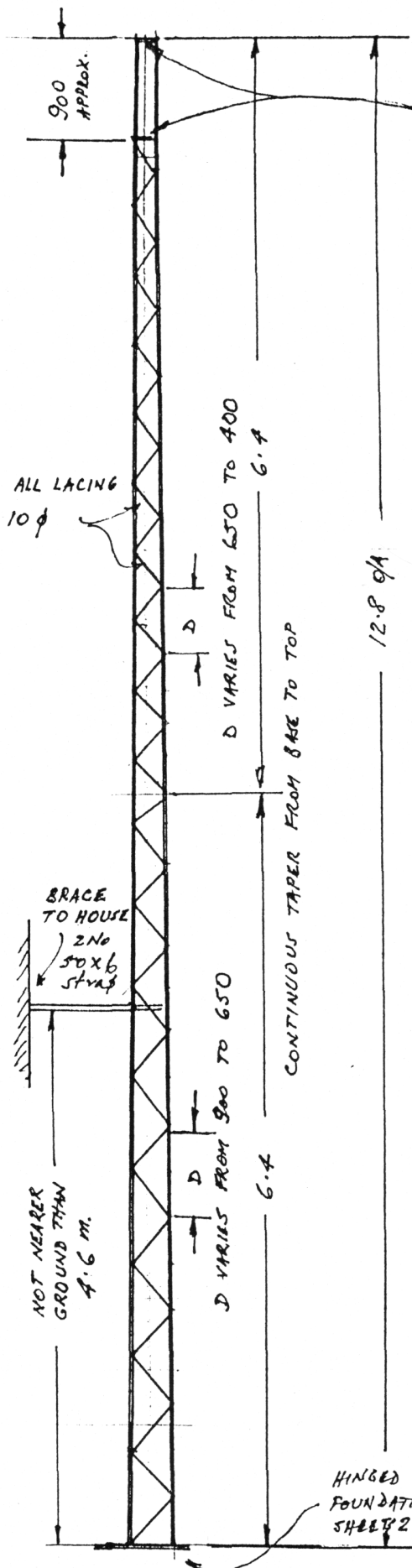
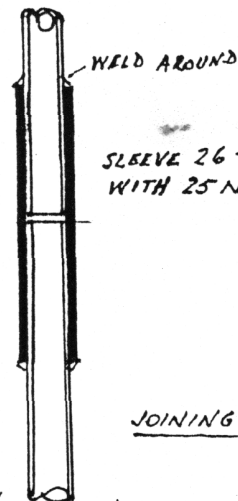
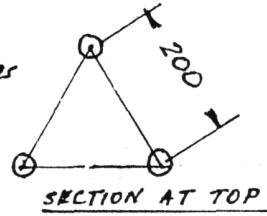


