

**DISEÑO Y CALCULO DE UN PUENTE DE PLACA Y VIGAS
(EN CONCRETO REFORZADO)**

**FACULTAD DE INGENIERIA
PROGRAMA DE ING. CIVIL**

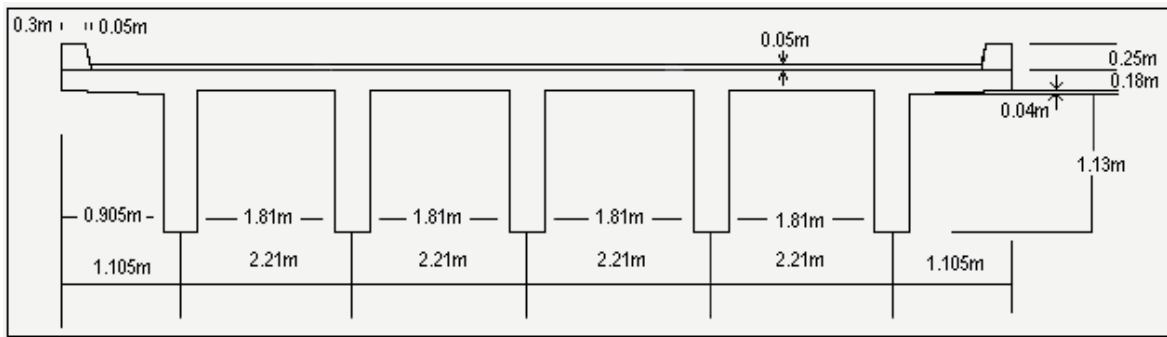
UNIVERSIDAD TECNOLOGICA DEL CHOCO

"Diego Luis Córdoba"

Diseñar un puente de placa y viga que tiene una luz de calculo $L = 19\text{m}$ para tres vías de transito y un camión de diseño C – 40 con los siguientes $F_y = 60000\text{PSI}$ (4200Kg/cm^2) y un $F'_c = 3500\text{PSI}$ (280Kg/cm^2). La altura del estribo es de 7.8m y el nivel freático esta a 3.0 m por encima de la cota de fundación, la capacidad portante del suelo es de 28 T/m^2 , el factor de fricción $\mu = 0.5$ (deslizamiento) y el ángulo de fricción del suelo es de $\phi = 30^\circ$

DIMENSIONAMIENTO

- Numero de vigas = 3 vías de transito + 1 = 4; según la norma la separación máxima entre vigas de centro a centro debe ser máximo de 2.44m por esta razón se consideraron 5 vigas.
- Ancho de placa = $2(0.35) + 3.05 + 2(3.65) = 11.05\text{m}$
- Separación de vigas centro a centro = $\frac{\text{ancho de placa}}{\# \text{ Vigas}} = \frac{11.05}{5} = 2.21\text{m}$
- Ancho de viga = 0.4m
- Separación de vigas cara a cara = $2.21 - 0.4 = 1.81\text{m}$
- Voladizo = $\frac{\text{separación de viga cara a cara}}{2} = \frac{1.81\text{m}}{2} = 0.905\text{m}$
- Altura de viga = $h = 1.1 (0.15 + L/18) = 1.1 (0.15 + 19/18) = 1.35\text{m}$
- Longitud de apoyo de viga = $3/8h = 3/8(1.35) = 0.51\text{m}$
- Altura de placa = $0.1 + S/30 = 0.1 + 2.36/30 = 0.18\text{m}$
- Peralte = $0.1 + S/10 = 0.1 + 1.185/10 = 0.22\text{m}$



CHEQUEO COMO VIGA T

$$b' \leq 16t + b' = 0.4m < 16(0.18m) + 0.4m = 0.4m < 3.28m \text{ ¡ok!}$$

$$b' < L/4 = 0.4m < 19m/4 = 0.4m < 4.75m \text{ ¡ok!}$$

ANALISIS DE PLACA

CARGAS

a) Carga Muerta

$$\text{Peso propio de placa: } 0.18m \times 1m \times 2.4T/m^3 = 0.43T/m^2$$

$$\text{Peso propio capa de rodadura: } 0.05m \times 1m \times 2.2T/m^3 = 0.11T/m^2$$

$$W_m = 0.54T/m^2$$

b) Carga Viva

Rueda trasera: 7.5T

Impacto: $I = 16/(S + 40)$ $I = 16/(2.36m + 40)$ $I = 38\% > 30\%$ ¡no ok!

$I = 30\%$

Carga viva + impacto = $7.5T \times 1.3$ $C_v + I = 9.75T$

Carga viva reducida para tres vías de tránsito $90\% = 9.75 \times 0.90 = 8.78T$

MOMENTOS

MOMENTO EN EL APOYO Y LA LUCES INTERIORES

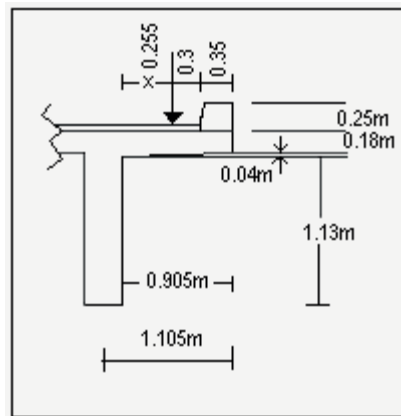
a) Carga Muerta

$$M = \frac{W_m \times S^2}{10} \qquad M = \frac{0.542T/m^2 \times (1.81m)^2}{10} \qquad M = 0.18T\text{-m}$$

b) Carga Viva

$$M = \frac{(S + 0.6)}{9.7} \times P \times 0.8 \qquad M = \frac{(1.81m + 0.6)}{9.7} \times 0.8 \times 8.78 \qquad M = 1.75T\text{-m}$$

MOMENTO EN EL VOLADIZO



a) Momento por Carga Muerta

Por peso C. rodadura: $2.2T/m^3 \times 0.05m \times 1m \times 0.555m \times (0.555/2) = 0.017T\text{-m}$

Por peso bordillo: $2.4T/m^3 \times 0.25m \times 0.35m \times 1m \times (0.905m - 0.35m/2) = 0.153T\text{-m}$

Por peso placa: $2.4T/m^3 \times 0.18m \times 0.905m \times 1m \times (0.905m/2) = 0.177T\text{-m}$

Por peralte: $2.4T/m^3 \times 0.04m \times 0.905m/2 \times 1m \times 1/3(0.905m) = 0.013T\text{-m}$



$M_t = 0.36T\text{-m}$

b) Momento por carga viva

$E = 0.8X + 1.14$ $E = 0.8(0.255m) + 1.14$ $E = 1.344m$

$M = \frac{C_v + I * (X)}{E}$ $M = \frac{8.78T \times 0.255m}{1.344}$ $M = 1.66T\text{-m}$

MOMENTOS ÚLTIMOS

MOMENTO EN EL APOYO Y LA LUCES INTERIORES

$$M_u = \gamma(M_{cm} + 1.67M_{cv} + I) \quad M_u = 1.3 (0.18T\text{-m} + 1.67(1.75T\text{-m}))$$

$$M_u = 4.03T\text{-m}$$

MOMENTO EN EL VOLADIZO

$$M_u = \gamma(M_{cm} + 1.67M_{cv} + I) \quad M_u = 1.3 (0.36T\text{-m} + 1.67(1.66T\text{-m}))$$

$$M_u = 4.09T\text{-m}$$

ARMADURA EN EL APOYO Y EN LUCES INTERIORES

$$M_u = 4.03T\text{-m} \quad F'_c = 280\text{Kg/cm}^2 \quad F_y = 4200\text{Kg/cm}^2 \quad h = 18\text{cm} \quad d = 15\text{cm}$$

$$b = 100\text{cm} \quad d' = 3\text{cm} \quad K = M_u/bxd^2 \quad K = 4.03T\text{-m}/(1\text{m} \times (15\text{cm})^2)$$

$$K = 0.0179T/\text{cm}^2 \quad \rho = 0.0050 \quad A_s = \frac{M_u}{F_y \times d} \quad A_s = 100 \times 15 \times 0.0050$$

$$A_s = 7.5\text{cm}^2 \quad S = 100 \times \frac{(1.27\text{cm})^2}{7.5\text{cm}^2}$$

$$S = 16\text{cm}, \text{ Colocar una } \emptyset \frac{1}{2}'' \text{ c/u } 16 \text{ cm}.$$

ARMADURA EN EL VOLADIZO

$$M_u = 4.09T\text{-m} \quad F'_c = 280\text{Kg/cm}^2 \quad F_y = 4200\text{Kg/cm}^2 \quad h = 22\text{cm} \quad d = 19\text{cm}$$

$$b = 100\text{cm} \quad d' = 3\text{cm} \quad K = M_u/bxd^2 \quad K = 4.09T\text{-m}/(1\text{m} \times (19\text{cm})^2)$$

$$K = 0.011T/\text{cm}^2 \quad \rho = 0.0031 \quad A_s = 100 \times 19 \times 0.0031 \quad A_s = 5.87\text{cm}^2$$

$$S = 100 \times (1.27\text{cm}^2/5.87\text{cm}^2)$$

S = 21.64cm, Colocar una $\emptyset \frac{1}{2}$ " c/u 21 cm.

ARMADURA DE DISTRIBUCIÓN

$$\% = 121/\sqrt{S} \quad \% = 121/\sqrt{1.81} \quad \% = 89.94\% > 67\% \quad A_s = 0.67 \times 7.5\text{cm}^2$$

$$A_s = 5.03\text{cm}^2 \quad S = 100 \times (1.27\text{cm}^2/5.03\text{cm}^2)$$

S = 25cm, Colocar una $\emptyset \frac{1}{2}$ " c/u 25 cm.

