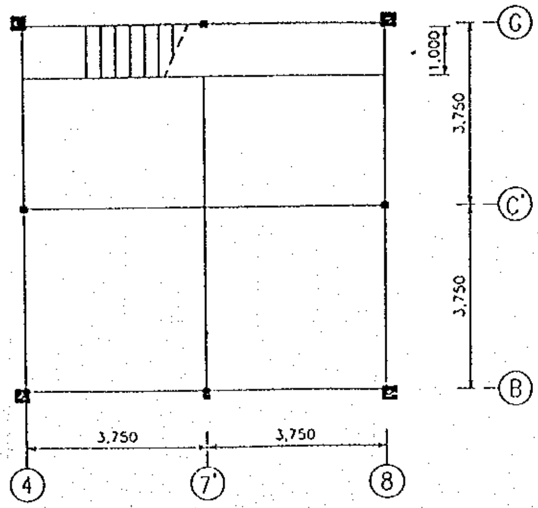
4.7. WEIR MANAGEMENT COMPLEX AND GATE CONTROL HOUSE

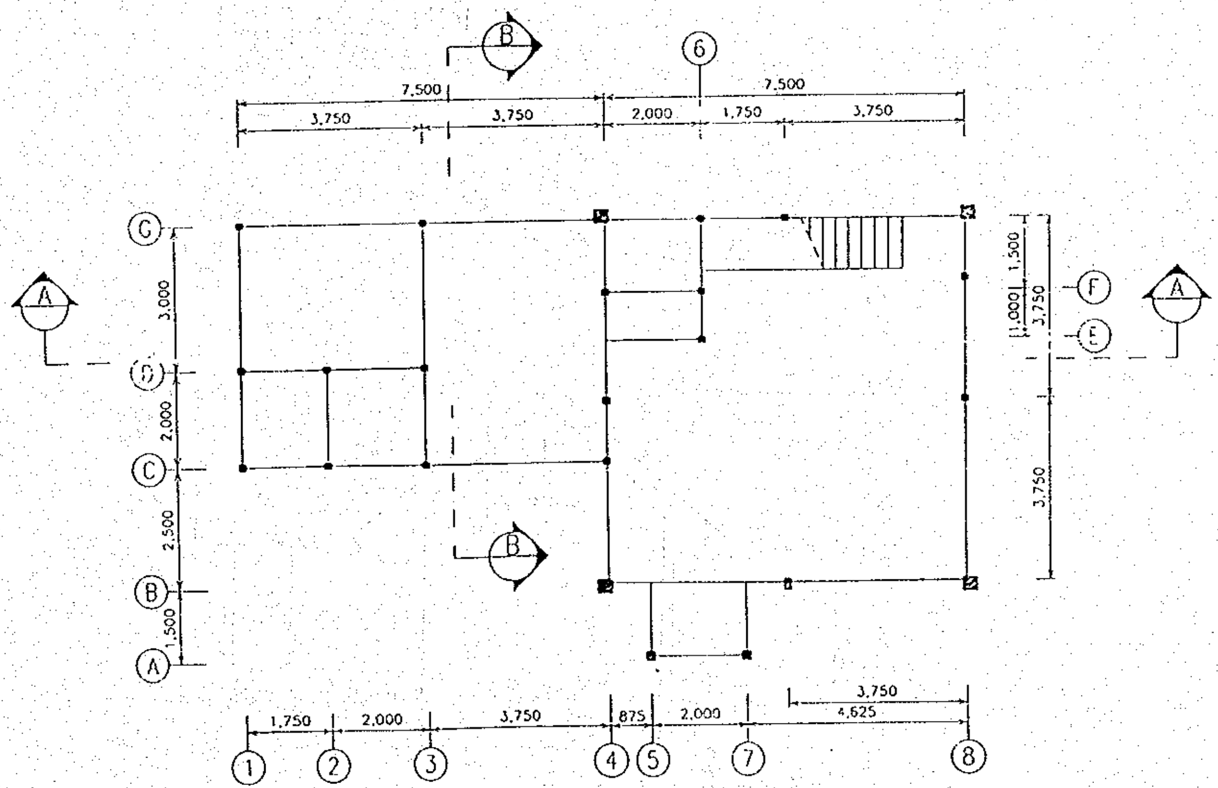
4.7.1. OPERATION/MANAGEMENT BUILDING STRUCTURE CALCULATION

1 STRUCTURE
2 DESIGN CONDITION
3 LOADING CONDITION
4 DESIGN OF PURLIN
5 DESIGN OF ROOF TRUSS
6 DATA FOR TWO STORIES
  a. DIMENSIONS
  b. DESIGN CONDITION
  c. LOADING CONDITION
  d. DESIGN OF REINFORCEMENT CONCRETE PLATE
7 DESIGN OF FOOTING

