PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. 1.

STEEL DESIGN.

PROJECT :

OWNER :

ARCHITECT :

ENGINEER :

DESIGN CRITERIA

A36 - STEEL : F_y = 2500 KSC.

: F_t = 1500 KSC.

TYPE OF CONSTRUCTION

STEEL STRUCTURE WITH CONNECTION BY WELDED

WELDING ELECTROD E 70

: F_u = 1470 KSC.

LOADING

GRAVITY LOAD OF DEAD LOAD + LIVE LOAD = 80 kg/m².

WIND LOAD

ความเร็วลมสูงสุดในรอบปี = 50 kg/m².

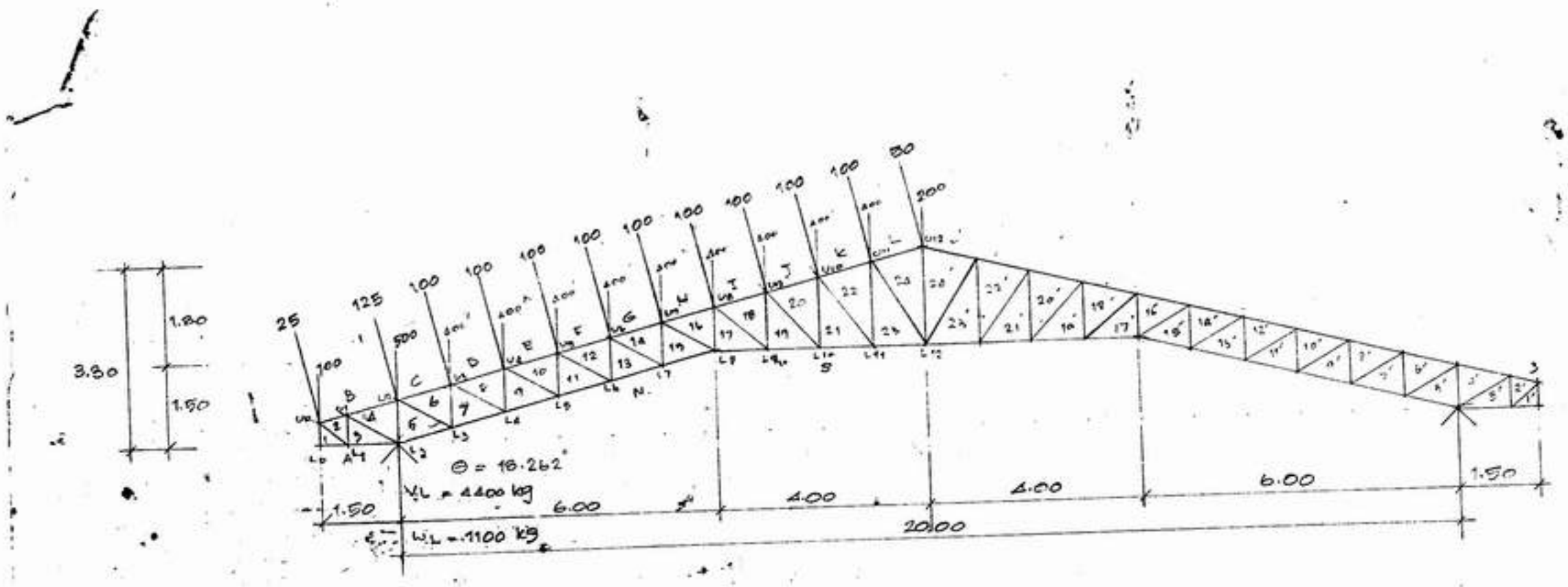
ความดันลมที่กระทำตั้งฉากกับหลังคา = $\frac{PB}{45} = \frac{50 \times 18.262^2}{45}$