ARMADURA DE TEMPERATURA

$$\rho = 0.002 \quad A_s = 0.002 \times 100\text{cm} \times 18\text{cm} = 3.6\text{cm}^2 \quad S = 100 \times (0.71\text{cm}^2/3.6\text{cm}^2)$$

S = 20cm, Colocar una $\emptyset \frac{3}{8}$ " c/u 20 cm.

DISEÑO DE VIGAS

VIGAS LONGITUDINALES INTERIORES

CARGAS

a) Carga Muerta

$$\text{Peso de placa: } 2.4\text{T/m}^3 \times 0.18\text{m} \times 2.21\text{m} = 0.96\text{T/m}$$

$$\text{Peso C. rodadura: } 2.2\text{T/m}^3 \times 0.05\text{m} \times 2.21\text{m} = 0.24\text{T/m}$$

$$\text{Peso nervio: } 2.4\text{T/m}^3 \times 0.4\text{m} \times 1.17\text{m} = 1.12\text{T/m}$$

$$\mathbf{WT = 2.32T/m}$$

$$\text{Peso viga diafragma apoyo} = 2.4\text{T/m}^3 \times 0.25\text{m} \times 1.17\text{m} \times 1.81\text{m} = 1.27\text{T}$$

$$\text{Peso viga diafragma central} = 2.4\text{T/m}^3 \times 0.25\text{m} \times 0.87\text{m} \times 1.81\text{m} = 0.94\text{T}$$

b) Carga Viva

$$\text{Rueda delantera: } 5\text{T}$$

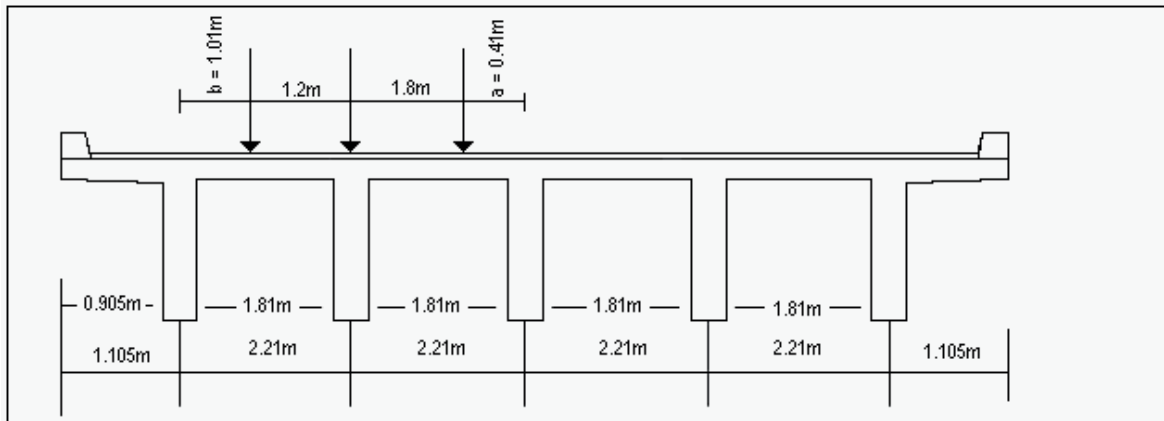
$$\text{Impacto: } I = 16/(S + 40) < 30\% \quad I = 16/(19 + 40) \quad I = 0.27 \quad I = 27\%$$

$$C_v + I = 1.27 \times 5\text{T} \quad C_v + I = 6.35\text{T} = P$$

$$\text{Carga viva reducida para tres vías de tránsito } 90\% = 6.35 \times 0.90 = 5.72\text{T}$$

FACTOR DE RUEDA

a) Cortante



$$Fr = P (1 + (a + b)/S)$$

$$Fr = P (1 + (0.41+1.01)/2.21))$$

$$Fr = 1.64P$$

b) Momento

$$Fr = S/1.80$$

$$Fr = 2.21/1.80$$

$$Fr = 1.23P$$

MOMENTOS

a) Por carga muerta

$$M = \frac{wl^2}{8} + \frac{PL}{4} = \frac{2.32 \times (19)^2}{8} + \frac{0.94 \times 19}{4} = 109.16T-m$$

b) Por carga viva (camión C₄₀)

$$M = \left[\frac{4}{L} \left(\frac{L+a}{2} \right)^2 - 6 \right] P Fr$$

$$M = \left[\frac{4}{19} \left(\frac{19 + 0.51}{2} \right)^2 - 6 \right] 5.72 \times 1.23$$

$$M = 98.74 \text{ T-m}$$

$$M_u = \gamma (M_{cm} + 1.67 M_{cv} + I) \quad M_u = 1.3 (109.16 \text{ T-m} + 1.67(98.74 \text{ T-m}))$$

$$M_u = 356.27 \text{ T-m}$$

$$\text{Colocando 4 capas de varillas } \# 8 \text{ tenemos: } d = 135 - \frac{(2.54(4) + 3.75(4))}{2}$$

Si tomamos una cuantía $\rho = 0.0033$ tendremos $a/d = 0.0581$, donde:

$$a = 0.0581 \times 122.42 \text{ cm} = 7.11 \text{ cm} < t (18 \text{ cm}) \text{ ¡ok! (Se diseña como viga rectangular)}$$

$$M_R = \phi \times 0.85 \times f'_c \times b \times a \times (d - a/2)$$

$$M_R = 0.9 \times 0.85 \times 280 \text{ kg/cm}^2 \times 221 \text{ cm} \times 7.11 \text{ cm} \times (122.42 \text{ cm} - 7.11 \text{ cm}/2)$$

$$M_R = 40006940.1 \text{ Kg-cm} \quad M_R = 400.06 \text{ T-m}$$

$$\text{Luego } M_u = 356.27 \text{ T-m} < M_R = 400.06 \text{ T-m} \text{ ¡ok!}$$

ENVOLVENTE DE MOMENTO

a) Carga Muerta

$$M = \frac{4f}{L^2} (Lx - x^2)$$

b) Carga Viva

$$M = \frac{4f}{(L-a)^2} (L-a)x - x^2$$

$$\rho = \frac{\phi F'_c}{F_y} \left(1 - \sqrt{1 - \frac{2.62 K}{F'_c}} \right)$$

$d = 122.42\text{cm}$ $d' = 12.58\text{cm}$ $b = 40\text{cm}$ $f_c = 280\text{K/cm}^2$
 $f_y = 4200\text{K/cm}^2$

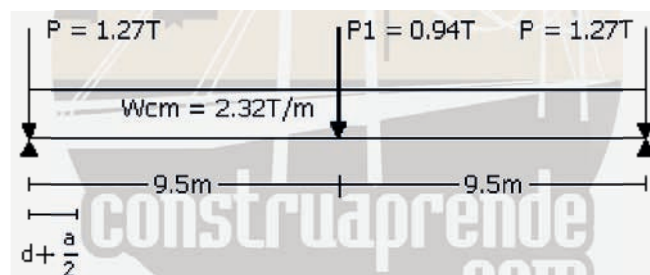
SECCIÓN	M_D	M_L	M_u	ρ_1	ρ	A_s	N° Barras
X (m)	(T-m)	(T-m)	(T-m)			(cm^2)	(# 8)
2	41.12	38.18	136.34	0.0064	0.0064	31.29	7
4	72.57	67.04	239.88	0.0118	0.0118	58.01	12
6	94.34	86.66	310.78	0.0160	0.0160	78.37	16
8	106.44	97.03	349.02	0.0184	0.0184	90.27	18
9.5	109.16	98.74	356.27	0.0189	0.0189	92.61	19

CORTANTE

Para los estribos se utilizarán barras # 4 de dos ramas.

a) Por carga muerta

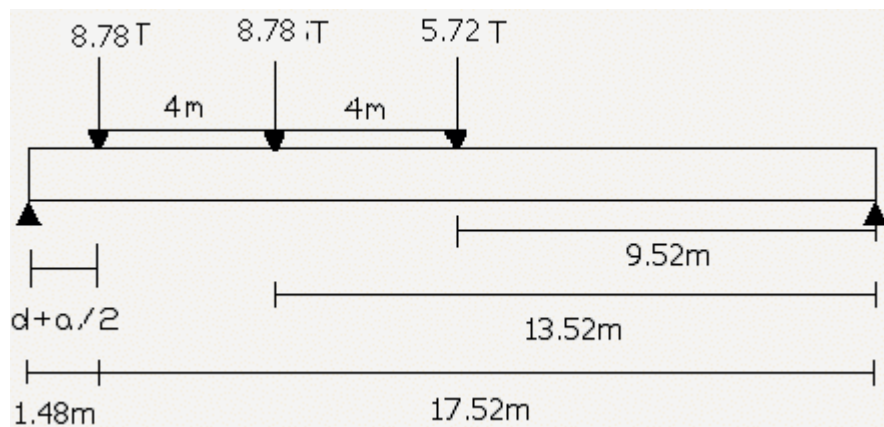
$$V_d = R_A$$



$$V_d = W_D \left(L/2 - \left(d + a/2 \right) \right) + P1/2$$

$$V_d = 2.32\text{T/m} \left(19\text{m}/2 - \left(1.22\text{m} + 0.51\text{m}/2 \right) \right) + 0.94\text{T}/2 = 19.09\text{T}$$

b) Por carga Viva



$$V_d = (R_t \times L_1 \times F_{rc} + R_t \times L_2 \times F_{rm} + R_d \times L_2 \times F_{rm})/19$$

$$V_d = (8.78T \times 17.52m \times 1.64 + 8.78T \times 13.52m \times 1.23 + 5.72T \times 9.52m \times 1.23)/19$$

$$V_d = 24.49T$$

CORTANTE ULTIMO

$$V_u = \gamma(V_D + 1.67V_L) \quad V_u = 1.3 (19.09T + 1.67(24.49T)) \quad V_u = 77.98T$$

$$\tau_u = V_u/(b \times d) \quad \tau_u = 77.98T/(0.4 \times 1.22) \quad \tau_u = 159.8T/m^2$$

$$\tau_u = 15.98Kg/cm^2$$

SEPARACION DE LOS ESTRIBOS

$$S_{\max} = d/2 \quad S_{\max} = 122.42 \text{ cm} / 2 \quad S_{\max} = 60 \text{ cm}$$

$$S = \frac{A_v * f_y}{(\gamma_u - \gamma_c) * b}$$

$$S = \frac{2 * (1.27 \text{ cm}^2) * 4200 \text{ kg/cm}^2}{(15.98 \text{ kg/cm}^2 - (0.85 * 0.53 * \sqrt{280 \text{ kg/cm}^2})) * 40 \text{ cm}}$$

$$S = 31 \text{ cm}$$

Colocar estribos # 4 cada 31 cms.