1. STRUCTURE



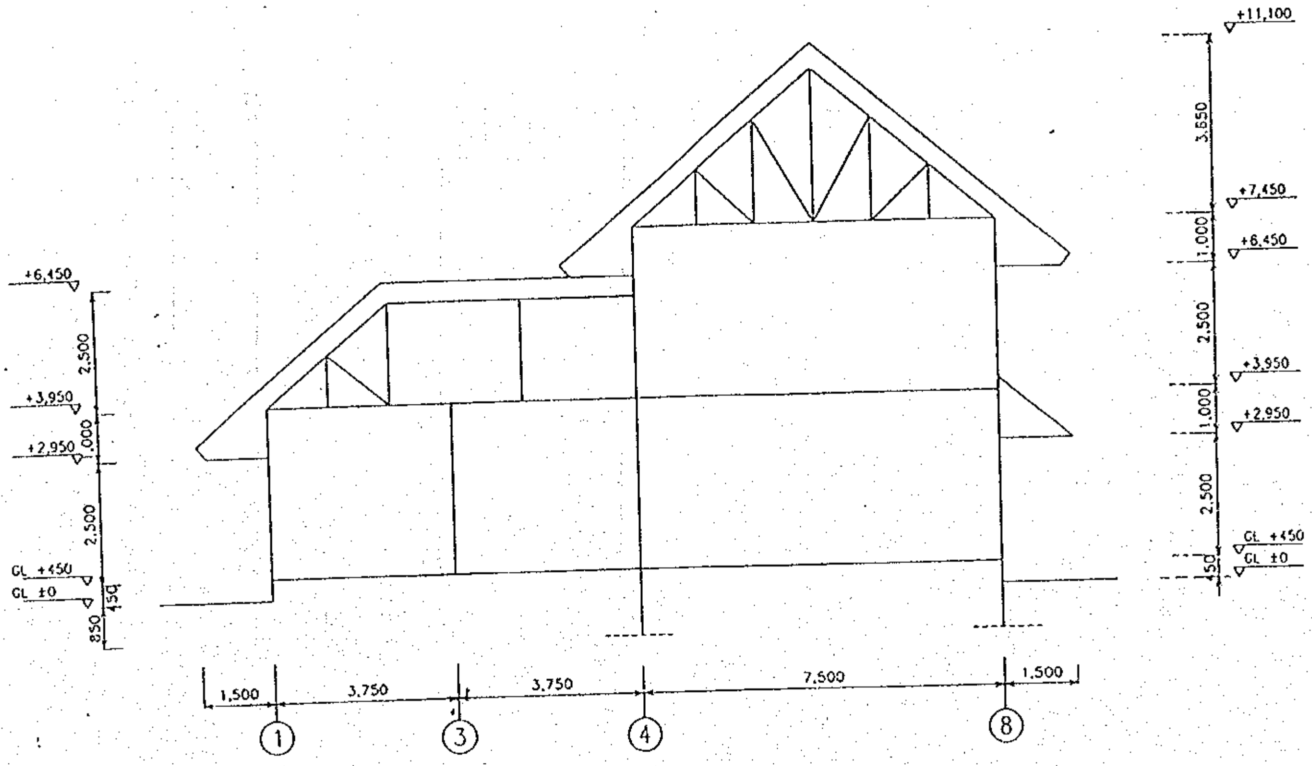
2nd FLOOR PLAN



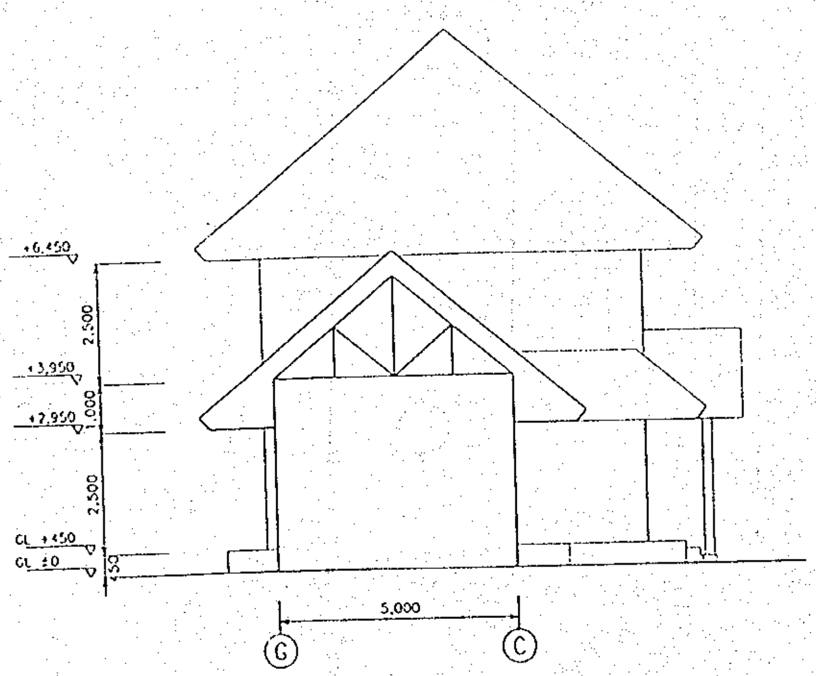
1st FLOOR PLAN

OPERATION/MANAGEMENT BUILDING

SIMONGAN WIER MANAGEMENT COMPLEX



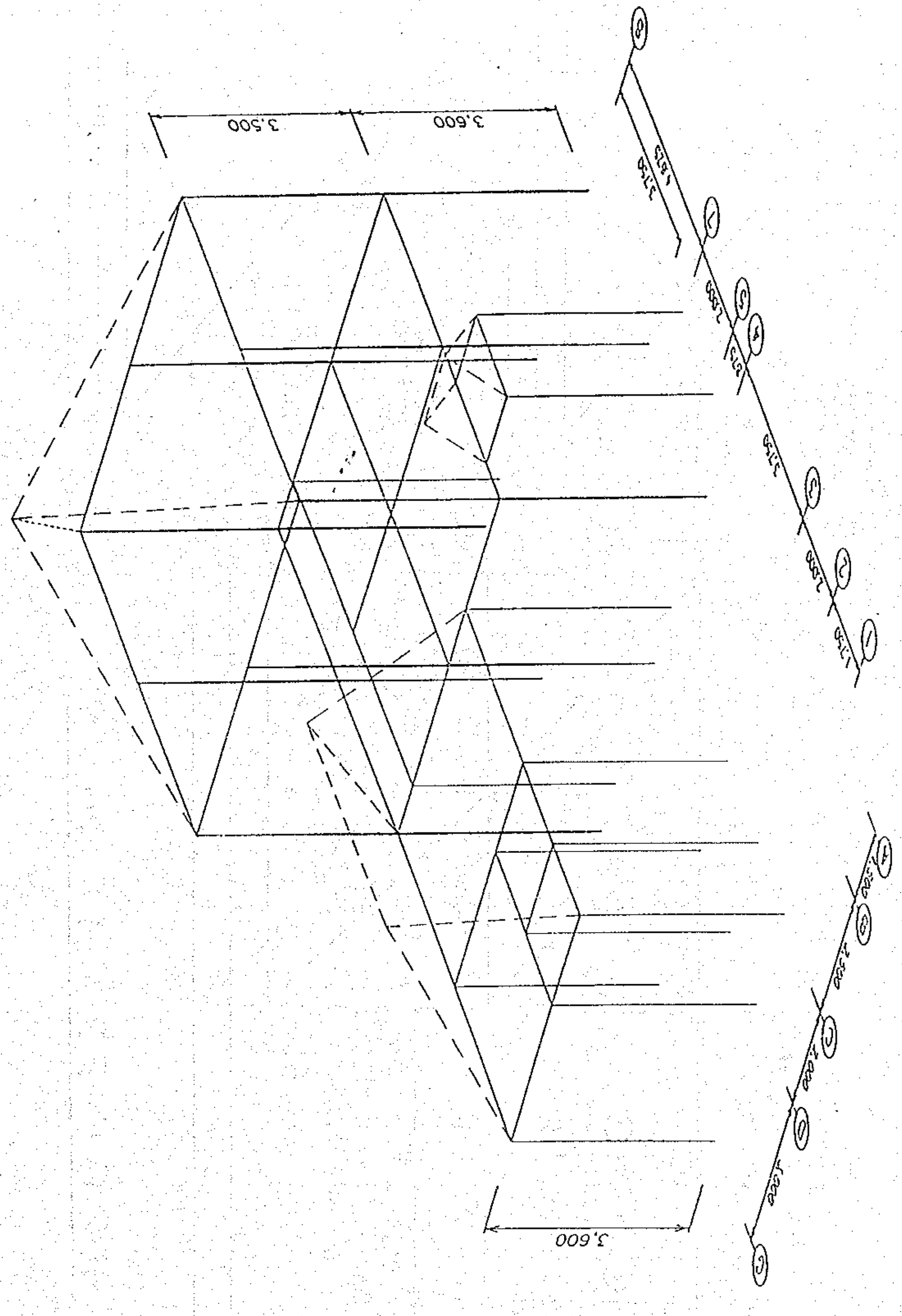
SECTION A-A



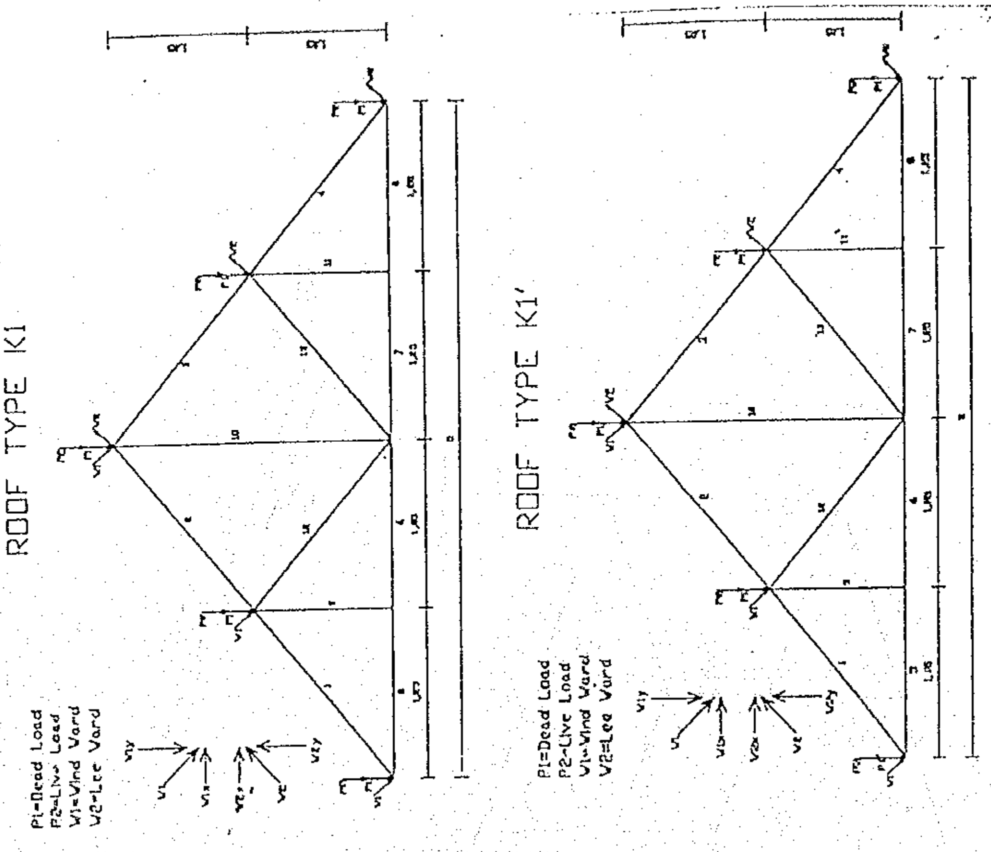
SECTION B-B

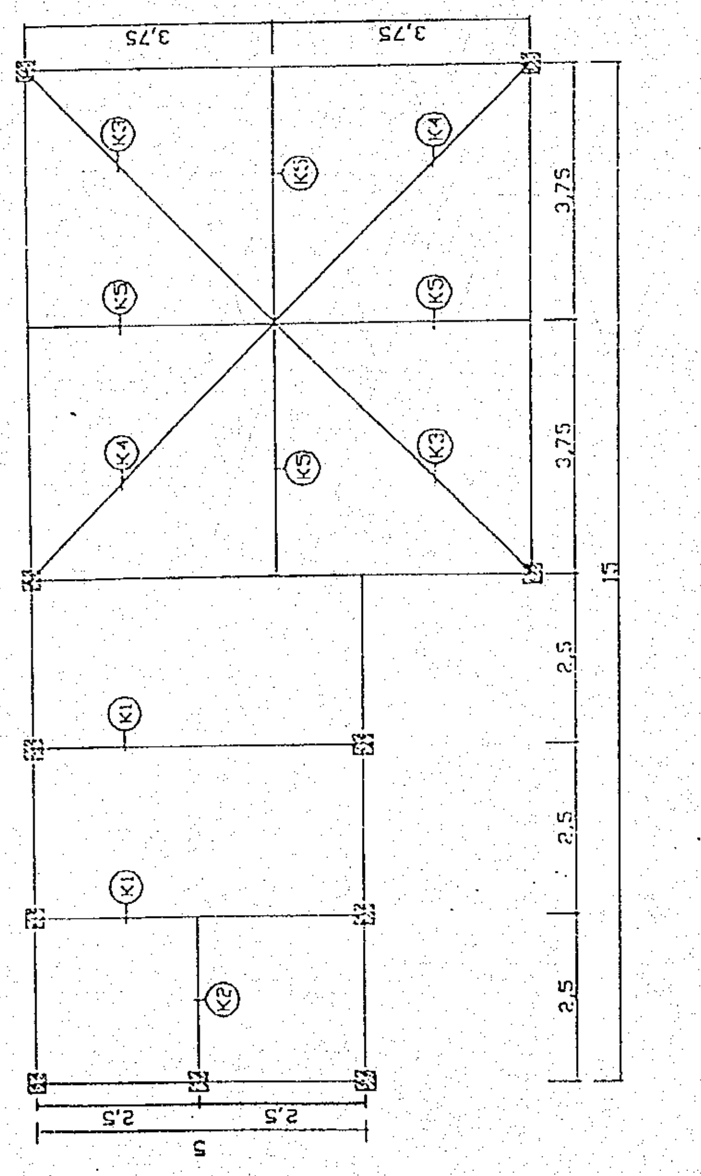
OPERATION/MANAGEMENT BUILDING

SIMONGAN WIER MANAGEMENT COMPLEX



ISOMETRY
OPERATION/MANAGEMENT BUILDING
SIMONGAN WEIR MANAGEMENT COMPLEX





ROOF FRAME PLAN

2. Design Condition

a) Roof truss members : - double angle steel
- Tensile strength (Fy) : 2400 kg/cm^2

b) Structural model : plane (xy axis) truss, linear elastic
c) Analysis method : static

3. Loading Condition

a) Dead Load :
- Roof cover (ceramic tile + timber rafter) = 70 kg/m^2
- Ceiling (fibre cement) = 10 kg/m^2
80 kg/m^2

b) Live load
- Weight of workers as point load = 100 kg

c) Wind load
- Wind pressure = 40 kg/m^2
- Pressure coefficient (f)
. wind ward -0.5
. lee ward -0.4

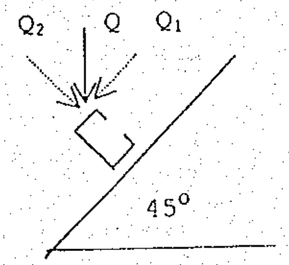
$W1 = 0.5 \times 40\ kg/m^2 = 20\ kg/m^2$
$W2 = 0.4 \times 40\ kg/m^2 = 16\ kg/m^2$

4. Design of purlin

- Purlin distance (c/c) = 1.77 m
- Purlin length = 2.50 m
- Purlin self weight say = 8.00 kg/m'

→ $q_1 = 1.77 \times 80\ kg/m^2 \approx 142\ kg/m'$
→ q_2 (self weight) $\approx 8\ kg/m'$

Q $\approx 150\ kg/m'$



$Q_1 = Q_2 = Q \cos 45^o$
$= 150 \cos 45^o$
$= 106\ kg/m'$

<u>Point load</u>

$Px = Py = P\cos \alpha = 100 \cos 45° \approx 71$ kg

<u>Bending moment</u>

$Mx = 1/8 \times q_1 \times L^2 + 1/P1 \times L$

$= 1/8 \times 106 \times 2.5^2 + 1/4 \times L$

$= 127.19$ kgm $= 12.719$ kgcm

<u>Try light lip channel type :</u>

150 x 50 x 50 x 4.5

$Ix = 438$ cm^4 ; $Wx = 58.4$ cm^3

$Iy = 71.4$ cm^4 ; $Wy = 13.2$ cm^3

<u>Stresses</u>

$\sigma_{all} = 0.6 \times Fy = 0.6 \times 2.400$

$= 1.440$ kg/cm^2

$\sigma = \sigma x \times \sigma y$

$= Mx/Wx + My/Wy$

$= (12{,}719/58.4) + (12{,}719/13.2)$

$= 1{,}181$ kg $< \sigma_{all} = 1.440$ kg/cm^2 (OK)

<u>Deflection</u>

$Fx = 5/384 \times Q1 \times L^4/EIx + 1PL^3/48EIx$

$= 5/384 \times 106 \times 250^4/(2.1 \times 10^6 \times 438) + 71{,}250^3/(48 \times 2.1 \times 10^6 \times 438)$

$= 0.09 + 0.03 = 0.12$ cm

$Fy = 0.36 + 0.15 = 0.51$ cm

$f = (0.12^2 + 0.51^2)^{1/2} = 0.52$ cm

$f = 0.52$ cm $< f_{all} = 1L/360 = 250/360 = 0.69$ (OK)

For purlin span up to 3.75 m, use the lip channel type of 2 x C 150 x 65 x 20 x 3.2 (in front to front arrangement)

5. Design of roof truss

a. Dead load

P1 = 2.50 x 1.77 x 80 = 354 kg

b. Wind load

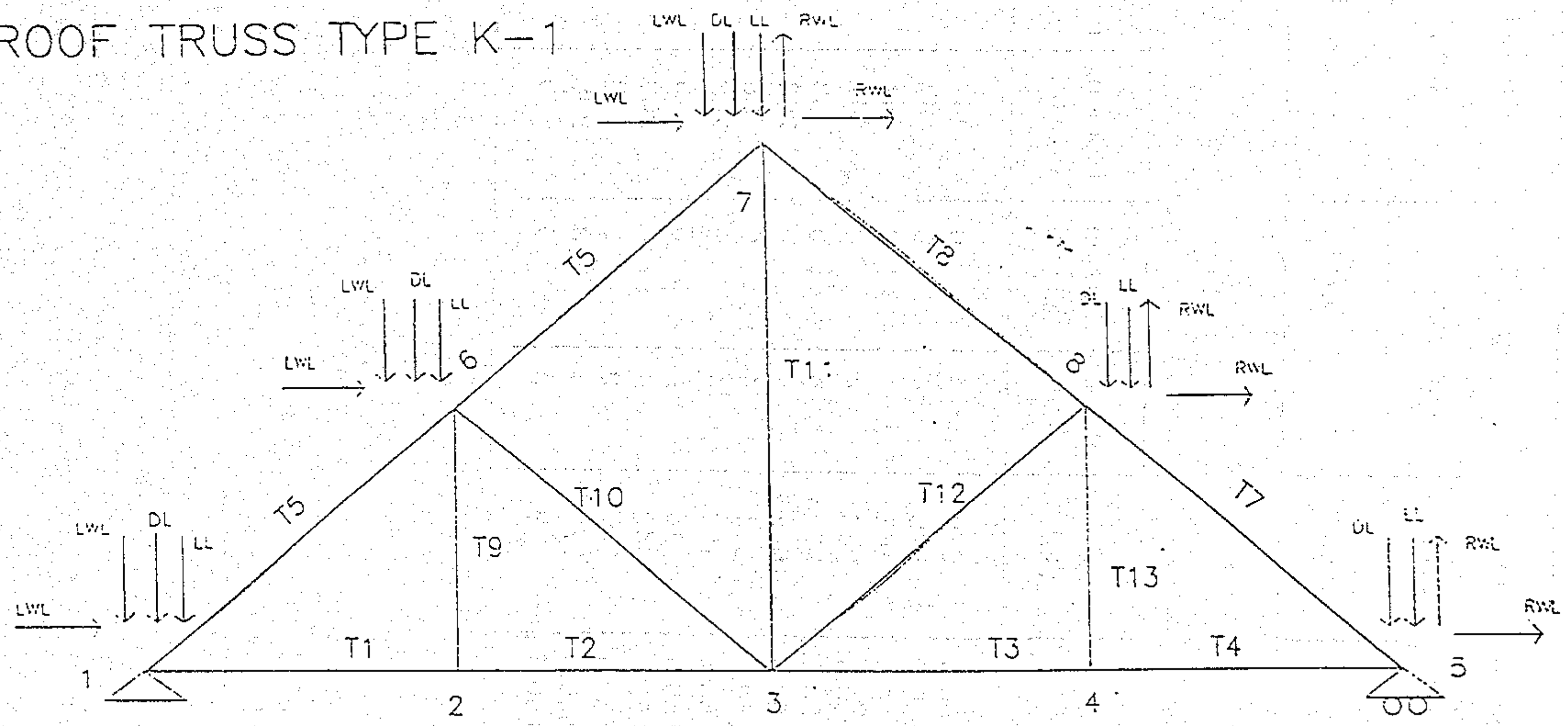
W1 = 2.50 x 1.77 x 20 = 89 kg

W2 = 2.50 x 1.77 x 16 = 71 kg

c. Live load

P2 = 100 kg

DESIGN LOADING
ROOF TRUSS TYPE K-1



NOTE
Dead load (DL) = 354 kg
Wind load :
- Left Wind Load (LWL) = 63 kg (downward)
63 kg (rightward)
- Right wind Load (RWL) = 50 kg (downward)
50 kg (rightward)
Live load (LL) = 100 kg.

Roof K-1

Prototype

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia. Bolt (cm)
L 50.50.5	0.8	2,400	3,700	1.4

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L 50.50.5	979	4	0	147	2	14
2	L 50.50.5	979	4	0	147	2	14
3	L 50.50.5	866	4	0	147	2	14
4	L 50.50.5	866	4	0	147	2	14
5	L 50.50.5	990	4	0	208	2	14
6	L 50.50.5	642	4	0	208	2	14
7	L 50.50.5	811	4	0	208	2	14
8	L 50.50.5	1,159	4	0	208	2	14
9	L 50.50.5	18	0	0	0	2	14
10	L 50.50.5	442	4	0	208	2	14
11	L 50.50.5	547	0	0	0	2	14
12	L 50.50.5	273	4	0	208	2	14
13	L 50.50.5	9	0	0	0	2	14

- Checking of members Strength of roof steel Truss Type K-1 base on the axial force:

  a. Due to Tensile force

  Maximum force on member T1 & T2 (loading Combination 2)
  Force $F = 979.7 \approx 980$ kg
  Length $L = 125$ cm

  Try : Double angle steel of 50.50.5
  Cross section area $A = 2 \times 4.8 = 9.6\ cm^2$

  $\sigma_{all} = 0.6 x Fy$
  $= 0.6 x 2,400 = 1,440\ kg/cm^2$

  Stress
  $\sigma = \frac{F}{A} = \frac{980}{9.6} = 102\ kg/cm^2 < \sigma_{all} = 1,440\ kg/cm^2$ (ok)

  b. Due to Compresion force

  Maximum force on member T5 (loading Combination 2)
  Force $F = 990.577 \approx 991$ kg (rounded)
  Length $L = 176.78$ cm

  Try : Double angle steel of 50.50.5
  Cross section area $A = 9.6\ cm^2$
  $ix = 1.51$ cm ; $Ix = 2x11 = 22\ cm^4$
  $\lambda = \frac{L}{ix} = \frac{176.78}{1.51} = 117.07 > 105$

  by Euler Formula

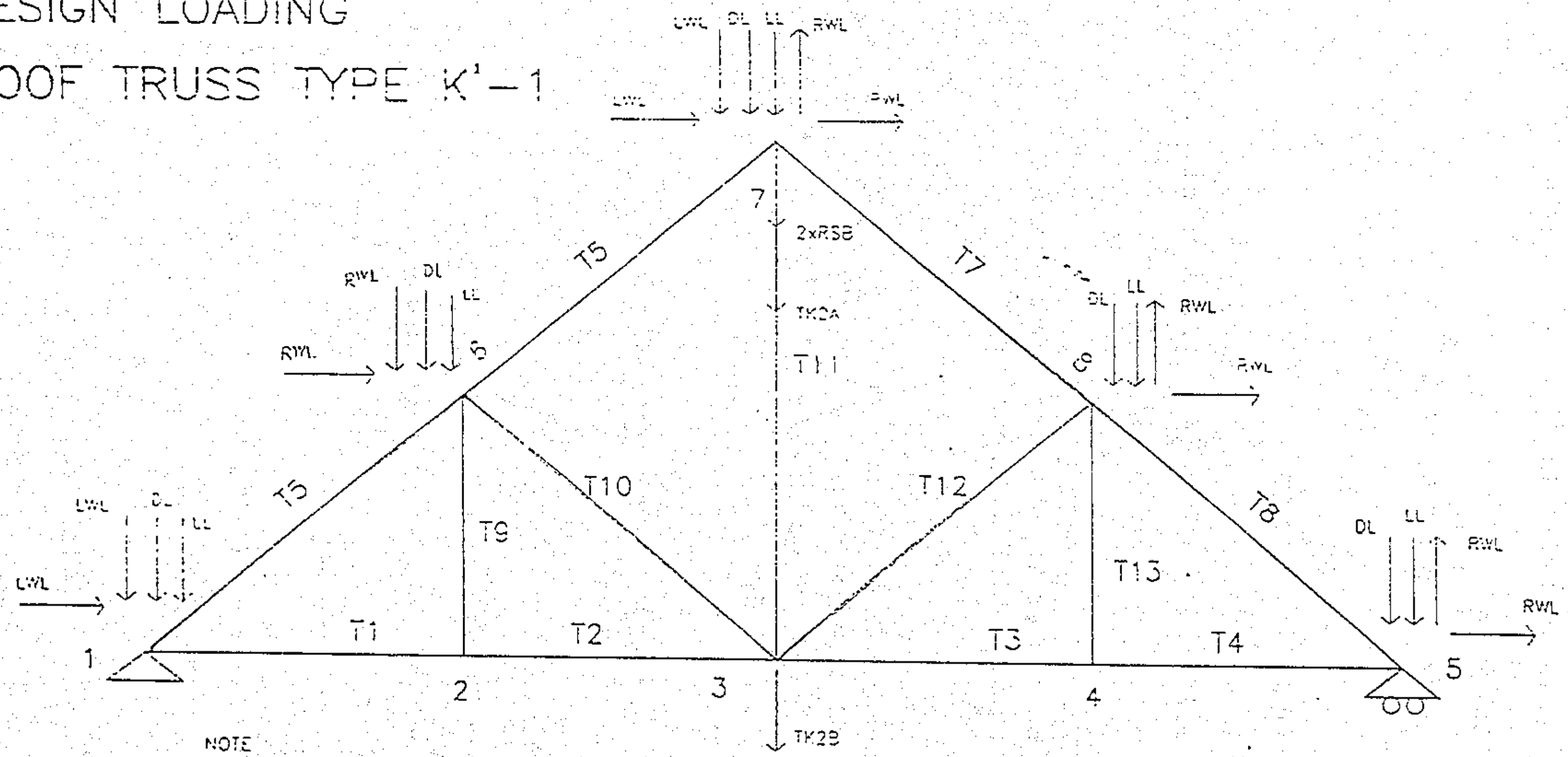
  $F_{all} = \frac{\pi^2 . E . Ix}{n . L^2}$; n = Safety Factor = 3

  $F_{all} = \frac{\pi^2 x (2.1x10^6) x 22}{3x(176.78)^2}$
  $= 4,863.56$ kg $> F = 991$ kg (ok)

Hence double angle steel of 50.50.5 can be used as the members of roof truss type K-1

DESIGN LOADING

ROOF TRUSS TYPE K'-1



NOTE
Dead load (DL) = 354 kg
- Roof Slope Beam (RSB) = 2 x 486 kg.
- Truss Type K-2 (TK2A) = 663 kg.
- Truss Type K-2 (TK2B) = 321 kg.

Wind load :
- Left Wind Load (LWL) = 63 kg (downward)
63 kg (rightward)
- Right Wind Load (RWL) = 50 kg (downward)
50 kg (rightward)

Live load (LL) = 100 kg.

