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Welded interior beam-column connections, Interim report, June 1955

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WELDED INTERIOR BEAM-COLUMN CONNECTIONS

Interim ~~Progress~~ Report 1

by

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Always make it a package: short text description.

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I. INTRODUCTION

The purpose of this research project, of which this is the first progress report, is to study restraining beam-column connections of both the direct-welded type and the type in which the beam is mounted on a seat angle or bracket and the top of the beam is secured to the column by means of a top plate. This project is an extension of a number of research projects conducted by Bruce Johnston and associates and of a project by Brandes and Mains, "Report of Tests of Top-Plate and Seat Building Connections" (Welding Journal, March, 1944).

The previous researches on restraining beam-column connections have not been carried to the point where definite conclusions, suitable for the designer, could be reached. In particular, information is lacking on the effect of restraining connections on column capacity, and also as to whether or not column stiffening is required and, when needed, how to design it. Information is also lacking concerning the designer's ability to estimate the moment-rotation curve, the degree of restraint, and the reserve strength of a designed assembly. Phase I of the program dealing with two-way connections, is designed to obtain information on these items. Heretofore, tests of beam-column connections have usually disregarded the axial column loads; in the present program the column is additionally subjected to an axial compression comparable to that existing in practice; further, at one stage of the tests the column axial load is raised to 1.67 times the "so-called" axial load while simultaneously carrying the two beam working load reactions and moments.

Examination is needed of other factors such as the effect of wind moments and the behavior of a four-way connection at a column. The effect of beams framing to a column from two or three sides such as takes place at a corner or side column was partially treated in the Brandes-Mains paper but not in sufficient detail to be useful to the designer. Development of a specific program to solve the above problems is somewhat dependent on the findings of Phase I and, therefore the test program presented here is limited to Phase I.

II. PHASE I TEST PROGRAM

Design, preparation, and testing of specimens similar to Fig. 1 for the purpose of determining the behavior and stress distribution in the columns. Attention is limited primarily to the study of what is considered to be the most important practical problem: Column stiffening requirements.

Series A. All beams to be 16 WF 36 and to be direct-welded to various sized columns as tabulated below. Note that no column stiffeners are used.

Specimen	A-1	Column	to	be	8WF31
"	A-2	"	"	"	8WF67
"	A-3	"	"	"	12WF40
"	A-4	"	"	"	12WF65
"	A-5	"	"	"	12WF99

Series B. Same as Series A except for the inclusion of column stiffeners as shown in Figure 3. The column stiffeners will have approximately the same thickness as the beam flange.

Specimen	B-6	Column to be 8WF31
"	B-7	" " " 8WF67
"	B-8	" " " 12WF40

Series C. Same beams and columns as in Series B, but the column stiffeners are to be placed parallel to the column web as in Figure 4.

Specimen	C-9	Column to be 8WF31
"	C-10	" " " 8WF67
"	C-11	" " " 12WF40

For convenience and to give some necessary stiffener and other details the complete Phase I program of test is given in Table I.

Table I: Program of Restrained Beam Tests - Two-Way Direct-Welded Connections

Series	Test No.	Column Size	Web "w"	Flg. "t"	Beam Size	Web "w"	Flg. "t"	Type	Dimensions	Joint Design
A	1	8WF31	.288	.433	16WF36	.299	.428	None	- -	F.1-A
	2	8WF67	.575	.933	"	"	"	"	- -	"
	3	12WF40	.294	.516	"	"	"	"	- -	"
	4*	12WF65								
	5*	12WF99								
B	6	8WF31	.288	.433	"	"	"	Fig. 1	3.5x7/16	F.1-A
	7**	8WF67	.575	.933	"	"	"	"	"	"
	8	12WF40	.294	.516	"	"	"	"	"	"
C	9	8WF31	.288	.433	"	"	"	Fig.1-B	7.1x5/16x22	F.1-B
	10**	8WF67	.575	.933	"	"	"	"	7.2x1/2x22	"
	11	12WF40	.294	.516	"	"	"	"	10.9x5/16x22	"

* Column weight may be changed dependent on results of previous tests
** May be omitted dependent on results of test of A-2

Loading Sequence

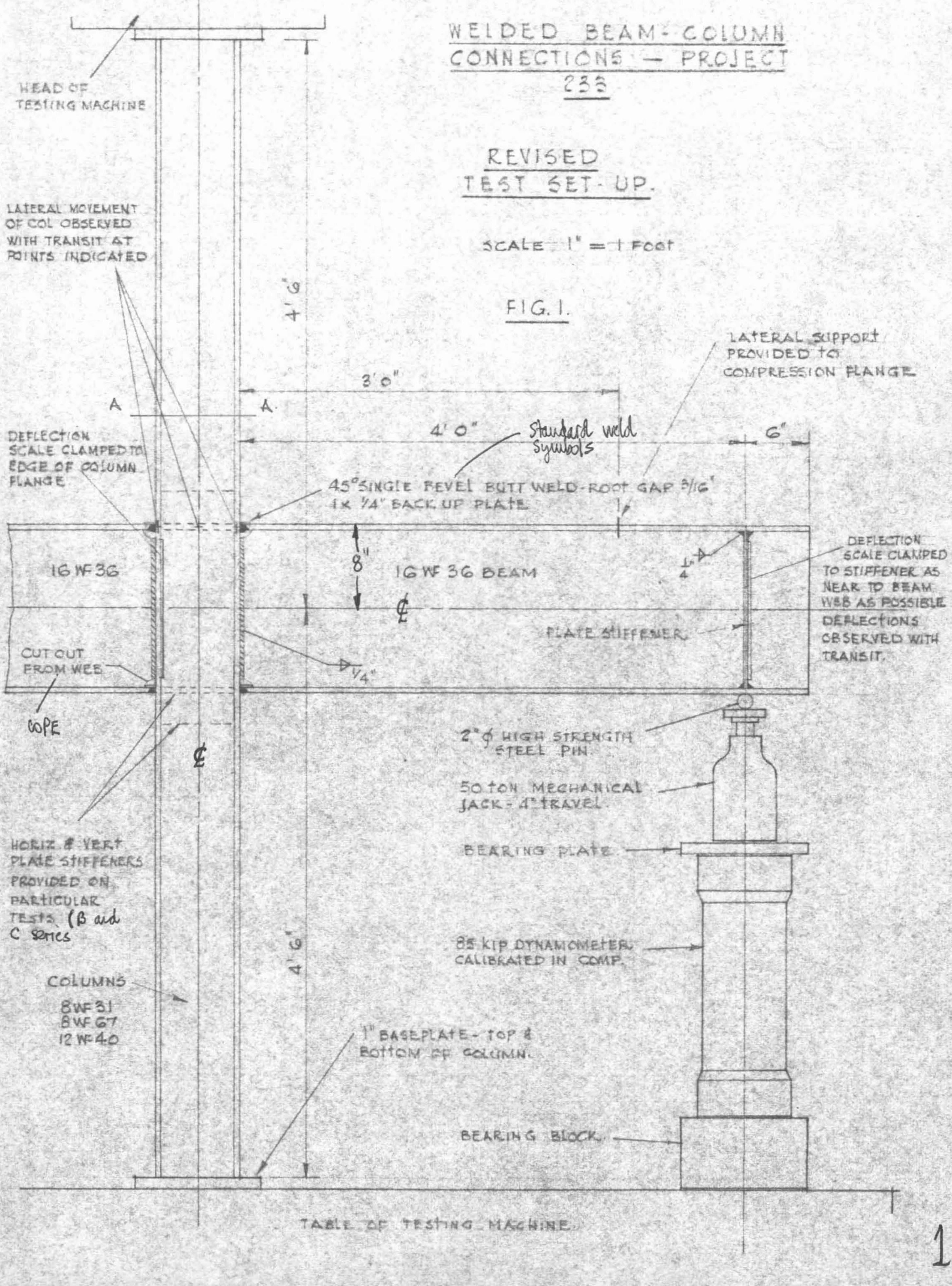
1. Load column by increments up to "working load" in the column (a unit stress in section A-A, Figure 1 of 14.5 ksi).
2. Load beams by increments up to "working load" on the beams, removing load from the column as necessary to maintain a unit stress at section A-A of 14.5 ksi.
3. Holding beam loads constant, raise column load to 1.67 times "working load".
4. Lower column load to working load and load beams by increments to failure.

WELDED BEAM-COLUMN CONNECTIONS — PROJECT 233

REVISED TEST SET-UP.

