

# **PRELIMINARY DESIGN SUB STRUCTURE ANALYSIS**

**PROJECT :**  
4 Floors Mosque Buiding  
Sangata – East Kalimantan 2008

**By:**  
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### Preliminary Design

Dari: "Makno Basoeki" <anugrah\_alam2000@yahoo.com>

Kepada: "ferih ferihst" <alco\_sby@yahoo.com>

1. Sub Structure Analysis.pdf (152KB),
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- 2.e Upper Structure Analysis.pdf (80KB),
- 2.f Upper Structure Analysis.pdf (135KB),
1. COLUMN PLAN N SCHEDULE.pdf (235KB),
- 1.1. COLUMN ANCHORAGE.pdf (37KB),
2. 2nd column b-c schedule.pdf (281KB),
3. 3rd column b-c schedule.pdf (376KB),
4. 4th column b-c schedule.pdf (299KB), 4. dome.pdf (89KB)

Selasa, 9 Desember, 2008 14:17

Kepada Yth  
Bp Ir. Gunawan  
Surabaya

Ass wr wb

Sesuai pembicaraan semalam, terlampir saya kirim hasil preliminary design.

Wass wr wb

makno basoeki

-- Pada Sen, 24/11/08, ferih ferihst <alco\_sby@yahoo.com> menulis:

Dari: ferih ferihst <alco\_sby@yahoo.com>  
Topik: File LAP-TANAH 2  
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## RUMUS JARAK RAUT. (PPBBI)

$2,5d \leq s \leq 7d$  atau  $14t$  ( Jarak dalam)

$1,5d \leq s_1 \leq 3d$  atau  $6t$  ( Jarak luar).

## RUMUS TERAK LAS

$$a < \boxed{\frac{t}{2} \cdot \sqrt{2}}$$

## Mutu Baja

Baja ST 41  $\rightarrow \sigma = 1666 \text{ kg/cm}^2 = 166,6 \text{ MPa}$

## MOSQUE UPPER STRUCTURAL ELEMENTS FOUNDATION ANALYSIS

### Baja Dipasaran

SB0  $\rightarrow$  WF 900. 200. 11. 16 + vute 2m (bentang 10m).

SB1  $\rightarrow$  WF 450. 200. 9. 14 (bentang 7,2m)

SB2  $\rightarrow$  WF 400. 200. 8. 13 (bentang 6m)

SB3  $\rightarrow$  WF 200. 100. 5,5. 8 (bentang 3m).

SB4  $\rightarrow$  WF 200. 100. 5,5. 8

SB5  $\rightarrow$  WF 800. 300. 14. 25 + vute 4m (bentang 18m)

SC1  $\rightarrow$  WF 600. 200. 11. 17 (4 70)

SC2  $\rightarrow$  WF 300. 200. 10. 16 (4 60)

## بسم الله الرحمن الرحيم

### Construction Design References :

- 1 Mekanika Tanah Oleh : Dr. Ir. D. Westley , Badan Penerbit Pekerjaan Umum
- 2 Foundation Analysis and Design, by J.E. Bowles
- 3 Peraturan Pembebanan Indonesia 1983
- 4 Peraturan Beton Bertulang Indonesia 1990
- 5 Peraturan Beton Bertulang Indonesia 1971
- 6 Peraturan Perencanaan Bangunan Baja Indonesia 1983
- 7 Peraturan Konstruksi Kayu Indonesia 1970
- 8 Dutch Cone Test (Sondir) and Bor Log - Soil Investigation Laboratory

### Load Data and Specific Gravity of Several Construction Material :

1	Wind Load	=	40 kg/m <sup>2</sup>
2	Rain Load	=	100 kg/m <sup>2</sup> for every 10 cm thickness
3	Live Load	=	500 kg/m <sup>2</sup>
4	Ceiling Ld	=	75 kg/m <sup>2</sup>
5	Roof Frame	=	150 kg/m <sup>2</sup>
6	Roof Cover	=	150 kg/m <sup>2</sup>
7	Specific Gravity ~Concrete	=	2.400 kg/m <sup>3</sup>
8	Specific Gravity ~Steel	=	7.850 kg/m <sup>3</sup>
9	Specific Gravity ~Mortar	=	2.200 kg/m <sup>3</sup>
10	Specific Gravity ~Water	=	1.000 kg/m <sup>3</sup>
11	Specific Gravity ~Sand	=	1.800 kg/m <sup>3</sup>

### Standard Quality Of Several Material :

1	Plain Reinforced Concrete Steel U-24 , mean $\sigma_{au}$ =	2.400 kg/cm <sup>2</sup> → $\sigma_{au}^*$ =	1.600
2	Deform Reinforced Concrete Steel U-32, mean $\sigma_{au}$ =	3.200 kg/cm <sup>2</sup> → $\sigma_{au}^*$ =	2.133
3	MessReinforced Concrete Steel ST-37, mean $\sigma_{au}$ =	3.700 kg/cm <sup>2</sup> → $\sigma_{au}^*$ =	2.467
4	Sub Structural Concrete K-225, mean $\sigma_{bk}^*$ =	225 kg/cm <sup>2</sup>	
5	Super Structural Concrete K-175, mean $\sigma_{bk}^*$ =	175 kg/cm <sup>2</sup>	
6	Non Structural Concrete K-125, mean $\sigma_{bk}^*$ =	125 kg/cm <sup>2</sup>	

### Structural Idealization :

- 1 Roof Structure is idealized simply supported on the Composite Concrete Structural Frame. So There is only Axial gravitation and wind lateral force acted on each simply support. No Torque or Moment at all.
- 2 The concrete structural frame is idealized as rigid composite concrete frame fixed supported on the foundation. So there are several forces acted to the based of each column :
  - a. Axial Gravitation Load.
  - b. Wind Lateral / Horizontal Load
  - c. Moment Load
- 3 The foundation is idealised as grid mat foundation combine to deep foundation. There are several assumption for this combined foundation , such as :
  - a. Pile or deep drilling strauss foundation prior to be constructed on the proper position and designated depth of foundation.
  - b. The structure of the grid mat is fully rigid, all super structure load distributed spreadly to all part of the foundation.
  - c. Before Concreting the top concrete plate of the grid mat, vertical drain work properly to drain the excess water on the designed consolidated soil layer , 10 meters.
  - d. Simultaneously work between grid mat and consolidated designed soil layer is designed to bear 4 storeys mosque gravity load and lateral load such as wind load without earthquake load since Kalimantan on free earthquake zone indicated.

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#### Brief Description of Soil Investigation Result :

- 1 Soil with poor bearing capacity.
- 2 Lack of conus bearing ( qc ) on shallow depth shown by decrease of number of blows of Standard Penetration Test (N)
- 3 Soil Investigation recommend deep foundation or pile foundation to 26 meters depth, taken 30 meters based on :
  - qc = 200 kg/cm<sup>2</sup>
  - JHP = 2158 kg/cm

#### Problem Solving Discussion :

- 1 Considering the probability of problem will be occurred as follows :
  - Foundation is Pile Foundation as the deep foundation.
  - Settlement is being process after the construction finish and house/building occupied, since shallow water level under ground zero and cohesive soil mechanical behaviour that resist water related to deduct bearing capacity and potential long term settlement will be occurred or settlement after construction which is very dangerous for the stability of the building.
  - Based on this condition, main problem is water dominated content on bearing soil cause lack of bearing capacity and settlement occurred during construction and after construction.
  - To solve this problem means :
    - a. Increase soil bearing capacity.
    - b. Develop settlement during construction.
    - c. Eliminated settlement after construction.
- 2 Based on the above soil condition and based on the proven similar case and sub structure system, we design the following "vertically drained grid-mat foundation system" :
  - Develop / implant vertical drain every +/- 0,75 meter and average to 8 meters depth , to drain the excess water to reach proper settlement due to building load.
  - Develop Grid Mat Foundation to buils the solid stiff full plate foundation system to eliminate the differential settlement probably occurred during the construction period (immediate settlement) due to building load and prepared vertical drain. This Grid Mat is constructed from designed ground beam 30x60 cm sectional dimension which is monolith constructed with ground slab.

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# I. Loading Calculation :

## I.1. Loading Of The Roof Floor :

### I.1.1. Dead Loading Of The Roof Floor :

a. Roof Wooden Frame	=	=	150 kg/m <sup>2</sup>
b. Ceiling Frame & Cover	=	=	75 kg/m <sup>2</sup>
c. Roof Cover / Genteng	=	=	150 kg/m <sup>2</sup>
Total Dead Load Of The Roof Floor ( Qdl )			<u>375 kg/m<sup>2</sup></u>

### I.1.2. Live Loading Of The Roof Floor :

a. Rain Water 20 cm thick	=	0,2 x 1000	=	200 kg/m <sup>2</sup>
b. Wind Load	=	=	=	80 kg/m <sup>2</sup>
c. Working Load	=	=	=	100 kg/m <sup>2</sup>
Total Live Load Of The Roof Floor ( Qdl )			=	<u>380 kg/m<sup>2</sup></u>
Design Load 2nd Floor			=	1.133 kg/m <sup>2</sup>

## I.2. Loading Of The 2nd, 3rd & 4th Floor :

### I.2.1. Dead Loading Of The 2nd, 3rd & 4th Floor :

a. Concrete own weight	=	0,12 x 2400	=	288 kg/m <sup>2</sup>
b. Ceiling Frame & Cover	=	=	=	75 kg/m <sup>2</sup>
c. Floor Mortar & Ceramic	=	=	=	110 kg/m <sup>2</sup>
Total Dead Load Of The 2nd, 3rd & 4th Floor ( Qdl )			=	<u>473 kg/m<sup>2</sup></u>

### I.2.2. Live Loading Of The 2nd, 3rd & 4th Floor :

a. Rain Water 10 cm thick	=	0,1 x 1000	=	100 kg/m <sup>2</sup>
b. Working Load	=	=	=	500 kg/m <sup>2</sup>
Total Dead Live Of The 2nd, 3rd & 4th Floor ( Qll )			=	<u>600 kg/m<sup>2</sup></u>
Design Load 2nd Floor			=	1.610 kg/m <sup>2</sup>

## I.3. Loading Of The 1st Floor (Ground Floor) :

### I.3.1. Dead Loading Of The Ground Floor :

a. Concrete own weight	=	0,12 x 2400	=	288 kg/m <sup>2</sup>
b. Pasir Urug Pondasi	=	0,25 x 1800	=	450 kg/m <sup>2</sup>
Total Dead Load Of Ground Floor (Qdl)			=	<u>738 kg/m<sup>2</sup></u>

### I.3.2. Live Loading Of The Foundation :

a. Rain Water 5 cm thick	=	0,05 x 1000	=	50 kg/m <sup>2</sup>
b. Working Load	=	=	=	500 kg/m <sup>2</sup>
Total Live Load Of Ground Floor (Qll)			=	<u>550 kg/m<sup>2</sup></u>
Design Load 2nd Floor			=	1.932 kg/m <sup>2</sup>

## II. Total Building Weight Calculation (TBW) :

$$TBW = 1,5 \times \Sigma (Q_{dli} + Q_{lli})$$

$$TBW = (1.133 + 3 \times 1.610 + 1.932)$$

$$TBW = 7.893 \text{ kg/m}^2 = 7.89 \text{ ton/m}^2 = 0,789 \text{ kg/cm}^2$$

$$\text{Luas Bangunan (A)} = 1.225 \text{ m}^2$$

$$\text{Total Beban} = TBW \cdot A = 9.668.925 \text{ kg}$$

$$\text{Total Beban} = TBW \cdot A = 9.669 \text{ ton}$$

## III. Soil Bearing Capacity Calculation :

III.1. Based on the worst Surabaya soil investigation data ( $\phi=20^\circ$ ), since shallow drilling can't be done on those area (wet mud layer and difficult to take sample for laboratory test) - First Assumption :

Refer to JE Bowles : Foundation Analysis and Design

$$q_{ult} = 1,3 c \cdot N_c + q \cdot N_q + 0,4 \cdot \gamma \cdot B \cdot N_\gamma$$

Where :

Angle of Repose of Dense Silty Soil  $\phi = 20^\circ$  to  $30^\circ$  , Take  $\phi = 20^\circ$

====> Found From Table 4-1 (Reference 1) :

$$\begin{aligned} N_c &= 17,70 \\ N_q &= 7,40 \\ N_\gamma &= 5,00 \end{aligned}$$

$$\begin{aligned} N_c^* &= 11,60 \\ N_q^* &= 3,90 \\ N_\gamma^* &= 1,70 \end{aligned}$$

$$\begin{aligned} \gamma &= 1.700,00 \text{ kg/m}^3 = 126 \text{ pcf} \\ \text{Depth} &= 100,00 \text{ cm} = 3,33 \text{ feet} \\ q &= \gamma \cdot \text{Depth} = 420 \text{ psf} \\ c &= q / N_c = 23,71 \text{ psf} \\ B &= 35 \text{ m} = 105 \text{ feet} \end{aligned}$$

$$q_{ult} = 1,3 c \cdot N_c + q \cdot N_q + 0,4 \cdot \gamma \cdot B \cdot N_\gamma$$

$$q_{ult} = 1,3 \times 23,71 \times 17,7 + 420 \times 7,4 + 0,4 \times 126 \times 35 \times 5$$

$$q_{ult} = 30.096,3 \text{ psf} = 16,7 \text{ kg/cm}^2$$

$$\text{Safety} = q_{ultimate} / TBW$$

$$\text{Safety} = 16,7 / 0,789$$

$$\text{Safety} = 21,2$$

====> before drained by vertical drain the foundation is safe supported before fully construction loading.

### III.2. Based on worst investigated soil data ( second assumption $\phi=5^\circ$ ) :

Refer to JE Bowles : 'Foundation Analysis and Design'

$$q_{ult} = 1,3 c.N_c + q.N_q + 0,4 \cdot \gamma \cdot B \cdot N_\gamma$$

Where :

Angle of Repose of Dense Silty Clay  $\phi = 5^\circ$   
 =====> Found From Table 4-1 (Reference 1) :

$$\begin{aligned} N_c &= 7,30 \\ N_q &= 1,60 \\ N_\gamma &= 0,50 \end{aligned}$$

$$\begin{aligned} N_c' &= 6,70 \\ N_q' &= 1,40 \\ N_\gamma' &= 0,20 \end{aligned}$$

$$\begin{aligned} \gamma &= 1.700,00 \text{ kg/m}^3 = 126 \text{ pcf} \\ \text{Depth} &= 100,00 \text{ cm} = 3,33 \text{ feet} \\ q &= \gamma \cdot \text{Depth} = 420 \text{ psf} \\ c &= q / N_c = 57,50 \text{ psf} \\ B &= 32,4 \text{ m} = 97,2 \text{ feet} \end{aligned}$$

$$q_{ult} = 1,3 c.N_c + q.N_q + 0,4 \cdot \gamma \cdot B \cdot N_\gamma$$

$$q_{ult} = 1,3 \times 57,5 \times 7,3 + 420 \times 1,6 + 0,4 \times 126 \times 39 \times 0,5$$

$$q_{ult} = 3.665,3 \text{ psf} = 3,0 \text{ kg/cm}^2$$

$$\text{Safety} = q_{ultimate} / \text{TBW}$$

$$\text{Safety} = 2,0 / 0,789$$

$$\text{Safety} = 2,535$$

=====> after fully drained by vertical drain the foundation is safe supported after fully construction loading.

### IV. Foundation Dimension Design :

#### IV.1. Checking The Heaviest Load Of The Column ( Calculated with tributary area method ) :

$$P_{u\_1} = \text{Area Tributary} \times \Sigma (Q-dl + Q-ll)i \quad ( 1.133 + 3 \times 1.610 + 1.932 )$$

$$P_{u\_1} = (7,2/2+6/2) \cdot (7,2/2+6/2) \cdot 3 \cdot ( 1932 ) + (7,2/2+6/2) \cdot (7,2/2+6/2) \cdot ( 1610 )$$

$$P_{u\_1} = 322605,36 \text{ kg}$$

$$P_{u\_1} = 323 \text{ ton}$$

$$P_{u\_2} = \text{Area Tributary} \times \Sigma (Q-dl + Q-ll)i$$

$$P_{u\_2} = (6/2+6/2) \cdot (7,2/2+6/2) \cdot 4 \cdot ( 1932 )$$

$$P_{u\_2} = 306028,8 \text{ kg}$$

$$P_{u\_2} = 306 \text{ ton}$$



$$P_u_3 = \text{Area Tributary} \times \Sigma (Q\text{-dl} + Q\text{-ll})$$

$$P_u_3 = (7,2/2) \times (7,2/2) \times 4 \times (1932)$$

$$P_u_3 = 100154,88 \text{ kg}$$

$$P_u_3 = 100 \text{ ton}$$

Hence , The Heaviest Column is  $P_1 = 323 \text{ ton}$

Column Diameter	=	70 cm
Plate Thickness	=	15 cm
The Grid Dimension Width	=	30 cm
The Grid Dimension Height	=	60 cm

Ponds Shearing Area	:	
a. Concete Slab	=	$((70 + 15) + (15 + 70)) \times 2 \times 15 \text{ cm}^2$
Fa	=	5100 cm <sup>2</sup>

b. Concete Grid	=	$((30 + 15) \times (60 + 15)) \times 4 \text{ cm}^2$
Fb	=	13.500 cm <sup>2</sup>

$$\text{Total Ponds Shearing Area} = F_a + F_b$$

$$\text{Total Ponds Shearing Area} = 18.600 \text{ cm}^2$$

$$\text{For K-300} \Rightarrow \text{Found } \tau \text{ pond} = 20,4 \text{ kg/cm}^2$$

$$P \text{ ponds Allowed} = A_t \times \tau = 379.440 \text{ kg} = 379 \text{ ton} > P_1 = 323 \text{ ton (o.k.)}$$

#### IV.2. Grid Dimension and Reinforcement :

$$\text{Top Ellips Dimension Shortest Radius (a)} = 2,8 \text{ meter}$$

$$\text{Top Ellips Dimension Longest Radius (b)} = 4,8 \text{ meter}$$

$$\text{Bottom Ellips Dimension Shortest Radius} = 4,8 \text{ meter} = (a + 2 \times h_{\text{rib}})$$

$$\text{Bottom Ellips Dimension Longest Radius} = 6,8 \text{ meter} = (b + 2 \times h_{\text{rib}})$$

$$q_{\text{distributed}} = P_u_3 / \Sigma L_{\text{rib}_i}$$

$$q_{\text{distributed}} = 59.000 / (4,8 + 6,8 + 6,8) = 3207 \text{ kg/m}$$

$$M_{\text{max}} = 1/12 \times q_{\text{dist}} \times L^2$$

$$M_{\text{max}} = 1/12 \times 3207 \times 6,8^2$$

$$M_{\text{max}} = 12357,64 \text{ kg.m}$$

$$M_{\text{max}} = 12,36 \text{ ton.m}$$

$$Q_{\text{max}} = 1/2 \times q_{\text{dist}} \times L$$

$$Q_{\text{max}} = 1/2 \times 3207 \times 6,8 = 10.904 \text{ kg} = 10,90 \text{ ton}$$

#### IV.2.1 Grid Flexural Reinforcement :

$$C_u = \frac{h}{\sqrt{\frac{M_{max} \times 1.5}{2 \times k_o \times \sigma_{bk} \times b}}}$$

$$C_u = 16,592025$$

$$100 q = 4,2$$

$$A = 4,2/100 \times 30 \times 54 \times 2 \times 0,5 \times 175 / 2780$$

$$\begin{array}{l} D 16 = 2,0096 \\ D 13 = 1,32665 \end{array}$$

$$A = 5,076259 \text{ cm}^2$$

$$\text{Used 4 D16} = 8,04 \text{ cm}^2 > A \text{ (o.k. Satisfy the technical requirement)}$$

#### IV.2.2 Grid Shearing Reinforcement :

$$T_{bz} = Q_u / (b \times z)$$

$$T_{bz} = 1,5 \times Q_{max} / (b \times z)$$

$$T_{bz} = 1,5 \times 10904 / (30 \times 0,9 \times 64)$$

$$T_{bz} = 9,5 \text{ kg/cm}^2$$

Used Ø 8-150 :

$$\tau_{au} = A_{au} \times \sigma_{au} \times 2 / (b \times a_s)$$

$$\tau_{au} = 3,14/4 \times 0,8^2 \times 2080 \times 2 / (10 \times 10)$$

$$\tau_{au} = 20,90 \text{ kg/cm}^2 > T_{bz} \text{ (o.k. Satisfy the technical requirement)}$$

#### IV.2.3. Slab Reinforcement :

$$TBW = 4,674 \text{ kg/m}^2 = 4,67 \text{ ton/m}^2 = 0,467 \text{ kg/cm}^2$$

$$M_{slab\_max} = 1/12 \times TBW \times L^2$$

$$M_{slab\_max} = 1/12 \times 4674 \times 2,8^2$$

$$M_{slab\_max} = 3053,68 \text{ kg.m}$$

$$\text{Slab Thickness} = 15 \text{ cm}$$

$$C_u = \frac{h}{\sqrt{\frac{M_{max} \times 1.5}{2 \times k_o \times \sigma_{bk} \times b}}}$$

$$C_u = 18.56886$$

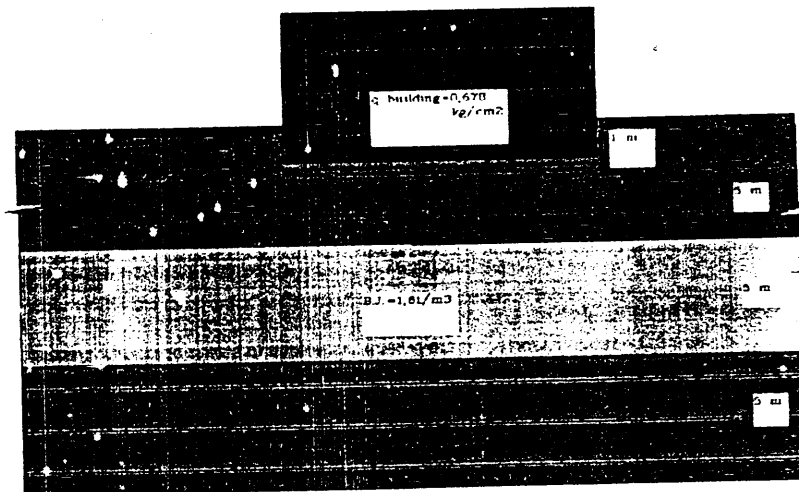
$$100 q = 4.2$$

$$A = 4.2/100 \times 100 \times 12.5 \times 2 \times 0.5 \times 175 / 2780$$

$$A = 3.10 \text{ cm}^2$$

Used 7 M 13 = 9.29 cm<sup>2</sup> > A (o.k. Satisfied the technical requiemant)  
 or # M 13-150

#### IV. Foundation Settlement Control :



$$P_o (1) = (2.2 \times 1.6 - 2.5 \times 1) = 1.02 \text{ ton/m}^2 = 0.102 \text{ kg/cm}^2$$

$$P_o (2) = (7.2 \times 1.6 - 7.5 \times 1) = 4.02 \text{ ton/m}^2 = 0.402 \text{ kg/cm}^2$$

$$P_o (3) = (12.2 \times 1.6 - 12.5 \times 1) = 7.02 \text{ ton/m}^2 = 0.702 \text{ kg/cm}^2$$

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Layer q=1,7xZ	Depth Z ( m )	m = B/Z = 13/Z	k = L/Z = 16,5/Z	$I \sigma$	$4 \times I \sigma$	$\Delta_P =$ $4 \times I \sigma \times q$	Po (kg/cm2)	P1	$\Delta_h$ ( mm )	S= $\Delta h/h \cdot H$ ( cm )
1 3,74	2,2	5,91	7,50	0,15	0,6	2,244	0,102	0,702	0,00014	0,28
2 12,24	7,2	1,81	2,29	0,09	0,36	4,4064	0,402	0,762	7E-05	0,315
3 20,74	12,2	1,07	1,35	0,06	0,24	4,9776	0,702	0,942	0,00011	0,77
									Total Settle- ment (cm)	1,365

Cc = Compression Index

h = sample thickness = 20 mm

Cc = 0,4216

$t_{90} = \frac{0,848 H^2}{C_v}$

Cv = Consolidation Index

$t_{90} = \frac{0,848 (400^2 + 1000^2)}{C_v}$

Cv = 1,8329 x 10<sup>-3</sup> mm<sup>2</sup>/detik

1,8329 x 10<sup>-3</sup>

$t_{90} = 532.294.372$  second

$t_{90} = 17$  years

Immediate Settlement will be occurred during the construction process, while long term settlement predicted will be occurred after 17 years building construction finished and approximately settle 1,4 cm in similar position of building. in other words the differential settlement will not be occurred, the long term settlement will be occurred same on every of building. Average speed of settlement every year = 1,4 cm / 17 year = 0,8 mm

STRONGLY REFERENCE AND RECOMMENDATION SHOULD BE APPLIED

IN ORDER TO REACHED IMMEDIATE SETTLEMENT DURING THE CONSTRUCTION, VERTICAL DRAIN SHOULD BE IMPLANTED UNTIL 10 METERS DEPTH. IN EVERY 0,75 METERS POINT TO POINT DISTANCE....