Roof K'-1

Prototype

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia. Bolt (cm)
L 50.50.5	0.8	2,400	3,700	1.4
P101.6B	0.8	2,400	3,700	1.4

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L 50.50.5	1,924	2	0	73	2	14
2	L 50.50.5	1,924	2	0	73	2	14
3	L 50.50.5	1,811	2	0	73	2	14
4	L 50.50.5	1,811	2	0	73	2	14
5	L 50.50.5	2,329	2	0	104	2	14
6	L 50.50.5	199	2	0	104	2	14
7	L 50.50.5	215	2	0	104	2	14
8	L 50.50.5	248	2	0	104	2	14
9	L 50.50.5	9	2	0	0	2	14
10	L 50.50.5	426	2	0	104	2	14
11	P101.6B	828	2	0	0	2	14
12	L 50.50.5	281	2	0	104	2	14
13	L 50.50.5	4	2	0	0	2	14

- Checking of members Strength of roof steel Truss Type K'-1 base on the axial force:

  a. Due to Tensile force

  Maximum force on member T1 & T2 (loading Combination 2)
  Force F = 2,485 kg
  Length L = 125 cm

  Try : Double angle steel of 50.50.5
  Cross section area A = 2 x 4.8 = 9.6 cm^2

  σ_{all} = 0.6xFy
  = 0.6x2,400 = 1,440 kg/cm^2

  Stress
  $\sigma = \frac{F}{A} = \frac{2,485}{9.6} = 258,85\ kg/cm^2 < \sigma_{all} = 1,440\ kg/cm^2$ (ok)

  b. Due to Compresion force

  Maximum force on member T8 (loading Combination 2)
  Force F = 3,289 kg (rounded)
  Length L = 176.78 cm (rounded)

  Try : Double angle steel of 50.50.5
  Cross section area A = 9.6 cm^2
  ix = 1.51 cm ; Ix = 2x11 = 22 cm^4
  $\lambda = \frac{L}{ix} = \frac{176.78}{1.51} = 117.07 > 105$

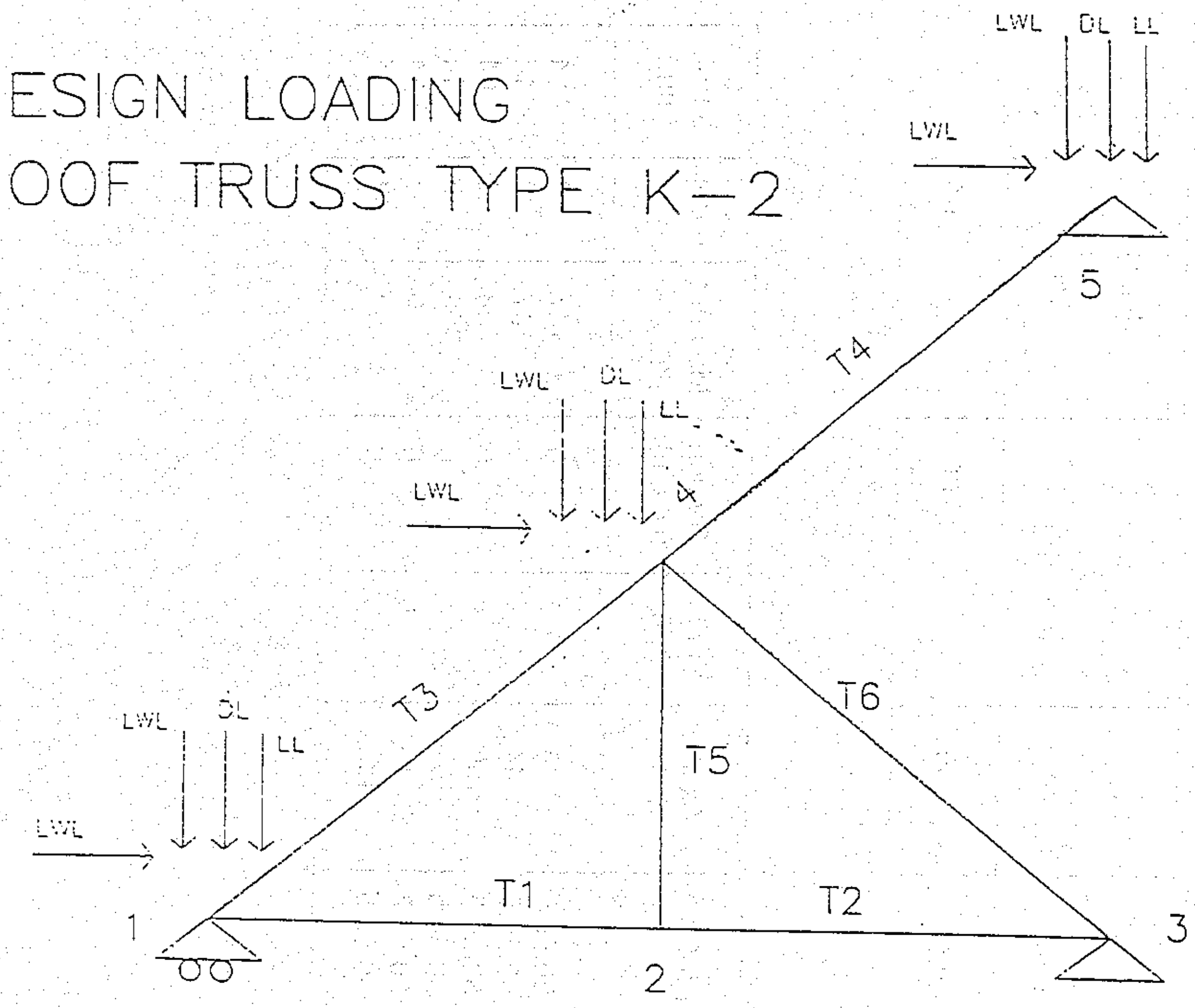
  by Euler Formula

  $F_{all} = \frac{\pi^2 . E . Ix}{n . L^2}$; n = Safety Factor = 3

  $F_{all} = \frac{\pi^2 x (2.1x10^6) x22}{3x(176.78)^2}$
  = 4,863.56 kg > F = 3,289 kg (ok)

Hence double angle steel of 50.50.5 can be used as the members of roof truss type K'-1

DESIGN LOADING
ROOF TRUSS TYPE K−2



NOTE
Dead load (DL) = 354 kg
Wind load :
− Left Wind Load (LWL) ≒ 63 kg (downward)
63 kg (rightward)

Live load (LL) = 100 kg.

Roof K-2

Prototype

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia. Bolt (cm)
L 50.50.5	0.8	2,400	3,700	1.4

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L 50.50.5	44	4	0	147	2	14
2	L 50.50.5	44	4	0	147	2	14
3	L 50.50.5	156	4	0	208	2	14
4	L 50.50.5	191	4	0	208	2	14
5	L 50.50.5	18	0	0	0	2	14
6	L 50.50.5	432	4	0	208	2	14

- Checking of members Strength of roof steel Truss Type K-2 base on the axial force:

  a. Due to Tensile force

  Maximum force on member T4 (loading Combination 2)
  Force $F = 201.19$ kg
  Length $L = 176.78$ cm

  Try : Double angle steel of 50.50.5
  Cross section area $A = 2 \times 4.8 = 9.6$ cm

  $$\sigma_{all} = 0.6xFy$$
  $$= 0.6x2{,}400 = 1{,}440 \text{ kg/cm}^2$$

  Stress
  $$\sigma = \frac{F}{A} = \frac{201.19}{9.6} = 20.96 \text{ kg/cm}^2 < \sigma_{all} = 1{,}440 \text{ kg/cm}^2 \text{ (ok)}$$

  b. Due to Compresion force

  Maximum force on member T6 (loading Combination 2)
  Force $F = 442.28$ kg
  Length $L = 176.78$ cm

  Try : Double angle steel of 50.50.5
  Cross section area $A = 2x4.8 = 9.6$ cm^2
  $ix = 1.51$ cm ; $Ix = 2x11 = 22$ cm^4
  $$\lambda = \frac{L}{ix} = \frac{176.78}{1.51} = 117.07 > 105$$

  by Euler Formula

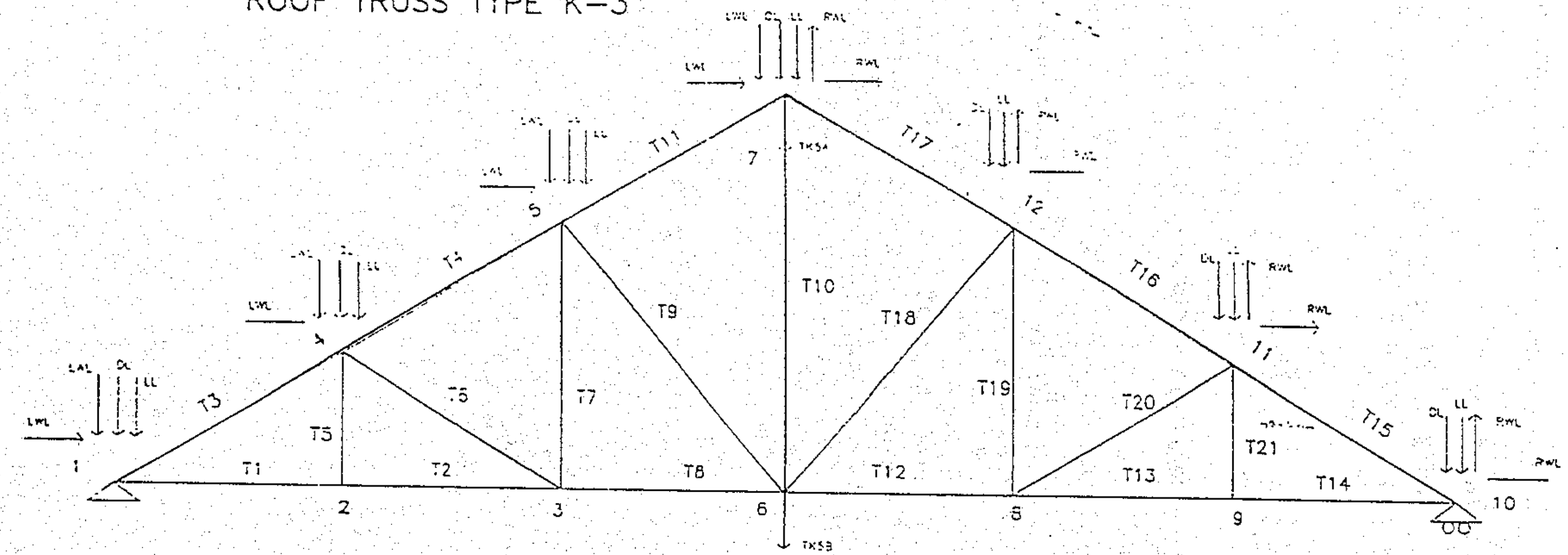
  $$F_{all} = \frac{\pi^2.E.Ix}{n.L^2} \quad ; n = \text{Safety Factor} = 3$$

  $$F_{all} = \frac{\pi^2 x(2.1x10^6)x22}{3x(176.78)^2}$$
  $$= 4{,}863.56 \text{ kg} > F = 442.28 \text{ kg (ok)}$$

Hence double angle steel of 50.50.5 can be used as the members of roof truss type K-2

DESIGN LOADING

ROOF TRUSS TYPE K–3



NOTE

Dead load (DL) = 354 kg

- Truss Type M-5 (TXS4) = [illegible] kg.
- Truss Type K-5 (TXS3) = 1,352 kg.

Wind load :

- Left Wind Load (LWL) = 53 kg (downward)
  53 kg (rightward)
- Right Wind Load (RWL) = 50 kg (downward)
  50 kg (rightward)

Live load (LL) = 100 kg.

Roof K-3

Prototype

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia. Bolt (cm)
L 60.60.6	0.8	2,400	3,700	1.7
P101.6B	0.8	2,400	3,700	1.7

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L 60.60.6	6,668	12	0	555	4	17
2	L 60.60.6	6,668	12	0	555	4	17
3	L 60.60.6	7,678	12	0	679	4	17
4	L 60.60.6	7,191	12	0	679	4	17
5	L 60.60.6	25	0	0	0	2	17
6	L 60.60.6	572	12	0	679	2	17
7	L 60.60.6	401	0	0	0	2	17
8	L 60.60.6	6,207	12	0	555	3	17
9	L 60.60.6	811	12	0	960	2	17
10	P101.6B	3,979	0	0	0	4	17
11	L 60.60.6	6,699	12	0	679	4	17
12	L 60.60.6	6,071	12	0	555	3	17
13	L 60.60.6	6,395	12	0	555	3	17
14	L 60.60.6	6,395	12	0	555	3	17
15	L 60.60.6	7,758	12	0	679	4	17
16	L 60.60.6	7,301	12	0	679	4	17
17	L 60.60.6	6,820	12	0	679	4	17
18	L 60.60.6	575	12	0	690	2	17
19	L 60.60.6	269	0	0	0	2	17
20	L 60.60.6	387	12	0	679	2	17
21	L 60.60.6	25	0	0	0	2	17

- Checking of members Strength of roof steel Truss Type K-3 base on the axial force:

  a. Due to Tensile force

  Maximum force on member T1 & T2 (loading Combination 2)
  Force $F = 8,524.58$ kg
  Length $L = 177$ cm

  Try : Double angle steel of 60.60.6
  Cross section area $A = 2 \times 6.91 = 13.81\ cm^2$

  $\sigma_{all} = 0.6xFy$
  $= 0.6x2,400 = 1,440\ kg/cm^2$

  Stress
  $$\sigma = \frac{F}{A} = \frac{8,524.58}{13.81} = 617.28\ kg/cm^2 < \sigma_{all} = 1,440\ kg/cm^2$$

  b. Due to Compresion force

  Maximum force on member T15 (loading Combination 2)
  Force $F = 4,812.42$ kg
  Length $L = 216.70$ cm

  Try : Double angle steel of 60.60.6
  Cross section area $A = 2 \times 6.91 = 13.82\ cm^2$
  $ix = 1.80$ cm ; $Ix = 2x22.8 = 45.6\ cm^4$
  $$\lambda = \frac{L}{ix} = \frac{216.7}{1.80} = 120.39 > 105$$

  by Euler Formula

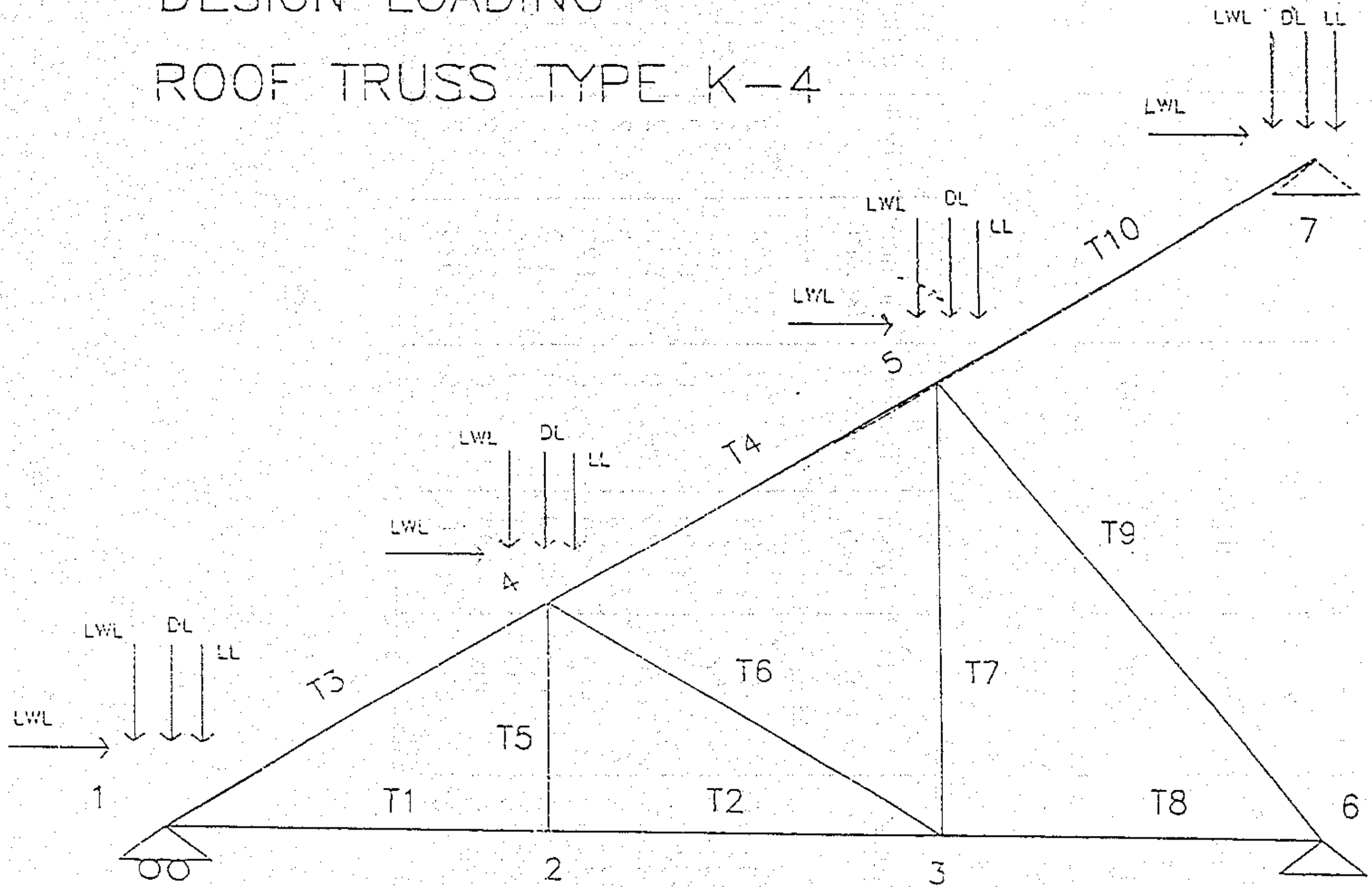
  $$F_{all} = \frac{\pi^2.E.Ix}{n.L^2} \quad ; n = \text{Safety Factor} = 3$$

  $$F_{all} = \frac{\pi^2 x (2.1x10^6) x 45.6}{3x(216.7)^2}$$
  $$= 6,708.80\ kg > F = 4,812.42\ kg\ (ok)$$

Hence double angle steel of 60.60.6 can be used as the members of roof truss type K-3

DESIGN LOADING

ROOF TRUSS TYPE K-4



NOTE
Dead load (DL) = 354 kg

Wind load :
- Left Wind Load (LWL) = 63 kg (downward)
63 kg (rightward)

Live load (LL) = 100 kg.