= 20.29 ≈ 20 kg

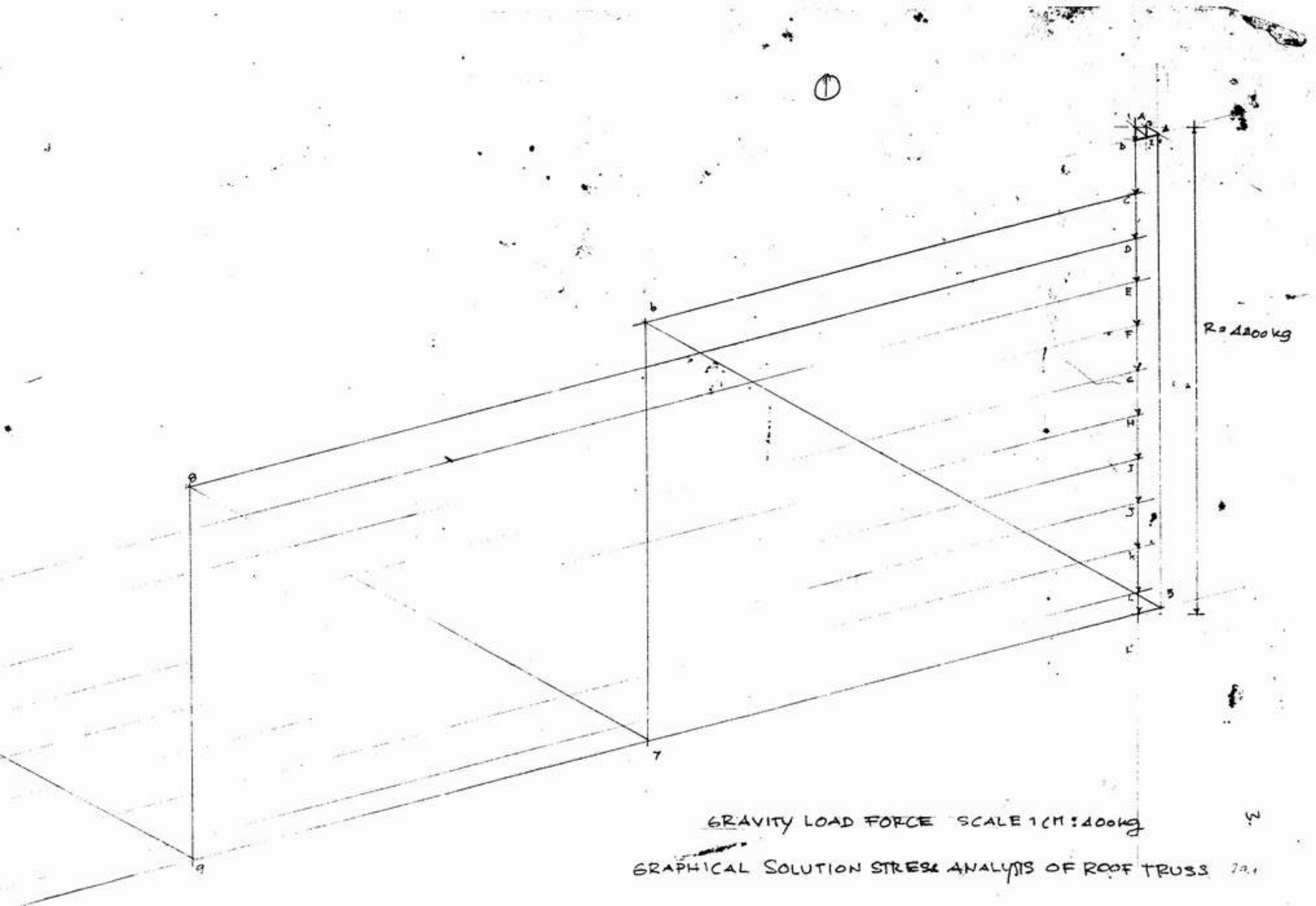
LOAD OF ROOF

GRAVITY LOAD = 80×5.0×1.0 = 400 kg

WIND LOAD = 20×5.0×1.0 = 100 kg



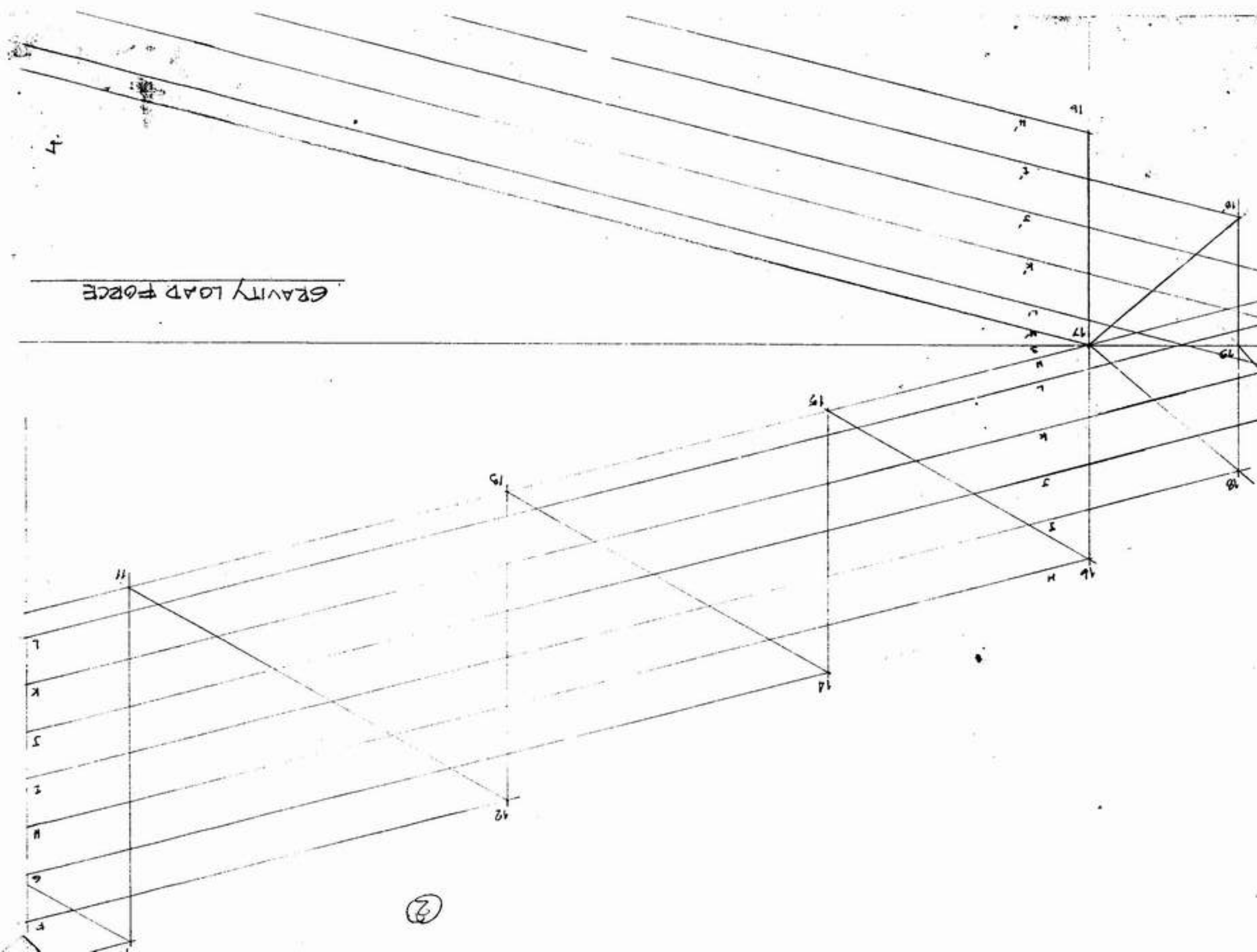
LINEAR SCALE 1:100 TRUSS T-1



