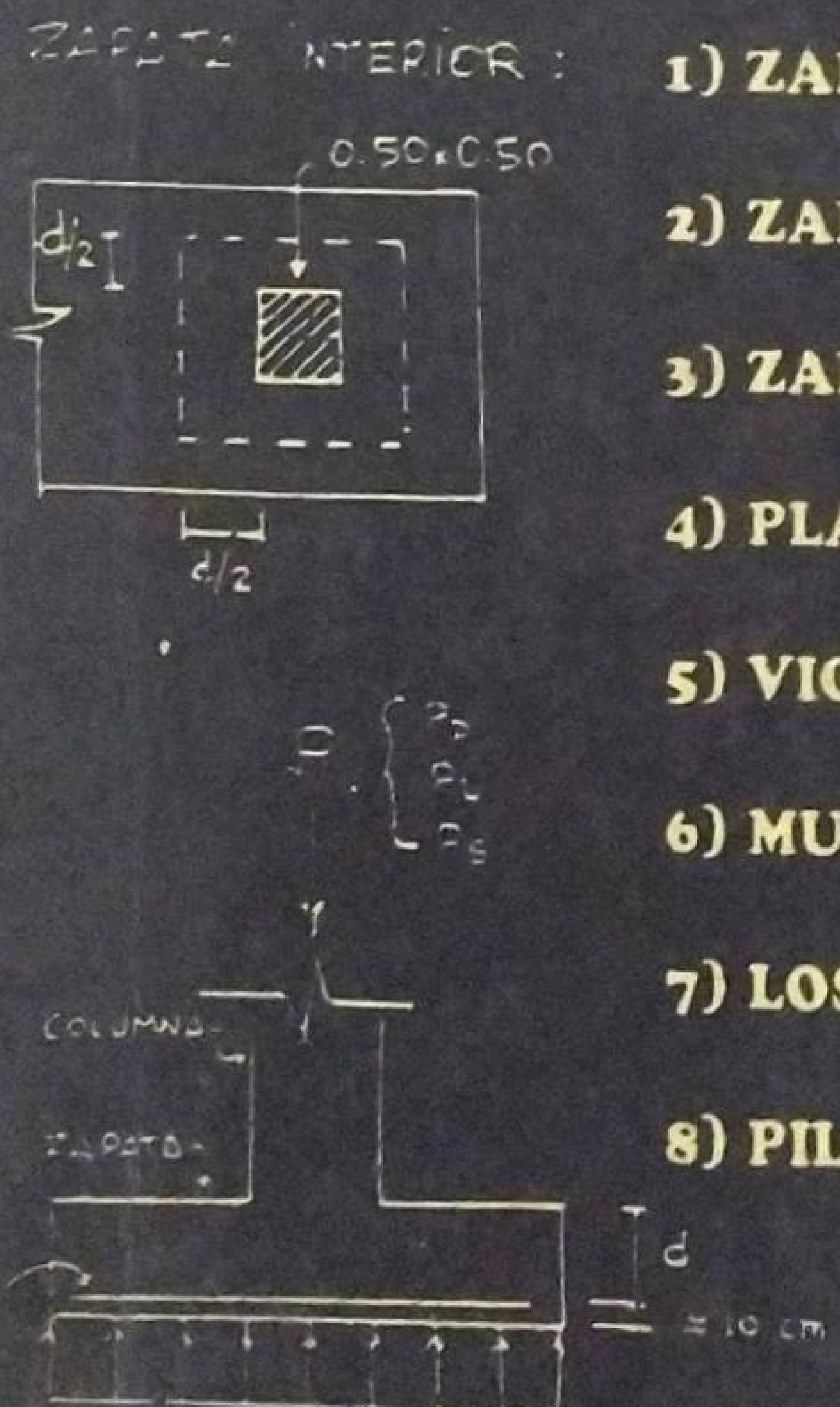
CONCRETO ARMADO II
TEORIA
PROBLEMAS RESUELTOS



ING. WALTER BARRENECHEA SOTO
C.I.P.

PROLOGO

La inquietud de la juventud Universitaria ha servido para elaborar el presente trabajo y recopilado y analizado, referente a todos los problemas mas importantes del curso de Concreto Armado II de la Facultad de Ing. Civil de la U.N.I.

El presente trabajo arduo, se divide en teoria y problemas resueltos contenido los capítulos de zapatas, placas, muros y losas en dos sentidos, respetando las normas del A.C.I.

En este trabajo es valido para todos las universidades ya que fueron hechas por profesores dedicados mas por vocación y por el alto nivel academico impartido en las aulas a pesar del pobre apoyo estatal.

Esperamos que este trabajo sirva de guia y práctica en provecho de nuestra sociedad en sacarla adelante, siendo los portavoces privilegiados a las mayorias y generadores de intelectuales consientes y no de insensibles.

Espero haya cumplido con mi deber de divulgar los conocimientos al alcande de todos (W.B.S.).

Ing. WALTER BARRENECHEA SOTO
CIP-CONTRATISTA-CONSULTOR
727269

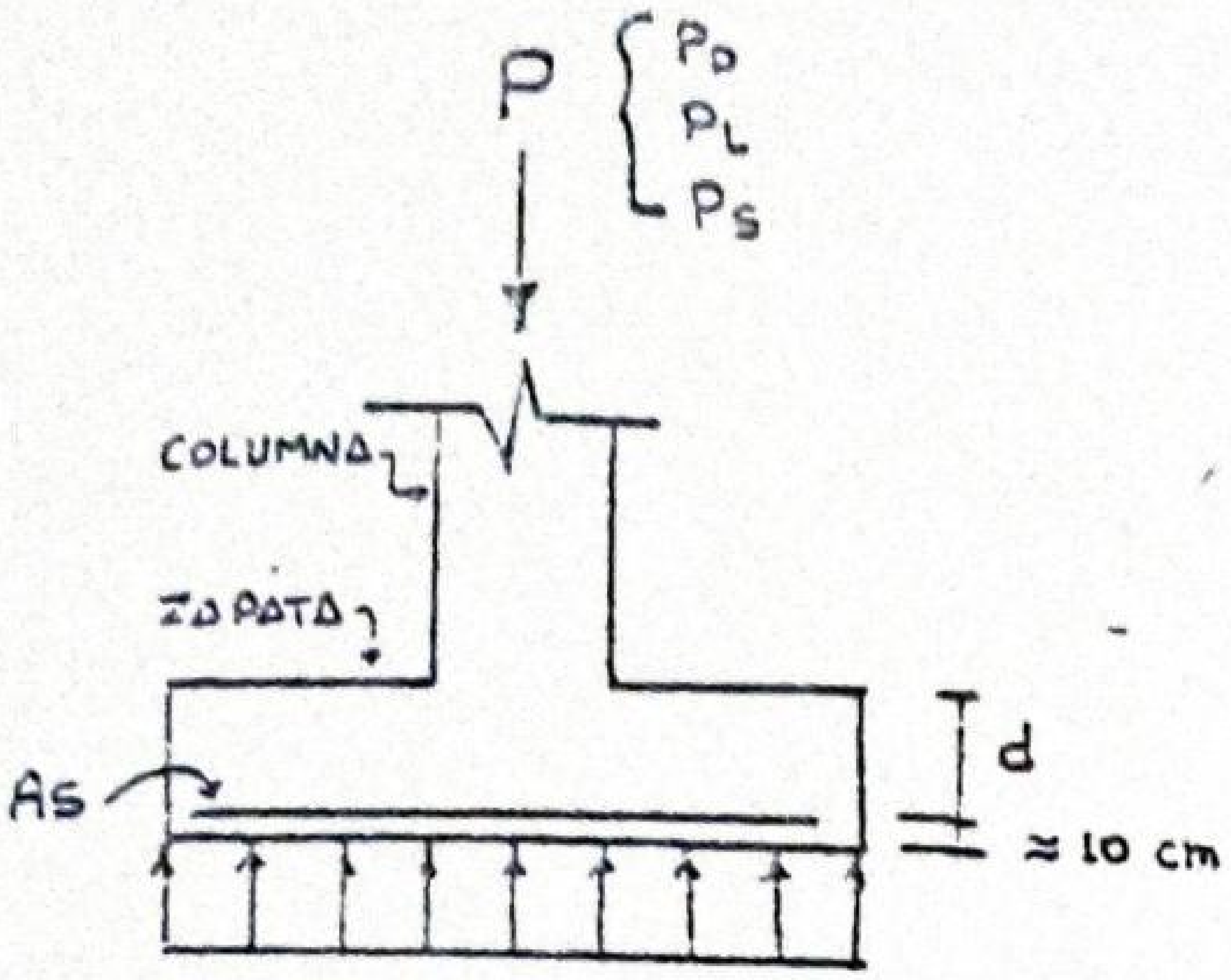
CONCRETO ARMADO II

TEORIA

PROBLEMAS RESUELTOS

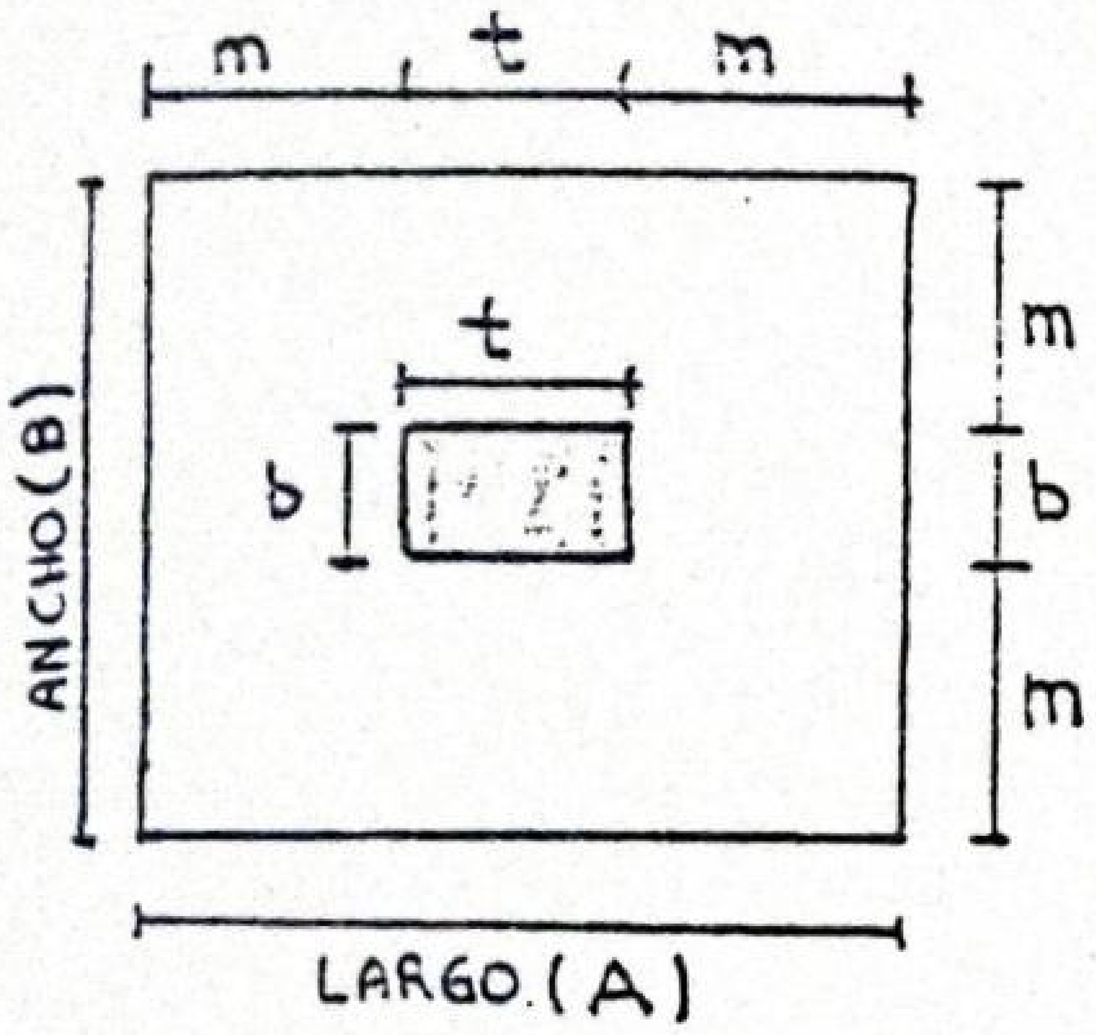
I.- TEORIA

DISEÑO DE ZAPATA INDIVIDUAL Y CONCENTRICA



σ_t = ESFUERZO DEL TERRENO

$$\sigma_t \le \sigma_{ADMISIBLE} = \sigma_s$$



A_z = AREA DE LA ZAPATA

$$A_z = A \cdot B$$

$$A_z = (t+2m)(b+2m)$$

A. DIMENSIONAMIENTO EN PLANTA :

(A_z, m) A SERVICIO :

$$A_z = \frac{P+P_z}{\sigma_s} \quad \text{CUANDO : } P = D+L \; \dots \text{ CARGAS VERTICALES } \dots (1)$$

$$A_z = \frac{P+P_z}{1.33\,\sigma_s} \quad \text{CUANDO : } P = D+L+S \; \dots\dots (2)$$

SE ESCOGE EL MAYOR

P_z = PESO PROPIO DE LA ZAPATA

CALCULO DEL PESO PROPIO DE LA ZAPATA : (P_Z)

(EL CUAL ES UN % DE LA CARGA A SERVICIO (P))

σ_s kg/cm²	P_z
4	0.04 P
3	0.06 P
2	0.08 P
1	0.10 P

COMO : $A_Z = (A \times B)$ ...(1)

$A \cdot B = (t + 2m)(b + 2m)$ ....(2)

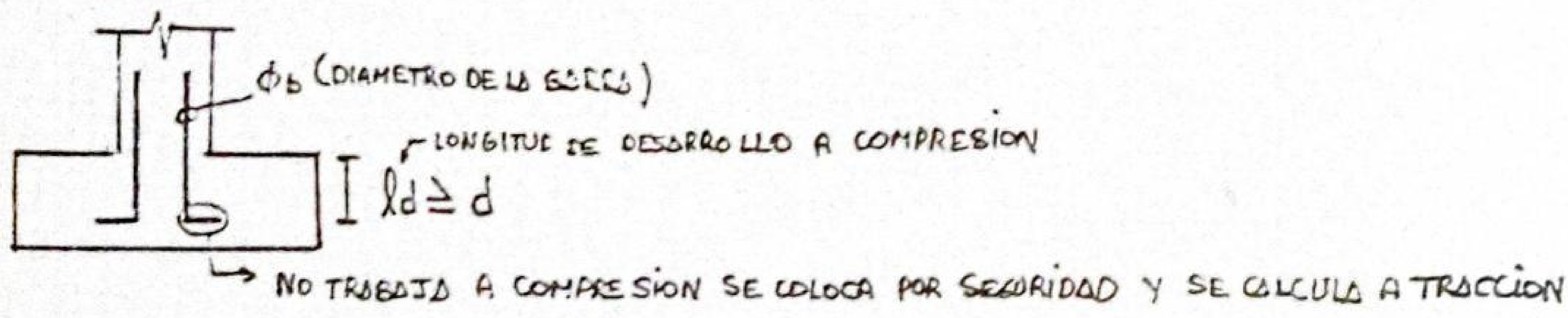
(1) = (2) : SE OBTIENE EL VALOR (m)

CONSIDERAR m = 5 Y UN POCO MAYOR AL CALCULADO CONSIGUIENDO ASI QUE $\sigma_t < \sigma_s$

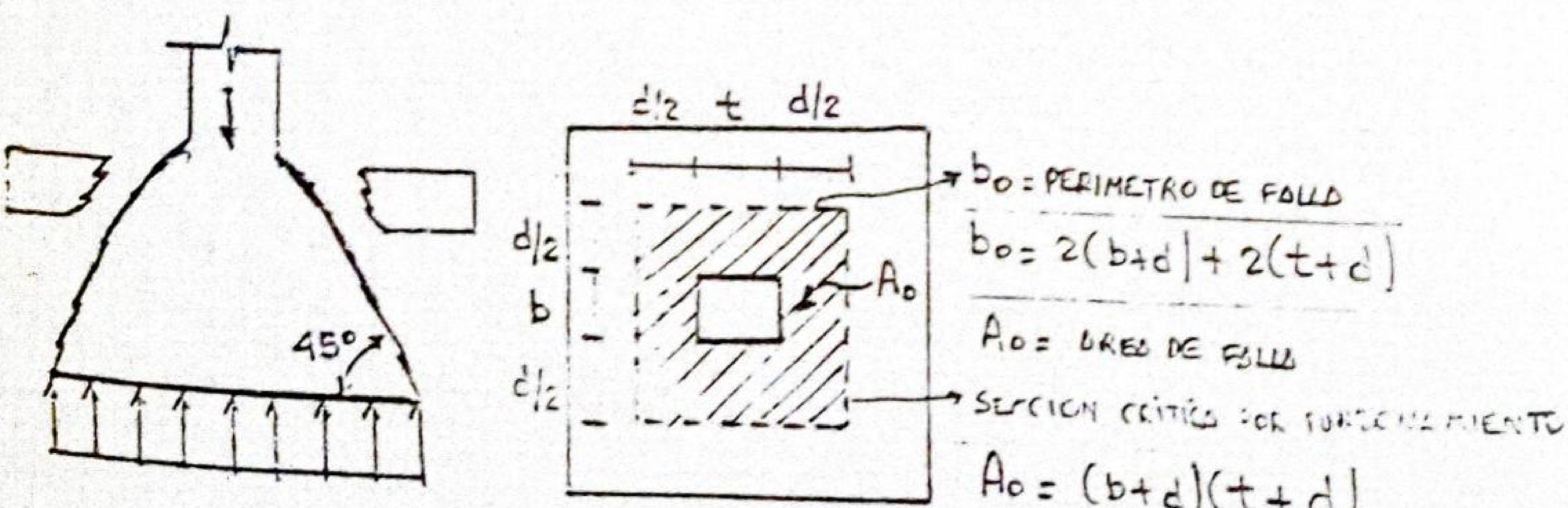
B. DIMENSIONAMIENTO EN ALTURA (d) A ROTURA O A SERVICIO

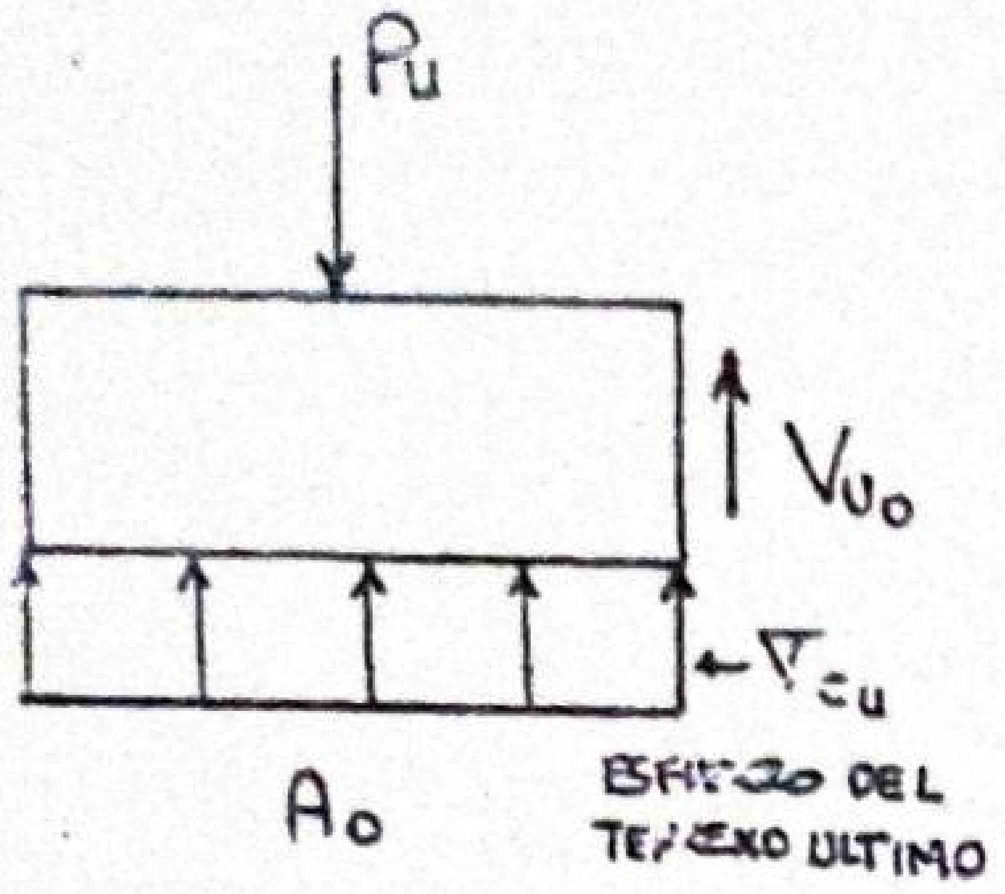
CALCULO DE P_u $\begin{cases} P_u = 1.5D + 1.8L \\ P_u = 1.25(D + L + S) \\ P_u = 0.9D + 1.1S \end{cases}$ SE ESCOGE EL MAYOR

1: POR LONGITUD DE ANCLAJE :



2: CORTE POR PUNZONAMIENTO (CORTE PERIMETRAL)





V_{Vo} = FUERZA CORTANTE EN TODO EL PERIMETRO DE FALLA

$$\sum F_{\text{VERTICALES}} = 0$$

$$A_o \cdot \sigma_{tu} + V_{Vo} - P_u = 0$$

$$V_{Vo} = P_u - A_o \cdot \sigma_{tu}$$

$$\sigma_{tu} = \frac{P_u}{(A)(B)}$$

$$\boxed{V_{Vo} = \sigma_{tu}\,(A \times B - A_o)}$$

ESFUERZO ACTUANTE ULTIMO :

$$\boxed{\upsilon_{uo} = \frac{V_{Vo}}{\phi \cdot b_o \cdot d} \leq \upsilon_{co}}$$ → ESFUERZO CORTANTE DEL CONCRETO A TODO EL ALREDEDOR

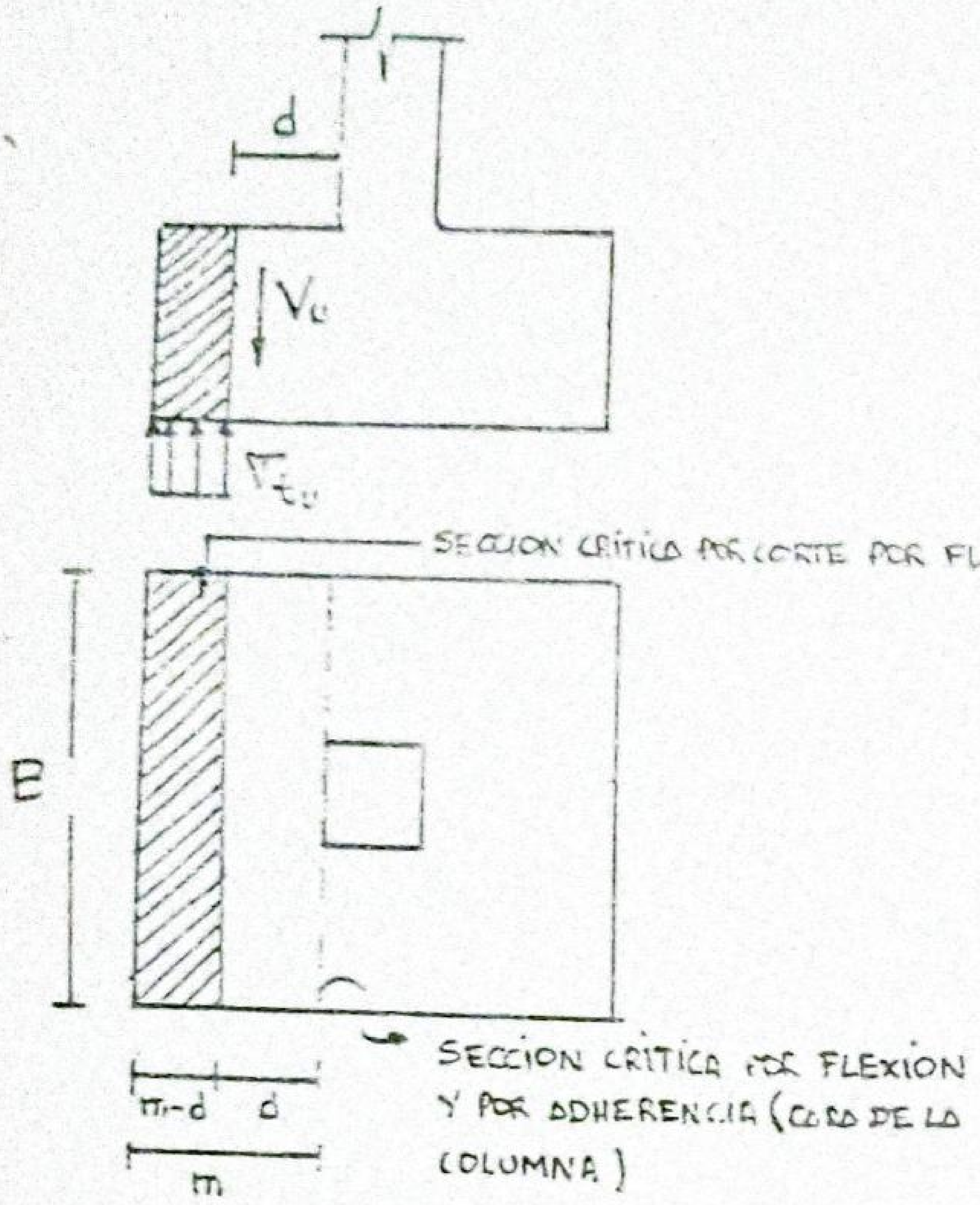
$$\phi = 0.85$$

SIENDO: $$\boxed{\upsilon_{co} = 0.27\left(2 + \frac{4}{\beta}\right)\sqrt{f'_c} \leq 1.1\sqrt{f'_c}}$$

DONDE $$\beta = \frac{\text{LADO LARGO DE LA COLUMNA } (t)}{\text{LADO CORTO DE LA COLUMNA } (b)}$$

NOTA : SE ASUME VALORES PARA "d" HASTA QUE SE CUMPLA QUE : $\upsilon_{uo} \leq \upsilon_{co}$

3. CORTE POR FLEXION (TRACCION DIAGONAL)



ÁREA ACHURADA

$$V_u = \sigma_{tu} . B (m-d) \quad \ldots \alpha$$

$$\nu_u = \frac{V_u}{\phi . d . B} \quad \ldots \beta$$

$\phi = 0.85$

α EN β :

ν_{CORTE}

$$\nu_u = \frac{\sigma_{tu}(m-d)}{\phi d} \leq 0.53 \sqrt{f'c}$$

NOTA: TAMBIÉN HAY QUE ASUMIR VALORES PARA "d" HASTA QUE $\nu_u \leq 0.53 \sqrt{f'c}$

PERO ES SUFICIENTE ASUMIR "d" PUNZONAMIENTO

4. SIN REFUERZO EN COMPRESION :

MOMENTO ÚLTIMO EN LA SECCION CRITICA POR FLEXION :

$$M_u = \sigma_{tu} . \frac{B . m^2}{2}$$

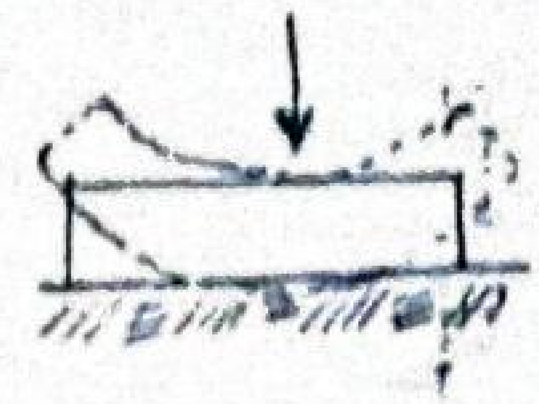
SI $\quad M_{ub} = K_{ub} B d^2 = M_u \quad$ ENTONCES $\quad d \geq \sqrt{\frac{M_u}{K_{ub} B}}$

PARA NO CONSIDERAR REFUERZO EN COMPRESION

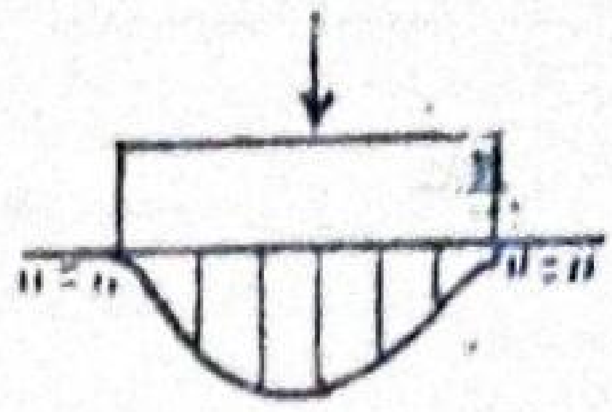
5. POR RIGIDEZ:

FLEXIBILIDAD DE LA CIMENTACION:

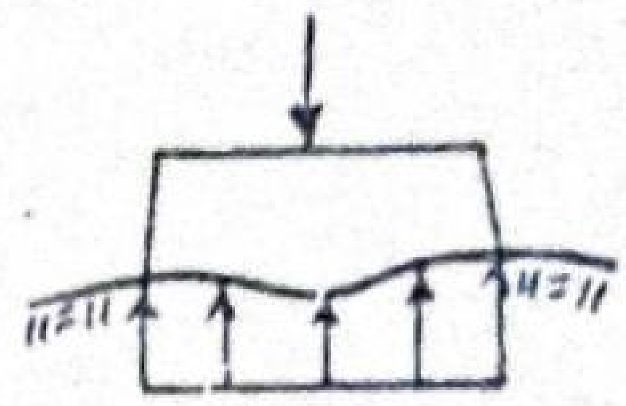
1. SI EL CIMIENTO ES EXTREMADAMENTE FLEXIBLE, RESPECTO AL SUELO



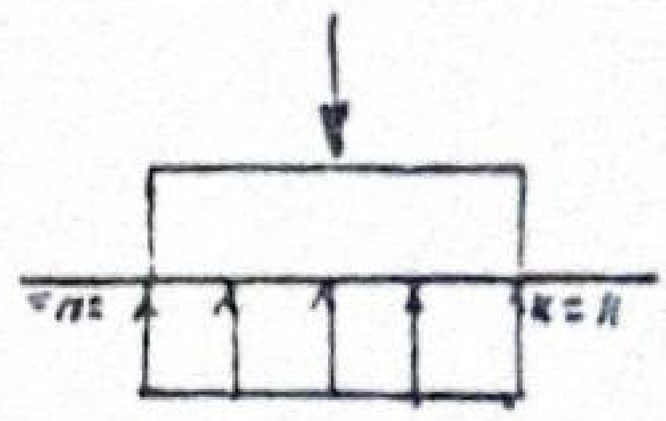
2. SI EL CIMIENTO ES MAS FLEXIBLE QUE EL SUELO; SE ORIGINA UNA LEVE DISTRIBUCION DE PRESIONES DEBAJO DE LA CARGA.



3. SI EL CIMIENTO TIENDE LA MISMA FLEXIBILIDAD DEL SUELO, SE PRODUCE UNA DISTRIBUCION DE PRESIONES SIGUIENDO LA DEFORMACION DEL SUELO, SE PRODUCEN ASENTAMIENTOS



4. CUANDO EL CIMIENTO ES MUY RIGIDO (NO DEFORMABLE) IMPONE AL SUELO LA CONDICION DE PRESIONES, ESTA ES LA SOLUCION OPTIMA PARA EL DISEÑO



PERALTE POR RIGIDEZ:

$$d \geq 1.45\,(A)\left(\frac{K_s \cdot A}{E_c}\right)^{1/3}$$

$A =$ MAYOR DIMENSION DE LA ZAPATA

$K_S =$ COEFICIENTE DE (BALASTO) SE HALLA POR MECANICA DE SUELOS DE LO CONTRARIO SE ADOPTA LO SIGUIENTE: *

$E_C = 15{,}000\sqrt{f'c}$

* COEFICIENTE DE BALASTO:

A. PARA SUELOS SIN COHESION O ARENA LIGERAMENTE COHESIVA

PARA UNA ZAPATA DE ANCHO "B" (cm) O PARA UNA ZAPATA CUADRADA DE LADO "B" (cm) EL VALOR DEL COEF. DE BALASTO PUEDE TOMARSE COMO: $K_S = K_{S1}\left[(B+30)/2B\right]^2$

Ks_1 = MODULO DE REACCION DEL SUELO CORRESPONDIENTE A UNA PLACA CUADRADA DE 1 PIE DE LADO O PLACA RECTANGULAR DE 1 PIE DE ANCHO

VALORES DE Ks_1 (Kg/cm^3)			
ARENA	SUELTA $\gamma = 1.3\ T/m^3$	MEDIA $\gamma = 1.6\ T/m^3$	DENSA $\gamma = 1.9\ T/m^3$
VALORES LIMITES EN CONDICIONES SECA Y HUMEDA	0.60 A 1.92	1.92 A 9.60	9.6 A 32.0
VALORES PROPUESTOS PARA ARENA SECA O HUMEDA	1.3	4.0	16.0
VALORES PROPUESTOS PARA ARENA SUMERGIDA.	0.8	2.5	10.0

DATOS QUE SE USA

B. PARA ARCILLA CONSISTENTE

a. PARA UNA ZAPATA DE ANCHO B (cm) Y UNA LONGITUD (LARGO): A (cm), EL COEF DE BALASTO SE CALCULA:

$$Ks = \left(\frac{20}{B}\right)\left(1 + \frac{15}{A}\right) Ks_1$$

b. PARA UNA ZAPATA CONTINUA DE ANCHO B (cm) SE TOMA $Ks = \frac{20}{B} Ks_1$

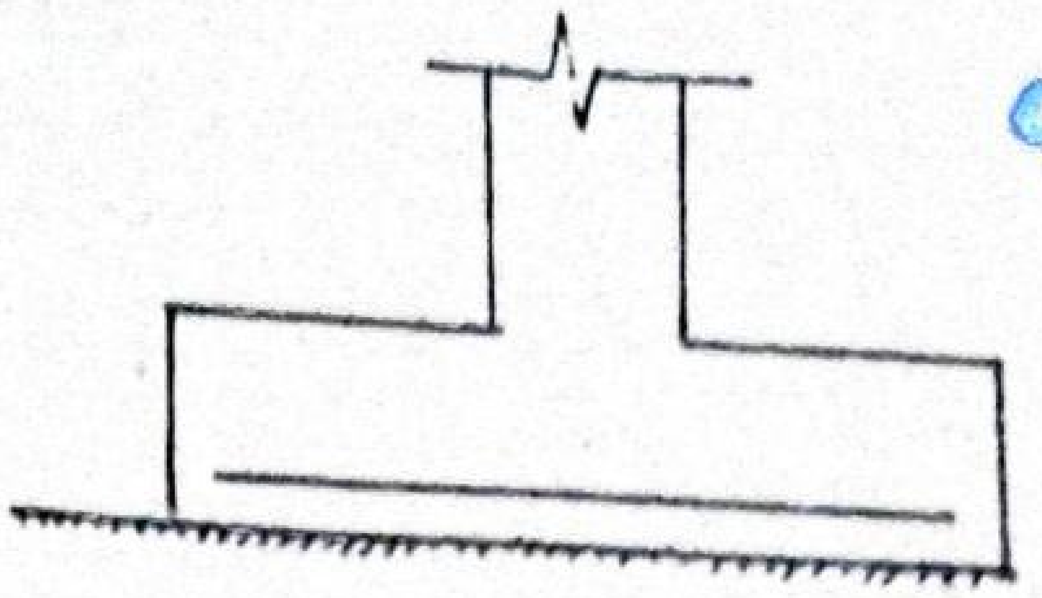
DONDE Ks_1, TOMA LOS SIGUIENTES VALORES:

VALORES DE Ks_1 (Kg/cm^3)			
CONSISTENCIA DE LA ARCILLA	BLANDA	MEDIA	DURA
RESISTENCIA A LA COMPRESION SIMPLE (Kg/cm^2)	1 – 2	2 – 4	4 ó mas
VALORES LIMITES	1.60 A 3.20	3.20 A 6.40	6.40
VALORES PROPUESTOS	2.5	5.0	10.0

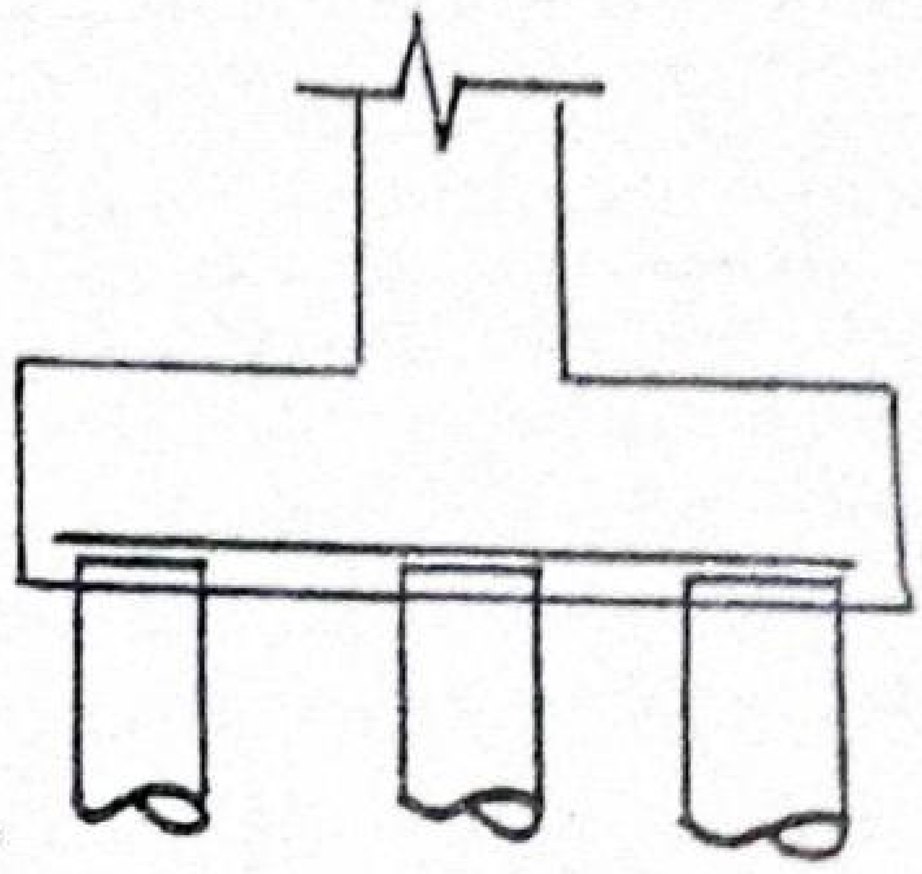
NOTA: DE LAS 5 CONDICIONES ANTERIORES SE TOMA EL MAYOR VALOR DE "d" Y CON ESTE SE HALLA EL REFUERZO

PERALTE MINIMO:

1.- SI LA ZAPATA SE APOYA EN EL SUELO



2. SI LA ZAPATA SE APOYA EN PILOTES

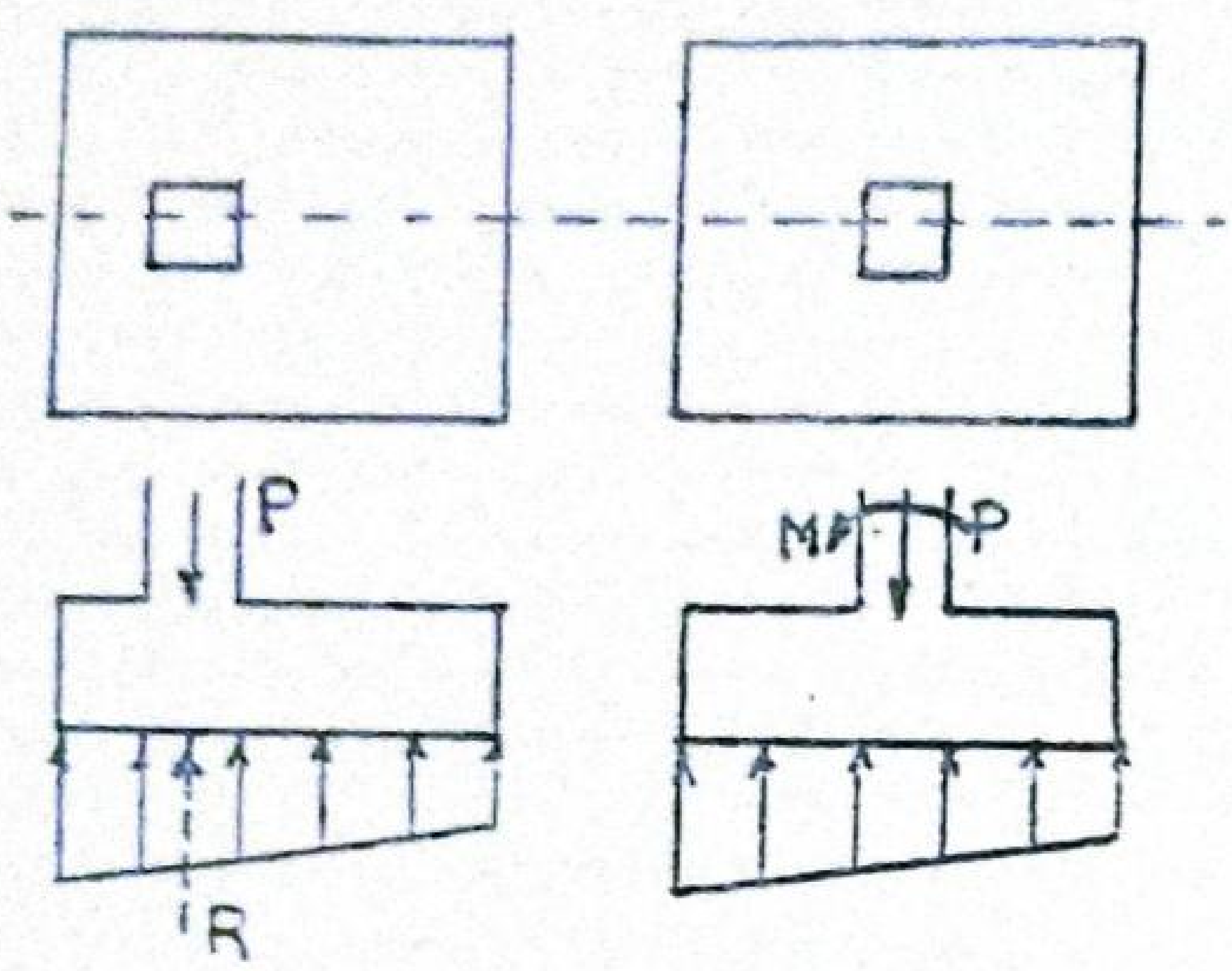


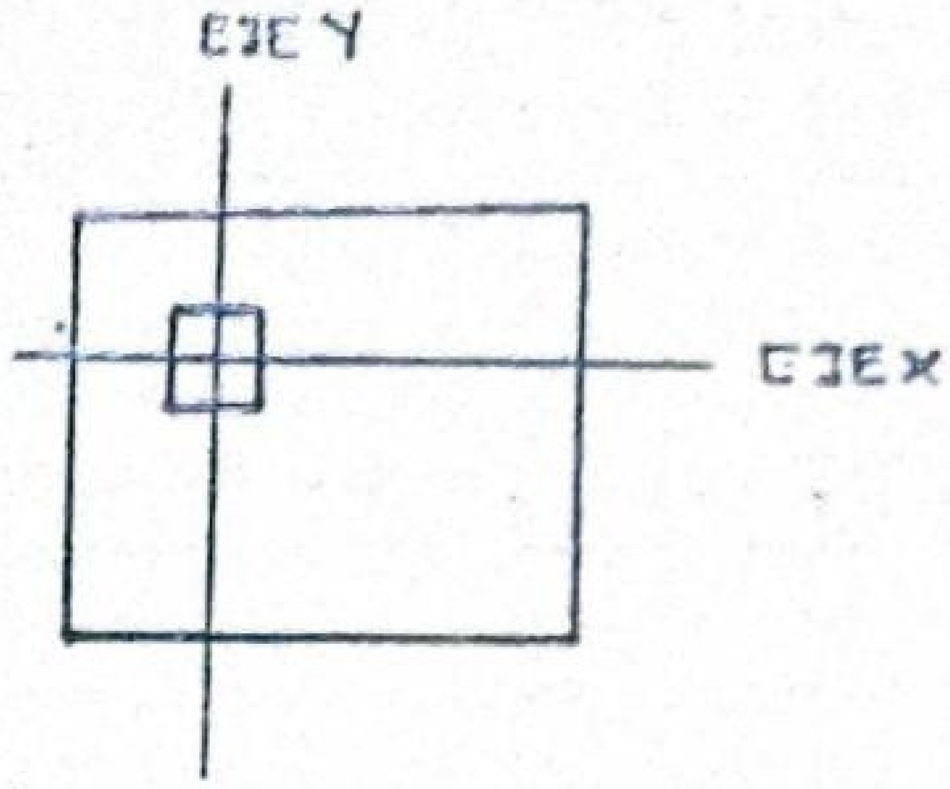
COLUMAS CIRCULARES O DE FORMA POLIGONAL REGULAR:

- SE LE CONSIDERA COMO UNA COLUMNA CUADRADA DE LA MISMA AREA PARA EFECTOS DE LOCALIZAR LAS SECCIONES CRITICAS POR CORTE, FLEXION Y ANCLAJE
- SE HACE EXTENSIVO ESTA CONSIDERACION A LOS PEDESTALES.

ZAPATAS CON CARGA EXCENTRICA:

CASO 1. CASO 2.





$$\sigma = \frac{P}{Az} + \frac{Mx.dy}{I_x} \pm \frac{My.dx}{I_y}$$

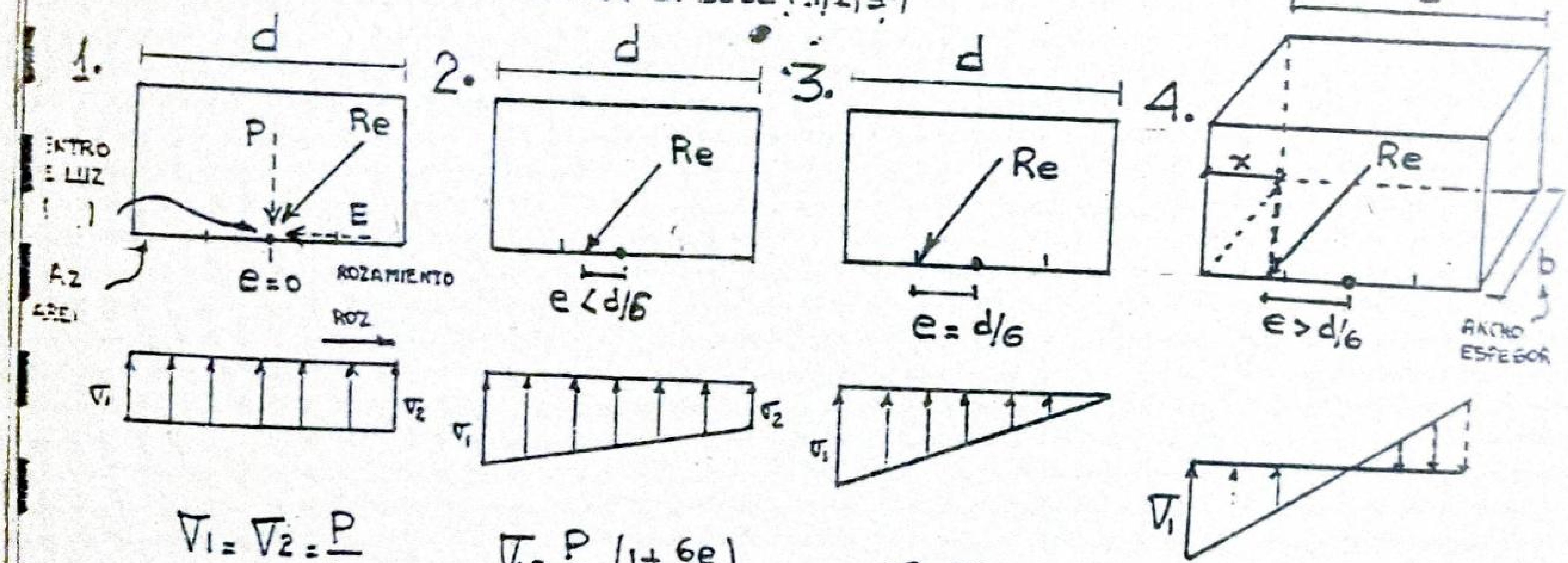
PRESION

PRINCIPIO DEL TERCIO MEDIO:

e = EXCENTRECIDAD

R_e = RESULTANTE

PRESION EN TODA LA BASE (1, 2, 3) PRESION PARCIAL EN LA BASE (4)



1. $\sigma_1 = \sigma_2 = \frac{P}{Az}$

2. $\sigma_1 = \frac{P}{Az}\left(1 + \frac{6e}{d}\right)$

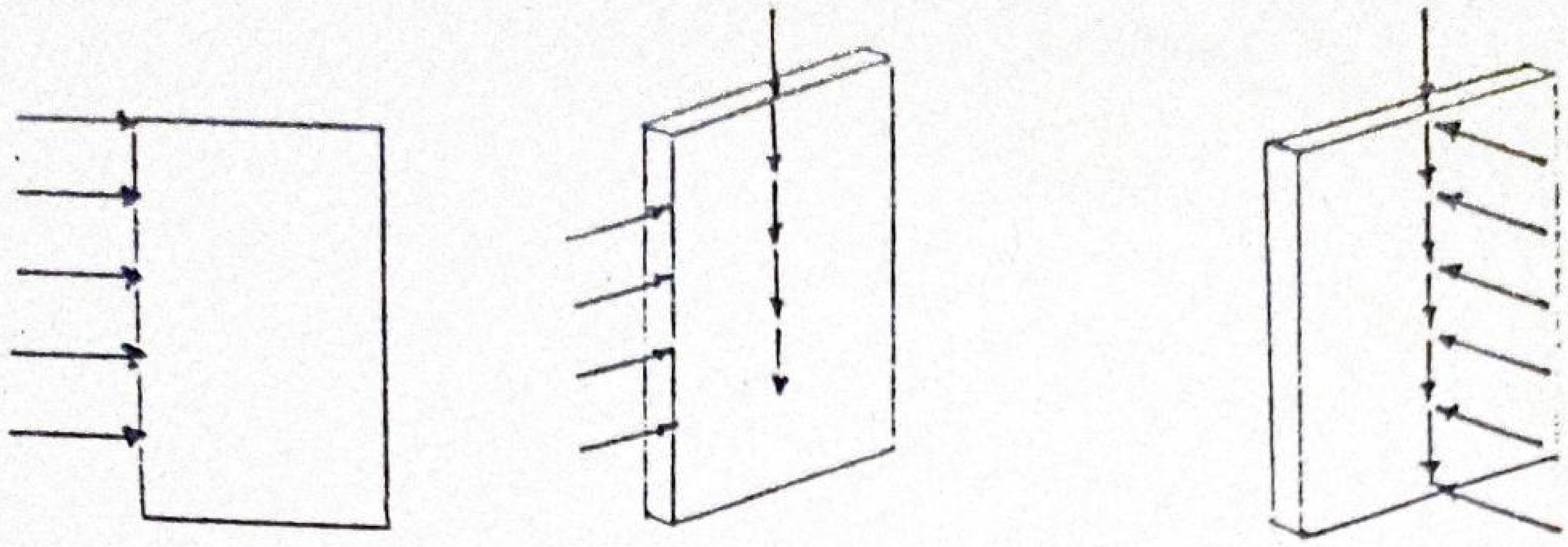
   $\sigma_2 = \frac{P}{Az}\left(1 - \frac{6e}{d}\right)$

3. $\sigma_1 = \frac{2P}{Az}$

   $\sigma_2 = 0$

4. $\sigma_1 = \frac{2P}{(3x)(b)}$

PLACAS



SE TOMA COMO:
MURO DE CORTE

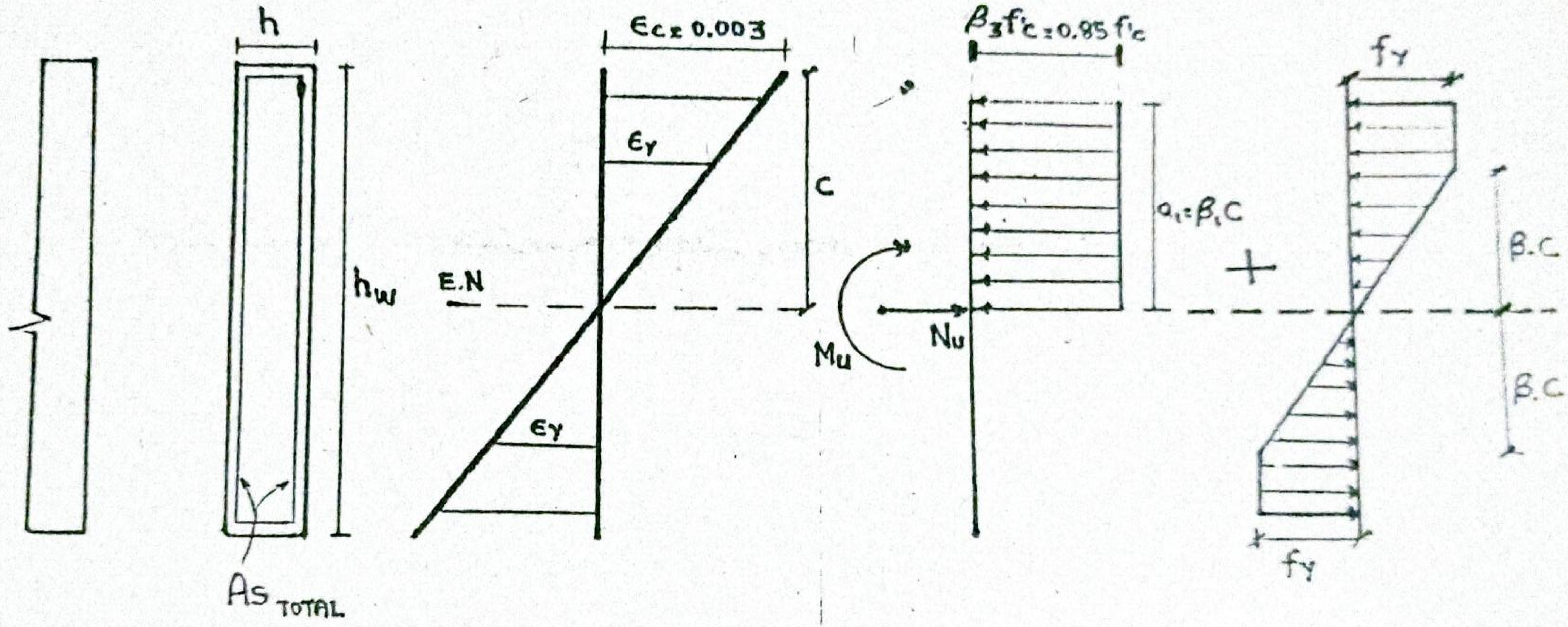
SE TOMA COMO:
COLUMNA

PLACAS: Tienen por función principal, en su propio plano plano, absorver un porcentaje de las fuerzas laterales aplicadas a la edificación, disminuyendo en este porcentaje el trabajo de los pórticos, en la misma dirección.

