STRUCTURAL DESIGN OF HYDRAULIC PRESS

by

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INTRODUCTION

The structural design of a hydraulic press is the purpose of this problem. This press is to be for use in Western Kansas Industry primarily. Since the industry in this area is devoted largely to the manufacture and prefabrication of metal products, the press will be designed for use in metal working. The author has had two years experience with Western Kansas Industry which has made him aware of the small manufacturer's need for an economical press.

Economics of Problem

Industry in this area is not overly blessed with capital to purchase plant equipment. This puts them at a disadvantage with their big brother industries in the east and other more industrially developed parts of the United States. The cost of a large metal working machine, so necessary to mass production, usually runs into five figures. Therefore, it is desired, if possible, to design a hydraulic press which would cost considerably less than machines already on the market. Most hydraulic presses are manufactured at distances of one thousand miles and over from Western Kansas. This causes the freight charges to increase cost of equipment by a considerable sum to the Western Kansas Manufacturer. Since these machines are required to have a strong frame due to their function, which makes the press very heavy, a saving in freight would result in a considerable saving. At present iron is used for these bulky machine frames. Kansas is not geographically located so the production of such a machine in

Kansas from iron is economically impossible. However, if a material which is available in Kansas, could be substituted for iron, freight costs might be decreased. If such an item were to be manufactured in Kansas an additional reduction in cost of manufacture could be realized, since labor and plant facilities do not cost as much as more industrialized areas of the United States.

Summary of Requirements

a). The cost of machine should be considerably less than present models on the market.

b). If possible the press should be designed in such a manner that it would be feasible to manufacture it in Kansas.

c). Flexibility is to be kept in mind at all phases of design. To make one press be adaptable to as many working operations as possible is of prime importance to the Western Kansas Manufacturer.

d). A press that can be transported relatively easily would be more desirable to the manufacturer.

e). Simplicity of design and operation will be a basic fundamental of the problem.

f). Exactness of operation of press should be carefully weighed against increase in cost for more exact operation of press.

Summary of Tentative Solutions

a). It is believed by the author that it is feasible to construct the press frame of prestressed concrete.

b). The shape of the frame shall be in the form of a C.

c). From previous experience with Western Kansas Industry it is believed that a press of 100 tons capacity would be most desirable.

In a previous design problem, a fixed portal frame was tried for a press of 50 tons capacity. The calculations indicated that the frame was much too awkward, therefore, a C frame is attempted in this 100 tons capacity press. The capacity of 100 tons is believed to be the optimum capacity required by most Western Kansas Manufacturers. Smaller presses can be built from available scrap steel of buildings and bridges that have been salvaged. Larger presses are generally not required except in rare instances and then the capacity would probably not be larger than 500 tons.

DESIGN STRESS ANALYSIS

PLATE I

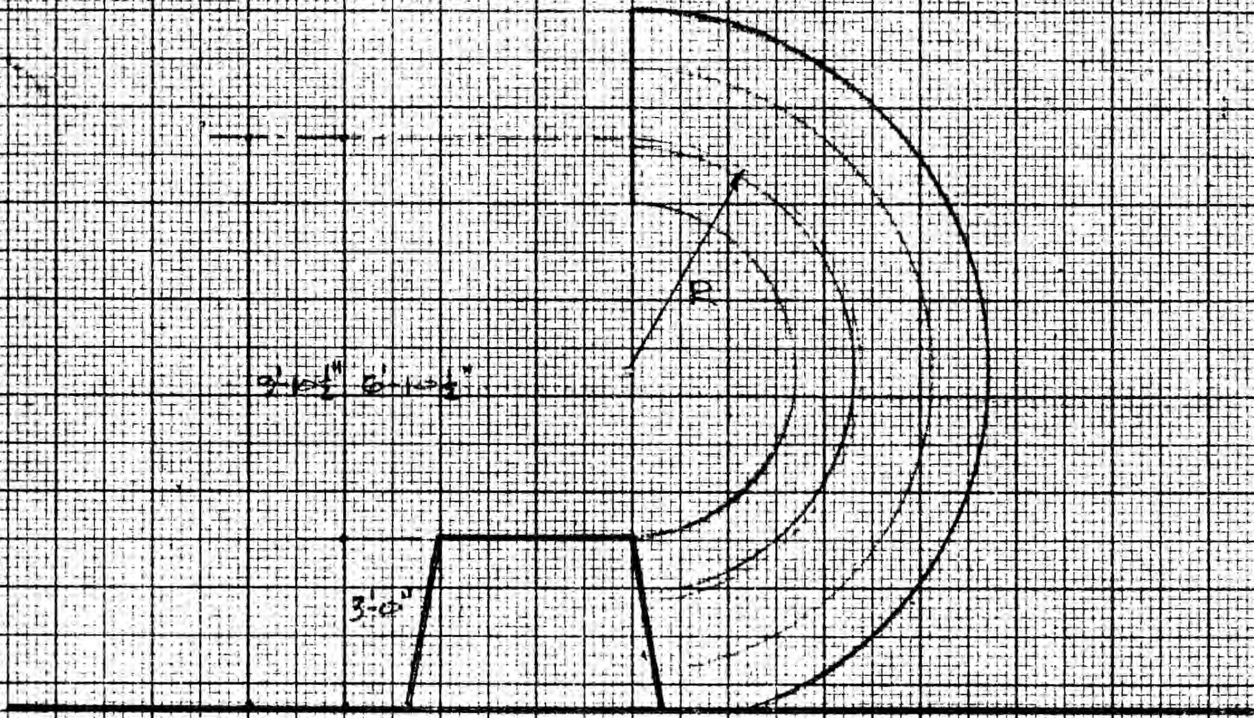


Fig. 1

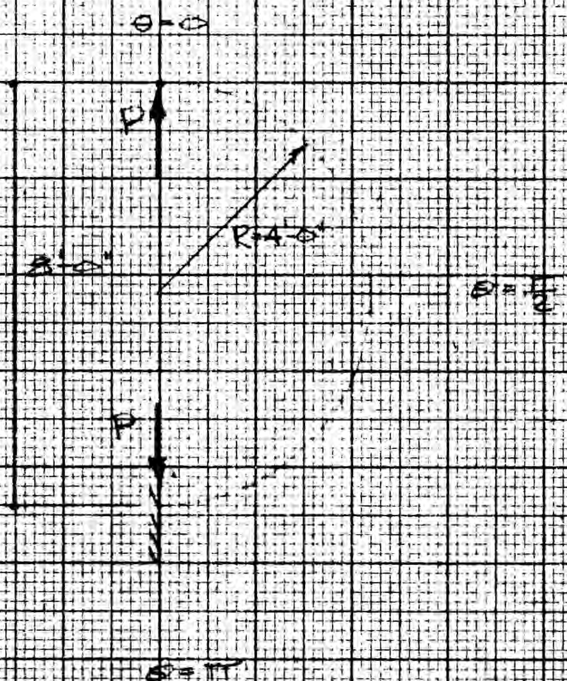


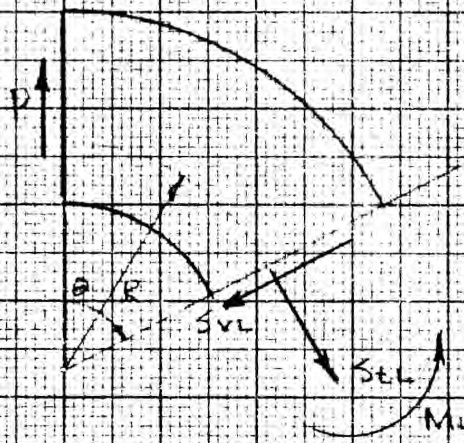
Fig. 2

EXPLANATION OF PLATE II

Fig. 3. Radial stress (S_{rL}) diagram due to live load.

Fig. 4. Tangential stress (S_{tL}) diagram due to live load.

PLATE II



M_L, S_{tL}, S_{rL} are considered positive as shown in diagram.

$S_{rL} = P \cos\theta$

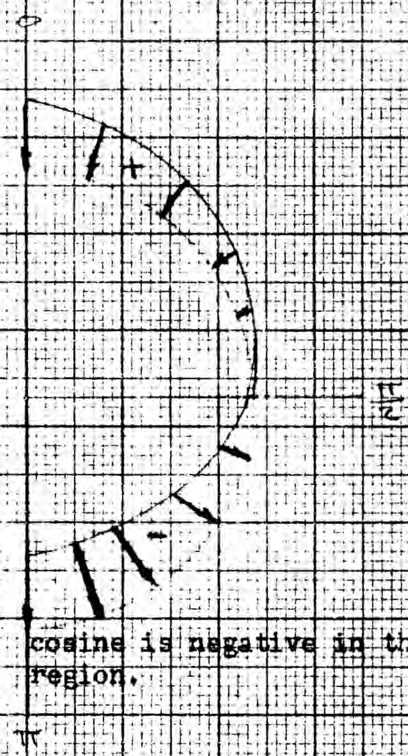
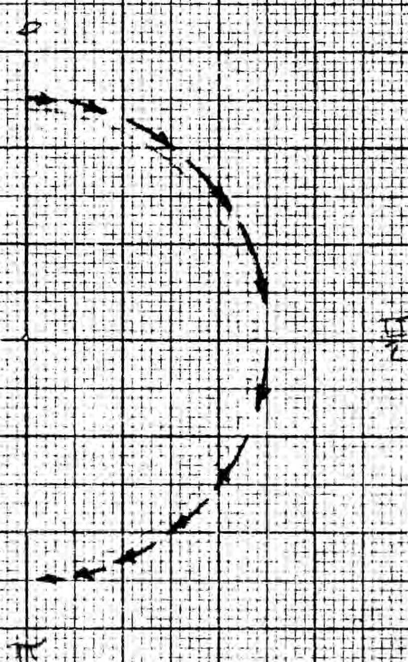


Fig. 3



$S_{tL} = P \sin\theta$

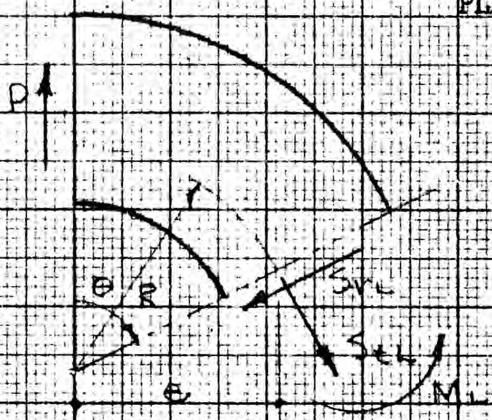
Fig. 4

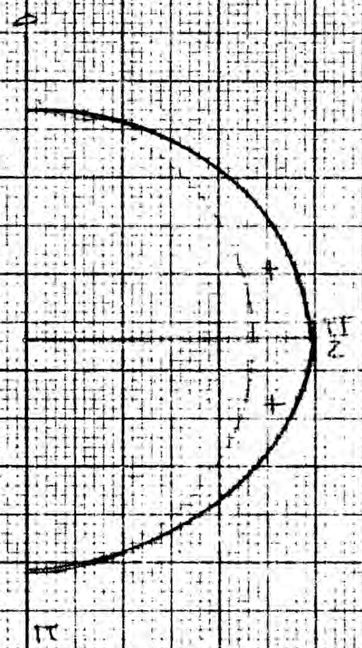
EXPLANATION OF PLATE III

Fig. 5. Moment (M_L) diagram due to live load.

Fig. 6. Radial stress (S_{rD}) diagram due to dead load.

PLATE III



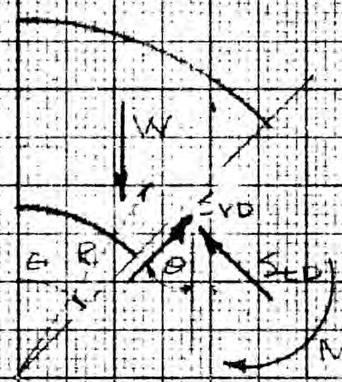


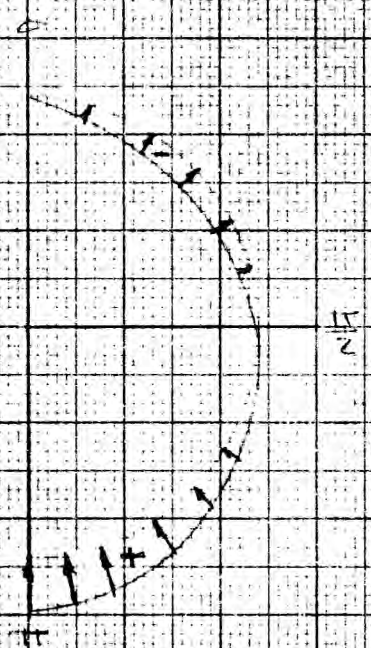
$M_L = Pe$

$e = R\sin\theta$

$M_L = PR\sin\theta$

Fig. 5





$W = wR\theta$; w = #/linear ft.

$$\cos\theta = -\frac{S_{rD}}{W}$$

$$S_{rD} = -W\cos\theta = -wR\theta\cos\theta$$

Fig. 6

EXPLANATION OF PLATE IV

Fig. 7. Tangential stress (S_{tD}) diagram due to dead load.

Fig. 8. Moment diagram (M_D) due to dead load.

PLATE IV

$$S_{tD} = -wR\theta \sin\theta$$

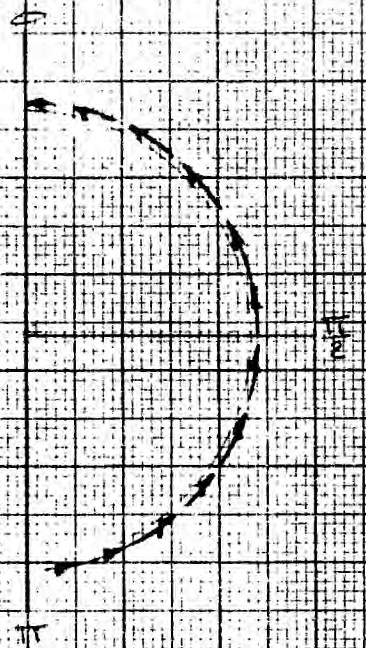
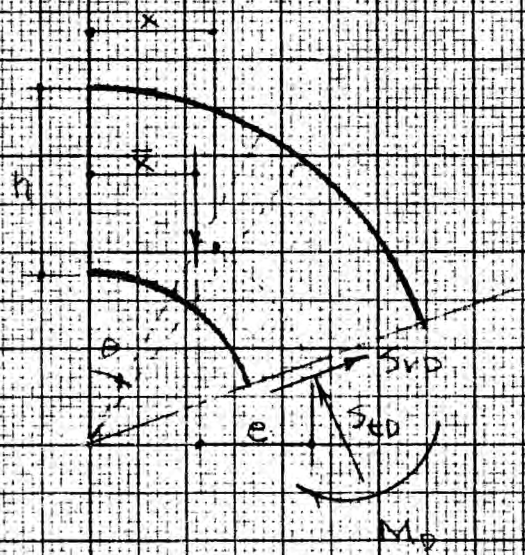
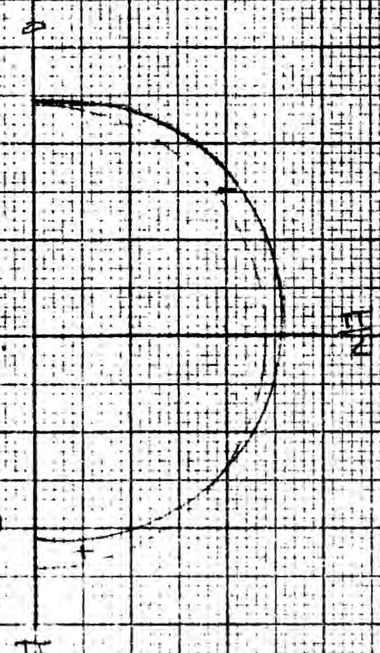


Fig. 7





$$M_D = -We = -wR^2\theta\left(\sin\theta - \frac{1-\cos\theta}{\theta}\right)$$

$$M_D = -wR^2(\theta\sin\theta - 1 + \cos\theta)$$

Fig. 8

Calculation of $\bar{x}$

For pictorial information regarding the following calculations the reader is referred to Fig. 8.

$$A\bar{x} = \int x\,dA$$

$$\int_0^\theta A(\theta)d\theta = \int_0^\theta \frac{(R + h/2)^2}{2}d\theta - \int_0^\theta \frac{(R - h/2)^2}{2}d\theta$$

$$= \frac{(R + h/2)^2}{2}\theta - \frac{(R - h/2)^2}{2}\theta$$

$$= \frac{\theta(R^2 + Rh + h^2/4 - R^2 + Rh - h^2/4)}{2}$$

$$= \theta\, Rh$$

Since $\sin\theta = x/R$, then $x = R\sin\theta$

$$\int_0^\theta xA(\theta)d\theta = \int_0^\theta R\sin\theta \left[\frac{(R + h/2)^2}{2} - \frac{(R - h/2)^2}{2}\right] d\theta$$

$$= R^2h\int_0^\theta \sin\theta d\theta$$

$$= - R^2h \cos\theta \Big|_0^\theta$$

$$= - R^2h \cos\theta + R^2h$$

$$\bar{x} = \frac{\int_0^\theta xA\,(\theta)d\theta}{\int_0^\theta A(\theta)d\theta}$$

$$\bar{x} = \frac{R^2h - R^2h\cos\theta}{\theta Rh}$$

then

$e = R\sin\theta - \bar{x}$, and

$$M_D = -We = - W(R\sin\theta - \bar{x}) = - W(\sin\theta - \frac{1 - \cos\theta}{\theta})\,R$$

Since $W = wR\theta$

$$M_D = -wR^2(\theta\sin\theta - 1 + \cos\theta)$$

ELASTIC DESIGN OF SECTION FOR FLEXURE

The elastic design of the press frame will be used instead of ultimate design because this frame will be under repeated loading throughout its lifetime. Therefore, fatigue of member may be a factor and it is believed elastic design will give the maximum safety against this type of failure. Also no tensile stress in the concrete will be designed for, if possible, for the same reason.

