AN EXPERIMENTAL COMPARISON OF STEEL COLUMN ECCENTRICITIES PRODUCED BY GUSSET AND SEATED BEAM CONNECTIONS

A Thesis

Presented to

the Faculty of the Department of Engineering

University of Manitoba

In Partial Fulfillment

of the Requirements for the Degree

Master of Science in Engineering

(Civil)

by

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Acknowledgment

The writer would like to express his sincere appreciation to Dominion Bridge Company Limited for their donation of the fabricated steel required in the construction of the Test Frame and the Load Frame.

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INTRODUCTION

The original objective of this thesis was to investigate the amount of eccentricity which must be provided for in the design of structural steel columns in buildings. However, after the experimental appartus was designed, fabricated and erected, time was limited. Therefore, the problem to be investigated was limited to an experimental comparison of steel column eccentricities produced by gusset and seated beam connections. It is hoped, however, that the original objective shall be reached, through further experimentation by others at a later date.

As both the gusset and seated beam connections are used in structural design, an experimental comparison as to their ability to transfer moment, caused by eccentric loading, was made.

First, it was necessary to design a Test Frame, on which the experiment could be carried out. In addition, a Load Frame was designed, in which the Test Frame might be loaded. This was done, and the plans turned over to Dominion Bridge Co. Limited, who fabricated the steel. Upon delivery, the steel was erected in the Materials Testing Laboratory.

The loads were applied to the Test Frame by means of Tension Bars which were first calibrated, in order that the amount of applied load might be known. Six gauges were placed at one section on the column leg and a preliminary test run to establish the points where gauges would be most advantageous. Gauges were then placed at six sections, distributed over the length of the column, and the frame was then ready to be tested.

Three tests were run as follows:

Test No. 1. Using the seated beam connections; a load was centrally applied on the lower beam of the Test Frame.

Test No. 2. Using the gusset connections; a load was centrally applied on the lower beam of the Test Frame.

Test No. 3. Using the seated beam connections; a load was centrally applied on the upper beam of the Test Frame.

For these three tests, the bending moment distribution in the column of the Test Frame was calculated, and a discussion of the results made.

A photograph of the Load Frame and the Test Frame, as assembled, immediately follows their design on page 15.

TEST FRAME

In the design of the Test Frame it was attempted to design a model, which would, as far as possible, resemble a bent of a two storey building. The storey heights of the model were made 4 ft. 6 ins. and beam spans 6 ft. 0 ins.

For the columns, it was necessary to use two angles, short legs back to back, in order to obtain an l/r ratio reasonably close to that of actual columns.

In the design of the beams, loading was considered as being applied at the one-third points. Two sets of beams, which were to be made up of channels back to back, were designed. One set was made stiff enough so as to have very little deflection and give a fairly well distributed load on the seat angle. The other set was designed for a stiffness in relation to the column, similar to that for a prototype, which would give results similar to actual field conditions. Due to a shortage of the smaller channels required in the latter set, the Dominion Bridge Co. Limited substituted channels a little larger than those requested, changing the column and beam stiffnesses from the desirable ratio.

FINAL DESIGN OF TEST FRAME

Columns

Using 2 angles $2\frac{1}{2}$ x 2 x $\frac{1}{4}$ s.l.b.b.

Unsupported length 4'-6" or 54"

Least r = 0.59

l/r = 54/0.59 = 91.5

For an unsupported length of 4'-6" the allowable concentric load is 27 kips.

The l/r ratio is suitable, as it is reasonably close to that of actual columns, therefore the 27 kips allowable concentric load will govern.

Apply loads of 2.5 kips at the one-third points.

Eccentricity of load = 1.667 + 0.54 = 2.11"

Equivalent concentric load = P + M Bx

= 2.5 +(2.5 x 2.11) 4.25 = 25.0 kips.

where $Bx = \frac{A}{SM.} = \frac{2.12}{0.5} = 4.25$

Therefore, this loading gives an equivalent concentric load approximately equal to the allowable of 27 kips.

Apply loads of 4.0 kips at the one-third points.

Equivalent concentric load = P + M Bx

4.0 +(4.0 x 2.11) 4.25 = 39.9 kips.

This loading is 148 % of the 27 kips allowable.

As this load is well under the failure load, it may be used to exaggerate the column bending conditions.

Beam Channels

The first set was designed for a stiffness in relation to the column, similar to that for a prototype.

Apply loads of 2.5 kips at the one-third points.

Use 2 channels 4 x 1-5/8 at 5.4 lbs.

$I = 2 \times 3.8 = 7.6''^4$

$M = 2500 \times 2 \times 12 = 60{,}000''\#$

$$S = \frac{60{,}000 \times 2}{7.6} = 17{,}200 \text{ p.s.i.}$$

The second set was designed stiff enough so as to have very little deflection.

Apply loads of 4.0 kips at the one-third points.

Use 2 channels 6 x 2 at 8.2 lbs.

$I = 2 \times 13.0 = 26.0''^4$

$M = 4000 \times 2 \times 12 = 96{,}000''\#$

$$S = \frac{96{,}000 \times 3}{26.0} = 11{,}100 \text{ p.s.i.}$$

The beams substituted for the first set are as follows;

Apply loads of 2.5 kips at the one-third points.

2 channels 5 x $1\frac{3}{4}$ at 6.7 lbs.

$I = 2 \times 7.4 = 14.8''^4$

$M = 2500 \times 2 \times 12 = 60{,}000''\#$

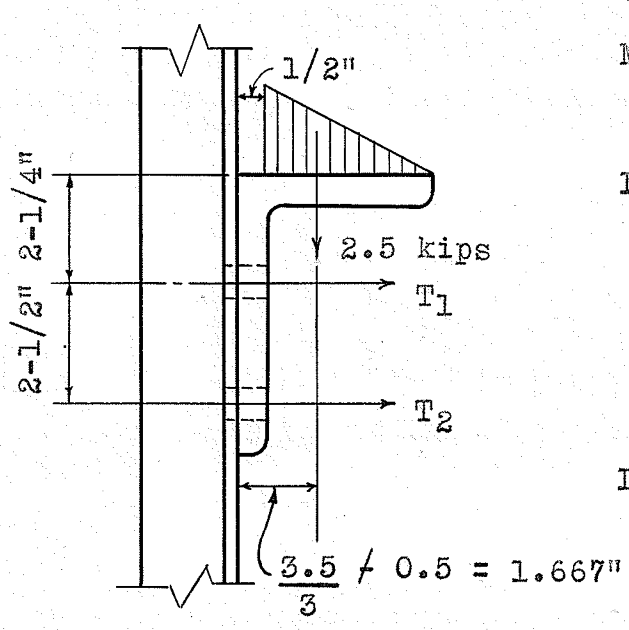
$$S = \frac{60{,}000 \times 2.5}{14.8} = 10{,}100 \text{ p.s.i.}$$

Seat Angles

Use four 5/8 inch diameter bolts.

Apply loads of 2.5 kips at the one-third points.

Design for flexure of vertical angle leg at net section on upper rivet line.



Use 6 x 4 x 5/8 angle

$M = 2500(1.667 - 0.312)$

$= 3390''\#$

Length of angle required

$$= \frac{6\ M}{t^2 f}$$

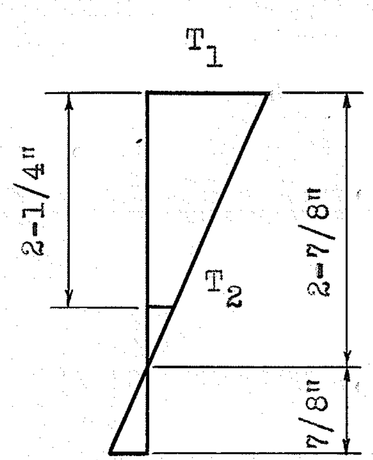
$$= \frac{6 \times 3390}{(0.625)^2 \times 18{,}000} = 2.9''$$

Length of angle available

$2(2.5) + 0.25 - 1.25$

$= 4.0''$ (Satisfactory)

Check for shear and tension in bolts.



$$S_s = \frac{2500}{4} = 625\ \#/\text{bolt}.$$

Allowable in shear = 3070 #/bolt.

(Satisfactory)

Moment of Inertia of bolt areas and compression area = $1/3 \times 4.75 \times (0.87)^3 + 2 \times 0.307(0.37^2 + 2.87^2)$

$= 6.04''^4$

Bolt tension = $\frac{2500 \times 1.667 \times 2.87}{6.04}$ = 1980 p.s.i.

Allowable in tension = 18,000 p.s.i. (Satisfactory)

Apply loads of 4.0 kips at the one-third points.

Design for flexure of vertical angle leg at net section on upper rivet line.

Consider same 6 x 4 x 5/8 angle

M = 4000 (1.667 - 0.312) = 5420"#

Length of angle required = $\frac{6 \times 5420}{(0.625)^2 \times 18{,}000}$ = 4.62"

Length of angle available = 4.0"

This will be satisfactory since at this loading the columns are at 148 % of their allowable.

Check for shear and tension in bolts.

$S_s = \frac{4000}{4}$ = 1000 #/bolt.

Allowable in shear = 3070 #/bolt. (Satisfactory)

Moment of Inertia of bolt areas and compression area = 6.04"^{4}

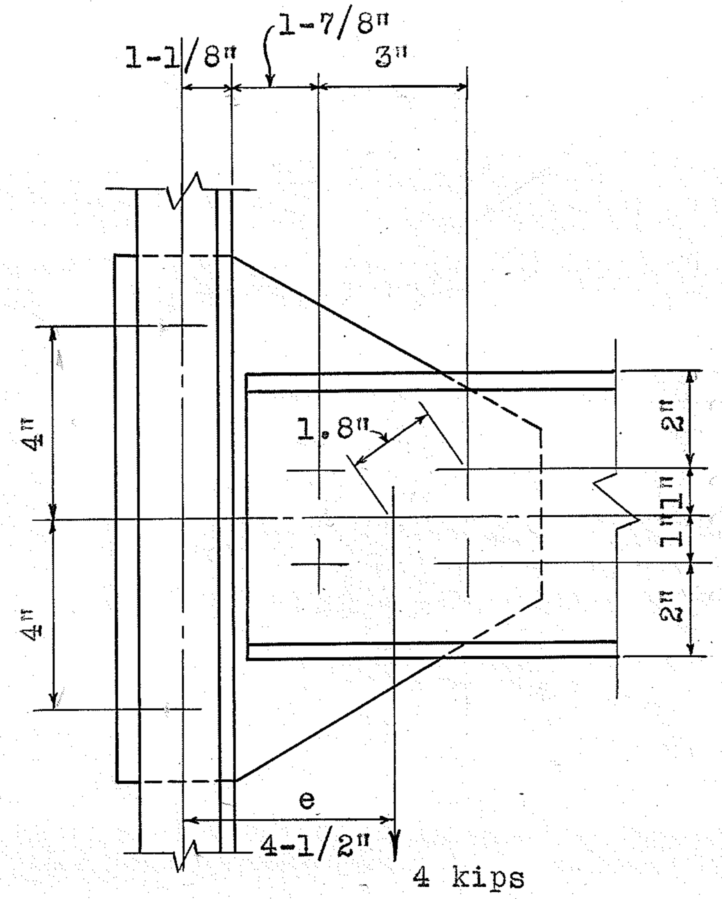
Bolt tension = $\frac{4000 \times 1.667 \times 2.87}{6.04}$ = 3,170 p.s.i.

Allowable in tension = 18,000 p.s.i. (Satisfactory)

Gussets

Use 5/8 inch diameter bolts.

Apply loads of 4.0 kips at the one-third points.



Use 3 bolts in the column.

$$S_s = \sqrt{\left(\frac{P}{n}\right)^2 + \left(\frac{Pec}{\Sigma y^2}\right)^2} = \sqrt{\left(\frac{4}{3}\right)^2 + \left(\frac{4 \times 4.5 \times 4}{2 \times (4)^2}\right)^2}$$

$$= \sqrt{1.78 + 5.06} = 2.62 \text{ kips/bolt.}$$

Allowable in shear = 2.93 kips/bolt.

(Satisfactory)

Use 4 bolts in the beam.

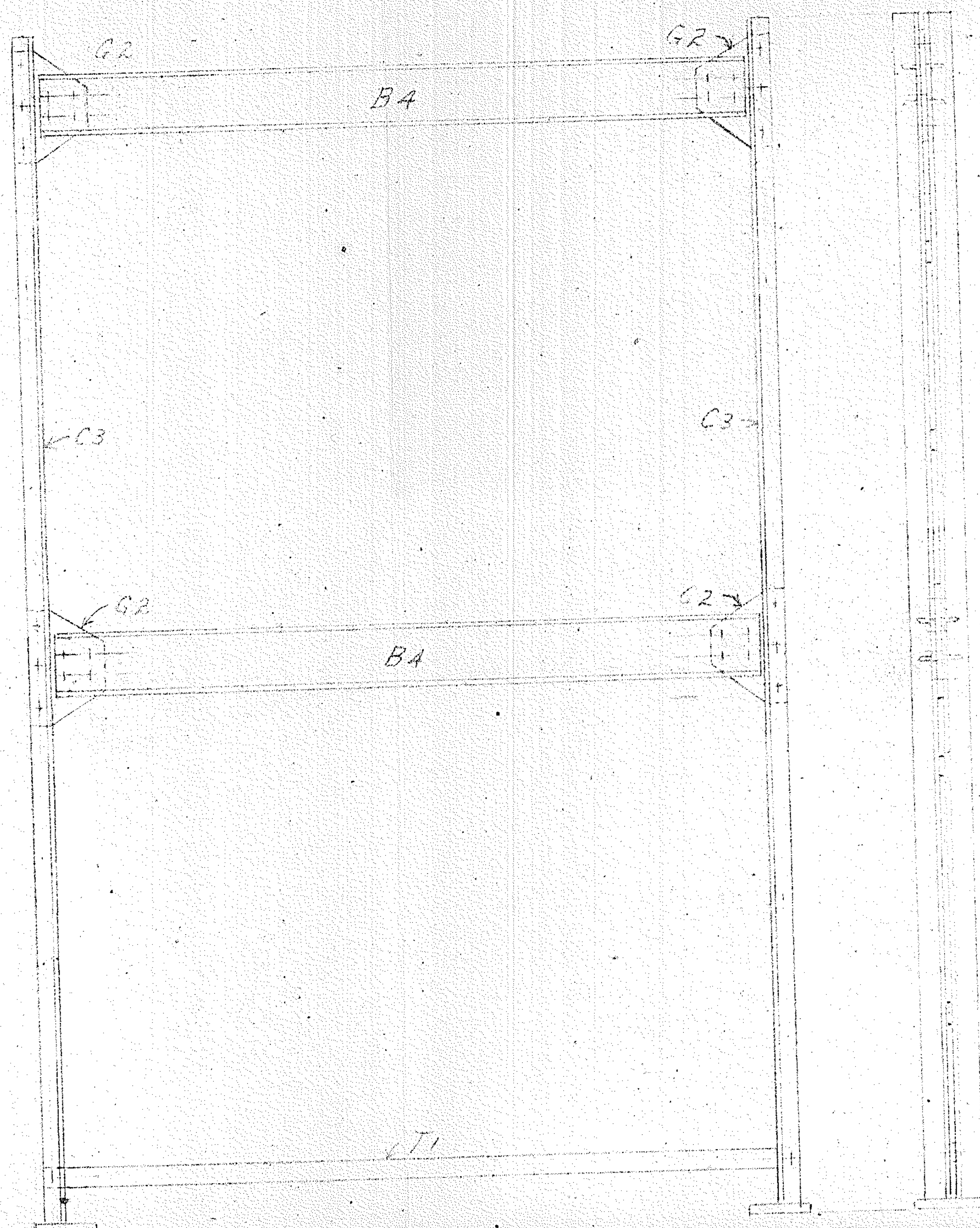
$$S_s = \sqrt{\left(\frac{4}{4}\right)^2 + \left(\frac{4 \times 4.5 \times 1.8}{4 \times (1.8)^2}\right)^2}$$

$$= \sqrt{1.0 + 6.25} = 2.69 \text{ kips/bolt.}$$

Allowable in shear = 2.93 kips/bolt.

(Satisfactory)

A detailed drawing of the Test Frame is shown on page 8.



ASSEMBLY DIAGRAM
scale $\frac{3}{4}$" = 1'-0"

NOTE
All holes drilled for $\frac{5}{8}$" ϕ bolts.

MATERIAL

Size
$2\frac{1}{2}$" x 2" x $\frac{1}{4}$" ∠ 9'-7" lg.
2 - 6" ['s @ 8.2# 5'-9" lg.
2 - 4" ['s @ 5.4# 5'-9" lg.
$\frac{1}{4}$" Pl. 11" x 12"
2" x $\frac{1}{4}$" bar 6'-3" lg.
$\frac{3}{4}$" Pl. 6" x 8"
$2\frac{1}{4}$" lg with hex. nuts and
2 washers each.
for $\frac{5}{8}$ ϕ bolt $\frac{1}{4}$" thick.

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DETAIL OF TEST FRAME

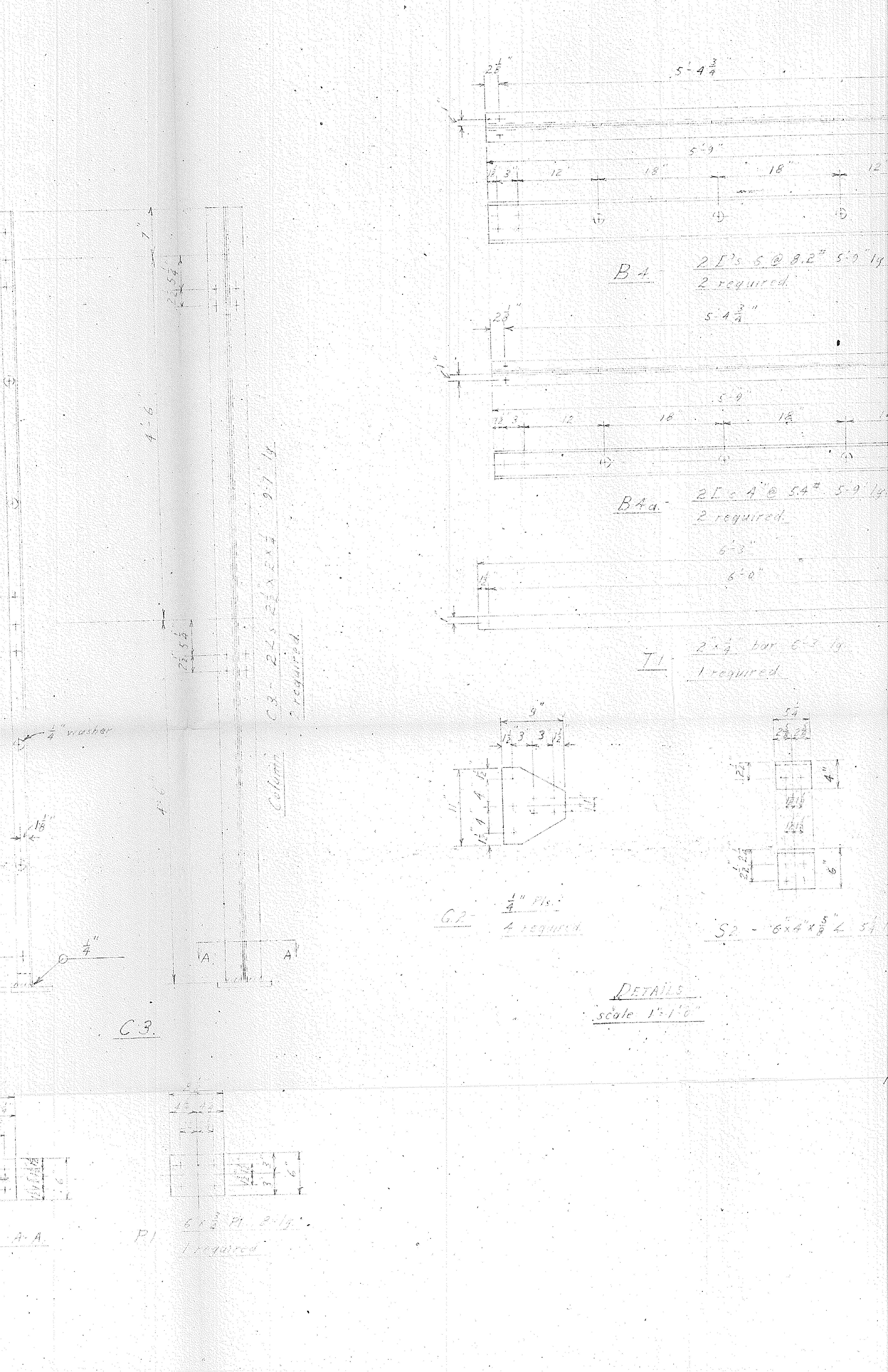
Scales: As shown

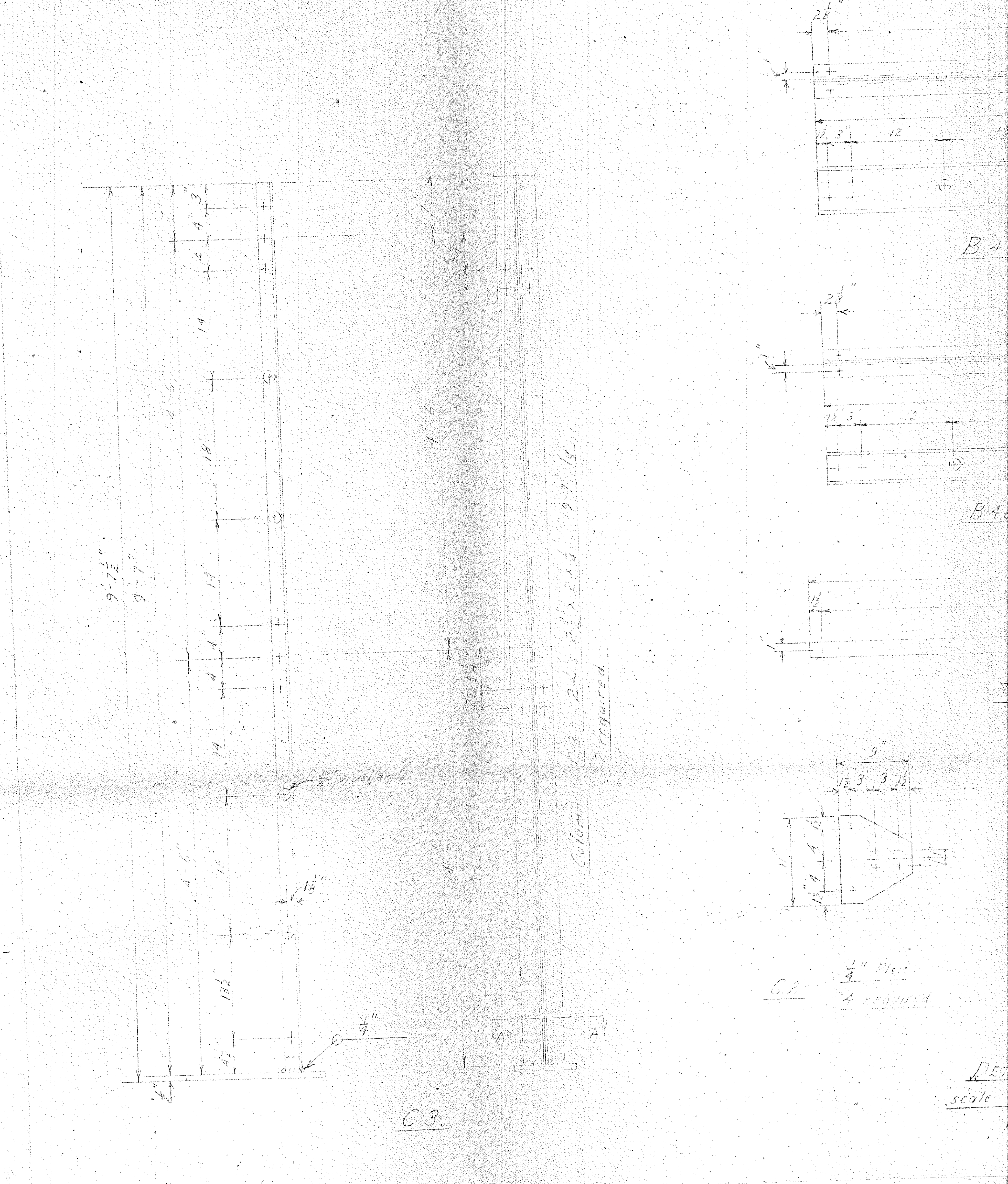
December, 1951

Dwg. No. 1

M. Sc. Course

F.W. Calle


C3 - 2Ls 2½ x 2½ x ¼ 9'-1" lg
2 required
Column
C3.
¼" washer
A-A
P1 6 x ⅜ Pl. 9" lg.
1 required
B4 2 [s 5 @ 8.2# 5'-9" lg
2 required
B4a. 2 [s 4" @ 5.4# 5'-9" lg
2 required
T1 2" x ¼" bar 6'-3" lg.
1 required
G2 ¼" Pls.
4 required
S2 - 6" x 4" x ⅝" ∠ 5¼ lg
DETAILS
scale 1"=1'-0"


Column C3 - 2Ls 2½ x 2½ x ¼ 9'7" lg.
2 required
¼" washer
C3.
¼" spacer
Section A-A.
P1 6 x ¾ Pl. 2' lg.
1 required
G2 ¼" Pls.
4 required

LOAD FRAME

The Load Frame was primarily designed as an enclosed frame in which the Test Frame could be loaded. It was made 9 ft. 0 ins. centre to centre of columns and of such height as to be connected to the ceiling of the Testing Laboratory.

It was recognized that such a frame could be used for testing other frames, large culverts, etc. after this test was completed. With this in mind, the frame was designed to withstand a centrally applied load of 20 kips.

FINAL DESIGN OF LOAD FRAME

Frame to take central load of 20 kips.

Load Channels

The beams, or load channels of the Load Frame were each made up of 2 channels.

Each channel to take 10 kips central load or 20 kips uniform load.

From A.I.S.C. 1 channel 10 x 2-5/8 at 15.3 lbs. will carry 19.9 kips uniform load if laterally supported.

Therefore, 4 inch diameter pipe spacers were used between the 2 load channels to give lateral support.

Load Channel Connection

Central load of 20 kips.

Load/channel = $\frac{20}{2}$ = 10 kips.

Use 1 - 1 inch diameter bolt in each end.

Load/bolt = $\frac{10}{2}$ = 5.0 kips.

Allowable in shear = 7.85 kips/bolt.

(Satisfactory)

The top load channels were coped at one end to frame into a concrete beam. This reduced the web area available to resist the shear.

Load channels 10 x 2-5/8 at 15.3 lbs.

Load/channel = 10 kips.

End reaction/channel = $\frac{10}{2}$ = 5 kips.

Area required in shear = $\frac{5000}{13,000}$ = 0.4 sq. ins.

Area provided = 4 x 1/4 = 1.0 sq. ins.

(Satisfactory)

Columns

The columns of the Load Frame were made up of 2 channels. In using channels, the load must be applied, so that the columns will always be in tension. One column was made 12 ft. 4-1/4 ins. high, as it was connected at the top to a floor beam.

The other column was made 14 ft. 1-1/4 ins. high as it was connected at the top to the ceiling.

Central load of 20 kips.

Load/column = $\frac{20}{2}$ = 10 kips.

Net area required = $\frac{10,000}{20,000}$ = 1/2 sq. in.

1 channel 3 x 1-1/2 at 4.1 lbs. gives

1.19 - (1 x 3/16) = 1.0 sq. in. (Satisfactory)

However, as 10 x 2-5/8 at 15.3 lbs. load channels were used, the columns were made of channels 10 x 2-5/8 at 15.3 lbs. and 4 inch diameter pipe spacers used for added rigidity.

The holes for the load channel connections were drilled on the centre line of the columns at intervals of 6 inches, which would allow the load channels to be moved up or down to any desired position.

<u>Base</u>

Central load of 20 kips.

$$M = \frac{PL}{4} = \frac{20,000 \times 9 \times 12}{4} = 540,000''\#$$

Use WF 12 x 12 at 65 lbs.