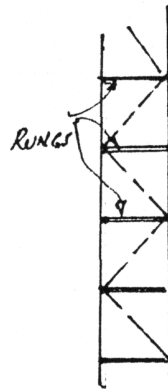


2 NO 6 MM PLATES WELDED TO LEGS AT CORNERS

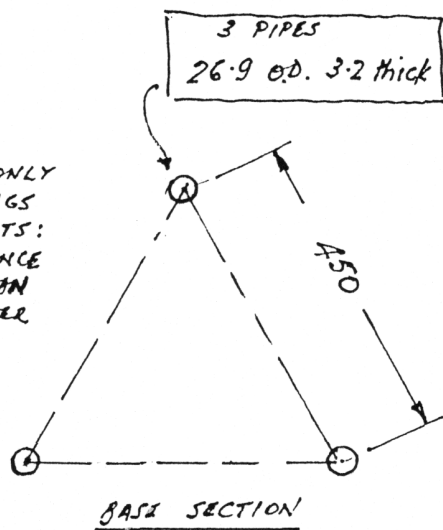


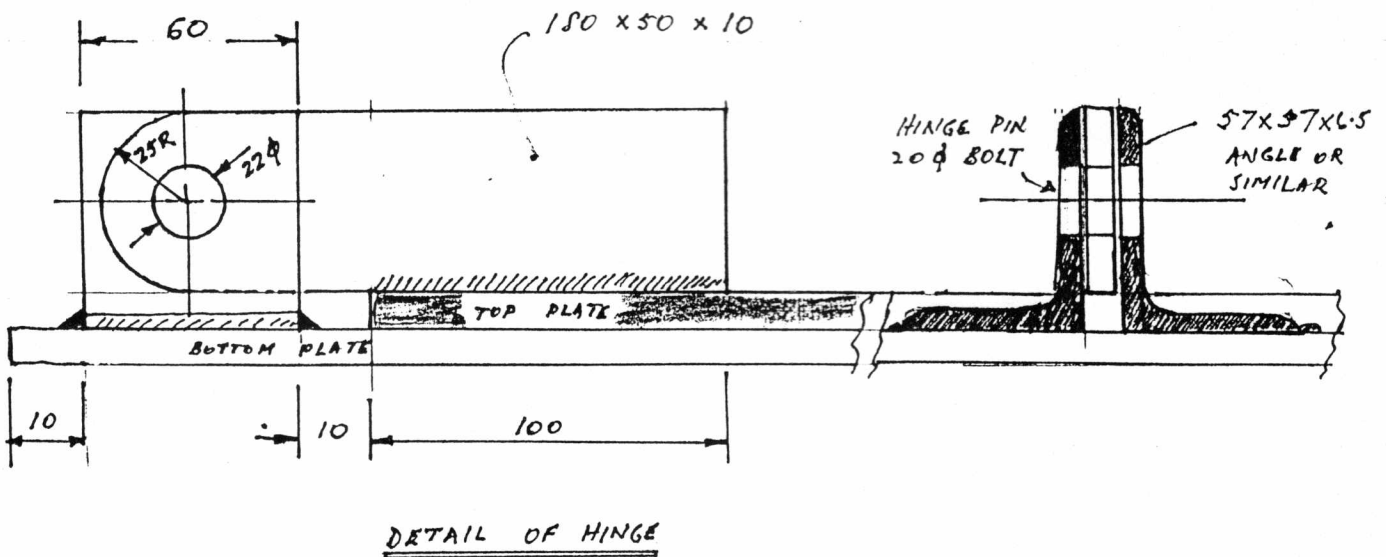
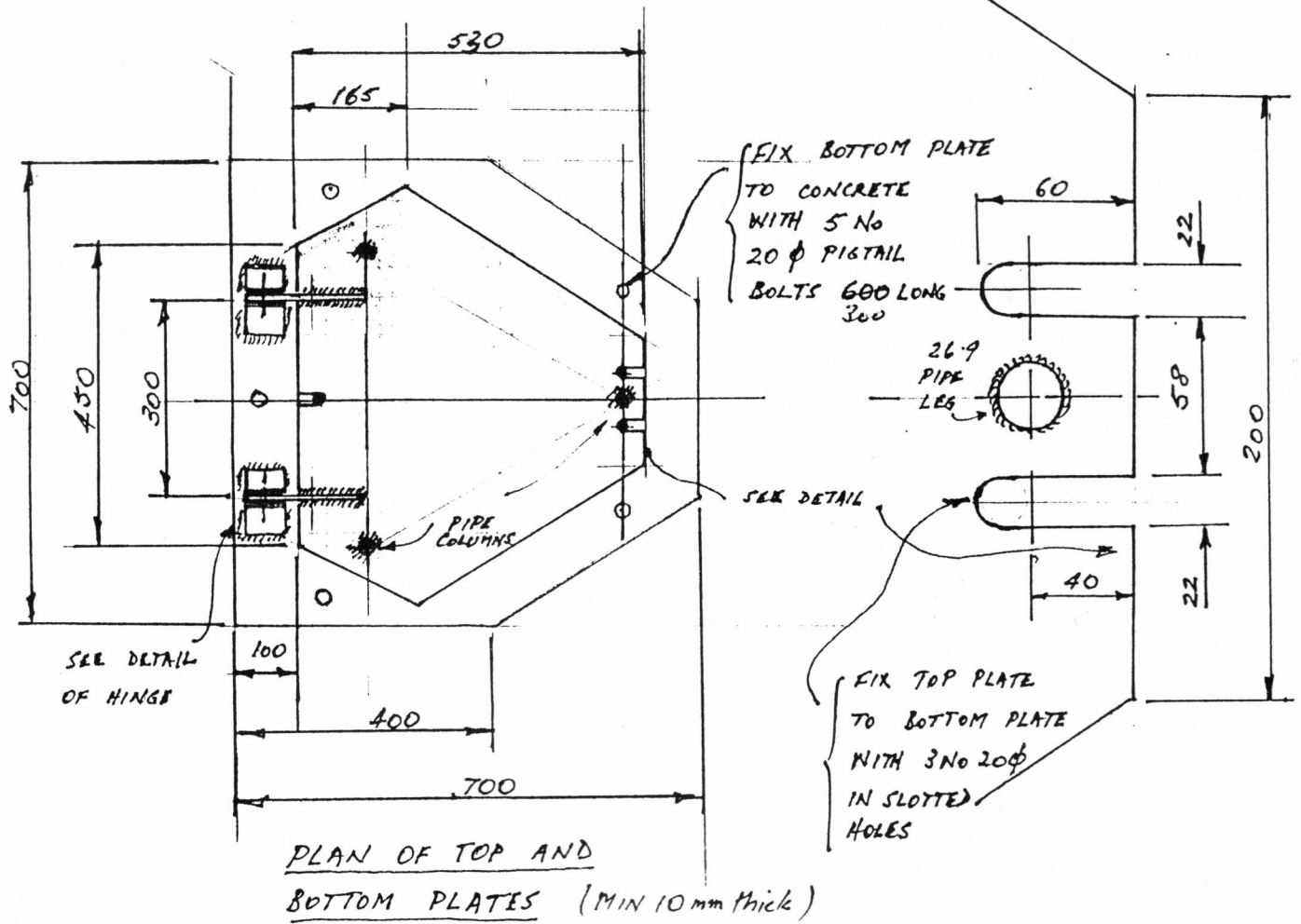
SLEEVE 26.9 O.D. PIPE WITH 25 N.B. (2.6 thickness)

