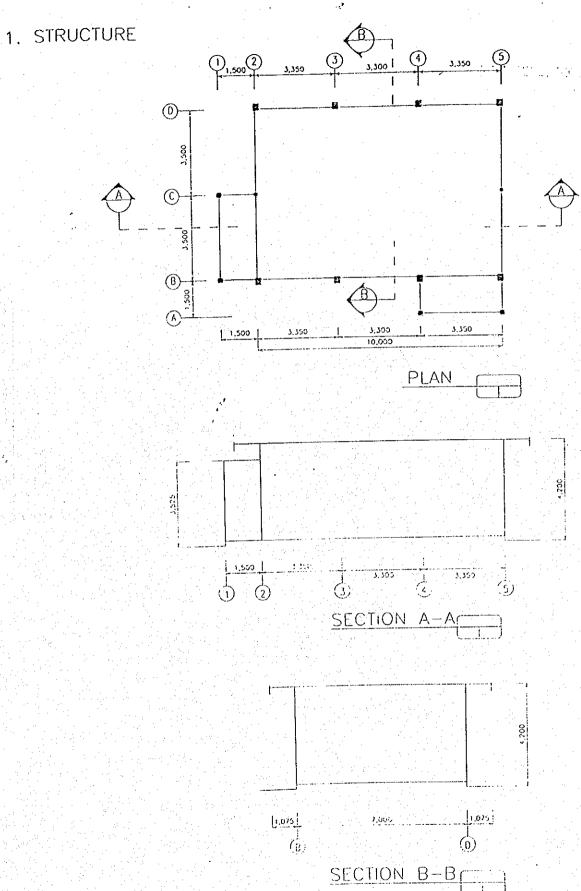
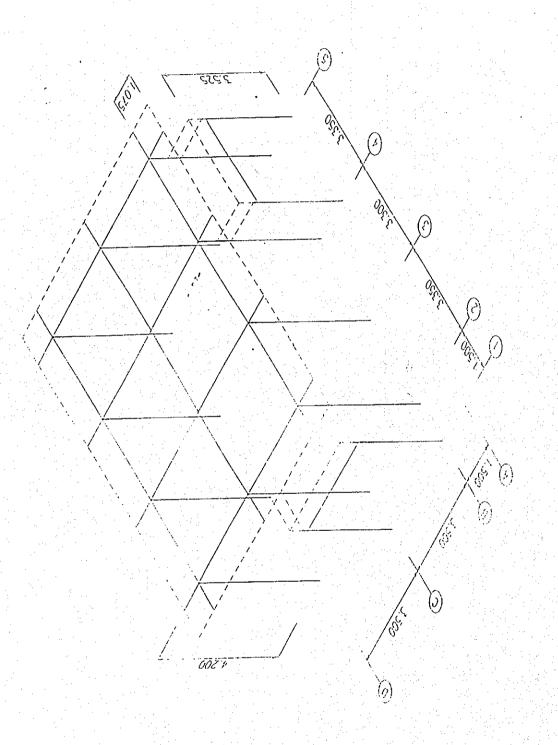
4.7.3. STORAGE HOUSE 1 STRUCTURE CALCULATION

- 1 STRUCTURE
- 2 DESIGN CONDITION
- 3 LOADING CONDITION
- 4 DESIGN OF FOOTING



STORAGE I SIMONGAN WIER MANAGEMENT COMPLEX



ISOMETRY
S T O R A G E 1
SINONGAN WEIR MANAGEMENT

2. Design Condition

a. Dimensions:

- width : 7.50 m - length : 10.00 m - height : 4.10 m

,b. Frame member

- Concrete, Fc = 225 kg/cm^2

- c. Structural model:
 - Space frame
 - Linear clastic
- d. Analysis method:

- Static

- 3. Loading condition
 - a. Dead load

- Roof cover (concrete slab 12 cm) = 288 kg/m^2 - Water stop (concrete 3 cm) = $\frac{72 \text{ kg/m}^2}{360 \text{ kg/m}^2}$

b. Live load

Weight of workers as point load = 100 kg

BEAN TYPE

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Bettom

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Stimp) (mm)

COULOM type 1

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COULOM type 2

Prototype			dia.	dia.	(c	fy	f∨	
b	h (m)	cover (cm)	main bar (cm)	stirrup (cm)	(kg/cm2)	(kg/cm2)	(kg/cm2)	
(cm)	(cm) 20		1.6	0.3	187	3,200	2,400	
20	2.0							į

		Earaa				Dasign		<u> </u>
Member	Frame ele Axial (kg)	men Force Moment-2 (kg.cm)	Moment-3 (kg.cm)	Main bar (mm)	Stirrup (mm)	Pu (kạ)	lylox (kg.cm)	Moy (kg.cm)
36 37	1,391 1,891	4 3	3,860 21,825	1 .	o3-70 o8-70	1,391 1,391	138,131 131,931	139,131 131,931

Checking of Beam reinforcement bar & stress

On Beam No. F12

Positive Bending Moment = 682,910 kgcm
b (width) = 25 cm
h_t (height) = 45 cm
Concrete cover = 4 cm
FC = 225 kg/cm²
$$\rightarrow$$
 $\sigma'_{b} = 130 kg/cm2$
Fu = 3,200 kg/cm² \rightarrow $\sigma_{a} = 2,600 kg/cm2$
ns $\phi_{0} = \frac{\sigma_{s}}{n} = \frac{2,600}{14 \times 130} = 1.43$

a) For Positive BM, M = 962,320 kgcm

 $\delta = 0.6$ (required of minimum compression reinforceme) 0.6 = 0.6 = 0.6 0.6 = 0.6 = 0.43

. Stresses

$$\frac{\overline{\sigma}_{a}}{\overline{\sigma}_{b}} = \frac{2,600 \text{ kg/cm}^{2}}{2,600} = 95.55 \text{ kg/cm}^{2} < \overline{\sigma}_{b}' = 130 \text{ kg/cm}^{2} \text{ (OK)}$$

$$\frac{\overline{\sigma}_{a}}{\overline{\sigma}_{b}} = \frac{2,600}{14 \times 1.985} = 95.55 \text{ kg/cm}^{2} < \overline{\sigma}_{b}' = 130 \text{ kg/cm}^{2} \text{ (OK)}$$

$$\sigma_{a} = \frac{\overline{\sigma_{a}}}{\phi'} = \frac{2,600}{2.83} = 918.72 \text{ kg/cm}^2 < \sigma_{a} = 2,600 \text{ kg/cm}^2 \text{ (OK)}$$

. Reinforcement bar

hereforement bar
$$= \frac{\omega bh}{14} = 0.1071 \times .25 \times 41 = 7.84 \text{ cm}^2$$

A steel (compression) = $\delta \times A$ steel (tensile) = $0.6 \times 7.84 \text{ cm}^2 = 4.704 \text{ cm}^2$

· Checking of Column reinforcement bar & stress

Random sampling On Column No. F32

Positive Bending Moment = 264,916 kgcm
b (width) = 25 cm
h_t (height) = 30 cm
Concrete cover = 4 cm

$$= 30 - 4 = 26$$
 cm
 $= 30 - 4 = 26$ cm
 $= 225$ kg/cm² $\longrightarrow \overline{\sigma}'_{b} = 130$ kg/cm²
FC = 3,200 kg/cm² $\longrightarrow \overline{\sigma}_{a} = 2,600$ kg/cm²
 $= 14$
 $= 30 - 4 = 26$ cm
 $= 225$ kg/cm² $\longrightarrow \overline{\sigma}'_{b} = 130$ kg/cm²
 $= 3,200$ kg/cm² $\longrightarrow \overline{\sigma}_{a} = 2,600$ kg/cm²
 $= 14$

a) For Positive BM M = 726,866 kgcm

$$Ca = \frac{h}{\sqrt{\frac{nM}{b\sigma_*}}} = \frac{26}{\sqrt{\frac{14 \times 264,916}{25 \times 2600}}}$$

$$\delta = 1 \text{ (for symetrical reinforcement)}$$

$$\phi' = 3.390$$
 $\phi' = 0.09492$

Stresses

$$\frac{\overline{\sigma_{a}}}{\overline{\sigma_{b}}} = \frac{2,600 \text{ kg/cm}^{2}}{\frac{14 \times 2.279}{1600}} = \frac{2,600}{\frac{14 \times 2.279$$

. Reinforcement

A =
$$\omega$$
bh = $\frac{0.09492}{14}$ × 25 × 26 = 4.4 cm²

DESIGN OF FOOTING

All of footing design are represented by support reaction of joint no.183 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 :

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

$$\sigma \max = 9.916 \times 10^{3} \div 2.0923 \times 10^{4} \div 4.997 \times 10^{4}$$

$$(150)^{2} = 1/6 \times 150 \times 150^{2} + 1/6 \times 150 \times 150^{2}$$

$$= 0.44 \div 0.04 \div 0.09$$

$$= 0.57 \times g/cm^{2} < \sigma/all = 1,0 \times g/cm^{2} \text{ (ok)}$$

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

= 0.31 kg/cm²

When earthquake occur (loading Combination 3., Support reaction of joint 3

```
## 9.533 K4 kg

## 6.3578 M4 kgom

## 19.1868 E4 kgom

## 5hear # # 477 kg

Shear p = 1,1121 kg
```

le boul stress beneth faming to

σ + 1 sg/cm² () . The second of the secon

All of forting concrete reintorcement is calculated by The settled a Inconesiat Code).