GRAVITY LOAD FORCE SCALE 1 CM : 400 kg

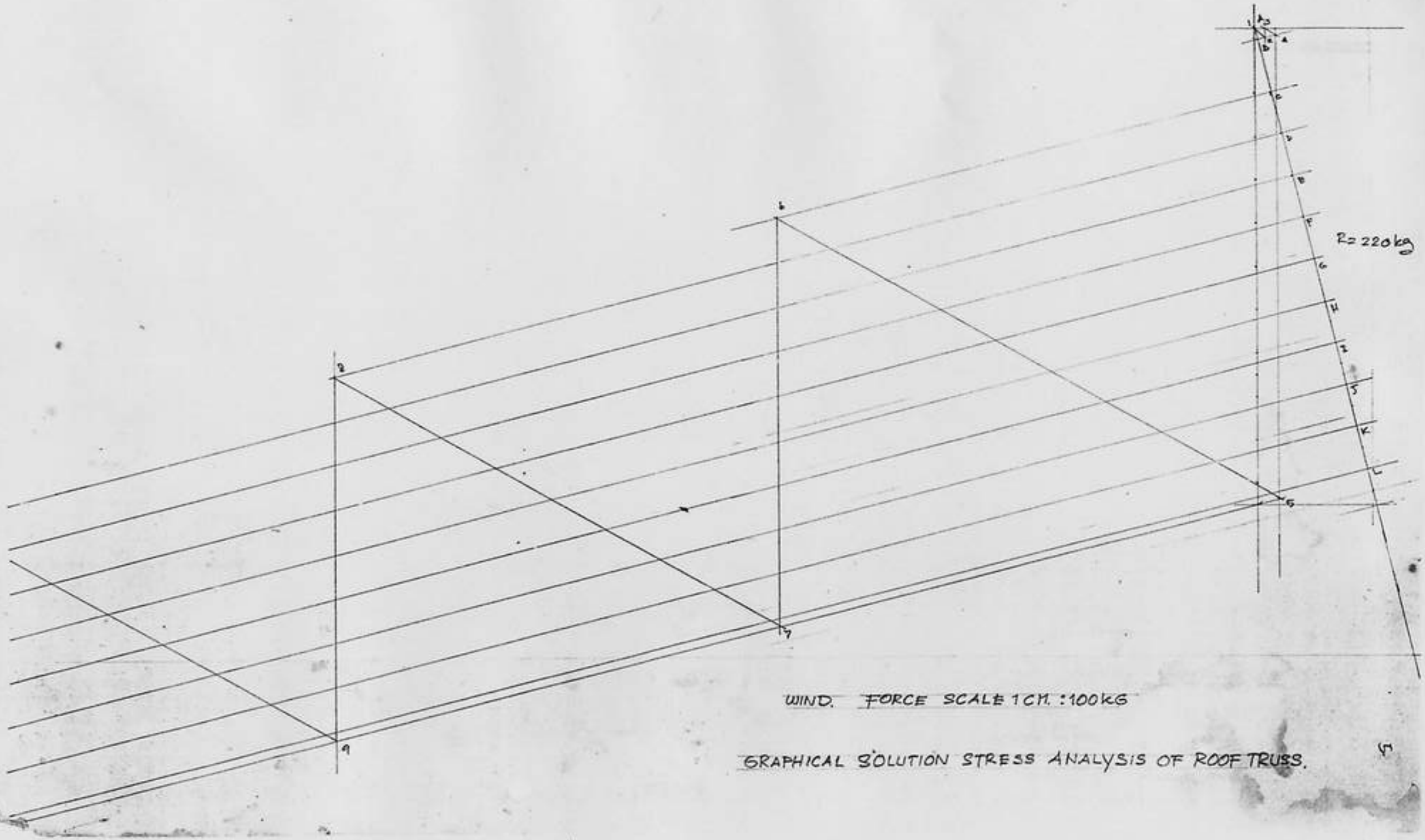
GRAPHICAL SOLUTION STRESS ANALYSIS OF ROOF TRUSS

GRAVITY LOAD FORCE



(2)

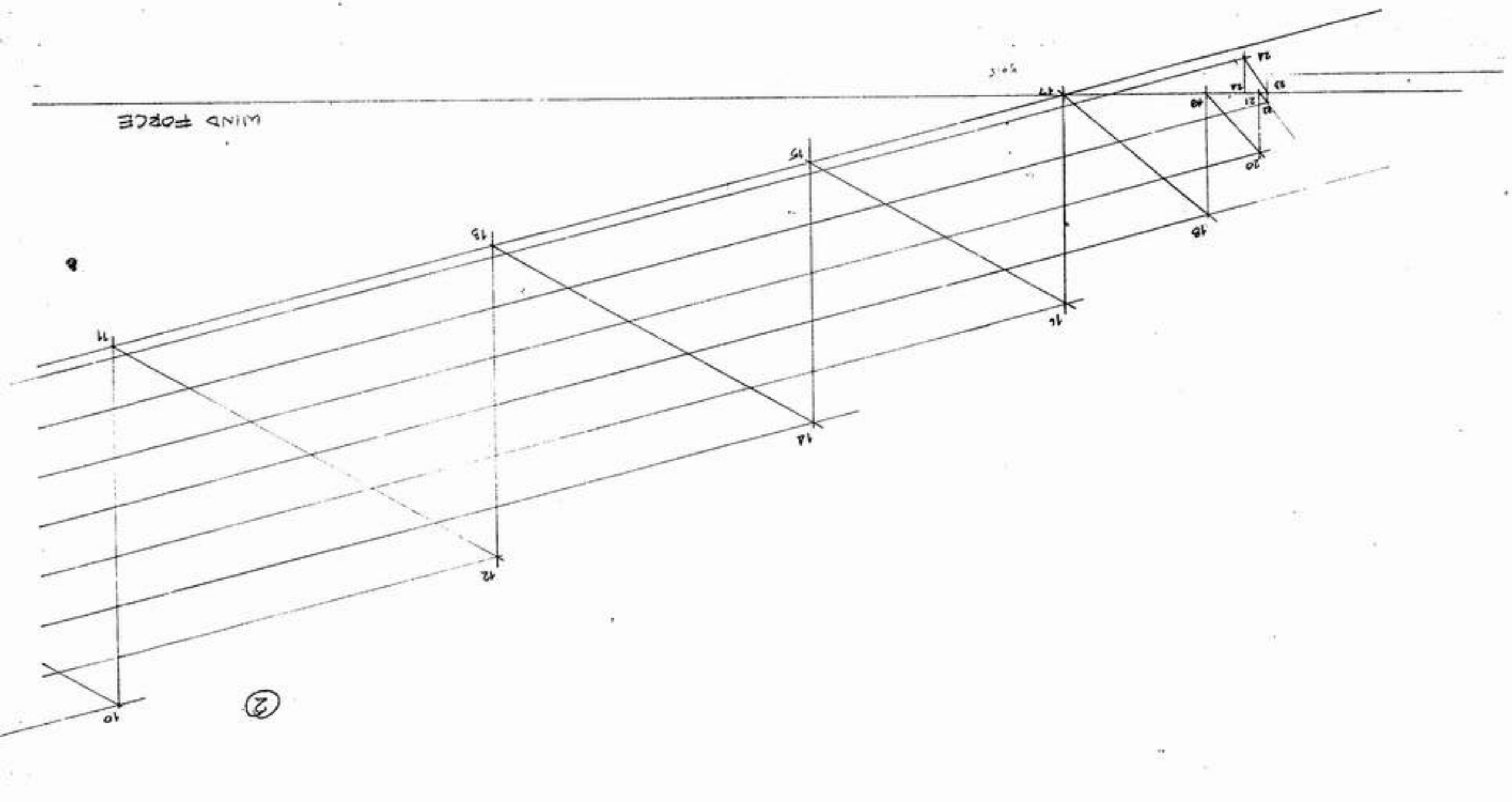
(1)