Los Muros de corte deben diseñarse para resistir las combinaciones de momento de volteo, cargas verticales y cortantes, deben tenerse precauciones adecuadas para transmitir los momentos del muro, las cargas verticales y los cortantes a la cimentación o apoyos. El uso de muros (Placas) es mayor cada día debido a la utilidad que dan, y a las cargas que son capaces de resistir así como a la restricción, al desplazamiento lateral que proporcionan. Según el A.C.I los muros pueden diseñarse por:

- Método de diseño por Flexo-compresión
- Método empírico de diseño.

DISEÑO COMO MURO DE CORTE



DISEÑO POR FLEXION

CALCULOS:

VALOR DE β : $\beta = \dfrac{f_y}{6000}$

UBICACION DEL E.N : $C = \left(\dfrac{q+\alpha}{2q + 0.85\beta_1}\right) l_w$

$q = \dfrac{\rho_v . f_y}{f'c}$

$\alpha = \dfrac{Nu}{l_w . h . f'_c}$

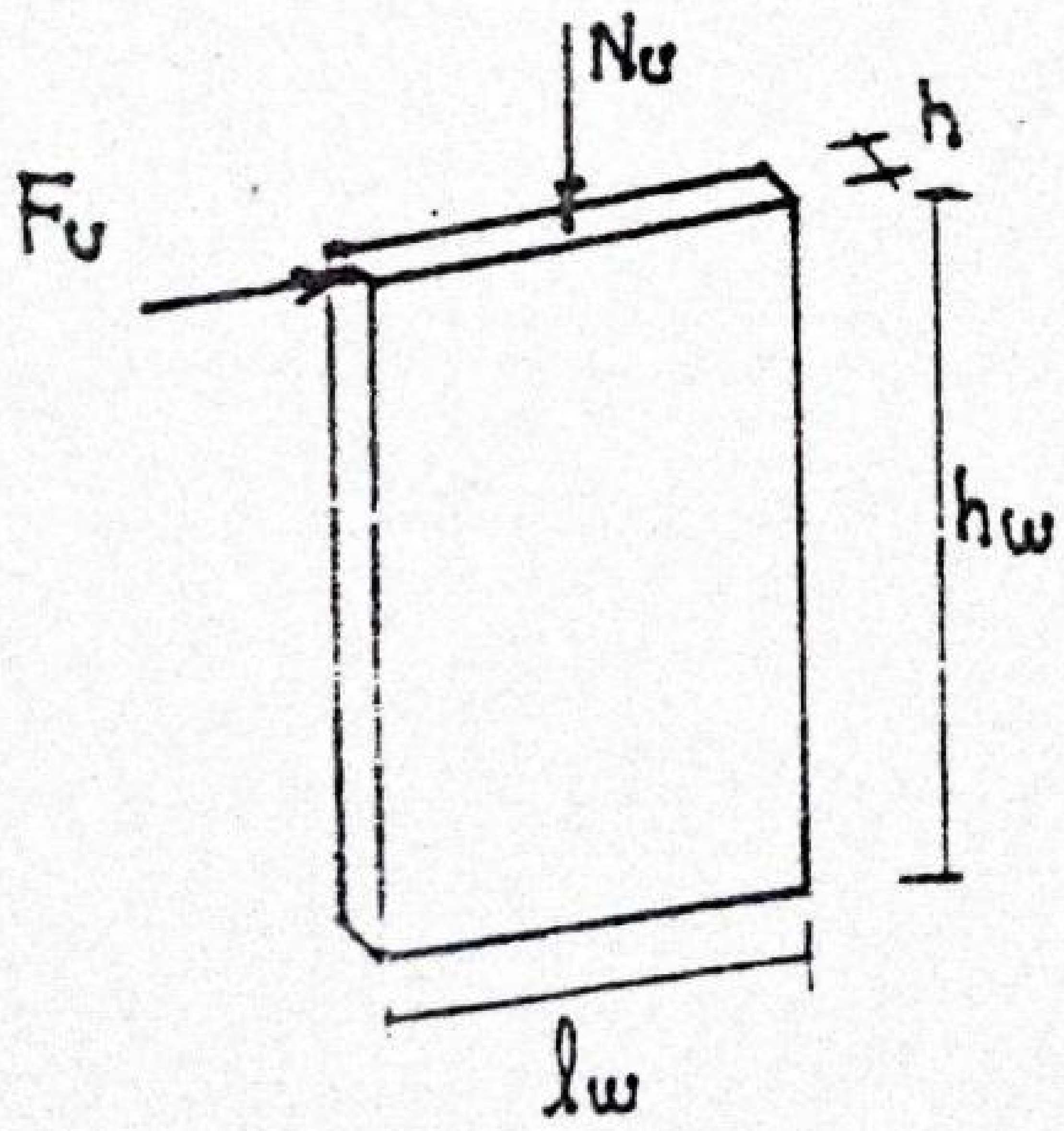
CUANTIA VERTICAL : $\rho_v = \dfrac{As}{l_w . h}$

$\phi = 0.70$

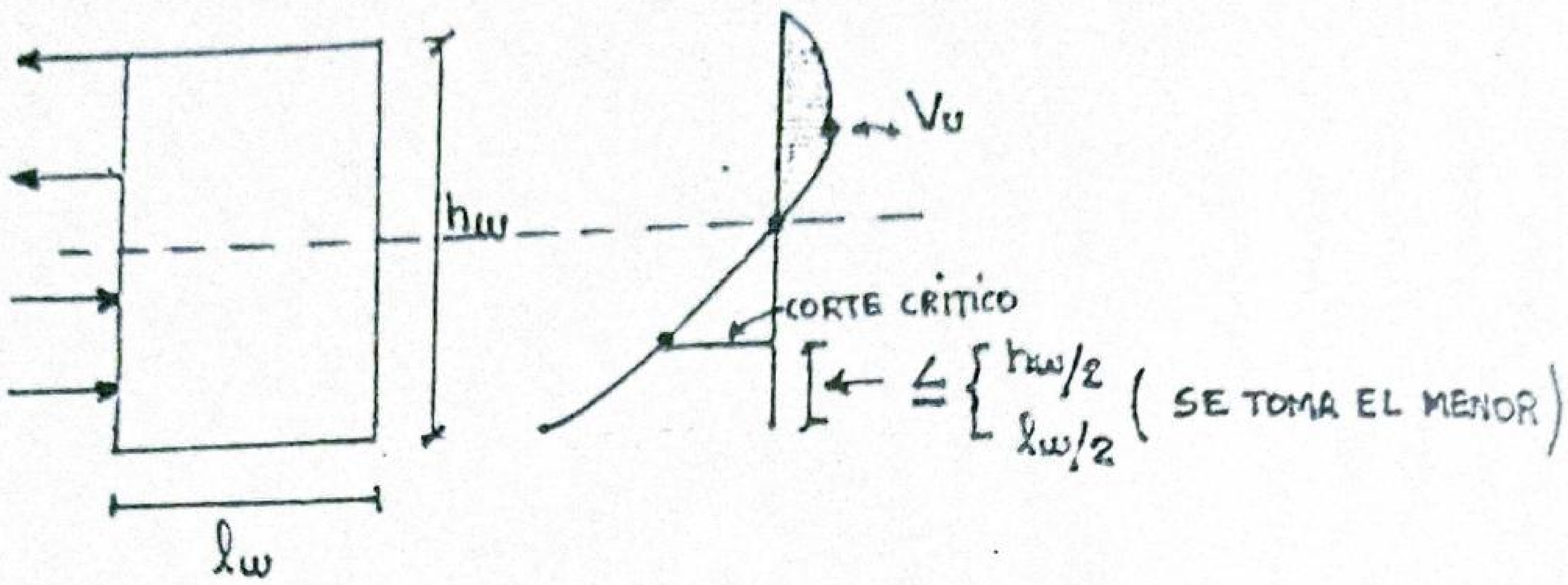
MOMENTO ULTIMO RESISTENTE : $Mu = \phi . As . f_y . l_w \left[\left(1 + \dfrac{Nu}{As . f_y}\right)\left(\dfrac{1}{2} - \dfrac{\beta_1 C}{2 l_w}\right) - \dfrac{C^2}{l_w^2}\left(1 + \dfrac{\beta^2}{3} - \beta_1\right)\right]$

$$Mu = \frac{\phi . As . f_y . l_w}{2}\left[1 + \frac{Nu}{As . f_y}\right]\left(1 - \frac{C}{l_w}\right)$$

PROCEDIMIENTO :
- SE ASUME UN As
- SE CALCULA ρ_v, q
- SE CALCULA Mu QUE RESISTE EL ESFUERZO Y SE LO COMPA. A CON EL DEL ANALISIS.
- SE ASUME UN As MAS EXACTO...



DISEÑO POR CORTE :



- CORTE ACTUANTE (v_u)

$$v_u = \frac{V_u}{\phi h (0.80 l_w)} \leq 2.7\sqrt{f'c}$$

$\phi = 0.85$

$d = 0.80\, l_w$

- CORTE RESISTENTE ADMISIBLE

$$(v_c) \leq \begin{cases} 0.87\sqrt{f'c} + \dfrac{N_u}{4 . l_w . h} \\[2ex] 0.16\sqrt{f'c} + \dfrac{l_w\left[0.33\sqrt{f'c} + 0.2\left(\dfrac{N_u}{l_w . h}\right)\right]}{\underbrace{\left(\dfrac{M_u}{V_u} - \dfrac{l_w}{2}\right)}} \end{cases}$$

SI SALE NUMERO NEGATIVO NO SE TOMARA EN CUENTA ESTE VALOR

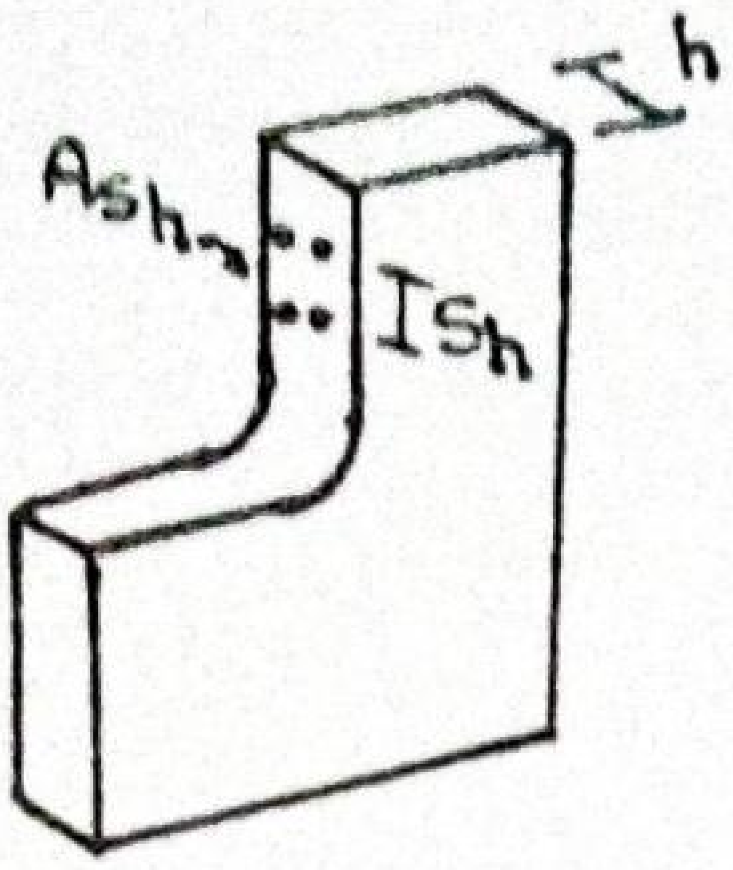
- SI : $v_u \leq v_c/2$ SE COLOCA REFUERZO MINIMO DONDE ES MAYOR POR CORTE

SE TOMARAN : $\begin{cases} \rho_h = 0.0025 & \text{(CUANTIA HORIZONTAL)} \\ \rho_v = 0.0015 & \text{(CUANTIA VERTICAL)} \end{cases}$

SI $\phi \leq 5/8''$ Y $f_y \geq 4200\ k/cm^2$ $\begin{cases} \rho_h = 0.0020 & \text{(CUANTIA HORIZONTAL)} \\ \rho_v = 0.0012 & \text{(CUANTIA VERTICAL)} \end{cases}$

SI $v_u > v_c/2$ { SE TOMARA O DISEÑARA REFUERZO HORIZONTAL Y VERTICAL POR CORTE

REFUERZO HORIZONTAL (EN UNA SECCION)



$$\frac{As_h}{S_h} = \frac{(v_u - v_c)}{f_y} \cdot h$$

$$\rho_h = \frac{As_h}{S_h \cdot h} \geq 0.0025$$

$$S_h \leq \begin{cases} l_w/5 \\ 3h \\ 45\text{ cm} \end{cases}$$

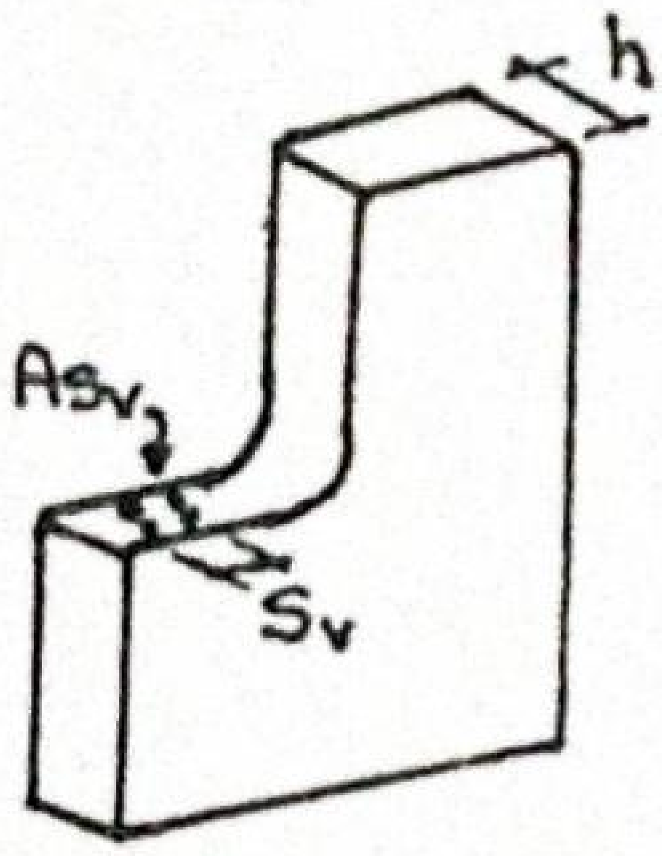
REFUERZO VERTICAL (EN TODA LA SECCION)

ACERO TOTAL VERTICAL → SE TOMARA EL MAYOR ENTRE As (POR FLEXION) y As_v

$$\rho_v = \frac{As_v}{h \cdot l_w} \geq \begin{cases} 0.0025 + \left(2.5 - \frac{h_w}{l_w}\right)(\rho_h - 0.0025) \\ 0.0025 \end{cases}$$

← SE ESCOGE EL MAYOR

h_w = ALTURA LIBRE CORRESPONDIENTE A UN PISO



$$S_v \leq \begin{cases} l_w/3 \\ 3h \\ 45\text{ cm} \end{cases}$$

SE ESCOGE EL MENOR

NOTA: COMO SI FUERA UNA VIGA PARA LOS TANTEOS SE ASUME $a = 0.2d$. LUEGO SE CALCULA As Y SE MULTIPLICA As POR 3 ENTONCES SE TENDRA As_h o As_v.

DISEÑO COMO COLUMNA:

$e = M_u/P_u$

si $e > h/6$: SE DISEÑARA COMO COLUMNA ESBELTA EMPLEANDO LAS CURVAS DE ITERACION

si $e \leq h/6$: SE DISEÑARA POR EL METODO SIMPLIFICADO DE MUROS.

METODO SIMPLIFICADO DE MUROS EMPIRICO:

$$\boxed{P_U = 0.55\,\phi\, f'_c A_s \left(1 - \left\{\frac{l_c}{40h}\right\}^2\right)}$$

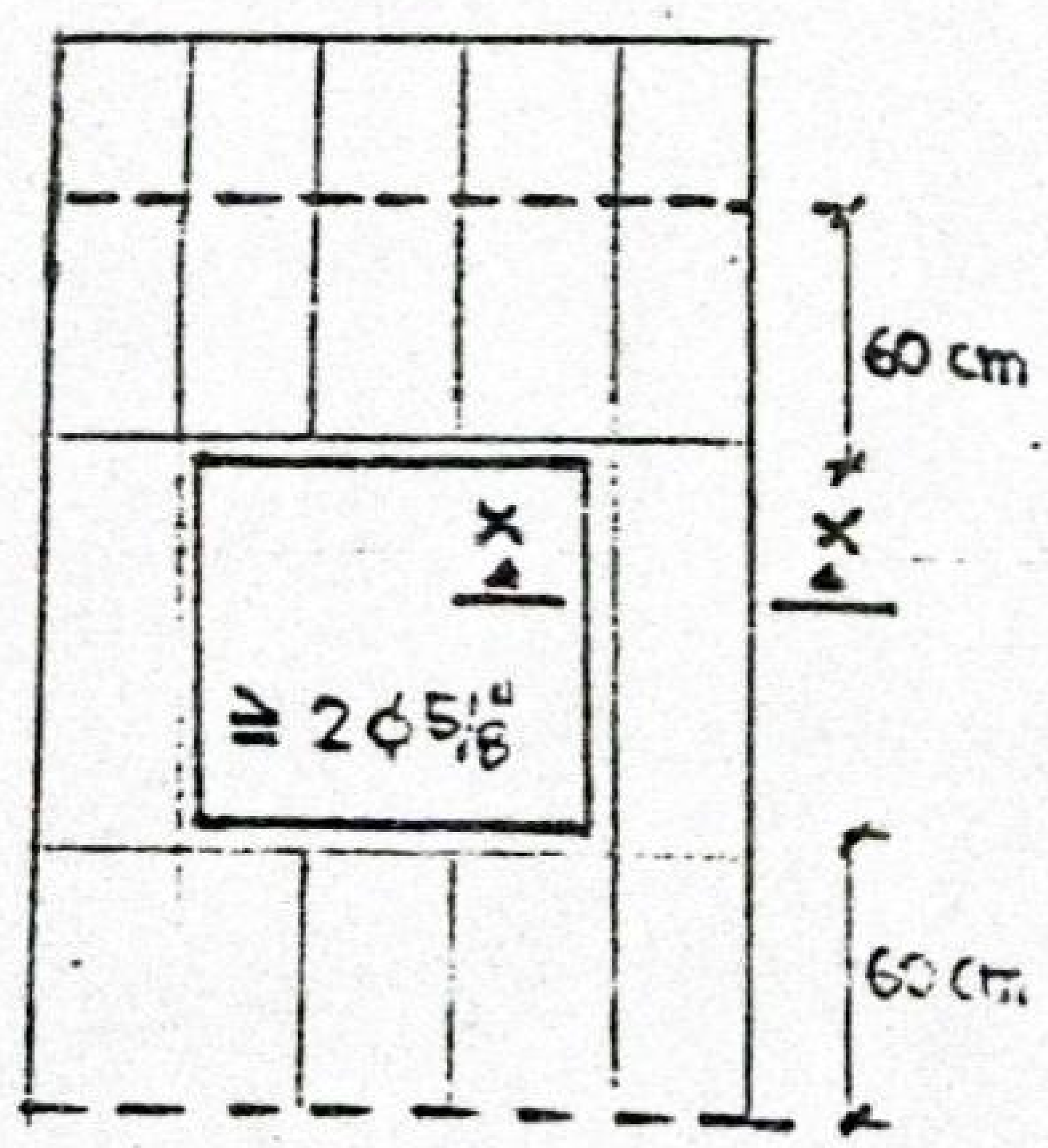
– CARGA ULTIMA QUE RESISTE ESTE MURO

A_G: AREA $= l_w \cdot h$ $(\phi = 0.70)$

l_c = DISTANCIA VERTICAL ENTRE LOS APOYOS (PISO A PISO)

CONSIDERACIONES DE ESPESOR DE PLACAS Y REFUERZOS:

- El espesor del muro o de la placa en edificios será como mínimo de 15 cm en los 4.5 mt superiores, y por cada 7.5 mt o fracción medidos hacia abajo, debe aumentarse en 2.5 cm.
- En sotanos (muros exteriores o expuestos a la intemperie o fuego) el espesor mínimo será de 20 cm. Si el espesor del Muro 25 cm, el refuerzo debe colocarse en 2 capas, una capa consiste en NO menos de la mitad y NO más de los 2/3 del refuerzo requerido.
- La separación máxima de refuerzos debe ser de 45 cm, con un diametro mínimo de 3/8".
- En Vanos (aberturas) aparte del refuerzo mínimo debe colocarse alrededor de la abertura, refuerzo 2 Ø 5/8", con 60 cm más allá de la abertura.

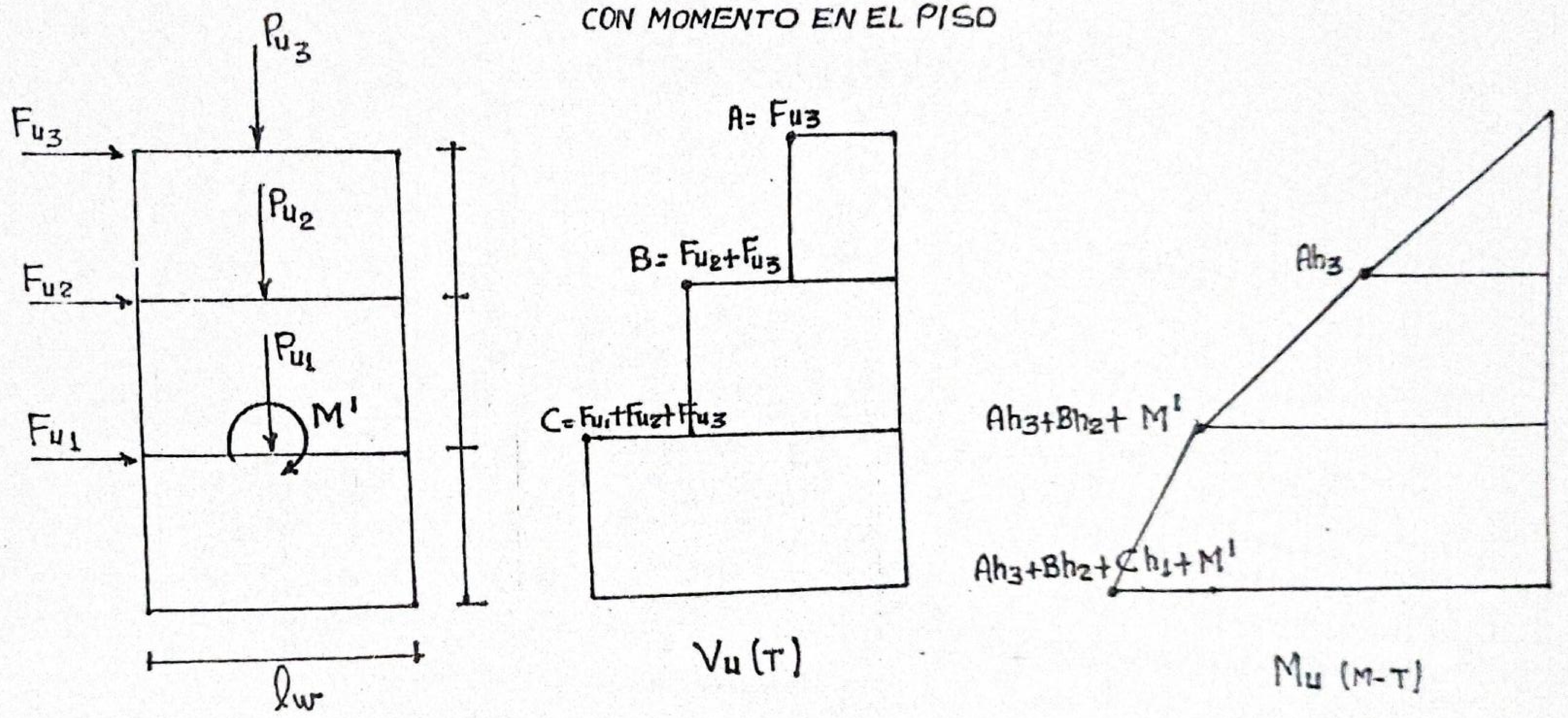


CORTE X-X

EL REFUERZO HORIZONTAL ES EL PRINCIPAL EN UN MURO DE CORTE

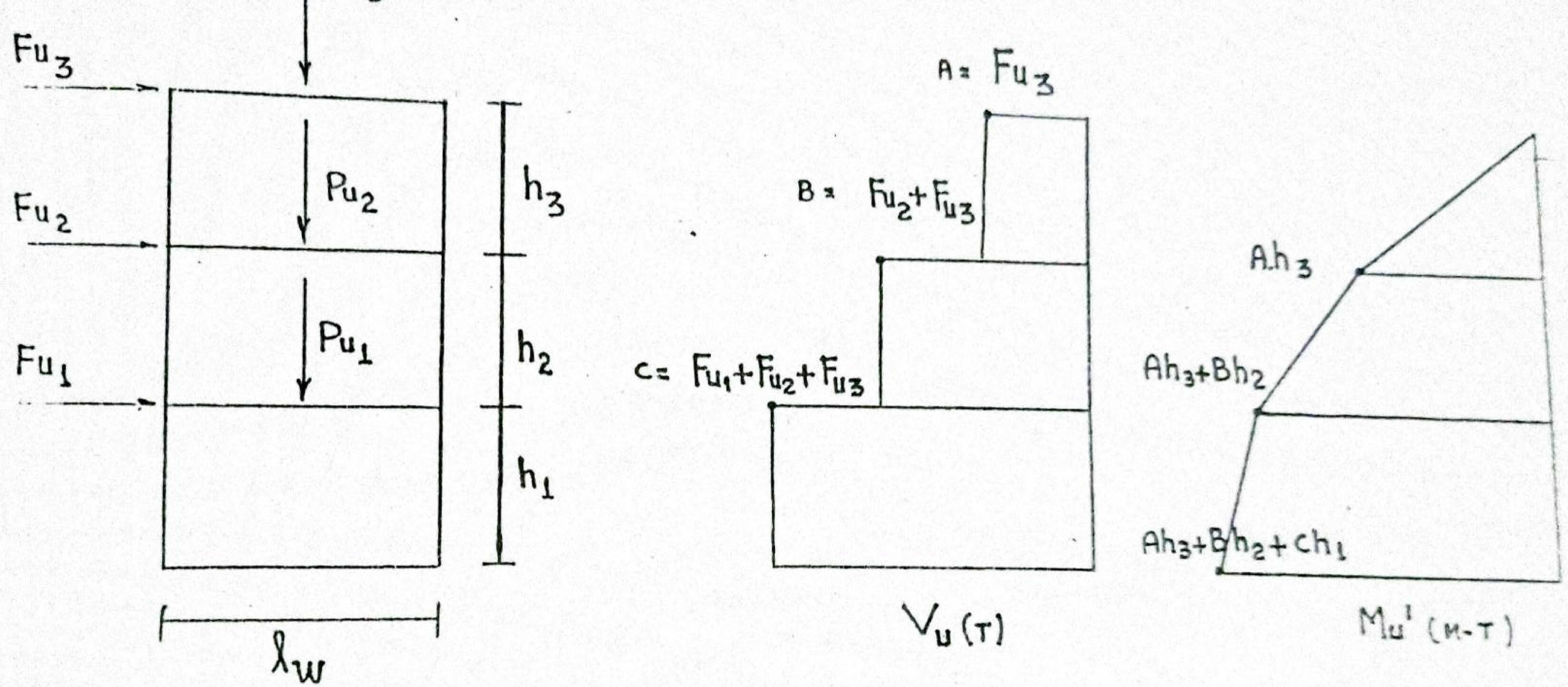
DISEÑO DE PLACAS

CON MOMENTO EN EL PISO



DIAGRAMAS DE DISEÑO

DISEÑO DE PLACAS



DIAGRAMAS DE DISEÑO

DISEÑO POR FLEXION:

CALCULOS PREVIOS

$\beta_1 = 0.85$ si $f'c \leq 280\ K/cm^2$

SI $f'c > 280$ SE DISMINUIRA 0.05 POR CADA $70\ K/cm^2$ DE AUMENTO.

VALORES CONSTANTES

$\phi = 0.90$ EN CALCULO DE AREAS (As)

$\phi = 0.70$ EN CALCULO DE MOMENTO (M_u)

$$\beta = \frac{f_y}{6000}$$

CALCULO DE

$$M_u' = Ah_3 + Bh_2 + ch_1$$

$$N_u = P_{u1} + P_{u2} + P_{u3}$$

$$d = 0.8\ l_w$$

PROCEDIMIENTO:

CALCULO:

$$As = \frac{M_u'}{\phi f_y d}$$

ASUMIENDO: $As_T = \overset{\text{NUMERO DE PISOS}}{3}\, As$ (AREA TOTAL)

$$\alpha = \frac{N_u}{l_w\, h\, f'c}$$

CUANTIA: $\rho_v = \dfrac{As}{l_w . h}$ (h: ESPESOR DE LA PLACA)

$$q = \frac{\rho_v . f_y}{f'c}$$

$$C = l_w \left[\frac{q + \alpha}{2q + 0.85\beta_1}\right]$$

$$M_u = \overset{\phi}{0.70}\, As . f_y\, l_w \left[\left(1 + \frac{N_u}{As f_y}\right)\left(0.5 - \frac{\beta_1 . C}{2 l_w}\right) - \frac{C^2}{l_w^2}\left(1 + \frac{\beta^2}{3} - \beta_1\right)\right] \text{cm-T}$$

$$\frac{M_u\ \text{cm-T}}{100} = M_u\ \text{m-T}$$

CONDICION $M_u \geq M_u'$ OK!

SI $M_u \leq M_u'$ DIMENSIONAR l_w
AUMENTAR

DISEÑO POR CORTE :

CALCULO DEL CORTE MAXIMO ADMISIBLE : $\phi = 0.85$

$V_u = Pu_1 + Pu_2 + Pu_3$

$\phi V_n = \phi\, 2.65\sqrt{f'c}\,.h.d$

si $V_u \leq \phi V_n$ "h" CORRECTO OK!

si $V_u > \phi V_n$ "h INCORRECTO DIMENSIONAR

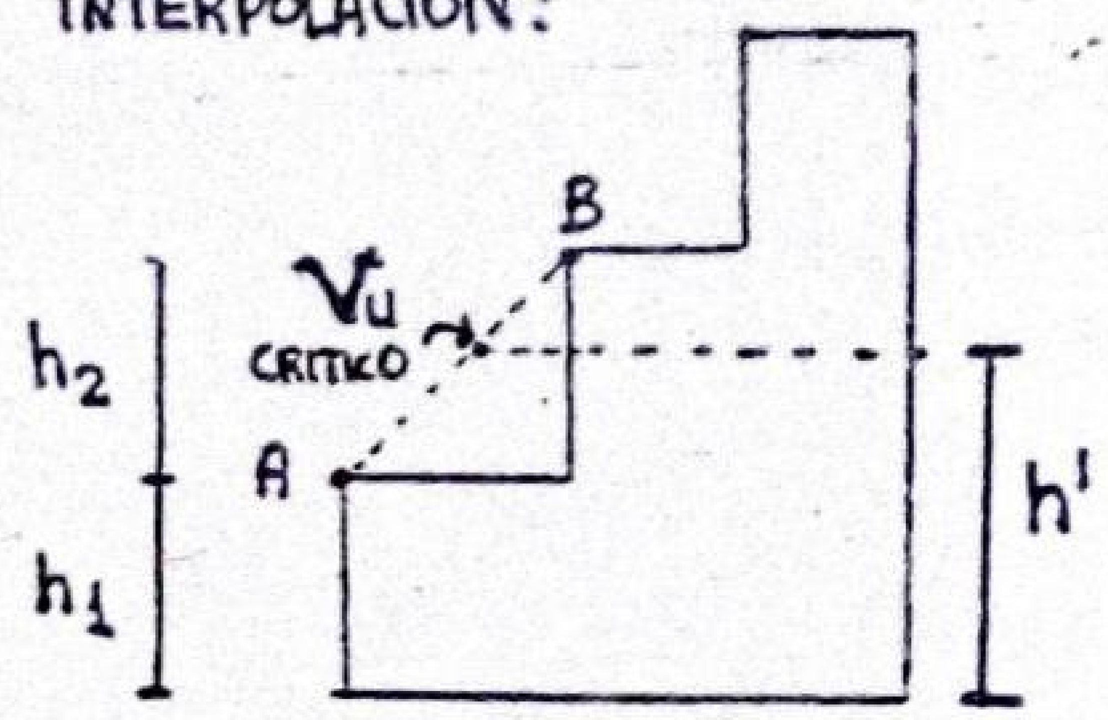
SECCION CRITICA POR CORTE :

$V_{u\,critico}$ $\begin{cases} h_w/2 \\ l_w/2 \end{cases}$ SE TOMA EL MENOR (LA CUAL SERA LA ALTURA DEL $V_{u\,critico}$) (h')

SE DETERMINARA SU VALOR EN LA CORTANTE

INTERPOLACION :

$$V_{u\,critico} = \frac{(A-B)(h_2+h_1-h')}{h_2} + B$$
(K_E)



LUEGO ↳ $\begin{cases} v_u' = \dfrac{V_{u\,critico}}{\phi\, d.h} \\ v_u = 2.7\sqrt{f'c} \end{cases}$ $v_u' < v_u$ OK!

CONDICION

CALCULO "V_c"

V_c ⇒ SE TOMA EL MENOR VALOR

$$V_c = 0.87\sqrt{f'c} + \frac{N_u}{4\, l_w h} \quad K/cm^2$$

$$V_c = 0.16\sqrt{f'c} + l_w\left[\frac{0.33\sqrt{f'c} + 0.2\left(\frac{N_u}{l_w.h}\right)}{\left(\frac{M_u}{V_u} - \frac{l_w}{2}\right)}\right] \quad K/cm^2$$

$\therefore\ v_u' > V_c/2$ ⇒ PROPORCIONAREMOS REFUERZO HORIZONTAL Y VERTICAL

REFUERZO HORIZONTAL

- CALCULO DE CUANTIA
  - $\rho_h = \dfrac{v_u' - v_c}{f_y}$
  - $\rho_{MIN} = 0.0025$
  - SE TOMA EL MAYOR VALOR
- CALCULO DE ESPACIAMIENTO S_h
  - $l_w/5$
  - $3h$
  - $45\ cm$
  - SE TOMA EL MENOR VALOR
- AREA DE ACERO
  - $As_h = \rho_h . S_h . h$
  - N° VARILLAS ϕ 1/2" @ S_h

REFUERZO VERTICAL

- CALCULO DE CUANTIA
  - $0.0025 + 0.5\left(2.5 - \dfrac{h_w}{l_w}\right)(\rho_h - 0.0025)$
  - 0.0015
  - SE TOMA EL MAYOR VAL(
- CALCULO DE ESPACIAMIENTO S_v
  - $l_w/3$
  - $3h$
  - $45\ cm$
  - SE TOMA EL MENOR VALOR
- AREA DE ACERO
  - $As_v = \rho_v . h . l_w$
  - N° VARILLAS ϕ 5/8" @ 0.125

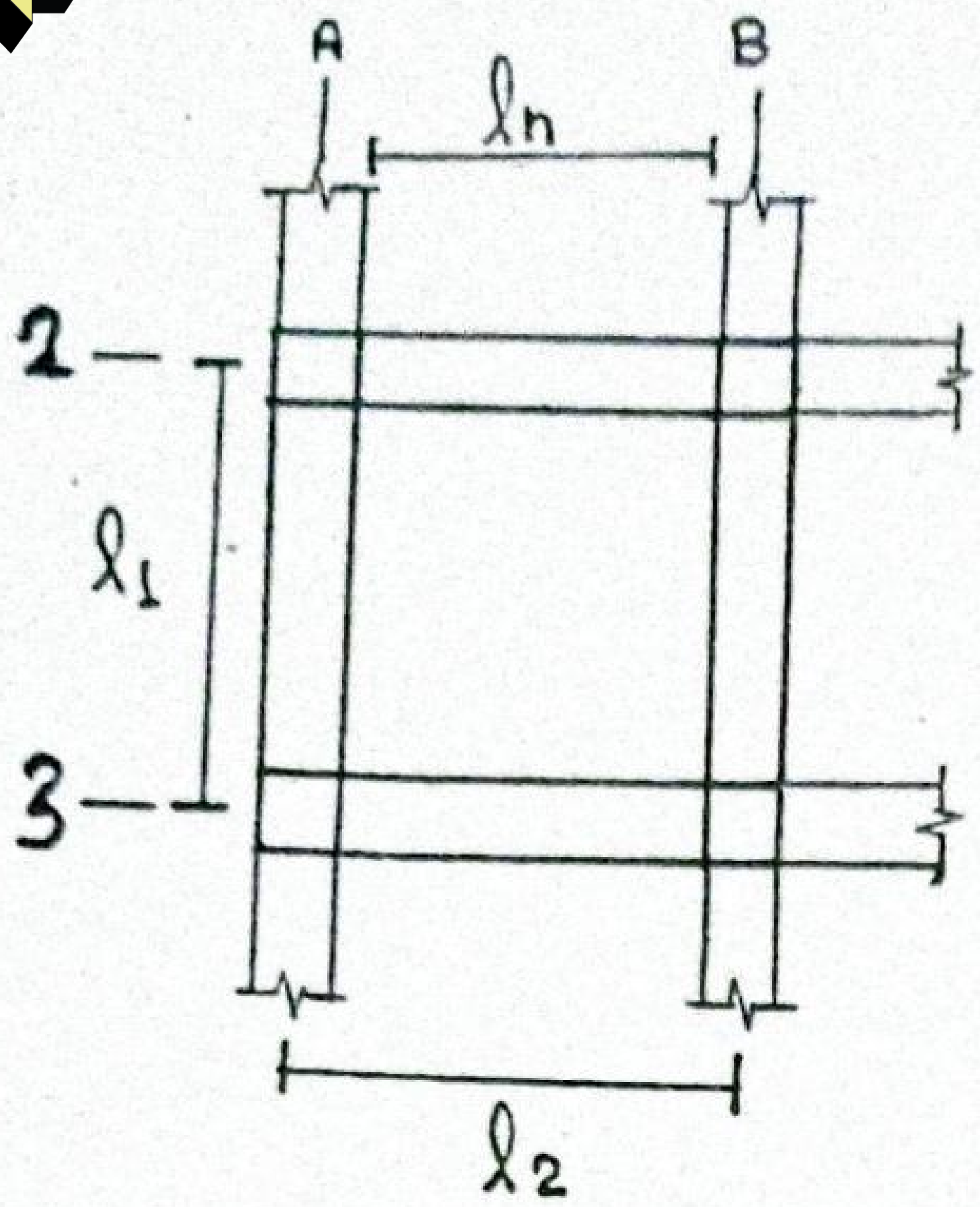
COMPARACION As

- As_v
- $As_{FLEXION}$
- SE TOMA EL MAYOR VALOR

CADA HILERA : N° VARILLAS TOTAL/2 ϕ 5/8 @ 0.125 m

(PARES)

CALCULO DEL ESPESOR DE LOSA : (h_f)



ALTURA DE VIGA $\left[\begin{array}{l} l_1 \\ l_2 \end{array}\right.$ SE TOMA EL MAYOR $h = \dfrac{l}{12}$

h_f $\left[\right.$ SE ASUME. TENIENDO EN CUENTA $h_f \geq 13\,cm$

MOMENTO DE INERCIA DE VIGAS

VIGA DE BORDE $I_b = \dfrac{b_1 h^3}{12} + \dfrac{(h-h_f)h_f^3}{12} + \left(\dfrac{h-h_f}{2}\right)^2 (h-h_f)(h_f$

VIGA CENTRAL $I_c = \dfrac{b_2 h^3}{12} + \dfrac{2(h-h_f)(h_f)^3}{12} + \left(\dfrac{h-h_f}{2}\right)^2 (2)(h-h_f)(h_f$

MOMENTO DE INERCIA DE LOSAS

$$I_{S_A} = \left(\frac{l_2 + b_1}{2}\right)\frac{h_f^3}{12}$$

$$I_{S_B} = \frac{l_2 h_f^3}{12}$$

$$I_{S_2} = I_{S_3} = \frac{l_1 h_f^3}{12}$$

CALCULO DE α

$$\alpha_A = \frac{I_b}{I_{SA}}$$

$$\alpha_B = \frac{I_c}{I_{SB}}$$

$$\alpha_2 = \alpha_3 = \frac{I_c}{I_{S2}}$$

DONDE $\alpha_m = \dfrac{\alpha_A + \alpha_B + \alpha_2 + \alpha_3}{4}$

CALCULO DE ℓ_n, β

CONSIDERANDO $\ell_2 > \ell_1$

$$\ell_n = \ell_2 - \left(\frac{b_1 + b_2}{2}\right)$$

$$\ell' = (\ell_1 - b_2)$$

$$\beta = \frac{\ell_n}{\ell'}$$

CALCULO DE β_s

$$\beta_s = \frac{2\ell_2 + \ell_1}{2(\ell_2 + \ell_1)}$$

CALCULO REAL DEL ESPESOR DE LOSA (h_f)

ESPESOR DE LOSA PARA NO CHEQUEAR DEFLEXIONES: CONDICION: $\frac{\text{CLARO LARGO}}{\text{CLARO CORTO}} \leq 2$

SI CUMPLE SEGUIMOS CON LOS CALCULOS

$$h = \left[\frac{\ell_n(800 + 0.071 f_y)}{36{,}000 + 5{,}000\beta\left[\alpha_m - 0.5(1-\beta_s)\left(1 + \frac{1}{\beta}\right)\right]}\right]$$

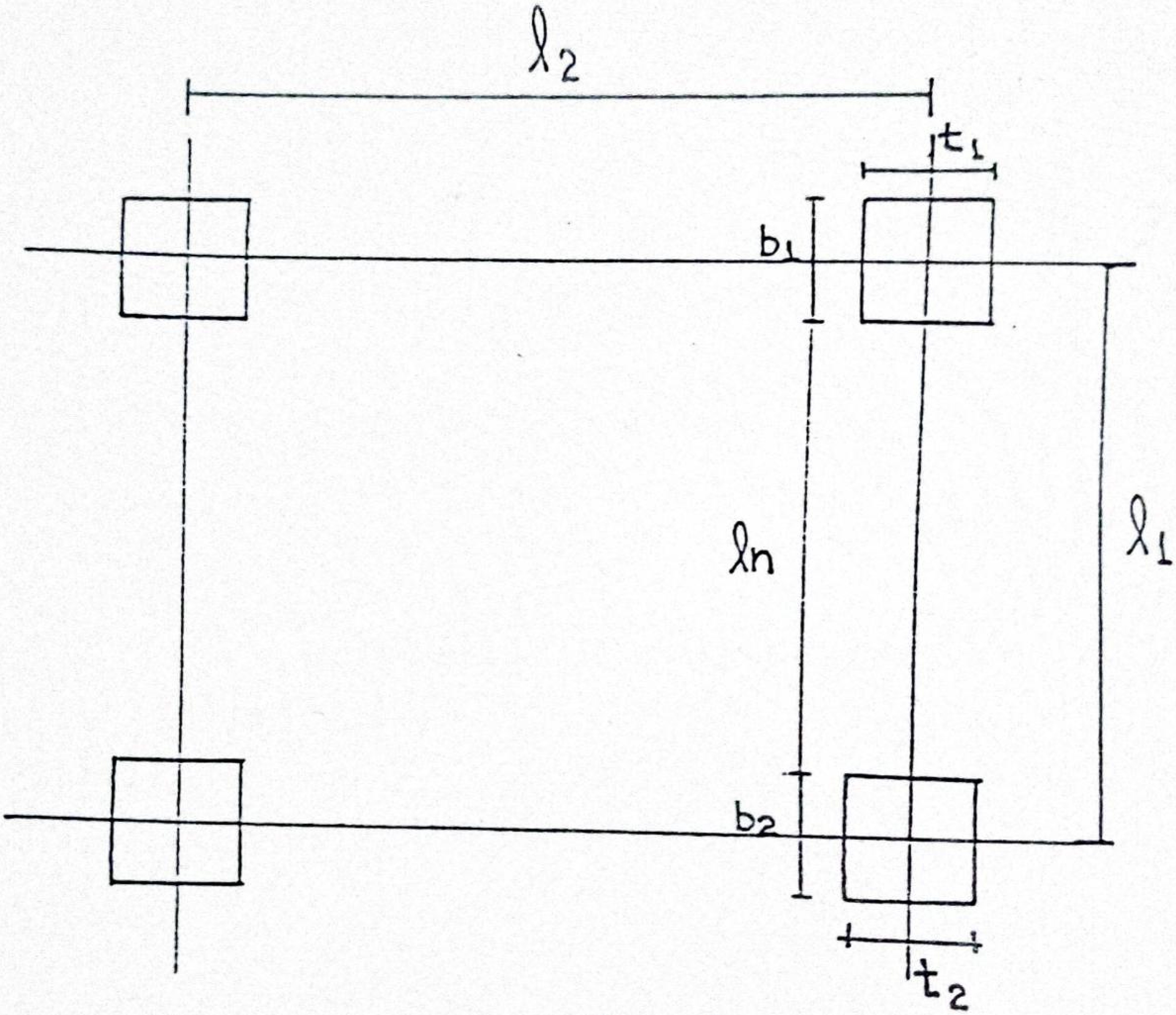
$$h' \geq \left[\frac{\ell_n(800 + 0.071 f_y)}{36{,}000 + 5{,}000\beta(1 + \beta_s)}\right]$$

$$h'' \leq \left[\frac{\ell_n(800 + 0.071 f_y)}{36{,}000}\right]$$

CONDICION: $h' \leq h \leq h''$

- SI CUMPLE $h = h_{f\,REAL}$
- NO CUMPLE $h_{f_{ASUMIDO}} = h_{f\,REAL}$

LOSAS SIN VIGAS DE BORDE



ASUMIENDO $l_1 > l_2$

CALCULO DEL ESPESOR DE LA LOSA (h)

CALCULOS

$$\ell n = \ell_1 - \left(\frac{b_1 + b_2}{2}\right)$$

$$\beta = \frac{\ell_1}{\ell_2} \quad \text{CONDICION.} \quad \beta \leq 2$$

CALCULO DEL ESPESOR DE LOSA (h)

$$h' \geq 12.5 \text{ cm (NORMA ACI 1977)}$$

$$h' \leq \frac{\ell n(800 + 0.071 f_y)}{36{,}000}$$

SE TOMA EL MAYOR VALOR

$$h = 1.10 h'$$

PARA LOSAS SIN VIGAS DE BORDE DEBERAN SER AUMENTADAS UN MINIMO DE 10% DE SU ESPESOR

VERIFICACION POR ESFUERZO DE CORTE :

CONSIDERAMOS UN ϕ 5/8" (1.98 Cm)

CALCULO "d"

RECUBRIMIENTO = 2cm

$$d = h - \text{RECUBRIMIENTO} - \phi/2$$

CALCULO DE W_u

CARGA MUERTA (W_D) → PESO PROPIO = (2400 K/M³)(d)
TABIQUERIA = 100 K/M²
ACABADOS = 100 K/M²
] W_D

CARGA VIVA (W_L)

$$W_u = 1.5\, W_D + 1.8\, W_L$$

CALCULO DE Lo , bo

$$L_o = (t_1 + d)$$

$$b_o = 2(b_1 + t_1 + 2d)$$

VERIFICACION DE CORTE POR FLEXION:

FORMULAS

$$V_U = W_U[l_1 \times l_2 - L_o^2] \quad \ldots\ldots (1)$$

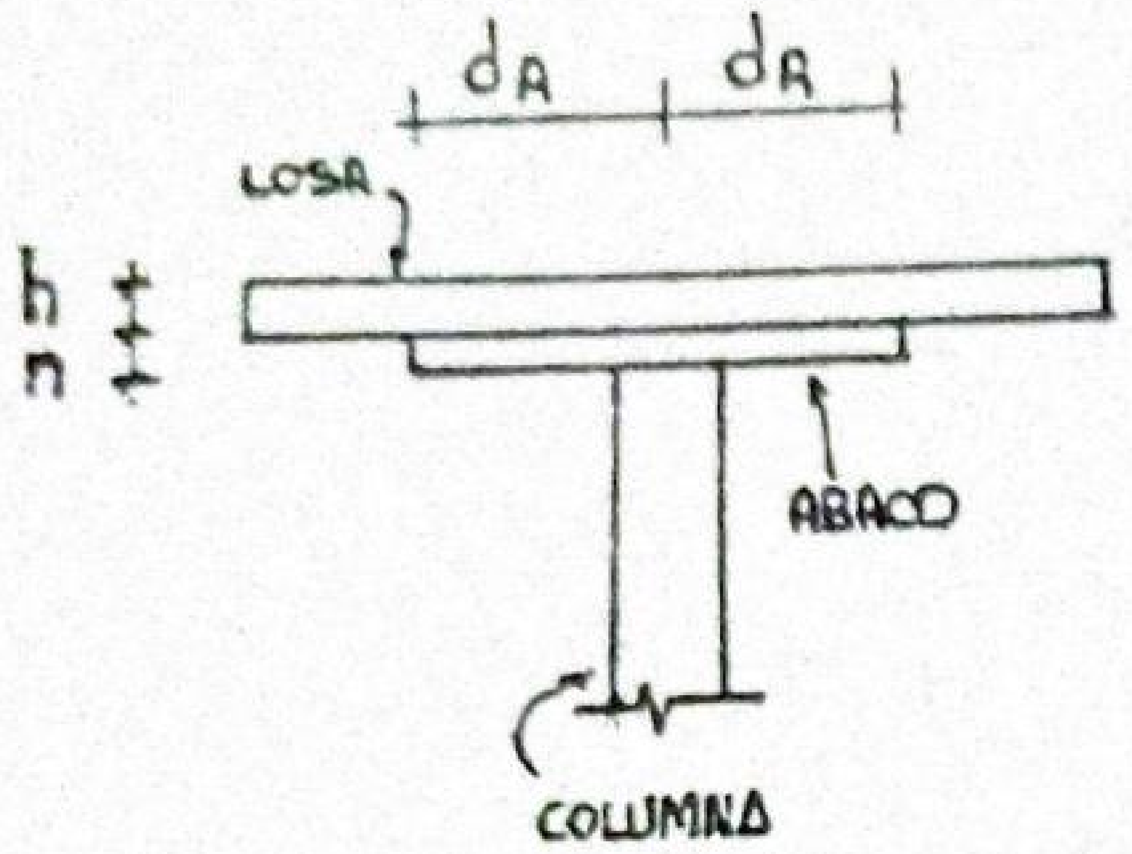
$$v_u = \frac{V_u}{b_o \cdot d} \quad \ldots\ldots (2)$$

$$v_c = \phi\, 0.27\left(2 + \frac{4}{\beta}\right)\sqrt{f'c}$$

$$v_u \leq \phi\, 1.1\sqrt{f'c}$$

SI CUMPLE : OK!

NO CUMPLE : SE DEBE USAR ABACOS



$d = h + n$

$$L_o = [t_2 + d]$$

$$b_o = [2(b_2 + t_2 + 2d)]$$

EN (1) y (2)

$v_u < \phi\, 1.1\sqrt{f'c}$ OK!

