ÍNDICE							
Categoría	**Número de Documento**	**Fecha**	**Descripción**	**Observaciones**	**Tomo**	**N° de Folio Inicio**	**N° de Folio final**
Estructuras			Continuación Estructuras Informe estudio y diseño de estructuras de puentes		42	8201	8400

Continuación Estructuras Informe estudio y diseño de estructuras de puentes

$$\frac{}{\Phi b_v d_v}$$ *(Eq. 5.8.2.9-1)*

- Maximum spacing for transverse reinforcement (s_{max})

v_u = 0.41 (MPa) < $0.125 \cdot f'_c$ = 4.31 (MPa)

$\therefore$ s_{max} = Min[$0.8 d_v$, 24.0(mm)] = 609.6 (mm) *(Eq. 5.8.2.7-1)*

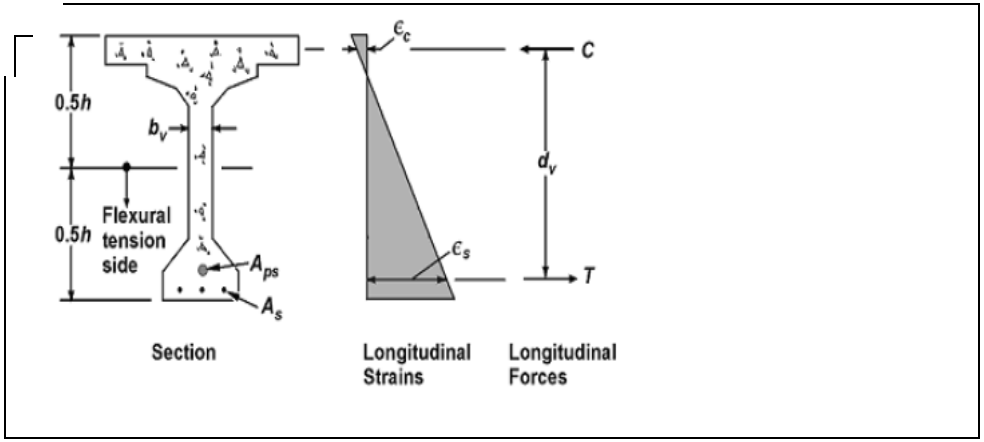
$\therefore$ s = 100.0 (mm) ≤ s_{max} **OK**

3) Minimum required transverse reinforcement ($A_{v,min}$)

$$A_{v,min} = 0.0316 \sqrt{f'_c} \frac{b_v s}{f_y} = 25.9 \ (mm^2)$$ *(Eq. 5.8.2.5-1)*

$\therefore$ A_v = 258.064 (mm^2) ≥ $A_{v,min}$ * Defining "OK" or "NG" will be checked in checking Shear resistance by transverse reinforcement

4) Longitudinal Strain(ε_s)



$$\varepsilon_s = \frac{(|M_u|/d_v + 0.5N_u + |V_u - V_p| - A_{ps}f_{po})}{E_s A_s + E_p A_{ps}} = -0.00312 \ , \ 0.0 \le \varepsilon_s \le 0.006$$ *(Eq. 5.8.3.4.2-4)*

$\therefore \quad \varepsilon_s = 0.00000$

where, $|M_u| = \text{Max}[\, M_u \,,\ |V_u - V_p\, d_v \,] = \text{Max}[\ 3850.81\ ,\ 207.46\ ]$

$= 3850.812$ (kN•m)

$f_{po} = 0.7 f_{pu} = 1304.28$ (MPa)

5) Component of shear resisted by tensile stresses in concrete (V_c)

- Factor indicating diagonally cracked concrete to transmit tension (β).

$$\beta = \frac{4.8}{(1+750\varepsilon_s)} = 4.800 \quad \because \quad A_v \geq A_{v,min}$$ *(Eq. 5.8.3.4.2-1)*

- Nominal shear resistance provide by concrete

$V_c = 0.0316\beta \sqrt{f'_c}\, b_v d_v = 826.71$ (kN) *(Eq. 5.8.3.3-3)*

6) Component of shear resisted by transverse reinforcement (V_s)

- Area of transverse reinforcement required *(See 5.8.2.4)*

$0.5\Phi(V_c+V_p)$	$\Phi(V_c+V_p)$	V_u
383.37 kN	766.75 kN	154.34 kN

$V_u < 0.5\Phi(V_c+V_p) \quad \therefore$ No shear reinforcing

- Component of shear resisted by transverse reinforcement (V_s)

$$V_s = \frac{A_v \cdot f_y \cdot d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} = 3094.87 \text{ (kN)}$$ *(Eq. 5.8.3.3-4)*

where, $\theta = 29+3500\varepsilon_s = 29.000$ (deg.) *(Eq. 5.8.3.4.2-3)*

$\alpha = 90.000$ (deg.)

7) Shear resistance.

- Nominal shear strength *(Eq. 5.8.3.3.-1)*

$V_c + V_s + V_p = 3946.82$ (kN) $>$ $0.25 \cdot f'_c b_v d_v + V_p = 3072.07$ (kN)

$\therefore \quad V_n = 0.25 \cdot f'_c b_v d_v + V_p = 3072.07$ (kN)

- Factored shear resistance (V_r).

$\therefore$ $V_r = \Phi V_n = 2764.86$ (kN) $\geq V_u = 154.34$ (kN) **OK**

8) Interface Shear

- Nominal interface shear resistance (V_{ni})

$V_{ni} = c \cdot A_{cv} + \mu(A_{vf} \cdot f_y + P_c) = 1120.47$ (kN) *(Eq. 5.8.4.1-3)*

The nominal shear resistance, Vni, used in the design shall not be greater than the lesser of:

$V_{ni} \leq K1 \cdot f`c \cdot Acv = 224.86$ (kN) *(Eq. 5.8.4.1-4)*

$V_{ni} \leq K2 \cdot Acv = 337.29$ (kN) *(Eq. 5.8.4.1-5)*

$\therefore$ $V_{ni} = 224.86$ (kN)

Cohesion and Friction Factors (Case) = Roughened(CIP slab) *(Eq. 5.8.4.3)*

where,

c	=	1.93	(MPa)
μ	=	1.00	
K_1	=	0.30	
K_2	=	12.41	(MPa)
A_{cv}	=	27178.00	(mm^2)
A_{vf}	=	2580.64	(mm^2)
F_y	=	413.69	(MPa)
P_c	=	0.42	(kN)

- Factored Interface shear force (V_{ui}) *(Eq. 5.8.4.2-2)*

$\therefore$ $V_{ui} = V_u / b_{vi} \cdot d_v = 0.06$ (kN)

where,

V_u	=	154.34	(kN)
b_{vi}	=	1070.00	(mm)
d_v	=	1606.93	(mm) (kN)

- Factored Interface shear resistance (V_{ri}) *(Eq. 5.8.4.1-2)*

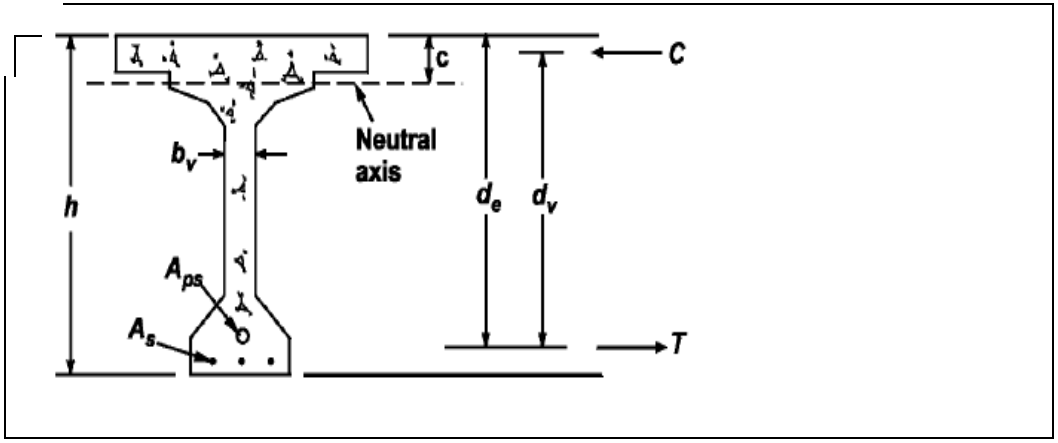
$\therefore$ $V_{ri} = \Phi V_{ni} = 224.86$ (kN) $\geq V_{ui} = 0.06$ (kN) **OK**

where, Φ = 1.0 *(Eq. 5.7.2.1)*

■ **Minimum Shear**

- Section type : Nonsegmental-Solid
- The Strength Limit Load Combination : cLCB1
- The factored Shear force : V_u = -37.54 (kN)
- Factored moment : M_u = 2129.67 (kN•m)
- Factored axial force : N_u = -3251.77 (kN)
- Resistance factor for shear : Φ = 0.90
- Component of prestressing force in direction of the shear force : $V_p = \Sigma A_{ps} \cdot f_{e(z\text{-}dir)}$ = 25.23 (kN)

1) Effective depth



b_v = 220.0 (mm) : Effective web width

$d_e = \dfrac{A_{ps}f_{ps}d_p + A_s f_s d_s}{A_{ps}f_{ps} + A_s f_s}$ = 1658.4 (mm) : Effective depth for Bending

d_v = Max [$\dfrac{M_n}{A_{ps}f_{ps} + A_s f_s}$, $0.9d_e$, $0.72h$] : Effective depth for shear

= Max [1606.9 , 1492.6 , 1375.2]

= 1606.931 (mm)

2) Maximum spacing for transverse reinforcement (s_{max})

- Shear stress on concrete (v_u).

$$v_u = \frac{|V_u - \Phi V_p|}{\Phi b_v d_v} = 0.19 \text{ (MPa)} \qquad \textit{(Eq. 5.8.2.9-1)}$$

- Maximum spacing for transverse reinforcement (s_{max})

v_u = 0.19 (MPa) < $0.125 \cdot f'_c$ = 4.31 (MPa)

$\therefore$ s_{max} = Min[$0.8d_v$, 24.0(mm)] = 609.6 (mm) *(Eq. 5.8.2.7-1)*

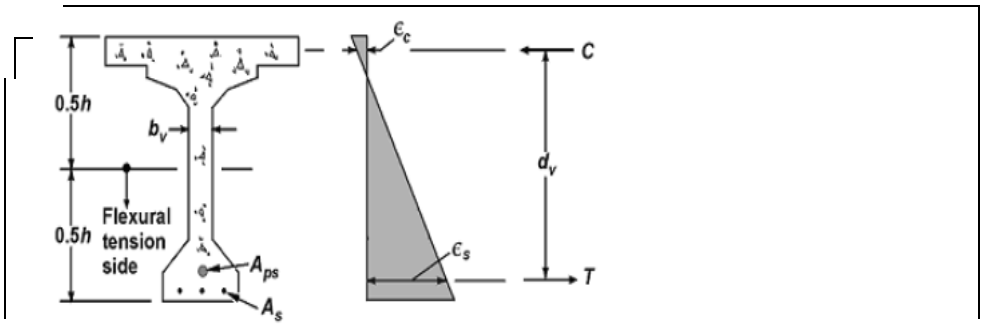
$\therefore$ s = 100.0 (mm) ≤ s_{max} **OK**

3) Minimum required transverse reinforcement ($A_{v,min}$)

$$A_{v,min} = 0.0316 \sqrt{f'_c} \frac{b_v s}{f_y} = 25.9 \text{ (mm}^2\text{)} \qquad \textit{(Eq. 5.8.2.5-1)}$$

$\therefore$ A_v = 258.064 (mm^2) ≥ $A_{v,min}$ * Defining "OK" or "NG" will be checked in checking Shear resistance by transverse reinforcement

4) Longitudinal Strain(ε_s)



Section Longitudinal Strains Longitudinal Forces

$$\varepsilon_s = \frac{(|M_u|/d_v + 0.5N_u + |V_u - V_p| - A_{ps}f_{po})}{E_sA_s + E_pA_{ps}} = -0.00435 \quad , \quad 0.0 \le \varepsilon_s \le 0.006 \qquad \textit{(Eq. 5.8.3.4.2-4)}$$

$\therefore \quad \varepsilon_s = 0.00000$

where, $|M_u| = \text{Max}[\, M_u \,,\ |V_u - V_p\, d_v\,] = \text{Max}[\ 2129.67\ ,\ 100.88\ ]$

$= 2129.667 \ \text{(kN•m)}$

$f_{po} = 0.7f_{pu} = 1304.28 \ \text{(MPa)}$

5) Component of shear resisted by tensile stresses in concrete (V_c)

- Factor indicating diagonally cracked concrete to transmit tension (β).

$$\beta = \frac{4.8}{(1+750\varepsilon_s)} = 4.800 \quad \because \quad A_v \ge A_{v,min} \qquad \textit{(Eq. 5.8.3.4.2-1)}$$

- Nominal shear resistance provide by concrete

$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d_v = 826.71 \ \text{(kN)}$ *(Eq. 5.8.3.3-3)*

6) Component of shear resisted by transverse reinforcement (V_s)

- Area of transverse reinforcement required *(See 5.8.2.4)*

$0.5\Phi(V_c+V_p)$	$\Phi(V_c+V_p)$	V_u
383.37 kN	766.75 kN	37.54 kN

$V_u < 0.5\Phi(V_c+V_p) \quad \therefore$ No shear reinforcing

- Component of shear resisted by transverse reinforcement (V_s)

$$V_s = \frac{A_v \cdot f_y \cdot d_v(\cot\theta + \cot\alpha)\sin\alpha}{s} = 3094.87 \ \text{(kN)} \qquad \textit{(Eq. 5.8.3.3-4)}$$

where, $\theta = 29+3500\varepsilon_s = 29.000 \ \text{(deg.)}$ *(Eq. 5.8.3.4.2-3)*

α = 90.000 (deg.)

7) Shear resistance.

- Nominal shear strength *(Eq. 5.8.3.3.-1)*

$V_c + V_s + V_p$ = 3946.82 (kN) > $0.25 \cdot f'_c b_v d_v + V_p$ = 3072.07 (kN)

$\therefore$ V_n = $0.25 \cdot f'_c b_v d_v + V_p$ = 3072.07 (kN)

- Factored shear resistance (V_r).

$\therefore$ V_r = ΦV_n = | 2764.86 | (kN) ≥ V_u = | -37.54 | (kN) **OK**

8) Interface Shear

- Nominal interface shear resistance (V_{ni})

V_{ni} = $c \cdot A_{cv} + \mu(A_{vf} \cdot f_y + P_c)$ = 1120.47 (kN) *(Eq. 5.8.4.1-3)*

The nominal shear resistance, Vni, used in the design shall not be greater than the lesser of:

$V_{ni} \leq K1 \cdot f`c \cdot Acv$ = 224.86 (kN) *(Eq. 5.8.4.1-4)*

$V_{ni} \leq K2 \cdot Acv$ = 337.29 (kN) *(Eq. 5.8.4.1-5)*

$\therefore$ V_{ni} = 224.86 (kN)

Cohesion and Friction Factors (Case) = Roughened(CIP slab) *(Eq. 5.8.4.3)*

where,

c	=	1.93	(MPa)
μ	=	1.00	
K_1	=	0.30	
K_2	=	12.41	(MPa)
A_{cv}	=	27178.00	(mm^2)
A_{vf}	=	2580.64	(mm^2)
F_y	=	413.69	(MPa)
P_c	=	0.42	(kN)

- Factored Interface shear force (V_{ui}) *(Eq. 5.8.4.2-2)*

$\therefore$ V_{ui} = V_u / $b_{vi} \cdot d_v$ = -0.01 (kN)

where, V_u = -37.54 (kN)
b_{vi} = 1070.00 (mm)
d_v = 1606.93 (mm) (kN)

▪ Factored Interface shear resistance (V_{ri}) *(Eq. 5.8.4.1-2)*

∴ $V_{ri} = \Phi V_{ni}$ = 224.86 (kN) ≥ V_{ui} = 0.01 (kN) **OK**

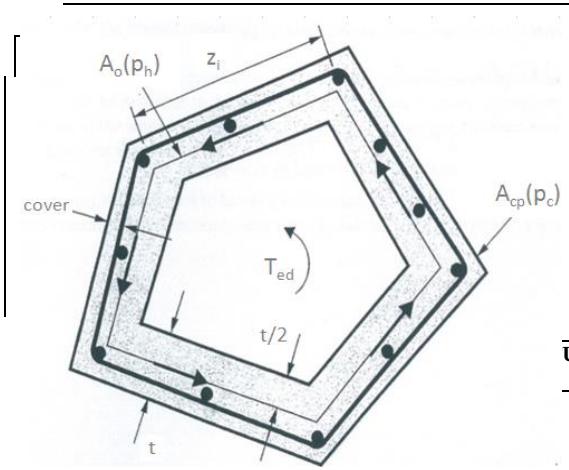
where, Φ = 1.0 *(Eq. 5.7.2.1)*

4.Torsional design for a section

■ **Case of $V_{max.}$**

- Section type : Nonsegmental-Solid
- The Strength Limit Load Combination : cLCB1
- The factored Torsional moment : T_u = 18.70 (kN•m)
- The factored Shear force : V_u = 154.34 (kN)
- Factored moment : M_u = 3850.81 (kN•m)
- Factored axial force : N_u = -3230.52 (kN)
- Resistance factor for shear : Φ = 0.90
- Component of prestressing force in direction of the shear force : $V_p = \Sigma A_{ps} \cdot f_{e(z\text{-}dir)}$ = 25.23 (kN)

1) Notation



A_o = Area enclosed by the shear flow path, including any area of holes therein
= 1.386E+06 (mm^2)
p_h = Perimeter of the centerline of the closed tranverse torsion reinforcement.
= 11532.3 (mm)
A_{cp} = Total area enclosed by outside

Perimeter of the concrete section.

= 1.386E+06 (mm^2)

p_c = The length of the outside perimeter of concrete section.

= 387.1 (mm)

2) Checking Torsional Effects

- Torsional cracking moment (T_{cr}).

$$T_{cr} = 0.125\cdot\sqrt{f'_c}\ \frac{A_{cp}^2}{p_c}\left(1+\frac{f_{pc}}{0.125\sqrt{f'_c}}\right)^{(1/2)} = 569.17 \text{ (kN•m)} \quad \textit{(Eq. 5.8.2.1-4)}$$

where,

A_{cp}^2/p_c = 160441577 (mm^3)

f_{pc} = *Compressive stress in concrete due to effective prestress only.*

$$= \frac{\Sigma A_{ps}\cdot f_{e(x\text{-}dir)}}{A_g} + \frac{\Sigma A_{ps}\cdot f_{e(x\text{-}dir)}\cdot e_p\cdot y_{joint}}{I_y} = 2.48 \text{ (MPa)}$$

T_u = | 18.70 | (kN•m) ≤ $0.25\Phi T_{cr}$ = | 128.06 | (kN•m) *(Eq. 5.8.2.1-3)*

∴ $T_u \le 0.25\Phi T_{cr}$, Ignore Torsional Effects.

■ **Case of $V_{min.}$**

- Section type : Nonsegmental-Solid
- The Strength Limit Load Combination : cLCB1
- The factored Torsional moment : T_u = -46.57 (kN•m)
- The factored Shear force : V_u = -37.54 (kN)
- Factored moment : M_u = 2129.67 (kN•m)
- Factored axial force : N_u = -3251.77 (kN)
- Resistance factor for shear : Φ = 0.90
- Component of prestressing force in direction of the shear force : $V_p = \Sigma A_{ps}\cdot f_{e(z\text{-}dir)}$ = 25.23 (kN)

1) Checking Torsional Effects

- Torsional cracking moment (T_{cr}).

$$T_{cr} = 0.125\cdot\sqrt{f'_c}\ \frac{A_{cp}^2}{p_c}\left(1+\frac{f_{pc}}{0.125\sqrt{f'_c}}\right)^{(1/2)} = 569.17\ (kN\bullet m) \qquad \textit{(Eq. 5.8.2.1-4)}$$

where,

A_{cp}^2/p_c = 160441577 (mm^3)

f_{pc} = *Compressive stress in concrete due to effective prestress only.*

$$= \frac{\Sigma A_{ps}\cdot f_{e(x\text{-}dir)}}{A_g} + \frac{\Sigma A_{ps}\cdot f_{e(x\text{-}dir)}\cdot e_p\cdot y_{joint}}{I_y} = 2.48\ (MPa)$$

$$T_u = |-46.57|\ (kN\bullet m) \le 0.25\Phi T_{cr} = |128.06|\ (kN\bullet m) \qquad \textit{(Eq. 5.8.2.1-3)}$$

$\therefore$ $T_u \le 0.25\Phi T_{cr}$, Ignore Torsional Effects.

■ **Case of T_{max}.**

- Section type : Nonsegmental-Solid
- The Strength Limit Load Combination : cLCB1
- The factored Torsional moment : T_u = -46.57 (kN•m)
- The factored Shear force : V_u = -37.54 (kN)
- Factored moment : M_u = 2129.67 (kN•m)
- Factored axial force : N_u = -3251.77 (kN)
- Resistance factor for shear : Φ = 0.90
- Component of prestressing force in direction of the shear force : $V_p = \Sigma A_{ps}\cdot f_{e(z\text{-}dir)}$ = 25.23 (kN)

1) Checking Torsional Effects

- Torsional cracking moment (T_{cr}).

$$T_{cr} = 0.125\cdot\sqrt{f'_c}\ \frac{A_{cp}^2}{p_c}\left(1+\frac{f_{pc}}{0.125\sqrt{f'_c}}\right)^{(1/2)} = 569.17\ (kN\bullet m) \qquad \textit{(Eq. 5.8.2.1-4)}$$

where,

A_{cp}^2/p_c = 160441577 (mm^3)

f_{pc} = *Compressive stress in concrete due to effective prestress only.*

$$= \frac{\Sigma A_{ps} \cdot f_{e(x\text{-}dir)}}{A_g} + \frac{\Sigma A_{ps} \cdot f_{e(x\text{-}dir)} \cdot e_p \cdot y_{joint}}{I_y} = 2.48 \text{ (MPa)}$$

T_u = | -46.57 | (kN•m) ≤ $0.25\Phi T_{cr}$ = | 128.06 | (kN•m) *(Eq. 5.8.2.1-3)*

∴ $T_u \le 0.25\Phi T_{cr}$, Ignore Torsional Effects.

