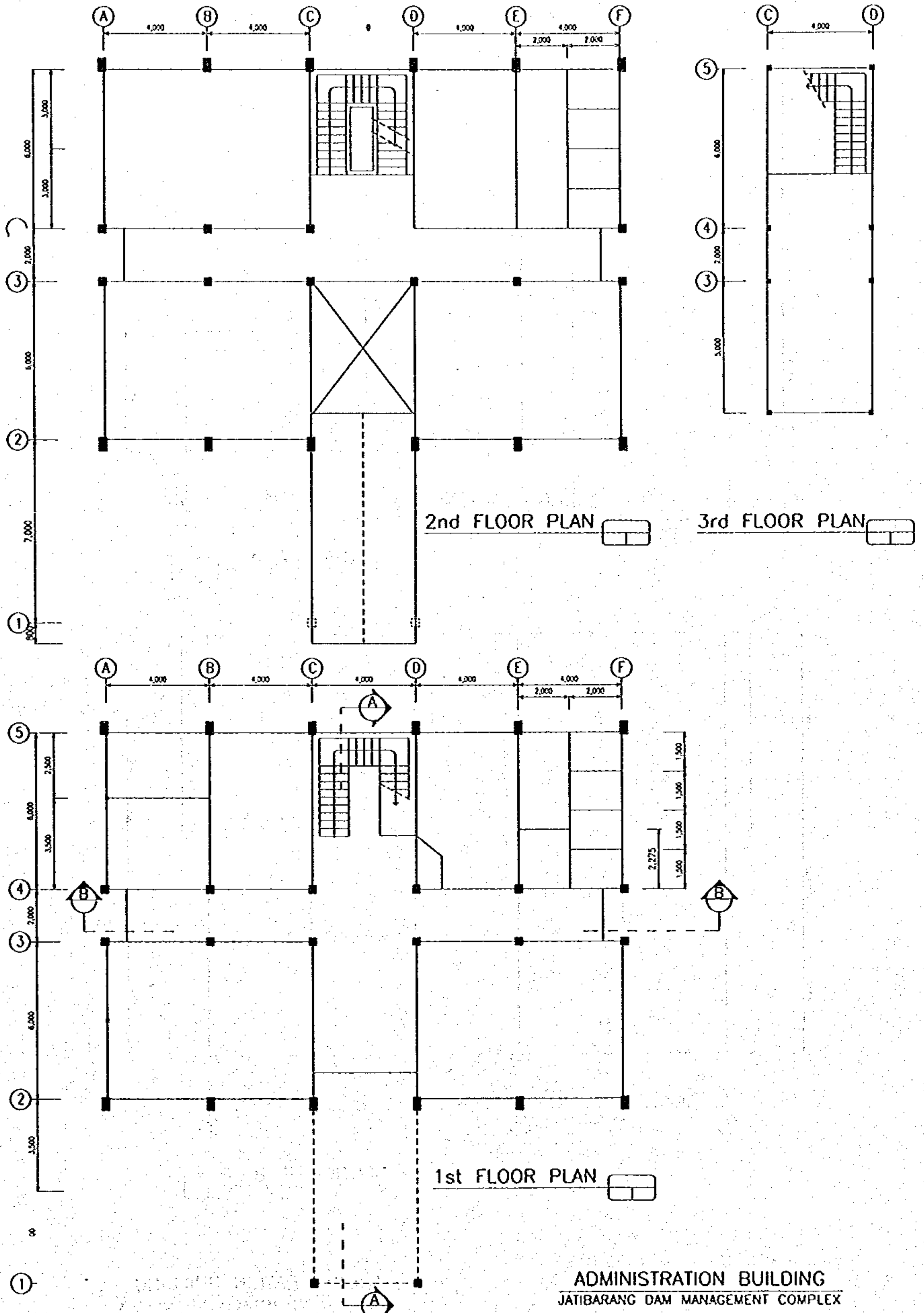
Chapter 5
ARCHITECTURAL DESIGN

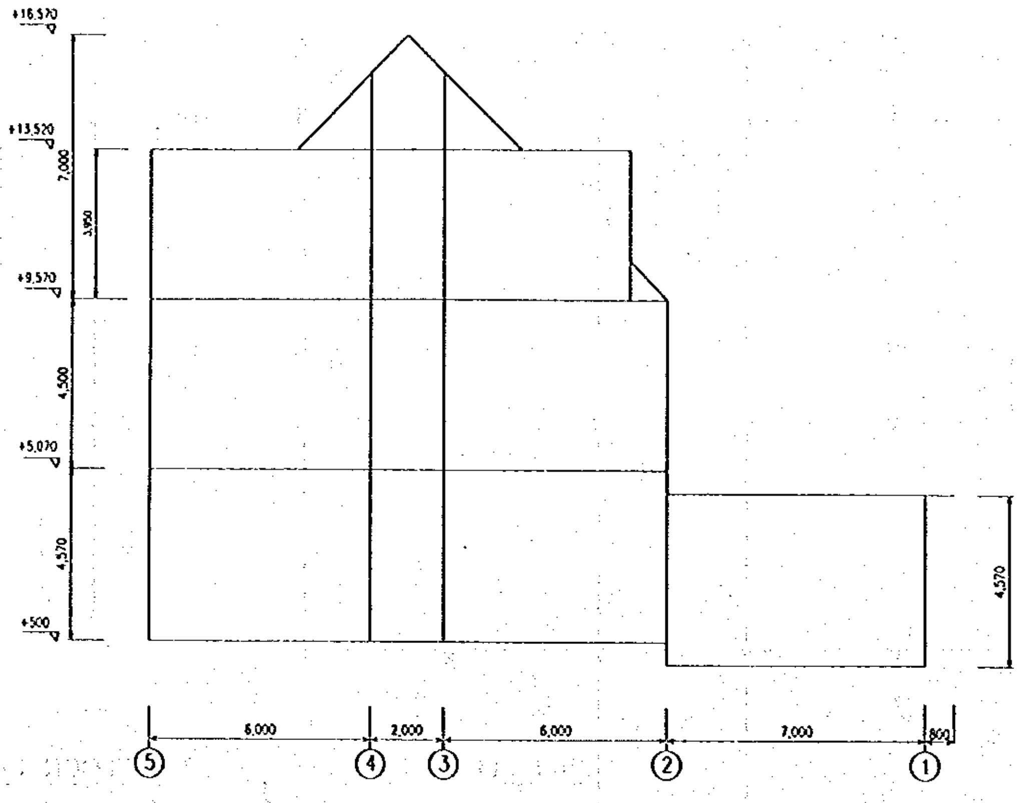
5.1 Design of Dam Management Complex

5.1.1 Administration Building
Structural Calculation

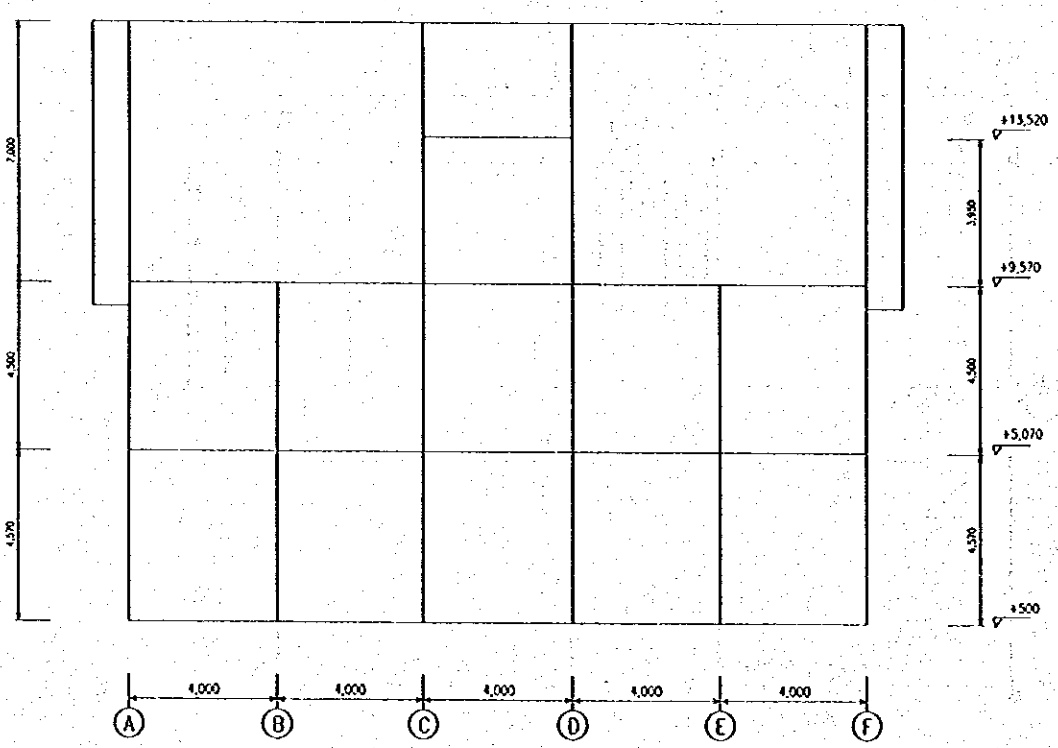
1. STRUCTURE



ADMINISTRATION BUILDING
JATIBARANG DAM MANAGEMENT COMPLEX

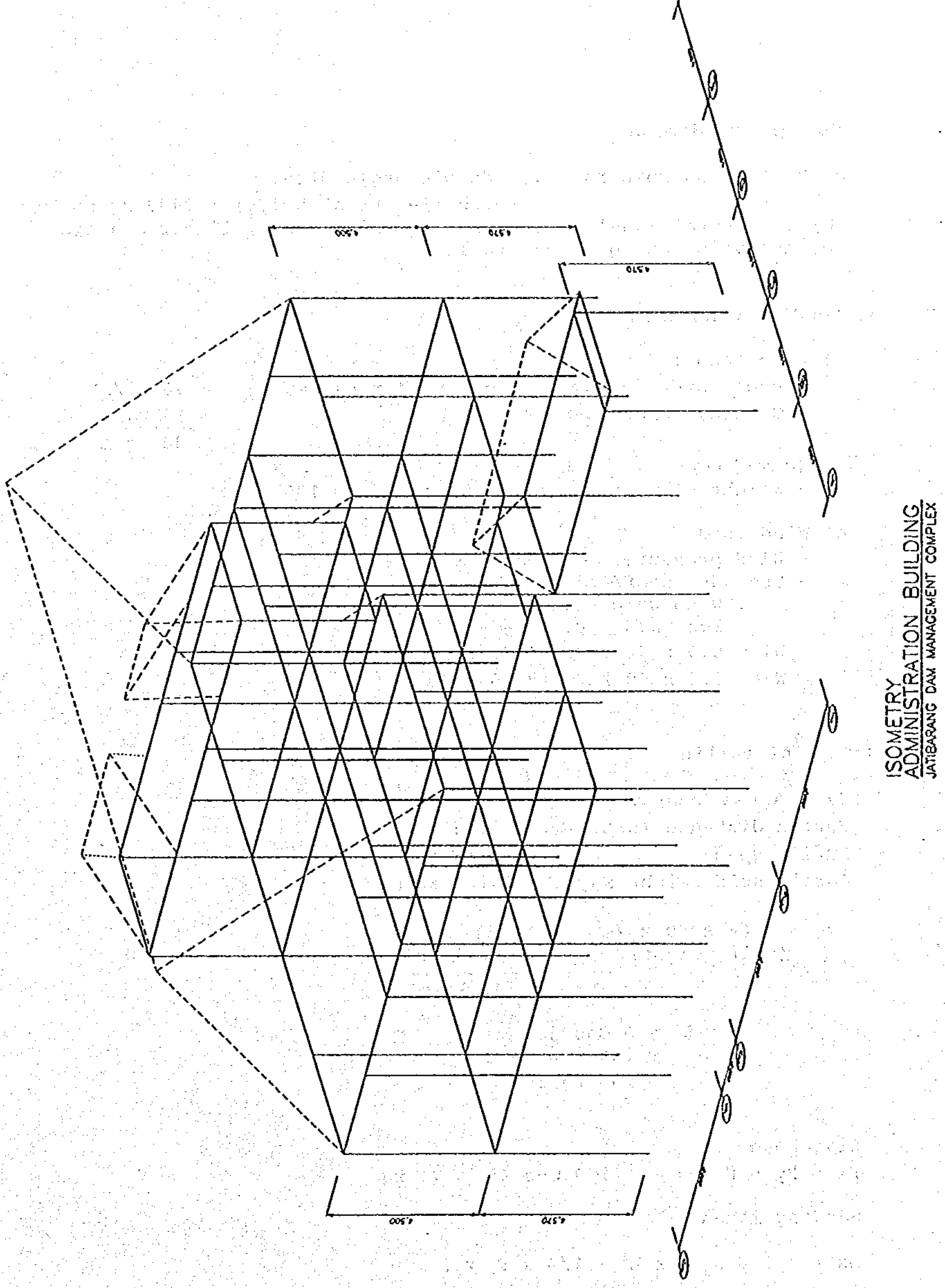


SECTION A–A



SECTION B–B

ADMINISTRATION BUILDING
JATIBARANG DAM MANAGEMENT COMPLEX


ISOMETRY
ADMINISTRATION BUILDING
JATIBARANG DAM MANAGEMENT COMPLEX

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

A. Roof Truss Type K-1
- Purlin distance (c/c) = 1.63 m
- Purlin span = 4.00 m
- Purlin self weight say = 15.00 kg/m'

$q_1 = 1.63 \times 80\ kg/m^2 \approx 131\ kg/m'$

q_2 (self weight) = 15 kg/m'

Q = 146 kg/m'

$Q_1 = Q_2 = Q \cos 45^o$

$= 146 \cos 45^o$

$\approx 105\ kg/m'$

- Live Load

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

- Bending moment

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

$Mx = 1/8 \times 105 \times 4^2 + 1/4 \times 71 \times 4 = 281\ kgm$

$My = Mx = 281\ kgm = 28{,}100\ kgcm$

- 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

$$\sigma = \sigma x + \sigma y$$
$$= \frac{Mx}{Wx} + \frac{My}{Wy}$$
$$= \frac{28,100}{88.6} + \frac{28,100}{73.2} = 317.16 + 383.88$$
$$= 701.04\ kg < \sigma_{all} = 1,400\ kg/cm^2\ (OK)$$

- Deflection

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

$$= \frac{5}{384} \times 1.05 \times \frac{400^4}{2.1x10^6x664} + \frac{1}{48} \frac{71 \times 400^3}{2.1x10^6x664}$$

$$= 0.25 + 0.07 = 0.32\ cm$$

$$fy = 0.35 + 0.09 = 0.44\ cm$$

$$f = (0.32^2 + 0.44^2)^{1/2} = 0.54\ cm$$

$$f = 0.54\ cm < \bar{f}_{all} = \frac{1}{360} L = \frac{400}{360} = 1.11\ cm\ (OK)$$

B. Roof Truss Type K-2

- Purlin distance (c/c) = 1.41 m
- Purlin span = 2.50 m
- Purlin self weight say = 15.00 kg/m'

$q_1 = 1.41 \times 80\ kg/m^2 \approx 113\ kg/m'$
q_2 (self weight) = 8 kg/m'

Q = 121 kg/m'

$$Q_1 = Q_2 = Q \cos 45^o$$
$$= 121 \cos 45^o$$
$$\approx 86\ kg/m'$$

- Live Load

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

- Bending moment

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

$Mx = 1/8 \times 86 \times 2.5^2 + 1/4 \times 71 \times 2.5 \approx 112$ kgm

$My = Mx = 112$ kgm $= 11{,}200$ kgcm

- Try Purlin of Lip Channel type :

150 x 50 x 50 x 3.2

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

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

- Stresses

$$\sigma = \sigma x + \sigma y = \frac{Mx}{Wx} + \frac{My}{Wy} = \frac{11{,}200}{58.4} + \frac{11{,}200}{13.2} = 191.78 + 848.48$$

$$= 1{,}040.26\ kg < \sigma_{all} = 1{,}400\ kg/cm^2\ (OK)$$

- Deflection

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

$$= \frac{5}{384} \times 0.86 \times \frac{250^4}{2.1x10^6x438} + \frac{1}{48}\frac{71 \times 250^3}{2.1x10^6x438}$$

$$= 0.05 + 0.03 = 0.08\ cm$$

$$fy = 0.29 + 0.15 = 0.44\ cm$$

$$f = (0.08^2 + 0.44^2)^{1/2} = 0.45\ cm$$

$$f = 0.45\ cm < f_{all} = \frac{1}{360} L = \frac{250}{360} = 0.69\ cm\ (OK)$$

5. Design of Roof Truss

A. Roof Truss type K1

a. Dead load

- $P_1 = 4.00 \times (131 + 15) = 584$ kg

b. Wind load

- $W_1 = 4.00 \times 1.63 \times 20 = 130.40$ kg
- $W_2 = 4.00 \times 1.63 \times 16 = -104.32$ kg