HENCE, SETTLEMENT CALCULATED HERE ONLY LONG TERM SETTLEMENT....

# V. PILE FOUNDATION CALCULATION :

Maximum Point/Column Load = 323 ton = 322.605 kg

Pile is designed to reach more than 26 meters depth or 30 meters depth refer to the following data :

Conus resistance qc = 200 kg/cm<sup>2</sup>

Accummulated Friction JHP = 2158 kg/cm

Pile Capacity Calculation :

$P_{\text{allowed}} = JHP \times \text{Circle Length} / 5 + qc \times \text{Section Area} / 3$

$P_{\text{allowed}} = 2158 \times (4 \times 40) / 5 + 200 \times 40 \times 40 / 3$

$P_{\text{allow}} = 175.723 \text{ kg} = 176 \text{ ton} \rightarrow \text{for 1 single pile}$

$P_{\text{allow}} = 527.168 \text{ kg} = 527 \text{ ton} \rightarrow \text{for 3 group pile}$

Hence  $P_{\text{allowed}}$  for 3 pile for 1 column = 527 ton >  $P_1 \text{ Max/Column Load} = 323 \text{ ton}$   
 -----> OK ....satisfy the required capqacity

Similar calculation for another position of column listed as following sketch

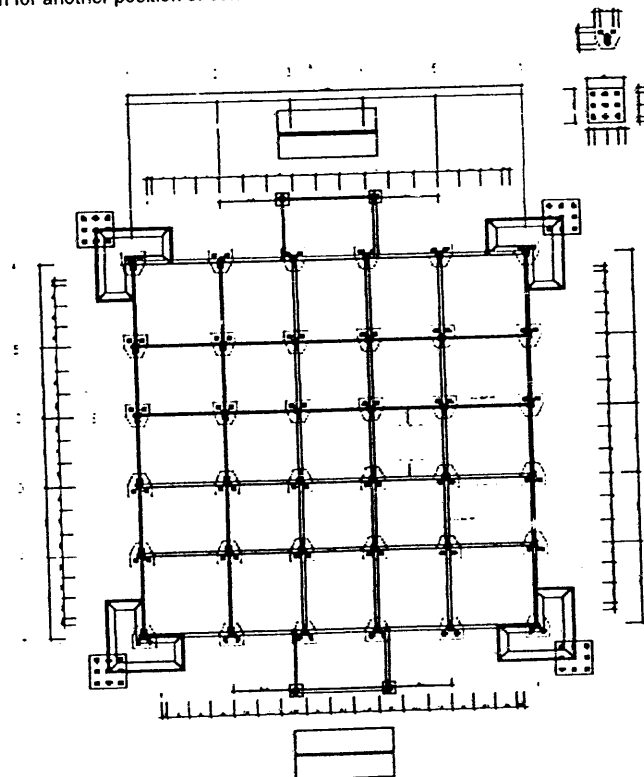
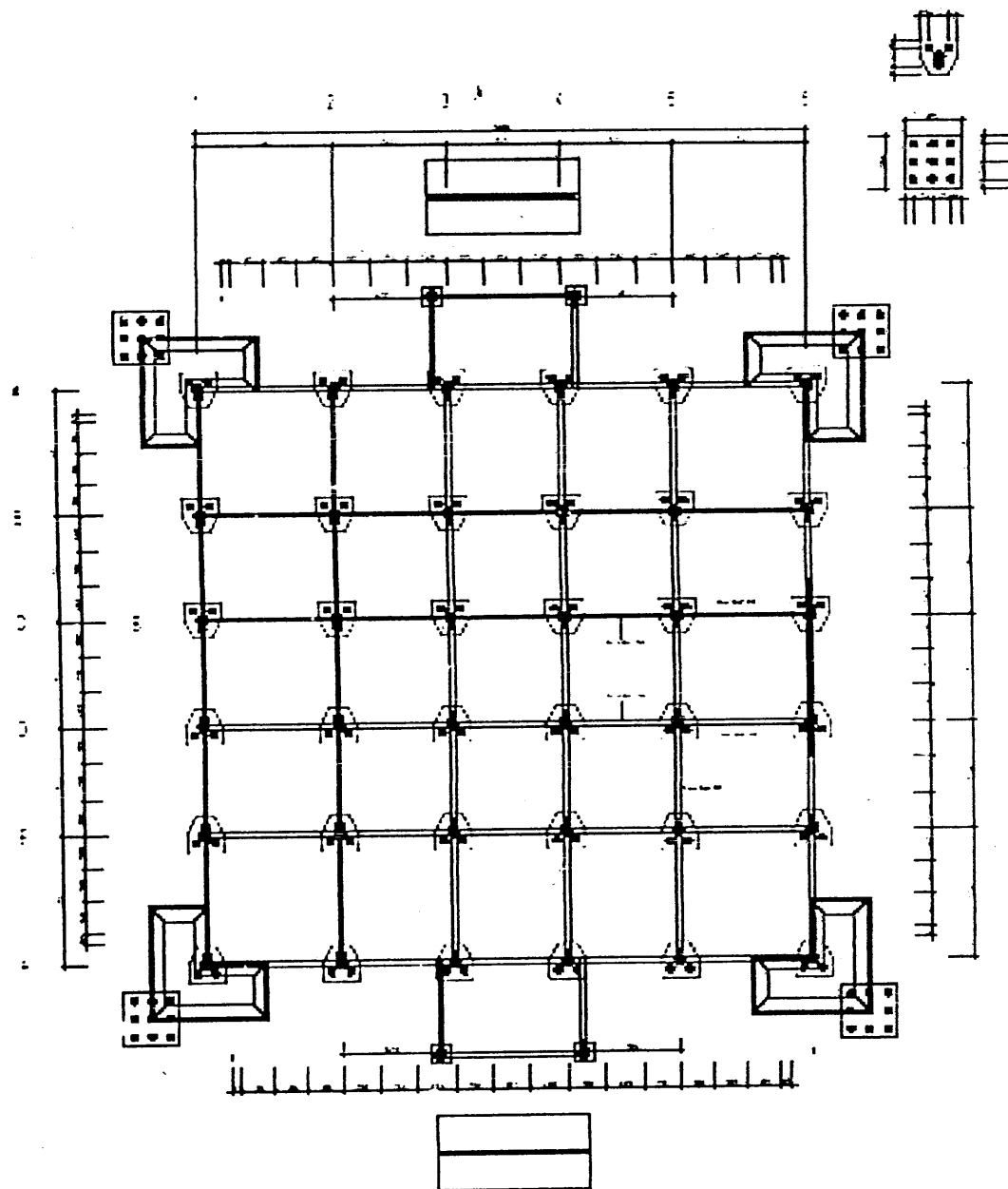


FIG. KOTEDUP ANA GROUND 4-41

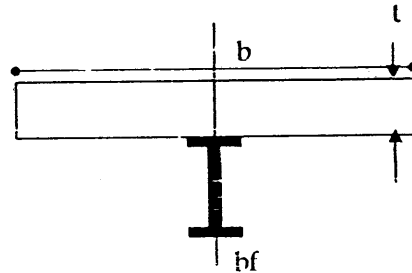


FOUNDATION PLAN & GROUND BEAM  
•• 2245 1/30

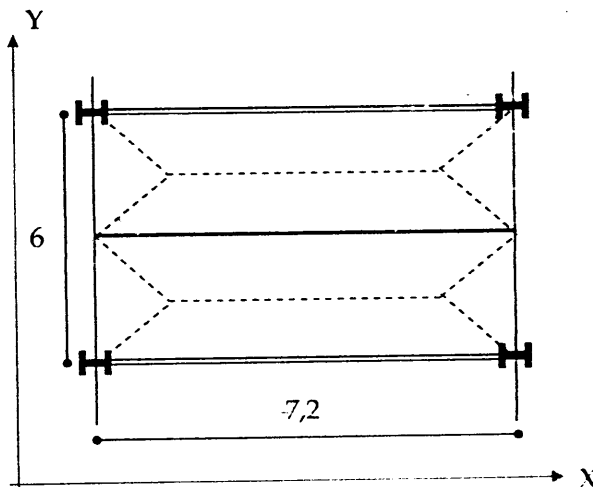
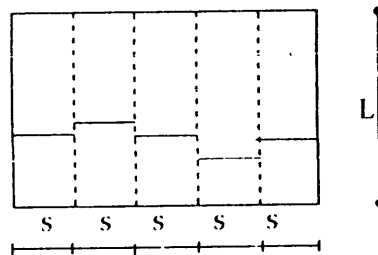
**MOSQUE UPPER STRUCTURAL ELEMENTS  
STEEL - CONCRETE COMPOSITE ANALYSIS**

## Mosque Upper Structural Elements Steel-Concrete Composite Analysis

### Beam Composite (X Axis)



$b = L / 4$  ;  $L = \text{beam length}$   
 $b = S$  ;  $S = \text{beam space}$   
 $b = bf + 16t$  ;  $t = \text{depth of slab}$



$t \text{ Slab} = 120 \text{ mm}$   
 $L = 7200 \text{ mm}$   
 $S = 3000 \text{ mm}$



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Using **WF450x200x66.2**

$$h_f = 446 \text{ mm}$$

$$b_f = 199 \text{ mm}$$

$$t_w = 8 \text{ mm}$$

$$t_f = 12 \text{ mm}$$

$$s_w = 66,2 \text{ kg/m}' = 0,6 \text{ kN/m}' \quad (\text{sw} = \text{Self Weight})$$

$$b = 7200 / 4 = 1800 \text{ mm}$$

$$b = 3000 \text{ mm}$$

$$b = 199 + 16 \times 120 = 2119 \text{ mm}$$

$$b_{\min} = 1800 \text{ mm}$$

### DATA SPESIFICATION

Steel Grade A-36

$$F_y = 240 \text{ Mpa}$$

$$f_c' = 20 \text{ Mpa}$$

$$E_c = 4700 \sqrt{f_c'} = 4700 \sqrt{20} = 21019 \text{ Mpa}$$

$$E_s = 200.000 \text{ Mpa}$$

$$n = E_s / E_c = 200000 / 21019 = 9$$

$$\Delta_D \max = 40 \text{ mm}$$

$$\Delta_L \max = L / 360 = 7200 / 360 = 20 \text{ mm}$$

### SUPERIMPOSED LOAD

$$\text{Live Load} = 6 \text{ kN/m}^2$$

$$\text{Partition} = 1 \text{ kN/m}^2$$

$$\text{Ceiling} = 0,75 \text{ kN/m}^2$$

$$\text{Superimposed} = 7,75 \text{ kN/m}^2$$

$$Q_{LL} = 775 \text{ kg/m}^2$$

### DEAD LOAD

$$\text{Slab} = 0,12 \times 24 = 2,88 \text{ kN/m}^2$$

$$\text{Finishing \& Ceiling} = 1,6 \text{ kN/m}^2$$

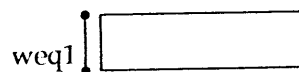
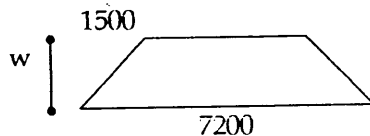
$$\text{Bondek } t = 1 \text{ mm} = 0,13 \text{ kN/m}^2$$

$$Q_{DL} = 4,61 \text{ kN/m}^2$$

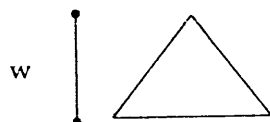
$$Q_{DL} = 461 \text{ kg/m}^2$$

### LOAD Coefisien

$$Q_u = 1,5 (Q_{DL} + Q_{LL}) = 1854 \text{ kg/m}^2$$



$$weq1 = w[1 - 4/3(a/L)^2] = w[1 - 4/3(1500/7200)^2] = 0,94213 w$$



$$weq2 = 2/3 w$$

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**SOLUTION**

**1) Find Bending Moment**

a) DEAD LOAD

Self Weight = 0,600 kN/m'

Slab =  $0,94212962962963 \times 4,61 \times S = 0,94212962962963 \times 4,61 \times 3$

Slab Weight = 13,030 kN/m'

Total = 13,630 kN/m'

Others 5% x Total =  $5\% \times 13,6296527777778 = 0,681 \text{ kN/m'}$

wD = 14,311 kN/m'

MD =  $1/8 wL^2 = 1/8 \times 14,311 \times 7,2^2 = 92,735 \text{ kN-m}$

VD =  $1/2 wL = 1/2 \times 14,311 \times 7,2 = 51,52 \text{ kN}$

b) SUPERIMPOSED LOAD

wL =  $0,94212962962963 \times 7,75 \times S = 0,94212962962963 \times 7,75 \times 3$

wL = 21,905 kN/m'

ML =  $1/8 wL^2 = 1/8 \times 21,905 \times 7,2^2 = 141,944 \text{ kN-m}$

VL =  $1/2 wL = 1/2 \times 21,905 \times 7,2 = 78,858 \text{ kN}$

Vmax =  $(14,311 + 21,905) \times 7,2/2 = 130,378 \text{ kN}$

**2) Find Ss (Fb = 0.66 Fy)**

Fb =  $0.66 Fy = 0.66(240) = 160 \text{ Mpa}$

Zs =  $MD/Fb = 92735000 / 160 = 579593,75 \text{ mm}^3 = 580 \text{ cm}^3$

Ztr min =  $(MD + ML)/Fb = (92735000 + 141944000) / 160 = 1466743,75 \text{ mm}^3 = 1467 \text{ cm}^3$

I min =  $(5/384) (wL^4)/(Es \times \Delta) = (5/384)(14,311)(7200^4)/(200000 \times 40) = 62596314 \text{ mm}^4 = 6260 \text{ cm}^4$

**3) Find Steel Beam with minimum**

Zs,min = 580 cm<sup>3</sup>

Is,min = 6260 cm<sup>4</sup>

Using **WF450x200x66.2**

Zs = 1290 cm<sup>3</sup>

Is = 28700 cm<sup>4</sup>

As = 84,3 cm<sup>2</sup>

**4) Calculate Characteristic of Composite Section**

b = 1800 mm

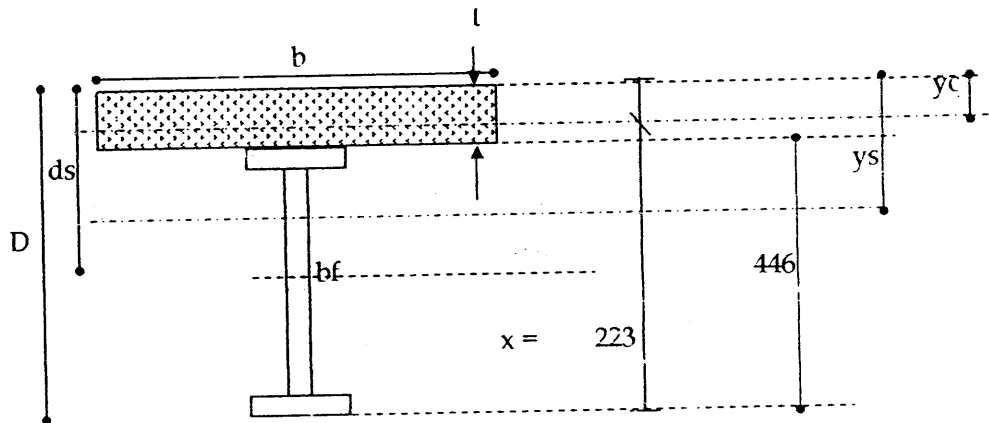
t = 120 mm

ds = 343 mm = (hf/2 + t)

D = 566 mm = (hf + t)

n = 9

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#### Neutral Axis within Concrete Slab

$$r = (2 A_s) / [(b)(d_s)]$$

$$r = (2 \times 8430) / (1800 \times 343)$$

$$r = 0,027308066$$

$$nr = 9 \times 0,02731 = 0,245773$$

$$y = (d_s/2) \times [\sqrt{\{(nr)(4+nr)\}} - nr] \quad y \leq t$$

$$y = (343/2) \times [\sqrt{\{0,24577 \times (4 + 0,24577)\}} - 0,24577]$$

$$y = 133 \text{ mm}$$

$$I_c = (b y^3) / (3n) + I_s + (A_s)(d_s - y)^2$$

$$I_c = 815605466,7 \text{ mm}^4$$

$$I_c = 81561 \text{ cm}^4$$

#### Section Modulus

$$Z_{bc} = I_c / (D - y) = 1883615 \text{ mm}^3$$

$$Z_{tc} = I_c / (t - y) = -6,3E+07 \text{ mm}^3$$

$$Z_c = I_c / y = 6132372 \text{ mm}^3$$

#### Neutral Axis in Steel Beam

$$y = [(bt^2)/(2n) + (A_s)(d_s)] / [(bt)/n + A_s] \quad y > t$$

$$y = [(1800 \times 120^2)/(2 \times 9) + 8430 \times 343] / [1800 \times 120/9 + 8430]$$

$$y = 102 \text{ mm}$$

$$I_c = (bt^3)/(12n) + (bt)/n \times (y - 0,5 t)^2 + I_s + A_s (d_s - y)^2$$

$$I_c = 847758830 \text{ mm}^4$$

$$I_c = 84776 \text{ cm}^4$$

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#### Section Modulus

$$\begin{aligned} Z_{bc} &= I_c / (D - y) = 1827066 \text{ mm}^3 \\ Z_{tc} &= I_c / (y - t) = -4,7E+07 \text{ mm}^3 \\ Z_c &= I_c / y = 8311361 \text{ mm}^3 \end{aligned}$$

#### Neutral Axis in Steel Beam

$$\begin{aligned} y &= 102 \text{ mm} \\ I_c &= 847758830 \text{ mm}^4 \\ Z_{bc} &= 1827066 \text{ mm}^3 \\ Z_{tc} &= -47097713 \text{ mm}^3 \\ Z_c &= 8311361 \text{ mm}^3 \end{aligned}$$

$I_c$  = Second Moment of Area of Composite Section  
 $Z_{bc}$  = Section Modulus of bottom flange Steel Beam, Composite Section  
 $Z_{tc}$  = Section Modulus of top flange Steel Beam, Composite Section  
 $Z_c$  = Section Modulus of Concrete Slab, Composite Section

$$\begin{aligned} \text{Check } Z_{bc} > Z_{tr} ? & \quad \text{if YES = OK; if NO = not OK} \\ 1827066 > 1466744 & \quad \text{OK} \end{aligned}$$

$$\begin{aligned} \text{Check for max } Z_{tr} \\ Z_{bc, \max} &= [1.35 + 0.35(ML / MD)] Z_s \\ &= [1.35 + 0.35(141,944 / 92,735)] \times (1290000) \\ &= 2432584,445 > 1827066 \text{ OK for using } Z_{bc} = 1827066 \text{ mm}^3 \end{aligned}$$

#### 5) Control Final Stress

Start

$$\begin{aligned} f_b &= MD / Z_s \\ f_b &= 92735000 / 1290000 \\ &= 71,89 \text{ Mpa} < 160 \text{ Mpa} \quad \text{OK} \end{aligned}$$

Final  
top

$$\begin{aligned} f_b &= ML / (n Z_c) \\ &= 141944000 / (9 \times 8311361) \\ &= 1,9 \text{ Mpa} < (0.45 \times 20) = 9 \text{ Mpa} \quad \text{OK} \end{aligned}$$

Bottom

$$\begin{aligned} f_b &= ML / Z_{bc} = 141944000 / (1827066) \\ &= 77,69 \text{ Mpa} < 160 \text{ Mpa} \quad \text{OK} \end{aligned}$$

$$\begin{aligned} f_b &= (ML + MD) / Z_{bc} = (141944000 + 92735000) / (1827066) \\ &= 128,45 \text{ Mpa} < 160 \text{ Mpa} \quad \text{OK} \end{aligned}$$