5.Crack check

■ **Bottom**

- The maximum Service Limit Load Combination : cLCB20
- The maximum spacing of steel reinforcement in layer closet to the tension face(s_{max}) *(Eq. 5.7.3.4-1)*

$$s_{max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2d_c = 4611.0 \text{ (mm)} \ge s_{use} = 152.5 \text{ (mm)}$$ **OK**

where,

$$\beta_s = 1 + \frac{d_c}{0.7(h-d_c)} = 1.038$$

d_c = 50.0 (mm)

γ_e = 1.00 : Exposure factor

f_{ss} = 25.060 (MPa) : Tensile stress in bar at the service limit state

3.4.15.1. VERIFICACIÓN POR MOMENTO FLECTOR

Tabla 16. Verificación de capacidad de diseño a flexión positiva

Elem	Positive/ Negative	Muy (tonf*m)	Mcr (tonf*m)	Mny (tonf*m)	PhiMny (tonf*m)	Ratio (Muy/PhiMny)	PhiMny/ min(1.33Muy,Mcr)	CHK
1	Negative	-110.76	392.79	345.21	345.21	0.32	2.34	OK
2	Negative	-48.38	335.32	300.00	300.00	0.16	4.66	OK
3	Negative	0.00	281.90	257.58	257.58	0.00	larger than 100	OK
4	Negative	0.00	233.10	219.03	219.03	0.00	larger than 100	OK
5	Negative	0.00	189.50	185.38	185.38	0.00	larger than 100	OK
6	Negative	0.00	151.66	157.67	157.67	0.00	larger than 100	OK
7	Negative	0.00	120.42	136.69	136.69	0.00	larger than 100	OK
8	Negative	0.00	96.08	122.64	122.64	0.00	larger than 100	OK
9	Negative	0.00	79.23	114.98	114.98	0.00	larger than 100	OK
10	Negative	0.00	70.34	112.19	112.19	0.00	larger than 100	OK
11	Negative	0.00	70.55	112.19	112.19	0.00	larger than 100	OK
12	Negative	0.00	79.66	114.98	114.98	0.00	larger than 100	OK
13	Negative	0.00	96.63	122.64	122.64	0.00	larger than 100	OK
14	Negative	0.00	120.99	136.69	136.69	0.00	larger than 100	OK
15	Negative	0.00	152.11	157.67	157.67	0.00	larger than 100	OK
16	Negative	0.00	189.53	185.38	185.38	0.00	larger than 100	OK
17	Negative	0.00	232.68	219.03	219.03	0.00	larger than 100	OK
18	Negative	0.00	281.05	257.58	257.58	0.00	larger than 100	OK
19	Negative	-52.08	334.08	300.00	300.00	0.17	4.33	OK
20	Negative	-116.00	391.19	345.21	345.21	0.34	2.24	OK

Elem	Positive/ Negative	Muy (tonf*m)	Mcr (tonf*m)	Mny (tonf*m)	PhiMny (tonf*m)	Ratio (Muy/PhiMny)	PhiMny/ min(1.33Muy,Mcr)	CHK
21	Negative	-192.08	451.77	392.01	392.01	0.49	1.53	OK

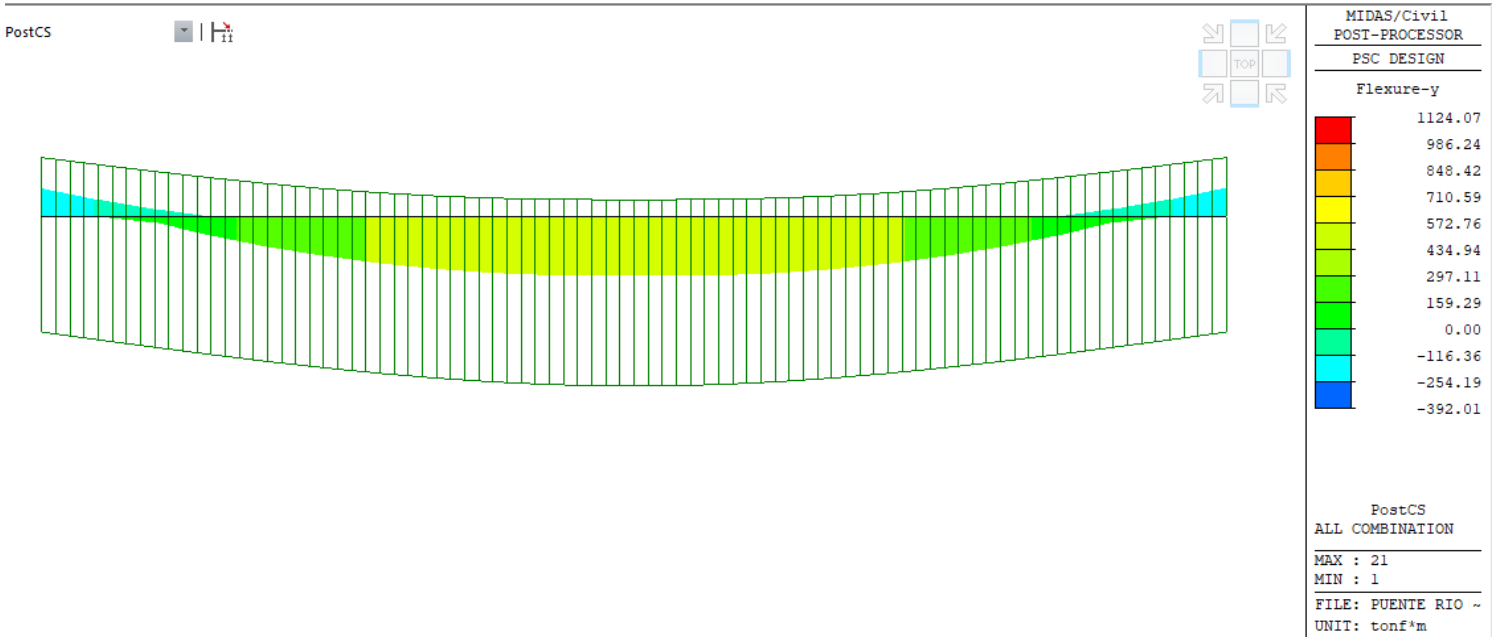


Figura 168. Verificación de capacidad a diseño de flexión positiva

3.4.15.2. VERIFICACIÓN POR FUERZA CORTANTE

Tabla 17. Verificación de capacidad de diseño a fuerza cortante

Elem	Max/Min	CHK	Vu (tonf)	Mu (tonf*m)	Vn (tonf)	Phi	Vc (tonf)	Vs (tonf)	Vp (tonf)	PhiVn (tonf)	de (m)	dv (m)	theta ([deg])	beta	Avs (m^2)	Ast (m^2)	Al (m^2)	bv (m)	Vri (tonf)	Vui (tonf)	Interface Shear(CHK)
1	Max	OK	-71.40	-110.76	294.40	0.90	49.74	248.15	28.52	264.96	0.91	1.38	31.10	3.31	0.00	0.00	0.00	0.22	22.93	-0.03	OK
1	Min	OK	-118.96	-95.57	232.05	0.90	22.15	181.38	28.52	208.84	0.91	1.38	39.54	1.47	0.00	0.00	0.00	0.22	22.93	-0.05	OK
2	Max	OK	-60.77	-48.38	293.68	0.90	67.97	266.92	27.79	264.31	0.82	1.38	29.29	4.52	0.00	0.00	0.00	0.22	22.93	-0.03	OK
2	Min	OK	-102.64	-23.51	253.64	0.90	26.71	199.13	27.79	228.28	0.82	1.38	36.94	1.78	0.00	0.00	0.00	0.22	22.93	-0.05	OK
3	Max	OK	-50.67	4.20	292.45	0.90	72.14	270.08	26.56	263.20	1.37	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.02	OK
3	Min	OK	-88.07	37.31	292.45	0.90	72.14	270.08	26.56	263.20	1.37	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.04	OK
4	Max	OK	-41.46	47.55	291.30	0.90	72.34	270.81	24.69	262.17	1.43	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.02	OK
4	Min	OK	-74.49	87.62	291.30	0.90	72.34	270.81	24.69	262.17	1.43	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.03	OK
5	Max	OK	-32.84	82.45	300.46	0.90	75.42	282.34	22.51	270.42	1.49	1.44	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.01	OK
5	Min	OK	-61.66	128.31	300.46	0.90	75.42	282.34	22.51	270.42	1.49	1.44	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.03	OK
6	Max	OK	-25.35	109.52	307.55	0.90	78.15	292.58	19.51	276.80	1.54	1.49	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.01	OK
6	Min	OK	-50.10	160.07	307.55	0.90	78.15	292.58	19.51	276.80	1.54	1.49	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.02	OK
7	Max	OK	-16.45	347.88	312.89	0.90	80.48	301.29	16.28	281.60	1.59	1.53	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.01	OK
7	Min	OK	-39.17	184.51	312.89	0.90	80.48	301.29	16.28	281.60	1.59	1.53	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.02	OK
8	Max	OK	-8.15	373.77	315.53	0.90	82.32	308.19	12.12	283.98	1.62	1.57	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
8	Min	OK	-29.61	201.57	315.53	0.90	82.32	308.19	12.12	283.98	1.62	1.57	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.01	OK
9	Max	OK	0.01	388.32	316.06	0.90	83.62	313.04	7.87	284.45	1.65	1.59	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
9	Min	OK	-20.37	212.07	316.06	0.90	83.62	313.04	7.87	284.45	1.65	1.59	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.01	OK
10	Max	OK	6.99	391.99	313.27	0.90	84.30	315.59	2.58	281.94	1.66	1.61	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
10	Min	OK	-12.58	216.52	313.27	0.90	84.30	315.59	2.58	281.94	1.66	1.61	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
11	Max	OK	14.17	392.62	313.26	0.90	84.30	315.59	2.57	281.94	1.66	1.61	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.01	OK
11	Min	OK	-5.00	216.99	313.26	0.90	84.30	315.59	2.57	281.94	1.66	1.61	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
12	Max	OK	21.63	389.73	316.02	0.90	83.62	313.04	7.83	284.42	1.65	1.59	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.01	OK
12	Min	OK	2.06	213.02	316.02	0.90	83.62	313.04	7.83	284.42	1.65	1.59	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK

Elem	Max/Min	CHK	Vu (tonf)	Mu (tonf*m)	Vn (tonf)	Phi	Vc (tonf)	Vs (tonf)	Vp (tonf)	PhiVn (tonf)	de (m)	dv (m)	theta ([deg])	beta	Avs (m^2)	Ast (m^2)	Al (m^2)	bv (m)	Vri (tonf)	Vui (tonf)	Interface Shear(CHK)
13	Max	OK	30.67	375.68	315.44	0.90	82.32	308.19	12.04	283.90	1.62	1.57	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.01	OK
13	Min	OK	10.29	202.72	315.44	0.90	82.32	308.19	12.04	283.90	1.62	1.57	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
14	Max	OK	40.03	350.02	312.74	0.90	80.48	301.29	16.13	281.46	1.59	1.53	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.02	OK
14	Min	OK	18.40	173.48	312.74	0.90	80.48	301.29	16.13	281.46	1.59	1.53	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.01	OK
15	Max	OK	50.85	311.90	307.33	0.90	78.15	292.58	19.29	276.60	1.54	1.49	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.02	OK
15	Min	OK	25.47	111.78	307.33	0.90	78.15	292.58	19.29	276.60	1.54	1.49	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.01	OK
16	Max	OK	62.08	261.81	300.21	0.90	75.42	282.34	22.25	270.19	1.49	1.44	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.03	OK
16	Min	OK	32.85	83.73	300.21	0.90	75.42	282.34	22.25	270.19	1.49	1.44	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.01	OK
17	Max	OK	74.58	200.39	291.03	0.90	72.34	270.81	24.42	261.93	1.43	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.03	OK
17	Min	OK	41.30	47.59	291.03	0.90	72.34	270.81	24.42	261.93	1.43	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.02	OK
18	Max	OK	87.79	124.93	292.16	0.90	72.14	270.08	26.27	262.95	1.37	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.04	OK
18	Min	OK	50.30	2.82	292.16	0.90	72.14	270.08	26.27	262.95	1.37	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.02	OK
19	Max	OK	102.08	34.91	293.39	0.90	72.14	270.08	27.50	264.05	1.30	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.04	OK
19	Min	OK	60.23	-51.29	293.39	0.90	72.14	270.08	27.50	264.05	0.82	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.03	OK
20	Max	OK	117.23	-69.99	294.12	0.90	63.40	263.06	28.23	264.71	0.91	1.38	29.64	4.22	0.00	0.00	0.00	0.22	22.93	0.05	OK
20	Min	OK	70.91	-115.29	294.12	0.90	72.14	270.08	28.23	264.71	0.91	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.03	OK
21	Max	OK	133.30	-190.04	274.10	0.90	31.64	214.01	28.45	246.69	1.01	1.38	34.97	2.11	0.00	0.00	0.00	0.22	22.93	0.06	OK
21	Min	OK	82.11	-189.61	294.34	0.90	55.46	255.13	28.45	264.91	1.01	1.38	30.40	3.69	0.00	0.00	0.00	0.22	22.93	0.04	OK

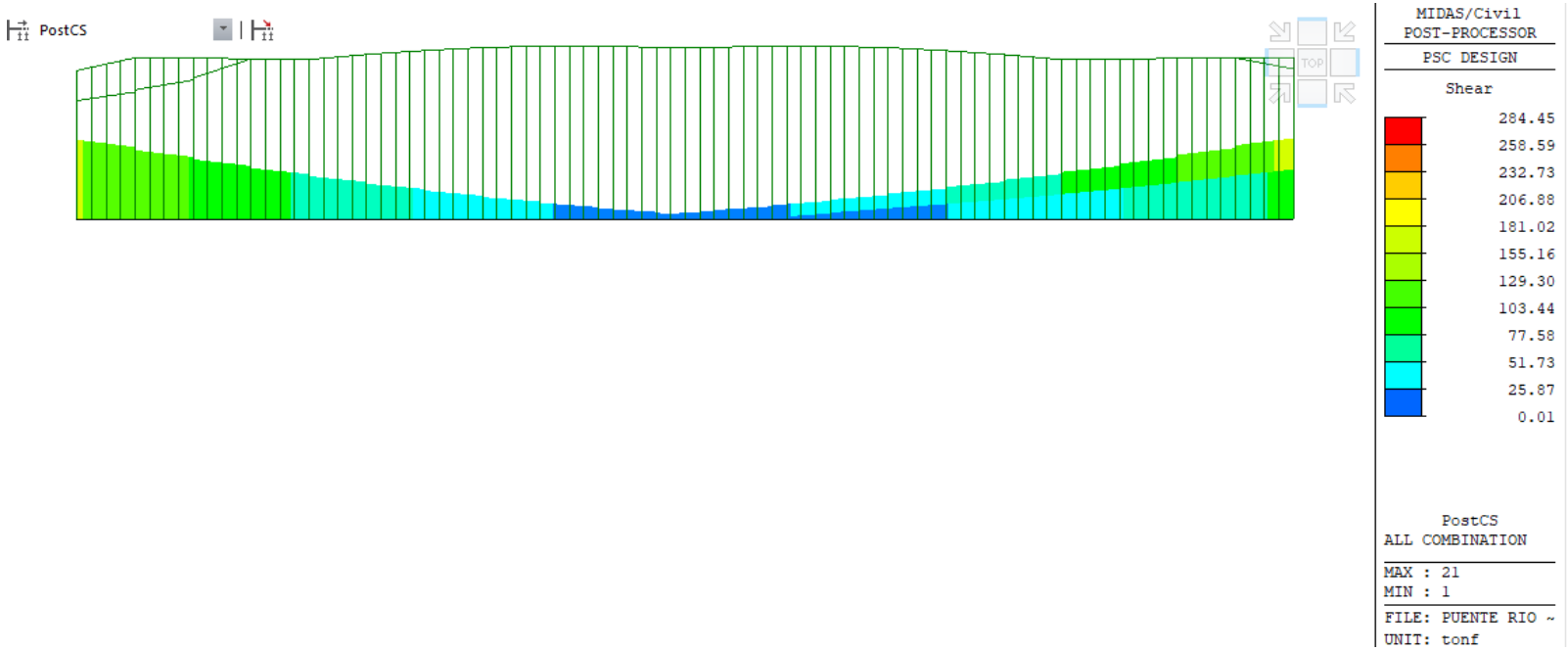


Figura 169. Verificación de capacidad a fuerza cortante

3.4.15.3. VERIFICACIÓN POR CORTANTE Y TORSIÓN COMBINADA

Tabla 18. Verificación de capacidad de diseño a cortante y torsión combinada

Elem	Max/Min	Vu (tonf)	Mu (tonf*m)	Tu (tonf*m)	Vn (tonf)	Tn (tonf*m)	Phi	Phi_t	Vc (tonf)	Vs (tonf)	Vp (tonf)	PhiVn (tonf)	Phi_tTn (tonf*m)	de (m)	dv (m)	theta ([deg])	beta	bv (m)	bv_min (m)	CHK
1	T-Max	-118.96	-95.57	-24.42	232.05	365.60	0.90	0.90	22.15	181.38	28.52	208.84	329.04	0.91	1.38	39.54	1.47	0.22	0.13	OK
1	V-Max	-71.40	-110.76	-11.50	294.40	500.18	0.90	0.90	49.74	248.15	28.52	264.96	450.16	0.91	1.38	31.10	3.31	0.22	0.09	OK
1	V-Min	-118.96	-95.57	-24.42	232.05	365.60	0.90	0.90	22.15	181.38	28.52	208.84	329.04	0.91	1.38	39.54	1.47	0.22	0.13	OK
2	T-Max	-102.64	-23.51	-6.37	253.64	401.38	0.90	0.90	26.71	199.13	27.79	228.28	361.24	0.82	1.38	36.94	1.78	0.22	0.12	OK
2	V-Max	-60.77	-48.38	-1.53	293.68	538.01	0.90	0.90	67.97	266.92	27.79	264.31	484.21	0.82	1.38	29.29	4.52	0.22	0.08	OK
2	V-Min	-102.64	-23.51	-6.37	253.64	401.38	0.90	0.90	26.71	199.13	27.79	228.28	361.24	0.82	1.38	36.94	1.78	0.22	0.12	OK
3	T-Max	-58.60	127.83	11.17	292.45	544.38	0.90	0.90	72.14	270.08	26.56	263.20	489.94	1.37	1.38	29.00	4.80	0.22	0.08	OK
3	V-Max	-50.67	4.20	1.80	292.45	544.38	0.90	0.90	72.14	270.08	26.56	263.20	489.94	1.37	1.38	29.00	4.80	0.22	0.07	OK
3	V-Min	-88.07	37.31	0.22	292.45	544.38	0.90	0.90	72.14	270.08	26.56	263.20	489.94	1.37	1.38	29.00	4.80	0.22	0.10	OK
4	T-Max	-46.85	201.33	12.30	291.30	544.38	0.90	0.90	72.34	270.81	24.69	262.17	489.94	1.43	1.38	29.00	4.80	0.22	0.06	OK
4	V-Max	-41.46	47.55	2.59	291.30	544.38	0.90	0.90	72.34	270.81	24.69	262.17	489.94	1.43	1.38	29.00	4.80	0.22	0.06	OK
4	V-Min	-74.49	87.62	1.78	291.30	544.38	0.90	0.90	72.34	270.81	24.69	262.17	489.94	1.43	1.38	29.00	4.80	0.22	0.09	OK
5	T-Max	-35.81	261.23	11.33	300.46	544.38	0.90	0.90	75.42	282.34	22.51	270.42	489.94	1.49	1.44	29.00	4.80	0.22	0.05	OK
5	V-Max	-32.84	82.45	2.41	300.46	544.38	0.90	0.90	75.42	282.34	22.51	270.42	489.94	1.49	1.44	29.00	4.80	0.22	0.05	OK
5	V-Min	-61.66	128.31	1.42	300.46	544.38	0.90	0.90	75.42	282.34	22.51	270.42	489.94	1.49	1.44	29.00	4.80	0.22	0.07	OK
6	T-Max	-25.88	309.97	9.74	307.55	544.38	0.90	0.90	78.15	292.58	19.51	276.80	489.94	1.54	1.49	29.00	4.80	0.22	0.04	OK
6	V-Max	-25.35	109.52	1.96	307.55	544.38	0.90	0.90	78.15	292.58	19.51	276.80	489.94	1.54	1.49	29.00	4.80	0.22	0.04	OK
6	V-Min	-50.10	160.07	0.48	307.55	544.38	0.90	0.90	78.15	292.58	19.51	276.80	489.94	1.54	1.49	29.00	4.80	0.22	0.06	OK
7	T-Max	-16.45	347.88	8.14	312.89	544.38	0.90	0.90	80.48	301.29	16.28	281.60	489.94	1.59	1.53	29.00	4.80	0.22	0.03	OK
7	V-Max	-16.45	347.88	8.14	312.89	544.38	0.90	0.90	80.48	301.29	16.28	281.60	489.94	1.59	1.53	29.00	4.80	0.22	0.03	OK
7	V-Min	-39.17	184.51	-0.48	312.89	544.38	0.90	0.90	80.48	301.29	16.28	281.60	489.94	1.59	1.53	29.00	4.80	0.22	0.04	OK
8	T-Max	-8.15	373.77	6.74	315.53	544.38	0.90	0.90	82.32	308.19	12.12	283.98	489.94	1.62	1.57	29.00	4.80	0.22	0.02	OK
8	V-Max	-8.15	373.77	6.74	315.53	544.38	0.90	0.90	82.32	308.19	12.12	283.98	489.94	1.62	1.57	29.00	4.80	0.22	0.02	OK
8	V-Min	-29.61	201.57	-1.23	315.53	544.38	0.90	0.90	82.32	308.19	12.12	283.98	489.94	1.62	1.57	29.00	4.80	0.22	0.03	OK
9	T-Max	0.01	388.32	5.52	316.06	544.38	0.90	0.90	83.62	313.04	7.87	284.45	489.94	1.65	1.59	29.00	4.80	0.22	0.01	OK

Elem	Max/Min	Vu (tonf)	Mu (tonf*m)	Tu (tonf*m)	Vn (tonf)	Tn (tonf*m)	Phi	Phi_t	Vc (tonf)	Vs (tonf)	Vp (tonf)	PhiVn (tonf)	Phi_tTn (tonf*m)	de (m)	dv (m)	theta ([deg])	beta	bv (m)	bv_min (m)	CHK
9	V-Max	0.01	388.32	5.52	316.06	544.38	0.90	0.90	83.62	313.04	7.87	284.45	489.94	1.65	1.59	29.00	4.80	0.22	0.01	OK
9	V-Min	-20.37	212.07	-1.77	316.06	544.38	0.90	0.90	83.62	313.04	7.87	284.45	489.94	1.65	1.59	29.00	4.80	0.22	0.02	OK
10	T-Max	6.99	391.99	4.53	313.27	544.38	0.90	0.90	84.30	315.59	2.58	281.94	489.94	1.66	1.61	29.00	4.80	0.22	0.00	OK
10	V-Max	6.99	391.99	4.53	313.27	544.38	0.90	0.90	84.30	315.59	2.58	281.94	489.94	1.66	1.61	29.00	4.80	0.22	0.00	OK
10	V-Min	-12.58	216.52	-2.13	313.27	544.38	0.90	0.90	84.30	315.59	2.58	281.94	489.94	1.66	1.61	29.00	4.80	0.22	0.01	OK
11	T-Max	14.17	392.62	3.75	313.26	544.38	0.90	0.90	84.30	315.59	2.57	281.94	489.94	1.66	1.61	29.00	4.80	0.22	0.01	OK
11	V-Max	14.17	392.62	3.75	313.26	544.38	0.90	0.90	84.30	315.59	2.57	281.94	489.94	1.66	1.61	29.00	4.80	0.22	0.01	OK
11	V-Min	-5.00	216.99	-2.46	313.26	544.38	0.90	0.90	84.30	315.59	2.57	281.94	489.94	1.66	1.61	29.00	4.80	0.22	0.01	OK
12	T-Max	21.63	389.73	3.41	316.02	544.38	0.90	0.90	83.62	313.04	7.83	284.42	489.94	1.65	1.59	29.00	4.80	0.22	0.01	OK
12	V-Max	21.63	389.73	3.41	316.02	544.38	0.90	0.90	83.62	313.04	7.83	284.42	489.94	1.65	1.59	29.00	4.80	0.22	0.01	OK
12	V-Min	2.06	213.02	-3.24	316.02	544.38	0.90	0.90	83.62	313.04	7.83	284.42	489.94	1.65	1.59	29.00	4.80	0.22	0.00	OK
13	T-Max	10.29	202.72	-4.27	315.44	544.38	0.90	0.90	82.32	308.19	12.04	283.90	489.94	1.62	1.57	29.00	4.80	0.22	0.00	OK
13	V-Max	30.67	375.68	3.06	315.44	544.38	0.90	0.90	82.32	308.19	12.04	283.90	489.94	1.62	1.57	29.00	4.80	0.22	0.02	OK
13	V-Min	10.29	202.72	-4.27	315.44	544.38	0.90	0.90	82.32	308.19	12.04	283.90	489.94	1.62	1.57	29.00	4.80	0.22	0.00	OK
14	T-Max	18.65	185.60	-5.45	312.74	544.38	0.90	0.90	80.48	301.29	16.13	281.46	489.94	1.59	1.53	29.00	4.80	0.22	0.00	OK
14	V-Max	40.03	350.02	2.55	312.74	544.38	0.90	0.90	80.48	301.29	16.13	281.46	489.94	1.59	1.53	29.00	4.80	0.22	0.02	OK
14	V-Min	18.40	173.48	-4.74	312.74	544.38	0.90	0.90	80.48	301.29	16.13	281.46	489.94	1.59	1.53	29.00	4.80	0.22	0.00	OK
15	T-Max	28.10	160.67	-6.88	307.33	544.38	0.90	0.90	78.15	292.58	19.29	276.60	489.94	1.54	1.49	29.00	4.80	0.22	0.01	OK
15	V-Max	50.85	311.90	1.79	307.33	544.38	0.90	0.90	78.15	292.58	19.29	276.60	489.94	1.54	1.49	29.00	4.80	0.22	0.03	OK
15	V-Min	25.47	111.78	-0.77	307.33	544.38	0.90	0.90	78.15	292.58	19.29	276.60	489.94	1.54	1.49	29.00	4.80	0.22	0.01	OK
16	T-Max	37.87	127.39	-8.43	300.21	544.38	0.90	0.90	75.42	282.34	22.25	270.19	489.94	1.49	1.44	29.00	4.80	0.22	0.02	OK
16	V-Max	62.08	261.81	0.85	300.21	544.38	0.90	0.90	75.42	282.34	22.25	270.19	489.94	1.49	1.44	29.00	4.80	0.22	0.04	OK
16	V-Min	32.85	83.73	-1.24	300.21	544.38	0.90	0.90	75.42	282.34	22.25	270.19	489.94	1.49	1.44	29.00	4.80	0.22	0.01	OK
17	T-Max	48.69	84.82	-10.01	291.03	544.38	0.90	0.90	72.34	270.81	24.42	261.93	489.94	1.43	1.38	29.00	4.80	0.22	0.02	OK
17	V-Max	74.58	200.39	-0.09	291.03	544.38	0.90	0.90	72.34	270.81	24.42	261.93	489.94	1.43	1.38	29.00	4.80	0.22	0.05	OK
17	V-Min	41.30	47.59	-1.71	291.03	544.38	0.90	0.90	72.34	270.81	24.42	261.93	489.94	1.43	1.38	29.00	4.80	0.22	0.02	OK
18	T-Max	60.14	32.33	-10.94	292.16	544.38	0.90	0.90	72.14	270.08	26.27	262.95	489.94	1.37	1.38	29.00	4.80	0.22	0.03	OK
18	V-Max	87.79	124.93	-0.44	292.16	544.38	0.90	0.90	72.14	270.08	26.27	262.95	489.94	1.37	1.38	29.00	4.80	0.22	0.06	OK
18	V-Min	50.30	2.82	-1.89	292.16	544.38	0.90	0.90	72.14	270.08	26.27	262.95	489.94	1.37	1.38	29.00	4.80	0.22	0.02	OK