WIND. FORCE SCALE 1 CM. : 100 KG

GRAPHICAL SOLUTION STRESS ANALYSIS OF ROOF TRUSS.

WIND FORCE



(2)

PROJECT. : _____ SHEET. NO.

OWNER. : _____ 6.

ARCHITECT. : _____

ENGINEER. : _____

DESIGN ROOF TRUSS. GRAPHICAL SOLUTION.

MEMBER	LENGTH (m.)	GRAVITY LOAD (kg)	WIND LOAD (kg)	P. DESIGN (kg)
$U_0 - U_1$		100	20	
$U_1 - U_2$		220	55	
$U_2 - U_3$		4680	1160	
$U_3 - U_4$		9040	2230	
$U_4 - U_5$		12920	3170	
$U_5 - U_6$		16240	3990	
$U_6 - U_7$		19120	4670	
$U_7 - U_8$		21480	5220	
$U_8 - U_9$		22800	5350	
$U_9 - U_{10}$		23320	5480	
$U_{10} - U_{11}$		23420	5500	28920 (115509)
$U_{11} - U_{12}$		23240	5430	
$L_0 - L_1$		0	0	
$L_1 - L_2$		100	20	
$L_2 - L_3$		220	55	
$L_3 - L_4$		4680	1160	
$L_4 - L_5$		9040	2230	
$L_5 - L_6$		12920	3170	
$L_6 - L_7$		16240	3990	
$L_7 - L_8$		19120	4670	
$L_8 - L_9$		20840	5220	

PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. 7.

DESIGN ROOF TRUSS GRAPHICAL SOLUTION.

MEMBER.	LENGTH (m)	GRAVITY LOAD (kg)	WIND LOAD (kg)	P. DESIGN.
$L_9 - L_{10}$		22120	5350	
$L_{11} - L_{12}$		22720	5480	28200 (COMP)
$U_0 - L_1$		160	35	
$U_2 - L_3$		5400	1340	6740 (TENS)
$U_3 - L_4$		4840	1190	
$U_4 - L_5$		4280	1050	
$U_5 - L_6$		3720	910	
$U_6 - L_7$		3160	760	
$U_7 - L_8$		2600	600	
$U_8 - L_9$		1680	390	
$L_9 - L_{10}$		760	170	
$U_{10} - L_{11}$		180	30	
$U_{11} - L_{12}$		280	90	
$U_0 - L_0$		0	0	
$U_1 - L_1$		80	25	
$U_2 - L_2$		4280	1080	5360 (COMP)
$U_3 - L_3$		3800	950	
$U_4 - L_4$		3400	850	
$U_5 - L_5$		3000	750	
$U_6 - L_6$		2600	650	

PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. 8.

DESIGN ROOF TRUSS GRAPHICAL SOLUTION.

MEMBER	LENGTH (m.)	GRAVITY LOAD (KG)	WIND LOAD KG	P. DESIGN.
$U_7 - L_7$		2200	550	
$U_8 - L_8$		1800	450	
$U_9 - L_9$		1080	250	
$U_{10} - L_{10}$		560	180	
$U_{11} - L_{11}$		120	25	
$U_{12} - L_{12}$		200	70	

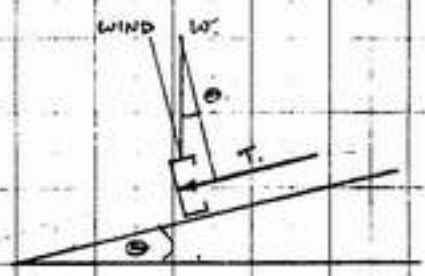
PROJECT. : _____ SHEET. NO.

OWNER. : _____

ARCHITECT. : _____ 01.

ENGINEER. : _____

DESIGN SAG ROD.



θ = 18.262

W = 80 x 2.50 x 11.50 = 2300 kg

T = 2300 Sin θ = 720 kg

A_{net} = T/Ft = 720/1500 = 0.48 cm^2.

USE STEEL ROUND BAR Ø 12 mm. ($A = 1.13\ cm^2$).

PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. 10.

DESIGN PURLIN.

SPAN = 5.00 m.

SPACING = 1.04 m.