VIGAS LONGITUDINALES EXTERIORES

CARGAS

a) Carga Muerta

$$\text{Peso de placa: } 2.4 \text{ T/m}^3 \times 0.18 \text{ m} \times 2.21 \text{ m} = 0.95 \text{ T/m}$$

$$\text{Peso C. rodadura: } 2.2 \text{ T/m}^3 \times 0.05 \text{ m} \times 1.86 \text{ m} = 0.20 \text{ T/m}$$

$$\text{Peso nervio: } 2.4 \text{ T/m}^3 \times 0.4 \text{ m} \times 1.17 \text{ m} = 1.12 \text{ T/m}$$

$$WT = 2.27 \text{ T/m}$$

Peso viga diafragma apoyo = $1.27T/2 = 0.64T$

Peso viga diafragma central = $0.94T/2 = 0.47T$

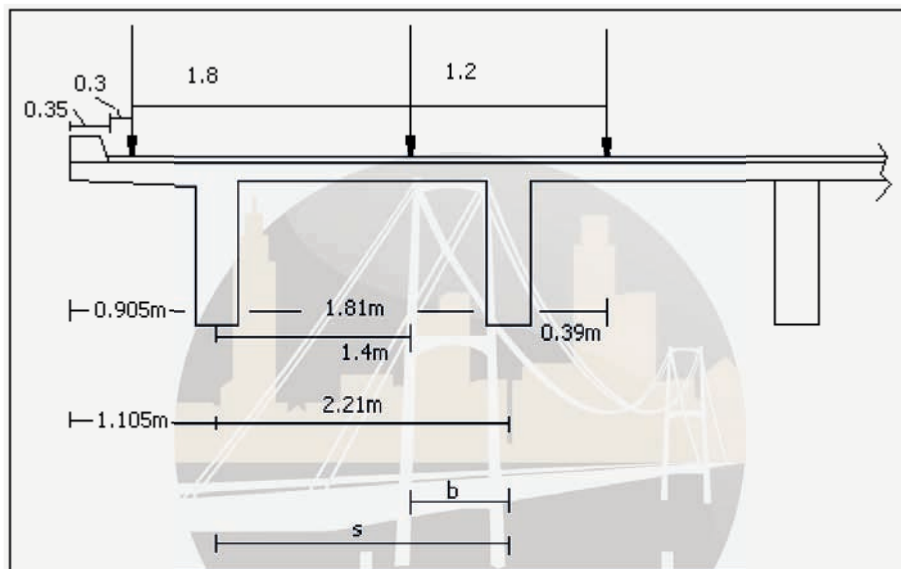
b) Carga Viva

Rueda delantera: 5T

Impacto: $I = 16/(S + 40) < 30\%$ $I = 16/(19 + 40)$ $I = 0.27$ $I = 27\%$

$C_v + I = 1.27 \times 5T$ $C_v + I = 6.35T = P$

Carga viva reducida para tres vías de tránsito 90% = 5.72T



$Fr = P (1 + (a + b)/S)$

$Fr = P (1 + (0 + 0.81)/2.21))$

Factor de Rueda para Cortante

$Frc = 1.37P$

Factor de Rueda para Momento $F_{rm} = 1.37P$

MOMENTOS

c) Por carga muerta

$$M = \frac{wl^2}{8} + \frac{PL}{4} = \frac{2.27 \times (19)^2}{8} + \frac{0.47 \times 19}{4} = 104.67T\text{-m}$$

d) Por carga viva (camión C₄₀)

$$M = \left[\frac{4}{L} \left(\frac{L+a}{2} \right)^2 - 6 \right] P F_r$$

$$M = \left[\frac{4}{19} \left(\frac{19+0.51}{2} \right)^2 - 6 \right] 5.72 \times 1.37$$

$$M = 109.97T\text{-m}$$

$$M_u = \gamma(M_{cm} + 1.67M_{cv} + I) \quad M_u = 1.3 (104.67T\text{-m} + 1.67(109.97T\text{-m}))$$

$$M_u = 374.82T\text{-m}$$

Colocando 4 capas de varillas # 8 tenemos: $d = 135 - \frac{(2.54(4) + 3.75(4))}{2}$

Si tomamos una cuantía $\rho = 0.0033$ tendremos $a/d = 0.0581$, donde:

$$a = 0.0581 \times 122.42\text{cm} = 7.11\text{cm} < t \text{ (18cm) ¡ok! (Se diseña como viga rectangular)}$$

$$M_R = \phi \times 0.85 \times f'_c \times b \times a \times (d - a/2)$$

$$M_R = 0.9 \times 0.85 \times 280 \text{ kg/cm}^2 \times 221 \text{ cm} \times 7.11 \text{ cm} \times (122.42 \text{ cm} - 7.11 \text{ cm}/2)$$

$$M_R = 40006940.1 \text{ Kg-cm}$$

$$M_R = 400.06 \text{ T-m}$$

$$\text{Luego } M_u = 374.82 \text{ T-m} < M_R = 400.06 \text{ T-m ¡ok!}$$

ENVOLVENTE DE MOMENTO

a) Carga Muerta

$$M = \frac{4f}{L^2} (Lx - x^2)$$

b) Carga Viva

$$M = \frac{4f}{(L-a)^2} (L-a)x - x^2$$

$$\rho = \phi \frac{F'_c}{F_y} \left(1 - \sqrt{1 - \frac{2.62 K}{F'_c}} \right)$$

$$d = 122.42 \text{ cm}$$

$$d' = 12.58 \text{ cm}$$

$$b = 40 \text{ cm}$$

$$f_c = 280$$

$$f_y = 4200$$

$$a = 0.51 \text{ m}$$

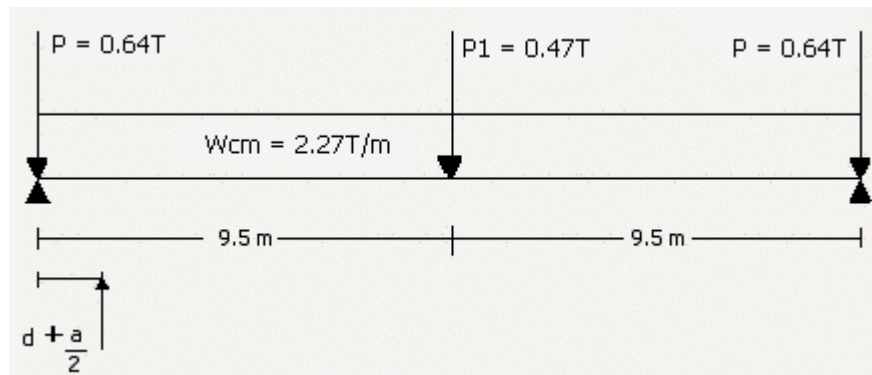
SECCIÓN (m)	M _D (T-m)	M _L (T-m)	M _u (T-m)	ρ ₁	ρ	As	N° Barras (# 8)
2	39.43	42.51	143.55	0.0068	0.0068	33.30	7
4	69.59	74.65	252.53	0.0126	0.0126	61.70	12
6	90.46	96.50	327.10	0.0170	0.0170	83.25	17
8	102.06	108.06	367.28	0.0197	0.0197	96.47	19
9.5	104.67	109.97	374.82	0.0202	0.0202	98.92	20

CORTANTE

Se utilizarán para los estribos, barras # 4 de dos ramas.

a) Por carga muerta

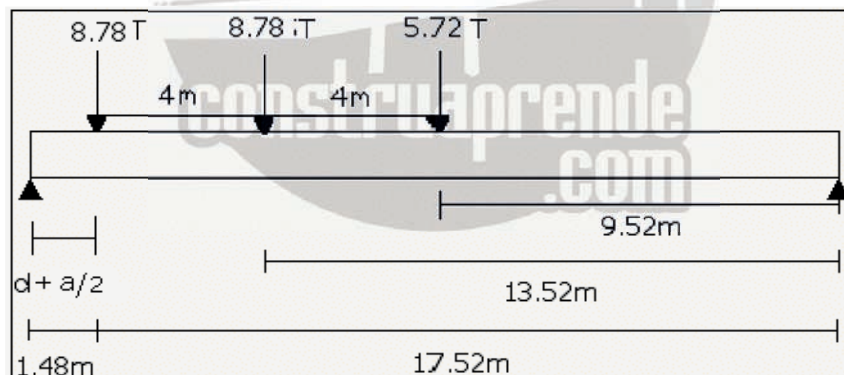
$$V_d = R_A$$



$$V_{dD} = W_D (L/2 - (d + a/2)) + P1/2$$

$$V_{dD} = 2.27T/m (19m/2 - (1.22m + 0.51m/2)) + 0.47T/2 = 18.45T$$

b) Por carga Viva



$$V_{dL} = (R_t \times L_1 \times F_{rc} + R_t \times L_2 \times F_{rm} + R_d \times L_2 \times F_{rm})/19$$

$$V_{dL} = (8.78T \times 17.52m \times 1.37 + 8.78T \times 13.52m \times 1.37 + 5.72T \times 9.52m \times 1.37)/19$$

$$V_{dL} = 23.58T$$

CORTANTE ULTIMO

$$V_u = \gamma(V_D + 1.67V_L) \quad V_u = 1.3 (18.45T + 1.67(23.58T)) \quad V_u = 75.18T$$

$$\tau_u = V_u / (b \times d) \quad \tau_u = 75.18T / (0.4 \times 1.22) \quad \tau_u = 154.06T/m^2$$

$$\tau_u = 15.41Kg/cm^2$$

SEPARACION DE LOS ESTRIBOS

$$S_{max} = d/2 \quad S_{max} = 122.42 \text{ cm} / 2 \quad S_{max} = 60 \text{ cm}$$