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$$f_b(\text{final}) \quad MD/Z_s + ML/Z_{bc} = \frac{71,89 + 77,69}{149,58 \text{ Mpa}} = < 160 \text{ Mpa} \quad \text{OK}$$

#### 6) Shear Connector

$$V_h = (0.85 f_c' b t) / 2 = (0.85 \times 20 \times 1800 \times 120) / 2 = 1836000 \quad \text{N}$$

or

$$V_h = A_s F_y / 2 = 8430 \times 240 / 2 = 1011600 \quad \text{N}$$

$$V_h = 1011600 \text{ N} = 1011,6 \text{ kN}$$

Table of Shear Connector (AISC spesification)

$\emptyset - t$	$f_c' = 20$	$f_c' = 25$	$f_c' = 30$
12 - 50	22,7	24,5	26,2
16 - 65	35,6	38,3	41,0
20 - 75	51,2	56,6	59,2
22 - 90	69,4	74,7	80,0

$$\text{Using } \emptyset \quad 16 - 65 \quad q = 35,6 \text{ kN}$$

$$N_1 = V_h / q = 1011,6 / 35,6 = 28,41573$$

using 30 units

$$N_2 = N_1 ((Z_{tr} M) / (Z_s M_{max}) - 1) / (Z_{tr} / Z_s - 1)$$

$$M = \text{Momen at Concetrated Load} = 0 \text{ kNm}$$

$$M_{max} = \text{Momen max at mid span} = 234,679 \text{ kNm} \quad (\text{MDL} + \text{MLL})$$

$$Z_{tr} = 1827066 \text{ mm}^3$$

$$Z_s = 1290000 \text{ mm}^3$$

Using 2 connectors for every section with spacing center to center

$$w_{min} = 4 \emptyset = 4 \times 16 = 64 \text{ mm}$$

$$\text{using } w = b_f / 2 = 199 / 2 = 99,5 \text{ mm} > 64 \text{ mm} \quad \text{OK}$$

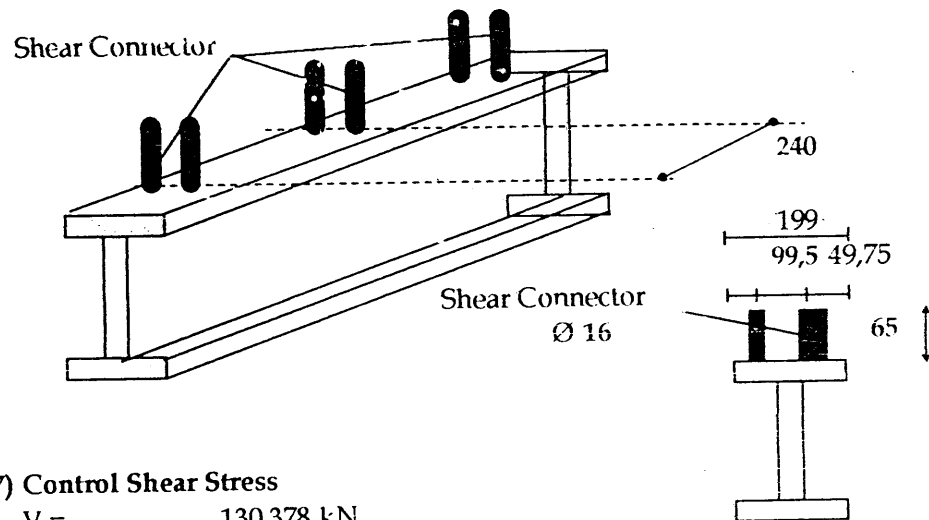
Space between longitudinal connector is

$$s = L / (2 N / 2) = 7200 / (2 \times 30 / 2) = 240 \text{ mm}$$

$$\text{Check } \min = 6 \emptyset = 6 \times 16 = 96 \text{ mm}$$

$$\max = 8 t = 8 \times 120 = 960 \text{ mm}$$

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#### 7) Control Shear Stress

$$\begin{aligned}
 V &= 130,378 \text{ kN} \\
 f_v &= V / (d \cdot t_w) = 130378 / (446 \times 8) = \\
 f_v &= 36,54 \text{ Mpa} < (0,4 \times 240) = 96 \text{ Mpa} \quad \text{OK}
 \end{aligned}$$

#### 8) Control Deflection

$$\begin{aligned}
 \Delta_L &= (5/384) (wL^4) / (EI) \\
 &= (5/384) (21,905 \times 7200^4) / (200000 \times 847758830) \\
 &= 4,52 \text{ mm} < 20 \text{ mm} \quad \text{OK}
 \end{aligned}$$

#### 9) RECAPITULATION

$F_c' = 20 \text{ Mpa}$   
 Beam Steel Grade A-36  
 $F_y = 240 \text{ Mpa}$

Using WF450x200x66.2

Shear Connector  $\varnothing 16 - 65$

per section	2 units
space	99,5 mm
longitudinal	240 mm

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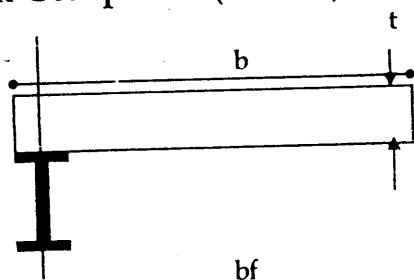
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## Mosque Upper Structural Elements Steel-Concrete Composite Analysis

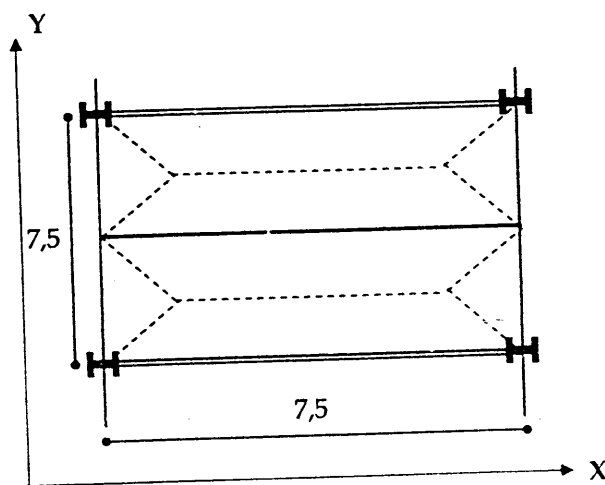
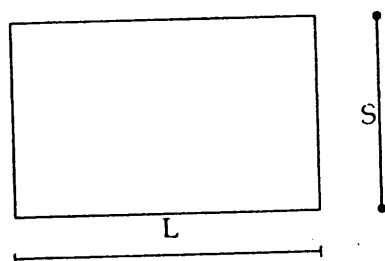
### Beam Composite (Y Axis)



$b = L / 12$  ;  $L$  = beam length

$b = S / 2$  ;  $S$  = beam space

$b = bf + 6t$  ;  $t$  = depth of slab



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t Slab = 120 mm

L = 7500 mm

S = 7500 mm

Using WF450x200x66.2

hf = 446 mm

bf = 199 mm

tw = 8 mm

tf = 12 mm

sw = 66,2 kg/m' = 0,6 kN/m' (sw = Self Weight)

b = 7500 / 12 = 625 mm

b = S/2 = 7500/2 = 3750 mm

b = 199 + 6 x 120 = 919 mm

b min = 625 mm

### DATA SPESIFICATION

Steel Grade A-36

Fy = 240 Mpa

fc' = 20 Mpa

Ec =  $4700 \sqrt{fc'} = 4700 \sqrt{20} = 21019$  Mpa

Es = 200.000 Mpa

n = Es/Ec = 200000 / 21019 = 9

$\Delta_D \max = 40$  mm

$\Delta_L \max = L/360 = 7500 / 360 = 20,83333$  mm

### SUPERIMPOSED LOAD

Live Load = 6 kN/m<sup>2</sup>

Partition = 1 kN/m<sup>2</sup>

Ceiling = 0,75 kN/m<sup>2</sup>

Superimposed = 7,75 kN/m<sup>2</sup>

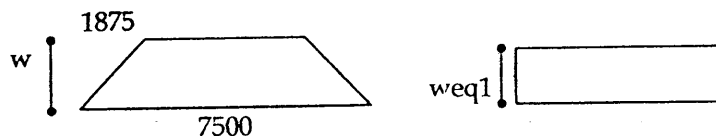
### DEAD LOAD

Slab = 0,12 x 23.5 = 2,82 kN/m<sup>2</sup>

Bondek t = 1 mm = 0,13 kN/m<sup>2</sup>

2,95 kN/m<sup>2</sup>

### LOAD Coefisien



$$weq1 = w[1 - 4/3(a/L)^2] = w[1 - 4/3(1875/7500)^2] = 0,916667 w$$



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$$weq2 = 2/3 w = 0,67 w$$

### SOLUTION

#### 1) Find Bending Moment

##### a) DEAD LOAD

Self Weight + others =

$$\text{Slab} = 0,67 \times 2,95 \times S = 0,67 \times 2,95 \times 1,875 =$$

$$\begin{array}{r} 0,600 \text{ kN/m}^1 \\ 3,706 \text{ kN/m}^1 \\ \hline \text{Total} \quad 4,306 \text{ kN/m}^1 \\ 0,215 \text{ kN/m}^1 \end{array}$$

$$\text{Others } 5\% \times \text{Total} = 5\% \times 4,306 =$$

$$\begin{array}{r} wD = 4,521 \text{ kN/m}^1 \\ PD = 51,52 \text{ kN} \end{array}$$

$$VD = (51,52 + 4,521 \times 7,5) / 2 =$$

$$MD = 1/8 wL^2 = 1/8 \times 4,521 \times 7,5^2 =$$

$$MD = 1/4 (PD) L = 1/4 \times 51,52 \times 7,5 =$$

$$\begin{array}{r} 42,714 \text{ kN} \\ 31,788 \text{ kN-m} \\ 96,6 \text{ kN-m} \\ \hline MD = 128,388 \text{ kN-m} \end{array}$$

##### b) SUPERIMPOSED LOAD

$$wL = 0,67 \times 7,75 \times S = 0,67 \times 7,75 \times 1,875 =$$

$$PL =$$

$$\begin{array}{r} 9,736 \text{ kN/m}^1 \\ 78,858 \text{ kN} \end{array}$$

$$VL = (78,858 + 9,736 \times 7,5) / 2 =$$

$$ML = 1/8 wL^2 = 1/8 \times 9,736 \times 7,5^2 =$$

$$ML = 1/4 (PL) L = 1/4 \times 78,858 \times 7,5 =$$

$$\begin{array}{r} 75,939 \text{ kN} \\ 68,456 \text{ kN-m} \\ 147,859 \text{ kN-m} \\ \hline ML = 216,315 \text{ kN-m} \end{array}$$

$$V_{\max} = 42,714 + 75,939 =$$

$$118,653 \text{ kN}$$

#### 2) Find Ss (Fb = 0.66 Fy)

$$Fb = 0.66 Fy = 0.66(240) =$$

$$160 \text{ Mpa}$$

$$Zs = MD / Fb = 128388000 / 160 =$$

$$802425 \text{ mm}^3 =$$

$$802 \text{ cm}^3$$

$$Z_{tr \min} = (MD + ML) / Fb = (128388000 + 216315000) / 160 =$$

$$2154393,75 \text{ mm}^3 =$$

$$2154 \text{ cm}^3$$

$$I_{\min} = (5/384) (wL^4) / (Es \times \Delta) = (5/384) (4,521) (7500^4) / (200000 \times 40) =$$

$$23282432,56 \text{ mm}^4 =$$

$$2328 \text{ cm}^4$$

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### 3) Find Steel Beam with minimum

$$Z_{s,min} = 802 \text{ cm}^3$$

$$I_{s,min} = 2328 \text{ cm}^4$$

Using WF450x200x66.2

$$Z_s = 1290 \text{ cm}^3$$

$$I_s = 28700 \text{ cm}^4$$

$$A_s = 84,3 \text{ cm}^2$$

### 4) Calculate Characteristic of Composite Section

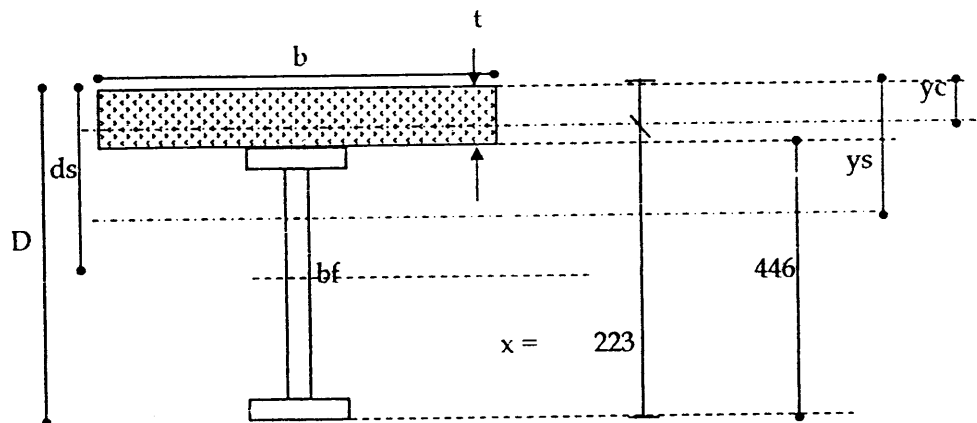
$$b = 625 \text{ mm}$$

$$t = 120 \text{ mm}$$

$$d_s = 343 \text{ mm} = (h_f/2 + t)$$

$$D = 566 \text{ mm} = (h_f + t)$$

$$n = 9$$



### Neutral Axis within Concrete Slab

$$r = (2 A_s) / [(b)(d_s)]$$

$$r = (2 \times 8430) / (625 \times 343)$$

$$0,07864723$$

$$nr = 9 \times 0,07865 = 0,707825$$

$$y = (d_s/2) \times [\sqrt{\{(nr)(4+nr)\}} - nr]$$

$$(343/2) \times [\sqrt{\{0,70783 \times (4 + 0,70783)\}} - 0,70783]$$

$$192 \text{ mm}$$

$$I_c = (b y^3) / (3n) + I_s + (A_s)(d_s - y)^2$$

$$I_c = 643052430 \text{ mm}^4$$

$$64305 \text{ cm}^4$$

$$y \leq t$$

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#### Section Modulus

$$\begin{aligned} Z_{bc} &= I_c / (D - y) = 1719392 \text{ mm}^3 \\ Z_{tc} &= I_c / (t - y) = -8931284 \text{ mm}^3 \\ Z_c &= I_c / y = 3349231 \text{ mm}^3 \end{aligned}$$

#### Neutral Axis in Steel Beam

$$y = \frac{[(bt^3)/(2n) + (As)(ds)]}{[(bt)/n + As]} \quad y > t$$

$$\frac{[(625 \times 120^3)/(2 \times 9) + 8430 \times 343]}{[625 \times 120/9 + 8430]} = 142 \text{ mm}$$

$$I_c = (bt^3)/(12n) + (bt)/n \times (y - 0.5 t)^2 + I_s + As (ds - y)^2$$

$$693613763,3 \text{ mm}^4$$

$$69361 \text{ cm}^4$$

#### Section Modulus

$$\begin{aligned} Z_{bc} &= I_c / (D - y) = 1635882 \text{ mm}^3 \\ Z_{tc} &= I_c / (y - t) = 31527898 \text{ mm}^3 \\ Z_c &= I_c / y = 4884604 \text{ mm}^3 \end{aligned}$$

$$y = 142 \text{ mm}$$

$$I_c = 693613763,3 \text{ mm}^4$$

$$Z_{bc} = 1635882 \text{ mm}^3$$

$$Z_{tc} = 31527898 \text{ mm}^3$$

$$Z_c = 4884604 \text{ mm}^3$$

Check  $Z_{bc} > Z_{tr} ?$

$$1635882 > 1154394$$

Neutral Axis in Steel Beam

Second Moment of Area of Composite Section

Sect Mod of bottom flange Steel Beam, Comp Sect

Sect Mod of top flange Steel Beam, Comp Section

Sect Modulus of Concrete Slab, Composite Section

if YES = OK; if NO = not OK

OK

Check for max  $Z_{tr}$

$$Z_{bc,max} = [1.35 + 0.35(ML/MD)] Z_s$$

$$[1.35 + 0.35(216,315/128,388)] \times (1290000)$$

$$2502211,457 > 1635882 \text{ OK for using } Z_{bc} =$$

$$1635882 \text{ mm}^3$$

#### 5) Control Final Stress

Start

$$f_b = MD / Z_s = 128388000 / 1290000$$

$$f_b = 99,53 \text{ Mpa} < 160 \text{ Mpa}$$

OK

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#### Final Top

$$f_b = ML / (n Z_c) = 216315000 / (9 \times 4884604) = 4,92 \text{ Mpa} < (0,45 \times 20) = 9 \text{ Mpa} \quad \text{OK}$$

#### Bottom

$$f_b = ML / Z_{bc} = 216315000 / (1635882) = 132,23 \text{ Mpa} < 160 \text{ Mpa} \quad \text{OK}$$

$$f_b = (ML + MD) / Z_{bc} = (216315000 + 128388000) / (1635882) = 130,71 \text{ Mpa} < 160 \text{ Mpa} \quad \text{OK}$$

$$f_b(\text{final}) \quad MD/Z_s + ML/Z_{bc} = 99,53 + 132,23 = 141,76 \text{ Mpa} < 160 \text{ Mpa} \quad \text{OK}$$

#### 6) Shear Connector

$$V_h = (0,85 f_c' b t) / 2 = (0,85 \times 20 \times 625 \times 120) / 2 = 637500 \text{ N}$$

or

$$V_h = A_s F_y / 2 = 8430 \times 240 / 2 = 1011600 \text{ N}$$

$$V_h = 637500 \text{ N} \quad 637,5 \text{ kN}$$

#### Table of Shear Connector (AISC spesification)

$\emptyset - l$	$f_c' = 20$	$f_c' = 25$	$f_c' = 30$
12 - 50	22,7	24,5	26,2
16 - 65	35,6	38,3	41,0
20 - 75	51,2	56,6	59,2
22 - 90	69,4	74,7	80,0

Using  $\emptyset$  16 - 65  $q = 35,6 \text{ kN}$

$$N_1 = V_h / q = 637,5 / 35,6 = 17,907303$$

using 18 units

$$N_2 = N_1 ((Z_{tr} M) / (Z_s M_{max}) - 1) / (Z_{tr} / Z_s - 1)$$

$$M = \text{Momen at Concetrated Load} = 244,459 \text{ kNm} \quad (\text{MDPL} + \text{MLPL})$$

$$M_{max} = \text{Momen max at mid span} = 344,703 \text{ kNm} \quad (\text{MDL} + \text{MLL})$$

$$Z_{tr} = 1635882 \text{ mm}^3$$

$$Z_s = 1290000 \text{ mm}^3$$

$$N_2 = 18 \times [(1635882 \times 244,459) / (1290000 \times 344,703 - 1) / (1635882 / 1290000 - 1)] = 6,757674115 \text{ using } 8 \text{ units}$$

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Using 2 connectors for every section with spacing center to center

$$w_{min} = 4 \varnothing = 4 \times 16 = 64 \text{ mm}$$

$$\text{using } w = bf / 2 = 199 / 2 = 99,5 \text{ mm} > 64 \text{ mm} \quad \text{OK}$$

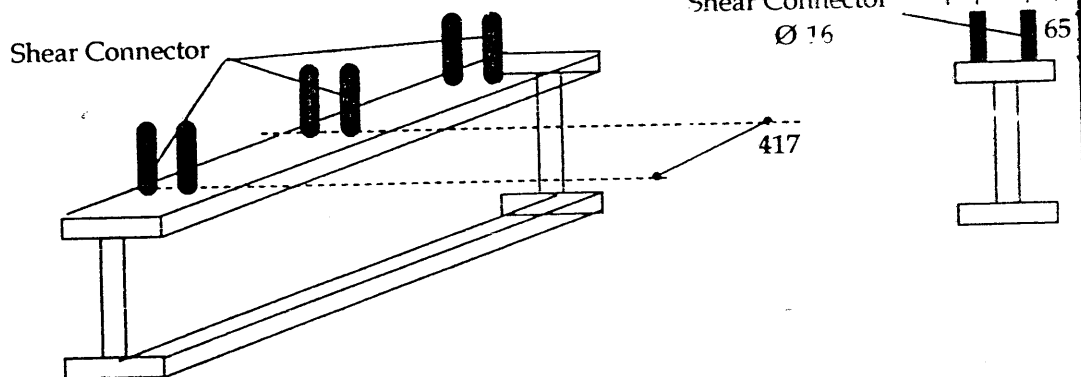
Space between longitudinal connector is

$$s = L / (2 N / 2) = 7500 / (2 \times 18 / 2) = 417 \text{ mm}$$

Check

$$\min = 6 \varnothing = 6 \times 16 = 96 \text{ mm}$$

$$\max = 8 t = 8 \times 120 = 960 \text{ mm}$$



#### 7) Control Shear Stress

$$V = 118,653 \text{ kN}$$

$$f_v = V / (d \cdot t_w) = 118653 / (446 \times 8) =$$

$$f_v = 33,25 \text{ Mpa} < (0,4 \times 240) = 96 \text{ Mpa} \quad \text{OK}$$

#### 8) Control Deflection

$$\Delta_L = (5/384) (w L^4) / (EI)$$

$$(5/384) (9,736 \times 7500^4) / (200000 \times 693613763)$$

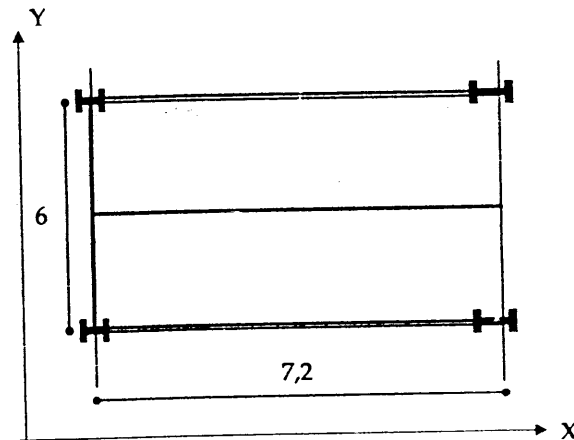
$$2,89 \text{ mm} < 20,83 \text{ mm} \quad \text{OK}$$

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## 9) RECAPITULATION

$F_c' = 20 \text{ Mpa}$   
 Beam Steel Grade A-36  
 $F_y = 240 \text{ Mpa}$   
 Using WF450x200x66.2  
 Shear Connector  $\varnothing 16 - 65$   
     per section           2 units  
     space               99,5 mm  
     longitudinal       417 mm