$W_{1X} = W_{1Y} = 130.40 \text{ Cos } 45^o = 92.21$ kg
$W_{2X} = W_{2Y} = -104.32 \text{ Cos } 45^o = -73.77$ kg

c. Live load

- $P_2 = 100$ kg

B. Roof Truss Type K-2

a. Dead load

- $P_1 = 2.50 \times 121 \approx 303$ kg

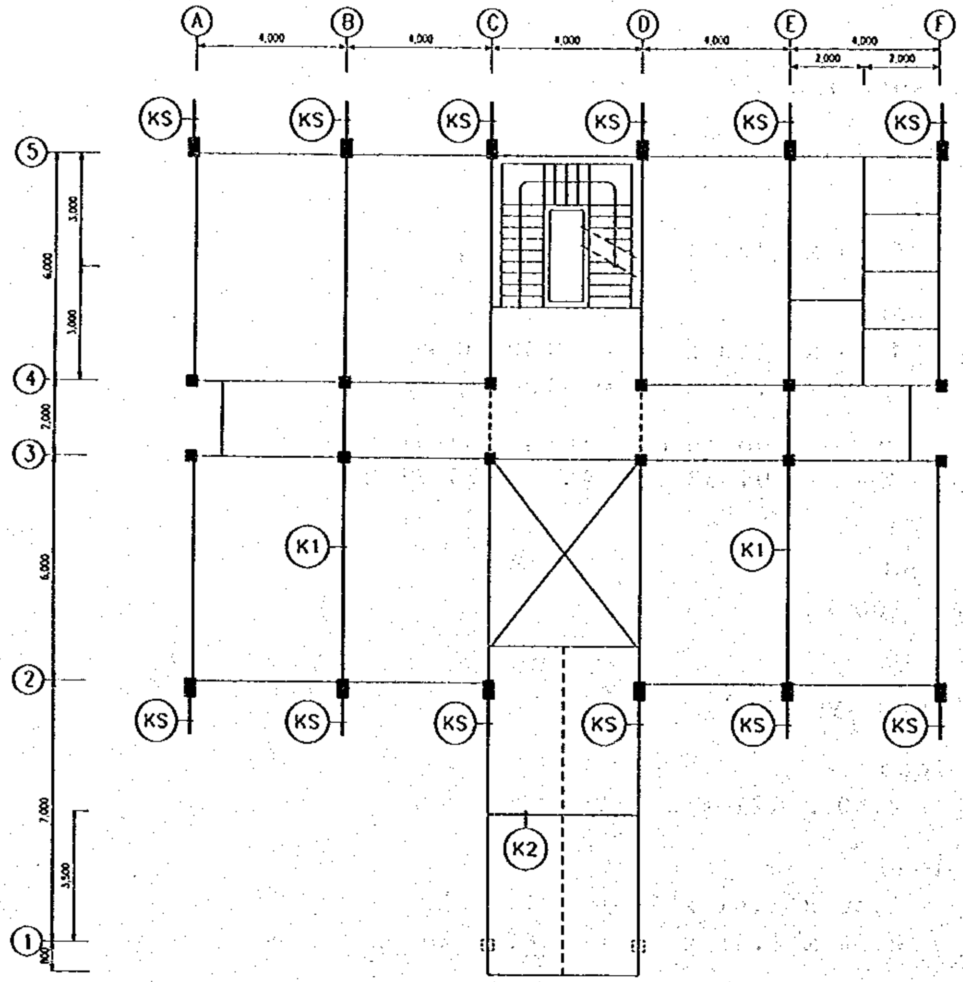
b. Wind load

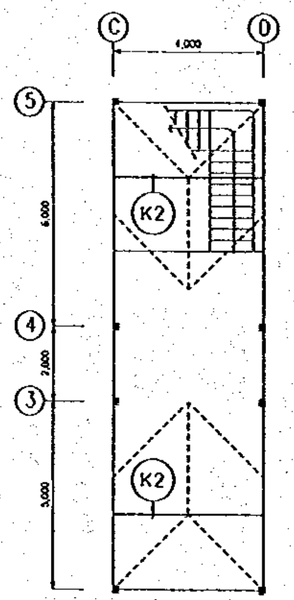
- $W_1 = 2.50 \times 1.41 \times 20 = 71.00$ kg
- $W_2 = 2.50 \times 1.41 \times 16 = -56.40$ kg

$W_{1X} = W_{1Y} = 71.00 \text{ Cos } 45^o = 50.20$ kg
$W_{2X} = W_{2Y} = -56.40 \text{ Cos } 45^o = -39.89$ kg

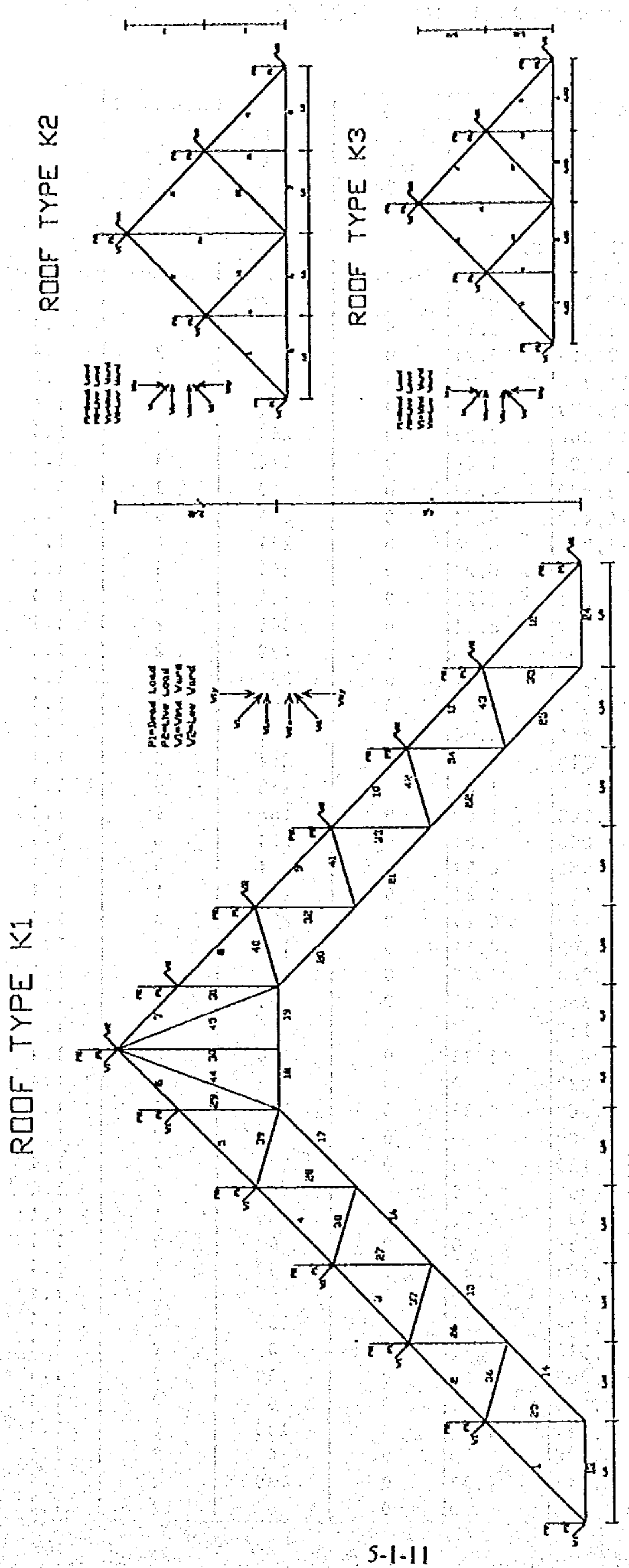
c. Live load

- $P_2 = 100$ kg





ROOF PLAN
ADMINISTRATION BUILDING
JATIBARANG DAM MANAGEMENT COMPLEX


ROOF TYPE K2
ROOF TYPE K3
ROOF TYPE K1

STEEL ROOF TRUSS JATIBARANG TYPE K-1
PROTOTYPE

PROFILE	PLATE THICKNESS (cm)	Fy (kg/cm2)	Fu (kg/cm2)	DIA.BOLT
L 50.50.5	1.0	3,700	2,400	1.4
L 60.60.6	1.0	3,700	2,400	1.4

FRAME ELEMENT FORCE

Member	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n bolt	dia.bolt (mm)
1	5,100	8.14	0	305	2	1.4
2	7,219	6.24	0	254	2	1.4
3	11,750	6.24	0	254	4	1.4
4	15,679	6.24	0	254	5	1.4
5	17,806	6.24	0	254	5	1.4
6	8,720	4.88	0	110	3	1.4
7	8,720	4.88	0	110	3	1.4
8	16,478	6.24	0	254	5	1.4
9	13,724	6.24	0	254	4	1.4
10	10,329	6.24	0	254	3	1.4
11	6,211	6.24	0	254	2	1.4
12	4,387	8.14	0	305	2	1.4
13	5,708	8.14	0	432	2	1.4
14	10,368	6.24	0	254	3	1.4
15	14,027	6.24	0	254	4	1.4
16	16,684	6.24	0	254	5	1.4
17	18,340	6.24	0	254	6	1.4
18	18,469	4.88	0	155	6	1.4
19	17,963	4.88	0	155	5	1.4
20	18,068	6.24	0	254	6	1.4
21	16,061	6.24	0	254	5	1.4
22	13,411	6.24	0	254	4	1.4
23	10,120	6.24	0	254	3	1.4
24	6,109	8.14	0	432	2	1.4
25	5,083	0	0	0	2	1.4
26	3,350	4.33	0	130	2	1.4
27	4,157	0	0	0	2	1.4
28	2,610	4.33	0	130	2	1.4
29	3,233	0	0	0	2	1.4
30	1,869	4.33	0	130	2	1.4
31	2,309	0	0	0	2	1.4
32	1,129	4.33	0	130	2	1.4
33	895	0	0	0	2	1.4
34	14,095	4.33	0	130	4	1.4
35	36	0	0	0	2	1.4
36	12,603	4.33	0	130	4	1.4
37	564	0	0	0	2	1.4
38	1,562	4.33	0	130	2	1.4
39	2,518	0	0	0	2	1.4
40	2,307	4.33	0	130	2	1.4
41	3,110	0	0	0	2	1.4
42	2,511	4.33	0	130	2	1.4
43	3,776	0	0	0	2	1.4
44	3,045	4.33	0	130	2	1.4
45	4,370	0	0	0	2	1.4

STEEL ROOF TRUSS JATIBARANG TYPE K-2
PROTOTYPE

PROFILE	PLATE THICKNESS (cm)	Fy (kg/cm2)	Fu (kg/cm2)	DIA.BOLT
L 50.50.5	0.5	3,700	2,400	1.4

FRAME ELEMENT FORCE

Member	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n bolt	dia.bolt (mm)
1	843	3.77	0	94.2	2	1.4
2	843	3.77	0	94.2	2	1.4
3	753	3.77	0	94.2	2	1.4
4	753	3.77	0	94.2	2	1.4
5	885	3.77	0	133.22	2	1.4
6	578	3.77	0	133.22	2	1.4
7	705	3.77	0	133.22	2	1.4
8	1,012	3.77	0	133.22	2	1.4
9	15	0	0	0	2	1.4
10	382	3.77	0	133.22	2	1.4
11	478	0	0	0	2	1.4
12	254	3.77	0	133.22	2	1.4
13	15	0	0	0	2	1.4

ROOF K-3

Jati barang Administration Building

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	Bolt diameter (cm)
L50.50.5	0.5	2,400	3,700	1.4

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L50.50.5	823	3	0	68	2	14
2	L50.50.5	823	3	0	6668	2	14
3	L50.50.5	288	3	0	686	2	14
4	L50.50.5	288	3	0	68	2	14
5	L50.50.5	856	3	0	96	2	14
6	L50.50.5	553	3	0	96	2	14
7	L50.50.5	681	3	0	96	2	14
8	L50.50.5	795	3	0	215	2	14
9	L50.50.5	12	0	0	0	2	14
10	L50.50.5	377	3	0	96	2	14
11	L50.50.5	525	0	0	0	2	14
12	L50.50.5	864	3	0	215	2	14
13	L50.50.5	25	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 T5 (loading Combination 2)
Force $F = 18{,}106$ kg
Length $L = 162.63$ cm

Try : Double angle steel of 70.70.7
Cross section area $A = 2 \times 9.40 = 18.80\ cm^2$

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

Stress
$$\sigma = \frac{F}{A} = \frac{18{,}106}{18.80} = 963\ kg/cm^2 < \sigma_{all} = 1{,}440\ kg/cm^2\ (OK)$$

b. Due to Compresion force

Maximum force on member T18 (loading Combination 2)
Force $F = 18{,}764$ kg
Length $L = 127.28$ cm

Try : Double angle steel of 70.70.7
Cross section area $A = 2 \times 9.40 = 18.80\ cm^2$
$ix = 2.28$ cm ; $Ix = 2x42.4 = 84.80\ cm^4$
$$\lambda = \frac{L}{ix} = \frac{127.28}{2.28} = 55.82 < 105$$
—— $\alpha = 0.788$

Stress :

$$\sigma = \alpha \times \sigma_{all}$$
$$= 0.788 \times 1{,}440$$
$$= 1{,}135\ kg/cm^2 < \sigma_{all} = 1{,}440\ kg/cm^2\ (OK)$$

Hence double angle steel of 70.70.7 can be used as the members of roof truss type K - 1

- **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 T1 & T2 (loading Combination 2)
Force F = 816 kg
Length L = 100 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

$$\sigma = \frac{F}{A} = \frac{816}{9.6} = 88.69 \text{ kg/cm}^2 < \sigma_{all} = 1{,}440 \text{ kg/cm}^2 \text{ (OK)}$$

b. Due to Compresion force

Maximum force on member T8 (loading Combination 2)
Force F = 973 kg (rounded)
Length L = 141.42 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

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

$$F_{all} = \frac{\pi^2 x(2.1x10^6)x22}{3x(176.78)^2}$$

$$= 4{,}863.56 \text{ kg} > F = 973 \text{ kg (OK)}$$

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

6. DATA FOR BUILDING

a. Dimensions

- length c - c column - 20,000 m
- width c - c column - 14,000 m
- height ground to 2^{nd} floor : 4,500 m
- height ground to 3^{rd} floor : 9,000 m
- height ground to roof truss : 13,100 m

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 K1 = 10.000 kg
- Truss type K2 = 2.000 kg

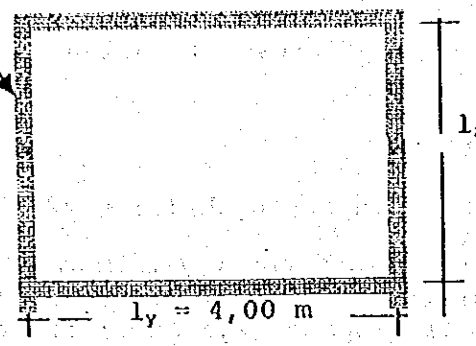
b) Slab dead load = 150 kg/m^2
c) Live load = 250 kg/m^2
d) Concrete self weight = 2.400 kg/m^3
e) Brick wall 15 cm thick = 250 kg/m^2
f) Soil Compression Stress = 20 kg/cm^2
(given by JICA Study Team)

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 :