Roof K-4

Prototype

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia. Bolt (cm)
L 60.60.6	0.8	2,400	3,700	1.7

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L 60.60.6	262	424	0	424	2	17
2	L 60.60.6	262	424	0	424	2	17
3	L 60.60.6	405	520	0	520	2	17
4	L 60.60.6	76	520	0	520	2	17
5	L 60.60.6	32	0	0	0	2	17
6	L 60.60.6	552	520	0	520	2	17
7	L 60.60.6	372	0	0	0	2	17
8	L 60.60.6	183	424	0	424	2	17
9	L 60.60.6	809	735	0	735	2	17
10	L 60.60.6	562	520	0	520	2	17

- Checking of members Strength of roof steel Truss Type K-4 base on the axial force:

a. Due to Tensile force

Maximum force on member T10 (loading Combination 2)
Force $F = 549.31$ kg
Length $L = 216.70$ cm

Try : Double angle steel of 60.60.6
Cross secsion area $A = 2 \times 6.91 = 13.82\ cm^2$

$$\sigma_{all} = 0.6 \times Fy = 0.6 \times 2{,}400 = 1{,}440\ kg/cm^2$$

Stress

$$\sigma = \frac{F}{A} = \frac{549.31}{13.82} = 39.75\ kg/cm^2 < \sigma_{all} = 1{,}440\ kg/cm^2$$

b. Due to Compresion force

Maximum force on member T9 (loading Combination 2)
Force $F = 782.41$ kg (rounded)
Length $L = 306.32$ cm (rounded)

Try : Double angle steel of 60.60.6
Cross section area $A = 2 \times 6.91 = 13.82\ cm^2$
$ix = 1.82$ cm ; $Ix = 2 \times 22.8 = 45.60\ cm^4$

$$\lambda = \frac{L}{ix} = \frac{306.32}{1.82} = 168.31 > 105$$

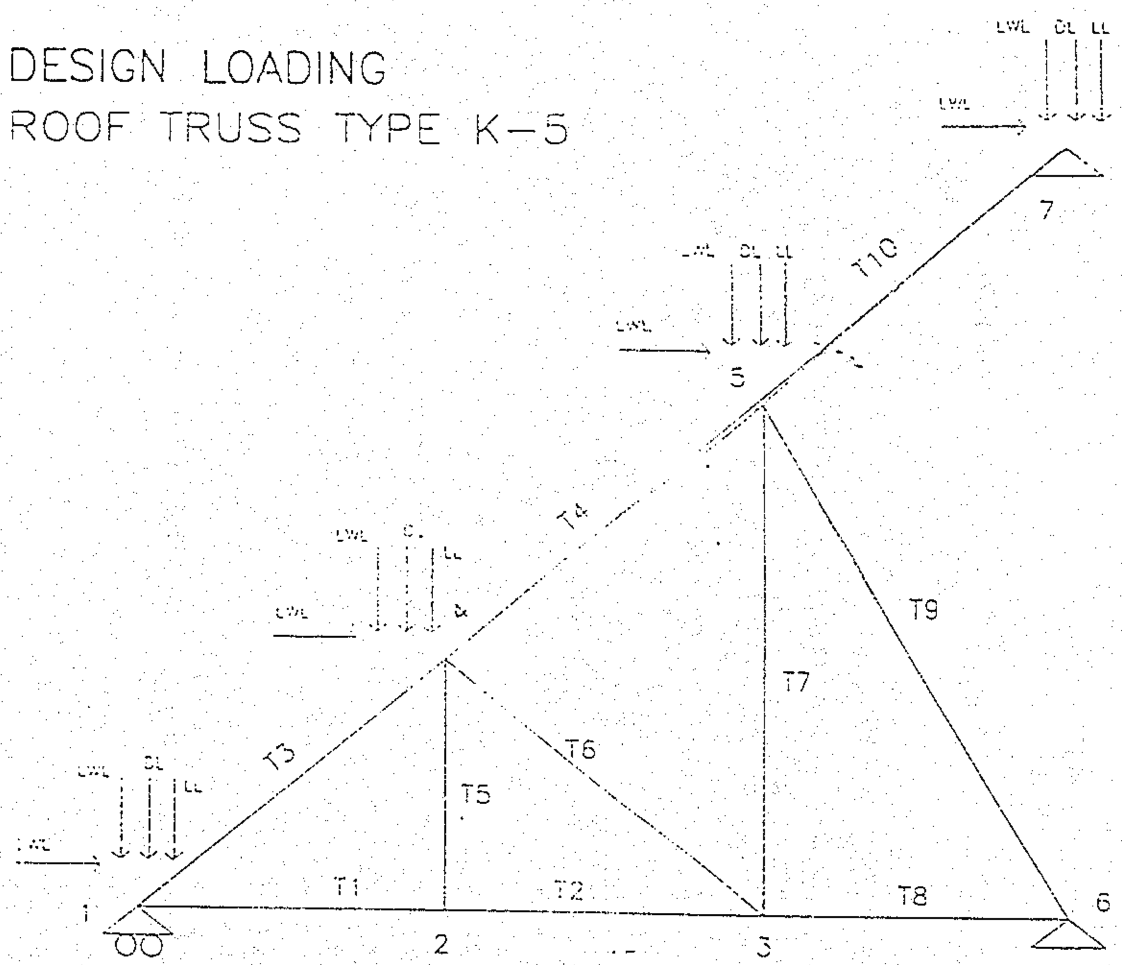
by Euler Formula

$$F_{all} = \frac{\pi^2 . E . Ix}{n . L^2} \quad ; n = \text{Safety Factor} = 3$$

$$F_{all} = \frac{\pi^2 \times (2.1 \times 10^6) \times 45.6}{3 \times (306.32)^2} = 3{,}357.47\ kg > F = 782.41\ kg\ (ok)$$

Hence double angle steel of 60.60.6 can be used as the members of roof truss type K-4

DESIGN LOADING
ROOF TRUSS TYPE K–5



NOTE
Dead load (DL) = 354 kg
Wind load :
– Left Wind Load (LWL) = 63 kg (downward)
63 kg (rightward)

Live load (LL) = 100 kg.

Roof K-5

Prototype

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia. Bolt (cm)
L 60.60.6	0.8	2,400	3,700	1.7

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L 60.60.6	171	6	0	211	2	17
2	L 60.60.6	171	6	0	211	2	17
3	L 60.60.6	325	6	0	299	2	17
4	L 60.60.6	35	6	0	299	2	17
5	L 60.60.6	13	0	0	0	2	17
6	L 60.60.6	456	6	0	299	2	17
7	L 60.60.6	368	0	0	0	2	17
8	L 60.60.6	146	6	0	211	2	17
9	L 60.60.6	718	6	0	473	2	17
10	L 60.60.6	395	6	0	229	2	17

- Checking of members Strength of roof steel Truss Type K-5 base on the axial force:

a. Due to Tensile force

Maximum force on member T10 (loading Combination 2)
Force $F = 395.73$ kg
Length $L = 176.78$ cm

Try : Double angle steel of 60.60.6
Cross section area $A = 2 \times 6.91 = 13.82\ cm^2$

$\sigma_{all} = 0.6 x Fy$
$= 0.6 x 2,400 = 1,440\ kg/cm^2$

Stress

$$\sigma = \frac{F}{A} = \frac{395.73}{13.82} = 28.6\ kg/cm^2 < \sigma_{all} = 1,440\ kg/cm^2$$

b. Due to Compresion force

Maximum force on member T9 (loading Combination 2)
Force $F = 718.79$ kg (rounded)
Length $L = 279.51$ cm (rounded)

Try : Double angle steel of 60.60.6
Cross section area $A = 2 \times 6.91 = 13.82\ cm^2$
$ix = 1.82$ cm ; $Ix = 2 x 22.8 = 45.60\ cm^4$

$$\lambda = \frac{L}{ix} = \frac{279.51}{1.82} = 153.58 > 105$$

by Euler Formula

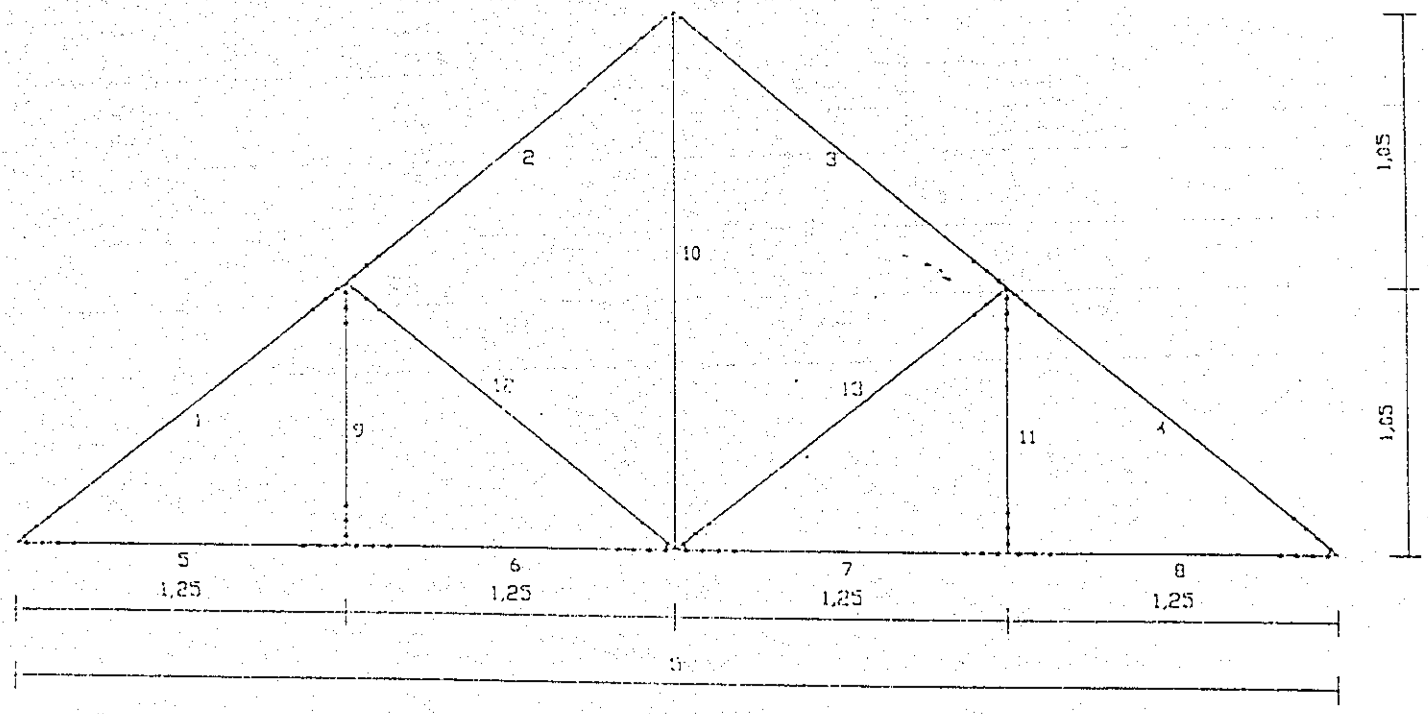
$$F_{all} = \frac{\pi^2 . E . Ix}{L^2}$$

$$F_{all} = \frac{\pi^2 x (2.1 x 10^6) x 45.6}{(279.51)^2}$$

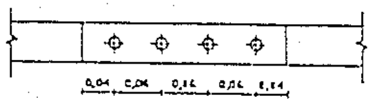
$= 4,032.44$ kg $> F = 697.32$ kg (ok)

Hence double angle steel of 60.60.6 can be used as the members of roof truss type K-5

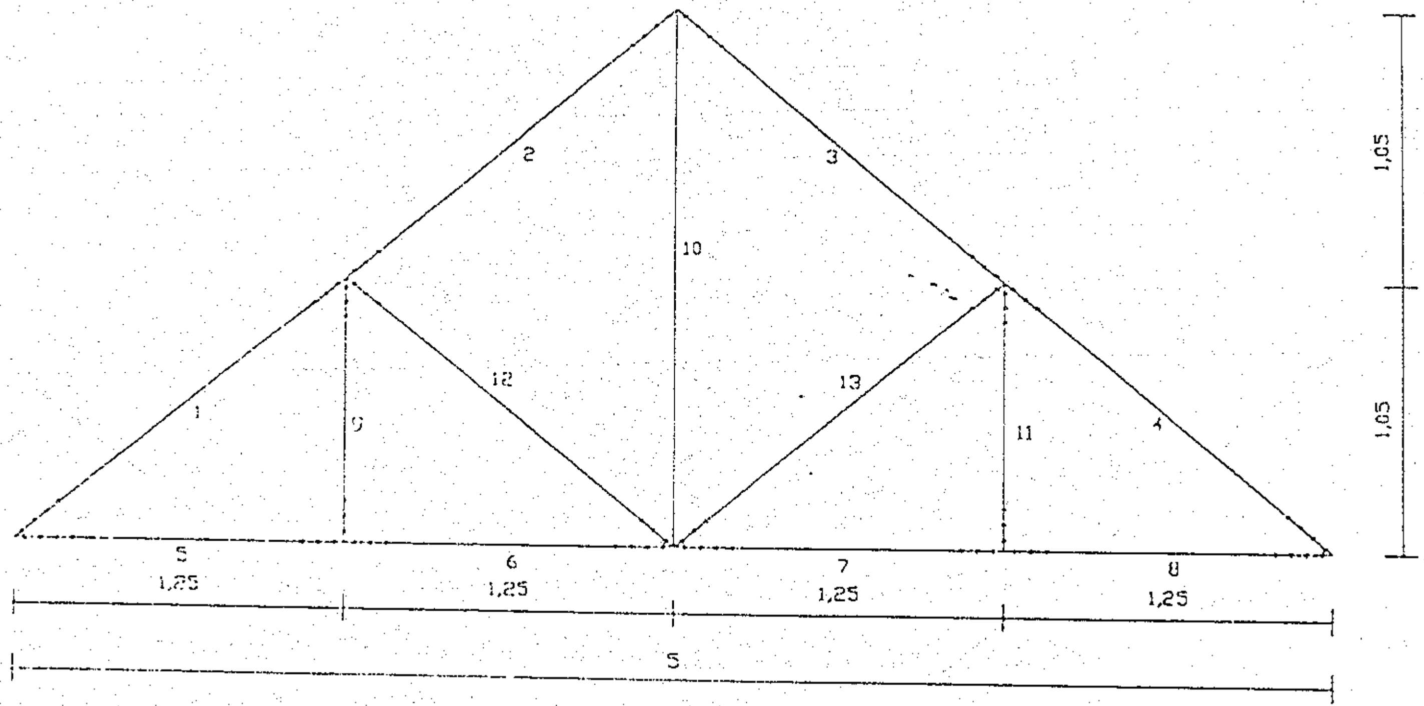
ROOF TYPE K1

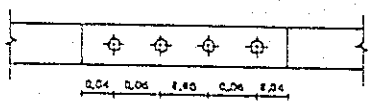


NO. Member	Profil	Bolt		Plate Thickness t (mm)
		Total	ø mm	
1 - 4	T 50.50.5	4 - 3	14	8
2 - 3	T 50.50.5	3 - 4	14	8
5 - 8	⊥ 50.50.5	4 - 3	14	8
6 - 7	⊥ 50.50.5	3 - 4	14	8
9 - 11	⊢ 50.50.5	3 - 3	14	8
10	⊥ 50.50.5	4 - 4	14	8
12 - 13	T 50.50.5	3 - 3	14	8