## Mosque Structural Elements Concrete Slab-Bondex Composite Analysis



t Slab = 120 mm  
 L = 7200 mm  
 S = 3000 mm

### DATA SPESIFICATION

Bondek

sheeting 0,75 mm

Fy = 500 Mpa

Fb =  $0.6 \times Fy = 0.6 \times 500 = 300 \text{ Mpa}$

fc' = 20 Mpa

fc' max =  $0.45 \times fc' = 0.45 \times 20 = 9 \text{ Mpa}$

Ec =  $4700\sqrt{(fc')} = 4700\sqrt{(20)} = 21019 \text{ Mpa}$

Es = 200.000 Mpa

n =  $Es/Ec = 200000 / 21019 = 9,5$

$\Delta_D \text{ max} = 40 \text{ mm}$

$\Delta_L \text{ max} = L/360 = 7200 / 360 = 20 \text{ mm}$

### SUPERIMPOSED LOAD

Live Load = 6 kN/m<sup>2</sup>

Partition = 1 kN/m<sup>2</sup>

Ceiling = 0,75 kN/m<sup>2</sup>

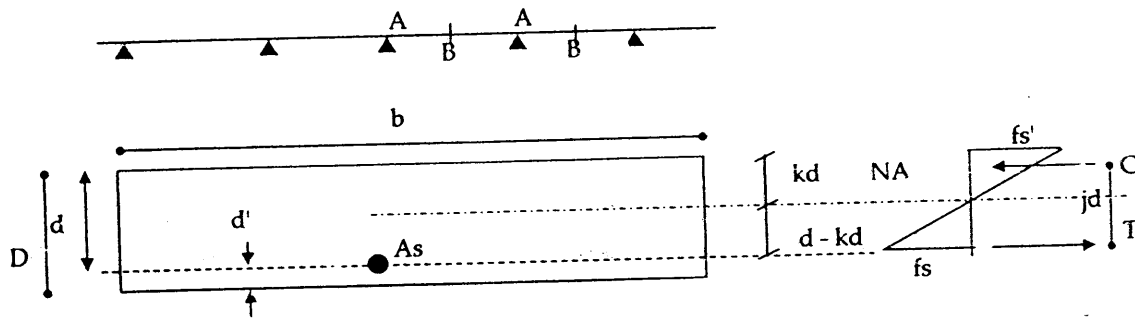
Superimposed = 7,75 kN/m<sup>2</sup>

### DEAD LOAD

Slab =  $0,12 \times 23,5 = 2,82 \text{ kN/m}^2$

Bondek t = 1 mm = 0,1 kN/m<sup>2</sup>

2,92 kN/m<sup>2</sup>



$fs'$  = "steel" stress at level of top fibre  
 $fc$  = concrete stress, top fibre =  $fs'/n$   
 $fs = (d-y)/y * fs' =$  steel stress at depth "d"  
 $kd$  = depth to elastic NA (Neutral Axis)  
 $n = Es/Ec$   
 $As$  = Bondek Area  
 $C$  = total concrete compression force  
 $T$  = total steel tension force

#### BONDEK properties

t bondek 0,75 mm

(from table 1)

Znt = 10,8 cm <sup>3</sup> Top Sheeting Modulus	elastic modulus, top of rib, negative moment regions (tensile stress)
Znb = 11,16 cm <sup>3</sup> Bottom Sheeting Modulus	elastic modulus, bottom pan, negative moment regions (compressive stress)
Zpt = 13,6 cm <sup>3</sup> Top Sheeting Modulus	elastic modulus, top of rib, positive moment regions (compressive stress)
Zpb = 52,6 cm <sup>3</sup> Bottom Sheeting Modulus	elastic modulus, bottom pan, positive moment regions (tensile stress)
Vmax = 19,74 kN	
Ip = 52,6 cm <sup>4</sup>	
In = 29,8 cm <sup>4</sup>	
Ie = 42,1 cm <sup>4</sup>	
Ab = 1240 mm <sup>2</sup>	
weight = 0,1 kN/m'	

#### Bondek with Slab

D = 120 mm

Code = 075120

Code of Bondex Sheet Section Properties

n = 9,5

(from table 2)

d' = 15,5 mm ;	d = D - d' = 120 - 15,5 = 104,5 mm
Mass = 275 kg/m <sup>2</sup>	
rib = 53 mm	rib height
kd = 39,4 mm	
ysb = 80,6 mm	(D - kd)
yst = 27,6	(D - kd - rib height)
Ic = 797 cm <sup>4</sup>	
Zc = 202,3 cm <sup>3</sup>	section modulus for concrete
Zb = 98,9 cm <sup>3</sup>	bottom section modulus for sheeting
Zt = 288,8 cm <sup>3</sup>	top section modulus for sheeting
Vmax = 13,97 kN	

$= Ic / kd$   
 $= Ic / ysb$   
 $= Ic / yst$



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#### Construction Load

$$\begin{aligned}
 \text{Mass} &= 275 \times 9.81 / 1000 = 2,7 \text{ kPa} \\
 \text{Uniform Construction Load} &= 1,9 \text{ kPa} \\
 \text{Total Const. Load} &= 4,6 \text{ kPa} \\
 \text{Total Const. Load per m'} &= 1 \text{ m} \\
 w &= 4,6 \times 1 = 4,6 \text{ kN/m width}
 \end{aligned}$$

$$\begin{aligned}
 \text{Max Negative Moment at A} &= -0.10 wL^2 = -0.10 \times 4,6 \times 3^2 = -4,14 \text{ kNm} \\
 \text{Max Positive Moment at B} &= +0.080 wL^2 = +0.08 \times 4,6 \times 3^2 = 3,312 \text{ kNm} \\
 \text{Max Reaction, internal span} &= 1.1 wL = 1.1 \times 4,6 \times 3 = 15,18 \text{ kN}
 \end{aligned}$$

#### Checking Sheeting under Construction Load

Negative Moment at A

$$\begin{aligned}
 \text{Top Sheeting Stress} \\
 f_{nt} &= 4140000 / 10800 = 383,33333 \text{ Mpa} > 300 \text{ Mpa (not OK)}
 \end{aligned}$$

$$\begin{aligned}
 \text{Bottom Sheeting Stress} \\
 f_{nb} &= 4140000 / 11160 = 370,96774 \text{ Mpa} > 300 \text{ Mpa (not OK)}
 \end{aligned}$$

Positive Momen at B

$$\begin{aligned}
 \text{Top Sheeting Stress} \\
 f_{pt} &= 3312000 / 13600 = 243,52941 \text{ Mpa} < 300 \text{ Mpa (OK)}
 \end{aligned}$$

$$\begin{aligned}
 \text{Bottom Sheeting Stress} \\
 f_{pb} &= 3312000 / 52600 = 62,965779 \text{ Mpa} < 300 \text{ Mpa (OK)}
 \end{aligned}$$

$$\begin{aligned}
 \text{Shear} \\
 V &= 15,18 \text{ kN} < 19,74 \text{ kN (OK)}
 \end{aligned}$$

#### Sheeting under Wet Concrete load only

$$\begin{aligned}
 \text{Mass} &= 275 \times 9.81 / 1000 = 2,7 \text{ kPa} \\
 w &= 2,7 \text{ kN/m width}
 \end{aligned}$$

$$\begin{aligned}
 \text{Max Negative Moment at A} &= -0.10 wL^2 = -0.10 \times 2,7 \times 3^2 = -2,43 \text{ kNm} \\
 \text{Max Positive Moment at B} &= +0.080 wL^2 = +0.08 \times 2,7 \times 3^2 = 1,944 \text{ kNm} \\
 \text{Max Reaction, internal span} &= 1.1 wL = 1.1 \times 2,7 \times 3 = 8,91 \text{ kN}
 \end{aligned}$$

#### Checking Sheeting under Wet Concrete ONLY

Positive Moment at B

$$\begin{aligned}
 \text{Top Sheeting Stress} \\
 f_{pt} &= 1944000 / 13600 = 142,94118 \text{ Mpa} < 300 \text{ Mpa (OK)} <<< \text{compression}
 \end{aligned}$$

$$\begin{aligned}
 \text{Bottom Sheeting Stress} \\
 f_{pb} &= 1944000 / 52600 = 36,958175 \text{ Mpa} < 300 \text{ Mpa (OK)} <<< \text{tension}
 \end{aligned}$$

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Maximum panel deflection under wet concrete,

$$\Delta_{max} = L / 270 = 11,11 \text{ mm}$$

$$\Delta_c = \frac{(0.0069 \text{ wc } L^4) / (E I_e)}{(0.0069 \times 2,7 \times 3000^4) / (200000 \times 421000)}$$

$$17,9219715 \text{ mm} > 11,11 \text{ mm (not OK)} \rightarrow \text{should be propped 3 pcs}$$

$$4,480492874 \text{ mm} < 11,11 \text{ mm (OK)} \rightarrow \text{save to be propped 3 pcs}$$

### Superimposed LOAD

Superimposed LOAD 7,75 kN/m

$$\begin{aligned} \text{Max Negative Moment at A} &= -0.10 \text{ wL}^2 = -0.10 \times 7,75 \times 3^2 = -6,975 \text{ kNm} \\ \text{Max Positive Moment at B} &= +0.091 \text{ wL}^2 = +0.091 \times 7,75 \times 3^2 = 6,34725 \text{ kNm} \\ \text{Max Reaction, internal span} &= 1.1 \text{ wL} = 1.1 \times 7,75 \times 3 = 25,575 \text{ kN} \end{aligned}$$

### Bond at C

$$V = 25,575 \text{ kN} > 13,97 \text{ kN (not OK)}$$

### Negative Reinforcement at A

$$\begin{aligned} \text{Design moment at A} &= 6,975 \text{ kNm} \\ \text{Mu} &= 1.8 \times 6,975 = 12,555 \text{ kNm} \end{aligned}$$

Use ultimate strength design procedure for Slab

$$\begin{aligned} F_c' &= 20 \text{ Mpa} \\ f_{sy} &= 500 \text{ Mpa (slab reinforcing)} \\ b &= 1000 \text{ mm} \\ h &= 120 \text{ mm} \\ A_t &= 265 \text{ mm}^2 \\ A_{min} &= 240 \text{ mm}^2 \end{aligned}$$

Using wiremesh D13 - 150 885 mm<sup>2</sup>

### Stresses at B

Top Sheeting Stress

$$f_t = 6347250 / 288800 = 21,978012 \text{ Mpa}$$

Bottom Sheeting Stress

$$f_b = 6347250 / 98900 = 64,178463 \text{ Mpa}$$

### Summary Stress at B

Condition	Bottom	Top
Wet concrete	37	-142,9
Superimposed	64,2	22
Total	101,2	-120,9
Allowable	250	250

$$= 0.50 \times 500$$

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#### Concrete Stress at B

$$\text{Allowable Stress} = 0.45 \times f_c' = 0.45 \times 20 = 9 \text{ Mpa}$$

$$6347250 / (202300 \times 9.5) = 3.3 \text{ Mpa} < 9 \text{ Mpa (OK)}$$

#### DEFLECTION UNDER SUPERIMPOSED LOAD

a)  $\Delta_{\max} = L / 480 = 3000 / 480 = 6.25 \text{ mm}$

$$\Delta_{\text{short}} = (0.0069)(wL^4) / (E I_{\text{crack}}) =$$

$$(0.0069 \times 7.75 \times 3000^4) / (200000 \times 7970000)$$

$$2.717361982 \text{ mm} < 6.25 \text{ mm (OK)}$$

b)  $\Delta_1 + \Delta_2 \leq (L / 500)$

$$L / 500 = 3000 / 500 = 6 \text{ mm}$$

Assume

$$\text{Superimposed load (short term)} = 3 \text{ kPa} \quad (= 50\% \text{ Live load})$$

$$\text{Superimposed load (long term)} = 7.75 \text{ kPa}$$

Effective second moment of area

$$I_e = (M_c / M)^3 (I_g - I_{\text{crack}}) + I_{\text{crack}}$$

$$I_{\text{crack}} = 7970000 \text{ mm}^4$$

$$I_g = (bD^3) / (12 n) + I_s =$$

$$I_s = 526000 \text{ mm}^4$$

$$I_g = (1000 \times 120^3) / (12 \times 9.5) + 526000$$

$$15683894.74 \text{ mm}^4$$

$$Z_g = I_g / (D/2) =$$

$$261398.2456 \text{ mm}^3$$

$$M_c = 0.625 n \sqrt{f_c'} Z_g = 0.625 \times 9.5 \sqrt{20} \times 261398 =$$

$$6940981.404 \text{ Nmm}$$

$$6.94 \text{ kNm} > 6.35 \text{ kNm}$$

$$M_c > M_{\text{slab}} = \text{slab should not crack; hence } I_e = I_g$$

Short time deflection

$$\Delta_1 = (0.0069)(wL^4) / (E I_e) =$$

$$(0.0069 \times 3 \times 3000^4) / (200000 \times 15683895)$$

$$0.534529219 \text{ mm}$$

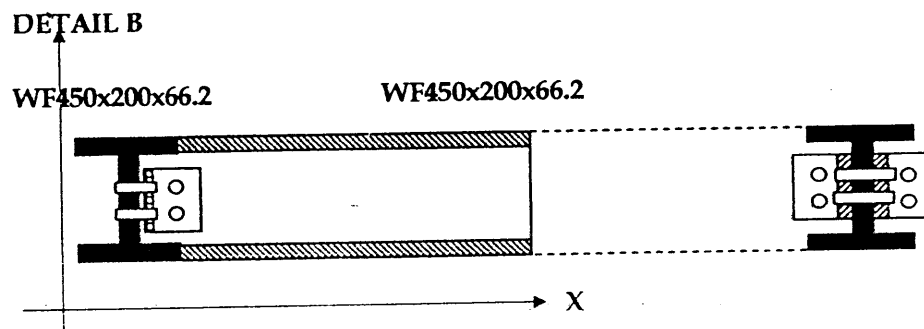
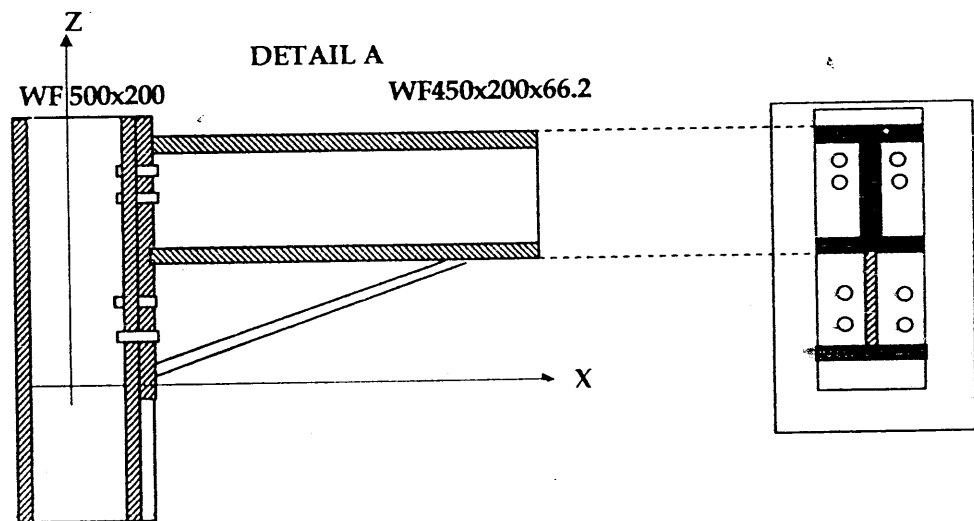
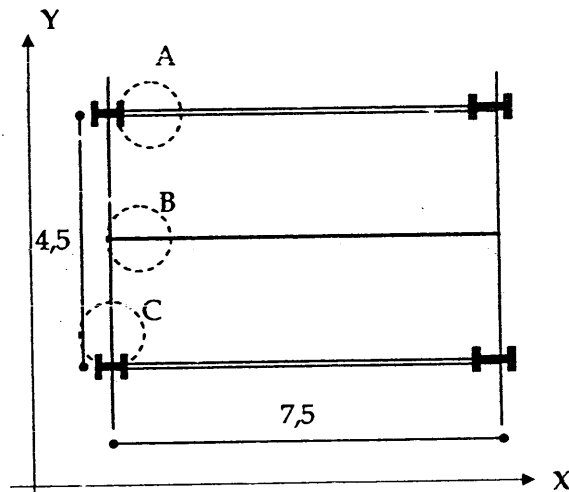
$$\Delta_2 = (0.0069 \times 7.75 \times 3000^4) / (200000 \times 15683895)$$

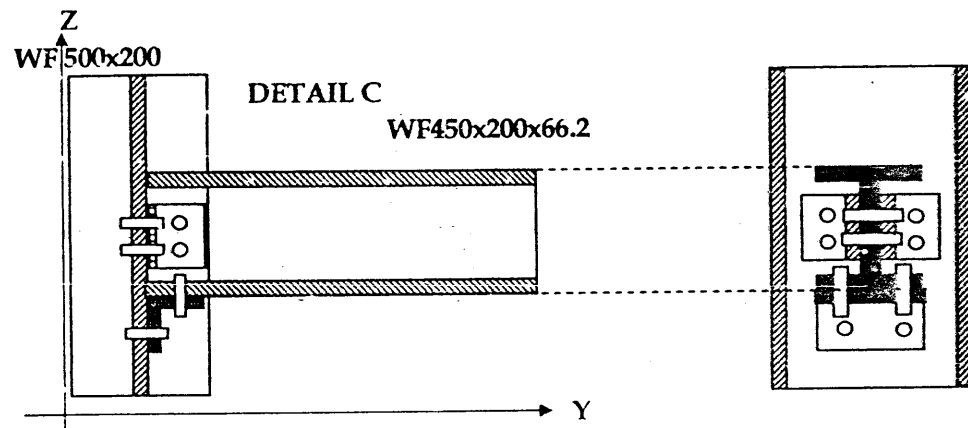
$$1.380867148 \text{ mm}$$

$$\Delta_1 + \Delta_2 = 0.53 + 1.38 = 1.92 \text{ mm} < 6 \text{ mm. (OK)}$$

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## CONNECTION DESIGN





DETAIL A	$V_x =$	130,378 kN
DETAIL B	$V_x =$	130,378 kN
DETAIL C	$V_y =$	118,653 kN

I) DETAIL A (see drawing above)

Connection to web of WF450x200x66.2 (beam)

Use A36 steel and A325 bolts

$t_w = 8$  mm

$F_u = 1.6 F_y = 1.6 \times 240 = 384$  Mpa

$F_{yb} = 200$  Mpa < Bolt A325-X >

$V_A = 130,378$  kN

$R_B = 1.5 F_u D t =$  D = bolt diameter

$1.5(384)(D)(8) = 4608 D$  (N) =  $4,608 D$  (kN)

$R_{DS} = 2(F_{yb})A_b =$   $A_b =$  Area of Bolt

$2(200)(A_b) = 400 A_b$  (N) =  $0,4 A_b$  (kN)

Bolt Diam	RB	RDS	R	n = V/R	
				n = 130,378 / R	
12	55,296	45,238934	45,238934	2,88199	4
16	73,728	80,424772	73,728	1,76836	2
19	87,552	113,41149	87,552	1,48915	2
22	101,376	152,05308	101,376	1,28608	2

Use 2 D 16

Eccentricty is negleted, (check by AISC 1.16.5.3)

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$$\text{End distance} \geq (2 V/n) / (F_u t)$$

$$(2)(130378/2)/(384 \times 8) = 42 \text{ mm}$$

$$\text{Required gage for WF450x200x66.2} \quad \text{web} = 42+16 = 58 \text{ mm}$$

$$\text{say } 60 \text{ mm}$$

$$\text{this mean a } 60+2 \times 25 = 110 \text{ mm} \quad \text{in leg}$$

b) Connection to flange of WF500x200x103 (column)

$$t_f = 19 \text{ mm}$$

$$\text{for D 16 bolts } A = 201 \text{ mm}^2$$

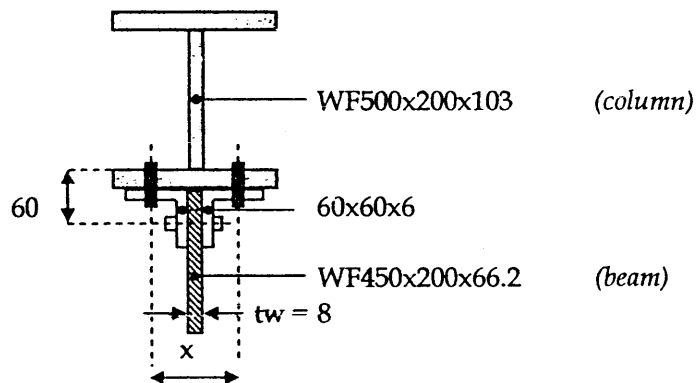
$$R_B = 1.5(384)(16)(19) = 175104 \text{ N} = 175,104 \text{ kN}$$

$$R_{SS} = (200)(201) = 40200 \text{ N} = 40,2 \text{ kN}$$

$$R = \min(R_B \text{ and } R_{SS}) = 40,2 \text{ kN}$$

$$n = V/R = 130,378 / 40,2 = 3,2432338 \quad \text{using } 4 \text{ bolts}$$

c) Outstanding legs connecting to WF500x200x103 (column)

