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Beam-to-Column Connections

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HIGHLIGHTS OF LEHIGH CONNECTION TESTS--
WEB CONNECTION TESTS 14-1 AND 14-3

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This work has been carried out as part of an investigation sponsored jointly by the American Iron and Steel Institute and the Welding Research Council.

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Highlights of Connection Tests 14-1 and 14-3

Tests of two of the four projected beam-to-column web connections were recently completed. They were Test 14-1, a flange-welded web-bolted connection and Test 14-3, a flange-bolted and web-bolted connection. Both tests exhibited a fracture in the tension flange connection plate while the specimens were in the ascending portion of the inelastic load-deflection curve. Further details of the tests and results are given on the accompanying pages.

Prior to further discussion of these tests, a brief description of the testing procedure will be given. After being placed in the testing machine and properly aligned, the column was loaded in 250 kip increments to a load of 1520 kips. This is equal to the value of the column axial load P obtained from $P/P_y = 0.5$ (1810 kips) minus 290 kips. The value of 290 kips is the beam load (V) calculated to cause M_p in the beam at the critical section, which is at a different location in each test. Both P and V values calculated above are based on using nominal yield stress values for the column and beam material respectively. The material used for both beams and columns of the tests was A572 Grade 50.

The beam was then loaded in increments of approximately 25 kips until deflections became excessive, at which time a deflection increment was applied. The value of the column load was adjusted at each increment to read 1520 kips plus the beam load V . Thus, the column in the top half of the assemblage had an axial load of $P + V$ and the column in the bottom half had a value of P . Once the plastic moment of the beam had been attained the value of the axial load in the upper

column was equal to the desired value of $P/P_y = 0.5$. Shown in Fig. 1 is a drawing of the assemblage and the loading scheme.

Connection 14-1

Connection 14-1 shown in Fig. 2 is a flange-welded, web-bolted connection. The beam flanges are connected to the column by means of flange connection plates. These plates, equal in thickness to the beam flange, are connected to the column flanges and web by fillet welds and to the beam flange by full penetration butt welds. The web of the beam is attached to a web connection plate by seven 7/8 in. dia. A490 bolts in 15/16 in. dia. holes. The web connection plate is fillet welded to the column web and flange connection plates.

The load-deflection curve plot of beam load V vs. beam deflection Δ is given in Fig. 4. This connection has a definite linear elastic $V-\Delta$ slope with the effect of yielding of the assemblage being indicated by the reduction in stiffness at higher load levels. The failure of this specimen occurred at a beam load of 273 kips which is 94 percent of the beam load to cause beam M_p . Failure of this specimen was due to tearing across the entire width of the tension flange connection plate as shown by the sketch in Fig. 3. On the south side of the connection, the tear is located in the region of the butt weld, but then progresses away from the weld in the region of the web connection plate. The failure was instantaneous with no evidence of tearing prior to the last load increment. With such a failure, the beam load dropped to zero immediately and therefore no unloading elastic slope could be obtained.

The elastic theoretical slope shown in the graph in Fig. 4 for comparison, is based only on beam bending and joint rotation due to the beam load and does not include items such as beam shear deformation,

loss of column stiffness due to axial load, or the effect of small end rotations at the top of the column. The theoretical horizontal line is the beam shear required to cause M_p in the beam based on nominal steel yield strength. This value was calculated for a critical beam section located eight inches from the column centerline.

Connection 14-3

Connection 14-3 shown in Fig. 5 is a flange-bolted, web-bolted connection. The beam flanges are bolted to beam flange connection plates by means of ten 1-in. dia. A490 bolts in 1-1/16 in. dia. holes. The two flange connection plates are fillet welded to the flanges and the web of the column. The web of the beam is connected to a web connection plate by seven 7/8 in. dia. A490 bolts in 15/16 in. dia. holes. The web connection plate is fillet welded to the two flange connection plates and the column web.

The load-deflection curve plot of beam load V vs. beam deflection Δ is given in Fig. 7. The plot shows an initial elastic slope up to approximately 90 kips and then a secondary linear slope up to a load of 200 kips. This general type of behavior of two distinct slopes agrees quite favorably with the results of tests on bolted connections recently completed in Phase Eleven of connection research activity. The load-deflection curve then gradually loses stiffness due to yielding of elements within the connection assemblage. The maximum load attained on this test was 289 kips which is 100 per cent of the beam load to cause beam M_p at the critical section. During the next load interval a tear developed in the tension flange connection plate as shown in Fig. 6 and the load dropped to 249 kips. During this load interval, the load reached a value of approximately 300 kips before the tear occurred. Due to the severity of the tear, no further loading was attempted and the beam was unloaded in two increments to obtain an elastic unloading curve.