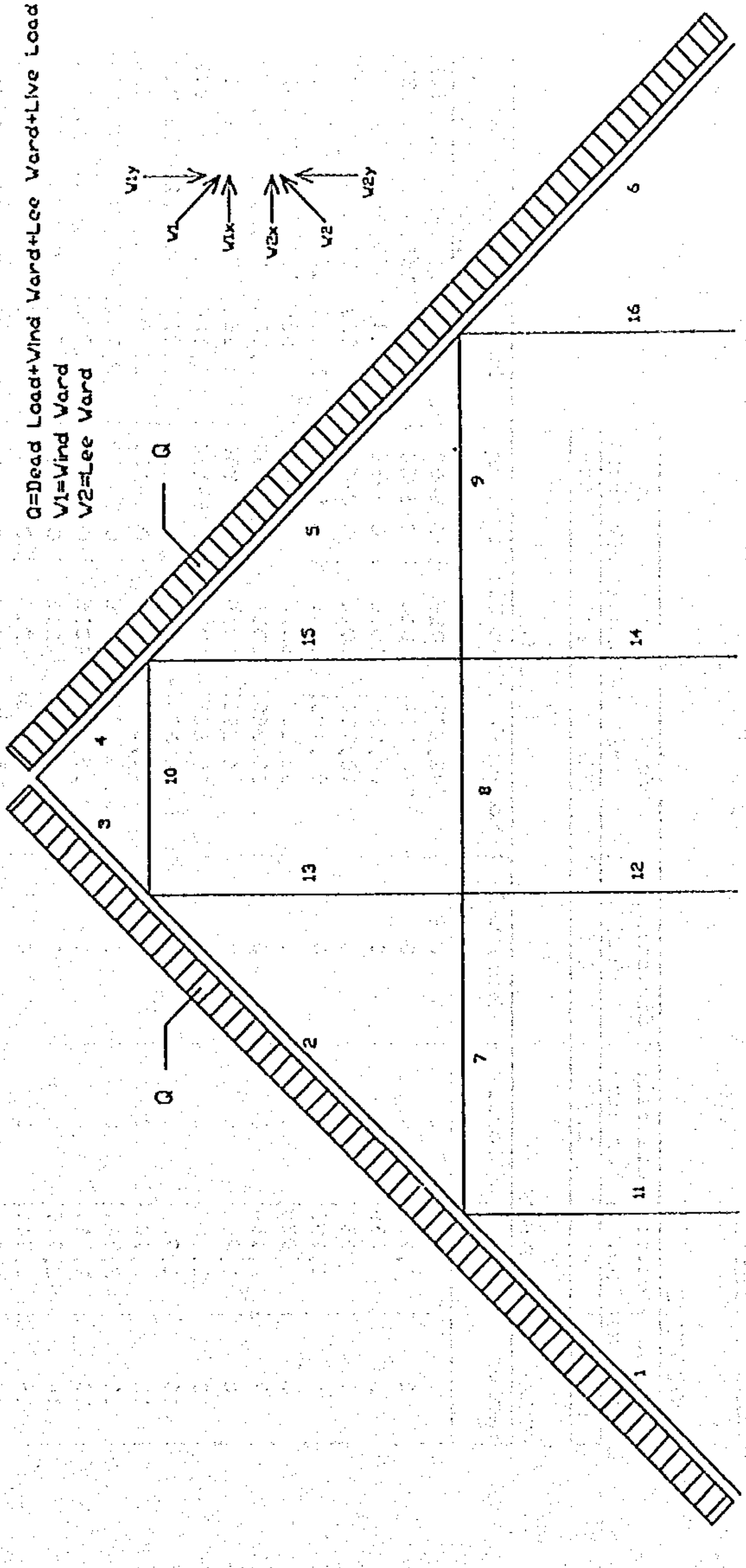
Plate self weight : 0.12 x 2,400 kg/m^3	=	288 kg/m^2
Plate dead load	=	150 kg/m^2
Live load	=	250 kg/m^2
q	=	688 kg/m^2



$l_y/l_x = 1.33$:
$M^-_{tx} = 0.001 \times 688 \times 3.00^2 \times 69 = 427.248$ kgm
$M^-_{ty} = 0.001 \times 688 \times 3.00^2 \times 57 = 352.944$ kgm
$M'_{lx} = 0.001 \times 688 \times 3.00^2 \times 31 = 191.952$ kgm
$M'_{ly} = 0.001 \times 688 \times 3.00^2 \times 19 = 177.648$ kgm
$M_{max} = 427.248$ kgm = 42,725 kgcm

$\delta = 0$ (single reinforcement)
$\phi = 2.73 > \phi_0 = \sigma_a / (n \times \sigma'_b) = 2,000 / (21 \times 70) = 1.36$ (OK)
$n\omega = 0.049$
$A_{steel} = \omega \times b \times h = 0.049/21 \times 100 \times 10 = 2.33$ cm^2
Used A_{steel} = dia. 10 - 15 cm = 5.5 cm^2 > 2.33 cm^2 (OK).

ROOF TYPE K5


Q=Dead Load+Wind Ward+Lee Ward+Live Load
V1=Wind Ward
V2=Lee Ward
Q
Q
V1y
V1
V1x
V2x
V2
V2y
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16

CONCRETE ROOF FRAME TYPE K-5 (OFFICE ADM. JATIBARANG)
COLOUM TYPE 1

PROTOTYPE

b (cm)	h (cm)	cover (cm)	dia.main bar (cm)	dia.stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
15	15	4	1.6	0.8	187	3200	2400

FRAME ELEMEN FORCE					Design				
member	Axial (kg)	Torsi (kg.cm)	Moment-2 (kg.cm)	Moment-3 (kg.cm)	Main Bar (mm)	Stirrup (mm)	Pu (kg)	Mox (kg.cm)	Moy (kg.cm)
1	928	0	0	92,107	8D16	O 8-50	928	111,811	111,811
2	1,298	0	0	90,109	8D16	O 8-50	1,297	109,320	109,320
3	239	0	0	64,929	4D16	O 8-50	239	79,686	79,686
4	239	0	0	43,714	4D16	O 8-50	239	79,686	79,686
5	1,298	0	0	90,109	8D16	O 8-50	1,297	109,320	109,320
6	928	0	0	92,107	8D16	O 8-50	928	111,811	111,811
7	1,186	0	0	15,865	4D16	O 8-150	1,186	79,072	79,072
8	1,268	0	0	6,293	4D16	O 8-150	1,268	79,016	79,016
9	1,186	0	0	15,865	4D16	O 8-150	1,186	79,072	79,072
10	1,701	0	0	18,303	4D16	O 8-150	1,701	78,712	78,712
11	3,200	0	0	5,373	4D16	O 8-150	3,197	77,520	77,520
12	2,744	0	0	700	4D16	O 8-150	2,746	77,844	77,844
13	1,744	0	0	16,878	4D16	O 8-150	1,745	78,680	78,680
14	2,744	0	0	700	4D16	O 8-150	2,746	77,844	77,844
15	1,744	0	0	16,878	4D16	O 8-150	1,745	78,680	78,680
16	3,200	0	0	5,373	4D16	O 8-150	3,197	77,520	77,520

COULOM TYPE 1

PROTOTYPE

b (cm)	h (cm)	cover (cm)	dia. main bar (cm)	dia. stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
30	50	4	1.6	1.0	187	3,200	2,400

Frame Element Force									
Member	Axial (kg)	Torsion	Moment-2	Moment-3	Main bar	Stirrup	Pu	Mox	Moy
3	15,028	234	348,639	239,997	8D16	o10-120	15,022	1,047,523	574,984
4	29,584	234	467,799	334,997	8D16	o10-120	29,573	8,871,941	653,585
5	32,936	234	241,402	177,641	8D16	o10-120	203,489	177,762	241,271
6	32,805	234	240,090	180,033	8D16	o10-120	203,482	180,003	240,327
7	29,590	234	469,625	297,105	8D16	o10-120	29,579	88,738	653,630
8	15,044	234	352,843	222,762	8D16	o10-120	15,037	1,047,706	575,076
21	14,946	234	325,542	236,494	8D16	o10-120	14,941	1,046,564	574,507
22	30,894	234	523,002	19,847	8D16	o10-120	30,884	92,653	663,380
23	31,804	234	402,075	176,760	8D16	o10-120	181,539	176,933	402,407
24	31,119	234	345,850	150,344	8D16	o10-120	31,108	150,288	65,030
25	30,146	234	531,633	276,978	8D16	o10-120	30,123	90,369	657,719
26	20,472	234	411,184	231,357	8D16	o10-120	20,460	110,931	605,320
27	1,947	269	261,296	187,703	8D16	o10-120	948	860,658	493,082
28	8,228	269	340,064	893	8D16	o10-120	8,226	24,679	533,672
29	11,973	269	642,531	201,638	16D16	o10-120	11,971	1,658,596	869,897
30	12,015	269	644,512	198,815	16D16	o10-120	12,013	1,658,665	869,969
31	8,228	269	340,821	40,132	8D16	o10-120	8,227	24,680	533,673
32	1,949	269	263,558	187,232	8D16	o10-120	1,949	880,879	493,092
45	1,940	269	276,427	187,036	8D16	o10-120	1,941	860,768	493,038
46	8,226	269	412,035	16,371	8D16	o10-120	8,225	24,674	533,662
47	11,702	269	527,665	198,006	12D16	o10-120	11,699	1,354,389	724,281
48	11,801	269	476,691	203,566	12D16	o10-120	11,799	1,355,241	724,505
49	5,817	269	419,003	3,739	8D16	o10-120	5,817	17,452	518,437
50	2,418	269	342,765	186,791	8D16	o10-120	2,419	887,148	496,157

COULOM TYPE 2

PROTOTYPE

b (cm)	h (cm)	cover (cm)	dia. main bar (cm)	dia. stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
30	30	4	1.6	1.0	187	3,200	2,400

Frame Element Force									
Member	Axial (kg)	Torsion	Moment-2	Moment-3	Main bar	Stirrup	Pu	Mox	Moy
1	2,147	95	17,660	19,050	8D16	o10-120	2,148	451,942	451,942
2	2,139	95	18,503	16,416	8D16	o10-120	2,141	451,912	451,912
9	16,144	95	123,909	235,609	8D16	o10-120	16,142	495,547	495,547
10	26,072	95	272,667	13,357	8D16	o10-120	26,079	62,590	535,484
11	33,587	95	185,943	146,955	8D16	o10-120	123,583	147,037	186,070
12	34,113	95	184,886	146,360	8D16	o10-120	123,788	146,364	184,840
13	26,199	95	269,562	13,827	8D16	o10-120	26,006	62,694	535,859
14	15,524	95	123,839	236,151	8D16	o10-120	15,519	496,069	496,069
15	16,083	95	137,747	235,439	8D16	o10-120	16,080	495,598	495,598
16	26,151	95	329,077	537,827	8D16	o10-120	28,169	67,605	541,507
17	34,244	95	273,518	106,734	8D16	o10-120	115,849	106,804	273,427
18	32,949	95	234,095	94,647	8D16	o10-120	32,970	94,708	553,898
19	27,454	95	316,673	20,980	8D16	o10-120	27,457	65,697	539,493
20	17,799	95	161,528	237,579	8D16	o10-120	116,167	237,815	161,518
33	3,241	109	118,288	177,421	8D16	o10-120	3,242	456,420	456,420
34	1,166	109	214,105	9,976	8D16	o10-120	1,167	447,851	447,851
35	16,039	109	337,829	161,232	12D16	o10-120	16,033	607,904	607,904
36	16,030	109	338,411	160,830	12D16	o10-120	16,033	607,904	607,904
37	1,166	109	212,155	10,421	8D16	o10-120	11,674	447,851	447,851
38	3,202	109	117,325	177,714	8D16	o10-120	3,203	456,266	456,266
39	3,199	109	106,315	177,355	8D16	o10-120	3,200	456,252	456,252
40	1,166	109	237,203	3,920	8D16	o10-120	1,167	447,851	447,851
41	11,030	109	324,111	156,308	8D16	o10-120	11,027	484,971	484,971
42	11,271	109	266,976	160,568	8D16	o10-120	11,267	485,729	485,729
43	1,166	109	227,522	15,849	8D16	o10-120	1,167	447,851	447,851
44	3,240	109	125,917	176,751	8D16	o10-120	3,241	456,419	456,419

COULOM TYPE 3

PROTOTYPE

b (cm)	h (cm)	cover (cm)	dia. main bar (cm)	dia. stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
20	20	4	1.6	0.8	187	3200	2400

Frame Element Force									
Member	Axial (kg)	Torsion	Moment-2	Moment-3	Main bar	Stirrup	Pu	Mox	Moy
53	2,515	9	69,704	35,806	4D16	o8-70	2,515	132,754	132,754
54	2,599	9	70,956	35,872	4D16	o8-70	2,597	132,893	132,893
55	3,178	9	69,799	31,837	4D16	o8-70	178	133,694	133,694
56	3,293	9	72,187	31,691	4D16	o8-70	3,293	133,850	133,850
59	2,963	9	108,284	13,690	4D16	o8-70	2,964	133,360	133,360
60	3,066	9	111,704	13,540	4D16	o8-70	3,067	133,499	133,499
61	1,856	9	96,261	43,314	8D16	o8-70	1,857	201,360	201,360
62	1,900	9	98,015	43,396	8D16	o8-70	1,901	201,237	201,237

COULOM TYPE 4

PROTOTYPE

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

Frame Element Force									
Member	Axial (kg)	Torsion (kg.cm)	Moment-2 (kg.cm)	Moment-3 (kg.cm)	Main bar (mm)	Stirrup (mm)	Pu (kg)	Mox (kg.cm)	Moy (kg.cm)
51	292	3	4,675	1,920	4D16	o8-40	292	65,878	65,678
52	292	3	5,014	1,927	4D16	o8-40	292	65,876	65,878
57	292	3	1,126	1,036	4D16	o8-40	292	65,878	65,878
58	.292	3	1,209	1,013	4D16	o8-40	292	65,876	65,878

BEAM TYPE a

PROTOTYPE

b (cm)	h (cm)	cover (cm)	dia. main bar (cm)	dia. stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
25	50	4	1.6	1	187	3200	2400

Frame Element Force					Design											
Member	Axial	Shear	Torsion	Moment	Main bar	Left bars			Mid bars			Right			Stirrup bars	Mu
	(kg)	(kg)	(kg.cm)	(kg.cm)	(mm)	top	middle	bottom	top	middle	bottom	top	middle	bottom	(mm)	(kg. cm)
97	0	8,410	112,297	837,001	D16	3D16	2012	2D16	2D16	2012	3D16	3D16	2012	4D16	o10-75	843,861
98	0	9,339	35,797	881,012	D16	3D16	2012	4D16	2D16	2012	3D16	5D16	2012	3D16	o10-120	1,020,331
99	0	857	117	602,558	D16	3D16	2012	2D16	3D16	2012	2D16	3D16	2012	2D16	o10-200	645,072
100	0	9,290	35,293	875,242	D16	5D16	2012	3D16	2D16	2012	3D16	3D16	2012	4D16	o10-120	1,020,331
101	0	8,341	111,145	828,163	D16	3D16	2012	4D16	2D16	2012	3D16	3D16	2012	2D16	o10-75	843,861
102	0	10,948	42,206	1,547,112	D16	5D16	2012	3D16	2D16	2012	3D16	5D16	2012	8D16	o10-100	161,354
103	0	10,818	46,851	1,546,878	D16	5D16	2012	8D16	2D16	2012	3D16	5D16	2012	3D16	o10-120	1,359,187
104	0	9,335	2,057	1,275,047	D16	4D16	2012	3D16	2D16	2012	3D16	5D16	2012	7D16	o10-120	1,359,187
105	0	9,625	3,183	1,275,818	D16	5D16	2012	7D16	2D16	2012	3D16	4D16	2012	3D16	o10-120	1,196,192
106	0	1,228	52,608	1,120,998	D16	6D16	2012	4D16	3D16	2012	2D16	2D16	2012	3D16	o10-75	1,020,331
107	0	7,055	149,477	889,598	D16	2D16	2012	3D16	2D16	2012	3D16	3D16	2012	5D16	o10-60	1,020,331
108	0	8,625	37,623	868,711	D16	3D16	2012	5D16	2D16	2012	3D16	3D16	2012	2D16	o10-120	1,020,331
109	0	11,055	32,588	1,075,787	D16	5D16	2012	3D16	2D16	2012	3D16	4D16	2012	6D12	o10-120	1,359,187
110	0	3,390	240,513	1,249,473	D16	4D16	2012	6D16	4D16	2012	6D16	5D16	2012	7D16	o10-40	1,359,187
111	0	10,414	106,455	1,250,833	D16	5D16	2012	7D16	2D16	2012	3D16	5D16	2012	3D16	o10-70	1,359,187
112	0	9,657	58,611	1,004,506	D16	4D16	2012	3D16	2D16	2012	3D16	3D16	2012	5D16	o10-100	1,020,331
113	0	2,444	174,392	1,002,574	D16	3D16	2012	5D16	3D16	2012	5D16	3D16	2012	5D16	o0-70	1,020,331
114	0	9,970	5,892	882,031	D16	3D16	2012	5D16	2D16	2012	3D16	4D16	2012	3D16	o10-120	1,020,331
115	0	8,783	37,160	894,074	D16	4D16	2012	3D16	2D16	2012	3D16	3D16	2012	5D16	o10-120	1,020,331
116	0	6,892	149,189	895,073	D16	3D16	2012	5D16	2D16	2012	3D16	3D16	2012	2D16	o10-60	1,020,331
117	0	1,211	52,140	1,116,115	D16	3D16	2012	2D16	3D16	2012	2D16	6D16	2012	4D16	o10-90	1,196,192
118	0	9,631	3,560	1,274,996	D16	4D16	2012	3D16	2D16	2012	3D16	5D16	2012	7D16	o10-120	1,359,187
119	0	9,388	2,526	1,274,380	D16	5D16	2012	7D16	2D16	2012	3D16	4D16	2012	3D16	o10-120	1,359,187
120	0	1,027	757	1,422,449	D16	5D16	2012	3D16	2D16	2012	3D16	5D16	2012	8D16	o10-120	1,613,543
121	0	12,608	8,747	1,422,457	D16	5D16	2012	8D16	2D16	2012	3D16	5D16	2012	3D16	o10-120	1,613,543
122	0	11,440	112,649	994,447	D16	4D16	2012	3D16	2D16	2012	3D16	3D16	2012	5D16	o10-60	1,020,331
123	0	10,385	36,379	1,037,295	D16	3D16	2012	5D16	2D16	2012	3D16	6D16	2012	4D16	o10-120	1,196,192
124	0	1,418	663	710,039	D16	4D16	2012	3D16	3D16	2012	2D16	3D16	2012	2D16	o10-200	843,861
125	0	9,308	35,620	870,291	D16	5D16	2012	3D16	2D16	2012	3D16	3D16	2012	4D16	o10-120	1,020,331
126	0	8,437	11,636	838,445	D16	3D16	2012	4D16	2D16	2012	3D16	3D16	2012	2D16	o10-80	843,861
140	0	11,937	221,210	654,087	D16	4D16	2012	3D16	3D16	2012	2D16	2D16	2012	3D16	o10-35	843,861
141	0	9,321	1,800	711,252	D16	2D16	2012	3D16	2D16	2012	3D16	4D16	2012	3D16	o10-120	645,072
143	0	6,022	7,591	547,951	D16	3D16	2012	2D16	2D16	2012	3D16	2D16	2012	3D16	o10-75	645,072
144	0	5,947	55,360	547,779	D16	2D16	2012	3D16	2D16	2012	3D16	3D16	2012	2D16	o10-120	645,072
146	0	5,968	55,085	541,427	D16	3D16	2012	2D16	2D16	2012	3D16	2D16	2012	2D16	o10-120	645072
147	0	6,000	7,832	541,255	D16	2D16	2012	3D16	2D16	2012	3D16	3D16	2012	2D16	o10-120	326,634
149	0	9,329	1,601	715,374	D16	4D16	2012	3D16	3D16	2012	4D16	2D16	2012	3D16	o10-120	843,861
150	0	11,890	22,387	651,861	D16	2D16	2012	3D16	3D16	2012	2D16	4D16	2012	3D16	o10-35	843,861