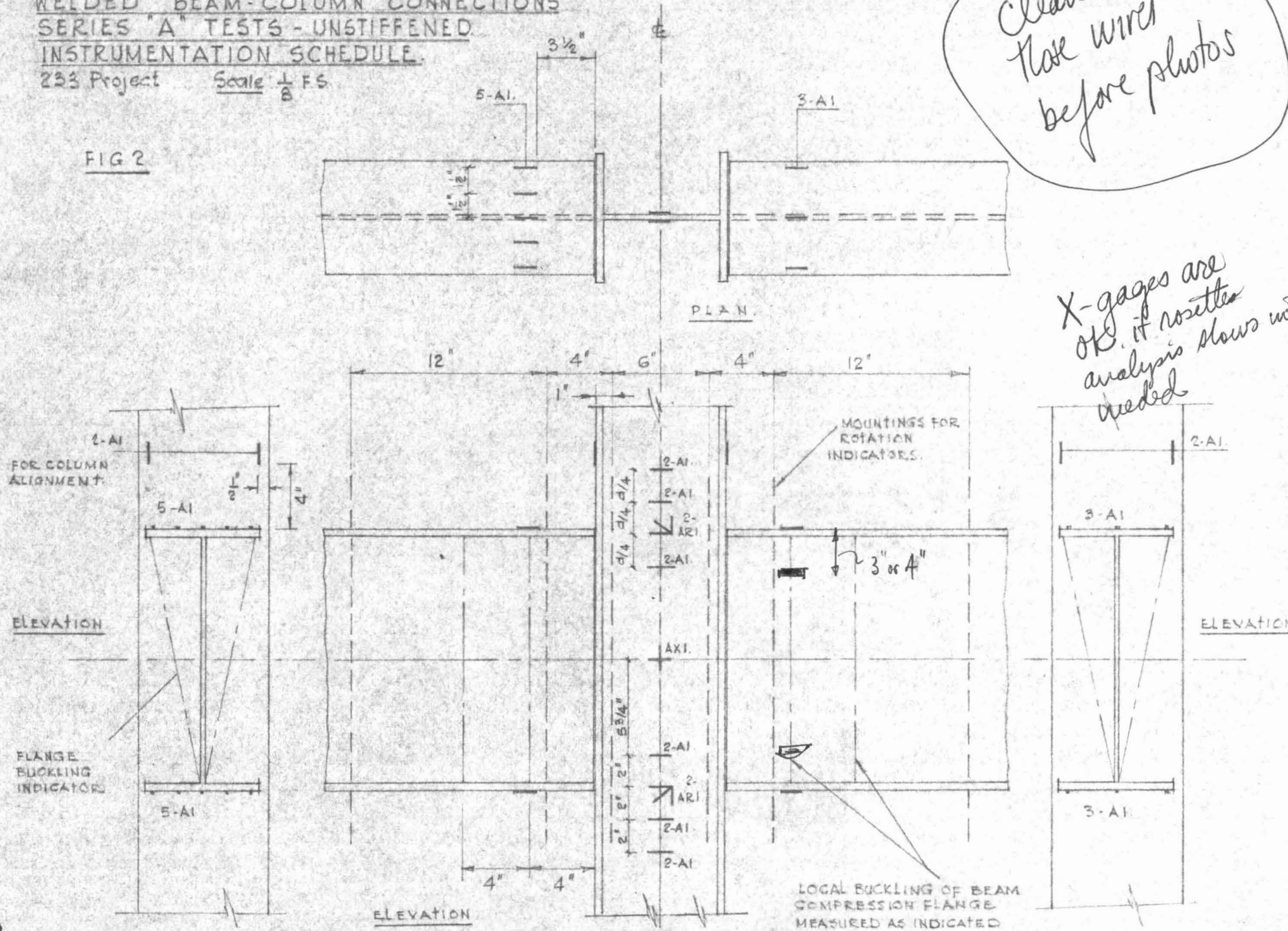
SCALE 1" = 1 FOOT

FIG. 1.



WELDED BEAM-COLUMN CONNECTIONS
SERIES "A" TESTS - UNSTIFFENED
INSTRUMENTATION SCHEDULE.
233 Project Scale $\frac{1}{8}$ F.S.

FIG 2



Clean up those wires before photos

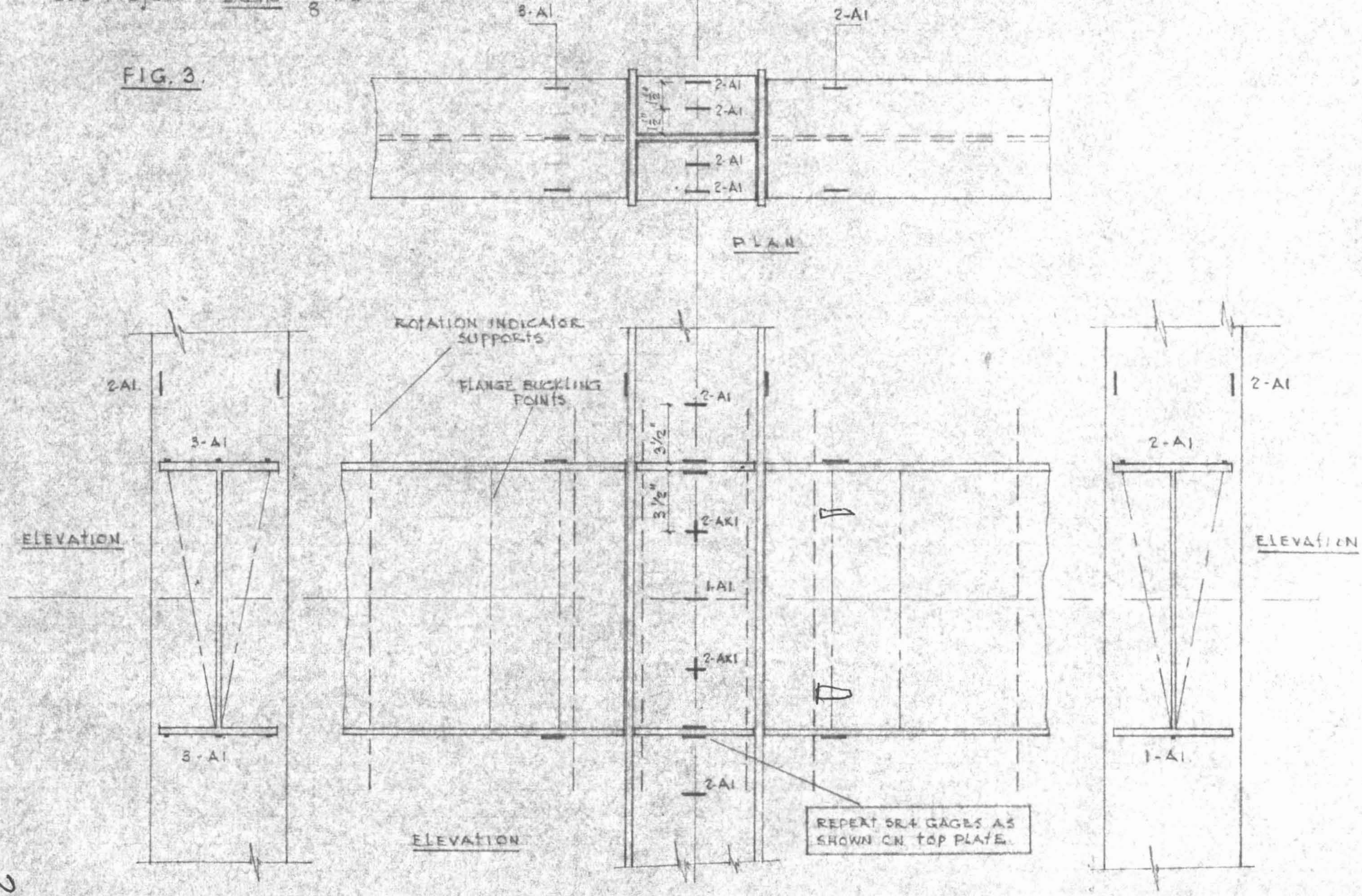
X-gages are OK if rosettes analysis shows not needed