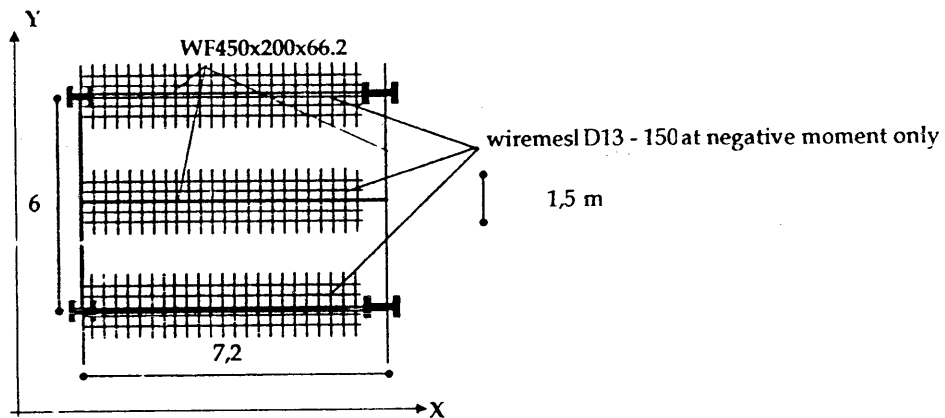


$$\text{Angle thickness} = (V/n) / [(1.5)(F_u)(D)] = (130378/4) / [(1.5)(384)(16)] = 3,53673 \text{ mm}$$

$$\text{use } 6 \text{ mm}$$

$$\text{use } 2 \text{ L } 60 \times 60 \times 6 \times 110 \text{ mm}$$

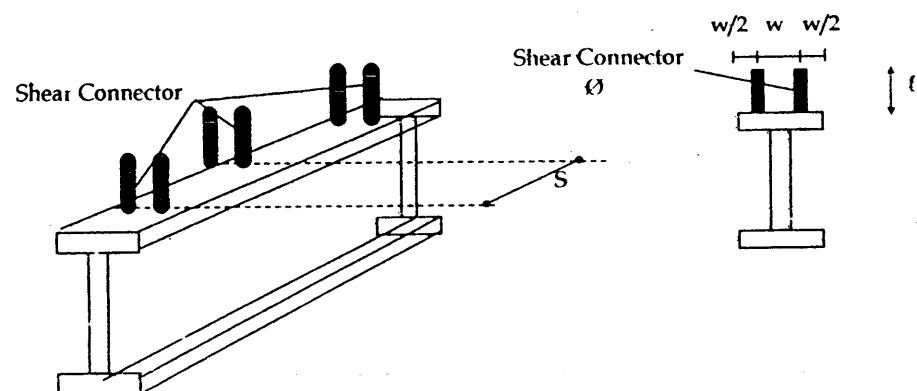
# OVERTOPPING REINFORCEMENT



GRADE Steel Beam A-36 WF450x200x66.2  
 Bondek sheeting thickness = 0,75 mm  
 t slab = 120 mm  
 $f_c' = 20$  Mpa

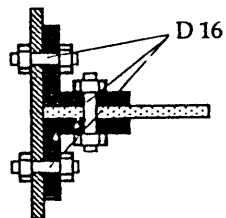
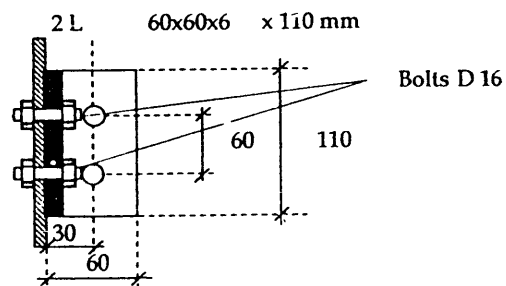
Reinforcement for negative moment only  
 wiremesh = D13 - 150  
 $f_y = 500$  Mpa

## Shear Connector at Steel Beam



Shear Connector	X-Axis		Y-Axis		units
	Ø	16 - 65	16 - 65	16 - 65	
	Ø	16	16	16	
	ℓ	65	65	65	
per section		2	2	2	
section space (w)		99,5	99,5	99,5	mm
longitudinal Spacing (S)		240	417	417	mm

### CONNECTION





# BONDEK SHEETING in COMPOSITE SLAB

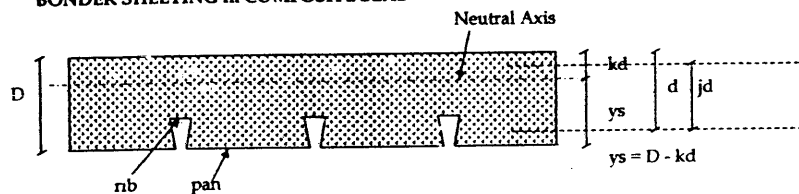


TABLE 1 : BONDEK SHEETING SECTION PROPERTIES (1000 mm WIDTH)

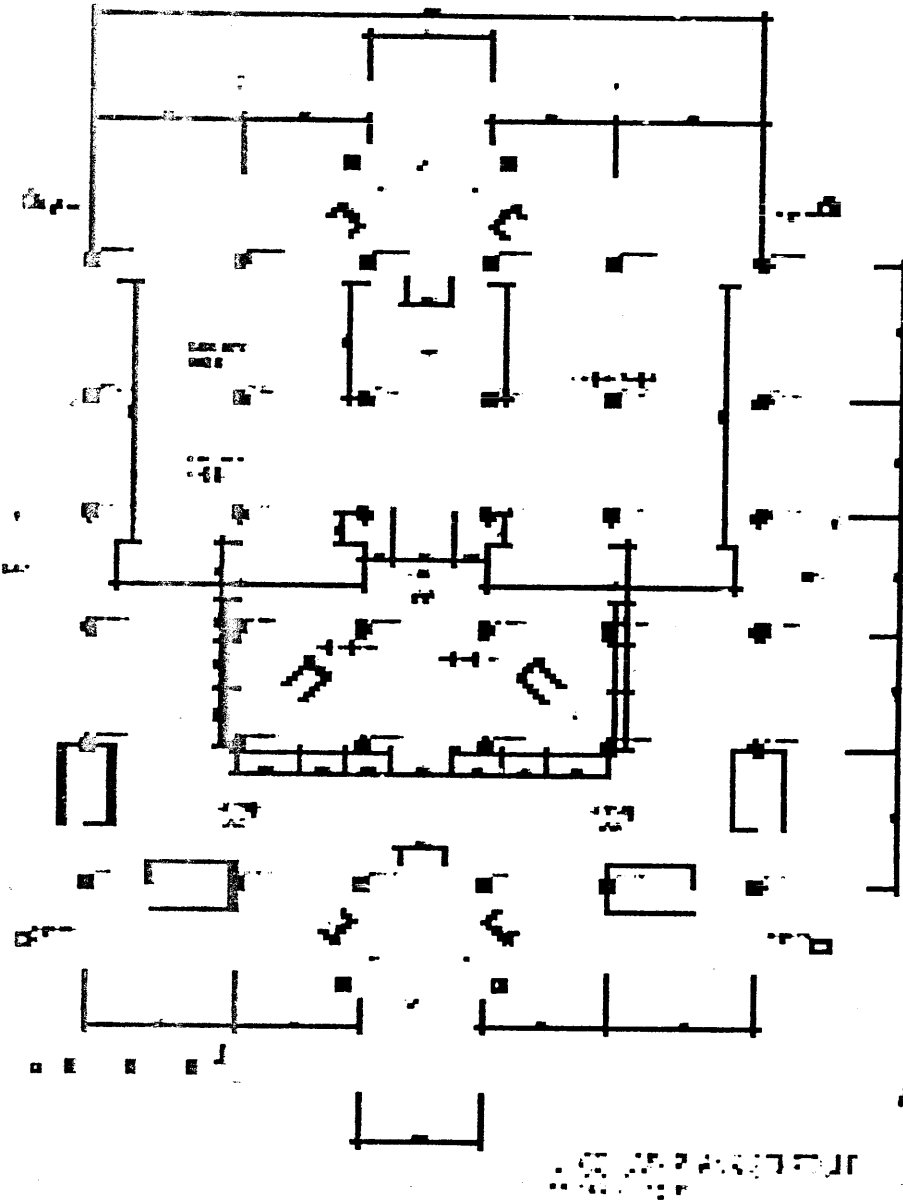
Steel thickness mm	Weight per unit area kPa	Cross section Area mm <sup>2</sup>	Distance to sheeting centroid from		Positive B.M Properties of sheeting			Negative B.M Properties of sheeting			Ie	Max Shear kN
			top	bottom	Ip	Zpt	Zpb	In	Znt	Znb		
mm	kPa	mm <sup>2</sup>	mm	mm	cm <sup>4</sup>	cm <sup>3</sup>	cm <sup>3</sup>	cm <sup>4</sup>	cm <sup>3</sup>	cm <sup>3</sup>	cm <sup>4</sup>	cm <sup>4</sup>
0,75	0,1	1240	38,7	15,5	52,6	13,6	33,92	29,8	10,8	11,16	42,1	19,74
1	0,13	1650	38,8	15,6	68,6	17,63	43,86	47,4	16,29	15	60,7	28,53

Ip = second moment area of sheeting profile for positive (sagging moment) regions  
 In = second moment area of sheeting profile for negative (hogging moment) regions  
 Zpt = elastic modulus, top of rib, positive moment regions (compressive stress)  
 Zpb = elastic modulus, bottom pan, positive moment regions (tensile stress)  
 Znt = elastic modulus, top of rib, negative moment regions (tensile stress)  
 Znb = elastic modulus, bottom pan, negative moment regions (compressive stress)  
 Ie = equivalent second moment of area for multiple span deflection calculations  
 Ip (1.26 - 0.26 Ip/In)

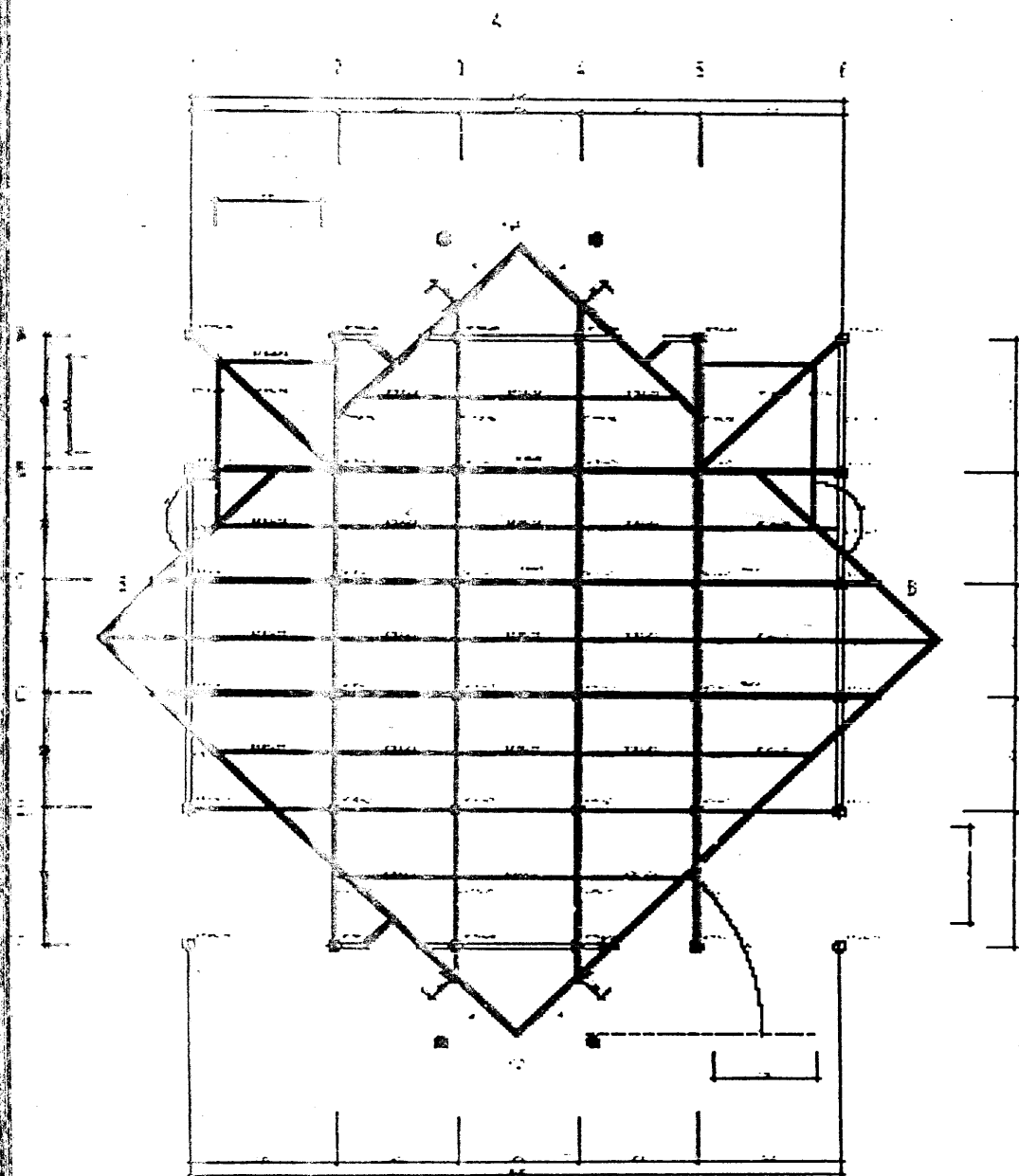
TABLE 2 : COMPOSITE "BONDEK" SLAB SECTION PROPERTIES per 1000 mm WIDTH (n = 9.5)

CODE	Steel thickness mm	Slab depth D mm	Slab Mass kg/m <sup>2</sup>	Effective Depth d mm	kd	ys D - kd	Ic	Zc	Zs (bottom)	jd	Bond Values = 0,27 Mpa			
											perimeter Σ O	bond c.g yb	lb	Max Shear kN
075090	0,75	90	206	74,5	31,8	58,2	3,9	123,7	67,6	63,9	691	22	47,4	8,84
075100		100	229	84,5	34,6	65,4	5,1	147,2	77,9	73,0	691	22	56,5	10,54
075110		110	252	94,5	36,9	73,1	6,4	174,4	88,0	82,2	691	22	65,7	12,26
075120		120	275	104,5	39,4	80,6	8,0	202,3	98,9	91,4	691	22	74,9	13,97
075130		130	299	114,5	41,7	88,3	9,7	232,5	109,8	100,6	691	22	84,1	15,69
075140		140	322	124,5	43,8	96,2	11,6	265,2	120,7	109,9	691	22	93,4	17,43
075150		150	345	134,5	45,9	104,1	13,7	299,2	131,9	119,2	691	22	102,7	19,16
100090	1,00	90	209	74,4	35,0	55,0	4,7	134,9	85,8	62,7	694	22	46,3	8,68
100100		100	232	84,4	37,9	62,1	6,1	161,5	98,6	71,8	694	22	55,4	10,38
100110		110	255	94,4	40,8	69,2	7,8	190,0	112,0	80,8	694	22	64,4	12,07
100120		120	278	104,4	43,5	76,5	9,6	221,1	125,7	89,9	694	22	73,5	13,77
100130		130	302	114,4	46,1	83,9	11,7	254,4	139,8	99,0	694	22	82,6	15,48
100140		140	325	124,4	48,5	91,5	14,1	290,3	153,9	108,2	694	22	91,8	17,20
100150		150	348	134,4	50,9	99,1	16,7	327,7	168,3	117,4	694	22	101,0	18,93
100160		160	371	144,4	53,1	106,9	19,5	367,7	182,7	126,7	694	22	110,3	20,67
100170		170	395	154,4	55,4	114,6	22,6	408,4	197,4	135,9	694	22	119,5	22,39
100180		180	418	164,4	57,5	122,5	26,0	451,9	212,1	145,2	694	22	128,8	24,13
100190		190	441	174,4	59,6	130,4	29,6	496,6	227,0	154,5	694	22	138,1	25,88
100200		200	464	184,4	61,7	138,3	33,5	542,4	242,0	163,8	694	22	147,4	27,62

Based on the Structural Analysis and dimensioning, founded the following structural element of each storeys :

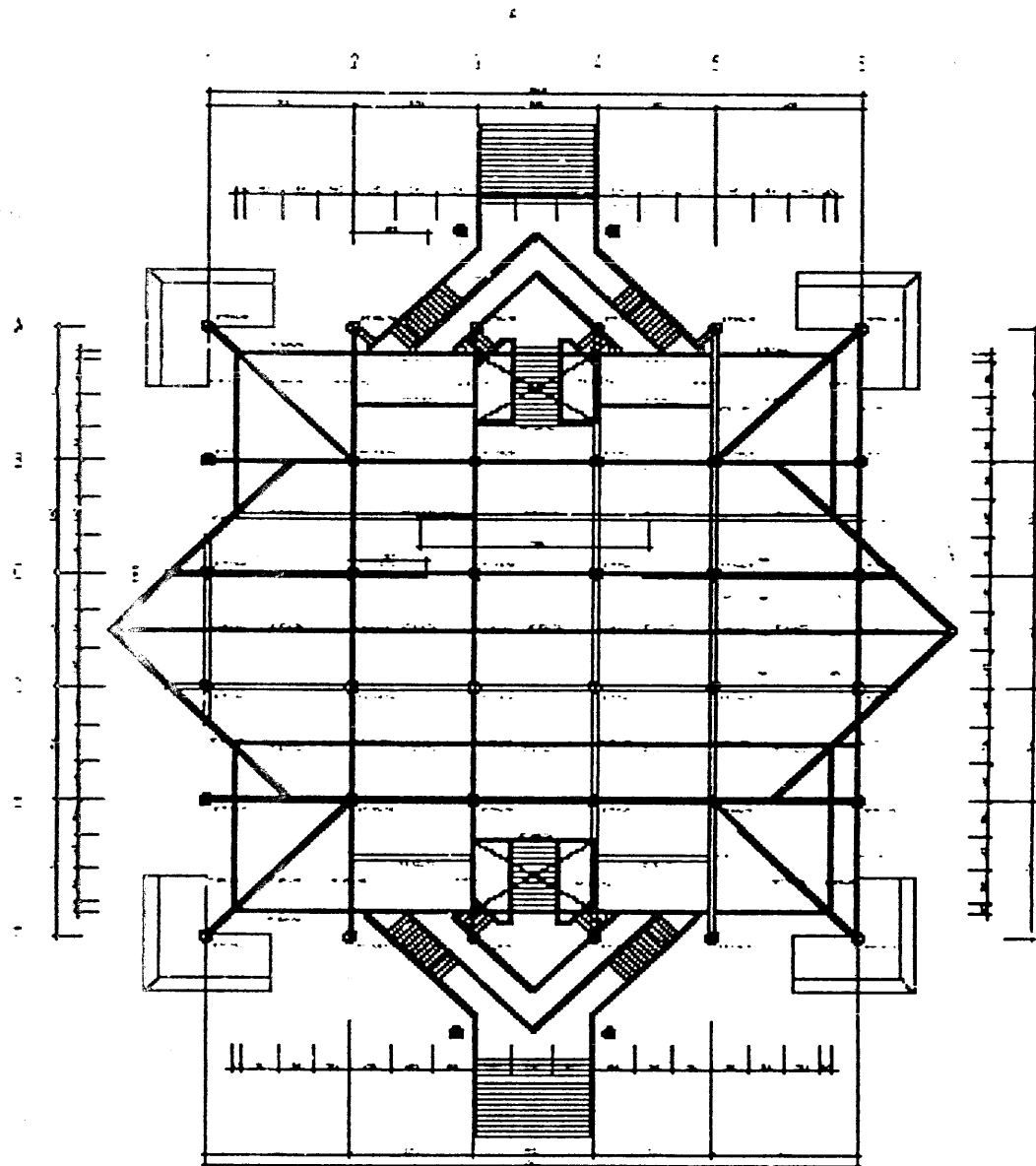


Initial Primary Structural Analysis and dimensioning, founded the following structural element of each storeys :



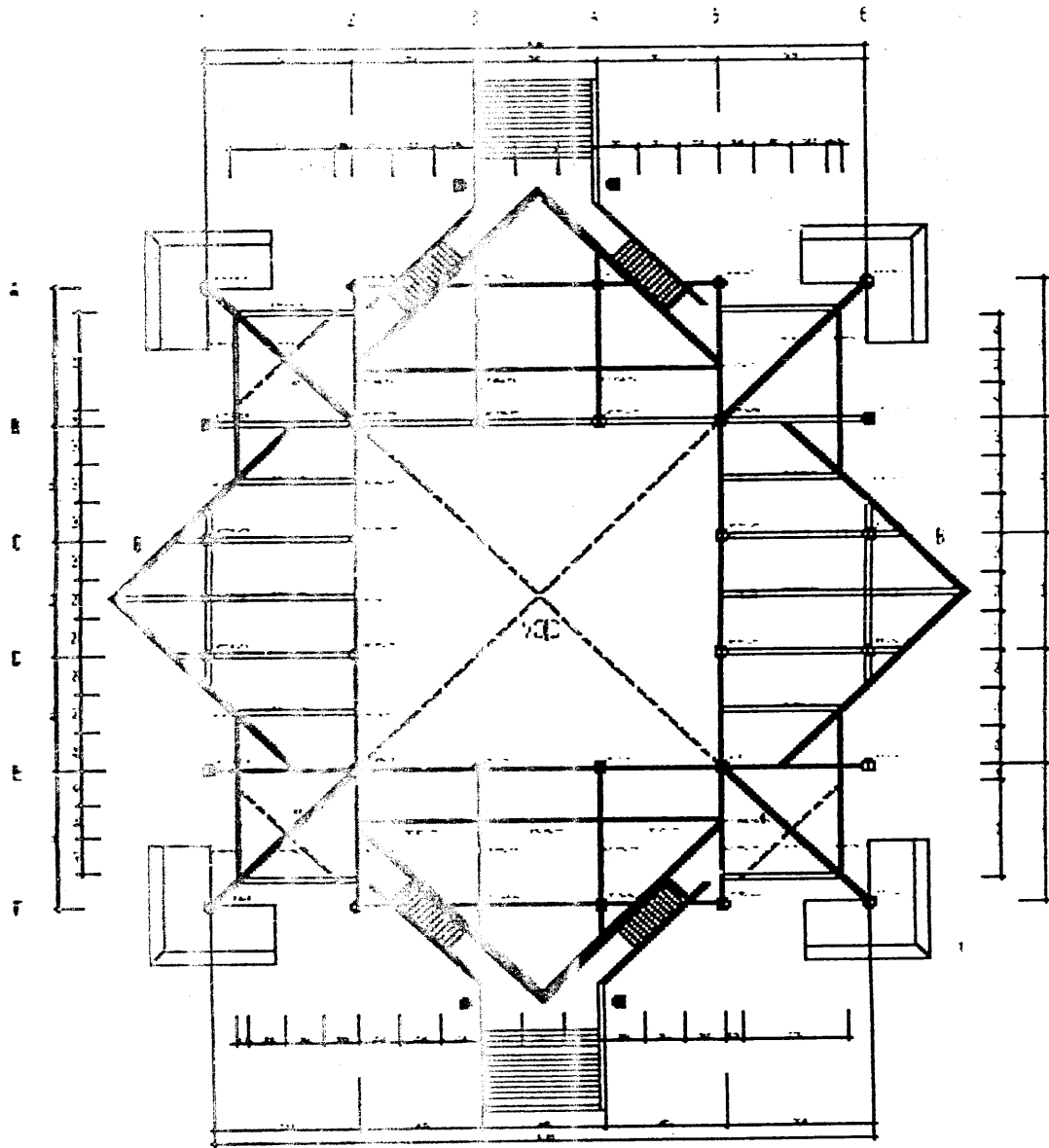
2nd FLOOR BEAM-COLUMN PLAN & SC-SCHEDULE  
 SCALE: 1/16" = 1'-0"