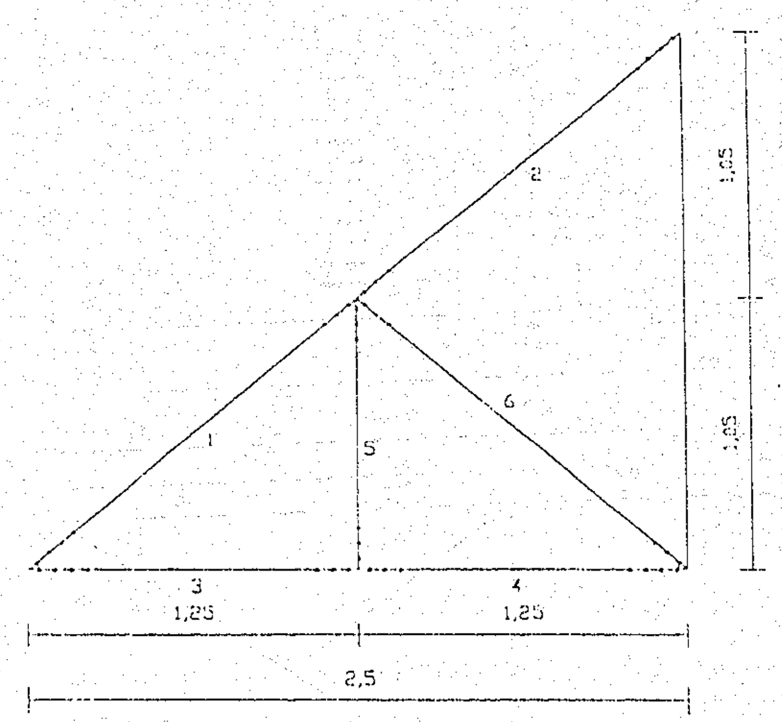
ROOF TYPE K1'



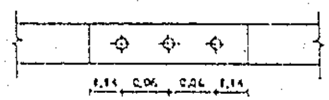


NO. Member	Profil	Bolt Total	Bolt ø mm	Plate Thickness t (mm)
1 = 4	T 50.50.5	4 - 3	14	8
2 = 3	T 50.50.5	3 - 4	14	8
5 = 8	⊥ 50.50.5	4 - 3	14	8
6 = 7	⊥ 50.50.5	3 - 4	14	8
9 = 11	⊢ 50.50.5	3 - 3	14	8
10	pipe ø 101,6	-	-	8
12 = 13	T 50.50.5	3 - 3	14	8

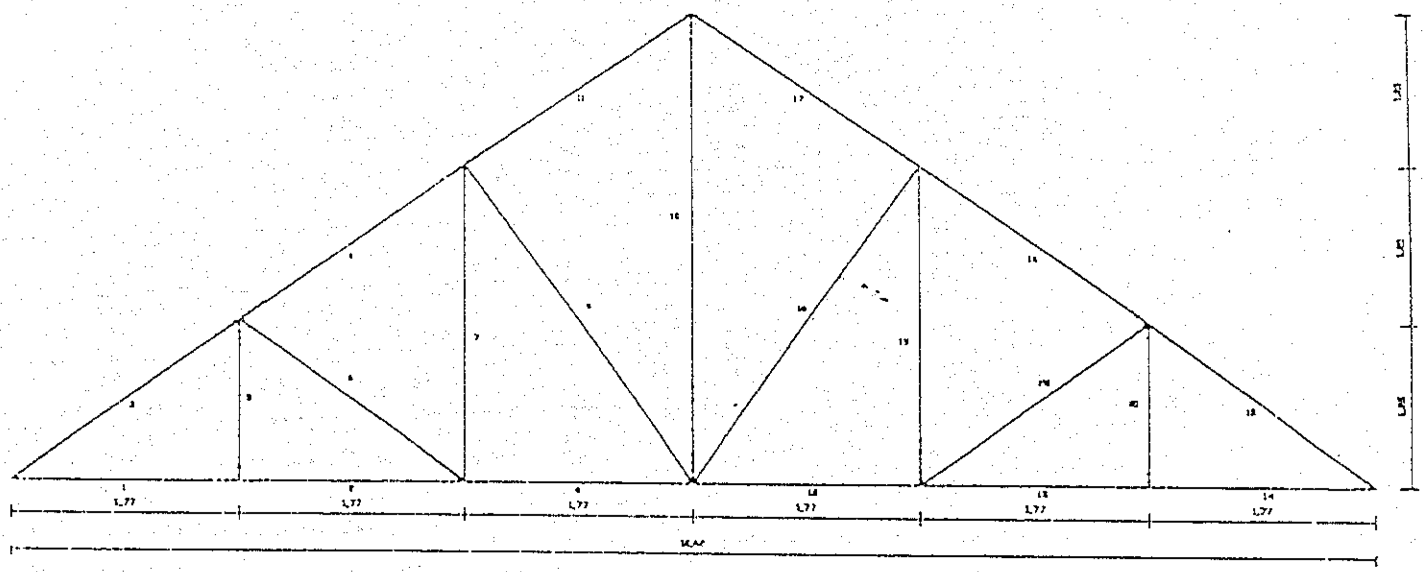
ROOF TYPE K2

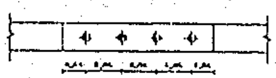


NO. Member	Profil	Bolt		Plate Thickness t (mm)
		Total	ø mm	
1	T 50.50.5	4 - 3	14	8
2	T 50.50.5	3 - 4	14	8
3	⊥ 50.50.5	4 - 3	14	8
4	⊥ 50.50.5	3 - 4	14	8
5	⊢ 50.50.5	3 - 3	14	8
6	T 50.50.5	3 - 3	14	8



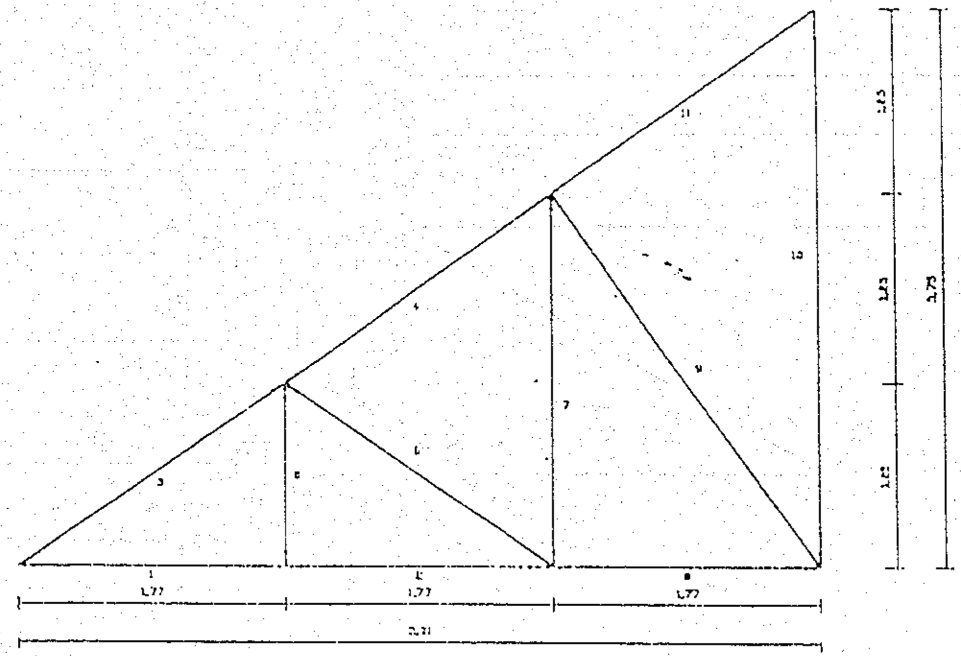
ROOF TYPE K3

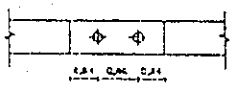




Nº Member	Profil	Bolt Total	Bolt ⌀ mm	Plate Thickness t (mm)
1	[illegible]	4 - 4	17	[illegible]
2	[illegible]	4 - 4	17	[illegible]
3	[illegible]	4 - 4	17	[illegible]
4	[illegible]	4 - 4	17	[illegible]
5	[illegible]	[illegible]	17	[illegible]
6	[illegible]	2 - 2	17	[illegible]
7	[illegible]	[illegible]	17	[illegible]
8	[illegible]	[illegible]	17	[illegible]
9	[illegible]	[illegible]	17	[illegible]
10	[illegible]	4 - 4	17	[illegible]
11	[illegible]	4 - 4	17	[illegible]
12	[illegible]	[illegible]	17	[illegible]
13	[illegible]	2 - 2	17	[illegible]
14	[illegible]	2 - 2	17	[illegible]
15	[illegible]	4 - 4	17	[illegible]
16	[illegible]	4 - 4	17	[illegible]
17	[illegible]	4 - 4	17	[illegible]
18	[illegible]	[illegible]	17	[illegible]
19	[illegible]	2 - 2	17	[illegible]
20	[illegible]	[illegible]	17	[illegible]
21	[illegible]	[illegible]	17	[illegible]

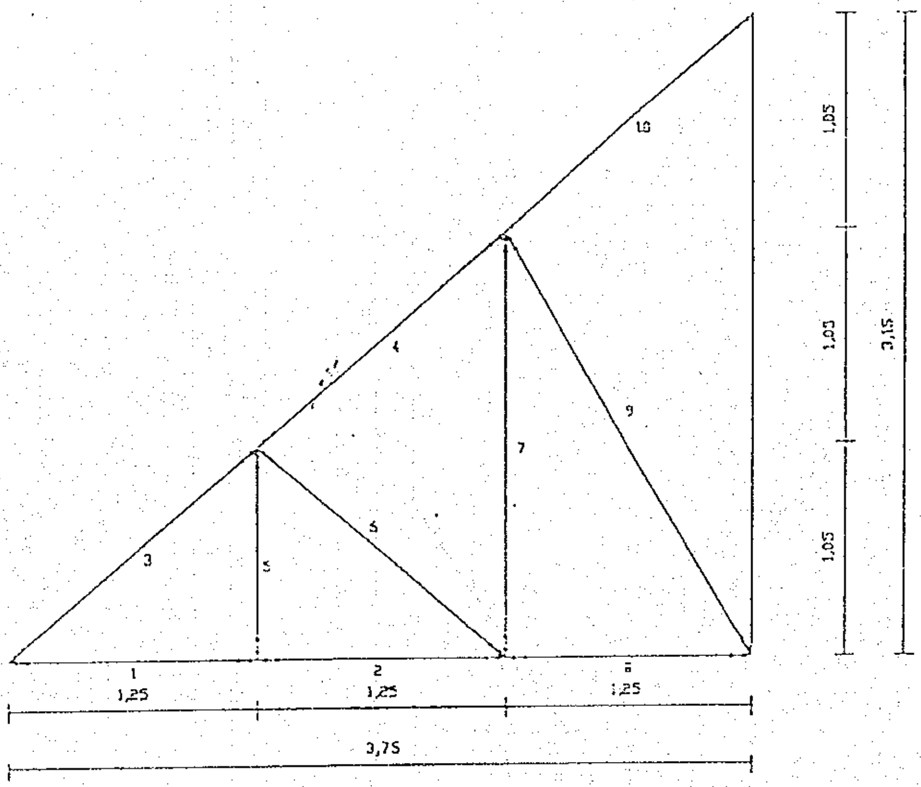
ROOF TYPE K4

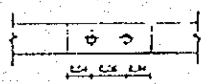




NO. Member	Profil	Bolt		Plate Thickness t (mm)
		Total	ø mm	
1	⊥ 60.60.6	2 - 2	17	8
2	⊥ 60.60.6	2 - 2	17	8
3	T 60.60.6	2 - 2	17	8
4	T 60.60.6	2 - 2	17	8
5	⊢ 60.60.6	2 - 2	17	8
6	T 60.60.6	2 - 2	17	8
7	⊢ 60.60.6	2 - 2	17	8
8	T 60.60.6	2 - 2	17	8
9	T 60.60.6	2 - 2	17	8
10	⊢ 60.60.6	2 - 2	17	8
11	T 60.60.6	2 - 2	17	8

ROOF TYPE K5



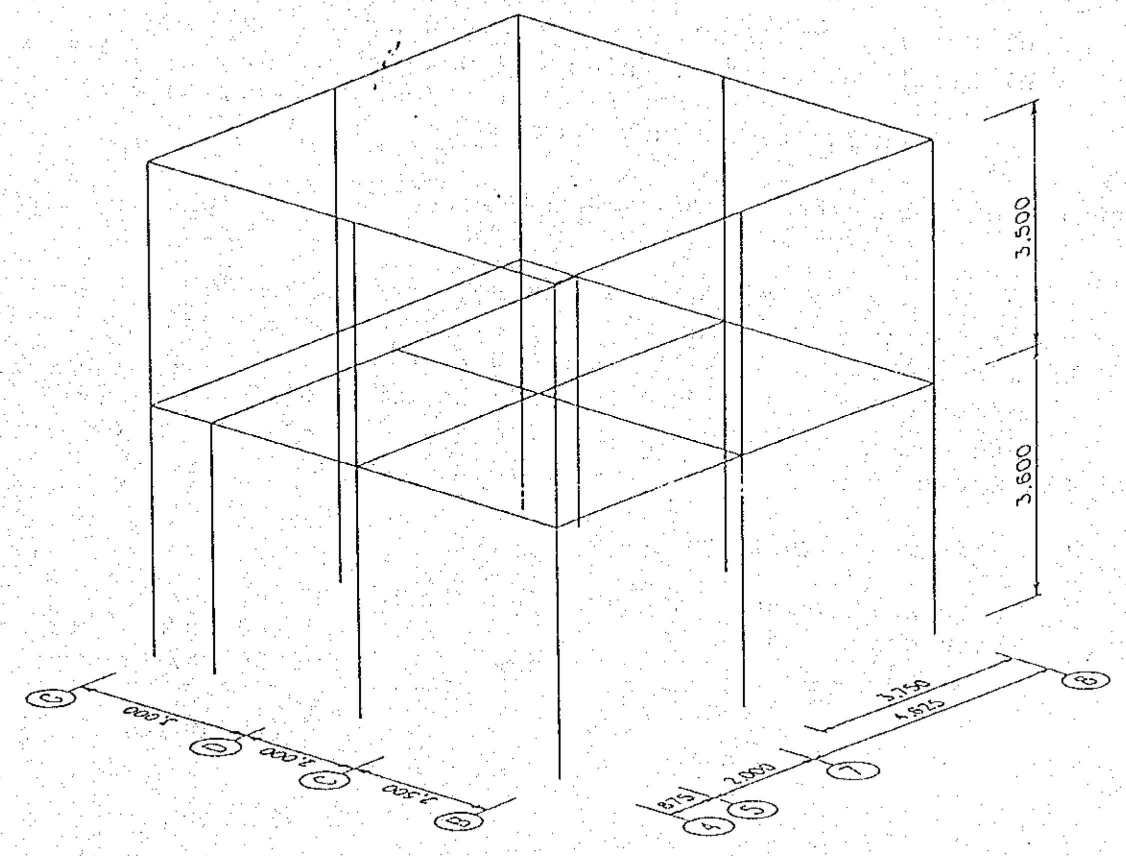


NO. Member	Profil	Bolt		Plate Thickness t (mm)
		Total	ø mm	
1	⊥ [illegible]	[illegible]	17	[illegible]
2	⊥ [illegible]	[illegible]	17	[illegible]
3	T [illegible]	[illegible]	17	[illegible]
4	T [illegible]	[illegible]	17	[illegible]
5	⊢ [illegible]	[illegible]	17	[illegible]
6	T [illegible]	[illegible]	17	[illegible]
7	⊢ [illegible]	[illegible]	17	[illegible]
8	⊥ [illegible]	[illegible]	17	[illegible]
9	T [illegible]	[illegible]	17	[illegible]
10	T [illegible]	[illegible]	17	[illegible]

6. DATA FOR TWO STORIES BUILDING

a. Dimension

- Length c-c column – 7,500 m
- Width c-c column 7,500 m
- Height ground to 2nd floor : 3,600 m
- Height ground to roof truss : 3,500 m