$I = 533.4''^4$ $\qquad y = 6''$

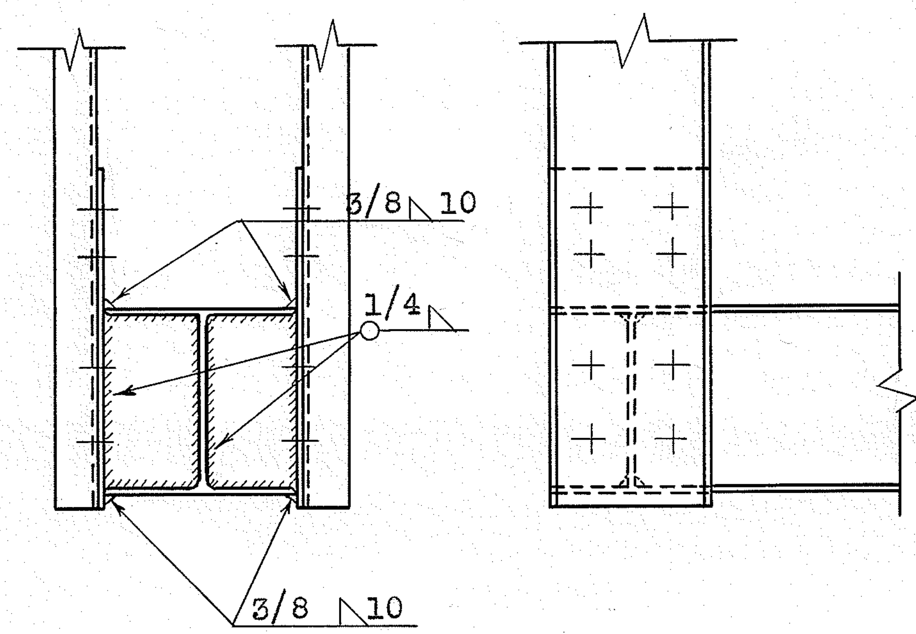
$$S = \frac{My}{I} = \frac{540,000 \times 6}{533.4} = 6070 \text{ p.s.i.}$$

$$\frac{ld}{bt} = \frac{9 \times 12 \times 12}{12 \times 5/8} = 173 \text{ which is less than } 600.$$

Therefore, allowable = 20,000 p.s.i. (Satisfactory)

Column To Base Connection

For the column to base connection, a 1/2 inch gusset plate was welded to the flanges of the base, and the column channels bolted to the gusset. The flanges of the beam were strengthened by welding in a 3/8 inch plate which was ground to fit.



Flange thickness of 12 x 12 WF at 65 lbs. = 5/8"

Web of 10 x 2-5/8 channel = 1/4"

Use 3/8" weld, throat = 0.265"

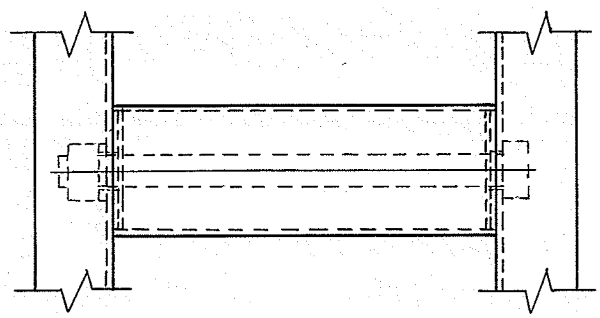
Length of weld required = $\frac{5000}{11,300 \times 0.265}$ = 1.67"

Weld the width of column channel = 10"

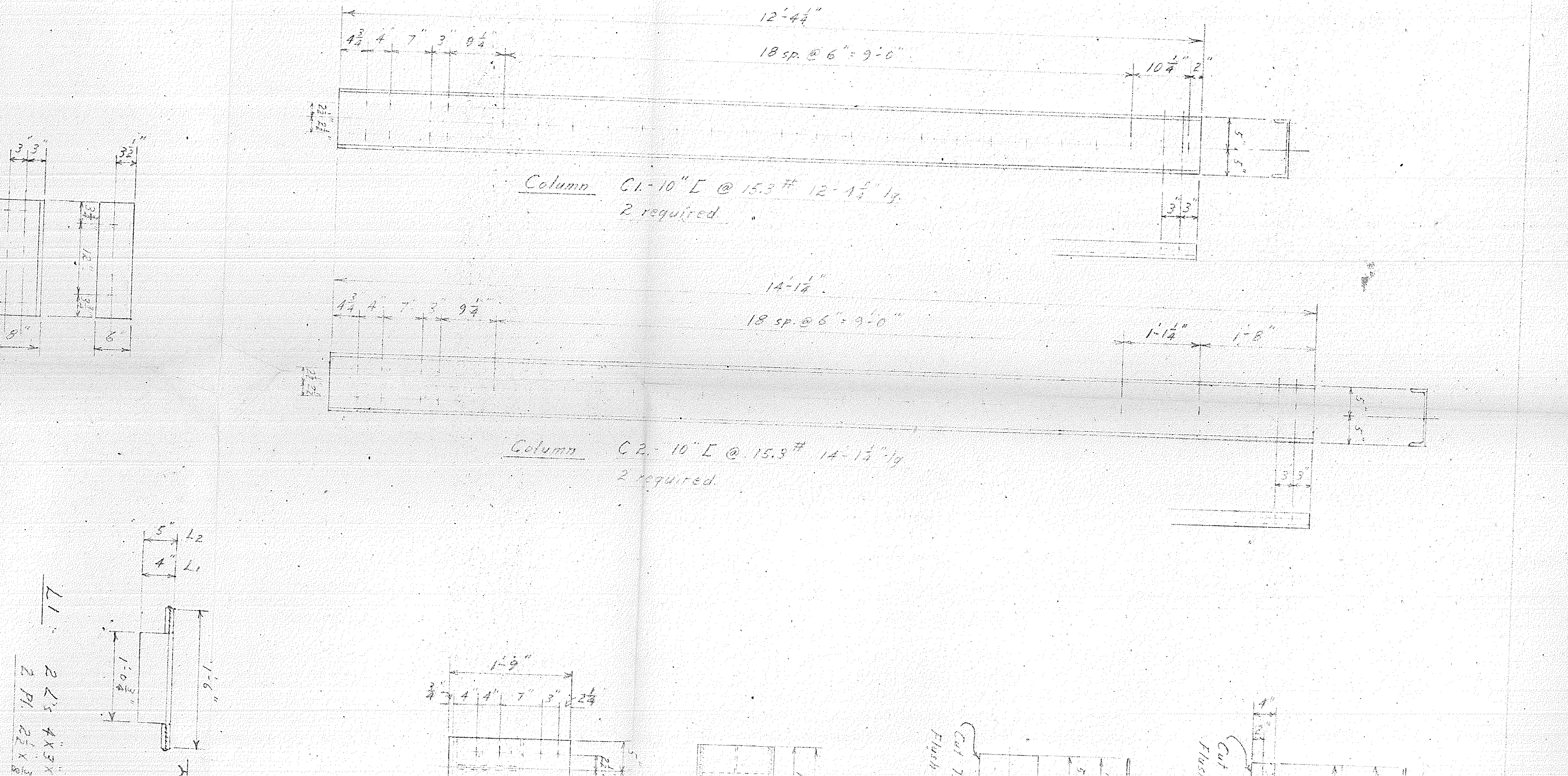
(Satisfactory)

Pipe Spacers

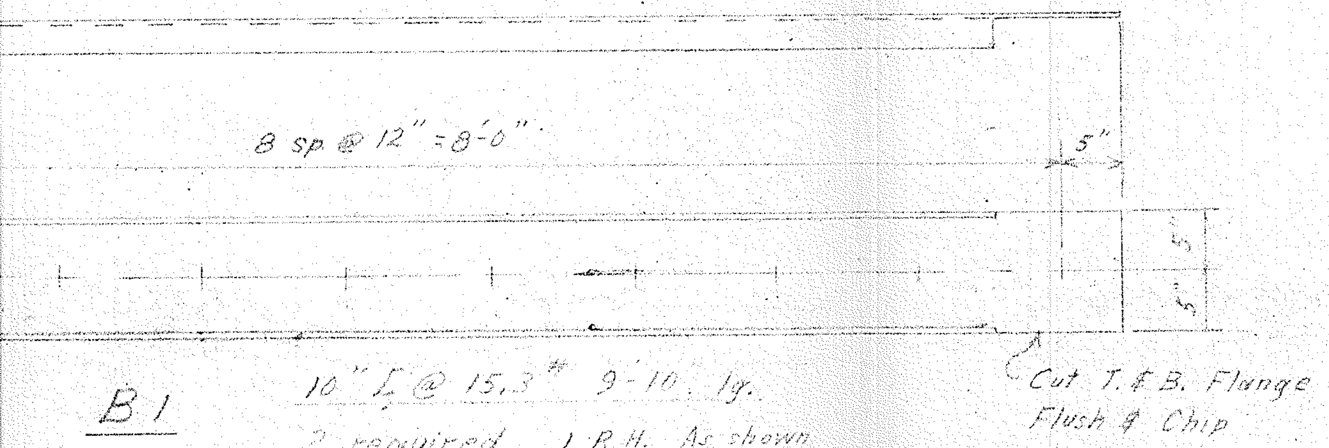
The pipe spacers were made up of 4 inch diameter pipe, with washers tack welded in each end in order to keep the bolt central.



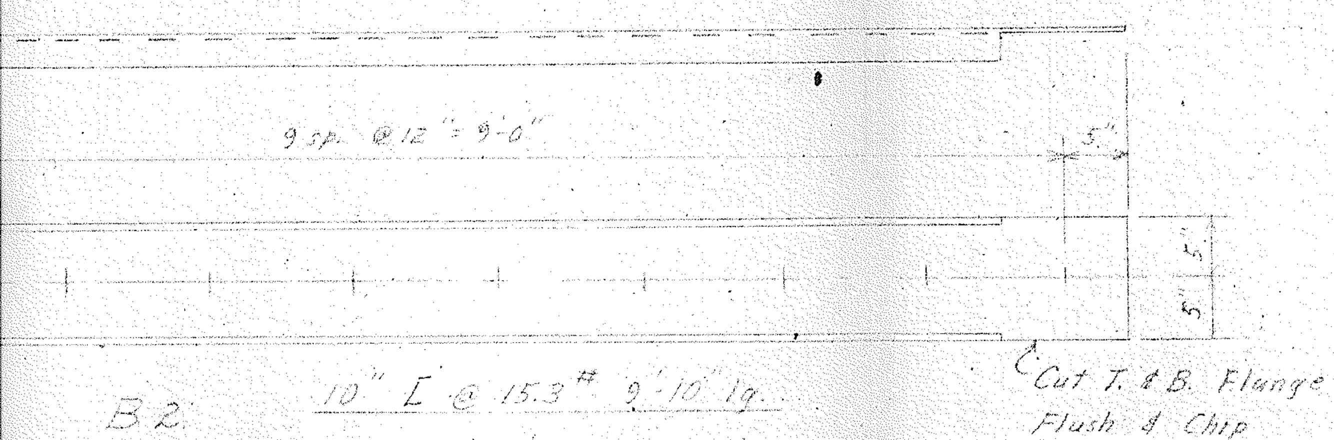
A detailed drawing of the Load Frame is shown on page 14.



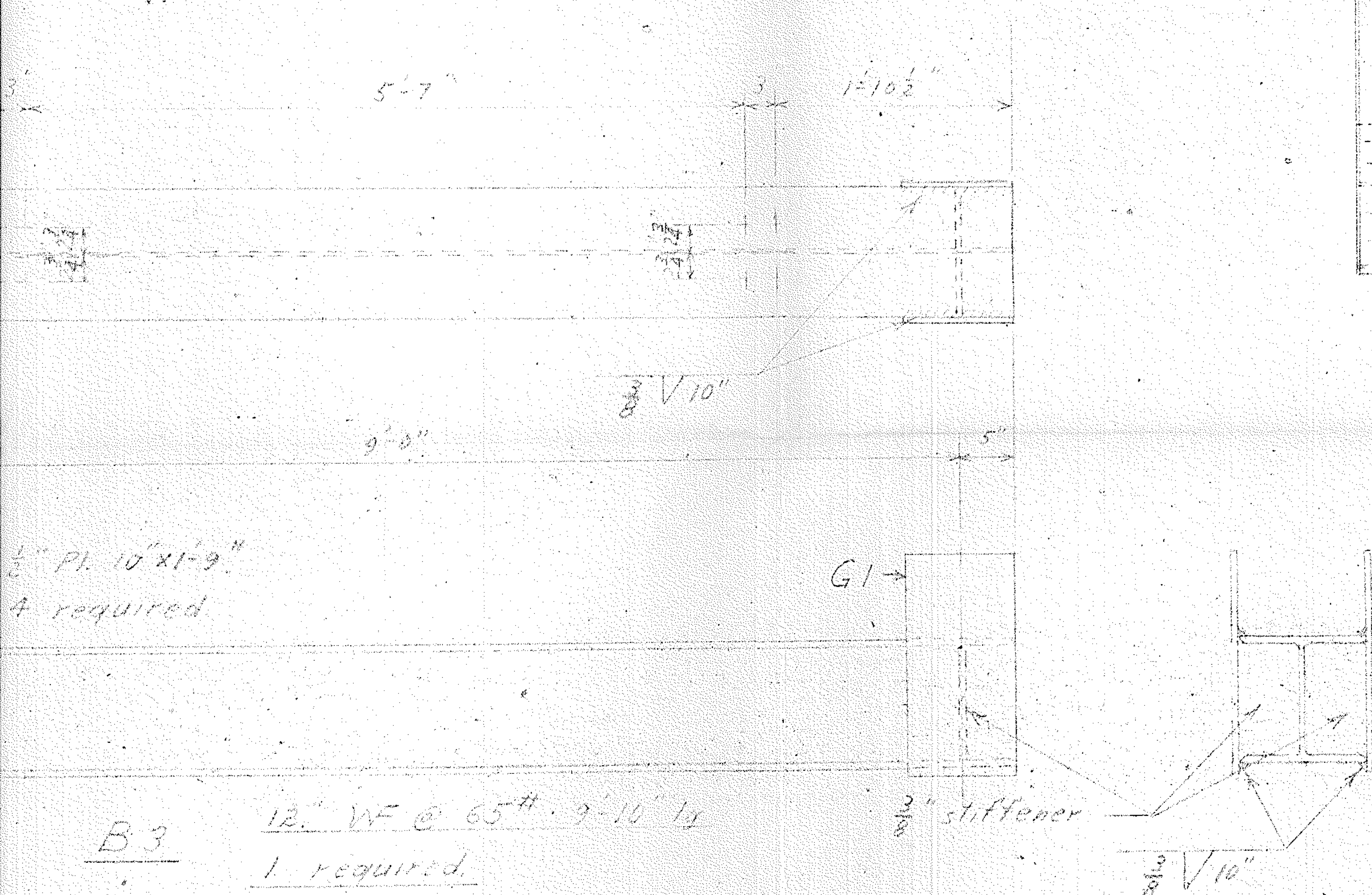
Column C1.-10" [@ 15.3# 12'-4½" lg.
2 required.
12'-4½"
18 sp. @ 6" = 9'-0"
10¼"
2"
5"
5"
3" 3"
Column C2.- 10" [@ 15.3# 14'-1½" lg.
2 required.
14'-1¼"
18 sp. @ 6" = 9'-0"
1'-1¼"
1'-8"
L1: 2 L's 4x3x
2 Pl. 2½x
1'-6"
1'-0¾"
1'-9"



B 1 — 10" [@ 15.3# 9'-10" lg.
2 required 1 R.H. As shown
1 L.H. Reverse

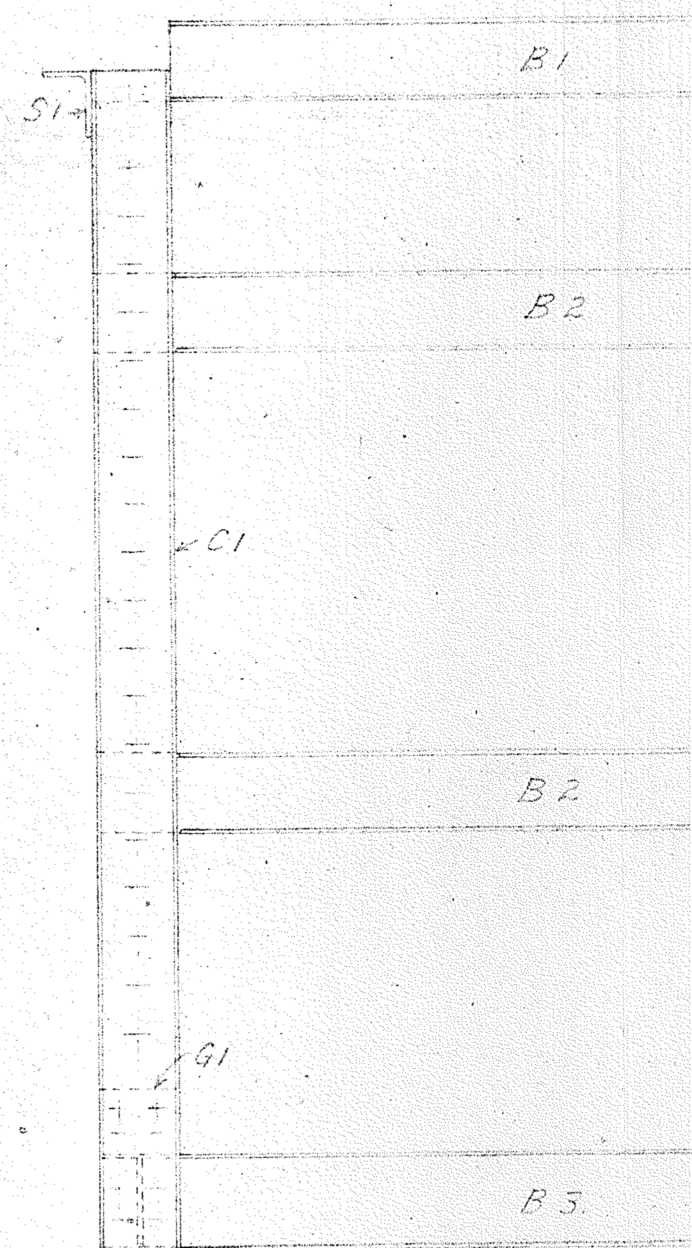


B 2. — 10" [@ 15.3# 9'-10" lg.
4 required.



B 3 — 12" WF @ 65# 9'-10" lg.
1 required.

DETAILS
scale: ¾" = 1'-0"



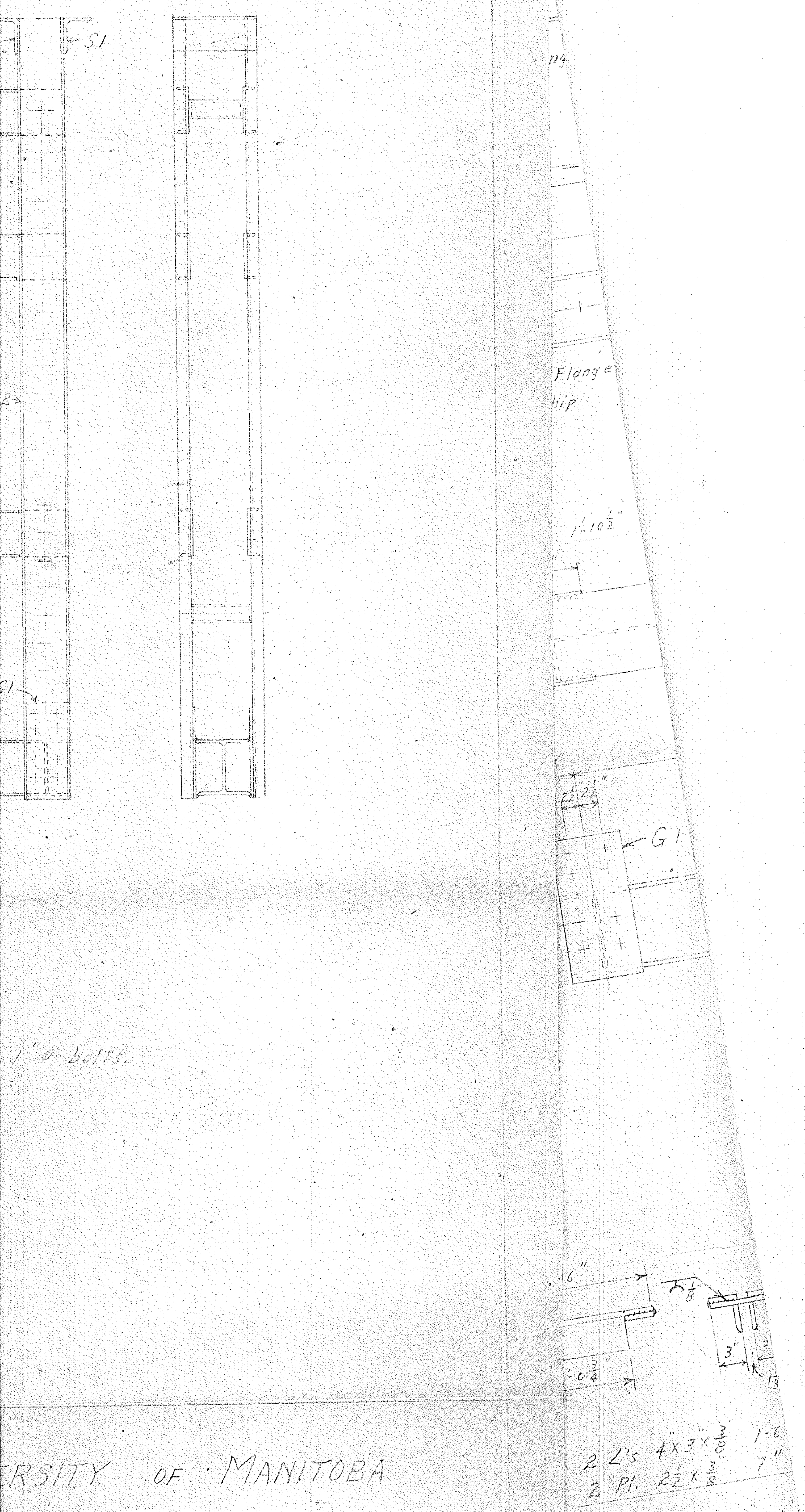
ASSEMBLY DIA
scale: ½" = 1'-0

NOTE:
All holes

BILL OF MATERIAL

No. req'd.	Symbol	Size
2	C1	10" [@ 15.3# 12'-4¼" lg.
2	C2	10" [@ 15.3# 14'-1¼" "
2	B1	10" [@ 15.3# 9'-10" "
4	B2	10" [@ 15.3# 9'-10" "
1	B3	12" WF @ 65# 9'-10" "
3	S1	8 x 6 x ½ L 1'-7½" "
4	G1	½" Pl. 10" x 1'-9"

lg. } 3 req'd.



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IL OF LOAD FRAME

ls shown Dwg. No. 2

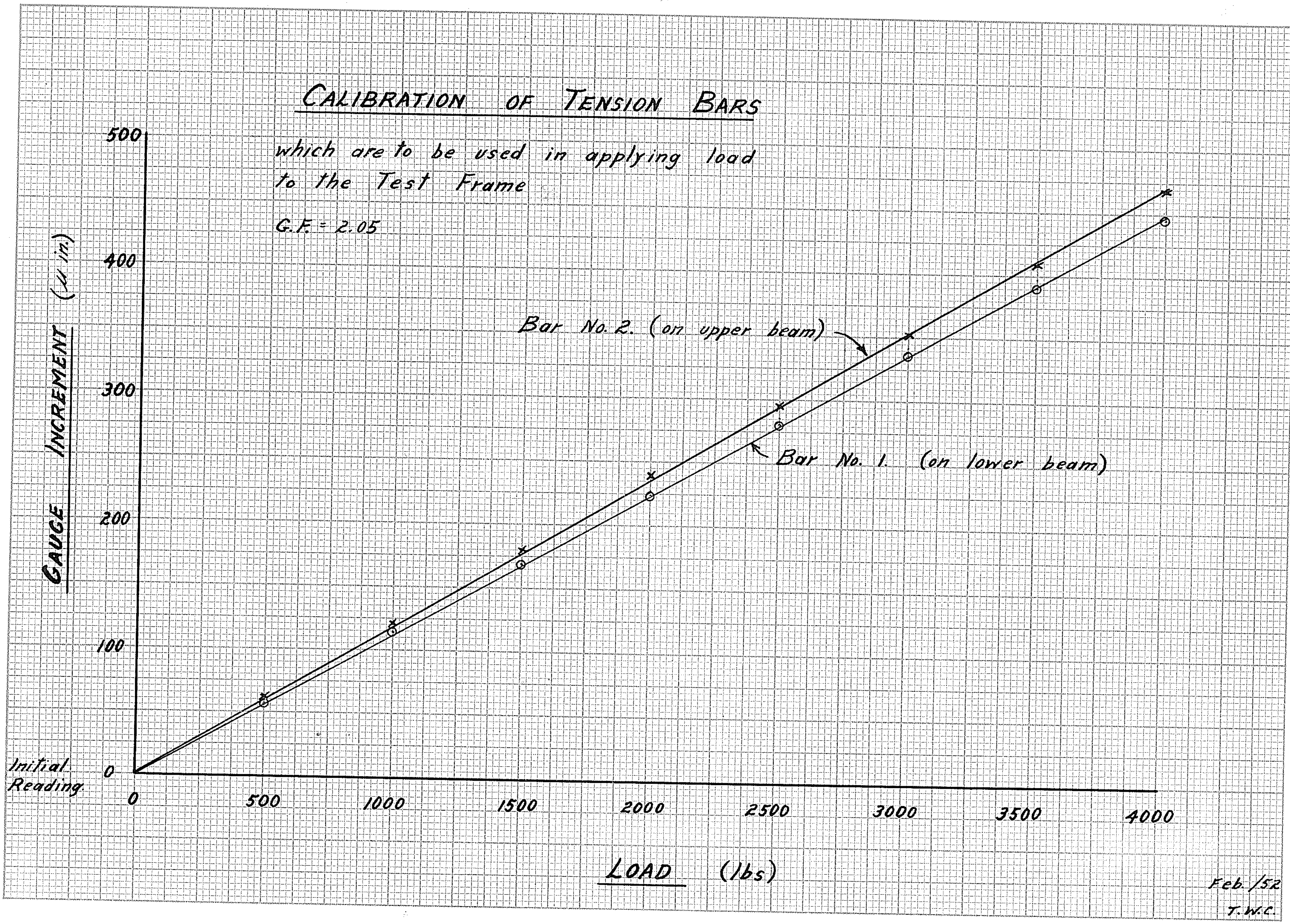
Load Frame and Test Frame

CALIBRATION OF TENSION LOADING BARS

As the loading was applied from the Load Frame to the Test Frame by means of Tension Bars, it was first necessary to calibrate these bars. This was done by placing a strain gauge on the Tension Bar, and loading the bar in a testing machine, taking care to keep the unit stress below the elastic limit of the bar. Gauge readings were taken at 500 lb. load increments and the gauge increment was calculated, taking the reading for zero load as zero strain.

The following table of values was recorded, and a Load vs Gauge Increment graph drawn.

Load (lbs.)	Bar No. 1		Bar No. 2	
	Gauge Reading	Increment	Gauge Reading	Increment
0	8233	0	7368	0
500	8291	58	7430	62
1000	8348	115	7489	121
1500	8404	171	7548	180
2000	8459	226	7607	239
2500	8517	284	7664	296
3000	8572	339	7720	352
3500	8628	395	7776	408
4000	8683	450	7834	466


CALIBRATION OF TENSION BARS
which are to be used in applying load
to the Test Frame
G.F. = 2.05
Bar No. 2. (on upper beam)
Bar No. 1. (on lower beam)
GAUGE INCREMENT (μ in)
500
400
300
200
100
0
Initial Reading
0
500
1000
1500
2000
2500
3000
3500
4000
LOAD (lbs)
Feb./52
T.W.C.

Now, by using the appropriate gauge increment, the tension bar could be used to obtain any desired load.

Tension Bar No. 1 was used to apply load to the lower beam of the "Test Frame."

Tension Bar No. 2 was used to apply load to the upper beam of the "Test Frame."