Preliminary Design of Frame

A method for determining the approximate depth of section is as follows.

$$h = k\sqrt{M}$$

h = approximate depth of beam in inches.

M = maximum bending moment in k.-ft.

k = a coefficient varying from 1.5 to 2.0.

Approximate design moment.

$$M = 100T(2k/T)(4.0ft.) = 800 \text{ k. -ft.}$$

then

$$h = 1.5\sqrt{800} = 42.3 \text{ in.}$$

Design Criteria

Assume the following characteristics of the materials to be used in the press frame.

$$f'_{ci} = 7{,}000psi$$

$f_c = 0.55f'_{ci} = 0.55(7{,}000) = 3{,}850psi$ for compression of post tensioned concrete members.

EXPLANATION OF PLATE V

Fig. 9. Preliminary design of member section.

Fig. 10. (a) Section properties. (b) Just after transfer, C at bottom kern point. (c) Under working load, C at top kern point.

PLATE V

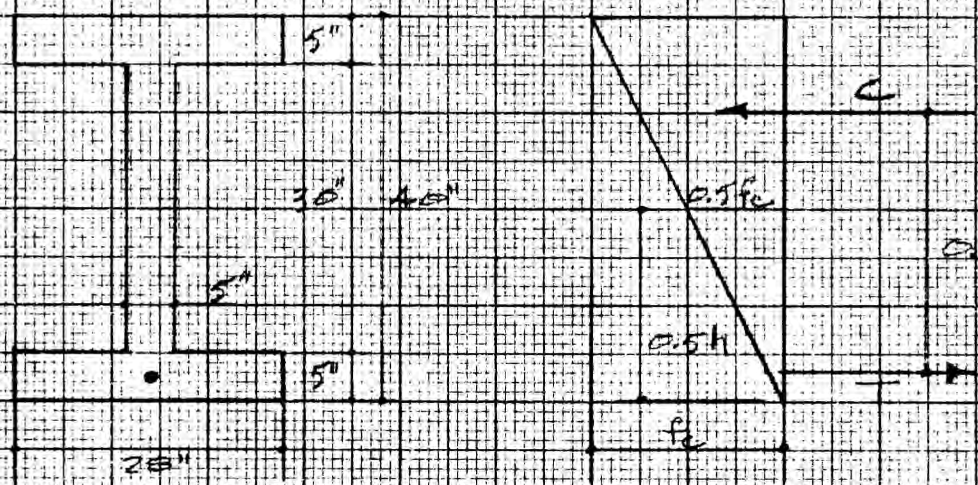


Fig. 9

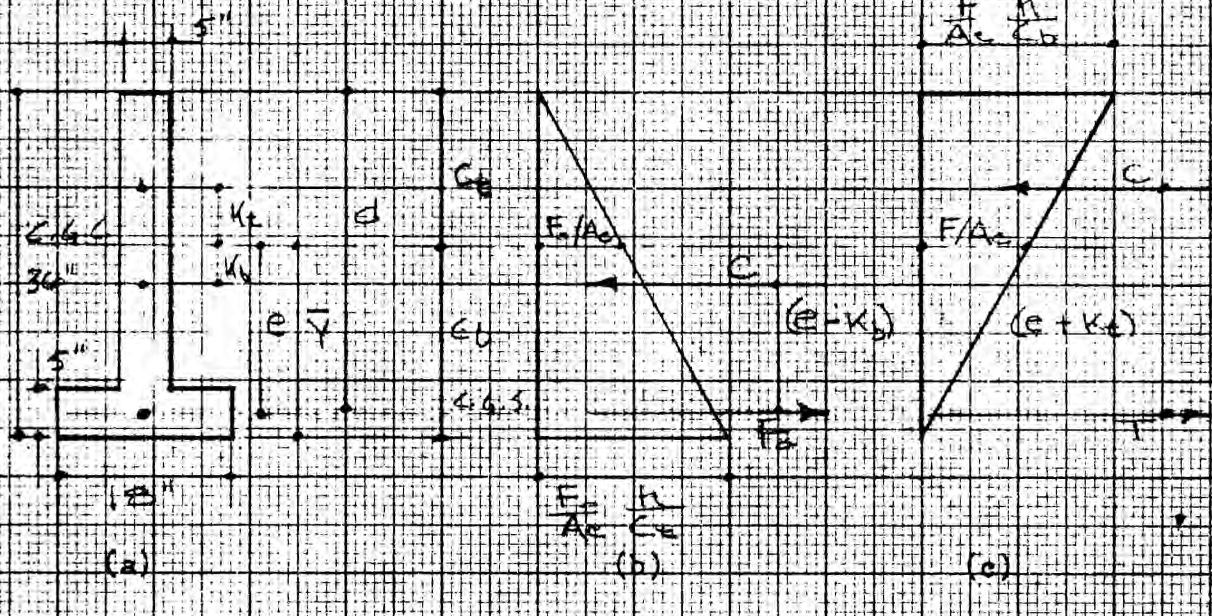


Fig. 10

$f_s' = 220{,}000$psi

$f_s = 0.70 f_s' = 0.70(220{,}000) = 154{,}000$psi

Assumed total prestress loss will be 15%.

Design Calculations

A symmetrical section will be assumed for the initial trial with the properties as indicated in figure 9.

$A_c = 2(5)(28)(5)(30)$

$= 430$ sq. ins.

$w = \dfrac{430(150)}{144} - 448$ lb./linear ft. of beam

$M_T = M_L + M_D$

$= PR\sin\Theta - wR^2(\Theta\sin\Theta - 1 + \cos\Theta)$

At Θ equal to $\pi/2$,

$M_T = 200(4)(\sin\pi/2) - 0.448(4)^2(\pi/2\ \sin\pi/2 - 1 + \cos\pi/2)$

$= 800 - 0.448(16)(\pi/2 - 1 + 0)$

$= 796$ ft.-k.

Under working load, the lever arm for the internal couple averages about 65% of the overall height h. However, this approximation applies to beams of standard construction and the structure under consideration deviates somewhat from the normal. In any case the design will proceed as if beam is standard and where alternations, as to design procedure, are necessary they will be attempted. Hence the effective prestress F will be.

$$F = T = \frac{M_T}{0.65h} = \frac{796(12)}{0.65(40)} = 368 \text{ k.}$$

$A_s = F/f_s = 368/154 = 2.39$ sq. ins.

The total prestress $A_s f_s$ is the force C on the section. This force will produce an average unit stress on the concrete of

$$C/A_c = T/A_c = A_s f_s/A_c$$

For preliminary design, this average stress can be assumed to be about 50% of the maximum allowable stress f_c, under the working load. Hence

$$A_s f_s/A_c = 0.50 f_c$$

$$A_c = \frac{A_s f_s}{0.50 f_c}$$

$$A_c = \frac{368}{0.50(3.850)} = 191 \text{ sq. ins. required.}$$

A_c required tends to indicate section assumed is somewhat too large. A revision of section is in order and since the M_D/M_T ratio will be small, an inverted T section will be assumed. An inverted T section is more practical because there is a great deal of material to resist the initial prestress. For properties of inverted T section see figure 10.

$$A_c = 5(31)(5)(18) = 245 \text{ sq. ins.}$$

$$w = \frac{245(150)}{144} = 255 \text{ lb./linear ft.}$$

$$M_T = 800 - 0.255(16)(0.57)$$

$$= 798 \text{ ft.-k.}$$

$$F = \frac{798(12)}{0.65(36)} = 409 \text{ k.}$$

$$A_s = 409/154 = 2.66 \text{ sq. ins.}$$

Elastic Design, No Tension In Concrete

$\sum M_{Bottom} = 0$

$$A\bar{y} = \int y\,dA$$

$$\bar{y} = \frac{5(36)(18) + 5(13)(2.5)}{245} = \frac{3,240 + 162.5}{245}$$

$$= 14.0 \text{ in.}$$

$$I = \frac{5(13)^3}{12} + 5(31)(6.5)^2 + \frac{18(5)^3}{12} + 5(18)(11.5)^2$$

$$= 12,500 + 6,560 + 187.5 + 11,900$$

$$= 31,148 \text{ in.}^4$$

$$K_t = \frac{I}{Ac_b} = \frac{31,148}{245(22)} = 5.78 \text{ in.}$$

$$K_b = \frac{I}{Ac_t} = \frac{31,148}{245(14)} = 9.1 \text{ in.}$$

When M_D/M_T is small, c.g.s. is located outside the kern just as much as the M_D will allow. (Fig. 10)

$$e - K_b = M_D/F_o; \quad F = 409 \text{ k.}$$

Assuming 15% prestress loss.

$$F_o = 409 + 0.15(409) = 470.4\text{k.}$$

$$e - K_b = 2.32/470.4 = 0.00493 \text{ in.}$$

Since $e - K_b$ is so small and K_b does not reach to the centroid of the bottom flange, the section will need to be revised.

Calculations for Revised Section (Fig. 11)

$$\bar{y} = \frac{12(20)(6) + 4\ (16)(20) + 6\ (12)(34)}{6(12) + 4(16) + 12(20)}$$

EXPLANATION OF PLATE IV

Fig. 11. Revised section properties.

Fig. 12. Revised section properties.

Fig. 13. Shearing and normal stresses in bending.

PLATE VI

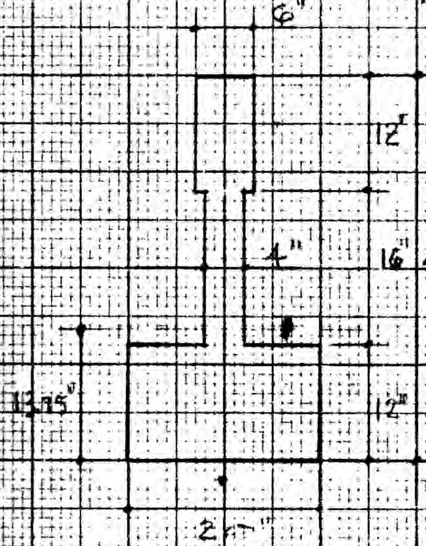


Fig. 11

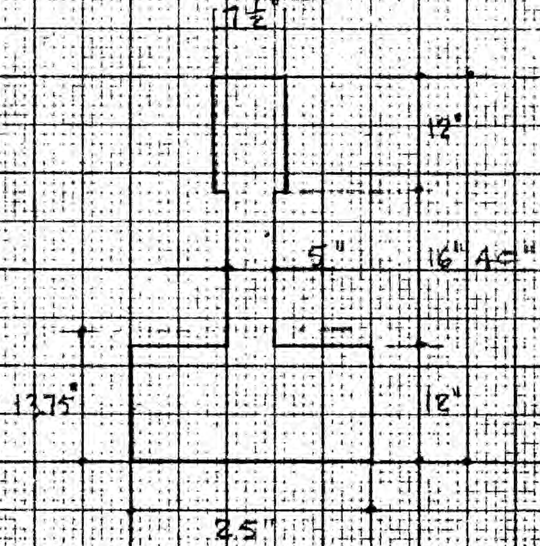


Fig. 12

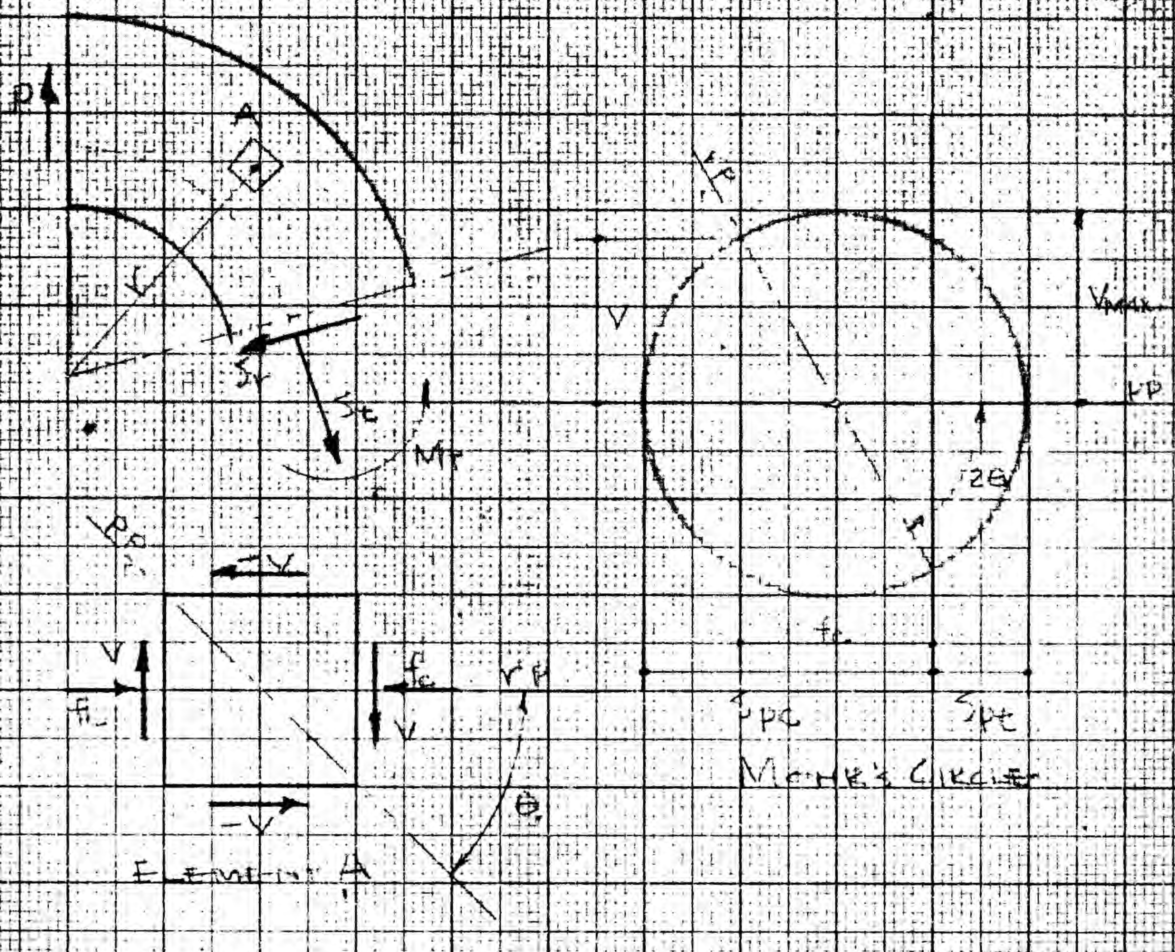


Fig. 13

$$\bar{y} = \frac{1440 + 1280 + 2450}{72 + 64 + 240}$$

$$= \frac{5170}{376} = 13.75 \text{ ins.}$$

$$I = \frac{20(12)^3}{12} + 12(20)(7.75)^2 + \frac{4(16)^3}{12} + 4(16)(6.25)^2 + \frac{6(12)^3}{12}$$

$$+ 6(12)(20.25)^2$$

$$= 2,880 + 14,400 + 1,370 + 2,500 + 864 + 29,700$$

$$= 51,714 \text{ in.}^4$$

$$K_b = \frac{I}{Ac_t} = \frac{51,714}{376(26.25)} = 5.24 \text{ ins.}$$

$$c_t = 40 - 13.75 = 26.25 \text{ ins.}$$

Locating the steel at the bottom kern point will make, $e = K_b = 5.25$ which will be the location of c.g.s.