BEAM TYPE b

PROTOTYPE

b (cm)	h (cm)	cover (cm)	dia. main bar (cm)	dia. stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
20	30	4	1.6	1	187	3200	2400

Frame Element Force					Design											
Member	Axial	Shear	Torsion	Moment	Main bar	Left bars			Mid bars			Right			Stirrup bars	Mu
	(kg)	(kg)	(kg cm)	(kg cm)	(mm)	top	middle	bottom	top	middle	bottom	top	middle	bottom	(mm)	(kg. cm)
63	0	4,865	5,769	357,015	D16	3D16	-	2D16	2D16	-	2D16	4D16	-	3D16	o10-75	400,403
64	0	4,902	11,130	362,856	D16	4D16	-	3D16	2D16	-	2D16	3D16	-	2D16	o10-75	400,403
65	0	350	123	38,368	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-120	230,335
66	0	48,932	11,166	361,032	D16	3D16	-	2D16	2D16	-	2D16	4D16	-	3D16	o10-75	400,403
67	0	4,894	5,769	358,804	D16	4D16	-	3D16	2D16	-	2D16	3D16	-	2D16	o10-75	400,403
68	0	3,048	87	202,085	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-75	400,403
69	0	4,310	1,660	345,321	D16	2D16	-	2D16	2D16	-	3D16	4D16	-	3D16	o10-75	400,403
70	0	4,209	888	344,196	D16	4D16	-	3D16	2D16	-	3D16	2D16	-	2D16	o10-75	400,403
71	0	4,214	999	345,343	D16	2D16	-	2D16	2D16	-	2D16	4D16	-	3D16	o10-75	400,403
72	0	4,314	1,616	346,365	D16	4D16	-	3D16	2D16	-	3D16	2D16	-	2D16	o10-75	400,403
73	0	6,857	36,257	377,116	D16	4D16	-	3D16	2D16	-	2D16	2D16	-	2D16	o10-75	400,403
74	0	6,988	28,329	553,968	D16	2D16	-	2D16	2D16	-	2D16	7D16	-	5D16	o10-75	230,335
75	0	6,652	7,010	527,227	D16	6D16	-	4D16	2D16	-	3D16	4D16	-	3D16	o10-75	594,258
76	0	1,738	26	188,722	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-120	230,335
77	0	6,653	6,960	527,713	D16	4D16	-	3D16	2D16	-	3D16	6D16	-	4D16	o10-75	594,258
78	0	6,986	28,433	53,263	D16	7D16	-	5D16	2D16	-	2D16	2D16	-	2D16	o10-75	566,582
79	0	6,858	38,025	377,582	D16	2D16	-	2D16	2D16	-	2D16	4D16	-	3D16	o10-75	400,403
80	0	6,871	36,513	377,617	D16	4D16	-	3D16	2D16	-	2D16	2D16	-	2D16	o10-75	400,403
81	0	7,009	32,551	559,147	D16	2D16	-	2D16	2D16	-	2D16	7D16	-	5D16	o10-75	566,582
82	0	6,508	4,587	504,557	D16	6D16	-	4D16	2D16	-	3D16	4D16	-	3D16	o10-75	594,258
83	0	3,297	3,272	299,666	D16	3D16	-	2D16	2D16	-	2D16	3D16	-	2D16	o10-75	326,635
84	0	6,520	6,672	512,052	D16	4D16	-	3D16	2D16	-	3D16	6D16	-	4D16	o10-85	594,257
85	0	6,976	30,208	549,349	D16	7D16	-	5D16	2D16	-	2D16	2D16	-	2D16	o10-75	566,582
86	0	6,904	39,811	380,613	D16	2D16	-	2D16	2D16	-	2D16	4D16	-	3D16	o10-75	400,403
87	0	4,428	1,595	390,895	D16	2D16	-	2D16	2D16	-	2D16	4D16	-	3D16	o10-75	400,403
88	0	6,849	1,360	479,752	D16	5D16	-	3D16	3D16	-	4D16	4D16	-	3D16	o10-75	472,737
89	0	1,951	2,063	207,925	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-120	230,335
90	0	4,052	1,932	327,210	D16	3D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-75	400,403
91	0	4,283	1,940	336,713	D16	4D16	-	3D16	2D16	-	3D16	2D16	-	2D16	o10-75	400,403
92	0	4,905	9,715	361,781	D16	3D16	-	2D16	2D16	-	2D16	4D16	-	3D16	o10-75	400,403
93	0	4,871	5,855	355,794	D16	4D16	-	3D16	2D16	-	2D16	3D16	-	2D16	o10-75	400,403
94	0	3,052	2,691	217,525	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-75	230,335
95	0	4,885	9,144	360,603	D16	3D16	-	2D16	2D16	-	2D16	4D16	-	3D16	o10-75	400,403
96	0	4,893	5,832	358,365	D16	4D16	-	3D16	2D16	-	2D16	3D16	-	2D16	o10-75	400,403
127	0	1,541	336	70,019	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-120	230,335
128	0	839	1,225	290,099	D16	3D16	-	2D16	2D16	-	2D16	3D16	-	2D16	o10-120	326,635

Frame Element Force					Design												
Member	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	Main bar (mm)	Left bars			Mid bars			Right			Stirrup bars (mm)	Mu (kg. cm)
						top	middle	bottom	top	middle	bottom	top	middle	bottom		
129	0	1,479	8,369	239,517	D16	3D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-120	326,635
130	0	1,865	6,615	293,326	D16	2D16	-	2D16	2D16	-	2D16	3D16	-	2D16	o10-120	326,635
131	0	1,483	1,359	308,436	D16	3D16	-	2D16	2D16	-	2D16	3D16	-	2D16	o10-120	326,635
132	0	1,540	16	68,065	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-120	230,335
135	0	340	10	18,170	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-120	230,335
137	0	347	24	20,714	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-120	326,635
139	0	349	91	3,498	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-120	230,335
142	0	2,910	4,295	301,758	D16	3D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-120	230,335
145	0	3,057	531	182,323	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-75	230,335
148	0	2,994	4,043	307,970	D16	2D16	-	2D16	2D16	-	2D16	3D16	-	2D16	o10-120	326,635
151	0	4,899	89	284,532	D16	3D16	-	2D16	2D16	-	3D16	3D16	-	2D16	o10-75	230,335
152	0	3,586	129	251,742	D16	2D16	-	2D16	2D16	-	3D16	2D16	-	2D16	o10-75	230,335
153	0	3,281	356	218,423	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-75	230,335
154	0	1,882	172	164,117	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	o10-120	230,336

BEAM TYPE c

PROTOTYPE

b (cm)	h (cm)	cover (cm)	dia. main bar (cm)	dia. stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
20	50	4	1.6	1	187	3200	2400

Frame Element Force					Design											
Member	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	Main bar (mm)	Left bars			Mid bars			Right			Stirrup bars (mm)	Mu (kg.cm)
						top	middle	bottom	top	middle	bottom	top	middle	bottom		
133	0	1,727	12,349	401,419	D16	2D16	2o12	2D16	2D16	2o12	2D16	2D16	2o12	2D16	o10-220	436,222
134	0	633	1,150	48,155	D16	2D16	2o12	2D16	2D16	2o12	2D16	2D16	2o12	2D16	o10-220	436,222
136	0	629	1,536	48,322	D16	2D16	2o12	2D16	2D16	2o12	2D16	2D16	2o12	2D16	o10-220	436,222
138	0	1,735	12,657	404,505	D16	2D16	2o12	2D16	2D16	2o12	2D16	2D16	2o12	2D16	o10-220	436,222

BEAM TYPE d

PROTOTYPE

b (cm)	h (cm)	cover (cm)	dia. main bar (cm)	dia. stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv (kg/cm2)
15	20	4	1.6	0.8	187	3200	2400

Frame Element Force					Design											
Member	Axial	Shear	Torsion	Moment	Main bar	Left bars			Mid bars			Right			Stirrup bars	Mu
	(kg)	(kg)	(kg.cm)	(kg.cm)	(mm)	top	middle	bottom	top	middle	bottom	top	middle	bottom	(mm)	(kg. cm)
155	0	201	1,027	18,008	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-50	131,270
156	0	207	9,420	20,973	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-50	131,270
157	0	209	9,484	21,329	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
158	0	199	996	17,582	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
159	0	270	1,340	32,544	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
160	0	101	37	8,461	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
161	0	279	1,323	36,180	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,277
162	0	195	1,123	16,952	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
163	0	206	6,157	20,577	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
164	0	209	5,991	21,371	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
165	0	200	1,561	17,866	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
166	0	277	1,337	34,506	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
167	0	111	13	7,615	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
168	0	270	1,314	32,256	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
169	0	173	313	15,143	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
170	0	173	304	10,281	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
171	0	1,209	100	127,384	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-50	131,270
172	0	861	1,827	122,370	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,281
173	0	299	422	68,944	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
174	0	1,129	2,080	158,633	D16	3D16	-	2D16	2D16	-	2D16	2D16	-	3D16	ø8-80	165,343
175	0	1,028	1,067	157,425	D16	2D16	-	3D16	2D16	-	2D16	2D16	-	2D16	ø8-80	165,343
176	0	995	1,050	151,626	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	3D16	ø8-80	131,298
177	0	1,093	2,087	152,751	D16	2D16	-	3D16	2D16	-	2D16	3D16	-	2D16	ø8-80	165,343
178	0	290	412	66,339	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,270
179	0	836	1,818	117,657	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,267
180	0	1,165	102	122,332	D16	2D16	-	2D16	2D16	-	2D16	2D16	-	2D16	ø8-80	131,281

CONCRETE ROOF FRAME TYPE K-5 (OFFICE ADM. JATIBARANG)
STAIR

PROTOTYPE

b (cm)	h (cm)	cover (cm)	dia.main bar (cm)	dia.stirrup (cm)	fc (kg/cm2)	fy (kg/cm2)	fv kg/cm2)
100	25	4	1.6	0.8	225	3200	2400

FRAME ELEMEN FORCE

member	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	Design Main Bar (mm)	Left Bar			Mid Bar			Right Bar			Stirrup (mm)	Mu (kg.cm)
						top	middle	bottom	top	middle	bottom	top	middle	bottom		
1	2,245	845	0	67,793	D16	5D16	-	3D16	3D16	-	5D16	3D16	-	5D16	O 8-250	543528
2	2,032	1276	0	67,793	D16	3D16	-	5D16	5D16	-	3D16	3D16	-	5D16	O 8-250	543231
3	764	1133	0	10,310	D16	5D16	-	3D16	3D16	-	5D16	3D16	-	5D16	O 8-250	541460
4	1,092	764	0	15,890	D16	3D16	-	5D16	5D16	-	3D16	5D16	-	3D16	O 8-250	539159
5	574	690	0	13,576	D16	5D16	-	3D16	5D16	-	3D16	5D16	-	3D16	O 8-250	540890
6	2,112	1244	0	19,541	D16	3D16	-	5D16	3D16	-	5D16	5D16	-	3D16	O 8-250	537420
7	1,460	2112	0	43,802	D16	3D16	-	5D16	5D16	-	3D16	5D16	-	3D16	O 8-250	538642
8	3,336	1257	0	55,081	D16	3D16	-	5D16	5D16	-	3D16	3D16	-	5D16	O 8-250	537938
9	2,930	2030	0	115,992	D16	3D16	-	5D16	3D16	-	5D16	3D16	-	5D16	O 8-250	544481

- Checking of Beam reinforcement bar & stress

On Beam No. F138

Maximum Bending Moment	= 404,505	kgcm	
b (width)	= 20	cm	
h_t (height)	= 50	cm	
Concrete cover	= 4	cm	
Fc	= 225	kg/cm²	$\longrightarrow \bar{\sigma}'_b = 130$ kg/cm²
Fu	= 3,200	kg/cm²	$\longrightarrow \bar{\sigma}_a = 2,600$ kg/cm²
ns	= 14		

$$\phi_0 = \frac{\bar{\sigma}_a}{n\,\bar{\sigma}'_b} = \frac{2,600}{14x130} = 1.43$$

For Maximum BM, M = 404,505 kgcm

$b = 20$
$h_t = 50\ ;\ d = 4 \longrightarrow h = h_t - d = 50 - 4 = 46$ cm

$$Ca = \frac{h}{\sqrt{\frac{nM}{b\sigma_a}}} = \frac{46}{\sqrt{\frac{14x404,505}{20x2,600}}} = 4.41$$

$\delta = 0.4$ (required of minimum compression reinforcement bar)
$\longrightarrow \phi = 1.546 > \phi_0 = 1.43$ (OK)
$\phi' = 2.103$
$n\omega = 0.0602$

. Stresses

$\bar{\sigma}_a = 2,600$ kg/cm²

$$\bar{\sigma}_b = \frac{\bar{\sigma}_a}{n\phi} = \frac{2,600}{14x1.546} = 120.13 \text{ kg/cm}^2 < \bar{\sigma}'_b = 130 \text{ kg/cm}^2 \text{ (OK)}$$

$$\sigma_a = \frac{\bar{\sigma}_a}{\phi'} = \frac{2,600}{2.103} = 1,236 \text{ kg/cm}^2 < \bar{\sigma}_a = 2,600 \text{ kg/cm}^2 \text{ (OK)}$$

. Reinforcement bar

$$A_{steel\ (tensile)} = \frac{\omega bh}{14} = 0.0043 \times 20 \times 46 = 4.01 \text{ cm}^2$$

$$A_{steel\ (compression)} = \delta \times A_{steel\ (tensile)}$$
$$= 0.4 \times 4.01 \text{ cm}^2 = 1.604 \text{ cm}^2$$

Used $A_{steel\ (tensile)}$ = 2 D 16 = 4.02 cm²(OK)

Used $A_{steel\ (compression)}$ = 2 D 16 = 4.02 cm²(OK)

- **Checking of Column reinforcement bar & stress**

On Column No. F25

Maximum Bending Moment	= 531,633 kgcm	
b (width)	= 30 cm	
h_t (height)	= 50 cm	
Concrete cover	= 4 cm	
$h = h_t - d$	= 50 - 4 = 46 cm	
Fc	= 225 kg/cm^2	$\longrightarrow \bar{\sigma}'_b$ = 130 kg/cm^2
Fu	= 3,200 kg/cm^2	$\longrightarrow \bar{\sigma}_a$ = 2,600 kg/cm^2
ns	= 14	

$$\phi_0 = \frac{\bar{\sigma}_a}{n\,\bar{\sigma}'_b} = \frac{2,600}{14x130} = 1.43$$

For Maximum BM M = 531,633 kgcm

$$Ca = \frac{h}{\sqrt{\frac{nM}{b\sigma_a}}} = \frac{46}{\sqrt{\frac{14x531,633}{30x2,600}}} = 4.71$$

$\delta = 1$ (for symetrical reinforcement)

$\longrightarrow \phi = 5.25 > \phi_0 = 1.43$ (OK)

$\phi' = 14.00$

$n\omega = 0.0164$

. Stresses

$\bar{\sigma}_a = 2,600$ kg/cm^2

$$\bar{\sigma}_b = \frac{\bar{\sigma}_a}{n\,\phi} = \frac{2,600}{14x5.25} = 35.37 \text{ kg/cm}^2 < \bar{\sigma}'_b = 130 \text{ kg/cm}^2$$

$$\sigma_a = \frac{\bar{\sigma}_a}{\phi'} = \frac{2,600}{14.00} = 185.71 \text{ kg/cm}^2 < \bar{\sigma}_a = 2,600 \text{ kg/cm}^2$$

. Reinforcement

$$A = \omega bh = \frac{0.0164}{14} x\ 30\ x\ 45 = 1.581 \text{ cm}^2$$

A_{steel} = 1.581cm^2 < 1 % x 1,500 cm^2 (sectional area of column)

Hence applied :

A_{steel} = 8 D 16

= 16.08 cm^2

$$= \frac{16.08}{30x50} x\ 100\ \%\ A_{concrete}$$

= 1.072 % $A_{concrete}$ (OK)

7. DESIGN OF FOOTING

All of footing Type - 1 design are represented by support reaction of joint no.7 of member F5 as the biggest of the frame element forces.