The elastic theoretical slope shown in the graph in Fig. 7 is again based only on beam bending and joint rotation due to the beam

load. The theoretical horizontal line is the beam shear required to cause M_p in the beam at the critical section. For this test, based upon experience of previous bolted connection studies, the critical section for computing the beam load required to cause M_p was taken as the outer row of flange bolts closest to the applied beam load.

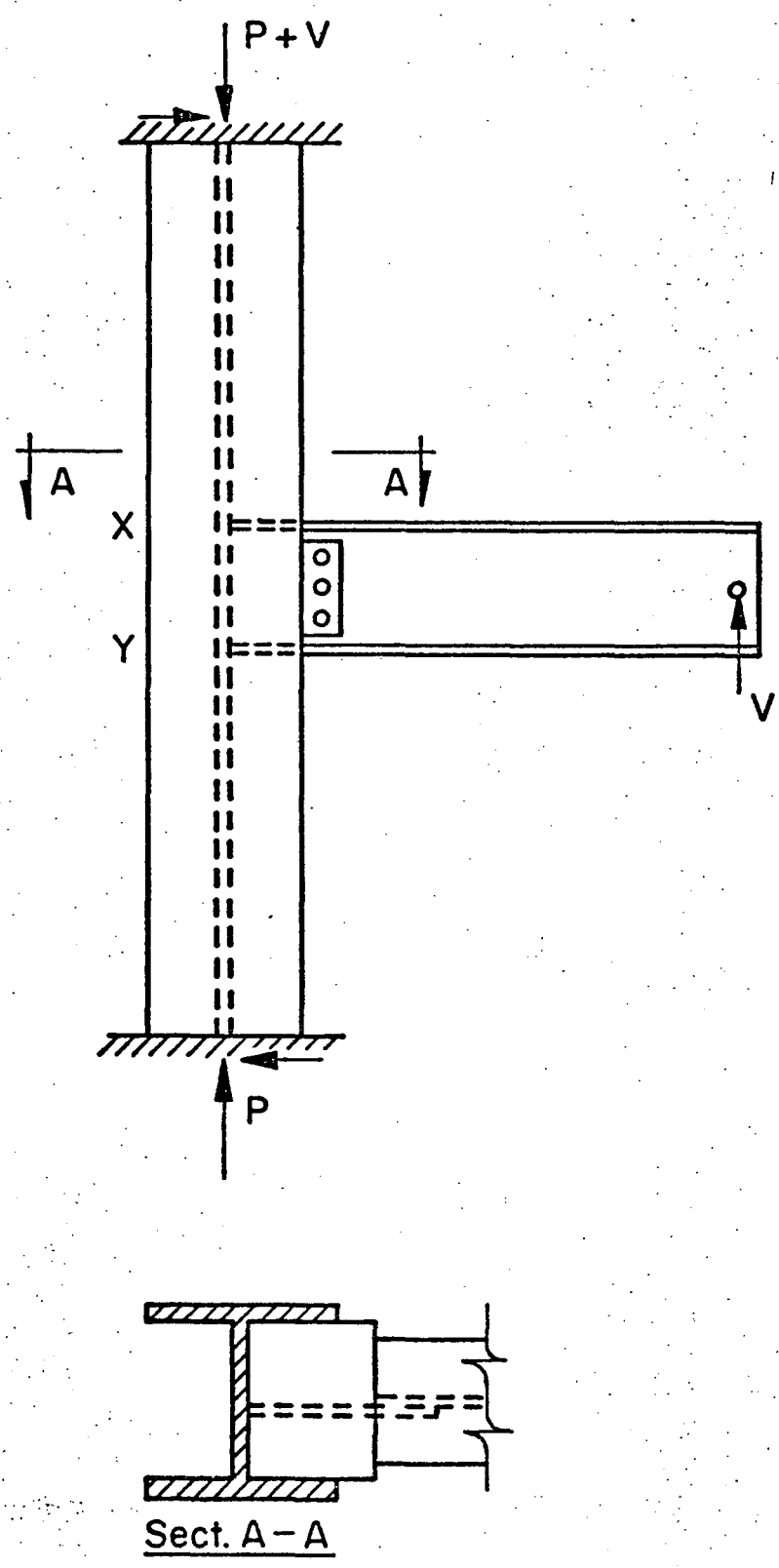


Fig. 1 Connection Assemblage and Loading Scheme