$$K_t = \frac{I}{Ac_b} = \frac{51,714}{376(13.75)} = 10 \text{ ins.}$$

Then effective prestress required is computed as

$$F = \frac{M_T}{e + K_t}$$

$$M_T = M_L + M_D; \quad w = \frac{376(150)}{144} = 392 \text{ lb./ft.}$$

$$= 800 - 0.392(16)(0.57)$$

$$= 800 - 3.57$$

$$= 796.4 \text{ ft.-k.}$$

$$F = \frac{796.4(12)}{5.25 + 10} = \frac{9550}{15.25} = 626 \text{ k.}$$

$$F_o = 626 + 0.15(626) = 626 + 94 = 720\text{k.}$$

The required concrete area under initial prestress is

$$A_c = \frac{F_0 h}{f_b c_t} = \frac{720(40)}{3,850(26.25)} = 285 \text{ sq.-in. required.}$$

The required concrete area under effective prestress after load is applied is.

$$A_c = \frac{Fh}{f_t c_b} = \frac{626(40)}{3.850(13.75)} = 473 \text{ sq. in. required.}$$

Actually A_c = 376 sq. in., therefore section will need to be revised again.

Calculations for Revised Section (Fig.12)

$$A_c = 25(12) + 5(16) + 12(7.5)$$

$$= 300 + 80 + 90 = 470 \text{ sq. in.}$$

$$w = \frac{470(150)}{144} = 490 \text{ lb./linear ft.}$$

$$M_T = 800 - 0.490(16)(0.57) = 800 - 4.47 = 795.5 \text{ ft.-k.}$$

$$\bar{y} = \frac{12(25)(6) + 5\,(16)(20) + 12(7.5)(34)}{470}$$

$$= \frac{1800 + 1600 + 3060}{470} = \frac{6,460}{470} = 13.75 \text{ in.}$$

$$I = \frac{25(12)^3}{12} + 25(12)(7.75)^2 + \frac{5(16)^3}{12} + 5(16)(6.25)^2 + \frac{7.5(12)^3}{12}$$

$$+ 7.5(12)(20.25)^2$$

$$= 3,600 + 18,000 + 1,710 + 3,140 + 108 + 37,100$$

$$= 63,658 \text{ in.}^4$$

$$K_b = \frac{I}{Ac_t} = \frac{63,658}{470(26.25)} = 5.16 \text{ ins.}$$

$$K_b = e$$

$$K_t = \frac{I}{A_c c_b} = \frac{63,658}{470(13.75)} = 9.85 \text{ ins.}$$

$$F = \frac{M_T}{e + K_t} = \frac{795.5(12)}{5.16 + 9.85} = 636 \text{ k.}$$

$$F_o = 636 + 0.15(636) = 636 + 95.4 = 731.4 \text{ k.}$$

Required A_c under initial prestress

$$A_c = \frac{F_o h}{f_b c_t} = \frac{731.4(40)}{3.85(26.25)} = 289 \text{ sq.-in.}$$

Required A_c under effective prestress and working load is

$$A_c = \frac{Fh}{f_t c_b} = \frac{636(40)}{3.85(13.75)} = 480 \text{ sq.-in.}$$

This is near enough to the actual area since some steel will be placed in the top side of the C frame to resist tensile stresses there during unloaded condition. These tensile stresses will exist on the top side in spite of the c.g.s. being located at K_b because of the physical shape of the C frame which causes M_D to be a negative moment rather than positive as in a normal loading condition. Also additional concrete area will be added when the fillets are added to the web near the flanges. This has been neglected in the previous calculations.

Steel Required

Using Roebling system of 7 - wire galvanized strands for post tensioning, $E_s = 25 \times 10^6$ psi.

Diameter of cable = 1 1/8 in.

$A_s = 0.751$ sq.-in.

Ultimate Strength = 156 k.

Design Load = 90 k.

Number of cables required $= \frac{F}{90} = \frac{636}{90} = 7.06$ cables

$$= \frac{F_o}{156} = \frac{731.4}{156} = 4.7 \text{ cables}$$

Shear Calculations

Some point where f_c is diminished, will often yield a higher principal tensile stress even though v is not maximum. Since we are concerned with the maximum principal tensile stress (S_{pt}) as indicated in Figure 13, rather than v, we will look for the value Θ which will give minimum f_c at this phase of design.

For I sections and T sections the junction of the web and the flange is often the critical point for maximum S_{pt}.

F = 636 k. for all values of Θ, approximately.

A_c = 470 sq. in.

c = 5.16 in.

I = 63,658 in.^{4}

S_t will help minimize f_c when it is maximum.

S_t = max. at $\Theta\ \frac{\pi}{2}$

$$= S_{tL} + S_{tD}$$

$$= P \sin\Theta - wR\Theta \sin\Theta$$

$$= P(1) - wR(\pi/2)(1)$$

$$= P - \frac{\pi}{2}wR$$

$P = 200\text{k.}$

$w = 0.490 \text{ k./ft.}$

$R = 4 \text{ ft.}$

$S_t = 200\text{k.} - \frac{\pi}{2}(0.490)(4)$

$= 200 - 3.08$

$= 197\text{k.}$

$V = S_r = S_{rL} + S_{rD} = 0 \text{ at } \Theta = \pi/2$

$M = M_L + M_D = PR \sin\Theta - wR^2(\Theta\sin\Theta - 1 + \cos\Theta)$

$= 200(4)(1) - 0.490(16)(0.570)$

$= 800 - 4.47$

$= 795 \text{ k.-ft.}$

Principal diagonal tension will be calculated at section where bottom flange connects to web and $\Theta = \pi/2$.

$f_c = - F/A + S_t/A \pm Fec/I \pm Mc/I$

$v = \frac{VQ}{Ib}$

$f_c = - 636{,}000/470 + 197{,}000/470 - 636{,}000(5.16)(1.75)/63{,}658$

$- 795{,}000(12)(1.75)/63{,}658$

$= - 1{,}355 + 420 - 90.2 - 262$

$= - 1{,}287 \text{ psi.}$

$v = 0$

$S_{pt} = \sqrt{v^2 \quad (f_c/2)^2} - f_c/2$

$= \sqrt{(643.5)^2} - 643.5$

$= 0$

Calculate S_{pt} when $\theta = 0$

$S_t = 0$

$M = 0$

$V = P \cos\theta = 200k.$

$f_c = - 1,355$ psi.

$$v = \frac{VQ}{Ib}$$

$$= \frac{200,000(12)(25)(7.75)}{63,658(5)}$$

$$= 1,462 \text{ psi.}$$

$S_{pt} = 1,462$ psi.

The above calculations indicate that in the final design diagonal tension will not be serious except at the end block where transverse tension will be somewhat serious. This will be taken into account when the final design is made.

REVIEW LOSS OF PRESTRESS

Since the hydraulic press frame being designed is to be factory produced, it will be assumed there will be no loss of prestress due to elastic shortening. The reasons are: First, the member is to be post-tensioned, therefore, elastic shortening will occur during jacking. Second, all cables will be jacked simultaneously, since it will be assumed the factory has the facilities to accomplish this economically. Also it will be mass produced product and it would be feasible for a manufacturer to invest in enough jacks to accomplish this.

The loss of prestress due to creep and shrinkage of concrete will also be attempted to be nullified by the following procedure. The member is to be post-tensioned, therefore, the tensioning will not be completed until after the shrinkage of the concrete has taken place. As to creep, the jacks will remain on the cables until such time as a greater percentage of creep has resulted. This time will need to be determined by experiment. Most data used to calculate the creep of concrete depends upon the creep coefficient (C_c) which is taken at the emperical value of 215.

For balancing the loss due to creep in the steel, the steel is to be overtensioned 5 - 10% and held there for 2 to 3 minutes. This practice will also help balance the friction losses in this member. These friction losses will undoubtedly be great due to curvature of frame.

The loss of prestress due to anchorage take up will be assumed as follows. The average value of anchorage device slippage is 0.1 in. For bearing anchorages, the heads and nuts will deform about 0.03 in. on the average. (9) The loss due to anchorage take up $\Delta f_s = \frac{\Delta a E_s}{L}$. (9)

$$\Delta f_{sa} = \Delta a E_s / L$$

$$\Delta a = 0.03 + 0.1 = 0.13 \text{ in.}$$

$$E_s = 25 \times 10^6 \text{ psi.}$$

$$L = 2\pi r/2 = \pi(48) = 151 \text{ in.}$$

$$\Delta f_{sa} = \frac{0.13(25)(10)^6}{151} = 21{,}500 \text{ psi.}$$

The frictional loss of prestress will now be taken into consideration. The overtensioning of the cables, as mentioned in the previous paragraph, is limited by the yield point, the creep limit, and the strength of the cable. It will be necessary to see if these limits will allow enough overtensioning to overcome frictional losses. Also the friction in the jacking system and anchorage system will cause the stress in the tendon to be less than that indicated on the pressure gauges. A loss of prestress due to curvature effect will result from friction and intended curvature of the tendons in addition to the wobble of the duct. This loss is dependent upon the coefficient of friction between the contact materials and the pressure exerted by the tendon on the concrete or ducts. The coefficient of friction (μ), in turn, depends on the smoothness and nature of the surfaces in contact, the amount and nature of lubricants, and sometimes the length of contact. The wobble effect of the duct is expressed by K, giving the loss of prestress per foot of length. A μ of 0.55 will be assumed since it is about the highest value of the determined coefficients. (9) Use of K of 0.0020 for a similar reason.

$$f_2 = f_1 e^{-\mu\theta - KL} \text{ (Unit stress at L)}$$

$$f_1 = \frac{f_2}{e^{-\mu\theta - KL}} = f_2 e^{\mu\theta + KL} \text{(Unit stress at jacking end)}$$

When $\theta = \pi/2$ and $L = r\theta = 4(\pi/2) = 6.29$ ft.

$$f_2 = F/A_s = \frac{636,000}{0.751(7)} = 121,000 \text{ psi.}$$

$$f_1 = 121,000\ e^{0.55(\pi/2) + 0.0020(6.29)}$$

$$\Delta f_{sf} = 291,000 - 121,000 = 170,000 \text{ psi.}$$

The minimum guaranteed ultimate strength of the Roebling System with a 1 1/8" cable being used is 156,000 lb.

$$f_s = \frac{156,000}{0.751} = 208,000 \text{ psi. guaranteed.}$$

Therefore, since $\Delta f_{sf} = 170,000$, the tendon may be overtensioned enough to reverse the total frictional loss of prestress. Then the final initial prestress will be $f_2 + \Delta f_{sa}$.

$$f_2 + \Delta f_{sa} = 121,000 + 21,500 = 142,500 \text{ psi.}$$

$$\text{\% of Prestress Loss} = \frac{21,500}{142,500} = 0.151 = 15.1\%$$ which is near the assumed value of 15% in the design calculations.

CALCULATION OF F AS A FUNCTION OF (θ)

If it were not for the loss of prestress due to the factors mentioned in the previous section, F would be constant as a function of θ. However, as a result of these factors, it decreases as θ increases.

At $\theta = 0$

$$F_2 = 731.4\text{k.}$$

At $\theta = \pi/2$

$$F_2 = 636 \text{ k.}$$

At $\theta = \pi$

$$F_2 = F_{1}e^{-\mu(\theta - \pi/2) - KL}$$

$$= \frac{636}{e^{0.55(\ \ /2) + 0.0020(6.29)}} = \frac{636}{e^{0.8766}}$$

$$= \frac{636}{2.41} = 264 \text{ k.}$$

EXPLANATION OF PLATE VII

Fig. 14. Prestress (F) as a function of (θ).

PLATE VII

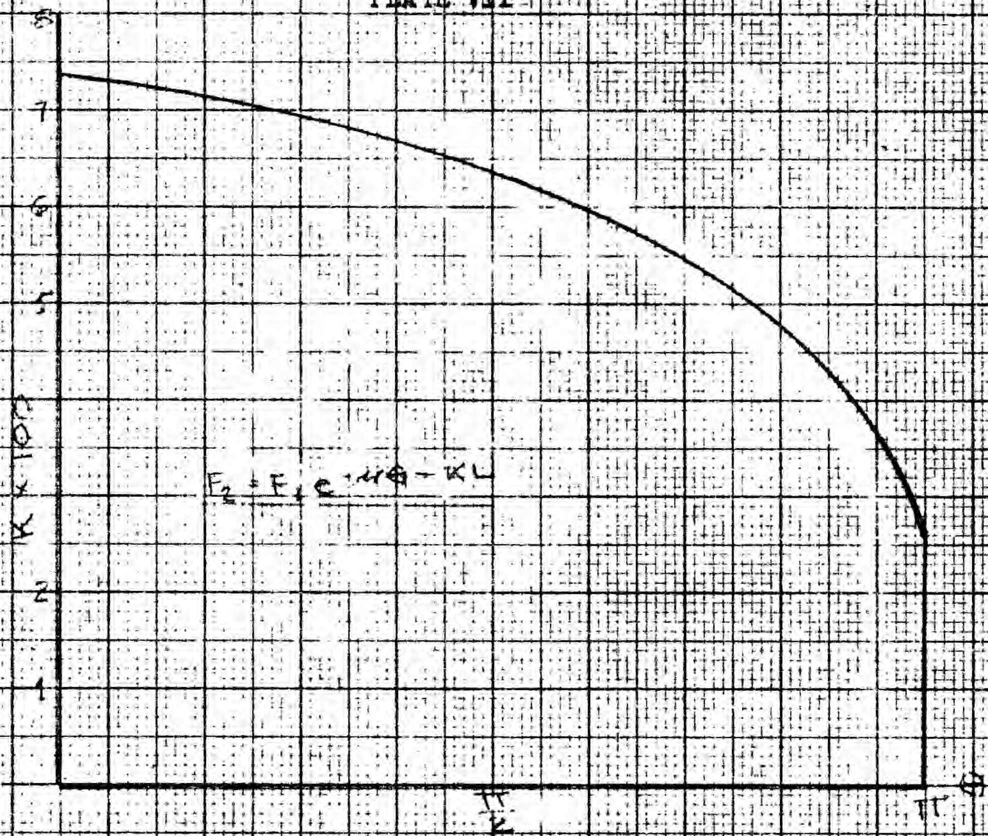


Fig. 14

REVIEW STRESS ANALYSIS

Table 1. Compiled data for calculation of stresses and moments.