1. Soil stress :

- For loading Combination 1, the element forces are :

N = 30,335 kg
Mx = 61,017 kgcm
Mz = 77,101 kgcm
Shear x = 514 kg
Shear z = 405 kg
Try size of Footing = (1.20 x 1.20) m^2

- Soil stress beneath footing :

$$\sigma = \frac{N}{A} \pm \frac{M_X}{W_X} \pm \frac{M_Z}{W_Z}$$

$$\sigma_{max} = \frac{30{,}335}{120x120} + \frac{61{,}017}{\frac{1}{6}x120x120^2} + \frac{77{,}101}{\frac{1}{6}x120x120^2}$$

σ_{max} = 2.11 + 0.21 + 0.27 = 2.59 kg/cm² < σ_{all} = 20 kg/cm² (OK)
σ_{min} = 2.11 - 0.21 - 0.27 = 1.63 kg/cm²

- For loading Combination 2 any Earthquake Force the element forces are :

N = 24,453 kg
Mx = 123,385 kgcm
Mz = 449,966 kgcm
Shear x = 1,740 kg
Shear z = 623 kg
Try size of Footing = (1.20 x 1.20) m^2

- Soil stress beneath footing :

$$\sigma = \frac{N}{A} \pm \frac{M_X}{W_X} \pm \frac{M_Z}{W_Z}$$

$$\sigma_{max} = \frac{24{,}453}{120x120} + \frac{123{,}385}{\frac{1}{6}x120x120^2} + \frac{449{,}966}{\frac{1}{6}x120x120^2}$$

σ_{max} = 1.70 + 0.43 + 1.56 = 3.69 kg/cm² < 1.5x σ_{all}= 30 kg/cm² (OK)
σ_{min} = 1.70 - 0.43 - 1.56 = -0.29 kg/cm²

note :
σ_{all} = 20 kg/cm²
= Allowable soil compression stress was given JICA Study Team

1. Concrete reinforcement bar :

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

Mz = 449,966 kgcm

Concrete : fc = 225 kg/cm² ⟶ $\sigma'b$ = 130 kg/cm²

Steel Bar : fy = 3200 kg/cm² ⟶ $\sigma'a$ = 2,600 kg/cm²

ns = 14

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

Footing slab thick ht = 25 cm ; width b = 120 cm
Cocrete cover d = 5 cm
h = ht - d = 25 - 5 = 20 cm.

$$Ca = \frac{h}{\sqrt{\frac{nxM}{bx\sigma'_a}}} = \frac{20}{\sqrt{\frac{14x449,966}{120x2,600}}} = 4.45$$

for δ = 0 (single reinforcement bar)

ϕ = 2.534 > ϕ_0 = 1.43 (OK)

$n\omega$ = 0.056

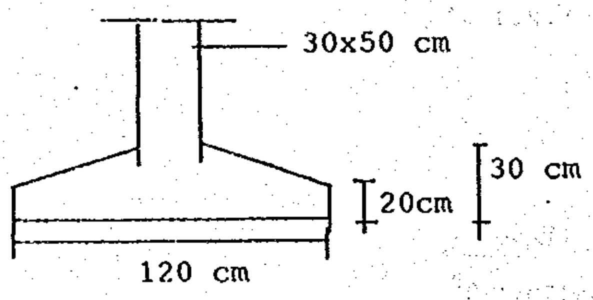
A = ωbh

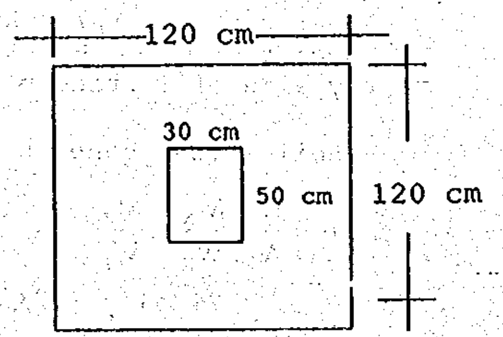
$$= \frac{0.056}{14} x 120 x 20 = 9,6 \text{ cm}^2$$

A_{stell} = D16 - 15 cm ≈ 7 x 2.01 = 14.07 cm² (OK)

Mx = 123,385 kgcm

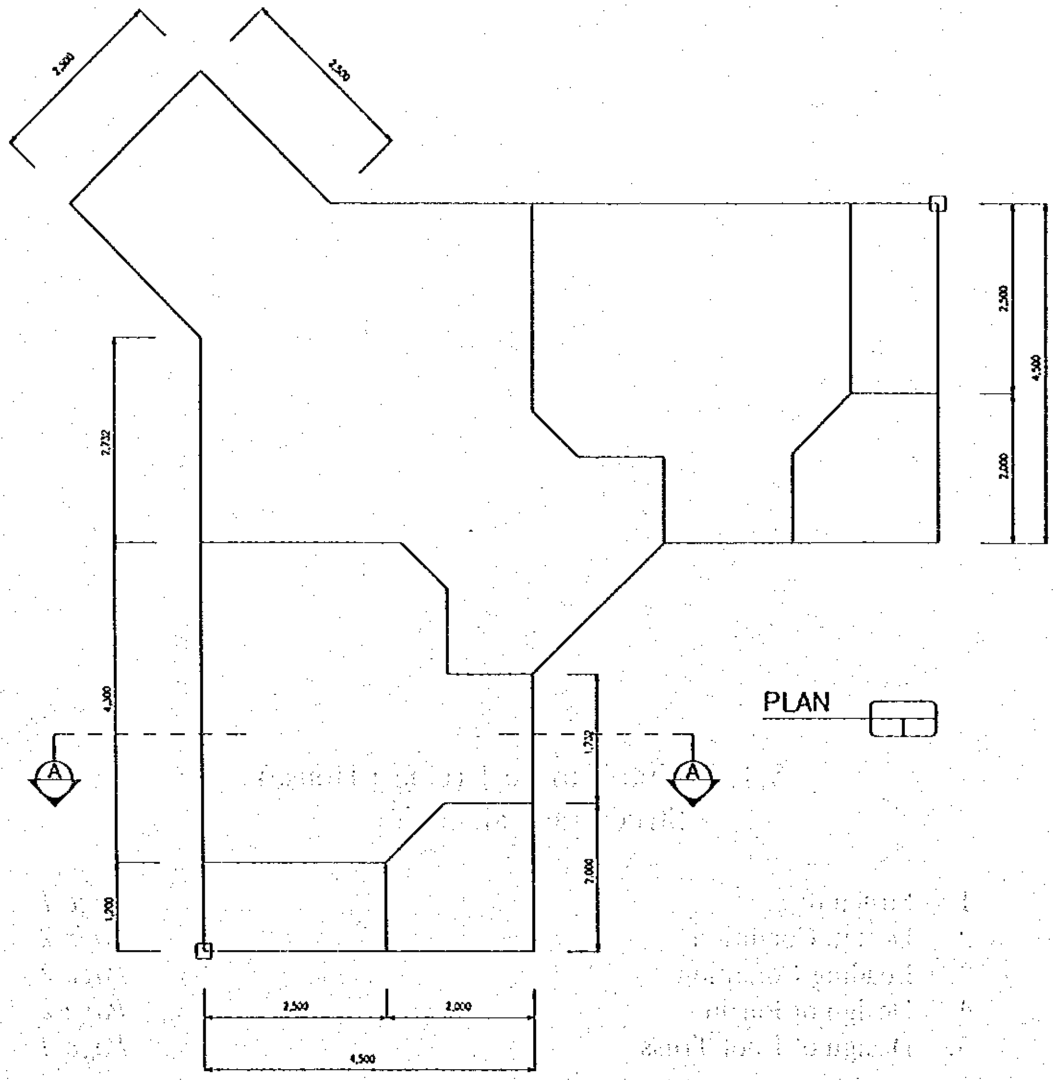
A_{stell} = D16 - 15 cm can be adopted



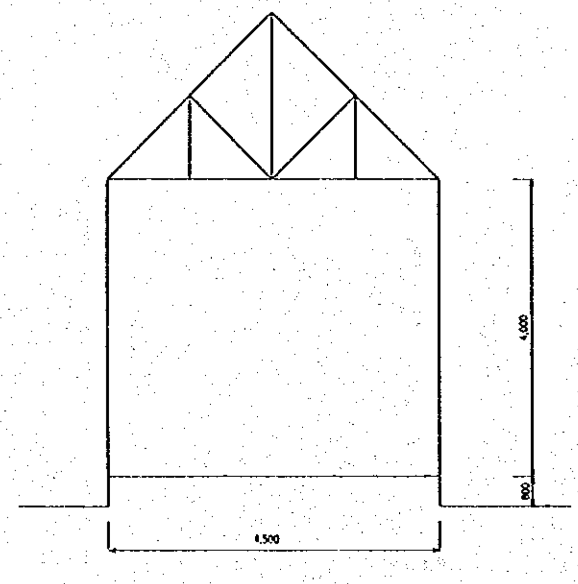


5.1.2 Staff House 1 (Guest House)
Structural Calculation

1. STRUCTURE



PLAN



SECTION A-A

GUEST HOUSE
JATIBARANG DAM MANAGEMENT COMPLEX

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

A. Roof Truss Type K-1

- Purlin distance (c/c) = 1.25 m
- Purlin span = 2.875 m
- Purlin self weight say = 15.00 kg/m'

$q_1 = 1.25 \times 80\ kg/m^2 \approx 100\ kg/m'$

q_2 (self weight) = 15 kg/m'

Q = 115 kg/m'

$$Q_1 = Q_2 = Q \cos 45^0 = 115 \cos 45^0 \approx 81.31\ kg/m'$$

- Live Load

$Px = Py = P \cos \alpha = 100 \cos 45^0 \approx 71\ kg$

- Bending moment

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

$Mx = 1/8 \times 115 \times 2.875^2 + 1/4 \times 71 \times 2.875 = 169.85\ kgm$

$My = Mx = 169.85\ kgm = 16{,}985\ kgcm$

- 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

$$\sigma = \sigma x + \sigma y = \frac{Mx}{Wx} + \frac{My}{Wy}$$
$$= \frac{16,985}{88.6} + \frac{16,985}{73.2} = 191.7 + 232.04$$
$$= 423.74 \text{ kg} < \sigma_{all} = 1,400 \text{ kg/cm}^2 \text{ (OK)}$$

- Deflection

$$fx = \frac{5}{384} \times Q_1 \times \frac{L^4}{EI_x} + \frac{1}{48}\frac{PL^3}{EI_x}$$
$$= \frac{5}{384} \times 1.15 \times \frac{287.5^4}{2.1x10^6 x664} + \frac{1}{48}\frac{71 \times 287.5^3}{2.1x10^6 x664}$$
$$= 0.07 + 0.00008 = 0.07 \text{ cm}$$

$$f = (0.07^2 + 0.07^2)^{1/2} = 0.1 \text{ cm}$$

$$f = 0.1 \text{ cm} < f_{all} = \frac{1}{360} L = \frac{287.5}{360} = 0.8 \text{ cm (OK)}$$

B. Roof Truss Type K-2

- Purlin distance (c/c) = 1.25 m
- Purlin span = 2.875 m
- Purlin self weight say = 15.00 kg/m'

q_1 = 1.25 x 80 kg/m² ≈ 100 kg/m'
q_2 (self weight) = 15 kg/m'
Q = 115 kg/m'

$Q_1 = Q_2 = Q \cos 45^o$
$= 115 \cos 45^o$
≈ 81.31 kg/m'

- Live Load

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

- Bending moment

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

$Mx = 1/8 \times 115 \times 2.875^2 + 1/4 \times 71 \times 2.875 = 169.85$ kgm

$My = Mx = 169.85$ kgm $= 16,985$ kgcm

- 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

$$\sigma = \sigma x + \sigma y = \frac{Mx}{Wx} + \frac{My}{Wy} = \frac{16,985}{88.6} + \frac{16,985}{73.2} = 191.7 + 232.04$$

$$= 423.74 \text{ kg} < \sigma_{all} = 1,400 \text{ kg/cm}^2 \text{ (OK)}$$

- Deflection

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

$$= \frac{5}{384} \times 1.15 \times \frac{287.5^4}{2.1x10^6x664} + \frac{1}{48}\frac{71 \times 287.5^3}{2.1x10^6x664}$$

$$= 0.07 + 0.00008 = 0.07 \text{ cm}$$

$$f = (0.07^2 + 0.07^2)^{1/2} = 0.1 \text{ cm}$$

$$f = 0.1 \text{ cm} < \bar{f}_{all} = \frac{1}{360} L = \frac{287.5}{360} = 0.8 \text{ cm (OK)}$$

C. Roof Truss Type K-3

- Purlin distance (c/c) = 1.41 m
- Purlin span = 2.50 m
- Purlin self weight say = 15.00 kg/m'

$q_1 = 1.41 \times 80\ kg/m^2 \approx 113\ kg/m'$
q_2 (self weight) $= 8\ kg/m'$

$Q = 121\ kg/m'$

$$Q_1 = Q_2 = Q \cos 45^o = 121 \cos 45^o \approx 86\ kg/m'$$

- Live Load
  $Px = Py = P \cos \alpha = 100 \cos 45^o \approx 71\ kg$

- Bending moment

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

  $Mx = 1/8 \times 86 \times 2.5^2 + 1/4 \times 71 \times 2.5 \approx 112\ kgm$

  $My = Mx = 112\ kgm = 11,200\ kgcm$

- Try Purlin of Lip Channel type :

  150 x 50 x 50 x 3.2
  $Ix = 438\ cm^4$; $Wx = 58.4\ cm^3$
  $Iy = 71.4\ cm^4$; $Wy = 13.2\ cm^3$

- Stresses

$$\sigma = \sigma x + \sigma y = \frac{Mx}{Wx} + \frac{My}{Wy} = \frac{11,200}{58.4} + \frac{11,200}{13.2} = 191.78 + 848.48$$

$$= 1,040.26\ kg < \sigma_{all} = 1,400\ kg/cm^2\ (OK)$$

- Deflection

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

$$= \frac{5}{384} \times 0.86 \times \frac{250^4}{2.1 x10^6 x438} + \frac{1}{48}\frac{71 \times 250^3}{2.1 x10^6 x438}$$

$$= 0.05 + 0.03 = 0.08\ cm$$

$$fy = 0.29 + 0.15 = 0.44\ cm$$

$$f = (0.08^2 + 0.44^2)^{1/2} = 0.45\ cm$$

$$f = 0.45\ cm < f_{all} = \frac{1}{360} L = \frac{250}{360} = 0.69\ cm\ (OK)$$

5. Design of Roof Truss

A. Roof Truss Type K-1

a. Dead load

- $P_1 = 2.875 \times (115 + 15) = 373.75$ kg

b. Wind load

- $W_1 = 2.875 \times 1.25 \times 20 = 71.88$ kg
- $W_2 = 2.875 \times 1.25 \times 16 = -57.50$ kg