TWO STORIES BUILDING ISOMETRY
OPERATION/MANAGEMENT BUILDING

SIMONGAN WEIR MANAGEMENT COMPLEX

b. Design Conditions

a) Concrete compression strength (K) = 225 kg/cm^2
b) Reinforcing bar ;
. Plain bar Fy = 2.400 kg/cm^2 (BJTP 24)
. Deformed bar Fy = 3.200 kg/cm^2 (BJTP 22)
c) Structural model : space (xyz axis) frame
d) Analysis method : static - rigid floor

c. Loading Conditions

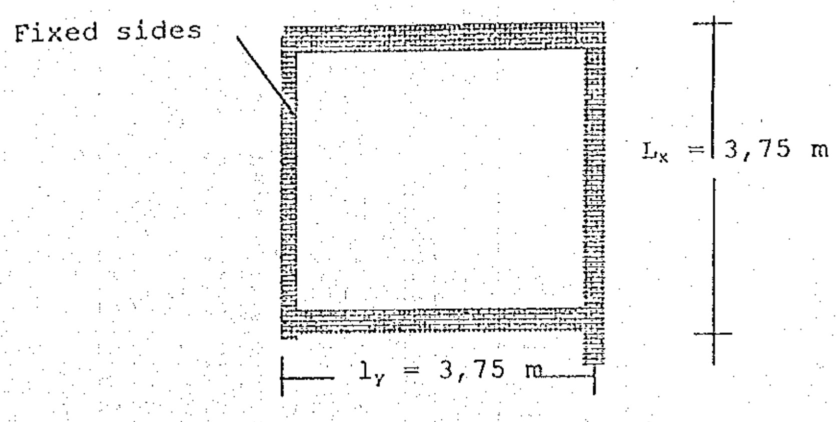
a) Roof load :
(as point load separated to 2 point)
- Truss type K3 = 7.000 kg
- Truss type K4 = 7.000 kg
- Truss type K5 = 2x2.000 kg = 4.000 kg
b) Slab dead load = 180 kg/m^2 (including pannel load 1000 kg)
c) Live load = 250 kg/m^2
d) Concrete self weight = 2.400 kg/m^3
e) Brick wall 0,15 cm thick = 250 kg/m^2
f) Soil Compression Stress = 1 kg/cm^2 (2 m depth)
(given by JICA Study Team)

d Design of reinforcement concrete plate :

Plate thickness	h_t	=	12 cm
Concrete cover	d	=	2 cm ; $h = h_t - d = 12 - 2 = 10$ cm.
Unit weight		=	2,400 kg/m^3
Compression stress	fc	=	225 kg/cm^2 ; σ'_b = 70 kg/cm^2 ; n = 21
Reinforcement bar	Fu	=	3,200 kg/ ; σ_a = 2,000 kg/cm^2
Plate area		=	(3.75 x 3.75) m^2

• Loading design :

Plate self weight : 0.12 x 2,400 kg/m^3	=	288 kg/m^2
Plate dead load	=	180 kg/m^2
Live load	=	250 kg/m^2
	q =	718 kg/m^2



l_y/l_x = 1.33 :

M^-_{tx} = 0.001 x 718 x 3.75^2 x 36 = 363.49 kgm

M^-_{ty} = 0.001 x 718 x 3.75^2 x 36 = 363.49 kgm

M^-_{lx} = 0.001 x 718 x 3.75^2 x 36 = 363.49 kgm

M^-_{ly} = 0.001 x 718 x 3.75^2 x 36 = 363.49 kgm

M_{max} = 363.49 kgm = 36.349 kgcm

$$C_a = \frac{h}{\sqrt{\dfrac{nxM}{bx\sigma_a}}} = \frac{10}{\sqrt{\dfrac{21x42,725}{100x2,000}}} = 4.72$$

δ = 0 (single reinforcement)

ϕ = 3.05 > ϕ_0 = σ_a / (n x σ'_b) = 2,000 / (21 x 70) = 1.36 (OK)

$n\omega$ = 0.041

A_{steel} = ω x b x h = 0.041/21 x 100 x 10 = 1.95 cm^2

Used A_{steel} = dia. 10 - 15 cm = 5.5 cm^2 > 1.95 cm^2 (OK).

BEAM type a

b (cm)	h (cm)	Cover (cm)	Diameter main bar (cm)	stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
25	50	4	1.6	0.8	187	3200	2400

Frame Element Force					Design											
Member	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	Main bar (mm)	Left bar			Mid bar			Right bar			Stirrup (mm)	Mu (kg.cm)
						Top	Middle	Bottom	Top	Middle	Bottom	Top	Middle	Bottom		
19	0	3,250	262,439	239,819	D16	3D16	2D16	4D16	3D16	2D16	4D16	4D16	2D16	3D16	o8-25	181,238
20	0	3,250	262,439	239,819	D16	4D16	2D16	3D16	3D16	2D16	4D16	3D16	2D16	4D16	o8-25	181,268
21	0	3,057	294,713	247,916	D16	3D16	2o12	2D16	2D16	2o12	3D16	2D16	2o12	3D16	o8-50	181,267
22	0	2,472	180,403	151,130	D16	3D16	2o12	3D16	3D16	2o12	3D16	3D16	2o12	3D16	o8-40	181,271
23	0	1,312	590,055	185,744	D16	3D16	2o12	2D16	2D16	2o12	3D16	2D16	2o12	3D16	o8-50	181,309
24	0	2,154	4,399	353,407	D16	2D16	2o12	3D16	2D16	2o12	3D16	3D16	2o12	2D16	o8-200	181,345
25	0	2,154	4,399	353,407	D16	3D16	2o12	2D16	2D16	2o12	3D16	2D16	2o12	3D16	o8-200	181,345
26	0	1,312	590,055	185,744	D16	3D16	2o12	2D16	2D16	2o12	3D16	2D16	2o12	3D16	o8-50	181,291
27	0	2,427	180,403	151,130	D16	3D16	2o12	3D16	3D16	2o12	3D16	3D16	2o12	3D16	o8-40	180,792
28	0	3,057	294,713	247,916	D16	3D16	2o12	3D16	3D16	2o12	3D16	3D16	2o12	3D16	o8-50	180,596
31	0	7,371	233,502	1,466,217	D16	4D16	2o12	4D16	4D16	2o12	4D16	5D16	2o12	8D16	o8-25	180,574
32	0	7,371	233,502	1,466,217	D16	5D16	2o12	6D16	4D16	2o12	4D16	4D16	2o12	4D16	o8-25	180,552

BEAM type b

b (cm)	h (cm)	Cover (cm)	Diameter main bar (cm)	stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
20	30	4	1.6	0.8	187	3200	2400

Frame Element Force					Design											
Member	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	Main bar (mm)	Left bar			Mid bar			Right bar			Stirrup (mm)	Mu (kg.cm)
						Top	Middle	Bottom	Top	Middle	Bottom	Top	Middle	Bottom		
29	0	6,470	5,974	619,017	D16	8D16	-	5D16	3D16	-	4D16	3D16	-	5D16	o8-70	181,238
30	0	6,470	5,974	619,017	D16	3D16	-	5D16	3D16	-	4D16	8D16	-	5D16	o8-70	181,268
33	0	7,455	-	733,592	D16	10D16	-	6D16	3D16	-	4D16	5D16	-	7D16	o8-70	181,267
34	0	5,761	-	572,202	D16	5D16	-	7D16	2D16	-	3D16	5D16	-	3D16	o8-70	181,271

BEAM type c

b (cm)	h (cm)	Cover (cm)	Diameter main bar (cm)	stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
20	30	4	1.2	0.8	187	3200	2400

Frame Element Force					D e s i g n											
Member	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	Main bar (mm)	Left bar			Mid bar			Right bar			Stirrup (mm)	Mu (kg.cm)
						Top	Middle	Bottom	Top	Middle	Bottom	Top	Middle	Bottom		
35	0	213	6,783	21,282	2o12	2o12	-	2o12	2o12	-	2o12	2o12	-	2o12	o8-70	82,408
36	0	213	6,783	21,282	2o12	2o12	-	2o12	2o12	-	2o12	2o12	-	2o12	o8-70	82,408
37	0	193	6,615	15,242	2o12	2o12	-	2o12	2o12	-	2o12	2o12	-	2o12	o8-70	82,408
38	0	246	6,431	28,656	2o12	2o12	-	2o12	2o12	-	2o12	2o12	-	2o12	o8-70	82,408
39	0	207	223	20,401	2o12	2o12	-	2o12	2o12	-	2o12	2o12	-	2o12	o8-70	82,408
40	0	207	223	20,401	2o12	2o12	-	2o12	2o12	-	2o12	2o12	-	2o12	o8-70	82,408
41	0	246	6,431	28,565	2o12	2o12	-	2o12	2o12	-	2o12	2o12	-	2o12	o8-70	82,408
42	0	130	6,615	15,212	2o12	2o12	-	2o12	2o12	-	2o12	2o12	-	2o12	o8-70	82,408

COLUMN type I

Prototype

b (cm)	h (cm)	Cover (cm)	Diameter Main Bar (cm)	Diameter Stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
30	30	4	1.6	0.8	187	3,200	2,400

Frame Element force				Design				
Member	Axial (kg)	Moment-2 (kg.cm)	Moment-3 (kg.cm)	Main Bar (mm)	Stirrup (mm)	Pu (kg)	Max (kg.cm)	May (kg.cm)
1	9,574	90,126	111,549	8D16	o8-100	9,573	461,621	461,621
3	9,574	90,126	111,549	8D16	o8-100	9,573	461,621	461,621
8	6,975	93,961	131,235	8D16	o8-100	6,975	453,477	453,477
10	6,975	93,961	32,808	8D16	o8-100	6,975	453,477	453,477
11	5,514	69,114	90,995	8D16	o8-100	5,514	448,579	448,579
13	5,514	69,114	90,995	8D16	o8-100	5,514	448,579	448,579
16	5,560	87,382	105,411	8D16	o8-100	5,560	448,735	448,735
18	5,560	87,382	105,411	8D16	o8-100	5,560	448,735	448,735

COLUMN type 2

Prototype

b (cm)	h (cm)	Cover (cm)	Diameter Main Bar (cm)	Diameter Stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
15	25	4	1.6	0.8	187	3,200	2,400

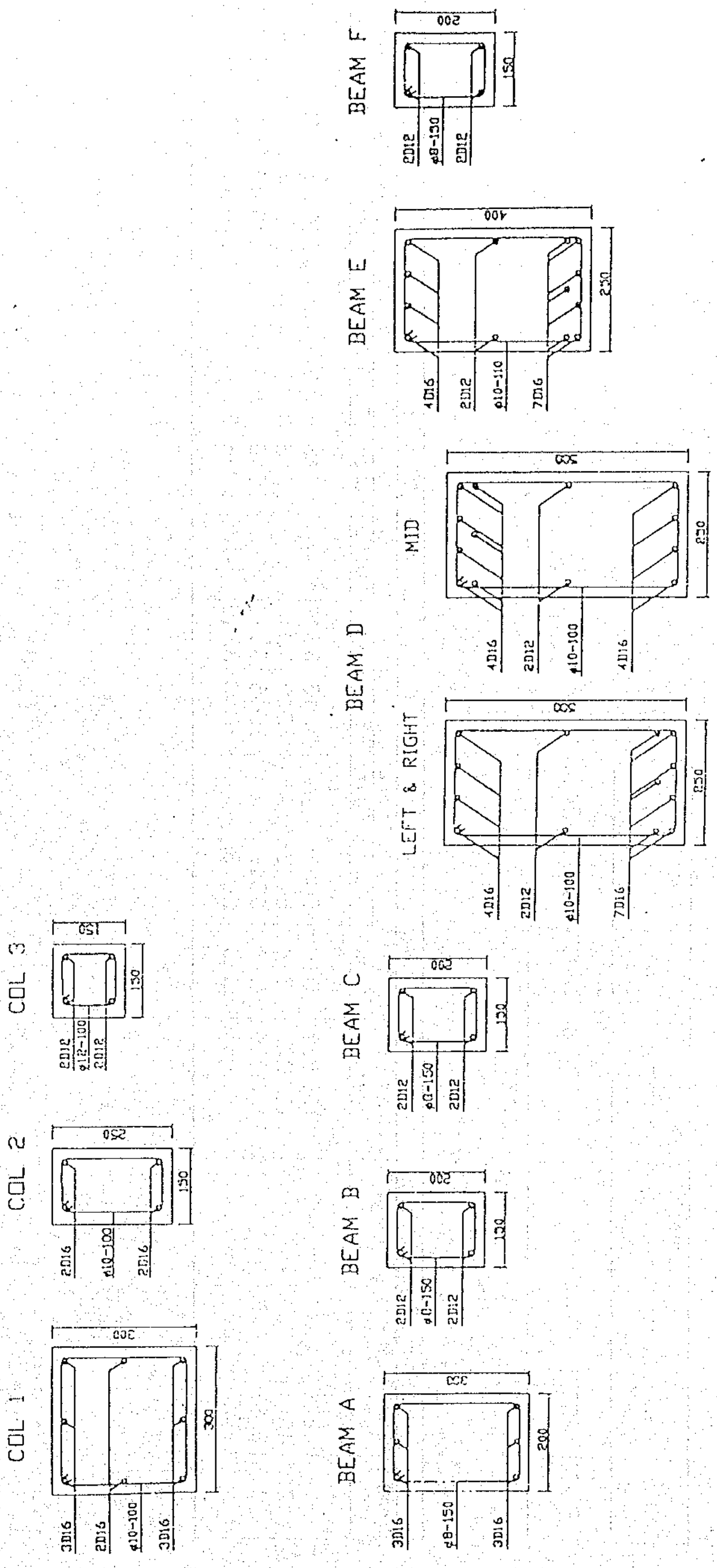
Frame Element force				Design				
Member	Axial (kg)	Moment-2 (kg.cm)	Moment-3 (kg.cm)	Main Bar (mm)	Stirrup (mm)	Pu (kg)	Max (kg.cm)	May (kg.cm)
2	9,574	90,126	111,549	8D16	o8-100	9,573	461,621	461,621
12	9,574	90,126	111,549	8D16	o8-100	9,573	461,621	461,621

COLUMN type 3

Prototype

b (cm)	h (cm)	Cover (cm)	Diameter Main Bar (cm)	Diameter Stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
15	15	4	1.6	0.8	187	3,200	2,400

Frame Element force				Design				
Member	Axial (kg)	Moment-2 (kg.cm)	Moment-3 (kg.cm)	Main Bar (mm)	Stirrup (mm)	Pu (kg)	Max (kg.cm)	May (kg.cm)
4	13,863	4,614	78,500	8D16	o8-50	30,675	78,569	78,569
5	13,863	4,614	78,500	8D16	o8-50	30,675	78,569	78,569
6	6,981	8,207	46,391	8D16	o8-50	32,405	46,407	46,407
7	6,981	8,207	46,391	8D16	o8-50	32,405	46,407	46,407
9	8,196	4,804	0	8D16	o8-50	43,081	120,920	120,920
14	1,635	6,710	65,399	8D16	o8-50	43,081	78,578	78,578
15	1,635	6,710	65,399	8D16	o8-50	1,635	78,578	78,578
17	1,660	399,504	0	8D16	o8-50	1,660	78,740	78,740


BEAM F
200
150
2D12
ϕ8-150
2D12
BEAM E
400
250
4D16
2D12
ϕ10-110
7D16
BEAM D
MID
250
4D16
2D12
ϕ10-100
4D16
LEFT & RIGHT
250
4D16
2D12
ϕ10-100
7D16
BEAM C
200
130
2D12
ϕ8-150
2D12
BEAM B
200
130
2D12
ϕ8-150
2D12
BEAM A
300
200
3D16
ϕ8-150
3D16
COL 1
300
300
3D16
2D16
ϕ10-100
3D16
COL 2
250
150
2D16
ϕ10-100
2D16
COL 3
150
150
2D12
ϕ12-100
2D12

BEAM type d

b (cm)	h (cm)	Cover (cm)	Diameter main bar (cm)	stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
100	25	4	1.6	0.8	187	3200	2400

Frame Element Force					Design											
Member	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	Main bar (mm)	Left bar			Mid bar			Right bar			Stirrup (mm)	Mu (kg.cm)
						Top	Middle	Bottom	Top	Middle	Bottom	Top	Middle	Bottom		
1	4223	3,377	-	155,569	D16	5D16	-	3D16	3D16	-	5D16	5D16	-	3D16	o8-250	534,436
2	4641	1,730	-	182,226	D16	5D16	-	3D16	3D16	-	5D16	5D16	-	3D16	o8-250	533,845
3	3387	656	-	90,055	D16	5D16	-	3D16	3D16	-	5D16	5D16	-	3D16	o8-250	535,618
4	956	3,387	-	90,055	D16	5D16	-	3D16	3D16	-	5D16	5D16	-	3D16	o8-250	539,468

7. DESIGN OF FOOTING

All of footing design are represented by support reaction of joint no.1&3 or column no.6 (the biggest) for loading Combination 1, the axial force :

N	=	9.916 E3 kg
Mx	=	2.092 E4 kg
Mz	=	4.997 E4 kg
Shear x	=	428 kg
Shear z	=	303 kg