θ (Deg.)	θ (Rad. x π)	S_{rL} (P)	S_{tL} (P)	M_L (PR)	S_{rD} (wR)	S_{tD} (wR)	M_D (wR^2)
0	0.000	1.000	0.000	0.000	-0.000	0.000	0.000
10	0.056	0.985	0.174	0.174	-0.172	-0.030	-0.015
20	0.111	0.940	0.342	0.342	-0.328	-0.119	-0.059
30	0.167	0.866	0.500	0.500	-0.454	-0.267	-0.128
40	0.222	0.766	0.644	0.644	-0.535	-0.448	-0.214
50	0.278	0.644	0.766	0.766	-0.563	-0.669	-0.313
60	0.334	0.500	0.866	0.866	-0.525	-0.909	-0.409
70	0.389	0.342	0.940	0.940	-0.418	-1.150	-0.492
80	0.445	0.174	0.985	0.985	-0.243	-1.380	-0.554
90	0.500	0.000	1.000	1.000	0.000	-1.570	-0.570
100	0.556	-0.174	0.985	0.985	0.304	-1.720	-0.546
110	0.611	-0.342	0.940	0.940	0.656	-1.805	-0.463
120	0.668	-0.500	0.866	0.866	1.050	-1.815	-0.315
130	0.722	-0.644	0.766	0.766	1.460	-1.740	-0.076
140	0.778	-0.766	0.644	0.644	1.870	-1.575	0.191
150	0.834	-0.866	0.500	0.500	2.270	-1.310	0.556
160	0.889	-0.940	0.342	0.342	2.620	-0.955	0.985
170	0.945	-0.985	0.174	0.174	2.921	-0.516	1.469
180	1.000	-1.000	0.000	0.000	3.141	0.000	2.000

EXPLANATION OF PLATE VIII

Fig. 15. Radial stress (S_{rL}), due to live load.

Fig. 16. Radial stress (S_{rD}), due to dead load.

PLATE VIII

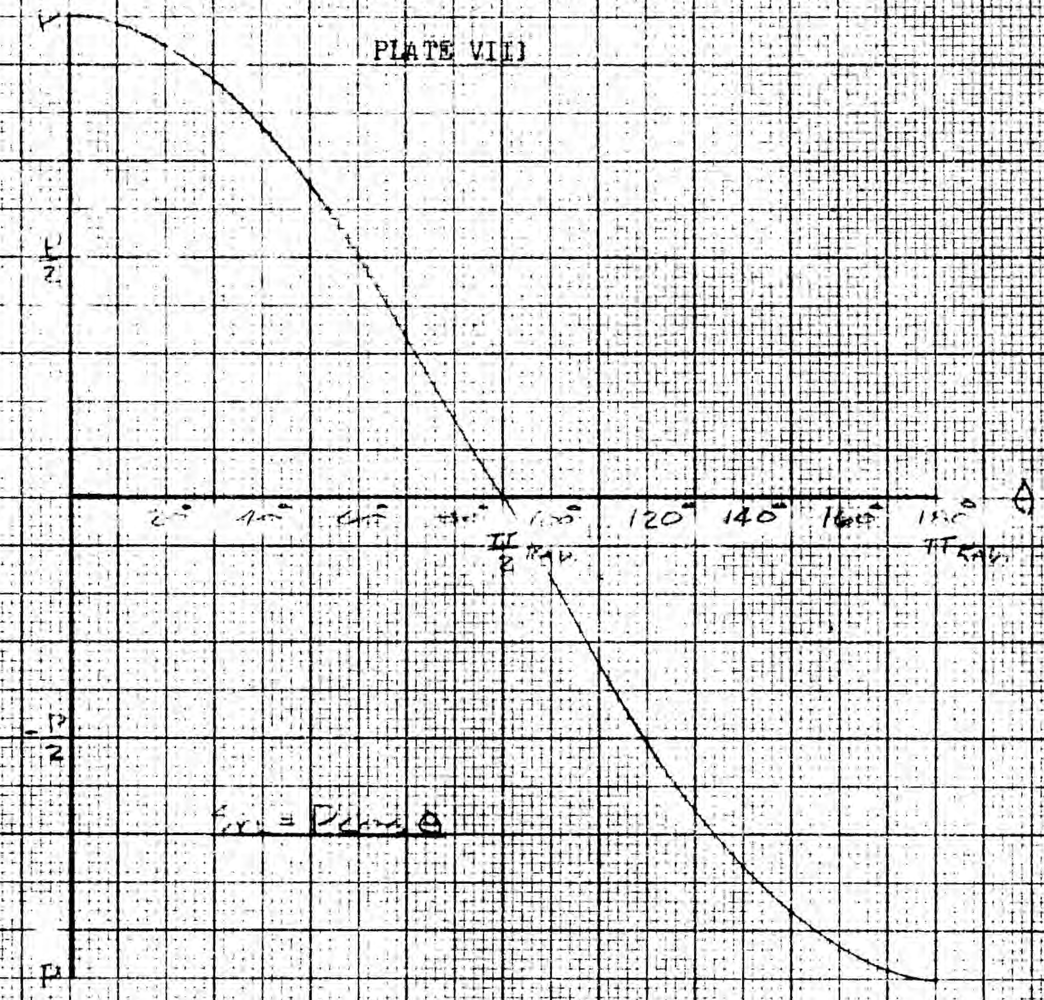


Fig. 15

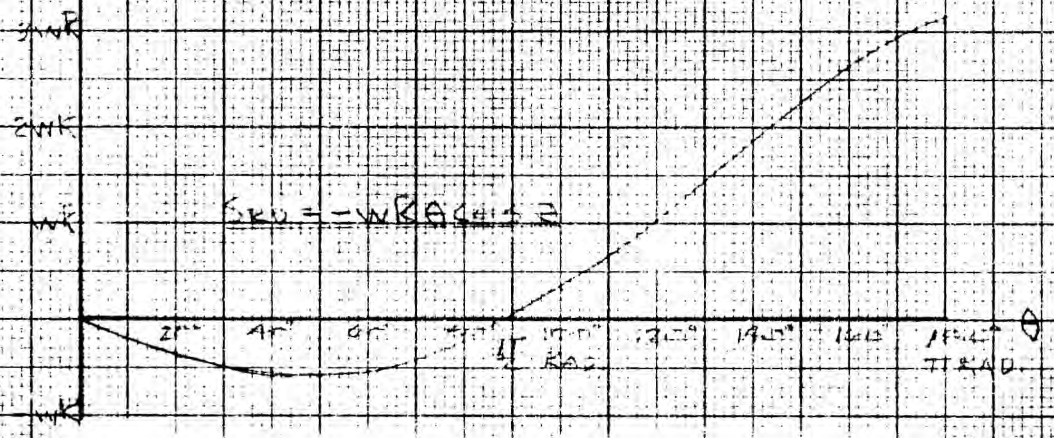


Fig. 16

EXPLANATION OF PLATE IX

Fig. 17. Tangential stress (S_{tL}), due to live load.

Fig. 18. Tangential stress (S_{tD}), due to dead load.

PLATE IX

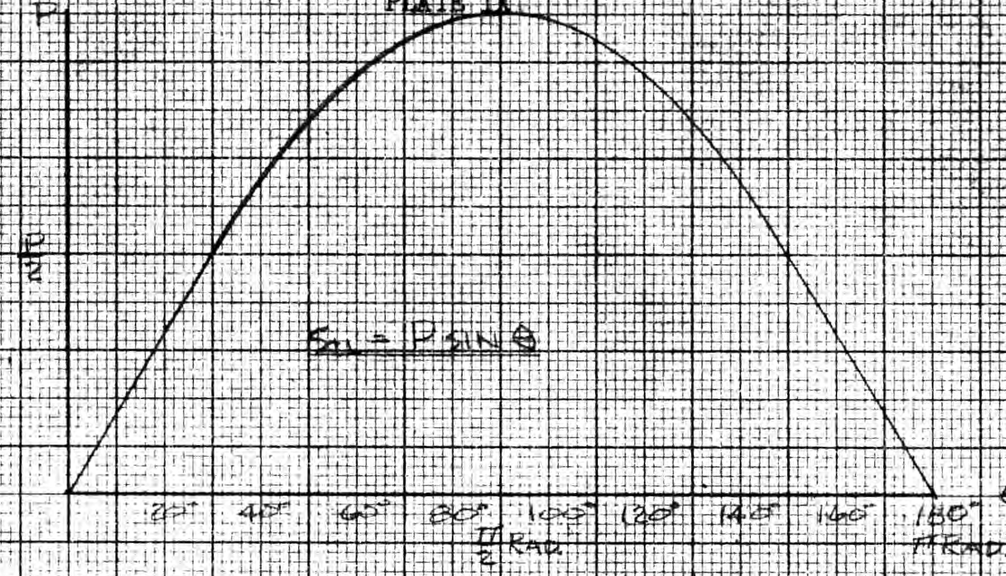


Fig. 17

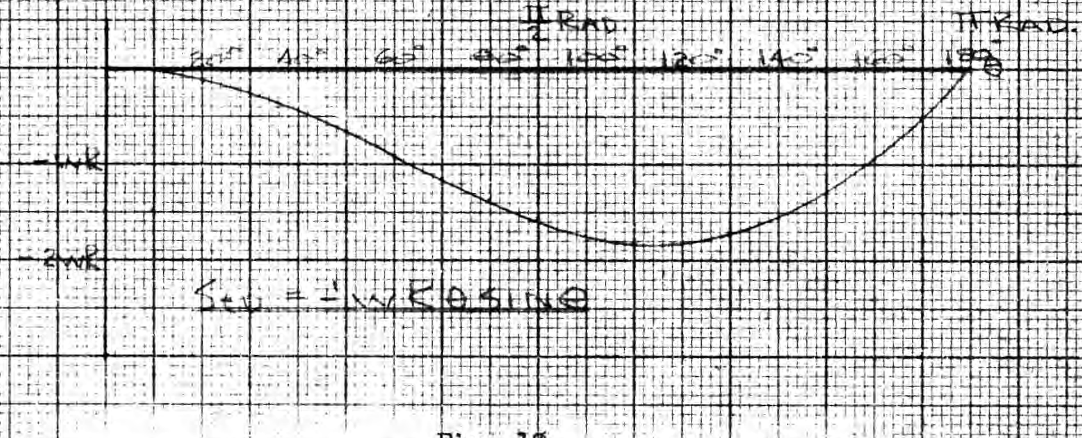


Fig. 18

EXPLANATION OF PLATE X

Fig. 19. Moment (M_L), due to live load.

Fig. 20. Moment (M_D), due to dead load.

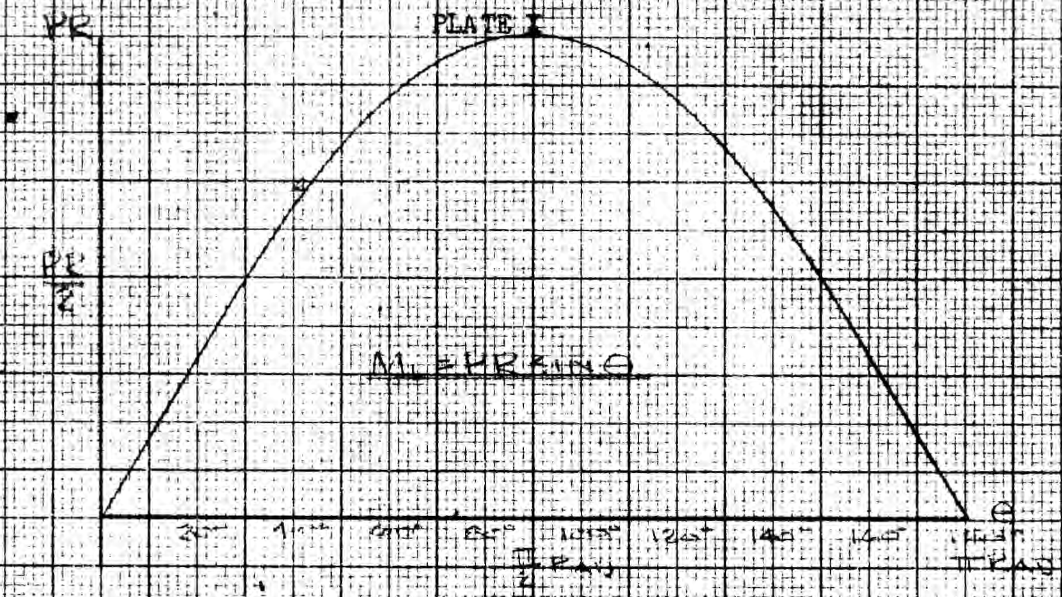


PLATE I
HR
HR/2
$M_1 = HR \sin\theta$
θ
π/2 RAD
π RAD

Fig. 19

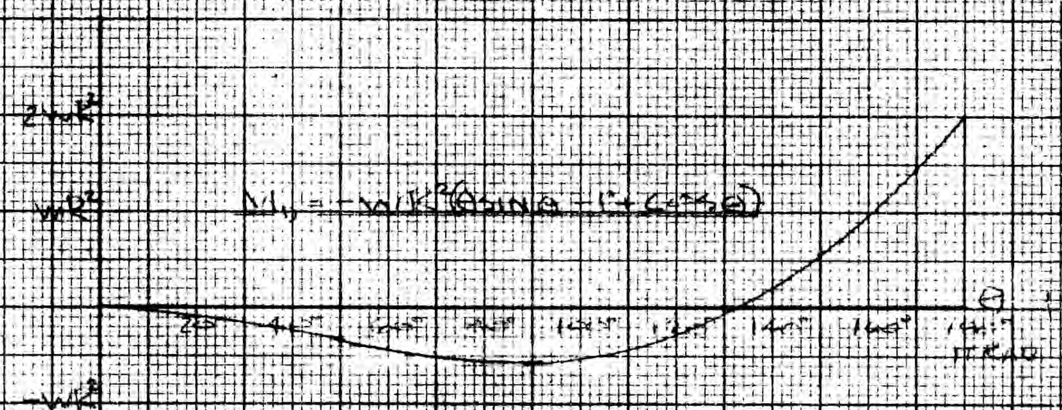

$2wR^2$
wR^2
$-wR^2$
$M_1 = -wR^2(\theta \sin\theta - 1 + \cos\theta)$
θ
π RAD

Fig. 20

BENDING STRESSES IN CURVED BEAMS

Assumptions:

1. The center line, joining the centroids of the cross sections of the bar, is a plane curve and that the cross sections have an axis of symmetry in this plane.

2. Transverse cross sections of the beam, originally plane and normal to the center of the beam, remain so after bending.

3. Lateral pressure between the longitudinal fibers is negligible.*

4. Also the usual assumptions of equilibrium, Hooke's Law is valid, homogenous prismatical beam, etc. While it is realized the particular application in this problem will be concrete the calculations will proceed on the elastic theory. Any revisions due to the plasticity of concrete will be made at a later phase.

Using ϵ = strain, E = modulus of elasticity, s = stress and M = moment, also see Figure 21. Since

$$E = s/\epsilon \; ; \quad s = \epsilon E$$

and $$\epsilon = \frac{y d\phi}{(R - n + y)d\theta}$$

then $$s = \left[\frac{y d\phi}{(R - n + y)d\theta}\right]E \qquad \text{Equation 1 (a)}$$

For equilibrium

$$\int_A s\,da = 0 = \frac{E d\phi}{d\theta}\int_A \frac{y\,da}{(R - n + y)}$$

also $$M = \int_A y s\,da - \frac{E d\phi}{d\theta}\int_A \frac{y^2\,da}{(R - n + y)}$$

* The exact theory shows that there is a certain radial pressure but that it has no substantial effect on the stress and can be neglected.

y can be expressed a $y = (R - n + y) - (R - n)$, then

$$M = \frac{Ed\mathbb{Q}}{d\theta}\int_A y \left[\frac{(R - n + y - (R - n)}{(R - n + y)}\right] da$$

$$= \frac{Ed\mathbb{Q}}{d\theta}\int_A yda - (R - n)\frac{Ed\mathbb{Q}}{d\theta}\int_A \frac{yda}{(R - n + y)}$$

since $\frac{Ed\mathbb{Q}}{d\theta}\int_A \frac{yda}{(R - n + y)} = \int_A sda = 0$

The intergral $\int_A yda = nA$ where n is the distance from the c.g. to N.A.

then $M = EnA \frac{d\mathbb{Q}}{d\theta}$

and $\frac{Ed\mathbb{Q}}{d\theta} = \frac{M}{nA}$

from equation 1 (a)

$$s = \frac{My}{nA(R - n + y)} \qquad \text{Equation 1 (b)}$$

In order to calculate s, n must be known which will be solved as follows. From previously $\int_A sda = 0 = \frac{Ed\mathbb{Q}}{d\theta}\int_A \frac{yda}{(R - n + y)} = \int_A \frac{ybdy}{(R - n + y)}$

The limits of the intergral will be $(R - h/2)$ and $(R + h/2)$.

since the $\int_A \frac{ybdy}{(R - n + y)}$ is an improper fraction, it can be reduced by division to a mixed form, consiting of the sum of a polynomial and a proper fraction.

$$\int_A \frac{ybdy}{(R - n + y)} = b\int_A dy - b(R - n)\int_A \frac{dy}{(R - n + y)}$$

and by intergration and applying limits

$$\Big[by - b(R - n)\ln(R - n + y)\Big]_{R - h/2}^{R + h/2} = 0$$

$$b(R + h/2 - R + h/2) - b(R - n)\Big[\ln(R + h/2) - \ln(R - h/2)\Big] = 0$$

$$h - (R - h)\ln\frac{R + h/2}{R - h/2} = 0$$

EXPLANATION OF PLATE XI

Fig. 21. Curved beam and curved beam cross section notation.