PRELIMINARY TEST

The preliminary test was performed to establish suitable locations for the strain gauges. Six gauges were placed on the column 2'-1" from the base. A load was then placed on the lower beam by Tension Bar No. 1 in increments of 500 lbs. up to a maximum of 3000 lbs., with gauge readings taken after each increment. The following table of results was obtained:

Load (lbs.)	Gauge						
	1 (μins.)	3 (μins.)	4 (μins.)	5 (μins.)	6 (μins.)	7 (μins.)	8 (μins.)
0	6000	5994	5948	6003	6002	5986	5982
500	6058	5990	5947	5998	5987	5979	5975
1000	6116	5992	5947	5996	5980	5972	5969
1500	6171	5996	5948	5990	5974	5967	5964
2000	6227	5991	5947	5986	5969	5963	5958
2500	6285	5992	5946	5982	5964	5956	5952
3000	6343	5990	5946	5979	5955	5950	5948
2500 lb. increment 500-3000		0	-1	-19	-32	-29	-27

The distance from the centre line of the gauges to the back of the angle (near gauge 8) is as follows:

Gauge	Distance	Gauge	Distance
3	1.80"	6	2.29"
4	1.80"	7	0.81"
5	0.54"	8	0.24"

A gauge location diagram for the Preliminary Test accompanies the graph described below.

The gauge increments for the 2500 lb. load increment were plotted on a graph according to their location on the angle. The construction of the remainder of the graph to obtain the axial load in the angle is described in the following paragraphs.

Theoretically, the axial load in a column passes through the centre of gravity of the column. In this case, where angles were used in the columns, the centre of gravity of each angle was in space, and it was impossible to measure the axial strain directly by means of a strain gauge. Thus it became necessary to calculate the strain at this point from strain readings taken on gauges which were on the angle. These gauges were 3, 4, 5, 6, 7 and 8 which were mentioned above. Gauge 1 is on Tension Bar No. 1.

After the gauge increments for these six gauges were plotted according to their location on the angle, lines through these points on each leg were drawn, thus giving a strain line for each leg. The strain on the Y-Y axis as measured by gauge 5, plus the difference between the strain at the corner near gauge 8 and the strain on the X-X axis at gauge 7, gave the strain at the centre of gravity of the angle. This then was the axial strain. This value was obtained graphically. The strain reading at the Y-Y axis (gauge 5) on the short leg was carried over to the back of the angle. Starting from this point a line was drawn parallel to the strain line for the long leg until it intersected the X-X axis at gauge 7. This point gave the axial strain for the angle, which should also be the axial strain for the column. From the graph the axial strain was found to be 21.0 microinches.

By actual calculation the axial strain = $\frac{P}{A\ E}$

Load applied = 2500 lbs.

Load/column = 1250 lbs.

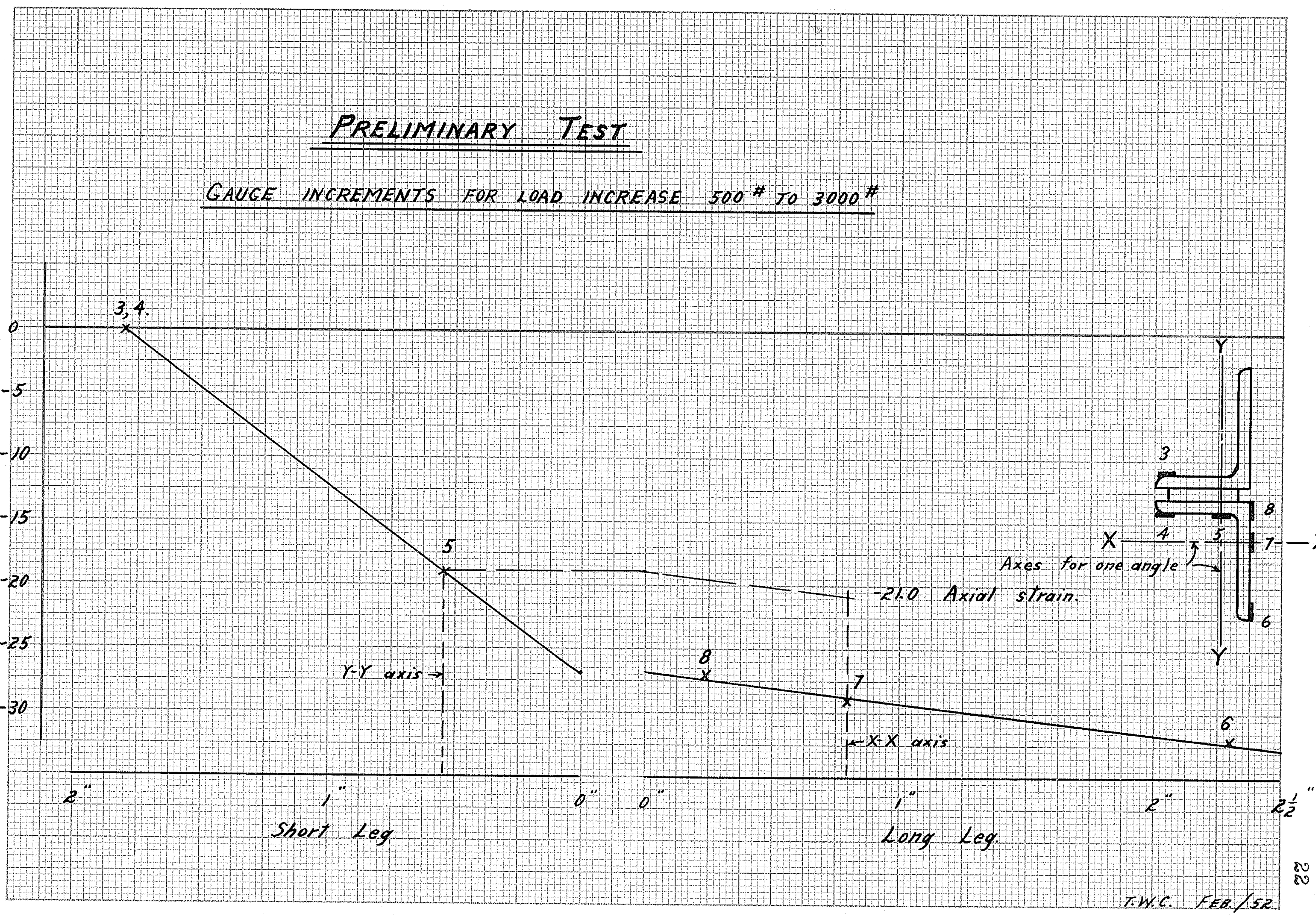
Area of column = 2.12 sq. ins.

$$\text{Axial Strain} = \frac{1250}{2.12 \times 30 \times 10^6} = 19.7 \times 10^{-6}$$

$$= 19.7 \text{ microinches.}$$

As these two results check closely, it was decided the axial strain could be calculated from the strain lines for each leg of the angle. The lines could be obtained by placing a strain gauge as close as possible to each corner of the angle. Thus three gauges were required, similarly placed to gauges 4, 8, and 6 at every point on the column, where readings desired.

Engraving, 7 × 10 in.
MADE IN U. S. A.



GAUGE GROUP LOCATIONS

The next step required was to find suitable locations on the column at which the sets of three gauges could be placed. The strain gauge switch box provided for a maximum of twenty gauges, two of which were required by the gauges on the Tension Loading Bars, leaving eighteen available for the column. As each set required three gauges, six sets could be used. These were placed at six points on the column, which were distributed as shown on the Gauge Location Diagram. The gauges could not all be placed at critical points, for two reasons. Firstly, the beam connections themselves interfered, and secondly, the critical points would not be the same for both the gusset and the seated beam connections.

The distance from the centre line of the gauges to the back of the angle, location and numbering of the gauges is shown in the following tables.

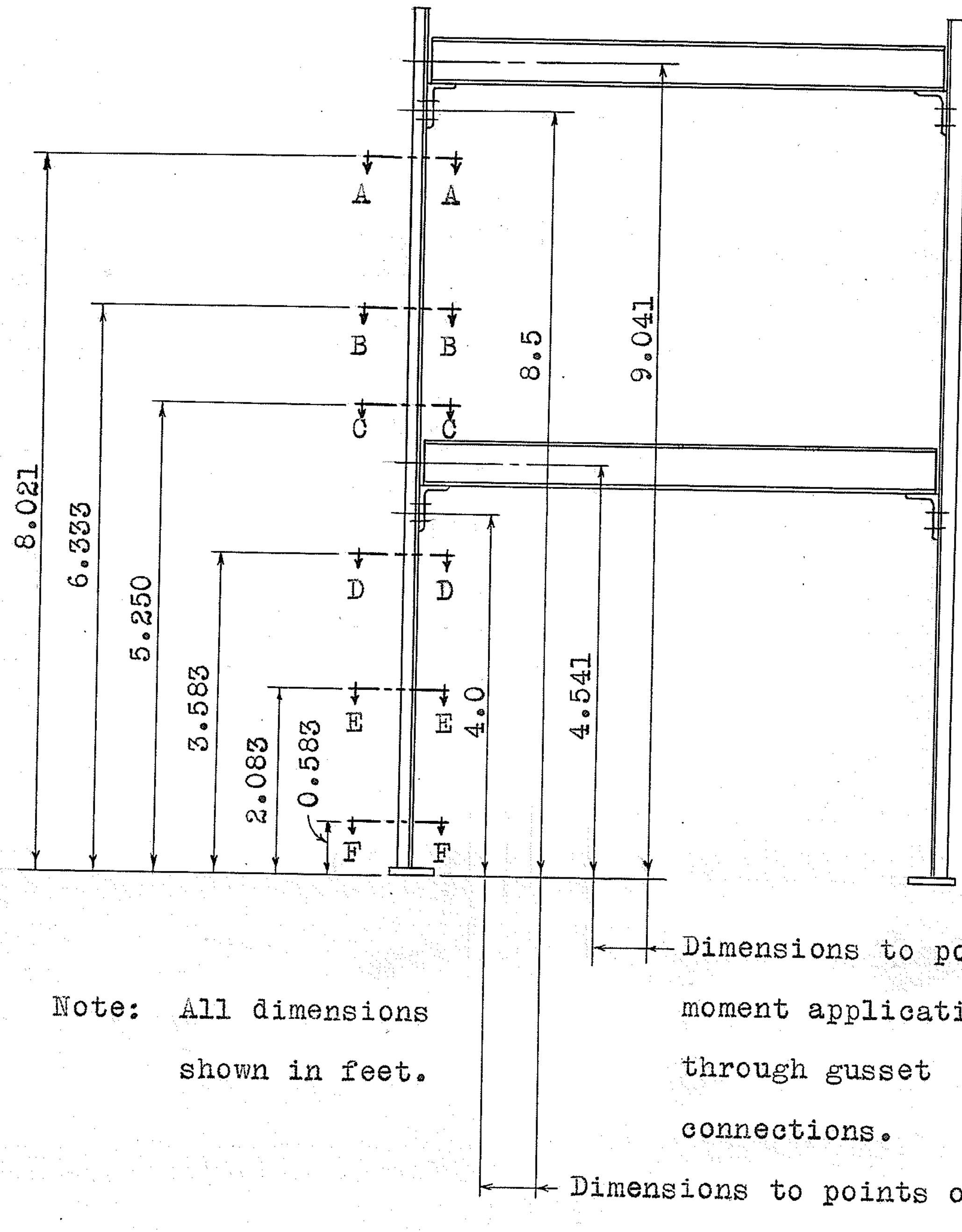
Distance from centre line of gauges to back of angle:

Gauge	Distance	Gauge	Distance	Gauge	Distance
3	1.80"	4	0.23"	5	2.30"
6	1.78	7	0.20	8	2.29
9	1.82	10	0.22	11	2.34
12	1.82	13	0.16	14	2.30
15	1.80	16	0.24	17	2.29
18	1.83	19	0.22	20	2.29

Location	Gauges
Load Gauge (lower beam)	1
Load Gauge (upper beam)	2
Section A-A	3, 4 and 5
Section B-B	6, 7 and 8
Section C-C	9, 10 and 11
Section D-D	12, 13 and 14
Section E-E	15, 16 and 17
Section F-F	18, 19 and 20

The numbering of the gauges at each section is shown in a diagram accompanying the graph for that section.

GAUGE LOCATION DIAGRAM



Dimensions to points of moment application through gusset connections.

Note: All dimensions shown in feet.

Dimensions to points of moment application through seated beam connections.

TEST NO. 1.

For this test the seated beam connections were used, and a load was centrally applied to the lower beam of the Test Frame. The beams made up of 2 channels 6 x 2 at 8.2 lbs. were used. They were connected to the seat angles by two bolts at each end. The load was applied by Tension Bar No. 1 in increments of 500 lbs. up to a maximum of 4000 lbs. Gauge readings were taken after each load increment, and the readings for zero load were checked after the maximum load was removed. The gauge readings were tabulated and are shown on page 28.

The gauge readings for zero load were taken as zero strain, and from them the gauge increments were measured. The increments for each individual gauge were plotted against the applied load and a strain line drawn. From each graph the total strain increment for that gauge, for a load increment of 4000 lbs., was found.

The total strain increments for the three gauges at each section were then plotted according to their location on the leg of the angle, and the axial strain calculated in the same manner as was used in the Preliminary Test.

The bending moment strain at the extreme outside fibre was found by subtracting the axial strain from the total strain at that point. The bending moment strain at the extreme inside fibre was found by subtracting the axial strain from the total strain at the X-X axis of the angle.

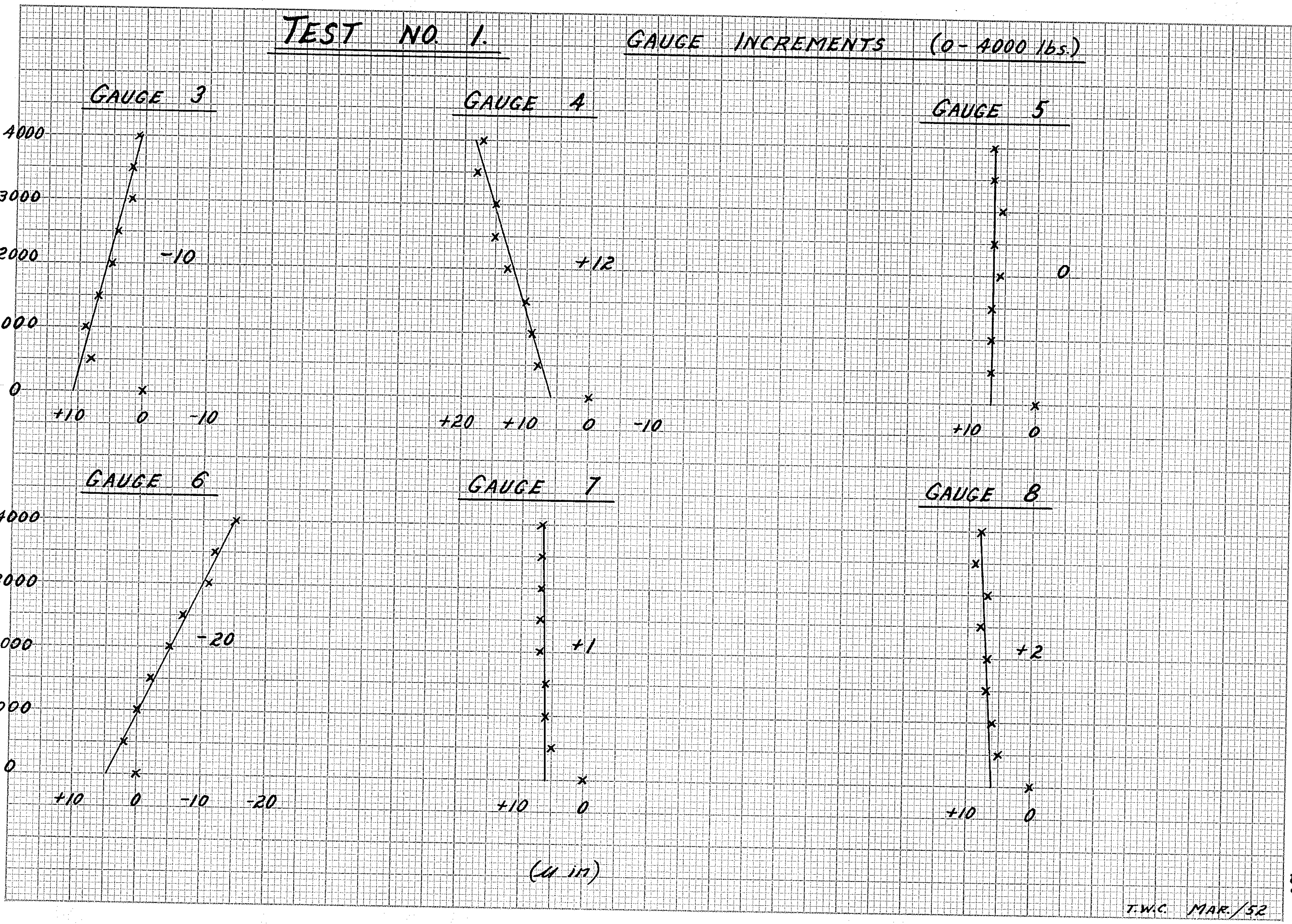
The bending moment at each section was calculated from the bending moment strains at the extreme fibres, and an average taken. A bending moment diagram for the column was then drawn as shown on page 42.

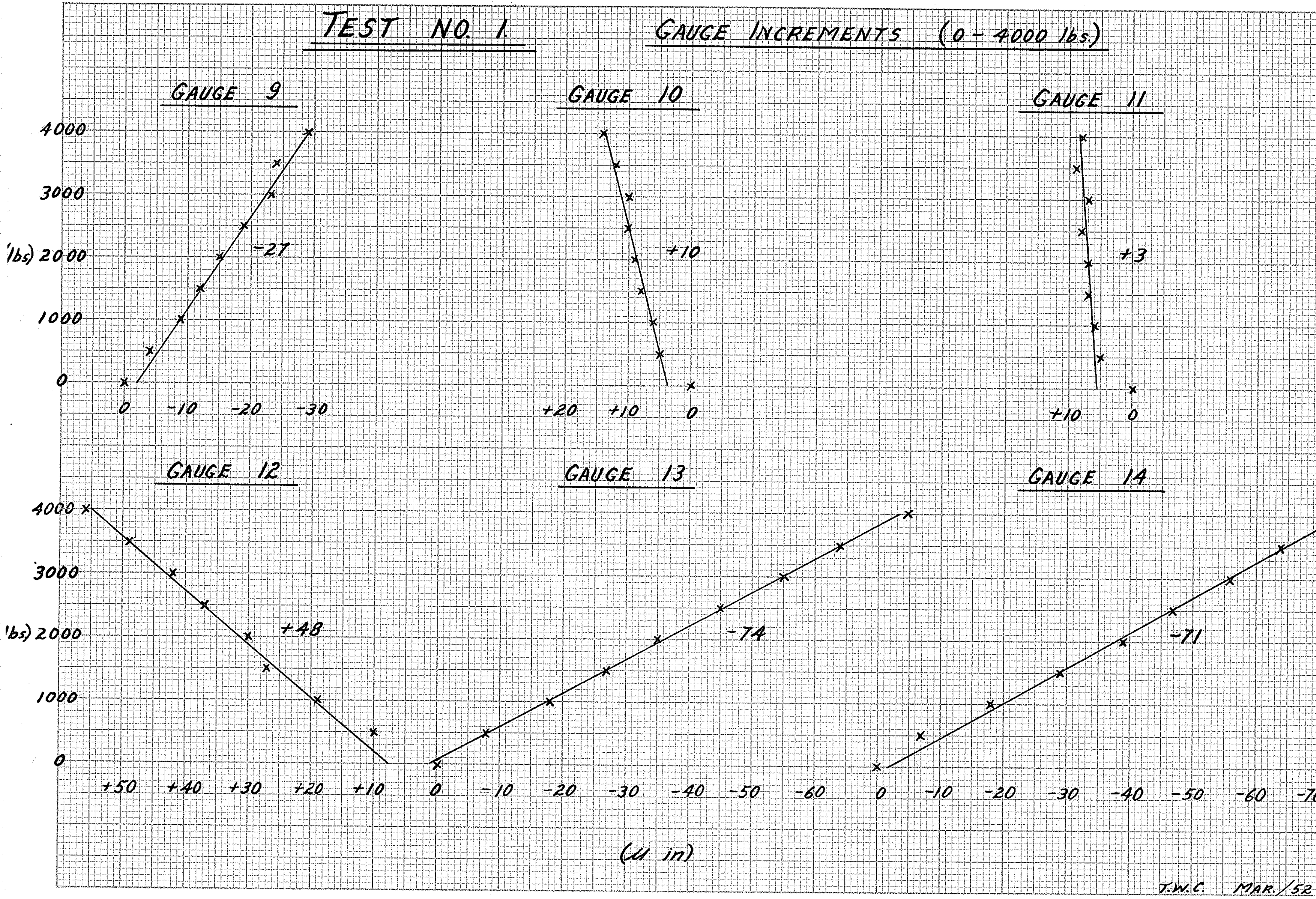
Test No. 1.

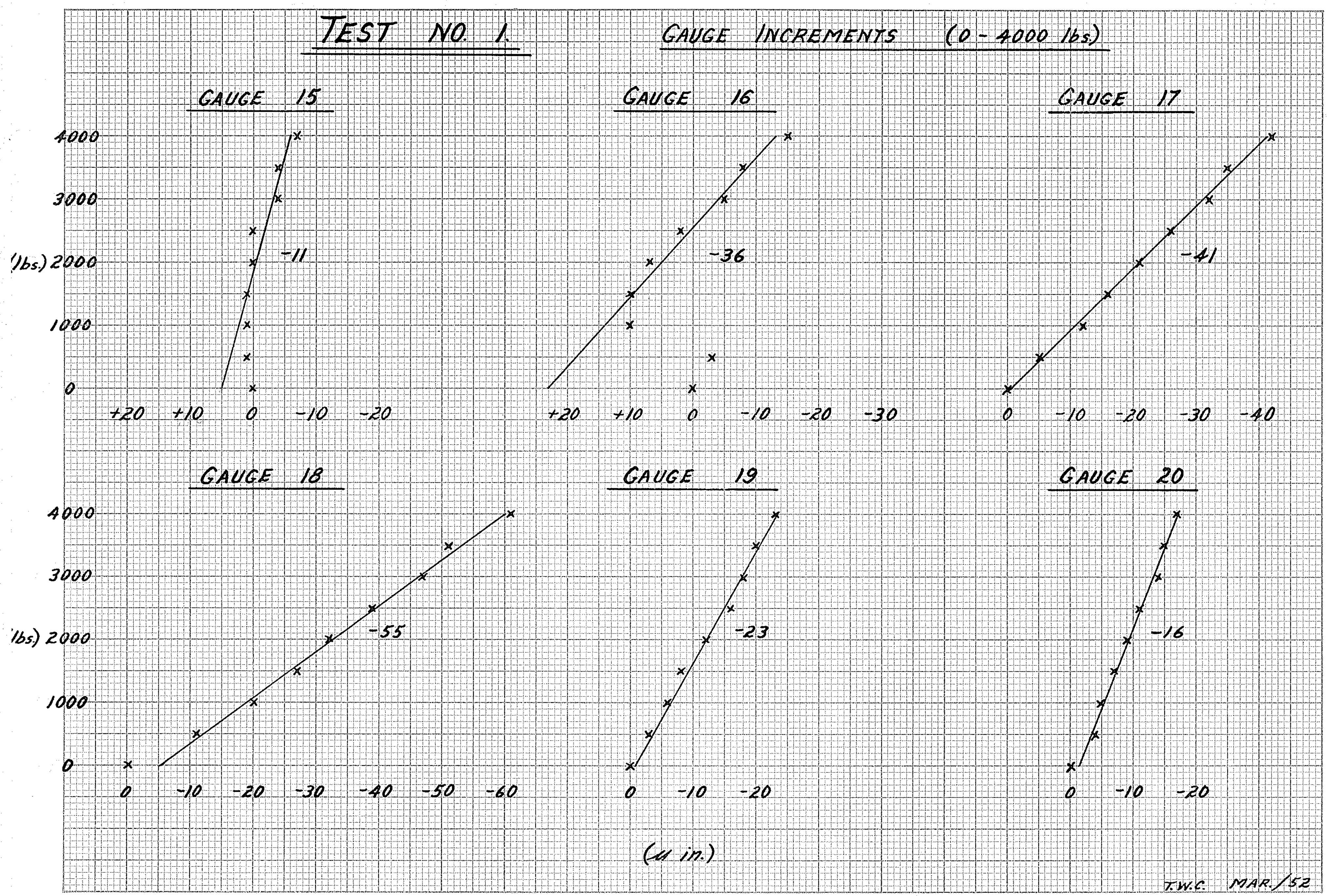
Load (lbs.)	Gauge								
	3	4	5	6	7	8	9	10	11
0	5998	6011	5973	5992	6003	5993	6024	5981	5983
500	6006	6019	5980	5994	6008	5998	6020	5986	5988
1000	6007	6020	5980	5992	6009	5999	6015	5987	5989
1500	6005	6021	5980	5990	6009	6000	6012	5989	5990
2000	6003	6024	5979	5987	6010	6000	6009	5990	5990
2500	6002	6026	5980	5985	6010	6001	6005	5991	5991
3000	6000	6026	5979	5981	6010	6000	6001	5991	5990
3500	6000	6029	5980	5980	6010	6002	6000	5993	5992
4000	5999	6028	5980	5977	6010	6001	5995	5995	5991

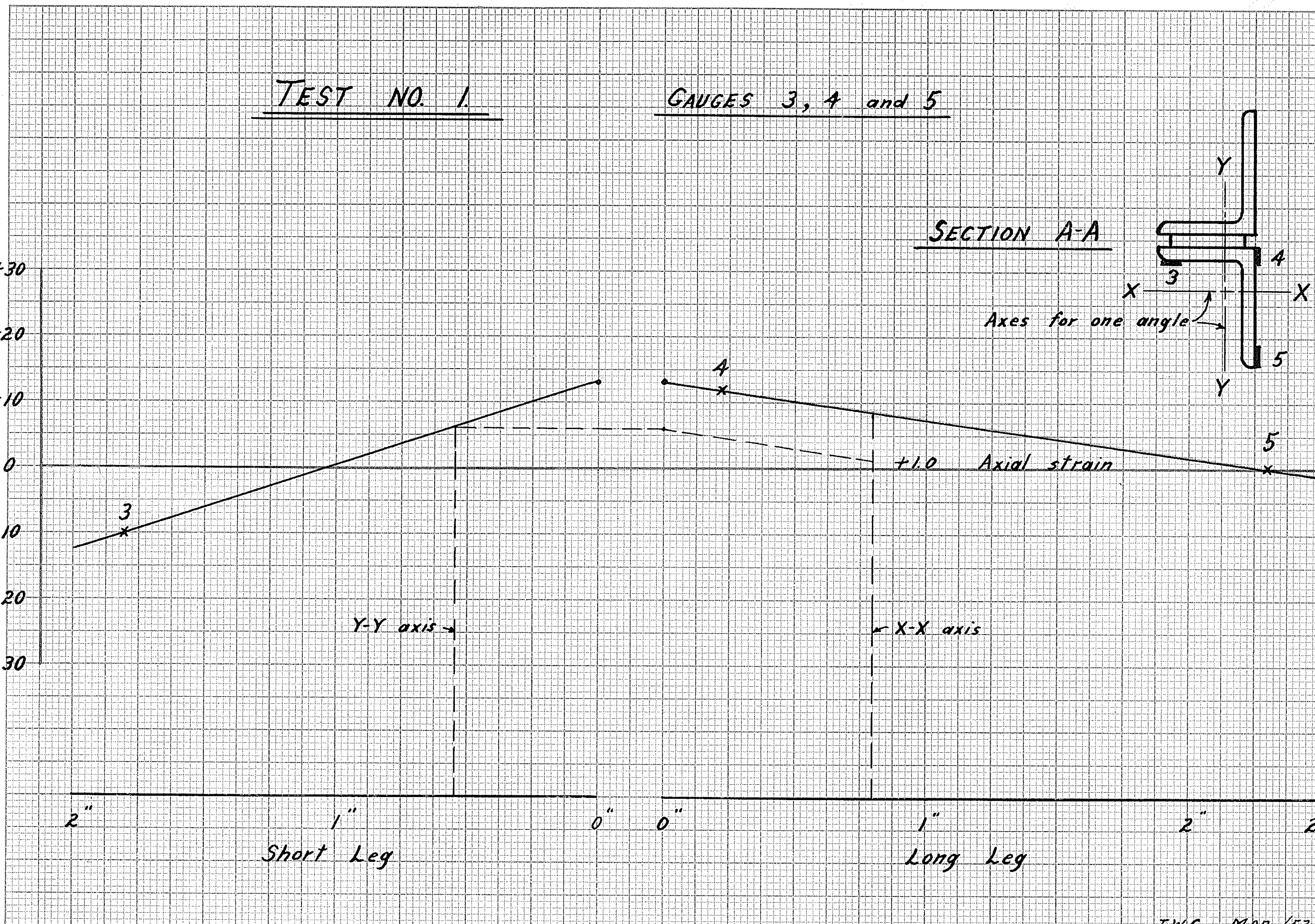
Load (lbs.)	Gauge								
	12	13	14	15	16	17	18	19	20
0	5970	6027	6016	5989	6005	5993	6019	5998	6016
500	5980	6019	6009	5990	6002	5988	6008	5995	6012
1000	5989	6009	5998	5990	6015	5981	5999	5992	6011
1500	5997	6000	5987	5990	6015	5977	5992	5990	6009
2000	6000	5992	5977	5989	6012	5972	5987	5986	6007
2500	6007	5982	5969	5989	6007	5967	5980	5982	6005
3000	6012	5972	5960	5985	6000	5961	5972	5980	6002
3500	6019	5963	5952	5985	5997	5958	5968	5978	6001
4000	6026	5952	5941	5982	5990	5951	5958	5975	5999