$W_{1X} = W_{1Y} = 71.88 \text{ Cos } 45^o = 50.77$ kg
$W_{2X} = W_{2Y} = -57.50 \text{ Cos } 45^o = -40.66$ kg

c. Live load

- $P_2 = 100$ kg

B. Roof Truss Type K-2

a. Dead load

- $P_1 = 2.875 \times (115 + 15) = 373.75$ kg

b. Wind load

- $W_1 = 2.875 \times 1.25 \times 20 = 71.88$ kg
- $W_2 = 2.875 \times 1.25 \times 16 = -57.50$ kg

$W_{1X} = W_{1Y} = 71.88 \text{ Cos } 45^o = 50.77$ kg
$W_{2X} = W_{2Y} = -57.50 \text{ Cos } 45^o = -40.66$ kg

c. Live load

- $P_2 = 100$ kg

C. Roof Truss Type K-3

a. Dead load

- $P_1 = 2.50 \times 121 \approx 303$ kg

b. Wind load

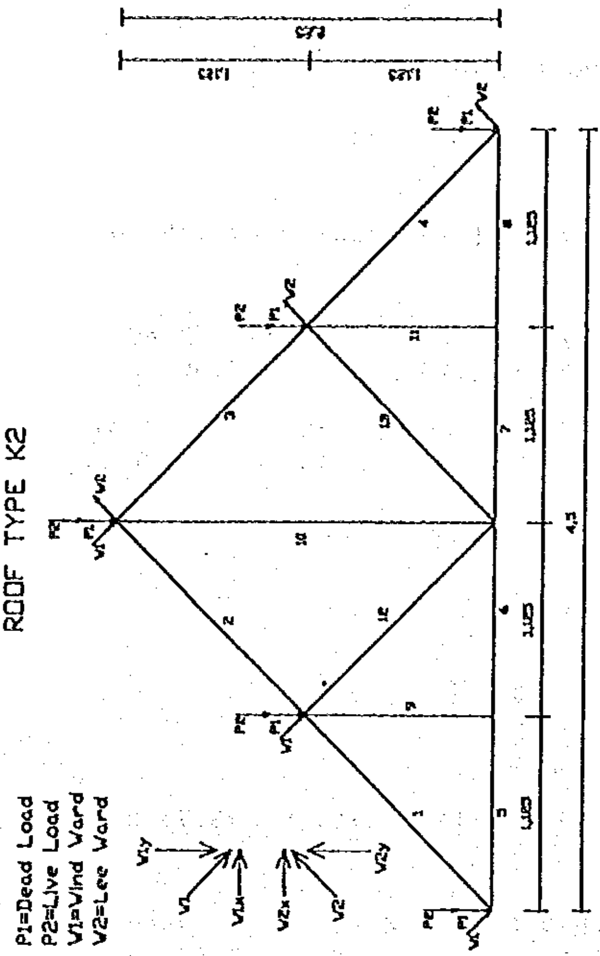
- $W_1 = 2.50 \times 1.41 \times 20 = 71.00$ kg
- $W_2 = 2.50 \times 1.41 \times 16 = -56.40$ kg

$W_{1X} = W_{1Y} = 71.00 \text{ Cos } 45^o = 50.20$ kg
$W_{2X} = W_{2Y} = -56.40 \text{ Cos } 45^o = -39.89$ kg

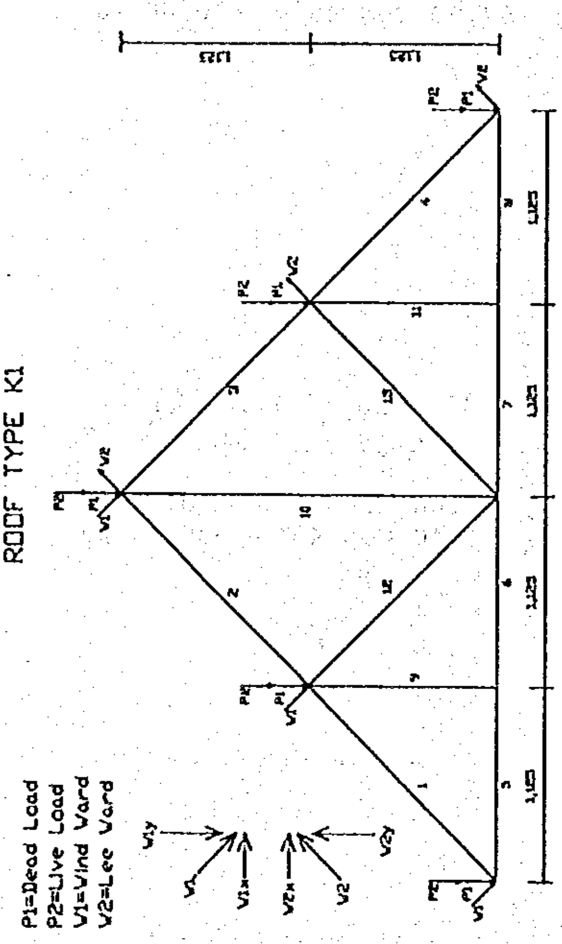
c. Live load

- $P_2 = 100$ kg

ROOF TYPE K2



ROOF TYPE K1



STEEL ROOF TRUSS QUEST HOUSE JATIBARANG TYPE K-1
PROTOTYPE

PROFILE	PLATE THICKNESS (cm)	Fy (kg/cm2)	Fu (kg/cm2)	DIA.BOLT
L 50.50.5	0.8	3,700	2,400	1.4

FRAME ELEMENT FORCE

Member	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n bolt	dia.bolt (mm)
1	998	4.24	0	167	2	1.4
2	649	4.24	0	192	2	1.4
3	853	4.24	0	192	2	1.4
4	1,200	4.24	0	167	2	1.4
5	1,011	4.24	0	119	2	1.4
6	1,011	4.24	0	119	2	1.4
7	901	4.24	0	119	2	1.4
8	901	4.24	0	119	2	1.4
9	17	0	0	0	2	1.4
10	678	0	0	0	2	1.4
11	17	0	0	0	2	1.4
12	533	4.24	0	167	2	1.4
13	378	4.24	0	167	2	1.4

STEEL ROOF TRUSS QUEST HOUSE JATIBARANG TYPE K-2
PROTOTYPE

PROFILE	PLATE THICKNESS (cm)	Fy (kg/cm2)	Fu (kg/cm2)	DIA.BOLT
L 50.50.5	0.8	3,700	2,400	1.4

FRAME ELEMENT FORCE

Member	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n bolt	dia.bolt (mm)
1	998	4.24	0	169	2	1.4
2	649	4.24	0	192	2	1.4
3	853	4.24	0	192	2	1.4
4	1,200	4.24	0	169	2	1.4
5	1,011	4.24	0	119	2	1.4
6	1,011	4.24	0	119	2	1.4
7	901	4.24	0	119	2	1.4
8	901	4.24	0	119	2	1.4
9	17	0	0	0	2	1.4
10	678	0	0	0	2	1.4
11	17	0	0	0	2	1.4
12	533	4.24	0	169	2	1.4
13	378	4.24	0	169	2	1.4

- **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 T4 (loading Combination 2)
Force $F = 1{,}200$ kg

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

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

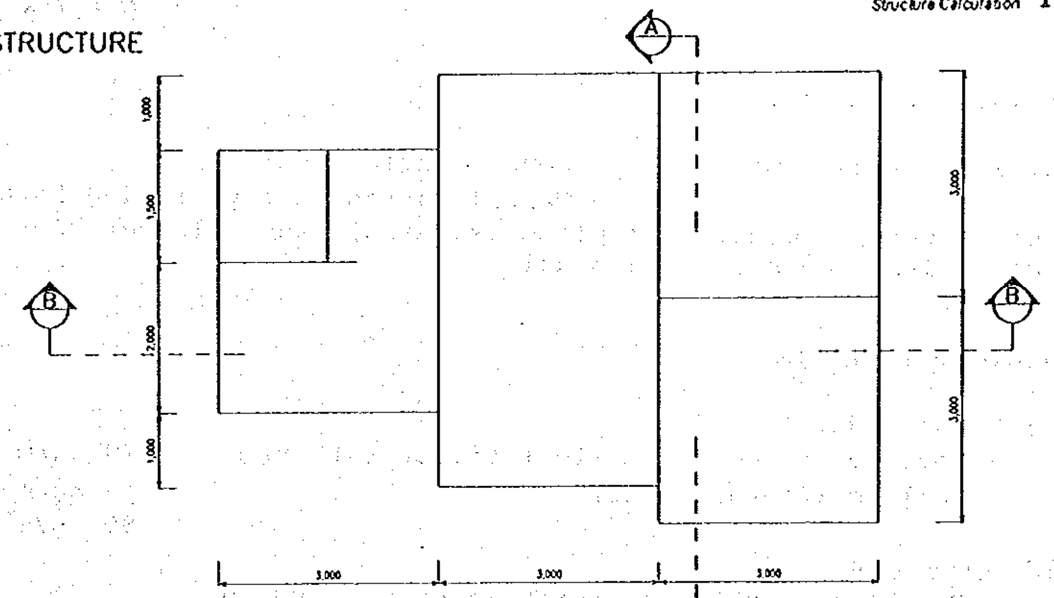
Stress

$$\sigma = \frac{F}{A} = \frac{1{,}200}{9.6} = 125\ kg/cm^2 < \sigma_{all} = 1{,}440\ kg/cm^2\ \text{(OK)}$$

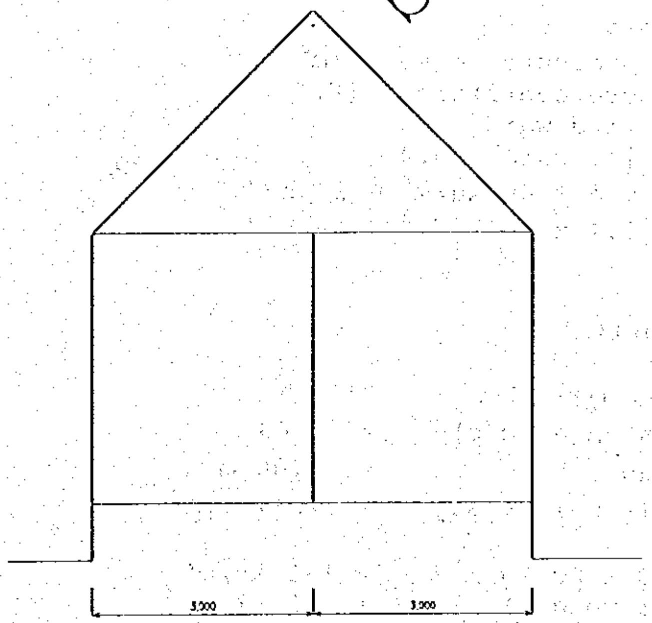
Hence double angle steel of 50.50.5 can be used as the members of roof truss type K - 1

5.1.3 Staff House 2
Structural Calculation

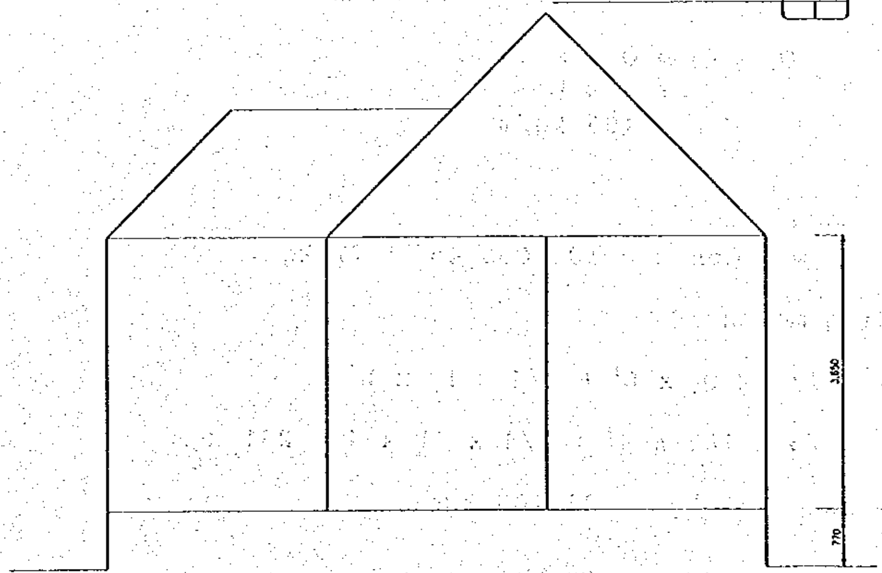
1. STRUCTURE



PLAN



SECTION A–A



SECTION B–B

STAFF HOUSE
JATIBARANG DAM MANAGEMENT COMPLEX

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 x 40 kg/m^2 = 20 kg/m^2

W2 = 0.4 x 40 kg/m^2 = 16 kg/m^2

4. Design of Purlin

A. Roof Truss Type K-1

- Purlin distance (c/c) = 1.63 m
- Purlin span = 4.00 m
- Purlin self weight say = 15.00 kg/m'

q_1 = 1.63 x 80 kg/m^2 ≈ 131 kg/m'

q_2 (self weight) = 15 kg/m'

Q = 146 kg/m'

$Q_1 = Q_2 = Q \cos 45^o$

$= 146 \cos 45^o$

≈ 105 kg/m'

- Live Load

Px = Py = P Cos ∝ = 100 Cos 45^o ≈ 71 kg

- Bending moment

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

$Mx = 1/8 \times 105 \times 4^2 + 1/4 \times 71 \times 4 = 281$ kgm

My = Mx = 281 kgm = 28,100 kgcm

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

  150 x 130 x 20 x 3.2
  $I_x = 664 cm^4$; $W_x = 88.6 cm^3$
  $I_y = 476 cm^4$; $W_y = 73.2 cm^3$

- Stresses

$$\sigma = \sigma_x + \sigma_y = \frac{M_x}{W_x} + \frac{M_y}{W_y} = \frac{28,100}{88.6} + \frac{28,100}{73.2} = 317.16 + 383.88$$

$$= 701.04 \text{ kg} < \sigma_{all} = 1,400 \text{ kg/cm}^2 \text{ (OK)}$$

- Deflection

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

$$= \frac{5}{384} \times 1.05 \times \frac{400^4}{2.1 \times 10^6 \times 664} + \frac{1}{48}\frac{71 \times 400^3}{2.1 \times 10^6 \times 664}$$

$$= 0.25 + 0.07 = 0.32 \text{ cm}$$

$$f_y = 0.35 + 0.09 = 0.44 \text{ cm}$$

$$f = (0.32^2 + 0.44^2)^{1/2} = 0.54 \text{ cm}$$

$$f = 0.54 \text{ cm} < f_{all} = \frac{1}{360} L = \frac{400}{360} = 1.11 \text{ cm (OK)}$$

B. Roof Truss Type K-2

- Purlin distance (c/c) = 1.41 m
- Purlin span = 2.50 m
- Purlin self weight say = 15.00 kg/m′

q_1 = 1.41 x 80 kg/m² ≈ 113 kg/m′
q_2 (self weight) = 8 kg/m′

Q = 121 kg/m′

$$Q_1 = Q_2 = Q \cos 45^\circ = 121 \cos 45^\circ \approx 86 \text{ kg/m}'$$

- Live Load

$$P_x = P_y = P \cos \alpha = 100 \cos 45^\circ \approx 71 \text{ kg}$$

- Bending moment

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

$$Mx = 1/8 \times 86 \times 2.5^2 + 1/4 \times 71 \times 2.5 \approx 112 \text{ kgm}$$

$$My = Mx = 112 \text{ kgm} = 11{,}200 \text{ kgcm}$$

- Try Purlin of Lip Channel type :

150 x 50 x 50 x 3.2
$Ix = 438 \text{ cm}^4$; $Wx = 58.4 \text{ cm}^3$
$Iy = 71.4 \text{ cm}^4$; $Wy = 13.2 \text{ cm}^3$

- Stresses

$$\sigma = \sigma x + \sigma y = \frac{Mx}{Wx} + \frac{My}{Wy} = \frac{11{,}200}{58.4} + \frac{11{,}200}{13.2} = 191.78 + 848.48$$

$$= 1{,}040.26 \text{ kg} < \sigma_{all} = 1{,}400 \text{ kg/cm}^2 \text{ (OK)}$$