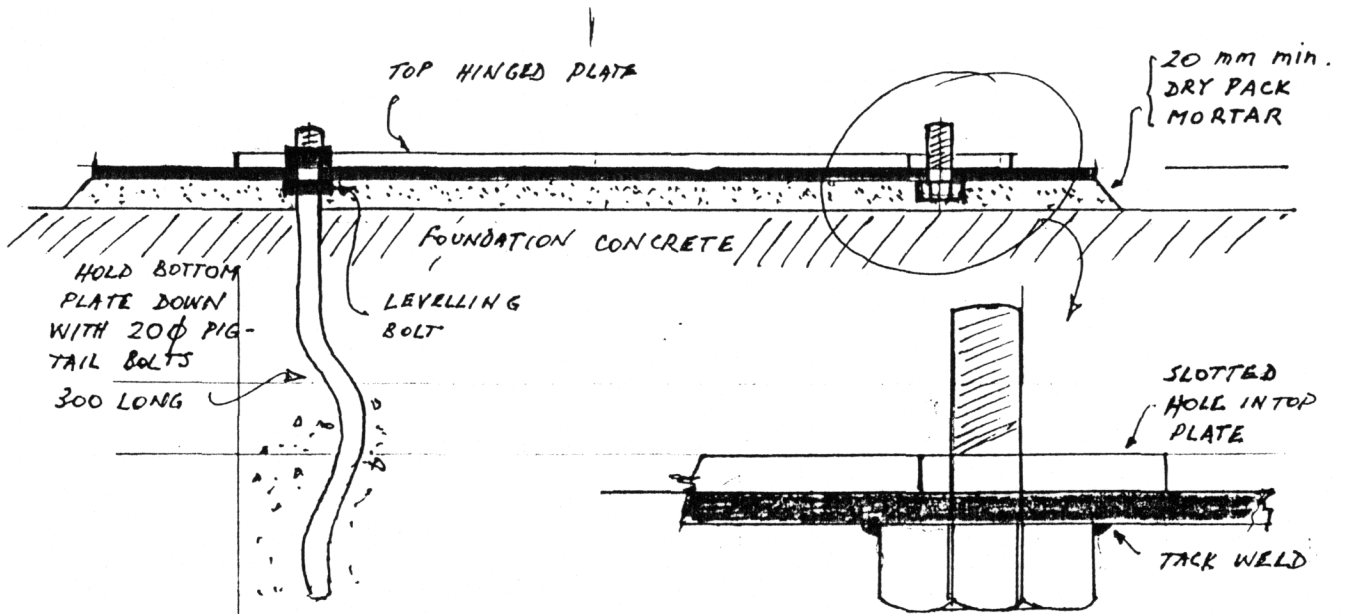
JOINING PIPES



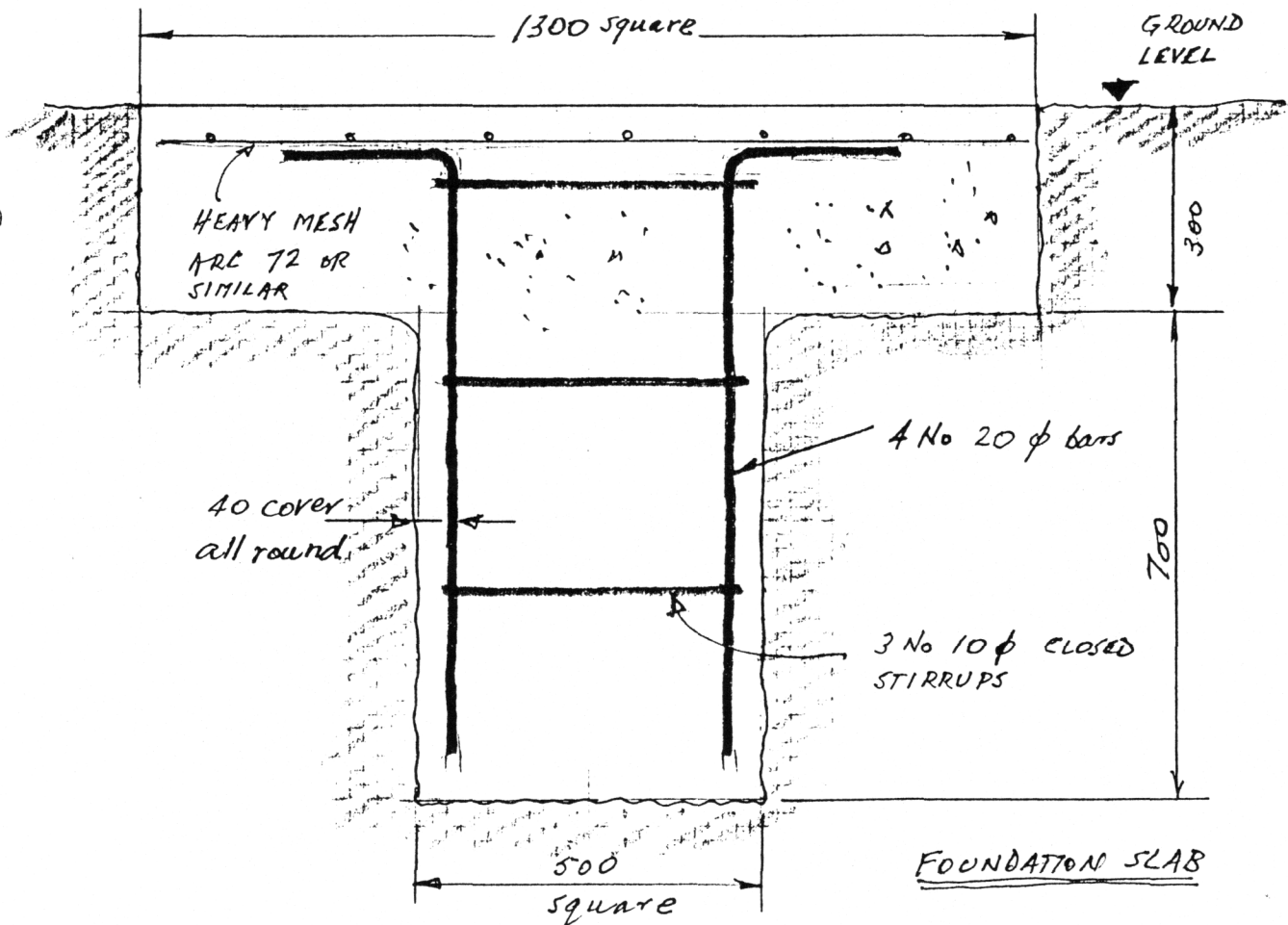
ON ONE SIDE ONLY ARRANGE RUNGS AT NODE POINTS: VERTICAL DISTANCE NOT LESS THAN 325 OR GREATER THAN 450.