Elem	Max/Min	Vu (tonf)	Mu (tonf*m)	Tu (tonf*m)	Vn (tonf)	Tn (tonf*m)	Phi	Phi_t	Vc (tonf)	Vs (tonf)	Vp (tonf)	PhiVn (tonf)	Phi_tTn (tonf*m)	de (m)	dv (m)	theta ([deg])	beta	bv (m)	bv_min (m)	CHK
19	T-Max	72.62	-30.94	-9.80	293.39	544.38	0.90	0.90	72.14	270.08	27.50	264.05	489.94	0.82	1.38	29.00	4.80	0.22	0.04	OK
19	V-Max	102.08	34.91	1.10	293.39	544.38	0.90	0.90	72.14	270.08	27.50	264.05	489.94	1.30	1.38	29.00	4.80	0.22	0.07	OK
19	V-Min	60.23	-51.29	-1.11	293.39	544.38	0.90	0.90	72.14	270.08	27.50	264.05	489.94	0.82	1.38	29.00	4.80	0.22	0.03	OK
20	T-Max	117.23	-69.99	7.67	294.12	530.24	0.90	0.90	63.40	263.06	28.23	264.71	477.21	0.91	1.38	29.64	4.22	0.22	0.08	OK
20	V-Max	117.23	-69.99	7.67	294.12	530.24	0.90	0.90	63.40	263.06	28.23	264.71	477.21	0.91	1.38	29.64	4.22	0.22	0.08	OK
20	V-Min	70.91	-115.29	2.20	294.12	544.38	0.90	0.90	72.14	270.08	28.23	264.71	489.94	0.91	1.38	29.00	4.80	0.22	0.04	OK
21	T-Max	133.30	-190.04	25.68	274.10	431.36	0.90	0.90	31.64	214.01	28.45	246.69	388.22	1.01	1.38	34.97	2.11	0.22	0.10	OK
21	V-Max	133.30	-190.04	25.68	274.10	431.36	0.90	0.90	31.64	214.01	28.45	246.69	388.22	1.01	1.38	34.97	2.11	0.22	0.10	OK
21	V-Min	82.11	-189.61	12.13	294.34	514.25	0.90	0.90	55.46	255.13	28.45	264.91	462.82	1.01	1.38	30.40	3.69	0.22	0.05	OK

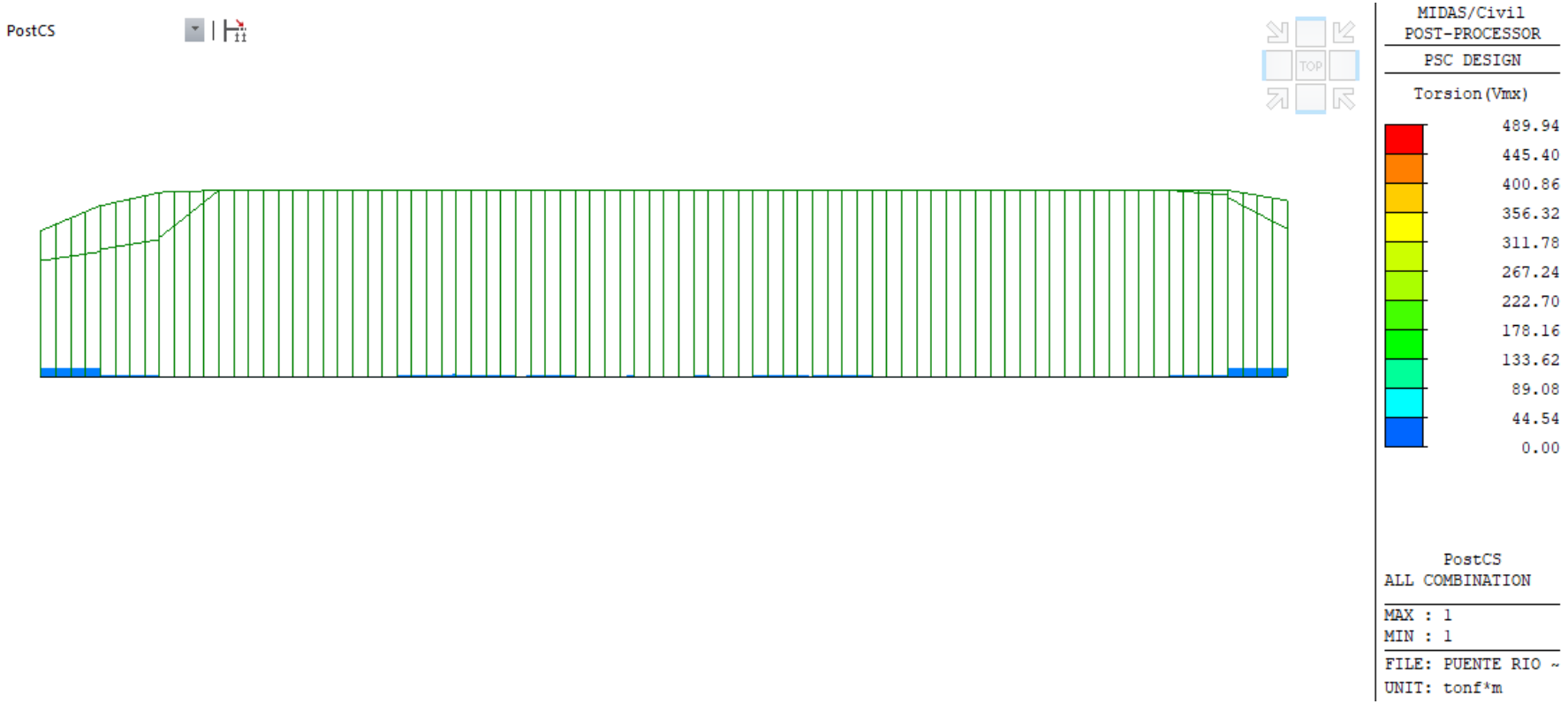


Figura 170. Verificación de capacidad de torsión combinada (Vmx)

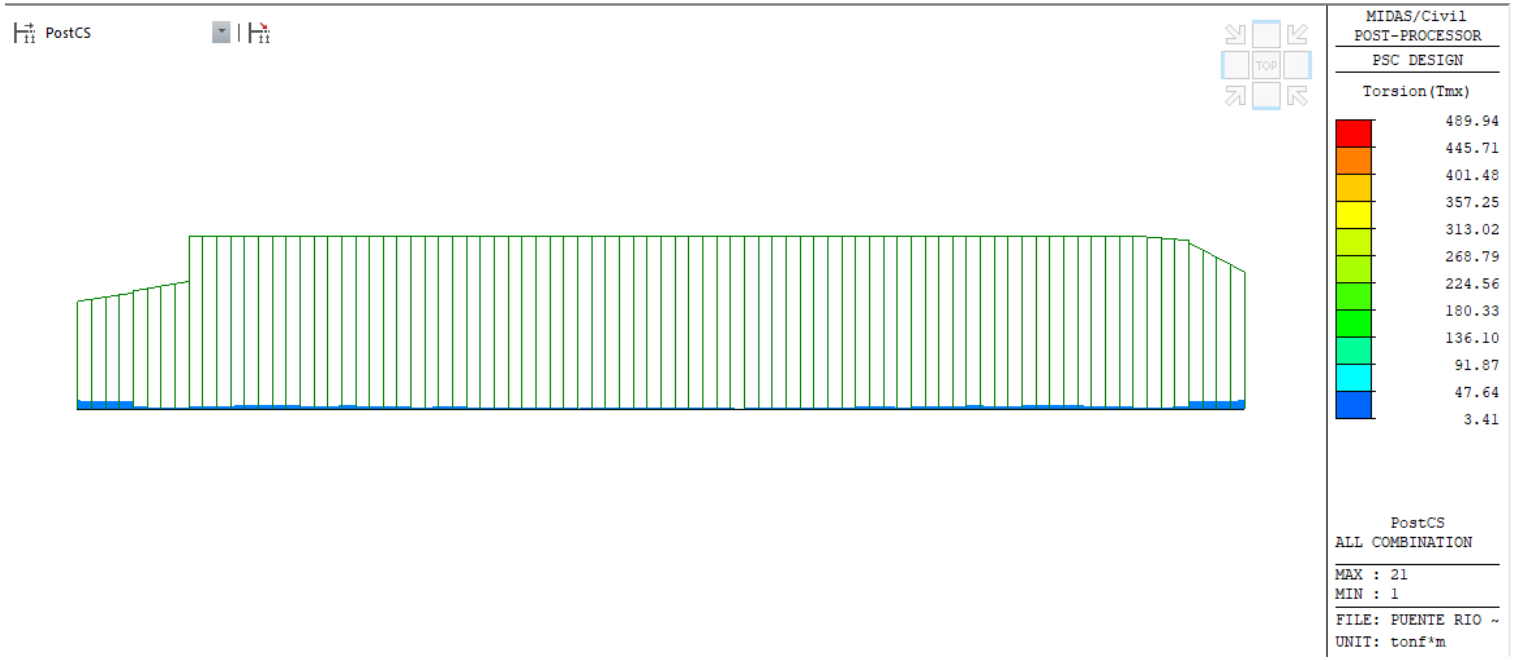


Figura 171. Verificación de capacidad de torsión combinada (Tmx)

3.4.15.4. VERIFICACIÓN POR ANCHO DE FISURA POR CARGAS DE SERVICIO

Elem	Top/Bottom	FT (kgf/cm^2)	FB (kgf/cm^2)	fss (kgf/cm^2)	s_use (cm)	s_max (cm)	CHK
1	Bottom	1.96	-7.28	50.13	15.25	2391.55	OK
2	Bottom	3.58	-13.46	92.60	15.25	1290.04	OK
3	Bottom	5.07	-19.18	132.00	15.25	902.01	OK
4	Bottom	6.35	-24.11	165.95	15.25	715.42	OK
5	Bottom	7.41	-28.19	194.02	15.25	610.46	OK
6	Bottom	8.36	-31.78	218.75	15.25	540.34	OK
7	Bottom	9.14	-34.62	238.27	15.25	495.25	OK
8	Bottom	9.66	-36.47	251.00	15.25	469.62	OK
9	Bottom	9.92	-37.31	256.74	15.25	458.90	OK
10	Bottom	9.91	-37.12	255.45	15.25	461.26	OK
11	Bottom	9.89	-37.17	255.78	15.25	460.66	OK
12	Bottom	9.92	-37.42	257.57	15.25	457.40	OK
13	Bottom	9.66	-36.66	252.34	15.25	467.08	OK
14	Bottom	9.14	-34.88	240.08	15.25	491.43	OK
15	Bottom	8.38	-32.11	221.01	15.25	534.70	OK
16	Bottom	7.42	-28.56	196.59	15.25	602.35	OK
17	Bottom	6.37	-24.57	169.12	15.25	701.84	OK
18	Bottom	5.09	-19.70	135.58	15.25	877.95	OK
19	Bottom	3.61	-14.03	96.55	15.25	1236.84	OK
20	Bottom	1.92	-7.62	52.45	15.25	2285.12	OK
21	Bottom	-0.19	-0.65	4.56	15.25	26366.36	OK

3.4.16. **ELEMENTOS VIGA I POSTENSADA – L=30.0m**

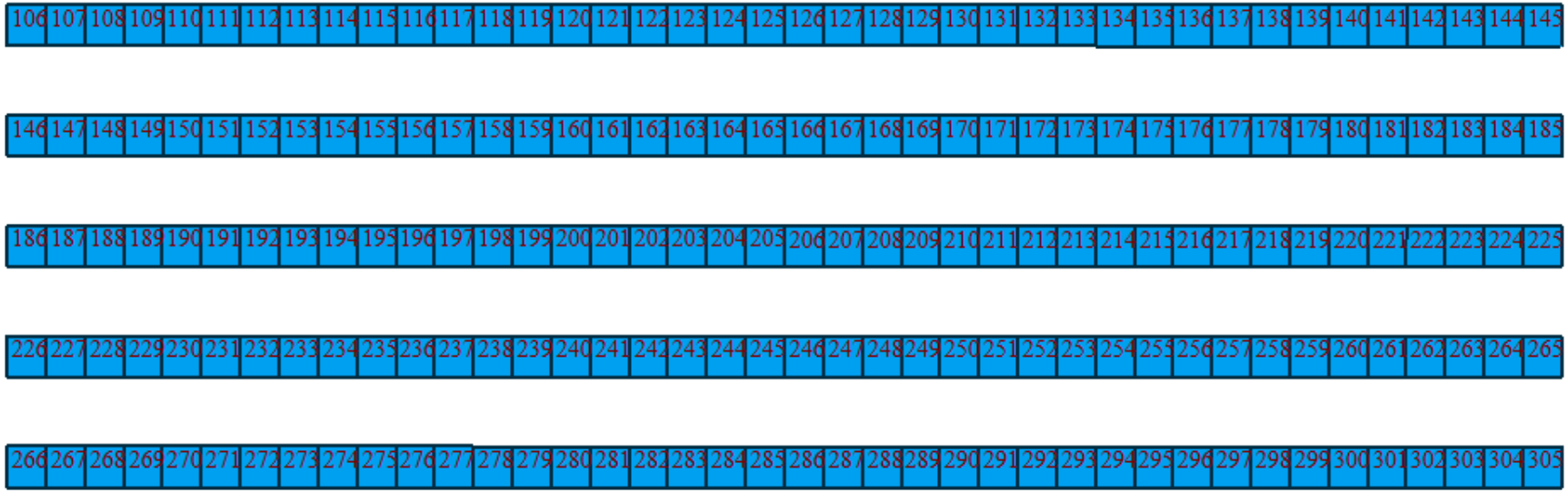


Figura 172. Número de elementos viga 30.0m

3.4.17. **GEOMETRÍA DE LOS CABLES Y FUERZA DE TENSIONAMIENTO**

Luz = 30.0m

El esfuerzo máximo permitido por la Norma antes de cualquier tipo de pérdidas corresponde a un 70*fpu, para efectos prácticos se tensionarán los cables a un esfuerzo igual a:

$$f_{pi} = 13300\ kg/cm^2$$

El cual corresponde a un 70% de fpu, de esta manera cada torón se tensionará a una fuerza de:

$$Fpi = 13300 \frac{kg}{cm^2} * 1.40\ cm^2\ = 18620\ kg = 18.62\ tn$$

Si cada cable tiene 15 torones, la carga total por cable es fpi*# torones = 15*18.62 = 279.30 ton (279300 kg)

Sobre la viga I postensada se dispondrán 3 cables de 15 torones de 5/8” en primera etapa de tensionamiento, mientras que para la segunda etapa de tensionamiento, se dispondrá de 2 cables de 15 torones de 5/8”, tal como se presenta en las siguientes figuras:

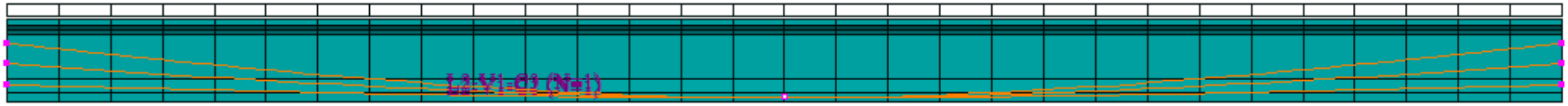


Figura 173. Configuración de los cables en la viga I postensada primera etapa

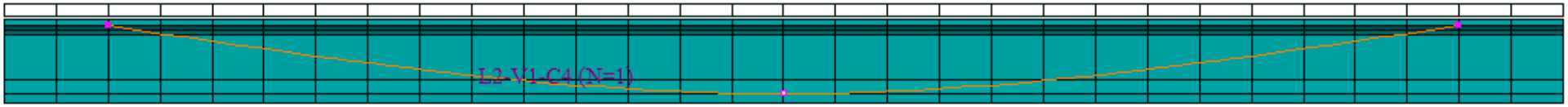


Figura 174. Configuración de los cables en la viga I postensada segunda etapa

Se presenta en las siguientes tablas la geometría de los cables y la variación tanto en los ejes coordenados X, Y la coordenada Z medida desde la cara inferior de la viga, para toda la longitud del puente

Tabla 19. Geometría y posición de los cables vigas – L=30.0m

TENSIONAMIENTO ETAPA 1											TENSIONAMIENTO ETAPA 2		
CABLE C1				CABLE C2				CABLE C3			CABLE C4		
7 torones				7 torones				7 torones			7 torones		
X (m)	Y (m)	Z (m)		X (m)	Y (m)	Z (m)		X (m)	Y (m)	Z (m)	X (m)	Y (m)	Z (m)
0.000	0.000	0.350		0.000	0.000	0.750		0.000	0.000	1.150	-	-	-
1.000	0.025	0.325		1.000	-0.025	0.685		1.000	0.000	1.045	-	-	-
2.000	0.050	0.300		2.000	-0.050	0.621		2.000	0.000	0.941	2.000	0.000	1.500
3.000	0.074	0.276		3.000	-0.074	0.558		3.000	0.000	0.839	3.000	0.000	1.350
4.000	0.098	0.252		4.000	-0.098	0.496		4.000	0.000	0.740	4.000	0.000	1.202
5.000	0.120	0.230		5.000	-0.120	0.437		5.000	0.000	0.644	5.000	0.000	1.057
6.000	0.142	0.208		6.000	-0.142	0.381		6.000	0.000	0.553	6.000	0.000	0.918
7.000	0.162	0.188		7.000	-0.162	0.328		7.000	0.000	0.468	7.000	0.000	0.786
8.000	0.181	0.169		8.000	-0.181	0.279		8.000	0.000	0.389	8.000	0.000	0.663
9.000	0.198	0.152		9.000	-0.198	0.235		9.000	0.000	0.318	9.000	0.000	0.550
10.000	0.213	0.137		10.000	-0.213	0.196		10.000	0.000	0.255	10.000	0.000	0.450
11.000	0.226	0.124		11.000	-0.226	0.163		11.000	0.000	0.202	11.000	0.000	0.365
12.000	0.236	0.114		12.000	-0.236	0.136		12.000	0.000	0.159	12.000	0.000	0.295
13.000	0.244	0.106		13.000	-0.244	0.117		13.000	0.000	0.127	13.000	0.000	0.243
14.000	0.248	0.102		14.000	-0.248	0.104		14.000	0.000	0.107	14.000	0.000	0.211
15.000	0.250	0.100		15.000	-0.250	0.100		15.000	0.000	0.100	15.000	0.000	0.200
16.000	0.248	0.102		16.000	-0.248	0.104		16.000	0.000	0.107	16.000	0.000	0.211
17.000	0.244	0.106		17.000	-0.244	0.117		17.000	0.000	0.127	17.000	0.000	0.243
18.000	0.236	0.114		18.000	-0.236	0.136		18.000	0.000	0.159	18.000	0.000	0.295
19.000	0.226	0.124		19.000	-0.226	0.163		19.000	0.000	0.202	19.000	0.000	0.365
20.000	0.213	0.137		20.000	-0.213	0.196		20.000	0.000	0.255	20.000	0.000	0.450
21.000	0.198	0.152		21.000	-0.198	0.235		21.000	0.000	0.318	21.000	0.000	0.550
22.000	0.181	0.169		22.000	-0.181	0.279		22.000	0.000	0.389	22.000	0.000	0.663
23.000	0.162	0.188		23.000	-0.162	0.328		23.000	0.000	0.468	23.000	0.000	0.786
24.000	0.142	0.208		24.000	-0.142	0.381		24.000	0.000	0.553	24.000	0.000	0.918
25.000	0.120	0.230		25.000	-0.120	0.437		25.000	0.000	0.644	25.000	0.000	1.057
26.000	0.098	0.252		26.000	-0.098	0.496		26.000	0.000	0.740	26.000	0.000	1.202
27.000	0.074	0.276		27.000	-0.074	0.558		27.000	0.000	0.839	27.000	0.000	1.350
28.000	0.050	0.300		28.000	-0.050	0.621		28.000	0.000	0.941	28.000	0.000	1.500
29.000	0.025	0.325		29.000	-0.025	0.685		29.000	0.000	1.045	-	-	-
30.000	0.000	0.350		30.000	0.000	0.750		30.000	0.000	1.150	-	-	-

3.4.18. **RESULTADOS DEL MODELO VIGAS - L=30.0m**

3.4.19. **DIAGRAMAS DE MOMENTO Y CORTANTE**

Se presentan los principales resultados del modelo estructural teniendo en cuenta los estados límites de resistencia, extremos y de servicio.