PARA LAS LOSAS CON ABACOS QUE SE EXTIENDE DESDE EL EJE DE APOYO DEBE TENER UNA DISTANCIA NO MENOR

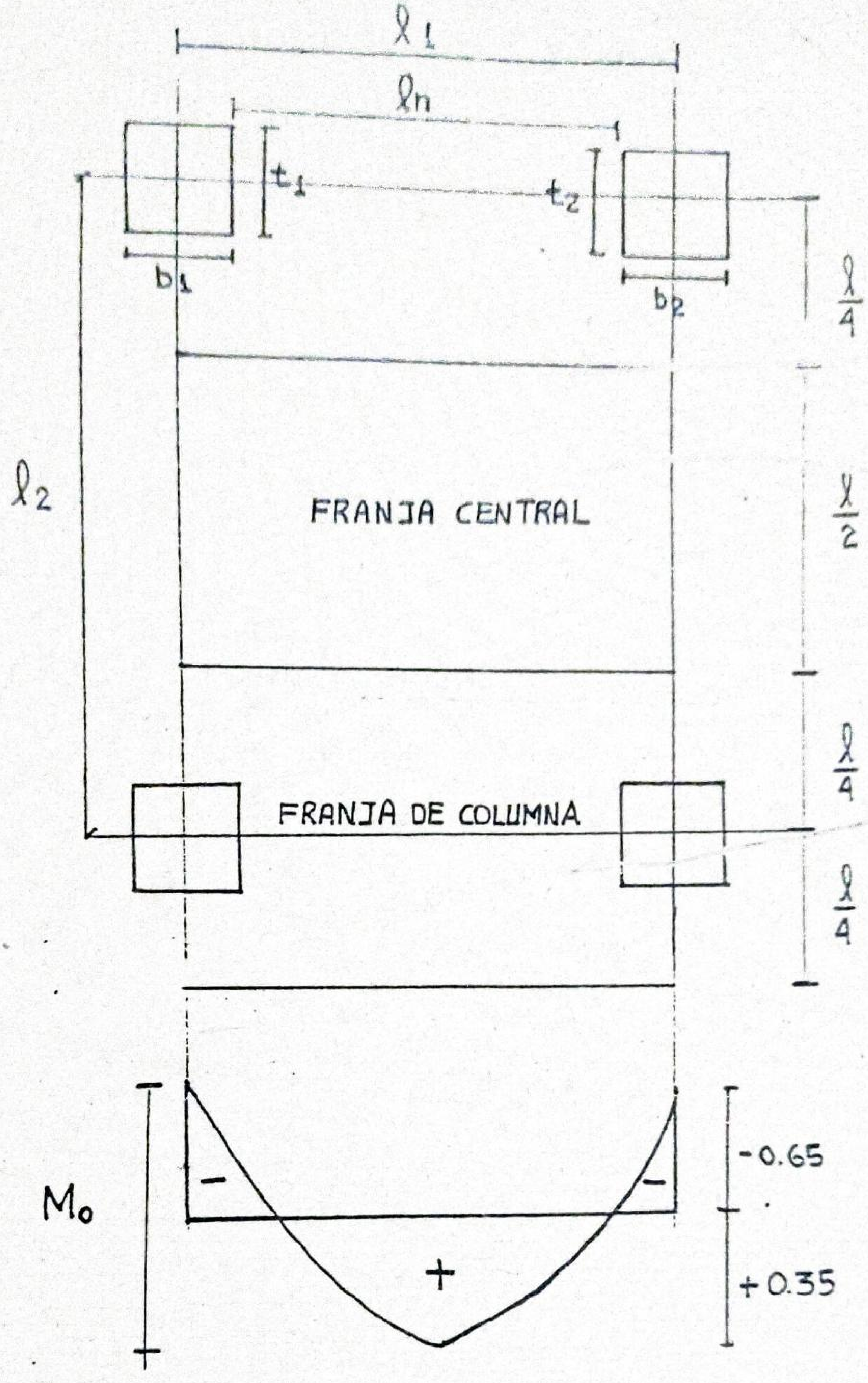
$$d_A \geq \frac{l}{6}$$

CALCULO DE MOMENTO

$$M_o = \frac{W_u\, l_2\, l_n^2}{8}$$

TABLA:

MOMENTOS PARA MARCOS INTERNOS		
M_o TOTAL	1.00 M_o	
MOMENTO +, −	−0.65 M_o	+0.35 M_o
FRANJA DE COLUMNA	75%	60%
FRANJA CENTRAL	25%	40%



l_1
l_n
t_1
t_2
b_1
b_2
l_2
$\frac{l}{4}$
$\frac{l}{2}$
$\frac{l}{4}$
$\frac{l}{4}$
FRANJA CENTRAL
FRANJA DE COLUMNA
M_o
$-$
$-$
$+$
-0.65
$+0.35$

CALCULO d

$$b = \frac{l_1}{2}$$

ASUMIENDO $\phi\, 1/2''$ (1.28) cm^2

$$d = \sqrt{\frac{M_{MAX}}{0.145\, b\, f'c}}$$

$$d = h - \text{RECUBRIMIENTO} - \phi/2$$

CALCULOS DE Mu

FRANJA DE COLUMNAS

$$-Mu_1 = 0.75\,(0.65\, M_o)$$
$$+Mu_2 = 0.60\,(0.35\, M_o)$$

FRANJA CENTRAL

$$-Mu_3 = 0.25\,(0.65\, M_o)$$
$$+Mu_4 = 0.40\,(0.35\, M_o)$$

CALCULO DE As

ASUMIR $a = 0.2d$

$$As = \frac{Mu}{0.9\, fy\, (d - a/2)}$$

$$a = \frac{As.fy}{0.85\, f'c . b}$$

$$b = \frac{l_1}{2}\ cm$$

a	As
a	As
a	As

TANTEAR MINIMO 3 VECES

SEPARACION

PARA ϕ .(cm^2)

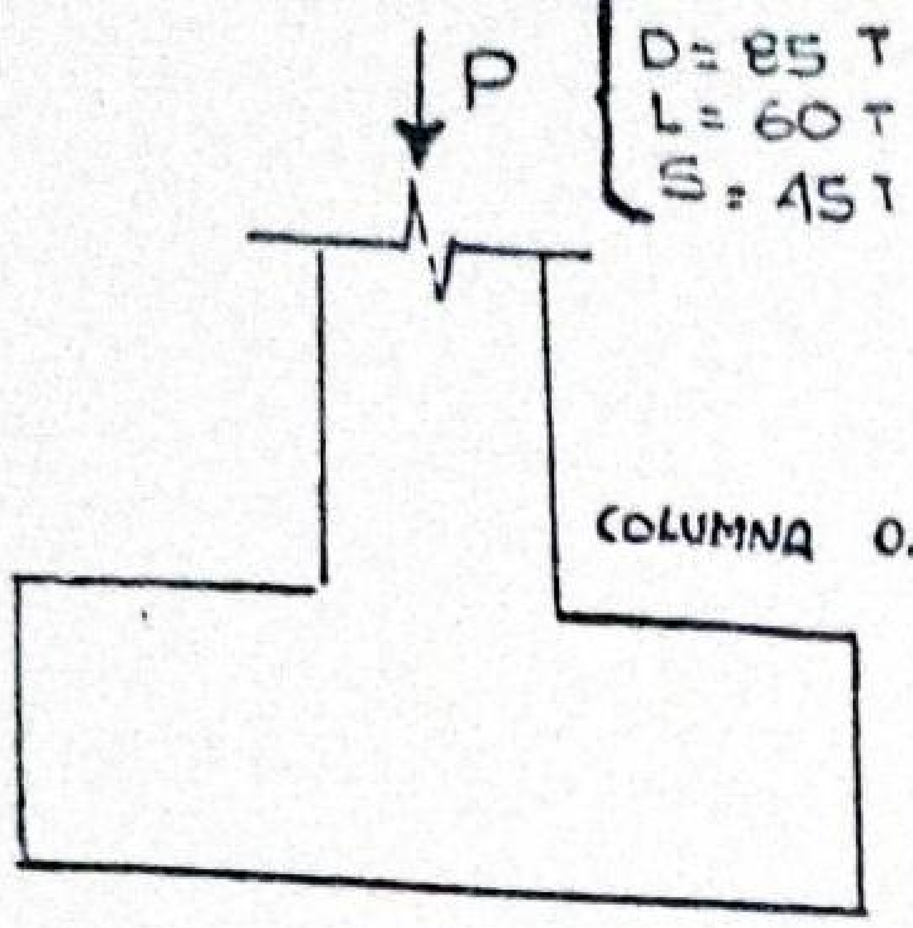
3/4 (2.85 cm^2)

$$As/ml = \frac{As\ cm^2}{b\ mt}$$

$$S = \frac{\phi \times 100}{As/ml}$$

II PROBLEMAS

1.- PROBLEMA N°(1) DISEÑAR LA ZAPATA QUE SE MUESTRA EN LA FIGURA :



$\begin{cases} D = 85\ T \\ L = 60\ T \\ S = 45\ T \end{cases}$

COLUMNA 0.5 × 0.6 mt

$\sigma_t = 2\ K/cm^2$ (ARCILLA MEDIA)

$f_y = 4200\ K/cm^2$

$f_c = 175\ K/cm^2$

A. DIMENSIONAMIENTO EN PLANTA :

$$A_Z = \frac{(1+0.08)(85+60)}{20\ TN/M^2} = 7.83\ M^2$$

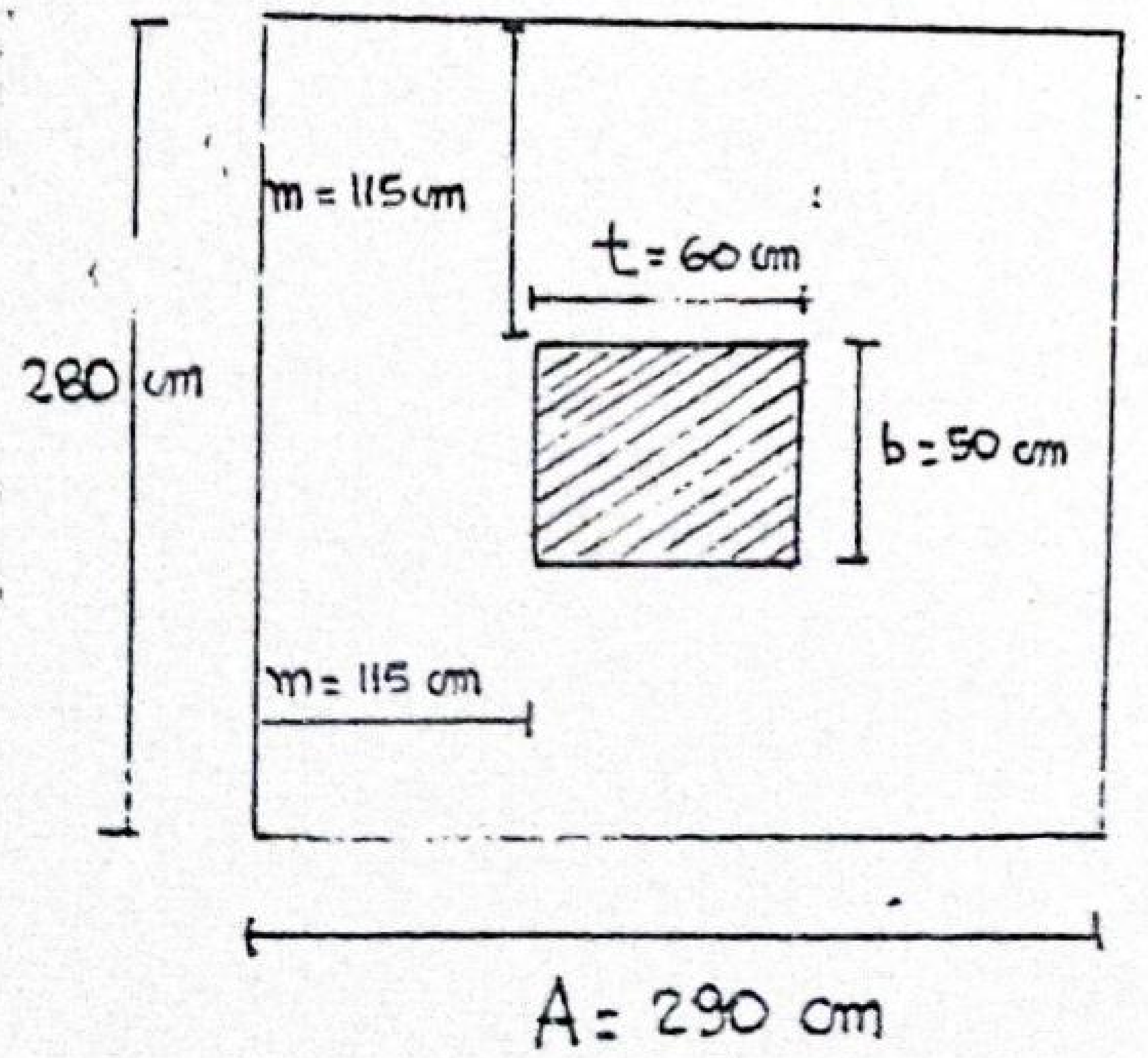
$\sim$ SE ESCOGE EL MAYOR : $A_Z = 78{,}300\ cm^2$

$$A_Z = \frac{(1+0.08)(85+60+45)}{1.33\,(20\ TN/M^2)} = 7.71\ M^2$$

COMO : $A_Z = (b+2m)(t+2m)$

$78{,}300 = (50+2m)(60+2m)$

DONDE $m = 112.433\ cm$ $\approx$ $m = 115\ cm \approx 1.15\ mt$



$A = t + 2m = 60 + 2(115) \approx 290\ cm$

$B = b + 2m = 50 + 2(115) \approx 280\ cm$

B. DIMENSIONAMIENTO EN ALTURA:

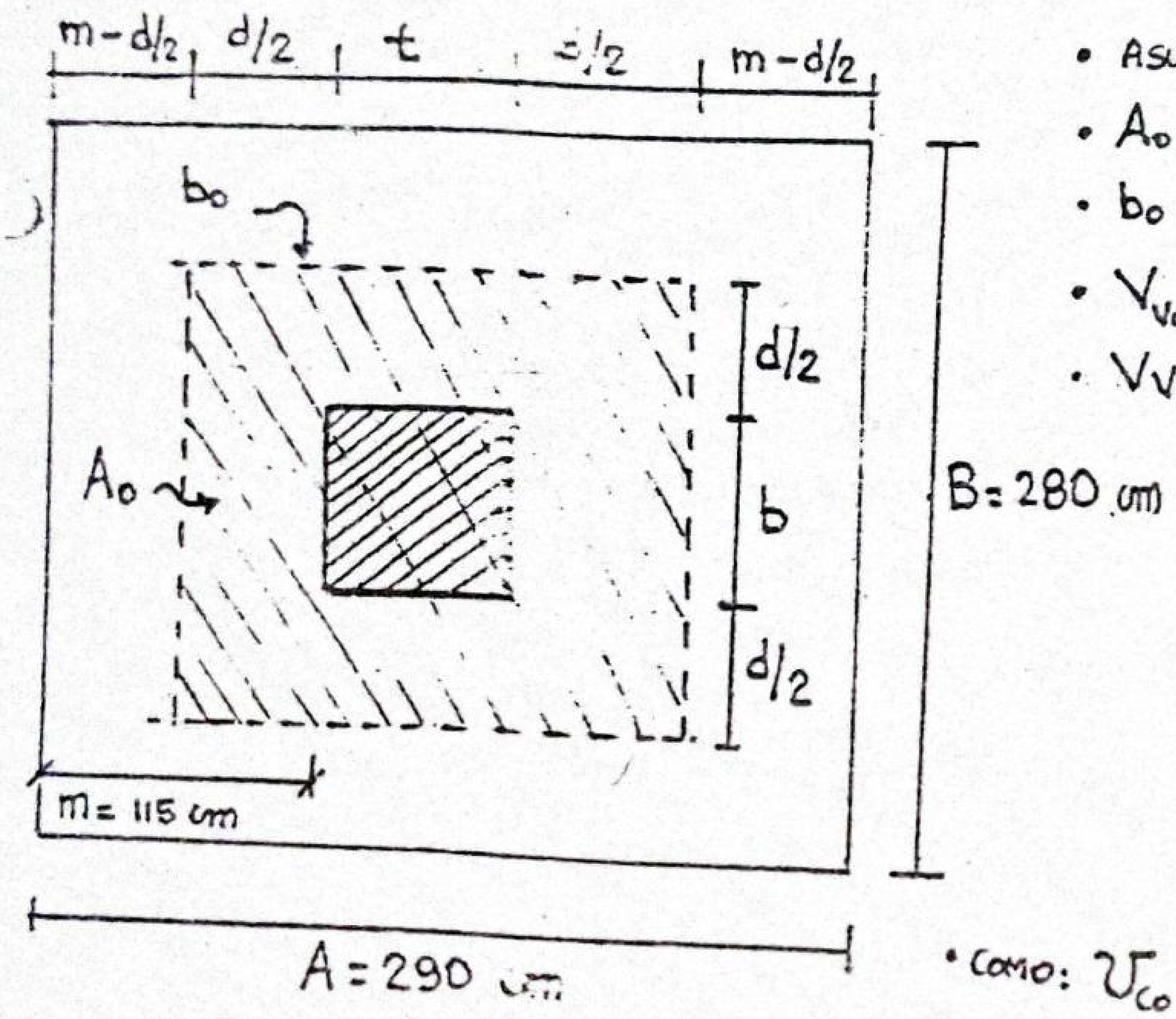
CALCULO DE P_U:

$$P_U = 1.5(85) + 1.8(60) = 235.5 \text{ TN}$$
$$P_U = 1.25(85+60+45) = 237.5 \text{ TN}$$
$$P_U = 0.90(85) + 1.10(45) = 126 \text{ TN}$$

SE ESCOGE EL MAYOR $P_U = 237.5T$

1. POR LONGITUD DE ANCLAJE: NO SE HACE

2. POR PUNZONAMIENTO (CORTE PERIMETRAL)

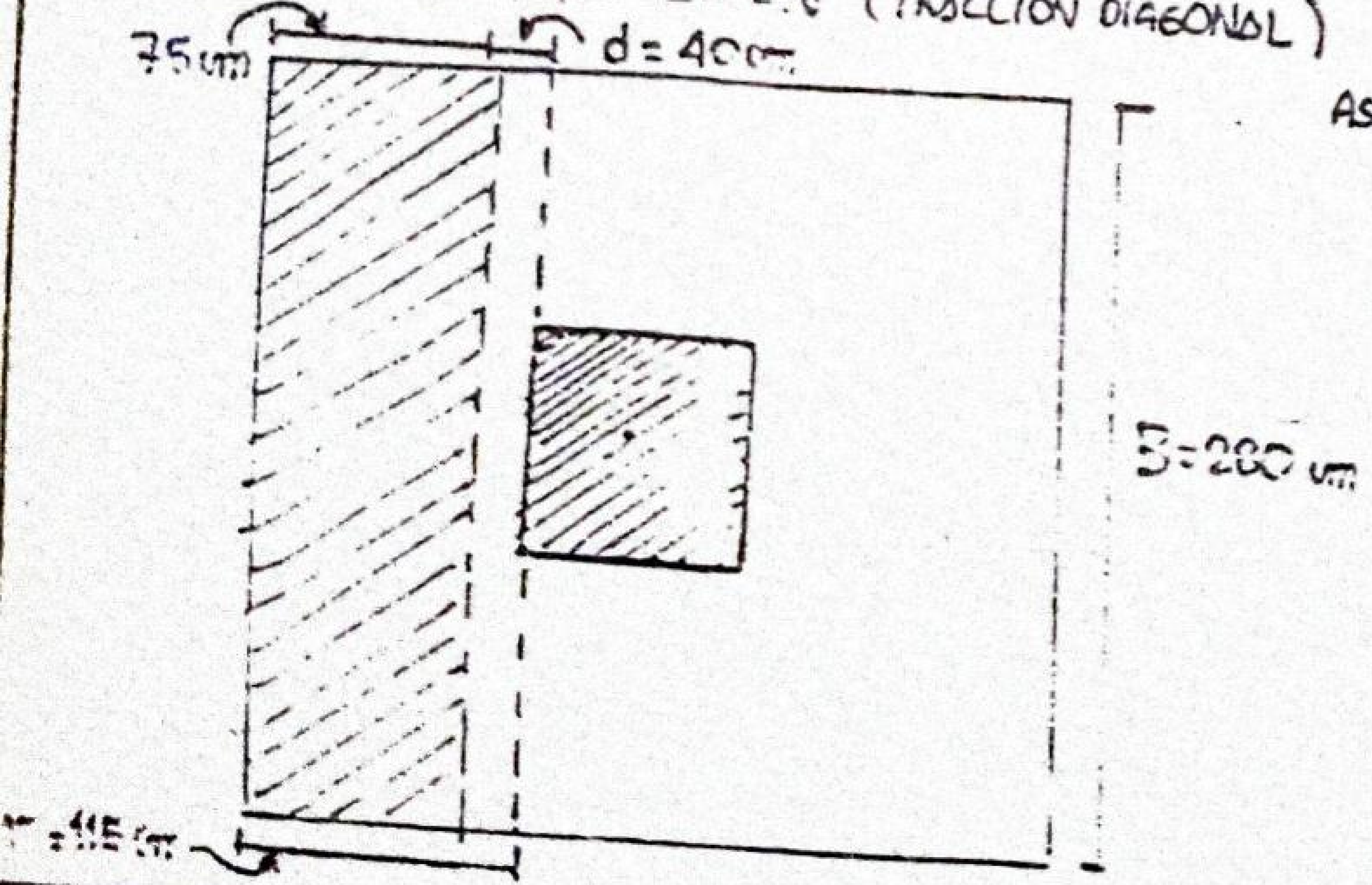


- $\sigma_{t_U} = \dfrac{237,500}{(290)(280)} = 2.92 \text{ K/cm}^2$
- ASUMIENDO: $d = 45$ cm
- $A_o = (50+45)(60+45) = 9,975 \text{ cm}^2$
- $b_o = 2(50+45+60+45) = 400$ cm
- $V_{Vo} = P_U - \sigma_{tU} \cdot A_o = 237,500 - 2.92(9,975)$
- $V_{Vo} = 208,373$ KG
- $v_{Vo} = \dfrac{208,373}{0.85(400)(45)} = 13.62 \text{ K/cm}^2$
- $v_{co} = 0.27\left(2 + \dfrac{4}{(60/50)}\right)\sqrt{175}$
- $v_{co} = 1.44\sqrt{175}$

- COMO: $v_{co} = 1.44\sqrt{175} > 1.1\sqrt{175} \} \rightarrow 14.55 \text{ K/cm}^2$

COMO $v_{Vo} = 13.62 < 14.55$ (OK!) SE TOMA $d = 45$ cm.

3: POR CORTE POR FLEXION (TRACCION DIAGONAL)



ASUMIENDO: $d = 40$ cm

- $V_U = 2.92(75)(280) = 61,320$ KG
- $v_U = \dfrac{61,320}{0.85(280)(40)} = 6.44 \text{ K/cm}^2$
- $v_c = 0.53\sqrt{175} = 7.011 \text{ KG/cm}^2$
- $v_U < v_c$ SE TOMARA $d = 40$ cm

REFUERZO EN COMPRESION :

$$M_U = 2.92(280)\frac{(115)^2}{2} = 5'406{,}350 \text{ cm.Kg} \qquad K_{U_b} = 29.2 \, K/cm^2$$

$$d \geq \sqrt{\frac{5'406{,}350}{(29.2)(280)}} \rightsquigarrow d \geq 25.7 \text{ cm} \approx 26 \text{ cm}.$$

PERALTE PARA NO USAR REFUERZO EN COMPRESION

- AREA DE ACERO (POR FLEXION)

$$\left.\begin{aligned} A_S &= \frac{M_U}{\phi . f_Y (d - a/2)} \\ a &= \frac{A_S . f_Y}{\beta_3 \cdot f'_c . B} \end{aligned}\right\} \begin{aligned} \phi &= 0.90 \\ \beta_3 &= 0.85 \end{aligned}$$

- ASUMIENDO INICIALMENTE :

$a = 6$ cm $\rightarrow A_S = 38.65$ cm^2

$a = 3.90$ cm $\rightarrow A_S = 37.59$ cm^2

$a = 3.79$ cm $\rightarrow A_S = 37.53$ cm^2

$a = 3.78$ cm $\rightarrow A_S = 37.53$ cm^2

5. POR RIGIDEZ :

- ARCILLA MEDIA : $K_{S1} = 5 \, K/cm^3$.

- $K_S = \left(\frac{20}{B}\right)\left(1 + \frac{15}{A}\right)(K_{S1}) = \left(\frac{20}{280}\right)\left(1 + \frac{15}{290}\right)(5) = 0.38$

- $d \geq 1.45(290)\left[\frac{(0.38)(290)}{15000\sqrt{175}}\right]^{1/3} \approx 34.56 \rightarrow d \geq 35$ cm

- ESFUERZO CORTANTE POR APLASTAMIENTO :

- $\nabla_a = \frac{237{,}500}{(50)(60)} = 79.17$ Kg/cm^2

- $\sqrt{\frac{A_2}{A_1}} = 2 \rightarrow \sqrt{\frac{A_2}{(50)(60)}} = 2 \rightsquigarrow A_2 = 12{,}000$ cm^2

- COMO : $A_Z = 78{,}300$ cm$^2 \rightarrow A_2 < A_Z$

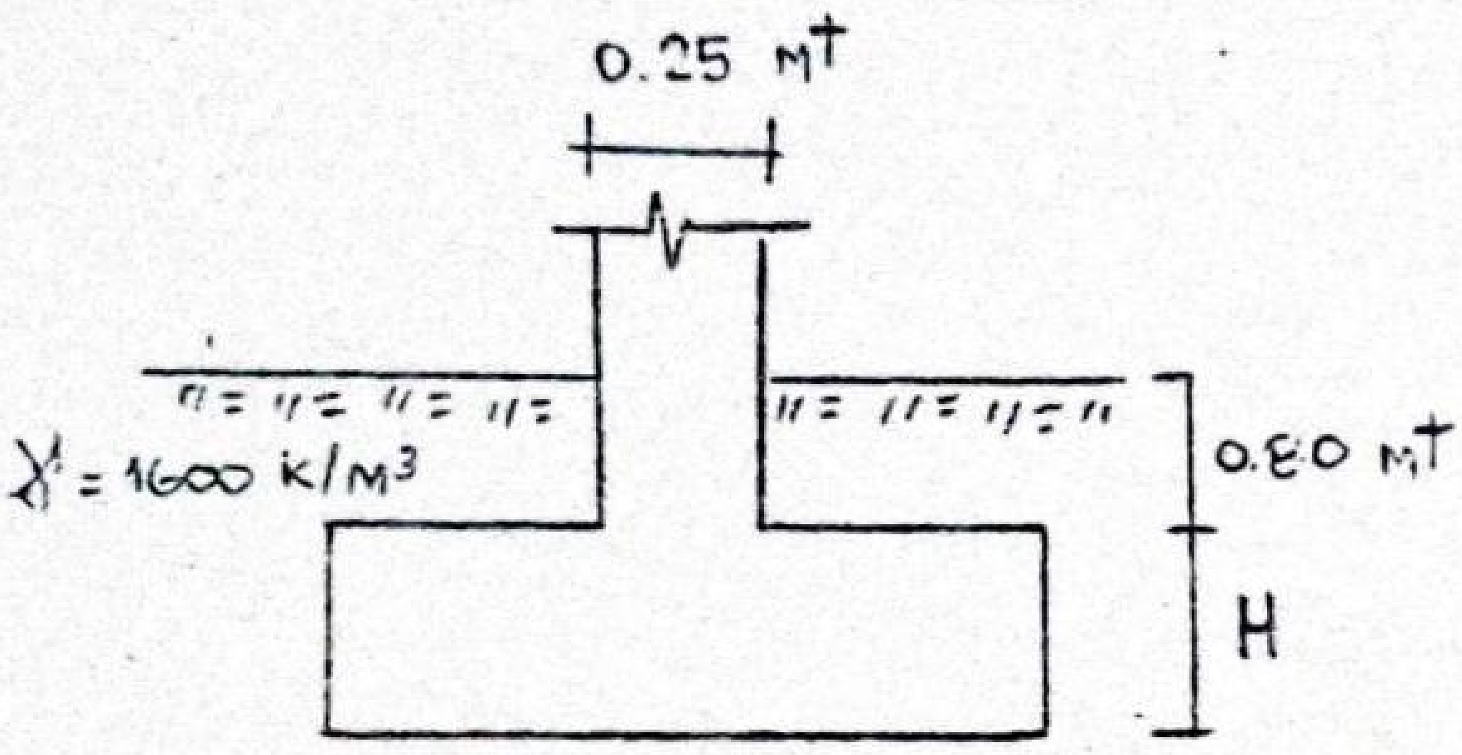
- POR LO TANTO SE TOMA :

$$\nabla_{ADMISIBLE} = (0.7)(0.85)(175)(2) \rightsquigarrow \nabla_{ADMISIBLE} = 205.25 \, K/cm^2$$

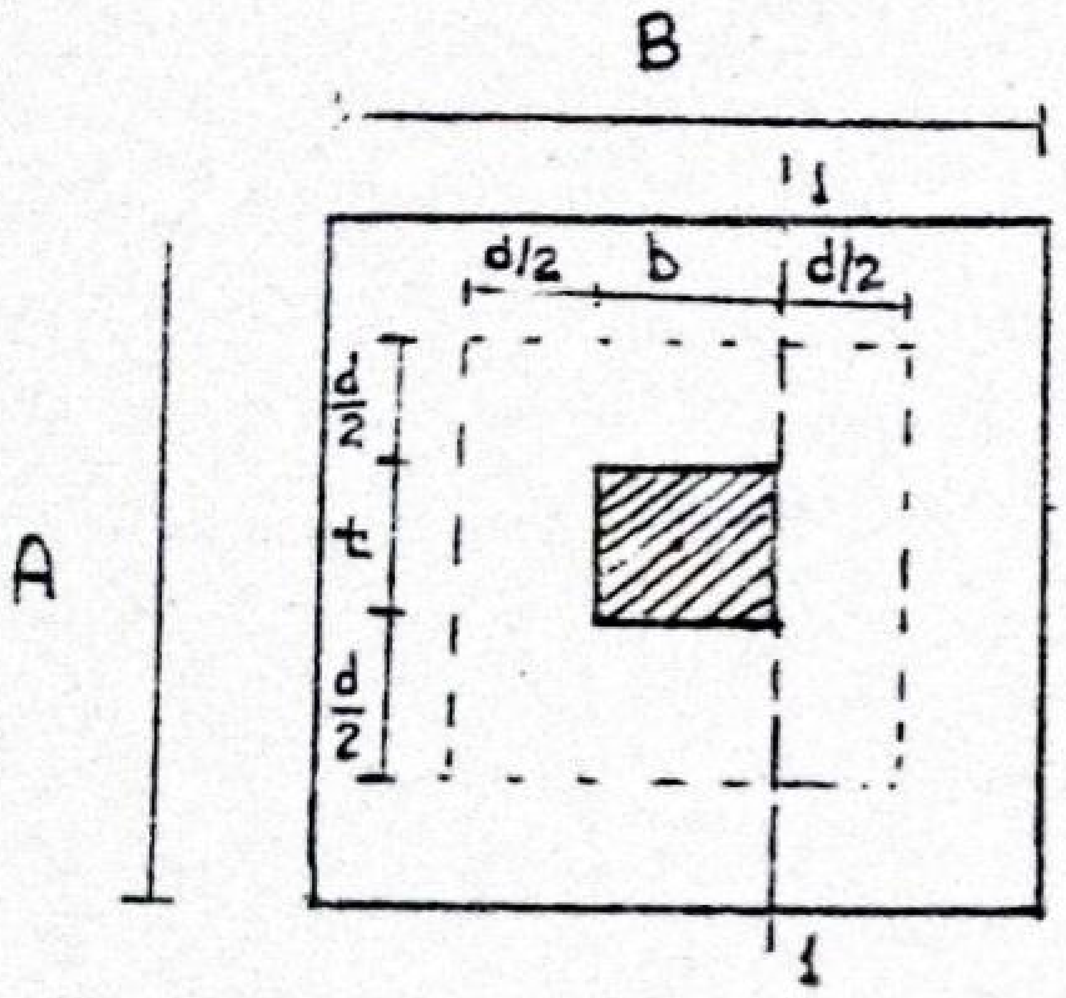
PROB (2) DIMENSIONAR UNA ZAPATA DE CONCRETO SIMPLE PARA TRANSMITIR LA SIGUIENTE CARGA $P = P_D + P_L$, EN DONDE $P_D = 30{,}000$ Kg $f_c = 100$ Kg/cm²

$P_L = 20{,}000$ Kg

NOTA UTILIZAR EL METODO ELASTICO



SOLUCION:



$P = 30{,}000 + 20{,}000 = 50{,}000$ Kg

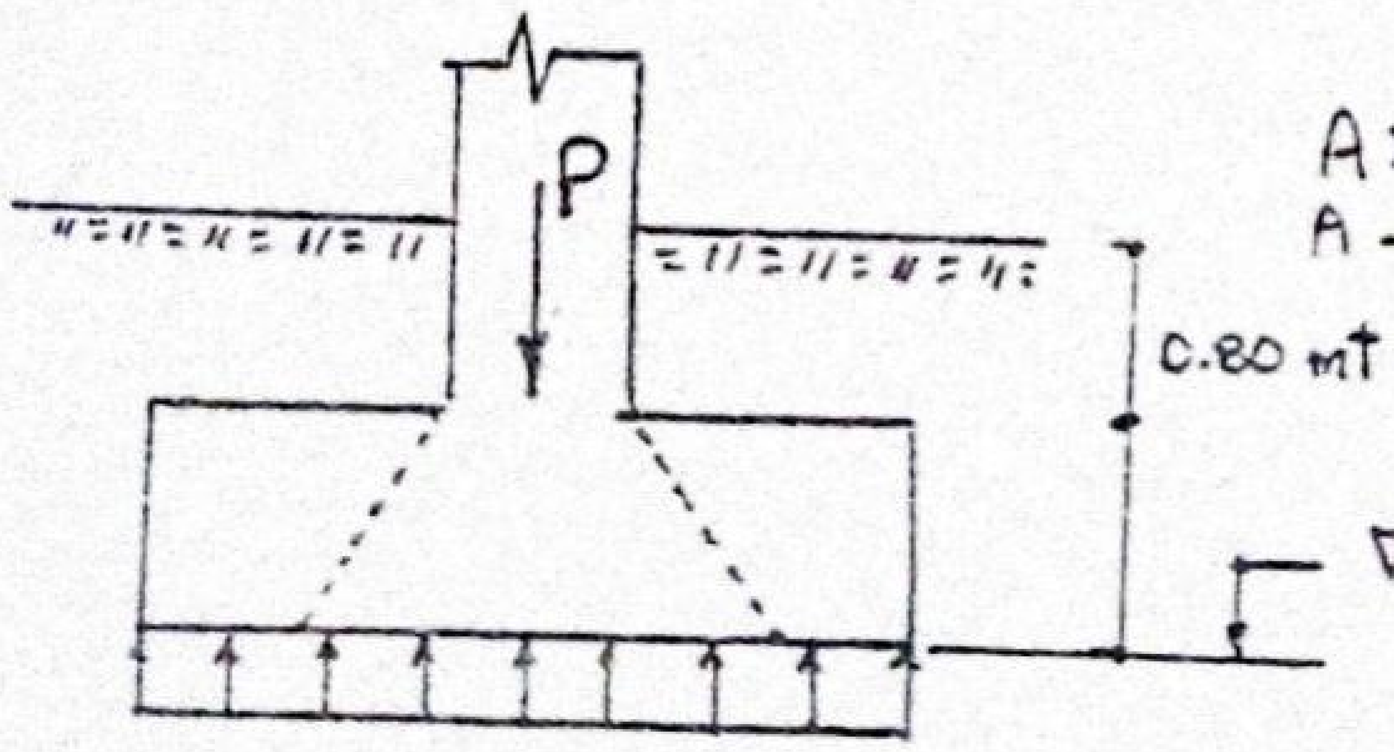
$P_{SUELO} = 0.80 \times 1{,}600 = 1{,}280$ K/m²

$P_{CONCRETO\ SIMPLE} = 0.80 \times 2{,}000 = 1{,}600$ K/m²

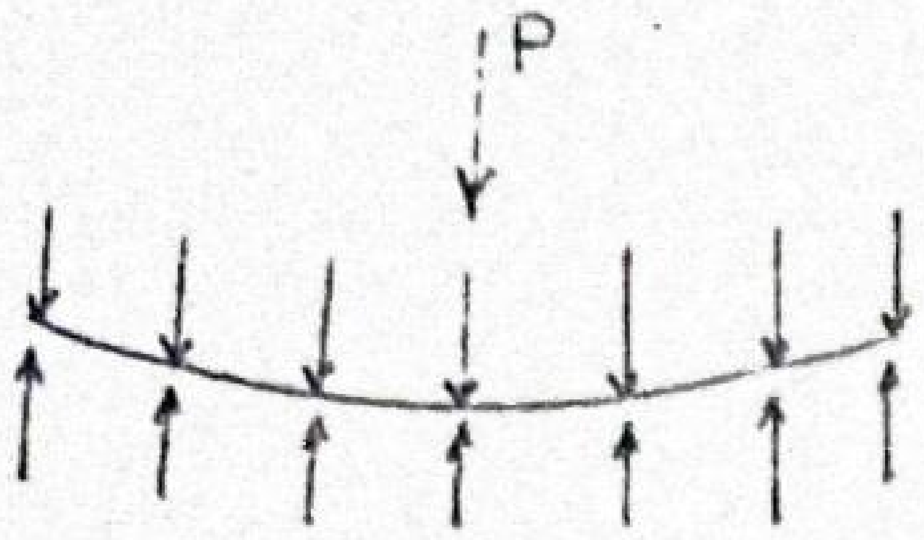
$\Sigma = 2{,}880$ K/m²

$$AREA = \frac{50{,}000 K}{(35{,}000 K/m^2 - 2880 K/m^2)} = 1.56 \, m^2$$

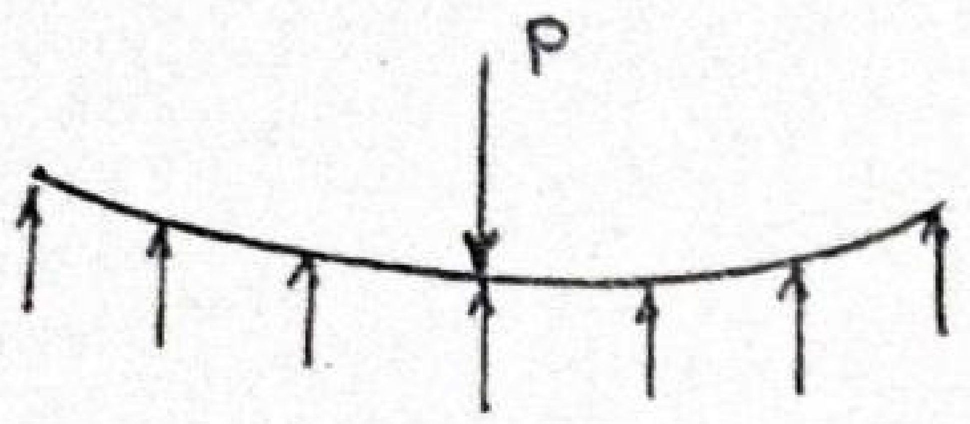
$$\left.\begin{array}{l} A \times B = 1.56 \, m^2 \\ A - B = 0.05 \, mt \end{array}\right\} \begin{array}{l} A = 1.275 \approx 1.30 \, mt \\ B = 1.225 \approx 1.25 \, mt \end{array}$$



$\sigma_t = 3.5$ K/cm² $\approx 35{,}000$ K/m²



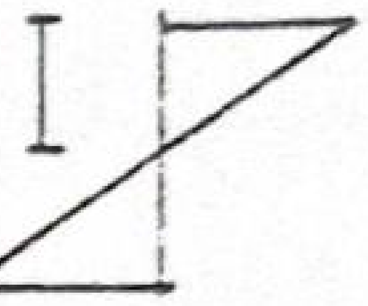
$$W_n = \frac{50{,}000}{130 \times 125} = 3.1\ \mathrm{K/cm^2}$$



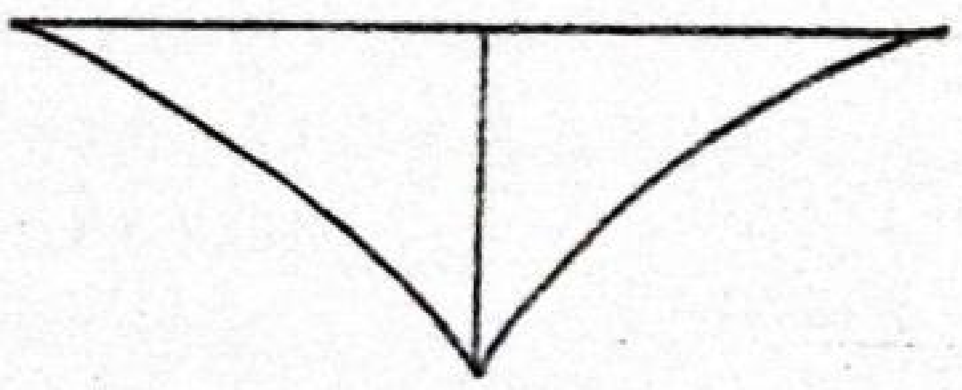
$W_n = 3.1\ \mathrm{K/cm^2}$

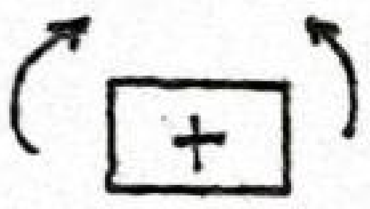
$$\sigma_c = \frac{My}{I} = \frac{M \cdot H/2}{A\frac{H^3}{12}} = \frac{6M_{1-1}}{AH^2}$$

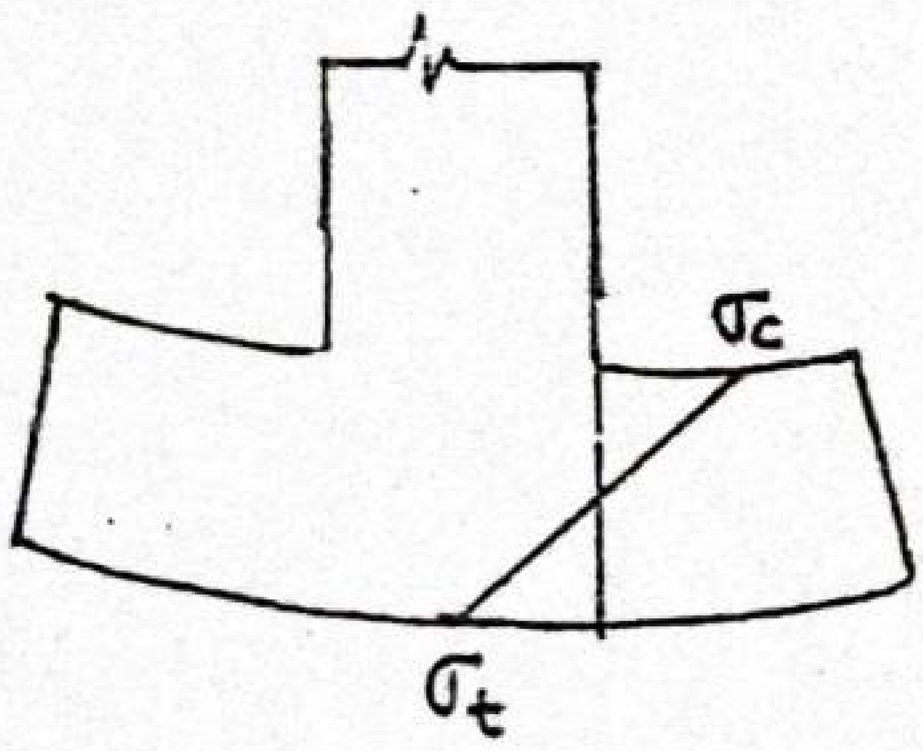
$$\sigma_t = \frac{My}{I} = \frac{6M_{1-1}}{AH^2}$$

$\frac{H}{2}$ 

D.M.F







FLEXION:

$$\sigma_c = f_c = 0.34 f'_c = 0.34 \times 100 = 34\ \mathrm{K/cm^2}$$

$$\sigma_t = f_t = 0.42\sqrt{f'_c} = 0.42\sqrt{100} = 4.2\ \mathrm{K/cm^2}$$

$\sigma_t < \sigma_c$

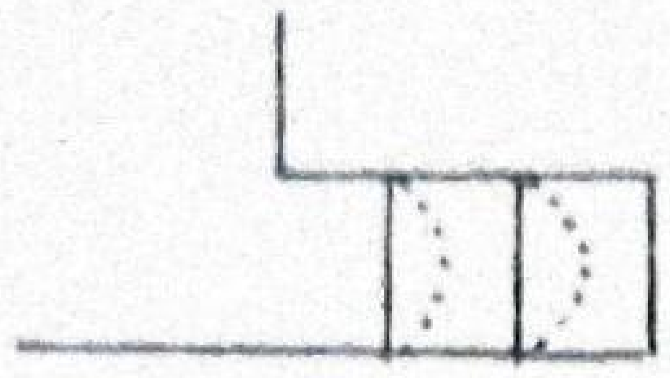
$\therefore\ H_t = \sqrt{\frac{6M_{1-1}}{A\,\sigma_t}} \Rightarrow H_t$ ES MAYOR $\qquad M_{1-1} = \frac{W_n \times n^2 \times A}{2}$ SI ES QUE $m = n$

$$M_{1-1} = (3.1)(130)(50)(25) = 503{,}750\ \mathrm{K_G\text{-}cm}$$

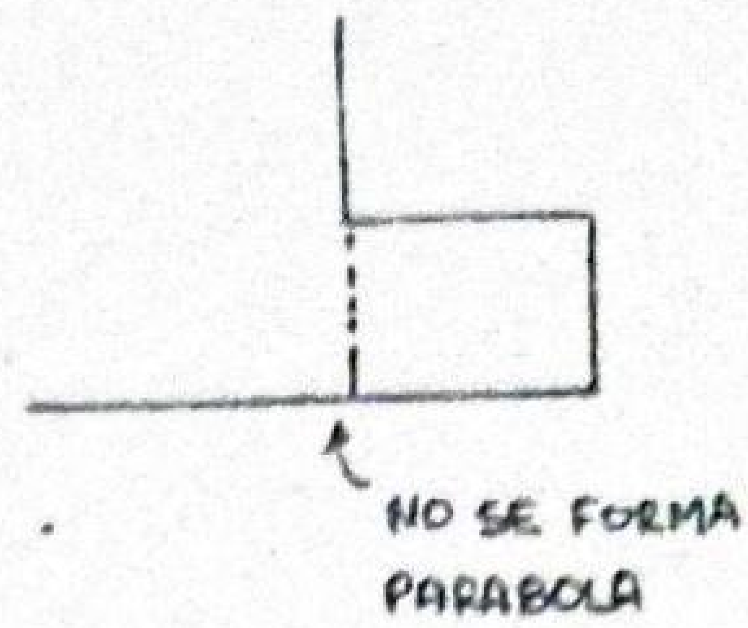
$A = 130\ \mathrm{cm}$

$\sigma_t = 4.2\ \mathrm{K/cm^2}$ $\quad\therefore\quad H_t = \sqrt{\frac{6 \times 503{,}750}{130 \times 4.2}} = 74.4\ \mathrm{cm} \Rightarrow \boxed{H = 75\ \mathrm{cm}}$

CORTE :



$$v = 1.5\,\frac{Q}{A}$$



TIPO VIGA :

a.- $v = \underset{\text{AREA}}{\frac{V_{1-1}}{A \times H}} < 0.29\sqrt{f'c} = 0.29\sqrt{100} = 2.9\ K/cm^2$

$$v = \underset{\text{AREA}}{\frac{V_{1-1}}{A \times H}} = \frac{(3.1\ K/cm^2)(130\ cm \times 50\ cm)}{(130 \times 75)\ cm^2} = 2.06\ K/cm^2$$

$2.06\ K/cm^2 < 2.9\ K/cm^2$

OK!

TIPO PUNZONAMIENTO :

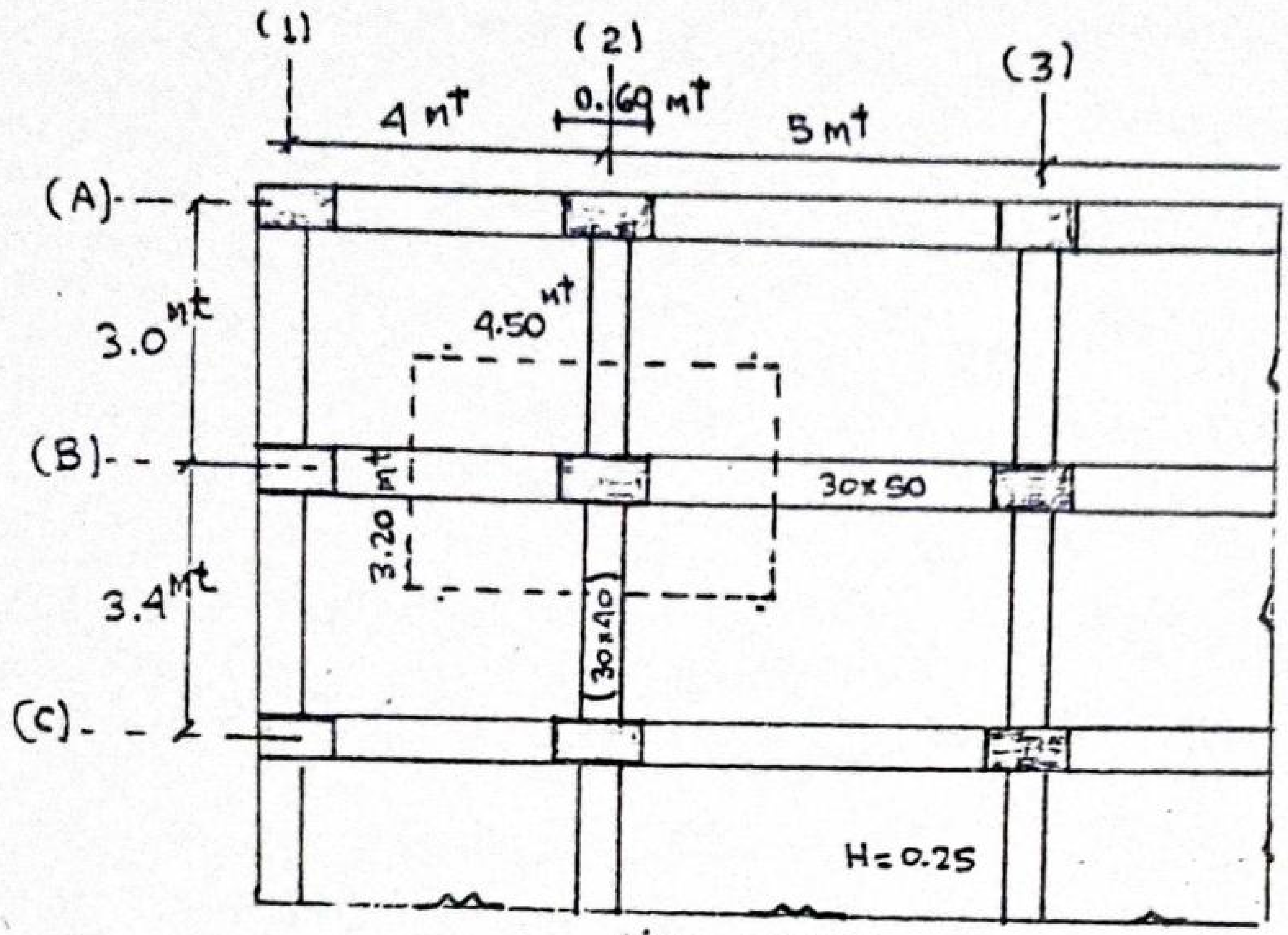
b.- $v_{cp} = \underset{\text{AREA}}{\frac{V}{A \times H}} = 3.1\,\frac{[130 \times 125 - (75+25)(75+30)]}{(100+105+100+105)\,75} = 0.58\ K/cm^2$

¡ ➔ CUMPLE !

$0.58\ K/cm^2 < 0.53\sqrt{100} = 5.3\ K/cm^2$

$0.58\ K/cm^2$ ➔ O.K!