$$\phi_0 = \frac{\sigma_E}{\sigma_{ED}} = \frac{2,600}{130 \times 14} = 1.43$$

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

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

for
$$-\delta = 1$$

for
$$0 = 1$$

 $\phi = 8.091 > \phi_0 = 1.43$ (ok)

$$\phi' = 89$$
; 100n $\omega = 0.69$

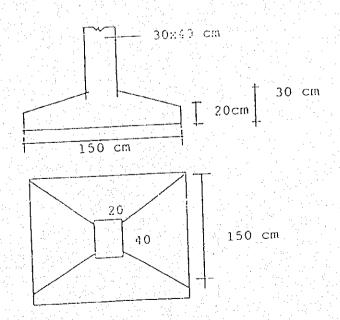
$$A = \omega bn$$

= $\frac{0.69 \times 150 \times 20}{300} = 20,7 \text{ cm}^2$

$$\frac{0.69}{100}$$
Astell = D16 - 15 cm(two way) \approx 11 \times 2.01 = 22.12 cm² (ok)

Mx = 2.0993E4 kgcm

Astell = Di6 - 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)

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

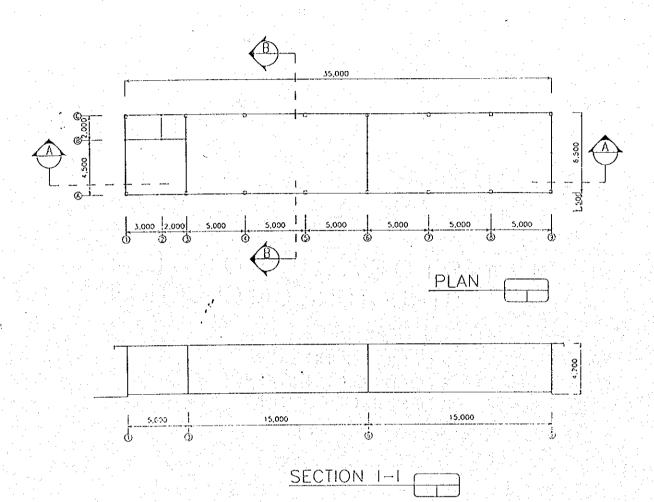
- Soil stress beneth foundation

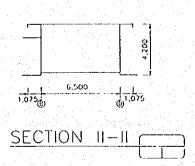
 $\sigma \max = 1.7025 \times 10^4 + 4.3720 \times 10^4 + 1.09209 \times 10^5 +$ $100x(750-750) 1/6x600x100^2 1/6x100x600^2$ 100x100 = 0.28 + 0.04 + 0.02= $0.34 \text{ kg/cm}^2 < \sigma \text{ all} = 1.0 \text{ kg/cm}^2 \text{ (ok)}$

4.7.4. STORAGE HOUSE 2 STRUCTURE CALCULATION

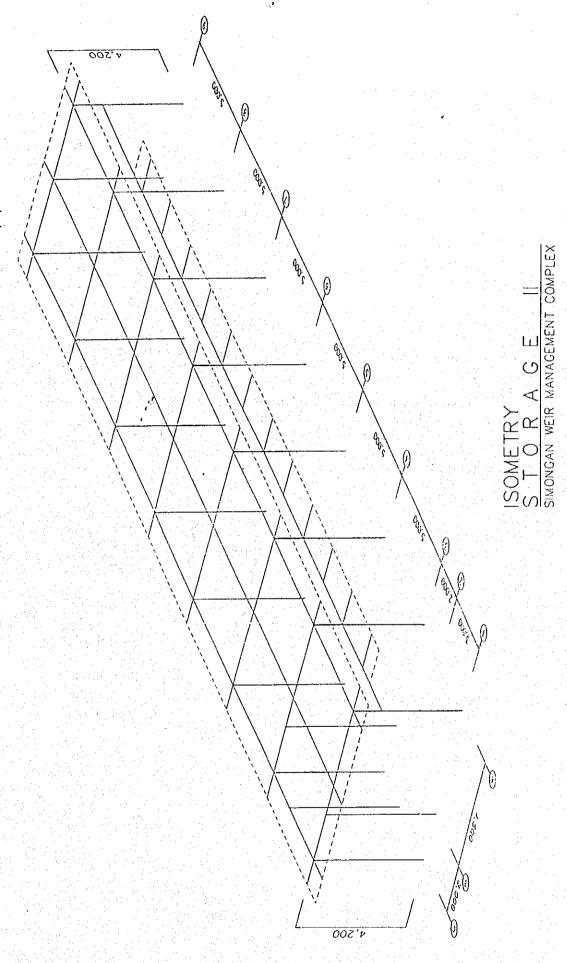
- 1 STRUCTURE
- 2 DESIGN CONDITION
- 3 LOADING CONDITION
- 4 DESIGN OF FOOTING

1. STRUCTURE





S T O R A G E II



2. Design Condition

- a) Roof slab
- concrette, thickness = 0,12 mm - main bar diameter = 10 mm
- Tie beam
- : Fc = 225 kg/cm2 on cubic sample, or Fc = 187 kg/cm2 on cylinder sample
 - main bar diameter = 16 mm
 - $F_V = 3200 \text{ kg/cm}2$
 - Stirrup bar diameter = 8,0 mm
 - $Fv = 2400 \text{ kg/cm}^2$

Column

- : Fc = 225 kg/cm^2 on cubic sample, or
 - Fc = 187 kg/cm^2 on cylinder sample
 - Main bar diameter = 16 mm
 - Fy = 3200 kg/cm2
 - Stirrup bar diameter = 0,8 mm
 - $Fy= 2400 \text{ kg/cm}^2$
- Foundation
- : Foot plate concrette
 - Fc = 225 kg/cm2 on cubic sample, or Fc = 187 kg/cm2 on cylinder sample
- b) Structural model
- : space (xyz axis) frame, linear elastic
- c) Analysis method
- : static

3. Loading Condition

- a) Dead Load :
- Concrette slab roof = $2400 \times 0.12 = 288 \text{ kg/m}^2$
- Water-stop slab roof = $2400 \times 0.03 = \frac{72 \text{ kg/m}^2}{360 \text{ kg/m}^2}$
- Concrette slab console = $2400 \times 0.12 = 288 \text{ kg/m}^2$
- Nater-stop slab console = $2400 \times 0.03 = \frac{72 \text{ kg/m}^2}{360 \text{ kg/m}^2}$
- Concrette slab leufel = $2400 \times 0.12 = 288 \text{ kg/m}^2$
- Water-stop slab leufel = 2400 x 0,03 = $\frac{72 \text{ kg/m}^2}{360 \text{ kg/m}^2}$
- Rolling door, say

 $= 50 \text{ kg/m}^2$

- b) Live load:
- Living load, say
- = 125 kg/m2

BEAM TYPE 5.

	2	(kg/cm2)	2.460
	À	(ka/cm2)	3,200
***************************************	ű	(kg/cm2)	187
	ų ų	stirrup (cm)	80
	da.	main bar (cm)	91.
,		cover (cm)	্য
		n (m)	25
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(cover	4	France elemen Force	Shear	(EX)	173	173	173	173	173	173	173	173	173
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2.400

λ**ί**υ (kg.cm)

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COULOW type 1

9:0101(ps 0 (cm)	h (cm)	ርመ)	gia. main bar (cm)	dia. stimup (cm)	(kg/cm2)	(kg/cm²)	fv (kg/cm2)
30	30	4	1.6	0.8	187	3,200	2,400

				1.00		·			· · ·
'n		Framë eli	emen Force				Dasign		
4	Member	Ayiai	Moment 2	Homent-3	Main bar	Stirrup	Pu	Mux	HUY
ļ	W.Zitioti	(kg).	(kg.cm)	(kg.cm)	(നന)	(mm)	(kg)	(kg.cm)	(kg.cm)
ļ		7.37							1.00
- [123	10,701	263,094	158,269	8016	or-120	10.684	470,339	470,339
1	124	16,870	260,228		8016	08-120	16,711	469,374	469,734
į	125	15.591	276,570		8016	08-120	15,936	38,245	
į	125	15 166	277,458		8015	08-120	16,006	. 38,415	471,311
j	127	16 429	289,302	392,084	6016	08-120	16,269	39,045	470,708
İ	128	16 079	300,409	94,436	8016	08-120	16,423	39 415	470,362
Í	123	17.382	302 804	583,290	8016	o8-120	100,891	74,595	303,856
1	130	10,833		186,158	12016	08-120	10,880	594,158	594,158
	131	12101		481,843	8016	08-120	12,517	31,381	472,638
	132	35,033				08-120	19,861	. 47,667	483,114
i	133	49 161	143,943			08-120	19,107	45,857	479,762
İ	134	49.346	144.432			08-120	19,157	46,002	480,031
I	135	43.607	151,339		8016	08-120	13,423		481,193
.	136	19 552	158,185		8D16	08-120	19,600	47,040	
	137	35 454	163,823			08-120	20,283	48,681	484,979
- }	138	12214	1			08-120	12,641	30,338	473,496
	139	13 393				08-120	13,724	32,938	
. }	140	38 589			8016	∙08-120	92,146	221,149	
į	141	50 668			.	08-120	92,146	.221,149	
.	112	50,896				08-120	32,148	221,149	
•	113	51 158		1		08-120	20,989	50,336	
. }	111	51 158	1			08-120	21,165		
- }	1.5	37 010			8016	08-120	21,850	52,438	
	148	13,505	4		8016	03-120	13,634	33,202	473,281
٠,	147	11 623		1	• 1.	08-120	11,650	471,661	471,861
.	118	17 670			8016	08-120	18,712	44,909	477,994
	149	17 474				08-120	17,472	41,933	472,404
	150	17 662				06-120	17,640	42,334	473,163
	151	17 330	1			08-120	17,907	42,976	474,371
	152	17,970	292,593			63-120	17,952	43,084	474,574
	153	19,261	231,219		0D16	08-120	19.242	46 180	480,383
	154	11.828			8018	o3-120	11,854	171,911	171,311
	155	12,837			6016	08-120	12,864	30,872	473,841
	156	20.277	52,893		8016	00-120	92,146	221,149	247,654
	157	19,040	1			08-120	92,146	221,149	282,558
÷	158	19,224		2,556	8016	08-120	92,145	221,119	251,520
	153	19,492			8D16	08-120	19,469	46,725	491,374
	160	19,535				n8-120	19,519	46,845	181,595
	151	20,306	1		8D16	o 0 120	20,793	49,902	487,212
	162	13,043		2,05€	8016	08-120	13,064	31,352	172,513
	'''		55,551	1					
	!	<u> </u>	!		 		·		