Fig. 22. Physical characteristics of composite curved beam cross section.

PLATE XI

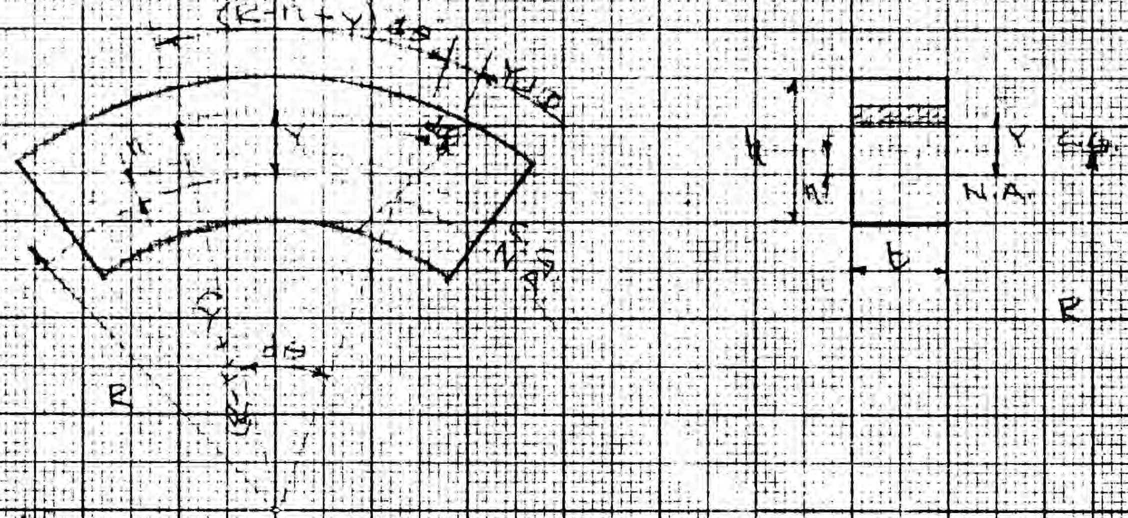


Fig. 21

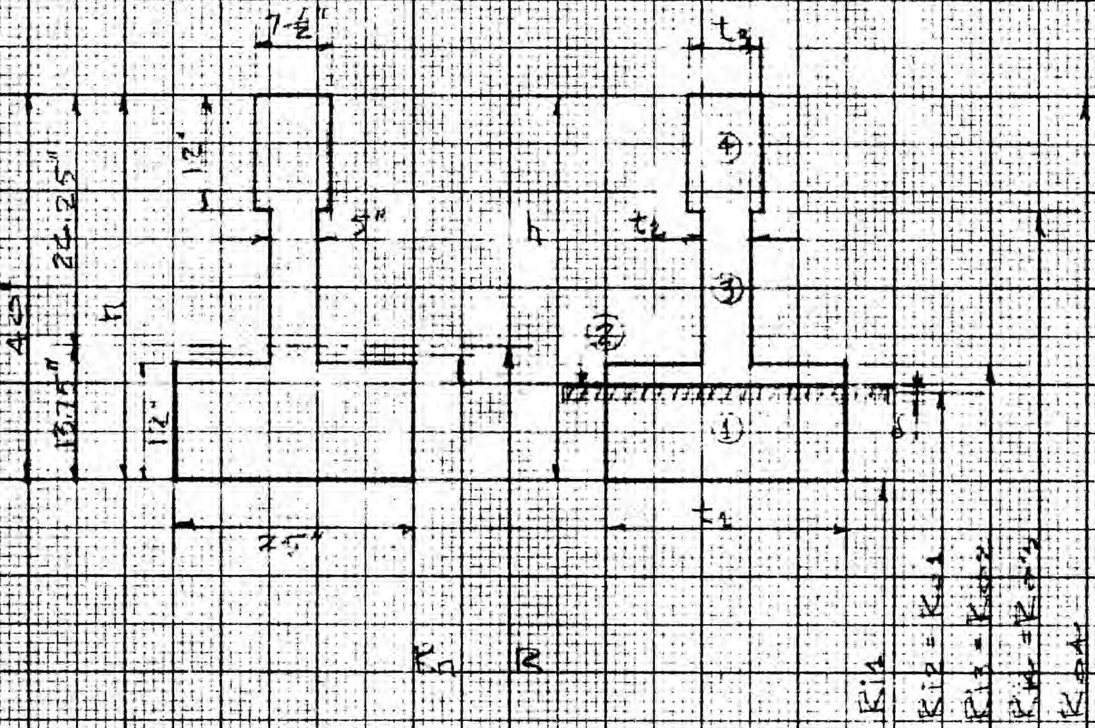


Fig. 22

$$h - (R - n)\ln \frac{R + h/2}{R - h/2}$$

$$n = R - \frac{h}{\ln \frac{R + h/2}{R - h/2}} \qquad \text{Equation 2}$$

This is the value of n for a rectangular cross section. For a composite section, R_n is (12)

$$R_n - \frac{A}{\ln \left[\left(\frac{R_{o1}}{R_{i1}}\right)^{t_1} \left(\frac{R_{o2}}{R_{i2}}\right)^{t_2} \left(\frac{R_{o3}}{R_{i3}}\right)^{t_3} \left(\frac{R_{o4}}{R_{i4}}\right)^{t_4} \cdots \left(\frac{R_{on}}{R_{in}}\right)^{t_n} \right]}$$

$$A = A_c + A_t; \quad A_t = n_c A_s; \quad n_c = \frac{E_s}{E_c} = \frac{25 \times 10^6}{7 \times 10^6} = 3.57$$

$$A_t = 3.57(0.751)(7) = 18.75 \text{ sq. in.}$$

$$A = 470 + 18.75 = 489 \text{ sq. in.}$$

The ducts for cables have been neglected in these calculations, however, the fillets at the junction of flange and web also have been neglected. It is believed these two omissions will approximately counter balance each other.

$$R_{o4} = 48 + 26.25 = 74.25 \text{ in.}$$

$$R_{o3} = R_{i4} = 74.25 - 12 = 62.25 \text{ in.}$$

$$R_{o1} = R_{i3} = 74.25 - 28 = 46.25 \text{ in.}$$

$$R_{i1} = 74.25 - 40 = 34.25 \text{ in.}$$

$$R_{o2} = R_{i2} = R - e = 48 - 5.16 = 42.84 \text{ in.}$$

$$R_n = \frac{A}{\ln \left[\left(\frac{46.25}{34.25}\right)^{25} \left(\frac{42.84}{42.84}\right)^{\infty} \left(\frac{62.25}{46.25}\right)^{5} \left(\frac{74.25}{62.25}\right)^{7.5} \right]}$$

$$= \frac{A}{\ln \left[(1.350)^{25} \quad (1)^{\infty} \quad (1.345)^{5} \quad (1.193)^{7.5} \right]}$$

$$R_n = \frac{489}{\ln(1,779 \times 4.410 \times 3.759)} = \frac{489}{\ln(29,420)} = \frac{489}{10.289} = 47.5 \text{ in.}$$

This indicates that the neutral axis, due to the frame being curved, has shifted 1/2 inch towards the center of curvature from the c.g.c. Stress in a curved beam is modified due to curvature by K_o or K_i.

$$f_o = K_o M c_o; \; K_o = \frac{I}{R_o A\bar{y}}$$

$$f_i = \frac{K_i M c_i}{I}; \; K_i = \frac{I}{R_i A\bar{y}}$$

f_c as a function of θ and the radius is

$$f_c(\theta, r_o) = F/A + \frac{S_t(\theta)}{A} \pm \frac{Fec_o}{R_o A\bar{y}} \pm \frac{Mc_o}{R_o A\bar{y}}$$

$$f_c(\theta, r_i) = F/A + \frac{S_t(\theta)}{A} \pm \frac{Fec_i}{R_i A\bar{y}} \pm \frac{Mc_i}{R_i A\bar{y}}$$

Calculation of c.g.c. using transformed section.

$$\bar{y} = \frac{25(12)(6) + 18.75(8.59) + 5(16)(20) + 12(7.5)(34)}{489}$$

$$= \frac{1800 + 161.2 + 1600 + 3060}{489}$$

$$= \frac{6,621.2}{489} = 13.5 \text{ in.}$$

when $\theta = 0^o$ and $r_i = R - \bar{y} = 48 - 13.5 = 34.5$ and

$$S_T = S_{tL} + S_{tD} = 0; \; M = M_L + M_D = 0$$

Under working load

$$f_c = -\frac{734}{489} + 0 - \frac{734(5.16)(13.5)}{34.5(489)(13.5)} + 0 = -1.5 - 0.225$$

$$= 1.725 \text{ k/in.}^2 = -1,725 \text{ psi.}$$

At transfer of prestress

$$f_c = -1,725 \text{ psi.}$$

when $\theta = 20^{o}$ and $r_i = 34.5$ and under working load

$$S_T = S_{tL} + S_{tD} = 0.35(200) - 0.12(0.490)(4)$$

$$= 70.00 - 0.235 - 69.77 \text{ k.}$$

$$M = M_L + M_D = 0.35(200)(4) - 0.1(0.490)(16)$$

$$= 280 - 0.784 - 279.2 \text{ ft.-k.}$$

$$f_c = - \frac{720}{489} + \frac{69.77}{489} - \frac{720(5.16)}{34.5(489)} + \frac{279.2(12)}{34.5(489)}$$

$$= - 1.472 + 0.1425 - 0.221 + 0.1982$$

$$= - 1.352 = - 1,352 \text{ psi.}$$

At transfer of prestress

$$f_c = - 1.472 - \frac{0.235}{489} - 0.221 - \frac{0.784(12)}{34.5(489)}$$

$$= - 1.472 - 0.001 - 0.221 - 0.001$$

$$= - 1.694 = - 1,694 \text{ psi.}$$

Since the remaining calculations made by the author are repetitious, only their answers will be indicated for reference.

when $\theta = 40^{o}$ and $r_i = 34.5$ and under working load

$$f_c = - 1,024 \text{ psi.}$$

At transfer of prestress

$$f_c = - 1,659 \text{ psi.}$$

when $\theta = 60^{o}$ and $r_i = 34.5$ and under working load

$$f_c = - 754 \text{ psi.}$$

At transfer of prestress

$$f_c = - 1,634 \text{ psi.}$$

when $\theta = 90^{o}$ and $r_i = 34.5$ and under working load

$$f = - 527 \text{ psi.}$$

At transfer of prestress

$f_c = - 1{,}504$ psi.

When $\theta = 120^{\circ}$, $r_i = 34.5$ and under working load

$f_c = - 499$ psi.

At transfer of prestress

$f_c = - 1{,}348$ psi

When $\theta = 140^{\circ}$, $r_i = 34.5$ and under working load

$f_c = - 581$ psi.

At transfer of prestress

$f_c = - 1{,}216$ psi.

When $\theta = 160^{\circ}$, $r_i = 34.5$ and under working load

$f_c = - 678$ psi.

At transfer of prestress

$f_c = - 1{,}009$ psi.

When $\theta = 180^{\circ}$, $r_i = 34.5$ and under working load

$f_c = - 600$ psi.

At transfer of prestress

$f_c = - 600$ psi.

When $\theta = 0^{\circ}$, $r_i = R - 1.5 = 48 - 1.5 = 46.5$ and under working load

$f_c = - 1{,}519$ psi.

At transfer of prestress

$f_c = - 1{,}519$ psi.

When $\theta = 20^{\circ}$, $r_i = 46.5$ and under working load

$f_c = - 1{,}331$ psi.

At transfer of prestress

$f_c = - 1{,}490$ psi.

When $\theta = 40^{o}$, $r_i = 46.5$ and under working load

$f_c = -\ 1{,}164$ psi.

At transfer of prestress

$f_c = -\ 1{,}460$ psi.

When $\theta = 60^{o}$, $r_i = 46.5$ and under working load

$f_c = -\ 1{,}015$ psi.

At transfer of prestress

$f_c = -\ 1{,}411$ psi.

When $\theta = 90^{o}$, $r_i = 46.5$ and under working load

$f_c = -\ 867$ psi.

At transfer of prestress

$f_c = -\ 1{,}322$ psi.

When $\theta = 120^{o}$, $r_i = 46.5$ and under working load

$f_c = -\ 790$ psi.

At transfer of prestress

$f_c = -\ 1{,}187$ psi.

When $\theta = 140^{o}$, $r_i = 46.5$ and under working load

$f_c = -\ 775$ psi.

At transfer of prestress

$f_c = -\ 1{,}071$ psi.

When $\theta = 160^{o}$, $r_i = 46.5$ and under working load

$f_c = -\ 741$ psi.

At transfer of prestress

$f_c = -\ 895$ psi.

When $\theta = 180^{o}$, $r_i = 46.5$ and under working load

$f_c = -\ 529$ psi.

At transfer of prestress

$f_c = -\ 529$ psi.

When $\theta = 0^{o}$, $r_i = R_n = 47.5$ and under working load

$f_c = -\ 1{,}500$ psi.

At transfer of prestress

$f_c = -\ 1{,}500$ psi.

When $\theta = 20^{o}$, $r_i = 47.5$ and under working load

$f_c = -\ 1{,}330$ psi.

At transfer of prestress

$f_c = -\ 1{,}473$ psi.

When $\theta = 40^{o}$, $r_i = 47.5$ and under working load

$f_c = -\ 1{,}176$ psi.

At transfer of prestress

$f_c = -\ 1{,}442$ psi.

When $\theta = 60^{o}$, $r_i = 47.5$ and under working load

$f_c = -\ 1{,}038$ psi.

At transfer of prestress

$f_c = -\ 1{,}394$ psi.

When $\theta = 90^{o}$, $r_i = 47.5$ and under working load

$f_c = -\ 898$ psi.

At transfer of prestress

$f_c = -\ 1{,}306$ psi.

When $\theta = 120^{o}$, $r_i = 47.5$ and under working load

$f_c = -\ 816$ psi.

At transfer of prestress

$f_c = -\ 1{,}172$ psi.

When $\theta = 140^{o}$, $r_i = 47.5$ and under working load

$f_c = - 793$ psi.

At transfer of prestress

$f_c = - 1,058$ psi.

When $\theta = 160^{o}$, $r_i = 47.5$ and under working load

$f_c = - 745$ psi.

At transfer of prestress

$f_c = - 884$ psi.

When $\theta = 180^{o}$, $r_i = 47.5$ and under working load

$f_c = - 532$ psi.

At transfer of prestress

$f_c = - 532$ psi.

When $\theta = 0^{o}$, $r_o = R + 26.5 - 12 = 62.50$ and under working load

$f_c = - 1,358$ psi.

At transfer of prestress

$f_c = - 1,358$ psi.

When $\theta = 20^{o}$, $r_o = 62.5$ and under working load

$f_c = - 1,316$ psi.

At transfer of prestress

$f_c = - 1,333$ psi.