2

copies each

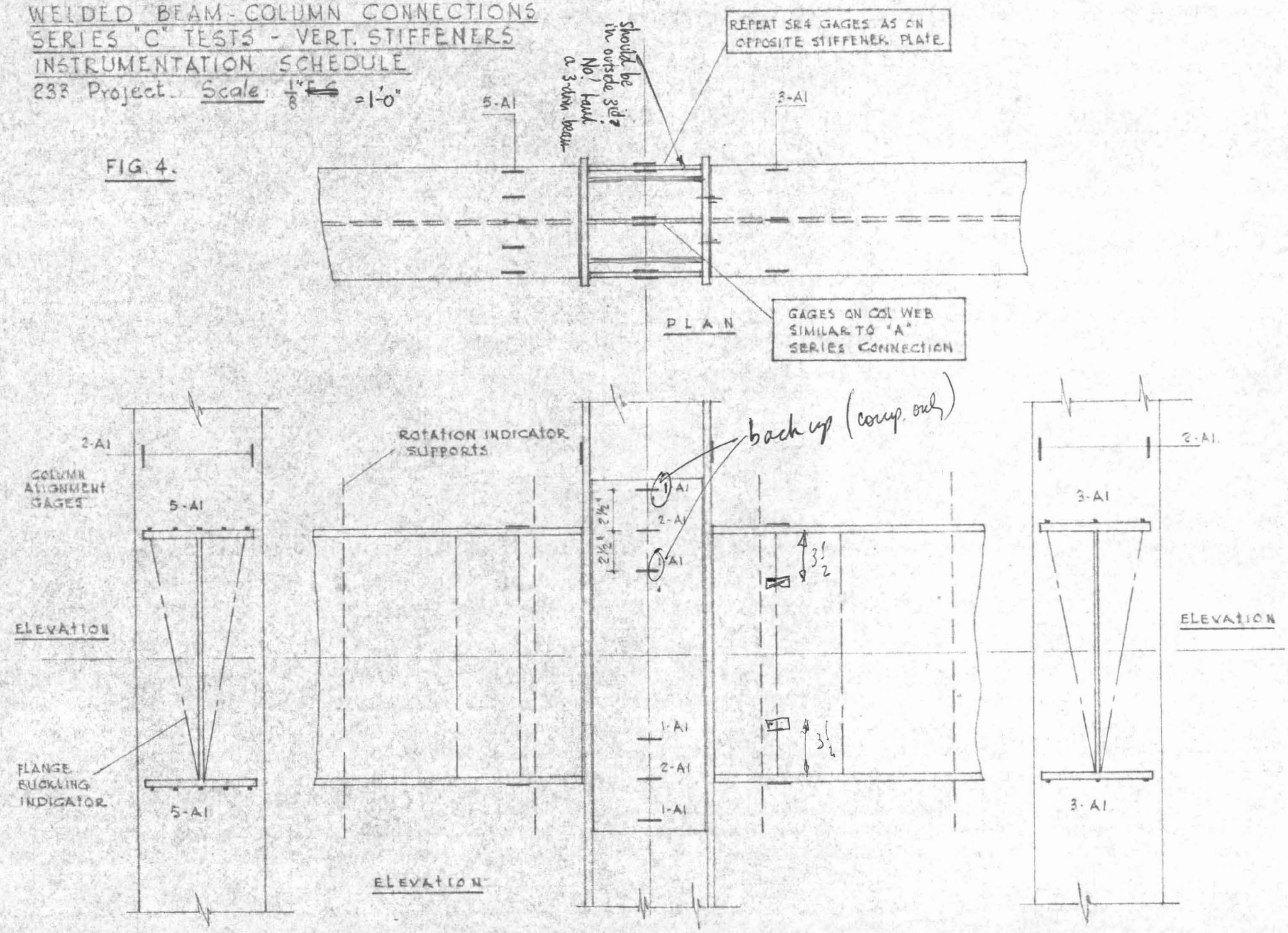
WELDED BEAM - COLUMN CONNECTIONS
SERIES "B" TESTS - HORIZ STIFFENERS
INSTRUMENTATION SCHEDULE
283 Project Scale $\frac{1}{8}$ F.S.

FIG. 3.

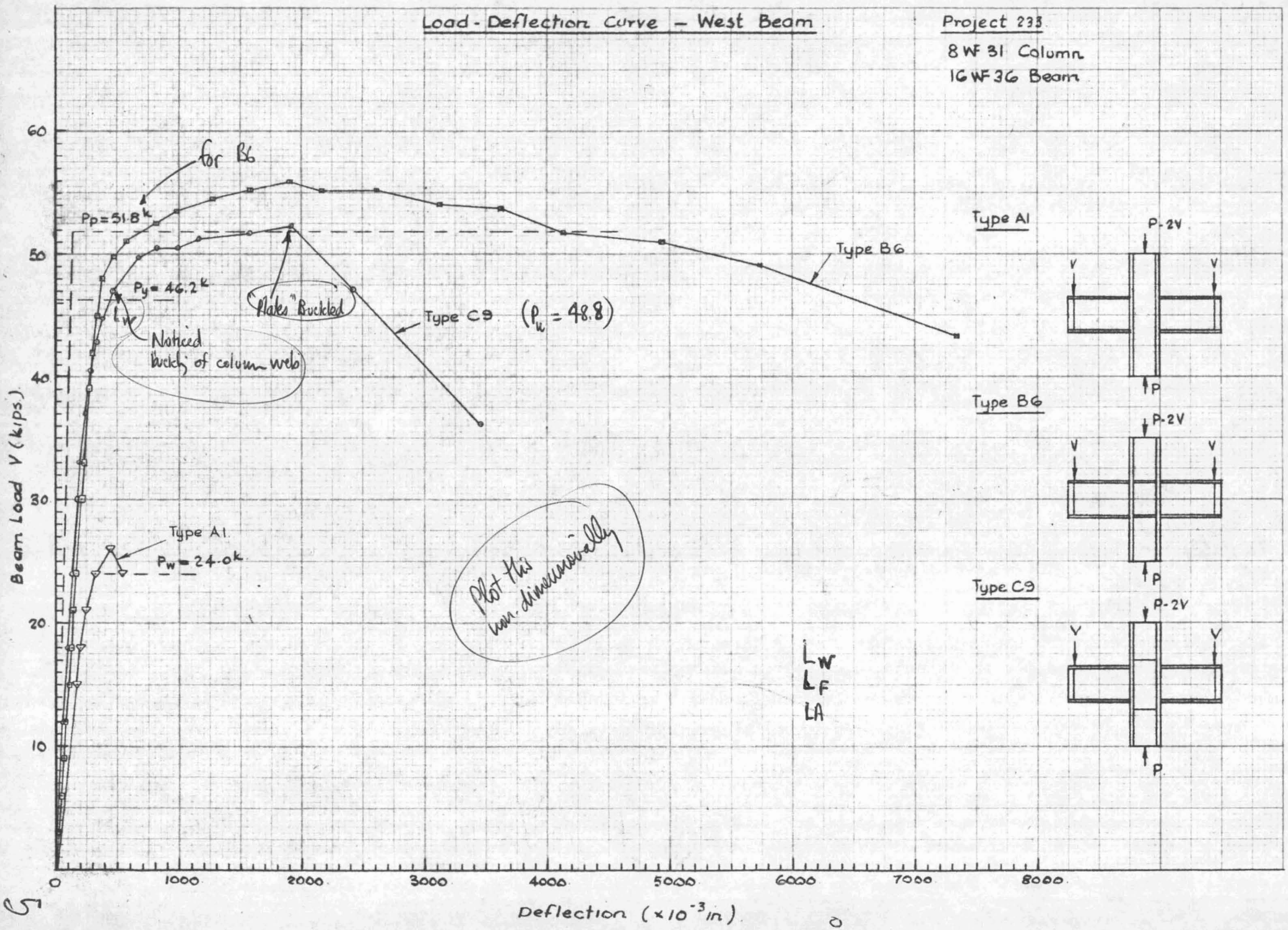


WELDED BEAM-COLUMN CONNECTIONS
SERIES "C" TESTS - VERT. STIFFENERS
INSTRUMENTATION SCHEDULE
233 Project. Scale: 1/8" = 1'-0"

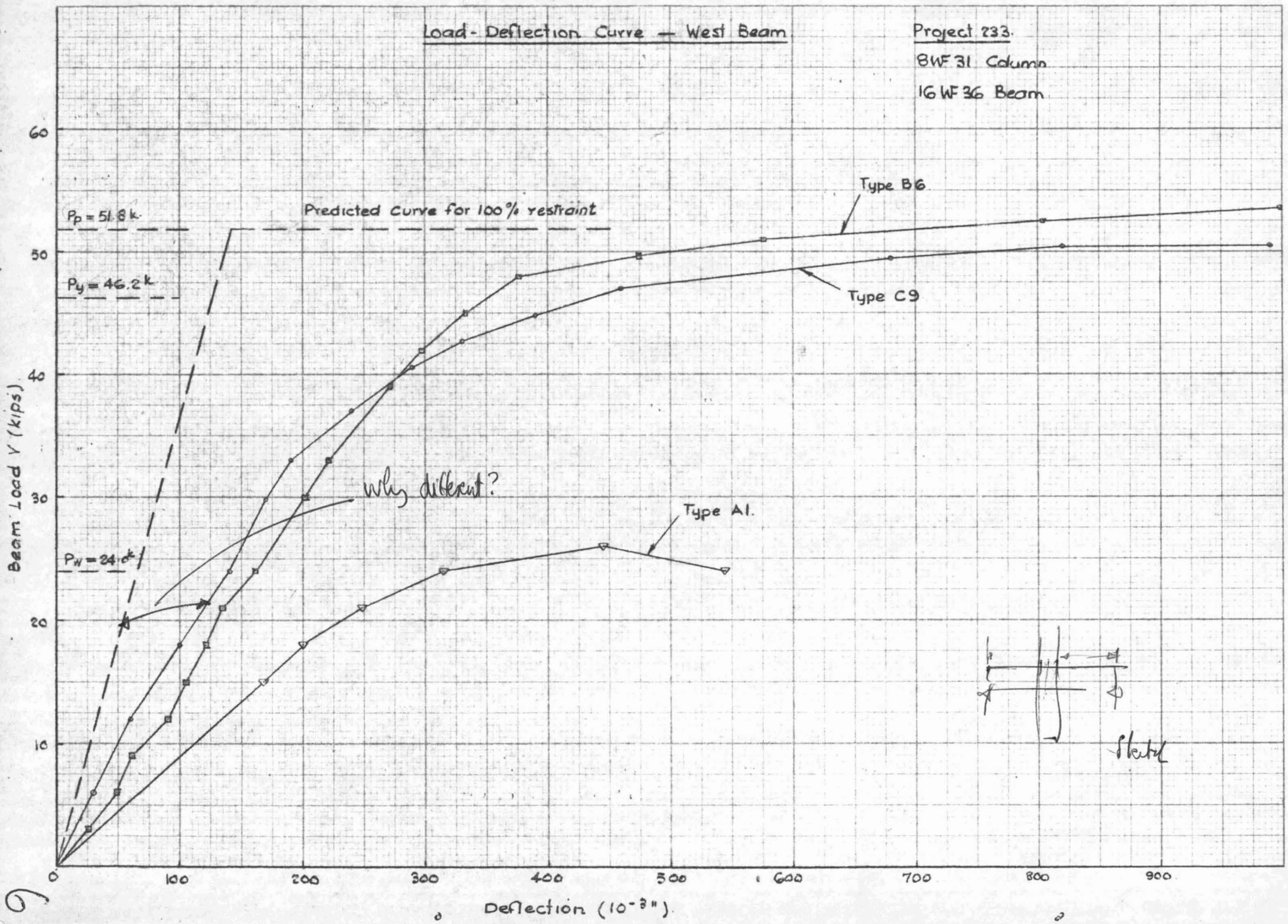
FIG. 4.