Checking of Beam reinforcement bar & stress

Random sample : On Beam No. F70

Positive Bending Moment = 745,962 kgcm
b (width) = 25 cm
h_t (height) = 45 cm
Concrete cover = 4 cm
Fc = 225 kg/cm²
$$\rightarrow$$
 $\sigma'_b = 130 kg/cm2$
Fu = 3,200 kg/cm² \rightarrow $\sigma_a = 2,600 kg/cm2
h0 = $\sigma_a = 2,600 = 1.43$$

a) For Positive BM, M = 745,962 kgcm

b = 25

$$h_t = 45$$
; $d = 4$, $d = 45 - 4 = 41$ cm

Ca = h =
$$\frac{41}{\sqrt{\frac{nM}{b\sigma_a}}}$$
 = 3.2.

$$\delta=0.6$$
 (required of minimum compression reinforcement bar)
 \Longrightarrow φ = 1.985 > φ_0 = 1.43 (OK)

$$\varphi'=2.820$$

$$n\omega=0.1071$$

. Stresses

$$\overline{\sigma}_{a} = 2,600 \text{ kg/cm}^{2}$$

$$\overline{\sigma}_{b} = \overline{\sigma}_{a} = 2,600 = 93.56 \text{ kg/cm}^{2} < \overline{\sigma}'_{b} = 130 \text{ kg/cm}^{2} \text{ (OK)}$$

$$\underline{n}\dot{\phi} = 14 \times 1.985$$

$$\sigma_a = \frac{1}{\sigma_a} = \frac{2,600}{2.820} = 921.99 \text{ kg/cm}^2 < \sigma_a = 2,600 \text{ kg/cm}^2 (OK)$$

Reinforcement bar

A steel { tensile } =
$$\frac{\omega \text{bh}}{14}$$
 = 0.1071 × 25 × 41 = 7.84 cm²

A steel (compression) = 8 × A steel (tensile)

$$= 0.6 \times 7.84 \text{ cm}^2 = 4.704 \text{ cm}^2$$

Used A
$$_{atcol}$$
 (tensile.) = 4 D 16 = 8.04 cm² (OK)

Used A
$$_{\text{atcel}}$$
 (compression) = 3 D 16 = 6.03 cm²(OK)

Checking of Column reinforcement bar & stress

On Column No. F139

```
= 555,303 \text{ kgcm}
Positive Bending Moment
                                      = 30
b (width)
                                                       cm
                                           30
ht (height)
Concrete cover
                                       = 30 - 4 = 26 cm
h = h_t - d
                                                                    \rightarrow \overline{\sigma'}_b = 130 \text{ kg/cm}^2
                                           225 	 kg/cm^2
FC
                                                                   \rightarrow \overline{\sigma}_{a} = 2,600 \text{ kg/cm}^2
                                           3,200 \text{ kg/cm}^2 -
Fu
ns
   \phi_0 = \overline{\sigma}_a = 2,600 =
         n о'ь 14х130
```

a) For Positive BM M = 178,266 kgcm

Ca = h = 26 = 2.61

$$\sqrt{\frac{\text{nM}}{\text{bo}_a}} \sqrt{\frac{14 \times 555,303}{30 \times .2600}}$$
 $\delta = 1$ (for symetrical reinforcement)

$$\phi = 1.778 > \phi_0 = 1.43 \text{ (OK)}$$

$$\phi' = 2.462$$

$$\eta\omega = 0.1705$$

Stresses

$$\frac{\overline{\sigma}_{a}}{\overline{\sigma}_{b}} = \frac{2,600 \text{ kg/cm}^{2}}{\frac{14 \times 1.778}{1600}} = \frac{2,600}{104.45} = \frac{104.45 \text{ kg/cm}^{2}}{\frac{104 \times 1.778}{1000}} = \frac{2,600}{\frac{104 \times 1.778}{1$$

Reinforcement

$$A = \omega bh = 0.1705 \times 30 \times 25 = 9.5 \text{ cm}^2$$

Hence applied:

A
$$_{\text{atcel}} = 8 \text{ D } 16$$

= 16.08 cm²

= 16.08 x 100 % A concrete

30x30

= 1.79 % A concrete (CK)

4 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 :

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

$$\sigma \max = 9.916 \times 10^{3} + 2.0923 \times 10^{4} + 4.997 \times 10^{4}$$

$$= 0.44 + 0.04 + 0.09$$

$$= 0.57 \text{ kg/cm}^{2} < \sigma' \text{ all} = 1.0 \text{ kg/cm}^{2} \text{ (ok)}$$

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

= 0.31 kg/cm²

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

```
HX = 9.533 E4 Kg MX = 6.3578 E4 Kg CM M2 = 19.1868 E4 kg CM Shear X = 477 kg Shear 2 = 1,1121 kg
```

anch soil stress beneth footing is

$$\sigma_{\text{max}} = \frac{9.533 \times 10^{3} + 6.3576 \times 10^{4} + 19.1866 \times 10^{5}}{(150)^{2} + (150 \times 150 \times 150^{2})} = \frac{1}{6 \times 150 \times 150^{2}} + \frac{19.1866 \times 10^{5}}{(150)^{2} + (150) \times 150^{2}} = \frac{1}{6 \times 150} \times \frac{1}{6 \times 150} = \frac{1}{6$$

:.::6 :

All of futing concrete reinforcement is calculated by "n" meriod (Indonesian Code). This was a first

 ns = 14

$$\phi_0 = \frac{\sigma_0}{\sigma_0 n} = \frac{2,600}{130 \times 14} = 1.43$$

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

$$h = hc - d$$

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

for $-\delta = 1$

 $\phi = 8.091 > \phi_0 = 1.43 \text{ (ok)}$

$$\phi' = 89$$
 ; $100n\omega = 0.69$

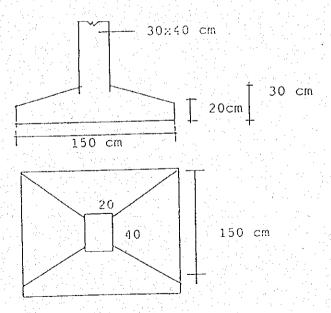
$$A = \omega bh$$

= $\frac{0.69}{100} \times 150 \times 20 = 20,7 \text{ cm}^2$

Astell = D16 - 15 cm(two way) \approx 11 x 2.01 = 22.12 cm² (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)

= 1.7025 E4 kg= 4.3720 E4 kgcm MX = 1.09209 E5 kgcm Μz Shear x = 387.5 kgbrickwall unit weight = 875 kg/m' (3m height) Shear z = 0

- Soil stress beneth foundation

 σ max = $\frac{1.7025 \times 10^4}{4.3720 \times 10^4} + \frac{1.09209 \times 10^5}{4.09209 \times 10^5} + \frac{875}{4.09209 \times 10^5}$ $\frac{100 \times (750 - 750)}{100 \times (750 - 750)} \frac{1/6 \times 600 \times 100^2}{1/6 \times 100 \times 600^2} \frac{100 \times 100}{100 \times 100}$ $= 0.28 \div 0.04 + 0.02$ $= 0.34 \text{ kg/cm}^2 < \sigma \text{ all} = 1.0 \text{ kg/cm}^2 \text{ (ok)}$

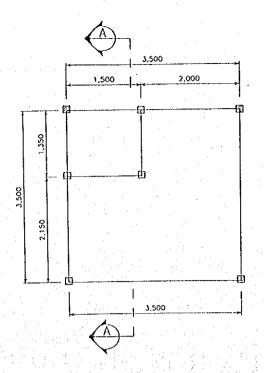
4.7.5. G U A R D H O U S E STRUCTURE CALCULATION