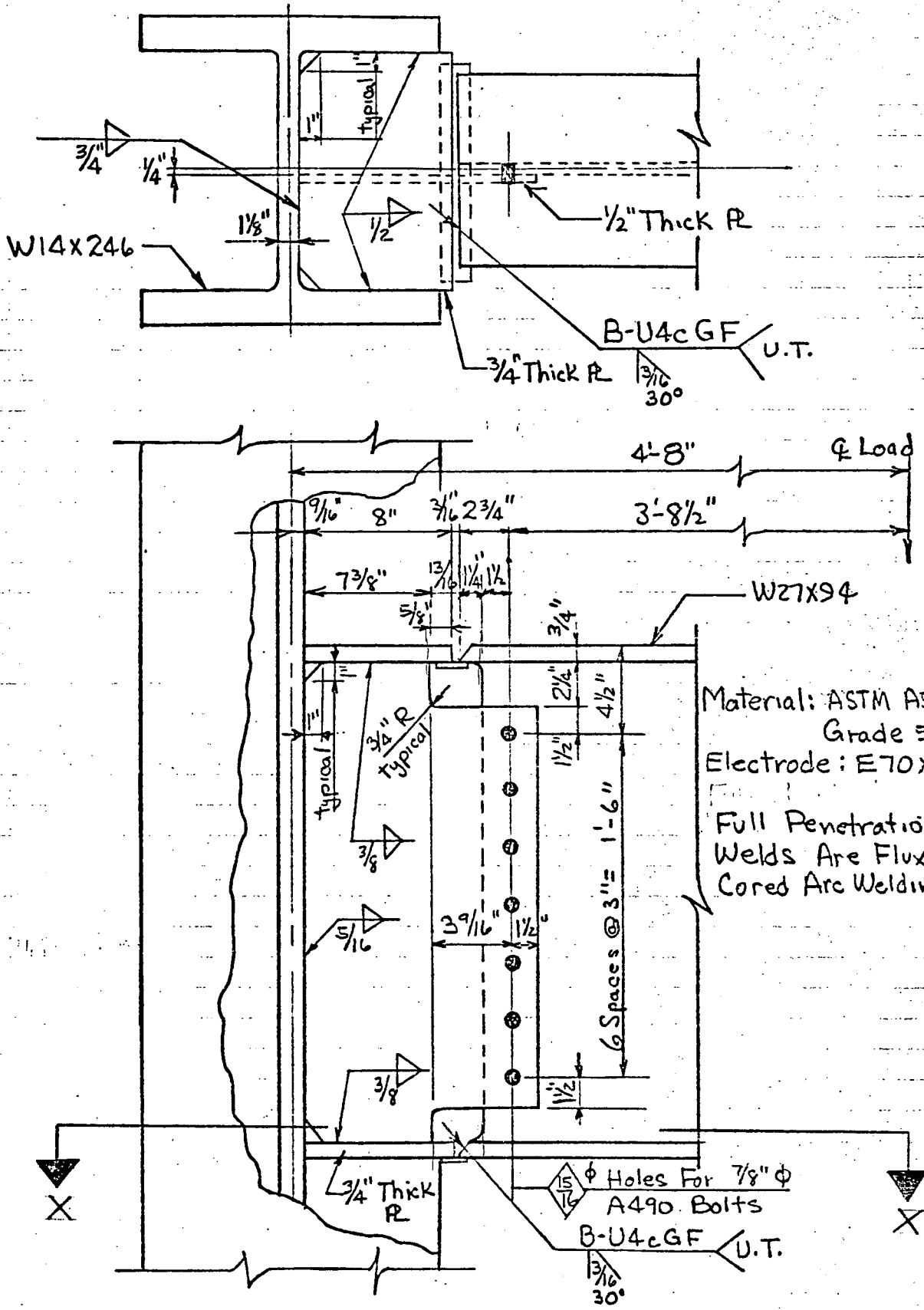


Fig 2 Connection 14-1

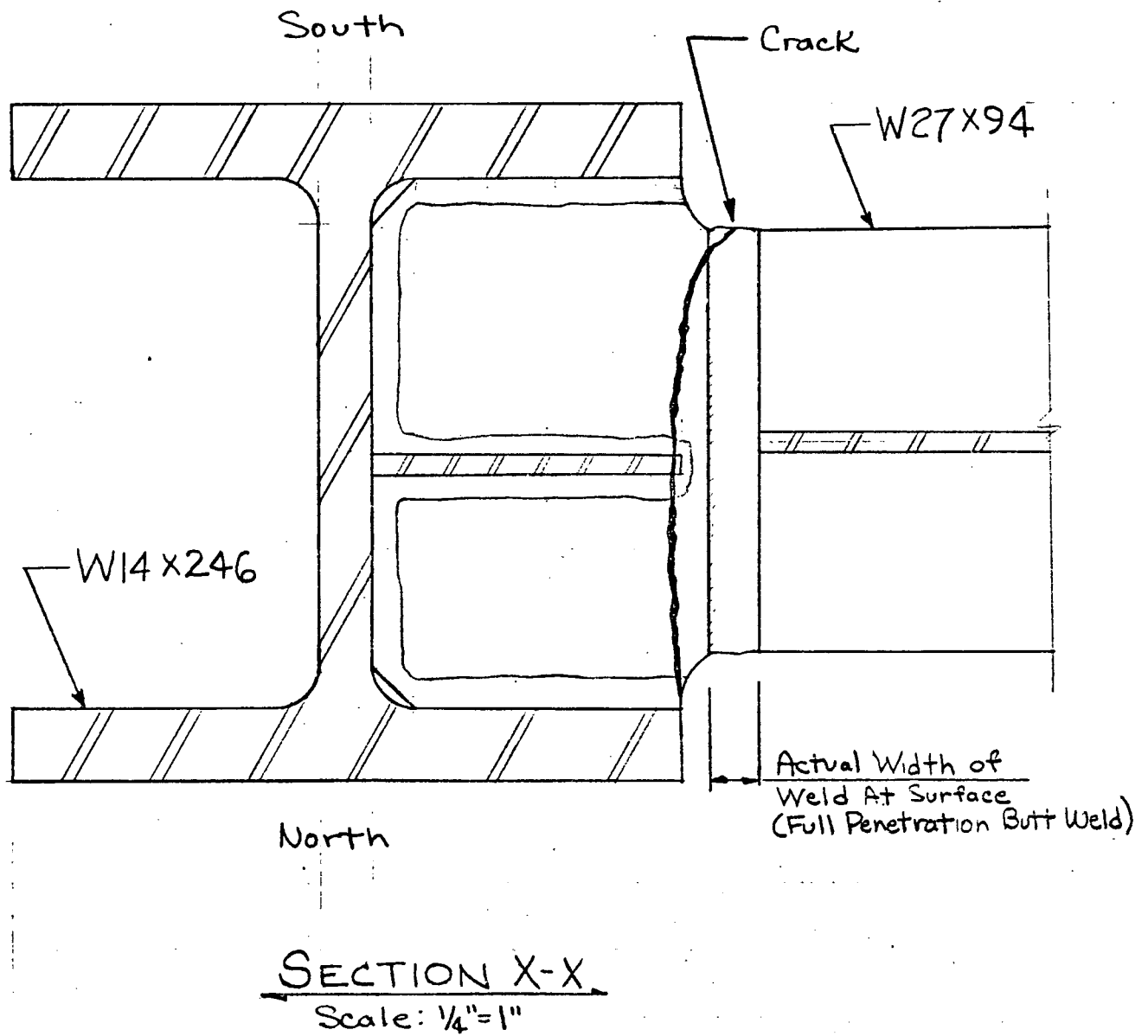
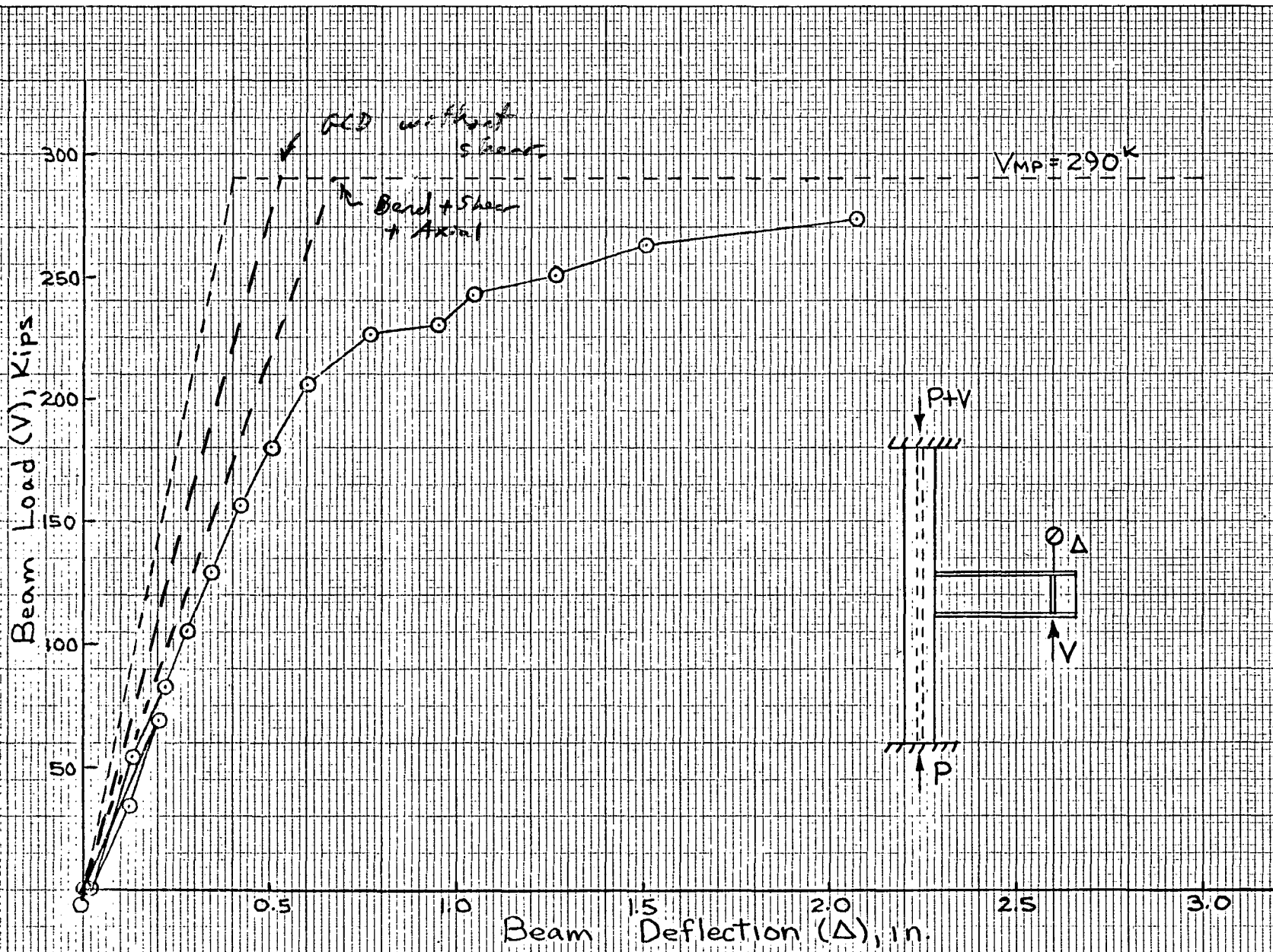
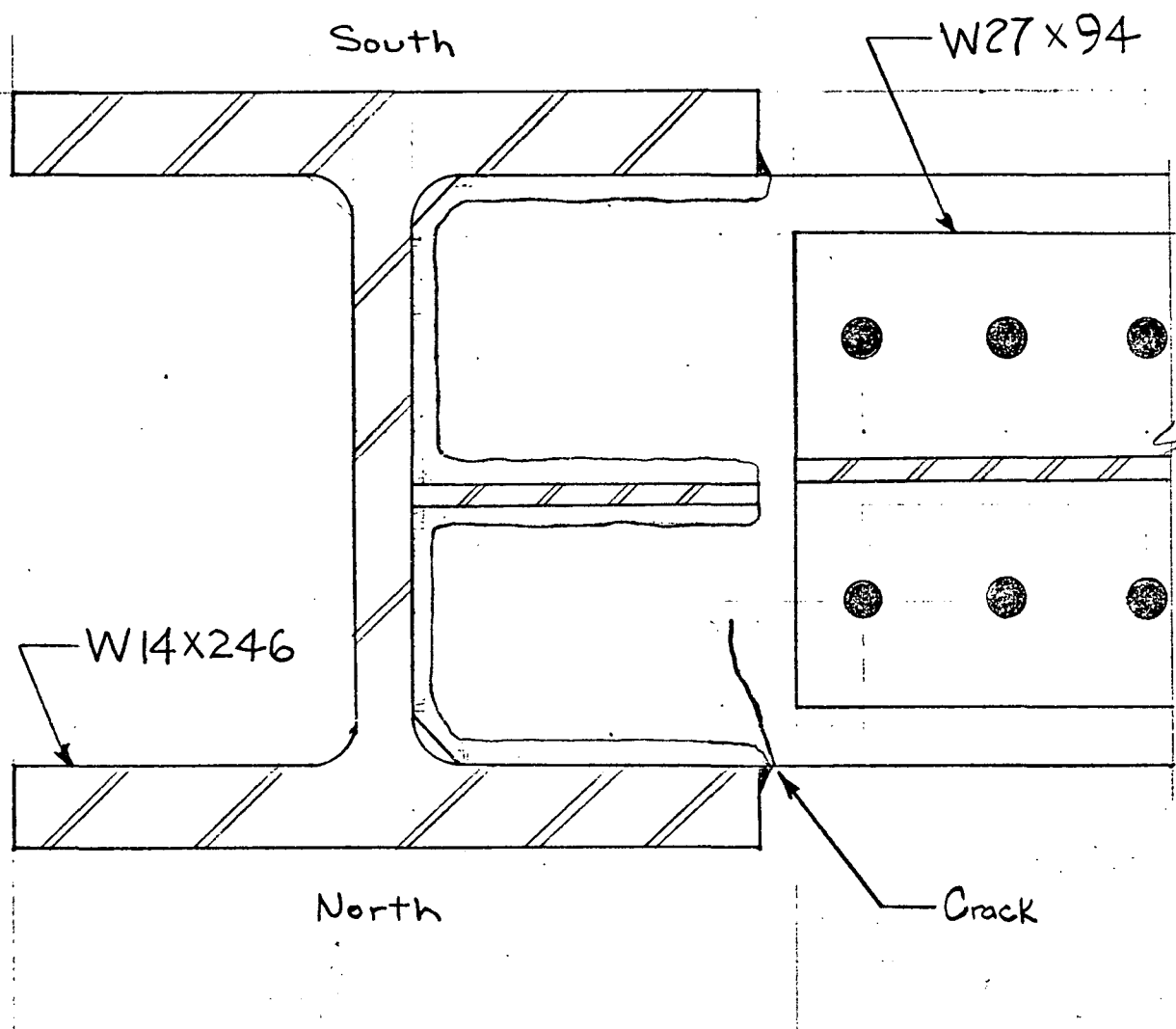