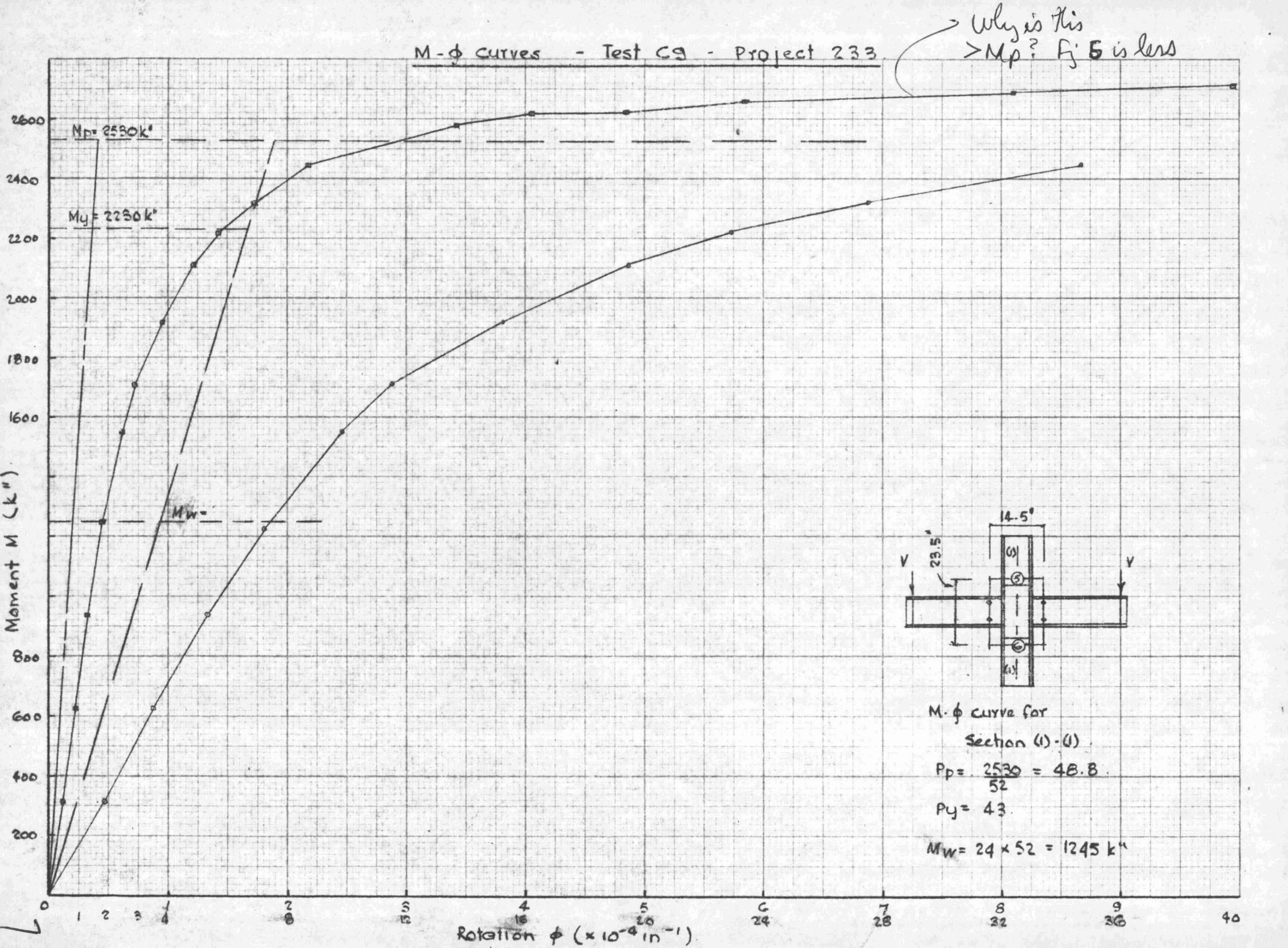
4

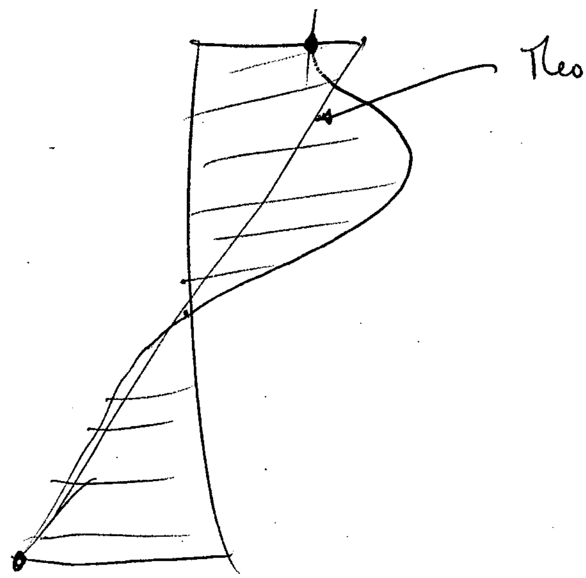

Load-Deflection Curve - West Beam
Project 233
8 WF 31 Column
16 WF 36 Beam
Beam Load V (kips.)
Deflection (×10⁻³ in)
0
1000
2000
3000
4000
5000
6000
7000
8000
10
20
30
40
50
60
for B6
Pp = 51.8k
Py = 46.2k
Lw
Noticed buck'g of column web
"Flakes" Buckled
Type C9
(Pu = 48.8)
Type B6
Type A1
Pw = 24.0k
Plot this non-dimensionally
Lw
LF
LA
Type A1
P-2V
V
V
P
Type B6
P-2V
V
V
P
Type C9
P-2V
V
V
P

5


Load - Deflection Curve — West Beam
Project 233.
8WF31 Column
16WF36 Beam
Predicted curve for 100% restraint
$P_p = 51.8^k$
$P_y = 46.2^k$
$P_w = 24.0^k$
Type B6
Type C9
Type A1.
Why different?
Sketch
Beam Load V (kips)
Deflection (10^{-3}")
0
100
200
300
400
500
600
700
800
900
10
20
30
40
50
60

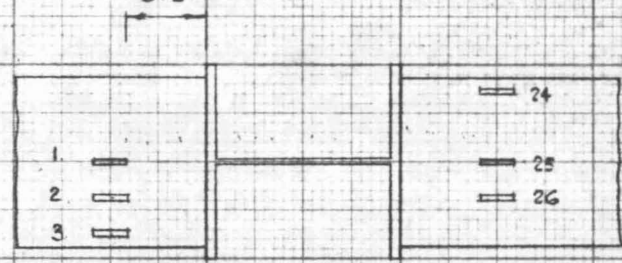
6


M-φ curves - Test C9 - Project 233
Why is this >Mp? Fig 5 is less
Mp= 2530 k'
My= 2230 k'
Mw=
Moment M (k")
2600
2400
2200
2000
1800
1600
800
600
400
200
Rotation φ (× 10⁻⁴ in⁻¹)
0
1 2 3 4 8 12 16 20 24 28 32 36 40
14.5"
23.5"
V
V
(1)
(5)
(6)
(1)
M·φ curve for
Section (1)-(1)
Pp = 2530 / 52 = 48.8
Py = 43
Mw = 24 × 52 = 1245 k"