METHOD OF FIXING PLATES TO FOUNDATION CONCRETE



27/7/83.

ANTENNA MASS DESIGN.

(DES 1045)

Des No SM1018.

Data:

location in terrain category 3.

no earthquake loading

design wind velocity for 50 year return period.

as per AS 1170 Pt 2 = 41 m/s.

mast vertical legs from 20 NB heavy wall pipe

22 O.D. x 3.2 wall.

form factor = 0.8

antenna weight = 10 kg.

frontal area of antenna 0.25 m^2 (assumed)

form factor for antenna 1.2

mast height 13106 mm.

Velocity multipliers for terrain category 3.

up to 5 m = 0.65.

" 10 m = 0.7

" 15 m = 0.75.

$$\therefore \text{Dynamic pressure } q_z = 0.6 (0.65 \times 41) \times 10^{-3} = 0.426 \text{ kPa.}$$

up to 5m

= 0.494 kPa up to 10m

= 0.567 kPa up to 15m.

$$\text{Frontal area of mast} = (3 \times 1 \times 0.022) \times 1.2$$

$$= 0.1 \text{ m}^2/\text{m}$$

The 1.2 factor is to allow for all the other bracing and struts.

No shielding is assumed.

Force acting on antenna

$$= 0.562 \times 0.25 \times 1.2$$

$$= 0.17 \text{ kN.}$$

load / metre length on mast over 10 m high

$$= 0.562 \times 0.1 \times 0.8$$

$$= 0.0454 \text{ kN/m.}$$

load / metre length on mast over 5 m high, but less than 10 m

$$= 0.494 \times 0.1 \times 0.8$$

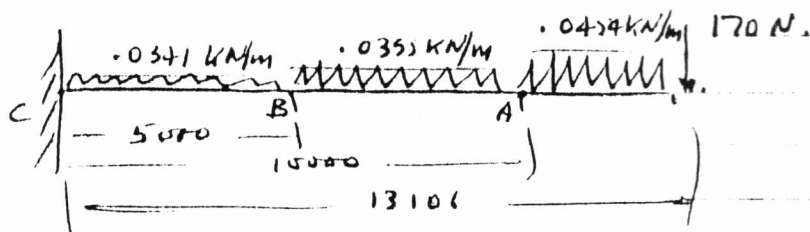
$$= 0.0395 \text{ kN/m.}$$

load / metre length on mast less than 5 m height

$$= 0.426 \times 0.1 \times 0.8$$

$$= 0.0341 \text{ kN/m.}$$

To determine bending moments along mast



$$\text{B.M. at A} = 170 \times 3106 + 45.4 \times 3.106 \times 3106/2.$$

$$= 0.75 \times 10^6 \text{ Nmm.}$$

$$\text{B.M. at B} = 170 \times 8106 + 45.4 \times 3.106 \times (5000 + 3106/2)$$

$$+ 39.5 \times 5 \times 5000/2.$$

$$= 2.796 \times 10^6 \text{ Nmm.}$$

$$\text{B.M. at C} = 170 \times 13106 + 45.4 \times 3.106 \times (10000 + 3106/2)$$