- Soil stress beneath footing :

$$\sigma = \frac{N}{A} \pm \frac{Mx}{Wx} \pm \frac{Mz}{Wz}$$

$$\sigma\ max = \frac{9.916 x 10^3}{(150)^2} + \frac{2.0923 X 10^4}{1/6 x 150 x 150^2} + \frac{4.997 x 10^4}{1/6 x 150 x 150^2}$$

$$= 0.44 + 0.04 + 0.09$$

$$= 0.57\ kg/cm^2 < \sigma\ all = 1,0\ kg/cm^2\ (ok)$$

$$\sigma\ min = 0.44 - 0.04 - 0.09$$

$$= 0.31\ kg/cm^2$$

When earthquake occur (loading Combination 3), Support reaction of joint 3 or column no.3 is :

N	=	9.533 E4 kg
Mx	=	6.3578 E4 kgcm
Mz	=	19.1868 E4 kgcm
Shear x	=	477 kg
Shear z	=	1,1121 kg

then soil stress beneth footing is

$$\sigma\ max = \frac{9.533 x 10^4}{(150)^2} + \frac{6.3578 x 10^4}{1/6 x 150 x 150^2} + \frac{19.1868 x 10^5}{1/6 x 150 x 150^2}$$

$$= 0.42 + 0.11 + 0.34$$

$$= 0.87\ kg/cm^2 < 1.5 \times \sigma_{all} = 1.5\ kg/cm^2\ (ok)$$

$$\sigma\ min = 0.42 - 0.11 - 0.34$$

$$= -0.03\ kg/cm^2$$

note :

$\sigma = 1\ kg/cm^2$
= Soil compression stress was given JICA Study Team

- All of footing concrete reinforcement is calculated by "n" method (Indonesian Code)

Mz = 8.177 E4 kgcm

Concrete : $fc = 225\ kg/cm^2$ ——— $\sigma'b = 130\ kg/cm^2$

Steel Bar : $fy = 3200\ kg/cm^2$ ——— $\sigma'a = 2600\ kg/cm^2$

ns = 14

$\phi_0 = \dfrac{\sigma_b}{\sigma_b n} = \dfrac{2,600}{130x14} = 1.43$

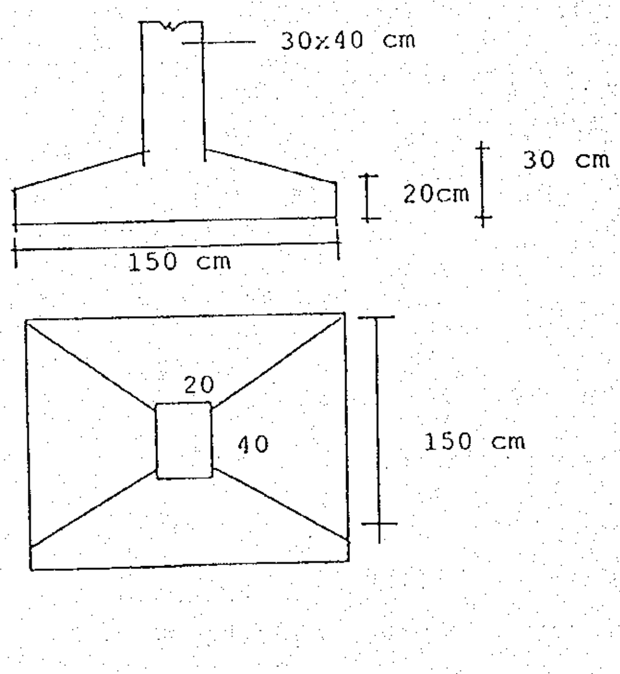
Footing slab thich ht = 25 cm ; b = 150 cm
Cocrete cover d = 5 cm
h = ht - d

$Ca = \dfrac{h}{\sqrt{\dfrac{nM}{b\sigma_a}}} = \dfrac{20}{\sqrt{\dfrac{14x49,970}{150x2600}}} = 14.93$

for - δ = 1
ϕ = 8.091 > ϕ_0 = 1.43 (ok)
ϕ' = 89 ; 100nω = 0.69

A = ωbh
= $\dfrac{0.69}{100}$ x 150 x 20 = 20,7 cm^2
Astell = D16 - 15 cm(two way) ≈ 11 x 2.01 = 22.12 cm^2 (ok)

Mx = 2.0993E4 kgcm
Astell = D16 - 15 cm can be adobted



Support reaction of joint no. 2, 4, 6, 7, 9 and 11 due to applied loading column no.2, 4, 5, 6, 7 and 9, each supported by their continous wet masonry foundation with 6 m length.
For example : column no.2 at joint no. 2 (loading Combination 1)

N = 1.7025 E4 kg
Mx = 4.3720 E4 kgcm
Mz = 1.09209 E5 kgcm
Shear x = 387.5 kg
Shear z = 0
brickwall unit weight = 875 kg/m' (3m height)

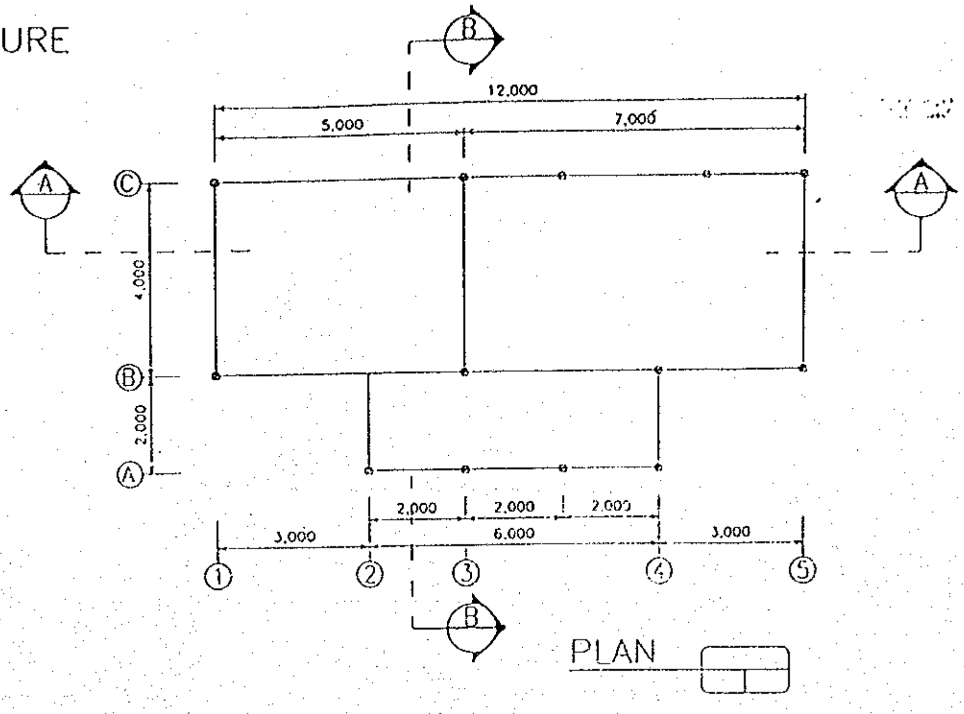
- Soil stress beneth foundation

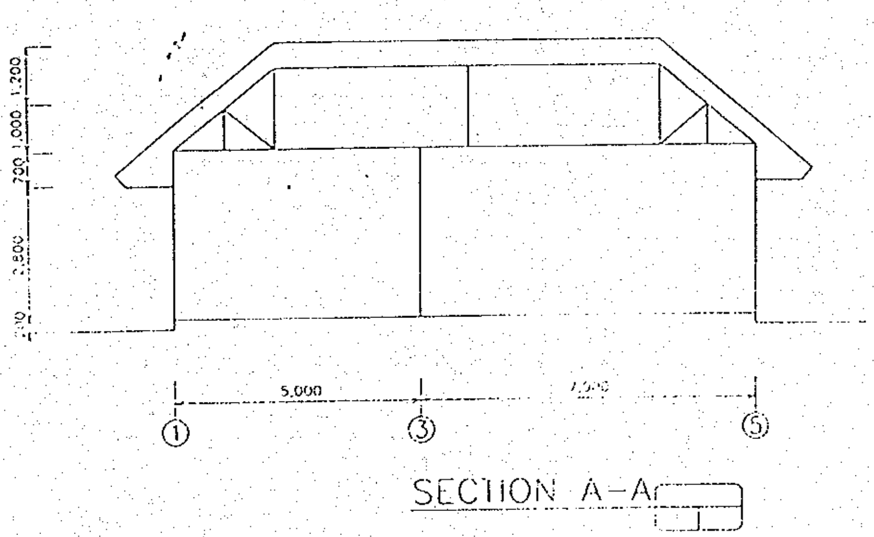
$$\sigma\ max = \frac{1.7025x10^4}{100x(750-750)} + \frac{4.3720x10^4}{1/6x600x100^2} + \frac{1.09209x10^5}{1/6x100x600^2} + \frac{875}{100x100}$$
$$= 0.28 + 0.04 + 0.02$$
$$= 0.34\ kg/cm^2 < \sigma\ all = 1,0\ kg/cm^2\ (ok)$$

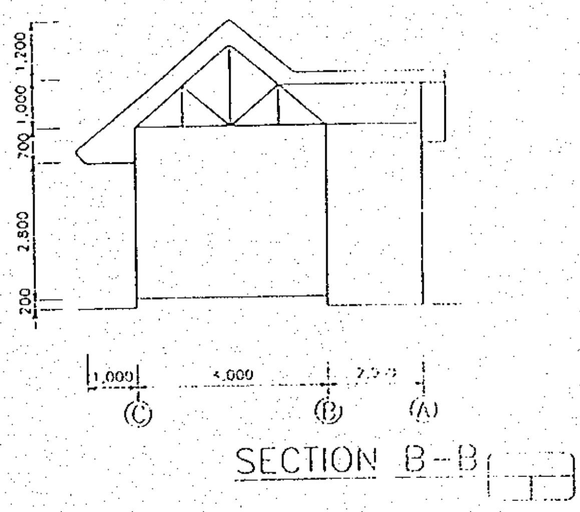
4.7.2. ELECTRICAL BULDING STRUCTURE CALCULATION