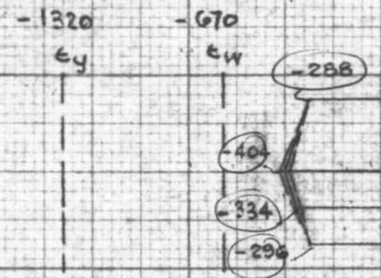


πeo

Strain Distribution in Beam Flanges - Test A1 Project 233



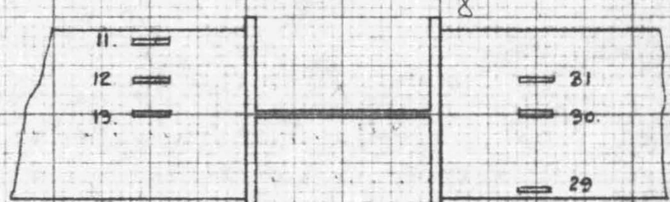
Plan of Compression Flange

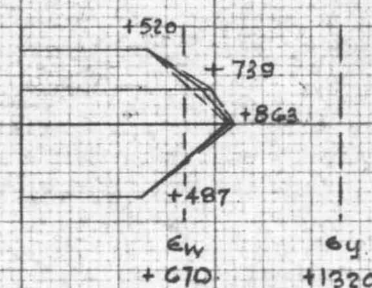


At working load

P_W = 24k on beams
= 131.8 on column

Tension Flange





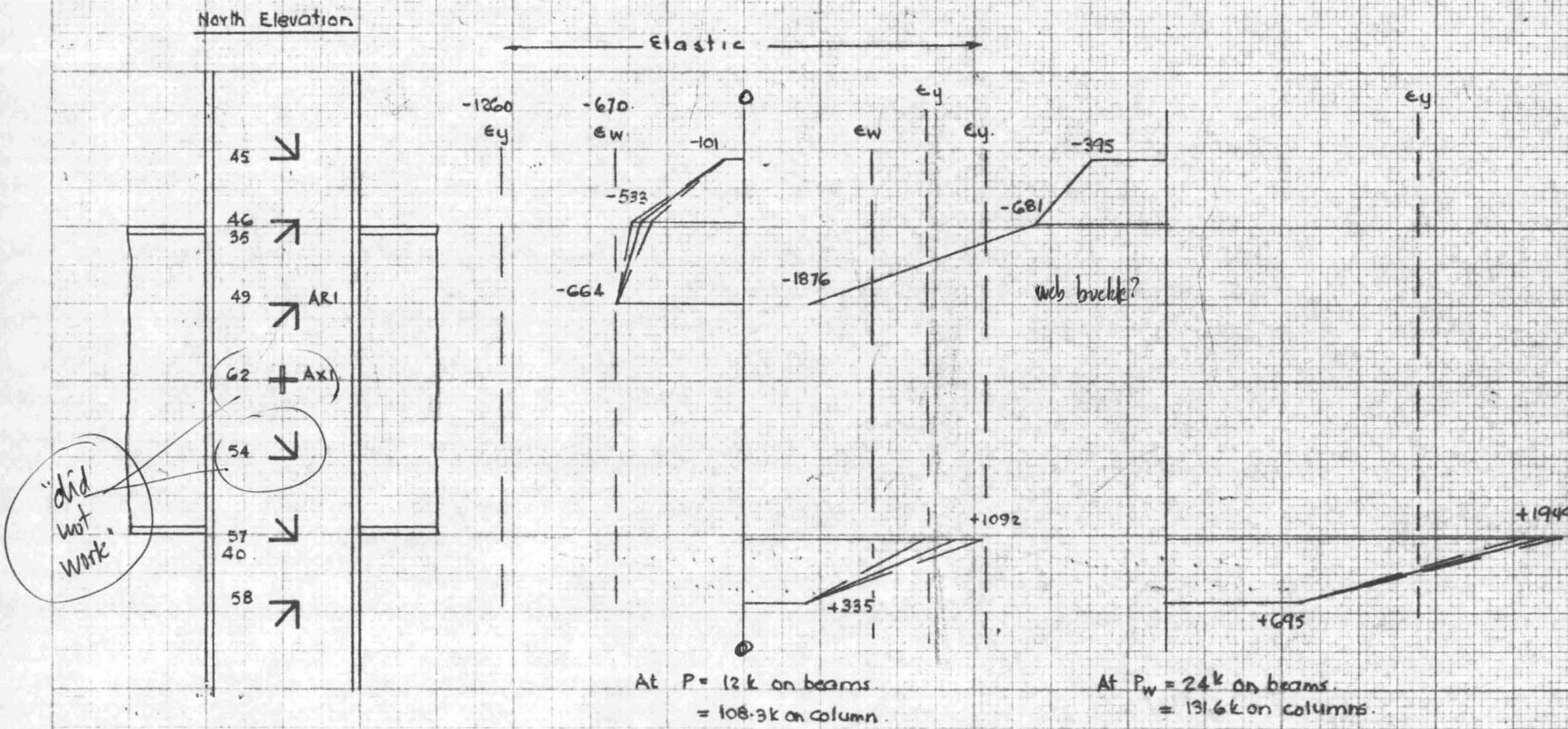
For 16 WF 36 σ_y = 39.6 ksi

E = 30000 ksi

At working load σ = 20 ksi $\epsilon_w = \frac{20}{30000} = 670 \times 10^{-6}$ in/in

At yield $\epsilon_y = \frac{39.6}{30000} = 1320 \times 10^{-6}$ in/in

Strain Distribution in Column Web - Test A1 Project 233.



σ_y = 37.9 ksi

E = 30000 ksi

$\epsilon_w = \frac{20}{30000} = 670 \times 10^{-6}$ in/in $\quad \epsilon_y = 1260 \times 10^{-6}$ in/in

Don't test until all gages are working

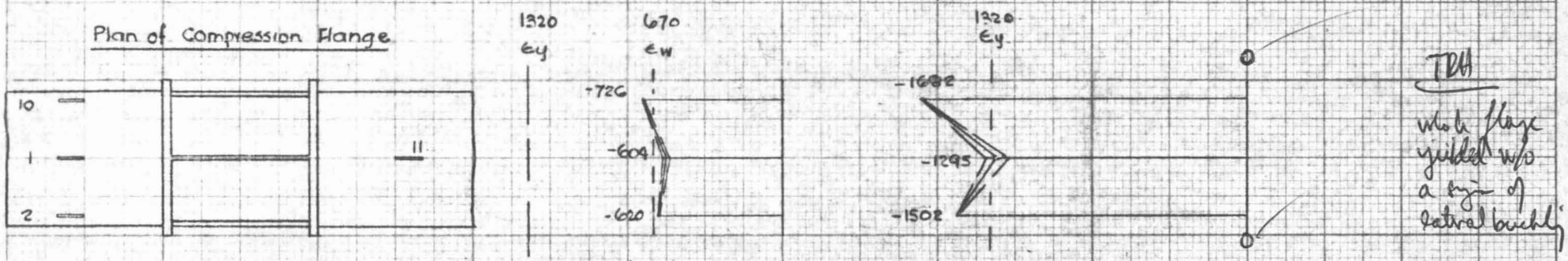
0

(12)

(18).