- Deflection

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

$$= \frac{5}{384} \times 0.86 \times \frac{250^4}{2.1\text{x}10^6\text{x}438} + \frac{1}{48}\frac{71 \times 250^3}{2.1\text{x}10^6\text{x}438}$$

$$= 0.05 + 0.03 = 0.08 \text{ cm}$$

$$fy = 0.29 + 0.15 = 0.44 \text{ cm}$$

$$f = (0.08^2 + 0.44^2)^{1/2} = 0.45 \text{ cm}$$

$$f = 0.45 \text{ cm} < f_{all} = \frac{1}{360} L = \frac{250}{360} = 0.69 \text{ cm (OK)}$$

5. Design of Roof Truss

A. Roof Truss Type K-1

a. Dead load

- $P_1 = 4.00 \times (131 + 15) = 584$ kg

b. Wind load

- $W_1 = 4.00 \times 1.63 \times 20 = 130.40$ kg
- $W_2 = 4.00 \times 1.63 \times 16 = -104.32$ kg

$W_{1X} = W_{1Y} = 130.40 \text{ Cos } 45^o = 92.21$ kg
$W_{2X} = W_{2Y} = -104.32 \text{ Cos } 45^o = -73.77$ kg

c. Live load

- $P_2 = 100$ kg
-

B. Roof Truss Type K-2

a. Dead load

- $P_1 = 2.50 \times 121 \approx 303$ kg

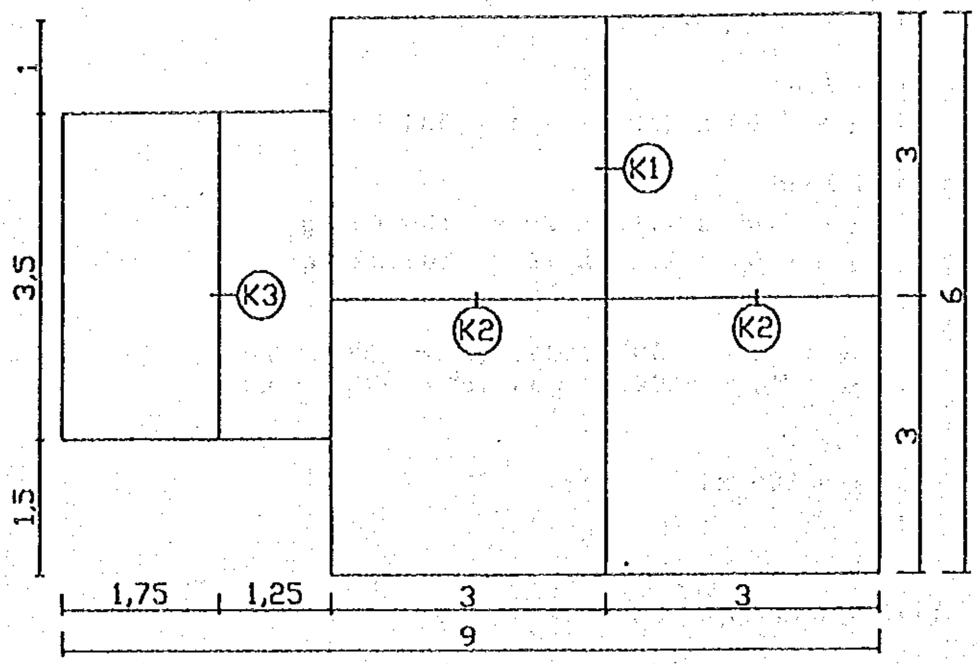
b. Wind load

- $W_1 = 2.50 \times 1.41 \times 20 = 71.00$ kg
- $W_2 = 2.50 \times 1.41 \times 16 = -56.40$ kg

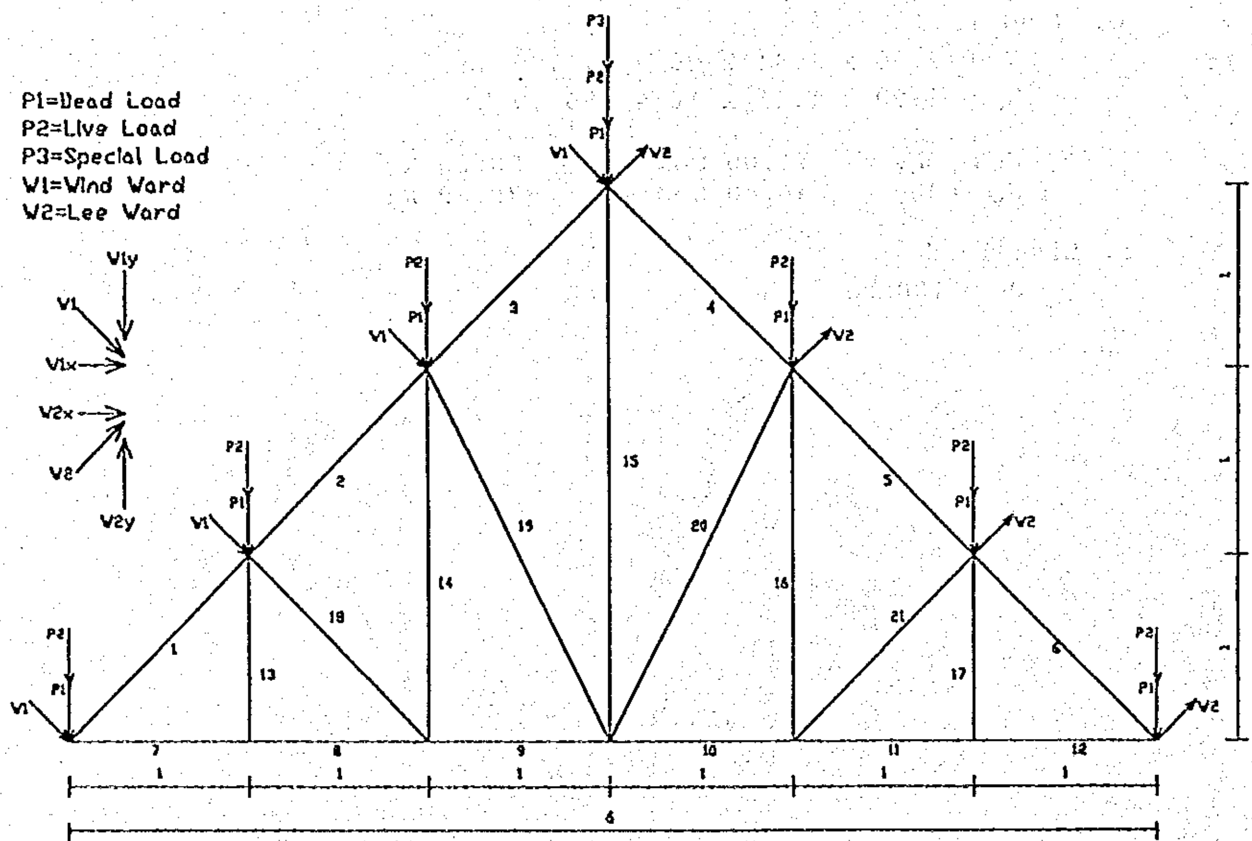
$W_{1X} = W_{1Y} = 71.00 \text{ Cos } 45^o = 50.20$ kg
$W_{2X} = W_{2Y} = -56.40 \text{ Cos } 45^o = -39.89$ kg

c. Live load

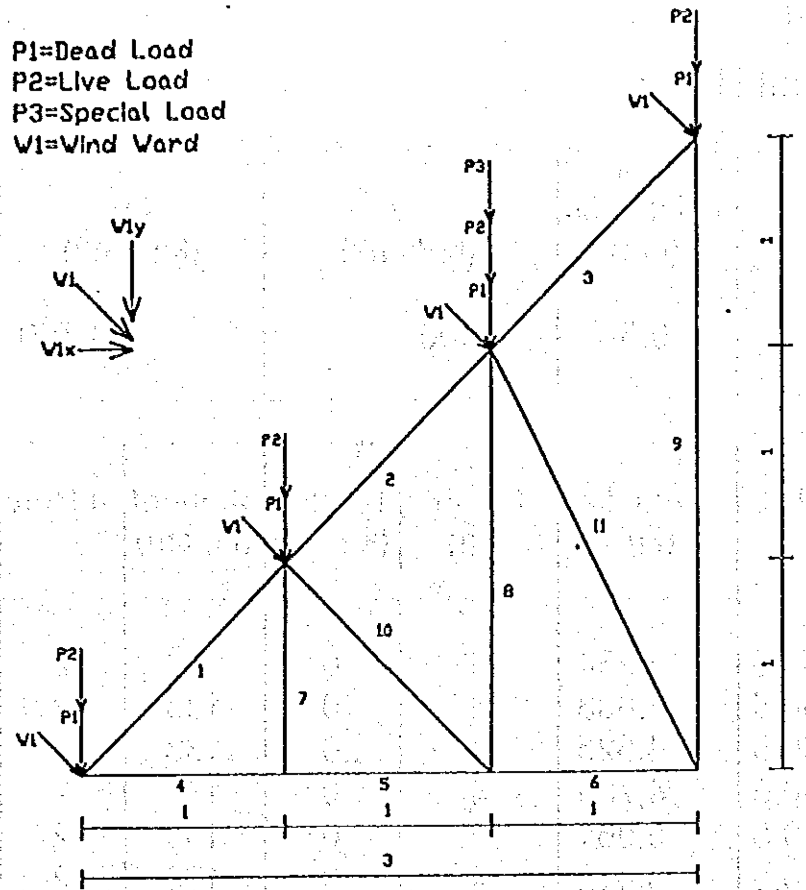
- $P_2 = 100$ kg


K1
K2
K2
K3
1
3,5
1,5
3
3
6
1,75
1,25
3
3
9

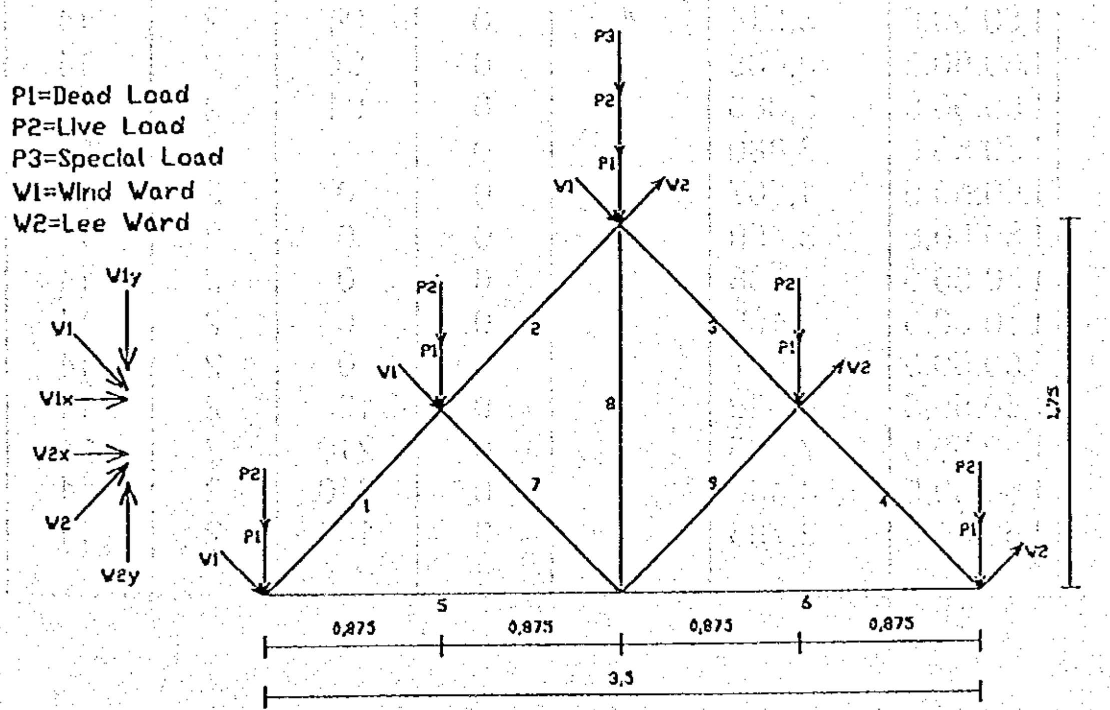
ROOF TYPE K1


P1=Dead Load
P2=Live Load
P3=Special Load
V1=Wind Ward
V2=Lee Ward
V1y
V1
V1x
V2x
V2
V2y
P3
P2
P1
V1
V2
6

ROOF TYPE K2



ROOF TYPE K3



ROOF K-1
Jatibarang Staff House

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	Bolt diameter (cm)
L50.50.5	0.5	2,400	3,700	1.4

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L50.50.5	5,234	4	0	133	2	14
2	L50.50.5	4,883	4	0	133	2	14
3	L50.50.5	4,525	4	0	133	2	14
4	L50.50.5	4,703	4	0	133	2	14
5	L50.50.5	5,061	4	0	133	2	14
6	L50.50.5	5,412	4	0	133	2	14
7	L50.50.5	4,132	4	0	94	2	14
8	L50.50.5	4,132	4	0	94	2	14
9	L50.50.5	3,814	4	0	94	2	14
10	L50.50.5	3,688	4	0	94	2	14
11	L50.50.5	3,880	4	0	94	2	14
12	L50.50.5	3,880	4	0	94	2	14
13	L50.50.5	1,507	4	0	0	2	14
14	L50.50.5	3,459	0	0	0	2	14
15	L50.50.5	1,388	0	0	0	2	14
16	L50.50.5	2.199	0	0	0	2	14
17	L50.50.5	1,507	0	0	0	2	14
18	L50.50.5	4,536	4	0	133	2	14
19	L50.50.5	7,305	4	0	210	2	14
20	L50.50.5	4,488	4	0	210	2	14
21	L50.50.5	2,783	4	0	133	2	14

ROOF K-2

Jatibarang Staff House

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	Bolt diameter (cm)
L50.50.5	0.5	2,400	3,700	1.4

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L50.50.5	424	4	0	133	2	14
2	L50.50.5	73	4	0	133	2	14
3	L50.50.5	376	4	0	133	2	14
4	L50.50.5	227	4	0	94	2	14
5	L50.50.5	227	4	0	94	2	14
6	L50.50.5	90	4	0	94	2	14
7	L50.50.5	16	0	0	0	2	14
8	L50.50.5	345	0	0	0	2	14
9	L50.50.5	453	4	0	133	2	14
10	L50.50.5	1,338	4	0	210	2	14

ROOF K-3
Jatibarang Staff House

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	Bolt diameter (cm)
L50.50.5	0.5	2,400	3,700	1.4

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L50.50.5	1,533	3	0	102	2	14
2	L50.50.5	1,194	3	0	103	2	14
3	L50.50.5	1,373	3	0	103	2	14
4	L50.50.5	1,711	3	0	103	2	14
5	L50.50.5	1,390	7	0	288	2	14
6	L50.50.5	1,264	7	0	288	2	14
7	L50.50.5	440	3	0	101	2	14
8	L50.50.5	516	0	0	0	2	14
9	L50.50.5	262	3	0	101	2	14

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

  Due to Tensile force

  Maximum force on member T10 (loading Combination 2)
  Force $F = 3{,}688$ kg

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

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

  Stress
  $\sigma = \frac{F}{A} = \frac{3{,}688}{9.6} = 384.17\ kg/cm^2 < \sigma_{all} = 1{,}440\ kg/cm^2$ (OK)

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

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

  Due to Tensile force

  Maximum force on member T10 (loading Combination 2)
  Force $F = 1{,}338$ kg

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

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

  Stress
  $\sigma = \frac{F}{A} = \frac{1{,}338}{9.6} = 139.38\ kg/cm^2 < \sigma_{all} = 1{,}440\ kg/cm^2$ (OK)