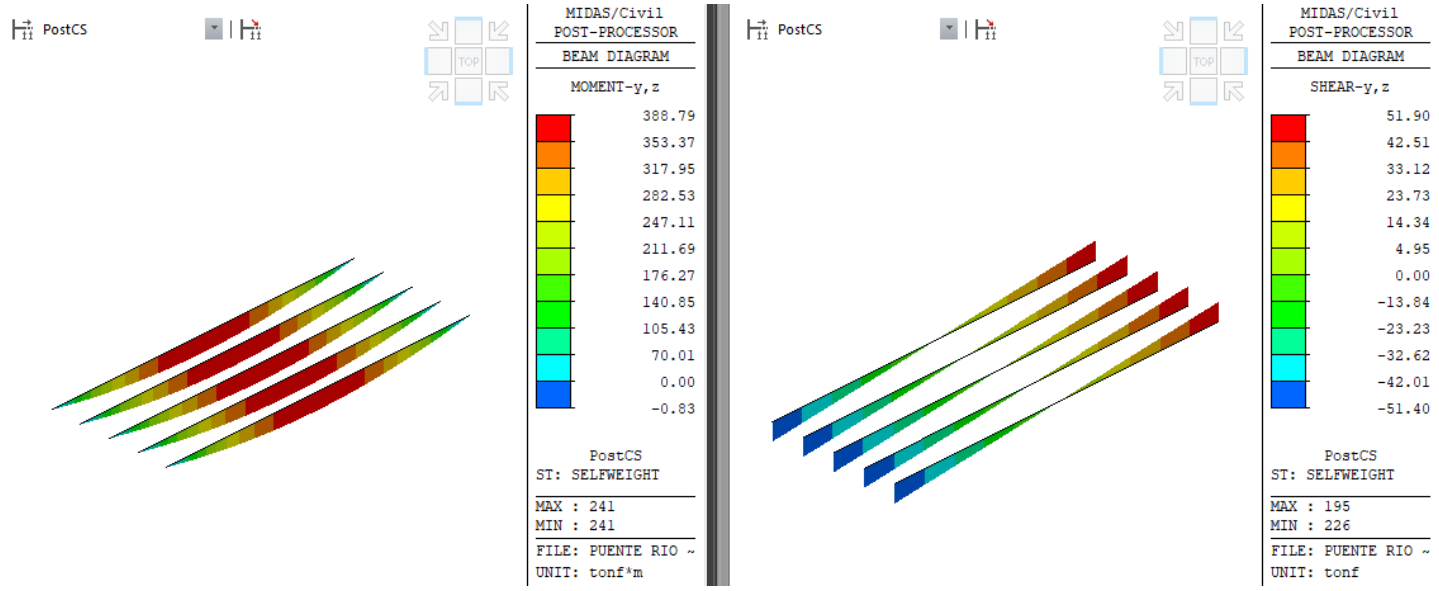


Figura 175. Resultados Momento flector y fuerza cortante para carga muerta

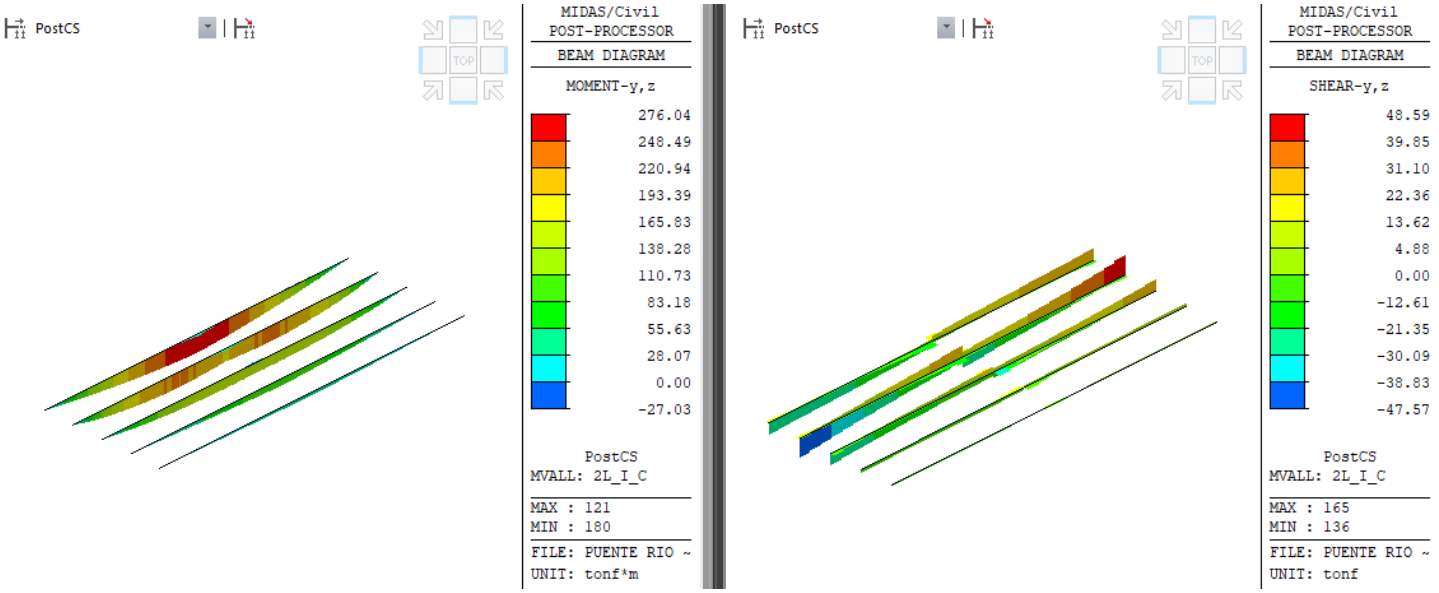


Figura 176. Resultados Momento flector y fuerza cortante para carga viva vehicular

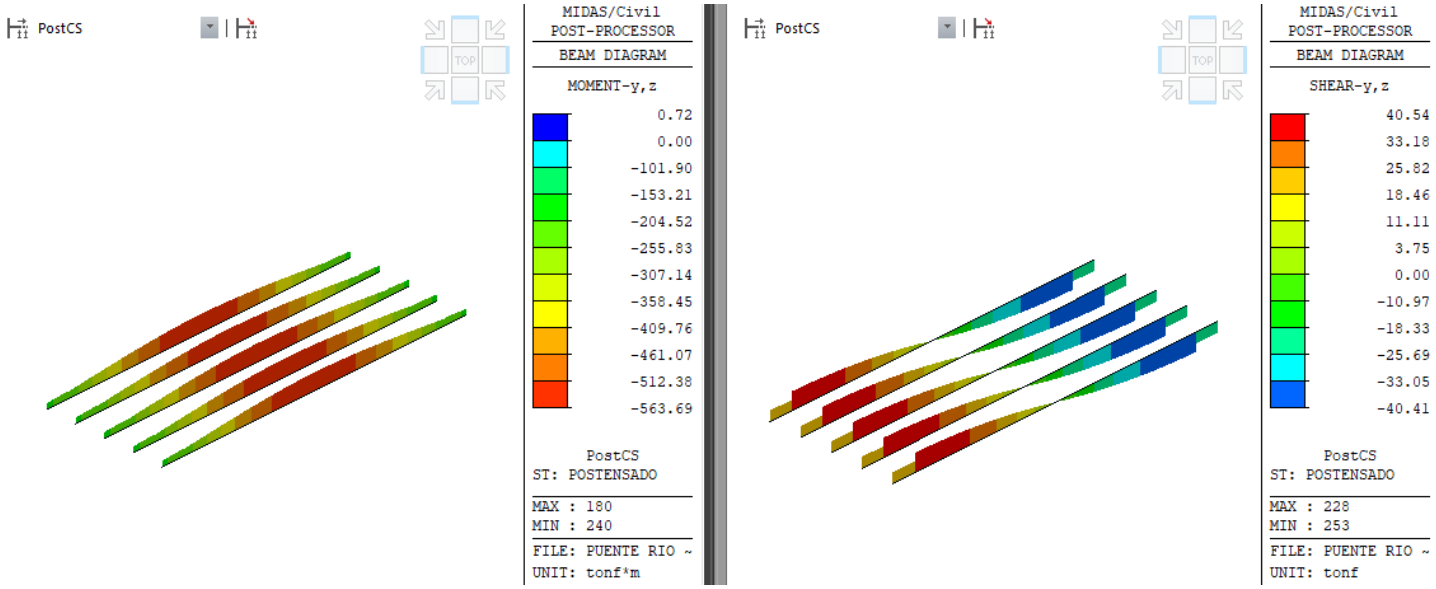


Figura 177. Resultados Momento flector y fuerza cortante para carga de postensado

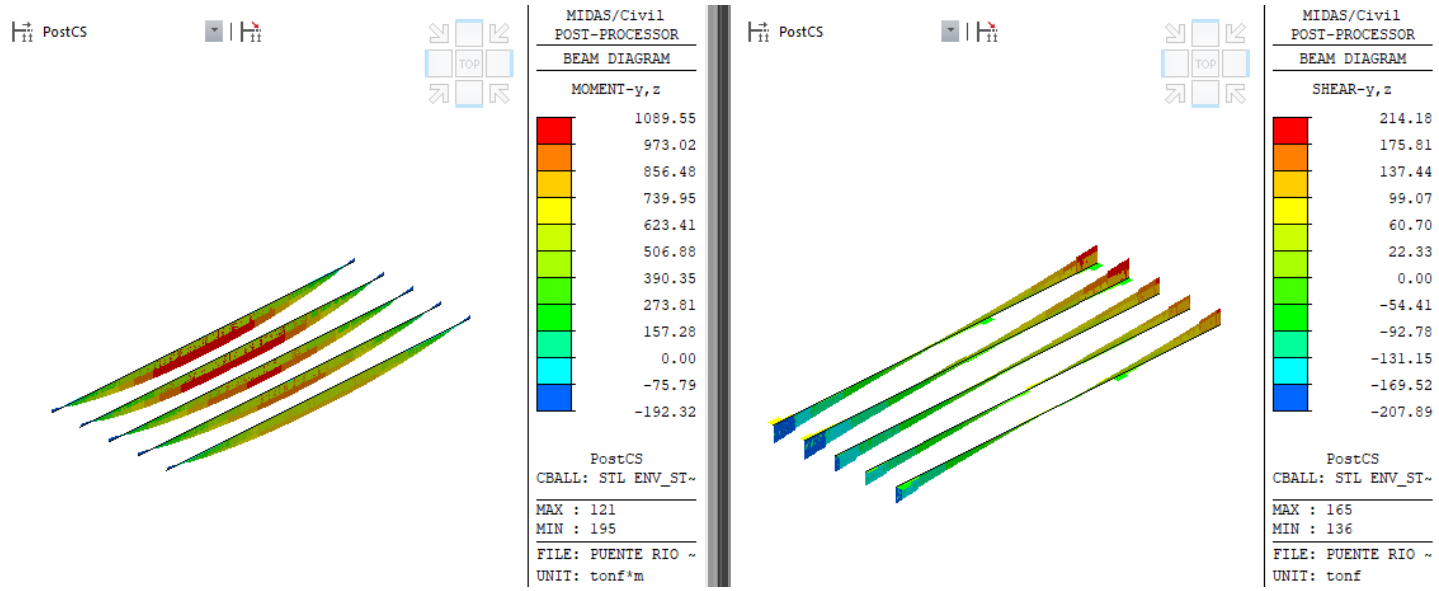


Figura 178. Resultados Momento cortante para la envolvente de resistencia

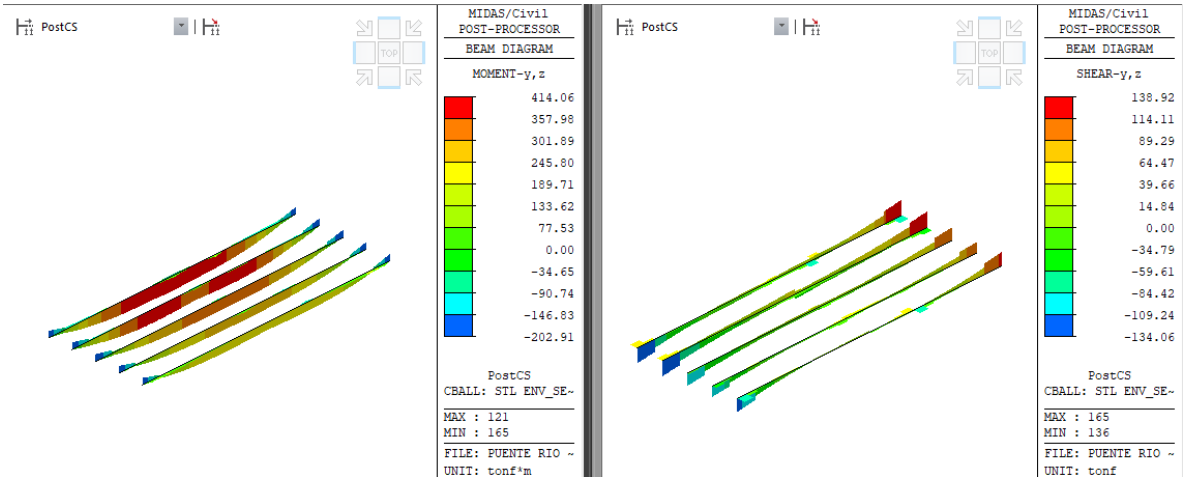


Figura 179. Resultados Momento flector y fuerza cortante en servicio

3.4.20. **REVISIÓN DE ESFUERZOS**

Se calculan los esfuerzos producidos en la viga I postensada debido a la combinación de Estado Límite de Servicio I y Estado Limite de Servicio III.

Para la revisión de los esfuerzos tanto en fibra superior como inferior se empleó la siguiente formulación:

$$\sigma_{sup} = \frac{P}{A} - M_P \frac{Y_{sup}}{I_g} - M_{DL} \frac{Y_{sup}}{I_g}$$

$$\sigma_{Inf} = \frac{P}{A} + M_P \frac{Y_{inf}}{I_g} + M_{DL} \frac{Y_{inf}}{I_g}$$

Donde:

$\boldsymbol{P}$ Carga axial por el tensionamiento de los cables (t)

$\boldsymbol{M_p}$ Momento por presforzado debido a la excentricidad de los cables.

$\boldsymbol{M_{DL}}$ Momento debido a la carga muerta, sobrecarga y carga viva.

Y_{sup} *e* Y_{inf} Son las distancias desde el centroide a la fibra extrema superior e inferior respectivamente.

I_g Inercia total de la sección

A Área de la sección transversal.

Se define los parámetros de diseño para la sección PSC (prestressed concrete)

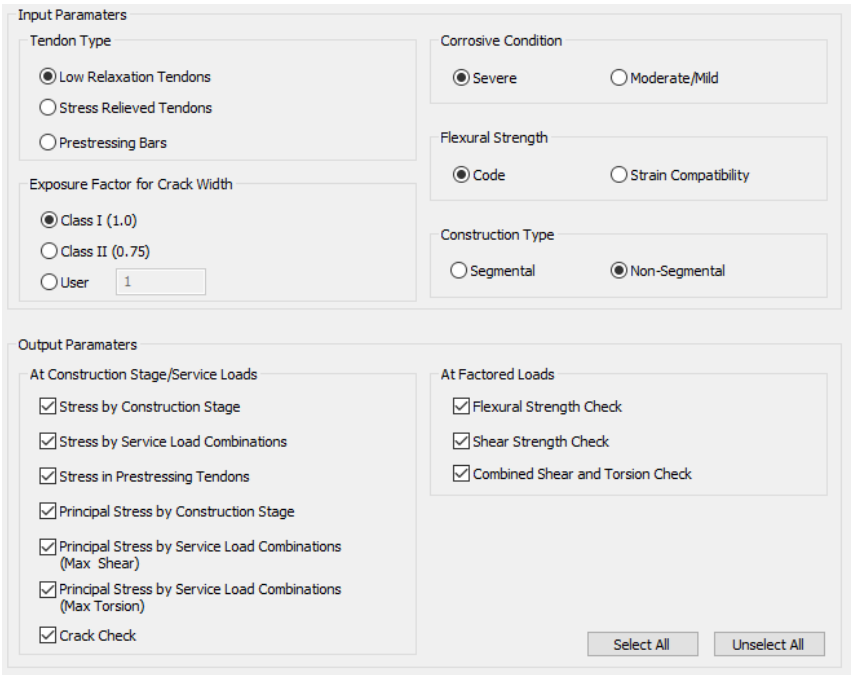


Figura 180. Parámetros de diseño de concreto postensado NO-SEGMENTAL

3.4.20.1. REVISIÓN DE ESFUERZOS EN ETAPAS CONSTRUCTIVAS:

Elem: número de elemento

Comp./Tens.: Compresión o Tensión

FT: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My) y fuerza axial en la fibra superior

FB: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My) y fuerza axial en la fibra inferior

FTL: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My), el eje menor (Mz) y la fuerza axial en la fibra superior izquierda

FBL: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My), el eje menor (Mz) y la fuerza axial en la fibra inferior izquierda

FTR: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My), el eje menor (Mz) y la fuerza axial en la fibra superior derecha

FBR: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My), el eje menor (Mz) y la fuerza axial en la fibra inferior derecha

ALW: Esfuerzo admisible en el hormigón en el estado límite de servicio.

CHK: comprobación de esfuerzos combinados para cargas de servicio

Tabla 20. Verificación de esfuerzos a compresión y tracción en fibra superior

Elem	Comp./Tens.	FT (kgf/cm^2)	FTL (kgf/cm^2)	FTR (kgf/cm^2)	ALW (kgf/cm^2)	CHK
106.0	Compression	42.00	42.27	41.73	175.32	OK
107.0	Compression	42.27	42.41	42.12	175.32	OK
108.0	Compression	42.84	42.89	42.79	175.32	OK
109.0	Compression	42.21	42.20	42.22	175.32	OK
110.0	Compression	41.28	41.22	41.33	175.32	OK
111.0	Compression	40.10	40.01	40.18	175.32	OK
112.0	Compression	38.74	38.64	38.85	175.32	OK
113.0	Compression	37.28	37.17	37.39	175.32	OK
114.0	Compression	35.78	35.68	35.88	175.32	OK
115.0	Compression	34.32	34.25	34.38	175.32	OK
116.0	Compression	32.99	32.89	33.09	175.32	OK
117.0	Compression	31.87	31.72	32.02	175.32	OK
118.0	Compression	31.03	30.86	31.20	175.32	OK
119.0	Compression	30.53	30.31	30.75	175.32	OK
120.0	Compression	30.45	30.15	30.75	175.32	OK
121.0	Compression	30.83	30.58	31.07	175.32	OK
122.0	Compression	31.61	31.44	31.79	175.32	OK
123.0	Compression	32.71	32.60	32.81	175.32	OK
124.0	Compression	34.05	34.04	34.07	175.32	OK
125.0	Compression	35.57	35.67	35.48	175.32	OK
126.0	Compression	36.89	36.89	36.90	175.32	OK
127.0	Compression	38.24	38.17	38.32	175.32	OK
128.0	Compression	39.55	39.44	39.65	175.32	OK
129.0	Compression	40.73	40.63	40.83	175.32	OK
130.0	Compression	41.73	41.65	41.82	175.32	OK
131.0	Compression	42.49	42.45	42.53	175.32	OK
132.0	Compression	42.93	42.96	42.91	175.32	OK
133.0	Compression	42.99	43.08	42.90	175.32	OK
134.0	Compression	41.70	41.83	41.56	175.32	OK
135.0	Compression	40.73	40.73	40.73	175.32	OK

Elem	Comp./Tens.	FT (kgf/cm^2)	FTL (kgf/cm^2)	FTR (kgf/cm^2)	ALW (kgf/cm^2)	CHK
106.0	Tension	42.00	42.27	41.73	-34.40	OK
107.0	Tension	42.27	42.41	42.12	-34.40	OK
108.0	Tension	42.84	42.89	42.79	-34.40	OK
109.0	Tension	42.21	42.20	42.22	-34.40	OK
110.0	Tension	41.28	41.22	41.33	-34.40	OK
111.0	Tension	40.10	40.01	40.18	-34.40	OK
112.0	Tension	38.74	38.64	38.85	-34.40	OK
113.0	Tension	37.28	37.17	37.39	-34.40	OK
114.0	Tension	35.78	35.68	35.88	-34.40	OK
115.0	Tension	34.32	34.25	34.38	-34.40	OK
116.0	Tension	32.99	32.89	33.09	-34.40	OK
117.0	Tension	31.87	31.72	32.02	-34.40	OK
118.0	Tension	31.03	30.86	31.20	-34.40	OK
119.0	Tension	30.53	30.31	30.75	-34.40	OK
120.0	Tension	30.45	30.15	30.75	-34.40	OK
121.0	Tension	30.83	30.58	31.07	-34.40	OK
122.0	Tension	31.61	31.44	31.79	-34.40	OK
123.0	Tension	32.71	32.60	32.81	-34.40	OK
124.0	Tension	34.05	34.04	34.07	-34.40	OK
125.0	Tension	35.57	35.67	35.48	-34.40	OK
126.0	Tension	36.89	36.89	36.90	-34.40	OK
127.0	Tension	38.24	38.17	38.32	-34.40	OK
128.0	Tension	39.55	39.44	39.65	-34.40	OK
129.0	Tension	40.73	40.63	40.83	-34.40	OK
130.0	Tension	41.73	41.65	41.82	-34.40	OK
131.0	Tension	42.49	42.45	42.53	-34.40	OK
132.0	Tension	42.93	42.96	42.91	-34.40	OK
133.0	Tension	42.99	43.08	42.90	-34.40	OK
134.0	Tension	41.70	41.83	41.56	-34.40	OK
135.0	Tension	40.73	40.73	40.73	-34.40	OK

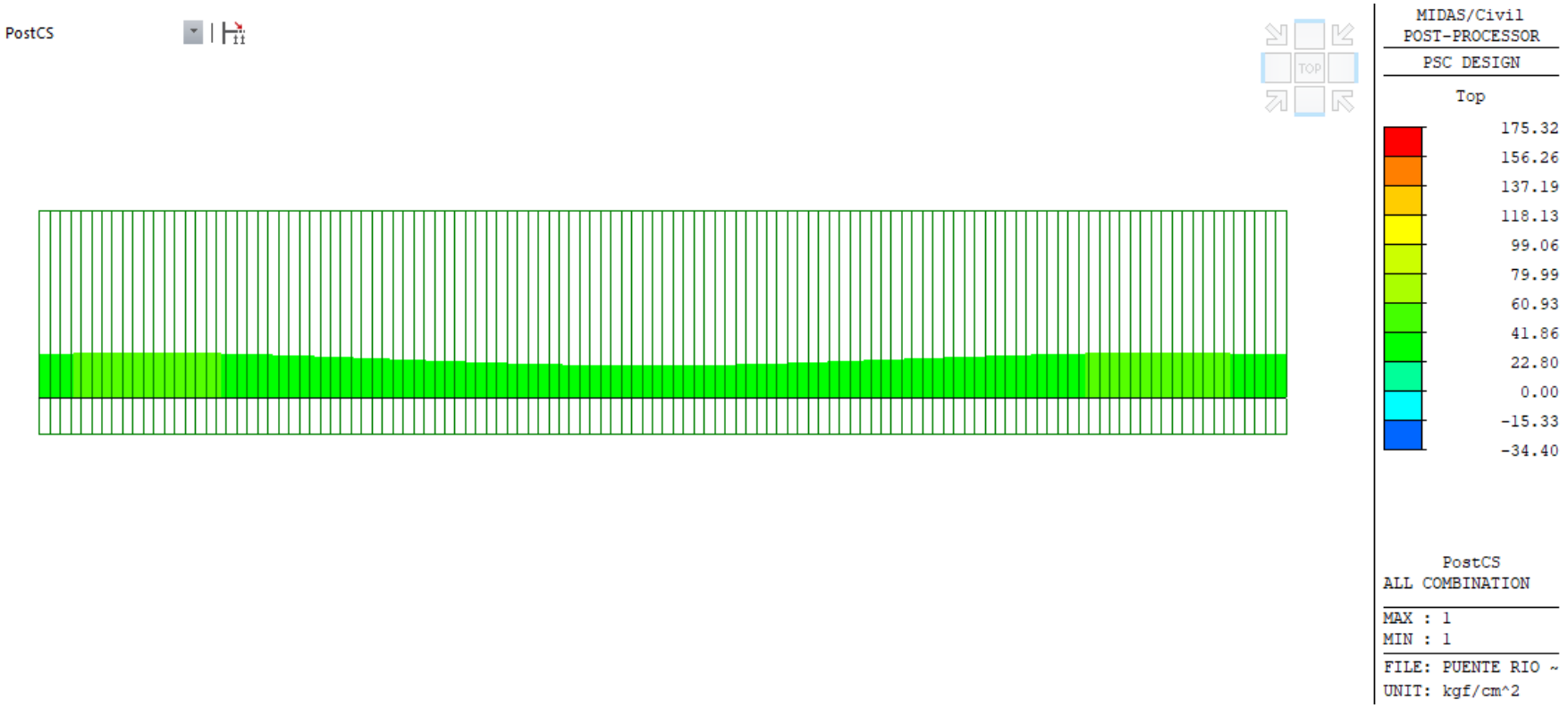


Figura 181. Verificación de esfuerzos a compresión y tracción en fibra superior

Tabla 21. Verificación de esfuerzos a compresión y tracción en fibra inferior

Elem	Comp./Tens.	FT (kgf/cm^2)	FTL (kgf/cm^2)	FTR (kgf/cm^2)	ALW (kgf/cm^2)	CHK
106.0	Compression	59.03	59.21	58.85	175.32	OK
107.0	Compression	58.81	58.91	58.71	175.32	OK
108.0	Compression	58.96	59.00	58.93	175.32	OK
109.0	Compression	59.73	59.72	59.74	175.32	OK
110.0	Compression	60.85	60.81	60.89	175.32	OK
111.0	Compression	62.24	62.18	62.29	175.32	OK
112.0	Compression	63.81	63.74	63.88	175.32	OK
113.0	Compression	65.53	65.46	65.60	175.32	OK
114.0	Compression	67.31	67.25	67.38	175.32	OK
115.0	Compression	69.05	69.01	69.10	175.32	OK
116.0	Compression	70.57	70.50	70.64	175.32	OK
117.0	Compression	71.84	71.74	71.94	175.32	OK
118.0	Compression	72.83	72.72	72.95	175.32	OK
119.0	Compression	73.48	73.34	73.63	175.32	OK
120.0	Compression	73.69	73.49	73.89	175.32	OK
121.0	Compression	73.08	72.92	73.25	175.32	OK
122.0	Compression	72.10	71.98	72.22	175.32	OK
123.0	Compression	70.82	70.75	70.89	175.32	OK
124.0	Compression	69.26	69.25	69.27	175.32	OK
125.0	Compression	67.54	67.60	67.48	175.32	OK
126.0	Compression	66.04	66.04	66.05	175.32	OK
127.0	Compression	64.51	64.46	64.56	175.32	OK
128.0	Compression	63.04	62.97	63.11	175.32	OK
129.0	Compression	61.71	61.64	61.78	175.32	OK
130.0	Compression	60.57	60.51	60.62	175.32	OK
131.0	Compression	59.67	59.64	59.70	175.32	OK
132.0	Compression	59.07	59.08	59.05	175.32	OK
133.0	Compression	58.87	58.93	58.81	175.32	OK
134.0	Compression	59.48	59.58	59.39	175.32	OK
135.0	Compression	60.43	60.43	60.43	175.32	OK