From Preliminary Structural Analysis and dimensioning, founded the following structural element of each storeys :



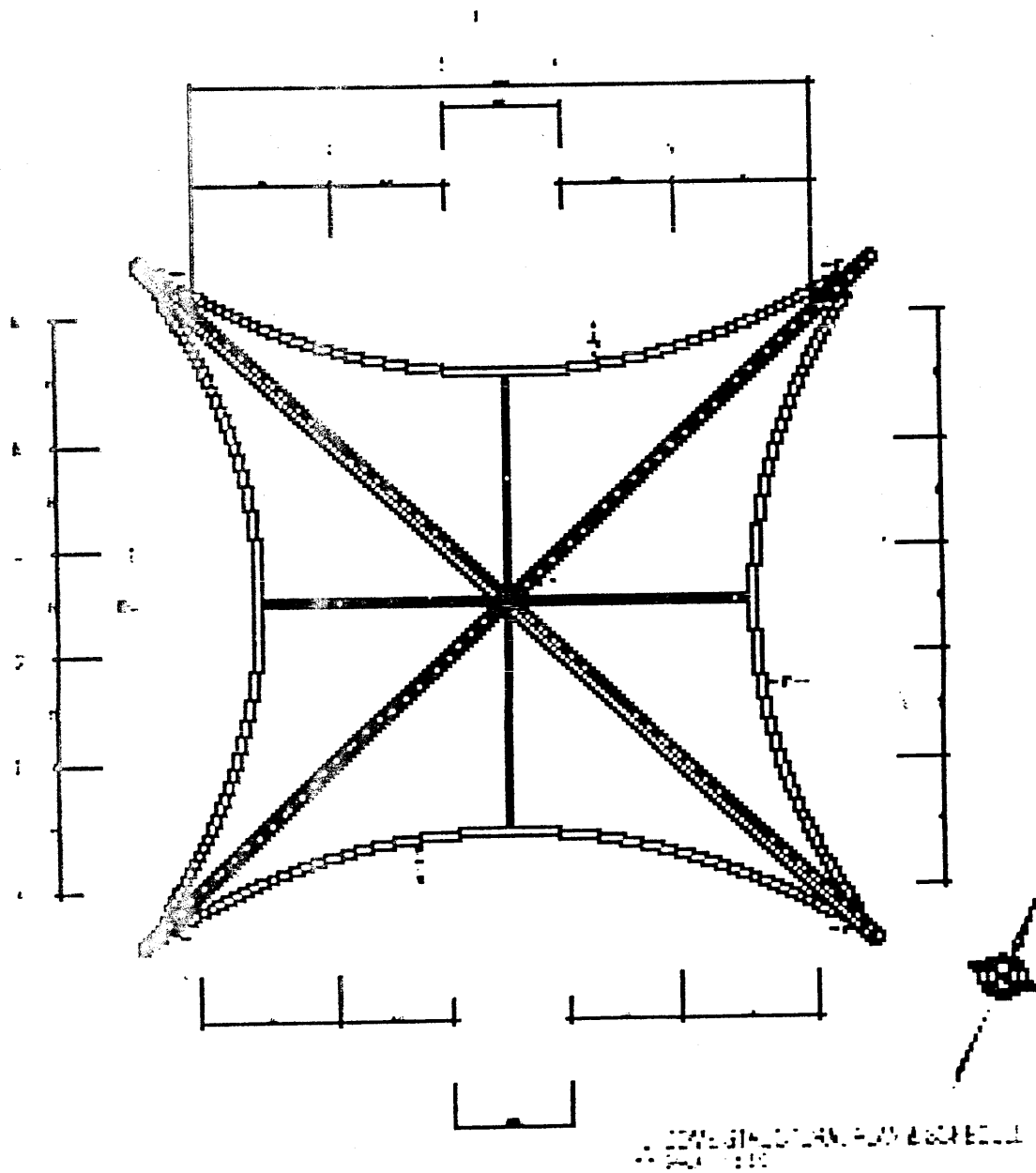
2ND FLOOR BEAM & COLUMN PLAN & SCHEDULE  
 29.4.2020

From Preliminary Structural Analysis and dimensioning, founded the following structural element of each storeys :

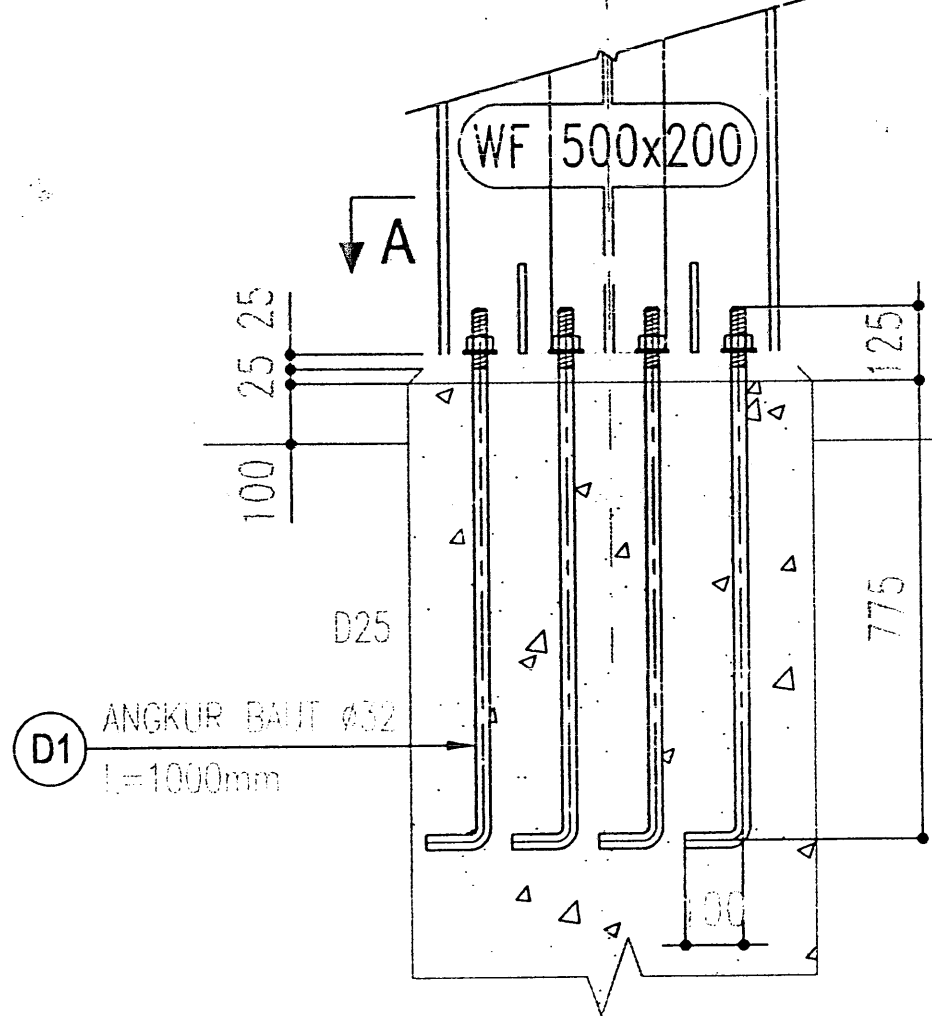


4th FLOOR BEAM-COLUMN PLAN & SCHEDULE

From Preliminary Structural Analysis and dimensioning, founded the following structural element of each storeys :