PROB (3) DISEÑAR UNA ZAPATA AISLADA DE UN EDIFICIO DE 8 PISOS DE ALTURA CUYA PLANTA Y ELEVACION SE MUESTRAN :

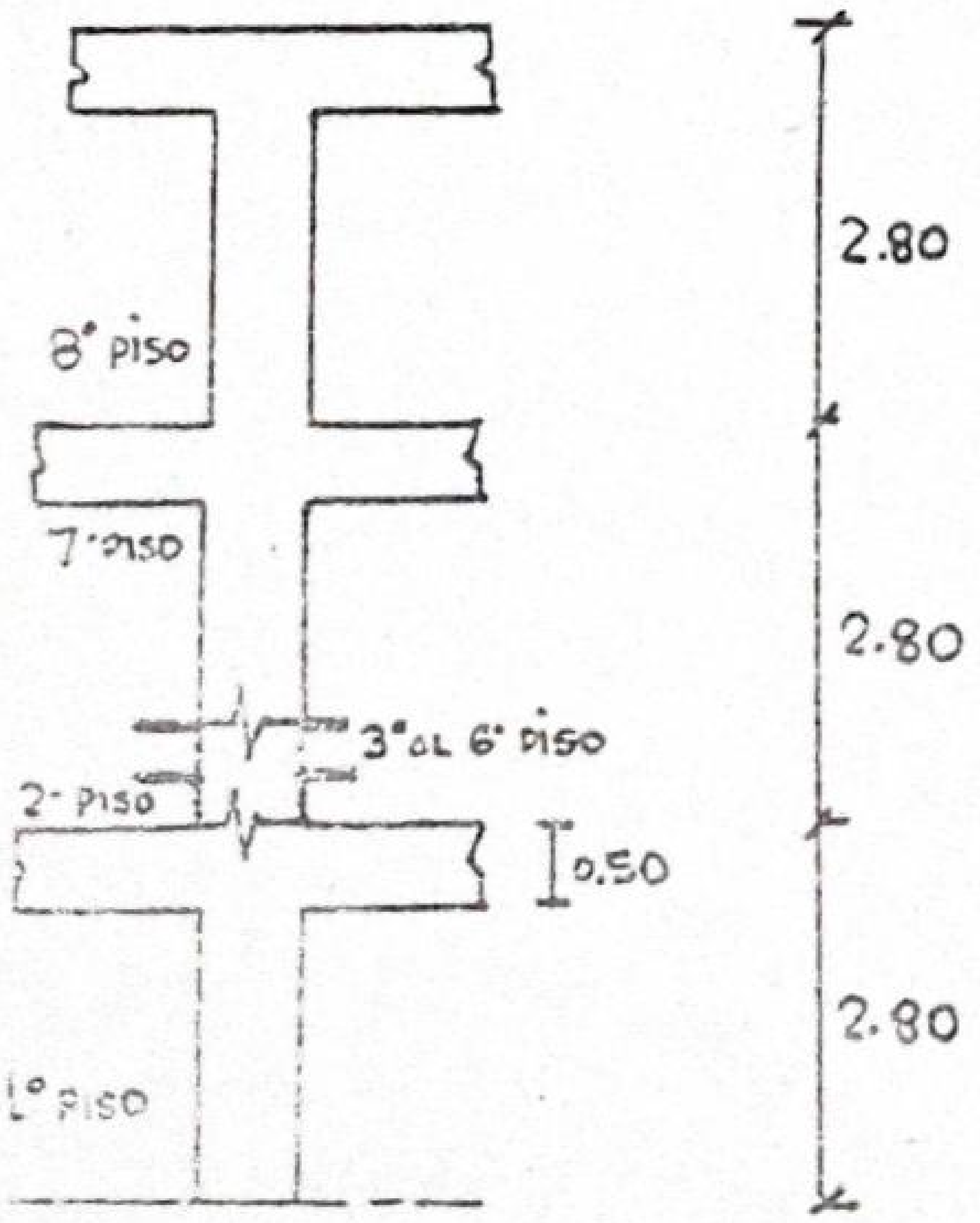


PISO TIPICO

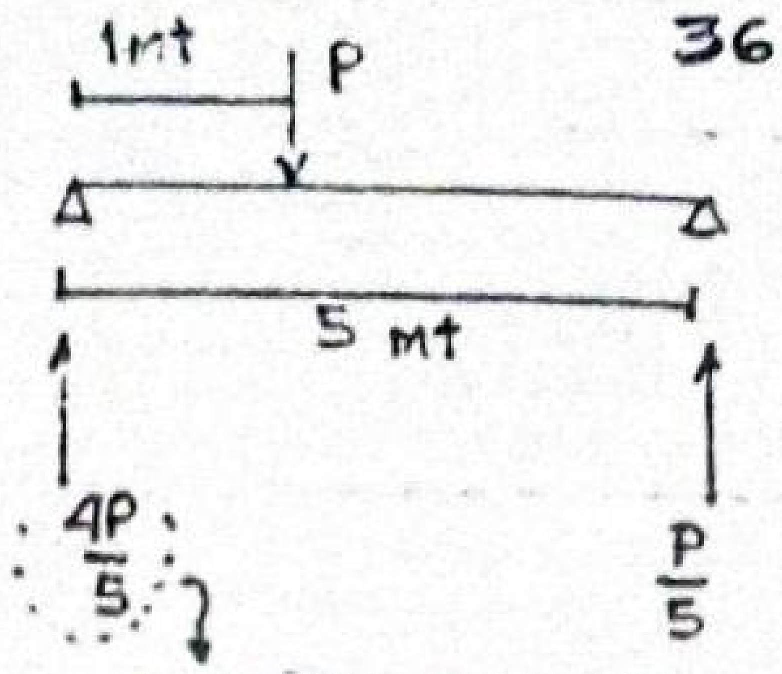
PLANTA

$f_c = 175\ K/cm^2$

$f_y = 2800\ K/cm^2$

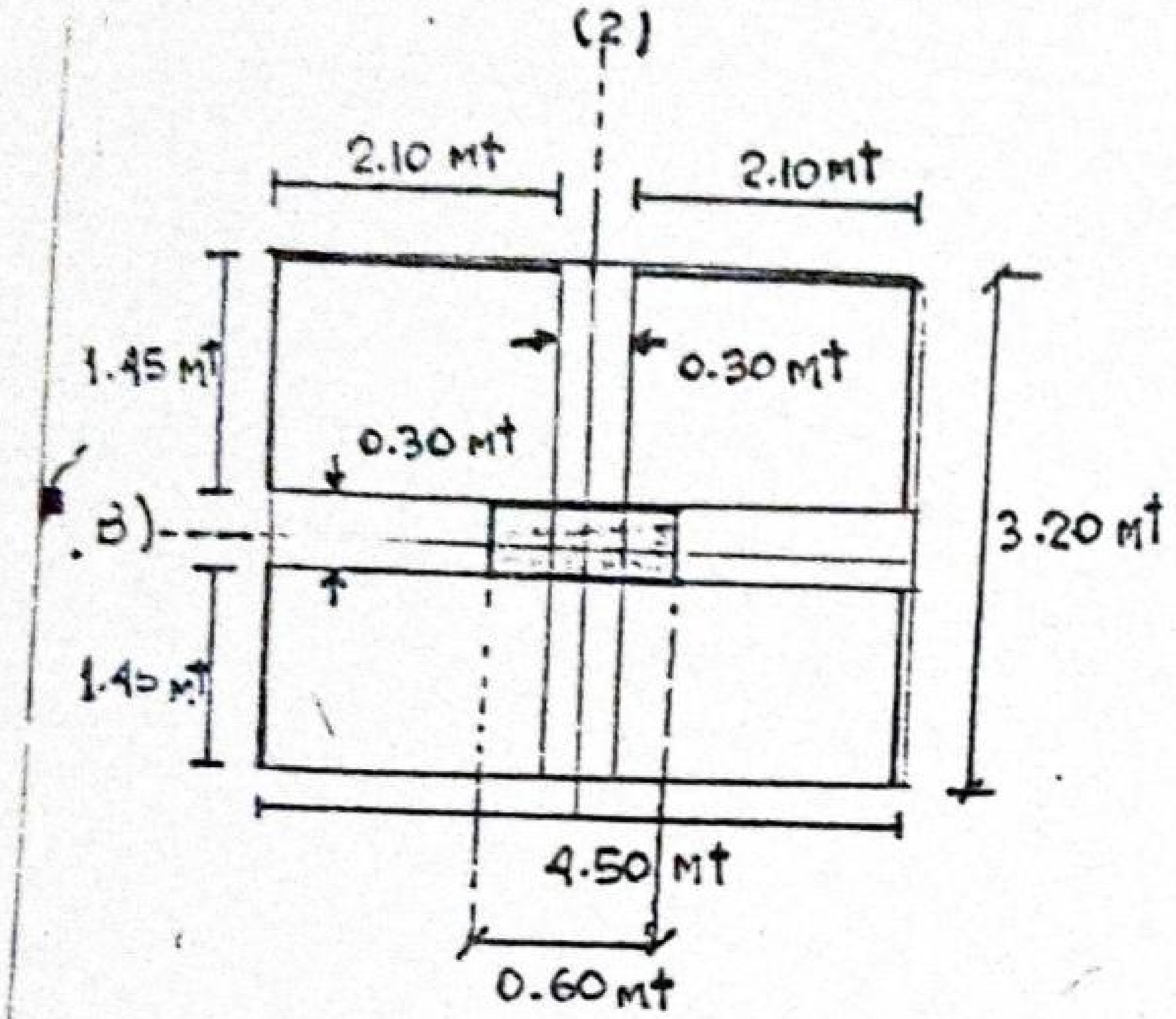


SOLUCION: SI ENTRE (2) y (3) HAY UN TABIQUE



ES EL PESO DE LA ZAPATA

1º METRADO DE CARGAS:



VIGA B : (4.50 - 0.60) = 3.90 mt

VIGA 2 : (3.20 - 0.30) = 2.90 mt

LOSA (2) : 1.45 + 1.45 = 2.90 mt

LOSA (B) : 2.10 + 2.10 = 4.20 mt

8vo PISO: CARGA PERMANENTE

P.P COLUMNA :	0.30 × 0.60 × 2.80 × 2400 =	1209.6
LOSA ALIGERADO :	3.50 × 2.90 × 4.20 =	4263.0
P. ACABADO :	3.20 × 4.50 × 100 =	1440.0
P.P VIGA B :	0.30 × 0.50 × 3.90 × 2400 =	1404.0
P.P VIGA 2 :	0.30 × 0.40 × 2.90 × 2400 =	835.2

$P_D = 9151.8$ KG

SOBRECARGA S/C : 3.20 × 4.50 × 300 = 4320.0 KG

SI TODOS LOS PISOS SON IGUALES:

$P_D = 8 \times 9151.8 = 73{,}214.4$ KG

$P_L = 8 \times 4320.0 = 34{,}560.0$ KG

CARGA ADMISIBLE DEL TERRENO : $\sigma_t = 2.5\ K/cm^2$

SEGUN LA FORMULA: $P.P.Z = \left(\frac{7 - \sigma_t}{60}\right) P_c$ $\quad P_c = P_D + P_L = 107{,}774.4$ KG

PESO PROPIO DE ZAPATA :

$$P.P.Z = \left(\frac{7 - 2.5}{60}\right)(107{,}774.4)$$

$$P.P.Z \approx 8{,}000\ KG$$

$$W = 107{,}774.4 + 8{,}000 = 115{,}774\ K$$

$$A_Z = \frac{115{,}774\ K}{25000\ K/m^2} = 4.631\ m^2 = A \times B$$

LA COLUMNA TIENE DIMENSIONES 0.30 × 0.60

1.90 × 2.20 = 4.18

2.00 × 2.30 = 4.60 (SE HALLA h Y LUEGO PESO ZAPATA < 8,000)

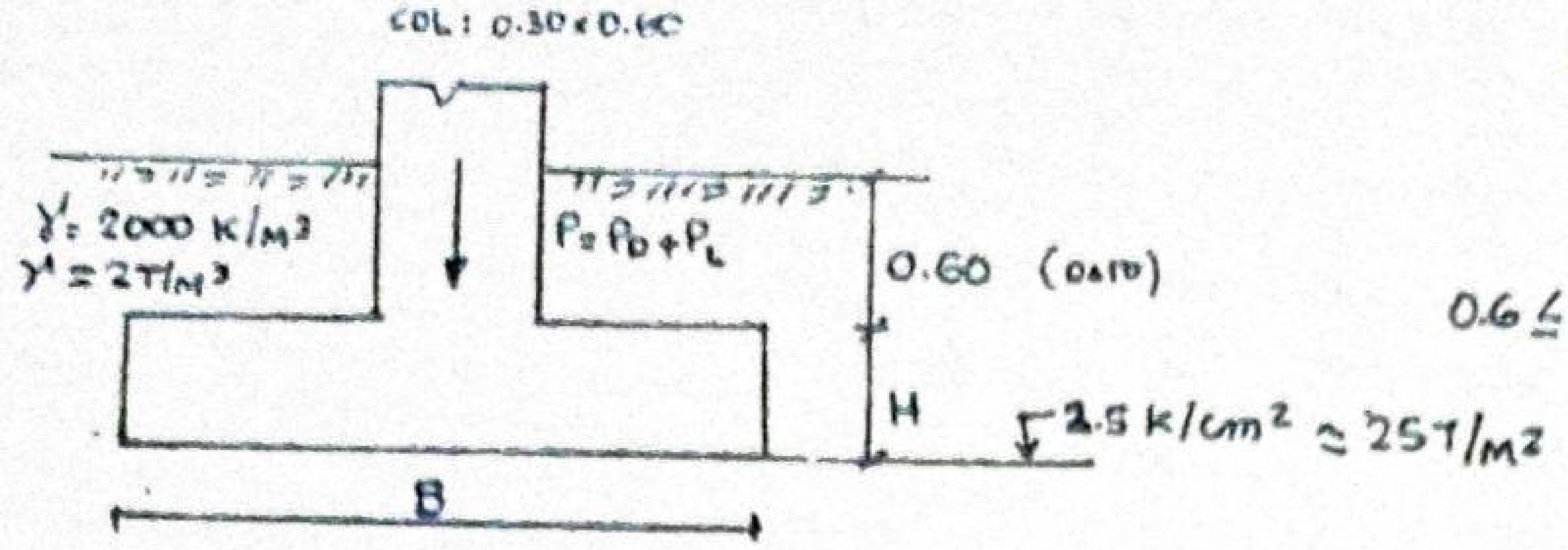
2.05 × 2.35 = 4.82

EL AREA DE LA ZAPATA SE DETERMINA POR CARGAS DE SERVICIO SEA:

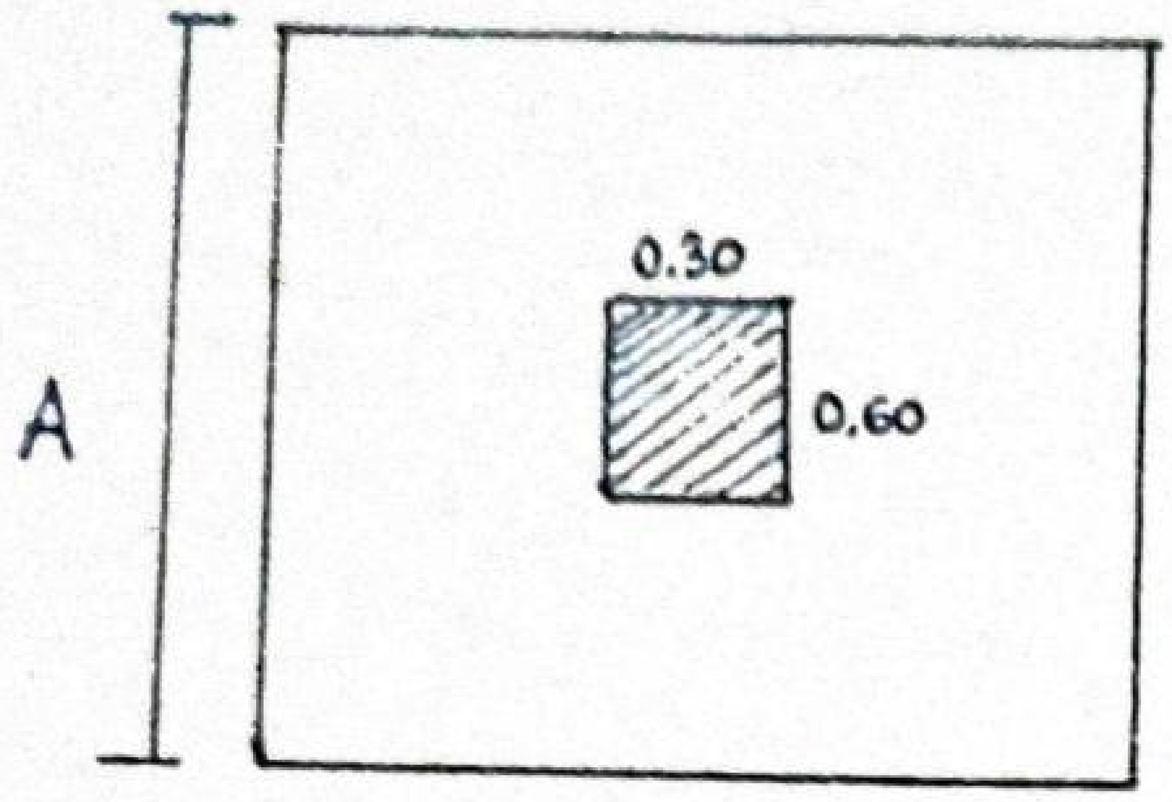
$P_D = 73214.4 \approx 74{,}000$ KG

$P_L = 34560.0 \approx 35{,}000$ KG

$P = 109{,}000\ KG \approx 109$ Tn



$0.6 \leq H \leq 0.7$ mt



AREA DE ZAPATA = A×B

$P_{SUELO} = 0.60 \times 2.00 = 1.20 \ T/m^2$

$P_{ZAPATA} = \overset{H + \text{ASUMIDO}}{0.60} \times 2.40 = 1.44 \ T/m^2$

CONCRETO ARMADO

$2.64 \ T/m^2$

$\nabla = \nabla_t - 2.64 = 25.0 - 2.64 = 22.36 \ T/m^2$

$A \times B = \dfrac{P}{\nabla} = \dfrac{109 \ T}{22.36 \ T/m^2} = 4.87 \ m^2$

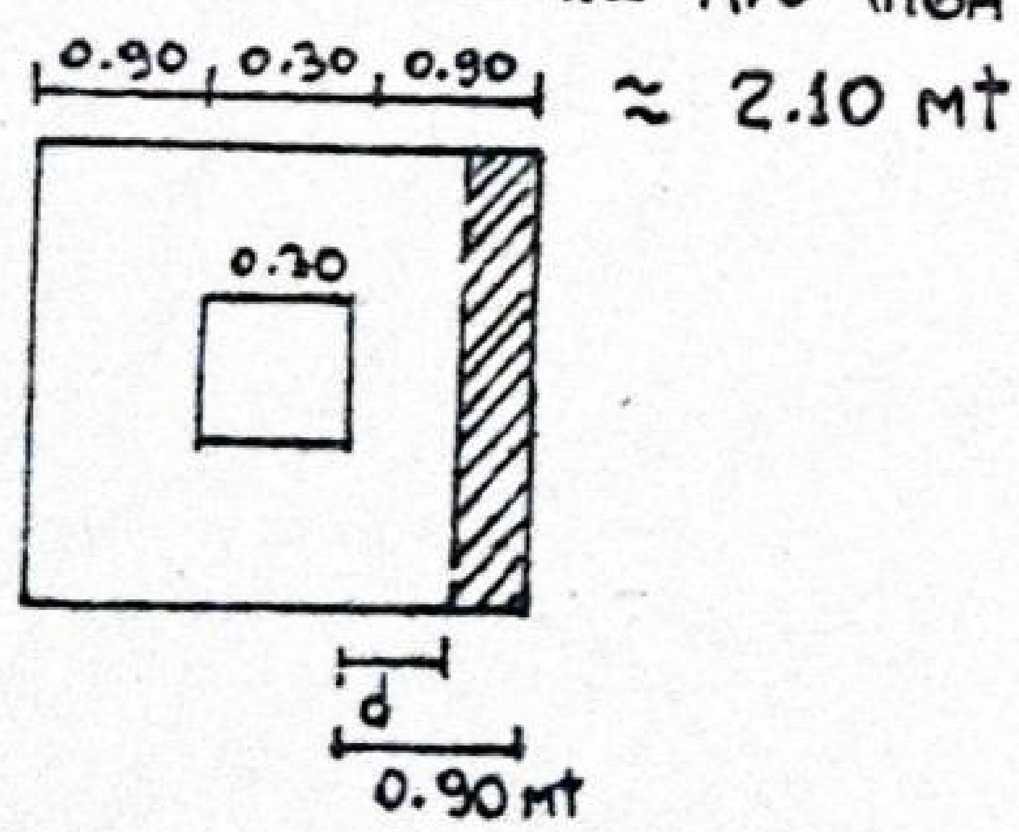
PLANTEANDO QUE LOS VOLADOS SON IGUALES $\Rightarrow$ $A - B = 0.30$, $A \times B = 4.87$ $\begin{cases} A = 2.36 \approx 2.40 \text{ mt} \\ B = 2.06 \approx 2.40 \text{ mt} \end{cases}$

CARGA UNIFORME EFECTIVA : $P_U = 1.5 \ P_D + 1.8 \ P_L$

$P_U = 1.5(74^T) + 1.8(35^T) = 174^T$

$W_{nu} = \dfrac{P_U}{A \times B} = \dfrac{174{,}000}{2.40 \times 2.40} = 3.45 \ K/cm^2 \approx 3.50 \ K/cm^2$

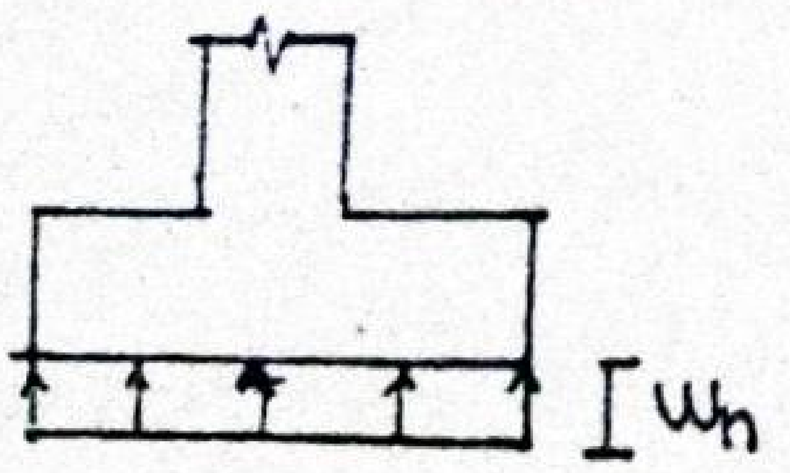
CORTE : CORTE TIPO VIGA :



$\phi V_c^{chad} = \phi . 0.53 \sqrt{f'c} \ b.d = 0.85 \times 0.53 \sqrt{175} \times 240 \times 52.5$

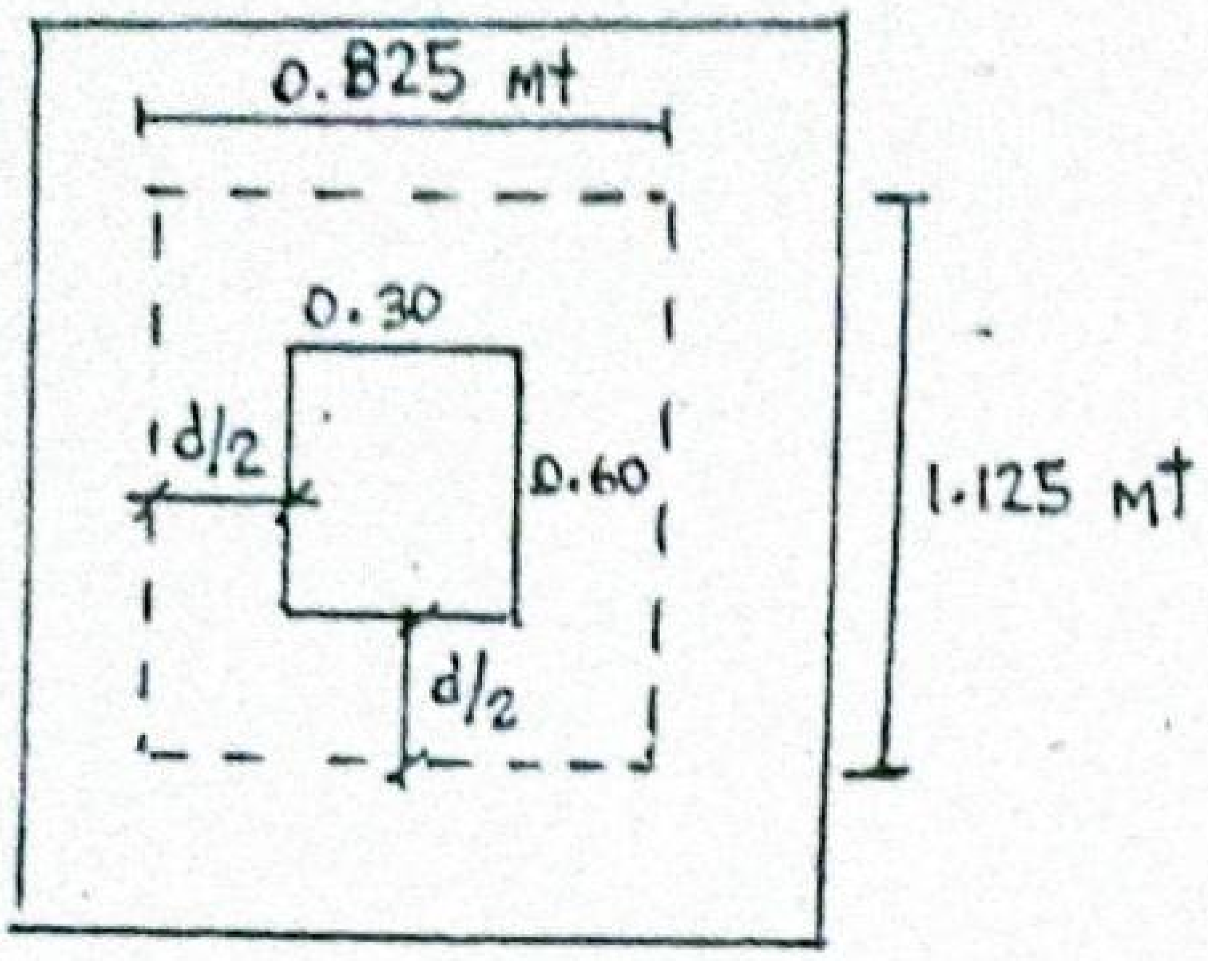
$\phi V_c = 75090.4$

$V_{cu} = \underbrace{W_n}_{3.5 \times 2.4} \times A (90 - 52.5) = 31{,}500 \ K_G < 75{,}090.4 \ OK$



PUNZONAMIENTO:

$H_{asumido} = 0.60 \qquad d = 0.60 - 0.075 = 0.525$



$V_U \leq \phi V_n$

$V_c = \left(0.53 + \frac{1.1}{\beta_c}\right)\sqrt{f'c}\,.b.d \leq 1.1\sqrt{f'c}\,.b.d$

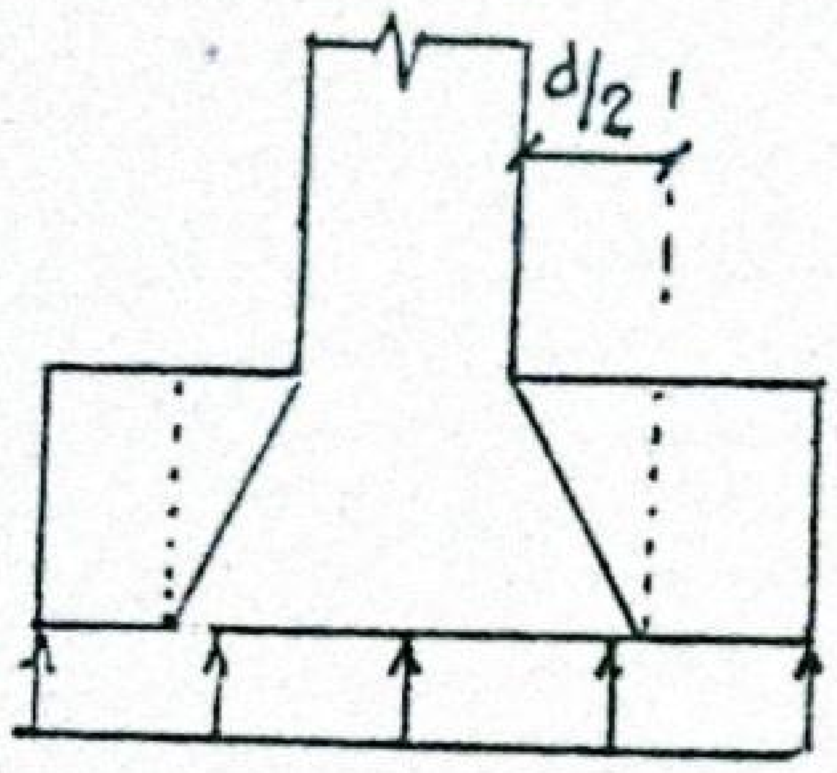
b_o = PERIMETRO SEGMENTADO

$b_o = 2(112.5 + 82.5) = 390$ cm

$\beta_c = \frac{\text{LADO LARGO COLUMNA}}{\text{LADO CORTO COLUMNA}} = \frac{.60}{.30} = 2$

$d = 52.5$ cm

$\phi = 0.85$



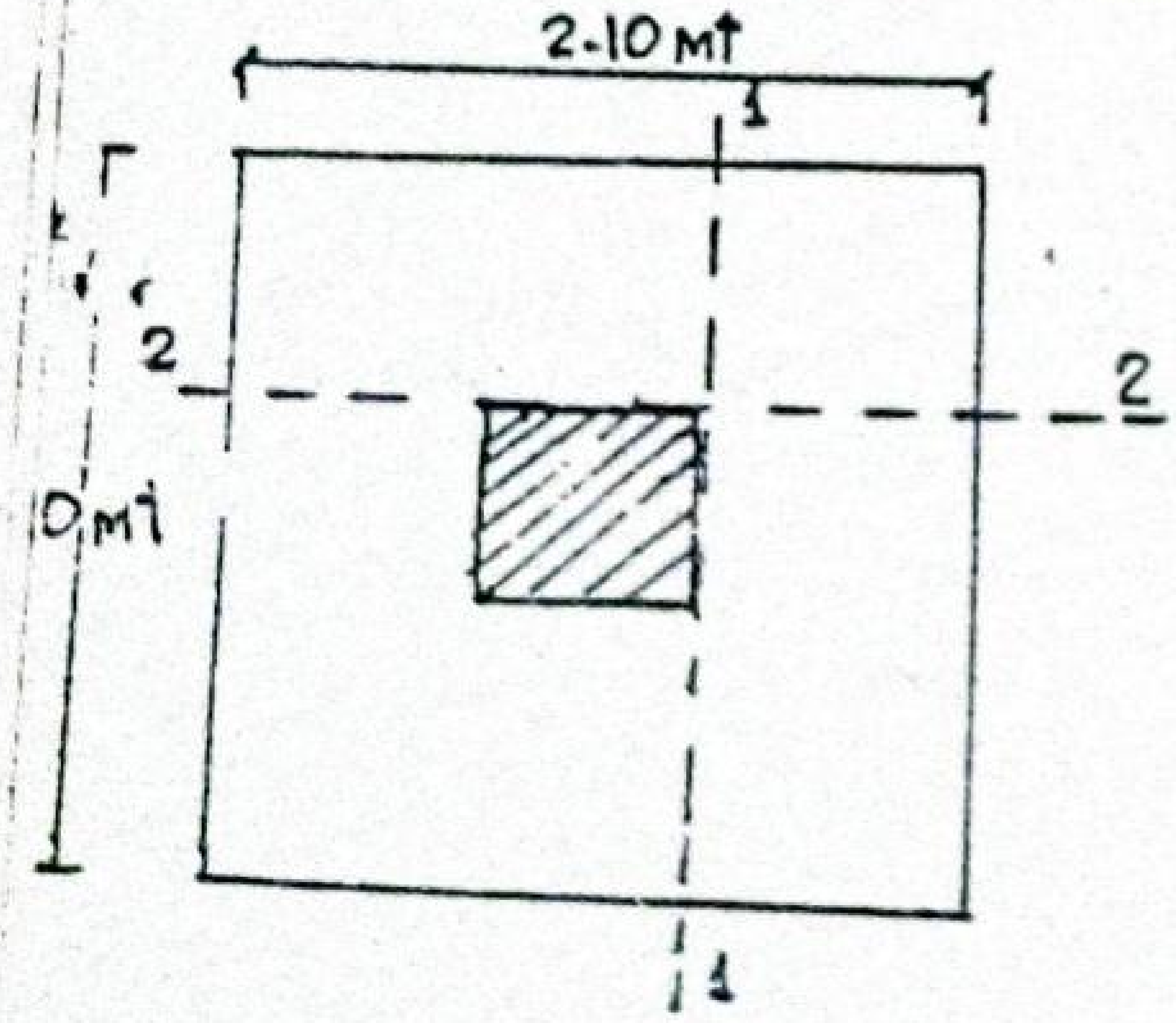
$\phi V_c = \phi\left(0.53 + \frac{1.1}{2}\right)\sqrt{175} \times 390 \times 52.5 = 248{,}648.4$

$V_U = W_n\left[A \times B - (t+d)(b+d)\right]$

$V_U = 3.50\left[240 \times 210 - (60+52.5)(30+52.5)\right] = 143{,}915.6$

$\therefore\ 143{,}915.6 < 248{,}648.4$ OK!

FLEXION: (A LA CARA DE LA COLUMNA)

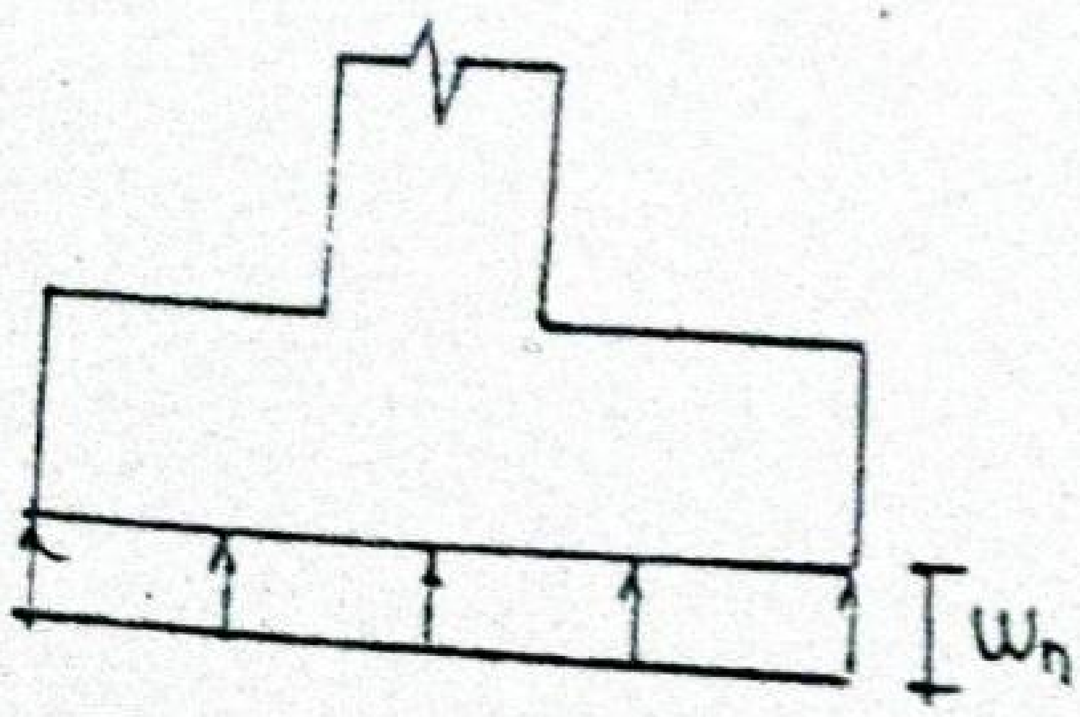


$* \ M_{1-1} = \frac{W_n \times A \times n^2}{2} = \frac{3.5 \times 2.40 \times (0.90)^2}{2}$

$M_{1-1} = 34.02^{\,T-M}$

$A_{S_{1-1}} = \left[0.85 - \sqrt{0.7225 - \frac{1.7 M_{1-1}}{\phi f'_c b d^2}}\right]\frac{f'_c}{f_y}.b.d$

REEMPLAZANDO VALORES TENEMOS: $A_{S_{1-1}} = 26.2$ cm^2



$\rho = \frac{A_{1-1}}{A \times B} = \frac{26.2}{240 \times 52.5} = 0.0021$

$\rho_{MAX} = 0.023 = 0.75\rho_b = 0.75(0.85)\frac{f'_c}{f_y} \cdot \beta_1\left(\frac{6000}{6000+f_y}\right)$

$A_{S\,MIN} = 0.002\,AH = 0.002 \times 240 \times 60 = 28.8$ cm^2

$As_{1-1} = 26.2\ cm^2 < As_{MIN} = 28.8\ cm^2$

$\therefore$ USAREMOS $As = 28.8\ cm^2 <> 15\ \phi\ 5/8''$ @ 16 cm c/u

$10\ \phi\ 3/4''$ @ 25 cm c/u

RECUBRIMIENTO 7.5 cm DESDE EL EXTREMO

$$* \ M_{2-2} = \frac{3.5 \times 210\ (0.90)^2}{2} = 29.76\ T\text{-}M$$

$$As_{2-2} = \left[0.85 - \sqrt{0.7225 - \frac{1.7\ M_{2-2}}{\phi f'c.bd^2}}\ \right]\left(\frac{f'c}{fy}\, bd\right)$$

REEMPLAZANDO VALORES OBTENEMOS: $As_{2-2} = 23.0\ cm^2$

$$\rho = \frac{As_{2-2}}{B \times d} = \frac{23.}{210 \times 52.5} = 0.0021$$

$$\rho_{MAX} = 0.023 > \rho_{2-2} = 0.0021$$

$As_{MIN} = 0.002 \times 210 \times 60 = 25.2\ cm^2$ $\qquad As_{MIN} = 25.2\ cm^2 > As_{2-2} = 23.0\ cm^2$

$\therefore$ USAMOS $As = 25.2\ cm^2 <> 13\phi 5/8''$ @ 16 cm c/u

$9\phi 3/4''$ @ 24 cm c/u

DISTRIBUCION DE ARMADURA EN ANCHO B = 2.10 MT

As EN ANCHO $(B = 2.10) = \frac{2}{R+1} \times As_{1-1}$ ...(α) NOTA: ESTO SE REALIZA CUANDO LAS ZAPATAS SON RECTANGULARES

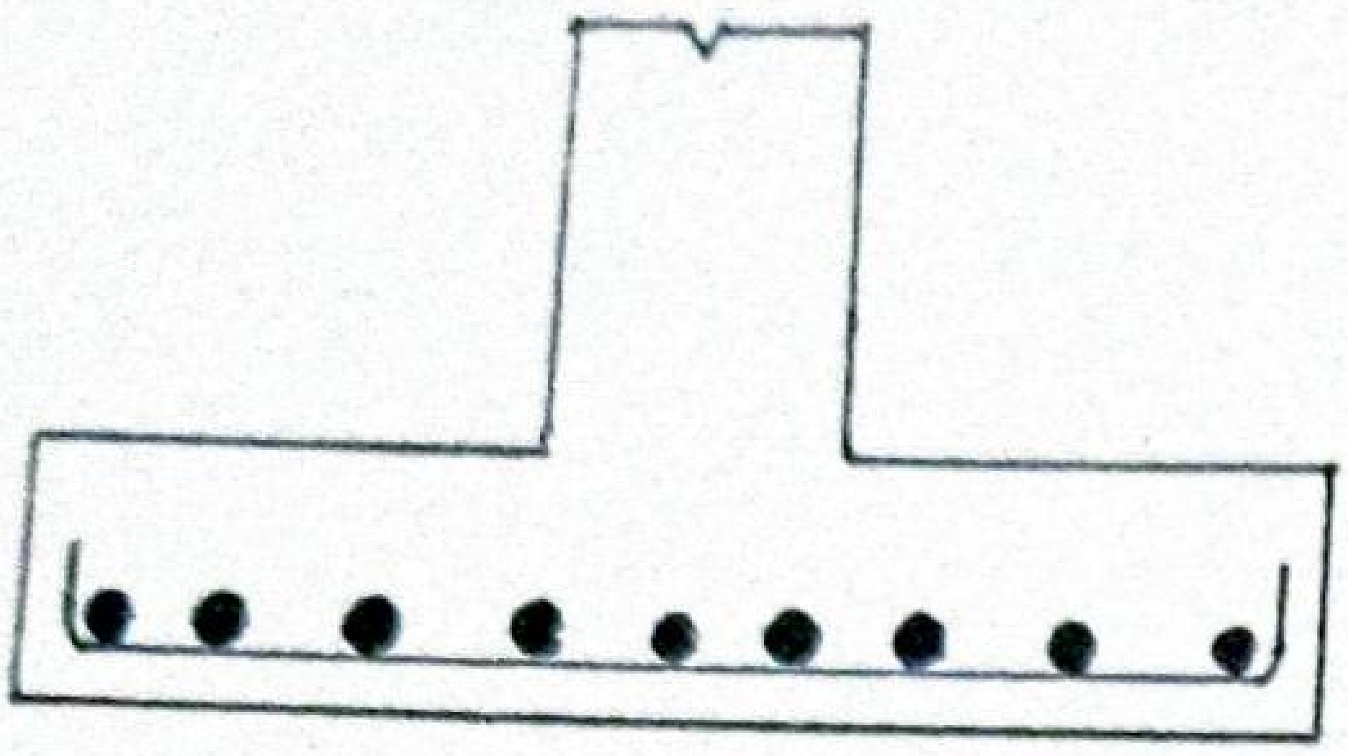
$$R = \frac{\text{LADO LARGO ZAPATA}}{\text{LADO CORTO ZAPATA}} = \frac{A}{B} = \frac{2.40}{2.10} = 1.14$$

EN (α) $As = \left(\frac{2}{1.14+1}\right)(28.8) = 26.78$

TAMBIEN: As EN ANCHO $(B = 2.10)$ $= \underbrace{\left(\frac{2}{1.14+1}\right)(10\phi 3/4'')}_{9.34\phi 3/4''} \approx 10\ \phi\ 3/4'' + 2\ \phi\ 3/4''$

$12\ \phi\ 3/4''$

* ℓ_{d_b}: LONGITUD DESARROLLO BASICO:



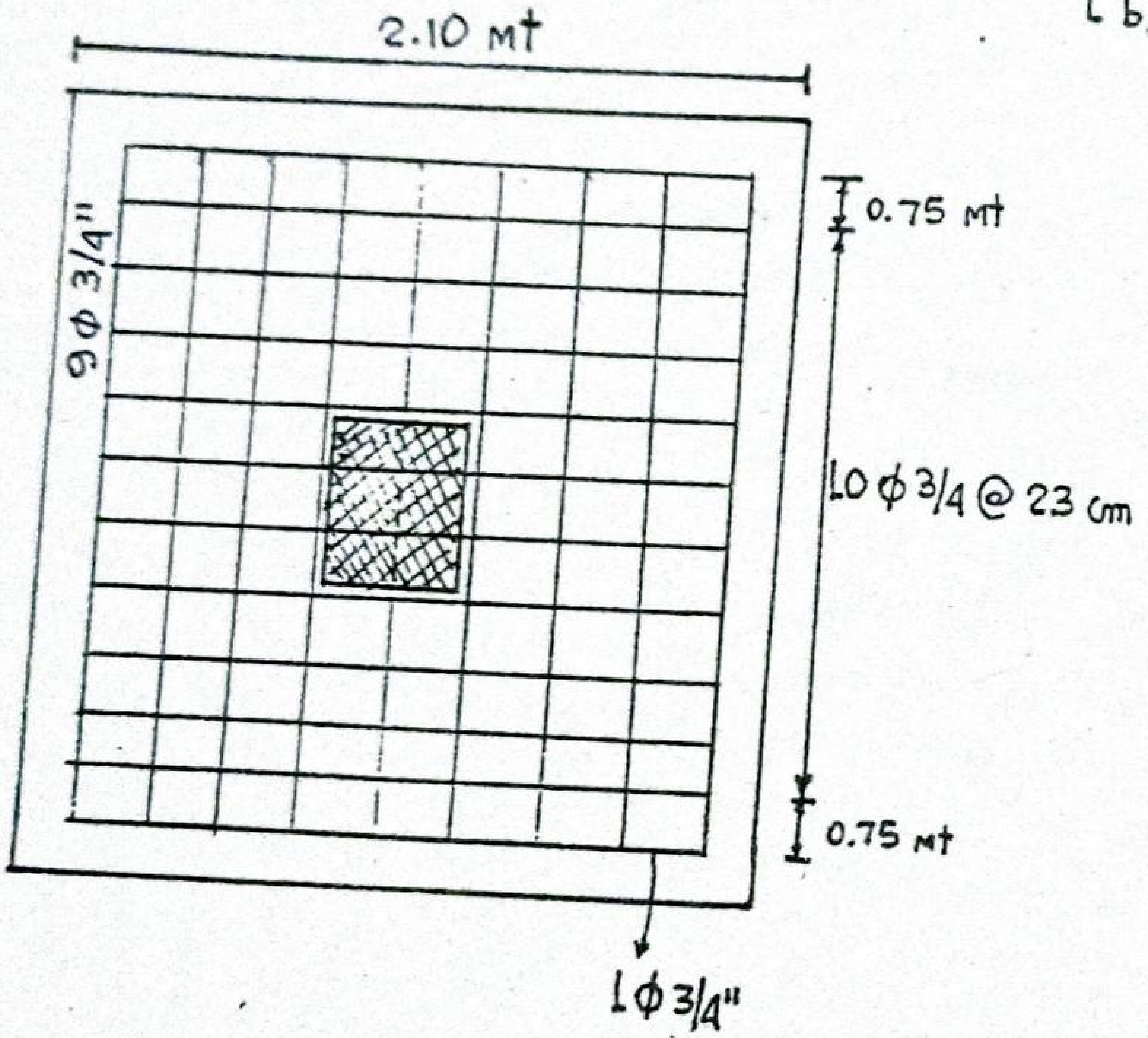
ANCLAJE Y ADHERENCIA:

- TRACCION:

$$\ell_{d_b} = 0.06\, A_b F_Y / \sqrt{f'c}$$

$$\ell_{d_b} = 0.006\, d_b f_Y$$

$$\ell_d \begin{cases} a) & 1.4 \\ b) & 0.8 \end{cases} \quad \ell d \geq 3.0 \text{ cm}$$



PARA EL PROBLEMA:

$d_b = 1.905$ cm

$A_b = 2.85$ cm²

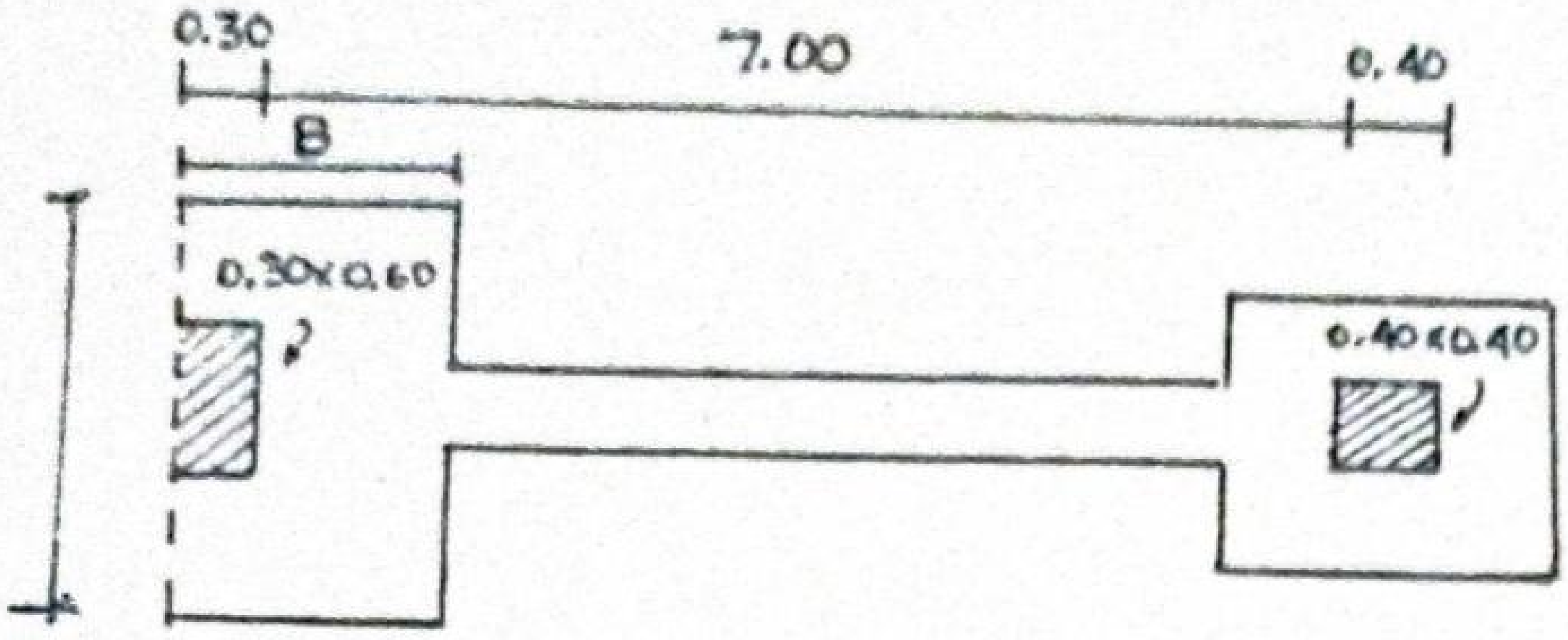
$f_Y = 2800$ K/cm²

$f'c = 175$ K/cm²

→ $\ell_{d_b} = 36.19$ cm ← TOMAMOS EL MAYOR.

$\ell_{b_d} = 32.0$ cm

PROB(4) DISEÑAR UNA ZAPATA CONECTADA DE LAS SIGUIENTES CARACTERISTICAS :



$\sigma_t = 4\ K/cm^2$

$f'_c = 175\ K/cm^2$

$f_Y = 4200\ K/cm^2$

$P_D = 100{,}000\ K_G$ $\qquad$ $P_D = 200{,}000\ K_G$

$P_L = 60{,}000\ K_G$ $\qquad$ $P_L = 80{,}000\ K_G$

SOLUCION :

$$P.P.Z_{\text{EXTERIOR}} = \left(\frac{7-\sigma_t}{60}\right) P_C = \left(\frac{7-4}{60}\right)(160{,}000) = 8{,}400$$

$$P_C = 100{,}000 + 60{,}000 = 160{,}000\ K_G$$

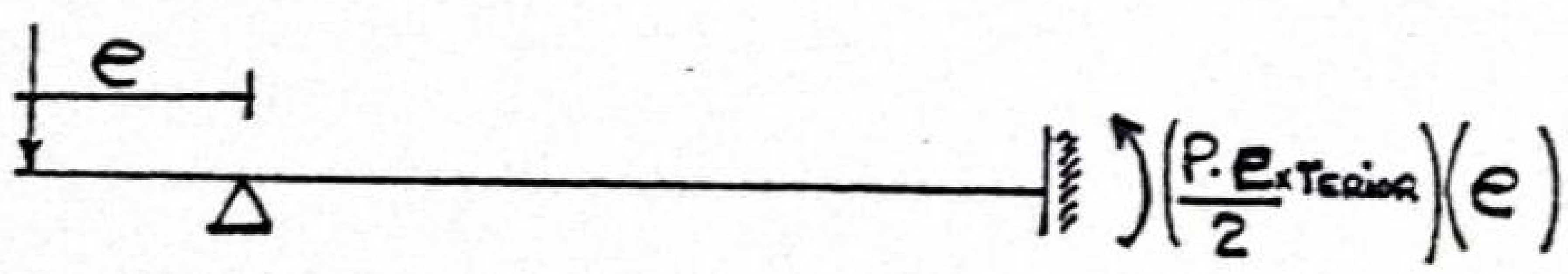
$$\sigma_t = \frac{P}{A_z} \rightarrow A_z = \frac{P}{\sigma_t} = \frac{168400}{40{,}000} = 4.22\ M^2$$

ASUMIMOS DIMENSIONES : B y 2B $\therefore$ $2B^2 = 4.22\ M^2$, $B = 1.5\ Mt$

PARA TENER EN CUENTA EL INCREMENTO DE CARGA POR REACCION HIPERESTATICA, REDONDEAMOS LA DIMENSIONES

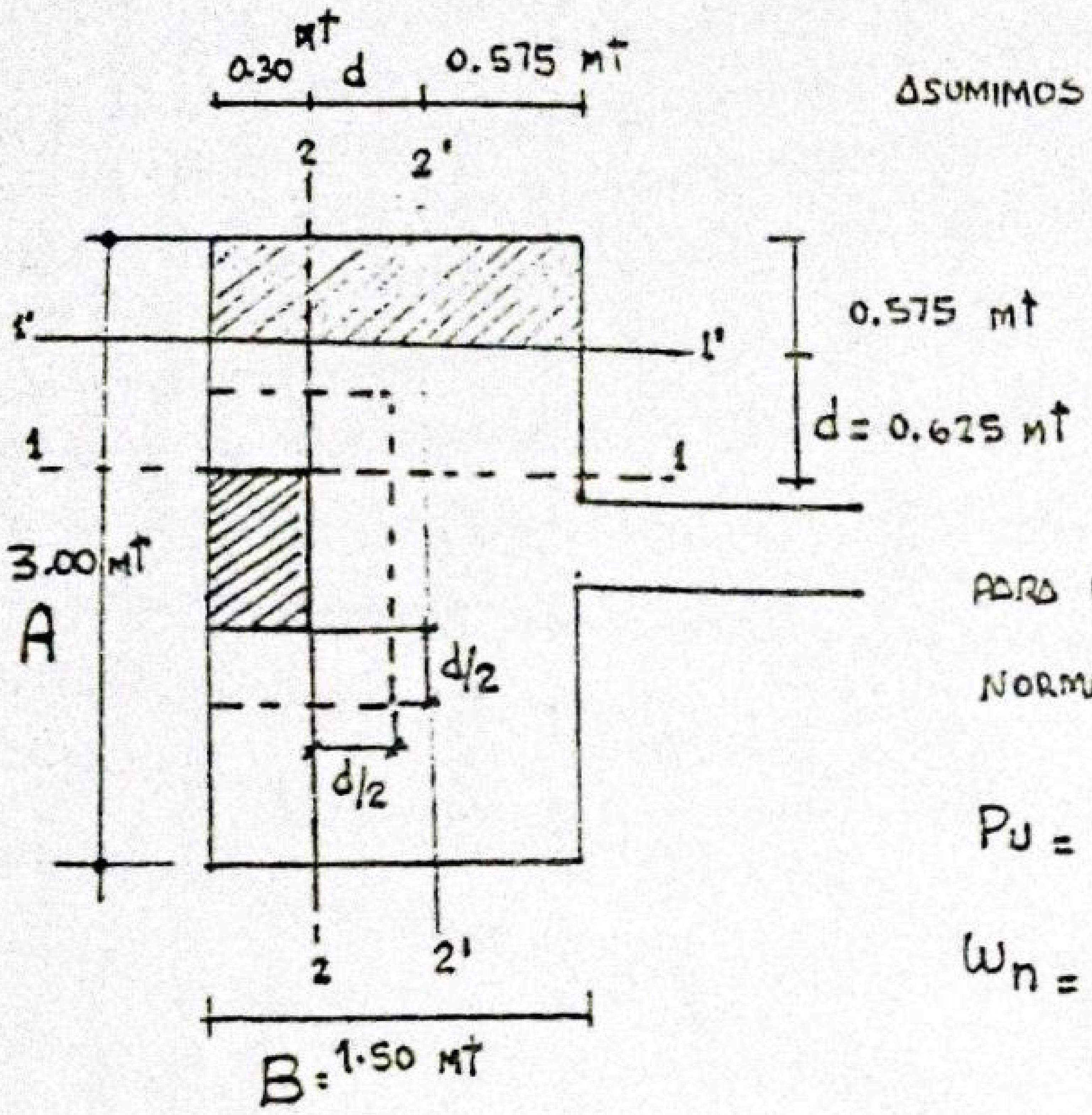
- $M_{\text{EXTERIOR}} = 160{,}000 \times 0.60 \approx 96{,}000\ K\text{-}Mt$

$(P \times e)$



TOMAMOS $B = 1.50\ Mt$

$2B = 3.00\ Mt$



ASUMIMOS $H = 0.70$ (LAS ZAPATAS NO DEBEN TENER ALTURA MENOR DE 60 cm)

RECUBRIMIENTO

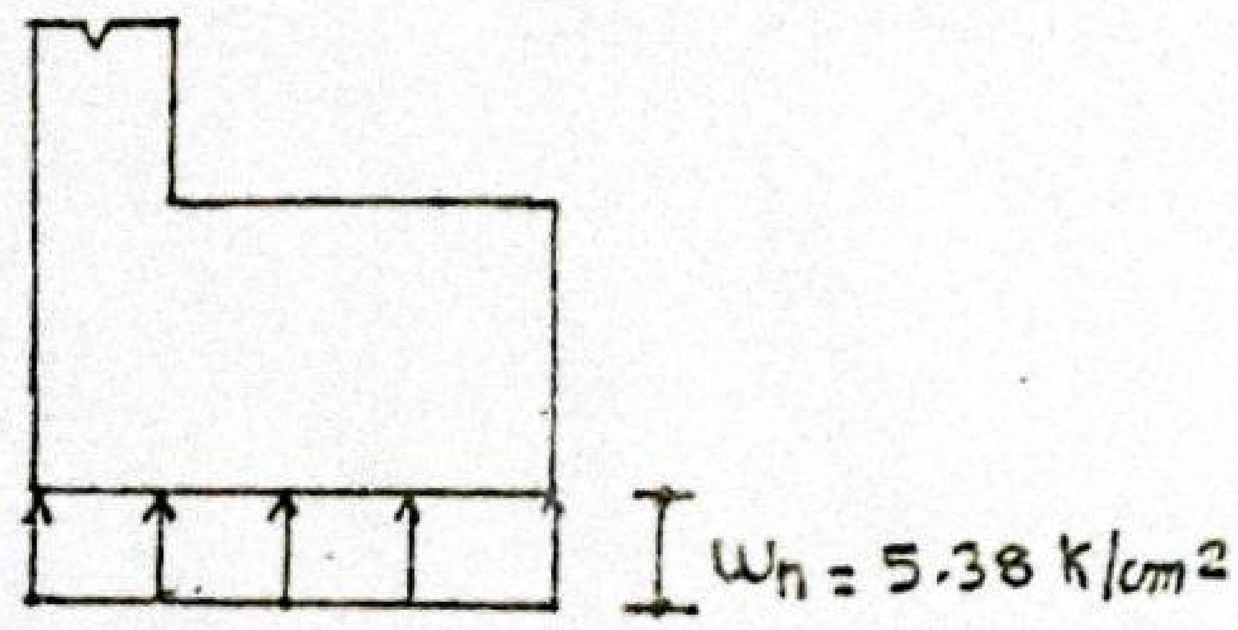
$d = 70\,cm - 7.5\,cm = 62.5\,cm \approx$

$d = 0.625\,mt$

PARA EL DISEÑO USAMOS EL METODO DE ROTURA Y LA NORMA A.C.I 318-83

$$P_u = 1.4\,(100{,}000) + 1.7\,(60{,}000) = 242{,}000\ K_G$$

$$W_n = \frac{242{,}000}{150 \times 300} = 5.38\ K/cm^2$$



VERIFICACION POR CORTE:

CUANDO LA VIGA ES DE MAYOR ALTURA QUE LA ZAPATA SOLO ES NECESARIO VERIFICAR EL CORTE POR VIGA, Y CUANDO SON DE LA MISMA ALTURA, VERIFICAR POR VIGA Y PUNZONAMIENTO.

a.- CORTANTE COMO VIGA:

* SECCION 1' - 1'

$$V = 5.38 \times 57.5 \times 150 = 46{,}402.5\ K_G$$

$$\phi V_c = \phi\, 0.53 \sqrt{f'c}\ B d = (0.85)(0.53)(\sqrt{175})(150)(62.5) = 55{,}870.8\ K_G > V$$

* SECCION 2' - 2'

$$V = 5.38 \times 300 \times 57.5 = 92{,}805\ K_G$$

SIENDO LA LONGITUD LIBRE 57.5 EN AMBAS SECCIONES LA SECCION 2' - 2' TAMBIEN SE CUMPLIRA b.- POR PUNZONAMIENTO:

$$V = W_n\left(A\times B - \left(\overset{30\,cm}{t} + \frac{d}{2}\right)\left(\overset{60\,cm}{b} + d\right)\right)$$

DE LA FIGURA:

$$V = 5.38\left(300\times 150 - \left(30 + \frac{62.5}{2}\right)(60 + 62.5)\right)$$

$$V = 201{,}733.18\ Kg$$

$$\phi V_c = \phi\left(0.53 + \frac{1.1}{\beta_c}\right)\sqrt{f'c}\; b_o . d = 0.85\left(0.53 + \frac{1.1}{2}\right)\sqrt{175}\,(2\overset{cm}{45})(6\overset{cm}{2.5}) = 185{,}955.$$

$$\beta_c = \frac{\text{LADO LARGO COLUMNA}}{\text{LADO CORTO COLUMNA}} = \frac{60}{30} = 2$$

$\therefore\ \phi V_c < V$ ¡NO PASA!