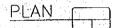
1 STRUCTURE

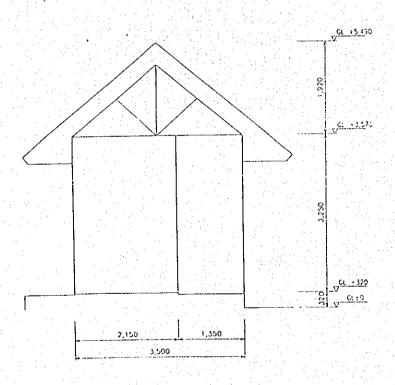
٠.,

- 2 DESIGN CONDITION
- 3 LOADING CONDITION
- 4 DESIGN OF PURLIN
- 5 DESIGN OF ROOF TRUSS

1. STRUCTURE



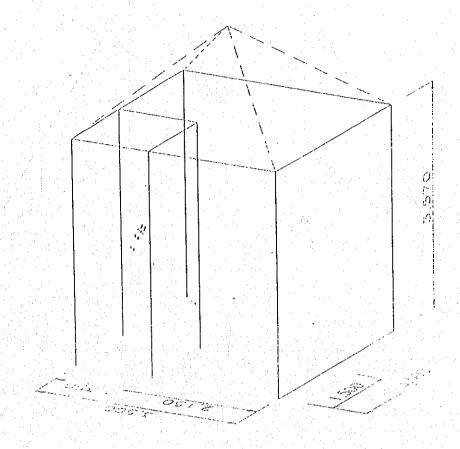




SECTION A-A

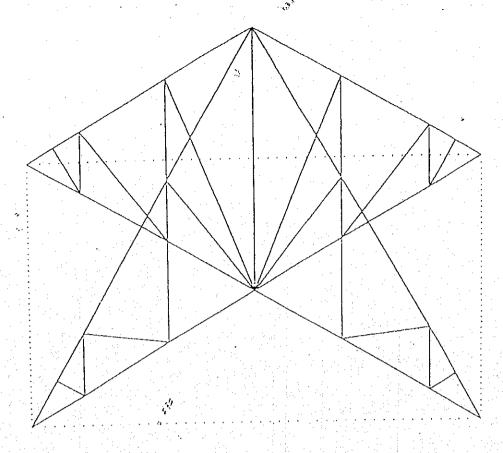
GUARD HOUSE

SIMONGAN WIER MANAGEMENT COMPLEX

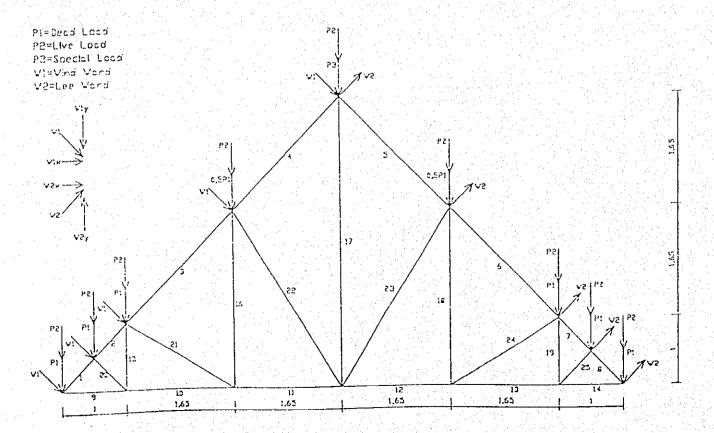


ISOMETRY GUARD HOUSE

TMORIGAN WEIR MANAGEMENT COMPLEX



SECURITY ROOF TYPE K1



2. Design Condition

- a. Dimensions
 - : 4.50 m - length
 - roof slope : 45°
 - b) Roof truss members : double angle steel
 - Tensile strength (Fy) : 2400 kg/cm²
 - 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/m²
 - Ceiling (fibre cement)

- $= 10 \text{ kg/m}^2$ 80 kg/m²
- 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
 - $w_1 = 0.5 \times 40 \text{ kg/m}^2 = 20 \text{ kg/m}^2$
 - $w2 = 0.4 \times 40 \text{ kg/m}^2 = 16 \text{ kg/m}^2$

4. Design of Roof Truss

- a. Dead load

 - $P_1 = (15X3.5) + (1.25X3.5X80) = 400 \text{ kg}$ $P_2 = (15X1.75) + (1.25X1.75X80) = 201.25 \text{ kg}$
 - = 150 Kg (PLN)- P
- b. Wind load
 - $-W_1 = 1.3 \times 3.5 \times 20 = 91 \text{ kg}$
 - $W_{1X} = W_{1Y} = 91 \text{ Sin } 45^{\circ} = 64.34 \text{ kg } \approx 65 \text{ KG}$
- c. Live load
 - $p_2 = 100 \text{ kg}$

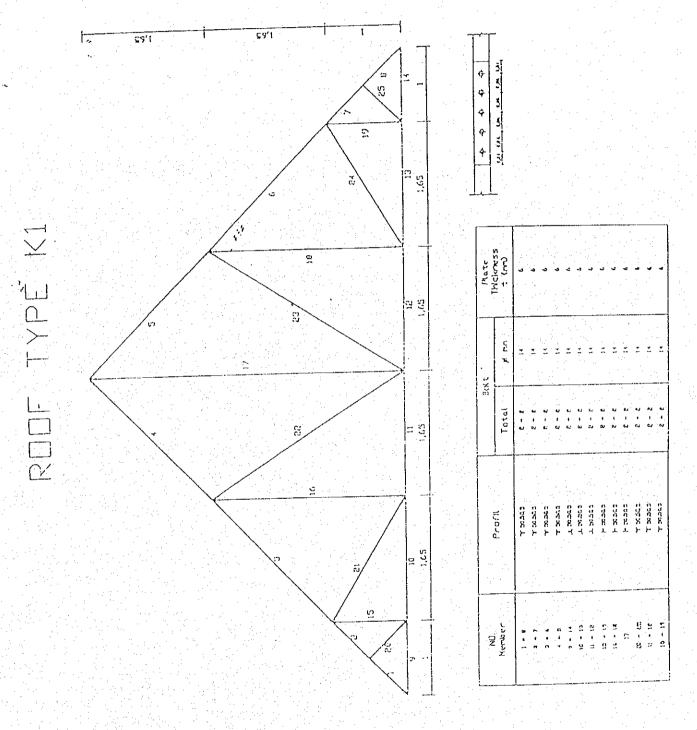
Roof Truss Security Base Simongan

 Prototype

 Profile
 Plate Thickness (kg/cm2) (kg/cm2)
 Fu (kg/cm2) (mm)
 dia. Bolt (mm)

 L 50.50.5
 0.5
 3,700
 2,400
 1.2

	E Cla	Axial	Shear	Torsion	Moment	n Bolt	d Boll
Member	Profile	(kg)	(kg)	(kg.cm)	(kg.cm)		(mm)
	 		7,437	<u>, , , , , , , , , , , , , , , , , , , </u>			
		714	2	0	33	2	1.2
1	L 50.50.5	719	2	0	33	2	12
2 3	L 50.50.5	55	3	0	102	2	12
3	L 50.50.5	31	3	0	101	2	12
4	L 50.50.5	25	3	0	101	2	12
5	L 50.50.5	63	2	0	102	2	12
6	L 50.50.5	723	2	0	33	2	12
7	L 50.50.5	728		0	33	2	12
, 8	L 50.50.5	506		0	94	2	12
9	L 50.50.5	510	1	0	73	2	12
10	L 50.50.5	41	3	0	72	2	
11	L 50.50.5	42	3	0	72		12
12	L 50.50.5	510	1	0	72	2	12
13	L 50.50.5	506		0	94	2	12
14 15	L 50.50.5	7		0	33		12
16	L 50.50.5	152	0	0	0		12
17	L 50.50.5	708	3	0	109	2	12
18	L 50.50.5	523	1	0	0		12
19	L 50.50.5	137	3	0	170		
20	L 50.50.5	240	0	0	1	2 2	12
21	L 50.50.5	138	3	1 .	1 1/2		The second second second
22	L 50.50.5			0	1		
23	L 50,50.5		3				
24	L 50.50.5		5 0	1.0	1 /		
25	L 50.50.5		3 2	C	33	2	12
					<u> </u>		<u> </u>



- 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 T24 F = 1,525 kgForce

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

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

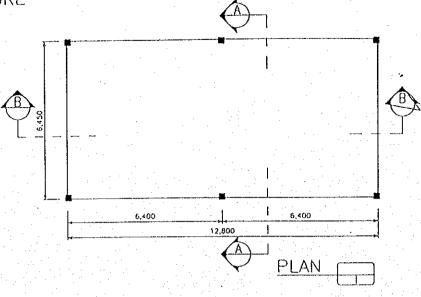
Stress
$$\sigma = \frac{F}{A} = \frac{1.525}{9.6} = 158.85 \text{ kg/cm}^2 < \sigma_{211} = 1.440 \text{ kg/cm}^2 \text{ (OK)}$$

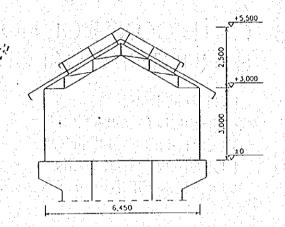
Hence double angle steel of 50.50.5 can be used as the members of roof truss type K - 1

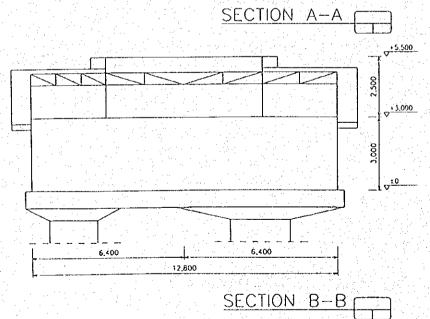
4.7.6. GATE CONTROL HOUSE 1 & 4 STRUCTURE CALCULATION