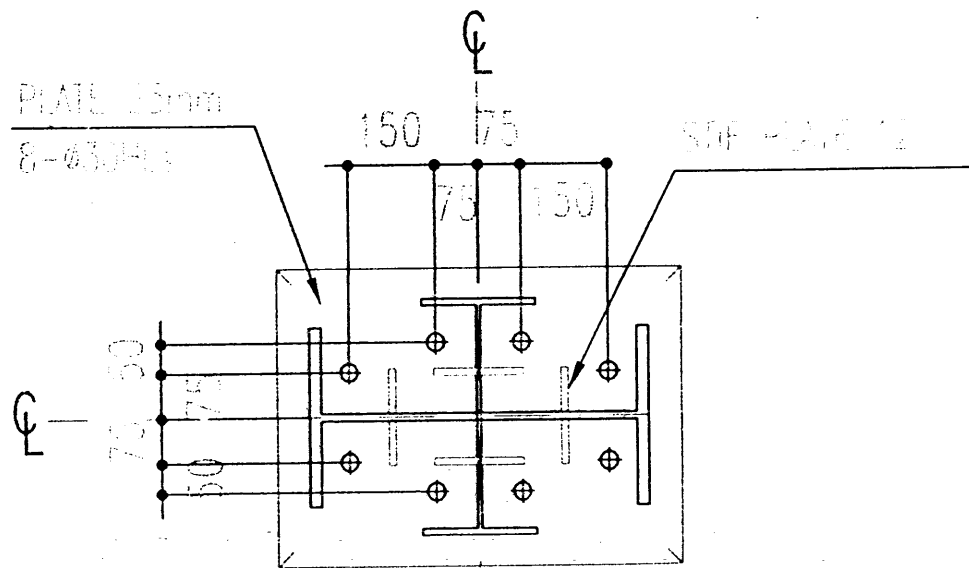


**LAMPIRAN**



## COLUMN ANCHORAGE

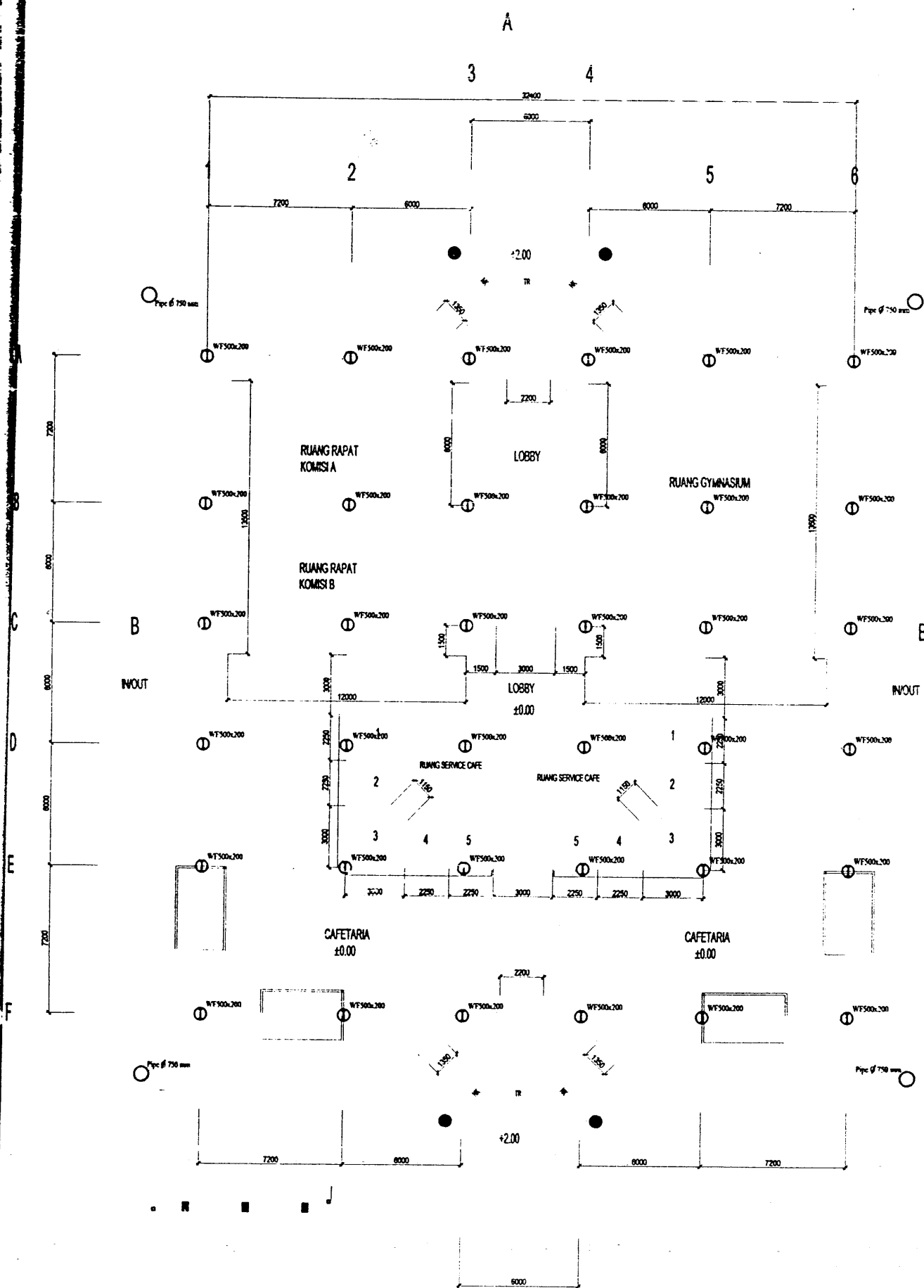
SCALE 1:150

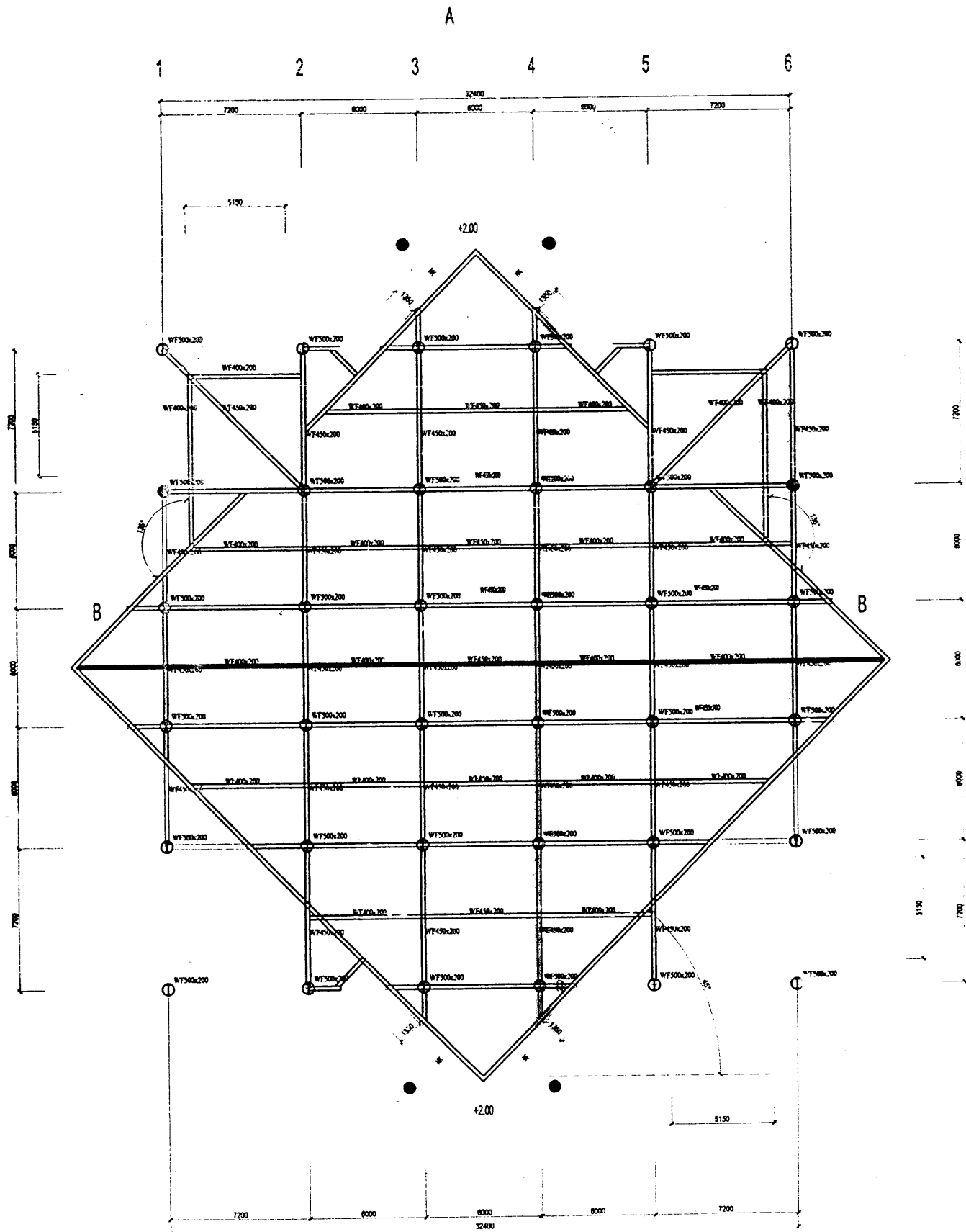


## SECTION - A

SCALE 1:150

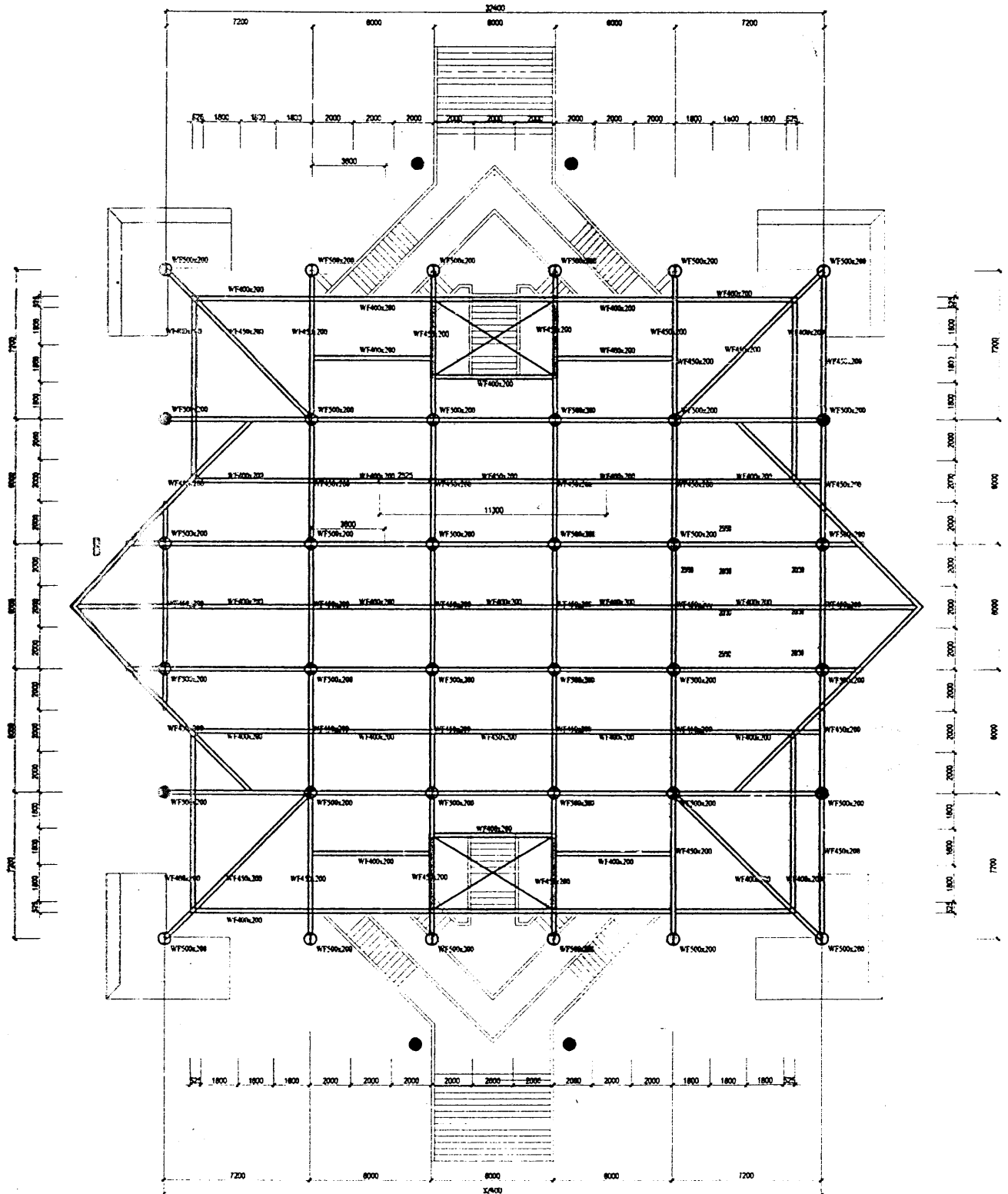




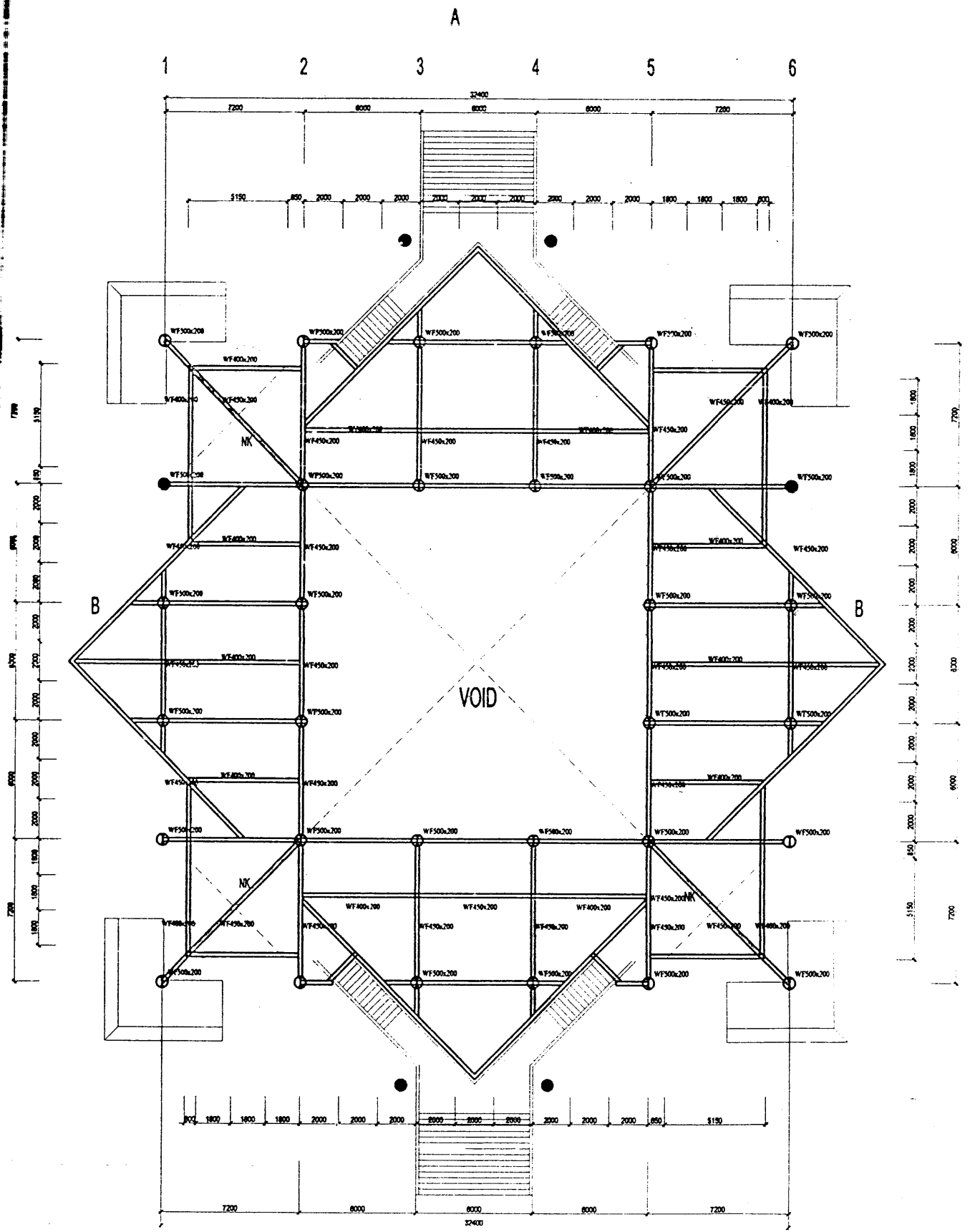


2nd FLOOR BEAM-COLUMN PLAN & SCHEDULE  
SCALE 1 : 300

1	2	3	4	5	6
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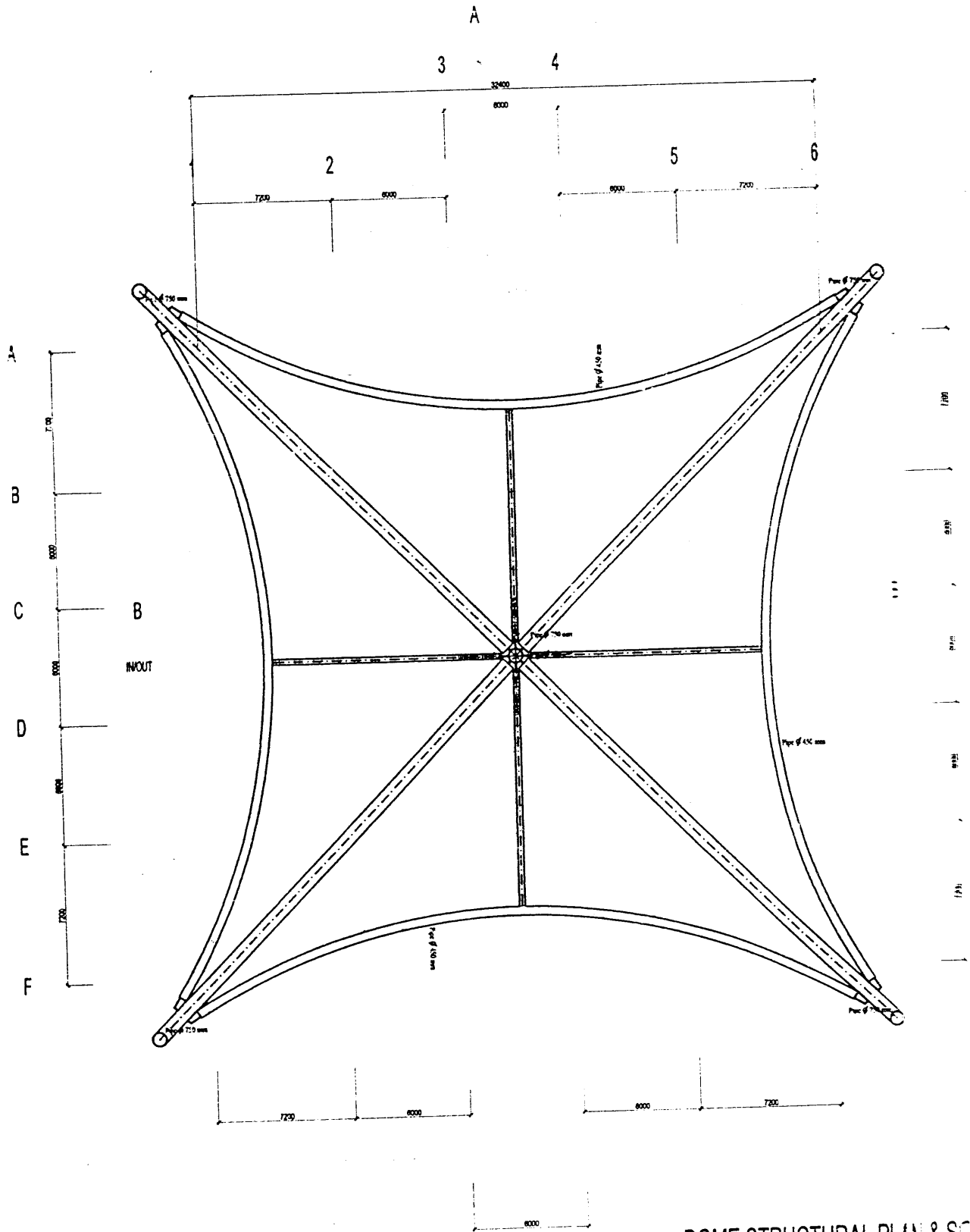


A



4th FLOOR BEAM-COLUMN PLAN & SCHEDULE  
SKALA 1 : 300

A



DOME STRUCTURAL PLAN & SCHEDULE  
SKALA 1 : 300

#### A. PERHITUNGAN GORDING :

Rencana profil C 125 . 50 . 20 . 3,2

$$F = 7,807 \text{ cm}^2$$

$$g = 6,31 \text{ kg/m'}$$

$$I_x = 181 \text{ cm}^4$$

$$I_y = 26,6 \text{ cm}^4$$

$$W_x = 29 \text{ cm}^3$$

$$W_y = 8,02 \text{ cm}^3$$

Jarak gording : 1,50 m

Pembebanan :

a. Beban mati :

$$\text{- Berat gording} = 7,807 \text{ kg/m'}$$

$$\text{- Berat atap galvalume } 5 \times 1,50 = 7,50 \text{ kg/m'}$$

$$q_m = 15,307 \text{ kg/m'}$$

$$M_x = 1/8 \times 15,307 \times \cos 10^\circ \times 6^2$$

$$= 67,835 \text{ kg m}$$

$$M_y = 1/8 \times 15,307 \times \sin 10^\circ \times (6/3)^2$$

$$= 1,329 \text{ kg m}$$

b. Beban hidup :

Diperhitungkan seorang pekerja (p) = 100 kg bekerja di tengah-tengah bentang :

$$M_x = 1/4 \times 100 \times \cos 10^\circ \times 6$$

$$= 147,72 \text{ kg m}$$

$$M_y = 1/4 \times 100 \sin 10^\circ \times 6/3$$

$$= 8,68 \text{ kg m}$$

c. Beban angin ( qw ) :

Koefisien angin :

$$C_1 = + 0,02 \alpha = 0,4$$

$$\text{Atau } C_1 = 0,2 \dots \dots \dots (\text{angin tiup})$$

$$C_2 = - 0,4 \dots \dots \dots (\text{angin isap})$$

Tekanan angin ( W ) di perhitungkan = 40 kg/m<sup>2</sup>

Tekanan angin yang bekerja pada gording adalah angin isap

$$q_w = -0,4 \times 40 \times 1,50 = 24 \text{ kg/m'}$$

momen yang terjadi :

$$M_x = -1/8 \times 24 \times 6,00^2 = -108 \text{ kg m}$$

$$M_y = 0$$

Kombinasi beban :

$$I = a + b$$

$$II = a + c$$

$$III = b + c$$

Kombinasi I yang menentukan

d. Beban kombinasi :

$$M_x = 67,835 + 147,72 = 215,555 \text{ kg m}$$

$$M_y = 1,329 + 8,68 = 10,009 \text{ kg m}$$

\* Kontrol tegangan :

$$\sigma_a = \frac{215,555}{29} + \frac{10,009}{8,02}$$

$$= 743,29 + 124,80 = 868,09 \text{ kg/cm}^2 < \sigma_a = 1600 \text{ kg/cm}^2$$

..... Okey.

\* Kontrol lendutan :

$$f_x = \frac{5}{384} \frac{q \cos \alpha l x^4}{E \cdot I_x} + \frac{1}{48} \frac{p \cos \alpha \cdot l x^3}{E \cdot I_x}$$

$$= \frac{5}{384} \frac{0,15307 \cos 10^\circ \cdot 600^4}{2,1 \times 10^{-6} \times 181} + \frac{1}{48} \frac{100 \cos 10^\circ 600^3}{2,1 \times 10^{-6} \times 181}$$

$$= 0,669 + 1,165 = 1,834 \text{ cm}$$

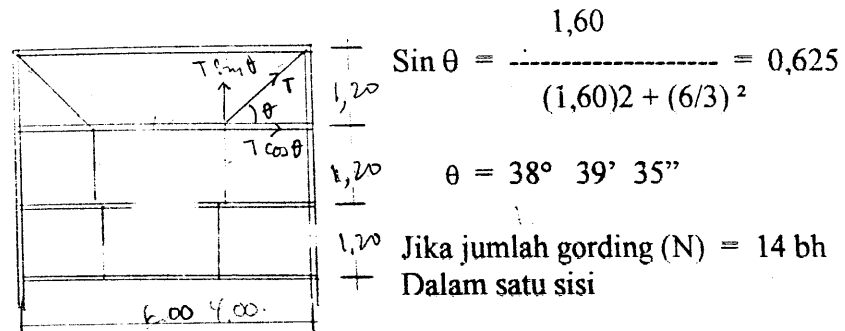
$$f_y = \frac{5}{384} \frac{0,15307 \sin 10^\circ (600/3)^4}{2,1 \times 10^{-6} \times 26,6} + \frac{1}{48} \frac{100 \sin 10^\circ (600/3)^3}{2,1 \times 10^{-6} \times 26,6}$$

$$= 0,0019 + 0,0518 = 0,0537 \text{ cm}$$

$$f = \sqrt{1,834 + 0,0537} = 1,374 \text{ cm}$$

$$\bar{f} = 1/180 L = 1/180 \times 600 = 3,33 \text{ cm} > f = 1,374 \text{ cm}$$

\* Perhitungan penggantung gording :



$$\sin \theta = \frac{1,60}{(1,60)^2 + (6/3)^2} = 0,625$$

$$\theta = 38^\circ 39' 35''$$

Jika jumlah gording (N) = 14 bh  
Dalam satu sisi

Keseimbangan gaya :

$$\Sigma V = 0$$

$$T \sin \theta = q \cdot \sin \alpha \cdot N \cdot L/3$$

$$T = \frac{15,307 \sin 10^\circ \times 14 (6,00/3)}{0,625} = 119,08 \text{ kg}$$

$$A = \frac{T}{\sigma_a} = \frac{119,08}{1600} = 0,074 \text{ cm}^2$$

Dipakai penggantung gording  $\phi 8 = 0,50 \text{ cm}^2 > 0,074 \text{ cm}^2$



## **B. PERHITUNGAN PORTAL BANGUNAN :**

### **I. Data-data profil baja yang dipergunakan :**

\* Kolom Wf . 350 . 175

$$\begin{aligned} g &= 49,6 \text{ kg/m} & ; W_x &= 775 \text{ cm}^3 \\ F &= 63,14 \text{ cm}^2 & ; W_y &= 112 \text{ cm}^3 \\ I_x &= 13.600 \text{ cm}^4 & ; i_x &= 14,7 \text{ cm} \\ I_y &= 984 \text{ cm}^4 & ; i_y &= 3,95 \text{ cm} \end{aligned}$$

\* Balok ikat (tie beam) WF . 200 . 100

$$\begin{aligned} g &= 21,3 \text{ kg/m} & ; W_x &= 184 \text{ cm}^3 \\ F &= 27,16 \text{ cm}^2 & ; W_y &= 26,8 \text{ cm}^3 \\ I_x &= 1.840 \text{ cm}^4 & ; i_x &= 8,24 \text{ cm} \\ I_y &= 134 \text{ cm}^4 & ; i_y &= 2,22 \text{ cm} \end{aligned}$$

\* Balok Wf 400 . 200

$$\begin{aligned} g &= 66 \text{ kg/m} & ; W_x &= 1.190 \text{ cm}^3 \\ F &= 84,12 \text{ cm}^2 & ; W_y &= 174 \text{ cm}^3 \\ I_x &= 23.700 \text{ cm}^4 & ; i_x &= 16,8 \text{ cm} \\ I_y &= 7.210 \text{ cm}^4 & ; i_y &= 4,54 \text{ cm} \end{aligned}$$

\* Balok konsol WF . 175 . 90

$$\begin{aligned} g &= 18,1 \text{ kg/m} & ; W_x &= 139 \text{ cm}^3 \\ F &= 23,04 \text{ cm}^2 & ; W_y &= 21,7 \text{ cm}^3 \\ I_x &= 1.210 \text{ cm}^4 & ; i_x &= 7,26 \text{ cm} \\ I_y &= 97,5 \text{ cm}^4 & ; i_y &= 2,06 \text{ cm} \end{aligned}$$

\* Balok Mezanin Wf . 250 . 125

$$\begin{aligned} g &= 29,6 \text{ kg/m} & ; W_x &= 324 \text{ cm}^3 \\ F &= 37,66 \text{ cm}^2 & ; W_y &= 47 \text{ cm}^3 \\ I_x &= 4.050 \text{ cm}^4 & ; i_x &= 10,4 \text{ cm} \\ I_y &= 294 \text{ cm}^4 & ; i_y &= 2,14 \text{ cm} \end{aligned}$$

\* Balok Casteleted Wf . 360 . 125

pada potongan masif, dari hasil perhitungan di dapat :

$$\begin{aligned} g &= 29,6 \text{ kg/m} & ; W_x &= 515 \text{ cm}^3 \\ F &= 43.02 \text{ cm}^2 & ; W_y &= 49 \text{ cm}^3 \\ I_x &= 8.932 \text{ cm}^4 & ; i_x &= 14,41 \text{ cm} \\ I_y &= 294 \text{ cm}^4 & ; i_y &= 2,60 \text{ cm} \end{aligned}$$

\* Balok Casteleted Wf . 360 . 125

pada potongan berongga, dari hasil perhitungan di dapat :

$$\begin{aligned} g &= 29,6 \text{ kg/m}^3 ; Wx = 503 \text{ cm}^3 \\ F &= 29,82 \text{ cm}^2 ; Wy = 24 \text{ cm}^3 \\ Ix &= 8.399 \text{ cm}^4 ; ix = 16,78 \text{ cm} \\ Iy &= 147 \text{ cm}^4 ; iy = 2,22 \text{ cm} \end{aligned}$$

\* Balok diafragma Wf . 150 . 150

$$\begin{aligned} g &= 31,5 \text{ kg/m}^3 ; Wx = 219 \text{ cm}^3 \\ F &= 40,14 \text{ cm}^2 ; Wy = 75,1 \text{ cm}^3 \\ Ix &= 1.640 \text{ cm}^4 ; ix = 6,39 \text{ cm} \\ Iy &= 563 \text{ cm}^4 ; iy = 3,75 \text{ cm} \end{aligned}$$

\* Balok kukuk Wf . 150 . 75

$$\begin{aligned} g &= 14 \text{ kg/m}^3 ; Wx = 88,8 \text{ cm}^3 \\ F &= 17,85 \text{ cm}^2 ; Wy = 13,2 \text{ cm}^3 \\ Ix &= 666 \text{ cm}^4 ; ix = 6,11 \text{ cm} \\ Iy &= 49,5 \text{ cm}^4 ; iy = 1,66 \text{ cm} \end{aligned}$$

\* Balok Casteleted WF 550 . 175

Pada potongan masif, dari hasil perhitungan di dapat :

$$\begin{aligned} g &= 49,6 \text{ kg/m}^3 ; Wx = 1.370 \text{ cm}^3 \\ F &= 75,46 \text{ cm}^2 ; Wy = 116 \text{ cm}^3 \\ Ix &= 36.553 \text{ cm}^4 ; ix = 22,01 \text{ cm} \\ Iy &= 984 \text{ cm}^4 ; iy = 3,61 \text{ cm} \end{aligned}$$

\* Balok Casteleted Wf . 550 . 175

pada potongan berongga dari hasil perhitungan di dapat :

$$\begin{aligned} g &= 49,6 \text{ kg/m}^3 ; Wx = 1.262 \text{ cm}^3 \\ F &= 47,46 \text{ cm}^2 ; Wy = 113 \text{ cm}^3 \\ Ix &= 18.294 \text{ cm}^4 ; ix = 19,63 \text{ cm} \\ Iy &= 983 \text{ cm}^4 ; iy = 4,55 \text{ cm} \end{aligned}$$

## II. Pembebanan :

### 1. Pembebanan pada portal kuda – kuda :

#### \* Beban pada atap kukuk :

- Berat gording + atap :  
 $(15,307 \times 6,00) 4/5,08 = 72,32 \text{ kg/m'}$
- B.s Wf 150 .75 = 14 kg/m'
- Berat penggantung 20 % = 2,8 kg/m'

---

$$\begin{aligned} \text{qm1} &= 89,12 \text{ kg/m'} \\ \text{- Beban hidup : } 100/5,08 &= 19,68 \text{ kg/m'} \end{aligned}$$

---

$$q1 = 109 \text{ kg/m'}$$

#### \* Beban pada atap kuda – kuda :

- Berat gording + atap :  
 $(15,307 \times 6,00) 17/24,87 = 62,78 \text{ kg/m'}$
- B.s Wf 360 . 125 = 29,6 kg/m'
- Berat penggantung 20 % = 5,92 kg/m'

---

$$\begin{aligned} \text{qm2} &= 98,30 \text{ kg/m'} \\ \text{- Beban hidup } 100 \times 5 / 24,87 &= 20,10 \text{ kg/m'} \end{aligned}$$

---

$$\begin{aligned} q2 &= 118,40 \text{ kg/m'} \\ \infty q2 &= 118 \text{ kg/m'} \end{aligned}$$

#### \* Beban pada balok ikat :

- B .s Balok Wf 200 . 100 = 21,3 kg/m

---

$$\infty q3 = 22 \text{ kg/m'}$$

#### \* Beban pada konsol pendek :

- B . s Balok crane Wf 350 . 175 : 49,6 kg/m' x 6,00 = 297,6 kg
- Beban crane kapasitas 5t = 5.000 kg

---

$$\begin{aligned} \Sigma p &= 5.297,6 \text{ kg} \\ \infty p &= 5.300 \text{ kg} \end{aligned}$$

## 2. Pembebanan pada portal mezanin :

\* Beban pada balok Wf . 250 . 125 :

- Berat lantai :  $0,12 \times 1,50 \times 2400 = 432 \text{ kg/m'}$
- Berat keramik :  $2 \times 24 \times 1,50 = 72 \text{ kg/m'}$
- Berat Bordex :  $7 \times 1,50 = 10,5 \text{ kg/m'}$
- B . s Balok Wf 250 . 125 =  $29,6 \text{ kg/m'}$
- Berat dinding partisi :  $0,04 \times 3 \times 1000 = 120 \text{ kg/m'}$

---


$$q_4 = 664 \text{ kg/m'}$$

$$= 375 \text{ kg/m'}$$


---

- Beban hidup :  $250 \times 1,50$

---


$$q_4 = 1.039 \text{ kg/m'}$$


---

\* Beban pada balok Wf 360 . 125 :

- B . s Balok Wf 360 . 125 =  $29,6 \text{ kg/m'}$

---


$$\infty q_5 = 30 \text{ kg/m'}$$


---

\* Beban pada balok diafragma Wf 150 . 150

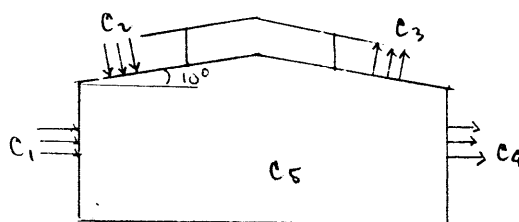
- B . s Balok Wf 150 . 150 =  $31,5 \text{ kg/m'}$

---


$$\infty q_6 = 32 \text{ kg/m'}$$


---

## 3. Pembebanan pada portal kuda-kuda akibat angin :



Koefesien angin bangunan tertutup :

$$C1 = + 0,9 \text{ ( tiup )}$$

$$C2 = + 0,02 \alpha - 0,4$$

$$\text{Atau } C2 = 0$$

$$C3 = - 0,4 \text{ ( isap )}$$

$$C4 = - 0,4 \text{ ( isap )}$$

$$C5 = - 0,4 \text{ ( isap )}$$

Tekanan angin (w) di perhitungkan =  $40 \text{ kg/m}^2$

Jadi tekanan angin untuk masing-masing bidang

$$Qw1 = + 0,9 \times 40 \times 6,00 = 216 \text{ kg/m' ( tekan )}$$

$$Qw2 = 0 \times 40 \times 6,00 = 0 \text{ kg/m' ( isap )}$$

$$Qw3 = - 0,4 \times 40 \times 6,00 = 96 \text{ kg/m' ( isap )}$$

$$Qw4 = - 0,4 \times 40 \times 6,00 = 96 \text{ kg/m' ( isap )}$$

$$Qw5 = - 0,4 \times 40 \times 6,00 = 96 \text{ kg/m' ( isap )}$$

### III Perhitungan Portal kuda - kuda :

#### 1. Kuda-kuda kukuk WF 150 . 75 :

Dari perhitungan Staad III di peroleh ( batang S-7 ) :

Axial : 215 kg

Shear : 401 kg

Moment : 602 kg m

#### Kontrol tegangan :

$$\sigma_a = \omega_x \frac{N_{max}}{F} + \frac{M_{max}}{W_x}$$

Tekuk terhadap sumbu x :

$$\lambda_x = \frac{L_{kx}}{i_x} = \frac{0,7 \times 406}{6,11} = 46,51$$

tabel ----->  $\omega_x = 1,161$

$$\sigma_a = 1,161 \times \frac{215}{17,85} + \frac{60200}{88,8}$$

$$= 692 \text{ kg/cm}^2 < \sigma_a = 1600 \text{ kg/cm}^2 \dots\dots\dots \text{Okey}$$

#### Ditinjau tegangan kip :

$$h/t_b = 150/5 = 30 < 75$$

$$l/h = 406/15 = 27$$

$$1,25 \frac{b}{t_s} = 1,25 \times \frac{75}{7} = 13,39$$

$l/h > 1,25 \frac{b}{t_s} \dots\dots\dots$  Balok dengan penampang tidak berubah bentuk

$$c_1 = \frac{l \cdot h}{b \cdot t_s} = \frac{406 \times 1,5}{7,5 \times 0,7} = 116 < 250$$

$$\text{maka } \sigma_{kip} = \sigma_a = 1600 \text{ kg/cm}^2 > \sigma_a = 692 \text{ kg/cm}^2 \dots\dots\dots \text{Okey}$$

**Kontrol lendutan :**

Dari perhitungan Staad III di peroleh  $\delta^o = 0,1 \text{ cm}$   
 $\delta = 1/360 \times 406 = 1,127 \text{ cm} > \delta^o = 0,1 \text{ cm}$   
..... okey.

