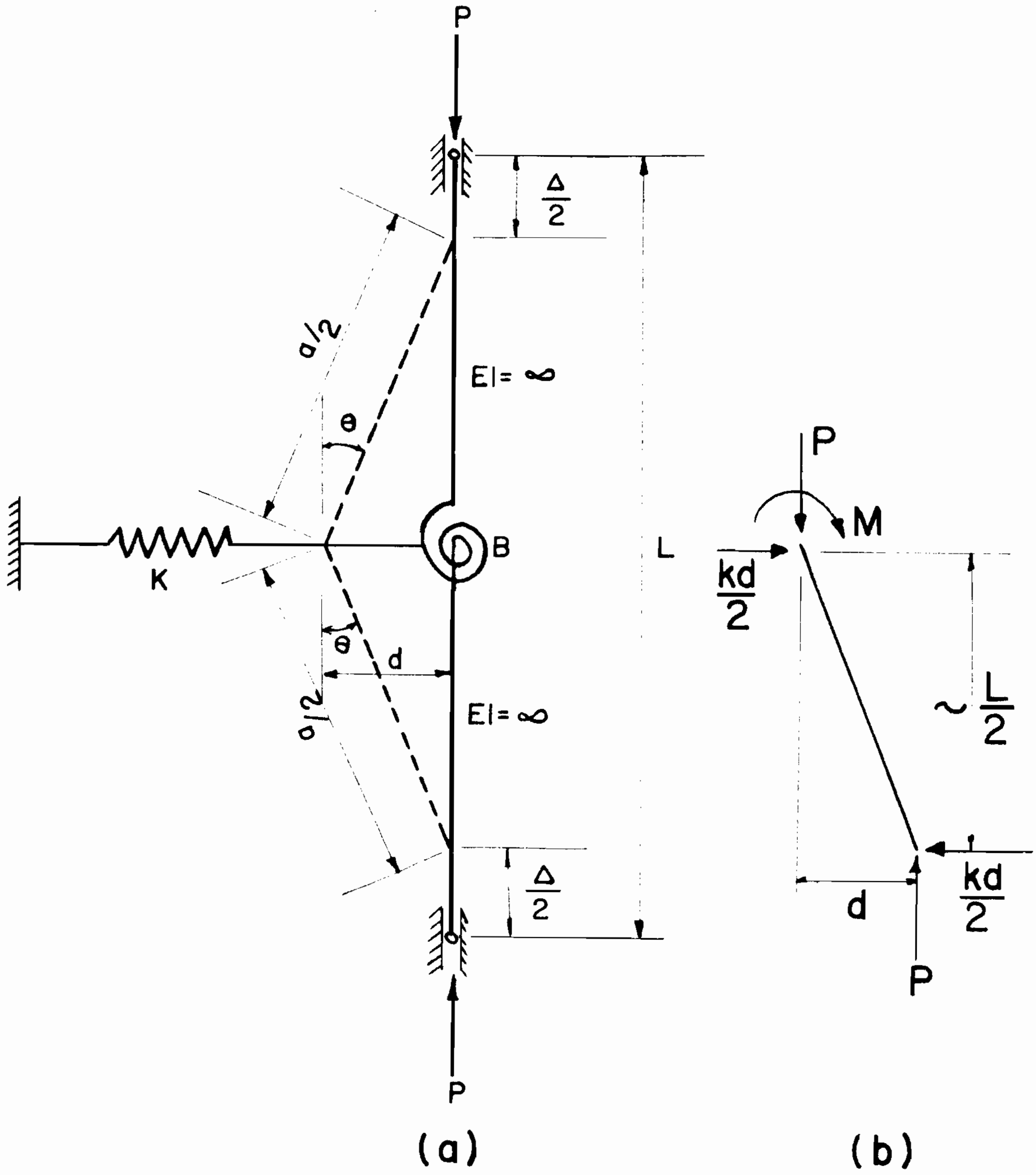
FIGURE 26

CORNELL CO-OPERATIVE SOCIETY, ITHACA, N.Y.



DIAPHRAGM IN SHEAR

FIGURE 27



IDEAL RESTRAINED COLUMN MODEL

FIGURE 28