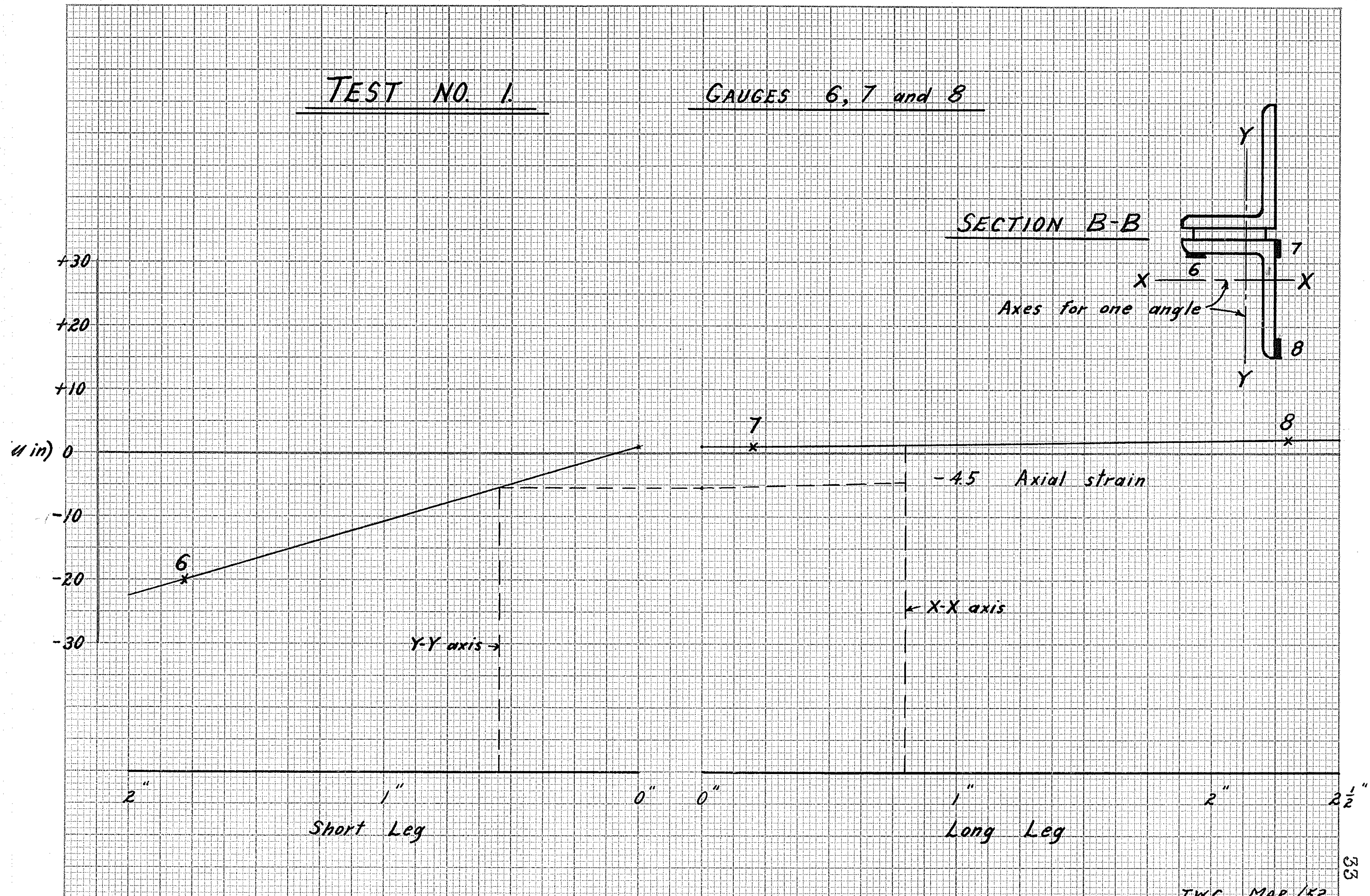
Note: All gauge readings are in microinches.

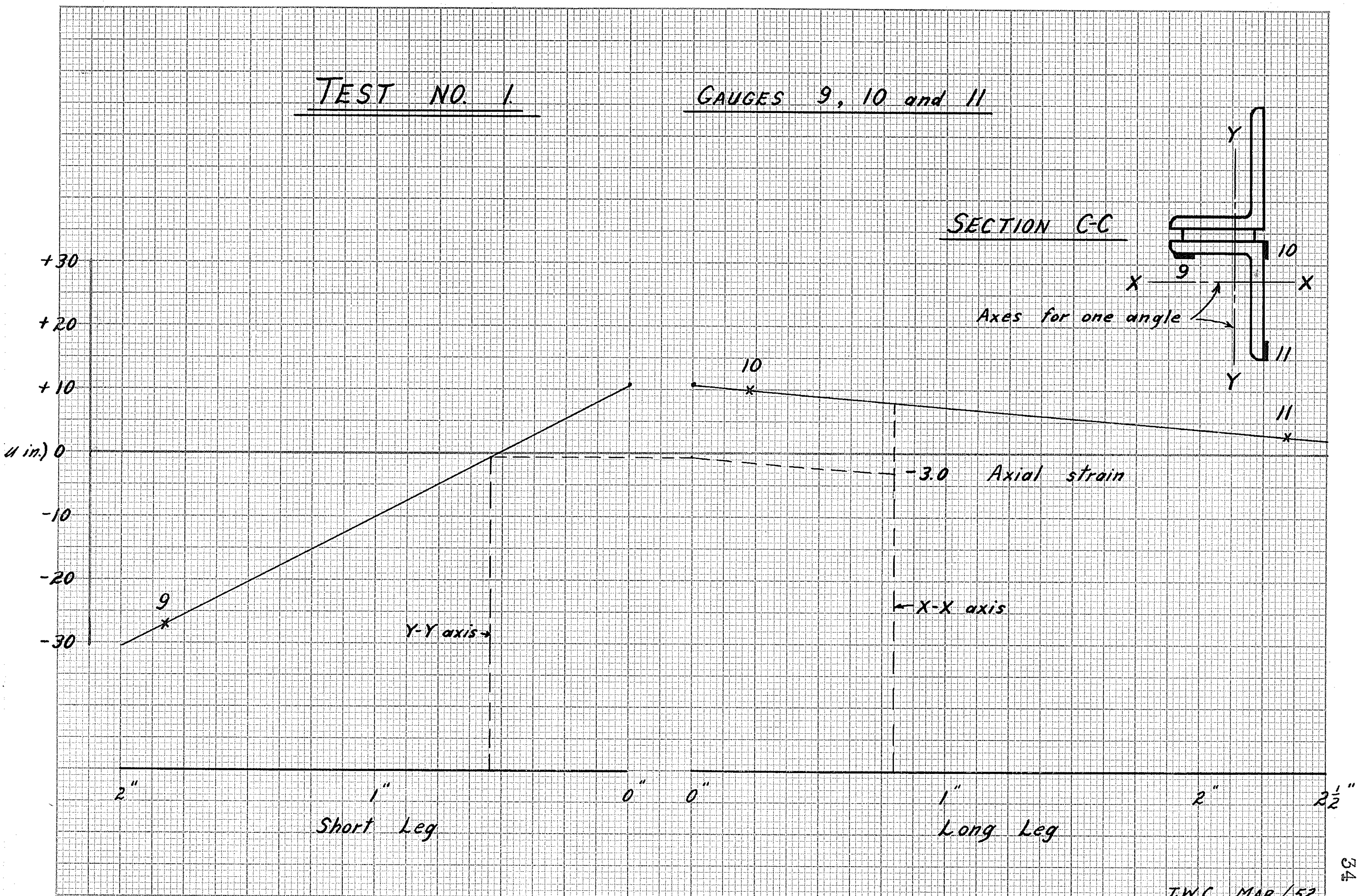

TEST NO. 1.
GAUGE INCREMENTS (0-4000 lbs.)
GAUGE 3
-10
+10 0 -10
GAUGE 4
+12
+20 +10 0 -10
GAUGE 5
0
+10 0
GAUGE 6
-20
+10 0 -10 -20
GAUGE 7
+1
+10 0
GAUGE 8
+2
+10 0
4000
3000
lbs) 2000
1000
0
(μ in)
T.W.C. MAR./52

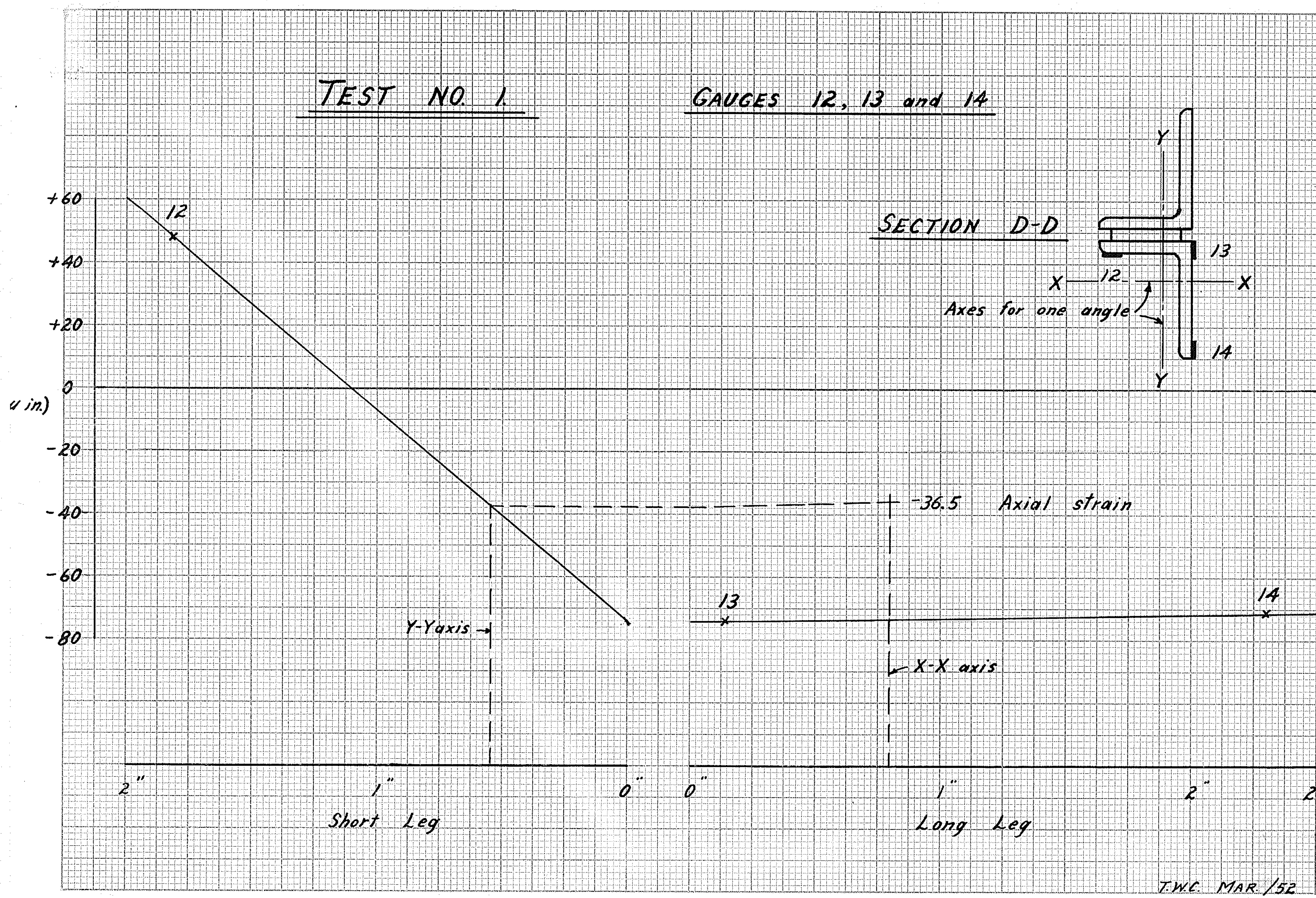

TEST NO. 1.
GAUGE INCREMENTS (0 - 4000 lbs.)
GAUGE 9
-27
GAUGE 10
+10
GAUGE 11
+3
GAUGE 12
+48
GAUGE 13
-74
GAUGE 14
-71
4000
3000
(lbs) 2000
1000
0
0 -10 -20 -30
+20 +10 0
+10 0
+50 +40 +30 +20 +10
0 -10 -20 -30 -40 -50 -60
0 -10 -20 -30 -40 -50 -60 -70
(μ in)
T.W.C. MAR./52

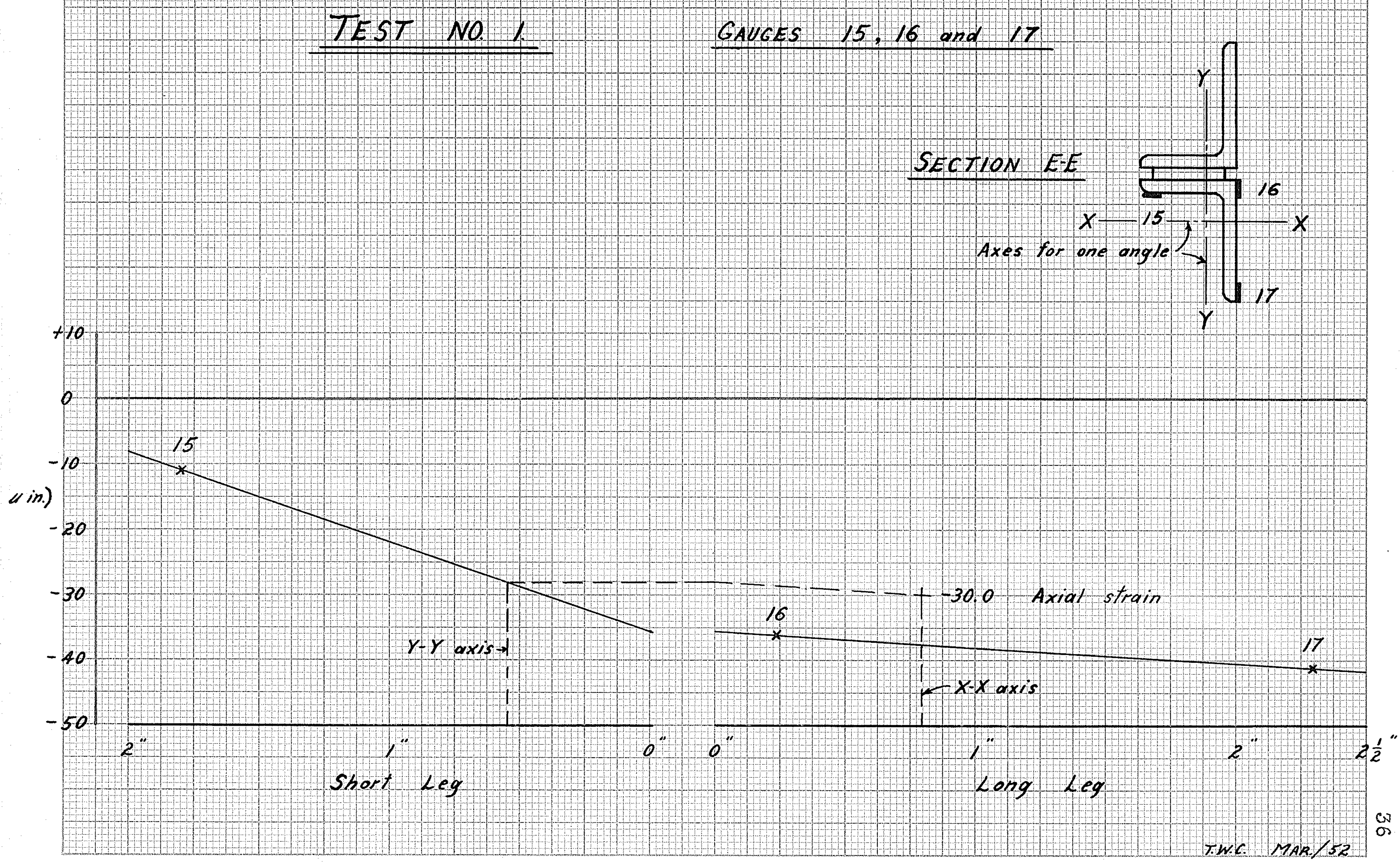

TEST NO. 1.
GAUGE INCREMENTS (0 - 4000 lbs.)
GAUGE 15
-11
GAUGE 16
-36
GAUGE 17
-41
GAUGE 18
-55
GAUGE 19
-23
GAUGE 20
-16
(lbs.)
4000
3000
2000
1000
0
+20
+10
0
-10
-20
-30
-40
-50
-60
(μ in.)
T.W.C. MAR./52

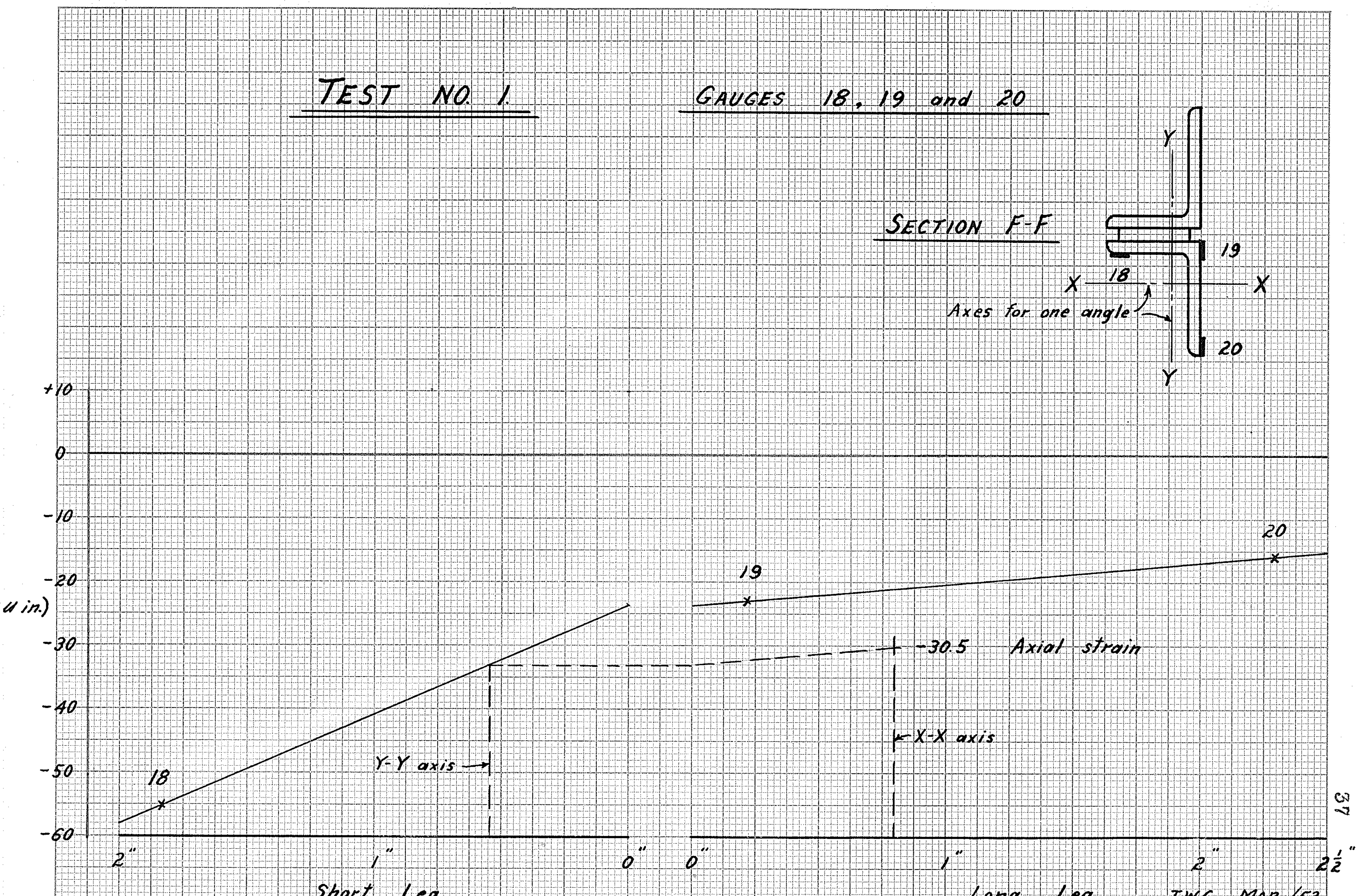

TEST NO. 1.
GAUGES 3, 4 and 5
SECTION A-A
Y
Y
X
X
3
4
5
Axes for one angle
+30
+20
+10
u in) 0
-10
-20
-30
4
5
3
+1.0 Axial strain
Y-Y axis
X-X axis
2"
1"
0"
0"
1"
2"
2½"
Short Leg
Long Leg
T.W.C. MAR./52


TEST NO. 1.
GAUGES 6, 7 and 8
SECTION B-B
Axes for one angle
X
X
Y
Y
6
7
8
+30
+20
+10
(µ in) 0
-10
-20
-30
-4.5 Axial strain
Y-Y axis
X-X axis
2"
1"
0"
0"
1"
2"
2½"
Short Leg
Long Leg
T.W.C. MAR/52


TEST NO. 1
GAUGES 9, 10 and 11
SECTION C-C
Y
X
9
10
11
Axes for one angle
+30
+20
+10
u in.) 0
-10
-20
-30
-3.0
Axial strain
X-X axis
Y-Y axis
2"
1"
0"
Short Leg
Long Leg
2½"
T.W.C. MAR./52


TEST NO. 1
GAUGES 12, 13 and 14
SECTION D-D
Axes for one angle
X
X
Y
Y
12
13
14
+60
+40
+20
0
-20
-40
-60
-80
in.)
-36.5 Axial strain
Y-Y axis
X-X axis
2"
1"
0"
0"
1"
2"
2½"
Short Leg
Long Leg
T.W.C. MAR./52


TEST NO. 1
GAUGES 15, 16 and 17
SECTION E-E
Axes for one angle
X
Y
15
16
17
+10
0
-10
-20
-30
-40
-50
u in.)
-30.0 Axial strain
Y-Y axis
X-X axis
2"
1"
0"
0"
1"
2"
2 1/2"
Short Leg
Long Leg
T.W.C. MAR./52


TEST NO. 1.
GAUGES 18, 19 and 20
SECTION F-F
Y
X
18
X
19
20
Axes for one angle
Y
+10
0
-10
-20
-30
-40
-50
-60
u in.)
18
19
20
-30.5
Axial strain
Y-Y axis
X-X axis
2"
1"
0"
0"
1"
2"
2½"
Short Leg
Long Leg
T.W.C. MAR./52

Test No. 1.

Analysis For Bending Moment

<u>Section A-A</u> (Gauges 3, 4, and 5)

Axial strain = $+ 1 \mu$in.

Bending Moment strain at outside edge of column

$$= -12 - (+1) = -13 \mu \text{ins.}$$

Bending Moment stress at outside edge of column

$$= -13 \times \frac{30 \times 10^6}{10^6} = -13 \times 30 = -390 \text{ p.s.i.}$$
(Compression)

Bending Moment strain at inside edge of column

$$= +8 - (+1) = +7 \mu \text{ins.}$$

Bending Moment stress at inside edge of column

$$= +7 \times 30 = +210 \text{ p.s.i.}$$ (Tension)

Bending Moment (outside edge)

$$= \frac{f\,I}{y} = \frac{-390 \times 0.7}{1.46} = -390 \times 0.479 = -187''\#$$

Bending Moment (inside edge)

$$= \frac{f\,I}{y} = \frac{+210 \times 0.7}{0.54} = +210 \times 1.296 = +272''\#$$

Average Bending Moment at Section A-A

$$= \frac{187 + 272}{2} = 229''\#$$

<u>Section B-B</u> (Gauges 6, 7, and 8)

Axial strain = -4.5μins.

Bending Moment strain at outside edge of column

= - 22.5 - (- 4.5) = - 18μins.

Bending Moment stress at outside edge of column

= - 18 x 30 = - 540 p.s.i. (Compression)

Bending Moment strain at inside edge of column

= + 1.5 - (- 4.5) = + 6μins.

Bending Moment stress at inside edge of column

= + 6 x 30 = + 180 p.s.i. (Tension)

Bending Moment (outside edge) = - 540 x 0.479 = - 259"#

Bending Moment (inside edge) = + 180 x 1.296 = + 233"#

Average Bending Moment at Section B-B = 246"#

<u>Section C-C</u> (Gauges 9, 10, and 11)

Axial strain = - 3.0μins.

Bending Moment strain at outside edge of column

= - 31 - (- 3) = - 28μins.

Bending Moment stress at outside edge of column

= - 28 x 30 = - 840 p.s.i. (Compression)

Bending Moment strain at inside edge of column

= + 8 - (- 3) = + 11μins.

Bending Moment stress at inside edge of column

= + 11 x 30 = +330 p.s.i. (Tension)

Bending Moment (outside edge) = - 840 x 0.479 = - 403"#

Bending Moment (inside edge) = + 330 x 1.296 = + 428"#

Average Bending Moment at Section C-C = 416"#

<u>Section D-D</u> (Gauges 12, 13 and 14)

Axial strain = - 36.5 μ ins.

Bending Moment strain at outside edge of column

= + 60 - (- 36.5) = + 96.5 μ ins.

Bending Moment stress at outside edge of column

= + 96.5 x 30 = + 2895 p.s.i. (Tension)

Bending Moment strain at inside edge of column

= - 73 - (- 36.5) = - 36.5 μ ins.

Bending Moment stress at inside edge of column

= - 36.5 x 30 = - 1095 p.s.i. (Compression)

Bending Moment (outside edge)

= + 2895 x 0.479 = + 1387"#

Bending Moment (inside edge)

= - 1095 x 1.296 = - 1419"#

Average Bending Moment at Section D-D = 1403"#

<u>Section E-E</u> (Gauges 15, 16 and 17)

Axial strain = - 30 μ ins.

Bending Moment strain at outside edge of column

= - 8 - (-30) = + 22 μ ins.

Bending Moment stress at outside edge of column

= + 22 x 30 = + 660 p.s.i. (Tension)

Bending Moment strain at inside edge of column

= - 37.5 - (- 30) = - 7.5 μ ins.

Bending Moment stress at inside edge of column

= - 7.5 x 30 = - 225 p.s.i. (Compression)

Bending Moment (outside edge) = + 660 x 0.479 = + 316"#

Bending Moment (inside edge) = - 225 x 1.296 = - 292"#

Average Bending Moment at Section E-E = 304"#

<u>Section F-F</u> (Gauges 18, 19 and 20)

Axial strain = - 30.5 μins.