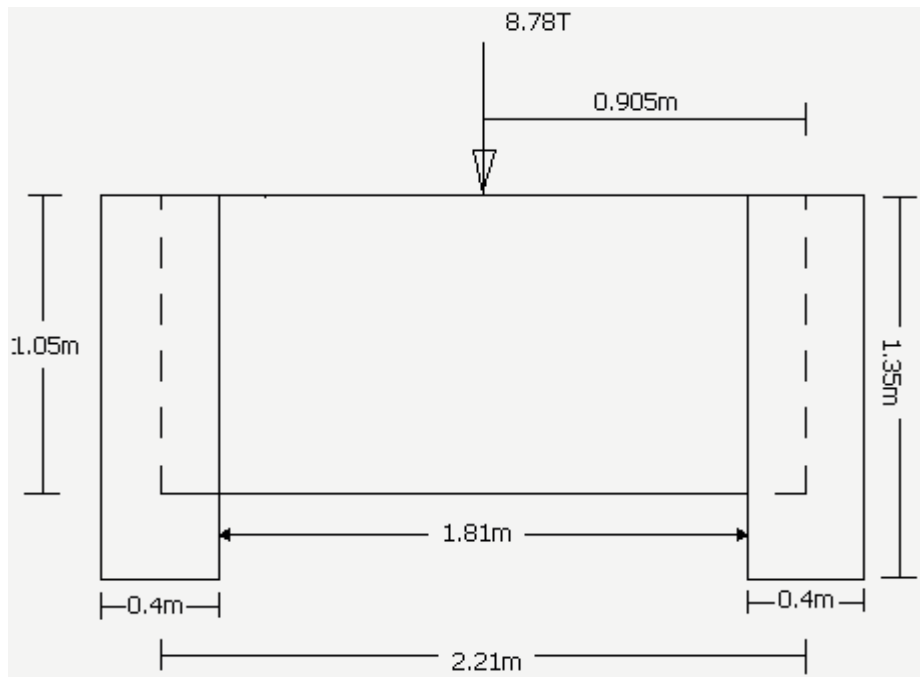
$$S = \frac{A_v \times f_y}{(\tau_u - \tau_c) \times b}$$

$$S = \frac{2 \times (1.27 \text{ cm}^2) \times 4200 \text{ kg/cm}^2}{(15.41 \text{ kg/cm}^2 - (0.85 \times 0.53 \times \sqrt{280 \text{ kg/cm}^2})) \times 40 \text{ cm}}$$

$$S = 33.88 \text{ cm}$$

Colocar estribos # 4 cada 33cms.

DISEÑO DE LA VIGA DIAFRAGMA



CARGAS

- Peso propio viga diafragma = $2.4T/m^3 \times 0.25m \times 1.05m = 0.63T/m$
- Carga viva = Peso rueda trasera + impacto = 8.78T

MOMENTO

A) Por carga muerta

Momento por Carga Muerta

$$M = WxL^2/10 \quad M = \frac{0.63T/m \times (1.81m)^2}{10} \quad M = 0.21T - m$$

Cortante por Carga Muerta

$$Vd = W (L/2 - (d)) \quad Vd = 0.63T/m (2.21m/2 - (0.98m)) \quad Vd = 0.079T$$

B) Por carga viva

$$V = 8.78T \times 1.105/2.21 = 4.39T$$

$$M = 4.39T \times 1.105m = 4.85T\text{-m}$$

$$\text{Momento ultimo: } Mu = 1.3 \times (M_{cm} + 1.67(M_{cv})) = 1.3 \times (0.21 + 1.67 \times 4.85) = 10.8T\text{-m}$$

ARMADURA

$$b = 25\text{cm}$$

$$d = 100\text{cm}$$

$$h = 105\text{cm}$$

$$K = 1080T\text{-cm} / (25 \times 100^2) = 0.0043$$

$$\rho = 0.0012$$

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

Colocar una 2 ϕ 6/8"

Por norma colocar 3 ϕ 6/8" arriba y abajo

CORTANTE ULTIMO

$$V_u = \gamma(V_D + 1.67V_L)$$

$$V_u = 1.3 (0.079T + 1.67(4.39T))$$

$$V_u = 9.63T$$

$$\mathcal{V}_u = V_u / (b \times d)$$

$$\mathcal{V}_u = 9.63T / (0.25 \times 0.98)$$

$$\mathcal{V}_u = 39.31T/\text{m}^2$$

$$\mathcal{V}_u = 3.93\text{Kg}/\text{cm}^2$$

$$\phi \mathcal{V}_c = 0.53 \times 0.85 \times \sqrt{(280\text{Kg}/\text{cm}^2)}$$

$$\phi \mathcal{V}_c = 7.54 \text{ Kg}/\text{cm}^2$$

$$\phi \mathcal{V}_s = 3.93\text{Kg}/\text{cm}^2 - 7.54 \text{ Kg}/\text{cm}^2$$

$$\phi \mathcal{V}_s = 3.61 \text{ Kg}/\text{cm}^2$$

$$\phi \mathcal{V}_s \approx \phi \mathcal{V}_c / 2 \quad \text{coloco mínimo refuerzo a cortante}$$

$$S = \frac{A_v \times f_y}{3.5 \times b}$$

$$S = \frac{2 \times 1.27\text{cm} \times 4200\text{Kg}/\text{cm}^2}{3.5 \times 25\text{cm}}$$