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

1. STRUCTURE

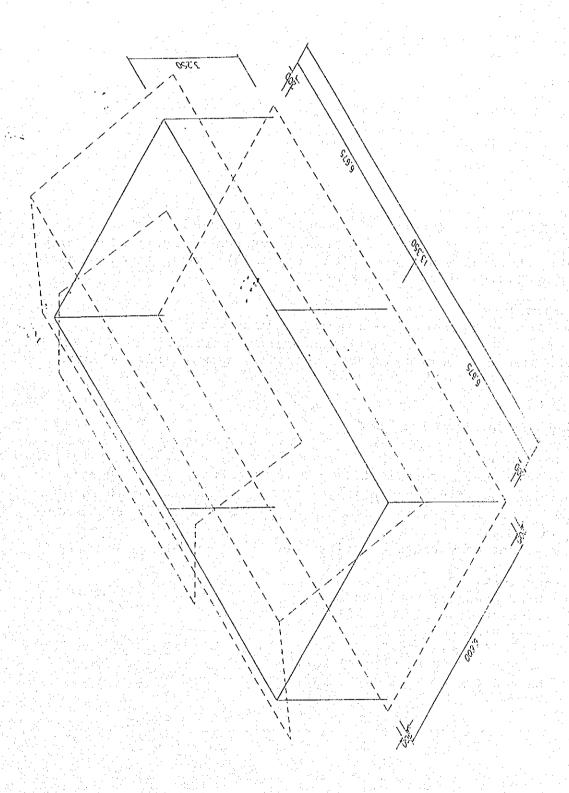






GATE CONTROL HOUSE II (1,4)

SIMONGAN WIER MANAGEMENT COMPLEX



ISOMETRY

GATE CONTROL HOUSE II

SIMONGAN WEIR MANAGEMENT COMPLEX

2. Design Condition

Dimensions

- a) Framing
 - width : 6.45 m - length : 13.35 m
 - height : 3.00 m
- b) Roof
 - length : 6.45 m - height : 1.57 m
 - roof slope : 45°

Roof truss :

- a) Roof truss members : double angle steel
 - Tensile strength (Fy) : 2400 kg/cm²
- b) Structural model : plane (xy axis) truss, linear elastic
- c) Analysis method : static

Framing:

- a) Framing members : biaxial column
 - Characteristic strength (Fc') = 225 kg/cm²
- b) Structural model: space (xyz axis) rame, linier elastic
- c) anaysis method : static

3. Loading Condition for roof

- a) Dead Load:
 - Roof cover (ceramic tile + timber rafter) = 70 kg/m^2
 - Ceiling (fibre cement) $\frac{= 10 \text{ kg/m}^2}{80 \text{ kg/m}^2}$
- 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
 - $W1 = 0.5 \times 40 \text{ kg/m}^2 = 20 \text{ kg/m}^2$
 - $W2 = 0.4 \times 40 \text{ kg/m}^2 = 16 \text{ kg/m}^2$

4. Design of Purlin

- A. Roof Truss Type K-1
- Purlin distance (c/c) = 1.63 m
- = 165.00 m- Purlin span
- Purlin self weight say = 15.00 kg/m'

$$q_1 = 1.63 \times 80 \text{ kg/m}^2 \approx 131 \text{ kg/m}'$$
 $q_2 \text{ (self weight)} = 15 \text{ kg/m}'$
 $Q = 146 \text{ kg/m}'$

$$Q_1 = Q_2 = Q \cos 45^{\circ}$$

= 146 Cos 45°
 $\approx 105 \text{ kg/m}'$

- Live Load $P_X = P_Y = P \cos \alpha = 100 \cos 45^{\circ} \approx 71 \text{ kg}$
- Bending moment

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

$$Mx = 1/8 \times 105 \times 1.65^2 + 1/4 \times 71 \times 1.65 = 65.02 \text{ kgm}$$

$$My = Mx = 65.02 \text{ kgm} = 6502 \text{ kgcm}$$

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

$$Ix = 664 cm^4$$
; $Wx = 88.6 cm^3$

$$Iy = 476 \text{ cm}^4$$
; $Wy = 73.2 \text{ cm}^3$

Stresses

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

$$=$$
 Mx + My

$$= 6502 + 6502 = 107.25 + 88.825$$

= 196.07 kg
$$< \sigma_{all} = 1,400 \text{ kg/cm}^2 \text{ (OK)}$$

Deflection

$$fx = 5 \times Q_1 \times L^4 + 1 PL^3$$

=
$$5 \times 1.05 \times 165^{\circ} + 1 \quad 71 \times 165^{\circ}$$

384 $2.1 \times 10^{6} \times 664 \quad 48 \quad 2.1 \times 10^{6} \times 664$

$$= 0.532 + 0.0047 = 0.5362$$
 cm

$$f = (0.5362^2 + 0.5362^2)^{1/2} = 0.76$$
 cm

Design of Roof Truss

a. Dead load

$$- P_1 = 1.65 \times (131 + 15) = 240.9 \text{ kg}$$

b. Wind load
$$- W_1 = 1.65 \times 1.63 \times 20 = 53.76 \text{ kg}$$

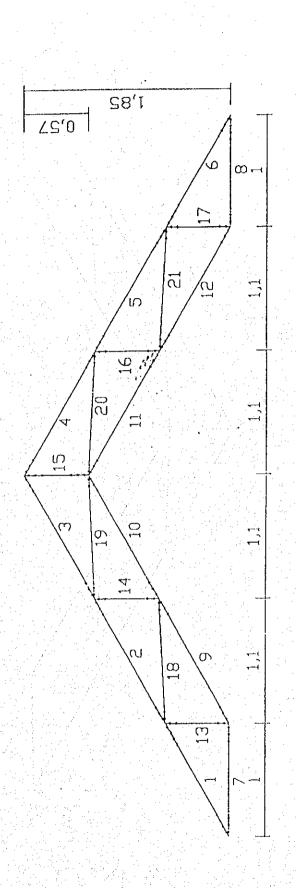
$$- W_2 = 1.65 \times 1.63 \times 16 = -43.032 \text{ kg}$$

$$W_{1X} = W_{1Y} = 53.76 \text{ Cos } 45^\circ = 38.01 \text{ kg}$$

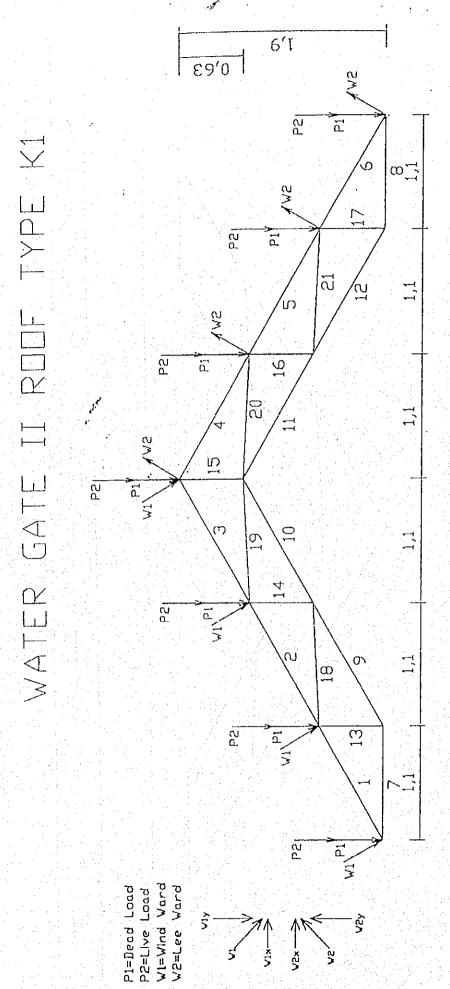
$$W_{2X} = W_{2Y} = -43.03 \text{ Cos } 45^\circ = -30.41 \text{ kg}$$

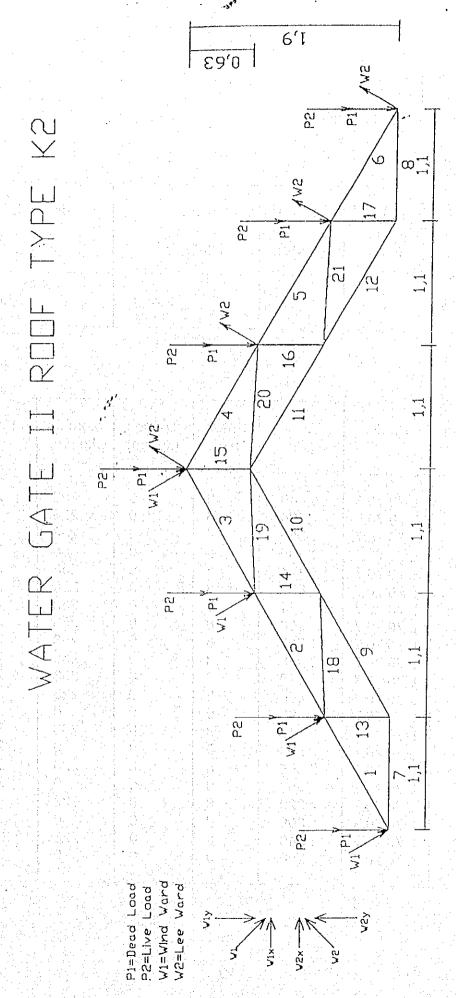
c. Live load
$$- p_2 = 100 \text{ kg}$$

WATER GATE ROOF TYPE K1



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	\ \		0.04 0.05					:					
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•		√ —¹											
	Plate	Thickness t (nm)	10	9	91	0,1	01	9.	93	01	01	91	9.1
		בנו אַ	14	<u> </u>	**	**	1	2	*	*		1	*1
	Bolt	Total	¥ ¥	e 1	D - D	ю !	9 1 3	C I C	נט נט	2 - 2	3 - 3	ល វ ប	ณ ย
	# V	11011	T75.75.7	T78.75.7	7.27.27	L75.75.7	175.75.7	T73.75.7	F 50.50.5	H 50.50.5	F 50.50.5	L 30.50.3	T 50 30:3
	ND.	Member	9 = 1	n 1 N	3 11 4	7 - 8	3 - 12	10 - 11	(1 = 61	14 = 16	n	18 = 81	19 - 20