DEBEMOS MODIFICAR LA ALTURA

$$b_o = \text{PERIMETRO SEGMENTADO} = 2\left(\underset{cm}{30} + \frac{6\overset{cm}{2}.5}{2}\right) + 1\overset{cm}{22.5} = 2.45\ mt = 245\ cm$$

TOMAMOS $H = 0.80\ mt$ DONDE $d = 72.5\ cm.$

$$V = 5.38\left(300\times 150 - \left(30 + \frac{72.5}{2}\right)(60 + 72.5)\right) = 194{,}873\ Kg$$

$$V = 194{,}873\ Kg.$$

$$\phi V_c = 0.85\left(0.53 + \frac{1.1}{2}\right)\left(\sqrt{175}\right)(265)(72.5) = 233{,}317\ Kg$$

$$b_o = 2\left(30 + \frac{72.5}{2}\right) + (60 + 72.5) = 265\ cm$$

$\phi V_c > V$ (OK!)

$\therefore\ H = 0.80\ mt$

C. POR FLEXION: SECCION 1-1

$$M = 5.38\times 1.50\times(120)^2\left(\tfrac{1}{2}\right) = 58{,}104\ K\text{-}m$$

$$M = W_n \times A \times n^2\left(\tfrac{1}{2}\right)$$

$$n = (300 - 60)/2 = 120\ cm.$$

HALLAMOS MOMENTO RESISTENTE DE LA SECCION:

$$M_u = \phi\,\beta\, f'c\, d^2\, \omega_{max}(1 - 0.59\,W_{MAX})$$

$$M_u = 0.90\times 175\times 1.5\,(72.5)^2(0.3187)(1 - 0.59(0.3187))$$

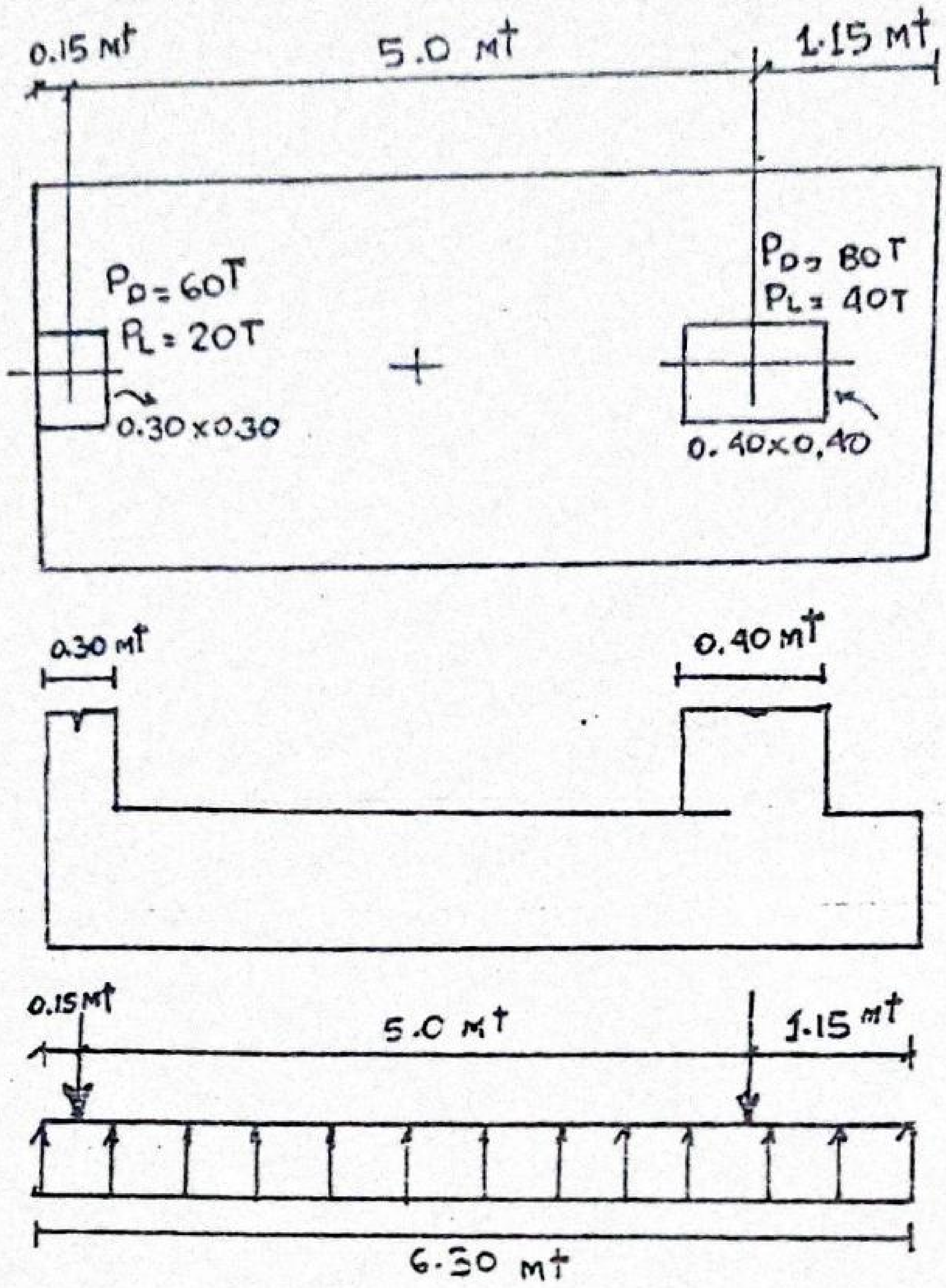
$$M_u = 321{,}642.57\ K\text{-}m > M$$

UTILIZANDO: $W = 0.85 - \sqrt{0.7225 - \dfrac{1.7\,M_u}{\phi f'c\,B\,d^2}}$

$$A_{s_{1\text{-}1}} = \left(0.85 - \sqrt{0.7225 - \frac{1.7\,M_u}{\phi f'c\, b d^2}}\right)\frac{f'c}{f_y}\, bd$$

Y ASÍ CALCULAMOS A_{s_1} Y A_{s_2}: //

PROB (5) CALCULAR Y DISEÑAR LA ZAPATA COMBINADA PARA LAS SIGUIENTES CARACTERISTICAS



$f_c = 210\ K/cm^2$

$\sigma_t = 2\ K/cm^2 \approx 20T/m^2$

SOLUCION:

ⓐ AREA DE ZAPATA COMBINADA

PESO PROPIO SUELO $= 0.40_M \times 1600\ K/M^3 = 0.64\ T/M^2$

PESO PROPIO ZAPATA $= \overset{H=ASUMIDO}{0.60} \times 2400\ K/M^3 = 1.44\ T/M^2$

$\overline{2.08\ T/M^2}$

$\sigma_n = 20 - 2.08 = 17.92\ T/M^2$

AREA ZAPATA COMBINADA $= L \cdot B = \dfrac{(P_{D_T} + P_{L_T})}{\sigma_n} = \dfrac{(60+80+20+40)}{17.92} = 11.16\ M^2$

$P_e = 80^T$

$P_i = 120^T \sim R = 200^T$ UBICACION DE RESULTANTE P_e, P_i

$$X_r = \frac{(80)(0.15) + 120(5.15)}{200} = 3.15 \text{ mt}$$

$$L = 2X_r = 2(3.15) = 6.30 \text{ mt}$$

$$B = \frac{11.16}{6.30} = 1.77 \approx 1.80 \text{ mt}$$

$L = 6.30$ mt

$B = 1.80$ mt

NOTA: AL CENTRAR LA CARGA SE BUSCA UNIFORMIZAR LA REACCION DEL SUELO

b.- CARGA UNIFORME REPARTIDA EFECTIVA :

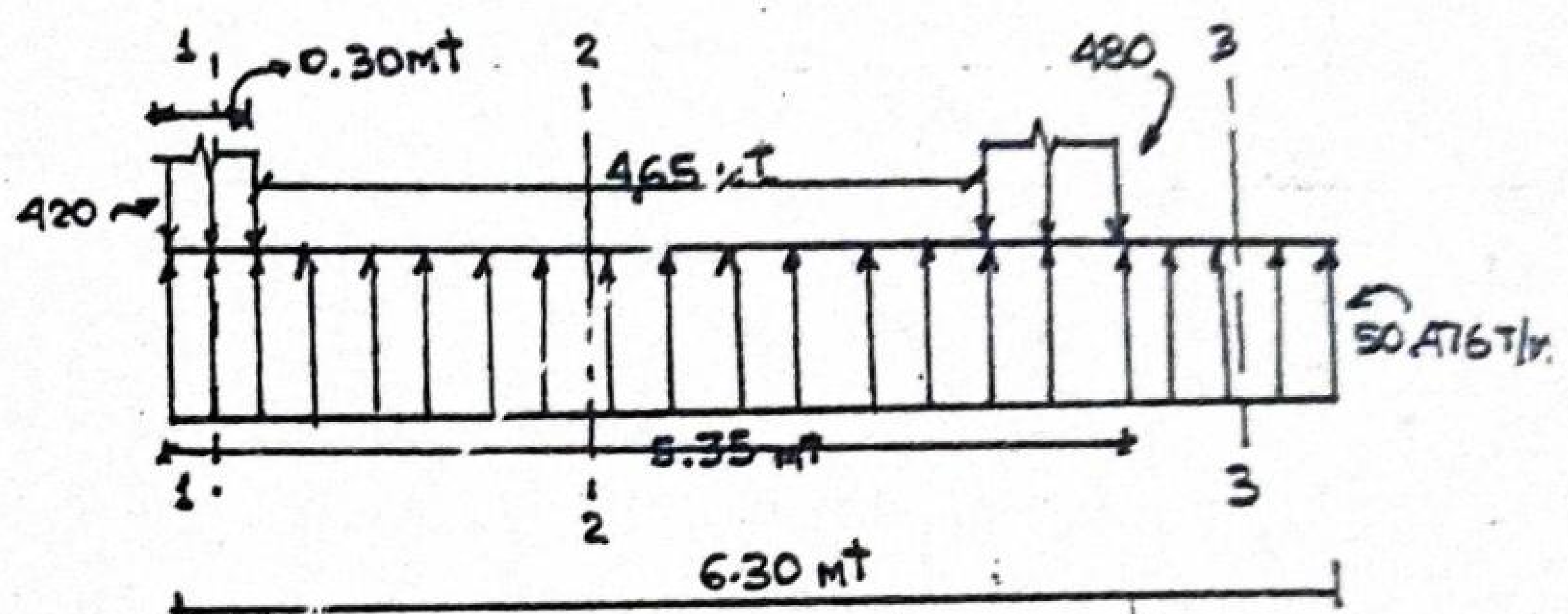
$$P_u = 1.5D + 1.8L \begin{cases} D = 60^T,\ L = 20^T \rightarrow 1.5(60) + 1.8(20) = 126 \text{ Tn} \\ D = 80^T,\ L = 40^T \rightarrow 1.5(80) + 1.8(40) = 192 \text{ Tn} \end{cases}$$

TOTAL = 318 Tn

$$W'_n = \frac{318}{6.3 \times 1.8} = 28.042 \text{ T/m}^2 \rightarrow W_n = (28.042)(1.8 \text{ mt}) = 50.476 \text{ T/m}$$

$$P_{EXT} = \frac{126 \text{ Tn}}{0.30 \text{ mt}} = 420 \text{ T/m}$$

$$P_{INT} = \frac{192 \text{ T}}{0.40 \text{ m}} = 480 \text{ T/m}$$



$\uparrow \boxed{+} \downarrow$

* TRAMO $0 \le x \le 0.30$
1-1

$$V = (50.476 - 420)X \begin{cases} x = 0 & V = 0 \\ x = 0.30 & V = -110.857^T \end{cases}$$

* TRAMO $0.30 \le x \le 4.95$
2-2

$$V = 50.476X - 420(0.30)$$

$$\begin{cases} X = 4.95 & V = 123.856^T \\ X = 2.50 & V = 0 \end{cases}$$

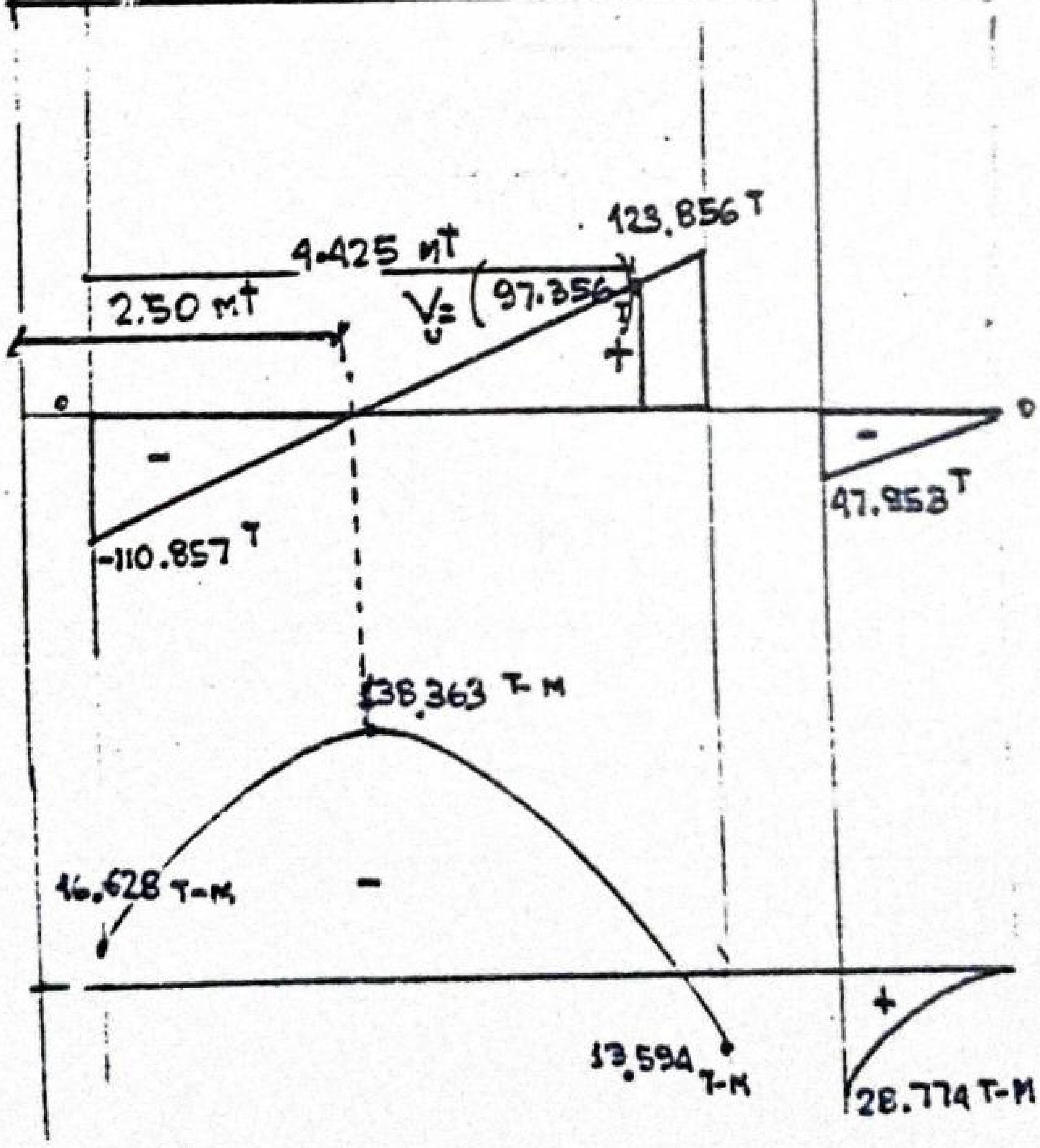
$d = 0.525$

$X = 4.95 - 0.525 = 4.425$

$X = 4.425 \quad V = 97.356$ T

* TRAMO 3-3 : $50.476X - 126 - 192$

$\{ X = 5.35 \quad V = -47.953^T$



d.- CALCULO DE MOMENTOS FLECTORES $\left(\curvearrowleft \boxed{+} \curvearrowright\right)$

TRAMO 1-1 $\quad 0 \leq X \leq 0.30 \qquad M = 50.476\frac{X^2}{2} - 126(X-0.15)$ $\quad \begin{cases} X=0 & M=0 \\ X=0.30 & M=-16.628 \text{ T-M} \end{cases}$

TRAMO 2-2 $\quad 0.30 \leq X \leq 4.95 \qquad M = 50.476\frac{X^2}{2} - 126(X-0.15)$ $\quad \begin{cases} X=2.5 & M_{MAX} = -138.362 \text{ T-M} \\ X=4.95 & M=+13.594 \text{ T-M} \end{cases}$

TRAMO 3-3 $\quad 5.35 \leq X \leq 6.30 \qquad M = 50.476\frac{X^2}{2} - 126(X-0.15) - 192(X-5.15)$ $\quad \begin{cases} X=5.35 & M=+28.774 \text{ T-M} \\ X=6.30 & M=0 \end{cases}$

VERIFICACION POR CORTE: ASUMIENDO $H = 0.60$ mt

* CORTE TIPO VIGA : $V_u = 97{,}356$ KG (DEL DIAGRAMA DE CORTANTES)

A_V = AREA DE ACERO DE ESTRIBOS

$$\phi V_c = 0.85 \times 0.53\sqrt{f'c}\cdot B\cdot d$$

$$\phi V_c = 0.85 \times 0.53\sqrt{210} \times 180 \times 52.5 = 61{,}693 \text{ KG}$$

COMO $V_u > \phi V_c \Rightarrow$ USAREMOS ESTRIBOS

$$A_{V_{MIN}} = 3.52\,\frac{b_w S}{f_y}$$

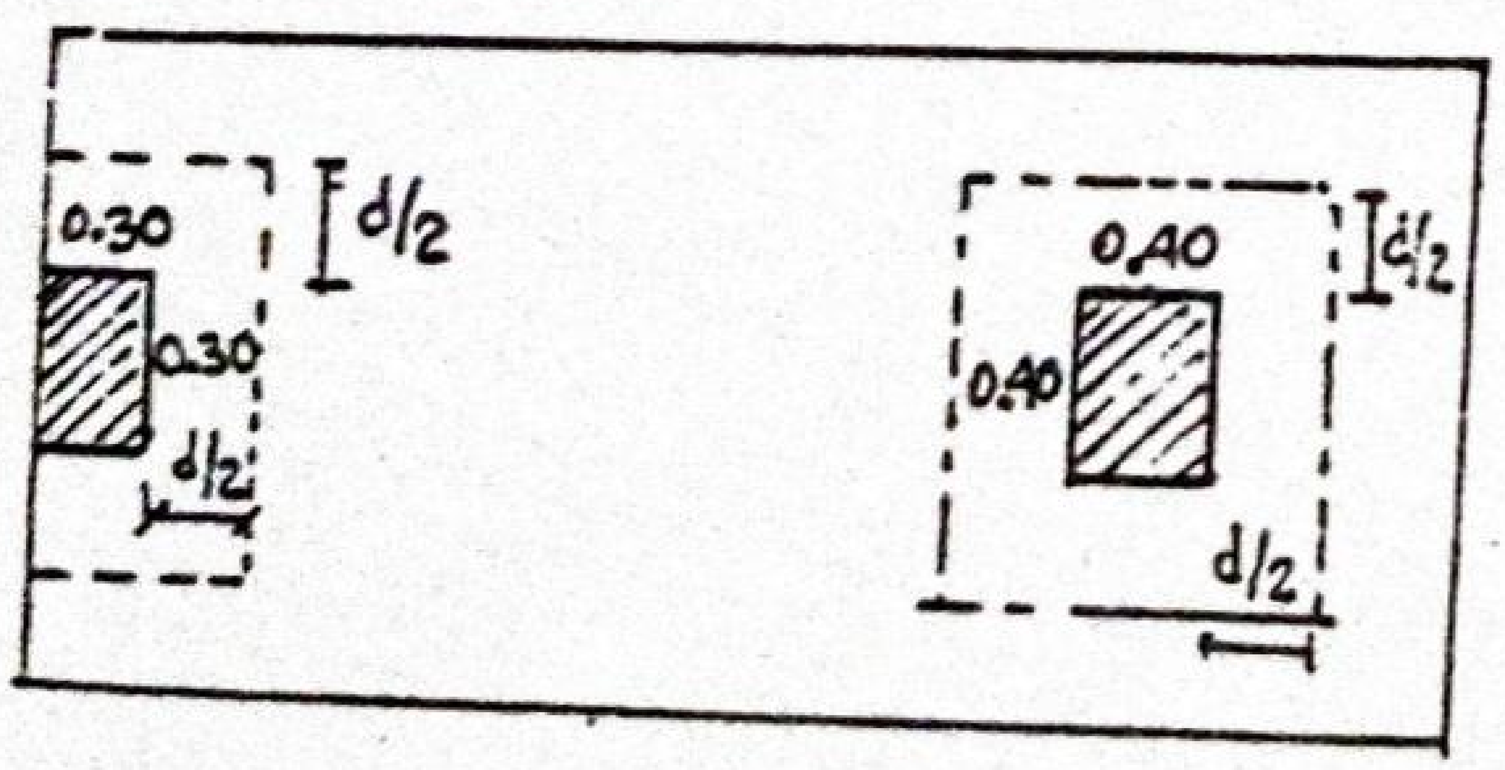
$$S = \frac{\phi A_v f_y \cdot d}{V_u - \phi V_c} = \frac{0.85(2\times1.98)(4200)(52.5)}{97{,}356 - 61{,}693} = 20.8 \text{ cm.}$$

$$S_{MAX} = \frac{d}{2}$$

* PUNZONAMIENTO:

$d = 52.5$ cm

COLUMNA EXTERNA: $P_e = 126{,}000$ KG



$$V_{u_e} = P_e - W_n\left(30 + \frac{d}{2}\right)(30 + d)$$

$$V_{u_e} = 126{,}000 - 2.8042\left(30 + \frac{52.5}{2}\right)(30 + 52.5) = 112{,}987$$

$$\phi V_{c_e} = \phi\, 1.1\sqrt{f'c}\; b_o\, d = 0.85 \times 1.1\sqrt{210}(56.25\times2 + 82.5\times1) \times (52.5$$

$$\phi V_{c_e} = 138{,}712 \text{ KG} \qquad \phi V_{c_e} > V_{u_e} \quad \text{OK!}$$

$$W_n = \frac{318{,}000}{630 \times 180} = 2.8042$$

b_o = PERIMETRO $\boxed{}$

COLUMNA INTERNA: $P_i = 192{,}000$ KG

$$V_{u_i} = 192{,}000 - 2.8042(40 + 52.5)(40 + 52.5) = 168{,}067 \text{ K}$$

$$\phi V_{c_i} = 0.85 \times 1.1\sqrt{210}\,(92.5 \times 4)(52.5)$$

$$\phi V_{c_i} = 263{,}198 \text{ KG} \qquad \phi V_{c_i} > V_{u_i} \quad \text{OK!}$$

FLEXION LONGITUDINAL: $M_{MAX} = 138{,}363$ K-mt

$$A_s = \left[0.85 - \sqrt{0.7225 - \frac{\overset{cte}{1.7} \times 138{,}363}{0.9 \times 210 \times 1.80\, d^2}}\right]\left(\frac{f'_c\, B d}{f_y}\right)$$

$B = 1.80$ mt

$d = 52.5$ cmt

$A_s = 77.2\ cm^2 <> \underbrace{15\,\phi\,1''}_{75\ cm^2}$

$$\rho = \frac{A_s}{B \cdot d} = \frac{77.2}{180 \times 52.5} = 0.00816$$

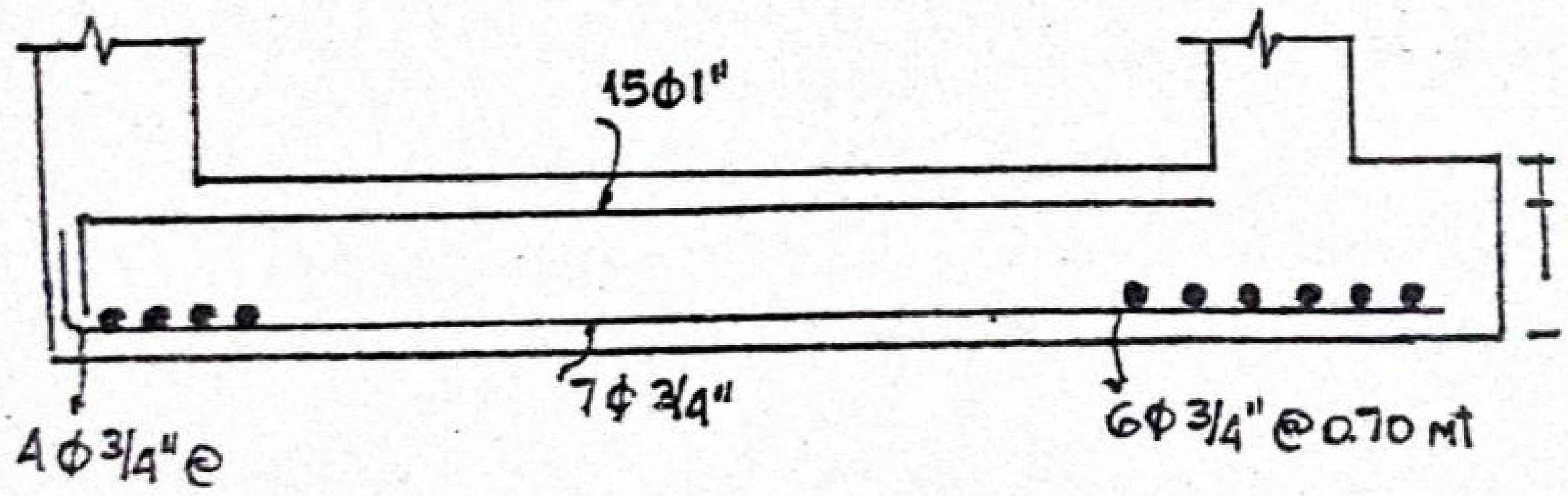
$$\rho_{MAX} = 0.75\left(0.85 \times 0.85 \times \frac{210}{4200}\left(\frac{6000}{6000+4200}\right)\right) = 0.016$$

$$A_{s\,MIN} = \rho_{min} \times \underset{asumido}{B \times H} = \underbrace{0.0018}_{para\ f_y = 4200} \times 180 \times 60 = 19.44\ cm^2$$

COLOCACION DEL ACERO LONGITUDINAL

CON ENCOFRADO → 7.5 cm $180 - 2(7.5) = 165$ $\frac{165}{15-1} = 11.78 \simeq 12$ cm

SIN ENCOFRADO → 4.5 cm



$$l_{db} = \frac{0.06\, A_b \cdot F_y}{\sqrt{f'_c}} = \frac{0.06 \times 5.07 \times 4200}{\sqrt{210}} = 88.2\ cm$$

$$l_{db} = 0.006\, d_b f_y = 0.006 \times 2.54 \times 4200 = 64\ cm$$

CAPA SUPERIOR

$$l_{d\phi 1''} = 1.4 \times \overset{\text{EN VEZ DE 0.8}}{1} \times 88.2 = 123\ cm$$

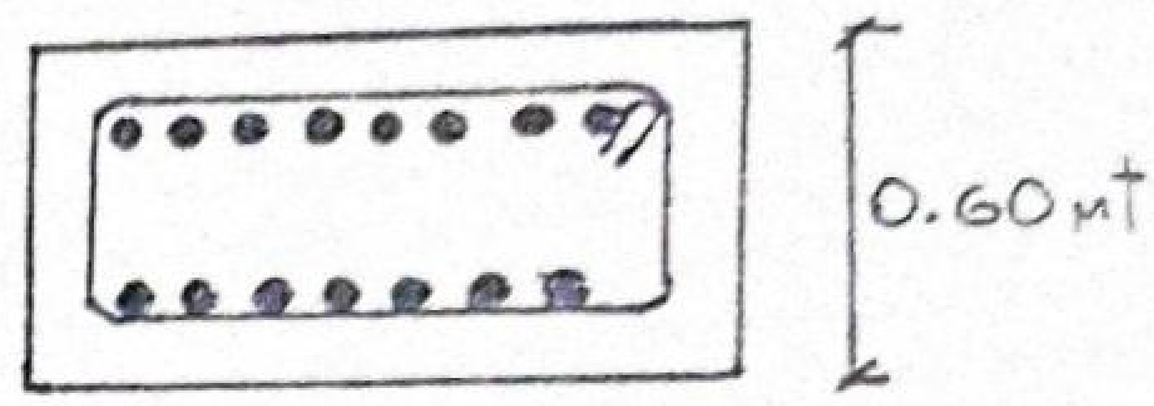
VOLADO: $M = 28775 \text{ K-M} \rightarrow A_s = 11.8\ cm^2 < A_{s\,min} = 19.44\ cm^2$

USAMOS $A_{s\,MIN} = 19.44\ cm^2 <> \underbrace{7\phi 3/4"}_{19.95\ cm^2}$

$$s = \frac{(180-15)}{7-1} = 27.5 \approx 27\ cm \text{ (ESPACIAMIENTO)}$$

$$l_{db} = \frac{0.06 \times 2.85 \times 4200}{\sqrt{210}} = 50$$

CORTE 1-1



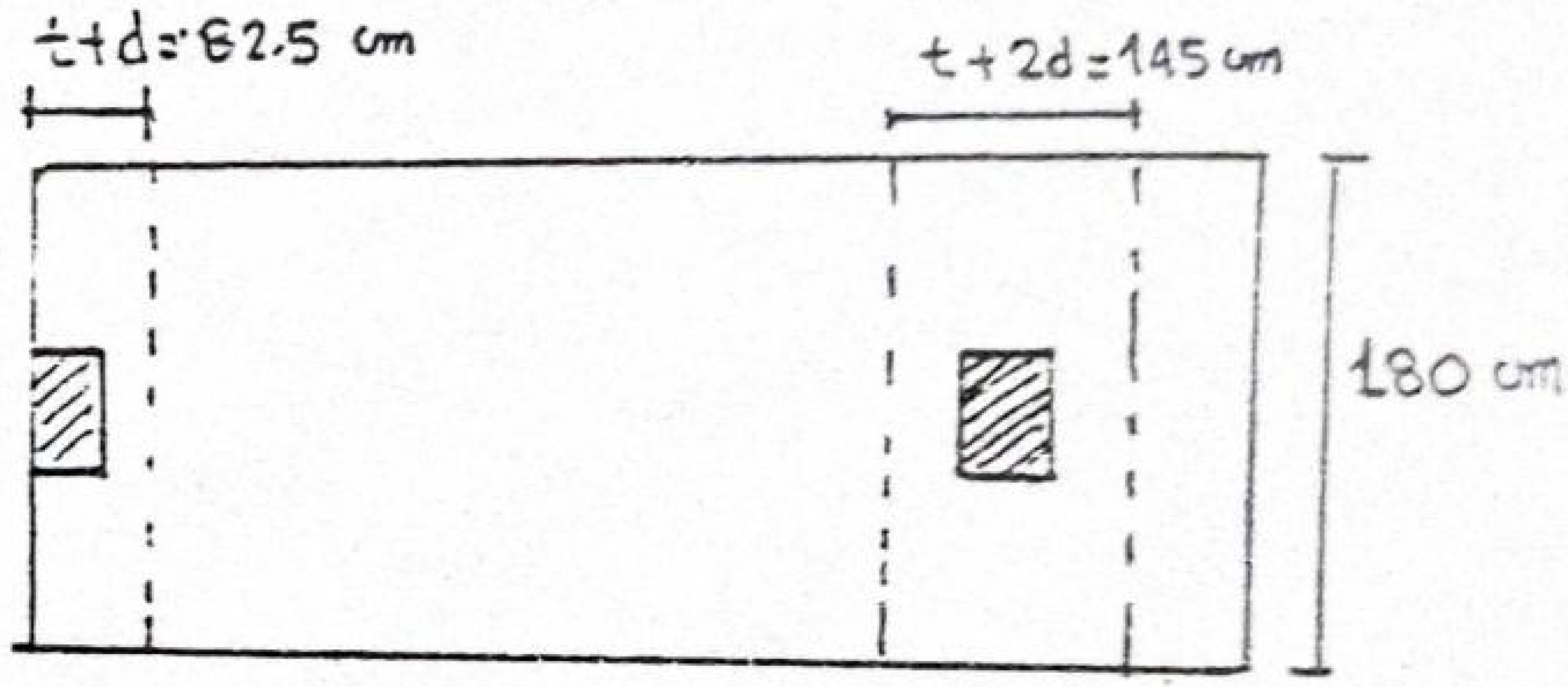
$$l_{db} = 0.06 \times \underset{d_b}{1.905} \times 4200 = 48$$

CAPA INFERIOR:

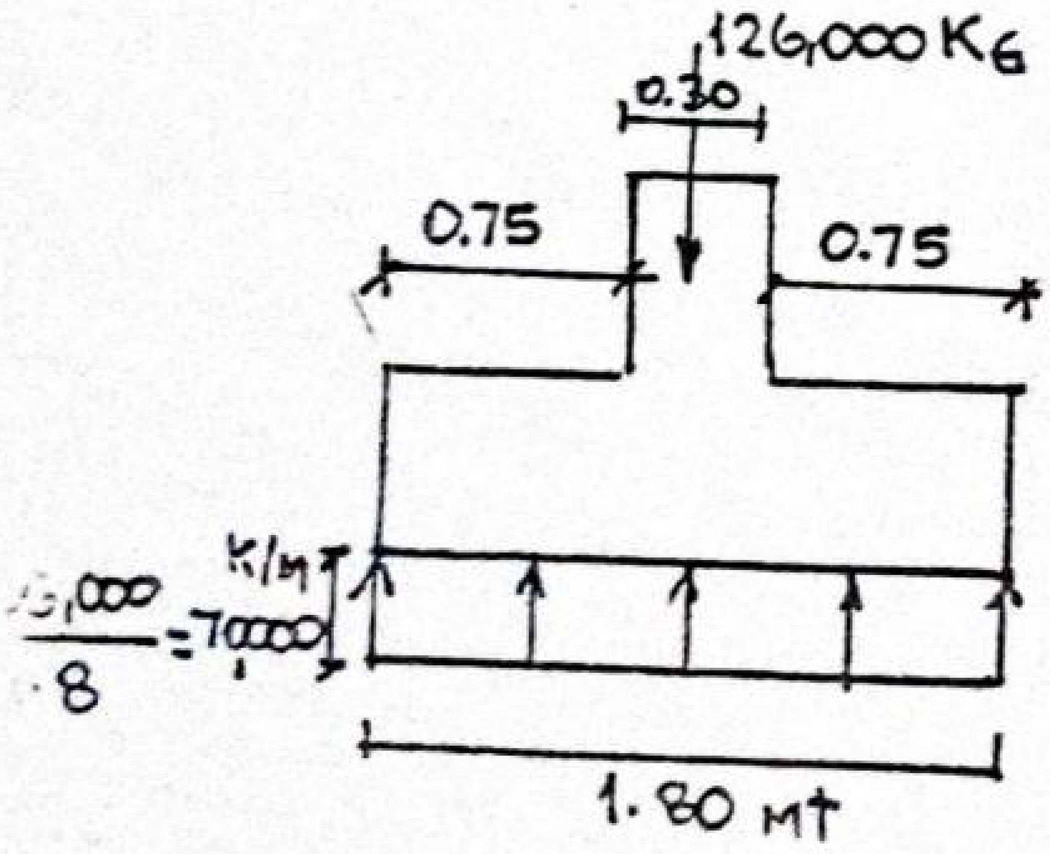
$l_{d\,3/4"} = 1 \times 0.8 \times 50 = 40 \qquad 40 < 82.5$

↳ DEBIDO A ESPACIAMIENTO Y RECUBRIMIENTO

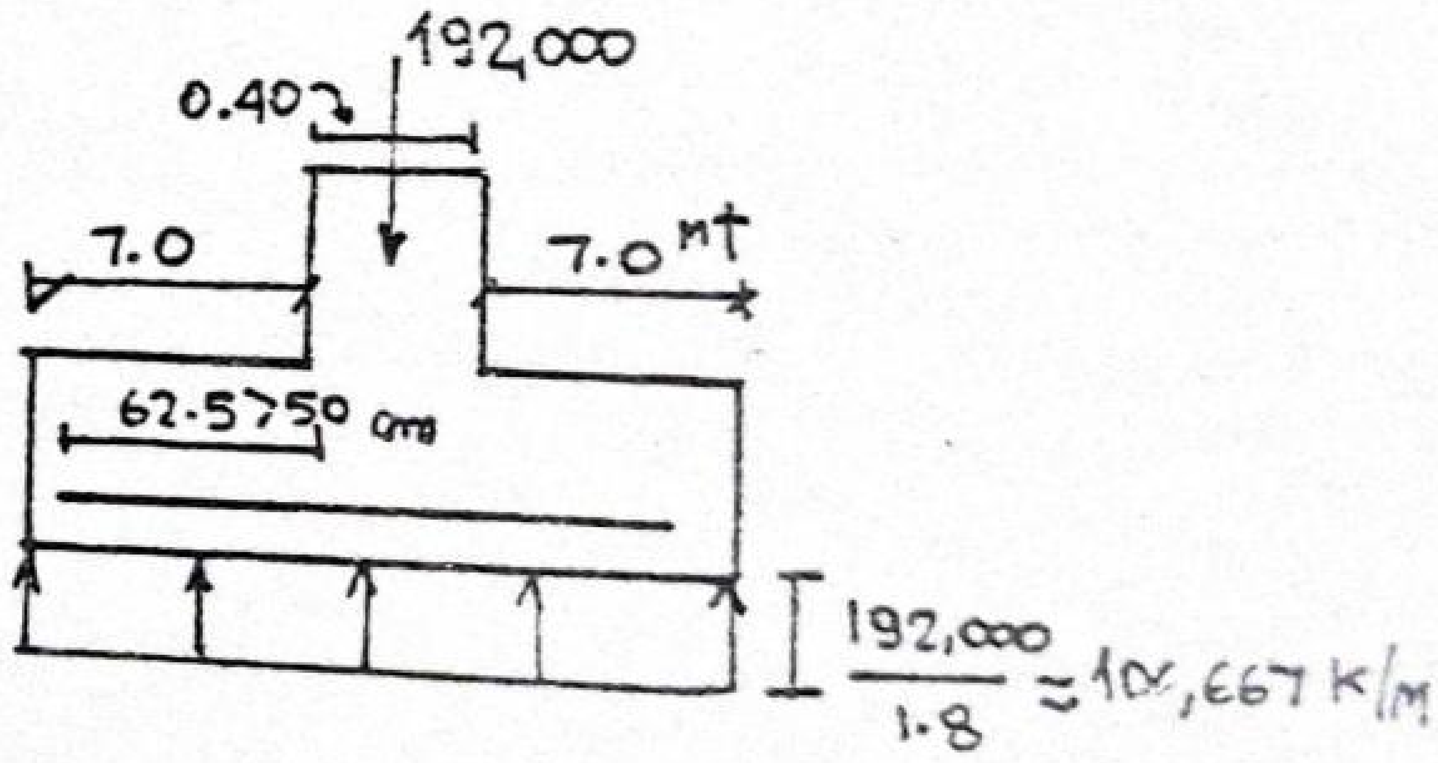
VIGAS TRANSVERSALES:



* BAJO COLUMNA EXTERIOR .30x30



* BAJO COLUMNA INTERIOR DE 0.40x0.40



$$M_u = 70{,}000 \times 0.75 \times \frac{0.75}{2} = 19{,}688 \text{ K-M}$$

$$A_s = \left[0.85 - \sqrt{0.7225 - \frac{1.7\,M_u}{0.9 \times 210 \times 82.5 \times 52.5}}\right] \frac{f'_c}{f_y}(82.5)(52.5)$$

$A_s = 10.2\ cm^2 \qquad l_{db\,3/4"} = 50\ cm$

$A_{s\,MIN} = 8.9\ cm^2,\ 4\phi 3/4" <> 11.4\ cm^2$

$$M_u = 106{,}667 \times (0.7)\left(\frac{0.7}{2}\right) = 26{,}133\ K_G$$

$A_s = 13.5\ cm^2;\ A_{s\,MIN} = 15.1\ cm^2 <> \overbrace{6\phi 3/4"}^{17.1\ cm^2}$

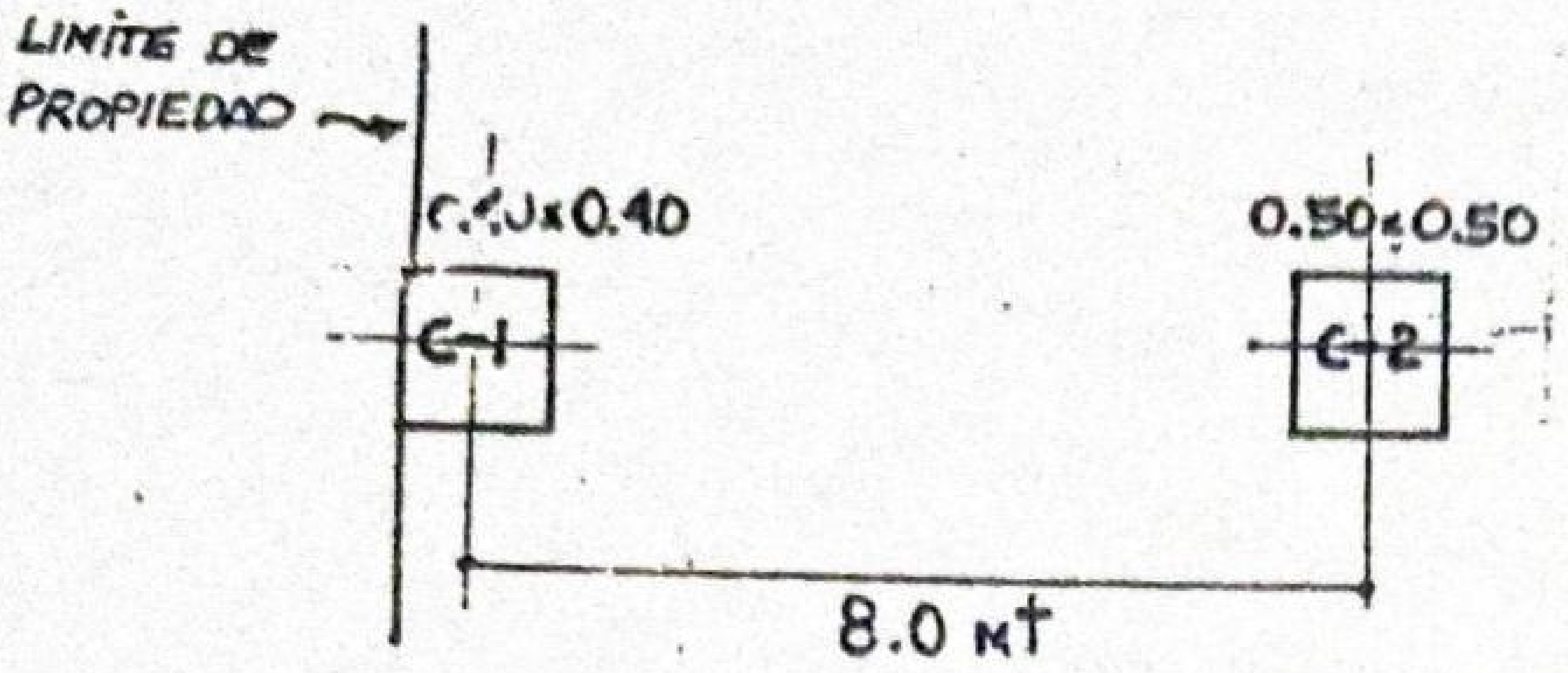
NOTA: MOMENTO DE IZQUIERDA A DERECHA ≠ MOMENTO DE DERECHA A IZQUIERDA TOMAR EL MAYOR DE ELLOS

$M_{ID} = 138{,}363$

$M_{DI} = 144{,}363 \quad (16\phi 1")$

* DE IGUAL MANERA SE VERIFICAN LOS ESTRIBOS

PROB (6) DISEÑO LA CIMENTACION DE COLUMNA C-1, CON C-2 DE TIPO DE ZAPATA COMBINADA



$\sigma_t = 3.8\ K/cm^2$

$f'c = 210\ K/cm^2$

$f_y = 4200\ K/cm^2$

$P_e \begin{cases} D = 92{,}928 \\ L = 24{,}000 \end{cases}$ $\quad P_i \begin{cases} D = 185{,}856 \\ L = 48{,}000 \end{cases}$

PASO 1:

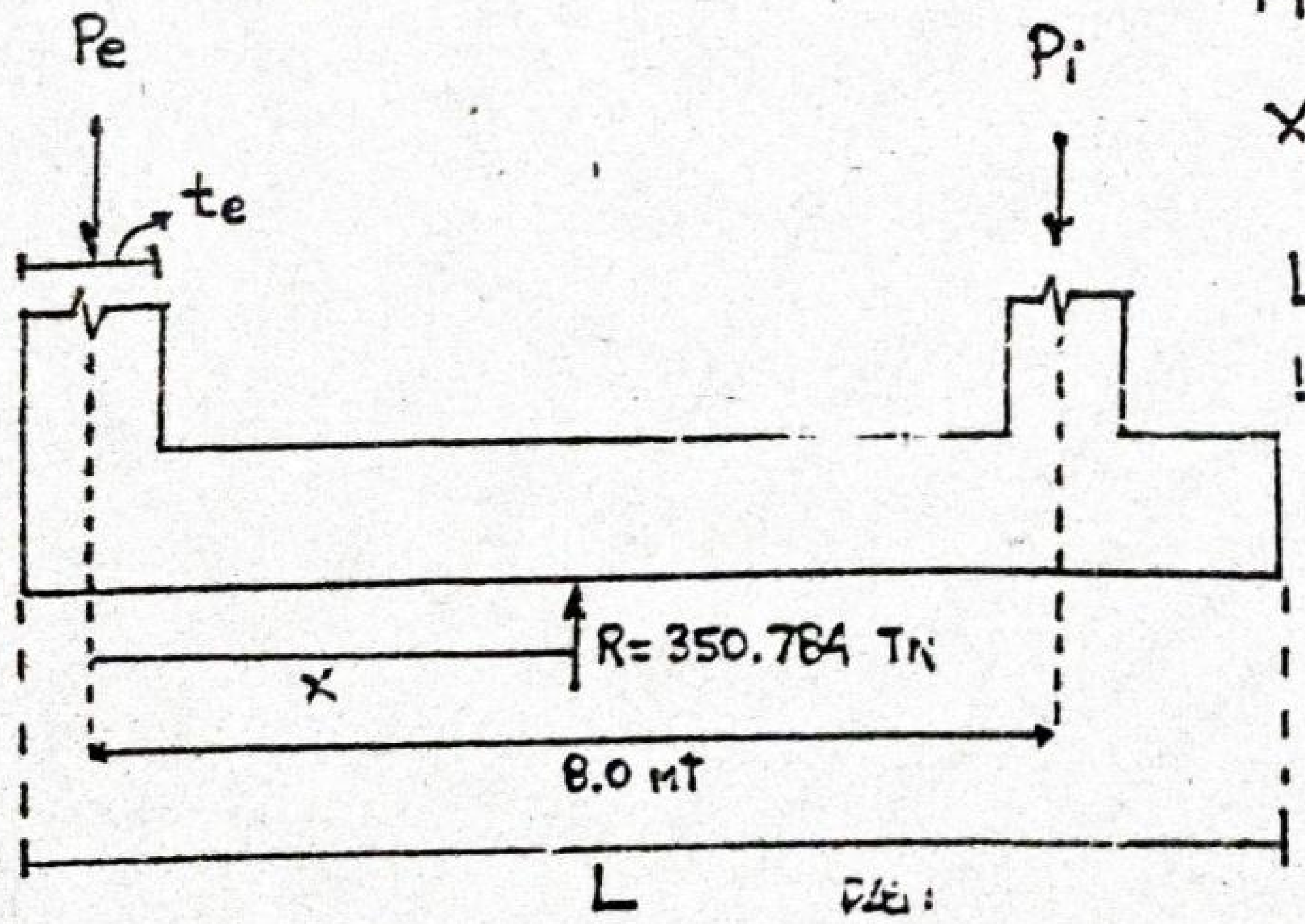
CALCULOS: $\begin{cases} \sigma_t = 3\ K/cm^2 & \text{—— } 9\% \\ \sigma_t = 3.8\ K/cm^2 & \text{—— } x \\ \sigma_t = 4\ K/cm^2 & \text{—— } 6\% \end{cases}$ $\quad x = 6.6\%$

$P_e = 92{,}928 + 24{,}000 = 116{,}928$

$P_i = 185{,}856 + 48{,}000 = 233{,}856$

$+ \Big\rangle\ 350{,}784\ K_G \approx 350.784\ TN$

$$A_z = \frac{P_e + P_i + P_z}{\sigma_t} = \frac{P_e + P_i + 0.066(P_e + P_i)}{38} = \frac{1.066(350.784)}{38} = 9.84\ TN$$



$\Sigma M_o = 0$

$P_i(8.0) = R\,X$

$X = \dfrac{233.856\,(8.0)}{350.784} = 5.33\ mt$

$L = 2(x + te/2) = (5.33 + 0.40/2)(2$

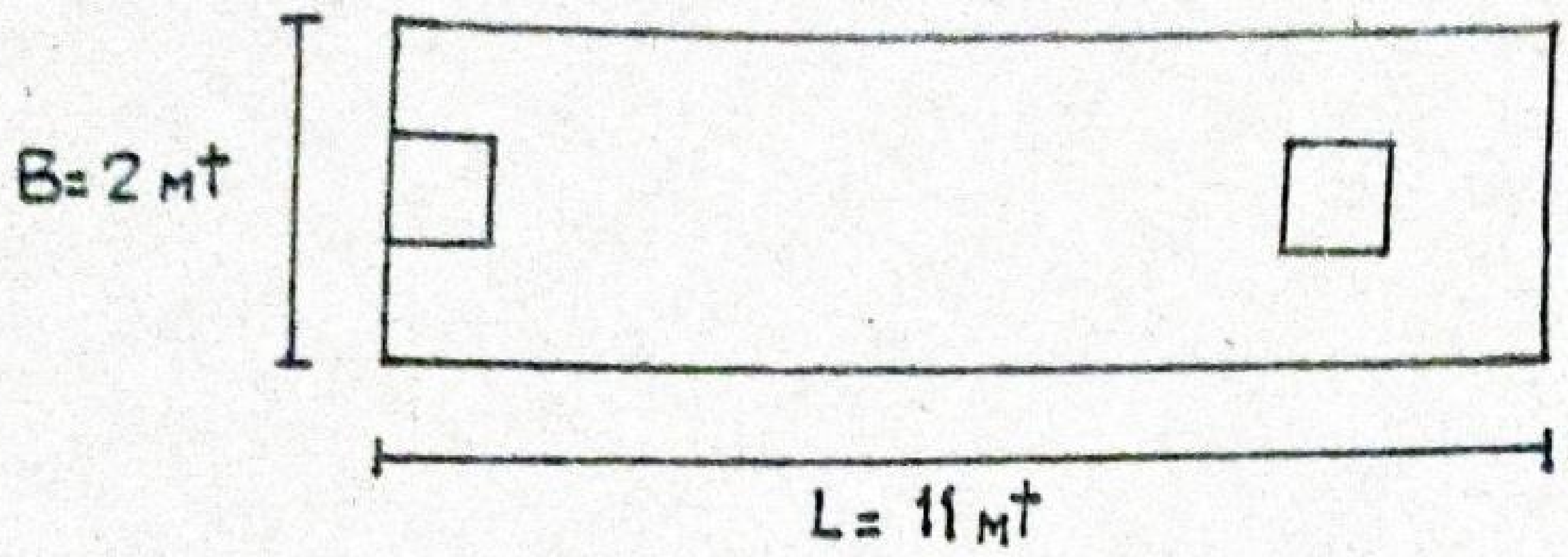
$L = 11.06\ mt$

$L = 11.0\ mts$

PASO 2: CALCULO DE "B" :

$$A_Z = B \times L \qquad B = \frac{A_Z}{L} = \frac{9.84}{11} = 0.90 \simeq 1 \text{ mt}$$

POR DISEÑO CONSTRUCTIVO, $B = 2$ mt $\therefore A_T = 2 \times 11 = 22 \text{ m}^2$



PASO 3: CALCULO σ_{t_u} (ROTURA):

* $P_{e_u} = 1.5D + 1.8L = 1.5(92.93) + 1.8(24) = 182.60$ TN
* $P_{i_u} = 1.5D + 1.8L = 1.5(185.86) + 1.8(48) = 365.20$ TN