**2. Kuda-kuda balok casteleted Wf 360 . 125 :**

Dari perhitungan Staad III di peroleh ( batang S – 214 )

Axial : 1.541 kg

Shear : 957 kg

Moment : 3.046 kg m

**Kontrol tegangan :**

$$\sigma_a = \omega_x \frac{N \text{ max}}{F} + \frac{M \text{ max}}{W_x}$$

Tekuk terhadap sumbu x :

$$\lambda_x = \frac{L_k x}{i_x} = \frac{0,7 \times 2335}{16,78} = 97,40$$

Tabel ----->  $\omega_x = 1,772$

$$\sigma_a = 1,772 \times \frac{1541}{29.82} + \frac{304600}{503}$$

$$= 91,57 + 605,56 = 697,14 \text{ kg/cm}^2 < \sigma_a = 1600 \text{ kg/m}^2$$

..... Okey

**Di tinjau tegangan kip :**

$$h/t_b = 360/6 = 60 < 75$$

$$L/h = 2335/36 = 64,86$$

$$1,25 \frac{b}{t_s} = 1,25 \times \frac{125}{9} = 17,36$$

$L/h > 1,25 \ b/t_s$  ..... Balok dengan penampang tidak berubah bentuk.

$$C1 = \frac{L \cdot h}{b \cdot t_s} = \frac{3374 \times 36}{12,5 \times 0,9} = 10.796,8 > 250$$

$$C3 = 0,21 (1 + \beta^*) (3 - 2 \beta^*) E / \sigma$$

$$\beta^* = 0,21 \frac{M_{ki} + M_{ka}}{2 M_{jep.}}$$

$$= \frac{4296 + 6808}{2 \times 3046} = 1,823$$

$$C3 = 0,21 (1 + 1,823) (3 - 2 \times 1,823) \frac{2,1 \times 10^6}{1600}$$

$$= 574$$

$$C1 \geq C3$$

$$\sigma_{kip} = \frac{C3}{C1} \times 0,7 \sigma_a$$

$$= \frac{574}{10.796} \times 0,7 \times 1600 = 59 \text{ kg/cm}^2 < \sigma_a = 697,14 \text{ kh/cm}^2$$

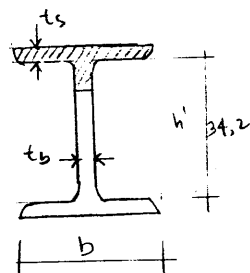
..... okey.

Kestabilan kip tidak terpenuhi sehingga perlu bracing, di coba setiap jarak 2,00 m.

Sehingga  $L = 200 \text{ cm}$

$$\frac{h}{t_b} = 60 < 75$$

$$\frac{L}{h} = \frac{200}{36} = 5,56 < 17,36 \quad \text{Balok dengan penampang dapat berubah bentuk.}$$



$$\text{Luas } A' = 12,5 \times 0,9 + 1/6 \times 34,2 \times 0,6$$

$$= 14,67 \text{ cm}^2$$

$$i_y = \sqrt{\frac{1/2 \cdot I_y}{A'}} = \sqrt{\frac{1/2 \cdot 147}{14,67}} = 2,24 \text{ cm}$$

$$= 0 + 98 = 98 \text{ kg/cm}^2 < \sigma_a = 1600 \text{ kg/cm}^2$$

..... Okey

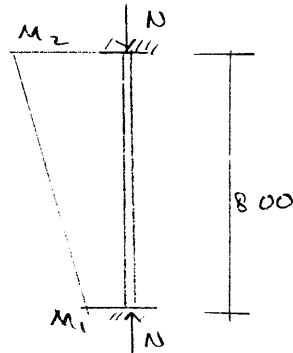
**Kontrol lendutan :**

Dari perhitungan Staad III di peroleh  $\delta^o = 0,5 \text{ cm}$

$$\delta = 1/180 \times 152 = 0,84 \text{ cm} > \delta^o = 0,5 \text{ cm}$$

..... Okey

#### 4. Kolom Wf 350 .175 :



Batang S - 227

Dari perhitungan Staad III di dapat :

$$\begin{aligned} \text{Axial} &= 7.179 \text{ kg} \\ M1 &= -3.305 \text{ kg m} \\ M2 &= -5.551 \text{ kg m} \\ \text{Shear} &= 1265 \text{ kg} \end{aligned}$$

**Kontrol tegangan :**

$$\begin{aligned} \beta &= 0,6 + 0,4 \frac{M1}{M2} \geq 0,4 \\ &= 0,6 + 0,4 \frac{-3304}{-5551} \geq 0,4 \\ &= 0,838 > 0,4 \end{aligned}$$

\* Panjang tekuk :

$$\begin{aligned} GA &= 1 \text{ (ujung kolom jepit)} \\ &\quad \frac{Ic}{Lc} \quad \frac{13.600}{800} \\ &\quad \Sigma \frac{Ic}{Lc} \\ GB &= \frac{\Sigma \frac{Ib}{Lb}}{\Sigma \frac{Ib}{Lb}} = \frac{8.399}{2335} = 4,726 \end{aligned}$$

$\left. \begin{array}{l} GA = 1 \\ GB = 4,726 \end{array} \right\} \text{ Dari nomogram di dapat } \kappa = 0,25$   
(untuk kolom tak bergoyang)



$$L_{kx} = 0,25 \times 800 = 200 \text{ cm}$$

$$\lambda_x = \frac{L_{kx}}{i_x} = \frac{200}{14,7} = 13,60 \rightarrow \text{tabel } \omega_x = 1,00, \sigma_{ex} = 122.640 \text{ kg/cm}^2$$

$$l_{ky} = 0,25 \times 800 = 200 \text{ cm}$$

$$\lambda_y = \frac{l_{ky}}{i_y} = \frac{200}{3,95} = 50,63 \rightarrow \text{tabel } \omega_y = 1,193, \sigma_{ex} = 8.290 \text{ kg/cm}^2$$

\* Tekuk arah x :

$$\sigma_a = \omega_x \frac{N}{A} + \beta \frac{n_x}{n_x - 1} \frac{M_x}{W_x}$$

$$n_x = \frac{A \sigma_{ex}}{N} = \frac{63,14 \times 122.640}{7.179} = 1.078$$

$$\sigma_a = 1,00 \times \frac{7179}{63,14} + 0,838 \times \frac{1078}{1078-1} \times \frac{555100}{775}$$

$$= 113,69 + 600,78 = 714,47 \text{ kg/cm}^2 < \sigma_a = 1600 \text{ kg/cm}^2$$

..... Okey

\* Tekuk arah y :

$$\sigma_a = \omega_y \frac{N}{A}$$

$$= 1,193 \times \frac{7179}{63,14} = 135,64 \text{ kg/cm}^2 < \sigma_a = 1600 \text{ kg/cm}^2$$

..... Okey.

\* Tegangan pada ujung kolom :

$$\sigma_a = \frac{N}{A} + \frac{M_x}{W_x}$$

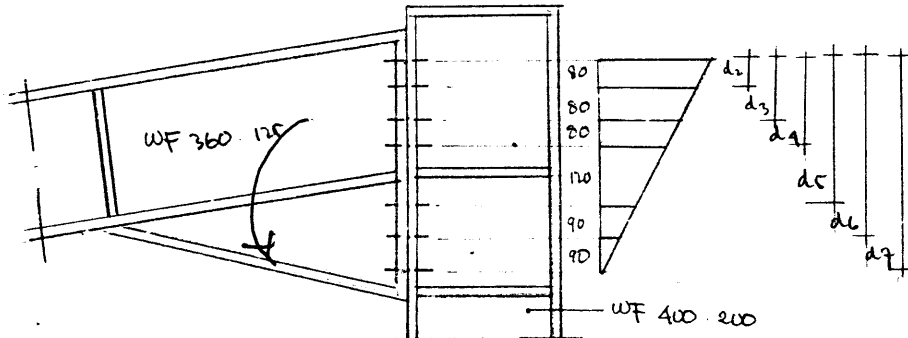
$$= \frac{7179}{63,14} + \frac{555100}{775} = 829,95 \text{ kg/cm}^2 < \sigma_a = 1600 \text{ kg/cm}^2$$

..... Okey

\*> Profil kolom Wf 350 . 175 bisa di pakai

## 5. Perhitungan Sambungan :

### a. Sambungan pada kolom dan kuda-kuda



Momen maximum yang bekerja pada balok dan kolom sebesar

M max = 6.808 kg m (batang S-22)

$$\begin{aligned} d1 &= 0 & ; d5 &= 36 \text{ cm} \\ d2 &= 8 \text{ cm} & ; d6 &= 45 \text{ cm} \\ d3 &= 16 \text{ cm} & ; d7 &= 52 \text{ cm} \\ d4 &= 24 \text{ cm} \end{aligned}$$

$$\begin{aligned} \Sigma di^2 &= d1^2 + d2^2 + d3^2 + d4^2 + d5^2 + d6^2 + d7^2 \\ &= 0^2 + 8^2 + 16^2 + 24^2 + 36^2 + 45^2 + 52^2 \\ &= 6.921 \text{ cm}^2 \end{aligned}$$

Tegangan maximum baut terjadi pada baut nomor 7

$$K7 = \frac{M \times d7}{\Sigma di^2} = \frac{680800 \times 52}{6921} = 5.115 \text{ kg}$$

Tegangan maximum yang terjadi :

$$\sigma_{\max} = \frac{5115}{2(1/4 \pi d^2)} = \frac{5115}{2(1/4 \pi \times 2,0^2)} = 814 \text{ kg/cm}^2$$

$< \sigma_a = 5630 \text{ kg/cm}^2$

..... Okey

Akibat gaya geser :

Dari Staad III pada batang S-22 di dapat  
Shear : 14.151 kg

$$K_v = \frac{K_v}{n} = \frac{14.151}{2 \times 7} = 1.010 \text{ kg}$$

Tegangan geser yang terjadi :

$$\tau^o = \frac{K_v}{\frac{1}{4} \pi d^2} = \frac{1010}{\frac{1}{4} \pi 2^2} = 321,49 \text{ kg/cm}^2$$

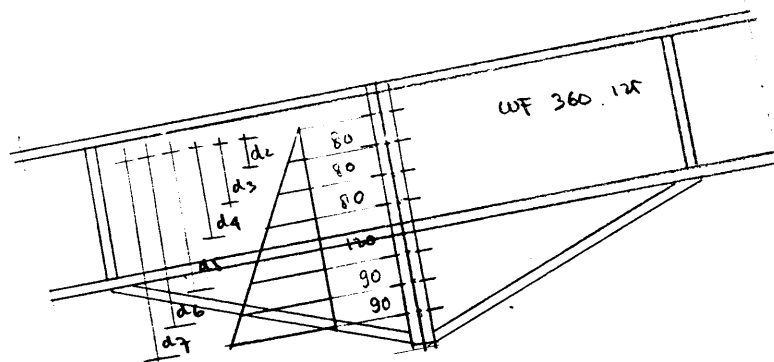
$$\tau = 0,6 \sigma_a = 3378 \text{ kg/cm}^2 > \tau^o$$

..... Okey

\*> jadi sambungan baut 7  $\phi$  20 atau 7  $\phi$  7/8 (type A 325) aman di pakai

**b. Sambungan pada balok kuda-kuda :**

Karena panjangnya lebih dari 12,00 m, maka perlu sambungan.



Momen maximum yang bekerja pada balok  
Sebesar M max = 3.046 kg m (batang S- 214)  
Shear = 628 kg

$$\begin{aligned} d1 &= 0 & ; d5 &= 36 \text{ cm} \\ d2 &= 8 \text{ cm} & ; d6 &= 45 \text{ cm} \\ d3 &= 16 \text{ cm} \\ d4 &= 24 \text{ cm} \end{aligned}$$

$$\begin{aligned}\Sigma d_i^2 &= d_1^2 + d_2^2 + d_3^2 + d_4^2 + d_5^2 + d_6^2 + d_7^2 \\ &= 0^2 + 8^2 + 16^2 + 24^2 + 36^2 + 45^2 + 52^2 \\ &= 6.921 \text{ cm}^2\end{aligned}$$

Tegangan inaximum baut terjadi pada baut nomor 7

$$K_7 = \frac{M \times d_7}{\Sigma d_i^2} = \frac{304600 \times 52}{6921} = 2288 \text{ kg}$$

Tegangan maximum yang terjadi :

$$\sigma_{\max} = \frac{2.288}{2(1/4 \pi 2,7^2)} = 199 \text{ kg/cm}^2 < \bar{\sigma}_a = 5630 \text{ kg/cm}^2$$

..... Okey

akibat gaya geser :

$$K_v = \frac{628}{2 \times 7} = 45 \text{ kg}$$

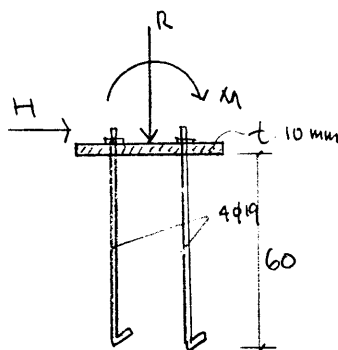
Tegangan geser yang terjadi :

$$\tau^o = \frac{45}{1/4 \pi 2,7} = 21,22 \text{ kg/cm}^2 < \tau^o = 3378 \text{ kg/cm}^2$$

..... Okey

\*> jadi sambungan baut 7  $\phi$  27 atau 7  $\phi$  9/8 (type A 325) aman di pakai

## 6. Perhitungan plat landas dan anker :



Dari Staad III di dapat

Joint 166 sbb :

Force -x = 1265 kg

Force -y = 7179 kg

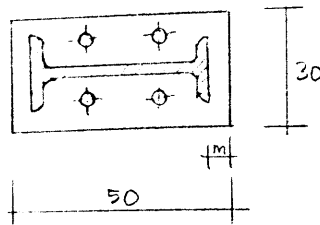
Moment = - 3304 kg m

\* Perhitungan plat landas :

luas alas (A) :

$$A' = \frac{R}{\sigma_b} = \frac{7179}{60} = 119,65 \text{ cm}^2$$

$$A = 30 \times 50 = 1500 \text{ cm}^2 > A'$$

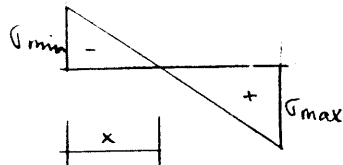


$$\sigma_b^o = \frac{7179}{1500} = 4.786 \text{ kg/cm}^2$$

ukuran plat landas 30 x 50 cm

$$m = 7,5 \text{ cm}$$

$$n = 6,25 \text{ cm}$$



$$M = \frac{\sigma_b m^2}{2} = \frac{4,786 \times 7,5}{2} = 134,6 \text{ kg cm}$$

$$t = \frac{134,6}{4 \times 1600} = 0,021 \text{ cm}$$

di pakai tebal plat (t) = 1,0 cm  
..... Okey

\* Perhitungan anker :

$$\sigma = \frac{R}{b \cdot h} + \frac{M}{W}$$

$$\sigma_{\max} = \frac{7.179}{30 \times 50} + \frac{330400}{1/6 \times 30 \times 50^2}$$

$$= 4,786 + 26,432 = 31,218 \text{ kg/cm}^2 < \bar{\sigma}_{\text{beton}} = 60 \text{ kg/cm}^2$$

$$\sigma_{\min} = 4,786 - 26,432 = -21,646 \text{ kg/cm}^2$$

Tegangan minimum harus di tahan oleh anker

$$\frac{X}{50 - x} = \frac{21,646}{31,218}$$

$$31,218 x = 21,646 (50) - 21,47 \text{ cm}$$

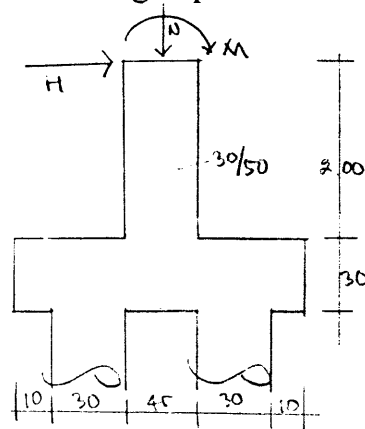
$$x = \frac{1082,3}{52,864} = 20,47 \text{ cm}$$

$$T = 20,47 (30) (1/2 \times 21,646) = 6.646 \text{ kg}$$

$$F = \frac{T}{\sigma_a} = \frac{6.646}{2.400} = 2,77 \text{ cm}^2$$

di pakai anker pada sisi tertarik  $2 \phi 19 = 5,68 \text{ cm}^2 > F$   
 di pakai anker pada sisi tertekan  $2 \phi 19 = 5,68 \text{ cm}^2$   
 Dengan mutu anker St-52.

## 7. Perhitungan pondasi Portal kuda – kuda :



Gaya – gaya yang bekerja :

Joint - 166

Force x = 12365 kg

Force y = 7179 kg

Moment = 3304 kg m

Data-data :

- kolom pendek : 30/50

- Dimensi pondasi : 1,25 x 1,25 m

- Strous  $2 \phi 30 - 3,50 \text{ m}$

### a. Perhitungan kolom pendek

$$Eou1 = Mu/Nu = 1,5 \times 3304 / 1,5 \times 7179 = 0,46 \text{ m}$$

$$Eou2 = 1/30 \cdot 0,50 = 0,016 \rightarrow \text{min} = 0,02 \text{ m}$$

$$Eou = eou1 + eou2 = 0,46 + 0,02 = 0,48 \text{ m}$$

$$\frac{Eou}{ht} = \frac{0,48}{0,50} = 0,96 \rightarrow \text{tabel } c1 = 1$$

$$c2 = 6,99$$

$$eu-1 = c1 \cdot c2 \left( \frac{lk}{100 \cdot ht} \right)^2 \cdot ht$$

$$= 1 \times 6,99 \left( \frac{0,7 \times 2,00}{100 \times 0,5} \right)^2 \cdot 0,5$$

$$= 0,0027 \text{ m}$$

$$eu-2 = 0,15 \times 0,50 = 0,075 \text{ m}$$

$$eau = eou + eu-1 + eu-2 = 0,48 + 0,0027 + 0,075 = 0,558 \text{ m}$$

$$\sigma'_{ou} = \frac{Nu}{b \cdot ht} = \frac{1,5 \times 7179}{30 \times 50} = 7,179 \text{ kg/cm}$$

$$\frac{\sigma'_{ou}}{2 \cdot ko \cdot \sigma_{bk}} = \frac{7,179}{2 \cdot 0,5 \cdot 225} = 0,0319$$

$$\frac{e_{au}}{ht} = \frac{0,558}{0,50} = 1,116$$

Dari nomogram di dapat  $q \text{ tot} = 0,08$

$$A_{tot} = 0,08 \cdot \frac{2 \times 0,5 \times 225}{2080} \times 30 \times 50$$

$$= 12,95 \text{ cm}^2$$

di pakai tulangan :  $8 \phi 16 = 16,08 \text{ cm}^2$

\* Tulangan geser :

$$Du = 1,5 \times 1265 \text{ kg} = 1897 \text{ kg}$$

$$\tau_{bu} = \frac{Du}{0,9 \times b \times ht} = \frac{1897}{0,9 \times 30 \times 50} = 1,405 \text{ kg/cm}^2$$

$$\tau_{bu} < \tau^*_{bu} = 9,5 \text{ kg/cm}^2$$

di pakai sengkang praktis  $\phi 6 - 20$

**b. Perhitungan poer / plat pondasi :**

Direncanakan dimensi poer/plat :  $125 \times 125 \text{ cm}$

Tabel plat =  $30 \text{ cm}$

Sehingga beban yang dipikul oleh plat :

$$Q = \frac{7179}{125 \times 125} = 0,4594 \text{ kg/cm}^2$$

atau  $q = 4594 \text{ kg/m}^2$

$$M_{\max} = \frac{1}{2} \cdot 4594 \cdot 0,625^2 = 897 \text{ kg m}$$

$$cu = \frac{1}{\sqrt{\frac{1,5 \times 897}{2 \times 0,5 \times 1,00 \times 225}}} = 10,22$$

$$\left. \begin{array}{l} \delta = 0,2 \\ c_u = 10,22 \end{array} \right\} q_{\min} = 0,0533$$

$$A = 0,0533 \times \frac{2 \times 0,5 \times 225}{2080} \times 25 \times 100 = 14,41 \text{ cm}^2$$

$$\text{Di pakai tulangan : } \phi 12 - 7,5 = 15,08 \text{ cm}^2$$

**c. Perhitungan Pondasi Strouss.**

Dari hasil laporan test sondir dengan faktor keamanan 3, metode Schmertmaan – Nottingham untuk  $\phi 30$  dengan kedalaman 3,5 m, kapasitas dukung ijin = 14 ton/tiang sehingga untuk 2 tiang  $N = 28 \text{ ton}$

\* Untuk Pondasi tepi :  
 $N = 7179 \text{ kg} < \bar{N} = 28.000 \text{ kg}$   
 ..... Okey

\* Untuk Pondasi tengah :  
 $N = 13.626 \text{ kg} < \bar{N} = 28.000 \text{ kg}$   
 ..... Okey