Bending Moment strain at outside edge of column

= - 58 - (- 30.5) = - 27.5 μins.

Bending Moment stress at outside edge of column

= - 27.5 x 30 = - 825 p.s.i. (Compression)

Bending Moment strain at inside edge of column

= - 21 - (- 30.5) = + 9.5 μins.

Bending Moment stress at inside edge of column

= + 9.5 x 30 = + 285 p.s.i. (Tension)

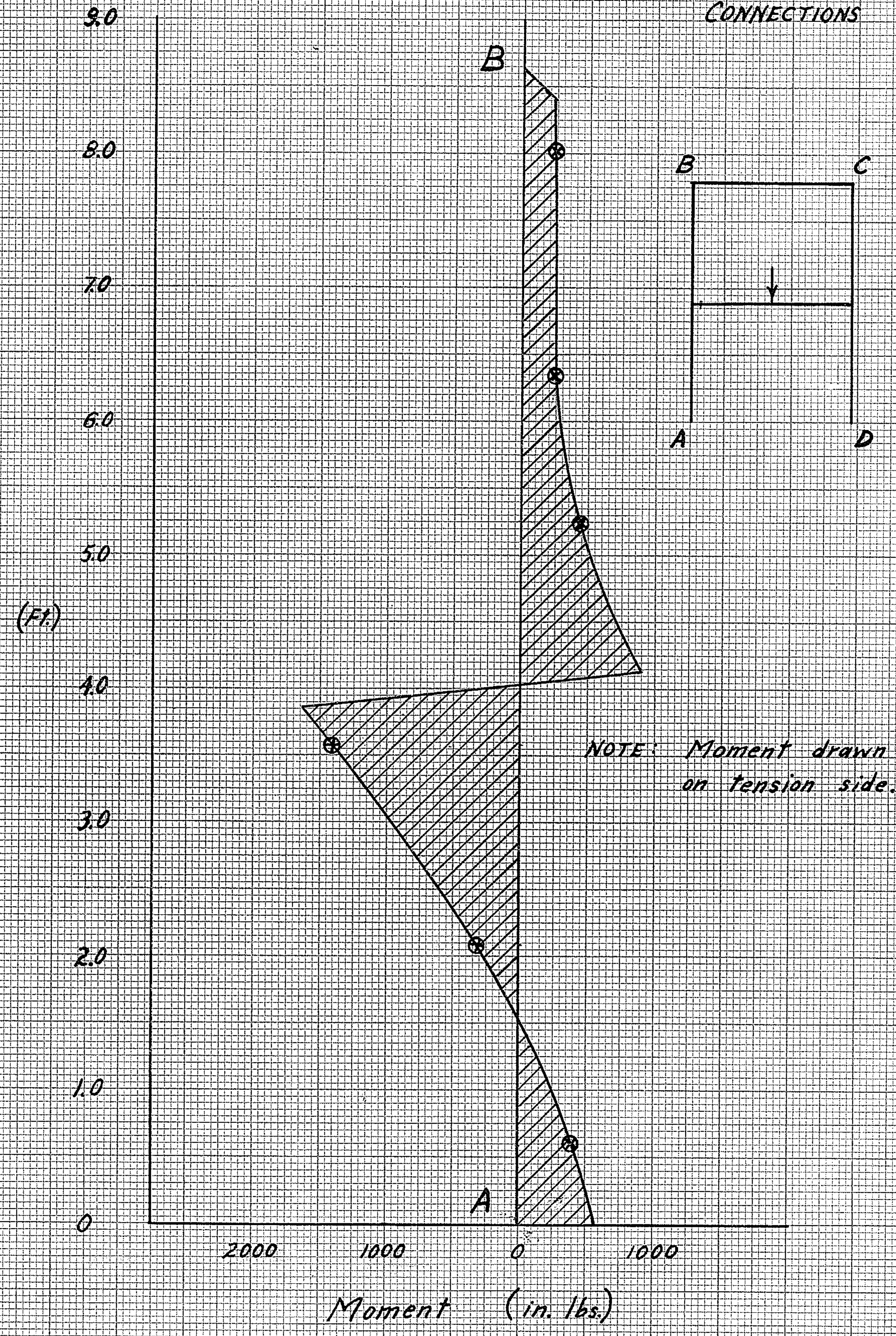
Bending Moment (outside edge) = - 825 x 0.479 = - 396"#

Bending Moment (inside edge) = + 285 x 1.296 = + 369"#

Average Bending Moment at Section F-F = 382"#

TEST NO. 1. BENDING MOMENT DIAGRAM FOR COLUMN AB

SEATED CONNECTIONS



T.W.C. MAR./52.

TEST NO. 2.

For this test the gusset connections were used, and a load was centrally applied on the lower beam of the Test Frame.

The same procedure of loading, taking gauge readings, and calculating the bending moments, as used in Test No. 1 was followed.

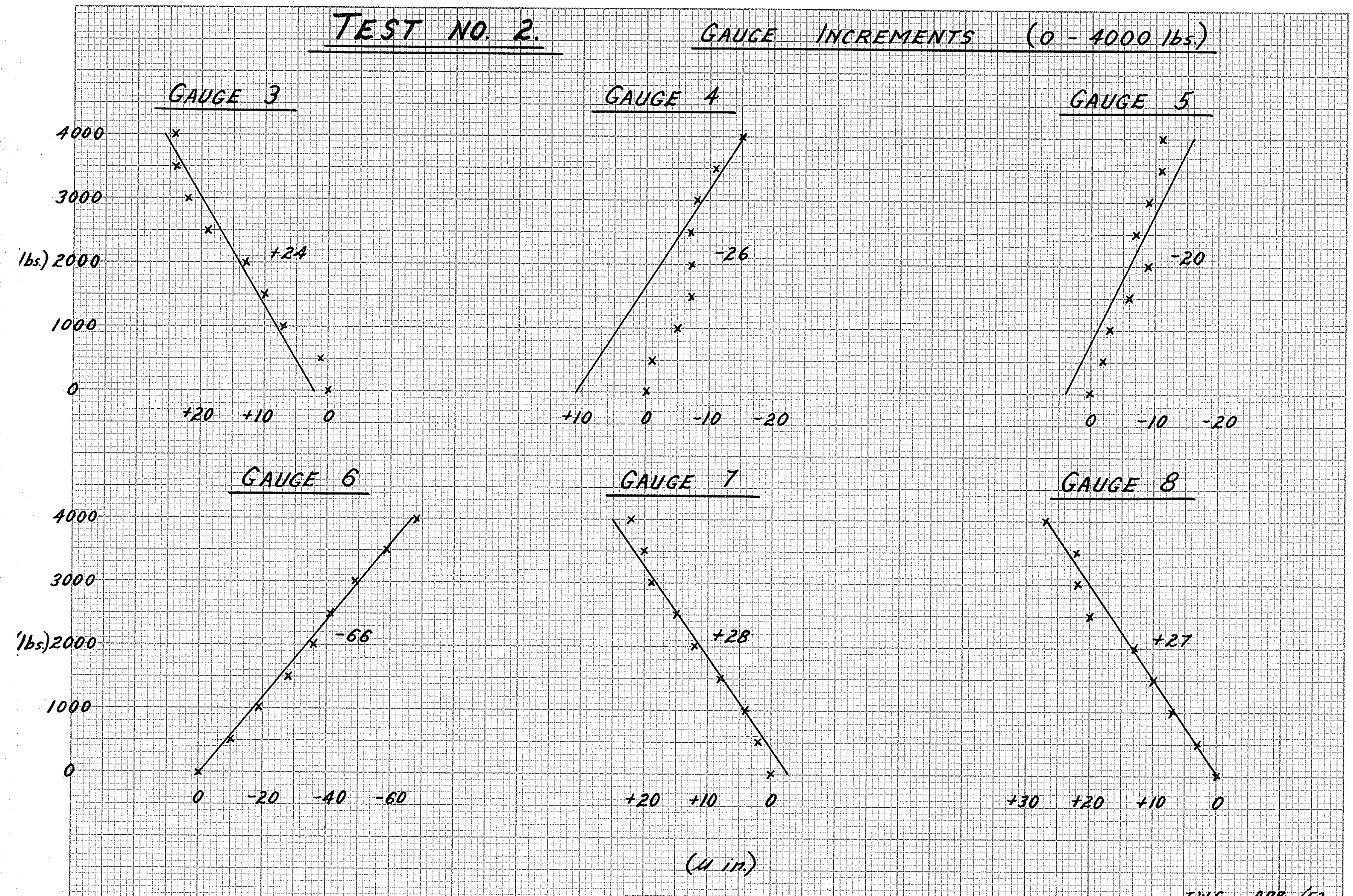
A bending moment diagram for the column was again drawn as shown on page 58.

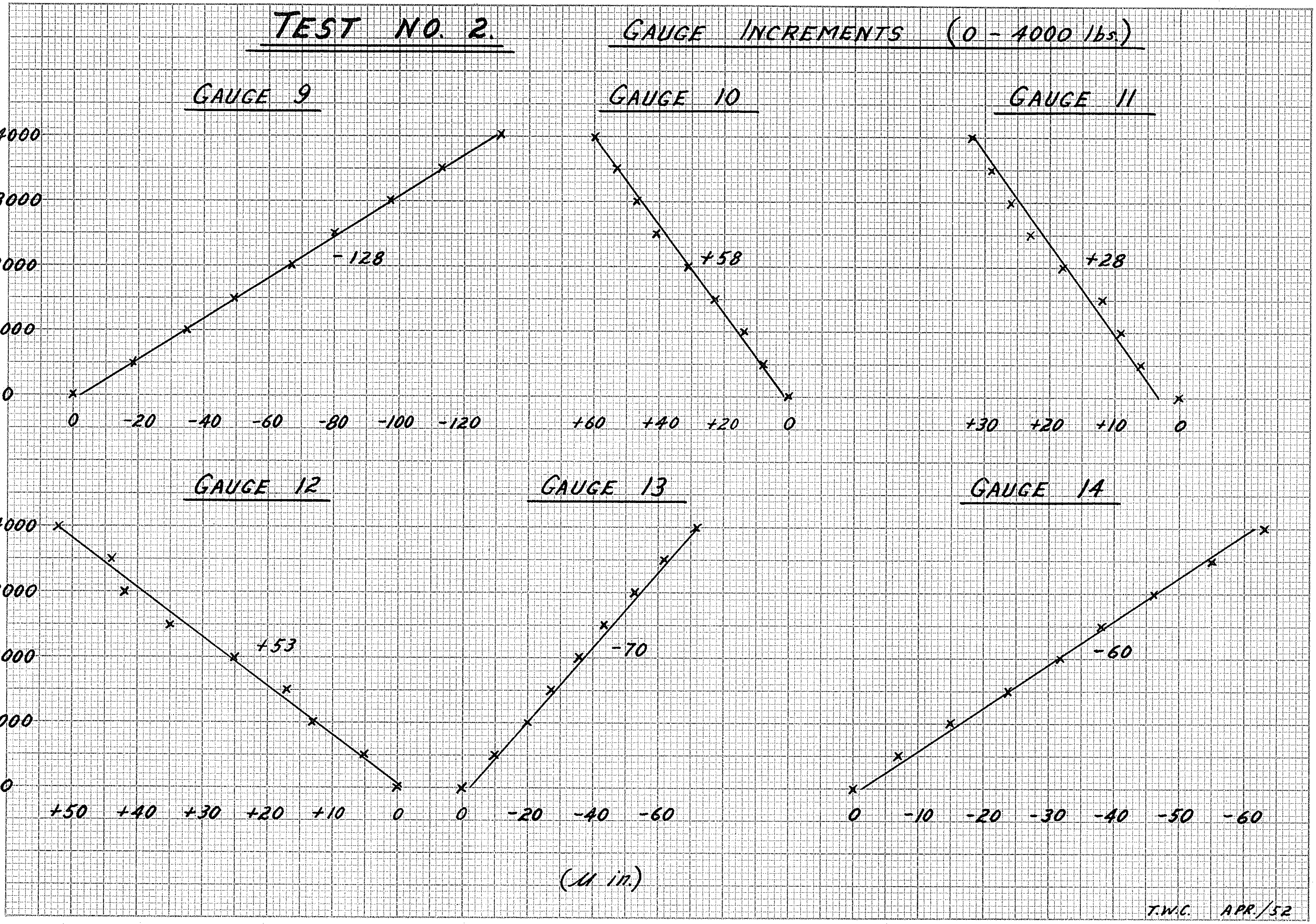
Test No. 2.

Load (lbs.)	Gauge 3	4	5	6	7	8	9	10	11
0	6060	6088	6051	6068	6087	6088	6118	6057	6060
500	6061	6087	6049	6058	6089	6091	6100	6065	6066
1000	6067	6083	6048	6049	6091	6095	6083	6071	6069
1500	6070	6081	6045	6040	6095	6098	6069	6080	6072
2000	6073	6081	6042	6032	6099	6101	6051	6088	6078
2500	6079	6081	6044	6027	6102	6108	6038	6098	6083
3000	6082	6080	6042	6019	6106	6110	6021	6104	6086
3500	6084	6077	6040	6009	6107	6110	6005	6110	6089
4000	6084	6073	6040	6000	6109	6115	5987	6117	6092

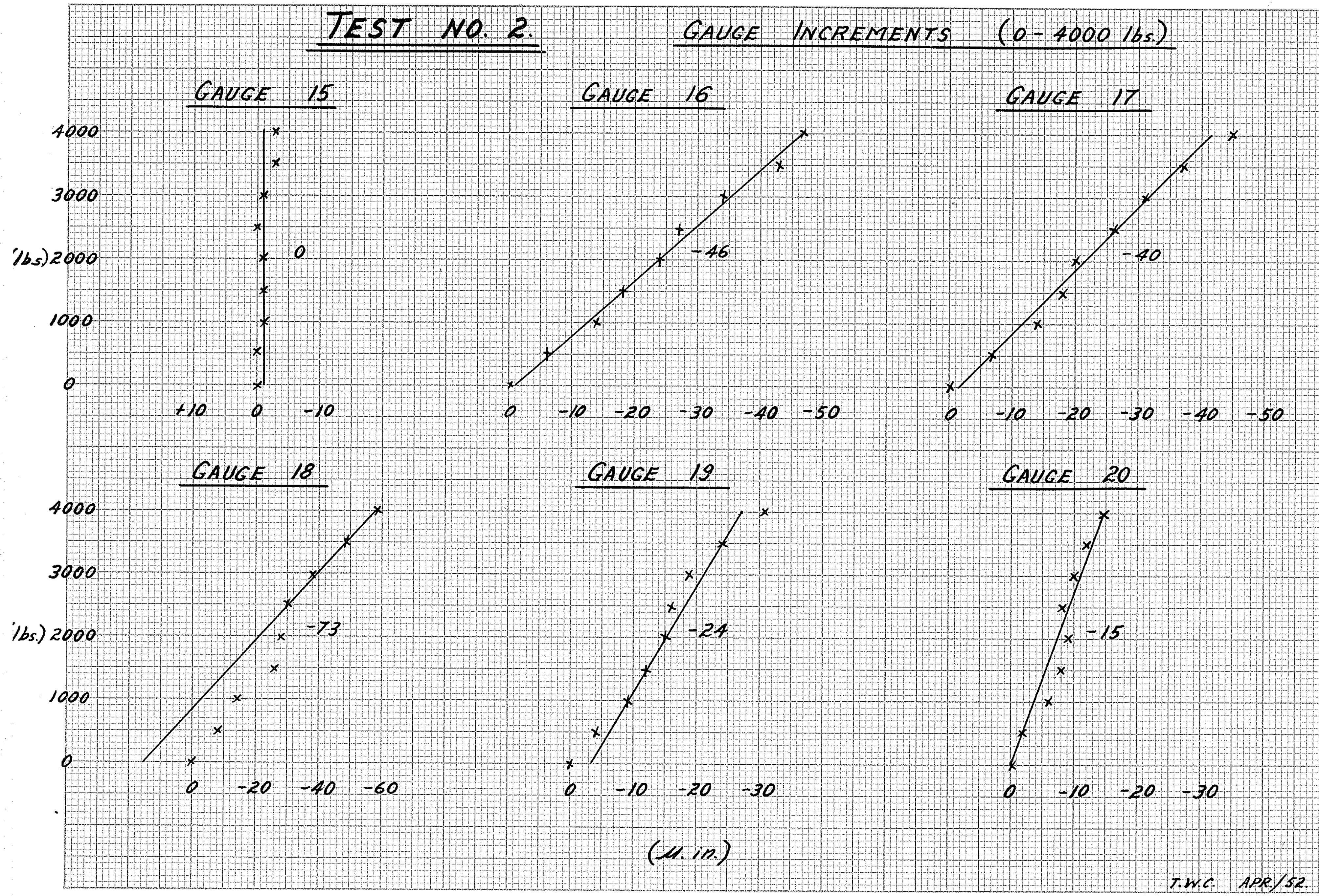
Load (lbs.)	Gauge 12	13	14	15	16	17	18	19	20
0	6066	6062	6063	6052	6095	6117	6118	6121	6129
500	6071	6052	6056	6052	6089	6110	6110	6117	6127
1000	6079	6042	6048	6051	6081	6103	6104	6112	6123
1500	6083	6035	6039	6051	6077	6099	6092	6109	6121
2000	6091	6026	6031	6051	6071	6097	6090	6106	6120
2500	6101	6019	6025	6052	6068	6091	6088	6105	6121
3000	6108	6009	6017	6051	6061	6086	6080	6102	6119
3500	6110	6000	6008	6049	6052	6080	6069	6097	6117
4000	6118	5990	6000	6049	6048	6072	6058	6090	6114

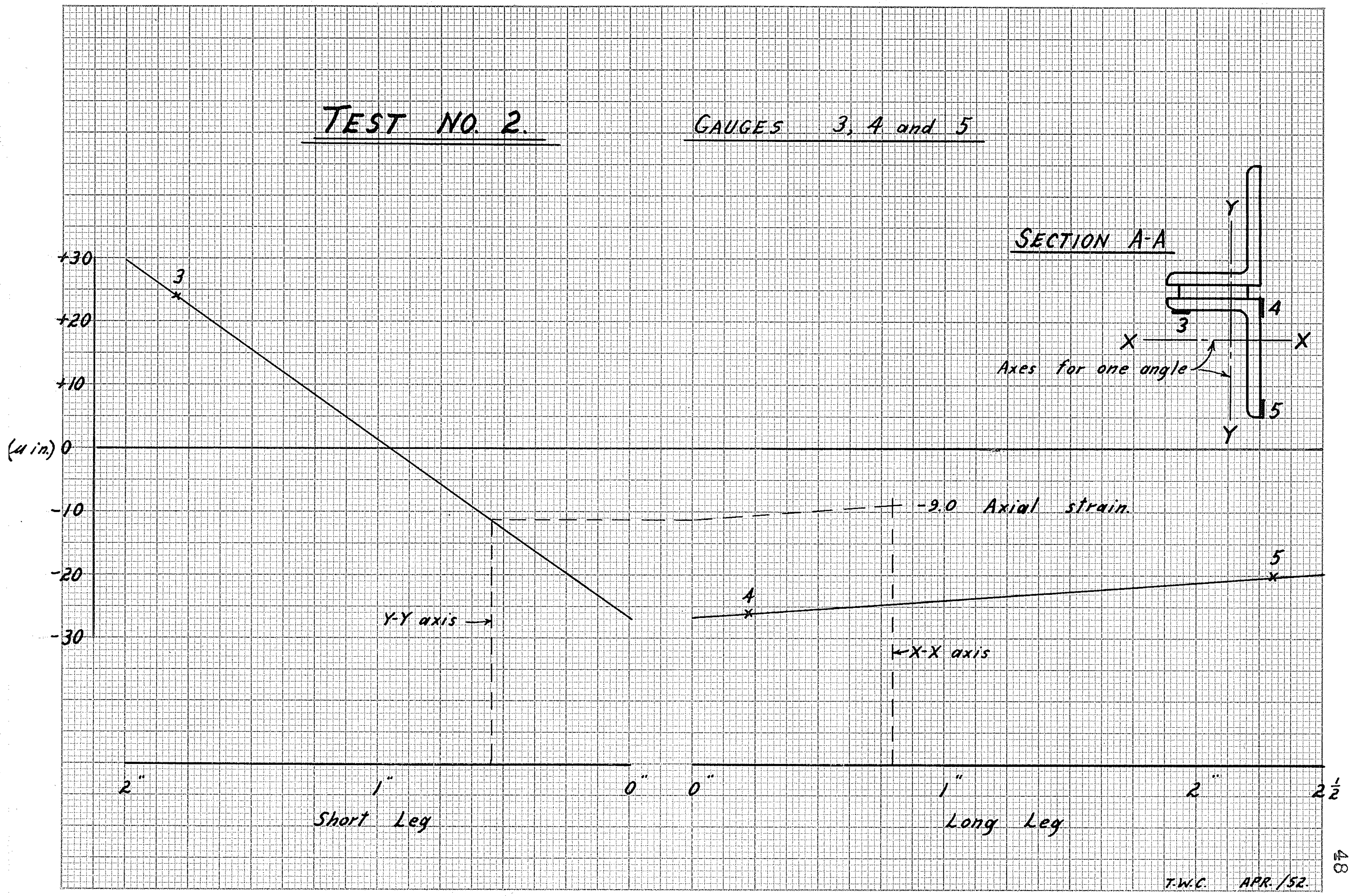
Note: All gauge readings are in microinches.

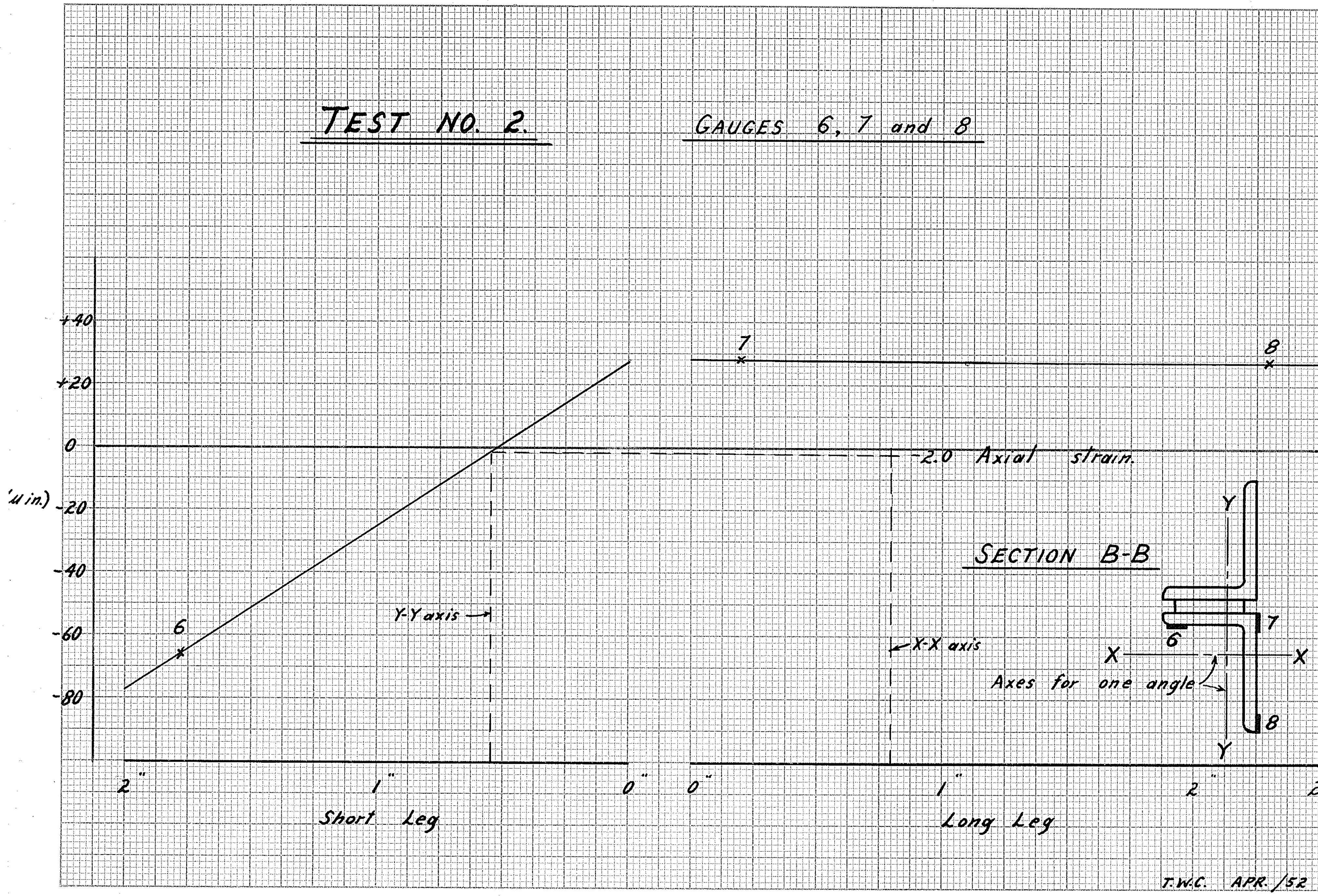

TEST NO. 2.
GAUGE INCREMENTS (0 - 4000 lbs.)
GAUGE 3
+24
GAUGE 4
-26
GAUGE 5
-20
GAUGE 6
-66
GAUGE 7
+28
GAUGE 8
+27
4000
3000
(lbs.) 2000
1000
0
+20 +10 0
+10 0 -10 -20
0 -10 -20
0 -20 -40 -60
+20 +10 0
+30 +20 +10 0
(μ in.)
T.W.C. APR. /52

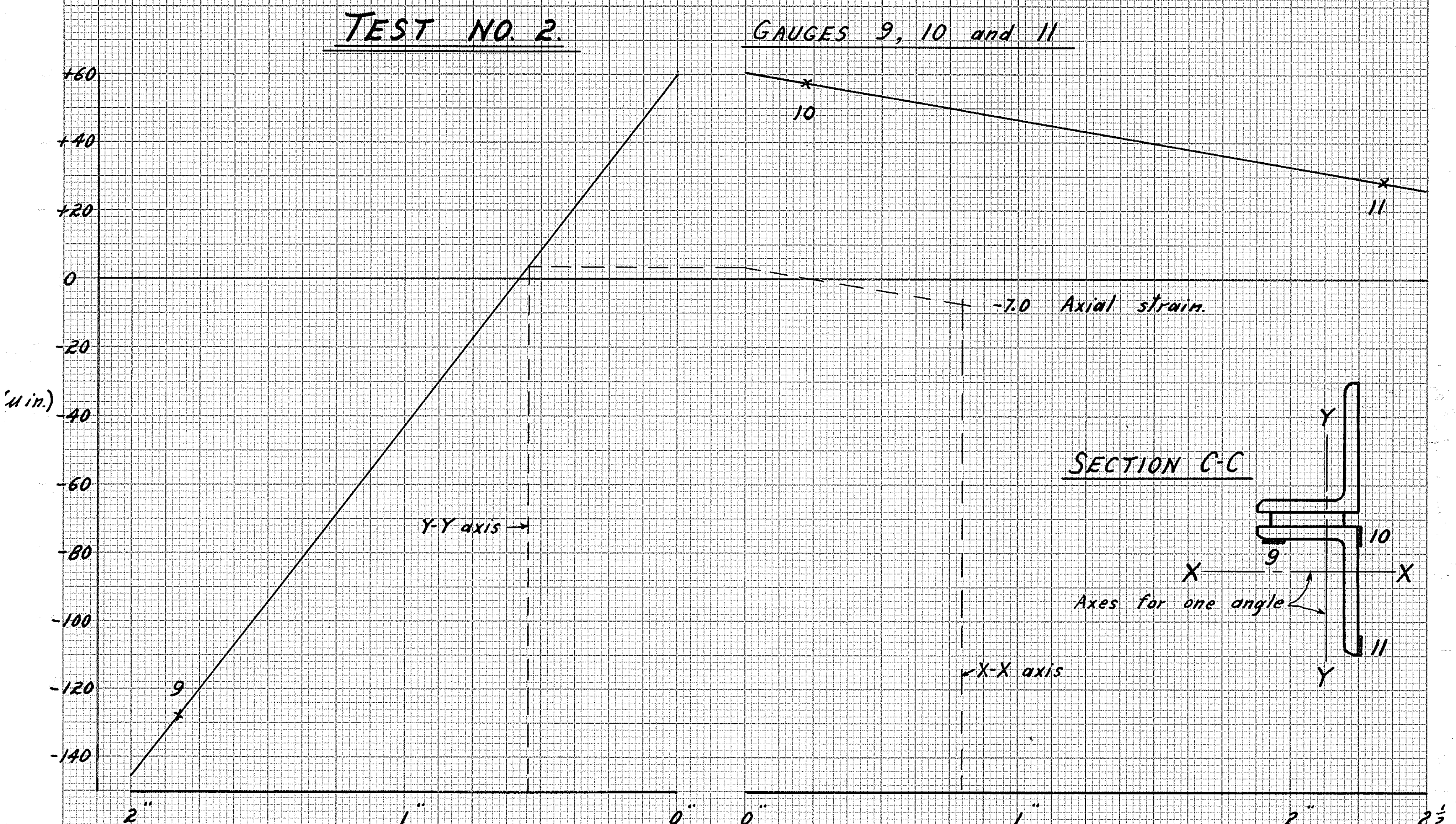

TEST NO. 2.
GAUGE INCREMENTS (0 - 4000 lbs.)
GAUGE 9
GAUGE 10
GAUGE 11
GAUGE 12
GAUGE 13
GAUGE 14
-128
+58
+28
+53
-70
-60
(lbs.)
4000
3000
2000
1000
0
0 -20 -40 -60 -80 -100 -120
+60 +40 +20 0
+30 +20 +10 0
+50 +40 +30 +20 +10 0
0 -20 -40 -60
0 -10 -20 -30 -40 -50 -60
(μ in.)
T.W.C. APR./52

Engraving, 7 × 10 in.
MADE IN U. S. A.