ASUMO: $h = 0.80 \rightarrow P_Z = L \times B \times h \times \sigma_{CONCRETO}$

$$P_Z = 11 \times 2 \times 0.8 \times 2.4 = 42.24$$

$$P_{Z_u} = 1.066(42.24) = 45 \text{ TN}$$

$$P_U = P_{e_u} + P_{i_u} + P_{Z_u} = 592.83 \text{ TN}$$

$$\sigma_{t_u} = \frac{592.83}{2 \times 11} = 26.95$$

$$\sigma_{t_u} = 27 \text{ T/m}^2$$

PASO 4: DIAGRAMA DE FUERZA CORTANTE Y MOMENTO FLECTOR

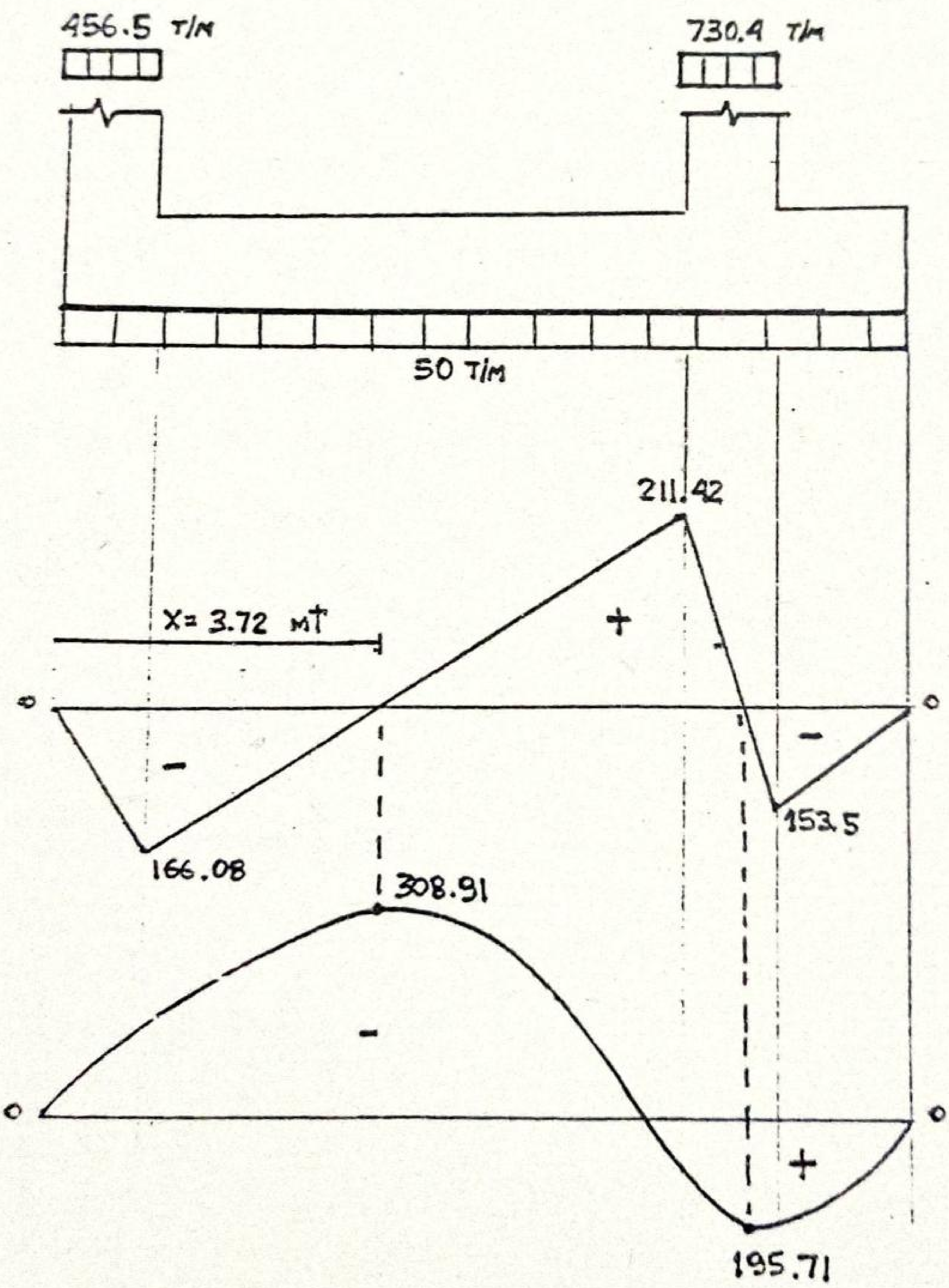
CALCULOS: $W_e = \frac{P_{eu}}{t_{e_1}} = \frac{182.60}{0.40} = 456.50$ T/m

$W_i = \frac{P_{iu}}{t_{e_2}} = \frac{365.20}{0.50} = 730.40$ T/m

$W_Z = \frac{P_{RU}}{L} = \frac{45.0}{11.0} = 4$ T/m

$W_t = \nabla t_u \times B = 27 \times 2 = 54$ T/m

$W_R = W_t - W_Z = 54 - 4 = 50$ T/m



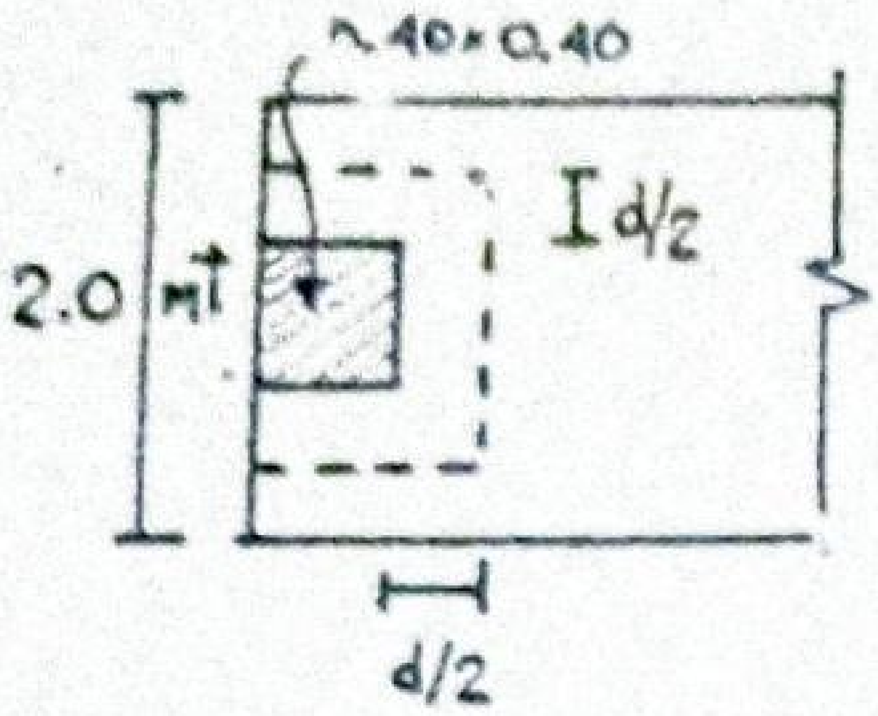
PASO 5: CALCULO DEL PERALTE:

A. CORTE POR PUNZONAMIENTO: ASUMIENDO $d = 1.2$ mt

ZAPATA EXTERIOR:

$b_o = 2(0.40 + 1.2/2) + 0.4 + 1.2 = 3.6$ mt

PERIMETRO



$A_o = (0.4 + 1.2)(0.4 + 1.2/2) = 1.6\ m^2$

$$V_{ue} = \frac{V_{ue}}{0.85 \times b_o d} = \frac{P_{ue} - A_o \cdot \sigma_{tu}}{0.85 \times b_o \cdot d} = \frac{182.6 - 1.6(27)}{0.85 \times 3.6 \times 1.2}$$

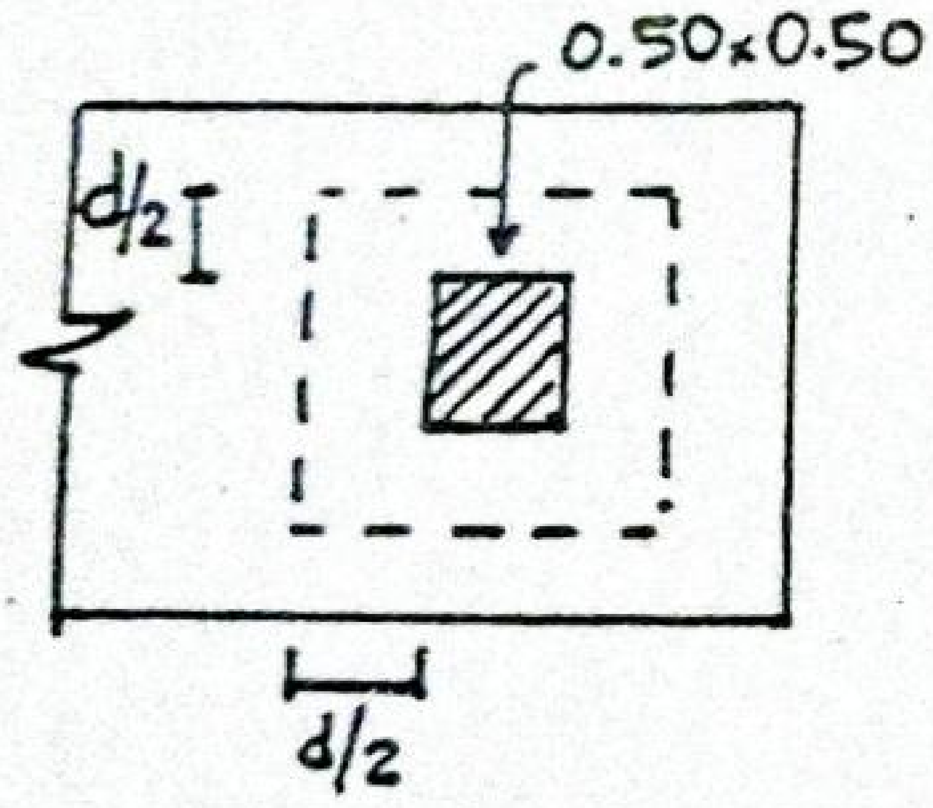
$V_{ue} = 37.96\ T/M^2 = 3.79\ K/cm^2$

$V_c = 0.27(2 + \frac{4}{\beta})\sqrt{f'c} = 0.27(2 + \frac{4}{1})\sqrt{210} = 23.48\ K/cm^2$

(PERO NO MAYOR DE) → $V_c = 1.1\sqrt{210} = 15.94\ K/cm^2$

∴ $3.79\ K/cm^2 < 15.94\ K/cm^2$ OK!

ZAPATA INTERIOR:



$b_o = 4(0.4 + 1.2) = 6.4$ mt

$A_o = (0.4 + 1.2)^2 = 2.56\ M^2$

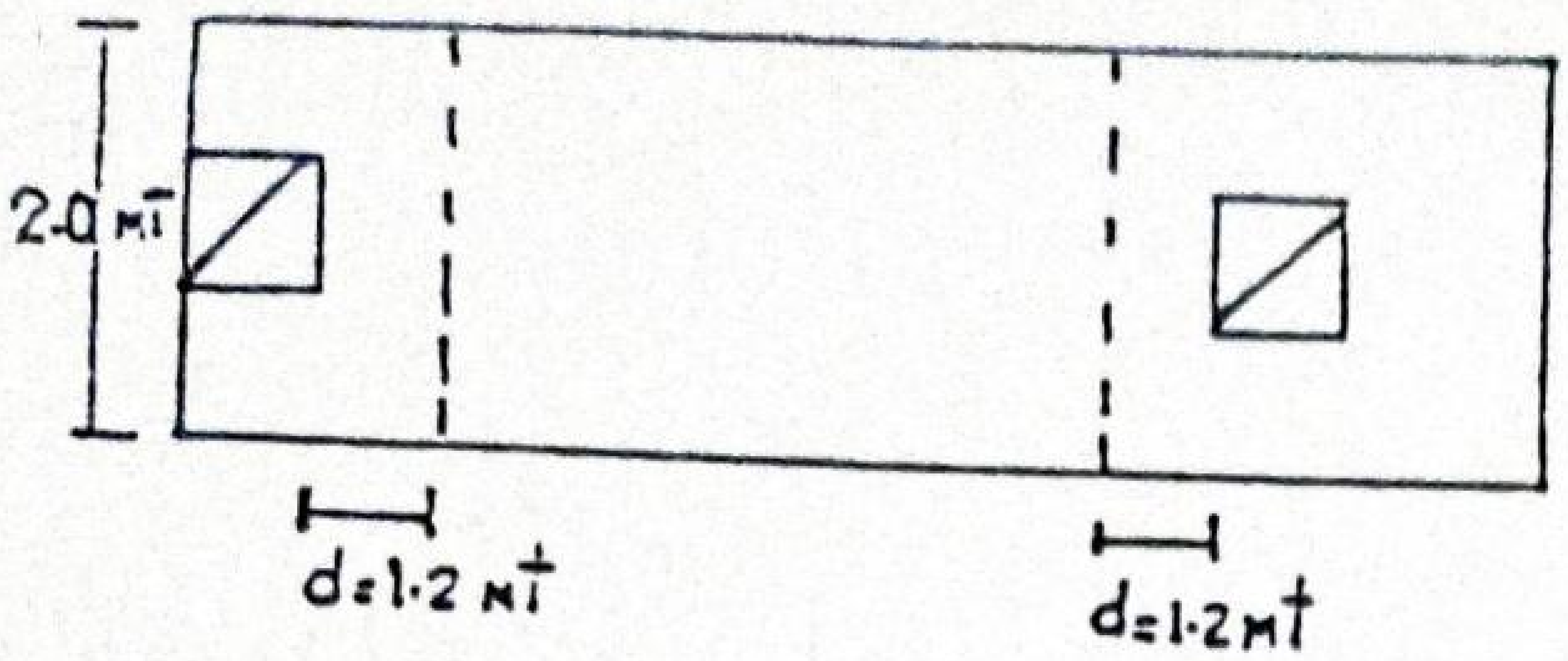
$$V_{ui} = \frac{365.2 - 2.56(27)}{0.85 \times 6.4 \times 1.2}$$

$V_{ui} = 45.36\ T/M^2 = 4.54\ K/cm^2$

∴ $V_{ui} = 4.54\ K/cm^2 < V_c = 15.94\ K/cm^2$ OK!

PASO 6: CORTE POR FLEXION:

A. ZAPATA EXTERIOR



$$V_{ue} = \frac{V_{MAX}^{(-)} - d(W_R)}{0.85 \times B \times d}$$

$$V_{ue} = \frac{166.08 - 1.2(50)}{0.85 \times 2 \times 1.2} = 52\ TN/M^2$$

$$V_c = 0.53\sqrt{210} = 7.68\ K/cm^2$$

$$V_{ue} = 5.2\ K/cm^2 < 7.68\ K/cm^2 \quad OK!$$

$$V_c/2 = 7.68/2 = 3.84\ K/cm^2$$

SEGUN R.N.C:

SI $V_u \leq V_c/2$ NO SE COLOCARA "As" VERTICAL MINIMA

$\therefore$ $5.2 \leq 3.84$ NO CUMPLE (SE COLOCARA Av_{min}

B. ZAPATA INTERIOR:

$$V_{ui} = \frac{V_{MAX}^{(+)} - d(W_R)}{0.85 \times B \times d}$$

$$V_{ui} = \frac{211.42 - 1.2(50)}{0.85 \times 2.0 \times 1.2} = 74.23\ T/M^2$$

$$V_{ui} = 7.42\ K/cm^2 < V_c = 7.68\ K/cm^2 \quad OK!$$

$V_{ui} > V_c/2$ (SE COLOCARA Av_{MIN})

NO CUMPLE

PASO 7: CALCULO DEL AREA DE ACERO:

1: LONGITUDINAL: SEGUN R.N.C (ZAPATAS) $\rho_{MIN} = 0.0017$

* PARA $As_{(-)} \rightarrow$ $Mu_{MAX} = 308.91$ M-T $< M_b$ (MOMENTO BALANCEADO)
(ENTRE COLUMNAS)

TANTEO: FORMULAS: $As = \dfrac{Mu}{\phi . f_y (d - a/2)} = \dfrac{30{,}891}{0.9(4.2)(120 - 30/2)} = 108.96 \text{ cm}^2$

$$a = \frac{As . f_y}{\beta_3 . f'c . b} = \frac{108.96 \times 4200}{0.9(210)(200)} = 12.1 \text{ cm}$$

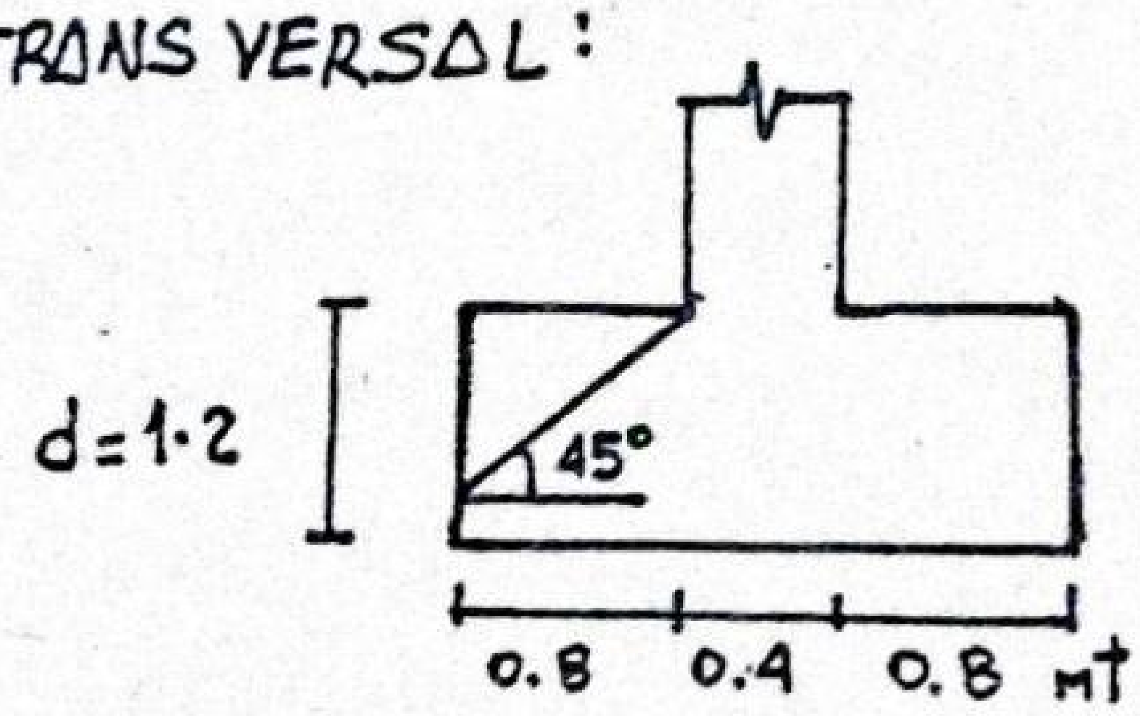
HASTA LLEGAR $As = 70.45 \text{ cm}^2$
$a = 8$ cm ($14 \phi 1''$ @ 0.125 mt)

* PARA $As_{(+)}$ (Z_{INT} DE COLUMNA)

$Mu = 195.71$ T-M $<$ M BALANCEADO

TANTEANDO $As = 44.05 \text{ cm}^2$
$a = 5$ cm ($9 \phi 1''$ @ 0.20 mts)

2: TRANSVERSAL:

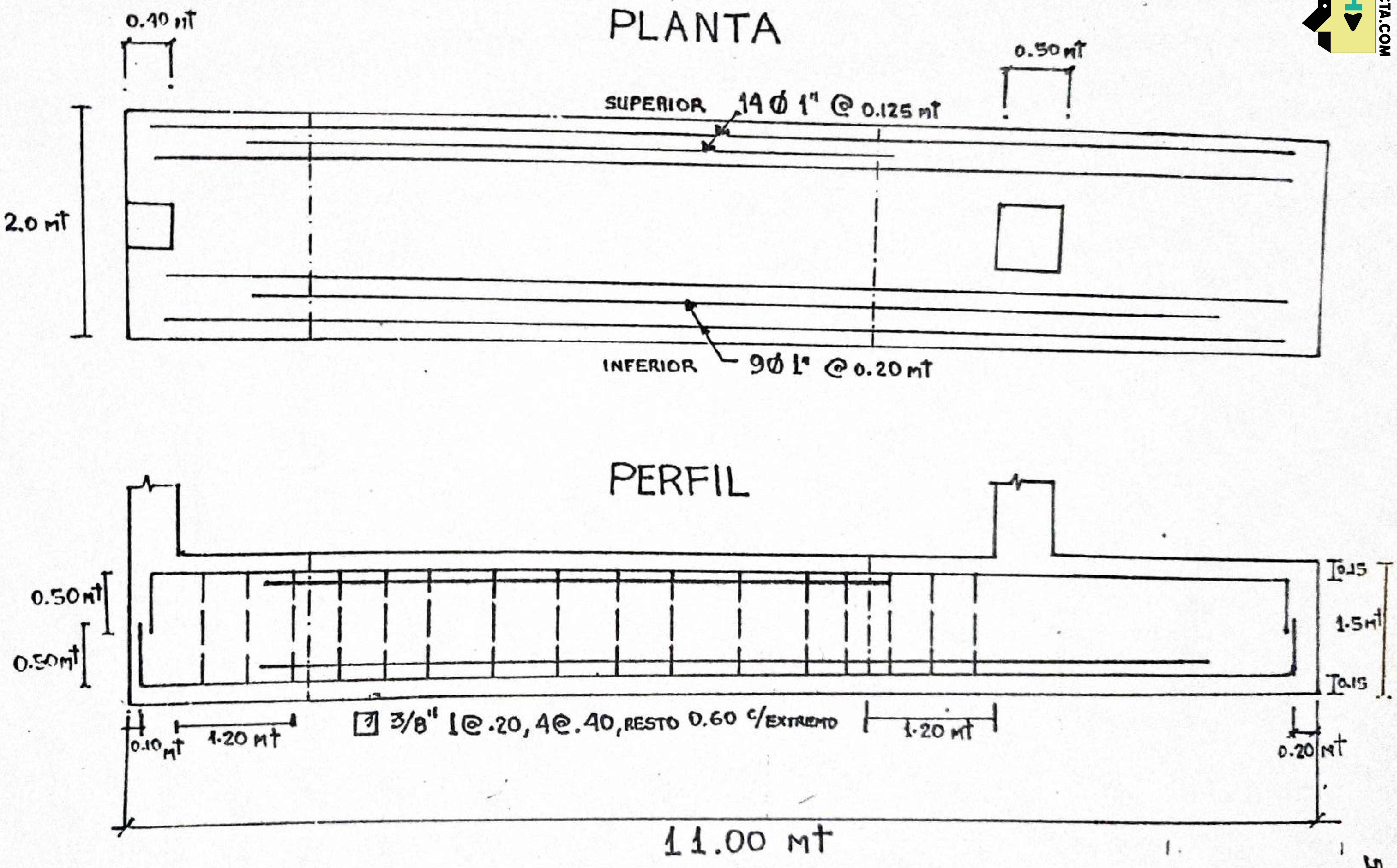


$\rightarrow$ NO SE NECESITARA VIGA TRANSVERSAL

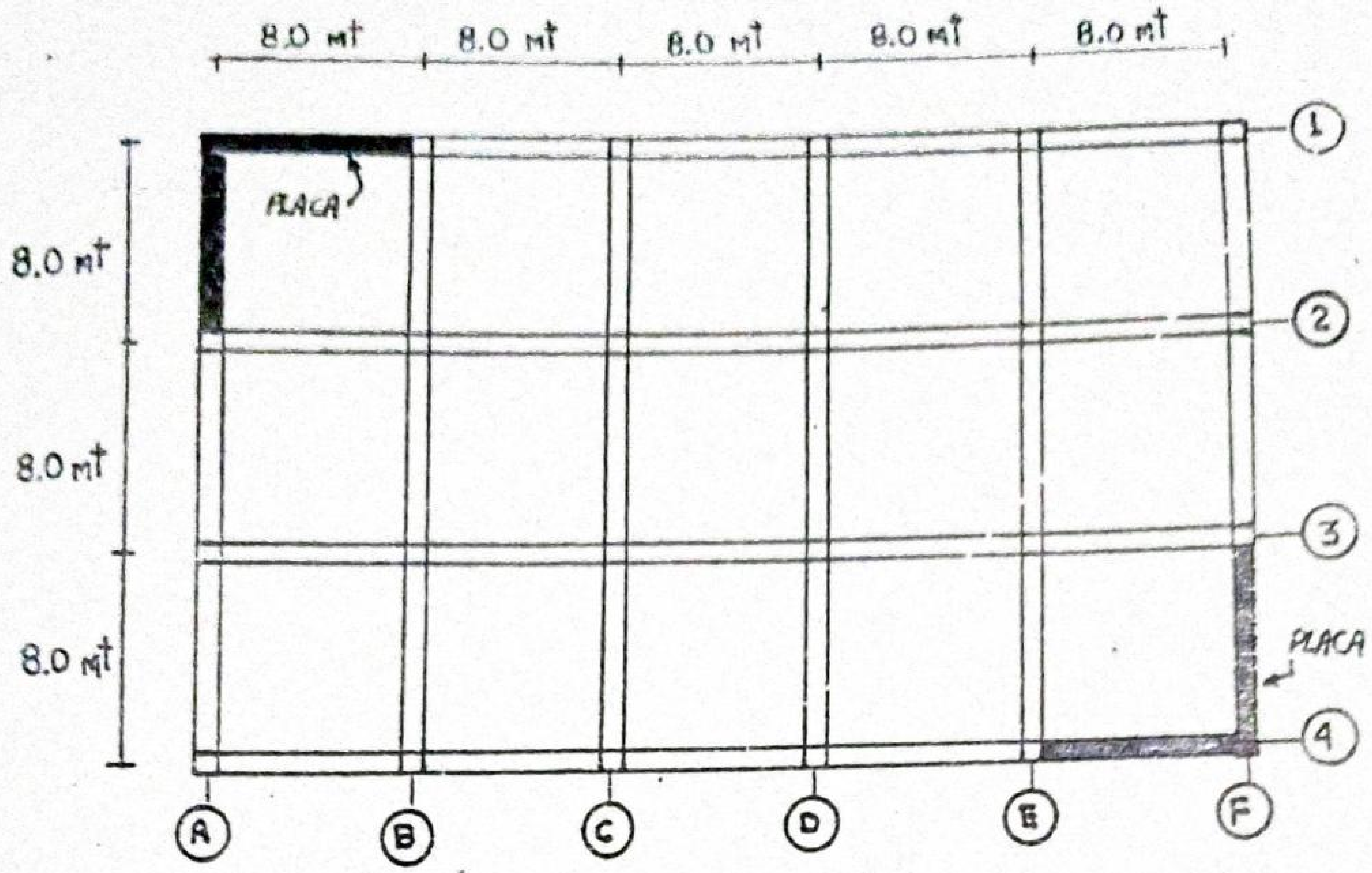
PARA LOS ESTRIBOS: $Av_{MIN} = \dfrac{3.52\, bw\, S_{MAX}}{f_y} = \dfrac{3.52 \times 200 \times 60}{4200} = 10.0$ cm

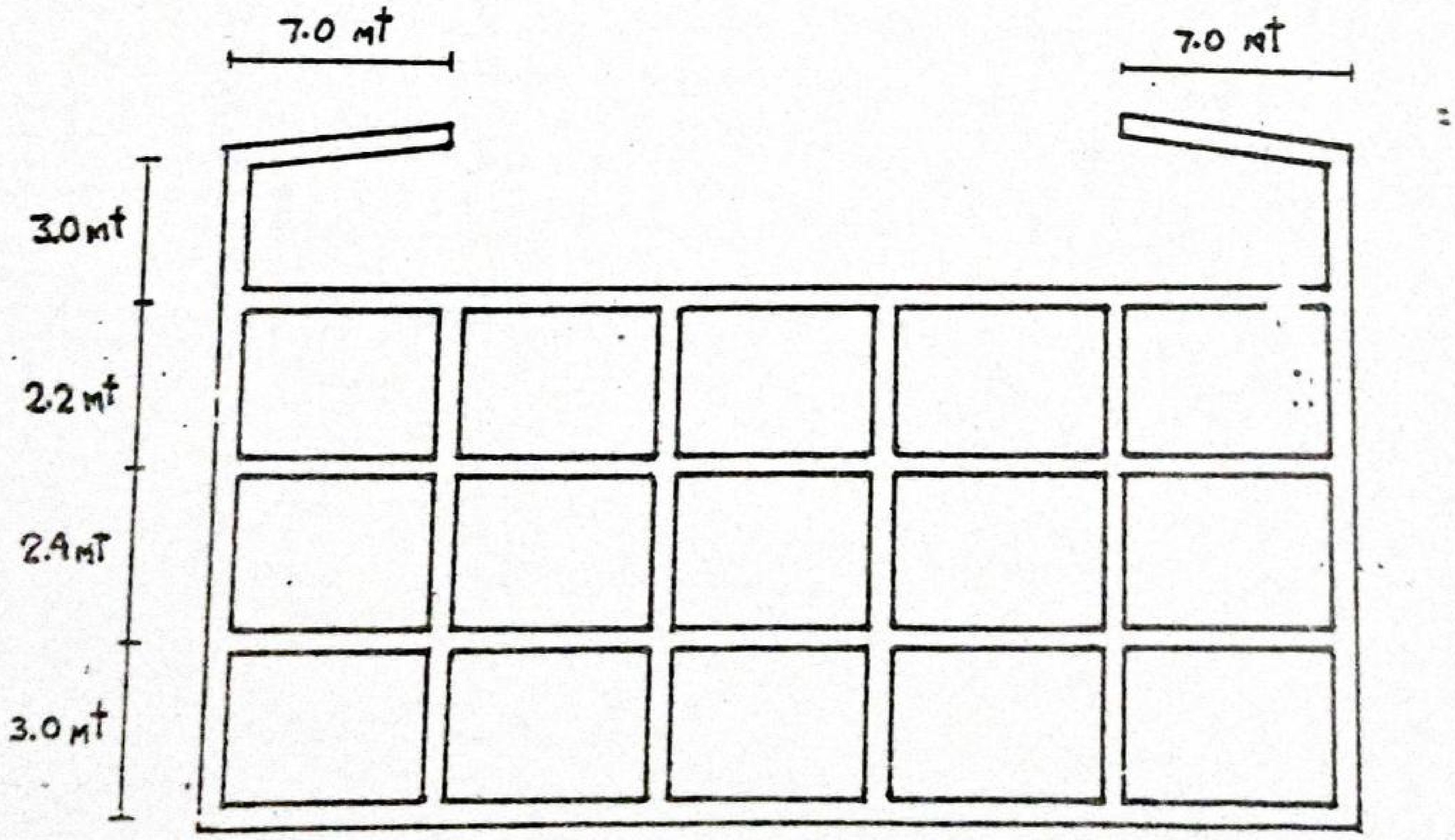
$S_{MAX} = \dfrac{d}{2} = \dfrac{1.2}{2} = 0.6$ mt

$Av_{MIN} = 10.06 \text{ cm}^2$ ($8 \phi 1/2''$)

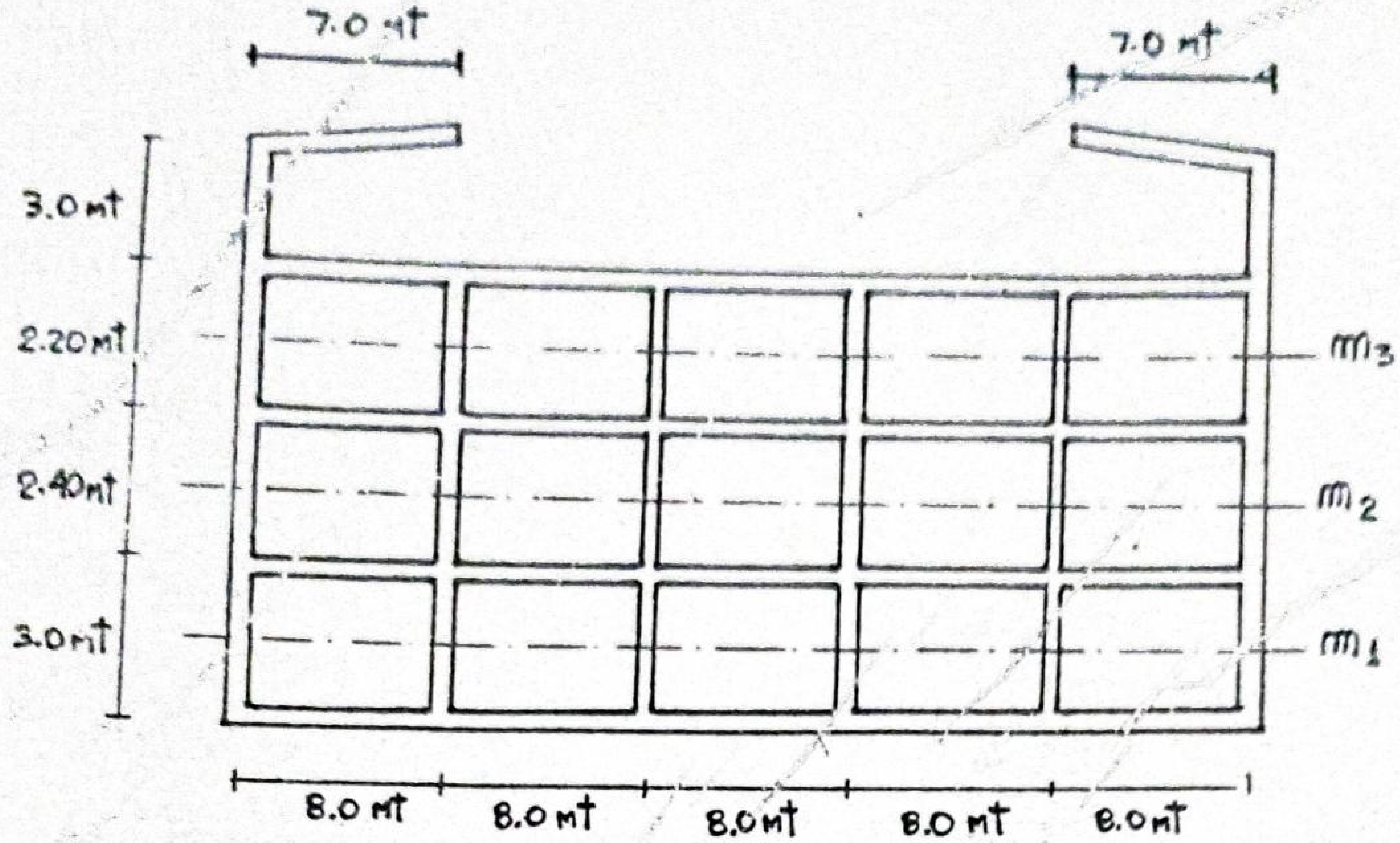

PLANTA
0.10 mt
0.50 mt
SUPERIOR 14 Ø 1" @ 0.125 mt
2.0 mt
INFERIOR 9 Ø 1" @ 0.20 mt
PERFIL
0.50 mt
0.50 mt
0.15
1.5 mt
0.15
0.10 mt
1.20 mt
3/8" 1@.20, 4@.40, RESTO 0.60 C/EXTREMO
1.20 mt
0.20 mt
11.00 mt

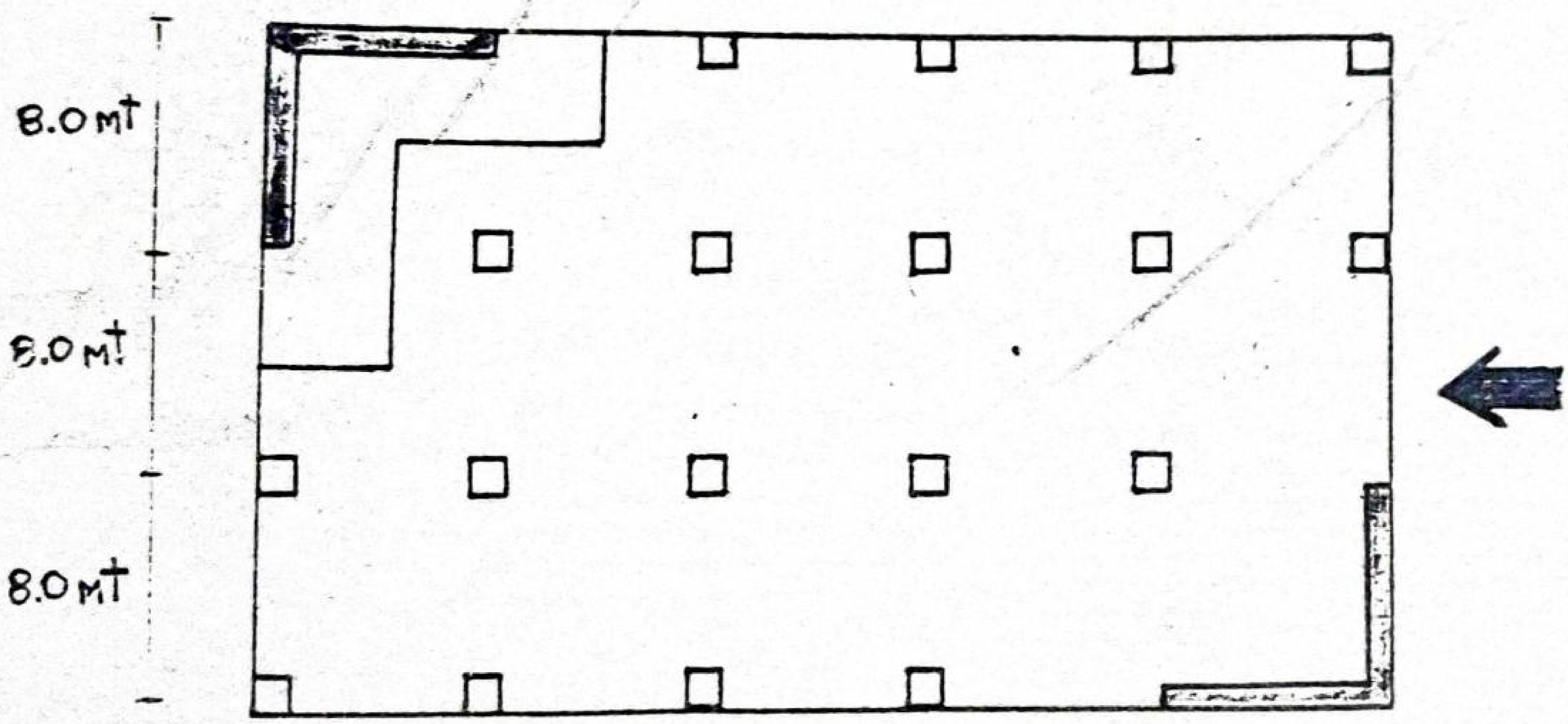
5(7) DISEÑAR:


8.0 mt
8.0 mt
8.0 mt
8.0 mt
8.0 mt
PLACA
8.0 mt
8.0 mt
8.0 mt
1
2
3
4
PLACA
A
B
C
D
E
F


7.0 mt
7.0 mt
3.0 mt
2.2 mt
2.4 mt
3.0 mt

PROB(7) DISEÑO DE PLACAS





CALCULO DE CARGAS (m_3):

COLUMNAS $= [4\times3\times(0.4\times0.4) + (8\times0.5\times0.5 + 10\times0.4\times0.4)\times\frac{2.2}{2}](2.4) = 14.11$ TN
4° NIVEL — 3° NIVEL

LOSA $= 0.25\times\underbrace{(24\times40)}_{\text{PERIMETRO}}\times2.4 = 576$ TN

VIGA $= 14\times7\times\left(\frac{0.50+0.125}{2}\right)\times0.40\times2.4 = 29.40$ TN

ALBAÑILERIA $= [2(24+40) - 2\times8]\times2.4\times0.15\times\underbrace{1.35}_{\gamma_{albañ.}} = 54.43$ TN

ALBAÑILERIA $= [2(24+40) - 4\times8]\times\frac{2.2}{2}\times0.15\times1.35 = 21.38$ TN

$\Sigma = 75.81$ TN

PLACA $= (8\times2)\times3\times0.2\times2.4 + (4\times8)\left(\frac{2.2}{2}\right)\times0.20\times2.4 = 39.94$ TN

W_{D1} = COLUMNAS + LOSA + VIGA + ALBAÑILERIA + PLACA $= 735.27$ TN

S/C $= 0.250\times24\times40 = 240$ TN

$W_{D1} = 735.27$ TN
$W_{L1} = 240.0$ TN

CALCULOS DE CARGAS (m_2):

COLUMNAS $= [8\times0.5\times0.5 + 10\times0.4\times0.4]\left(\frac{2.2+2.4}{2}\right)(2.4) = 19.87$ TN

LOSA $= 0.25\times(24\times40)\times2.4 = 576$ TN

ALBAÑILERIA $= [2(24+40) - 4\times8]\left(\frac{2.2+2.4}{2}\right)\times0.15\times1.35 = 44.71$ TN

PLACA $= 4\times8\left(\frac{2.2+2.4}{2}\right)\times0.2\times2.4 = 35.33$ TN

$W_{D2} = 675.91$ T
$W_{L2} = 240.0$ TN

W_{D2} = COLUMNAS + LOSA + ALBAÑILERIA + PLACA $= 675.91$ TN

CALCULOS DE CARGAS (m_3):

COLUMNA $= [8\times0.5\times0.5 + 10\times0.4\times0.4]\left(\frac{2.4+3.0}{2}\right)(2.4) = 23.33$ TN

LOSA $= 0.25(24\times40)\times2.4 = 576$ TN

ALBAÑILERIA $= [2(24+40) - 4\times8]\left(\frac{2.4+3.0}{2}\right)\times0.15\times1.35 = 52.49$ TN

PLACA $= 4\times8\left(\frac{2.4+3.0}{2}\right)\times0.20\times2.4 = 41.47$ TN

$W_{D3} = 693.29$ T
$W_{L3} = 240.0$ T

W_{D3} = COLUMNA + LOSA + ALBAÑILERIA + PLACA $= 693.29$ TN

FUERZA SISMICAS : $H = \dfrac{ZUSC.P}{R_d}$

Z = FACTOR ZONA → LIMA: ZONA I : Z = 1.0

U = FACTOR USO → EDIFICACION TIPO B U = 1.3

S = FACTOR SUELO → SUELO II S = 1.2 ∴ $T_s = 0.6$ DONDE: $0.3 \leq T_s \leq 0$

C = COEFICIENTE SISMICO :

$$C = \frac{0.8}{\frac{T}{T_s}+1} \quad \text{DONDE} \quad T = \frac{0.05\,H}{\sqrt{D}}$$

H = ALTURA TOTAL DEL EDIFICIO

D = DIMENSION DEL EDIFICIO EN LA DIRECCION CONSIDERADA DE LA FUERZA SISMICA.

H = 3+2.4+2.2+3 = 10.6 mt

D = 8×5 = 40 mt²

$$T = \frac{0.05(10.6)}{\sqrt{40}} = 0.083$$

$0.16 \leq C \leq 0.40 \ldots (1)$

$$C = \frac{0.8}{\frac{0.083}{0.6}+1} = 0.70$$

DE(1) C = 0.40

Rd = FACTOR DUCTIL : EDIFICACION EN QUE LAS FUERZAS ABSORVEN MUROS DE CORTES O ESTRUCTURAS SIMILARES (PLACAS) ∴ Rd = 3.0

VALOR DE "P" POR SER TIPO "B" (U=1.3)

$P = \Sigma W_D + 50\% \Sigma W_L$

P = 2104.5 + 0.5 (3×240)

∴ P = 2464.5 TN

$$H = \frac{(1 \times 1.3 \times 1.2 \times 0.4)}{3}(2464.5)$$

H = 512.62 TN

DISTRIBUCION DE LA FUERZA CORTANTES EN LOS NIVELES :

$\frac{ALTO}{ANCHO} > 6 \rightarrow f = 0.85$

$\frac{ALTO}{ANCHO} < 3 \rightarrow f = 1.00$

$\frac{10.6}{40} = 0.265 < 3$ ∴ f = 1.00

N	P_i (TN)	h_i (mt)	$P_i h_i$	$\frac{P_i h_i}{\Sigma P_i h_i}$	$F_i = f \frac{P_i \times h_i}{\Sigma P_i h_i} \times H$
3	855.27	7.6	6,500	0.491	251.70
2	795.91	5.4	4,298	0.324	166.09
1	813.29	3.0	2,440	0.184	94.32
		Σ =	13,238		

$$F = \frac{I_x}{\sum m I_x} F_i$$

$$I_{\square}^{P-1} = \frac{0.2 \times 8^3}{12} = 8.53$$

$$I_{\square}^{P-2} = \frac{8 \times 0.2^3}{12} = 0.0053 \qquad \sum_m I_x = N^\circ I$$

$$I_{\square}^{C-1} = \frac{0.4 \times 0.4^3}{12} = 0.00213$$

$$I_{\square}^{C-2} = \frac{0.5 \times 0.5^3}{12} = 0.0052$$

$$\sum_m I_x = 2 \times 8.53 + 2(0.0053) + 10(0.00213) + 8(0.0052) = 17.13$$

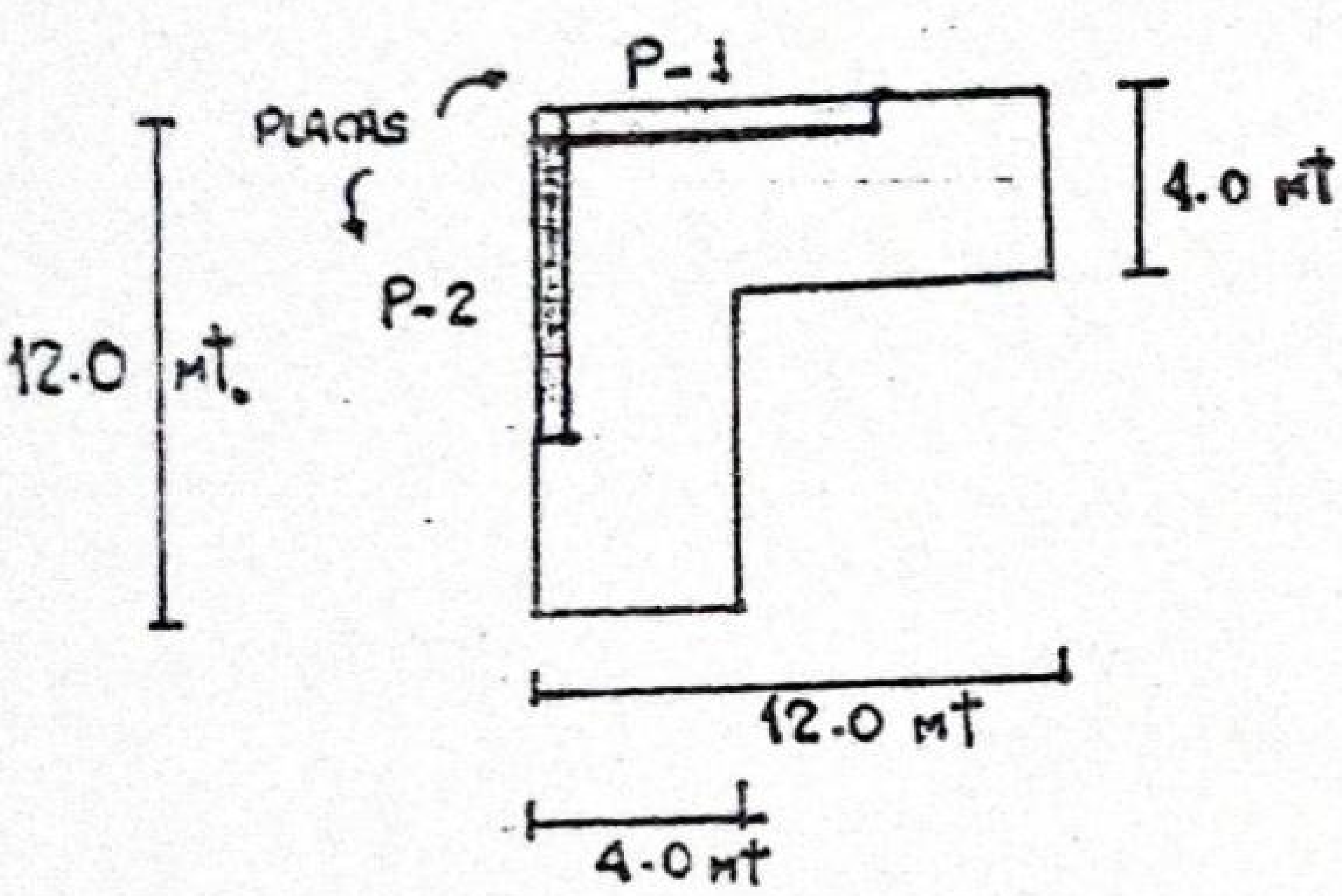
$$F = \frac{8.53}{17.13} \cdot F_i$$

$F_3 = 0.498 \times 251.70 = (125.31\ Tn)$

$F_2 = 0.498 \times 166.09 = (82.69\ Tn)$

$F_1 = 0.498 \times 94.32 =$ [redacted] $= (46.97\ Tn)$

DISTRIBUCION DE LAS CARGA VERTICALES EN CADA NIVEL:



AREA TRIBUTARIA: A_T

$$A_T = \frac{12 \times 4 + 8 \times 4}{2} = 40\ m^2$$

$$P = (\omega_D + \omega_L)\left(\frac{\text{AREA DE LA PLACA}}{\text{AREA TOTAL}}\right)$$

$$P_3 = (735.27 + 240.0)\left(\frac{40}{24 \times 40}\right) = 40.64\ Tn$$

$$P_2 = (675.91 + 240.0)\left(\frac{40}{24 \times 40}\right) = 38.16\ Tn$$

$$P_1 = (693.29 + 240.0)\left(\frac{40}{24 \times 40}\right) = 38.89\ Tn$$

MAYORANDO LAS CARGAS:

$Pu_1 = 38.89\ (1.65) = 64.17\ Tn$

$Pu_2 = 38.16\ (1.65) = 62.96\ Tn$

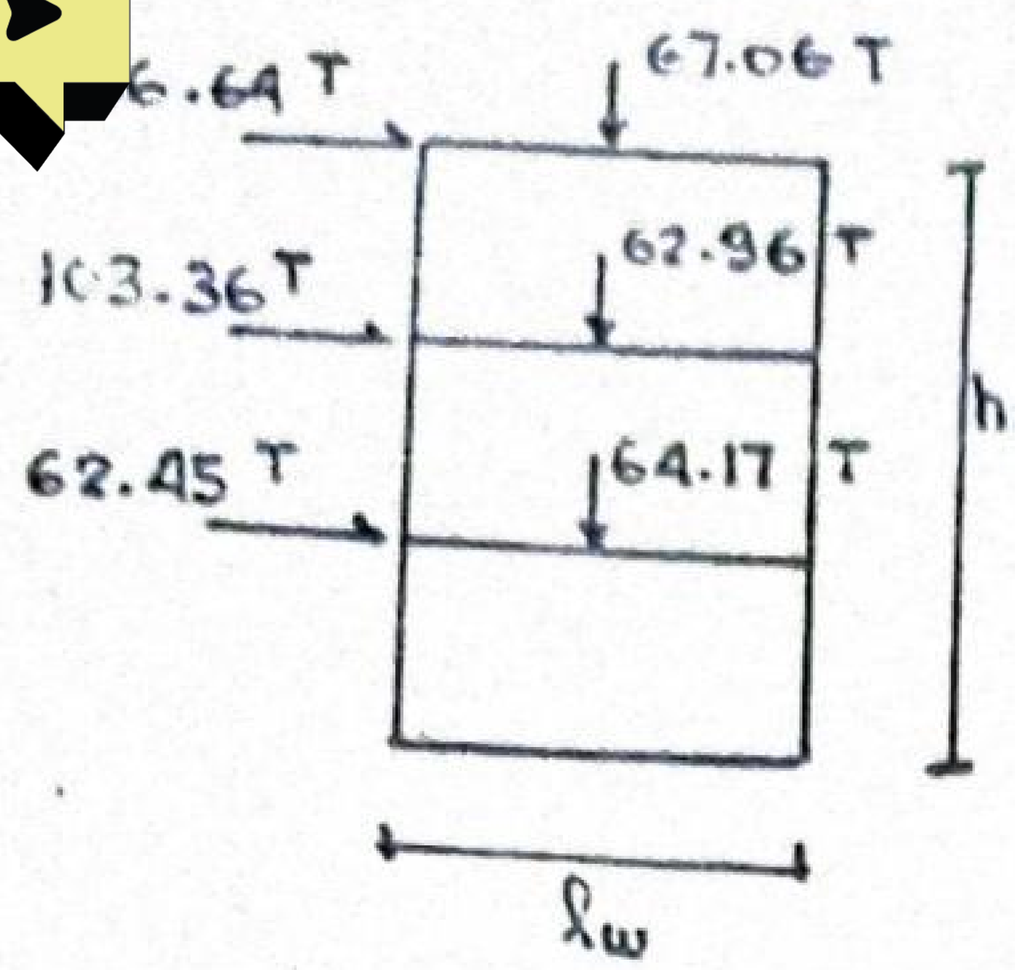
$Pu_3 = 40.64\ (1.65) = 67.06\ Tn$

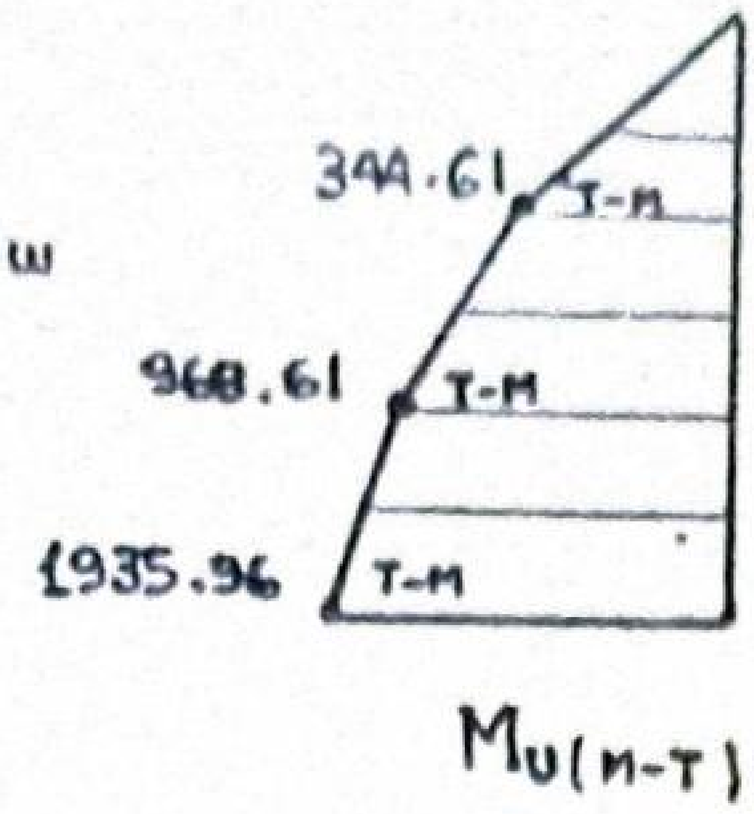
$Nu = 194.19\ Tn$

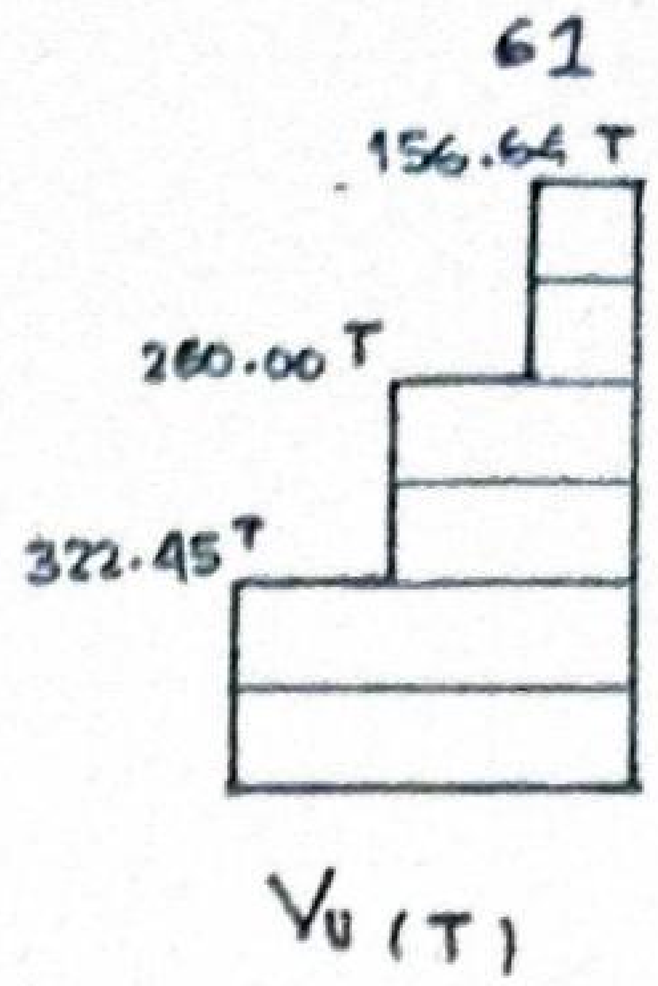
$Fu_1 = 46.97\ (1.25) = 62.45\ Tn$

$Fu_2 = 82.69\ (1.25) = 103.36\ Tn$

$Fu_3 = 125.31\ (1.25) = 156.64\ Tn$







DISEÑO POR FLEXION : $d = 0.8\, l_w = 0.8(800) = 640$ cm $\quad \phi = 0.9$

$$A_s = \frac{M_U}{\phi f_y . d} = \frac{1935.96}{0.9(4.2)(640)} = 80.02 \text{ cm}^2$$

ASUMIENDO $A_{s_{TOTAL}} = 3A_s = 3(80.02)$

$A_{s_T} = 240.06 \text{ cm}^2$

$$\alpha = \frac{N_U}{l_w . h . f'_c} = \frac{194.19}{800(20)(0.21)} = 0.0578 \qquad \beta_1 = 0.85$$

$$\beta = \frac{f_y}{6000} = \frac{4200}{6000} = 0.7 \; ; \qquad \rho_v = \frac{A_s}{l_w . h} = \frac{240.06}{800(20)} = 0.015$$

$$q = \frac{\rho_v . f_y}{f'_c} = \frac{0.015 \times 4200}{210} = 0.30$$

$$C = l_w \left(\frac{q + \alpha}{2q + 0.85\beta_1}\right) = 800\left(\frac{0.3 + 0.0578}{2(0.3) + (0.85)^2}\right) = 216.44$$

$$M_U = \phi A_s f_y l_w \left[\left(1 + \frac{N_U}{A_s f_y}\right)\left(\frac{1}{2} - \frac{\beta_1 C}{2 l_w}\right) - \frac{C^2}{l_w^2}\left(1 + \frac{\beta^2}{3} - \beta_1\right)\right]$$

$$M_U = 0.7(240.06)(4.2)(800)\left[\left(1 + \frac{194.96}{240.06 \times 4.2}\right)\left(\frac{1}{2} - \frac{0.85(216.44)}{2(800)}\right) - \left(\frac{216.44}{800}\right)^2\left(1 + \frac{0.7^2}{3} - 0.85\right.\right.$$

$M_U = 246520.08$ cm-T $\simeq 2465.2$ M-T

$\therefore$ $M_U = 2465.2$ mt-Tn $\geq M = 1935.96$ M-

OK!