Strain Distribution in Beam Flanges - Test C9 Project 233

Plan of Compression Flange

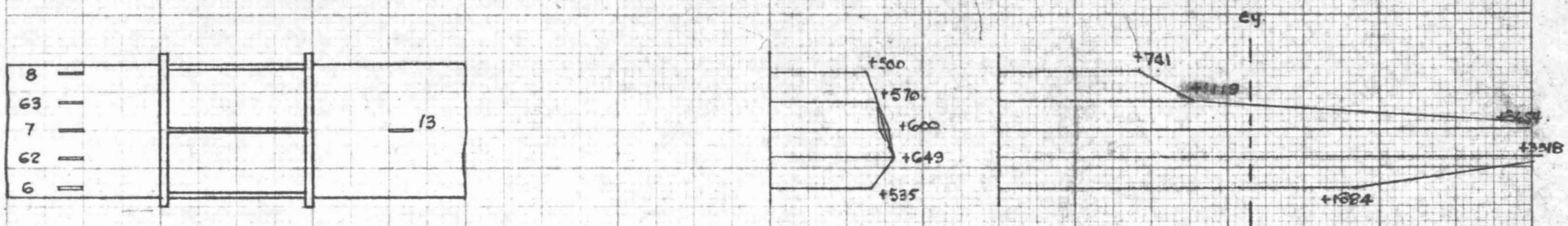


A. P_W = 24 k on beams
= 132 k on columns

B. P = 47.25 k on beams
= 132.8 k on column

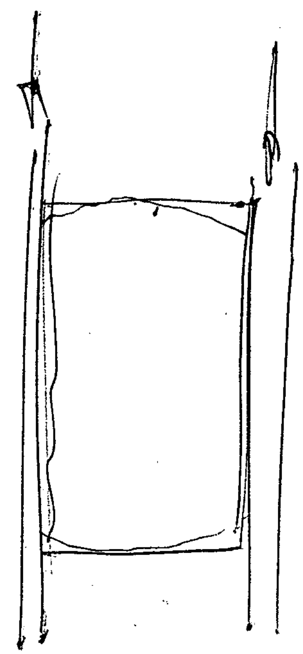
At working Load

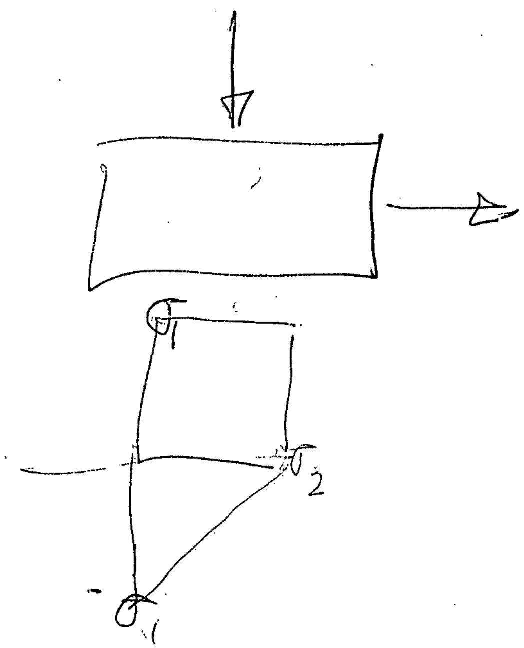
Plan of Tension Flange



ϵ_W = 670×10^{-6} in/in

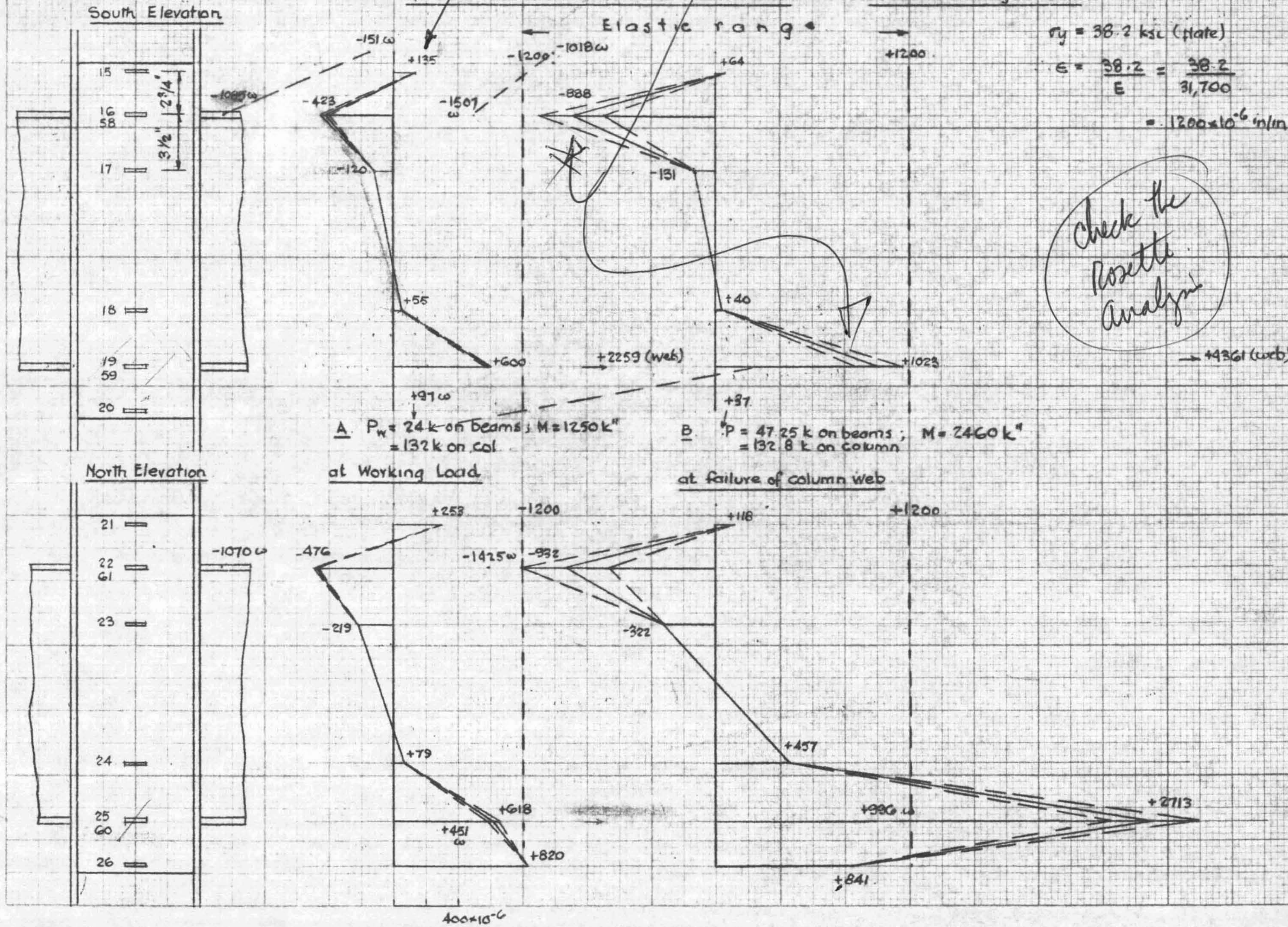