$$S = 121.92\text{cm}$$

$$S_{\max} = d/2$$

$$S_{\max} = 98\text{cm} / 2$$

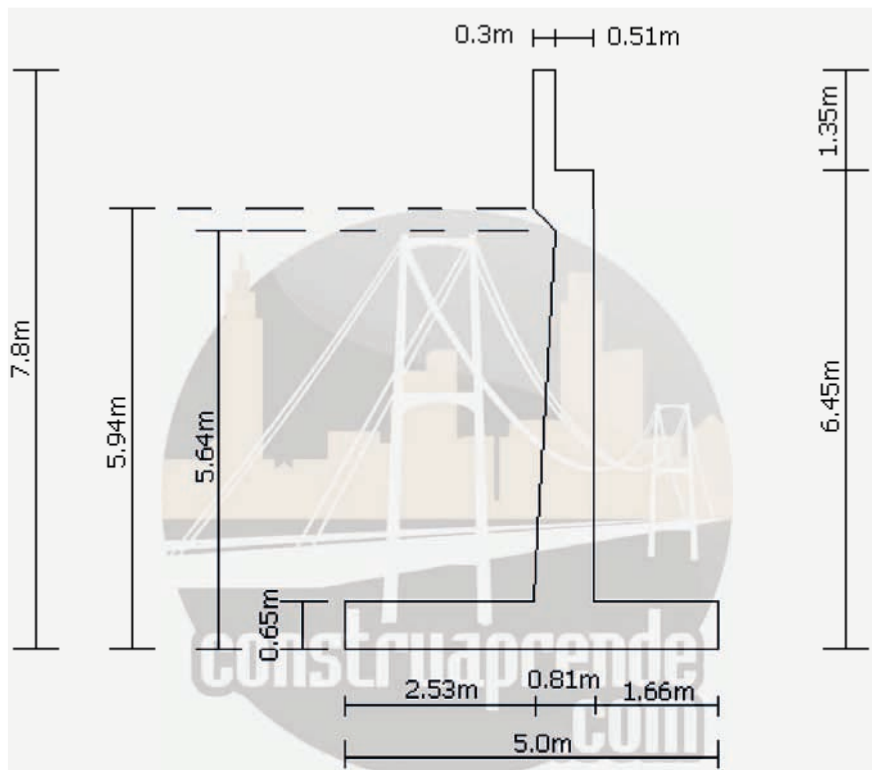
$$S_{\max} = 49 \text{ cm}$$

Colocar E ϕ 4/8 c/u 49cm

CALCULO DE LA INFRAESTRUCTURA

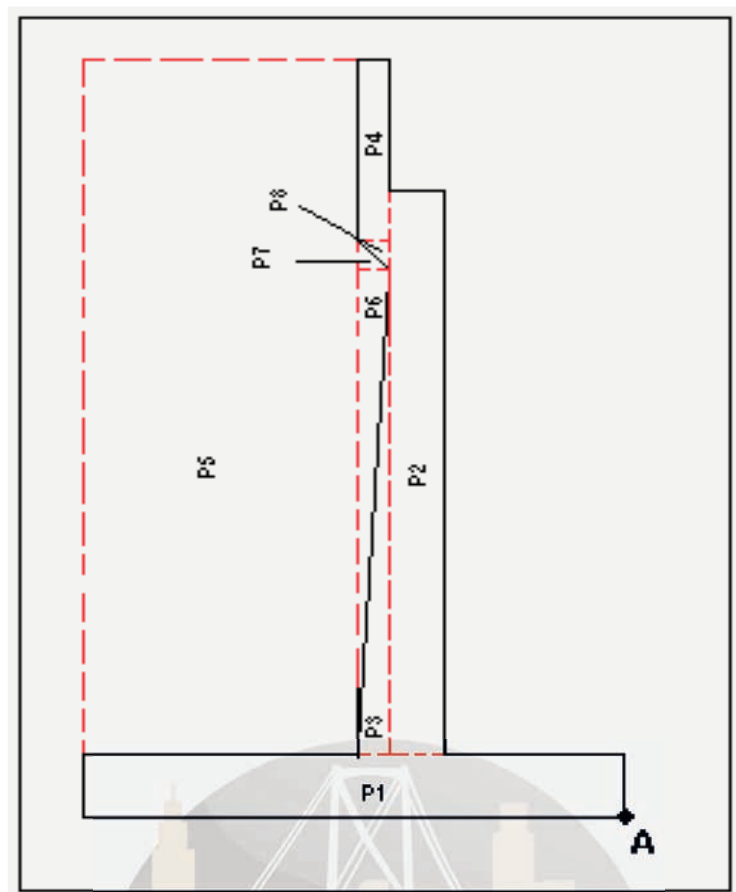
1. DIMENSIONAMIENTO

- Ancho de corona = $(h/24) \geq 0.3m = (7.8m/24) = 0.33m \approx 0.30m$
- Ancho caja del estribo = long. de apoyo de vigas = 0.51m
- Base del cimiento (B) = $(h/2 - 2h/3) = (3.9m - 5.2m) \rightarrow 5.00m$
- Altura del cimiento = $h/12 = 7.8m/12 = 0.65m$
- Base del vástago = $(h/8 - h/10) = (0.98m - 0.78m) \rightarrow 0.81m$
- Zarpa delantera = $(B/3) = 5.0m/3 = 1.66m$
- Zarpa trasera = $5 - (0.81 + 1.66) = 2.53m$
- Altura de caja = altura de la viga = 1.35m



2. CARGAS

a) Muerta infraestructura



P	Carga	P (Ton)	Xa (m)	MXa (Ton-m)	Ya (m)	MYa (Ton-m)
P1	2,4 x 5 x 1 x 0,65	7,80	2,500	19,50	0,325	2,535
P2	2,4 x 5,8 x 1 x 0,51	7,10	1,915	13,59	3,550	25,202
P3	(2,4 x 0,3 x 4,99 x 1)/2	1,80	2,270	4,08	2,313	4,155
P4	2,4 x 0,3 x 1 x 1,86	1,34	2,320	3,11	6,870	9,200
P5	1,8 x 2,53 x 1 x 7,15	32,56	3,735	121,62	4,225	137,571
P6	(1,8 x 0,3 x 4,99 x 1)/2	1,35	2,370	3,19	3,978	5,360
P7	(1,8 x 0,3 x 0,3 x 1)/2	0,08	2,370	0,19	5,740	0,465
P8	(2,4 x 0,3 x 0,3 x 1)/2	0,11	2,270	0,25	5,84	0,631
Σ		52,13		165,53		185,118

b) Muerta superestructura

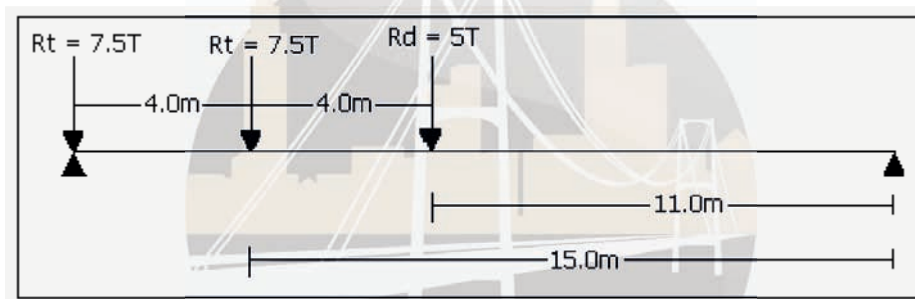
- Peso placa : $0.18\text{m} \times 11.05\text{m} \times 2.4\text{T/m}^3$ = 4.77T/m
 - Capa de rodadura : $0.05\text{m} \times 10.35\text{m} \times 2.2\text{T/m}^3$ = 1.14T/m
 - Bordillos : $0.325\text{m} \times 0.25\text{m} \times 2.4\text{T/m}^3 \times 2$ = 0.39T/m
 - Acartelamiento : $\frac{1}{2} \times 0.04\text{m} \times 0.905\text{m} \times 2.4\text{T/m}^3 \times 2$ = 0.09T/m
 - Vigas : $0.4\text{m} \times 1.17\text{m} \times 2.4\text{T/m}^3 \times 5$ = 5.62T/m
- $W = 12.01\text{T/m}$**

- Diafragma de apoyo : $1.27\text{T} \times 4 = 5.08\text{T}$
- Diafragma central : $0.94\text{T} \times 2 = 1.89\text{T}$

$$D = \frac{12.01 \times (19.51/2) + 5.08 + 1.89}{11.05} = 11.23\text{T/m}$$

$$M_{AD} = 11.23\text{T} \times 1.915\text{m} = 21.51\text{T-m}$$

c) Viva



$$RL = ((7.5 \times 1.64 \times 19) + (7.5 \times 1.23 \times 15) + (5 \times 1.23 \times 11)) \times 5 / 19$$

$$RL = 115.72\text{T}$$

$$RL = 115.72/11.05 = 10.47\text{T}$$

$$M_{AL} = 10.47\text{T} \times 1.915\text{m} = 20.05\text{T-m}$$

d) Flotacion (B)

El nivel de aguas freáticas se encuentra a 3.0m por encima del suelo de fundación.

$$B = \gamma \times V$$

$$B = 1 \times [(0.65\text{m} \times 5\text{m} \times 1\text{m}) + (0.51\text{m} \times 2.35\text{m} \times 1\text{m}) + (0.16\text{m} + 0.3\text{m})2.35\text{m}/2]$$

$$B = 4.99\text{T}$$

$$MB_A = 4.99\text{T} \times 2.34\text{m}$$

$$MB_A = 11.68\text{T-m}$$

e) Viento (W)

- Carga de Viento en Súper Estructura

○ Carga Muerta

$$W_D = \frac{0.059 \text{ T/m}^2 \times 18.49\text{m} \times 1.60\text{m}}{11.05\text{m}}$$

$$W_D = 0.16\text{T}$$

$$MW_{Da} = 0.16\text{T} \times 7\text{m} \quad MW_{Da} = 1.12\text{T-m}$$

○ Carga Viva

$$W_L = \frac{0.6\text{T/m} \times 19.51\text{m}}{11.05\text{m}}$$

$$W_L = 1.06\text{T}$$

$$MW_{LA} = 1.06\text{T} \times (7.8\text{m} + 1.83\text{m})$$

$$MW_{LA} = 10.21\text{T-m}$$

f) Fuerza Longitudinal (LF)

$$FL = \frac{0.05 \times 10.47 \times 3}{11.05}$$

$$FL = 0.14\text{T}$$

$$M_{FLA} = 0.14T \times (7.8 + 1.83) \quad M_{FLA} = 1.35T\text{-m}$$

g) Empuje de Tierra (E)

$$h' = 0.61\text{m} \quad \phi = 30^\circ$$

$$E = \frac{1}{2} \times \gamma \times K_a \times h (h + 2h') \quad K_a = \tan^2 (45 - \phi/2) \quad K_p = 1/K_a$$

$$K_a = 1/3 \quad K_p = 3$$

$$E = \frac{1}{2} \times 1.8T/m^3 \times 1/3 \times 7.8\text{m} (7.8\text{m} + 2 \times 0.61\text{m}) \quad E = 21.11T$$

$$\hat{Y} = \frac{h}{3} \frac{(h + 3h')}{h + 2h'} \quad \hat{Y} = \frac{7.8\text{m} (7.8\text{m} + 3(0.61))}{3 \times (7.8 + 2(0.61))} \quad \hat{Y} = 2.78\text{m}$$

$$ME_A = 21.11T \times 2.78\text{m}$$

$$ME_A = 58.69T\text{-m}$$

h) Fuerza de Sismo (EQ)

- Infraestructura (I)

$$EQ_I = P (0.04 - 0.06) \quad EQ_I = 52.13 (0.04) \quad EQ_I = 2.1T$$

$$\hat{Y} = MY/P \quad \hat{Y} = 185.118T\text{-m} / 52.13T \quad \hat{Y} = 3.55\text{m}$$

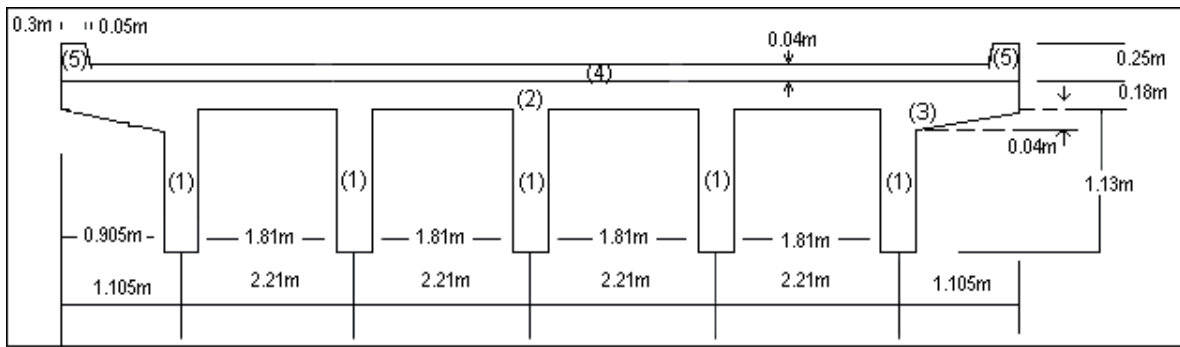
$$M_{EQIA} = 2.1T \times 3.55\text{m} \quad M_{EQIA} = 7.46T\text{-m}$$