DISEÑO POR CORTE : $V_{U_{CRITICO}} \begin{cases} h_w/2 = 7.6/2 = 3.8 \text{ mt} \rightarrow \text{LA MENOR} \\ l_w/2 = 8.0/2 = 4.0 \text{ mt} \end{cases}$

$V_{U_{CRITICO}} = 266$ TN $\rightarrow \therefore 3.8$ mt $\rightarrow 2^{\circ}$ NIVEL

$$v_U = \frac{266000}{\phi . d . h} = \frac{266000}{0.85(640)(20)} = 24.45 \text{ k/cm}^2$$

$$v_U \leq 2.7\sqrt{f'_c} = 2.7\sqrt{210} = 39.13 \text{ k/cm}^2$$

$24.45 \text{ k/cm}^2 < 39.13 \text{ k/cm}^2$ OK!

$$V_C = 0.87\sqrt{f'c} + \frac{Nu}{4\,lw\,h} = 0.87\sqrt{210} + \frac{194{,}190}{4(800)(20)} = 15.64\ k/cm^2$$

$$\frac{Mu}{Vu} - \frac{lw}{2} = \frac{1935.96}{194.19} - \frac{8}{2} = 5.97\ mt = 597\ cm.$$

TOMAMOS EL MENOR $V_c = 11.98$

$$V_C = 0.16\sqrt{f'c} + lw\left[\frac{0.33\sqrt{f'c} + 0.2\,(Nu/(lw.h))}{\left(\frac{Mu}{Vu} - \frac{lw}{2}\right)}\right]$$

$$V_C = 0.16\sqrt{210} + 800\left[\frac{0.33\sqrt{210} + 0.2\left(\frac{194{,}190}{800 \times 20}\right)}{597}\right] = 11.98\ K/cm^2$$

$$\therefore \quad V_U = 24.45\ K/cm^2 > V_C/2 = 11.98/2 = 5.99\ K/cm^2$$

$\therefore$ PROPORCIONAREMOS REFUERZO HORIZONTAL VERTICAL

HORIZONTAL: $\rho_h = \frac{Ash}{Sh \times h} = \frac{(V_U - V_C)}{f_Y} = \frac{(24.95 - 11.98)}{4200} = 0.00297 > \rho_{MIN} = 0.0025$

SE TOMA EL MAYOR

$$Sh \le \begin{cases} lw/5 = 800/5 = 160\ cm \\ 3h = 3(20) = 60\ cm \\ 45\ cm \end{cases}$$

EL MENOR:

$Ash = \rho_h \times Sh \times h$

SE ASUME $Sh = 0.40\ mt$ $\quad Ash = 0.00297 \times (40)(20)$

$Ash = 2.38\ cm^2$ $(2\phi\,1/2'' @ 0.40\ mt)$

VERTICAL: $\rho_V = \frac{Asv}{h\,lw}$

$$\rho_V \ge \begin{cases} 0.0025 + 0.5\left(2.5 - \frac{hw}{lw}\right)(\rho_h - 0.0025) = 0.0025 + 0.5\left(2.5 - \frac{300}{800}\right)(0.00297 - 0.0025) = 0 \\ 0.0015 \end{cases}$$

$hw \rightarrow$ 1º PISO

$0.0030 > 0.0015$ $\qquad \therefore \rho_V = 0.003$

$$Sv \le \begin{cases} lw/3 = 800/3 = 266.67\ cm \\ 3h = 3(20) = 60\ cm \\ 45\ cm \leftarrow \text{EL MENOR} \end{cases}$$

$Asv = \rho_V \times h \times lw$

$Asv = 0.003(20)(800) = 48\ cm^2$

COMPARANDO CON EL As CALCULADO POR FLEXION: TENEMOS QUE:

$Asv < As\,(FLEXION)$

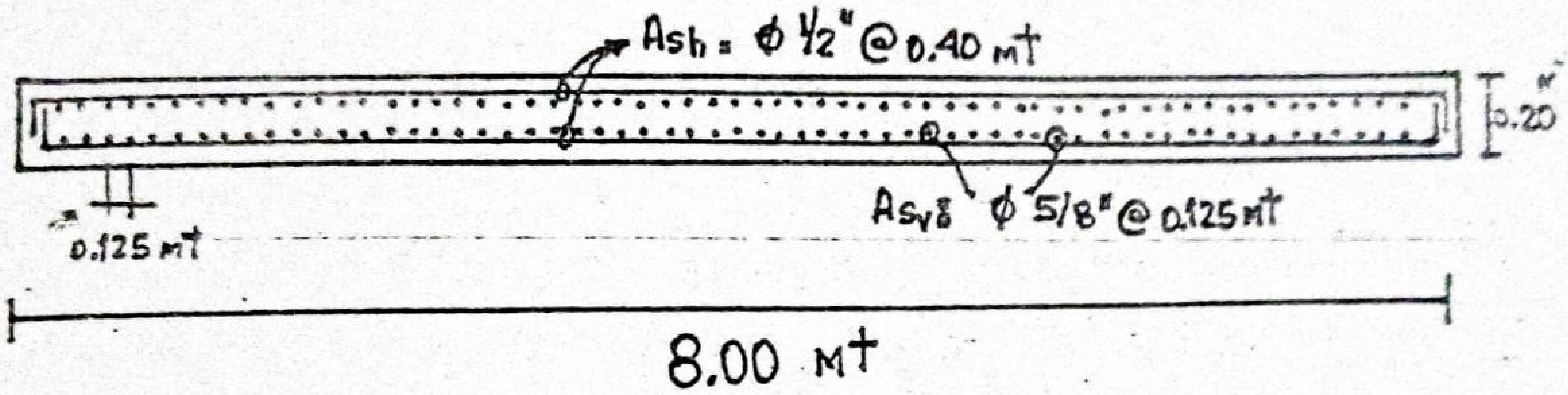
$48\ cm^2 < 240.06\ cm^2$ C

$\therefore Asv = 240.06\ cm^2$ $(122\phi\,5/8'') \rightarrow 61\phi\,5/8''$ CADA HILERA

SEPARACION $(S_{REAL}) = \frac{800 - 10}{60} = 13\ cm \rightarrow 0.13\ mt$

$\therefore$ $61\phi\,5/8''$ C/HILERA : $S_{MAX} = 0.125\ mt$

DISTRIBUCION EN PLACA

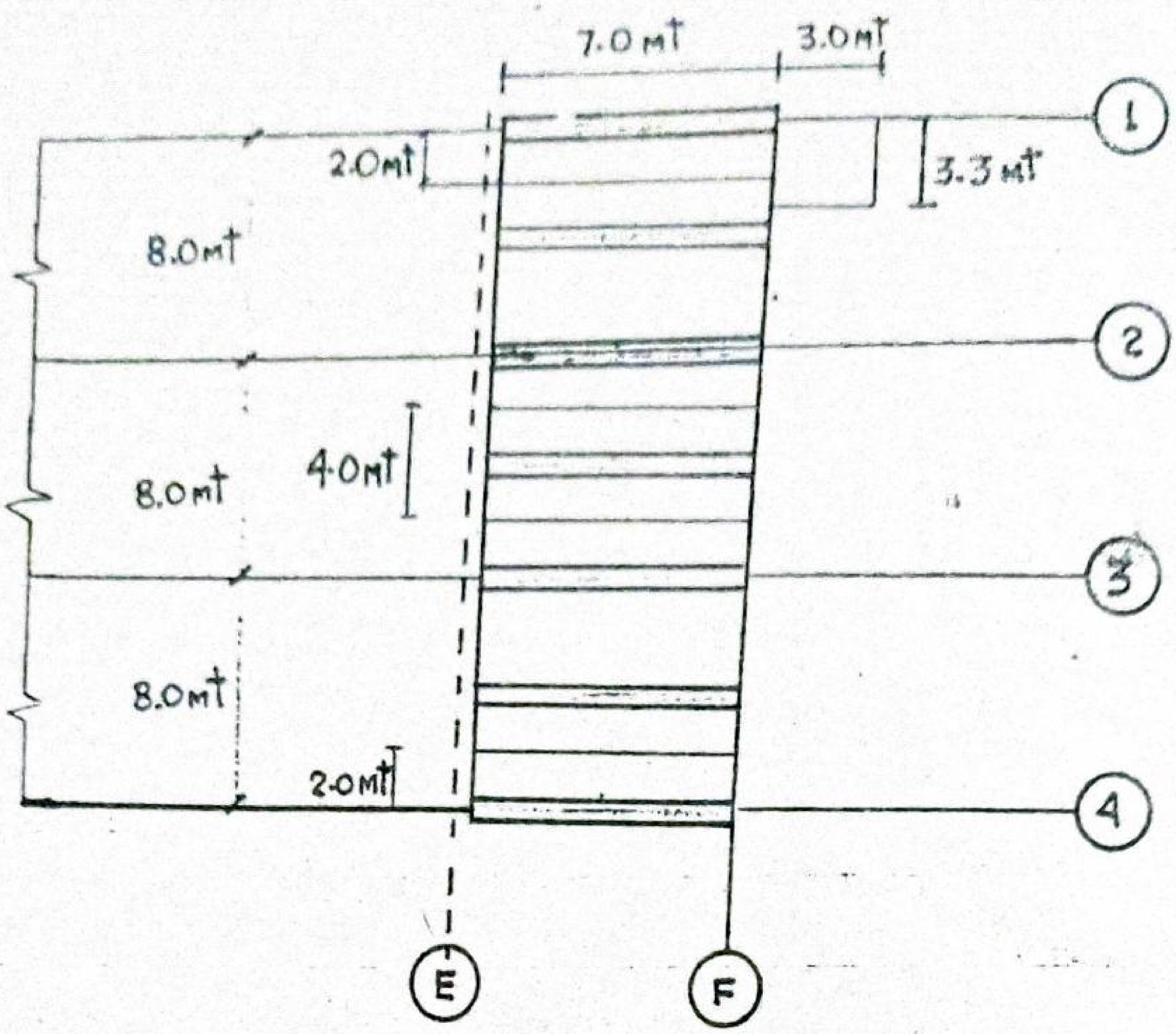


ESCALA:

H: 1/50

V: 1/20

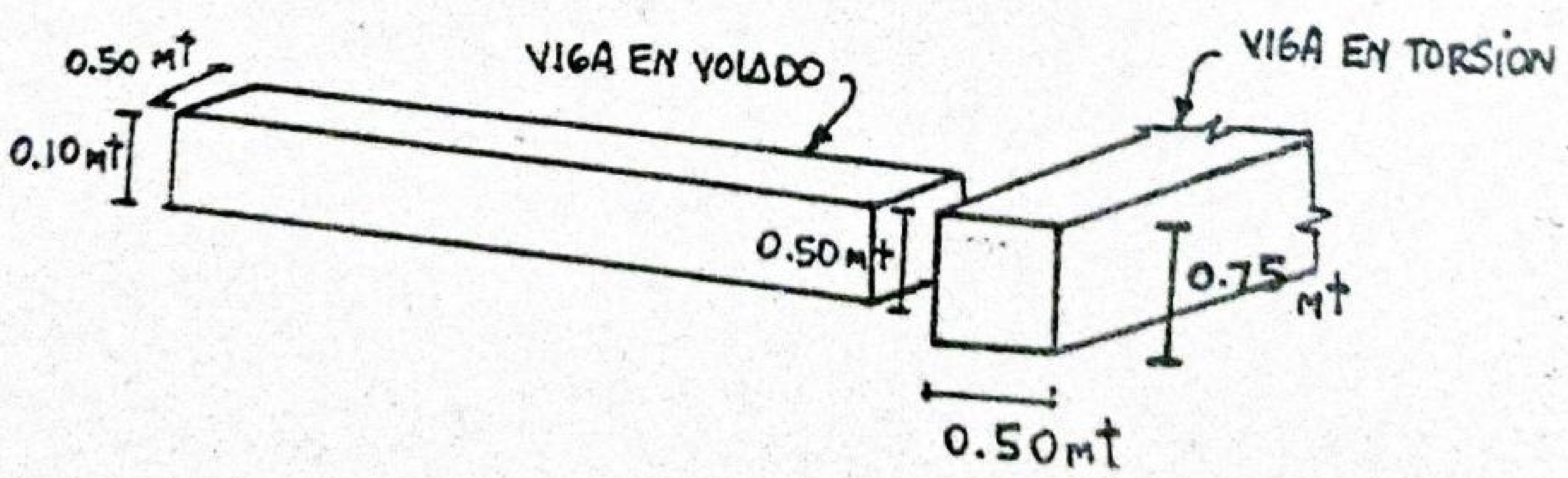
PROB (8) DISEÑO DE VIGA EN TORSION :



SOLUCION :

VIGA EN VOLADO: EJES (4)

ASUMIMOS LAS DIMENSIONES SIGUIENTES:



CAR. AS :

$W_D = \left(\frac{0.10+0.5}{2}\right)(7.0)(0.50)(2.4) = 2.52$ TN

S/c = 25 K/m² Atributaria = 2×7 = 14 mt

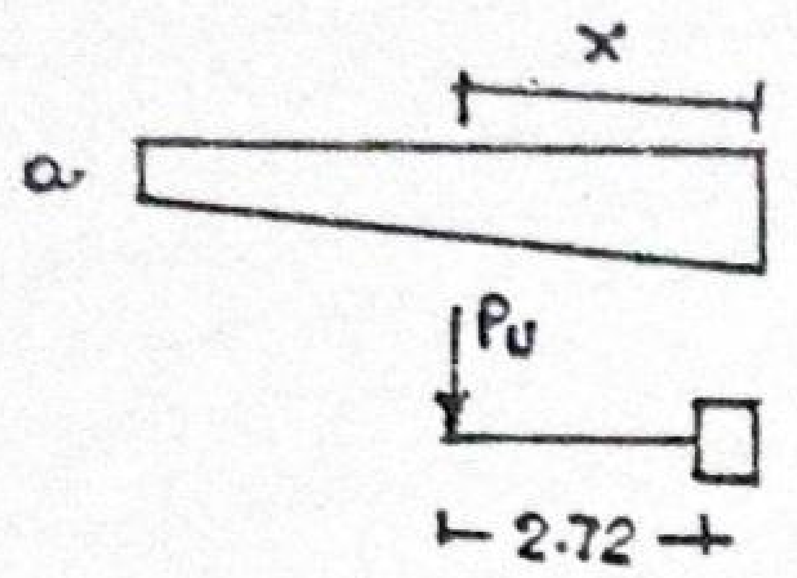
L = 14×25 = 350 Kg ≈ 0.35 TN

∴ 1.5(2.52)+1.8(0.3…

P_U … 41 TN

CALCULO DEL M_U.

$C.G = \frac{b}{3}\left(\frac{2a+b}{a+b}\right) = \frac{7}{3}\left(\frac{2(0.10)+0.5…}{0.10+0.50}\right)$ …72 mt



$\bar{X}$ = CG = 2.72 mt

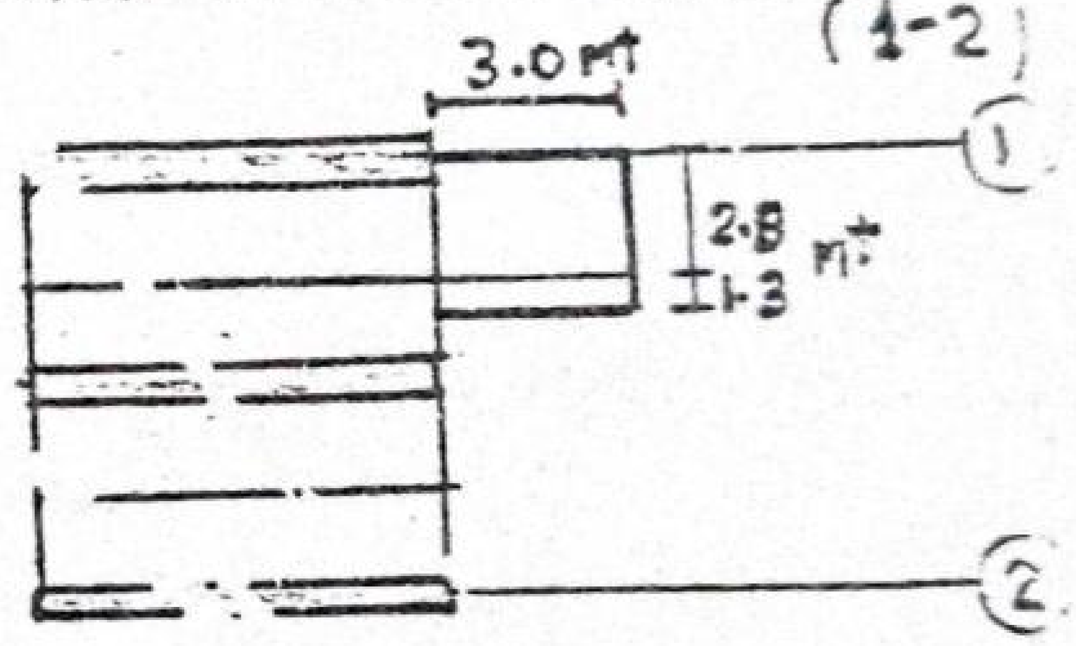
$M_U = P_U . \bar{X} = 4.41 \times 2.72 = 12$ TN-mt

VIGA EN VOLADO ENTRE EJES : (INTERMEDIOS)

W_D = 2.52 TN
S/c = 25 K/m²
W_L = 28×25 = 0.7 TN

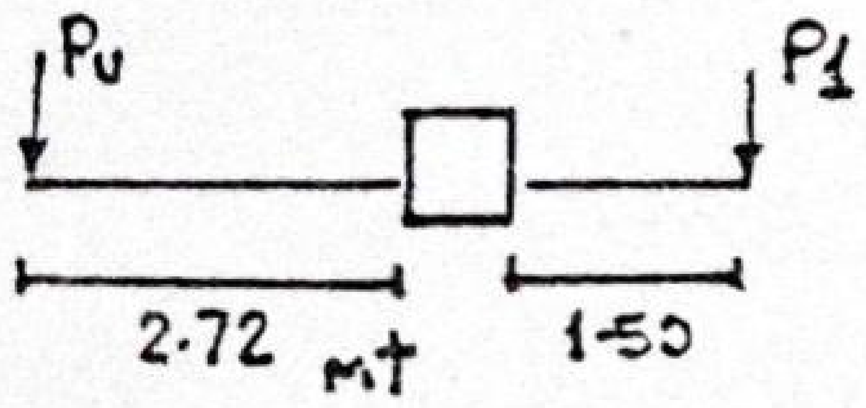
P_U = 1.5(2.52) + 1.8(0.7) = 5.04 TN
$M_U = P_U \times \bar{X}$ = 5.04×2.72 = 13.71 T-mt

CALCULO DEL PESO DE LA CARA POSTERIOR . VIGA EN VOLADO ENTRE EL EJE (1-2)



$\frac{l}{10} = \frac{3.0}{10} = 0.30$ cm

P_{LOSA} = 3×3.3×0.3×2.4 = 7.128 TN
P_1 = 3×1.3×0.3×2.4 = 2.81 TN



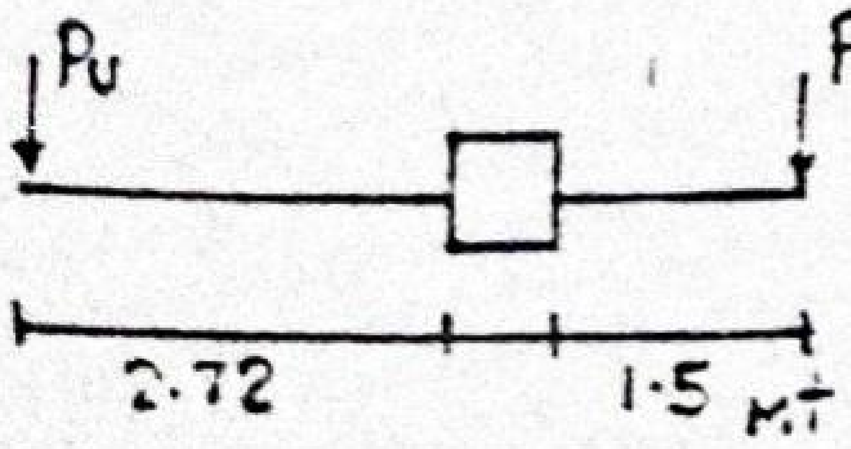
P_1 = 2.81 TN

W_D = 2.52 TN
W_L = 0.70 TN
P_U = 5.04 TN

M_U = (5.04×2.72) − (1.5×2.81) = 5 T-mt

VIGA DE VOLADO DEL EJE (1)

P_2 = 3×2×0.3×2.4 = 4.32 TN



P_U = 5.04 TN

M_U = (5.04×2.72) − (1.5×4.32) = 7.23 T-mt

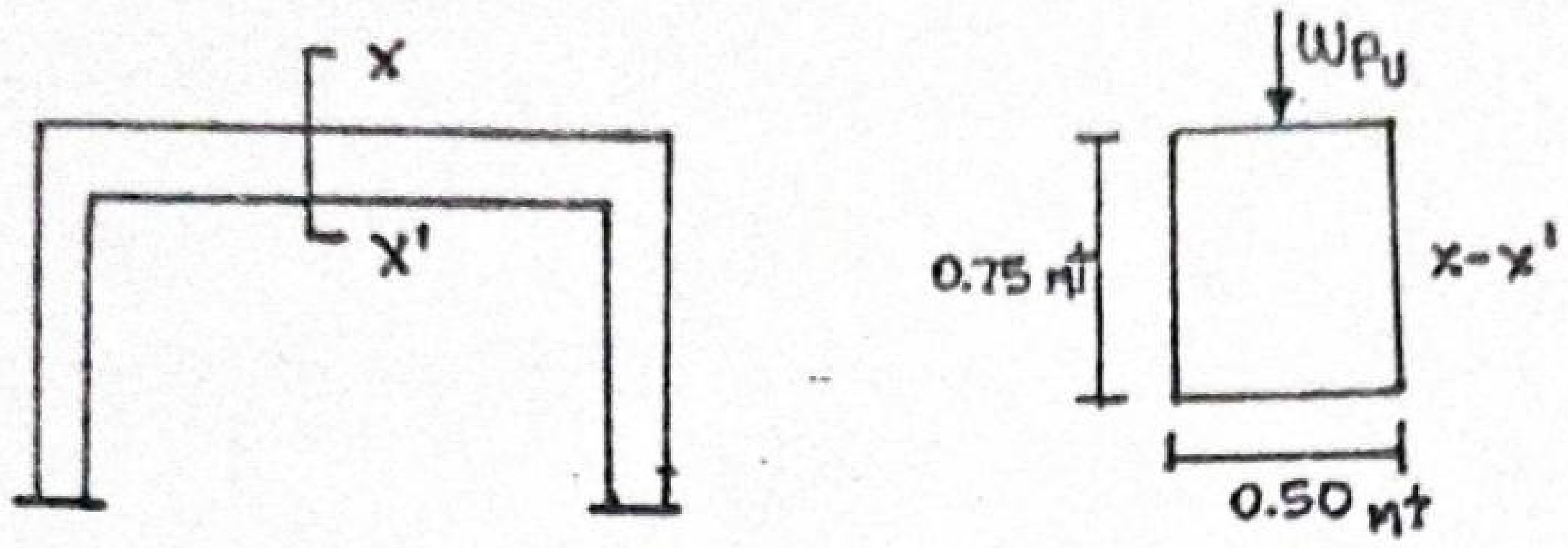
OBSERVAMOS QUE LOS EJES INTERMEDIOS SON LOS MAS CRITICOS

UNIFORMIZANDO: P_U PARA TODO EL TRAMO DE LA VIGA (CADA TRAMO EXISTE 3 VOLADOS)

$$P_U = 3(5.04) = 15.12 \text{ TN}$$

LONGITUD DE TRAMO DE VIGA : $8.0 - 0.5 = 7.5$ mt

$$W_{P_U} = \frac{15.12}{7.5} = 2.02 \text{ T/M}$$



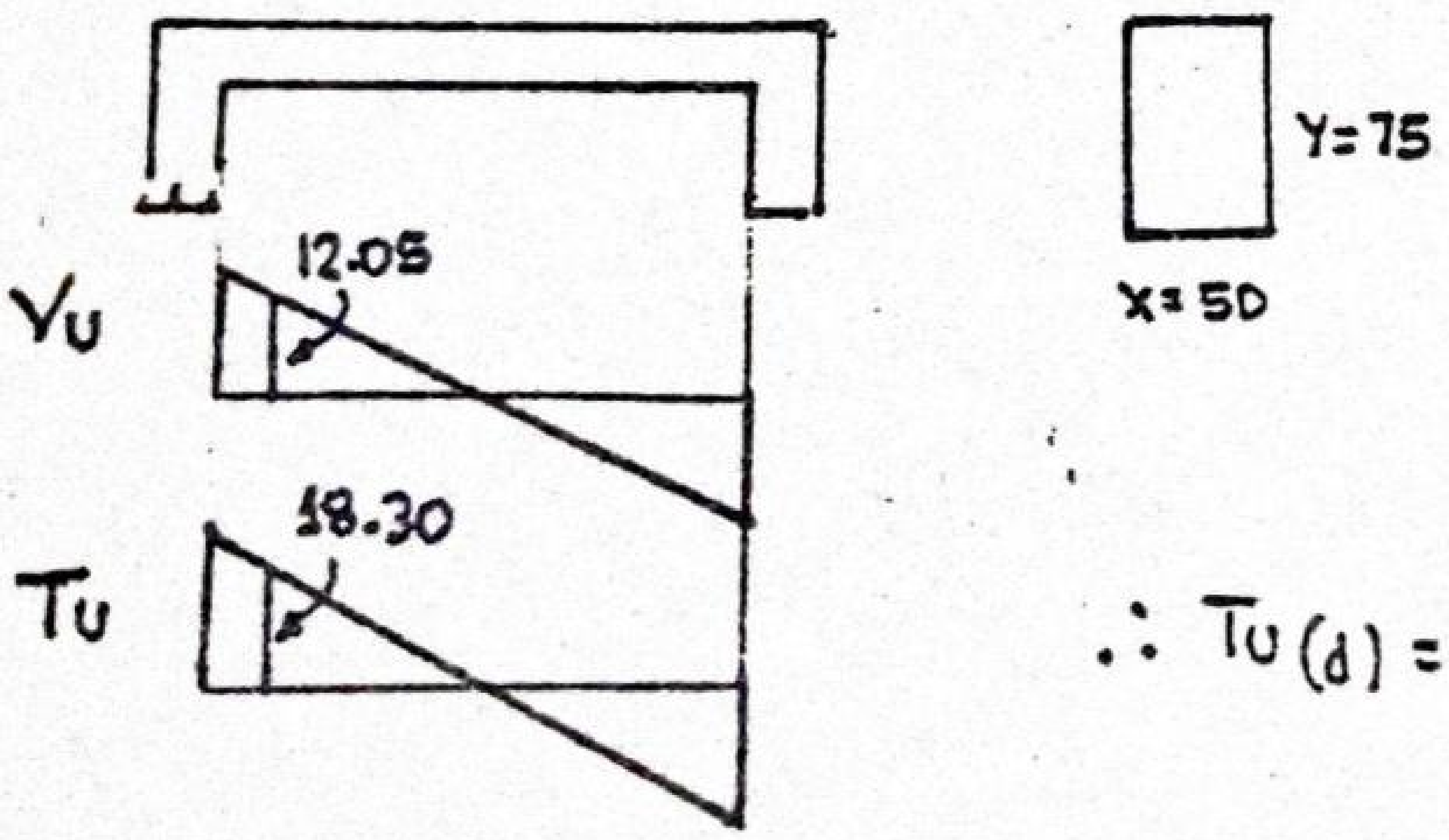
CALCULO DE M_t : $M_t = (2.72 + 0.25)(2.02) = 6$ M-TN.

CALCULO DE W_U : $W_U = 2.6 + (0.75 \times 0.50 \times 2.4)(1.5) = 3.95$ TN

CALCULO DE $V_{U(d)}$ y $T_{U(d)}$: TOMAMOS UN $d = 0.70$ mt

$$V_{U(d)} = W_U\left(\frac{l}{2} - d\right) = 3.95\left(\frac{7.5}{2} - 0.7\right) = 12.05 \text{ TN}$$

$$T_{U(d)} = M_t\left(\frac{l}{2} - d\right) = 6\left(\frac{7.5}{2} - 0.7\right) = 18.3 \text{ TN}$$



$$\Sigma x^2 y = (50)^2(75) = 187{,}500 \text{ cm}^3$$

$$\phi \times 0.13\sqrt{f'c}\,\Sigma x^2 y = 0.85 \times 1.3\sqrt{210} \times 187{,}500$$
$$= 300{,}243 \text{ cm-Kg}$$

$\therefore T_{U(d)} = 1'830{,}000$ cm-K $>$ 300,243 cm-K

SE DISEÑA POR TORSION

Si $T_{U(d)} > \phi T_c$ SE PROPORCIONA REFUERZO TRANSVERSAL Y LONGITUDINAL

$$C_t = \frac{bwd}{\Sigma x^2 y} = \frac{50 \times 70}{187{,}500} = 0.01867$$

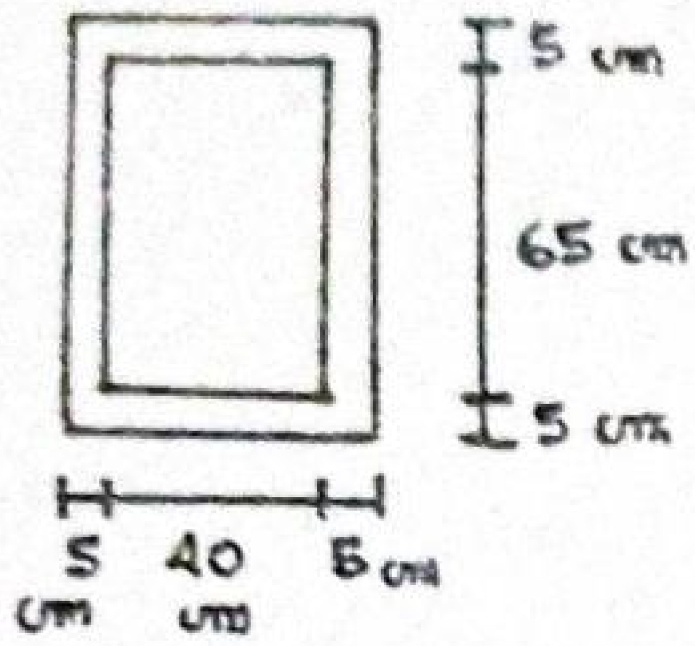
$$T_c = \frac{0.2\sqrt{f'c}\,\Sigma x^2 y}{\sqrt{1 + \left(\frac{0.4 V_U}{C_t T_U}\right)^2}} = \frac{0.2\sqrt{210}\,(187{,}500)}{\sqrt{1 + \left(\frac{0.4 \times 12050}{0.01867 \times 1830{,}000}\right)^2}} = 538{,}098.31 \text{ cm-K}$$

$$c = \frac{0.53\sqrt{f'c}\; b_w d}{\sqrt{1+\left(2.5\, C_t \,\frac{T_u}{V_u}\right)^2}} = \frac{0.53\sqrt{210}\times 50\times 70}{\sqrt{1+\left(2.5\times 0.01867\times \frac{1'830,000}{12050}\right)^2}} = 3,755.14 \text{ KG}$$

$\therefore \quad 1'830,000 > 0.85(538,098.31)$

$T_u > \phi T_c$

CALCULO DE REFUERZO TRANSVERSAL:



$X_1 = 40$ cm

$Y_1 = 65$ cm

$$S \leq \begin{cases} \frac{40+65}{2} = 26.75 \\ 30 \text{ cm} \end{cases} \Big\rangle \; S = 25 \text{ cm}$$

SE TOMA EL MENOR

$$\alpha_t = 0.66 + 0.33\left(\frac{Y_1}{X_1}\right) = 0.66 + 0.33\left(\frac{65}{40}\right) = 1.196$$

$\alpha_t : \therefore \quad 1.196 < 1.5$ OK!

$$A_t = \left[\frac{T_u}{\phi} - T_c\right]\left[\frac{S}{\alpha_t . X_1 . Y_1 f_y}\right]$$

$$A_t = \left[\frac{1'830,000}{0.85} - 538,098.31\right]\left[\frac{25}{1.196(40)(65)4200}\right] = 3.09 \text{ cm}^2$$

POR CADA RAMA

$\therefore$ 2 ◻ 1/2" @ 0.20 mt , 1 ◻ 3/8" @ 0.20 mt.

COMPROBANDO

$$T_s = \frac{A_t\, \alpha_t . X_1 . Y_1 . f_y}{S} = \frac{3.09\times 1.196\times 40\times 65\times 4200}{25} = 1'614,255.55 \text{ cm-K}$$

$4t_c = 4(538,098.31) = 2'152,393.24$ cm-K $\qquad T_s \leq 4t_c$ OK!

CUMPLE

CALCULO DE REFUERZO LONGITUDINAL

$$A_l = 2A_t\left[\frac{X_1+Y_1}{S}\right] = 2(3.09)\left|\frac{40+65}{25}\right| = 25.96 \text{ cm}^2 \quad \text{....(1)}$$

$$A_{l_1} = \left[\frac{28\, X_1 S}{f_y}\left(\frac{T_u}{T_u + \frac{V_u}{3C_t}}\right) - 2A_t\right]\left[\frac{X_1+Y_1}{S}\right] = \left[\frac{28(50)(25)}{4200}\left(\frac{1'830,000}{1'830,000 + \frac{12050}{3(0.01867)}}\right) - 2(3.09)\right]\left[\frac{40+65}{25}\right]$$

$A_{l_1} = 5.36$ cm^2

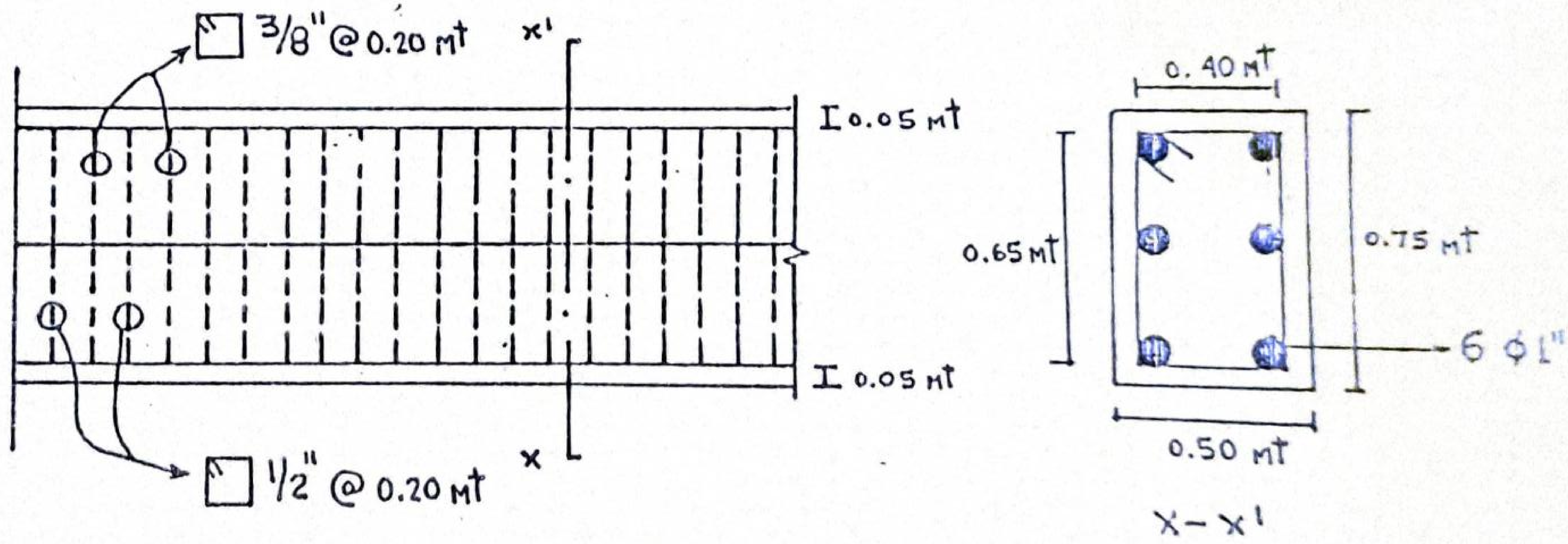
REEMPLAZANDO EN LA FORMULA A_{l_1}: $(2A_t)$ POR $\left(\frac{3.5\, b_w S}{f_y}\right) = \frac{3.5(50)(25)}{4200} = 1.041$

SE TENDRA $A_{l_2} = 26.95$ cm^2 POR LO TANTO $A_{l_2} > A_{l_1}$ $\quad \begin{cases} A_l = 25.96 \text{ cm}^2 \\ A_{l_2} = 26.95 \text{ cm}^2 \end{cases}$ ⟸

$\therefore$ SE ESCOGERA EL MAYOR

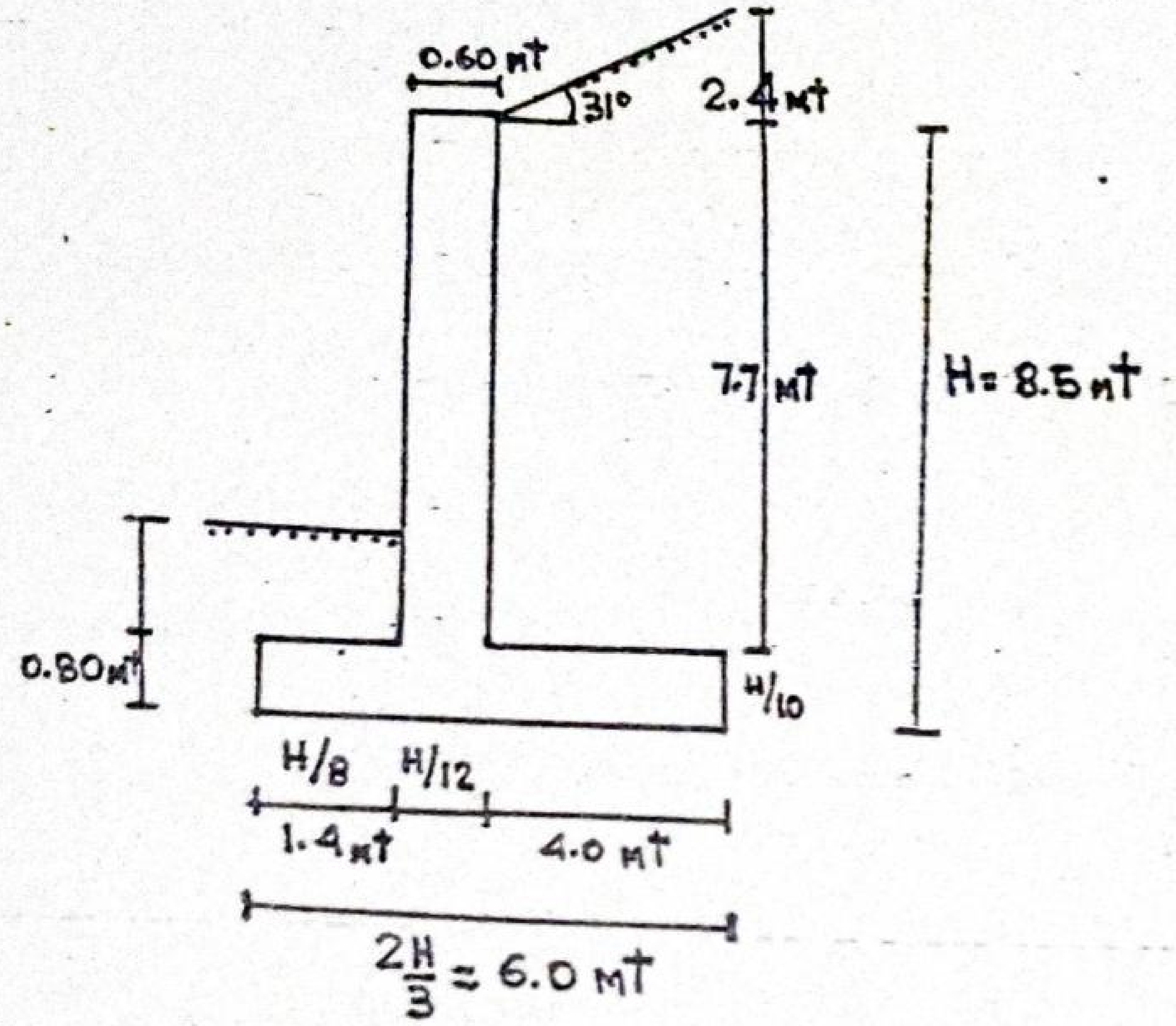
$A_l = 26.95$ cm$^2 \approx 6\phi 1"$ ($A_s = 30.40$ cm^2)

DISTRIBUCION DE VIGA EN TORSION

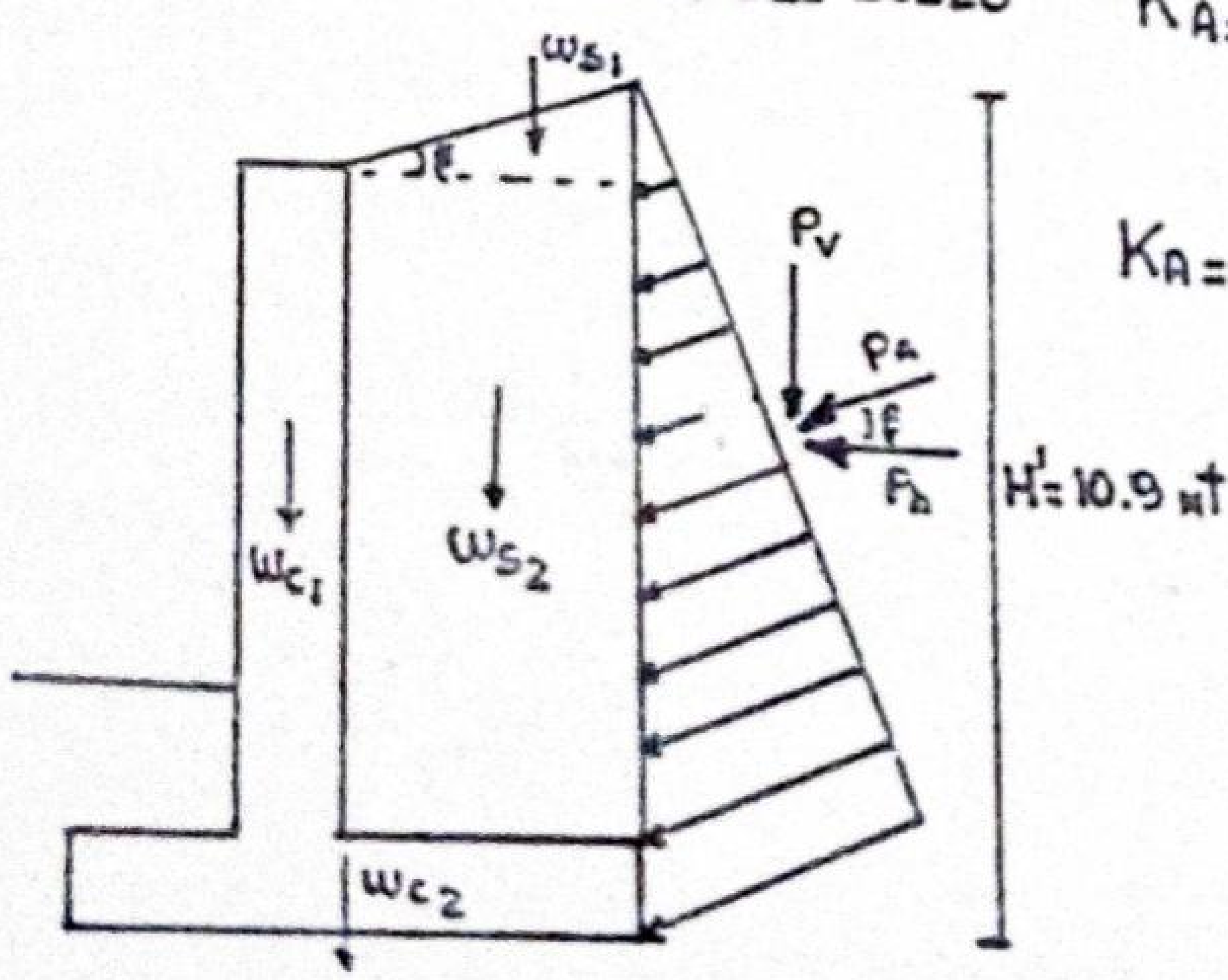


PREDIMENSIONAMIENTO:

DATOS: $\beta = 31°$; $H_1 = 7.0$ mt; $\gamma_s = 1.3$ T/m³; $\phi = 36°$



CALCULO DEL EMPUJE LATERAL DEL SUELO



$$K_A = \cos\beta\left(\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}\right)$$

$$K_A = \cos 31°\left(\frac{\cos 31 - \sqrt{\cos^2 31 - \cos^2 36}}{\cos 31 + \sqrt{\cos^2 31 - \cos^2 36}}\right)$$

$$K_A = 0.4314$$

$$P_A = \frac{1}{2}\gamma (H')^2 K_A = \frac{1}{2} \times 1.3 \times 10.5^2 \times 0.4314 = 33.32 \text{ Tn}$$

$$P_h = P_A \cos\beta = 33.32 \times \cos 31° = 28.56 \text{ Tn}$$

$$P_v = P_A \operatorname{sen}\beta = 33.32 \times \operatorname{sen} 31° = 17.16 \text{ Tn}$$

CALCULO DE LA ESTABILIDAD DEL MURO

POR VOLTEO:

$W_{S_1} = \left(\frac{2.4 \times 4}{2}\right)(1.3) = 6.24\ T_N$

$M_{S1} = 6.24 \times 4.67 = 29.12\ T_{N-M}$

$W_{S_2} = 7.7 \times 4 \times 1.3 = 40.04\ T_N$

$M_{S2} = 40.04 \times 4.0 = 160.16\ T_{N-M}$

$W_{C_1} = 0.8 \times 6 \times 2.4 = 11.52\ T_N$

$M_{C1} = 11.52 \times 3.0 = 34.56\ T_{N-M}$

$W_{C_2} = 0.6 \times 7.7 \times 2.4 = 11.09\ T_N$

$M_{C_2} = 11.09 \times 1.7 = 18.85\ T_{N-M}$

$\Sigma V = 6.24 + 40.04 + 11.52 + 11.09 + \underset{P_V}{\underline{17.16}} = 86.05\ T_N$

$\Sigma M = 29.12 + 160.16 + 34.56 + 18.85 = 242.69\ T\text{-}M$

MOMENTO DE VOLTEO:

$M_V = P_h \times \frac{H'}{3} = 28.56 \times \frac{10.9}{3} = 103.77\ T\text{-}M$

$F.S_V > 1.5$ FACTOR DE SEGURIDAD POR VOLTEO DEBE SER MAYOR DE 1.5

$F.S = \frac{\Sigma M}{M_V} = \frac{242.69}{103.77} = 2.34 > 1.5$ OK!

POR DESLIZAMIENTO:

$f = 0.9\ Tg\phi = 0.9\ Tg\ 36° = 0.654$

$F.R = \Sigma V \cdot f = 86.05 \times 0.654 = 56.28\ T_N$

$F.S_D > 1.8$ FACTOR DE SEGURIDAD POR DESLIZAMIENTO DEBE SER MAYOR DE 1.8

$F.S = \frac{F.R}{P_h} = \frac{56.28}{28.56} = 1.97 > 1.8$ OK!

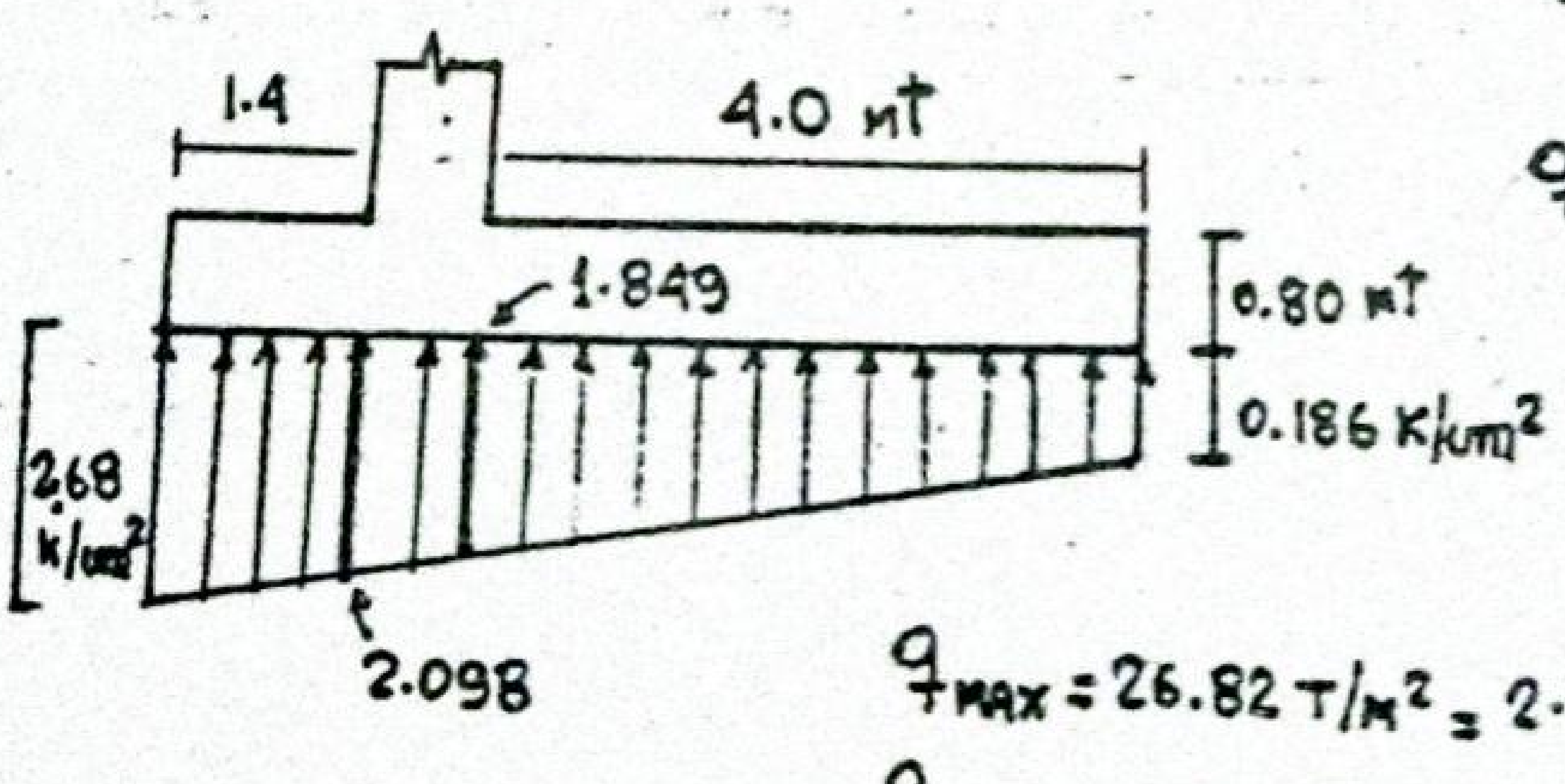
UBICACION DE LA RESULTANTE EN LA BASE

$\bar{X} = \frac{\Sigma M - M_V}{\Sigma V} = \frac{242.69 - 103.77}{86.05} = 2.13$

$e = \frac{B}{2} - \bar{X} = \frac{6}{2} - 2.13 = 0.87\ mt$

$L/6 = 1.00 > 0.87$ OK!