TEST NO. 2.
GAUGES 3, 4 and 5
SECTION A-A
Axes for one angle
X
X
Y
Y
3
4
5
+30
+20
+10
(μ in.) 0
-10
-20
-30
-9.0 Axial strain.
Y-Y axis
X-X axis
2"
1"
0"
0"
1"
2"
2½"
Short Leg
Long Leg
T.W.C. APR. /52.

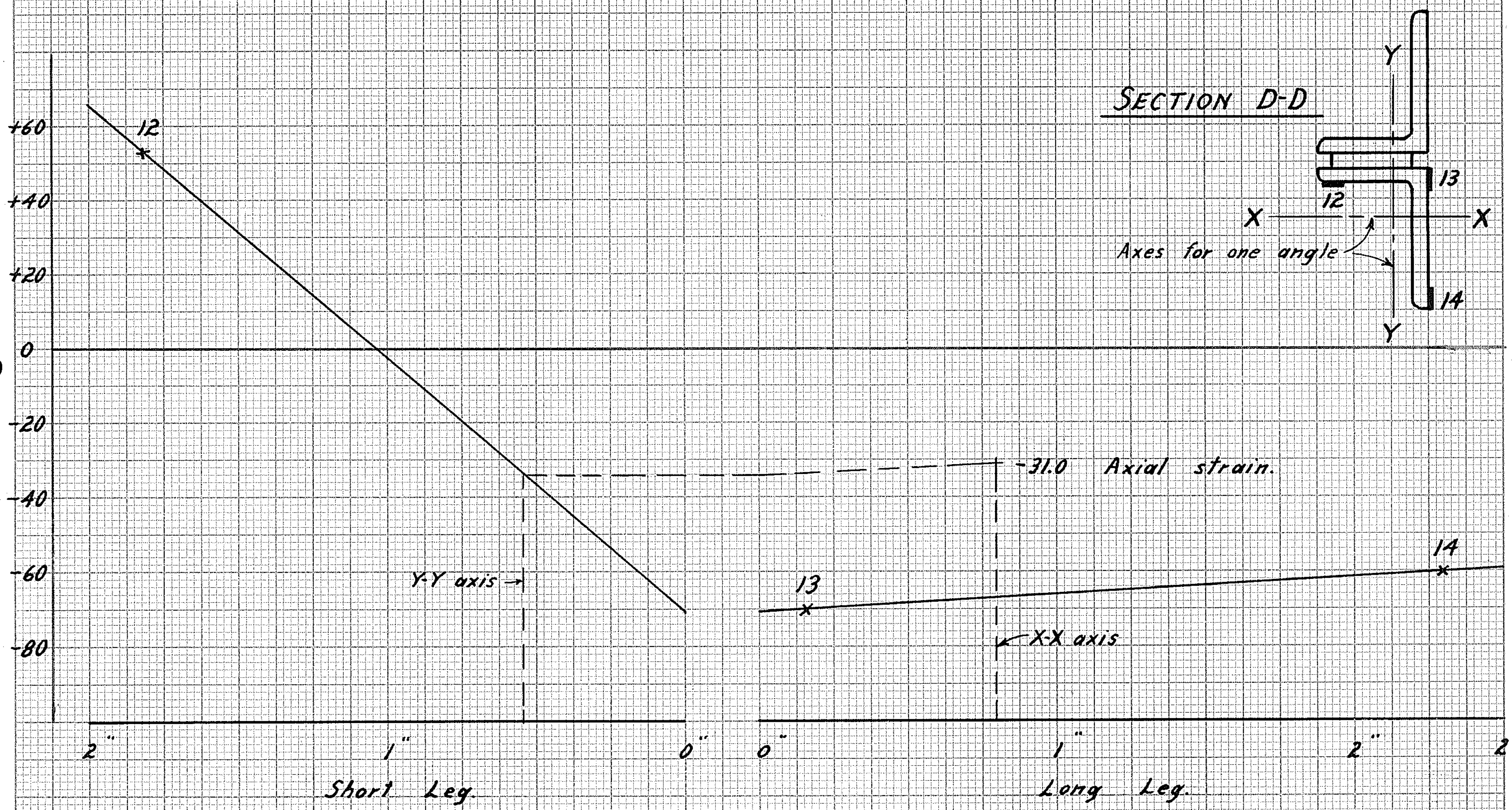

TEST NO. 2.
GAUGES 6, 7 and 8
+40
+20
0
(μ in.) -20
-40
-60
-80
6
7
8
Y-Y axis
X-X axis
-2.0 Axial strain.
SECTION B-B
Y
X
Axes for one angle
2"
1"
0"
0"
1"
2"
2½"
Short Leg
Long Leg
T.W.C. APR. /52


TEST NO. 2.
GAUGES 9, 10 and 11
+60
+40
+20
0
-20
(µ in.) -40
-60
-80
-100
-120
-140
9
10
11
-7.0 Axial strain.
Y-Y axis
X-X axis
SECTION C-C
Axes for one angle
X
Y
2"
1"
0"
0"
1"
2"
2½"

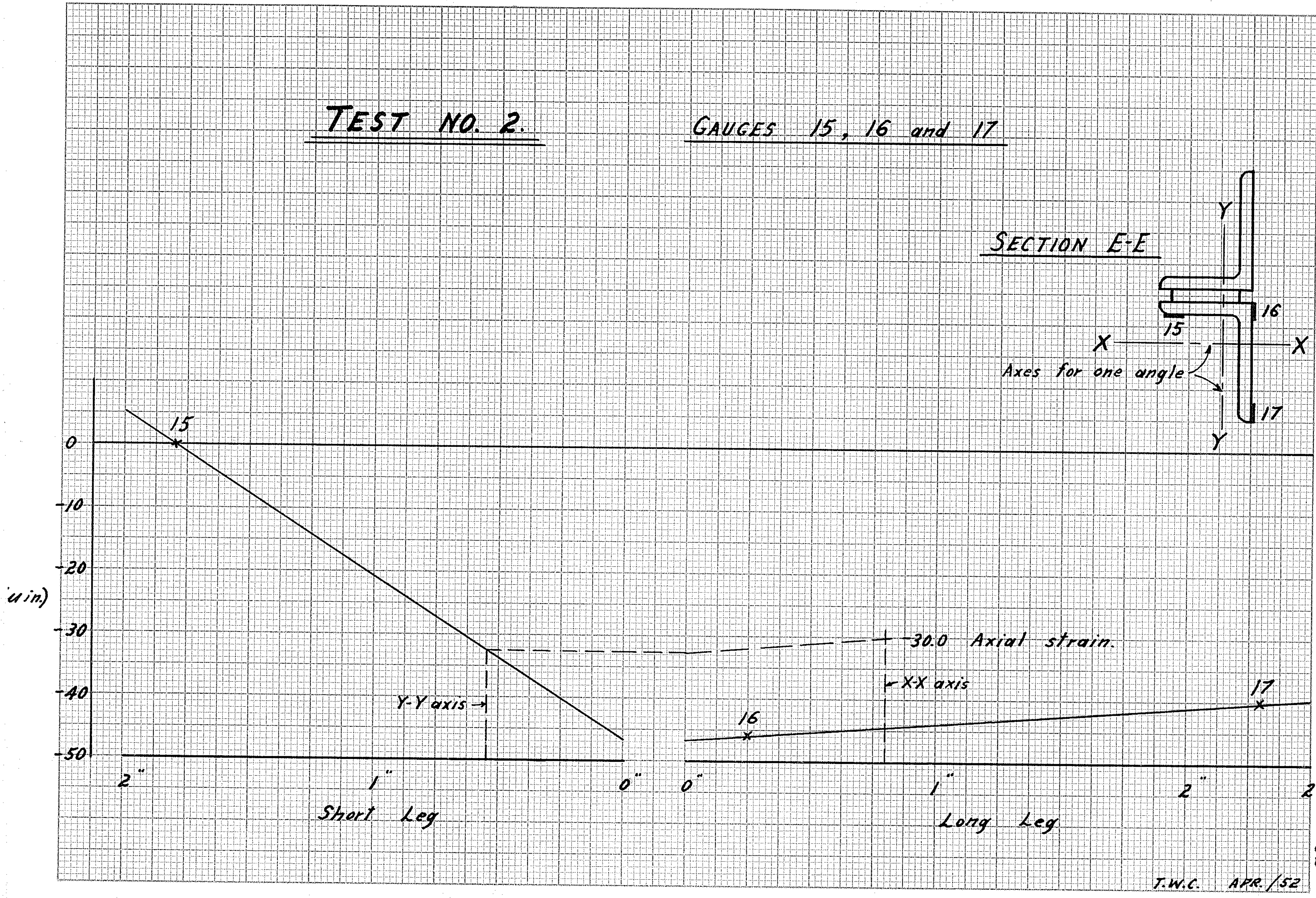
T.W.C. APR./52.

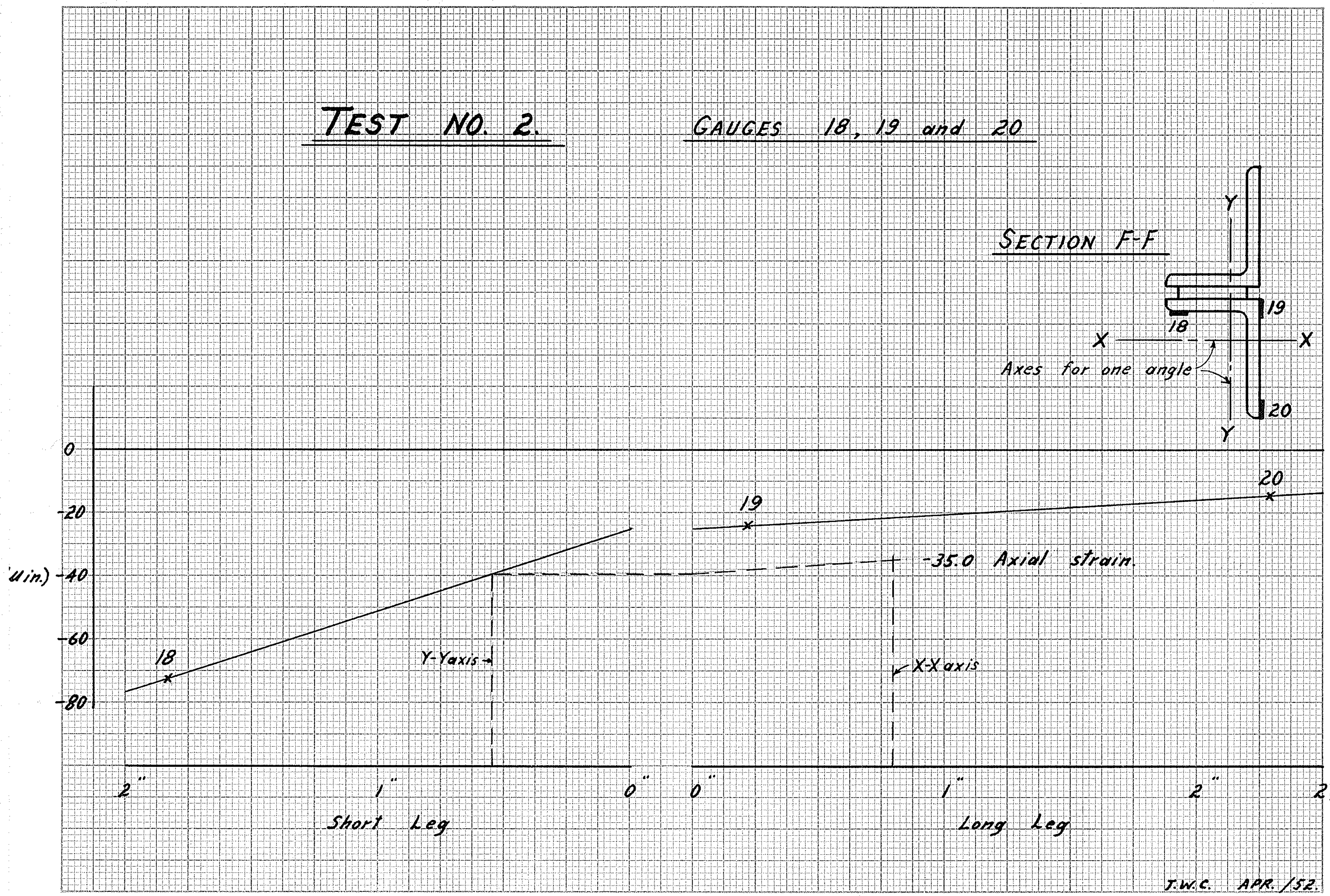
TEST NO. 2.

GAUGES 12, 13 and 14



T.W.C. APR./52.


TEST NO. 2.
GAUGES 15, 16 and 17
SECTION E-E
Axes for one angle
X
X
Y
Y
15
16
17
0
-10
-20
-30
-40
-50
u in.)
-30.0 Axial strain.
X-X axis
Y-Y axis
2"
1"
0"
0"
1"
2"
2½"
Short Leg
Long Leg
T.W.C. APR./52


TEST NO. 2.
GAUGES 18, 19 and 20
SECTION F-F
Y
Y
X
X
18
19
20
Axes for one angle
0
-20
-40
-60
-80
μin.)
18
19
20
-35.0 Axial strain.
Y-Yaxis
X-X axis
2"
1"
0"
0"
1"
2"
2½"
Short Leg
Long Leg
T.W.C. APR. /52.

Test No. 2.

Analysis For Bending Moment

<u>Section A-A</u> (Gauges 3, 4, and 5)

Axial strain = - 9.0 μ ins.

Bending Moment strain at outside edge of column

= + 30 - (- 9) = + 39 μ ins.

Bending Moment stress at outside edge of column

= + 39 x 30 = + 1170 p.s.i. (Tension)

Bending Moment strain at inside edge of column

= - 24.5 - (- 9) = - 15.5 μ ins.

Bending Moment stress at inside edge of column

= - 15.5 x 30 = - 465 p.s.i. (Compression)

Bending Moment (outside edge)

= + 1170 x 0.479 = + 560"#

Bending Moment (inside edge) = - 465 x 1.296 = - 602"#

Average Bending Moment at Section A-A = 581"#

<u>Section B-B</u> (Gauges 6, 7, and 8)

Axial strain = - 2.0 μ ins.

Bending Moment strain at outside edge of column

= - 77 - (- 2) = - 75 μ ins.

Bending Moment stress at outside edge of column

= - 75 x 30 = - 2250 p.s.i. (Compression)

Bending Moment strain at inside edge of column

= + 28 - (- 2) = + 30 μ ins.

Bending Moment stress at inside edge of column

= + 30 x 30 = + 900 p.s.i. (Tension)

Bending Moment (outside edge)

= - 2250 x 0.479 = - 1080"#

Bending Moment (inside edge)

= + 900 x 1.296 = + 1165"#

Average Bending Moment at Section B-B = 1122"#

<u>Section C-C</u> (Gauges 9, 10, and 11)

Axial strain = - 7.0 μins.

Bending Moment strain at outside edge of column

= - 146 - (- 7) = - 139 μins.

Bending Moment stress at outside edge of column

= - 139 x 30 = - 4170 p.s.i. (Compression)

Bending Moment strain at inside edge of column

= + 49 - (- 7) = + 56 μins.

Bending Moment stress at inside edge of column

= + 56 x 30 = + 1680 p.s.i. (Tension)

Bending Moment (outside edge)

= - 4170 x 0.479 = - 2000"#

Bending Moment (inside edge)

= + 1680 x 1.296 = + 2174"#

Average Bending Moment at Section C-C = 2087"#

<u>Section D-D</u> (Gauges 12, 13, and 14)

Axial strain = - 31.0μins.

Bending Moment strain at outside edge of column

= + 65 - (- 31) = + 96μins.

Bending Moment stress at outside edge of column

= + 96 x 30 = + 2880 p.s.i. (Tension)

Bending Moment strain at inside edge of column

= - 67 - (- 31) = - 36μins.

Bending Moment stress at inside edge of column

= - 36 x 30 = - 1080 p.s.i. (Compression)

Bending Moment (outside edge)

= + 2880 x 0.479 = + 1380"#

Bending Moment (inside edge)

= - 1080 x 1.296 = - 1400"#

Average Bending Moment at Section D-D = 1390"#

<u>Section E-E</u> (Gauges 15, 16, and 17)

Axial strain = - 30.0μins.

Bending Moment strain at outside edge of column

= + 5 - (- 30) = + 35μins.

Bending Moment stress at outside edge of column

= + 35 x 30 = + 1050 p.s.i. (Tension)

Bending Moment strain at inside edge of column

= - 44 - (- 30) = - 14μins.

Bending Moment stress at inside edge of column

= - 14 x 30 = - 420 p.s.i. (Compression)

Bending Moment (outside edge) = + 1050 x 0.479 = + 503"#

Bending Moment (inside edge) = - 420 x 1.296 = - 543"#

Average Bending Moment at Section E-E = 523"#

<u>Section F-F</u> (Gauges 18, 19, and 20)

Axial strain = - 35.0 μ/ins.

Bending Moment strain at outside edge of column

= - 77 - (- 35) = - 42 μ/ins.

Bending Moment stress at outside edge of column

= - 42 x 30 = - 1260 p.s.i. (Compression)

Bending Moment strain at inside edge of column

= - 22 - (- 35) = + 13 μ/ins.

Bending Moment stress at inside edge of column

= + 13 x 30 = + 390 p.s.i. (Tension)

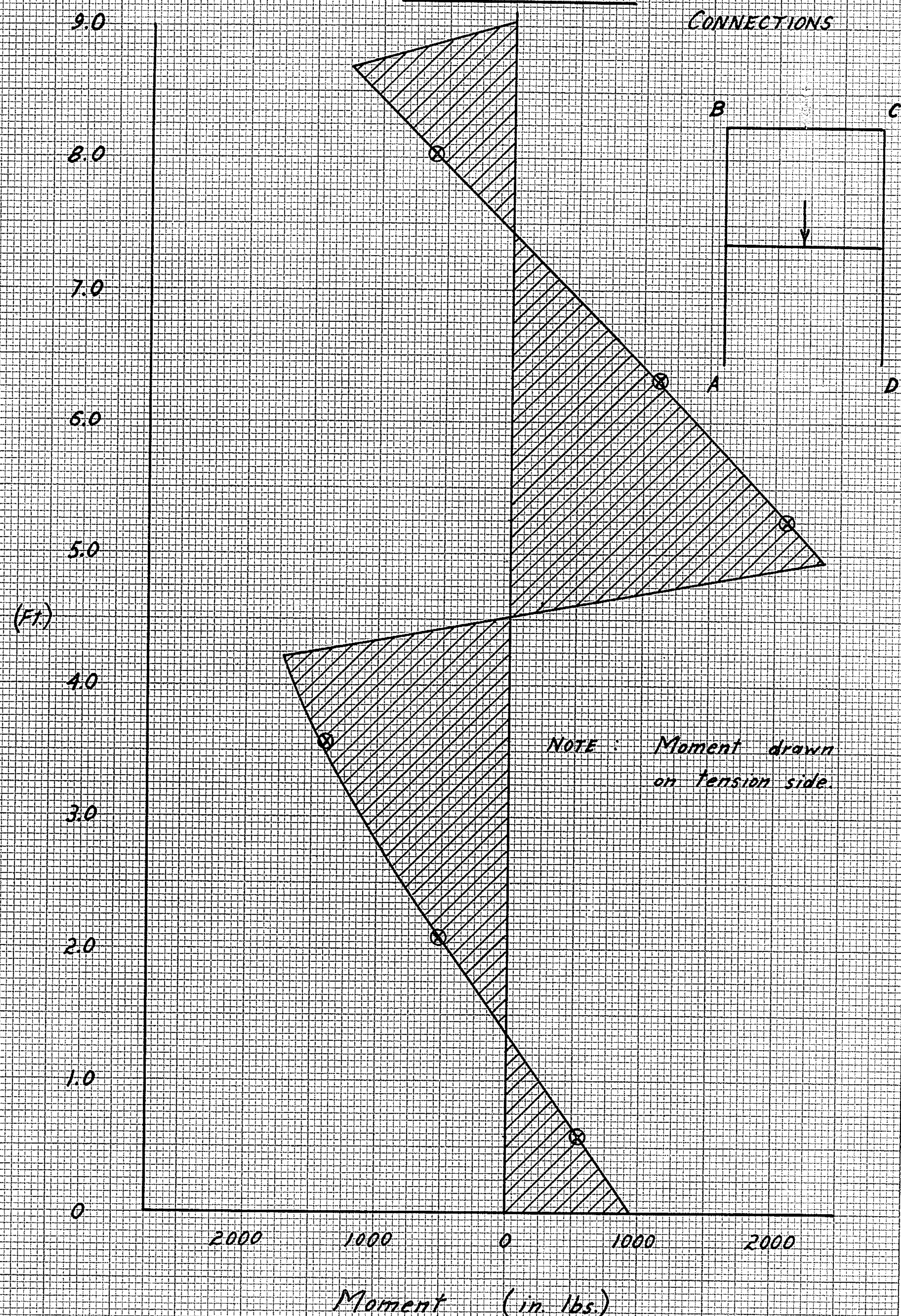
Bending Moment (outside edge) = - 1260 x 0.479 = - 603"#

Bending Moment (inside edge) = + 390 x 1.296 = + 505"#

Average Bending Moment at Section F-F = 554"#

TEST NO. 2. BENDING MOMENT DIAGRAM FOR COLUMN AB

GUSSET CONNECTIONS



NOTE: Moment drawn on tension side.

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TEST NO. 3.

For this test the seated beam connections were used, and a load was centrally applied to the upper beam of the Test Frame.

The same procedure of loading, taking gauge readings, and calculating the bending moments, as used in Test No. 1 was followed.

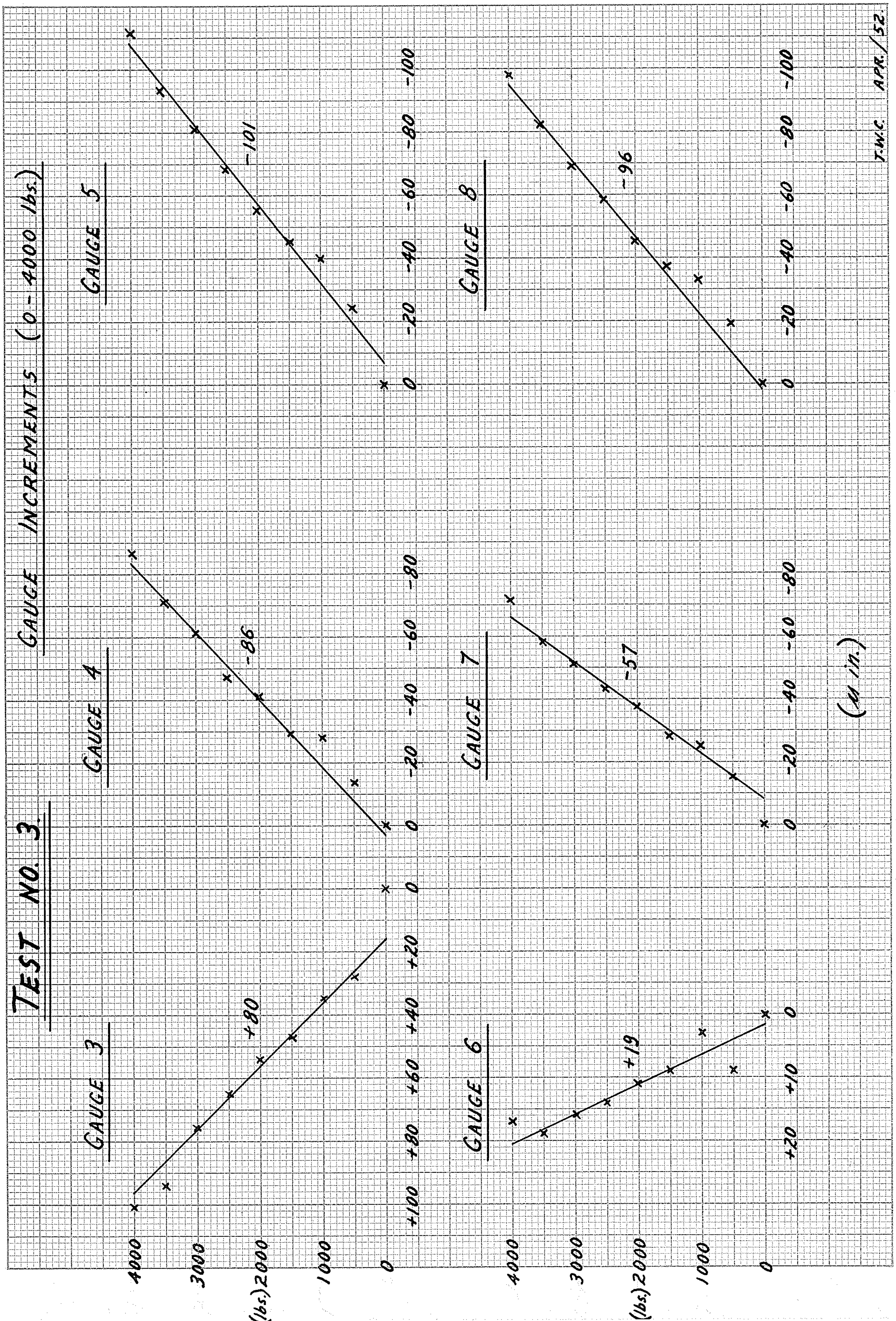
A bending moment diagram for the column was again drawn as shown on page 74. In addition, a bending moment diagram was drawn, combining the bending moments from Tests No. 1 and No. 3 and is shown on page 75. This considers the possibility of loading both beams simultaneously, using the seated beam connections.

Test No. 3.

Load (lbs.)	Gauge								
	3	4	5	6	7	8	9	10	11
0	6002	6137	5973	5989	5998	5985	6012	6021	5977
500	6030	6124	5949	5998	5983	5966	6005	6014	5962
1000	6037	6109	5933	5992	5973	5952	6000	6005	5949
1500	6049	6108	5928	5998	5970	5948	6000	6002	5945
2000	6056	6096	5918	6000	5960	5940	5998	5998	5941
2500	6067	6090	5905	6003	5955	5927	5999	5991	5929
3000	6078	6076	5892	6005	5947	5916	5992	5989	5919
3500	6096	6066	5880	6008	5940	5903	5982	5985	5910
4000	6103	6050	5861	6006	5927	5887	5976	5975	5895

Load (lbs.)	Gauge								
	12	13	14	15	16	17	18	19	20
0	5962	6020	6005	6043	6025	5985	6019	5983	6018
500	5949	6014	5990	6031	6010	5980	6010	5972	6024
1000	5940	6009	5985	6025	6003	5973	6007	5968	6018
1500	5936	6007	5982	6023	5993	5969	6003	5954	6011
2000	5931	6005	5980	6020	5998	5966	6003	5952	6013
2500	5920	6000	5965	6012	5981	5958	5998	5946	6006
3000	5913	5998	5957	6006	5976	5952	5995	5941	6002
3500	5904	5995	5948	5996	5971	5946	5988	5937	6000
4000	5891	5988	5935	5986	5960	5938	5980	5930	5993

Note: All gauge readings are in microinches.