Roof K-1 WATER GATE II SIMONGAN

Prototype

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia. Bolt (cm)
L 70.70.7	1	3,700	2,400	1.6

<u> jagot i Alfrantis k</u> a		· * * · · · · · · · · · · · · · · · · ·				Γ	
Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
		5.040	0		258	2	16
1	L 70.70.7	5,649	8	0	258 258	2	16
2	L 70.70.7	4,224	8	0	1.00	2	16
3	L 70.70.7	1,385	8 8	0	258	2	16
4	L 70.70.7	1,485		0	258	2	16
5	L 70.70.7	4,174	8	0	258		16
6 7	L 70.70.7	5,833	8	0	258	2 2	16
	L 70.70.7	131	8	0	258		
8	L 70.70.7	156	8	0	258	2	16
9	L 70.70.7	1,702	8	0	258	2	16 16
10	L 70.70.7	2,068	8	0	258	2	16
11	L 70.70.7	522	8	0	258	2	16
12	L 70.70.7	456	8	0	223	2	16
13	L 70.70.7	93	0	0	0	2	
14	L 70.70.7	1,340	8	0	223	2	16
15	L 70.70.7	809	0	0	0	2	16
16	L 70.70.7	2,554	8	0	223	2	16 16
17	L 70.70.7	358	0	0	0	2	1 . 1
18	L 70.70.7	2,229	8	0	223	2	16
19	L 70.70.7	809	0	0	0	2	16
20	L 70.70.7	1,340	8	0	223	2	16
21	L 70.70.7	290	0	0	0	2	16
		5, 2, 2, 3		19.5%		<u> </u>	

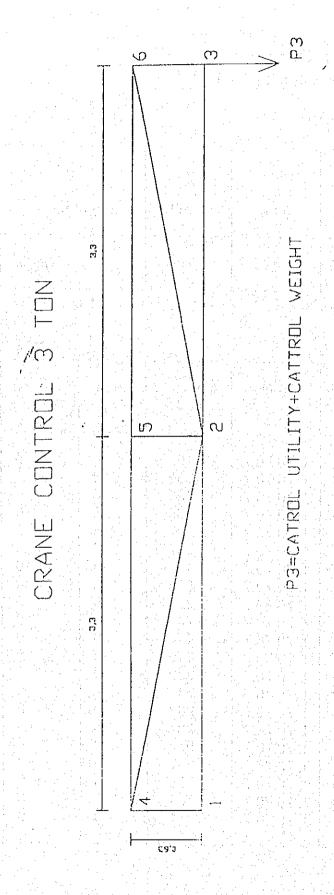
Roof K-2

WATER GATE II SIMONGAN

Prototype

Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia. Bolt (cm)
L 70.70.7	1	3,700	2,400	1.6

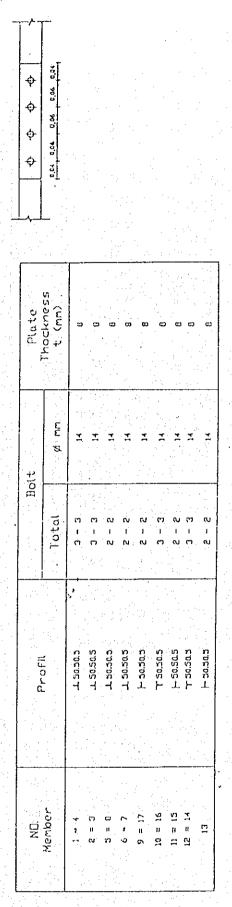
Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21	L 70.70.7 L 70.70.7	3,102 2,332 150 365 2,397 3,392 444 508 374 741 142 119 239 734 288 1,987 522 1,662 178 764 51	888888888080808080	000000000000000000000000000000000000000	258 258 258 258 258 258 258 258 258 258	2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2	16 16 16 16 16 16 16 16 16 16 16 16



12 - 16

59'0 1,65 σ 3 1,65 Ω. ដ 1,65 ſIJ ৩ Ξ 1,65 ហ ä S.

CRANE CONTROL 3 TON



Prototype Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia, Boli
L 70.70.7	1	3,700	2,400	1.6

Member	Profile	Axial (kg)	Shear (kg)	Torsion (kg.cm)	Moment (kg.cm)	n Bolt	d Bolt (mm)
1 2 3 4 5 6 7 8	L 70.70.7 L 70.70.7 L 70.70.7 L 70.70.7 L 70.70.7 L 70.70.7 L 70.70.7 L 70.70.7	10944 0 5216 16160 969 5321 4319 16476 3124	24 24 24 0 24 0 24 0	0 0 0 0 0 0 0	2009 2009 2009 2009 0 2009 0 2009 0	2 2 2	16 16 16 16 16 16 16 16

- Checking of members Strength of roof steel Truss Type Crane base on the axial force:
 - a. Due to Tensile force

Maximum force on member T8 Force
$$F = 16,476 \text{ kg}$$

Try : Double angle steel of
$$70.70.7$$
 Cross section area A = 18.8 cm²

$$\sigma_{s:1} = 0.6xFy$$

= 0.6x2,400 = 1,440 kg/cm²

Stress
$$\sigma = \frac{F}{A} = \frac{16,476}{18.8} = 876.38 \text{ kg/cm}^2 < \sigma_{311} = 1,440 \text{ kg/cm}^2 \text{ (OK)}$$

Hence double angle steel of 70.70.7 can be used as the members of roof truss type Crane

BEALL TYPE a Water Gate II Simongan

Prototype

٠.,		
	(), g/cm2) (kg/cm2)	2.400
	1y (3,0/cm2)	187 3.200
	fc (kg/cm2) (λg	187
	dia, Stirrup (cm)	9.0
	cover dia main bar dia Stirrup (cm) (cm)	1.6
	cover (cm)	4
	ر (E3)	45
	o (cus)	20

- 1	_			
	Stirrup	1	(EE)	08-150 08-150 08-150 08-150 08-150
			Boom	2016 2016 2016 2016 2016 2016 2016
	Right bases		Middle	2012 2012 2012 2012 2012 2012
			Top	3016 2016 3016 2016 2016 2016
Design			Botom	2016 2016 2016 2016 2015 2015
	Lidello bange	1000	Middlo	2012 2012 2012 2012 2012 2012
			Тор	2016 2016 2016 2016 2016 2016
			Botom	2016 2016 2016 2016 2016 2016
		Lon pages	Middle	2012 2012 2012 2012 2012 2012
			Top	2016 2016 2016 3016 2016 2016
		150 C C C C C C C C C C C C C C C C C C C	(mm)	000000
		LOCK OF	(ka.cm)	401,432 401,393 418,634 418,816 80,310 80,479
	Frame Element Force	Torsion	(Kg.cm)	25 25 25 25 25 25 25 25 25 25 25 25 25 2
	7 rame to	Shear	(kg)	2.571. 2.571. 2.695. 2.695. 835. 835.
		Avis	(ke)	1,251 1,251 1,278 1,278 2,199 2,199
		Linmhar	John Marie	L & 6 C C C

561,351 561,351 561,351 561,351 367,482 306,918

COULOM type I. Water Gate II Simengan

		(xg/cm2)	2,400
	<u>`</u>	(xc/cm2)	3,200
	ပ္ပ	(kg/cm2)	187
	cover. Ita, main badia, Stimp	(cm)	0.8
	ia, main ba	(cm)	1.5
	cover	(c::u)	*
	c	(cm)	25
adypo	ם	(Li	25

,								
200	Axia	Moment-2	Moment-3	Main bar	Stirrup	ā.	Mox	λο¦-
	(5x)	(kg.cm)	(ка.ст)	(mm)	(mm)	(xo)	(kg.cm)	(kg.cm)
· •-	\$436		•			8435	653534	,
7	10.58	21935	21936	4D16		64377		
c	4621		٠.			4622	•	٠.,
4	5548	į.		٠.	10	5546		
S	10670	٠,			٠.	54377	1.	144843
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	3256					32.56		ķ.
7	3256					3256	1.	
ñi	71:	 	٠.			177.4	196368	
iū	11.					1774	196368	366
				-				

Checking of Beam reinforcement bar & stress

Random sample On Beam No. F9

Positive Bending Moment = 418,884 kgcm
b (width) = 20 cm
h_t (height) = 45 cm
Concrete cover = 225 kg/cm²
$$\rightarrow \sigma'_b = 130 \text{ kg/cm}^2$$

FC = 3,200 kg/cm² $\rightarrow \sigma_a = 2,600 \text{ kg/cm}^2$
p $\sigma'_b = \frac{\sigma_a}{14 \times 130} = \frac{2,600}{14 \times 130} = 1.43$

For Positive BM, M = 418,884 kgcm

 $n\omega = 0.07759$

Stresses

$$\frac{\overline{\sigma}_{a}}{\overline{\sigma}_{b}} = \frac{2,600 \text{ kg/cm}^{2}}{n\phi} = \frac{2,600}{14 \times 2.33} = 79.71 \text{ kg/cm}^{2} < \overline{\sigma}_{b}' = 130 \text{ kg/cm}^{2} \text{ (OK)}$$

$$\frac{\overline{\sigma}_{a}}{n\phi} = \frac{2,600}{14 \times 2.33} = 825.40 \text{ kg/cm}^{2} < \overline{\sigma}_{a} = 2,600 \text{ kg/cm}^{2} \text{ (OK)}$$

$$\frac{\overline{\sigma}_{a}}{\phi'} = \frac{2,600}{3.15} = 825.40 \text{ kg/cm}^{2} < \overline{\sigma}_{a} = 2,600 \text{ kg/cm}^{2} \text{ (OK)}$$