W = 80 x 5 x 1.04 = 416 kg.

WIND = 20 x 5 x 1.04 = 100 kg

W_1 = 416 Cos 18.262 + 100 = 495 kg.

W_2 = 416 Sin 18.262 = 130 kg.

M_1 = $\frac{495 \times 5}{8}$ = 309 kg

M_2 = $\frac{1}{8}\left(130/2 \times 5/2\right)$ = 20 kg

Try $\square$ - 125 x 50 x 20 x 3.2 m.m.

A = 7.807 cm^2

Unit weight = 6.13 kg/m.

I_x = 181 cm^4 : I_y = 26.6 cm^4

Z_x = 29.0 cm^3 : Z_y = 8.02

$$f = \frac{309 \times 100}{29} + \frac{20 \times 100}{8.02} = 1066 + 249 = 1315 \text{ ksc.}$$

Fb = 0.6 Fy = 1500 ksc. > 1315 ksc.

USE PURLIN $\square$ - 125 x 50 x 20 x 3.2 m.m.

PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. 11.

DESIGN UPPER CHORD.

MEMBER $U_{10}-U_{11}$

COMPRESSION FORCE $P = 28,920$ kg.

LENGTH = 1.04 m.

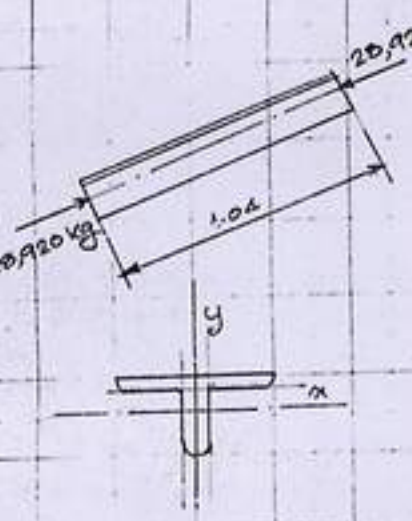
USE 2LS

ASSUMED $Fa = 0.4Fy = 1000$ ksc.

ASSUMED $A\ (1L) = \dfrac{P/2}{\text{Assumed Fa.}}$

$$A. = \frac{28,920/2}{1000} = 14.46\ cm^2$$

TRIAL 2LS 75×75×9mm. ($A\ 1L = 12.69\ cm^2$)



$r_x = r_y = 2.25$ cm.

FORMU $C_c = \sqrt{\dfrac{2\pi^2 E}{Fy}} = \sqrt{\dfrac{2\pi^2 \times 2.1\times10^6}{2500}} = 128.76$

$$KL/r = \frac{1.0\times1.04\times100}{2.25} = 46.22 < 200$$

$KL/r < C_c$ USE FORMULA

$$Fa = \frac{\left[1-\dfrac{(KL/r)^2}{2C_c^2}\right]Fy}{\dfrac{5}{3}+\dfrac{3}{8}\dfrac{(KL/r)}{C_c}-\dfrac{1}{8}\dfrac{(KL/r)^3}{C_c^3}}$$

$$Fa = \frac{\left[1-\dfrac{(46.22)^2}{2\times(128.76^2)}\right]2500}{\dfrac{5}{3}+\dfrac{3\times46.22}{8\times128.76}-\dfrac{(46.22)^3}{8\times(128.76^3)}} = \frac{2338.93}{1.795} = 1303\ ksc$$

$P = Fa \cdot A$

$= 1302\times(12.69\times2) = 33070$ kg $> 28,920$ kg.

USE 2LS 75×75×9mm. FOR UPPER CHORD

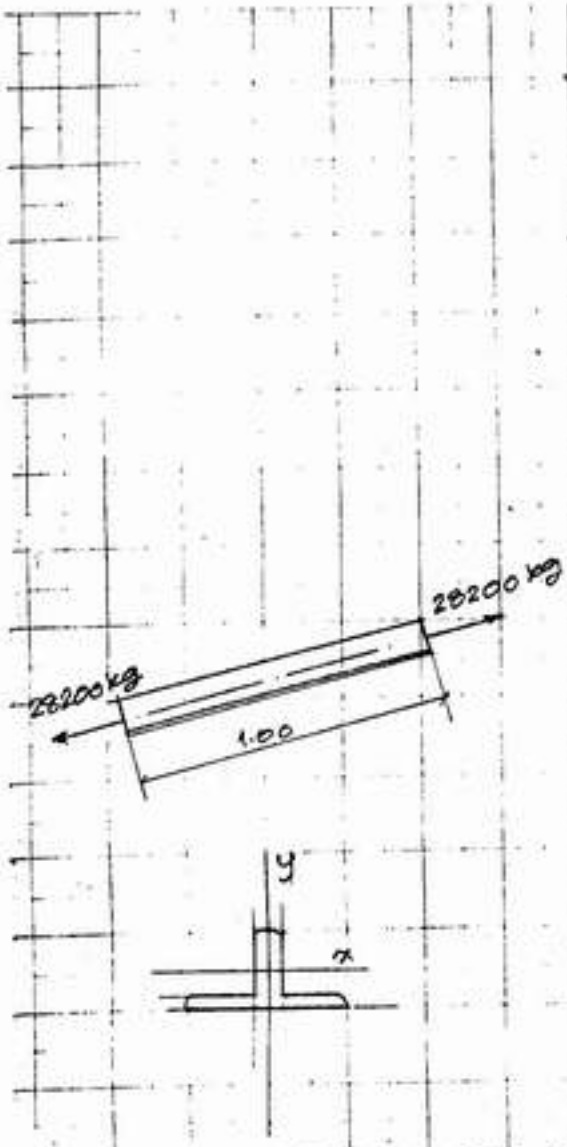
PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. 12.



DESIGN LOWER CHORD

MEMBER $L_{11}-L_{12}$

TENSION FORCE P = 28,200 kg

LENGTH = 1.00 m.

USE 2LS

$F_t = 0.6F_y = 1500$ kg/cm.