TEST NO. 3
GAUGE INCREMENTS (0-4000 lbs.)
GAUGE 3
+80
GAUGE 4
-86
GAUGE 5
-101
GAUGE 6
+19
GAUGE 7
-57
GAUGE 8
-96
(lbs.)
(μ in.)
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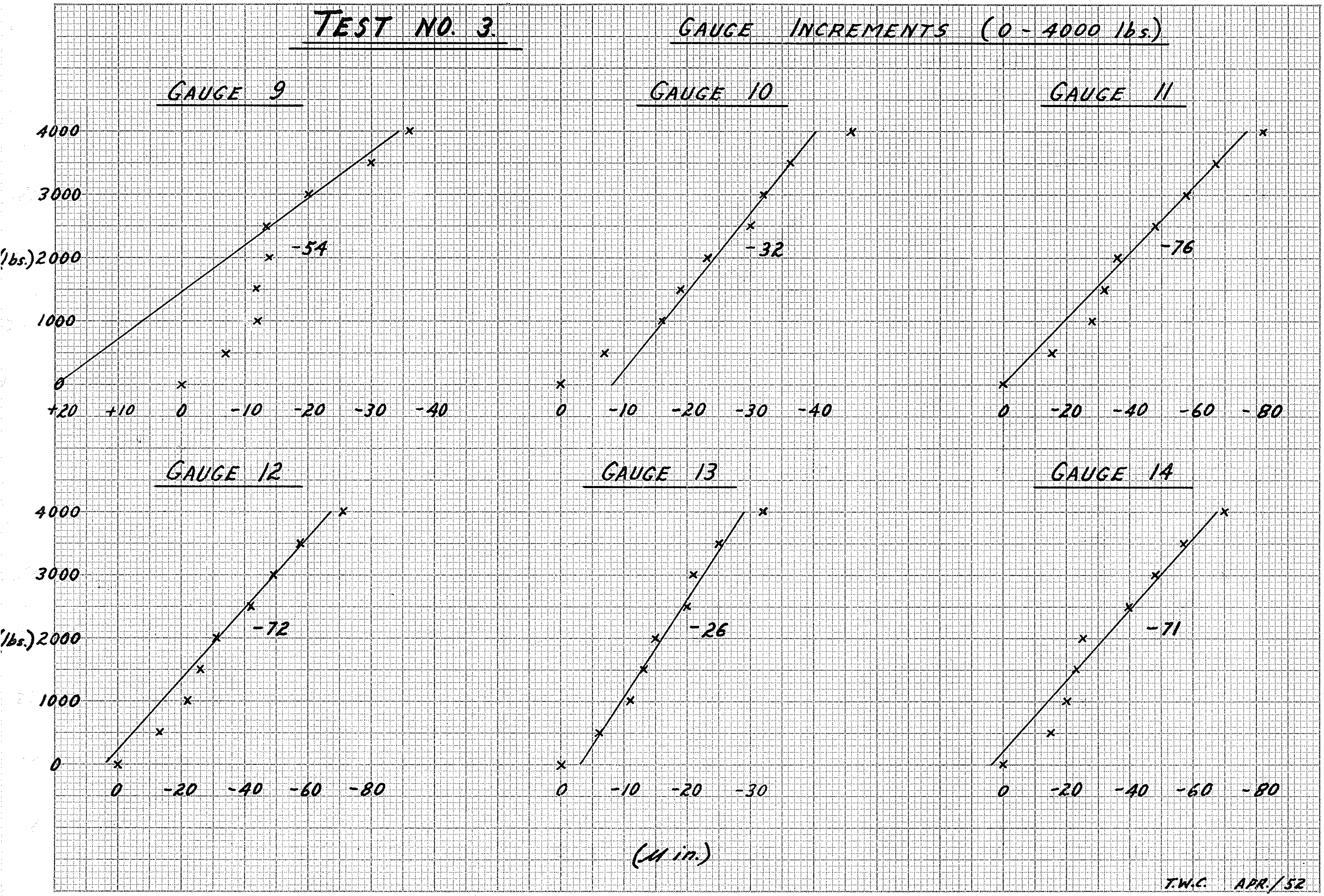
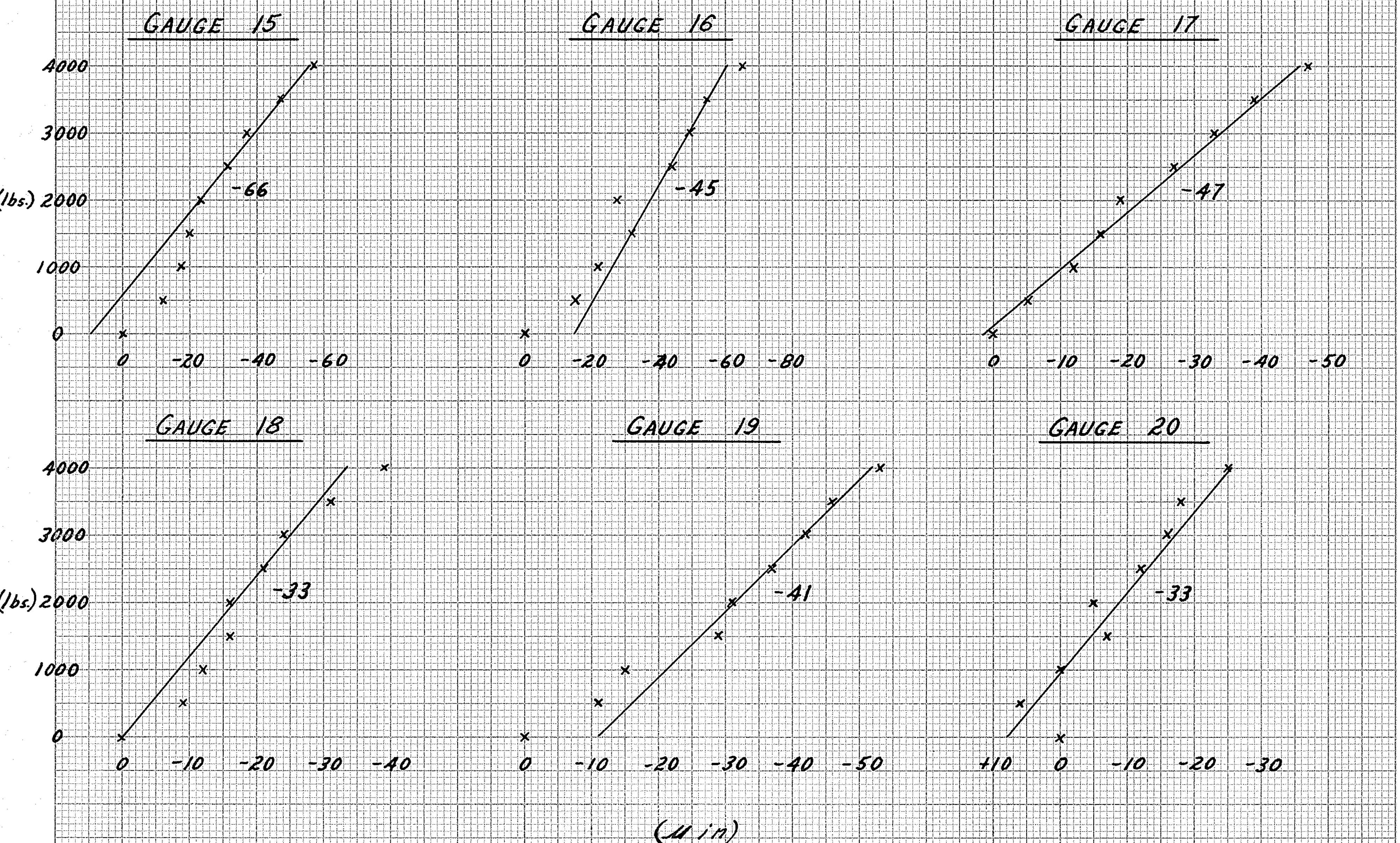
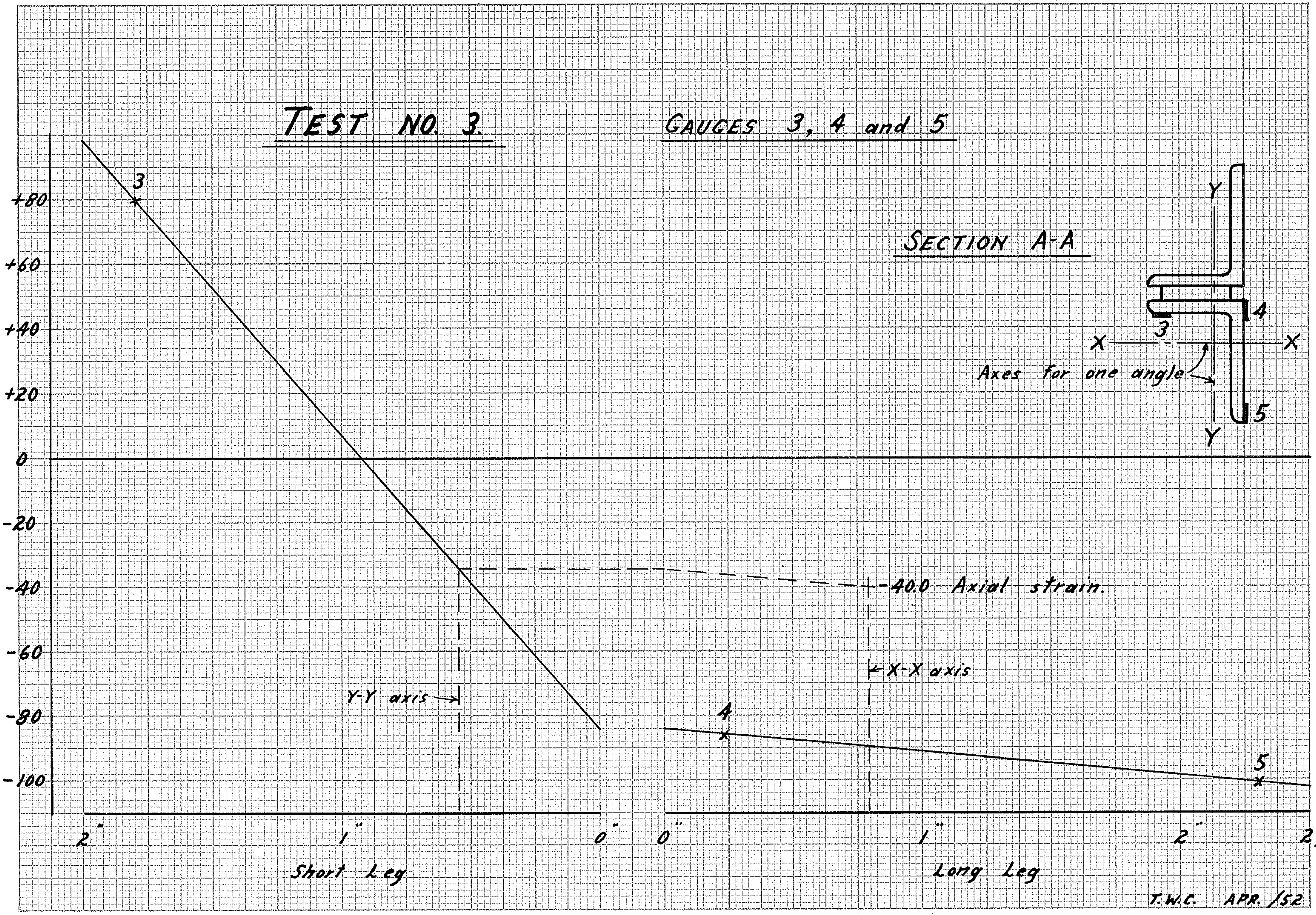

TEST NO. 3.
GAUGE INCREMENTS (0 - 4000 lbs.)
GAUGE 9
-54
GAUGE 10
-32
GAUGE 11
-76
GAUGE 12
-72
GAUGE 13
-26
GAUGE 14
-71
(lbs.)
4000
3000
2000
1000
0
+20 +10 0 -10 -20 -30 -40
0 -10 -20 -30 -40
0 -20 -40 -60 -80
0 -20 -40 -60 -80
0 -10 -20 -30
(μ in.)
T.W.C. APR./52

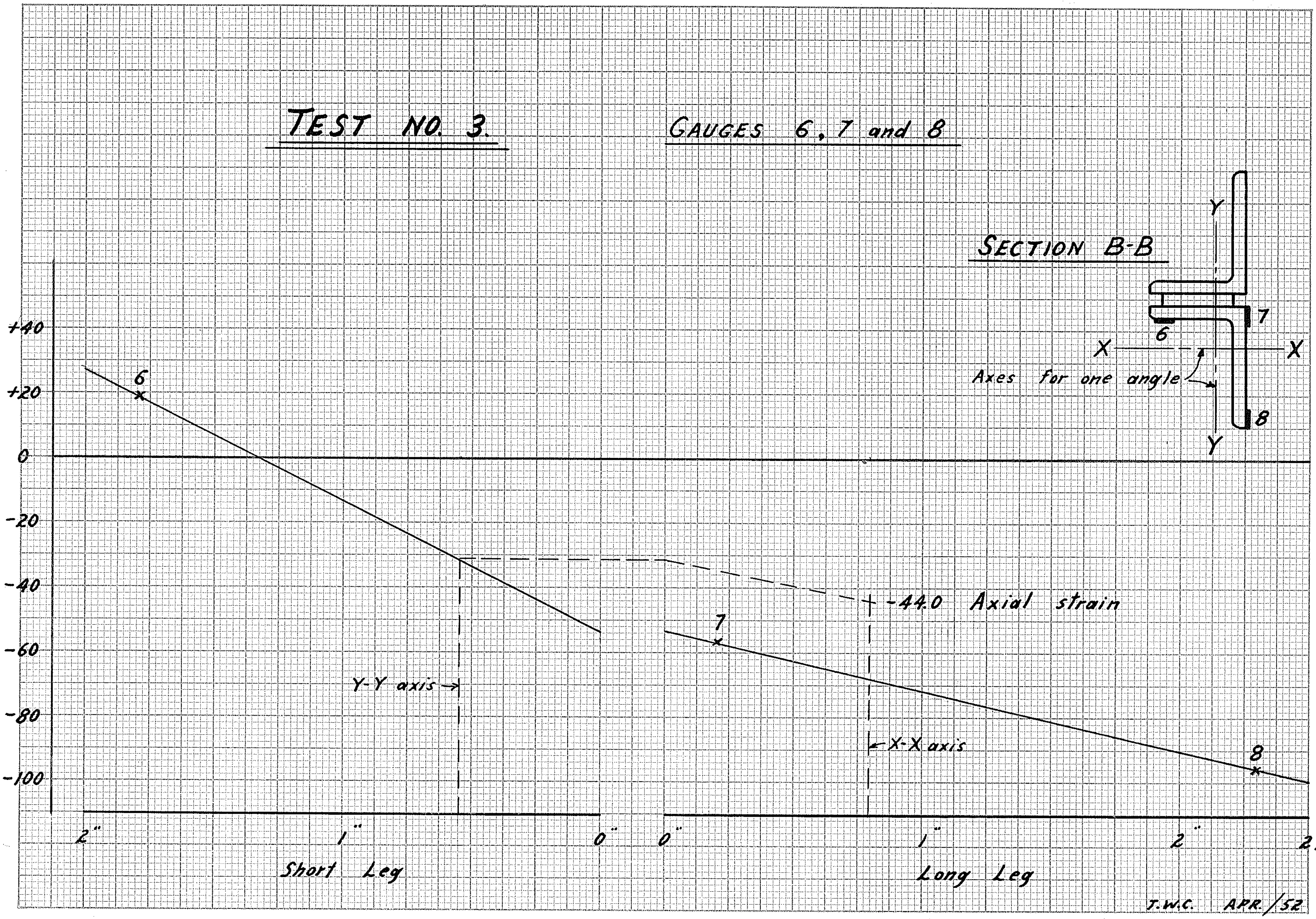
TEST NO. 3.

GAUGE INCREMENTS (0 - 4000 lbs.)



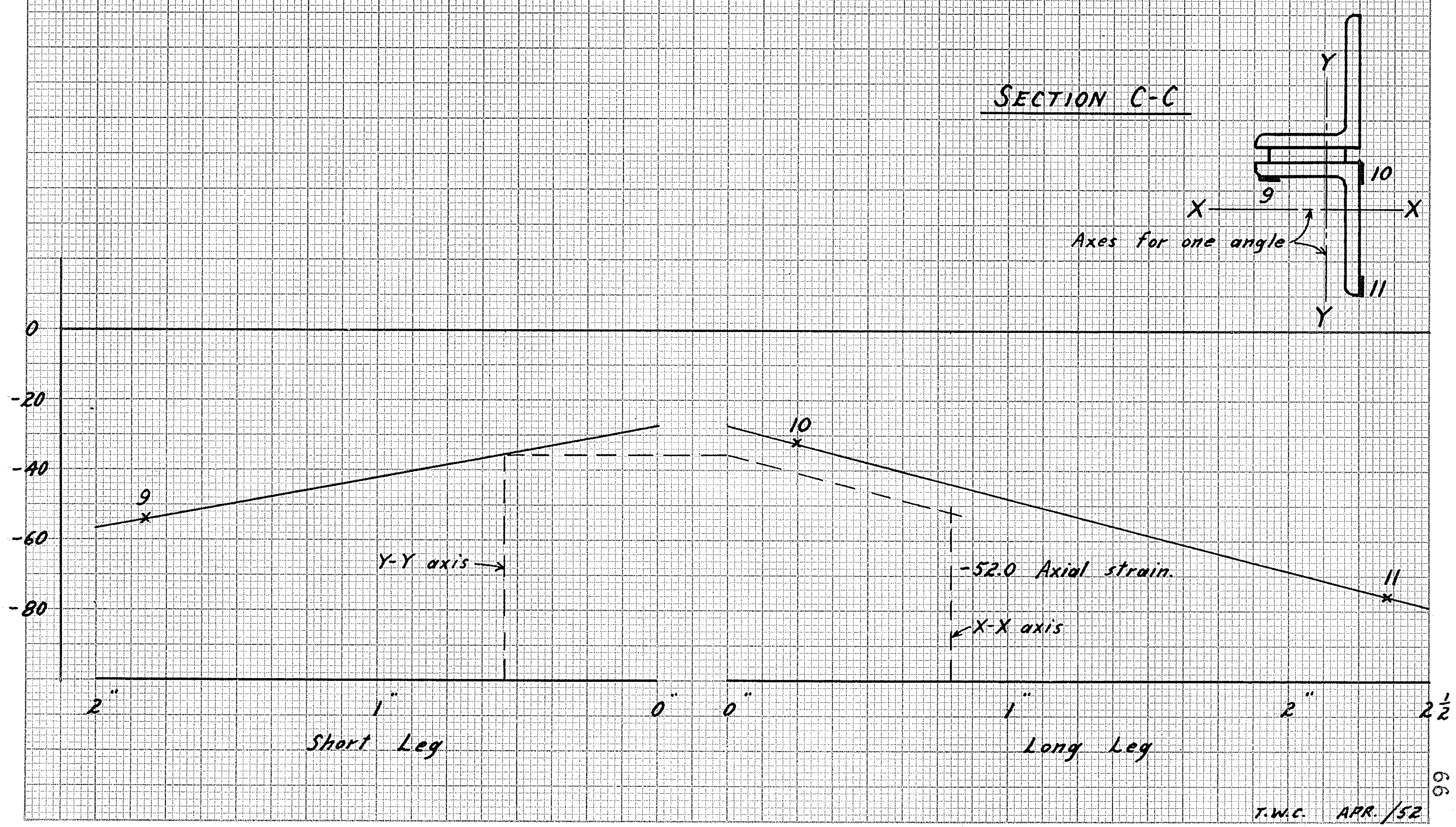
T.W.C. APR./52.


TEST NO. 3.
GAUGES 3, 4 and 5
SECTION A-A
Y
Y
X
X
3
4
5
Axes for one angle
+80
+60
+40
+20
0
-20
-40
-60
-80
-100
3
4
5
-40.0 Axial strain.
←X-X axis
Y-Y axis→
2"
1"
0"
0"
1"
2"
2 1/2"
Short Leg
Long Leg
T.W.C. APR. /52


TEST NO. 3.
GAUGES 6, 7 and 8
SECTION B-B
Y
X
X
Y
6
7
8
Axes for one angle
+40
+20
0
-20
-40
-60
-80
-100
6
7
8
-44.0 Axial strain
Y-Y axis
X-X axis
2"
1"
0"
0"
1"
2"
2 1/2"
Short Leg
Long Leg
T.W.C. APR./52

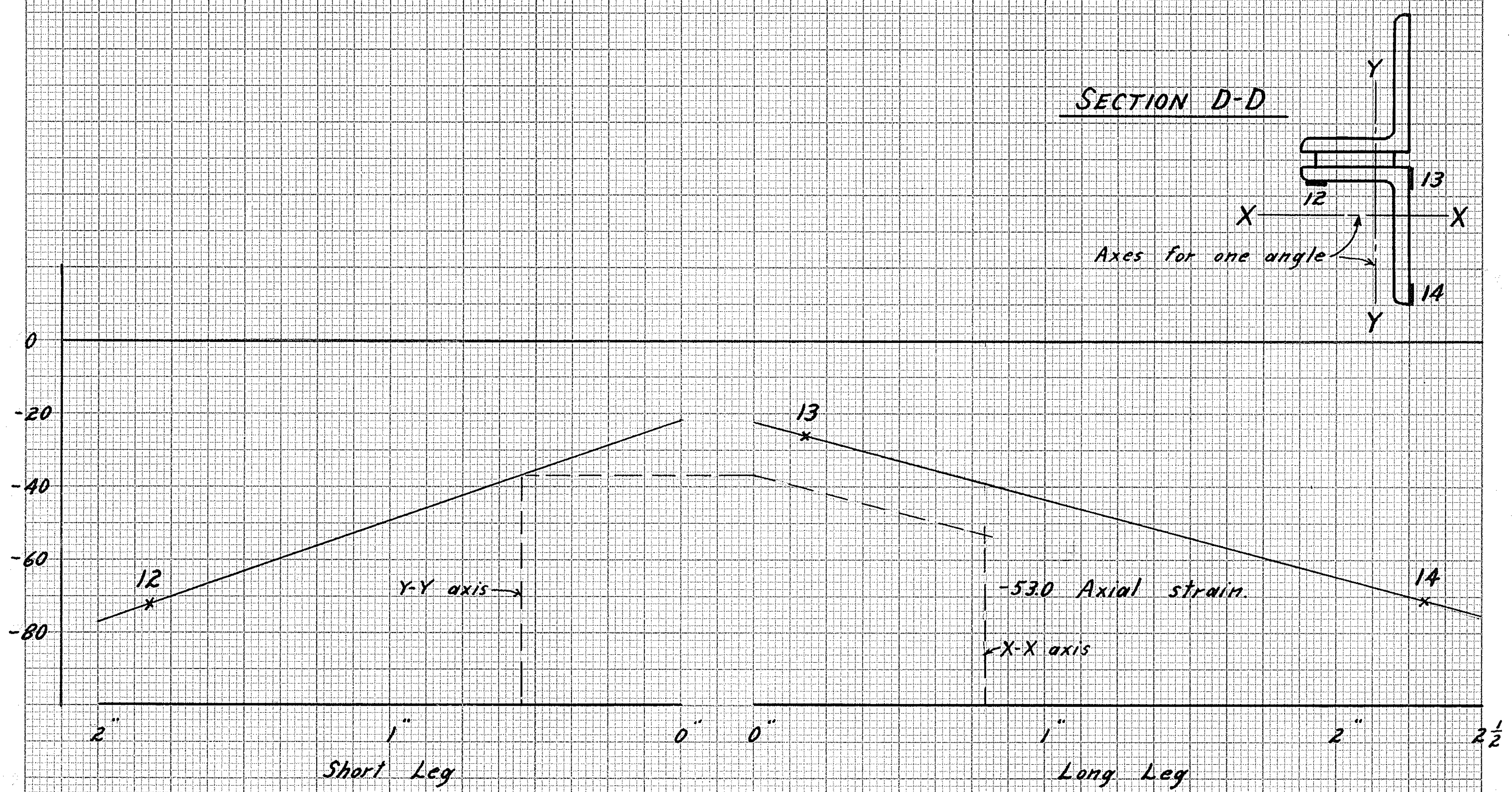
TEST NO. 3.

GAUGES 9, 10 and 11



TEST NO. 3.