CALCULO DE LA PRESION DEL SUELO:



$q = \frac{\Sigma V}{B} \pm \frac{\Sigma V \cdot 6 \cdot e}{B^2}$

$q = \frac{86.05}{6} \pm \frac{86.05 \times 6 \times 0.87}{6^2}$

$q = 14.34 \pm 12.48$

$q_{MAX} = 26.82\ T/M^2 = 2.68\ K/cm^2 < \sigma_S = 3.8\ K/cm^2$ OK!

$q_{MIN} = 1.86\ T/M^2 = 0.186\ K/cm^2$

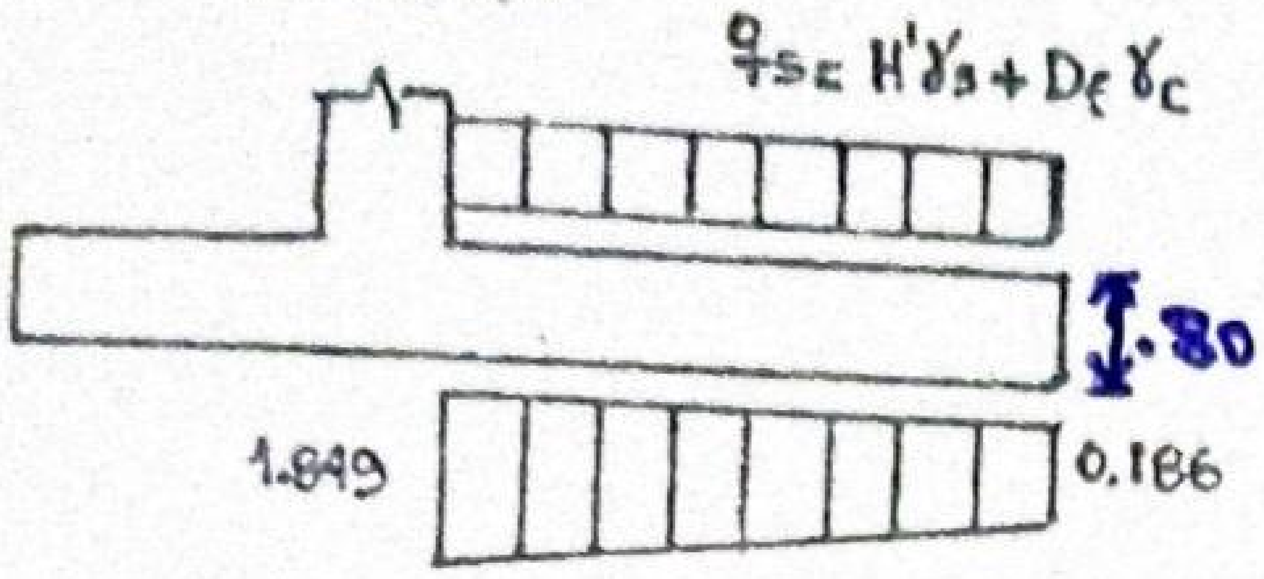
...LCULO DE ESFUERZOS: EN UNA FRANJA DE UN METRO EN LA PUNTA

$$M_{MAX} = 20.98 \times 1.4 \times \frac{1.4}{2} + \frac{5.82 \times 1.4}{2} \times \frac{2}{3}(1.4) = 24.36\ T\text{-}M$$

$$V_{MAX} = 20.98 \times 1.4 + 5.82 \times \frac{1}{2} = 32.282\ TN \qquad V = \frac{32,282}{100(80-8)} = 4.48\ K/cm^2$$

EN EL TALON:

$q_s = H'\gamma_s + D_f\gamma_c$



$$q_s = 8.5 \times 1.3 + 0.8 \times 2.4 = 12.97\ T/m^2 = 1.297\ K/cm^2$$

$$M_{MAX} = (12.97 - 1.86)\left(\frac{1}{2}\right) = 5.555\ Tn/m$$

$$V = \frac{5.555}{100(80-8)} = 0.77\ K/cm^2$$

EN LA PANTALLA:

VALORES h' = 2.5 mt 5.0 mt 7.7 mt

$$P_{Ah} = \left(\frac{1}{2}\right)(1.3)(h')^2(0.4314)\cos 31° = P_{Ah_1} = 1.3\ Tn \quad P_{Ah_2} = 8.65\ Tn \quad P_{Ah_3} = 14.25\ Tn$$

$$M_{MAX} = P_{Ah} \times \frac{h'}{3} = M_{MAX} = 1.25\ T\text{-}M \quad M_{MAX} = 14.42\ T\text{-}M \quad M_{MAX} = 36.58\ T\text{-}M$$

$$V_{MAX} = 36.58\ T/m \qquad V = \frac{36580}{100 \times 60} = 6.09\ K/cm^2$$

CALCULO DEL As:

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} \qquad a = \frac{A_s \cdot f_y}{\beta_3 \cdot f'_c\ b}$$

DONDE: $d = 50\ cm$, $b = 100\ cm$, $\phi = 0.9$, $\beta_3 = 0.85$

EN LA PUNTA:

$$M_u = M \times 1.65 = 24.36 \times 1.65 = 40.194\ T\text{-}M$$

$$M_u = 4019.4\ T\text{-}cm \rightarrow A_s = 22.39\ cm^2$$

$$S_{MAX} = \frac{100 \times 5.07 \rightarrow \phi 1''}{22.39} = 25\ cm \Rightarrow \phi 1'' @ 0.25\ mt$$

EN EL TALON:

$$M_u = M \times 1.65 = 27.00 \times 1.65 = 44.546\ T\text{-}M$$

$$M_u = 4454.6\ T\text{-}cm \rightarrow A_s = 25.07\ cm^2$$

$$S_{MAX} = \frac{100 \times 5.07}{25.07} = 20\ cm \Rightarrow \phi 1'' @ 0.20\ mt$$

EN LA PANTALLA:

$h = 2.5\ mt$

$$M_u = 1.25 \times 1.65 = 2.063\ T\text{-}M$$

$$M_u = 206.3\ T\text{-}cm \rightarrow A_s = 1.09\ cm^2$$

$$S = \frac{100 \times 1.09}{1.27} = 1.00\ mt \Rightarrow \phi 1/2'' @ 1.00\ mt$$

$h = 5.0$ mt

$Mu = 14.42 \times 1.65 = 23.793$ T-M

$Mu = 2379.3$ T-cm $\rightarrow As = 10.76\ cm^2$

$\rightarrow \phi\ 1/2"$ @ 0.30 mt, $\phi\ 5/8"$ @ 0.30 mt

$h = 7.7$ mt

$Mu = 36.58 \times 1.65 = 60.357$ T-M

$Mu = 6035.7$ T-cm $\rightarrow As = 34.71\ cm^2$

$\rightarrow \phi\ 1/2"$ @ 0.30 mt, $\phi\ 5/8"$ @ 0.30 mt, $\phi\ 3/4"$ @ 0.15 mt

CORTE ADMISIBLE (MIN)

$V_c = \phi \times 0.53\sqrt{f'c}$

$V_c = 0.85 \times 0.53\sqrt{210}$

$V_c = 6.53\ K/cm^2$

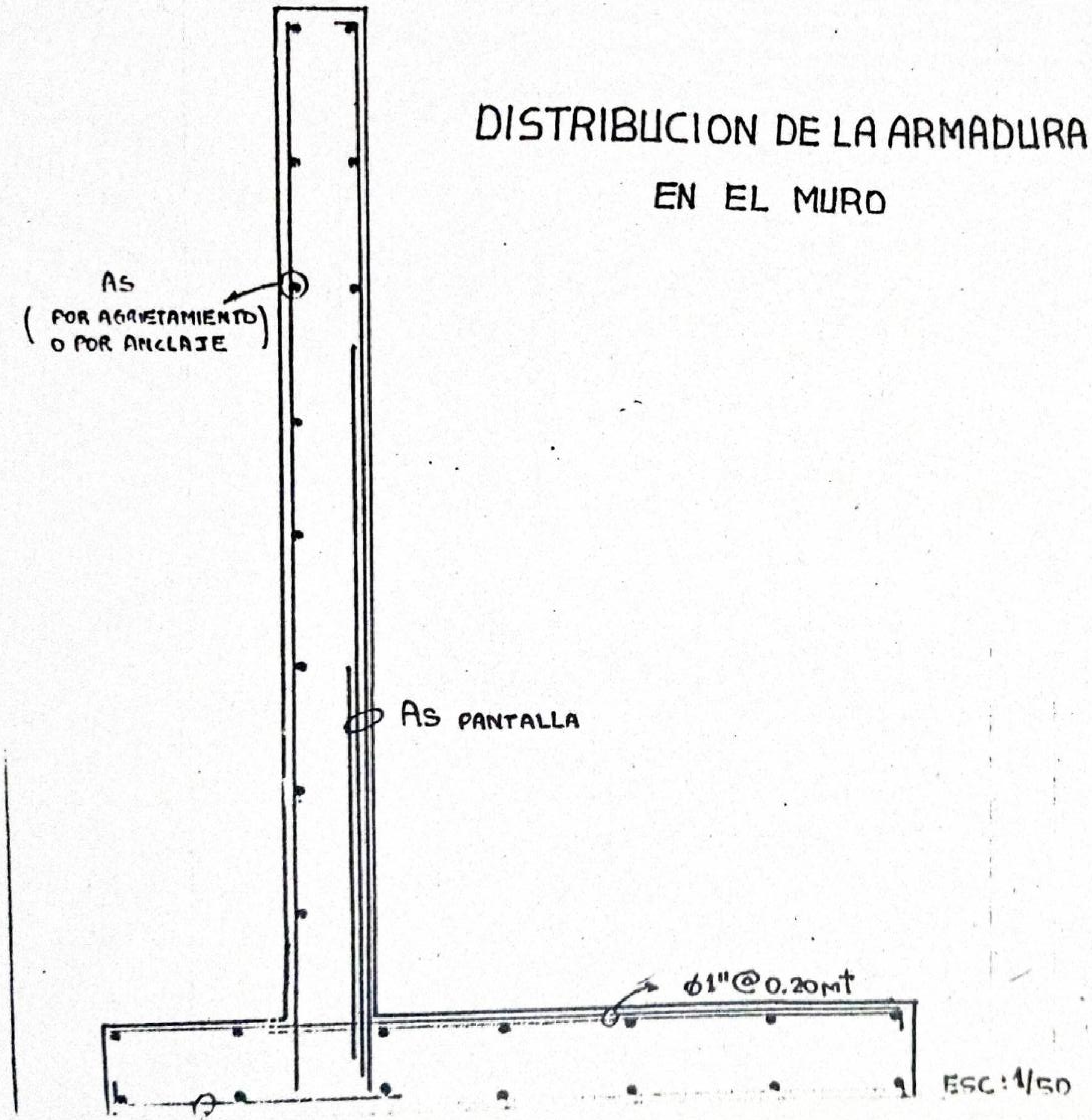
COMPROBANDO

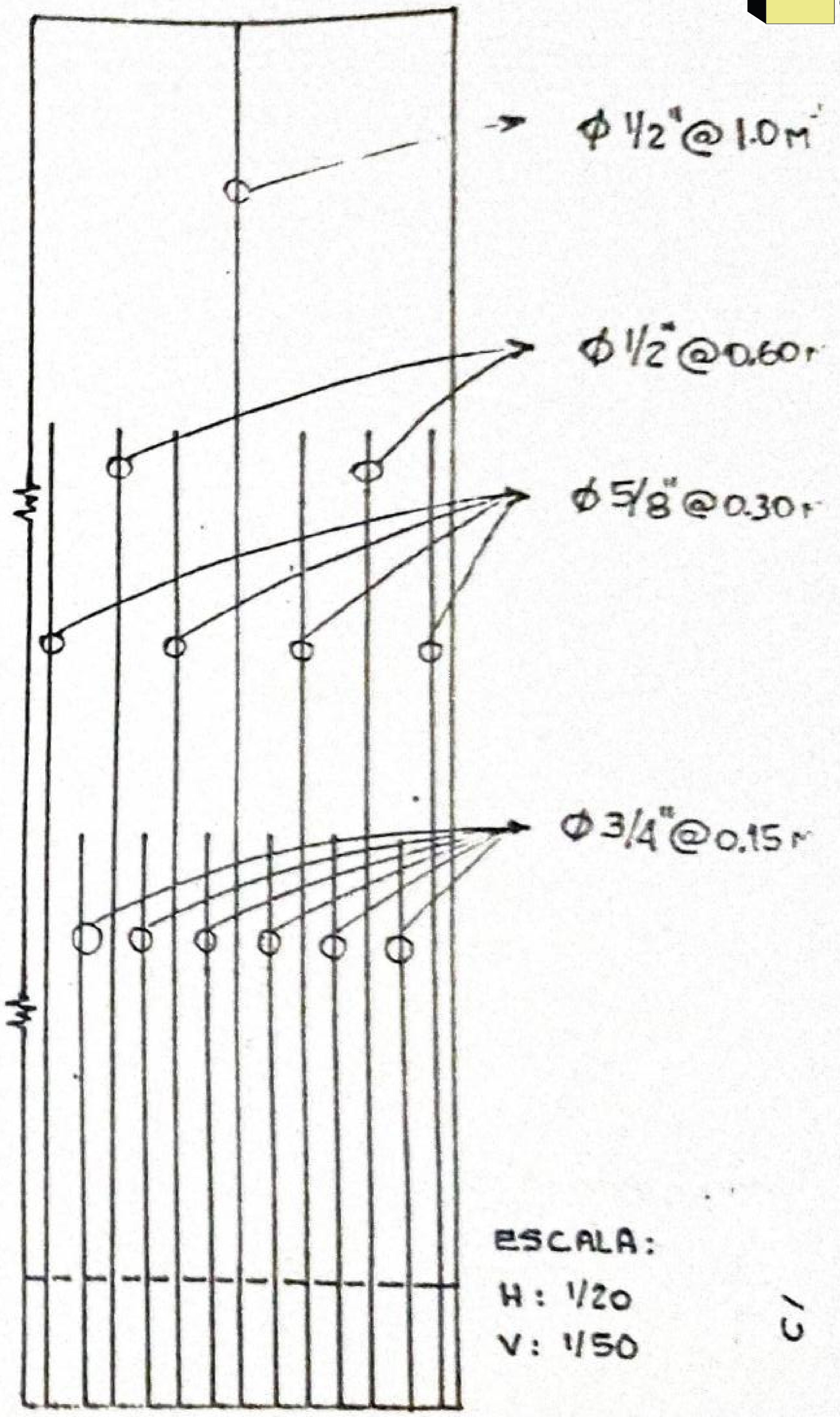
PUNTA $\rightarrow V = 4.48\ K/cm^2 \angle 6.53\ K/cm^2$ OK!

TALON $\rightarrow V = 0.77\ K/cm^2 \angle 6.53\ K/cm^2$ OK!

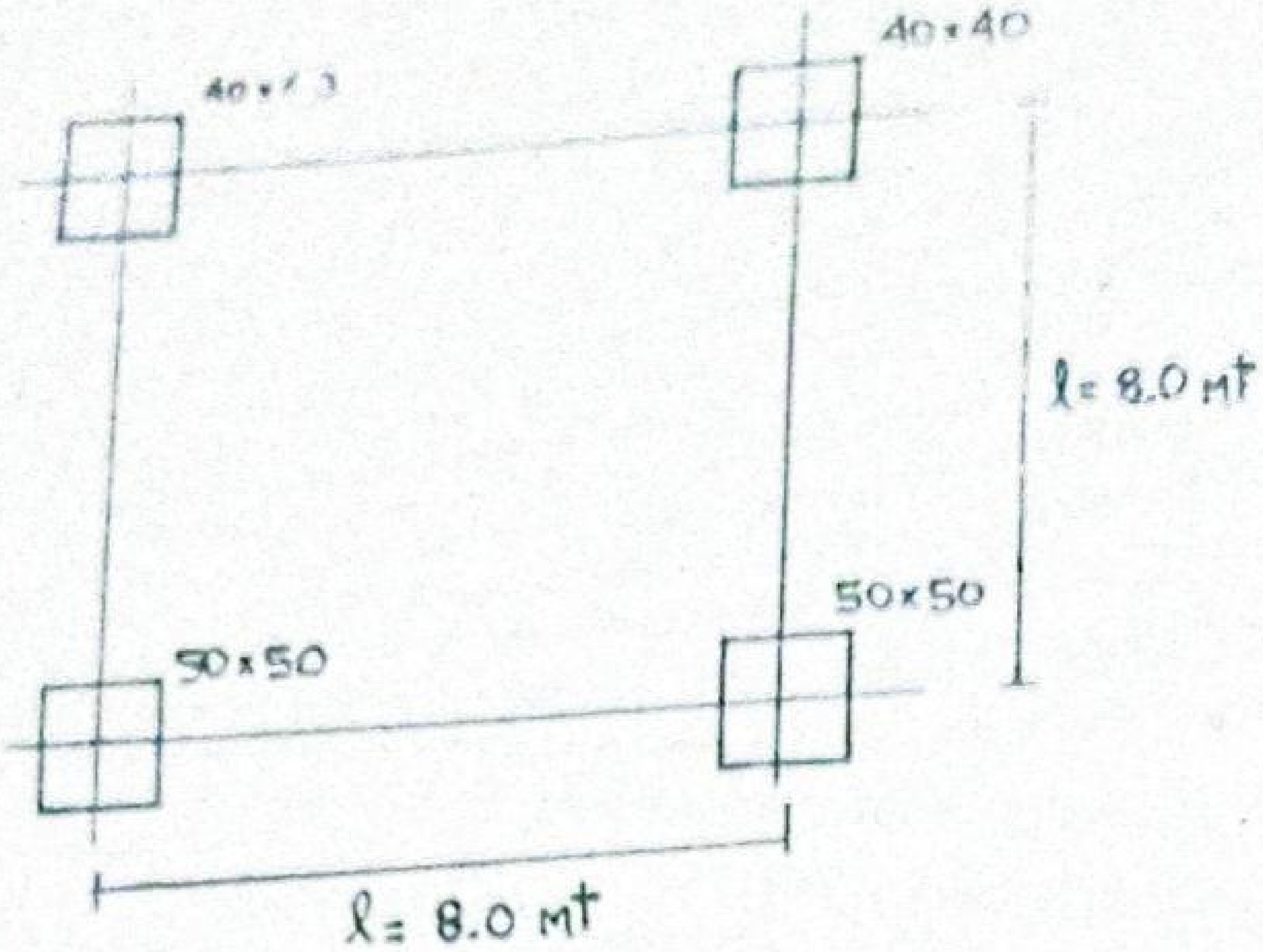
PANTALLA $\rightarrow V = 6.09\ K/cm^2 \angle 6.53\ K/cm^2$ OK!

DISTRIBUCION DE LA ARMADURA EN EL MURO





PROB(16) DISEÑO DE LOSA ARMADA EN DOS SENTIDOS



$f_y = 4200$ K/cm2

$f'_c = 210$ K/cm2

CALCULO DEL ESPESOR DE LA LOSA (h)

l_n = LONGITUD DEL CLARO LIBRE ENTRE LAS COLUMNAS EN LAS DOS DIRECCIONES

ESPESOR MINIMO e = 12.5 (NORMA ACI 1977)

$l_n = (8.0 - 0.2 - 0.25)$

$l_n = 7.55$ mt

$\beta = \frac{l_1}{l_2} \leq 2 \qquad \beta = \frac{8}{8} = 1 \qquad 1 \leq 2$ OK!

$$h \leq \frac{l_n(800 + 0.07 f_y)}{36{,}000} = \frac{755(800 + 0.07 \times 4200)}{36{,}000}$$

$h = 22.94$ cm $\rightarrow$ $h = 23$ cm

PARA LOSAS SIN VIGAS DE BORDES DEBERAN SER AUMENTADAS UN MINIMO DE 10 % DE SU ESPESOR.

$h = 23 \times 1.10 = 25.3$ cm $\rightarrow$ $h = 25$ cm

VERIFICO ESFUERZO DE CORTE:

CONSIDERANDO UN ϕ 5/8"

$d = h -$ RECUBRIMIENTO $- \phi/2 = 25 - 2 - 1.6 = 21.4$ cm

CARGA MUERTA = P.P + TABIQUERIA + ACABADOS

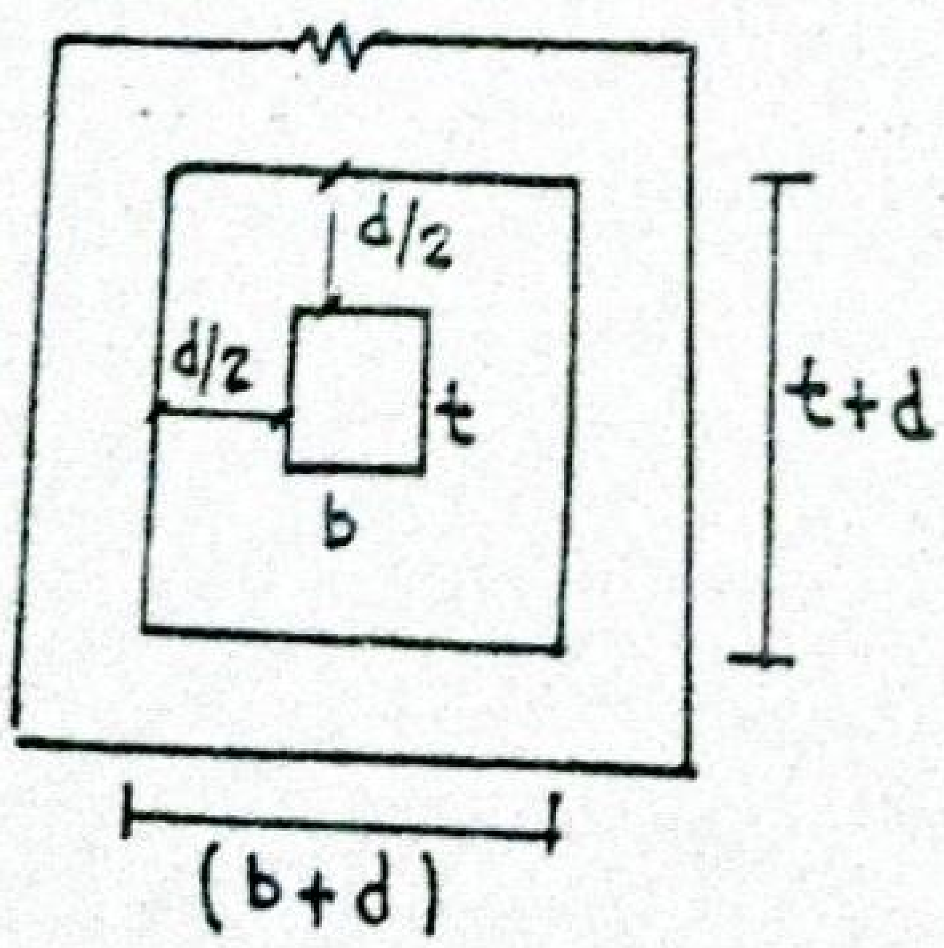
$W_D = 0.214 \times 2400 + 100 + 100 = 713.6$ K/m²

CARGA VIVA = PARQUEO DE AUTOS

$W_L = 250$ K/m²

$W_u = 1.5(713.6) + 1.8(250)$

$W_u = 1520.4$ K/m²



$b_o = 2(b + t + 2d) = 2(80 + 2(21.4))$

$b_o = 245.6$ cm

LADO = $(t + d) = (40 + 21.6) = 61.6$ cm

TE POR FLEXION:

$$V_U = W_U\left[l_1 . l_2 - (\text{LADO})^2\right]$$

$$V_U = 1520.4\left[8\times 8 - (0.616)^2\right] = 96{,}728.7$$

$$v_U = \frac{V_U}{b_o \times d} = \frac{96{,}728.7}{245.6\times 21.4} = 18.4\ \text{K/cm}^2$$

CORTE PERMISIBLE: $\beta = \frac{8}{8} = 1.0$

$$v_c = \phi \times 0.27\left(2 + \frac{4}{\beta}\right)\sqrt{f'c}$$

$$v_c = 0.85\times 0.27\left(2 + \frac{4}{1}\right)\sqrt{210}$$

$$v_c = 19.95\ \text{K/cm}^2$$

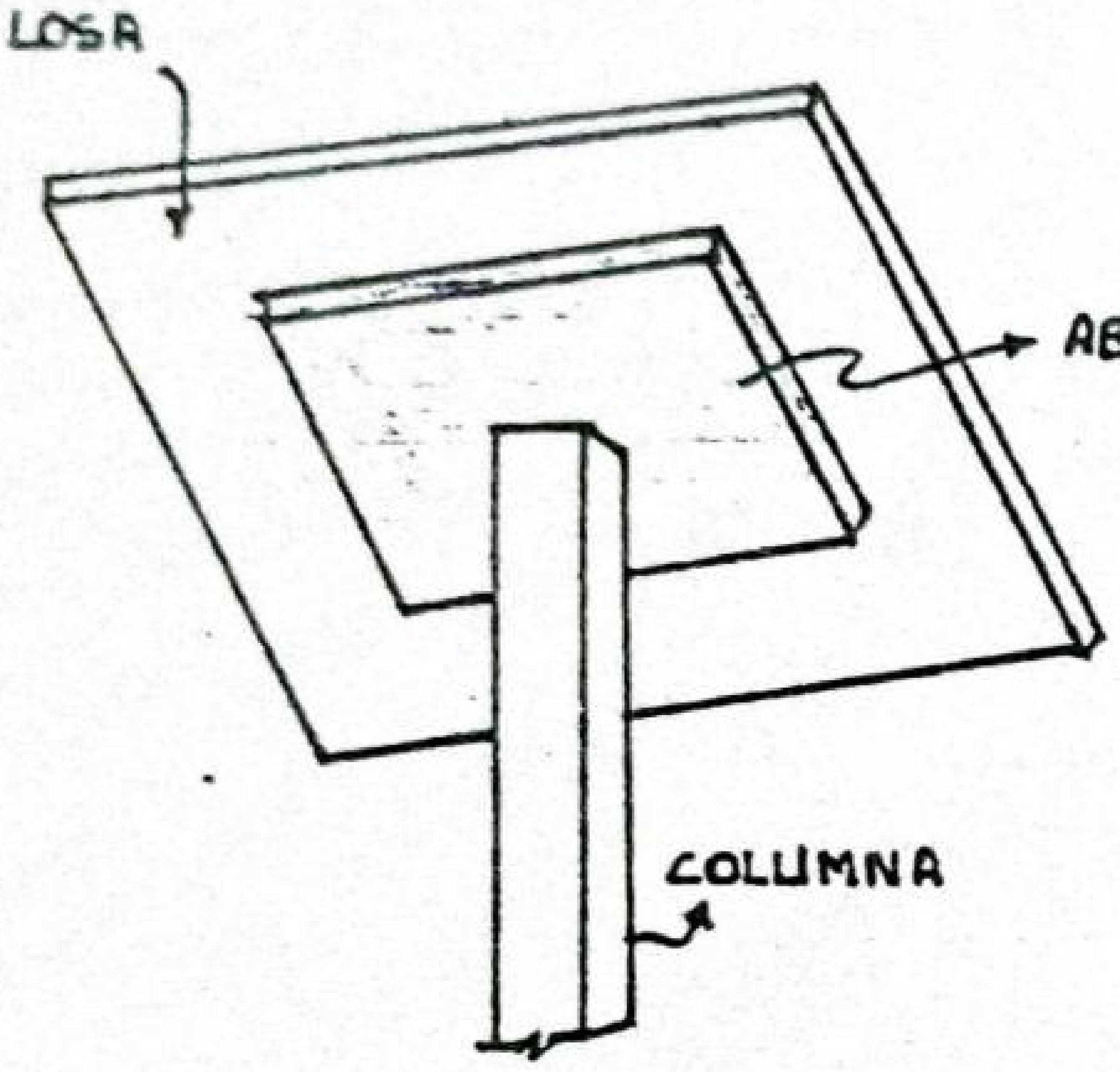
PERO NO MAYOR: $v_c = \phi \times 1.1\sqrt{f'c}$

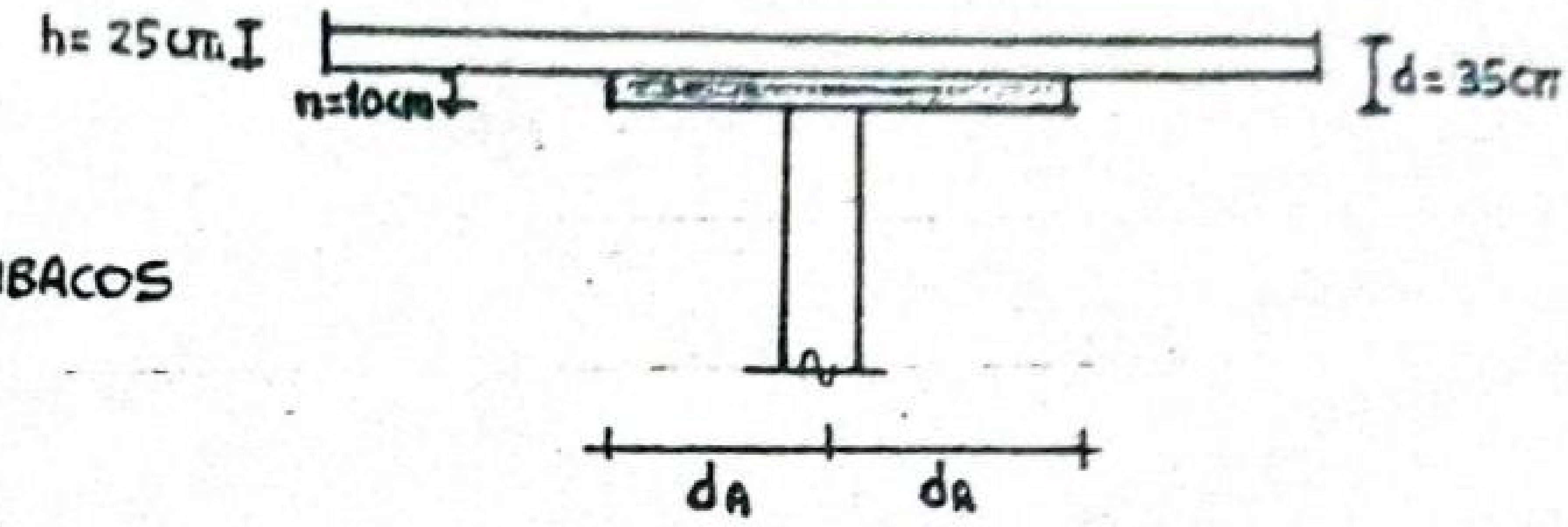
$$v_c = 0.85\times 1.1\sqrt{210}$$

$$v_c = 13.55\ \text{K/cm}^2$$

$\therefore\ v_U = 18.4 > v_c = 13.55$ NO CUMPLE !

SE DEBE USAR ABACOS





$$b_o = 2(b+t+2d) = 2\left[50+50+2(35)\right] = 340\ \text{cm}$$

$$\text{LADO} = (t+d) = 50+35 = 85\ \text{cm}$$

$$V_U = 1520.4\left[8\times 8 - (0.85)^2\right] = 96{,}207\ \text{Kg}$$

$$v_U = \frac{96207}{340\times 35} = 8.08 < V_c = 13.55\quad \text{OK!}$$

PARA LAS LOSAS CON ABACOS QUE SE EXTIENDE DESDE EL EJE DE APOYO DEBE TENER UNA DISTANCIA NO MENOR

$$d_A \geq \frac{L}{6} \qquad d_A \geq \frac{8.0}{6} = 1.33\ \text{mt}$$

$$d_A \approx 1.50\ \text{mt}$$

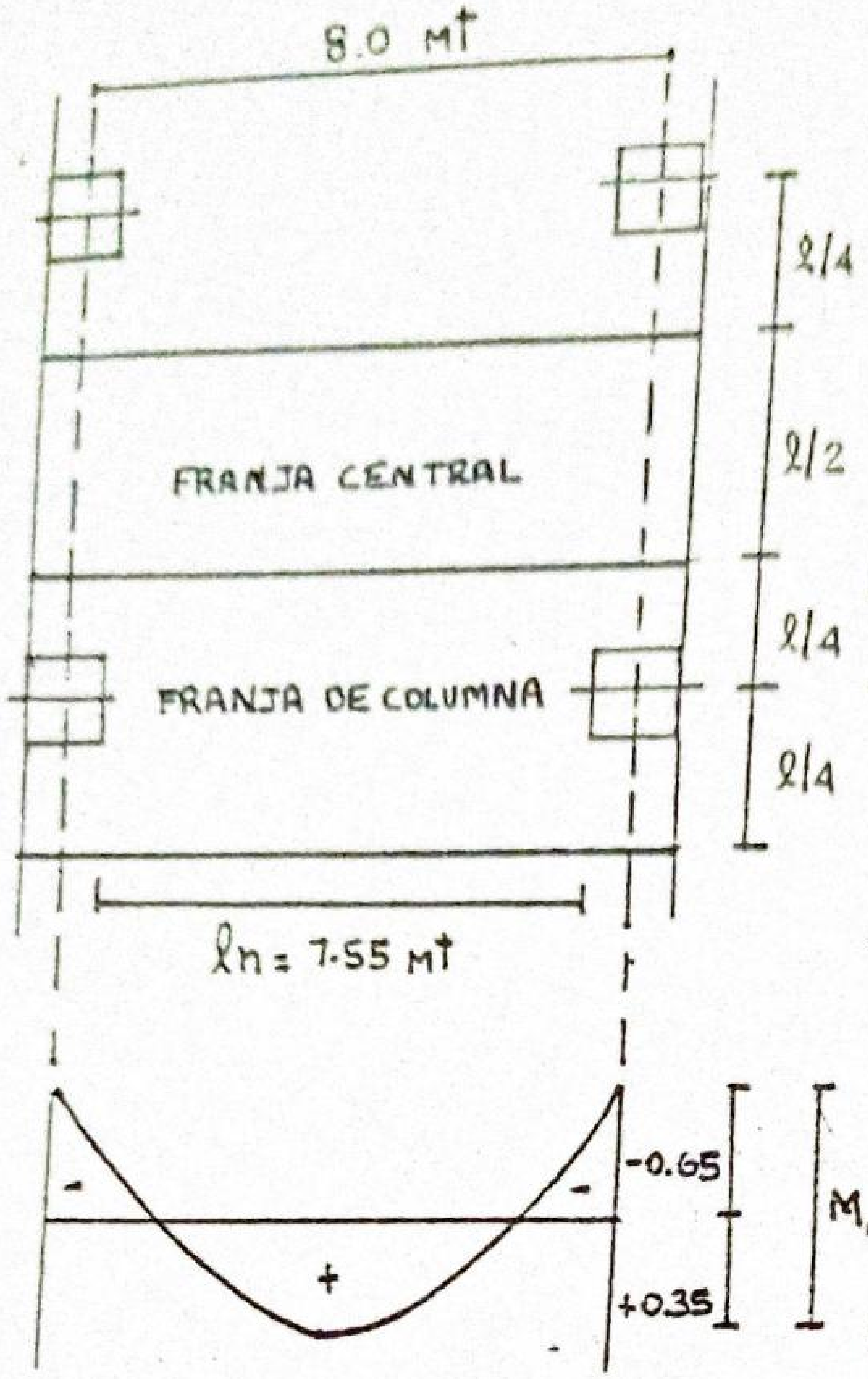
CALCULO DE MOMENTO:

TRAMO LARGO = TRAMO CORTO

$$M_o l = \frac{W_U . l_2 . l_n^2}{8}$$

$$M_o l = \frac{1520.4(8)(7.55)^2}{8}$$

$$M_o l = 86{,}666.6\ \text{K-Mt}$$



MOMENTOS PARA MARCOS INTERNOS		
M. TOTAL	1.00 M_o	
MOMENTO (+),(-)	$-0.65\,M_o$	$+0.35\,M_o$
FRANJA DE COLUMNA	75 %	60 %
FRANJA CENTRAL	25%	40%

FRANJA DE COLUMNAS:

$-M_u = -0.75\,(0.65\,M_o)$

$-M_u = 0.75\,[0.65\,(86{,}666.6)]$

$-M_u = 42{,}250$ K-M

$+M_u = 0.6\,[0.35\,M_o]$

$+M_u = 0.6\,[0.35\,(86{,}666.6)]$

$+M_u = 18{,}200$ K-mt

FRANJA CENTRAL

$M_u = 0.25\,[0.65\,M_o]$

$M_u = 0.25\,[0.65\,(86{,}666.6)]$

$-M_u = 14{,}083$ K-M

$+M_u = 0.40\,(0.35\,M_o)$

$+M_u = 12{,}133$ K-M

CALCULO (d_{MIN}):

PARA $M_{MAX} = 42{,}250$ K-M $\qquad b = \frac{8}{2} = 4$ mt

$$\rho = 0.18\,\frac{f_c}{f_y} = 0.18\,\frac{(210)}{4200} = 0.009$$

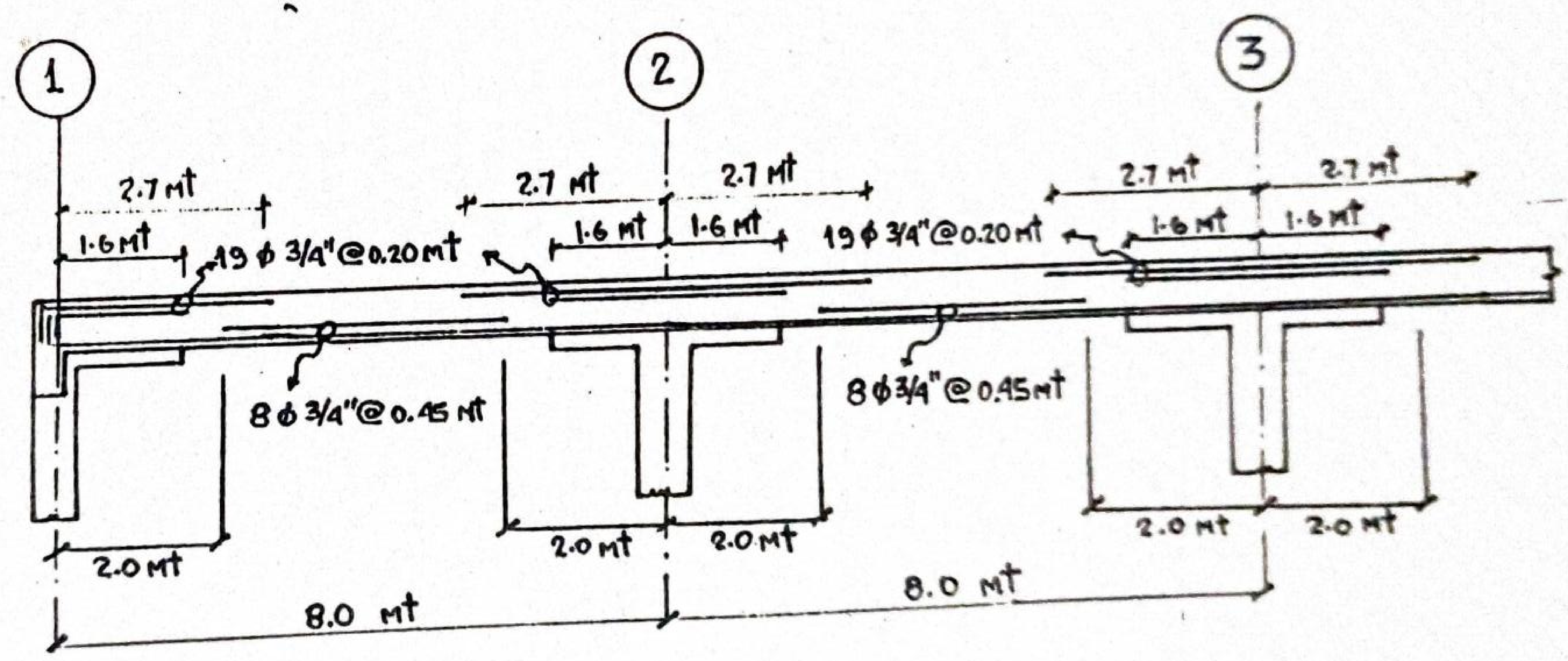
$$d = \sqrt{\frac{M_u}{0.145\,b f_c}} = \sqrt{\frac{42{,}500 \times 100}{0.145 \times 400 \times 210}} = 18.62\text{ cm}$$

$$d = h - \text{RECUBRIMIENTO} - \frac{\overset{1/2''}{\phi}}{2} = 25 - 2 - 0.65$$

$d = 22.35$ cm

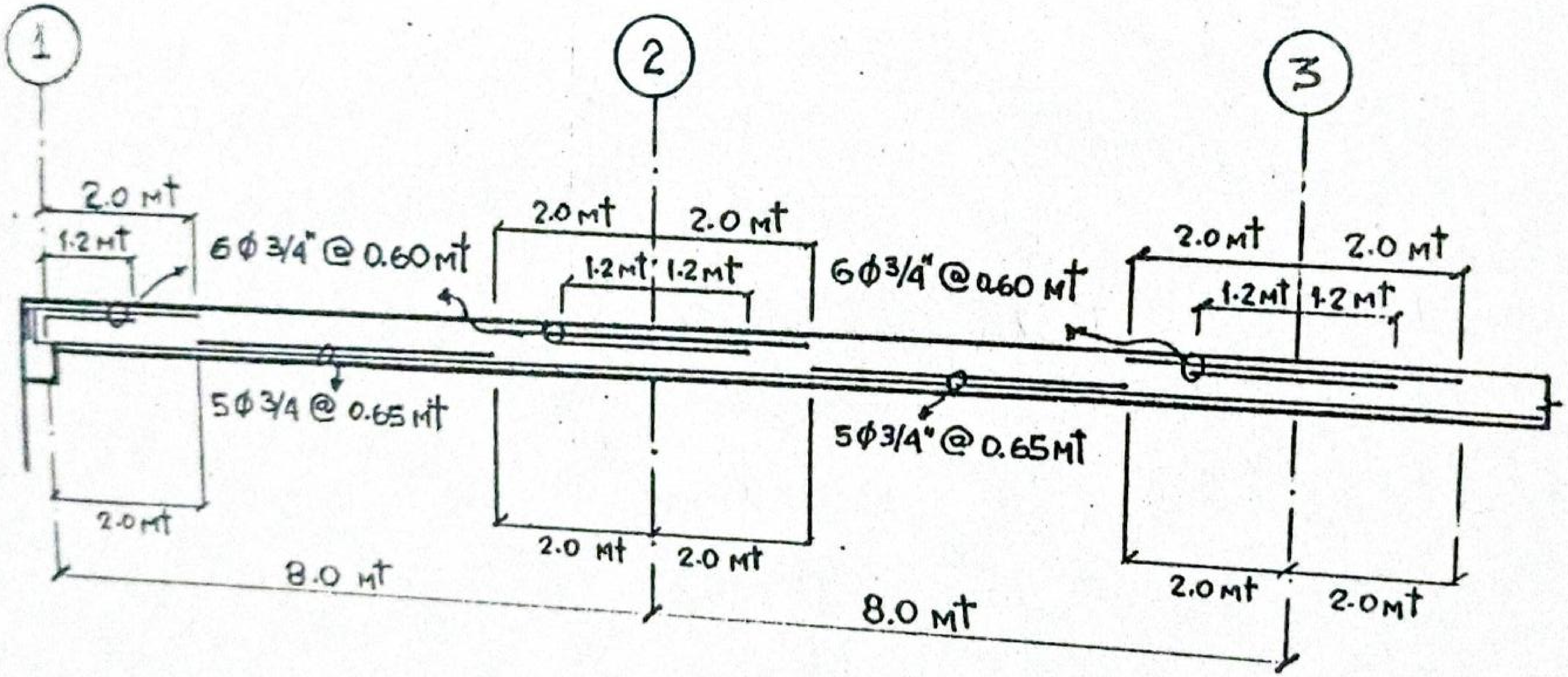
CALCULO DEL "As":

FRANJA COLUMNA	$-M_u = 42{,}250$ K-M	$A_s = 53.6\text{ cm}^2$	$a = 3.0$ cm	19 ϕ 3/4" @ 0.20 mt
	$+M_u = 18{,}200$ K-M	$A_s = 22.8\text{ cm}^2$	$a = 1.26$ cm	8 ϕ 3/4" @ 0.45 mt
FRANJA CENTRAL	$-M = 14{,}083$ K-M	$A_s = 17.0\text{ cm}^2$	$a = 1.00$ cm	6 ϕ 3/4" @ 0.60 mt
	$+M = 12{,}133$ K-M	$A_s = 14.66\text{ cm}^2$	$a = 1.53$ cm	5 ϕ 3/4 @ 0.65 mt



CORTE EN FRANJA COLUMNA.

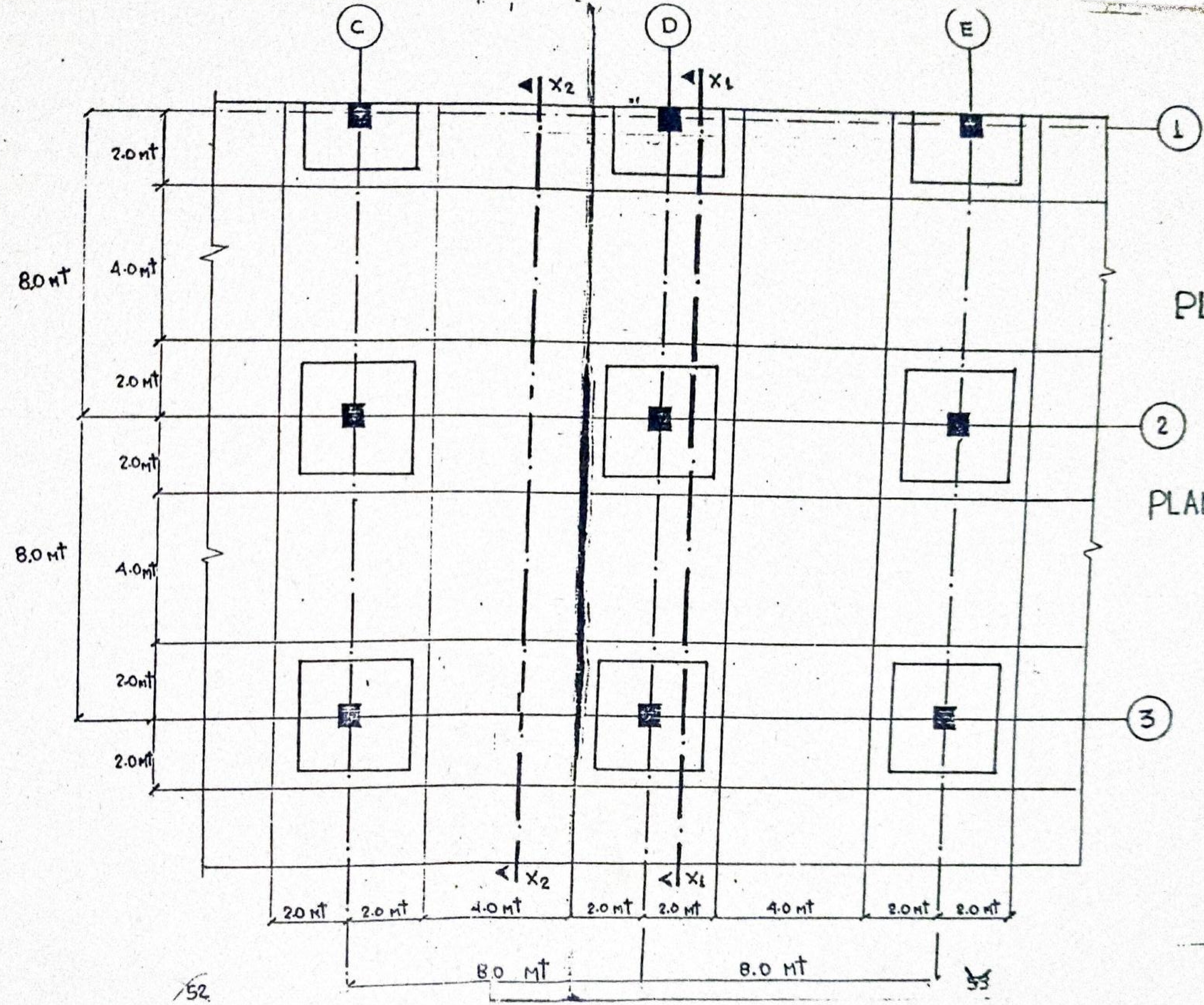
X' - X'



CORTE EN FRANJA CENTRAL $X_2 - X_2$

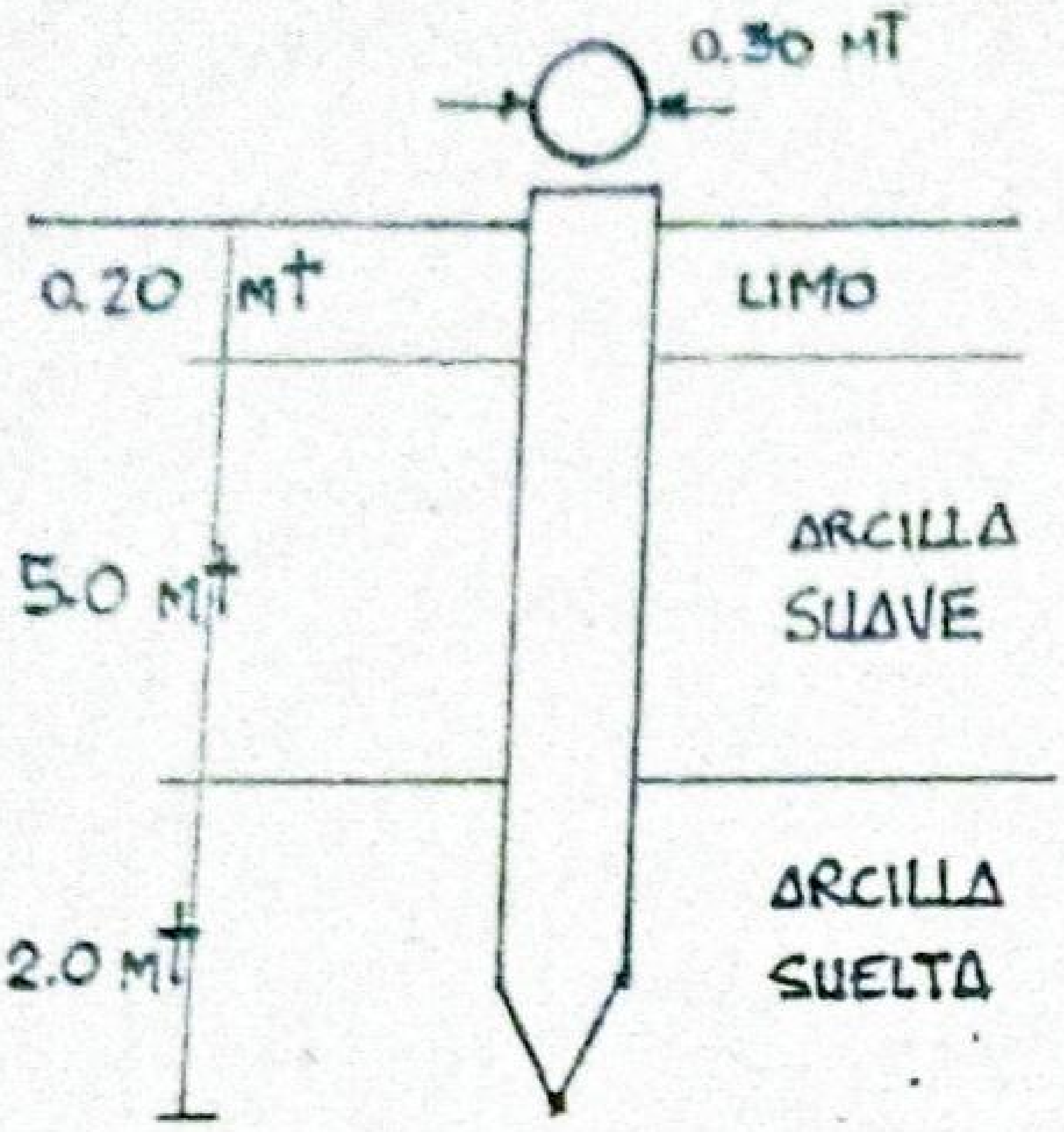
PLANO

PLANTA



/52

PROB (11) CALCULAR LA CARGA DE TRABAJO DEL PILOTE. SI EL FACTOR DE SEGURIDAD F.S = 2.



ARCILLA SUAVE: $c = 1.5\ T/M^2$, $\gamma' = 0.9\ T/M^3$, $F = 1.5\ T/M^2$

ARCILLA SUELTA: $c = 0$, $\phi = 30°$, $\gamma' = 1\ TN/M^3$, $F = 2\ TN/M^2$

SOLUCION:

POR TABLAS: PARA $\phi = 30°$ $\begin{cases} N_c = 37 \\ N_q = 23 \\ N_\gamma = 20 \end{cases}$

$$Q_F = \left[2\pi(0.15)(2) + 2\pi(0.15)(5)\right](1.5) = 10.838\ TN$$

$$Q_P = \pi(0.15)^2\left[(1.3)(0)(37) + \left[(1)(2) + (0.9)(5)\right]\right][23] + 0.6(1)(0.15)(20)]$$

$$Q_P = 10.695\ TN$$

$$Q_U = Q_P + Q_F = 10.695 + 10.838$$

$$\boxed{Q_U = 21.533\ TN}$$ RPTA