1 STRUCTURE
2 DESIGN CONDITION
3 LOADING CONDITION
4 DESIGN OF PURLIN
5 DESIGN OF ROOF TRUSS

1. STRUCTURE

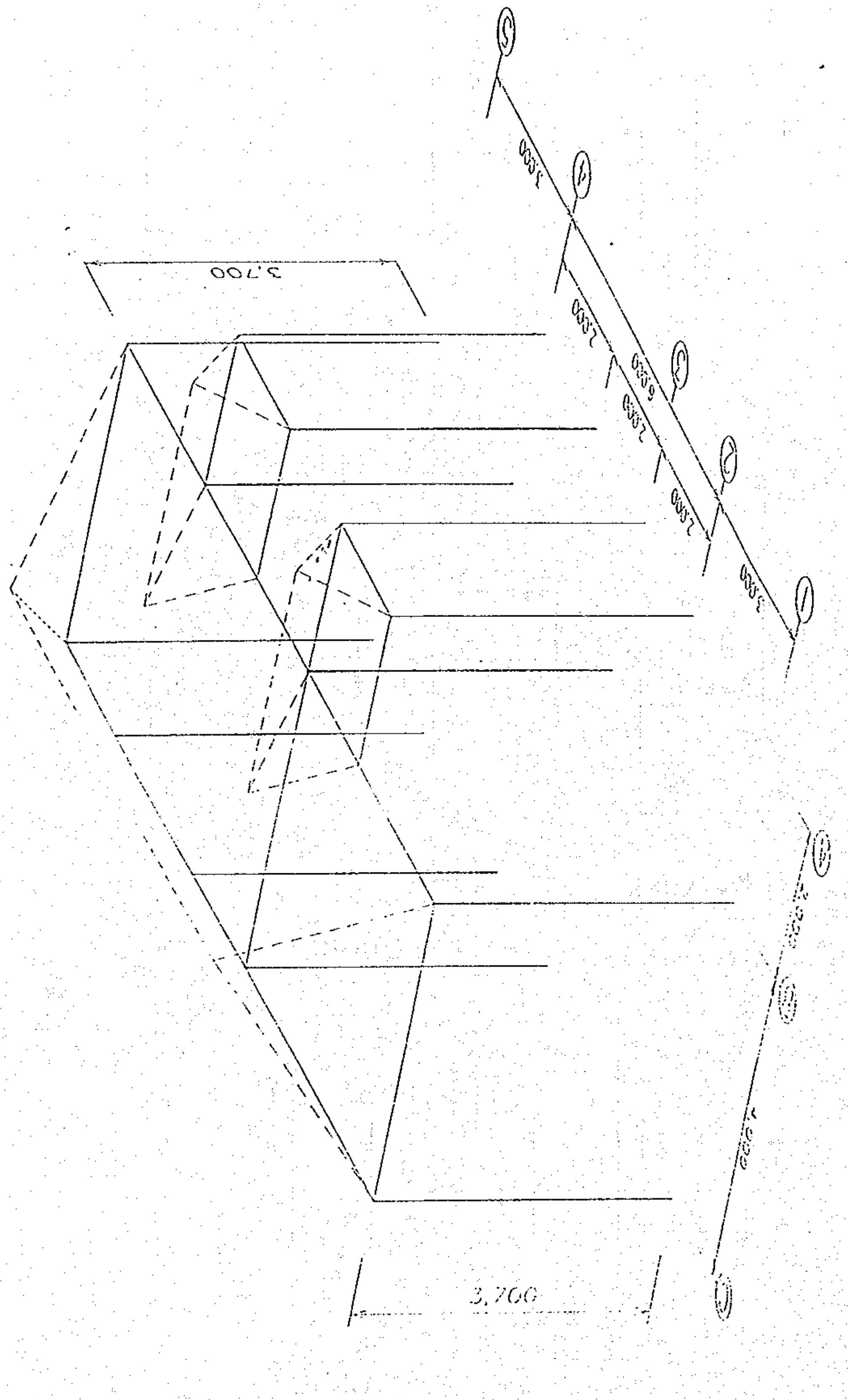




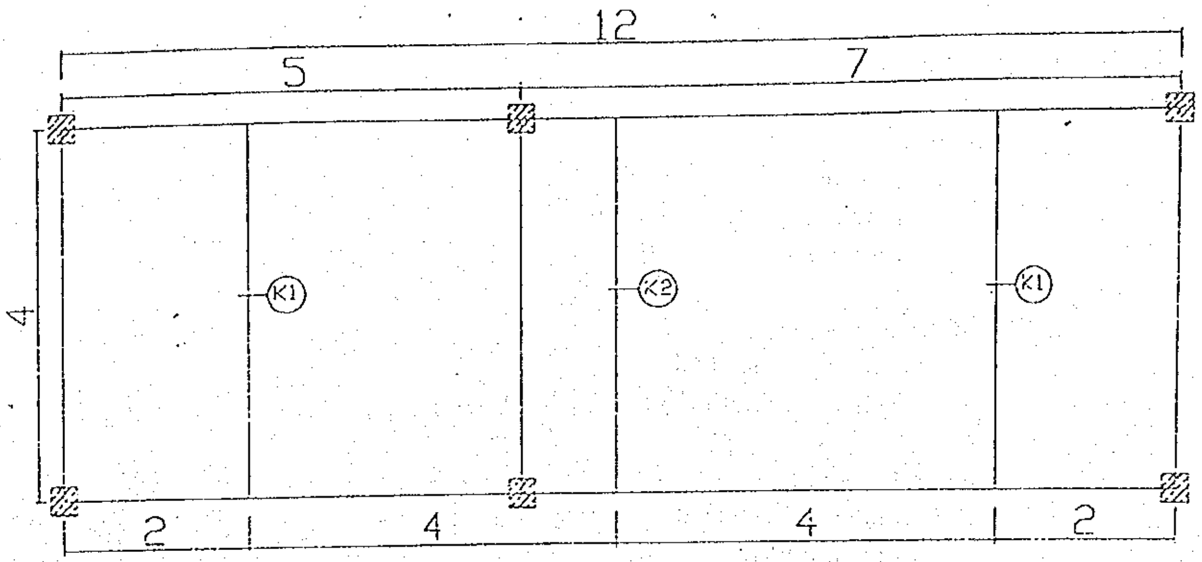


ELECTRICAL BUILDING

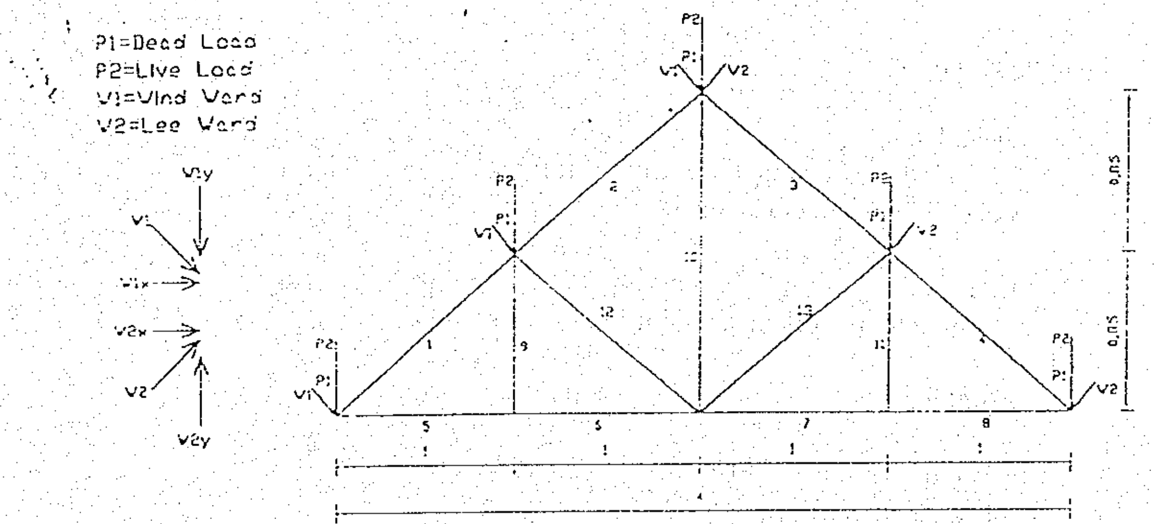
SIMONGAN WIER MANAGEMENT COMPLEX



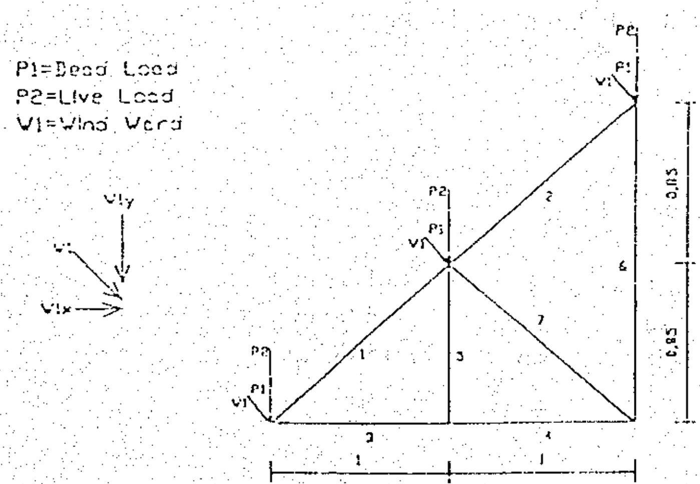
ISOMETRY
ELECTRICAL BUILDING
SIMONGAN WEIR MANAGEMENT COMPLEX



ROOF TYPE K1



ROOF TYPE K3



2. Design Condition

a. Dimensions :
- length : 4.00 m
- height : 1.70 m
- roof slope : 40°

b. Roof truss member :
- Double angle steel
- Tensile strength (Fy) = 2,400 kg/cm^2

c. Structural model :
- Plane (xy axis) truss
- Linear elastic

d. Analysis method :
- Static

3. Loading condition

a. Dead load
- Roof cover (ceramic tile + timber rafter) = 70 kg/m^2
- C e i l i n g (fiber cement) = 10 kg/m^2

b. Live load
Weight of workers as point load = 100 kg

c. Wind load

- Wind pressure = 40 kg/m^2
- Pressure koefficient (f)
. wind ward - 0.5 kg/m^2
. lee ward - 0.4

$W1 = 0.5 \times 40\ kg/m^2 = 20\ kg/m^2$

$W2 = 0.4 \times 40\ kg/m^2 = 16\ kg/m^2$

4. Design of Purlin

A. Roof truss type K-1

- Purlin distance (c/c) = 1.31 m
- Purlin span = 4.00 m
- Purlin self weight say = 15.00 kg/m

$q_1 = 1.31 \times 80\ kg/m^2 \approx 104.8\ kg/m'$
q_2 (self weight) $= 15\ kg/m'$
$Q = 119.8\ kg/m'$

$Q_1 = Q_2 = Q \cos 40^\circ$
$= 119.8 \cos 40^\circ$
$\approx 91.77\ kg/m'$

- Live Load
  $P_x = P_y = P \cos \alpha = 100 \cos 40^\circ \approx 76.6\ kg$

- Bending moment

  $Mx = 1/8 \times Q_1 \times L^2 + 1/4 \times P_1 \times L$

  $Mx = 1/8 \times 119.8 \times 4^2 + 1/4 \times 76.6 \times 4 = 286.2\ kgm$

  $My = Mx = 286.2\ kgm = 28,620\ cm$

- Try Purlin of Lip Channel (in front to front arrangement) type :

  150 x 130 x 20 x 3.2
  $Ix = 664 cm^4$; $Wx = 88.6\ cm^3$
  $Iy = 476\ cm^4$; $Wy = 73.2\ cm^3$

- Stresses
  $\sigma = \sigma x + \sigma y$
  $= \frac{Mx}{Wx} + \frac{My}{Wy}$
  $= \frac{28,620}{88.6} + \frac{28,620}{73.2} = 323.02 + 390.98$
  $= 714.0\ kg < \sigma_{all} = 1,400\ kg/cm^2$ (OK)

- Deflection
  $fx = \frac{5}{384} \times Q_1 \times \frac{L^4}{EI_x} + \frac{1}{48} \frac{PL^3}{EI_x}$

  $= \frac{5}{384} \times 0.9177 \times \frac{400^4}{2.1 \times 10^6 \times 664} + \frac{1}{48} \frac{76.6 \times 400^3}{2.1 \times 10^6 \times 664}$

  $= 0.22 + 0.07 = 0.29\ cm$

  $f = (0.29^2 + 0.29^2)^{1/2} = 0.41\ cm$

  $f = 0.41\ cm < \bar{f}_{all} = \frac{1}{360} L = \frac{400}{360} = 1.11\ cm$ (OK)

Design of Roof Truss

a. Dead load
- $P_1 = 4.00 \times (104.8 + 15) = 479.2$ kg

b. Wind load
- $W_1 = 4.00 \times 1.31 \times 20 = 104.8$ kg
- $W_2 = 4.00 \times 1.31 \times 16 = -83.84$ kg

$W_{1X} = W_{1Y} = 104.8 \text{ Cos } 40^o = 80.28$ kg
$W_{2X} = W_{2Y} = -83.84 \text{ Cos } 40^o = -64.23$ kg

c. Live load
- $P_2 = 100$ kg

B. Roof Truss Type K-2

- Purlin distance (c/c) = 1.31 m
- Purlin span = 4.00 m
- Purlin self weight say = 15.00 kg/m'

$q_1 = 1.31 \times 80 \text{ kg/m}^2 \approx 104.8$ kg/m'
q_2 (self weight) = 15 kg/m'

Q = 119.8 kg/m'

$Q_1 = Q_2 = Q \text{ Cos } 40^o$
$= 119.8 \text{ Cos } 40^o$
≈ 91.77 kg/m'

- Live Load
$Px = Py = P \text{ Cos } \alpha = 100 \text{ Cos } 40^o \approx 76.6$ kg

- Bending moment

$Mx = 1/8 \times Q_1 \times L^2 + 1/4 \times P_1 \times L$

$Mx = 1/8 \times 119.8 \times 4^2 + 1/4 \times 76.6 \times 4 = 286.2$ kgm

$My = Mx = 286.2$ kgm = 28,620 cm

- Try Purlin of Lip Channel (in front to front arrangement) type :

150 x 130 x 20 x 3.2
$Ix = 664 \text{cm}^4$; $Wx = 88.6 \text{ cm}^3$
$Iy = 476 \text{ cm}^4$; $Wy = 73.2 \text{ cm}^3$

- Stresses

$$\sigma = \sigma x + \sigma y$$
$$= \frac{Mx}{Wx} + \frac{My}{Wy}$$
$$= \frac{28,620}{88.6} + \frac{28,620}{73.2} = 323.02 + 390.98$$
$$= 714.0 \text{ kg} < \sigma_{all} = 1,400 \text{ kg/cm}^2 \text{ (OK)}$$

- Deflection

$$fx = \frac{5}{384} \times Q_1 \times \frac{L^4}{EI_x} + \frac{1}{48}\frac{PL^3}{EI_x}$$
$$= \frac{5}{384} \times 0.9177 \times \frac{400^4}{2.1\times10^6\times664} + \frac{1}{48}\frac{76.6 \times 400^3}{2.1\times10^6\times664}$$
$$= 0.22 + 0.07 = 0.29 \text{ cm}$$
$$f = (0.29^2 + 0.29^2)^{1/2} = 0.41 \text{ cm}$$
$$f = 0.41 \text{ cm} < f_{all} = \frac{1}{360}L = \frac{400}{360} = 1.11 \text{ cm (OK)}$$

5 Design of Roof Truss

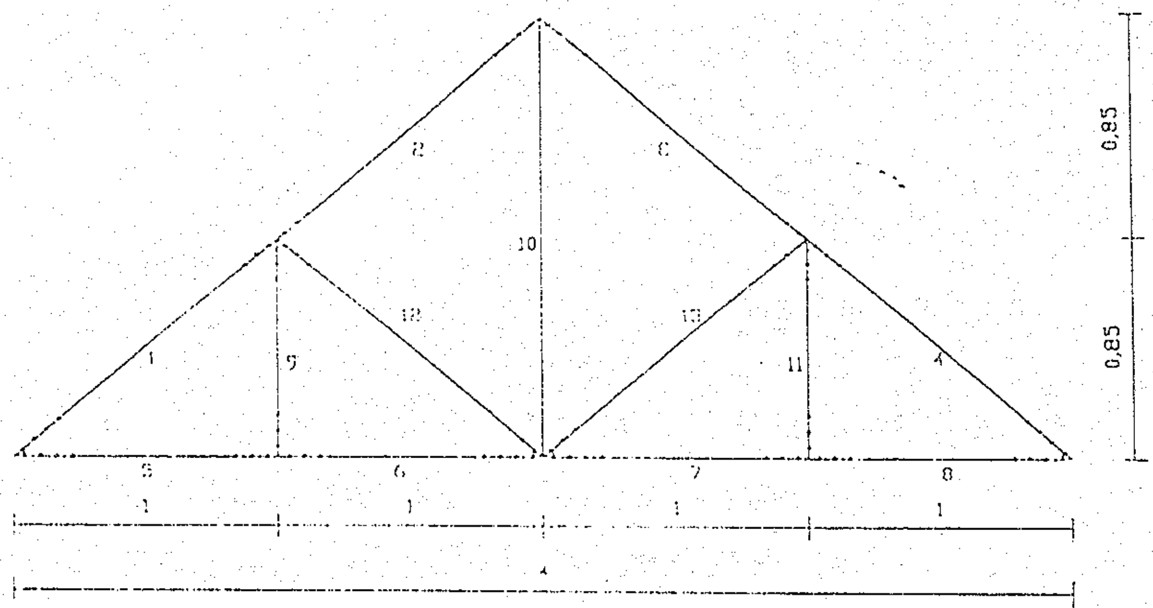
a. Dead load
- $P_1 = 4.00 \times (104.8 + 15) = 479.2$ kg

b. Wind load
- $W_1 = 4.00 \times 1.31 \times 20 = 104.8$ kg
  $W_2 = 4.00 \times 1.31 \times 16 = -83.84$ kg

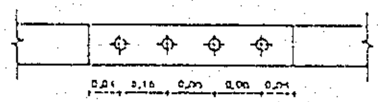
  $W_{1x} = W_{1y} = 104.8 \text{ Cos } 40 = 80.28$ kg
  $W_{2x} = W_{2y} = -83.84 \text{ Cos } 40^\circ = -64.23$ kg

c. Live load
- $P_2 = 100$ kg

ROOF TYPE K1



NO. Member	Profil	Bolt		Plate Thickness t (mm)
		Total	ø mm	
1 - 4	T 50.50.5	4 - 3	14	8
2 - 3	T 50.50.5	3 - 4	14	8
5 - 8	⊥ 50.50.5	4 - 3	14	8
6 - 7	⊥ 50.50.5	3 - 4	14	8
9 - 11	⊢ 50.50.5	3 - 3	14	8
10	Pipe ø 101,6	-	-	8
12 - 13	T 50.50.5	3 - 3	14	8



Roof K-1

ELECTRICAL BUILDING SIMONGAN

Prototype

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia. Bolt (cm)
L 50.50.5	0.8	2,400	3,700	1.4

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L 50.50.5	567	4	0	94	2	14
2	L 50.50.5	567	4	0	94	2	14
3	L 50.50.5	721	4	0	94	2	14
4	L 50.50.5	658	4	0	94	2	14
5	L 50.50.5	395	4	0	124	2	14
6	L 50.50.5	363	4	0	124	2	14
7	L 50.50.5	422	4	0	124	2	14
8	L 50.50.5	795	4	0	124	2	14
9	L 50.50.5	8	0	0	0	2	14
10	L 50.50.5	19	4	0	124	2	14
11	L 50.50.5	182	0	0	0	2	14
12	L 50.50.5	115	4	0	124	2	14
13	L 50.50.5	115	4	0	124	2	14

- Checking of members Strength of roof steel Truss Type K-1 base on the axial force:

a. Due to Tensile force

Maximum force on member T8 (loading Combination 2)
Force $F = 795$ kg

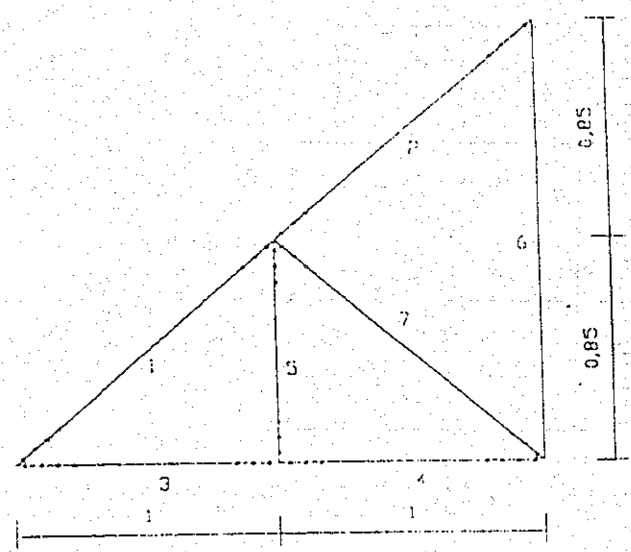
Try : Double angle steel of 50.50.5
Cross section area $A = 9.6$ cm^2

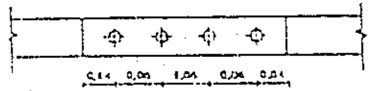
$\sigma_{all} = 0.6 \times Fy$
$= 0.6 \times 2{,}400 = 1{,}440$ kg/cm^2

Stress
$$\sigma = \frac{F}{A} = \frac{795}{9.6} = 82.81 \text{ kg/cm}^2 < \sigma_{all} = 1{,}440 \text{ kg/cm}^2 \text{ (OK)}$$

Hence double angle steel of 50.50.5 can be used as the members of roof truss type K - 1

ROOF TYPE K3





NO. Member	Profil	Bolt		Plate Thickness t (mm)
		Total	⌀ mm	
1	T 50.50.5	4 - 3	14	8
2	T 50.50.5	3 - 4	14	8
3	⊥ 50.50.5	4 - 3	14	8
4	⊥ 50.50.5	3 - 4	14	8
5	⊢ 50.50.5	3 - 3	14	8
6	pipe ⌀ 101,6	-	-	8
7	T 50.50.5	3 - 3	14	8

Roof K-2

ELECTRICAL BUILDING SIMONGAN

Prototype

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia. Bolt (cm)
L 50.50.5	0.8	2,400	3,700	1.4

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L 50.50.5	38	4	0	94	2	14
2	L 50.50.5	38	4	0	94	2	14
3	L 50.50.5	36	4	0	124	2	14
4	L 50.50.5	14	4	0	124	2	14
5	L 50.50.5	8	0	0	0	2	14
6	L 50.50.5	19	4	0	124	2	14

- Checking of members Strength of roof steel Truss Type K-3 base on the axial force:

a. Due to Tensile force

Maximum force on member T1 (loading Combination 2)
Force $F = 38$ kg

Try : Double angle steel of 50.50.5
Cross section area $A = 9.6\ cm^2$

$$\sigma_{all} = 0.6 \times Fy$$
$$= 0.6 \times 2{,}400 = 1{,}440\ kg/cm^2$$

Stress
$$\sigma = \frac{F}{A} = \frac{38}{9.6} = 3.96\ kg/cm^2 < \sigma_{all} = 1{,}440\ kg/cm^2\ \text{(OK)}$$

Hence double angle steel of 50.50.5 can be used as the members of roof truss type K - 2