Elem	Comp./Tens.	FT (kgf/cm^2)	FTL (kgf/cm^2)	FTR (kgf/cm^2)	ALW (kgf/cm^2)	CHK
106.0	Tension	59.03	59.21	58.85	-34.40	OK
107.0	Tension	58.81	58.91	58.71	-34.40	OK
108.0	Tension	58.96	59.00	58.93	-34.40	OK
109.0	Tension	59.73	59.72	59.74	-34.40	OK
110.0	Tension	60.85	60.81	60.89	-34.40	OK
111.0	Tension	62.24	62.18	62.29	-34.40	OK
112.0	Tension	63.81	63.74	63.88	-34.40	OK
113.0	Tension	65.53	65.46	65.60	-34.40	OK
114.0	Tension	67.31	67.25	67.38	-34.40	OK
115.0	Tension	69.05	69.01	69.10	-34.40	OK
116.0	Tension	70.57	70.50	70.64	-34.40	OK
117.0	Tension	71.84	71.74	71.94	-34.40	OK
118.0	Tension	72.83	72.72	72.95	-34.40	OK
119.0	Tension	73.48	73.34	73.63	-34.40	OK
120.0	Tension	73.69	73.49	73.89	-34.40	OK
121.0	Tension	73.08	72.92	73.25	-34.40	OK
122.0	Tension	72.10	71.98	72.22	-34.40	OK
123.0	Tension	70.82	70.75	70.89	-34.40	OK
124.0	Tension	69.26	69.25	69.27	-34.40	OK
125.0	Tension	67.54	67.60	67.48	-34.40	OK
126.0	Tension	66.04	66.04	66.05	-34.40	OK
127.0	Tension	64.51	64.46	64.56	-34.40	OK
128.0	Tension	63.04	62.97	63.11	-34.40	OK
129.0	Tension	61.71	61.64	61.78	-34.40	OK
130.0	Tension	60.57	60.51	60.62	-34.40	OK
131.0	Tension	59.67	59.64	59.70	-34.40	OK
132.0	Tension	59.07	59.08	59.05	-34.40	OK
133.0	Tension	58.87	58.93	58.81	-34.40	OK
134.0	Tension	59.48	59.58	59.39	-34.40	OK
135.0	Tension	60.43	60.43	60.43	-34.40	OK

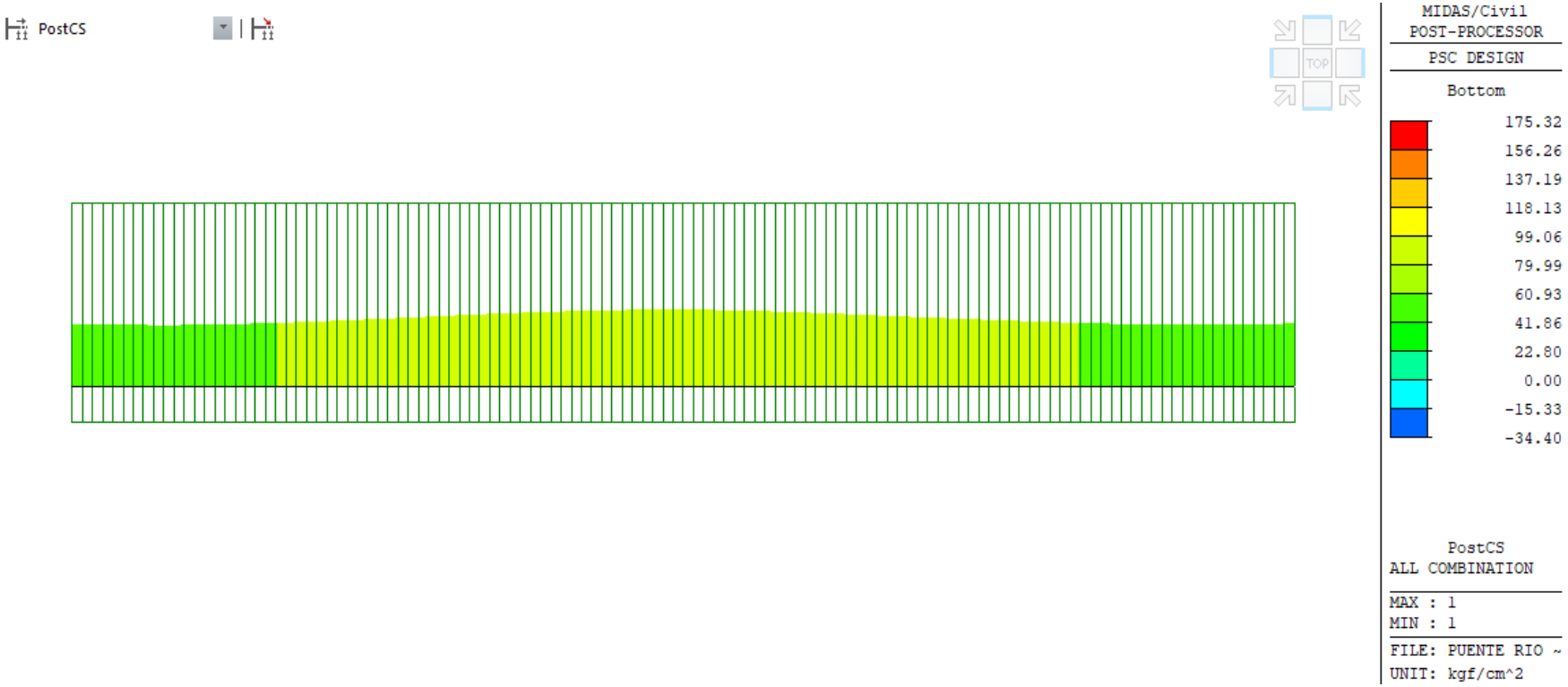


Figura 182. Verificación de esfuerzos a compresión y tracción en fibra inferior

3.4.20.2. REVISIÓN DE ESFUERZOS EN ETAPA DE SERVICIO:

Elem: número de elemento

Comp./Tens.: Esfuerzo de Compresión o Tensión

FT: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My) y fuerza axial en la fibra superior

FB: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My) y fuerza axial en la fibra inferior

FTL: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My), el eje menor (Mz) y la fuerza axial en la fibra superior izquierda

FBL: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My), el eje menor (Mz) y la fuerza axial en la fibra inferior izquierda

FTR: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My), el eje menor (Mz) y la fuerza axial en la fibra superior derecha

FBR: Esfuerzo combinado debido al momento de flexión sobre el eje mayor (My), el eje menor (Mz) y la fuerza axial en la fibra inferior derecha

ALW: Esfuerzo admisible en el hormigón en el estado límite de servicio.

CHK: comprobación de esfuerzos combinados para cargas de servicio

Tabla 22. Verificación de esfuerzos a compresión y tracción en fibra superior

Elem	Comp./Tens.	FT (kgf/cm^2)	FTL (kgf/cm^2)	FTR (kgf/cm^2)	ALW (kgf/cm^2)	CHK
106.0	Compression	52.33	61.24	43.43	210.92	OK
107.0	Compression	56.23	65.83	46.63	210.92	OK
108.0	Compression	78.94	87.36	70.53	210.92	OK
109.0	Compression	88.15	92.74	83.57	210.92	OK
110.0	Compression	90.46	93.38	87.54	210.92	OK
111.0	Compression	92.30	93.88	90.73	210.92	OK
112.0	Compression	93.72	94.30	93.14	210.92	OK
113.0	Compression	94.74	94.52	94.96	210.92	OK
114.0	Compression	95.48	94.86	96.11	210.92	OK
115.0	Compression	95.94	95.14	96.74	210.92	OK
116.0	Compression	96.18	95.38	96.97	210.92	OK
117.0	Compression	96.26	95.64	96.89	210.92	OK
118.0	Compression	74.27	76.65	71.89	210.92	OK
119.0	Compression	73.47	77.37	69.57	210.92	OK
120.0	Compression	72.97	79.51	66.42	210.92	OK
121.0	Compression	73.64	77.62	69.65	210.92	OK
122.0	Compression	74.77	77.40	72.14	210.92	OK
123.0	Compression	97.18	96.44	97.92	210.92	OK
124.0	Compression	97.44	94.14	100.74	210.92	OK
125.0	Compression	97.52	87.23	107.80	210.92	OK
126.0	Compression	97.05	94.61	99.50	210.92	OK
127.0	Compression	96.11	96.76	95.46	210.92	OK
128.0	Compression	94.79	96.80	92.79	210.92	OK
129.0	Compression	93.06	95.95	90.18	210.92	OK
130.0	Compression	90.89	94.67	87.11	210.92	OK
131.0	Compression	88.22	93.06	83.37	210.92	OK
132.0	Compression	78.53	86.46	70.61	210.92	OK
133.0	Compression	76.86	85.18	68.53	210.92	OK
134.0	Compression	51.01	57.84	44.18	210.92	OK
135.0	Compression	46.11	46.11	46.11	210.92	OK

Elem	Comp./Tens.	FT (kgf/cm^2)	FTL (kgf/cm^2)	FTR (kgf/cm^2)	ALW (kgf/cm^2)	CHK
106.0	Tension	52.40	61.11	43.68	-29.87	OK
107.0	Tension	56.42	65.77	47.07	-29.87	OK
108.0	Tension	83.52	90.02	77.02	-29.87	OK
109.0	Tension	85.74	90.42	81.07	-29.87	OK
110.0	Tension	87.51	90.52	84.50	-29.87	OK
111.0	Tension	88.83	90.51	87.15	-29.87	OK
112.0	Tension	89.76	90.44	89.07	-29.87	OK
113.0	Tension	90.33	90.35	90.32	-29.87	OK
114.0	Tension	90.64	90.26	91.02	-29.87	OK
115.0	Tension	90.69	90.16	91.22	-29.87	OK
116.0	Tension	90.56	90.08	91.04	-29.87	OK
117.0	Tension	90.32	90.11	90.53	-29.87	OK
118.0	Tension	90.05	90.07	90.03	-29.87	OK
119.0	Tension	89.84	90.16	89.52	-29.87	OK
120.0	Tension	89.80	90.44	89.15	-29.87	OK
121.0	Tension	90.02	90.57	89.47	-29.87	OK
122.0	Tension	90.57	91.00	90.14	-29.87	OK
123.0	Tension	91.17	90.76	91.59	-29.87	OK
124.0	Tension	91.76	88.69	94.82	-29.87	OK
125.0	Tension	92.20	82.09	102.30	-29.87	OK
126.0	Tension	92.15	89.86	94.43	-29.87	OK
127.0	Tension	91.64	92.34	90.93	-29.87	OK
128.0	Tension	90.78	92.84	88.73	-29.87	OK
129.0	Tension	89.55	92.49	86.61	-29.87	OK
130.0	Tension	87.90	91.72	84.07	-29.87	OK
131.0	Tension	85.77	90.66	80.89	-29.87	OK
132.0	Tension	83.15	89.17	77.13	-29.87	OK
133.0	Tension	77.09	85.23	68.96	-29.87	OK
134.0	Tension	51.12	57.84	44.41	-29.87	OK
135.0	Tension	46.11	46.11	46.11	-29.87	OK

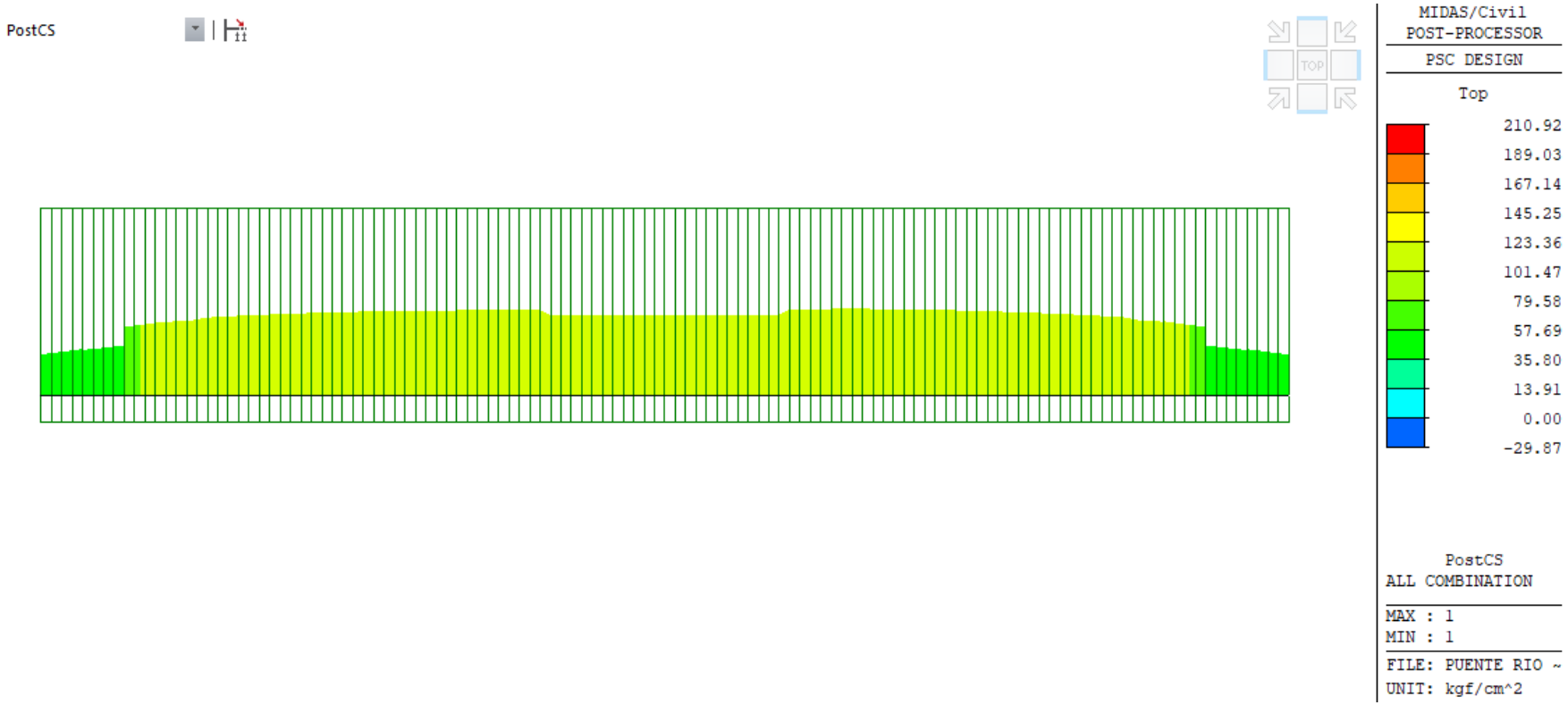


Figura 183. Verificación de esfuerzos a compresión y tracción en fibra superior

Tabla 23. Verificación de esfuerzos a compresión y tracción en fibra inferior

Elem	Comp./Tens.	FT (kgf/cm^2)	FTL (kgf/cm^2)	FTR (kgf/cm^2)	ALW (kgf/cm^2)	CHK
106.0	Compression	107.61	113.52	101.70	210.92	OK
107.0	Compression	90.90	97.27	84.53	210.92	OK
108.0	Compression	88.56	94.15	82.98	210.92	OK
109.0	Compression	52.98	56.02	49.93	210.92	OK
110.0	Compression	43.42	45.35	41.48	210.92	OK
111.0	Compression	35.89	36.93	34.84	210.92	OK
112.0	Compression	30.11	30.49	29.72	210.92	OK
113.0	Compression	26.65	26.51	26.80	210.92	OK
114.0	Compression	23.61	23.20	24.03	210.92	OK
115.0	Compression	21.68	21.15	22.21	210.92	OK
116.0	Compression	20.47	19.94	20.99	210.92	OK
117.0	Compression	19.88	19.47	20.30	210.92	OK
118.0	Compression	98.33	99.91	96.75	210.92	OK
119.0	Compression	101.58	104.16	98.99	210.92	OK
120.0	Compression	104.24	108.58	99.90	210.92	OK
121.0	Compression	101.18	103.83	98.54	210.92	OK
122.0	Compression	97.68	99.42	95.93	210.92	OK
123.0	Compression	18.84	18.35	19.34	210.92	OK
124.0	Compression	19.25	17.06	21.44	210.92	OK
125.0	Compression	20.30	13.47	27.12	210.92	OK
126.0	Compression	22.44	20.82	24.07	210.92	OK
127.0	Compression	24.91	25.34	24.48	210.92	OK
128.0	Compression	29.36	30.69	28.03	210.92	OK
129.0	Compression	35.43	37.34	33.51	210.92	OK
130.0	Compression	43.33	45.84	40.82	210.92	OK
131.0	Compression	53.28	56.50	50.07	210.92	OK
132.0	Compression	89.35	94.61	84.09	210.92	OK
133.0	Compression	96.45	101.98	90.93	210.92	OK
134.0	Compression	109.16	113.70	104.63	210.92	OK
135.0	Compression	128.83	128.83	128.83	210.92	OK

Elem	Comp./Tens.	FT (kgf/cm^2)	FTL (kgf/cm^2)	FTR (kgf/cm^2)	ALW (kgf/cm^2)	CHK
106.0	Tension	107.35	113.14	101.57	-29.87	OK
107.0	Tension	90.30	96.50	84.10	-29.87	OK
108.0	Tension	71.55	75.86	67.24	-29.87	OK
109.0	Tension	61.75	64.85	58.65	-29.87	OK
110.0	Tension	54.11	56.11	52.11	-29.87	OK
111.0	Tension	48.37	49.49	47.26	-29.87	OK
112.0	Tension	44.28	44.73	43.82	-29.87	OK
113.0	Tension	41.63	41.64	41.62	-29.87	OK
114.0	Tension	40.00	39.74	40.25	-29.87	OK
115.0	Tension	39.35	38.99	39.70	-29.87	OK
116.0	Tension	39.28	38.97	39.60	-29.87	OK
117.0	Tension	39.71	39.57	39.85	-29.87	OK
118.0	Tension	40.43	40.44	40.42	-29.87	OK
119.0	Tension	41.42	41.63	41.21	-29.87	OK
120.0	Tension	42.32	42.75	41.89	-29.87	OK
121.0	Tension	40.92	41.28	40.56	-29.87	OK
122.0	Tension	39.62	39.91	39.33	-29.87	OK
123.0	Tension	38.69	38.42	38.97	-29.87	OK
124.0	Tension	38.09	36.06	40.12	-29.87	OK
125.0	Tension	37.98	31.28	44.69	-29.87	OK
126.0	Tension	38.83	37.32	40.35	-29.87	OK
127.0	Tension	40.65	41.12	40.18	-29.87	OK
128.0	Tension	43.52	44.89	42.16	-29.87	OK
129.0	Tension	47.90	49.85	45.95	-29.87	OK
130.0	Tension	54.00	56.54	51.46	-29.87	OK
131.0	Tension	62.03	65.27	58.79	-29.87	OK
132.0	Tension	72.19	76.18	68.19	-29.87	OK
133.0	Tension	95.80	101.19	90.40	-29.87	OK
134.0	Tension	108.85	113.31	104.39	-29.87	OK
135.0	Tension	128.83	128.83	128.83	-29.87	OK