$A_{net} = \frac{P/2}{F_t} = \frac{28,200/2}{1500} = 9.4\ cm^2$

Try = 2LS 75x75x9 mm. ($A = 12.69\ cm^2$)

$I_x = 60.6\ cm^4$; $I_y = 60.6\ cm^4$

$r_x = 2.25$ cm. ; $r_y = 2.25$ cm.

$\frac{KL}{r} = \frac{1.0 \times 1.0 \times 100}{2.25} = 44.4 < 240$ ... ok.

USE 2 LS 75x75x 9 mm. FOR LOWER CHORD

PROJECT. : _____ SHEET. NO.

OWNER. : _____ 13.

ARCHITECT. : _____

ENGINEER. : _____

DESIGN OF VERTICAL MEMBER

MEMBER $U_2 - L_2$

COMPRESSION FORCE P = 5360 kg

LENGTH = 0.70 m.

USE 2 L's

ASSUMED Fa = 0.4 Fy = 1000 kg/c.

ASSUMED $A(IL) = \frac{5360}{1000} = 5.36\ cm^2$

TRIAL L-50x50x6 mm.

$A = 5.64\ cm^2$

$I_x = I_y = 12.6\ cm^4$

$Y_x = Y_y = 1.50$ cm.

$C_x = C_y = 1.44$ cm.

FORHE $C_c = \sqrt{\frac{2\pi^2 \times 2.1 \times 10^6}{2500}} = 128.76$

$KL/r = \frac{1.0 \times 0.70 \times 100}{1.50} = 46.66 < 200$

$KL/r < C_c$ USE FORMULA

$$Fa = \frac{\left[1 - \frac{(46.66)^2}{2 \times 128.76^2}\right] 2500}{\frac{5}{3} + \frac{3 \times 46.66}{8 \times 128.76} - \frac{46.66^3}{8 \times 128.76^3}} = \frac{2335.85}{180} = 1297\ kg/c$$

$P = Fa \cdot A = 1297 \times 5.64 = 7315\ kg > 5360\ kg$

USE L 50x50x6 mm. FOR VERTICAL MEMER.

PROJECT. :

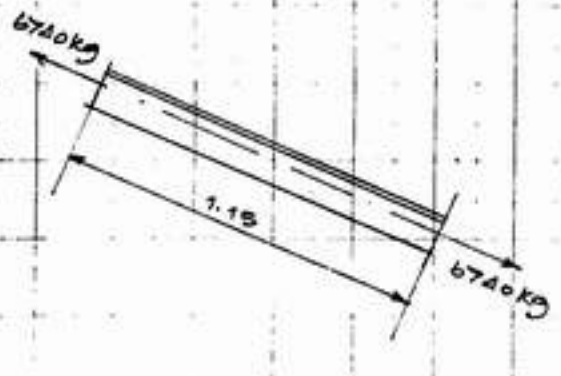
OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. 14.

DESIGN OF DIAGANAL MEMBER



MEMBER $U_2 - L_3$

TENSION FORCE P = 6740 kg.

LENGTH = 1.15 m.

Ft = 1500 ksc.

$A_{net} = \frac{P}{Ft} = \frac{6740}{1500} = 4.49\ cm^2$

Try LS 50x50x 6mm.

A = 5.60 cm^2

Ix = Iy = 12.24 cm^4

Yx = Yy = 1.50 cm.

Cx = Cy = 1.40 cm.

Zx = Zy = 3.08 cm^3

$KL/r = \frac{1.00 \times 1.15 \times 100}{1.50} = 76.66 < 240$

USE LS 50x50x 6mm. FOR DIAGANAL.

PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO.

15.

DESIGN OF CONNECTION FOR WELDED

MEMBER $U_2 - L_2$

COMPERSSION = 5360 kg

TH. 0.6 cm. LS 50x50x6 mm.

ขนาดใหญ่สุดของการเชื่อมแบบต่อทาบ 6 − 2 = 4 mm.

$P_w = 0.707 \times 0.4 \times 1470 = 415$ kg/cm

$\therefore L = \frac{5360}{415} = 12.91$ cm

$L_1 = \frac{1.44}{6.0} \times 12.91 = 3.09$ cm ⇒ 4 cm

$L_2 = \frac{4.56}{6.0} \times 12.91 = 9.81$ cm ⇒ 10 cm

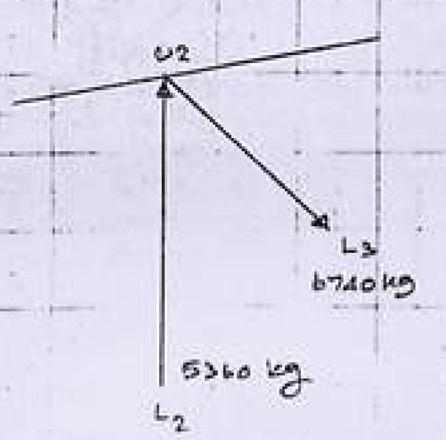
MEMBER $U_2 - L_3$

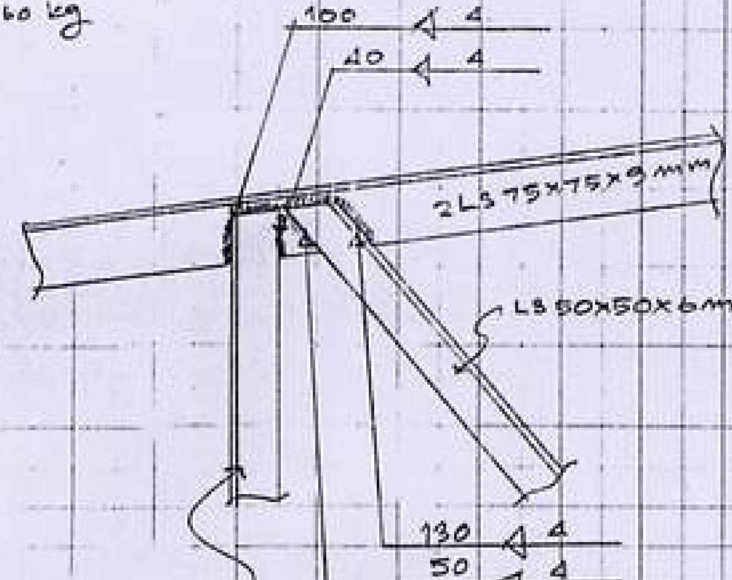
TENSION = 6740 kg

$L = \frac{6740}{415} = 16.24$ cm

$L_1 = \frac{1.44}{6} \times 16.24 = 3.9$ cm ⇒ 5 cm

$L_2 = \frac{4.56}{6} \times 16.24 = 12.30$ cm ⇒ 12 cm





PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. K.