Reinforcement bar

nforcement bar
$$A_{\text{steel (tensile)}} = \omega bh = 0.07759 \times 20 \times 41 = 4.54 \text{ cm}^2$$

$$A_{\text{steel (compression)}} = \delta \times A_{\text{steel (tensile)}}$$

$$= 0.6 \times 4.54 \text{ cm}^2 = 2.724 \text{ cm}^2$$

Used A steel (tensile) = 3 D 16 =
$$6.03 \text{ cm}^2$$
 (OK)
Used A steel (compression) = 2 D 16 = 4.02 cm^2 (OK)

Checking of Column reinforcement bar & stress

On Column No. F5

Positive Bending Moment = 61,589 kgcm
b (width) = 25 cm
h_t (height) = 25 cm
Concrete cover = 4 cm
h = h_t - d = 25 - 4 = 21 cm
Fc = 225 kg/cm²
$$\longrightarrow$$
 σ'_b = 130 kg/cm²
Fu = 3,200 kg/cm² \longrightarrow σ_a = 2,600 kg/cm²
ns = 14
 $\phi_0 = \overline{\sigma}_a = 2,600 = 1.43$

For Positive BM M = 61,589 kgcm

Ca = h = 21 = 5.77

$$\sqrt{\frac{\text{nM}}{\text{bo}_a}} \sqrt{\frac{14 \times 61,589}{25.7 \times 2600}}$$
 $\delta = 1$ (for symetrical reinforcement)

 $\phi = 3.762 > \phi_0 = 1.43$ (OK)

 $\phi' = 7.182$
 $\phi = 0.03243$

Stresses

$$\overline{\sigma}_{a} = 2,600 \text{ kg/cm}^{2}
\overline{\sigma}_{b} = \overline{\sigma}_{a} = 2,600 = 35.37 \text{ kg/cm}^{2} < \overline{\sigma}'_{b} = 130 \text{ kg/cm}^{2}
\overline{n} \phi = 14x5.25
\overline{\sigma}_{a} = 2,600 = 185.71 \text{ kg/cm}^{2} < \overline{\sigma}_{a} = 2,600 \text{ kg/cm}^{2}
\overline{\phi}' = 14.00$$

Reinforcement

$$A = \omega bh = 0.03243 \times 25 \times 21 = 1.22 \text{ cm}^2$$

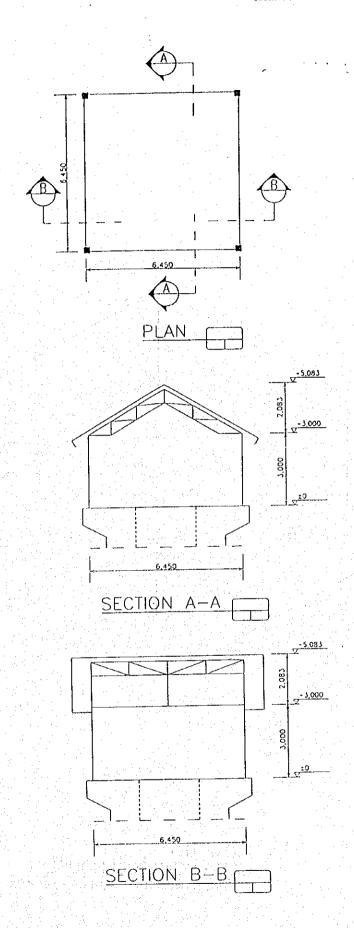
A $_{\text{steel}} = 1.22 \text{cm}^2 < 1 \% \times 1,500 \text{ cm}^2 \text{ (sectional area of column)}$

Hence applied:
A
$$_{\text{steel}} = 4 \text{ D } 16$$
 $= 8.04 \text{ cm}^2$
 $= \frac{8.04 \times 100 \text{ % A concrete}}{25 \times 25}$
 $= 0.0129 \text{ % A concrete} \text{ (OK)}$

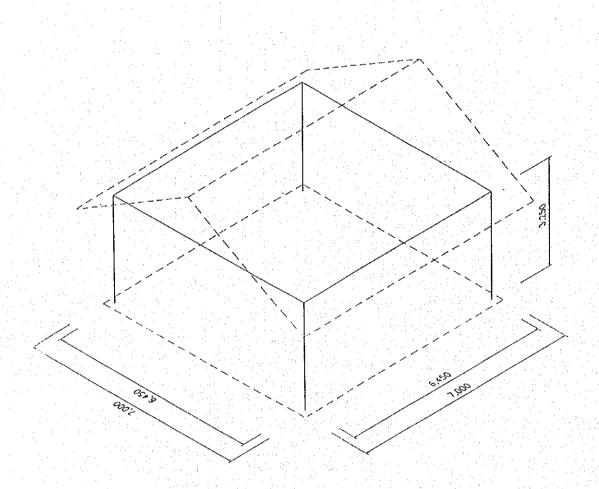
4.7.7. GATE CONTROL HOUSE 2 & 3 STRUCTURE CALCULATION

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

1. STRUCTURE



GATE CONTROL HOUSE 1 (2,3) SIMONGAN WIER MANAGEMENT COMPLEX



ISOMETRY

GATE CONTROL HOUSE I

SIMONGAN WEIR MANAGEMENT COMPLEX

2. Design Condition

a. Dimensions

- length : 6.45 m - roof slope : 30° - height : 1.85 m

b) Roof truss members : - double angle steel

- Tensile strength (Fy) : 2400 kg/cm²

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 - Ceiling (fibre cement) = $\frac{10 \text{ kg/m}^2}{80 \text{ kg/m}^2}$

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

 $w_1 = 0.5 \times 40 \text{ kg/m}^2 = 20 \text{ kg/m}^2$

 $W1 = 0.5 \times 40 \text{ kg/m}^2 = 16 \text{ kg/m}^2$ $W2 = 0.4 \times 40 \text{ kg/m}^2 = 16 \text{ kg/m}^2$

4. Design of Purlin

A. Roof Truss Type K-1

- Purlin distance (c/c) = 1.24 m

- Purlin span = 3.3 m

- Purlin self weight say = 15.00 kg/m'

 $q_1 = 1.24 \times 80 \text{ kg/m}^2 \approx 99.2 \text{ kg/m}'$

 q_2 (self weight) = 15 kg/m'

Q = 114.2 kg/m'

 $Q_1 = Q_2 = Q \cos 30^{\circ}$ = 114.2 Cos 30° $\approx 98.9 \text{ kg/m'}$

- Live Load $P_X = P_Y = P \cos \alpha = 100 \cos 30^{\circ} \approx 86.6 \text{ kg}$
- Bending moment

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

$$Mx = 1/8 \times 114.2 \times 3.3^2 + 1/4 \times 86.6 \times 3.3 = 226.9 \text{ kgm}$$

$$My = Mx = 226.9 \text{ kgm} = 26,900 \text{ kgcm}$$

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

$$Ix = 664 cm^4$$
; $Wx = 88.6 cm^3$

$$1y = 476 \text{ cm}^4$$
; Wy = 73.2 cm³

- Stresses

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

$$=$$
 Mx + My

$$= \frac{26,900}{88.6} + \frac{26,900}{73.2} = 303.61 + 372.57$$

=
$$676.18 \text{ kg} < \sigma_{all} = 1,400 \text{ kg/cm}^2 (OK)$$

- Deflection

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

$$= \underbrace{5}_{384} \times 1.142 \times \underbrace{330}_{2.1\times10} \times 664 + \underbrace{1}_{48} \underbrace{86.6}_{2.1\times10} \times 330^{3}$$

$$= 0.12 + 0.046 = 0.166$$
 cm

$$f = (0.166^2 + 0.166^2)^{1/2} = 0.234$$
 cm

$$f = 0.234 \text{ cm} < f_{ail} = \frac{1}{360} \text{ L} = \frac{330}{360} = 0.916 \text{ cm} (OK)$$

1.3

t Design of Roof Truss

- a. Dead load
 - $-P_1 = 3.30 \times (114.2 + 15) = 426.36 \text{ kg}$
 - Load of Cattroll = 3 ton
- b. Wind load
 - $W_1 = 3.30 \times 1.24 \times 20 = 81.84 \text{ kg}$
 - $W_2 = 3.30 \times 1.24 \times 16 = -65.47 \text{ kg}$

$$W_{1X} = 81.84 \text{ Cos } 30^{\circ} = 70.87 \text{ kg}$$

 $W_{1Y} = 81.84 \text{ sin } 30^{\circ} = 40.92 \text{ kg}$

$$W_{\rm v} = 81.84 \sin 30^{\circ} = 40.92 \text{ kg}$$

$$W_{2Y} = -65.47 \text{ sin } 30^{\circ} = -32.735 \text{ kg}$$

 $W_{2X} = -65.47 \text{ Cos } 30^{\circ} = -56.69 \text{ kg}$

$$W_{3V} = -65.47 \text{ Cos } 30^{\circ} = -56.69 \text{ kg}$$

- c. Live load
 - $p_2 = 100 \text{ kg}$

 Φ 58'1 ф ۷۵٬۵ 900 ф. 0,04 0,06 ∞--\$ WATER GATE 7×7 ROOF 21 ່ທ. 16 Plate Thickness t (nn) 20 0 0 0 0 0 0 0 0 Ü **** က 19 Bolt 14 Total ก เ ถ \wp α 7,27,277 7,27,277 T75.75.7 F 20.30.3 F 20.50.5 L 30.30.5 7.87.87 1 30.003 T25.25.T Profil 13 ND. Menber 18 1 E 9 = 12 10 = 11 13 = 17 14 = 16 13 8 - 6