ϵ_y = 1320×10^{-6} in/in

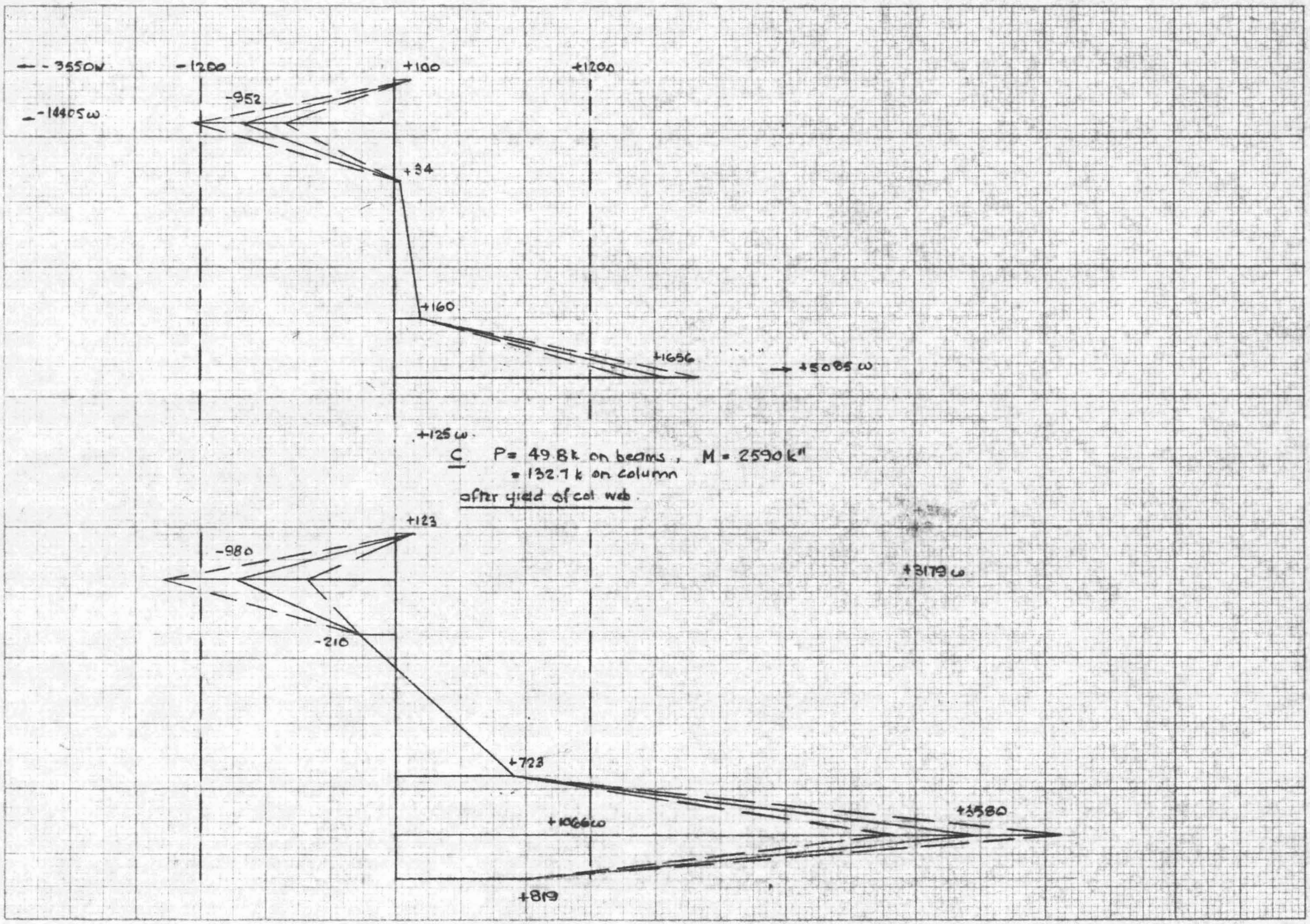



σ1
σ2
σ1

watch
yield cond

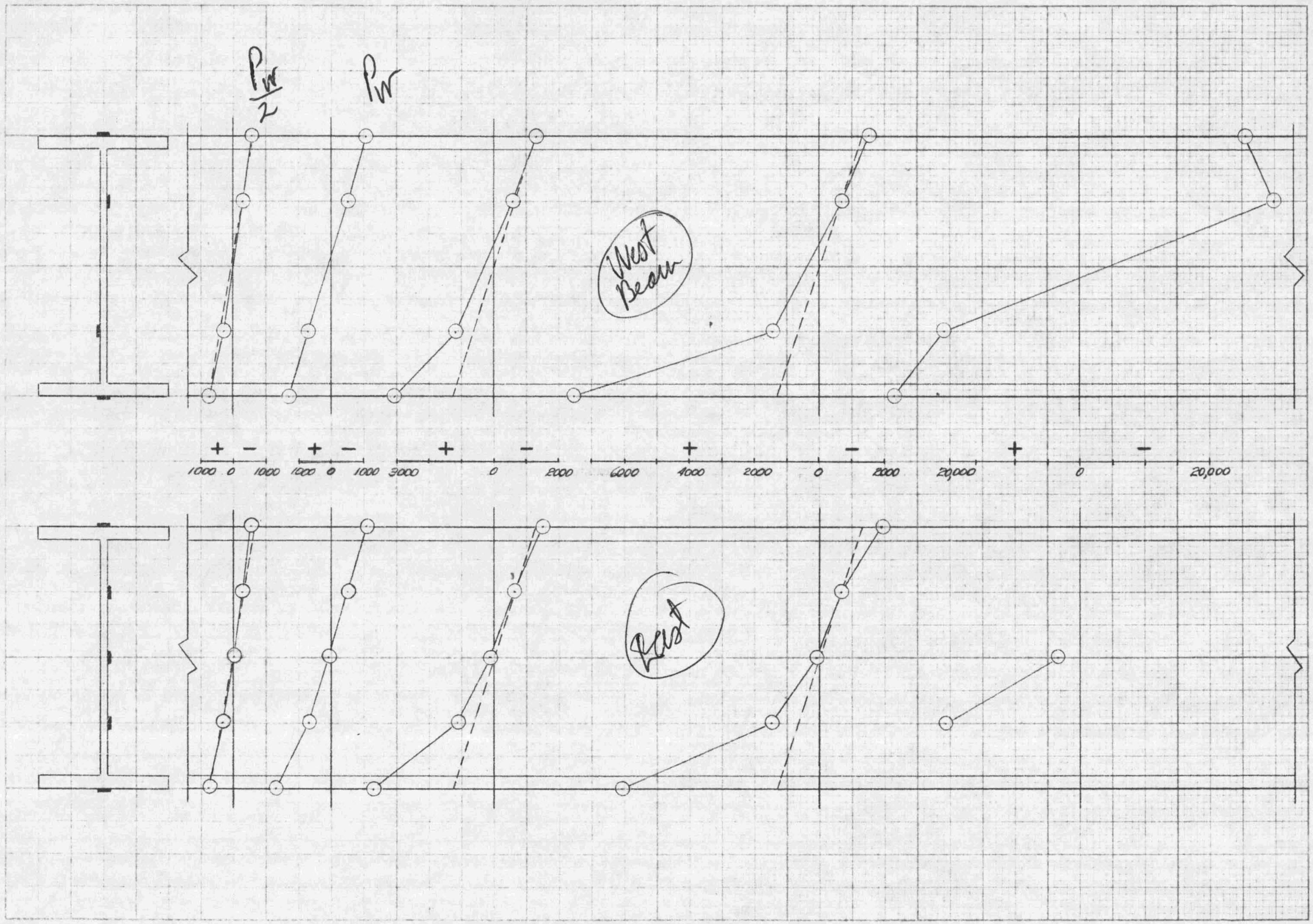
Strain Distribution in Stiffener Plates — Test C9 Project 233




-3550w
-1200
+100
+1200
-952
-1445w
+34
+160
+1656
+5085 w
+125 w
C P= 49.8k on beams, M = 2590 k"
= 132.7 k on column
after yield of col web
+123
-980
+3179 w
-210
+723
+3580
+1066 w
+819