DESIGN UPPER CHORD AND LOWER CHORD

MEMBER $U_1 - U_2$

COMPERSSION = 180 kg

LENGTH = 1.25 m.

Try = 1L^s 65x65x6mm. A = 7.53 cm^2

$r_x = r_y = 1.98$ cm.

$C_c = 128.76$

$KL/r = 63.13 < 200$

$$F_a = \frac{\left[1 - \frac{63.13^2}{2 \times 128.76^2}\right] 2500}{\frac{5}{3} + \frac{3 \times 63.13}{8 \times 128.76} - \frac{63.13^3}{8 \times 128.76^3}} = \frac{2199}{1.83} = 1198 \text{ KSC}$$

$P = 1198 \times 7.53 = 9,022$ kg

MEMBER $L_1 - L_2$

TENSION P = 143 kg Length = 1.25 m

F_t = 1500 KSC.

$A_{net} = 143/1500 = 0.09$ cm^2

Try = 1LS 65x65x6mm. A = 7.53 cm^2

$r_x = r_y = 1.98$ cm.

$KL/r = \frac{1.0 \times 1.25 \times 100}{1.98} = 63.13 < 240$ ... ok.

PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. 17.

DESIGN TRUSS T-2, T-3, T-4

MEMBER $U_0 - L_1$

TENSION. $P = 270$ kg.

Length $= 2.25$ m.

$F_t = 1500$ KSC

$A_{net} = \frac{270}{1500} = 0.18\ cm^2$

Try 1L3 50x50x6 mm. $A = 5.64\ cm^2$

$r_x = r_y = 1.50$ cm

$kL/r = \frac{1.0 \times 2.25 \times 100}{1.50} = 170 < 240$ ... ok

MEMBER $U_0 - L_0$

COMPERSSION $P = 300$ kg

LENGTH $= 2.10$ m.