When $\theta = 40^{o}$, $r_o = 62.5$ and under working load

$f_c = - 1,273$ psi.

At transfer of prestress

$f_c = - 1,305$ psi.

When $\theta = 60^{\circ}$, $r_o = 62.5$ and under working load

$f_c = -1{,}218$ psi.

At transfer of prestress

$f_c = -1{,}262$ psi.

When $\theta = 90^{\circ}$, $r_o = 62.5$ and under working load

$f_c = -1{,}134$ psi.

At transfer of prestress

$f_c = -1{,}183$ psi.

When $\theta = 120^{\circ}$, $r_o = 62.5$ and under working load

$f_c = -1{,}017$ psi.

At transfer of prestress

$f_c = -1{,}061$ psi.

When $\theta = 140^{\circ}$, $r_o = 62.5$ and under working load

$f_c = -925$ psi.

At transfer of prestress

$f_c = -955$ psi.

When $\theta = 160^{\circ}$, $r_o = 62.5$ and under working load

$f_c = -788$ psi.

At transfer of prestress

$f_c = -803$ psi.

When $\theta = 180^{\circ}$, $r_o = 62.5$

$f_c = 522$ psi.

At transfer of prestress

$f_c = -552$ psi.

When $\theta = 0^{\circ}$, $r_o = 74.5$ and under working load

f_c = - 1,348 psi.

At transfer of prestress

f_c = - 1,348 psi.

When θ = 20°, r_o = 74.5 and under working load

f_c = -1,315 psi.

At transfer of prestress

f_c = - 1,324 psi.

When θ = 40°, r_o = 74.5 and under working load

f_c = - 1,280 psi.

At transfer of prestress

f_c = - 1,296 psi.

When θ = 60°, r_o = 74.5 and under working load

f_c = - 1,231 psi.

At transfer of prestress

f_c = - 1,253 psi.

When θ = 90°, r_o = 74.5 and under working load

f_c = - 1,150 psi.

At transfer of prestress

f_c = - 1,174 psi.

When θ = 120°, r_o = 74.5 and under working load

f_c = - 918 psi.

At transfer of prestress

f_c = - 1,054 psi.

When θ = 140°, r_o = 74.5 and under working load

f_c = - 937 psi.

At transfer of prestress

$f_c = -651$ psi.

When $\theta = 160^{\circ}$, $r_o = 74.5$ and under working load

$f_c = -791$ psi.

At transfer of prestress

$f_c = -795$ psi.

When $\theta = 180^{\circ}$, $r_o = 74.5$ and under working load

$f_c = -486$ psi.

At transfer of prestress

$f_c = -486$ psi.

Shearing Stress (Conventional Theory)

$v = \frac{VQ}{Ib}$; $V = S_r = S_{rL} + S_{rD}$

When $\theta = 0^{\circ}$ and $r_i = 46.5$

$V = 200$ k.

$v = \frac{200(12)(25)(7.5)}{63,658(5)} = 200(0.007) = 1.414$ k./sq.in. $= 1,414$ psi.

When $\theta = 20^{\circ}$, $v = 1,322$ psi.

When $\theta = 40^{\circ}$, $v = 1,080$ psi.

When $\theta = 60^{\circ}$, $v = 700$ psi.

When $\theta = 90^{\circ}$, $v = 0$ psi.

When $\theta = 120^{\circ}$, $v = -693$ psi.

When $\theta = 140^{\circ}$, $v = -1,055$ psi.

When $\theta = 160^{\circ}$, $v = -1,260$ psi.

When $\theta = 180^{\circ}$, $v = -1,370$ psi.

EXPLANATION OF PLATE XII

Fig. 23. Diagram of f_c when r_i = 34.5 inches.

Fig. 24. Diagram of f_c when r_i = 46.5 inches.

Fig. 25. Diagram of f_c when r_i = 47.5 inches.

PLATE XII

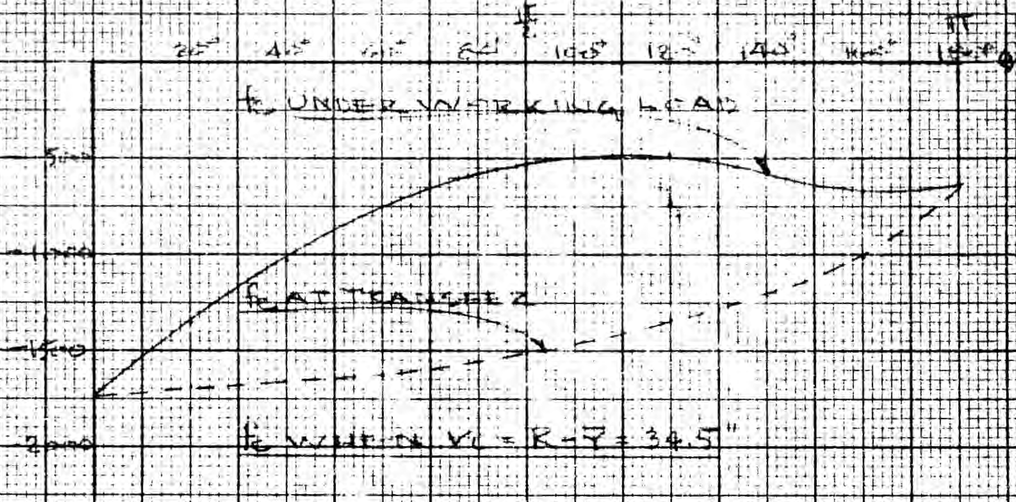


Fig. 23

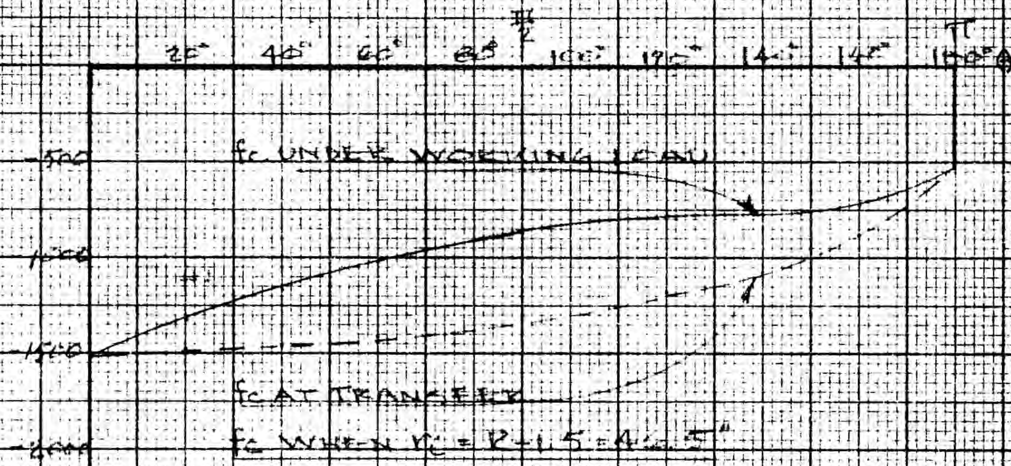


Fig. 24

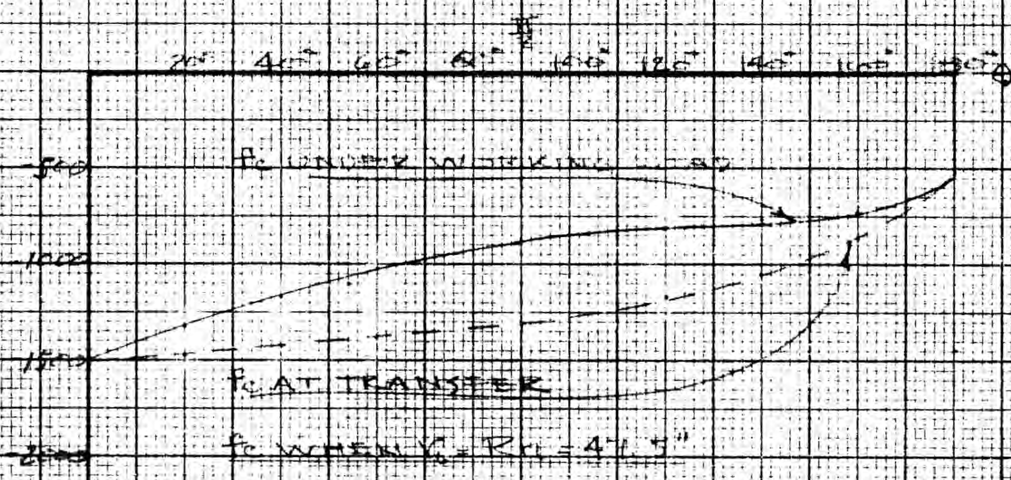


Fig. 25

EXPLANATION OF PLATE XIII

Fig. 26. Diagram of f_c when r_o = 62.5 inches.

Fig. 27. Diagram of f_c when r_o = 74.5 inches.

PLATE XIII

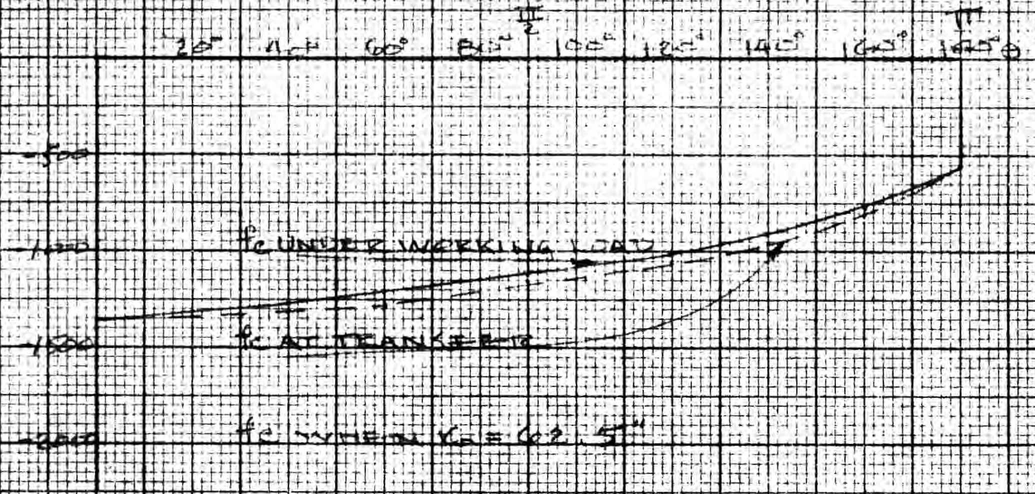


Fig. 26

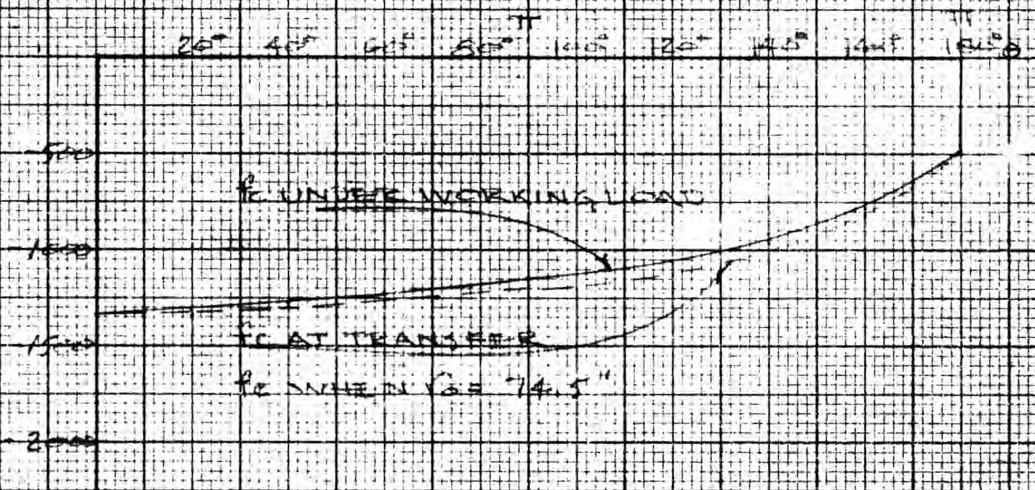


Fig. 27.

When $\theta = 0^{o}$ and $r_o = 62.5$

$v = 1{,}160$ psi.

When $\theta = 20^{o}$, $v = 1{,}089$ psi.

When $\theta = 40^{o}$, $v = 886$ psi.

When $\theta = 60^{o}$, $v = 574$ psi.

When $\theta = 90^{o}$, $v = 0$ psi.

When $\theta = 120^{o}$, $v = -570$ psi.

When $\theta = 140^{o}$, $v = -868$ psi.

When $\theta = 160^{o}$, $v = -1{,}035$ psi.

When $\theta = 180^{o}$, $v = -1{,}125$ psi.

Principal Tensile Stresses (S_{pt})

Conventional design for shear in prestressed concrete beams is based on the computation of the principal tensile stress in the beam rather than the shear. Since concrete beams are more likely to fail in tension than shear. Since the principal tensile stress does not occur when v is maximum but at some point where f_c is diminished. Therefore, the value for f_c calculated by the curved beam theory will be used in the calculation of S_{pt} rather than the straight beam theory since these values are smaller and probably more accurate. $v(\theta)$ is maximum when $r_i = 46.5''$.

When $\theta = 0^{o}$

$$S_{pt} = \sqrt{(1.414)^2 + (1500/2)^2} - 1500/2$$

$$= 1600 - 750 = 850 \text{ psi.}$$

When $\theta = 20^{o}$, $S_{pt} = 814$ psi.

When $\theta = 40^{o}$, $S_{pt} = 650$ psi.

When $\theta = 60^{o}$, $S_{pt} = 360$ psi.

EXPLANATION OF PLATE XIV

Fig. 28. Diagram of v when r_i = 46.5 inches.

Fig. 29. Diagram of v when r_o = 62.5 inches.

Fig. 30. Diagram of S_{pt} when r_i = 46.5 inches.

PLATE XIV

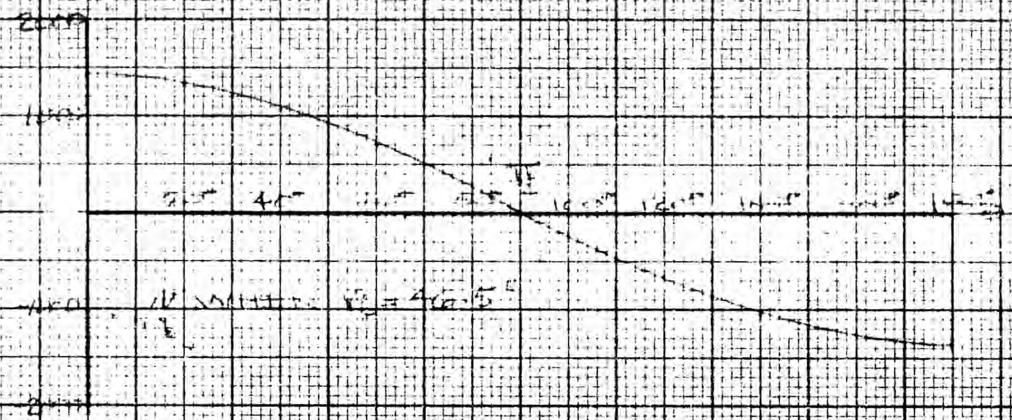


Fig. 28

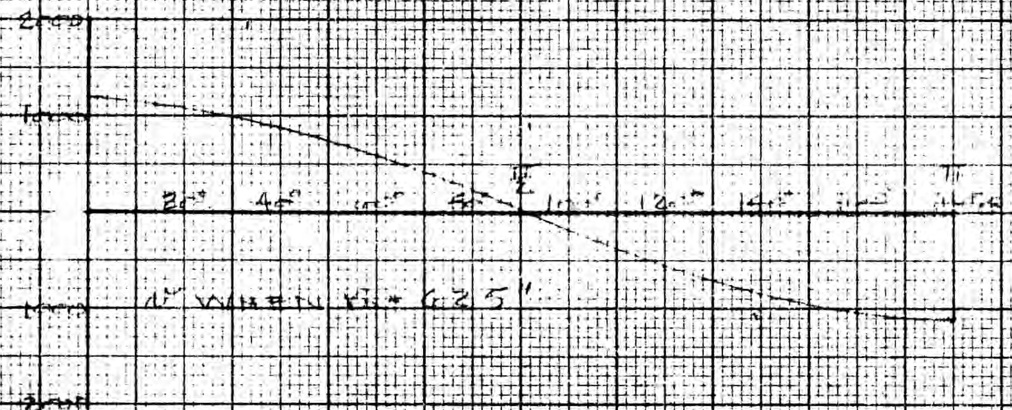


Fig. 29

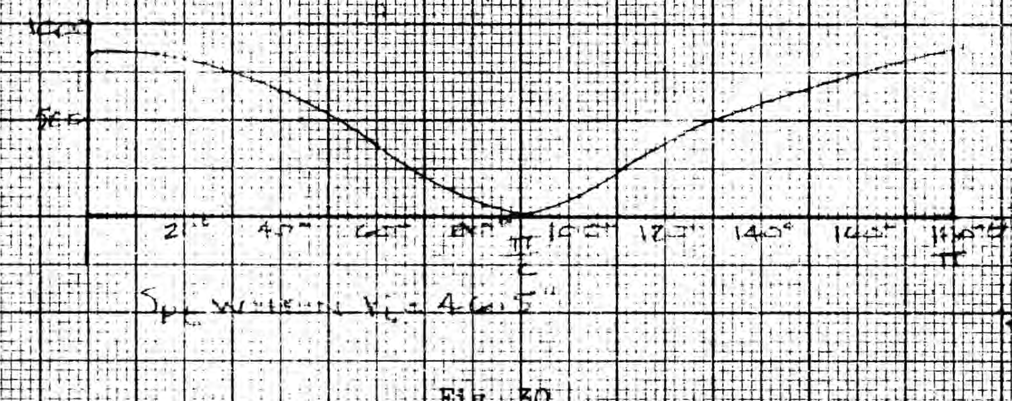


Fig. 30

When $\theta = 90^\circ$, $S_{pt} = 0$ psi.