Fig. 3 View Showing Failure Location of Test 14-1

Fig 4 Beam Load Vs Beam Deflection For Test 14-1

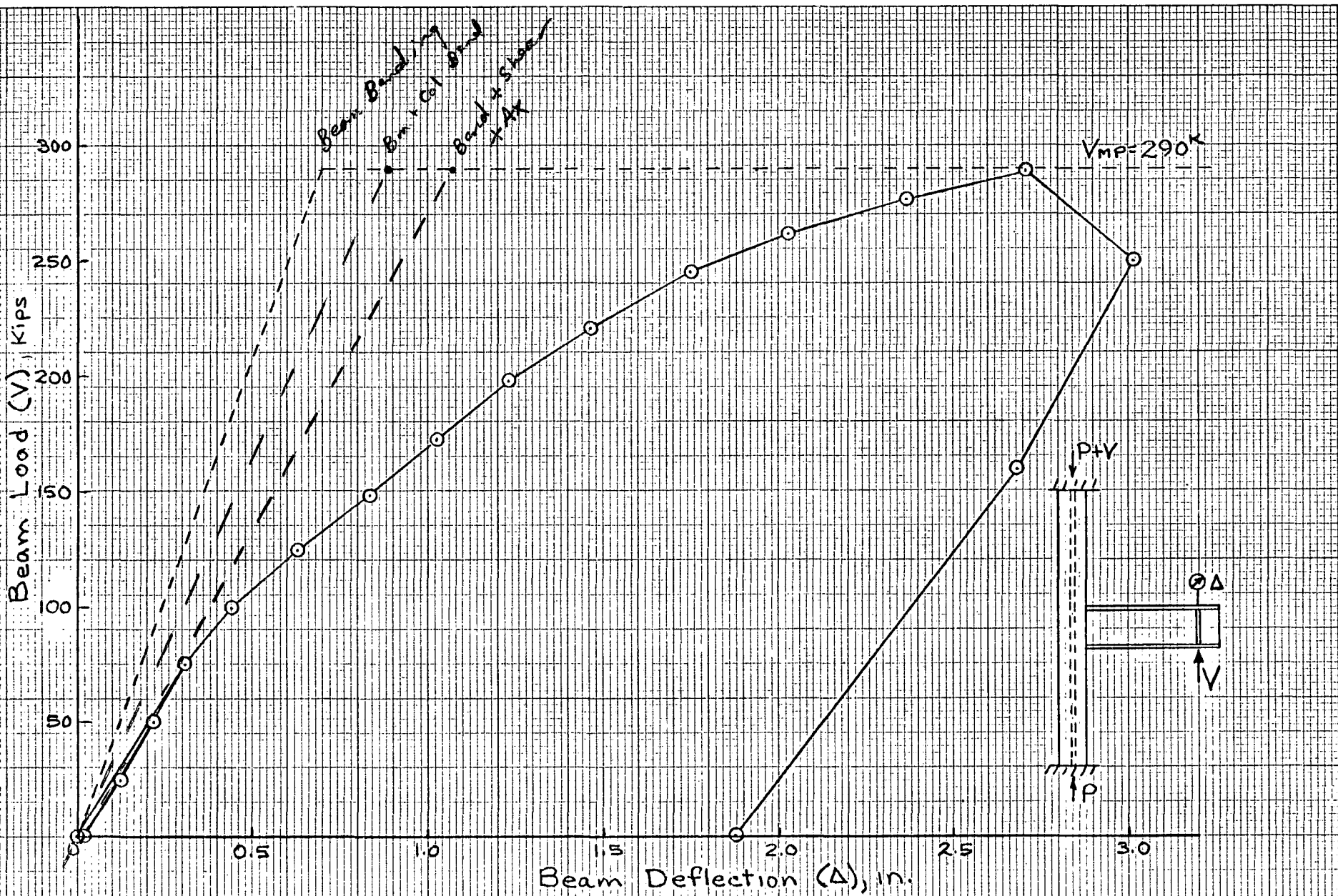


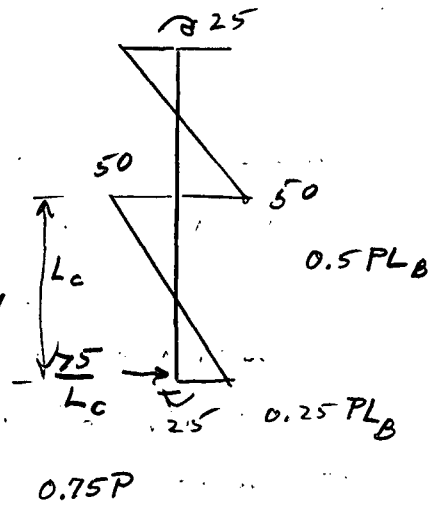
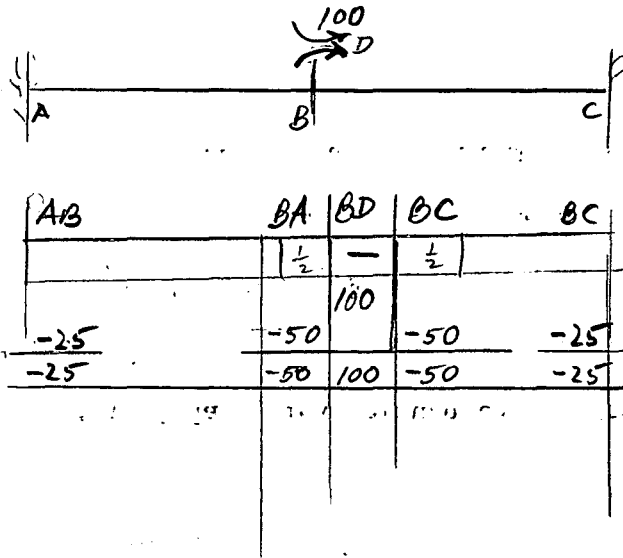
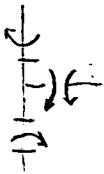


SECTION Y-Y
Scale: $\frac{1}{4}'' = 1''$

Fig. 6 View Showing Failure Location of Test 14-3

Fig 7 Beam Load Vs Beam Deflection For Test 14-3





$$\theta_B = \frac{1}{2} \frac{M_A L_c}{EI_c} - \frac{1}{2} \frac{M_B L_c}{EI_c} = \frac{1}{2} \frac{L_c}{EI_c} (M_A - M_B) = \frac{1}{2} \frac{L_c}{EI_c} (0.25 - 0.5) PL_B$$

$$\theta_B = -\frac{0.25 PL_B L_c}{2 EI_c}$$

$$\Delta_{beam} = \theta_B L_B + \frac{1}{3} \frac{PL_B^3}{EI_B} = \frac{1}{8} \frac{PL_B^2 L_c}{EI_c} + \frac{1}{3} \frac{PL_B^3}{EI_B}$$

col = W14x246 = $I_c = 1230$ weak axis = 3230 strong axis

Beam = W27x94 = $I_B = 3270$

$L_c = 108$ in

$L_B = 70$ in for 14-3

$L_B = 56$ in for 14-1

$$\frac{E \Delta}{P} = \frac{1}{8} \frac{L_B^2 L_c}{I_c} + \frac{1}{3} \frac{L_B^3}{I_B}$$

$$= \frac{1}{8} \frac{(70)^2 (108)}{1230} + \frac{1}{3} \frac{(70)^3}{3270}$$

$$\frac{E \Delta}{P} = \frac{53.78}{34.42} + \frac{34.964}{17.90} = \frac{88.7448}{52.3212}$$