Try $= 50 \times 50 \times 6$ mm $A = 5.64\ cm^2$

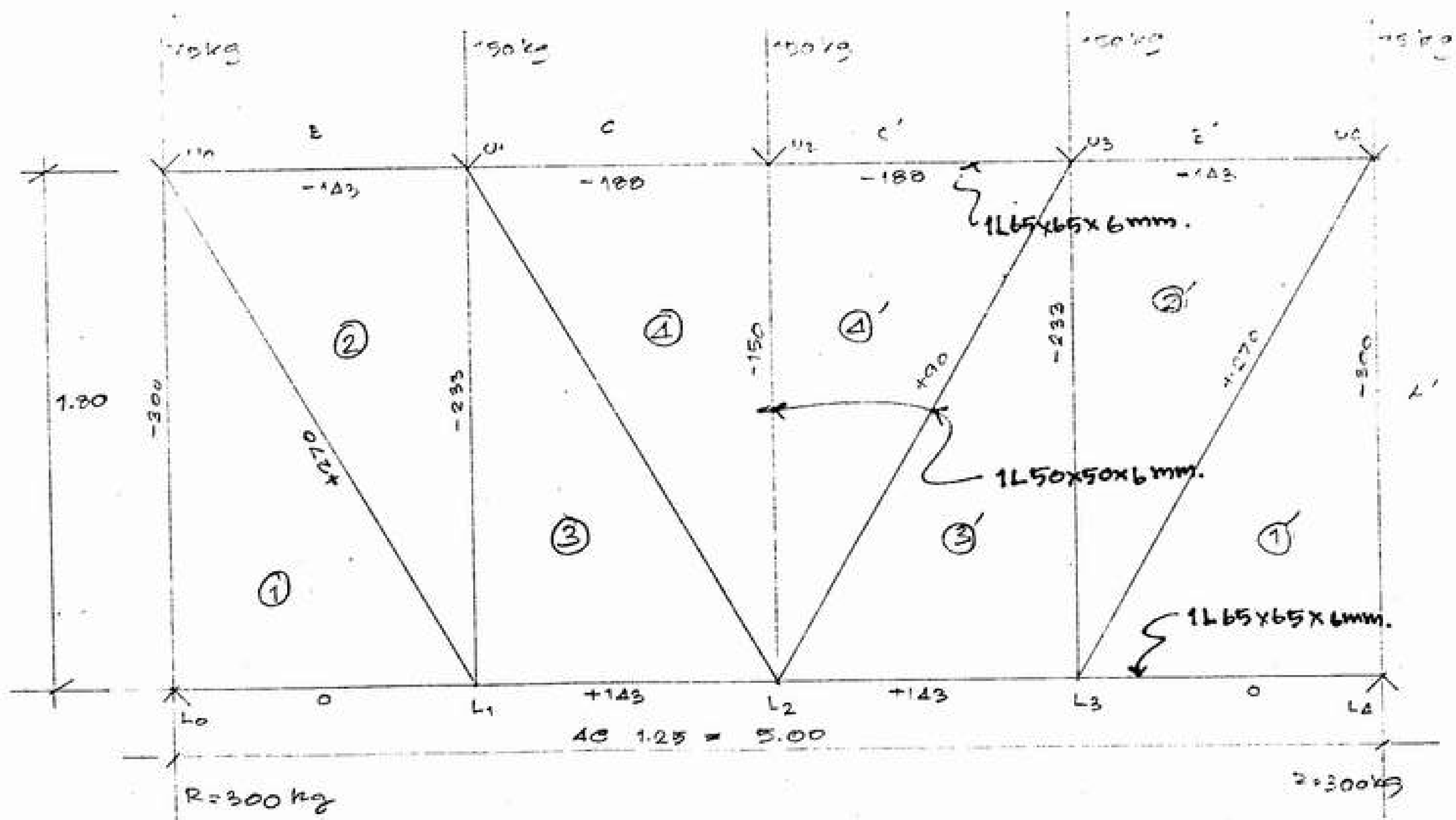
$r_x = r_y = 1.50$

$C_c = 128.76$

$kL/r = 140 < 200$

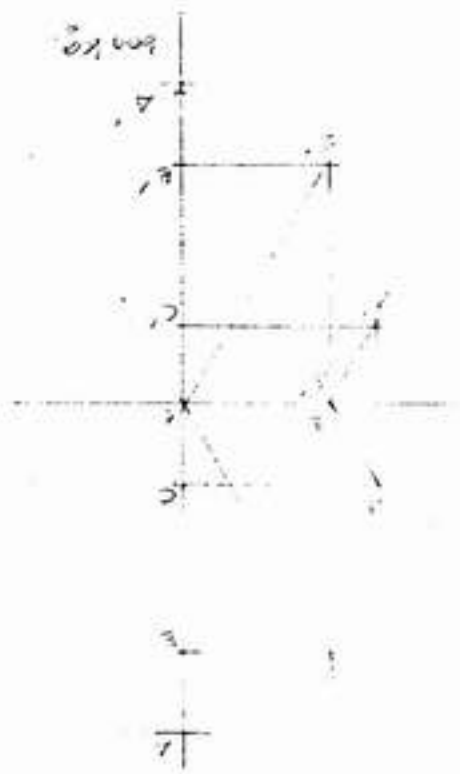
$$F_a = \frac{\left[1 - \frac{140^2}{2 \times 128.76^2}\right] 2500}{\frac{5}{3} + \frac{3 \times 140}{8 \times 128.76} - \frac{140^3}{8 \times 128.76^3}} = \frac{1022}{1.90} = 537$$

$P = 537 \times 5.64 = 3033$ kg ... OK.



1L65x65x6mm.
1L50x50x6mm.
1L65x65x6mm.
1.80
-300
+270
-233
-150
+90
-143
-180
-188
+143
0
L0
L1
L2
L3
L4
U2
U3
4@ 1.25 = 5.00
R=300 kg

SCALE 1:25


FORCE CONSTANTS

PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. 20.

REINFORCED CONCRETE DESIGN.

PROJECT :

OWNER :

ARCHITECT :

ENGINEER :

DESIGN CRITERIA

CONCRETE : f_c' = 173 KSC.

f_c = 65 KSC

n = 11

j = 0.876

k = 0.373

R = 10.62 KSC

V_c = 3.31 KSC

STEEL RB ≤ DB

f_s = 1200 KSC

f_y = 2400 KSC

f_v = 1200 KSC

LIVE LOAD :

FOR ROOF = 50 kg/m^2.

DEAD LOAD

FOR CONCRETE = 2400 kg/m^3.

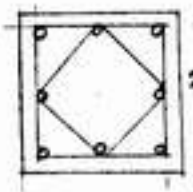
PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. 21.



$C_1 = 0.25 \times 0.25$

DESIGN COLUMN C1

SHORT COLUMN Max Load = 9,220 kg

Sect = 0.25 x 0.25 m.

h/t = 5.00/.25 = 20 > 15 LONG COLUMN.

As = 0.01 - 0.08

P = 0.85 x 625 x (.25 x 173 + 0.01 x 1200) = 29,351 kg

As = 0.01 x 625 = 6.25 cm^2 ⇒ 8 Ø 12mm 2/ø 6mm @0.20

LONG COLUMN.

P = 29,351 kg

R = 1.07 - 0.008 (h/r)

h = 1 x 5 x 100 = 500 cm.; r = 0.25 x 25 = 6.25 cm

h/r = 500/6.25 = 80 < 100

R = 1.07 - 0.008 (80) = 0.43 < 1.0

P' = 29,351 x 0.43 = 12,620 kg. > 9,220 kg. . . ok

PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO. 22.

DESIGN BEAM GB1

GB1 = 0.15×0.40

Sect = 0.15×0.40 d = 0.35

W = 744 kg

Moment = $0.10 \times 744 \times 5^2$ = 1860 kg-m.

Mc = $10.62 \times .15 \times 35^2$ = 1951 kg-m.

As = 5.05 cm^2 ⇒ 5ϕ12mm. As = 5.65 cm^2.

Vmax = 1860 kg

Vc = 3.81×15×35 = 2000 kg ⇒ Vϕ6mm @0.20 m.

DESIGN FOOTING

MAX LOAD = 9,220 kg

PF 10% = 922 kg

Tw = 10142 kg

USE PILE = I-18×18×14.00 m. SAFT LOAD = 7.5 T/PILE

USE PILE = 2 ต้น

$V = \frac{10142}{2}$ = 5071 kg

Moment = $5071 \left(\frac{0.50-0.25}{2}\right)$ = 887 kg-m.

d = 12.92 cm. USE d = 0.35 cm.

V = P' = 2536 kg

$v = \frac{2536}{50 \times 35}$ = 1.44 < $0.53\sqrt{173}$ = 6.97 ksc.

$A_s = \frac{887}{1200 \times .876 \times 35}$ = 2.41 cm^2 ⇒ 4ϕ12mm. As = 4.52 cm^2

$A_s' = 0.0029 \times 110 \times 50$ = 13.75 cm^2. ⇒ 13ϕ12mm. As = 14.69 cm^2

PROJECT. :

OWNER. :

ARCHITECT. :

ENGINEER. :

SHEET. NO.

27.

DETAIL FOOTING