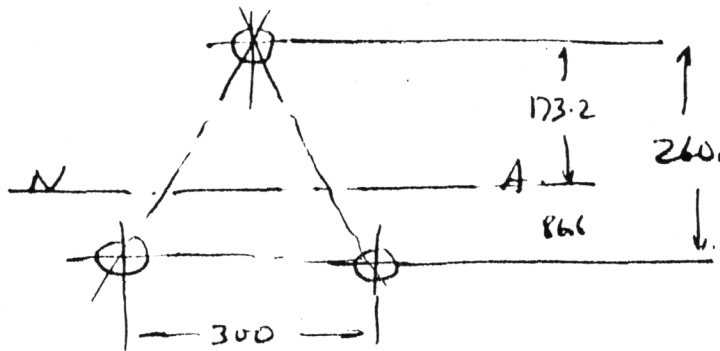
$$+ 39.5 \times 5 \times 7500 + 34.1 \times 5 \times 2500$$

$$= 5.765 \times 10^6 \text{ Nmm.}$$

To determine section properties

$$\begin{aligned} I_{xx} \text{ of } 2700 \times 3.2 \text{ tube} &= \frac{\pi}{64} [27^4 - 20.6^4] = 17247 \text{ mm}^4 \\ \times 4 &= \frac{\pi}{4} [27^2 - 20.6^2] = 239 \text{ mm}^2 \end{aligned}$$

Consider section properties at base.



$$\begin{aligned} I_{NA} &= 2 [17247 + 239 \times 86.6^2] + [17247 + 239 \times 173.2^2] \\ &= 10\,820\,000 \text{ mm}^4 \end{aligned}$$

$$z_{NA} = \frac{I}{y} = \frac{10\,820\,000}{86.6} = 124918 \text{ mm}^3$$

$$\text{or } \frac{10\,820\,000}{173.2} = 62459 \text{ mm}^3$$

$$\begin{aligned} \therefore \text{bending stress max} &= \pm \frac{M}{z} = \frac{5.765 \times 10^6}{62459} \\ &= \pm 92.3 \text{ MPa} \end{aligned}$$

From AS1250-1951. Steel Structures Code.

For a stress of 92.3 MPa on a strut made from a material with a yield of 250 MPa, the slenderness ratio is 90 say

$$\begin{aligned} \therefore \text{the length of the column} &= 0.7 \times 13.5 \times 90 \\ \text{between stays} &= 850 \text{ mm} \end{aligned}$$

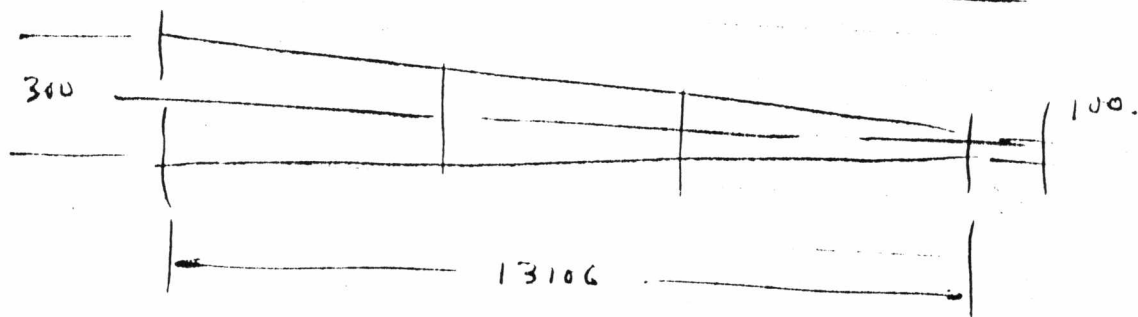
to allow for body forces, we shall reduce this to 750. This will also match the ladder stiles spacing at 250

$$\frac{d}{K} = \frac{750}{0.2 \times 13.5} = 79.36 \text{ say } 80.$$

$$\text{and } \sigma_{all} = 104 \text{ MPa c.f. } 92.5 \text{ MPa.}$$

This means that there is enough reserve to carry the self weight forces

Consider stresses at 5000 + 10000 levels.



to determine spacing of members at 5000 + 10000 levels.