GAUGES 12, 13 and 14



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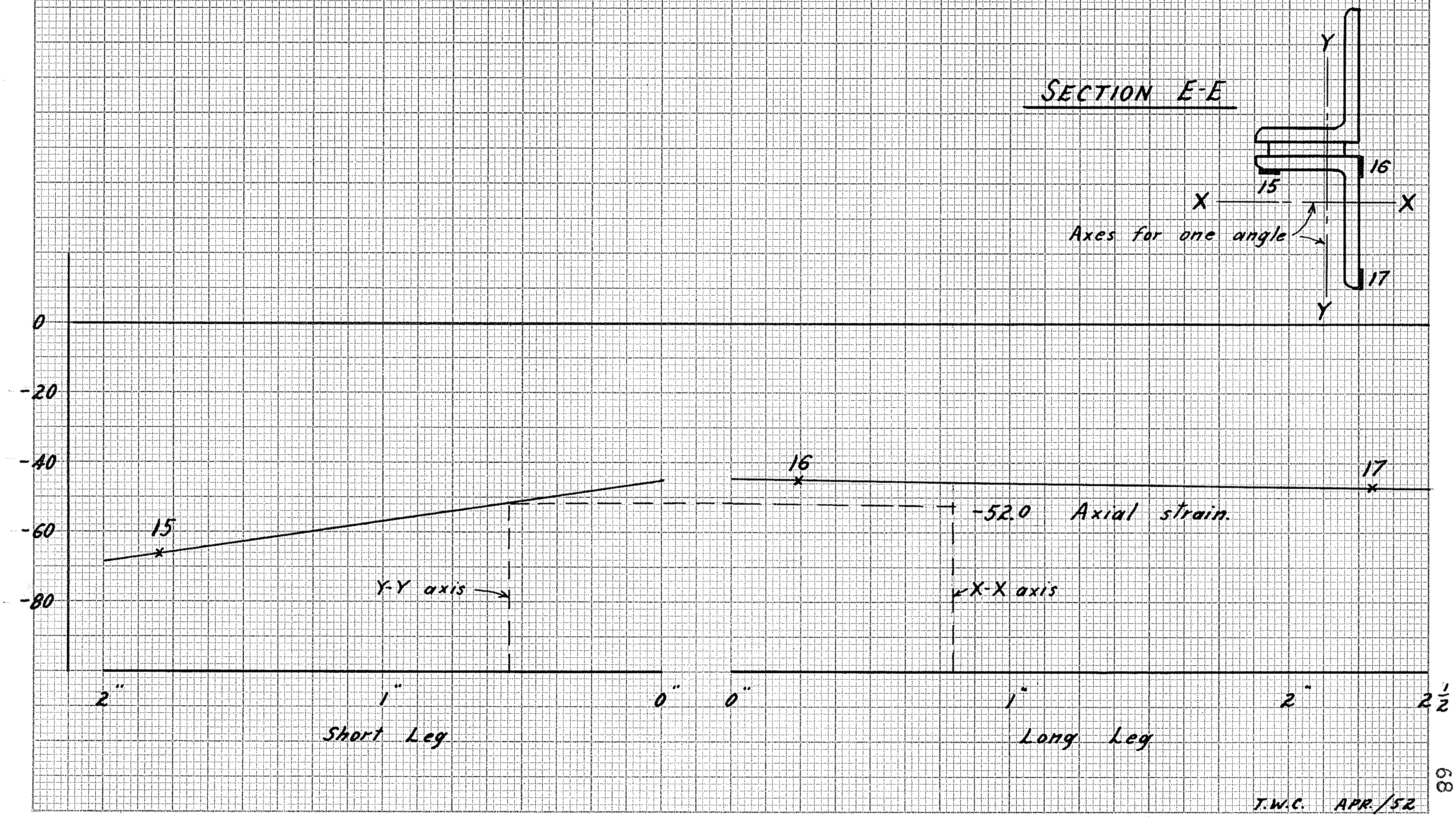
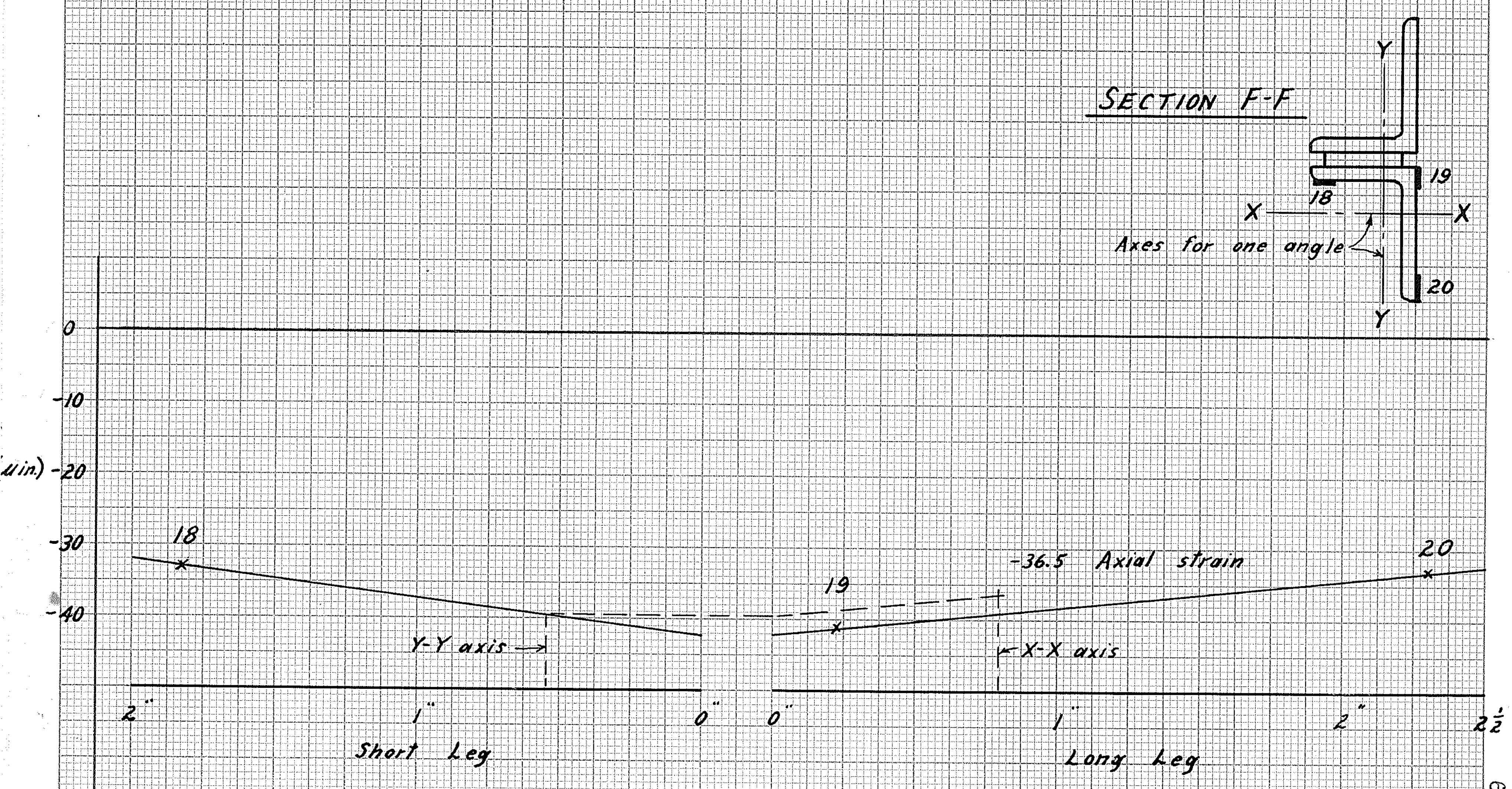
TEST NO. 3.

GAUGES 15, 16 and 17



TEST NO. 3.

GAUGES 18, 19 and 20



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Test No. 3.

Analysis For Bending Moment

<u>Section A-A</u> (Gauges 3, 4, and 5)

Axial strain = - 40 µins.

Bending Moment strain at outside edge of column

= + 98 - (- 40) = + 138 µins.

Bending Moment stress at outside edge of column

= + 138 x 30 = + 4140 p.s.i. (Tension)

Bending Moment strain at inside edge of column

= - 90 - (- 40) = - 50 µins.

Bending Moment stress at inside edge of column

= - 50 x 30 = - 1500 p.s.i. (Compression)

Bending Moment (outside edge)

= + 4140 x 0.479 = + 1983"#

Bending Moment (inside edge)

= - 1500 x 1.296 = - 1942"#

Average Bending Moment at Section A-A = 1962"#

<u>Section B-B</u> (Gauges 6, 7, and 8)

Axial strain = - 44 µins.

Bending Moment strain at outside edge of column

= + 27 - (- 44) = + 71 µins.

Bending Moment stress at outside edge of column

= + 71 x 30 = + 2130 p.s.i. (Tension)

Bending Moment strain at inside edge of column

= - 69 - (- 44) = - 25 μins.

Bending Moment stress at inside edge of column

= - 25 x 30 = - 750 p.s.i. (Compression)

Bending Moment (outside edge)

= + 2130 x 0.479 = + 1022"#

Bending Moment (inside edge) = - 750 x 1.296 = - 972"#

Average Bending Moment at Section B-B = 997"#

<u>Section C-C</u> (Gauges 9, 10, and 11)

Axial strain = - 52 μins.

Bending Moment strain at outside edge of column

= - 57 - (- 52) = - 5 μins.

Bending Moment stress at outside edge of column

= - 5 x 30 = - 150 p.s.i. (Compression)

Bending Moment strain at inside edge of column

= - 44 - (- 52) = + 8 μins.

Bending Moment stress at inside edge of column

= + 8 x 30 = + 240 p.s.i. (Tension)

Bending Moment (outside edge) = - 150 x 0.479 = - 72"#

Bending Moment (inside edge) = + 240 x 1.296 = + 311"#

Average Bending Moment at Section C-C = 191"#

<u>Section D-D</u> (Gauges 12, 13, and 14)

Axial strain = - 53 μins.

Bending Moment strain at outside edge of column

= - 78 - (- 53) = - 25 μins.

Bending Moment stress at outside edge of column

= - 25 x 30 = - 750 p.s.i. (Compression)

Bending Moment strain at inside edge of column

= - 40 - (- 53) = + 13 μins.

Bending Moment stress at inside edge of column

= + 13 x 30 = + 390 p.s.i. (Tension)

Bending Moment (outside edge) = - 750 x 0.479 = - 359"#

Bending Moment (inside edge) = + 390 x 1.296 = + 505"#

Average Bending Moment at Section D-D = 432"#

<u>Section E-E</u> (Gauges 15, 16, and 17)

Axial strain = - 52 μins.

Bending Moment strain at outside edge of column

= - 68 - (- 52) = - 16 μins.

Bending Moment stress at outside edge of column

= - 16 x 30 = - 480 p.s.i. (Compression)

Bending Moment strain at inside edge of column

= - 46 - (- 52) = + 6 μins.

Bending Moment stress at inside edge of column

= + 6 x 30 = + 180 p.s.i. (Tension)

Bending Moment (outside edge) = - 480 x 0.479 = - 230"#

Bending Moment (inside edge) = + 180 x 1.296 = + 233"#

Average Bending Moment at Section E-E = 232"#

<u>Section F-F</u> (Gauges 18, 19, and 20)

Axial strain = - 36.5 μins.

Bending Moment strain at outside edge of column

= + 32 - (- 36.5) = + 4.5 μins.

Bending Moment stress at outside edge of column

= + 4.5 x 30 = + 135 p.s.i. (Tension)

Bending Moment strain at inside edge of column

= - 39 - (- 36.5) = - 2.5 μins.

Bending Moment stress at inside edge of column

= - 2.5 x 30 = - 75 p.s.i. (Compression)

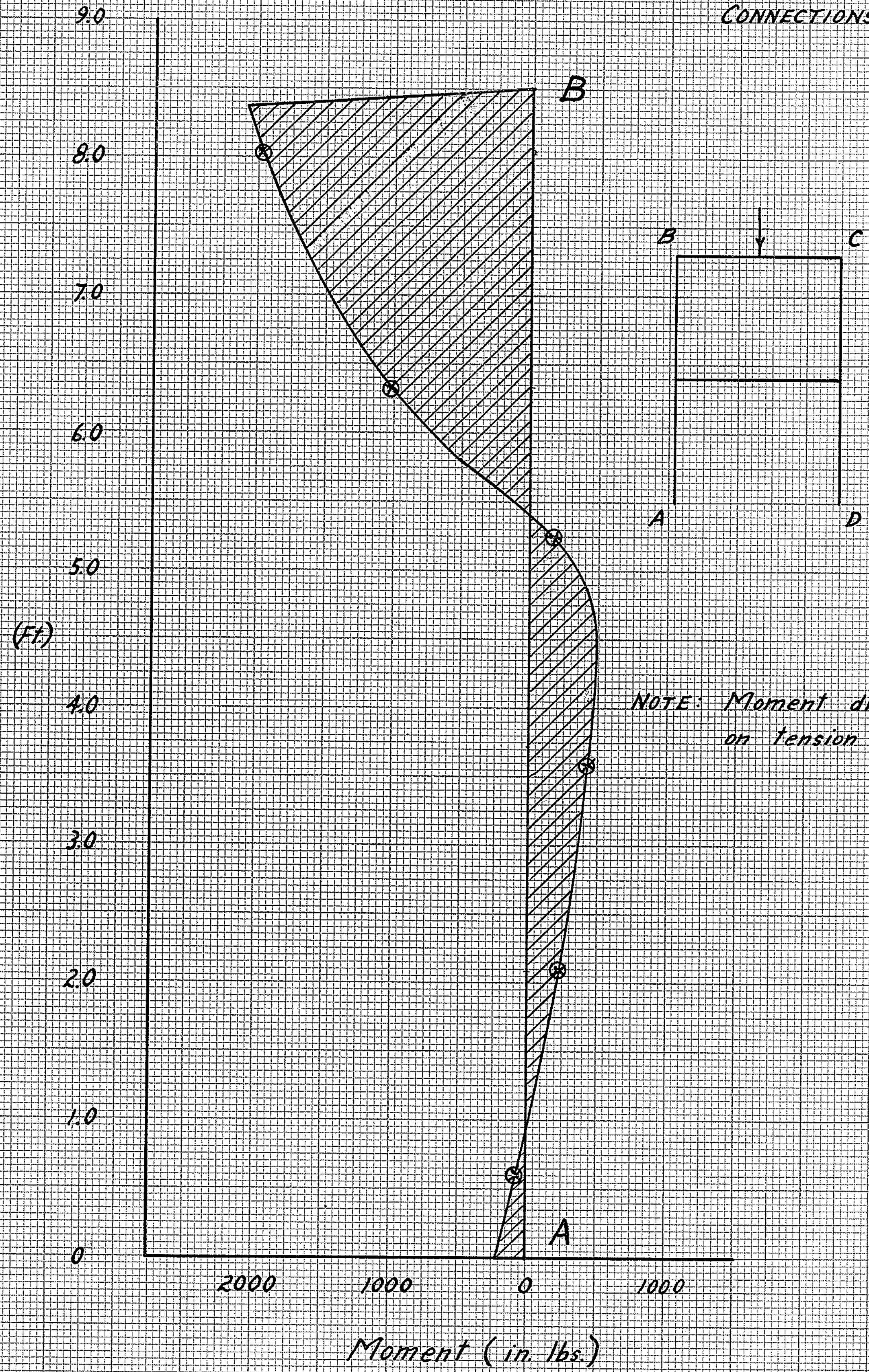
Bending Moment (outside edge) = + 135 x 0.479 = + 65"#

Bending Moment (inside edge) = - 75 x 1.296 = - 97"#

Average Bending Moment at Section F-F = 81"#

TEST NO. 3. BENDING MOMENT DIAGRAM FOR COLUMN AB

SEATED BEAM CONNECTIONS



NOTE: Moment drawn on tension side.

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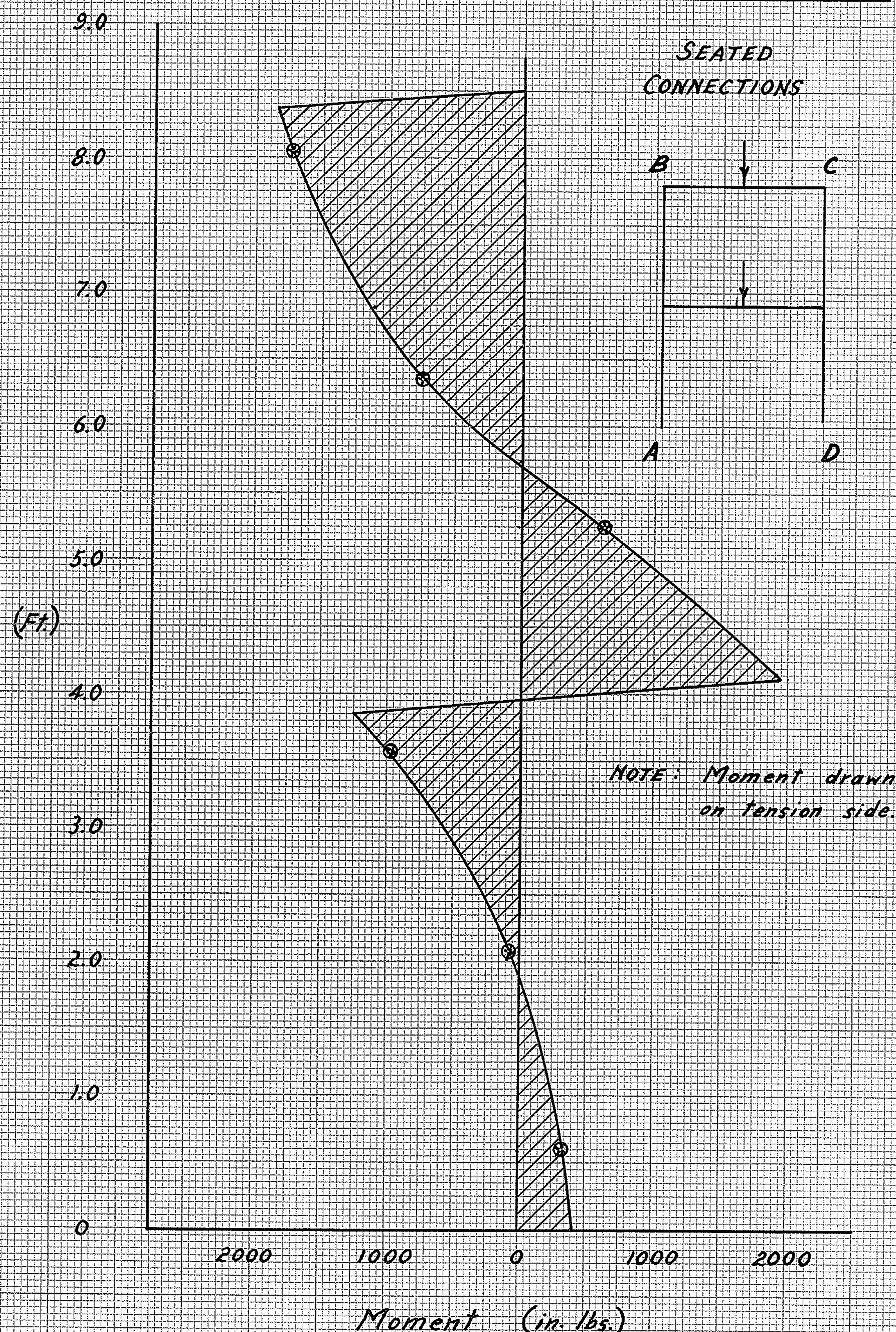
BENDING MOMENT DIAGRAM FOR COLUMN AB

COMBINING BENDING MOMENTS FROM TESTS NO. 1 AND NO. 3.



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DISCUSSION OF RESULTS.

A comparison of the column bending moment diagrams for Test No. 1 and Test No. 2 reveals that the gusset connection was more rigid than the seated beam connection since the moment produced in the column was much larger for the gussets. The total moment produced in the column through the seated beam connection was 2550 in. lbs. whereas for the gusset connection the total moment was 4050 in. lbs.

In the design of the seat angle, the eccentricity of the load from the column was calculated to be 1.667 inches. To the centre line of the column the total eccentricity would be $1.667 + 0.54 = 2.207$ inches. With a central load of 4 kips on the beam, the apparent moment produced in the column is Pe or $2000 \times 2.207 = 4414$ in. lbs. However, the bending moment diagram for Test No. 1 shows the total moment being taken in both tension and compression in the column, and the maximum moment to be designed for is 1650 in. lbs. This is less than one-half of the apparent moment. Although the experimental eccentricity is not known exactly, it appears the eccentricity used in design is much too large.

In the design of the gusset connection, the eccentricity was taken from the centre line of the column to the centre of the rivet group on the gusset, and was 4.5 inches. In this case, again using a central load of 4 kips on the beam, the apparent moment produced in the column is 2000 x 4.5 = 9000 in. lbs. The bending moment diagram for Test No. 2 shows the maximum moment to be designed for is 2350 in. lbs. This is close to one-quarter of the apparent moment, again showing the moment considered in design work as being much too high.

From the first two tests it seems safe to say that in designing columns, the load to consider would be the axial load plus the equivalent concentric load for one-half of the apparent moment.

The bending moment diagrams for Tests No. 1 and No. 2 follow the same general pattern except that the seated beam connection did not transfer much moment into the beam of the storey above the one where the load was applied. The gusset connection transfered about one-half of the maximum moment into the beam of the storey above the loaded storey. As the base of the column was bolted down in both cases, the moment in the column at the base, is proportional to the rest of the two moment diagrams. The point of contraflexure in the lower storey of the column agrees closely for both types of beam connections. For the gusset connections, the bending moments have almost a straight line relationship with the storey height.

For Test No. 3 the bending moment diagram shows the maximum moment to be at the point of application, and has a value of 2100 in. lbs. Even though the moment is taken in tension on only one side of the column at this point, the maximum moment is still less than one-half of the apparent moment of 4414 in. lbs. The moment at the storey below is one-quarter of the maximum, and two storeys below it is one-eighth of the maximum. At the same rate of reduction the moment could be neglected at about three storeys away from the point of application.

On page 75 a bending moment diagram is shown, combining the bending moments from Tests No. 1 and No. 3. This takes into account the possibility of having both beams loaded simultaneously. This combination of loading tends to bring the points of contraflexure nearer to the mid-height of the storeys. Thus the moment is the least at the point where the column action is generally considered the greatest.

For comparison purposes the moment distribution in the frame, considering all connections rigid, was calculated. The distribution and the resulting graph for a 4000 lb. load centrally applied to the lower beam are as follows:

Columns $I_{x-x} = 0.7''^4$

Beams $I_{x-x} = 26.0''^4$

Column stiffness factor(top storey)

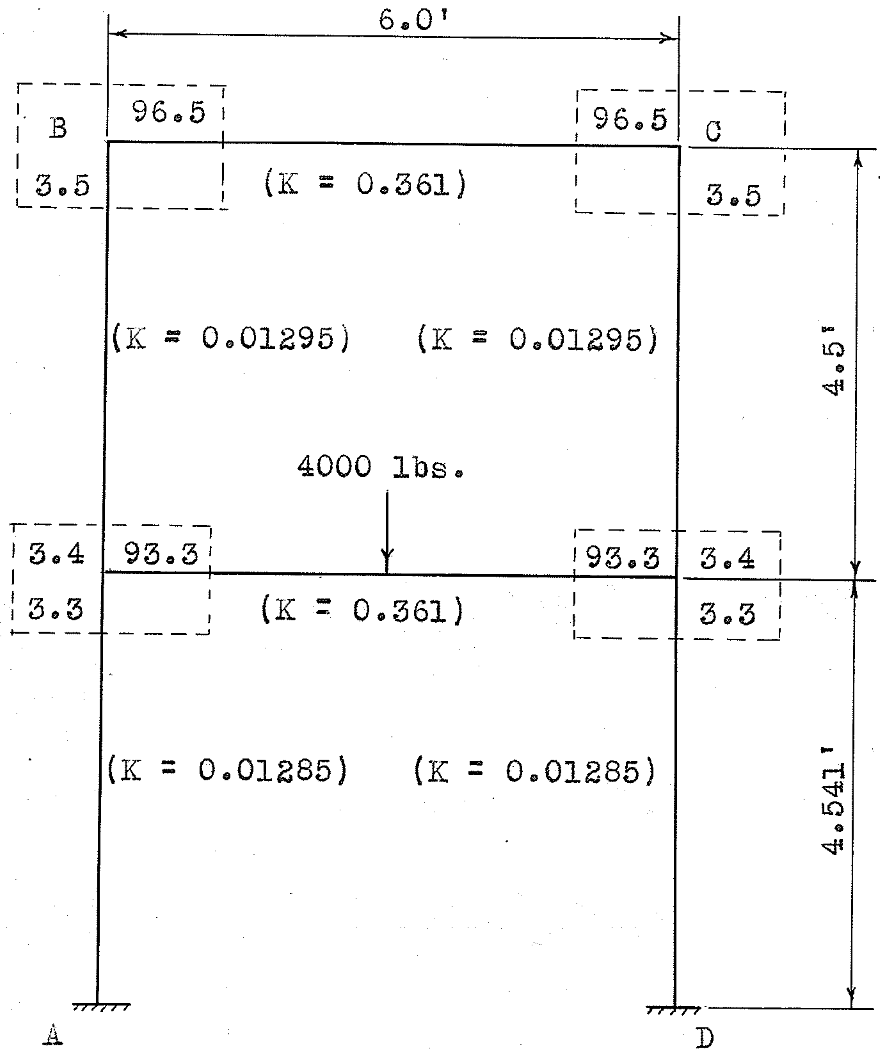
$$K = \frac{I}{L} = \frac{0.7}{4.5 \times 12} = 0.01295$$

Column stiffness (bottom storey)

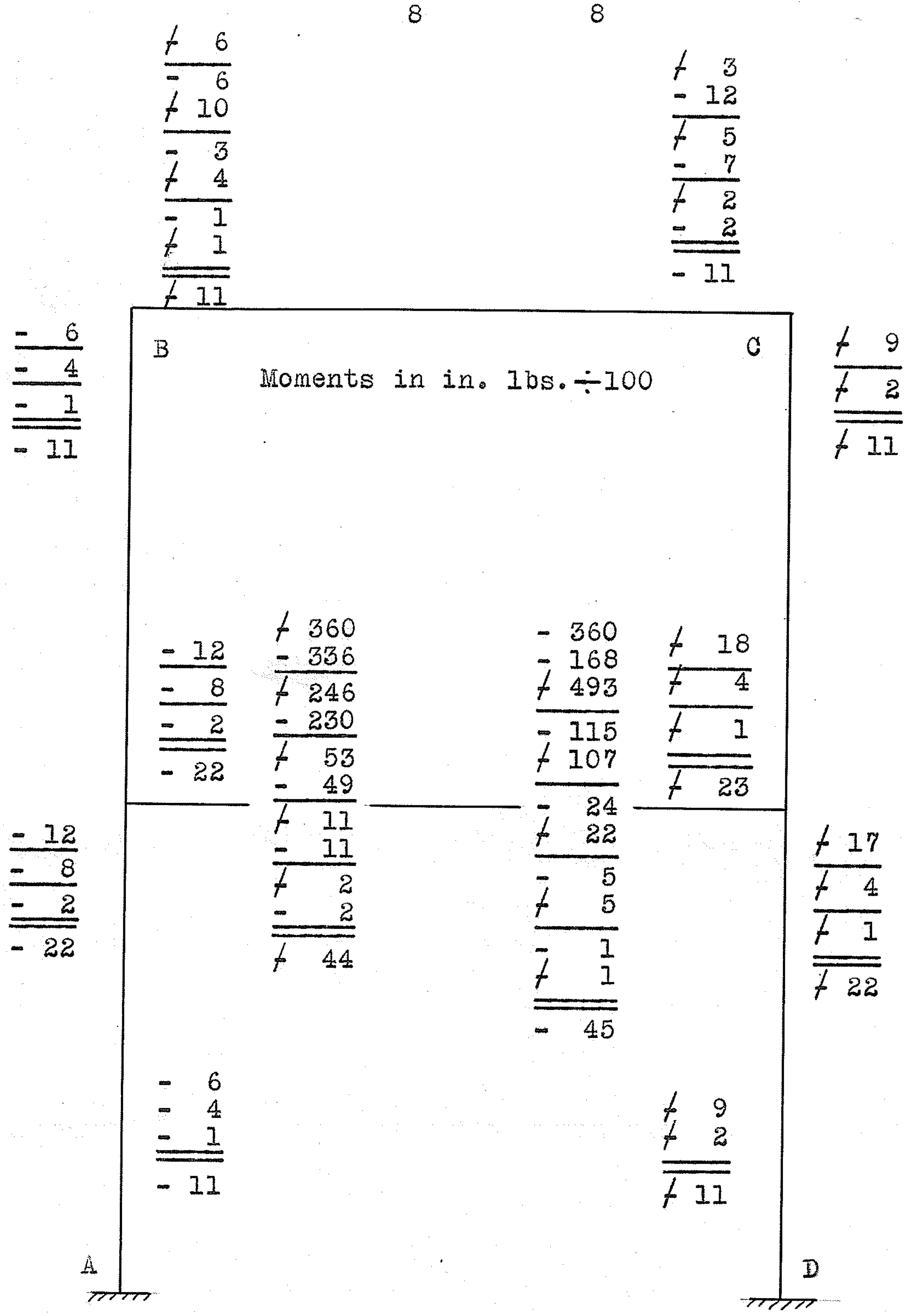
$$K = \frac{I}{L} = \frac{0.7}{4.541 \times 12} = 0.01285$$

Beam stiffness

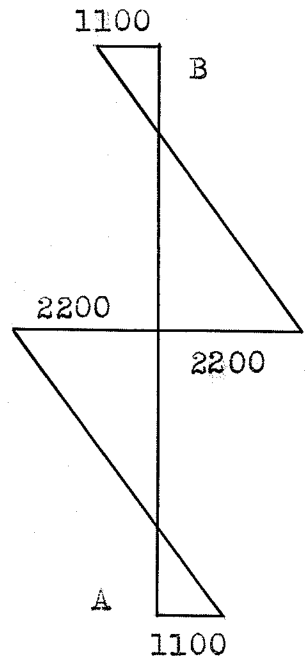
$$K = \frac{26.0}{6 \times 12} = 0.361$$



Fixed end moment $= \frac{P\,L}{8} = \frac{4000 \times 6 \times 12}{8} = 36{,}000''\#$



It is interesting to note that the accompanying bending moment diagram , which was plotted from values obtained by the moment distribution method, agrees very closely with the moment diagram for the gusset connections in Test No. 2.



Note: All moments in in. lbs. and drawn on the tension side.

It may be suggested that if further research is carried out on a similar frame in the future, the following points may be considered.

For the seated beam connection, a roller could be placed under the beam so that the point of load application, and therefore the exact eccentricity of the load might be known. For the gusset connection, one bolt could be used in the connection to achieve the same purpose.

In order to study the effect of eccentric loading on interior columns, a two panel frame could be used and the loads and their points of application varied, as would be the case in unequal floor loadings.

In conclusion, two statements appearing in the discussion may be repeated.

1. From the information collected, it seems safe to say that in designing columns, the load to consider would be the axial load plus the equivalent concentric load for one-half of the apparent moment.

2. As the points of contraflexure are not far from the mid-height of the storeys, it may be said that the bending moment is least at the point where the column action is generally considered the greatest.