- Super estructura (S)

$$\text{Peso de Súper Estructura} = 11.23T/m \times 11.05\text{m} \times 2 \quad W_{QS} = 248.18T$$

$$EQS = \frac{248.18T}{11.05\text{m}} \times 0.04 \quad EQS = 0.9T$$

Calculo del Centro de Masa de la Súper estructura



SECCIÓN	W	W	\bar{Y}	$W\bar{Y}$
1	2,4 X 0,4 X 1,17 X 5	5,62	0,59	3,29
2	2,4 X 0,18 X 11,05	4,77	1,26	6,01
3	2,4 X (0,04 X 0,905)/2 X 2	0,09	1,16	0,10
4	2,2 X 0,05 X 10,35	1,14	1,38	1,57
5	2,4 X 2 X (0,35+0,30) X 0,25/2	0,39	1,48	0,58
		12,01		10,97

$$\hat{Y} = M / P \quad \hat{Y} = 10.97/12.01 \quad \hat{Y} = 0.91m$$

$$M_{EQSA} = 0.9T \times (0.91m + 6.45m) \quad M_{EQSA} = 6.62T-m$$

CARGA	SENTIDO		MOMENTO	
D	↓	11.23T	M_{AD}	21.51T-m
L	↓	10.47T	M_{AL}	20.05T-m
B	↑	4.99T	M_{Ba}	11.68T-m
WD	→	0.16T	MW_{Da}	1.12T-m
WL	→	1.06T	MW_{La}	10.21T-m
LF	→	0.14T	M_{FLA}	1.35T-m
E	→	21.11T	M_{EA}	58.69T-m
EQ _I	→	2.1T	M_{EQIA}	7.46T-m
EQS	→	0.9T	M_{EQSA}	6.62T-m
P	↓	52.13T	M_{PA}	165.53T-m

ANALISIS DE ESTABILIDAD

❖ GRUPO I (ESTRIBO SIN CARGAS) (D + L + E + B + SF + CF) 100%

$$L = 0, SF = 0, CF = 0$$

ESTABILIDAD A LA CAPACIDAD PORTANTE DEL SUELO

$$\sigma_{\text{máx}} = \frac{P}{L^2} (4L - 6a) \quad \sigma_{\text{mín}} = \frac{P}{L^2} (6a - 2L) \quad a = \frac{\sum M_v - \sum M_h}{\Delta P}$$

CARGA	SENTIDO		MOMENTO	
P	↓	52.13T	M _{PA}	165.53T-m
B	↑	4.99T	M _{ba}	11.68T-m
E	→	21.11T	M _{EA}	58.69T-m

$$a = ((165.53\text{T-m} - 11.68\text{T-m}) - (58.69\text{T-m})) / (52.13\text{T} - 4.99\text{T}) = (95\text{T-m}) / (47.14\text{T})$$

$$a = 2.02\text{m}$$

$$P = 47.14\text{T}$$

$$(3.33) \geq a \geq (1.66) \quad \text{¡Ok!}$$

$$\sigma_{\text{máx}} = \frac{47.14\text{T}}{(5\text{m})^2} (4 \times 5\text{m} - 6 \times 2.02\text{m})$$

$$\sigma_{\text{máx}} = 14.86\text{T/m}^2 \leq 28\text{ T/m}^2 \quad \text{¡ok!}$$

$$\sigma_{\text{mín}} = \frac{47.14\text{T}}{(5\text{m})^2} (6 \times 2.02\text{m} - 2 \times 5\text{m})$$

$$\sigma_{\text{mín}} = 4.0\text{T/m}^2 \leq 28\text{ T/m}^2 \quad \text{¡ok!}$$



ESTABILIDAD AL VOLCAMIENTO (Fv)

$$F_v = \Sigma (\text{momentos estabilizadores}) / \Sigma (\text{momentos desestabilizadores}) \geq 1.5$$

$$F_v = (165.53\text{T-m} - 11.68\text{T-m}) / (58.69\text{T-m}) = 2.62 > 1.5 \text{ ¡ok!}$$

ESTABILIDAD AL DESLIZAMIENTO (Fd)

$$F_d = (\Sigma F_v \times \mu) / \Sigma F_H = (47.14\text{T} \times 0.5) / 21.11\text{T} = 1.12 < 1.5 \text{ No ¡ok!}$$

NOTA: El Estribo requiere llave o espolón.

❖ GRUPO I (ESTRIBO CARGADO) (D + L + E + B + SF + CF) 100%

$$SF = 0, CF = 0$$

ESTABILIDAD A LA CAPACIDAD PORTANTE DEL SUELO

$$\sigma_{\text{máx}} = \frac{P}{L^2} (4L - 6a) \quad \sigma_{\text{mín}} = \frac{P}{L^2} (6a - 2L) \quad a = \frac{\Sigma M_v - \Sigma M_h}{\Sigma P}$$

CARGA	SENTIDO		MOMENTO	
P	↓	52.13T	M _{PA}	165.53T-m
D	↓	11.23T	M _{AD}	21.51T-m
L	↓	10.47T	M _{AL}	20.05T-m
B	↑	4.99T	M _{ba}	11.68T-m
E	→	21.11T	M _{E_A}	58.69T-m

$$a = ((165.53 + 21.51 + 20.05 - 11.68) - (58.69)) / (52.13 + 11.23 + 10.47 - 4.99) \\ = (136.72\text{T-m}) / (68.84\text{T})$$

$$a = 1.99\text{m}$$

$$P = 68.84\text{T}$$

$$(3.33) \geq a \geq (1.66) \quad \text{¡Ok!}$$

$$\sigma_{\text{máx}} = \frac{68.84\text{T}}{(5\text{m})^2} (4 \times 5\text{m} - 6 \times 1.99\text{m})$$

$$\sigma_{\text{máx}} = 22.19\text{T/m}^2 \leq 28\text{ T/m}^2 \quad \text{¡ok!}$$

$$\sigma_{\text{min}} = \frac{68.84\text{T}}{(5\text{m})^2} (6 \times 1.99\text{m} - 2 \times 5\text{m})$$

$$\sigma_{\text{min}} = 5.34\text{T/m}^2 \leq 28\text{ T/m}^2 \quad \text{¡ok!}$$

ESTABILIDAD AL VOLCAMIENTO (Fv)

$$F_v = \Sigma (\text{momentos estabilizadores}) / \Sigma (\text{momentos desestabilizadores}) \geq 1.5$$

$$F_v = (165.53\text{T-m} + 21.51\text{T-m} + 20.05\text{T-m} - 11.68\text{T-m}) / (58.69\text{T-m}) = 3.84 > 1.5 \quad \text{¡ok!}$$

ESTABILIDAD AL DESLIZAMIENTO (Fd)

$$F_d = (\Sigma F_v \times \mu) / \Sigma F_H = (68.84\text{T} \times 0.5) / 21.11\text{T} = 1.63 > 1.5 \quad \text{¡ok!}$$

❖ **GRUPO I I (ESTRIBO CARGADO) (D + E + B + SF + W) 125%**

$$SF = 0$$

ESTABILIDAD A LA CAPACIDAD PORTANTE DEL SUELO

$$\sigma_{\text{máx}} = \frac{P}{L^2} (4L - 6a) \quad \sigma_{\text{mín}} = \frac{P}{L^2} (6a - 2L) \quad a = \frac{\Sigma M_v - \Sigma M_h}{\Delta P}$$

CARGA	SENTIDO		MOMENTO	
P	↓	52.13T	M _{PA}	165.53T-m
D	↓	11.23T	M _{AD}	21.51T-m
E	→	21.11T	M _{EA}	58.69T-m
B	↑	4.99T	M _{ba}	11.68T-m
WD	→	0.16T	M _{WDa}	1.12T-m

$$a = ((165.53 + 21.51 - 11.68) - (58.69 + 1.12)) / (52.13 + 11.23 - 4.99)$$

$$= (115.55\text{T-m}) / (58.37\text{T})$$

$$a = 1.98\text{m}$$

$$P = 58.37\text{T}$$

$$(3.33) \geq a \geq (1.66) \quad \text{¡Ok !}$$

$$\sigma_{\text{máx}} = \frac{58.37\text{T}}{(5\text{m})^2} (4 \times 5\text{m} - 6 \times 1.98\text{m})$$

$$\sigma_{\text{máx}} = 18.96\text{T/m}^2$$

$$\text{Aplicando el 125\% : } \sigma_{\text{máx}} = (18.96\text{T/m}^2) / 1.25 = 15.17\text{T/m}^2 \leq 28\text{T/m}^2 \quad \text{¡ok !}$$

$$\sigma_{\text{min}} = \frac{58.37\text{T}}{(5\text{m})^2} (6 \times 1.98\text{m} - 2 \times 5\text{m})$$

$$\sigma_{\text{min}} = 4.39\text{T/m}^2$$

$$\text{Aplicando el 125\% : } \sigma_{\text{min}} = (4.39\text{T/m}^2) / 1.25 = 3.51\text{T/m}^2 \leq 28\text{T/m}^2 \quad \text{¡ok !}$$

ESTABILIDAD AL VOLCAMIENTO (Fv)

$$F_v = \Sigma (\text{momentos estabilizadores}) / \Sigma (\text{momentos desestabilizadores}) \geq 1.5$$

$$F_v = (165.53\text{T-m} + 21.51\text{T-m} - 11.68\text{T-m}) / (58.69\text{T-m} + 1.12\text{T-m}) = 2.93 > 1.5 \quad \text{¡ok !}$$

ESTABILIDAD AL DESLIZAMIENTO (Fd)

$$F_d = (\Sigma F_v \times \mu) / \Sigma F_H = (58.37T \times 0.5) / (21.11T + 0.16T) = 1.37 < 1.5 \quad \text{No ¡ok!}$$

NOTA: El Estribo requiere llave o espolón.