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

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

Due to Tensile force

Maximum force on member T4 (loading Combination 2)
Force $F = 1{,}711$ kg

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

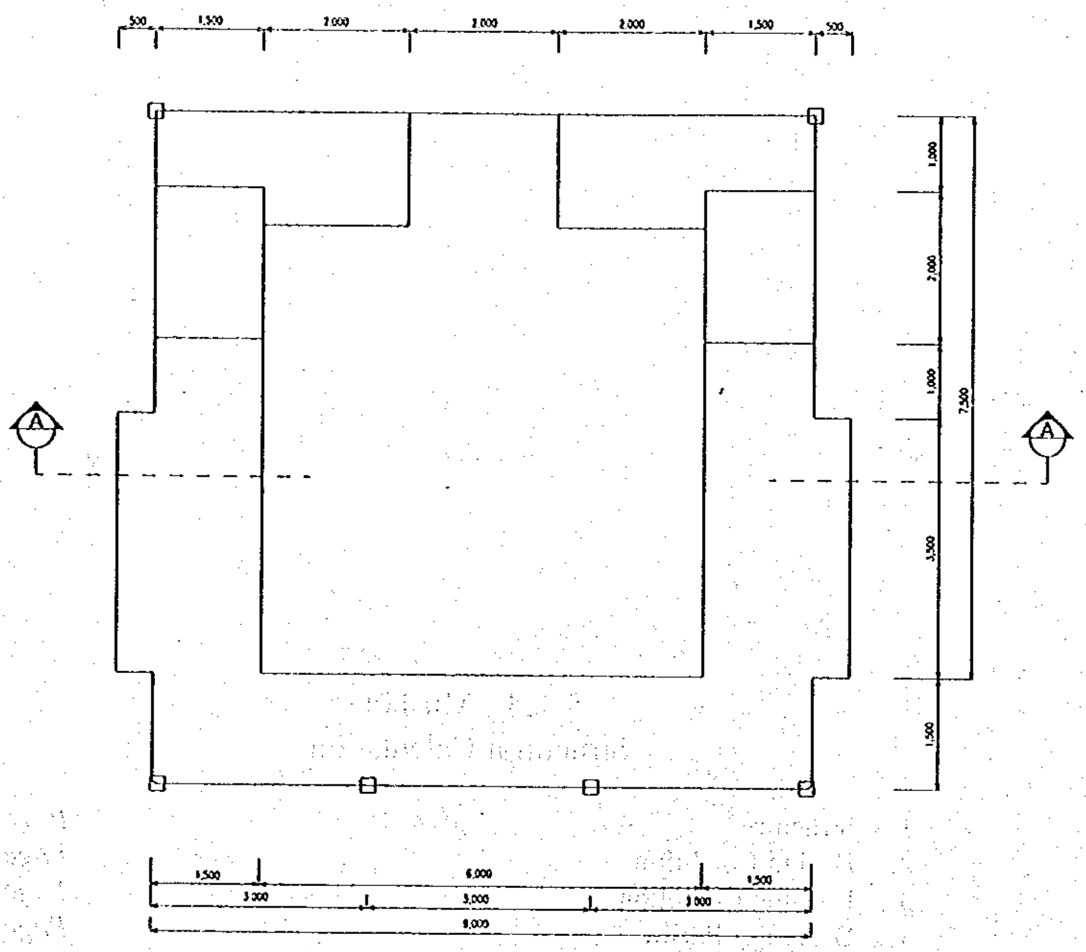
$\sigma_{all} = 0.6xFy$
$= 0.6x2{,}400 = 1{,}440\ kg/cm^2$

Stress
$\sigma = \dfrac{F}{A} = \dfrac{1{,}711}{9.6} = 178.23\ kg/cm^2 < \sigma_{all} = 1{,}440\ kg/cm^2$ (OK)

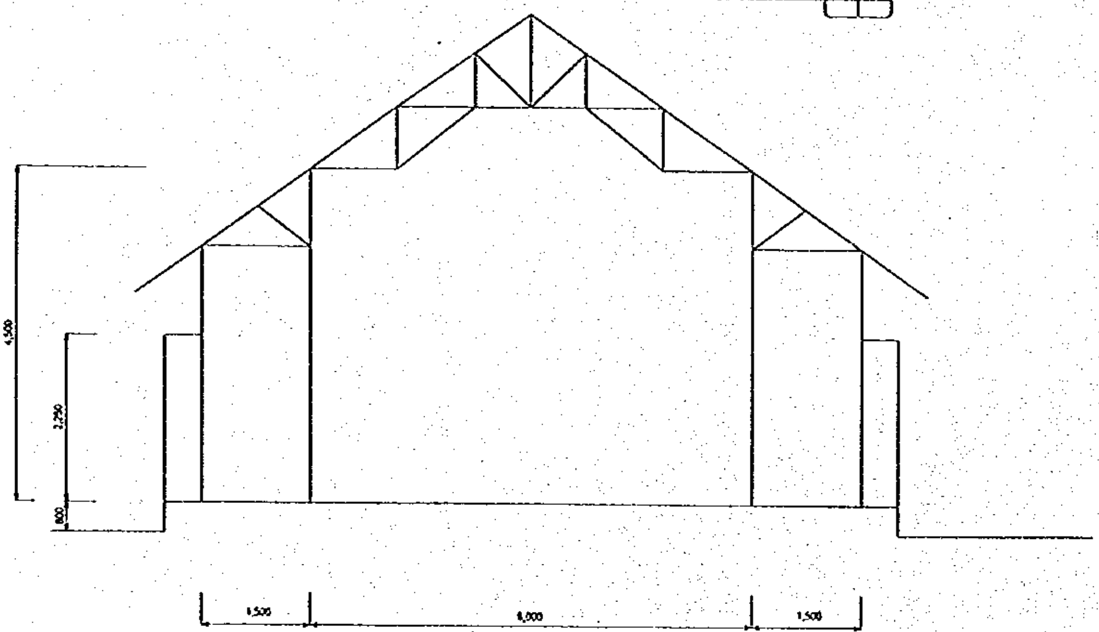
Hence double angle steel of 50.50.5 can be used as the members of roof truss type K - 3

5.1.4 Mushola
Structural Calculation

1. STRUCTURE



PLAN



SECTION A-A

MUSHOLA
JATIBARANG DAM MANAGEMENT COMPLEX

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 x 40 kg/m^2 = 20 kg/m^2

W2 = 0.4 x 40 kg/m^2 = 16 kg/m^2

4. Design of Purlin

A. Roof Truss Type K-1
- Purlin distance (c/c) = 1.35 m
- Purlin span = 3.00 m
- Purlin self weight say = 15.00 kg/m'

q_1 = 1.35 x 80 kg/m^2 ≈ 108 kg/m'

q_2 (self weight) = 15 kg/m'

Q = 123 kg/m'

$Q_2 = Q \cos 35^o$
$= 123 \cos 35^o$
$= 100.75$
≈ 101 kg/m'

- Live Load

$Py = P \cos \alpha = 100 \cos 35^o = 81.9 \approx 82$ kg

- Bending moment

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

$Mx = 1/8 \times 101 \times 3^2 + 1/4 \times 82 \times 3 = 175.125$ kgm

$Mx = 175.125$ kgm = 17,512.5 kgcm

- 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

$$\sigma = \sigma x + \sigma y$$
$$= \frac{Mx}{Wx} + \frac{My}{Wy}$$
$$= \frac{17,512.5}{88.6} + \frac{17,512.5}{73.2} = 197.658 + 239.241$$
$$= 436.89 \text{ kg} < \sigma_{all} = 1,400 \text{ kg/cm}^2 \text{ (OK)}$$

- Deflection

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

$$= \frac{5}{384} \times 1.01 \times \frac{300^4}{2.1 \times 10^6 \times 664} + \frac{1 \times 82 \times 300^3}{48 \ 2.1 \times 10^6 \times 664}$$

$$= 0.076 + 0.00011 = 0.0765 \text{ cm}$$

$$fy = 0.0765 \text{ cm}$$

$$f = (0.0765^2 + 0.0765^2)^{1/2} = 0.1082 \text{ cm}$$

$$f = 0.1082 \text{ cm} \leq f_{all} = \frac{1}{360} L = \frac{400}{360} = 1.11 \text{ cm (OK)}$$

B. Roof Truss Type K-2 (1/2 K1)

- Purlin distance (c/c) = 1.35 m
- Purlin span = 3.00 m
- Purlin self weight say = 15.00 kg/m'

$q_1 = 1.35 \times 80 \text{ kg/m}^2 \approx 108$ kg/m'
q_2 (self weight) = 15 kg/m'

Q = 123 kg/m'

$$Q_2 = Q \cos 35^o$$
$$= 123 \cos 35^o$$
$$= 100.75$$
$$\approx 101 \text{ kg/m'}$$

- Live Load

  $Py = P \cos \alpha = 100 \cos 35^0 = 81.9 \approx 82$ kg

- Bending moment

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

  $Mx = 1/8 \times 101 \times 3^2 + 1/4 \times 82 \times 3 = 175.125$ kgm

  $Mx = 175.125$ kgm $= 17{,}512.5$ kgcm

- 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

$$\sigma = \sigma x + \sigma y$$

$$= \frac{Mx}{Wx} + \frac{My}{Wy}$$

$$= \frac{17{,}512.5}{88.6} + \frac{17{,}512.5}{73.2} = 197.658 + 239.241$$

$$= 436.89 \text{ kg} < \sigma_{all} = 1{,}400 \text{ kg/cm}^2 \text{ (OK)}$$

- Deflection

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

$$= \frac{5}{384} \times 1.01 \times \frac{300^4}{2.1x10^6x664} + \frac{1 \times 82 \times 300^3}{48\ 2.1x10^6x664}$$

$$= 0.076 + 0.00011 = 0.0765 \text{ cm}$$

$$fy = 0.0765 \text{ cm}$$

$$f = (0.0765^2 + 0.0765^2)^{1/2} = 0.1082 \text{ cm}$$

$$f = 0.1082 \text{ cm} < f_{all} = \frac{1}{360} L = \frac{400}{360} = 1.11 \text{ cm (OK)}$$

5. Design of Roof Truss

A. Roof Truss Type K-1

a. Dead load
- $P_1 = 3.00 \times (108 + 15) = 369$ kg

b. Wind load
- $W_1 = 3.00 \times 1.35 \times 20 = 81.0$ kg
- $W_2 = 3.00 \times 1.35 \times 16 = -64.8$ kg

$W_{1X} = 81.0 \text{ Cos } 35^o = 66.3$ kg
$W_{1y} = 81.0 \text{ Sin } 35^o = 46.0$ kg
$W_{2X} = -64.8 \text{ Cos } 35^o = -53.0$ kg
$W_{2y} = -64.8 \text{ Sin } 35^o = -37.0$ kg

c. Live load
- $P_2 = 100$ kg

B. Roof Truss Type K-2 (1/2 K1)

a. Dead load
- $P_1 = 3.00 \times (108 + 15) = 369$ kg

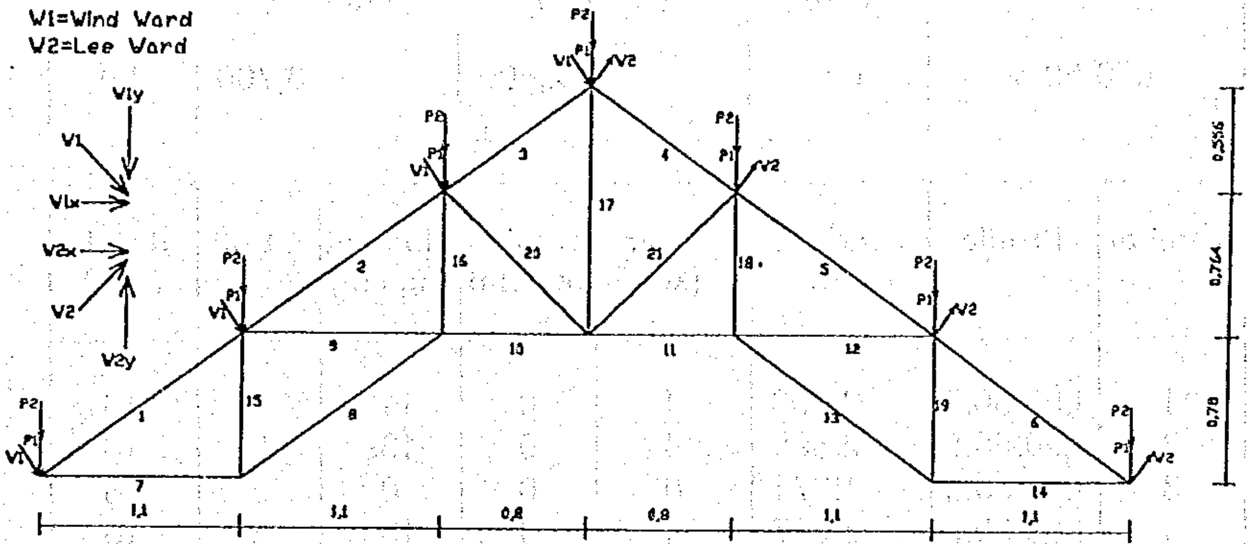
b. Wind load
- $W_1 = 3.00 \times 1.35 \times 20 = 81.0$ kg

$W_{1X} = 81.0 \text{ Cos } 35^o = 66.3$ kg
$W_{1y} = 81.0 \text{ Sin } 35^o = 46.0$ kg

c. Live load
- $P_2 = 100$ kg

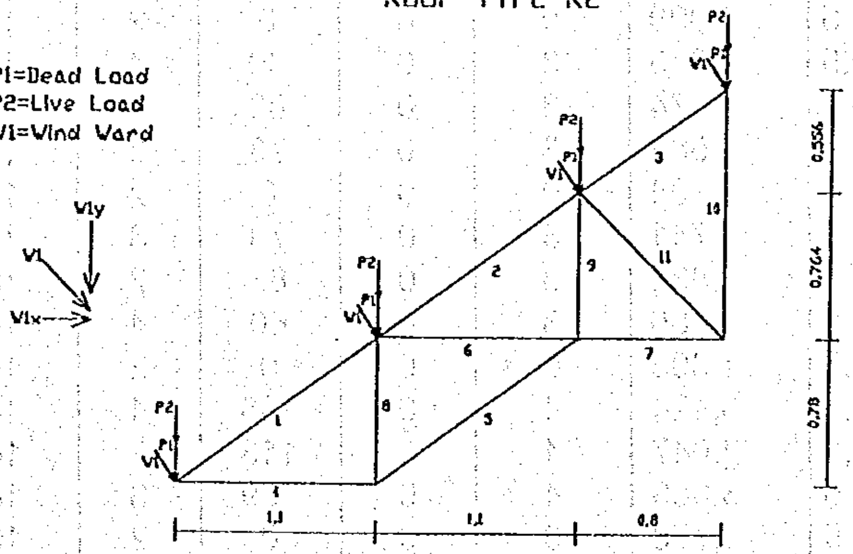
P1=Dead Load
P2=Live Load
W1=Wind Ward
V2=Lee Ward

ROOF TYPE K1



ROOF TYPE K2

P1=Dead Load
P2=Live Load
W1=Wind Ward



ROOF K-1
Jatibarang Mushola

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	Bolt diameter (cm)
L50.50.5	0.5	2,400	3,700	1.4

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L50.50.5	2,793	4	0	113	2	16
2	L50.50.5	2,895	4	0	139	2	16
3	L50.50.5	1,971	0	0	0	2	16
4	L50.50.5	3,227	4	0	139	3	16
5	L50.50.5	2,019	4	0	113	2	16
6	L50.50.5	5,419	4	0	138	4	16
7	L50.50.5	1,999	0	0	0	2	16
8	L50.50.5	4,813	3	0	30	4	16
9	L50.50.5	2,174	3	0	83	2	16
10	L50.50.5	3,588	3	0	73	3	16
11	L50.50.5	2,794	0	0	0	2	16
12	L50.50.5	3,741	3	0	73	3	16
13	L50.50.5	1,836	3	0	83	2	16
14	L50.50.5	1,585	3	0	60	4	16
15	L50.50.5	1,803	0	0	0	2	16
16	L50.50.5	5,424	4	0	138	4	16
17	L50.50.5	2,047	4	0	113	2	16
18	L50.50.5	3,089	4	0	139	3	16
19	L50.50.5	1,778	0	0	0	2	16
20	L50.50.5	2,518	4	0	113	2	16
21	L50.50.5	3,021	4	0	139	2	16

ROOF K-2
Jatibarang Mushola

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	Bolt diameter (cm)
L50.50.5	0.5	2,400	3,700	1.4

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1	L50.50.5	781	4	0	133	2	14
2	L50.50.5	703	4	0	139	2	14
3	L50.50.5	544	44,444	0	0	2	14
4	L50.50.5	980	4	0	139	2	14
5	L50.50.5	729	4	0	113	2	14
6	L50.50.5	694	4	0	138	2	14
7	L50.50.5	572	0	0	0	2	14
8	L50.50.5	708	3	0	60	2	14
9	L50.50.5	9,982	3	0	83	2	14
10	L50.50.5	873	3	0	73	2	14

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

Due to Tensile force

Maximum force on member T16 (loading Combination 2)
Force $F = 5,424.000$ kg

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

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

Stress

$$\sigma = \frac{F}{A} = \frac{5,424}{9.6} = 565\ kg/cm^2 < \sigma_{ijin} = 1,440\ kg/cm^2\ (OK)$$

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

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

Due to Tensile force

Maximum force on member T9 (loading Combination 2)
Force $F = 9,982$ kg

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

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

Stress

$$\sigma = \frac{F}{A} = \frac{9,983}{9.6} = 1,040\ kg/cm^2 < \sigma_{ijin} = 1,440\ kg/cm^2\ (OK)$$

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