Roof Truss K1 Water Gate Simongan 7 x 7

Profile Profile	Plate Thickness (cm)	Fy (kg/cm2)	Fu (kg/cm2)	dia, Bolt
L 75.75.7 L 50.50.5	1	3,700 3,7∞	2.400 2.400	

L			7.14			Moment	n Bot	d Boit
ſ	Member	Profile	Axial	Shear	Torsion		., 50	(mm)
ı	1110111201		(kg)	(kç)	(kg.cm)	(kg.cm)		
ł						277	2	14
ļ	1	L 75.75.7	938	9	0	277	. 4	14
1	2	L 75.75.7	1.783	9	0	277	- 6	14
Ì	3	L 75.75.7	2.006	9	0	277	6	14
1	_ Z	L 75.75.7	2.006	9	0	1	7	14
	5	L 75.75.7	1.783	9	0	277	2	14
. 1	6	L 75.75.7	9.389	9	0	277	2	14
	7	L 75.75.7	808	9	0	277	2	14
	8	L 75.75.7	977	S	0		4	14
	ğ	L 75.75.7	1.763	9	0	: _ :	,	14
	10	L 75.75.7	1.783	9	0	1 1 1 1 1 1		14
	11	L 75.75.7	977	9	0	40.0	2	14
	12	L 50.50.5	808	9	0	277	2	14
	13	L 50.50.5	521	0	0		2	
	14	L 50.50.5	736	4	0	1	2	. 14
	15	1.50.50.5	258	. 0	0	1 1 1 1	2	14
		L 50.50.5	1	1 .	0	1,144	, 2	14
	1€	L 50.50.5	1		0	0	2	14
	17	L 50.50.5			0	1.144	2	14
	18	L 50.50.5	1 3 1 1 1		l c	0	2	14
	15	L 50.50.5			C	1.144		14
	20		In a factor of	1	d c)	2	14
	21	L 50.50.5	321					
	1			1	1	J		

- Checking of members Strength of roof steel Truss Type K-1 base on the axial force:
 - a. Lue to Tensile force

Maximum force on member T6 Force F = 9,389 kg

Try : Double angle steel of 75.75.7 Cross section area A = 20/2 cm²

 σ_{a1} , = 0.6×Fy = 0.6×2,400 = 1,440 kg/cm²

Stress $\sigma = \frac{F}{A} = \frac{9,389}{20.2} = 464.6 \text{ kg/cm}^2 < \sigma_{all} = 1,440 \text{ kg/cm}^2 \text{ (OK)}$

Hence double angle steel of 75.75.7 can be used as the members of roof truss type K - 1

BEAM TYPE a Water Gate Simongam 7 x 7

Prototype

		100
2	(kg/cm2)	2,400
À	kg/cm2) (hg/cm:2) (3.200
ı	(kg/cm2)	137
dia, Stirrup	(cm)	9.0
cover dia, main bar dia, Stirrup	(cm)	9.1
COVER	(cm)	7
-	(cm)	Ç
ြ	(נונו)	30

				8	60	w	0	=	ø	ø	හ	
	Stirrup		(mm)									
		Botom		2D16	2D16	2D16	2D16	2D16	2D16	2016	2016	
	Right baars	Middle		2012	2012	2012	2012	2012	2012	2012	2012	. F . a . z
		Top		2D16	2015	2D16	2016	2016	2016	2016	2D16	
Dosign	5	Botom		2D16	2016	2016	2D16	2D16.	2016	2016	2016	
	Middle baars	Middle		2012	2012	2012	2012	2012	2012	2012	2012	
		Top		2016	2016	2016	2016	2D16	2D16	2016.	2D16	
		Botom		2D16	2016	2016	2016	2016	2D16	2016	2016	
	Lon baars	Middle		2012	2012	2012	2012	2012	2012	2012	2012	
		Top		2016	2D16	2D16	2016	2016	2016	2D16	2016	
	Main Bar	(mm)		15	16	16	16	16	91	16	16	
J	Moment	(kg.cm)		1.612	10.425	15.287	25.054	22.786	303,730	22.141	211,800	
Frame Element Forc	Torsion	(kg.cm)		1,736	1.48	200	1.232	358	336	378	373	
Frame	Shear	(kg)		1.703	2.068	1.737	189	3.585	420 3.368	38	3,730	
	Axial	(kg)		943	8	663	1.403	1.299	1.420	1.465	1.455	, v.
	Member				۲,	n	4	Ŋ	9	. 7	8	

BEAM TYPE b. Water Cate Simongan 7 × 7

Prodetype

2.	į	
2	(Kg/cm2)	2.400
 *	(kg/cm2)	3.200
33	(kg/cm2)	187
dua Stirrup	(cm)	8 O
ia, main baldia	(cm)	1.6
COVCI	(cm)	*
-	(cip)	25
٦	(cus)	20

		Framo Element Force	nent Force								Design					
Member	lcixA	Shear	Torsion	Moment	Main Bar		Left baars.		4	Aiddle baars			Right baars		Stirrup	Nfu
)	(kg)	(ka)	(kg.cm)	(kg.cm)	(mm)	Jop	Middle	Botom	do_t	Middlo	Botom	Top	Middle	Botom		
															(mm)	(kg.cm)
21	81	2.808	340	257.104	16	4D16		3016	2016		2016	2016	_	2016	හ	334,120
22	133	199			16	2016	•	2016	2016	•	2016	2016	•	2016	ສ	333,904
2	0.01	661	164	242.325	16	2D1C	•	2D16	3016		2016	.2016		2016	80	334.108
24	0	396	0	43.560	9	2D1G		2D16	2016		2016	2016	•	2D16	బ	335.628
55	0	396	0	43,560	9	2016	•	2016	2D16		2016	2D16		2016	83	335.574
56	0	395	0	43,560	5	2016	•	2D16	2D16		2016	2016	•	2016	60	333,618

BEAM TYPE c Water Gate Simongan 7 x 7

Prototype

2	(cultury)	7. 15 780	2.400
.≥	()	(10)(117)	3.200
٢	3	(XQ)Cm2)	187
Sire Chire	בסיפר בום, השות במבות, ביווינים	(E)	_ຍ 0
	ia. main ca L	(m ₀)	1.6
	2000	(cm)	4
	£	(cm)	20
	م	(cu3)	20

	· 2	_	(kg.cm)	334.120 333.904 334.108 335.578 335.574
	currup		(mm)	0 0 0 0 0 0
		Botom		2016 2016 2016 2016 2016 2016
	Right baars	Midello		
			25	2016 2016 2016 2016 2016 2016
Design		J	DOCUM	2016 2016 2016 2016 2016 2016
	Afiddle bases		Midale	
		,	Top	2016 2016 2016 2016 2016 2016
			Edom	2016 2016 2016 2016 2016 2016
	And the	Len pagis	Midalo	
			Тор	2016 2016 2016 2016 2016 2016
		Main Bar	(mm)	16 16 16 16 16 16
		Torsion Moment Mair	(kg.cm) (kg.cm)	2.594 2.836 1.911 8.036 2.610 14.533 2.348 28.582 1.859 879 2.381 29.460
	Jeni Force	Torsion	(ka.cm)	2.594 1.911 2.610 2.348 1.859
	Frame Element Force	Shear	(ka)	414 109 177 377 7
	.	Axial	(2)	18.816 952 1.622 2.018 933
		Jambar	3	9 0 1 1 2 2 1 2 2 4 1

COULOM type I Water Gate Simongan 7 x 7

	Prototype -						617	6/
	b	h			dia. Stirrup (cm)	(kg/cm2)	(kg/cm2)	(kg/cm2)
ı	(cm)	(cm)	(cm)	(cm)	(CIII)	(Kg/OIIIZ)	(kg/oi//2)	\ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \
	20	20	4	1.6	8,0	187	3.200	2.400
								L

	Frame Ele	ment Force		10 A. F		Design		· · · · · · · · · · · · · · · · · · ·
Member	Axial (kg)	Moment-2 (kg.cm)	Moment-3 (kg.cm)	Main bar (mm)	Stirrup (mm)	Pu (kg)	Mox (kg.cm)	Moy (kg.cm)
15 16 17 18	3608 3767 5206	49077 29110	170177 284533	4D16 8D16	08-90 08-90 08-90 08-90	3607 3766 5204 5407	200561 201074 328775 328024	20056 201074 328775 328024

COULOM type II Water Gate Simongan 7 x 7

	Prototype					()	(,	fy
Ī	h	h	cover	lia, main ba	oia. Stirrup	10	13	14
٠	· ·			(cm)	(cm)	(kg/cm2)	(kg/cm2)	(kg/cm2)
ı	(cm) [(cm)	(cm)	City		V-3		
Ť						1	2.7	
	วร	26	Α.	1.6	8.0	187	3.200	2.400
.	25	23		' ' ' '				

	Frame Ele	ment Force			er in the first of	Design		
Member	Axial (kg)	Moment-2 (kg.cm)	Moment-3 (kg.cm)	Main bar (mm)	Stimup (mm)	Pu (kg)	Mox (kg.cm)	Moy (kg.cm)
19	1808	638	638		08-90 08-90	1808 1609	76483 76865	76483 76865