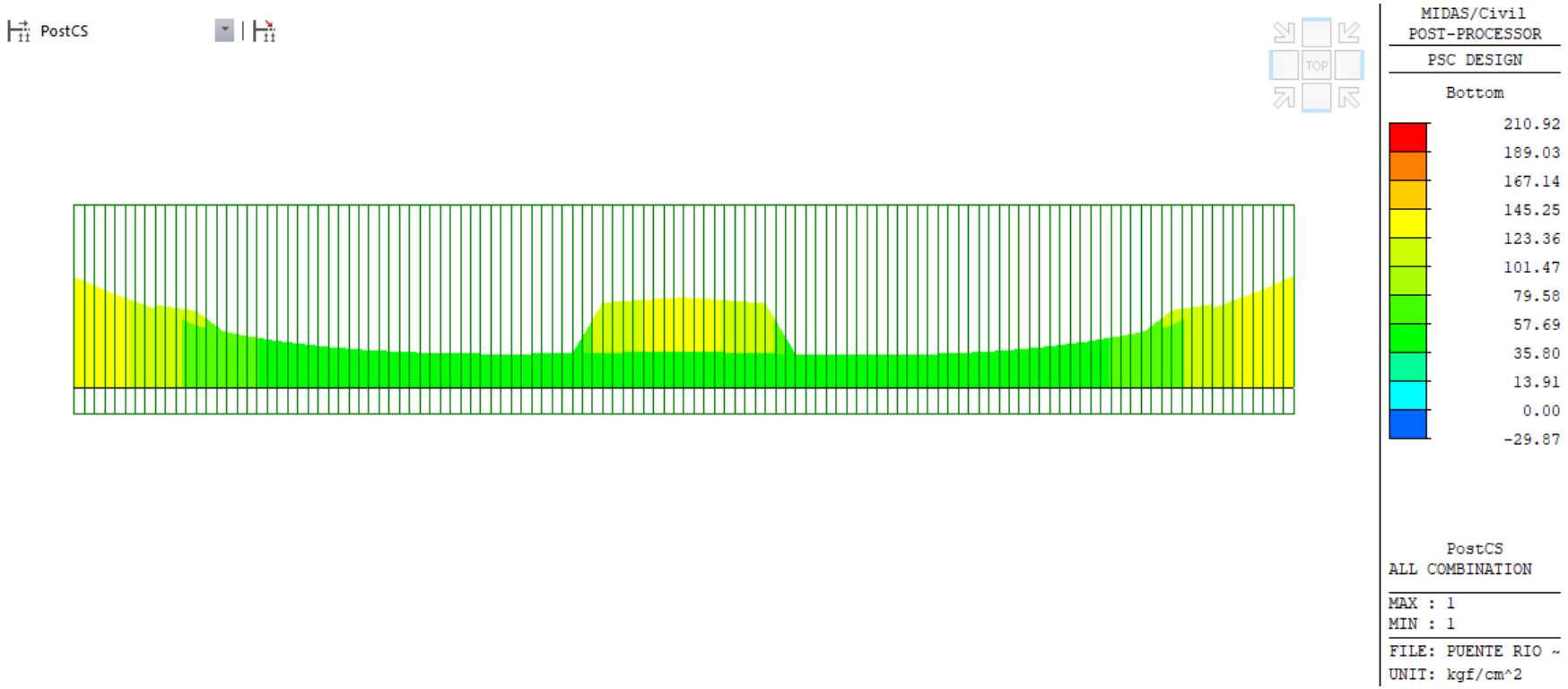


Figura 184. Verificación de esfuerzos a compresión y tracción en fibra inferior

3.4.21. **PERDIDAS INSTANTÁNEAS DE CABLES DE POSTENSADO**

CABLES C1

Elem	Force (tonf)	Force (After Immediate Loss) : A (tonf)	Elastic Deform. Loss : B (tonf)	Force(Elastic Loss)/ Force(Immediate Loss)	Creep/Shrinkage Loss (tonf)	Relaxation Loss (tonf)	Force(After All Loss)/ Force(After Immediate Loss)
The Loss of tendon group [L2-V1-C1] at the stage of [CS1]							
106.00	130.34	120.64	0.11	1.00	-2.70	-1.13	0.93
107.00	130.34	120.74	0.12	1.00	-2.70	-1.13	0.93
108.00	130.34	120.86	0.13	1.00	-2.72	-1.14	0.93
109.00	130.34	120.97	0.14	1.00	-2.75	-1.14	0.93
110.00	130.34	121.11	0.15	1.00	-2.79	-1.15	0.93
111.00	130.34	121.25	0.16	1.00	-2.85	-1.16	0.93
112.00	130.34	121.39	0.18	1.00	-2.90	-1.16	0.93
113.00	130.34	121.56	0.19	1.00	-2.97	-1.17	0.93
114.00	130.34	121.73	0.20	1.00	-3.03	-1.18	0.93
115.00	130.34	121.90	0.21	1.00	-3.10	-1.19	0.94
116.00	130.34	122.10	0.22	1.00	-3.16	-1.20	0.94
117.00	130.34	122.30	0.23	1.00	-3.21	-1.21	0.94
118.00	130.34	122.50	0.24	1.00	-3.25	-1.22	0.94
119.00	130.34	122.73	0.25	1.00	-3.27	-1.23	0.94
120.00	130.34	122.94	0.25	1.00	-3.28	-1.24	0.94
121.00	130.34	123.13	0.25	1.00	-3.26	-1.25	0.94
122.00	130.34	123.35	0.24	1.00	-3.22	-1.26	0.95
123.00	130.34	123.55	0.23	1.00	-3.17	-1.27	0.95
124.00	130.34	123.75	0.21	1.00	-3.11	-1.28	0.95
125.00	130.34	123.94	0.14	1.00	-3.05	-1.29	0.95
126.00	130.34	124.11	0.20	1.00	-2.99	-1.30	0.95
127.00	130.34	124.28	0.19	1.00	-2.94	-1.31	0.95

128.00	130.34	124.44	0.18	1.00	-2.88	-1.31	0.95
129.00	130.34	124.58	0.17	1.00	-2.83	-1.32	0.96
130.00	130.34	124.72	0.15	1.00	-2.79	-1.33	0.96
131.00	130.34	124.79	0.14	1.00	-2.75	-1.33	0.96
132.00	130.34	124.67	0.13	1.00	-2.73	-1.33	0.96
133.00	130.34	124.56	0.12	1.00	-2.71	-1.32	0.96
134.00	130.34	124.47	0.10	1.00	-2.71	-1.31	0.95
135.00	130.34	124.38	0.09	1.00	-2.72	-1.31	0.95

CABLES C2

Elem	Force (tonf)	Force (After Immediate Loss) : A (tonf)	Elastic Deform. Loss : B (tonf)	Force(Elastic Loss)/ Force(Immediate Loss)	Creep/Shrinkage Loss (tonf)	Relaxation Loss (tonf)	Force(After All Loss)/ Force(After Immediate Loss)
The Loss of tendon group [L2-V1-C2] at the stage of [CS1]							
106.00	130.34	122.36	0.08	1.00	-2.55	-1.21	0.94
107.00	130.34	122.47	0.09	1.00	-2.57	-1.22	0.94
108.00	130.34	122.60	0.10	1.00	-2.61	-1.22	0.94
109.00	130.34	122.74	0.11	1.00	-2.65	-1.23	0.94
110.00	130.34	122.91	0.12	1.00	-2.69	-1.24	0.94
111.00	130.34	123.10	0.14	1.00	-2.75	-1.25	0.94
112.00	130.34	123.30	0.15	1.00	-2.82	-1.26	0.95
113.00	130.34	123.53	0.17	1.00	-2.89	-1.27	0.95
114.00	130.34	123.78	0.18	1.00	-2.97	-1.28	0.95
115.00	130.34	124.03	0.20	1.00	-3.05	-1.29	0.95
116.00	130.34	123.88	0.21	1.00	-3.12	-1.29	0.95
117.00	130.34	123.56	0.22	1.00	-3.18	-1.27	0.95
118.00	130.34	123.26	0.23	1.00	-3.23	-1.25	0.95

119.00	130.34	122.90	0.24	1.00	-3.26	-1.24	0.94
120.00	130.34	122.57	0.24	1.00	-3.27	-1.22	0.94
121.00	130.34	122.19	0.23	1.00	-3.25	-1.20	0.94
122.00	130.34	121.83	0.22	1.00	-3.21	-1.18	0.93
123.00	130.34	121.52	0.21	1.00	-3.15	-1.17	0.93
124.00	130.34	121.20	0.21	1.00	-3.08	-1.15	0.93
125.00	130.34	120.90	0.25	1.00	-3.01	-1.14	0.93
126.00	130.34	120.65	0.17	1.00	-2.93	-1.13	0.93
127.00	130.34	120.39	0.15	1.00	-2.86	-1.12	0.92
128.00	130.34	120.15	0.14	1.00	-2.80	-1.10	0.92
129.00	130.34	119.95	0.13	1.00	-2.74	-1.09	0.92
130.00	130.34	119.75	0.12	1.00	-2.69	-1.08	0.92
131.00	130.34	119.57	0.11	1.00	-2.65	-1.08	0.92
132.00	130.34	119.42	0.10	1.00	-2.61	-1.07	0.92
133.00	130.34	119.28	0.09	1.00	-2.58	-1.06	0.92
134.00	130.34	119.16	0.08	1.00	-2.55	-1.06	0.91
135.00	130.34	119.07	0.08	1.00	-2.52	-1.05	0.91

CABLES C3

Elem	Force (tonf)	Force (After Immediate Loss) : A (tonf)	Elastic Deform. Loss : B (tonf)	Force(Elastic Loss)/ Force(Immediate Loss)	Creep/Shrinkage Loss (tonf)	Relaxation Loss (tonf)	Force(After All Loss)/ Force(After Immediate Loss)
The Loss of tendon group [L2-V1-C3] at the stage of [CS1]							
106.00	130.34	118.03	0.07	1.00	-2.39	-1.00	0.91
107.00	130.34	118.17	0.08	1.00	-2.44	-1.01	0.91
108.00	130.34	118.33	0.09	1.00	-2.50	-1.02	0.91

109.00	130.34	118.51	0.10	1.00	-2.54	-1.03	0.91
110.00	130.34	118.74	0.11	1.00	-2.59	-1.04	0.91
111.00	130.34	119.00	0.12	1.00	-2.66	-1.05	0.91
112.00	130.34	119.25	0.14	1.00	-2.73	-1.06	0.91
113.00	130.34	119.57	0.16	1.00	-2.82	-1.08	0.92
114.00	130.34	119.92	0.17	1.00	-2.91	-1.09	0.92
115.00	130.34	120.26	0.19	1.00	-3.00	-1.11	0.92
116.00	130.34	120.67	0.21	1.00	-3.09	-1.13	0.93
117.00	130.34	121.11	0.22	1.00	-3.16	-1.15	0.93
118.00	130.34	121.52	0.23	1.00	-3.22	-1.17	0.93
119.00	130.34	122.02	0.24	1.00	-3.26	-1.19	0.94
120.00	130.34	122.48	0.24	1.00	-3.28	-1.22	0.94
121.00	130.34	121.89	0.24	1.00	-3.25	-1.19	0.94
122.00	130.34	121.39	0.23	1.00	-3.20	-1.16	0.93
123.00	130.34	120.98	0.22	1.00	-3.13	-1.14	0.93
124.00	130.34	120.54	0.20	1.00	-3.05	-1.12	0.92
125.00	130.34	120.13	0.18	1.00	-2.96	-1.10	0.92
126.00	130.34	119.79	0.17	1.00	-2.88	-1.09	0.92
127.00	130.34	119.44	0.15	1.00	-2.79	-1.07	0.92
128.00	130.34	119.12	0.14	1.00	-2.72	-1.06	0.91
129.00	130.34	118.87	0.12	1.00	-2.65	-1.04	0.91
130.00	130.34	118.61	0.11	1.00	-2.59	-1.03	0.91
131.00	130.34	118.38	0.10	1.00	-2.54	-1.02	0.91
132.00	130.34	118.20	0.09	1.00	-2.50	-1.01	0.91
133.00	130.34	118.04	0.08	1.00	-2.46	-1.00	0.91
134.00	130.34	117.90	0.08	1.00	-2.39	-1.00	0.90
135.00	130.34	117.80	0.07	1.00	-2.33	-0.99	0.90

CABLES C4

Elem	Force (tonf)	Force (After Immediate Loss) : A (tonf)	Elastic Deform. Loss : B (tonf)	Force(Elastic Loss)/ Force(Immediate Loss)	Creep/Shrinkage Loss (tonf)	Relaxation Loss (tonf)	Force(After All Loss)/ Force(After Immediate Loss)
The Loss of tendon group [L2-V1-C4] at the stage of [CS2]							
108.00	130.34	116.15	0.01	1.00	-0.94	-0.92	0.89
109.00	130.34	116.32	0.02	1.00	-0.87	-0.93	0.89
110.00	130.34	116.56	0.02	1.00	-0.80	-0.94	0.89
111.00	130.34	116.82	0.02	1.00	-0.73	-0.95	0.90
112.00	130.34	117.17	0.02	1.00	-0.66	-0.97	0.90
113.00	130.34	117.58	0.03	1.00	-0.61	-0.98	0.90
114.00	130.34	117.99	0.03	1.00	-0.57	-1.00	0.91
115.00	130.34	118.51	0.03	1.00	-0.53	-1.03	0.91
116.00	130.34	119.09	0.03	1.00	-0.51	-1.05	0.91
117.00	130.34	119.73	0.04	1.00	-0.50	-1.08	0.92
118.00	130.34	120.34	0.04	1.00	-0.50	-1.11	0.92
119.00	130.34	121.08	0.04	1.00	-0.50	-1.15	0.93
120.00	130.34	121.79	0.04	1.00	-0.50	-1.18	0.93
121.00	130.34	120.90	0.04	1.00	-0.50	-1.14	0.93
122.00	130.34	120.16	0.04	1.00	-0.49	-1.10	0.92
123.00	130.34	119.55	0.04	1.00	-0.50	-1.08	0.92
124.00	130.34	118.91	0.03	1.00	-0.51	-1.05	0.91
125.00	130.34	118.33	0.03	1.00	-0.53	-1.02	0.91
126.00	130.34	117.81	0.03	1.00	-0.57	-0.99	0.90
127.00	130.34	117.39	0.03	1.00	-0.61	-0.98	0.90
128.00	130.34	116.99	0.02	1.00	-0.66	-0.96	0.90
129.00	130.34	116.64	0.02	1.00	-0.73	-0.94	0.89
130.00	130.34	116.37	0.02	1.00	-0.80	-0.93	0.89

131.00	130.34	116.14	0.02	1.00	-0.87	-0.92	0.89
132.00	130.34	115.96	0.02	1.00	-0.94	-0.91	0.89
133.00	130.34	115.85	0.01	1.00	-1.01	-0.91	0.89

Elem	Force (After Immediate Loss) : A (tonf)	Force (After All Loss) (tonf)	Elastic Deform. Loss : B (tonf)	Force(Elastic Loss)/ Force(Immediate Loss)	Creep/Shrinkage Loss (tonf)	Relaxation Loss (tonf)	Force(After All Loss)/ Force(After Immediate Loss)
108.00	251.66	226.12	0.19	1.00	-21.63	-4.10	0.90
109.00	251.89	226.42	0.30	1.00	-21.65	-4.13	0.90
110.00	252.20	224.46	0.70	1.00	-24.37	-4.06	0.89
111.00	252.57	225.49	0.93	1.00	-23.88	-4.13	0.89
112.00	253.05	226.50	1.23	1.00	-23.56	-4.21	0.90
113.00	253.55	227.44	1.59	1.01	-23.42	-4.28	0.90
114.00	254.20	228.37	1.99	1.01	-23.47	-4.36	0.90
115.00	254.94	229.21	2.43	1.01	-23.70	-4.45	0.90
116.00	255.65	229.91	2.88	1.01	-24.10	-4.52	0.90
117.00	256.55	230.62	3.34	1.01	-24.65	-4.61	0.90
118.00	257.40	231.18	3.79	1.01	-25.32	-4.69	0.90
119.00	258.47	231.82	4.21	1.02	-26.08	-4.79	0.90
120.00	259.45	232.31	4.60	1.02	-26.87	-4.87	0.90
121.00	260.68	232.94	4.94	1.02	-27.68	-4.98	0.89
122.00	261.79	233.46	5.21	1.02	-28.46	-5.08	0.89
123.00	263.17	234.22	5.41	1.02	-29.17	-5.20	0.89
124.00	263.97	234.45	5.52	1.02	-29.76	-5.27	0.89
125.00	262.44	232.71	5.52	1.02	-30.16	-5.09	0.89
126.00	260.84	230.90	5.39	1.02	-30.41	-4.91	0.89
127.00	259.49	230.07	5.51	1.02	-30.17	-4.78	0.89
128.00	258.00	229.10	5.51	1.02	-29.75	-4.64	0.89
129.00	256.79	228.49	5.40	1.02	-29.15	-4.54	0.89
130.00	255.46	227.80	5.20	1.02	-28.44	-4.43	0.89
131.00	254.40	227.31	4.92	1.02	-27.67	-4.34	0.89
132.00	253.25	226.73	4.59	1.02	-26.87	-4.24	0.90
133.00	252.32	226.28	4.20	1.02	-26.09	-4.17	0.90

134.00	251.33	225.70	3.78	1.02	-25.34	-4.08	0.90
135.00	250.55	225.19	3.33	1.01	-24.68	-4.01	0.90
136.00	249.72	224.54	2.88	1.01	-24.13	-3.93	0.90
137.00	249.06	223.91	2.42	1.01	-23.73	-3.86	0.90
138.00	248.39	223.11	1.99	1.01	-23.49	-3.79	0.90
139.00	247.81	222.23	1.58	1.01	-23.45	-3.72	0.90
140.00	247.36	221.33	1.22	1.00	-23.60	-3.65	0.89
141.00	246.92	220.33	0.91	1.00	-23.92	-3.58	0.89
142.00	246.60	221.84	0.46	1.00	-21.61	-3.62	0.90
143.00	246.32	221.36	0.26	1.00	-21.64	-3.59	0.90

3.4.22. **PERDIDAS A LARGO PLAZO DE CABLES DE POSTENSADO**

CABLES C1

Elem	Force (tonf)	Force (After Immediate Loss) : A (tonf)	Force (After All Loss) (tonf)	Elastic Deform. Loss : B (tonf)	Force(Elastic Loss)/ Force(Immediate Loss)	Creep/Shrinkage Loss (tonf)	Relaxation Loss (tonf)	Force(After All Loss)/ Force(After Immediate Loss)
The Loss of tendon group [L2-V1-C1] at the stage of [CS22]								
106.00	130.34	120.64	115.47	0.63	1.01	-12.24	-2.17	0.89
107.00	130.34	120.74	116.15	0.91	1.01	-11.85	-2.20	0.89
108.00	130.34	120.86	116.00	0.87	1.01	-11.96	-2.20	0.89
109.00	130.34	120.97	116.12	0.95	1.01	-11.93	-2.22	0.89
110.00	130.34	121.11	116.16	1.01	1.01	-11.96	-2.23	0.89
111.00	130.34	121.25	116.12	1.05	1.01	-12.04	-2.25	0.89
112.00	130.34	121.39	116.04	1.08	1.01	-12.15	-2.26	0.89
113.00	130.34	121.56	115.91	1.10	1.01	-12.28	-2.27	0.89
114.00	130.34	121.73	115.77	1.11	1.01	-12.44	-2.28	0.89

115.00	130.34	121.90	115.61	1.12	1.01	-12.60	-2.29	0.89
116.00	130.34	122.10	115.47	1.12	1.01	-12.74	-2.31	0.89
117.00	130.34	122.30	115.34	1.12	1.01	-12.87	-2.32	0.88
118.00	130.34	122.50	115.23	1.12	1.01	-12.98	-2.34	0.88
119.00	130.34	122.73	115.16	1.12	1.01	-13.05	-2.36	0.88
120.00	130.34	122.94	115.13	1.12	1.01	-13.09	-2.38	0.88
121.00	130.34	123.13	115.18	1.11	1.01	-13.03	-2.40	0.88
122.00	130.34	123.35	115.27	1.11	1.01	-12.94	-2.43	0.88
123.00	130.34	123.55	115.36	1.09	1.01	-12.84	-2.46	0.89
124.00	130.34	123.75	115.39	1.03	1.01	-12.74	-2.48	0.89
125.00	130.34	123.94	115.27	0.87	1.01	-12.70	-2.50	0.88
126.00	130.34	124.11	115.74	1.05	1.01	-12.42	-2.53	0.89
127.00	130.34	124.28	115.98	1.10	1.01	-12.23	-2.56	0.89
128.00	130.34	124.44	116.11	1.09	1.01	-12.10	-2.58	0.89
129.00	130.34	124.58	116.17	1.06	1.01	-12.00	-2.60	0.89
130.00	130.34	124.72	116.19	1.02	1.01	-11.94	-2.62	0.89
131.00	130.34	124.79	116.13	0.95	1.01	-11.93	-2.62	0.89
132.00	130.34	124.67	115.99	0.86	1.01	-11.98	-2.61	0.89
133.00	130.34	124.56	115.75	0.74	1.01	-12.08	-2.59	0.89
134.00	130.34	124.47	115.44	0.62	1.01	-12.28	-2.56	0.89
135.00	130.34	124.38	114.71	0.33	1.00	-12.71	-2.52	0.88

CABLES C2

Elem	Force (tonf)	Force (After Immediate Loss) : A (tonf)	Force (After All Loss) (tonf)	Elastic Deform. Loss : B (tonf)	Force(Elastic Loss)/ Force(Immediate Loss)	Creep/Shrinkage Loss (tonf)	Relaxation Loss (tonf)	Force(After All Loss)/ Force(After Immediate Loss)
The Loss of tendon group [L2-V1-C2] at the stage of [CS22]								
106.00	130.34	122.36	115.65	0.42	1.00	-11.86	-2.35	0.89

107.00	130.34	122.47	116.02	0.60	1.00	-11.69	-2.37	0.89
108.00	130.34	122.60	115.56	0.49	1.00	-12.03	-2.37	0.89
109.00	130.34	122.74	115.69	0.60	1.00	-12.01	-2.39	0.89
110.00	130.34	122.91	115.78	0.70	1.01	-12.02	-2.41	0.89
111.00	130.34	123.10	115.82	0.79	1.01	-12.07	-2.43	0.89
112.00	130.34	123.30	115.82	0.87	1.01	-12.16	-2.45	0.89
113.00	130.34	123.53	115.77	0.93	1.01	-12.28	-2.47	0.89
114.00	130.34	123.78	115.68	0.99	1.01	-12.42	-2.49	0.89
115.00	130.34	124.03	115.56	1.03	1.01	-12.58	-2.52	0.89
116.00	130.34	123.88	115.43	1.05	1.01	-12.73	-2.49	0.89
117.00	130.34	123.56	115.29	1.07	1.01	-12.88	-2.45	0.88
118.00	130.34	123.26	115.17	1.07	1.01	-13.00	-2.42	0.88
119.00	130.34	122.90	115.05	1.05	1.01	-13.10	-2.37	0.88
120.00	130.34	122.57	114.96	1.03	1.01	-13.15	-2.34	0.88
121.00	130.34	122.19	115.04	1.04	1.01	-13.08	-2.30	0.88
122.00	130.34	121.83	115.14	1.04	1.01	-12.98	-2.27	0.88
123.00	130.34	121.52	115.29	1.04	1.01	-12.83	-2.24	0.88
124.00	130.34	121.20	115.51	1.06	1.01	-12.64	-2.22	0.89
125.00	130.34	120.90	115.92	1.19	1.01	-12.36	-2.20	0.89
126.00	130.34	120.65	115.72	0.98	1.01	-12.34	-2.17	0.89
127.00	130.34	120.39	115.72	0.89	1.01	-12.25	-2.14	0.89
128.00	130.34	120.15	115.75	0.83	1.01	-12.15	-2.12	0.89
129.00	130.34	119.95	115.77	0.76	1.01	-12.07	-2.10	0.89
130.00	130.34	119.75	115.73	0.68	1.01	-12.03	-2.08	0.89
131.00	130.34	119.57	115.64	0.59	1.01	-12.02	-2.06	0.89
132.00	130.34	119.42	115.51	0.49	1.00	-12.04	-2.04	0.89
133.00	130.34	119.28	115.35	0.38	1.00	-12.08	-2.02	0.89
134.00	130.34	119.16	115.62	0.43	1.00	-11.87	-2.02	0.89
135.00	130.34	119.07	115.29	0.26	1.00	-12.02	-2.00	0.88

CABLES C3

Elem	Force (tonf)	Force (After Immediate Loss) : A (tonf)	Force (After All Loss) (tonf)	Elastic Deform. Loss : B (tonf)	Force(Elastic Loss)/ Force(Immediate Loss)	Creep/Shrinkage Loss (tonf)	Relaxation Loss (tonf)	Force(After All Loss)/ Force(After Immediate Loss)
The Loss of tendon group [L2-V1-C3] at the stage of [CS22]								
106.00	130.34	118.03	115.90	0.27	1.00	-11.43	-1.92	0.89
107.00	130.34	118.17	116.07	0.42	1.00	-11.43	-1.93	0.89
108.00	130.34	118.33	115.32	0.25	1.00	-11.96	-1.92	0.88
109.00	130.34	118.51	115.49	0.39	1.00	-11.95	-1.94	0.89
110.00	130.34	118.74	115.61	0.52	1.00	-11.96	-1.97	0.89
111.00	130.34	119.00	115.69	0.63	1.01	-12.01	-2.00	0.89
112.00	130.34	119.25	115.72	0.74	1.01	-12.09	-2.03	0.89
113.00	130.34	119.57	115.69	0.83	1.01	-12.21	-2.06	0.89
114.00	130.34	119.92	115.62	0.91	1.01	-12.36	-2.09	0.89
115.00	130.34	120.26	115.52	0.97	1.01	-12.52	-2.12	0.89
116.00	130.34	120.67	115.40	1.02	1.01	-12.69	-2.16	0.89
117.00	130.34	121.11	115.29	1.06	1.01	-12.84	-2.20	0.88
118.00	130.34	121.52	115.18	1.08	1.01	-12.97	-2.24	0.88
119.00	130.34	122.02	115.09	1.08	1.01	-13.07	-2.28	0.88
120.00	130.34	122.48	115.04	1.08	1.01	-13.12	-2.33	0.88
121.00	130.34	121.89	115.10	1.07	1.01	-13.05	-2.27	0.88
122.00	130.34	121.39	115.19	1.06	1.01	-12.94	-2.22	0.88
123.00	130.34	120.98	115.30	1.03	1.01	-12.80	-2.19	0.88
124.00	130.34	120.54	115.42	0.98	1.01	-12.64	-2.15	0.89
125.00	130.34	120.13	115.52	0.93	1.01	-12.48	-2.11	0.89
126.00	130.34	119.79	115.62	0.87	1.01	-12.32	-2.08	0.89