When $\theta = 120^\circ$, $S_{pt} = 400$ psi.

When $\theta = 140^\circ$, $S_{pt} = 568$ psi.

When $\theta = 160^\circ$, $S_{pt} = 579$ psi.

When $\theta = 180^\circ$, $S_{pt} = 893$ psi.

Assuming that there is to be no tension allowed in the concrete, the web reinforcement will absorb all the diagonal tension. When $\theta = 180^\circ$, where S_{pt} is maximum.

$$\tan 2\theta = \frac{v}{f_c/2} = \frac{1125(2)}{525} = 4.29$$

$$2\theta = 76^\circ - 52'; \quad \theta = 38^\circ - 26' = 38.5^\circ$$

The plane of principal tension is rotated clockwise 38.5° from the tangent to the radius at $\theta = 180^\circ$. The spacing of stirrups measured at $r_i = 46.5''$ is as follows:

$$S = \frac{A_v f_v \sin\theta}{S_{pt} b}$$

Try #4 bars as double stirrups.

$$S = \frac{2(0.20)(20{,}000)(0.623)}{893(5)} = 1.115''$$

Try #8 bars as double stirrups.

$$S = \frac{2(0.79)(20{,}000)(0.623)}{893(5)} = 4.41''$$

Use #8 bars at 4" o.c.

Arc L = θr; $\theta = \frac{4.5}{46.5} = 0.0968$ radians. $= 5.55^\circ$

When $\theta = 140^\circ$

$$\tan 2\theta = \frac{864(2)}{750} = 2.31$$

$2\theta = 69^{o} - 3'; \quad \theta = 34.5^{o}$

$$S = \frac{2(0.79)(20,000)(0.566)}{568(5)} = 6.3''$$

$$\theta = \frac{6.3}{46.5} = 0.135 \text{ radians.} = 7.76^{o}$$

The web reinforcement is to be placed 5^{o} o.c. radially for when θ goes from 0^{o} to 50^{o}, then 8^{o} o.c. when θ goes from 50^{o} to 130^{o}, then 5^{o} o.c. for the final 50^{o}.

Stresses Straight Beam Theory

$$f_c = F/A + S\,(\theta)/A \pm Fey/I \pm My/I$$

When $\theta = 90^{o}$ and $r_i = 34.5$, when under working load

$$f_c = -636/489 + 196.9/489 - \frac{636(5.16)(13.5)}{63,658} + \frac{795.3(12)(13.5)}{63,658}$$

$$= -\,1.3 + 0.402 = 0.696 + 0.203 = -\,1.391 = -\,1,391 \text{ psi.}$$

At transfer of prestress

$$f_c = -\,1.3 - 0.006 - 0.696 - 4.70(12)(13.5)/63,658$$

$$= -\,2.014 = -\,2,014 \text{ psi.}$$

When $\theta = 90^{o}$ and $r_o = 74.5$, when under working load

$$f_c = -\,3,513 \text{ psi.}$$

At transfer of prestress

$$f_c = 36 \text{ psi.}$$

COLUMN ACTION DUE TO PRESTRESS

If the prestressing element is in direct contact with the concrete all along its length, there will be no column action in the member due to prestress. When a column deflects, additional moment in a section is

EXPLANATION OF PLATE XV

Fig. 31. Diagram of v as a function of r when $\theta = 0^{\circ}$.

Fig. 32. (a) Diagram of f_c as a function of r when $\theta = 90^{\circ}$ and under working load, as calculated by curved beam theory.

Fig. 32. (b) Diagram of f_c as a function of r when $\theta = 90^{\circ}$ and at transfer of prestress, as calculated by curved beam theory.

Fig. 32. (c) Diagram of f_c as a function of r when $\theta = 90^{\circ}$ and under working load, as calculated by straight beam theory.

Fig. 32. (d) Diagram of f_c as a function of r when $\theta = 90^{\circ}$ and at transfer of prestress, as calculated by straight beam theory.

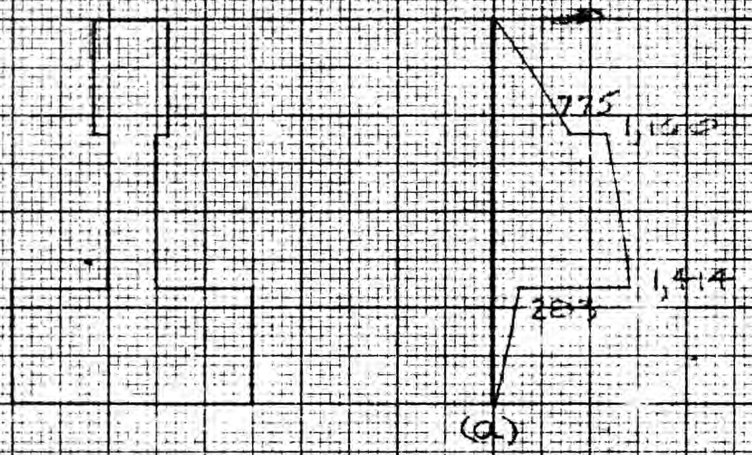

775
1,160
1,414
283
(a)

Fig. 31

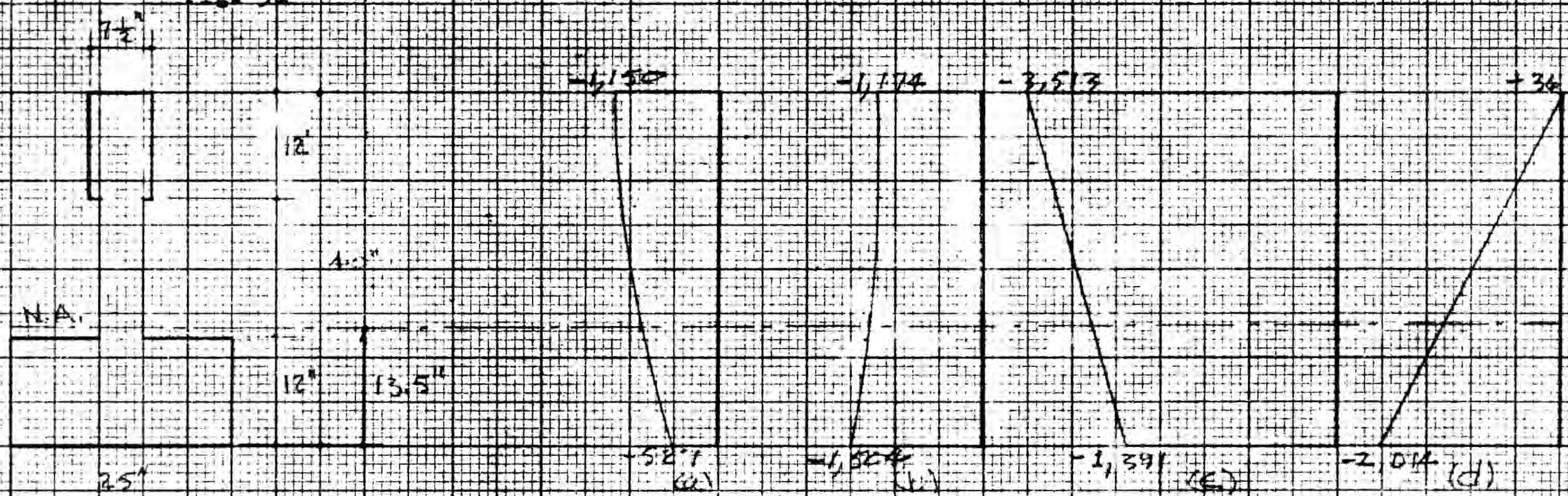

7½"
12"
4.5"
N.A.
12"
13.5"
25"
-1,150
-527
(a)
-1,174
-1,524
(b)
-3,513
-1,391
(c)
+36
-2,014
(d)

Fig. 32

created by the deflection, because the external load now acts with a greater eccentricity on that section. This additional moment is the cause of column action. However, in the case here concerned with the externally applied load is not compressive and also the prestress tendon is confined to a duct which is fixed in position. Therefore, eccentricity of the tendon does not change. So long as the steel and concrete deflect together, there is no change in the eccentricity of the prestress on the concrete, no matter how the member is deflected. Hence, there is no change in moment due to any deflection of the member and no column action.

SELECTION OF HYDRAULIC COMPONENTS

The selection of a hydraulic cylinder will be based upon an operating pressure of 2000 - 3000 psi. Operating pressure of above 3000 psi. require pumps and valves that are considerably more expensive. To obtain a 100 Ton press a pressure area of $\frac{200,000}{3,000} = 67$ sq. ins. is required. Assuming a 5 1/2 " piston rod and a 12" cylinder bore,

$$A = \pi(6)^2 - \pi(2.75)^2 = 113 - 23.8 = 89.2 \text{ sq. ins.}$$

This cylinder will be used even though it is somewhat larger than called for. The reason is the next standard size is too small and the writer also assumed 100% efficiency from the hydraulic system. Assuming 75% efficiency which is a good average value, the area required is 67 x 1.25 = 83.9 sq. ins. The stroke of this size cylinder will be 16 7/16 inches. Volume of oil pumped per stroke is 2(89.2)(16.437) = 2,930 in.^{3} which equals 12.7 gallons. A 30 gallon per minute pump would produce approximately three full length strokes per minute. This does not seem like

EXPLANATION OF PLATE XVI

Fig. 33. Typical Cross Section of C Frame. Scale 1 1/2" = 1'-0".

PLATE XVI

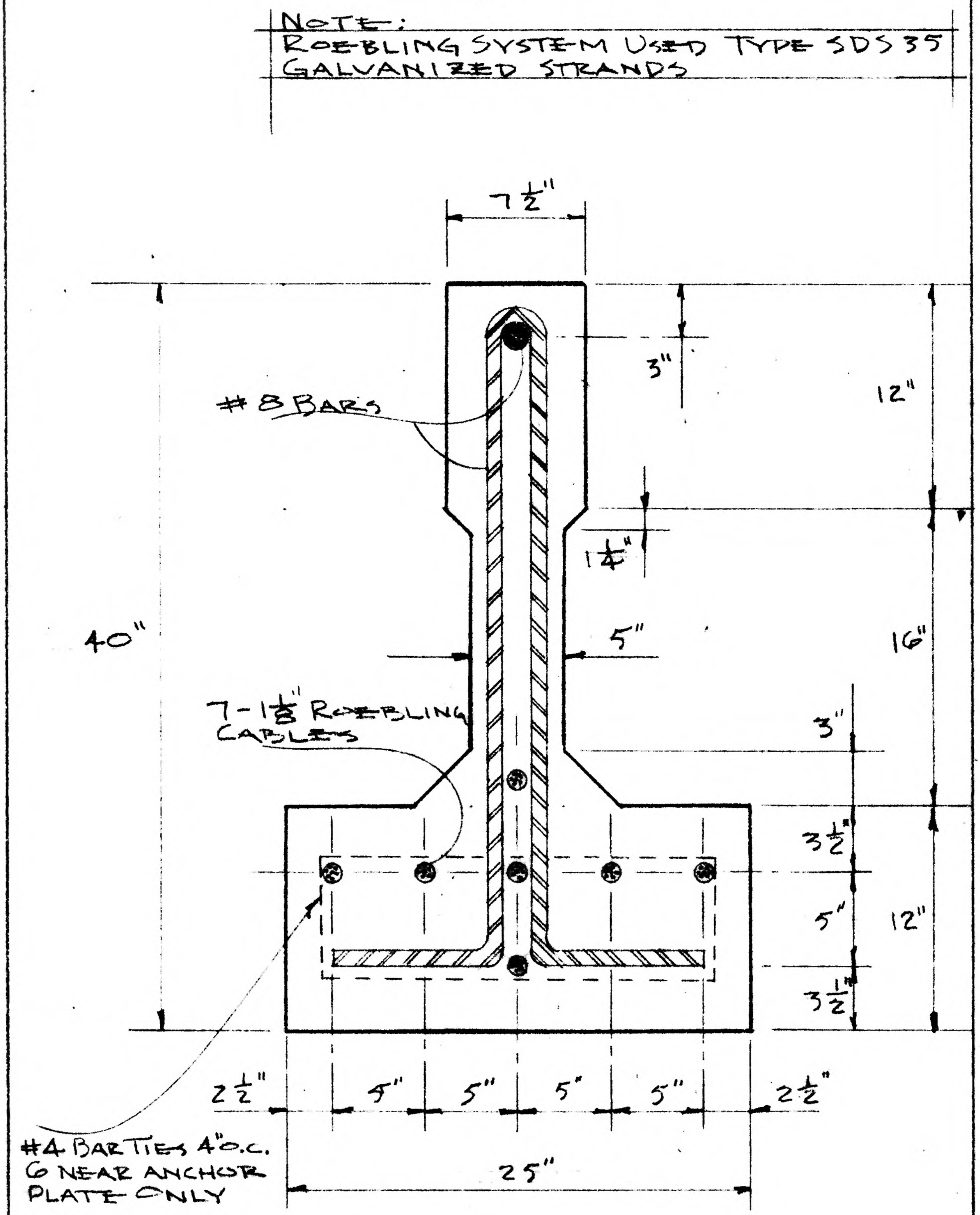


Fig. 33

EXPLANATION OF PLATE XVII

Fig. 34. Front Elevation of Hydraulic Press indicating structural C Frame only. Scale 1/2" = 1'-0".