By proportion.

at 5000 level.

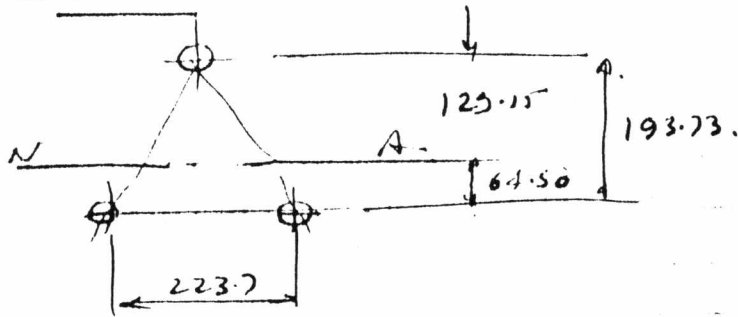
$$\begin{array}{l} 200 \text{ in } 13106 \\ x \text{ in } 8106 \end{array}$$

$$\therefore x = \frac{8106}{13106} \times 200 = 123.7$$

$$\therefore \text{spacing} = 123.7 + 100 = \underline{\underline{223.7}}$$

at 10000 level spacing is 147.4.

at 5500 level.



$$I_{NA} = 2[17247 + 239 \times 64.56^2] + [17247 + 239 \times 129.15^2]$$

$$= 6\,032\,000 \text{ mm}^4$$

$$Z_{NA} = \frac{6\,032\,000}{64.56} = 93\,400 \text{ mm}^3$$

$$r = \frac{6\,032\,000}{129.15} = 46\,700 \text{ mm}^3$$

$$\therefore \text{bending stress max} = \pm \frac{M}{Z} = \frac{2.796 \times 10^6}{46\,700}$$

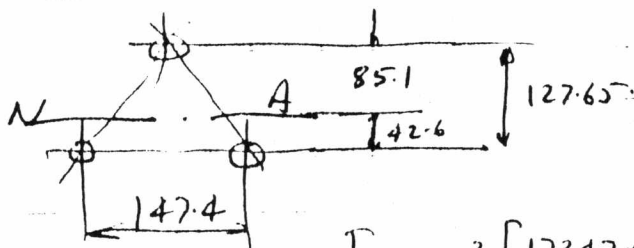
$$= \pm 59.87 \text{ MPa}$$

$\therefore$ slenderness ratio can be 120.

$$\text{i.e. } L = 120 \times 0.7 \times 13.5$$

$$= 1134 \text{ mm}$$

at 10000 level.



$$I_{NA} = 2[17247 + 239 \times 42.6^2] + [17247 + 239 \times 85.1^2]$$

$$= 2\,648\,000 \text{ mm}^4$$

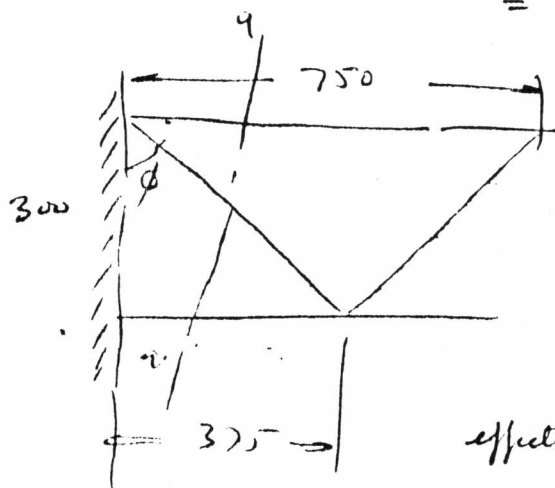
$$Z_{NA} = \frac{2\,648\,000}{42.6} = 62\,160 \text{ mm}^3$$

$$r = \frac{2\,648\,000}{85.1} = 31\,080 \text{ mm}^3$$

$$\therefore \text{Max Bending stress} = \frac{0.15 \times 10^6}{31080} = \pm 24 \text{ MPa.}$$