127.00	130.34	119.44	115.69	0.80	1.01	-12.18	-2.05	0.89
128.00	130.34	119.12	115.72	0.72	1.01	-12.08	-2.01	0.89
129.00	130.34	118.87	115.68	0.62	1.01	-12.00	-1.99	0.89
130.00	130.34	118.61	115.60	0.51	1.00	-11.96	-1.96	0.89
131.00	130.34	118.38	115.48	0.39	1.00	-11.95	-1.93	0.89
132.00	130.34	118.20	115.32	0.26	1.00	-11.97	-1.91	0.88
133.00	130.34	118.04	115.16	0.12	1.00	-11.99	-1.88	0.88
134.00	130.34	117.90	115.94	0.30	1.00	-11.42	-1.90	0.89
135.00	130.34	117.80	115.92	0.19	1.00	-11.32	-1.89	0.89

CABLES C4

Elem	Force (tonf)	Force (After Immediate Loss) : A (tonf)	Force (After All Loss) (tonf)	Elastic Deform. Loss : B (tonf)	Force(Elastic Loss)/ Force(Immediate Loss)	Creep/Shrinkage Loss (tonf)	Relaxation Loss (tonf)	Force(After All Loss)/ Force(After Immediate Loss)
The Loss of tendon group [L2-V1-C4] at the stage of [CS22]								
108.00	130.34	116.15	117.68	0.18	1.00	-9.64	-1.82	0.90
109.00	130.34	116.32	117.79	0.22	1.00	-9.58	-1.84	0.90
110.00	130.34	116.56	117.89	0.27	1.00	-9.52	-1.87	0.90
111.00	130.34	116.82	117.98	0.31	1.00	-9.48	-1.90	0.91
112.00	130.34	117.17	118.06	0.35	1.00	-9.46	-1.94	0.91
113.00	130.34	117.58	118.10	0.40	1.00	-9.46	-1.98	0.91
114.00	130.34	117.99	118.10	0.44	1.00	-9.49	-2.03	0.91
115.00	130.34	118.51	118.09	0.48	1.00	-9.54	-2.08	0.91
116.00	130.34	119.09	118.06	0.52	1.00	-9.60	-2.14	0.91
117.00	130.34	119.73	118.01	0.55	1.00	-9.67	-2.20	0.91
118.00	130.34	120.34	117.96	0.58	1.00	-9.74	-2.27	0.91
119.00	130.34	121.08	117.92	0.59	1.00	-9.79	-2.34	0.90
120.00	130.34	121.79	117.88	0.60	1.00	-9.83	-2.41	0.90

121.00	130.34	120.90	117.92	0.58	1.00	-9.79	-2.32	0.90
122.00	130.34	120.16	117.96	0.56	1.00	-9.73	-2.25	0.91
123.00	130.34	119.55	118.00	0.53	1.00	-9.67	-2.18	0.91
124.00	130.34	118.91	118.02	0.49	1.00	-9.60	-2.12	0.91
125.00	130.34	118.33	118.05	0.45	1.00	-9.54	-2.06	0.91
126.00	130.34	117.81	118.06	0.41	1.00	-9.50	-2.01	0.91
127.00	130.34	117.39	118.06	0.38	1.00	-9.47	-1.96	0.91
128.00	130.34	116.99	118.04	0.34	1.00	-9.47	-1.92	0.91
129.00	130.34	116.64	117.97	0.30	1.00	-9.49	-1.88	0.91
130.00	130.34	116.37	117.89	0.26	1.00	-9.53	-1.85	0.90
131.00	130.34	116.14	117.80	0.23	1.00	-9.58	-1.83	0.90
132.00	130.34	115.96	117.71	0.19	1.00	-9.62	-1.80	0.90
133.00	130.34	115.85	117.64	0.15	1.00	-9.65	-1.79	0.90

3.4.23. **ALARGAMIENTO DE CABLES DE POSTENSADO**

Se calcularán los alargamientos en los cables luego del tensionamiento, para el cálculo de alargamientos se consideran únicamente las pérdidas instantáneas, utilizando la expresión:

$$\delta = \frac{P * L}{A * E}$$

	Tendon Name	Tendon Elongation		Element Elongation		Summation	
		Begin (cm)	End (cm)	Begin (cm)	End (cm)	Begin (cm)	End (cm)
	L2-V1-C1	19.5509	0.0000	0.0837	0.0000	19.6347	0.0000
	L2-V1-C2	0.0000	19.4046	0.0000	0.0830	0.0000	19.4877
	L2-V1-C3	9.8848	9.8154	0.0756	0.0751	9.9604	9.8905
	L2-V1-C4	8.5730	8.5014	0.0325	0.0322	8.6055	8.5336
	L2-V2-C1	19.5510	0.0000	0.0837	0.0000	19.6347	0.0000
	L2-V2-C2	0.0000	19.4046	0.0000	0.0830	0.0000	19.4877
	L2-V2-C3	9.8848	9.8154	0.0756	0.0751	9.9604	9.8905
	L2-V2-C4	8.5730	8.5014	0.0325	0.0322	8.6055	8.5336
	L2-V3-C1	19.5510	0.0000	0.0837	0.0000	19.6347	0.0000
	L2-V3-C2	0.0000	19.4046	0.0000	0.0830	0.0000	19.4877
	L2-V3-C3	9.8848	9.8154	0.0756	0.0751	9.9604	9.8905
	L2-V3-C4	8.5730	8.5014	0.0325	0.0322	8.6055	8.5336
	L2-V4-C1	19.5510	0.0000	0.0837	0.0000	19.6347	0.0000
	L2-V4-C2	0.0000	19.4046	0.0000	0.0830	0.0000	19.4877
	L2-V4-C3	9.8848	9.8154	0.0756	0.0751	9.9604	9.8905
	L2-V4-C4	8.5730	8.5014	0.0325	0.0322	8.6055	8.5336
	L2-V5-C1	19.5510	0.0000	0.0837	0.0000	19.6347	0.0000
	L2-V5-C2	0.0000	19.4046	0.0000	0.0830	0.0000	19.4877
	L2-V5-C3	9.8848	9.8154	0.0756	0.0751	9.9604	9.8905
	L2-V5-C4	8.5730	8.5014	0.0325	0.0322	8.6055	8.5336

Figura 185. Alargamientos teóricos de cables de postensado

3.4.24. REVISIÓN DE CAPACIDAD A FLEXIÓN, CORTANTE Y TORSIÓN

Se verifica la sección crítica en el centro de la luz, es decir a 10.50 m.

Se revisa la capacidad del puente teniendo en cuenta la resistencia a flexión de los cables de postensado, la capacidad del puente se tomó como:

$$\emptyset M_n = \emptyset * A_{ps} * F_{ps} * \left(d_p - \frac{a_p}{2}\right)$$

Donde:

$\boldsymbol{A_{ps}}$ Área de acero presforzado.

$\boldsymbol{f_{ps}}$ Tensiones efectivas de presfuerzo, calculadas según el 5.7.3.2.2)

$\boldsymbol{d_p}$ Profundidad efectiva de presforzado, distancia desde la fibra extrema a compresión al centroide del acero de presfuerzo, debe evaluarse si la sección está sometida a un momento positivo o negativo.

$\boldsymbol{a}$ Profundidad del bloque de compresión del concreto, calculada como $a_p = \beta * C_p$

Siendo C la distancia del eje neutro a la fibra extrema a compresión.

$$C_p = \frac{A_{ps} * f_{pu} + A_s * fs - A's * f's}{0.85 * f'c * \beta * b + k * A_{ps} * \frac{f_{pu}}{d_p}}$$

1. Design Condition

Design Code	Element	Node(I/J)	Section
AASHTO-LRFD2016	121	I	Composite

■ Section Properties

- Gross section

		Before	After
H	(mm)	1610.0	1910.0
B	(mm)	1070.0	2840.0
C_{zp}	(mm)	785.2	341.9
C_{zm}	(mm)	824.8	1268.1
C_{zps}	(mm)		641.9
C_{zms}	(mm)		391.9
A_g	(mm^2)	6.759.E+05	1.256.E+06
I_y	(mm^4)	2.234.E+11	5.142.E+11
S_t	(mm^3)	2.845.E+08	6.533.E+08
S_b	(mm^3)	2.708.E+08	1.762.E+08
S_{ts}	(mm^3)		8.010.E+08
S_{bs}	(mm^3)		1.312.E+09

- Transformed section

		Before	After
H	(mm)	1610.0	1910.0
B	(mm)	1070.0	2840.0
C_{zp}	(mm)	809.5	371.0
C_{zm}	(mm)	800.5	1239.0
C_{zps}	(mm)		671.0
C_{zms}	(mm)		421.0
A_g	(mm^2)	7.219.E+05	1.302.E+06
I_y	(mm^4)	2.426.E+11	5.572.E+11
S_t	(mm^3)	2.996.E+08	6.537.E+08
S_b	(mm^3)	3.030.E+08	1.958.E+08
S_{ts}	(mm^3)		7.662.E+08
S_{bs}	(mm^3)		1.221.E+09

■ Materials

- Concrete

	f'_c (MPa)	E_c (Mpa)	$f_r=0.20\sqrt{f'_c}$ (Mpa)	β_1
Girder	34.474	28091.2	3.083	0.800
Slab	27.579	25125.5	2.758	0.850

* β_1 is 0.85 in lower f'c 4ksi, the others are 0.85-0.05(f'c-4.0)≥0.65

- Prestressing steel Information

No.	Tendon Name	Bond Type	d_p (mm)	A_{ps} (mm^2)	Strength (MPa)		E_p (MPa)
					f_{py}	f_{pu}	
1	S_L2-V1-C4	Bond	1710.1	970.9	1676.9	1863.3	196500.6

2	S_L2-V1-C3	Bond	1810.1	970.9	1676.9	1863.3	196500.6
3	S_L2-V1-C2	Bond	1810.1	970.9	1676.9	1863.3	196500.6
4	S_L2-V1-C1	Bond	1810.2	970.9	1676.9	1863.3	196500.6

* d_p is Distance from extreme compression fiber to centroid of prestressing tendon.

- Longitudinal non prestressed steel reinforcement Information.

Girder		Slab		Bottom for Flexure		Top for Flexure		Torsion
Es (MPa)	fy (MPa)	Es (MPa)	fy (MPa)	ds (mm)	As (mm^2)	ds (mm)	As (mm^2)	A_l (mm^2)
199948.0	413.7	199948.0	413.7	1139.6	3709.7	0.0	0.0	3709.7

* d_s or d'_s is Distance from extreme compression fiber to centroid of non prestressing reinforcement.

- Tranverse non prestressed steel reinforcement Information.

E_s (MPa)	f_y (MPa)	α (deg.)	Shear		Torsion	
			A_v (mm^2)	s (mm)	A_t (mm^2)	s_t (mm)
199948.0	413.7	90.0	258.1	100.0	258.1	100.0

* α is angle between longitudinal and stirrup.

■ **Sectional forces/stresses due to effective prestress**

No.	Tendon Name	Bond Type	e_p (mm)	$A_{ps}f_{e(z\text{-}dir)}$ (kN)	$A_{ps}f_{e(x\text{-}dir)}$ (kN)	$A_{ps}f_{e(x\text{-}dir)}e_p$ (kN•m)
1	S_L2-V1-C4	Bond	-1039.1	0.00	1079.32	1121.54
2	S_L2-V1-C3	Bond	-1139.1	0.00	1059.62	1207.03
3	S_L2-V1-C2	Bond	-1139.1	0.00	1060.24	1207.72
4	S_L2-V1-C1	Bond	-1139.1	0.00	1064.77	1212.89
			Σ	0.00	4263.95	4749.17

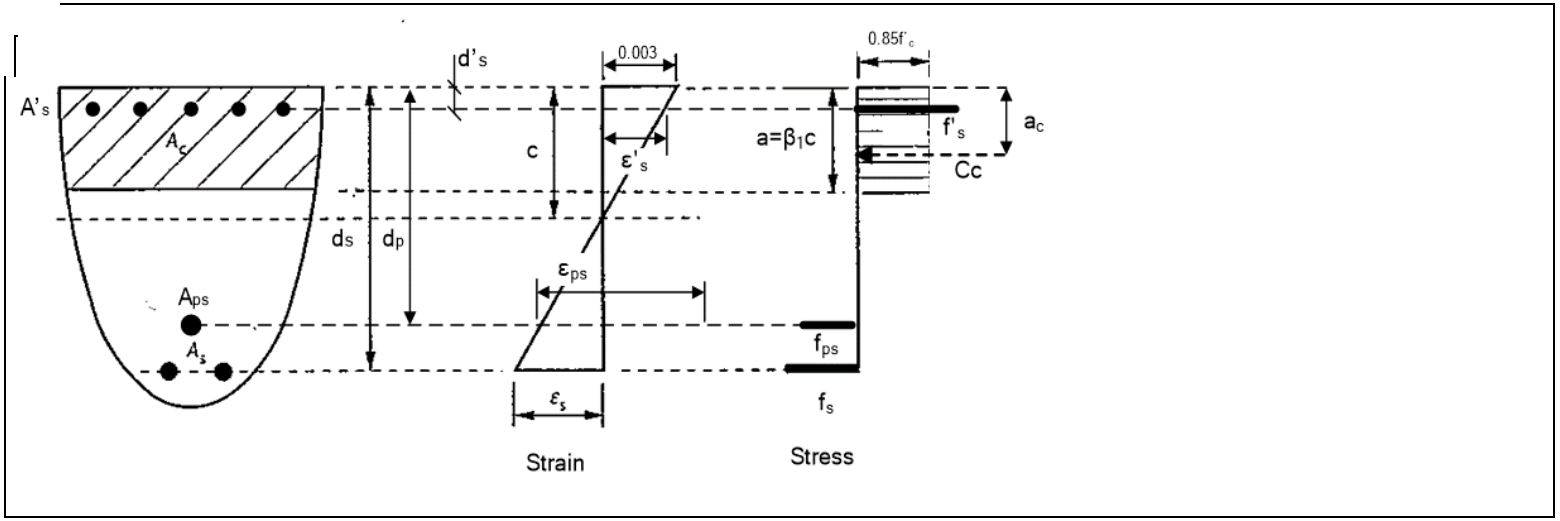
* e_p is Distance from centroid of section to centroid of tendon

2.Flexure design for a section

■ **Moment Direction :** **Positive**

- Method of Calculation : Code
- The maximum Strength Limit Load Combination : cLCB1
- The maximum factored moment (M_u) : 10684.849 (kN•m)

1) Depth of neutral axis to compression face (c). *(see. 5.7.3.1)*



Axial force in concrete (compressive zone)					
c	$a=\beta_1 c$	A_c	a_c	$C_c=0.85f'_cA_c$ (kN)	$C_c(c-a_c)$ (kN•m)
152.017	129.214	366968.08	64.61	8602.53	751.94

Axial force in reinforcement steels			
tensile zone		compressive zone	
$T_s=A_sf_s$ (kN)	$\Sigma A_sf_s(d_s-c)$	$C_s=A'_sf'_s$ (kN)	$\Sigma A'_sf'_s(c-d'_s)$ (kN•m)

1534.64	1515.58	0.00	0.00

No.	Axial force in tendons(Bond) by Code				
	Tendon Name	k	f_{ps} (MPa)	$T_{ps}=A_{ps}f_{ps}$ (kN)	$A_{ps}f_{ps}(d_p-c)$ (kN•m)
1	S_L2-V1-C4	0.280	1816.89	1764.02	2748.57
2	S_L2-V1-C3	0.280	1819.45	1766.50	2929.10
3	S_L2-V1-C2	0.280	1819.45	1766.50	2929.10
4	S_L2-V1-C1	0.280	1819.45	1766.50	2929.10
			Σ	7063.53	11535.87

* $f_{ps} = f_{pu}(1-k\ c/d_p)$ $\quad k = 2(1.04-f_{py}/f_{pu})$

c (mm)	Compression Force (kN)		Tensile Force (kN)		Tolerance (C/T)
	C_c	C_s	T_s	T_{ps}	
152.017	8602.53	0.00	1534.64	7063.53	1.00051

2) Flexural resistance

- Nominal resistance(Mn). *(see. 5.7.3.2)*

$M_n = \Sigma A_s f_s(d_s-c)+\Sigma A_{ps}f_{ps}(d_p-c)+C_c(c-a_c)+\Sigma A'_s f'_s(c-d'_s) =$ 13803.39 (kN•m)

- Resistance factor(Φ).

$d_t =$ 1860.0 (mm)

$\varepsilon_t = 0.003(d_t/c-1) =$ 0.03371 : Net tensile strain in the extreme tension steel

$\therefore\ \Phi =$ 1.00 : Tension Controlled Range, $0.005 \le \varepsilon_t$

- Factored resistance($M_r = \Phi M_n$)

$M_r =$ 13803.39 (kN•m) ≥ $M_u =$ 10684.85 (kN•m) **OK**

3) Minimum reinforcement

▪ Judgement *(see. 5.7.3.3.2)*

M_r = 13803.39 (kN•m) < $1.33M_u$ = 14210.85 (kN•m)

∴ check mimimum reinforcement

▪ Cracking moment.(M_{cr}) *(Eq. 5.7.3.3.2-1)*

$M_{cr} = \gamma_3 [(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}) S_c - M_{dnc}(S_c/S_{nc}-1)]$ = 8764.89 (kN•m)

where,

M_{dnc} = 1.918E+03 (kN•m) : *Non-Composite dead load moment*

S_c = 4.497E+08 (mm^3) : *Composite Section Modulus for extreme fiber*

S_{nc} = 3.030E+08 (mm^3) : *Non-Composite Section Modulus for extreme fiber*

γ_1 = 1.60 : *flexural cracking variability factor*

γ_2 = 1.10 : *Prestress variability factor*

γ_3 = 1.00 : *Ratio of yield strength to ultimate tensile strength of the reinforcement*

f_{cpe} = *Compressive stress in concrete due to effective prestress only.*

$= \Sigma A_{ps} \cdot f_{e(x\text{-}dir)} / A_g + \Sigma A_{ps} \cdot f_{e(x\text{-}dir)} \cdot e_p / S_b$ = 15.11 (MPa)

▪ Checking minimum reinforcement

M_r = 13803.39 (kN•m) ≥ M_{cr} = 8764.89 (kN•m) **OK**

■ **Moment Direction :** **Negative**

- Method of Calculation : Code
- The maximum Strength Limit Load Combination : cLCB3
- The maximum factored moment (M_u) : 0.000 (kN•m)

1) Depth of neutral axis to compression face (c). *(see. 5.7.3.1)*

Axial force in concrete (compressive zone)					
c	$a=\beta_1 c$	A_c	a_c	$C_c=0.85f'_cA_c$ (kN)	$C_c(c-a_c)$ (kN•m)
244.346	207.694	147404.62	103.81	3455.49	485.63

Axial force in reinforcement steels			
tensile zone		compressive zone	
$T_s=A_sf_s$ (kN)	$\Sigma A_sf_s(d_s-c)$	$C_s=A'_sf'_s$ (kN)	$\Sigma A'_sf'_s(c-d'_s)$ (kN•m)
1014.19	894.62	454.71	83.06

No.	Axial force in tendons(Bond) by Code				
	Tendon Name	k	f_{ps} (MPa)	$T_{ps}=A_{ps}f_{ps}$ (kN)	$A_{ps}f_{ps}(d_p-c)$ (kN•m)
1	S_L2-V1-C4	0.280	1225.39	1189.73	-52.94
2	S_L2-V1-C3	0.280	586.56	569.49	-82.29
3	S_L2-V1-C2	0.280	586.56	569.49	-82.29
4	S_L2-V1-C1	0.280	586.56	569.49	-82.29
			Σ	2898.22	-299.81

* $f_{ps} = f_{pu}(1-k\ c/d_p)$ $\quad k = 2(1.04-f_{py}/f_{pu})$

c (mm)	Compression Force (kN)		Tensile Force (kN)		Tolerance (C/T)
	C_c	C_s	T_s	T_{ps}	
244.346	3455.49	454.71	1014.19	2898.22	0.99943

2) Flexural resistance

- Nominal resistance(Mn). *(see. 5.7.3.2)*

  $M_n = \Sigma A_sf_s(d_s-c)+\Sigma A_{ps}f_{ps}(d_p-c)+C_c(c-a_c)+\Sigma A'_sf'_s(c-d'_s) =$ 1163.51 (kN•m)

- Resistance factor(Φ).

  $d_t =$ 1560.0 (mm)

  $\varepsilon_t = 0.003(d_t/c-1) =$ 0.01615 : Net tensile strain in the extreme tension steel

  ∴ $\Phi =$ 1.00 : Tension Controlled Range, $0.005 \le \varepsilon_t$

- Factored resistance($M_r = \Phi M_n$)

M_r = 1163.51 (kN•m) ≥ M_u = 0.00 (kN•m) **OK**

3) Minimum reinforcement

- Judgement *(see. 5.7.3.3.2)*

M_r = 1163.51 (kN•m) ≥ $1.33M_u$ = 0.00 (kN•m)

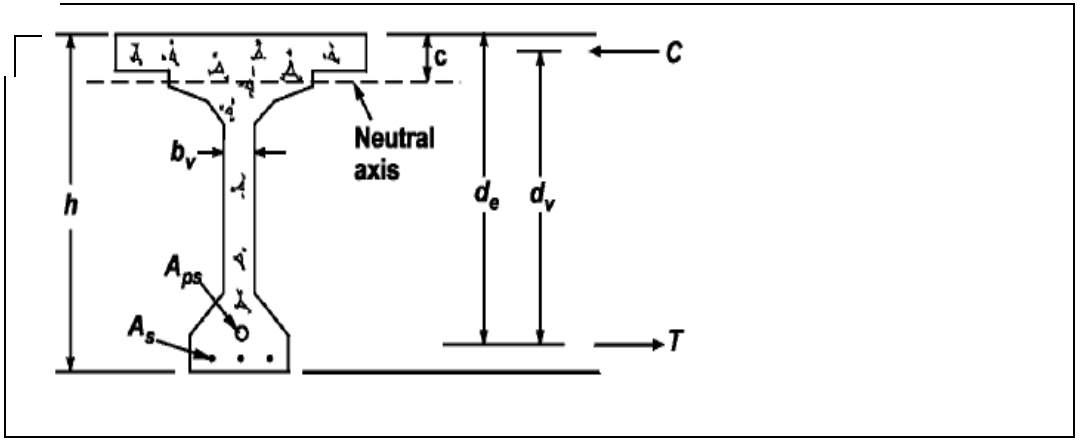
∴ No check.

3.Shear design for a section

■ **Maximum Shear**

- Section type : Nonsegmental-Solid
- The Strength Limit Load Combination : cLCB1
- The factored Shear force : V_u = 148.13 (kN)
- Factored moment : M_u = 10684.85 (kN•m)
- Factored axial force : N_u = -4528.74 (kN)
- Resistance factor for shear : Φ = 0.90
- Component of prestressing force in direction of the shear force : V_p = $\Sigma A_{ps} \cdot f_{e(z\text{-}dir)}$ = 0.00 (kN)

1) Effective depth



b_v = 220.0 (mm) : Effective web width

$d_e = \frac{A_{ps}f_{ps}d_p+A_sf_sd_s}{A_{ps}f_{ps}+A_sf_s}$ = 1670.0 (mm) : Effective depth for Bending

d_v = Max [$\frac{M_n}{A_{ps}f_{ps}+A_sf_s}$, $0.9d_e$, $0.72h$] : Effective depth for shear

= Max [1605.4 , 1503.0 , 1375.2]

= 1605.388 (mm)

2) Maximum spacing for transverse reinforcement (s_{max})

- Shear stress on concrete (v_u).