❖ **GRUPO I I I (ESTRIBO CARGADO)** $(D + L + E + B + CF + 0.3WD + SF + WL + LF)$ 125%

$$SF = 0, CF = 0$$

ESTABILIDAD A LA CAPACIDAD PORTANTE DEL SUELO

$$\sigma_{\max} = \frac{P}{L^2} (4L - 6a) \quad \sigma_{\min} = \frac{P}{L^2} (6a - 2L) \quad a = \frac{\Sigma M_v - \Sigma M_h}{\Delta P}$$

CARGA	SENTIDO		MOMENTO	
D	↓	11.23T	M_{AD}	21.51T-m
L	↓	10.47T	M_{AL}	20.05T-m
B	↑	4.99T	M_{ba}	11.68T-m
0.3WD	→	0.048T	$M_{W_{Da}}$	0.336T-m
WL	→	1.06T	$M_{W_{LA}}$	10.21T-m
LF	→	0.14T	M_{FLA}	1.35T-m
E	→	21.11T	M_{EA}	58.69T-m
P	↓	52.13T	M_{PA}	165.53T-m

$$a = ((165.53 + 20.05 + 21.51 - 11.68) - (58.69 + 1.35 + 10.21 + 0.336)) / (52.13 + 11.23 + 10.47 - 4.99)$$

$$= (124.824T\text{-m})/(68.84T)$$

$$a = 1.81\text{m}$$

$$P = 68.84T$$

$$(3.33) \geq a \geq (1.66) \quad \text{¡Ok !}$$

$$\sigma_{\text{máx}} = \frac{68.84T (4 \times 5\text{ m} - 6 \times 1.81\text{m})}{(5\text{ m})^2}$$

$$\sigma_{\text{máx}} = 25.17T/\text{m}^2$$

$$\text{Aplicando el 125\% : } \sigma_{\text{máx}} = (25.17T/\text{m}^2)/1.25 = 20.13T/\text{m}^2 \leq 28\text{ T/m}^2 \quad \text{¡ok !}$$

$$\sigma_{\text{min}} = \frac{68.84T (6 \times 1.81\text{m} - 2 \times 5\text{ m})}{(5\text{ m})^2}$$

$$\sigma_{\text{min}} = 2.37T/\text{m}^2$$

$$\text{Aplicando el 125\% : } \sigma_{\text{min}} = (2.37T/\text{m}^2)/1.25 = 1.894T/\text{m}^2 \leq 28\text{ T/m}^2 \quad \text{¡ok !}$$

ESTABILIDAD AL VOLCAMIENTO (Fv)

$$F_v = \Sigma (\text{momentos estabilizadores}) / \Sigma (\text{momentos desestabilizadores}) \geq 1.5$$

$$F_v = (165.53T\text{-m} + 21.51T\text{-m} + 20.05T\text{-m} - 11.68T\text{-m}) / (58.69T\text{-m} + 1.35T\text{-m} + 10.21T\text{-m} + 0.336T\text{-m}) = 2.77 > 1.5 \quad \text{¡ok !}$$

ESTABILIDAD AL DESLIZAMIENTO (Fd)

$$F_d = (\Sigma F_v \times \mu) / \Sigma F_H = (68.84T \times 0.5) / (21.11T + 0.14T + 1.06T + 0.048T) = 1.54 > 1.5 \quad \text{¡ok !}$$

❖ **GRUPO V I I (ESTRIBO CARGADO) (D + E + B + SF + EQ) 133%**

$$SF = 0$$

ESTABILIDAD A LA CAPACIDAD PORTANTE DEL SUELO

$$\sigma_{\text{máx}} = \frac{P}{L^2} (4L - 6a) \quad \sigma_{\text{mín}} = \frac{P}{L^2} (6a - 2L) \quad a = \frac{\sum M_v - \sum M_h}{\Delta P}$$

CARGA	SENTIDO		MOMENTO	
D	↓	11.23T	M _{AD}	21.51T-m
B	↑	4.99T	M _{ba}	11.68T-m
EQ _I	→	2.1T	M _{EQIA}	7.46T-m
EQ _S	→	0.9T	M _{EQSA}	6.62T-m
E	→	21.11T	M _{E_A}	58.69T-m
P	↓	52.13T	M _{PA}	165.53T-m

$$a = ((165.53 + 21.51 - 11.68) - (58.69 + 6.62 + 7.46)) / (52.13 + 11.23 - 4.99) \\ = (102.59\text{T-m}) / (58.37\text{T})$$

$$a = 1.76\text{m}$$

$$P = 58.37\text{T}$$

$$(3.33) \geq a \geq (1.66) \quad \text{¡Ok !}$$

$$\sigma_{\text{máx}} = \frac{58.37\text{T}}{(5\text{m})^2} (4 \times 5\text{m} - 6 \times 1.76\text{m})$$

$$\sigma_{\text{máx}} = 22.04\text{T/m}^2$$

$$\text{Aplicando el 133\% : } \sigma_{\text{máx}} = (22.04\text{T/m}^2) / 1.33 = 16.57\text{T/m}^2 \leq 28\text{ T/m}^2 \quad \text{¡ok !}$$

$$\sigma_{\text{mín}} = \frac{58.37\text{T}}{(5\text{m})^2} (6 \times 1.76\text{m} - 2 \times 5\text{m})$$

$$\sigma_{\text{mín}} = 1.31\text{T/m}^2$$

$$\text{Aplicando el 133\% : } \sigma_{\text{mín}} = (1.31\text{T/m}^2) / 1.33 = 0.99\text{T/m}^2 \leq 28\text{ T/m}^2 \quad \text{¡ok !}$$

ESTABILIDAD AL VOLCAMIENTO (Fv)

$$F_v = \Sigma (\text{momentos estabilizadores}) / \Sigma (\text{momentos desestabilizadores}) \geq 1.5$$

$$F_v = (165.53T\text{-m} + 21.51T\text{-m} - 11.68T\text{-m}) / (58.69T\text{-m} + 6.62T\text{-m} + 7.46T\text{-m}) = 2.41 > 1.5 \text{ ¡ok !}$$

ESTABILIDAD AL DESLIZAMIENTO (Fd)

$$F_d = (\Sigma F_v \times \mu) / \Sigma F_H = (58.37T \times 0.5) / (21.11T + 0.9T + 2.1T) = 1.22 < 1.5 \text{ No ¡ok !}$$

NOTA: El Estribo requiere llave o espolón.

DISEÑO DEL VASTAGO

BASE DEL VASTAGO

$$\begin{aligned} h &= 7.15\text{m} & h' &= 0.61\text{m} & \phi &= 30^\circ & K_a &= 1/3 & K_p &= 3 \\ E &= \frac{1}{2} \times \gamma \times K_a \times h (h + 2h') & E &= \frac{1}{2} \times 1.8T/\text{m}^3 \times 1/3 \times 7.15\text{m} (7.15\text{m} + 2 \times 0.61\text{m}) \\ E &= 17.95T \end{aligned}$$

$$\hat{Y} = \frac{h}{3} \frac{(h + 3h')}{h + 2h'} \qquad \hat{Y} = \frac{7.15\text{m} (7.15\text{m} + 3(0.61))}{3 \times (7.15 + 2(0.61))} \qquad \hat{Y} = 2.56\text{m}$$

$$M_{E_U} = 1.6 \times 17.95T \times 2.56\text{m} \qquad M_{E_U} = 73.52T\text{-m}$$

$$F'_c = 280\text{Kg}/\text{cm}^2 \qquad F_y = 4200\text{Kg}/\text{cm}^2 \qquad h = 81\text{cm} \qquad d = 74\text{cm}$$

$$b = 100\text{cm} \qquad d' = 7\text{cm} \qquad K = M_u / b x d^2 \qquad K = 73.52T\text{-m} / (1\text{m} \times (74\text{cm}^2)^2)$$

$$K = 0.0134T/\text{cm}^2 \qquad \rho = 0.0037 \qquad A_s = 100 \times 74 \times 0.0037 \qquad A_s = 27.38\text{cm}^2$$

$$S = 100 \times (2.85\text{cm}^2 / 27.38\text{cm}^2) \qquad S = 10.41\text{cm}, \text{ Colocar una } \phi 6/8'' \text{ c/u } 10 \text{ cm.}$$

HACIENDO UN CORTE A $h = 3.6\text{m}$

$$h = 3.6\text{m} \qquad h' = 0.61\text{m} \qquad \phi = 30^\circ \qquad K_a = 1/3 \qquad K_p = 3$$

$$E = \frac{1}{2} \times \gamma \times K_a \times h (h + 2h') \quad E = \frac{1}{2} \times 1.8T/m^3 \times 1/3 \times 3.6m (3.6m + 2 \times 0.61m)$$

$$E = 5.21T$$

$$\hat{Y} = \frac{h (h + 3h')}{3 (h + 2h')} \quad \hat{Y} = \frac{3.6m (3.6m + 3(0.61))}{3 \times (3.6 + 2(0.61))} \quad \hat{Y} = 1.35m$$

$$ME_U = 1.6 \times 5.21T \times 1.35m \quad ME_U = 11.25T \cdot m$$

$$F'_c = 280Kg/cm^2 \quad F_y = 4200Kg/cm^2 \quad h = 66Cm \quad d = 59cm$$

$$b = 100cm \quad d' = 7cm \quad K = Mu/bxd^2 \quad K = 11.25T \cdot m / (1m \times (59cm^2)^2)$$

$$K = 0.0032T/cm^2 \quad \rho = 0.0009 \quad A_s = 100 \times 59 \times 0.0009 \quad A_s = 5.31Cm^2$$

$$S = 100 \times (2.85Cm^2 / 5.31Cm^2) \quad S = 53.7Cm, \text{ Colocar una } \emptyset 6/8'' \text{ c/u } 20 \text{ cm.}$$

ARMADURA DE DISTRIBUCIÓN

$$A_s = 0.0012 \times 100 \times 81 \quad A_s = 9.72Cm^2 \quad S = 100 \times (1.27Cm^2 / 9.72Cm^2)$$

$$S = 13.07Cm, \text{ Colocar una } \emptyset 4/8'' \text{ c/u } 13 \text{ cm.}$$

Colocar 4/8'' C/U 26Cm a cada lado del estribo.