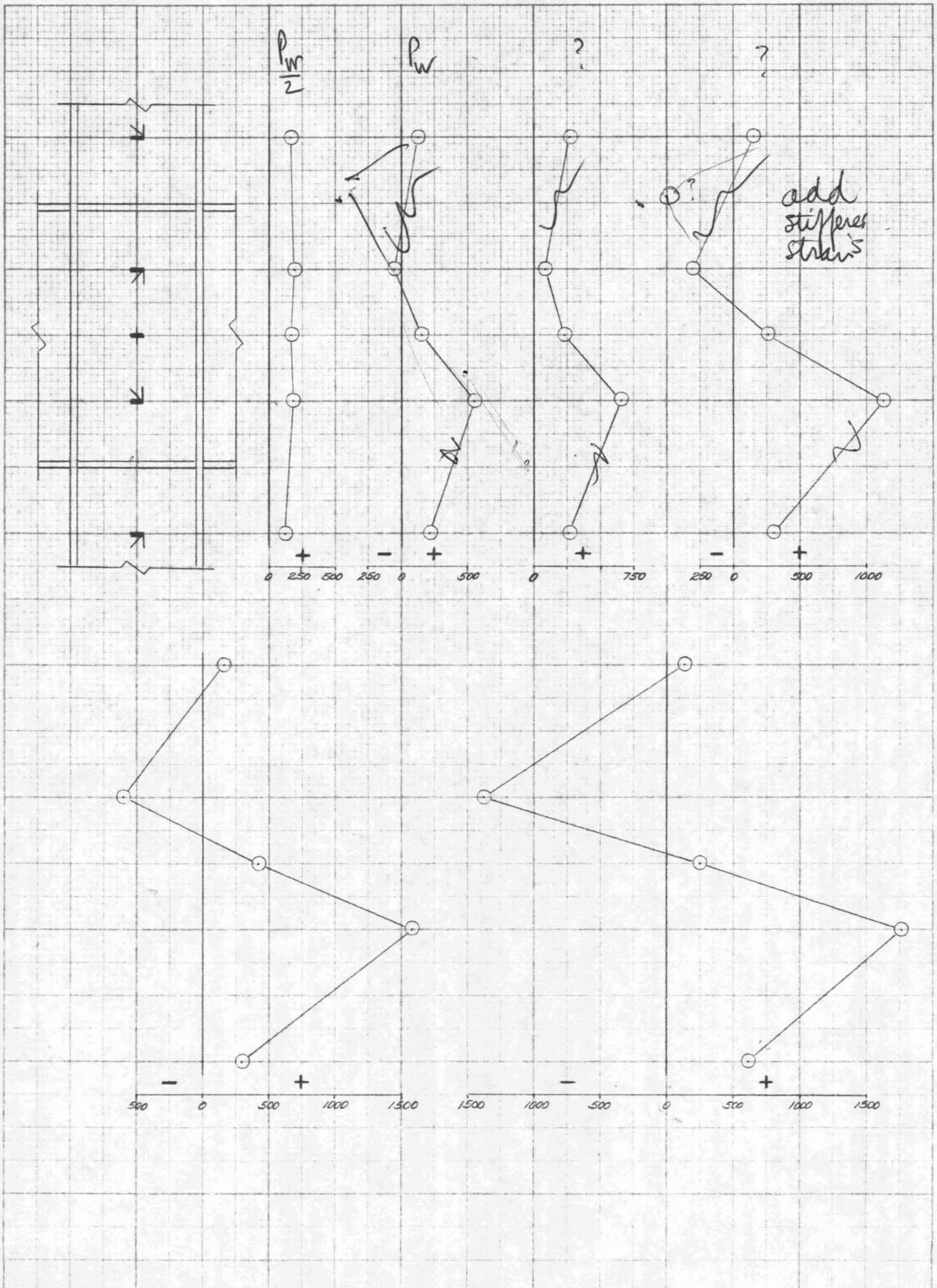
12

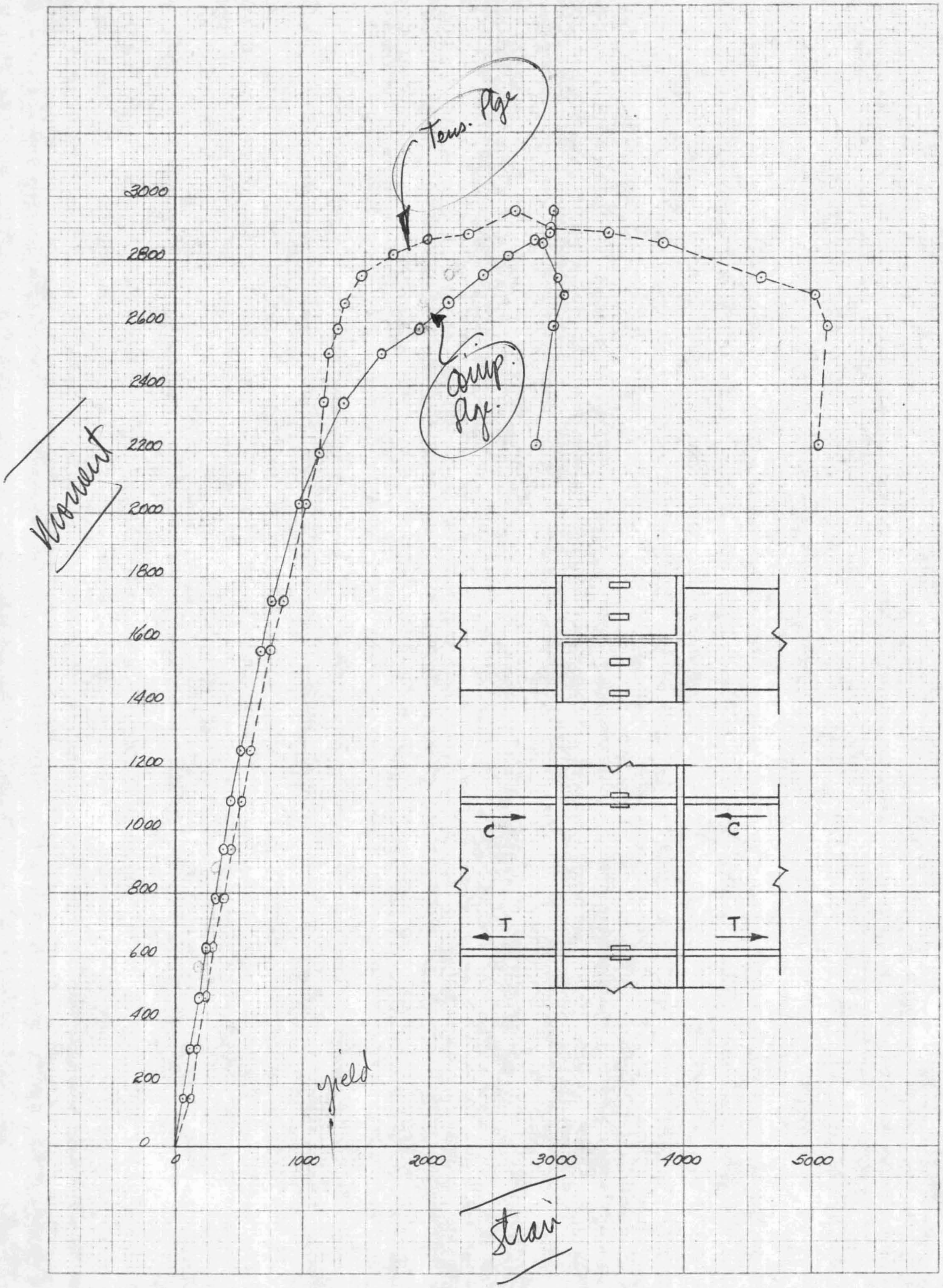
(B6)

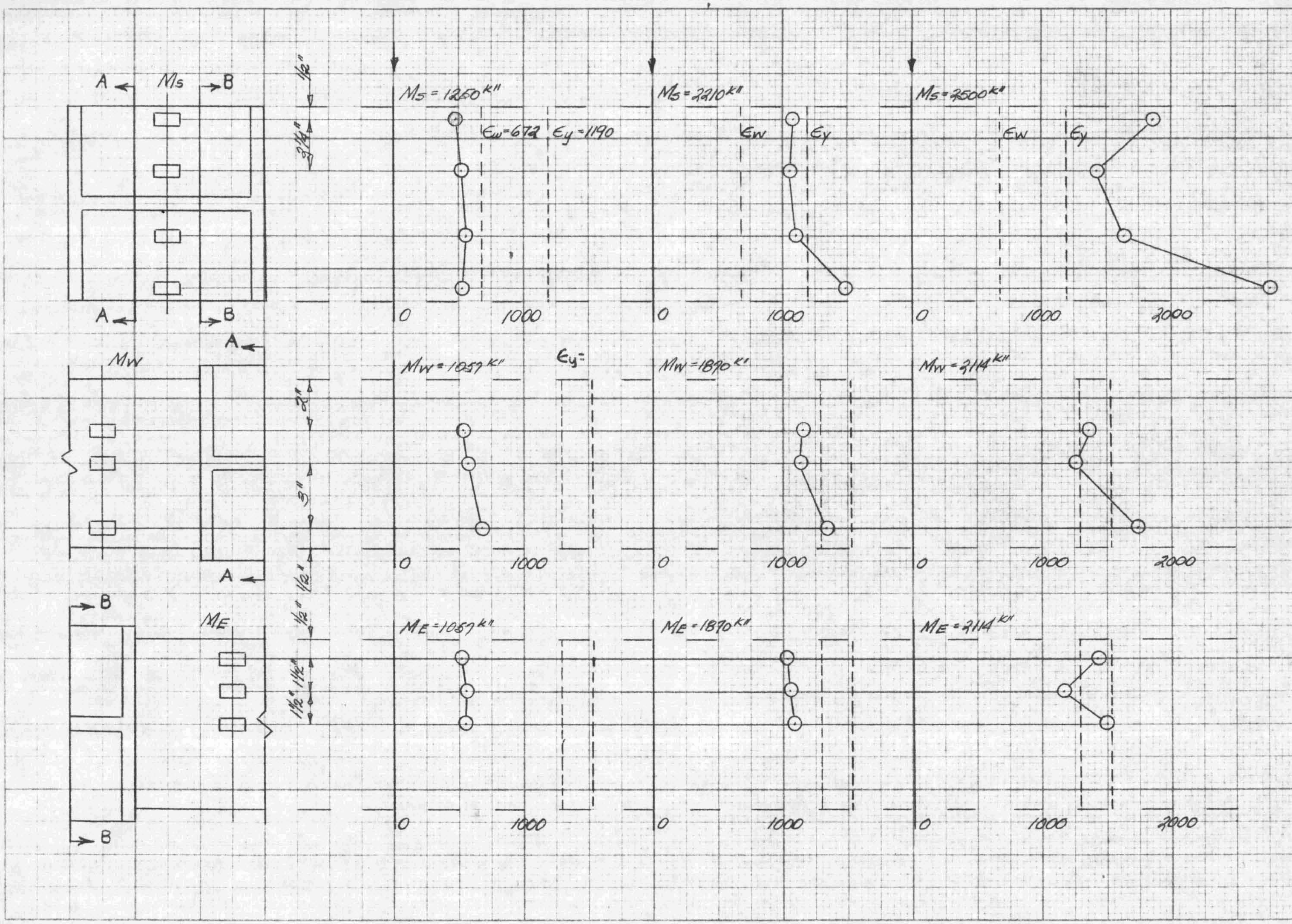


$\frac{P_W}{2}$
P_W
West Beam
+ − + − + − + − + −
1000 0 1000 1000 0 1000 3000 0 2000 6000 4000 2000 0 2000 20,000 0 20,000
East

13


Pw/2
Pw
?
?
add
stiffener
strains
+
0 250 500
−
+
250 0 500
+
0 750
−
+
250 0 500 1000
−
+
500 0 500 1000 1500
−
+
1500 1000 500 0 500 1000 1500


Tens. Flge
Comp. Flg.
Moment
Strain
yield
3000
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0
1000
2000
3000
4000
5000
C
C
T
T


A
Ms
B
A
B
1/2"
3 1/4"
Ms = 1250 k"
Ew = 672
Ey = 1190
0
1000
Ms = 2210 k"
Ew
Ey
0
1000
Ms = 2500 k"
Ew
Ey
0
1000
2000
Mw
A
A
2"
3"
1/2"
Mw = 1057 k"
Ey =
0
1000
Mw = 1890 k"
0
1000
Mw = 2114 k"
0
1000
2000
B
ME
B
1/2"
1 1/2"
1 1/2"
ME = 1057 k"
0
1000
ME = 1890 k"
0
1000
ME = 2114 k"
0
1000
2000