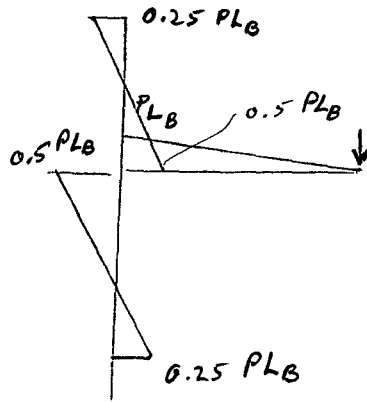
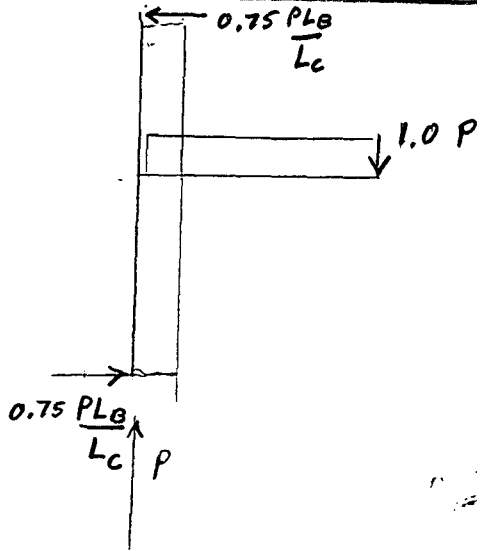
$\Delta = 0.0030601658 P = 0.8874$ in when $P = 290$ for 14-3

$\Delta = 0.0018041808 P = 0.523$ in when $P = 290$ for 14-1

Glenn gets 0.7 in
0.4 in why?

3-15-79

GCD



$$G = \frac{E}{2(1+\nu)}$$

$$= \frac{E}{2.6}$$

Shear Deflection = $K \int_0^L \frac{V_v}{GA} dx$

K = 1.2 rectangle
K = 1.0 WF

$$\Delta = 1.0 \left[\frac{L_B P}{G A_B} \right] + \frac{1.2}{G A_c} \left[L_c \frac{0.75 L_B}{L_c} \frac{0.75 P L_B}{L_c} \right] \times 2$$

$$\frac{G \Delta}{P} = 1.0 \frac{L_B}{A_B} + \frac{2.4}{i_c} \left(\frac{9}{16} \right) \frac{L_c}{A_c} \left(\frac{L_B}{L_c} \right)^2 = 1.0 \frac{L_B}{A_B} + 1.35 \frac{L_c}{A_c} \left(\frac{L_B}{L_c} \right)^2$$

$L_B = 56$ for 14-1

$A_B = 27.7$

or $26.91 \times 0.490 = 13.185$

$L_B = 70$ for 14-3

$A_c = 72.3$

or $15.945 \times 3.626 = 57.82$

$L_c = 108$ in for both

$$\frac{G \Delta}{P} = \frac{1.0(56)}{13.185} + \frac{1.35(108)}{57.82} \left(\frac{56}{108} \right)^2 = 4.247 + 0.677966 = 4.9252$$

$$\Delta = 4.9252 \frac{P}{G} = \frac{4.252(290)}{11,153.8} = 0.128055 \text{ in for 14-1}$$

$$\frac{G \Delta}{P} = \frac{1.0(70)}{13.185} + 1.35 \frac{(108)}{57.82} \left(\frac{70}{108} \right)^2 = 5.309 + 1.05932 = 6.3684$$

$$\Delta = \frac{6.3684 P}{G} = \frac{6.368(290)}{11,153.8} = 0.165578 \text{ in for 14-3}$$

$$\text{Axial} = \frac{P L_c}{A_c E} = \frac{290(108)}{29000(72.3)} = 0.0149377$$

Summary	14-1	14-3
Bending	0.523	0.887
Shear	0.128	0.166
Axial	0.015	0.015
	<u>0.666</u>	<u>1.068</u>