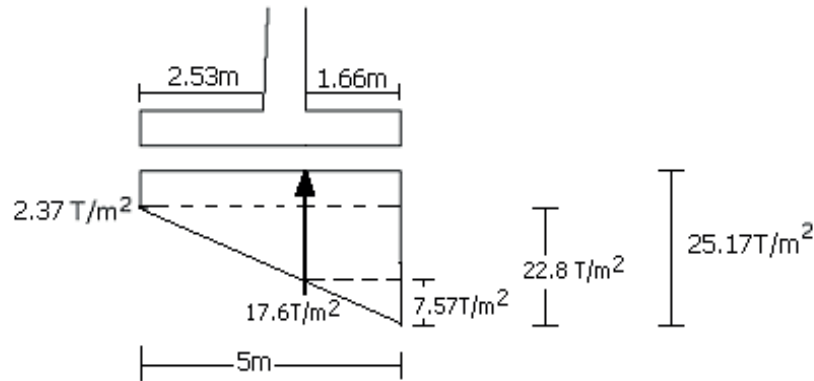
ARMADURA DE DISTRIBUCIÓN EN LA CARA POSTERIOR (LADO SIN RELLENO)

$$A_s = 0.0016 \times 100 \times 81 \quad A_s = 12.96Cm^2 \quad S = 100 \times (2Cm^2 / 12.96Cm^2)$$

$$S = 15.43Cm, \text{ Colocar una } \emptyset 5/8'' \text{ c/u } 15 \text{ cm.}$$

DISEÑO DE LA ZARPA DELANTERA

Tomamos el Grupo 1 (cargado)



$$M_u = 1.6 (17.6 \times (1.66)^2 / 2) + (\frac{1}{2} \times 7.57 \times (1.66)^2 \times 2/3) - 2.4 \times 0.65 \times (1.66)^2 / 2$$

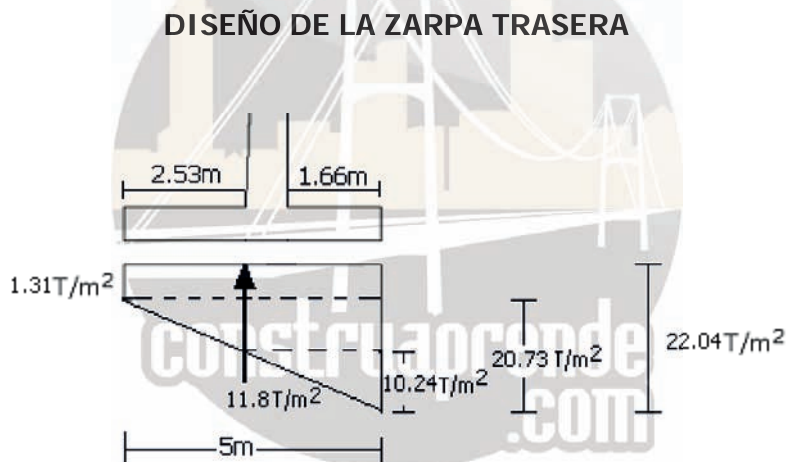
$$M_u = 46.49 \text{ T-m}$$

$$F'_c = 280 \text{ Kg/cm}^2 \quad F_y = 4200 \text{ Kg/cm}^2 \quad h = 65 \text{ cm} \quad d = 58 \text{ cm}$$

$$b = 100 \text{ cm} \quad d' = 7 \text{ cm} \quad K = M_u / b x d^2 \quad K = 46.49 \text{ T-m} / (1 \text{ m} \times (58 \text{ cm})^2)$$

$$K = 0.0138 \text{ T/cm}^2 \quad \rho = 0.0038 \quad A_s = 100 \times 58 \times 0.0035 \quad A_s = 21.99 \text{ cm}^2$$

$$S = 100 \times (2.85 \text{ cm}^2 / 21.99 \text{ cm}^2) \quad S = 12.96 \text{ cm}, \text{ Colocar una } \phi 6/8'' \text{ c/u } 12 \text{ cm}.$$



$$M_u = 1.6 ((1.8 \times 7.15 \times (2.53)^2 / 2) + (2.4 \times 0.65 \times (2.53)^2 / 2)) - [(\frac{1}{2} \times 10.49 \times (2.53)^2 \times 1/3) - (1.31 \times (2.53)^2 / 2)]$$

$$M_u = 49.3 \text{ T-m}$$

$$F'_c = 280\text{Kg/cm}^2 \quad F_y = 4200\text{Kg/cm}^2 \quad h = 65\text{cm} \quad d = 58\text{cm}$$

$$b = 100\text{cm} \quad d' = 7\text{cm} \quad K = M_u/bxd^2 \quad K = 49.3\text{T-m}/(1\text{m} \times (58\text{cm})^2)$$

$$K = 0.015\text{T/cm}^2 \quad \rho = 0.004 \quad A_s = 100 \times 58 \times 0.004 \quad A_s = 23.37\text{cm}^2$$

$$S = 100 \times (2.85\text{cm}^2/23.37\text{cm}^2) \quad S = 12.20\text{cm}, \text{ Colocar una } \phi 6/8'' \text{ c/u } 12 \text{ cm}.$$

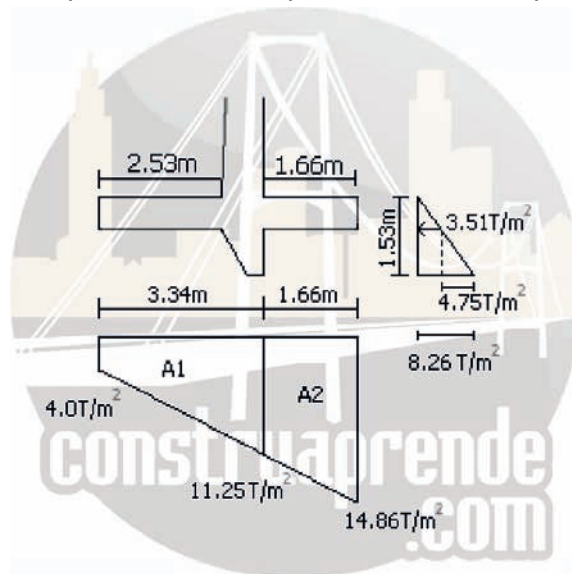
ARMADURA DE DISTRIBUCIÓN

$$A_s = 0.0012 \times 100 \times 65 \quad A_s = 7.8\text{cm}^2 \quad S = 100 \times (1.27\text{cm}^2/7.8\text{cm}^2)$$

$$S = 16\text{cm}, \text{ Colocar una } \phi 6/8'' \text{ c/u } 16 \text{ cm}.$$

DISEÑO DEL ESPOLON

Al observar las diferentes hipótesis de carga para determinar el factor de seguridad al deslizamiento vemos que la hipótesis del grupo I (estribo sin cargas) es la que presenta el menor factor de seguridad, por lo tanto el espolón se diseñará para esta hipótesis.



$$\frac{1}{2} \times A1 + \text{Tang } \phi \times A2 + \frac{1}{2} \times \gamma \times K_p \times h^2 = 1.5 E$$

$$A1 = (4.0 + 11.25) \times 3.34 / 2 = 25.48$$

$$A2 = (11.25 + 14.86) \times 1.66 / 2 = 21.67$$

$$Kp = 3$$

$$\tan 30^\circ = 0.58$$

$$(0.5 \times 25.48) + (0.58 \times 21.67) + (0.5 \times 1.8 \times 3 \times h^2) = (1.5 \times 21.11)$$

$$h = 1.53\text{m}$$

$$\sigma_{Ep1} = \gamma \times h \times Kp = 1.8\text{T/m}^3 \times 1.53\text{m} \times 3 = 8.26\text{T/m}^2$$

$$\sigma_{Ep2} = 1.8\text{T/m}^3 \times 0.65\text{m} \times 3 = 3.51\text{ T/m}^2$$

$$Mu = 1.6 \times (((4.75 \times 0.88^2 \times (2/3)/2) + (3.51 \times (0.88^2)/2)) = 4.14\text{T-m}$$

$$F'_c = 280\text{Kg/cm}^2 \quad F_y = 4200\text{Kg/cm}^2 \quad h = 0.81\text{Cm} \quad d = 74\text{cm}$$

$$b = 100\text{cm} \quad d' = 7\text{cm} \quad K = Mu/bxd^2 \quad K = 4.14\text{-m}/(1\text{m} \times (74\text{cm})^2)$$

$$K = 0.0008 / \text{cm}^2 \quad \rho = 0.0002 \quad As = 100 \times 74 \times 0.0002 \quad As = 1.48\text{Cm}^2$$

$$S = 100 \times (1.27\text{Cm}^2/1.48\text{Cm}^2) \quad S = 86\text{Cm}, \text{ Colocar una } \varnothing 6/8'' \text{ c/u } 30\text{cm}.$$



BIBLIOGRAFIA

TRUJILLO OROZCO. José Eusebio. Diseño de Puentes. Universidad Industrial de Santander. Ediciones UIS. 2da Edición 1993. pág. 111 – 120, 177 – 191.