PLATE XVII

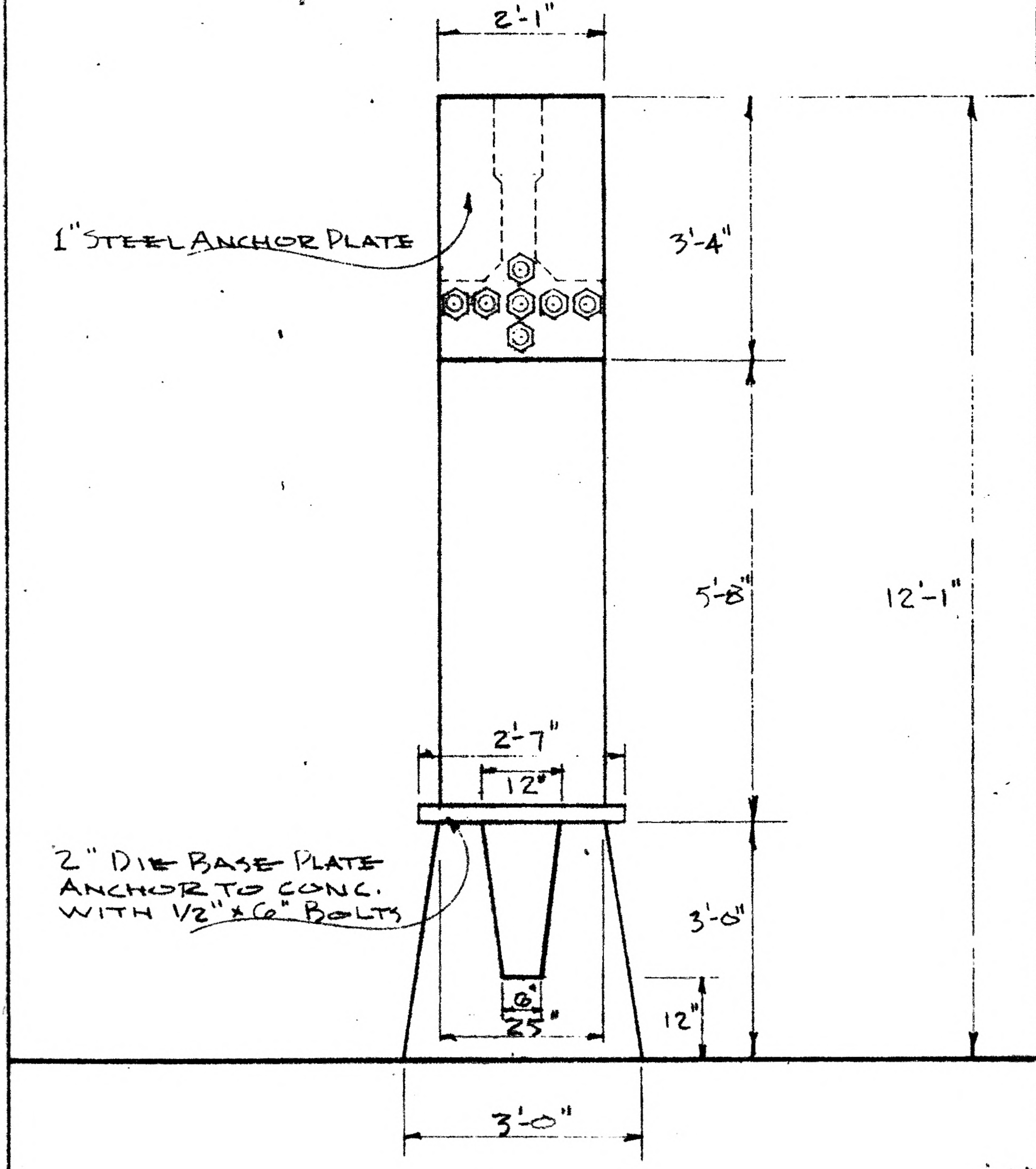


Fig. 34.

EXPLANATION OF PLATE XVIII

Fig. 35. Side Elevation of Hydraulic Press indicating structural C Frame only and the location of web reinforcement. Scale 1/2" = 1'-0".

PLATE XVIII

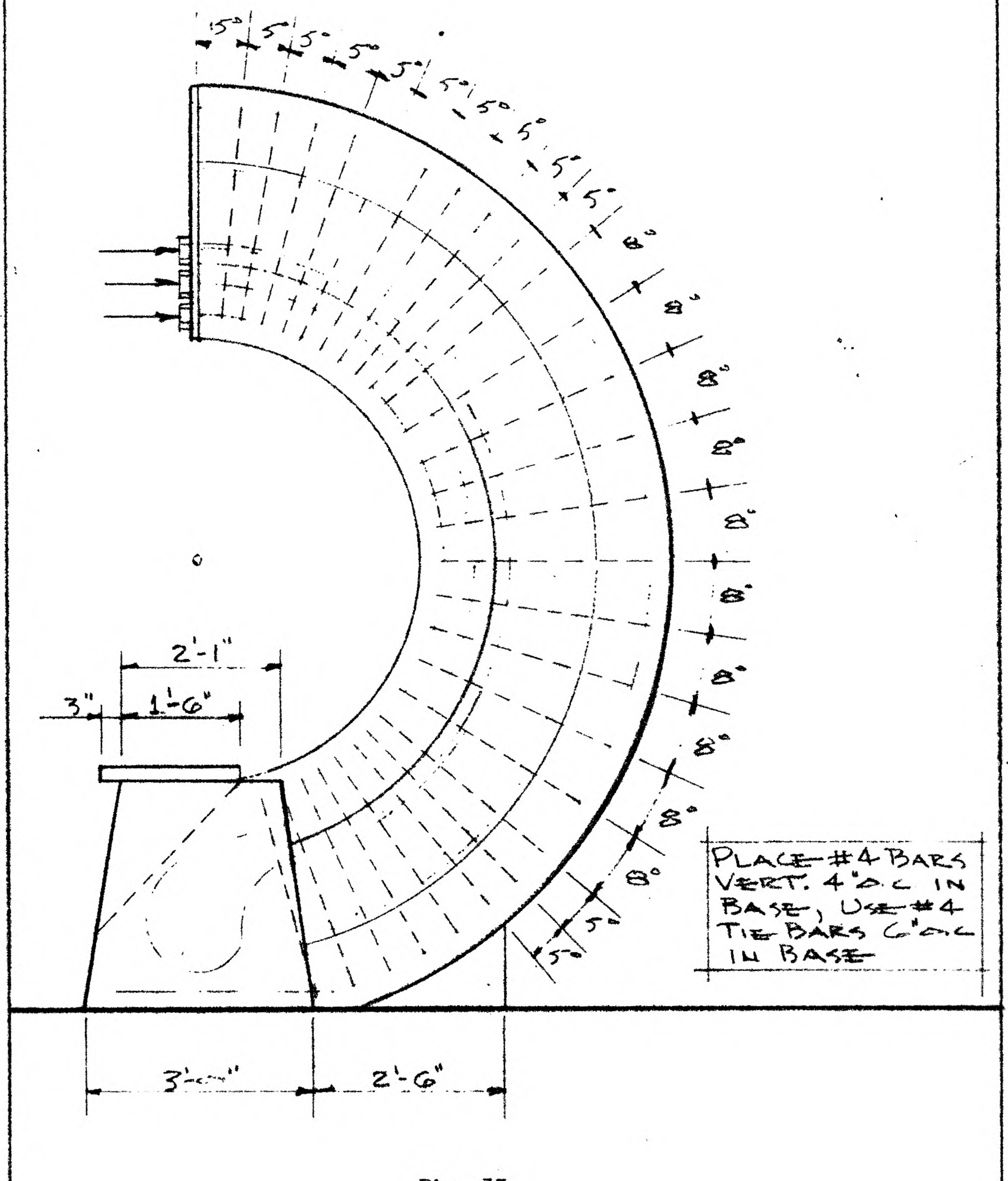


Fig. 35

EXPLANATION OF PLATE XIX

Fig. 36. Side Elevation indicating principal mechanical components.

PLATE XIX

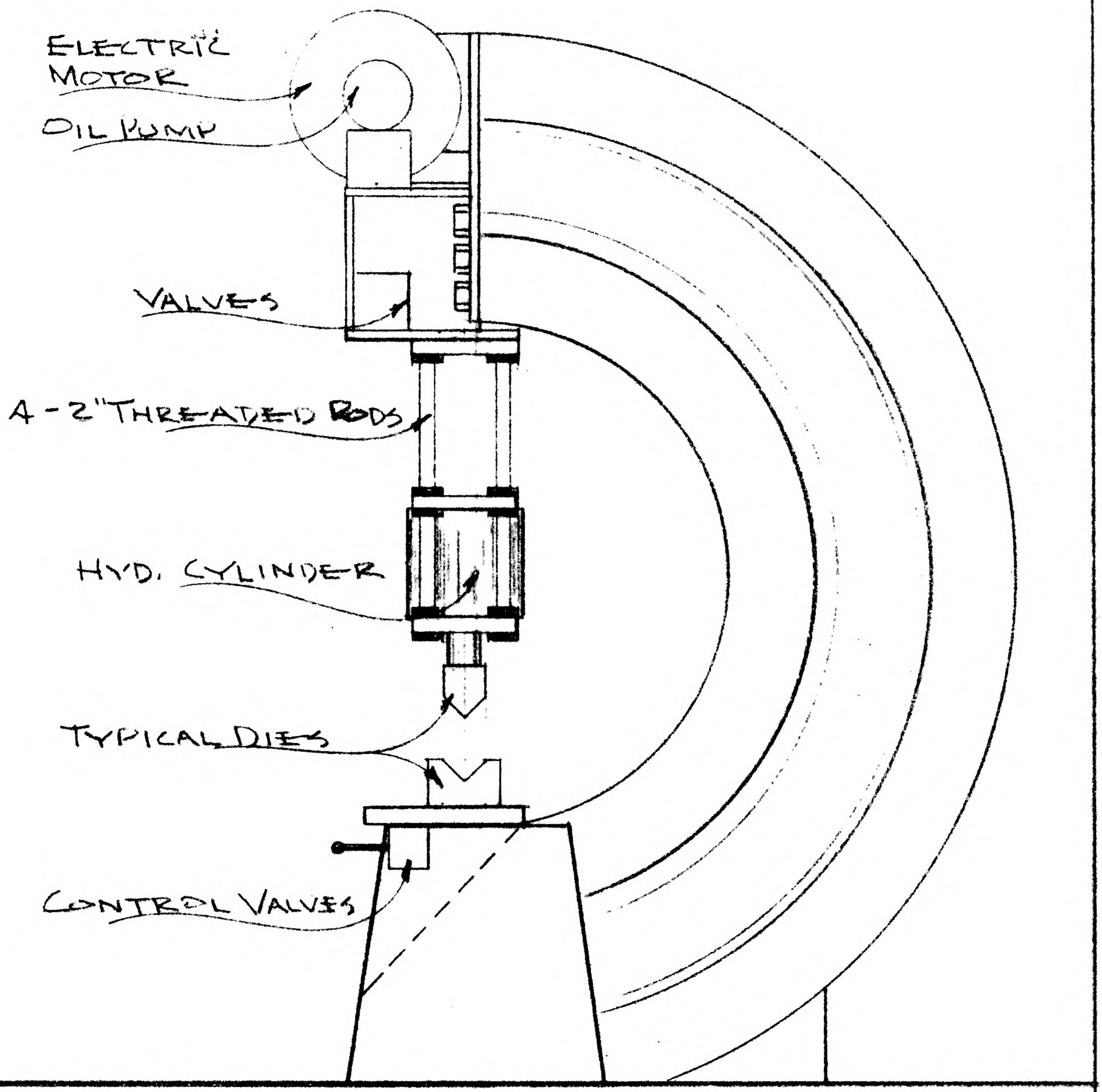


Fig. 36

a very fast operating press, however, if the length of stroke required for a particular operation is 1/4 of a full stroke, the operation cycle will be twelve per minute. The approximate power required to operate a press of the capacity we are here concerned with will be,

$$\text{Hp} = \frac{\text{G.P.M. x total head(including friction)}}{\text{3,960 x efficiency of pump}}$$

$$\text{Friction head (ft.)} = \frac{\text{pipe length(ft.) x } [\text{velocity of flow(fps)}]^2 0.02}{\text{5.367 x diameter of pipe (inches)}}$$

Efficiency varies from 50% to 85%.

Assume the approximate length of pipe utilized to be 56 ft. Since tap into hydraulic cylinder will be for a 2 1/2" pipe nipple, a 2" diameter flow area will be assumed. Then the velocity of flow will be as follows:

$$\text{v(fps)} = \frac{\text{30 G.P.M.}}{\text{7.481 G.P.ft.}^3 \text{ x } (1/12)^2\text{ft.}^2 \text{ x 60 spM.}} = \text{3.0.8 fps}$$

$$\text{Friction head (ft.)} = \frac{56(3.08)^2(0.02)}{5.367(2)} = 0.992 \cong 1 \text{ ft.}$$

$$\text{Pressure head (ft.)} = \frac{\text{3,000 psi. x 144 in.}^2\text{p ft.}^2}{\text{62.5\#/cubic ft.}} = \text{6,900 ft.}$$

$$\text{Hp Required} = \frac{30(6,900 + 1)}{3,960(0.75)} = 69.8 \cong 70 \text{ Hp}$$

SUMMARY OF MANUFACTURING REQUIREMENTS

Concrete

Physical characteristics:

7,000 psi. 28 day compressive strength.

Moist cured at 70^{o}F.

Reinforcement

Physical characteristics:

Intermediate grade 20,000 psi.

Prestress System

Physical characteristics:

Roebling system used.

7 wire galvanized strands for Post - Tensioning.

1 1/8" diameter strand.

Minimum guaranteed ultimate strength of 156,000 lbs.

Design load of 90,000 lbs.

Anchorage fitting is Type SDS35.

Electric Motor

Physical characteristics:

70 horsepower required.

Induction motor, squirrelcage type.

Hydraulic Pump

Physical characteristics:

30 G.P.M. capacity, operating pressure 2,000 to 3,000 psi.

Gear type pump.

Hydraulic Cylinder

Physical characteristics:

Bore size, 12", piston shaft 5 1/2", port size 2 1/2".

Stroke 16 7/16", operating pressure 2,000 to 3,000 psi.

ACKNOWLEDGMENTS

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STRUCTURAL DESIGN OF HYDRAULIC PRESS

by

RICHARD JOSEPH HORNUNG

B. S., Kansas State University, 1958

AN ABSTRACT OF A MASTER'S REPORT

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Departments of Architecture and Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

1961

It is the purpose of this report to design the C Frame for a Hydraulic Press, using as the basic material, prestressed concrete. This hydraulic press is to be designed for the use of Western Kansas Manufacturers. These manufacturers have a great need for an economical press to keep up with competition of Eastern Manufacturers. This is the basic reason behind the attempt to use prestressed concrete for the frame rather than cast iron.

The preliminary design is based upon conventional design of prestressed concrete where applicable. That is, it is designed on an elastic straight line theory. Flexural tension in concrete is not allowed in the design. This is because the frame is part of a machine which will be under repeated loadings and fatigue may be a factor. It is believed if all flexural stresses in the concrete remain in compression and elastic range the chances of fatigue failure will be minimized.

The Roebling system of prestressing was used for post-tensioning. Since the curvature of this frame is rather sharp, a cable system is desirable. The anchorage device of the Roebling system also facilitates easier retensioning of cables because of its bolt and nut arrangement. This is especially important since these prestress losses were assumed taken up by delayed jacking.

Bending stresses in curved beams are known to vary from thos calculated by the straight line method. The reason is the neutral axis shifts towards the center of curvature. Then in order for equilibrium to exist the stress diagram must be a curve. The methods vary for calculation of these stresses in composite beams. Some are not practical for application. Fred B. Seely has derived a solution which is more difficult to apply than that derived by Glen Murphy. Therefore, the latter solution has been used in this paper. In either case the method for arrival at the shift of the neutral axis is approximately the same. In the problem here concerned with the shift of the neutral axis only amounts to 1/4 inch. It would be an interesting project to test the stresses in this structure with resistance type strain gauges, strageti-cally placed, to see how far the actual stresses vary from calculated values.

The shearing stresses are somewhat serious in this frame, in that the principal diagonal tensile stresses are high. This is reflected in the amount of radial stirrups required. Possibly improvement could be made by the increase of web thickness, however, this would increase the weight of the structure which would be undesirable from the mobility standpoint.

The desirn accomplished in this paper is believed by the writer to still be somewhat too bulky to be the most desired press by the Western Kansas Manufacturer. Possibly a third design should be accomplished before manufacture of a press of this type. A C Frame somewhat flattened on the order of a jaw may prove to be more satisfactory.