Consider shear stress at base.

$$\begin{aligned} \text{Total shear} &= 0.17 + 0.454 \times 3.1 + 0.0395 \times 5 + 0.034 \times 5 \\ &= 0.6782 \text{ kN.} \end{aligned}$$



Shear across section q-q.

$$\phi = \arctan 275/300 = 57.34^\circ.$$

$\therefore$ force in members (2 off).

$$= 678 / \cos 57.34^\circ$$

$$= 1085 \text{ N.}$$

i.e. 543 N / member.

$$\text{effective length of member} = 480 \text{ mm} \times 0.7.$$

$$\frac{l}{r} = \frac{480 \times 0.7}{0.5 \times r} = 224$$

Assume 6 ϕ rod. $A = 28 \text{ mm}^2.$

$$\therefore \text{stress} = \frac{543}{28} = 19.2 \text{ MPa.}$$

all stress in steel from table G.1.1. AS 1250
for ϕ = 250 MPa

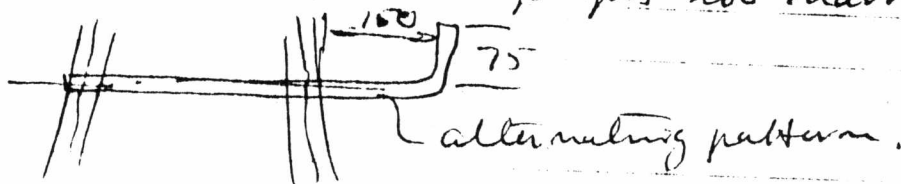
$$\sigma_{all} = 18 \text{ MPa} \approx \text{that above}$$

$\therefore$ adopt 6 ϕ rods.

Ladder stiles.

make 2 from 10 ϕ MS rod.

Suggest that above 5000 level, stiles be extended by 150 with a 75 mm high end return because space between vertical legs gets too narrow.



Hold-down bolts.

Max tensile load in bolt is of the same order as that of the legs.

From page 3 we have a stress of 92.3 MPa in the legs.

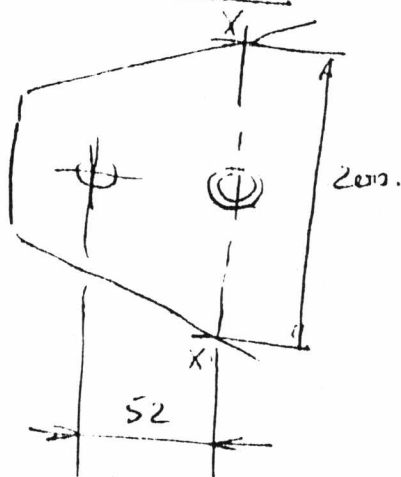
Using this figure and a cross area of 237 mm^2 for the vertical legs, the actual force in the leg is

$$\sigma \times A = 92.3 \times 237 \\ = 21878 \text{ N.}$$

Using a stress of 120 MPa in the hold-down bolt, the area req'd = 182 mm^2

$$\equiv 15 \text{ mm } \phi$$

suggest we use 18ϕ hold down bolts
(or $\frac{3}{4}'' \phi$)

Base plate.

$$\text{At } XX \quad Z \text{ of plate using } 12 \text{ k} \\ = \frac{200 \times 12^2}{6} \\ = 4800 \text{ mm}^3.$$

Load in bolt in this case = 17000 N .

$$\text{B.M. } XX = 17000 \times 52 \text{ Nm.}$$

$$\therefore \sigma = \frac{17000 \times 52}{4800}$$

$$= 185 \text{ MPa.} = 75\% \text{ of } \sigma_{\text{allow.}}$$

$\therefore$ adopt 16 pl.

$$Z = 8533$$

$$\sigma = \frac{17000 \times 52}{8533} = 103 \text{ MPa. ok.}$$

Joseph Q/Borg (M/E
Arch.)