$v_u = \frac{|V_u - \Phi V_p|}{\Phi b_v d_v}$ = 0.47 (MPa) *(Eq. 5.8.2.9-1)*

- Maximum spacing for transverse reinforcement (s_{max})

v_u = 0.47 (MPa) < $0.125 \cdot f'_c$ = 4.31 (MPa)

∴ s_{max} = Min[$0.8d_v$, 24.0(mm)] = 609.6 (mm) *(Eq. 5.8.2.7-1)*

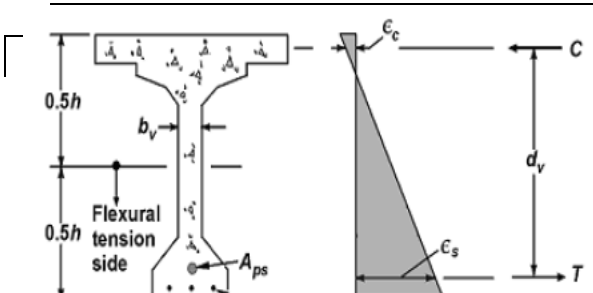
∴ s = 100.0 (mm) ≤ s_{max} **OK**

3) Minimum required transverse reinforcement ($A_{v,min}$)

$A_{v,min} = 0.0316 \sqrt{f'_c} \frac{b_v s}{f_y}$ = 25.9 (mm^2) *(Eq. 5.8.2.5-1)*

∴ A_v = 258.064 (mm^2) ≥ $A_{v,min}$ * Defining "OK" or "NG" will be checked in checking Shear resistance by transverse reinforcement

4) Longitudinal Strain(ε_s)



$$\varepsilon_s = \frac{(|M_u|/d_v + 0.5N_u + |V_u - V_p| - A_{ps}f_{po})}{E_sA_s + E_pA_{ps}} = -0.00047 \quad , \quad 0.0 \le \varepsilon_s \le 0.006 \qquad \textit{(Eq. 5.8.3.4.2-4)}$$

$\therefore \quad \varepsilon_s = 0.00000$

where, $|M_u| = \text{Max}\,[\,M_u\,,\ |V_u - V_p\,d_v\,]$ = Max [10684.85 , 237.80]

$= 10684.849$ (kN•m)

$f_{po} = 0.7f_{pu} = 1304.28$ (MPa)

5) Component of shear resisted by tensile stresses in concrete (V_c)

- Factor indicating diagonally cracked concrete to transmit tension (β).

$$\beta = \frac{4.8}{(1+750\varepsilon_s)} = 4.800 \quad \because \quad A_v \ge A_{v,min} \qquad \textit{(Eq. 5.8.3.4.2-1)}$$

- Nominal shear resistance provide by concrete

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v = 825.91 \text{ (kN)} \qquad \textit{(Eq. 5.8.3.3-3)}$$

6) Component of shear resisted by transverse reinforcement (V_s)

- Area of transverse reinforcement required *(See 5.8.2.4)*

0.5Φ(V_c+V_p)	Φ(V_c+V_p)	V_u
371.66 kN	743.32 kN	148.13 kN

$V_u < 0.5\Phi(V_c+V_p)$ ∴ No shear reinforcing

- Component of shear resisted by transverse reinforcement (V_s)

$$V_s = \frac{A_v \cdot f_y \cdot d_v(\cot\theta+\cot\alpha)\sin\alpha}{s} = 3091.90 \text{ (kN)} \quad \textit{(Eq. 5.8.3.3-4)}$$

where, $\theta = 29+3500\varepsilon_s = 29.000$ (deg.) *(Eq. 5.8.3.4.2-3)*

$\alpha = 90.000$ (deg.)

7) Shear resistance.

- Nominal shear strength *(Eq. 5.8.3.3.-1)*

$V_c + V_s + V_p =$ 3917.82 (kN) > $0.25 \cdot f'_c b_v d_v + V_p =$ 3043.91 (kN)

∴ $V_n = 0.25 \cdot f'_c b_v d_v + V_p =$ 3043.91 (kN)

- Factored shear resistance (V_r).

∴ $V_r = \Phi V_n =$ 2739.52 (kN) ≥ $V_u =$ 148.13 (kN) **OK**

8) Interface Shear

- Nominal interface shear resistance (V_{ni})

$V_{ni} = c \cdot A_{cv} + \mu(A_{vf} \cdot f_y + P_c) = 1120.47$ (kN) *(Eq. 5.8.4.1-3)*

The nominal shear resistance, Vni, used in the design shall not be greater than the lesser of:

$V_{ni} \leq K1 \cdot f`c \cdot Acv = 224.86$ (kN) *(Eq. 5.8.4.1-4)*

$V_{ni} \leq K2 \cdot Acv = 337.29$ (kN) *(Eq. 5.8.4.1-5)*

∴ $V_{ni} = 224.86$ (kN)

Cohesion and Friction Factors (Case) = Roughened(CIP slab) *(Eq. 5.8.4.3)*

where, $c = 1.93$ (MPa)

$\mu = 1.00$

$K_1 = 0.30$

$K_2 = 12.41$ (MPa)

$A_{cv} =$ ####### (mm^2)

A_{vf} = 2580.64 (mm^2)
F_y = 413.69 (MPa)
P_c = 0.42 (kN)

- Factored Interface shear force (V_{ui}) *(Eq. 5.8.4.2-2)*

∴ $V_{ui} = V_u / b_{vi}.d_v$ = 0.06 (kN)

where, V_u = 148.13 (kN)
b_{vi} = 1070.00 (mm)
d_v = 1605.39 (mm) (kN)

- Factored Interface shear resistance (V_{ri}) *(Eq. 5.8.4.1-2)*

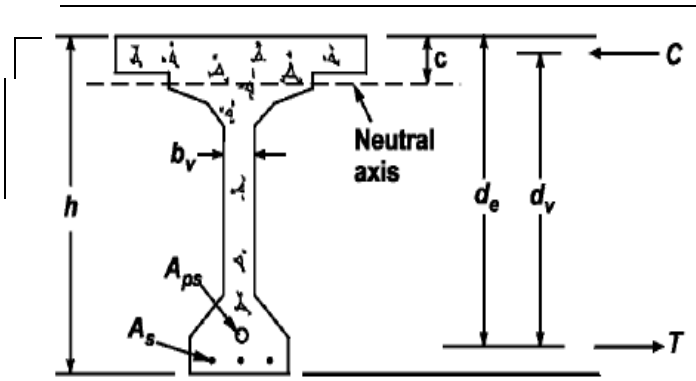
∴ $V_{ri} = \Phi V_{ni}$ = 224.86 (kN) ≥ V_{ui} = 0.06 (kN) **OK**

where, Φ = 1.0 *(Eq. 5.7.2.1)*

■ **Minimum Shear**

- Section type : Nonsegmental-Solid
- The Strength Limit Load Combination : cLCB1
- The factored Shear force : V_u = -158.36 (kN)
- Factored moment : M_u = 5947.49 (kN•m)
- Factored axial force : N_u = -4612.24 (kN)
- Resistance factor for shear : Φ = 0.90
- Component of prestressing force in direction of the shear force : $V_p = \Sigma A_{ps} \cdot f_{e(z\text{-}dir)}$ = 0.00 (kN)

1) Effective depth



b_v = 220.0 (mm) : Effective web width

$d_e = \frac{A_{ps}f_{ps}d_p + A_sf_sd_s}{A_{ps}f_{ps} + A_sf_s}$ = 1670.0 (mm) : Effective depth for Bending

d_v = Max [$\frac{M_n}{A_{ps}f_{ps} + A_sf_s}$, $0.9d_e$, $0.72h$] : Effective depth for shear

= Max [1605.4 , 1503.0 , 1375.2]

= 1605.388 (mm)

2) Maximum spacing for transverse reinforcement (s_{max})

- Shear stress on concrete (v_u).

$v_u = \frac{|V_u - \Phi V_p|}{\Phi b_v d_v}$ = 0.50 (MPa) *(Eq. 5.8.2.9-1)*

- Maximum spacing for transverse reinforcement (s_{max})

v_u = 0.50 (MPa) < $0.125 \cdot f'_c$ = 4.31 (MPa)

∴ s_{max} = Min[$0.8d_v$, 24.0(mm)] = 609.6 (mm) *(Eq. 5.8.2.7-1)*

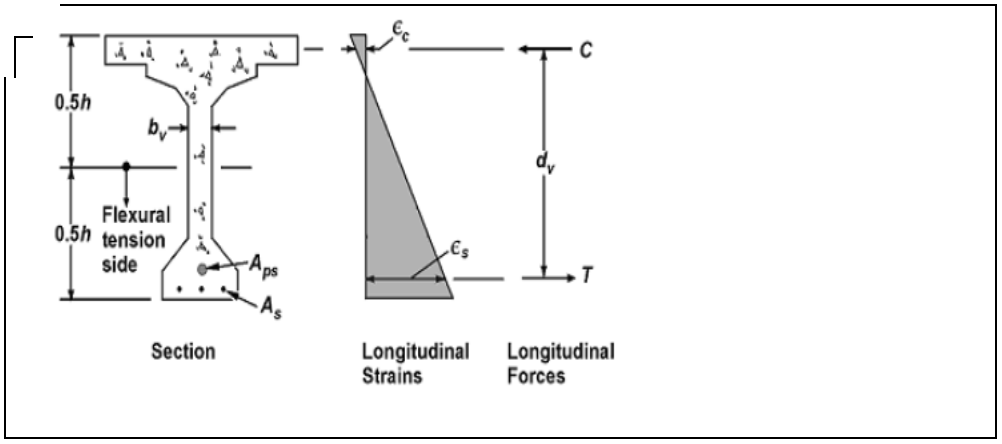
∴ s = 100.0 (mm) ≤ s_{max} **OK**

3) Minimum required transverse reinforcement ($A_{v,min}$)

$A_{v,min} = 0.0316 \sqrt{f'_c} \frac{b_v s}{f_y}$ = 25.9 (mm^2) *(Eq. 5.8.2.5-1)*

$\therefore$ A_v = 258.064 (mm^2) ≥ $A_{v,min}$ * Defining "OK" or "NG" will be checked in checking Shear resistance by transverse reinforcement

4) Longitudinal Strain(ε_s)



$$\varepsilon_s = \frac{(|M_u|/d_v + 0.5N_u + |V_u - V_p| - A_{ps}f_{po})}{E_sA_s + E_pA_{ps}} = -0.00314 \quad , \quad 0.0 \le \varepsilon_s \le 0.006 \qquad \textit{(Eq. 5.8.3.4.2-4)}$$

$\therefore$ ε_s = 0.00000

where, $|M_u| = \text{Max}[M_u, |V_u - V_p| d_v] = \text{Max}[5947.49, 254.23]$

$= 5947.494$ (kN•m)

$f_{po} = 0.7f_{pu} = 1304.28$ (MPa)

5) Component of shear resisted by tensile stresses in concrete (V_c)

- Factor indicating diagonally cracked concrete to transmit tension (β).

$$\beta = \frac{4.8}{(1+750\varepsilon_s)} = 4.800 \quad \because \quad A_v \ge A_{v,min} \qquad \textit{(Eq. 5.8.3.4.2-1)}$$

- Nominal shear resistance provide by concrete

$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v =$ 825.91 (kN) *(Eq. 5.8.3.3-3)*

6) Component of shear resisted by transverse reinforcement (V_s)

- Area of transverse reinforcement required *(See 5.8.2.4)*

$0.5\Phi(V_c+V_p)$	$\Phi(V_c+V_p)$	V_u
371.66 kN	743.32 kN	158.36 kN

$V_u < 0.5\Phi(V_c+V_p)$ ∴ No shear reinforcing

- Component of shear resisted by transverse reinforcement (V_s)

$$V_s = \frac{A_v \cdot f_y \cdot d_v(\cot\theta+\cot\alpha)\sin\alpha}{s} = 3091.90 \text{ (kN)}$$ *(Eq. 5.8.3.3-4)*

where, $\theta = 29+3500\varepsilon_s = 29.000$ (deg.) *(Eq. 5.8.3.4.2-3)*

$\alpha = 90.000$ (deg.)

7) Shear resistance.

- Nominal shear strength *(Eq. 5.8.3.3.-1)*

$V_c + V_s + V_p =$ 3917.82 (kN) > $0.25\cdot f'_c b_v d_v + V_p =$ 3043.91 (kN)

∴ $V_n = 0.25\cdot f'_c b_v d_v + V_p =$ 3043.91 (kN)

- Factored shear resistance (V_r).

∴ $V_r = \Phi V_n =$ | 2739.52 | (kN) ≥ $V_u =$ | -158.36 | (kN) **OK**

8) Interface Shear

- Nominal interface shear resistance (V_{ni})

$V_{ni} = c\cdot A_{cv} + \mu(A_{vf}\cdot f_y + P_c) =$ 1120.47 (kN) *(Eq. 5.8.4.1-3)*

The nominal shear resistance, Vni, used in the design shall not be greater than the lesser of:

$V_{ni} \le K1\cdot f`c\cdot Acv =$ 224.86 (kN) *(Eq. 5.8.4.1-4)*

$V_{ni} \le K2\cdot Acv =$ 337.29 (kN) *(Eq. 5.8.4.1-5)*

∴ $V_{ni} =$ 224.86 (kN)

Cohesion and Friction Factors (Case) = Roughened(CIP slab) *(Eq. 5.8.4.3)*

where, c = 1.93 (MPa)
μ = 1.00
K_1 = 0.30
K_2 = 12.41 (MPa)
A_{cv} = ######## (mm^2)
A_{vf} = 2580.64 (mm^2)
F_y = 413.69 (MPa)
P_c = 0.42 (kN)

- Factored Interface shear force (V_{ui}) *(Eq. 5.8.4.2-2)*

$\therefore$ $V_{ui} = V_u / b_{vi}.d_v$ = -0.06 (kN)

where, V_u = -158.36 (kN)
b_{vi} = 1070.00 (mm)
d_v = 1605.39 (mm) (kN)

- Factored Interface shear resistance (V_{ri}) *(Eq. 5.8.4.1-2)*

$\therefore$ $V_{ri} = \Phi V_{ni}$ = 224.86 (kN) ≥ V_{ui} = 0.06 (kN) **OK**

where, Φ = 1.0 *(Eq. 5.7.2.1)*

4.Torsional design for a section

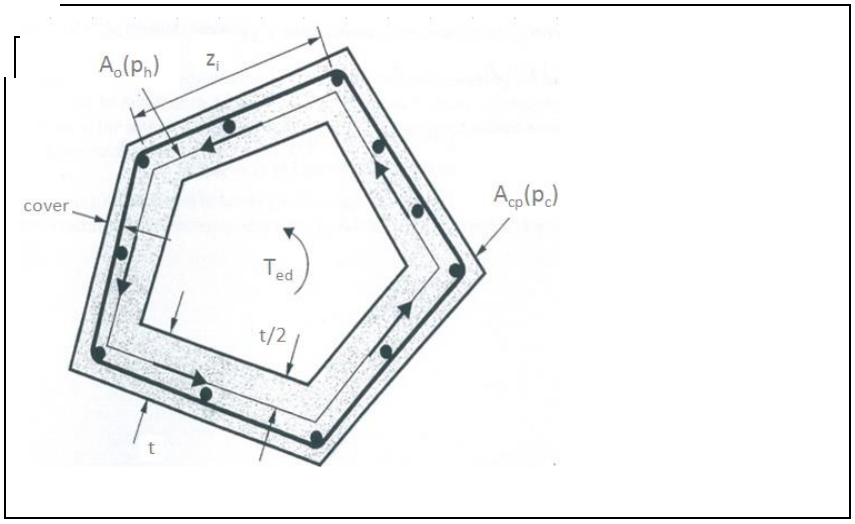
■ **Case of $V_{max.}$**

- Section type : Nonsegmental-Solid
- The Strength Limit Load Combination : cLCB1
- The factored Torsional moment : T_u = 30.71 (kN•m)
- The factored Shear force : V_u = 148.13 (kN)
- Factored moment : M_u = 10684.85 (kN•m)
- Factored axial force : N_u = -4528.74 (kN)
- Resistance factor for shear : Φ = 0.90

- Component of prestressing force

in direction of the shear force : $V_p = \Sigma A_{ps} \cdot f_{e(z\text{-}dir)} = 0.00$ (kN)

1) Notation



A_o = Area enclosed by the shear flow path, including any area of holes therein
= 1.386E+06 (mm^2)

p_h = Perimeter of the centerline of the closed tranverse torsion reinforcement.
= 11532.3 (mm)

A_{cp} = Total area enclosed by outside Perimeter of the concrete section.
= 1.386E+06 (mm^2)

p_c = The length of the outside perimeter of concrete section.
= 387.1 (mm)

2) Checking Torsional Effects

- Torsional cracking moment (T_{cr}).

$$T_{cr} = 0.125 \cdot \sqrt{f'_c} \; \frac{A_{cp}^2}{p_c} \left(1 + \frac{f_{pc}}{0.125\sqrt{f'_c}} \right)^{(1/2)} = 625.71 \text{ (kN•m)} \qquad \textit{(Eq. 5.8.2.1-4)}$$

where,

A_{cp}^2/p_c = 160441577 (mm^3)

f_{pc} = *Compressive stress in concrete due to effective prestress only.*

$$= \frac{\Sigma A_{ps} \cdot f_{e(x\text{-}dir)}}{A_g} + \frac{\Sigma A_{ps} \cdot f_{e(x\text{-}dir)} \cdot e_p \cdot y_{joint}}{I_y} = 3.40 \text{ (MPa)}$$

T_u = | 30.71 | (kN•m) ≤ $0.25\Phi T_{cr}$ = | 140.78 | (kN•m) *(Eq. 5.8.2.1-3)*

∴ $T_u \le 0.25\Phi T_{cr}$, Ignore Torsional Effects.

■ **Case of $V_{min.}$**

- Section type : Nonsegmental-Solid
- The Strength Limit Load Combination : cLCB1
- The factored Torsional moment : T_u = -59.06 (kN•m)
- The factored Shear force : V_u = -158.36 (kN)
- Factored moment : M_u = 5947.49 (kN•m)
- Factored axial force : N_u = -4612.24 (kN)
- Resistance factor for shear : Φ = 0.90
- Component of prestressing force in direction of the shear force : $V_p = \Sigma A_{ps} \cdot f_{e(z\text{-}dir)}$ = 0.00 (kN)

1) Checking Torsional Effects

▪ Torsional cracking moment (T_{cr}).

$$T_{cr} = 0.125 \cdot \sqrt{f'_c} \; \frac{A_{cp}^2}{p_c} \left(1 + \frac{f_{pc}}{0.125\sqrt{f'_c}} \right)^{(1/2)} = 625.71 \text{ (kN•m)} \qquad \textit{(Eq. 5.8.2.1-4)}$$

where,

A_{cp}^2/p_c = 160441577 (mm^3)

f_{pc} = *Compressive stress in concrete due to effective prestress only.*

$$= \frac{\Sigma A_{ps} \cdot f_{e(x\text{-}dir)}}{A_g} + \frac{\Sigma A_{ps} \cdot f_{e(x\text{-}dir)} \cdot e_p \cdot y_{joint}}{I_y} = 3.40 \text{ (MPa)}$$

$$T_u = \left| -59.06 \right| \text{ (kN•m)} \le 0.25\Phi T_{cr} = \left| 140.78 \right| \text{ (kN•m)} \qquad \textit{(Eq. 5.8.2.1-3)}$$

∴ $T_u \le 0.25\Phi T_{cr}$, Ignore Torsional Effects.

■ **Case of $T_{max.}$**

- Section type : Nonsegmental-Solid
- The Strength Limit Load Combination : cLCB1

- The factored Torsional moment : T_u = -59.06 (kN•m)
- The factored Shear force : V_u = -158.36 (kN)
- Factored moment : M_u = 5947.49 (kN•m)
- Factored axial force : N_u = -4612.24 (kN)
- Resistance factor for shear : Φ = 0.90
- Component of prestressing force in direction of the shear force : $V_p = \Sigma A_{ps} \cdot f_{e(z\text{-}dir)}$ = 0.00 (kN)

1) Checking Torsional Effects

- Torsional cracking moment (T_{cr}).

$$T_{cr} = 0.125 \cdot \sqrt{f'_c} \; \frac{A_{cp}^2}{p_c} \left(1 + \frac{f_{pc}}{0.125\sqrt{f'_c}} \right)^{(1/2)} = 625.71 \text{ (kN•m)} \qquad \textit{(Eq. 5.8.2.1-4)}$$

where,

A_{cp}^2/p_c = 160441577 (mm^3)

f_{pc} = *Compressive stress in concrete due to effective prestress only.*

$$= \frac{\Sigma A_{ps} \cdot f_{e(x\text{-}dir)}}{A_g} + \frac{\Sigma A_{ps} \cdot f_{e(x\text{-}dir)} \cdot e_p \cdot y_{joint}}{I_y} = 3.40 \text{ (MPa)}$$

T_u = |-59.06| (kN•m) ≤ $0.25\Phi T_{cr}$ = |140.78| (kN•m) *(Eq. 5.8.2.1-3)*

∴ $T_u \le 0.25\Phi T_{cr}$, Ignore Torsional Effects.

5.Crack check

■ **Bottom**

- The maximum Service Limit Load Combination : cLCB20
- The maximum spacing of steel reinforcement in layer closet to the tension face(s_{max}) *(Eq. 5.7.3.4-1)*

$$s_{max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2d_c = 1516.4 \text{ (mm)} \ge s_{use} = 152.5 \text{ (mm)} \quad \textbf{OK}$$

where, $\beta_s = 1 + \frac{d_c}{0.7(h-d_c)} = 1.038$

d_c = 50.0 (mm)

γ_e = 1.00 : Exposure factor

f_{ss} = 73.038 (MPa) : Tensile stress in bar at the service limit state

3.4.24.1. VERIFICACIÓN POR MOMENTO FLECTOR

Tabla 24. Verificación de capacidad de diseño a flexión positiva

Elem	Positive/ Negative	Muy (tonf*m)	Mcr (tonf*m)	Mny (tonf*m)	PhiMny (tonf*m)	Ratio (Muy/PhiMny)	PhiMny/ min(1.33Muy,Mcr)	CHK
106	Negative	-63.90	389.67	359.15	359.15	0.18	4.23	OK
107	Negative	0.00	326.52	326.92	326.92	0.00	larger than 100	OK
108	Negative	0.00	354.48	455.87	455.87	0.00	larger than 100	OK
109	Negative	0.00	280.88	403.43	403.43	0.00	larger than 100	OK
110	Negative	0.00	211.34	353.54	353.54	0.00	larger than 100	OK
111	Negative	0.00	146.31	307.15	307.15	0.00	larger than 100	OK
112	Negative	0.00	86.30	264.86	264.86	0.00	larger than 100	OK
113	Negative	0.00	31.70	227.41	227.41	0.00	larger than 100	OK
114	Negative	0.00	-17.09	195.34	195.34	0.00	larger than 100	OK
115	Negative	0.00	-59.64	168.61	168.61	0.00	larger than 100	OK
116	Negative	0.00	-95.46	148.01	148.01	0.00	larger than 100	OK
117	Negative	0.00	-124.25	133.52	133.52	0.00	larger than 100	OK
118	Negative	0.00	-145.61	124.57	124.57	0.00	larger than 100	OK
119	Negative	0.00	-159.27	120.12	120.12	0.00	larger than 100	OK
120	Negative	0.00	-164.70	118.64	118.64	0.00	larger than 100	OK
121	Negative	0.00	-160.16	120.12	120.12	0.00	larger than 100	OK
122	Negative	0.00	-147.49	124.57	124.57	0.00	larger than 100	OK
123	Negative	0.00	-127.13	133.52	133.52	0.00	larger than 100	OK
124	Negative	0.00	-99.36	148.01	148.01	0.00	larger than 100	OK
125	Negative	0.00	-64.64	168.61	168.61	0.00	larger than 100	OK
126	Negative	0.00	-21.91	195.34	195.34	0.00	larger than 100	OK

Elem	Positive/ Negative	Muy (tonf*m)	Mcr (tonf*m)	Mny (tonf*m)	PhiMny (tonf*m)	Ratio (Muy/PhiMny)	PhiMny/ min(1.33Muy,Mcr)	CHK
127	Negative	0.00	27.08	227.41	227.41	0.00	larger than 100	OK
128	Negative	0.00	81.91	264.86	264.86	0.00	larger than 100	OK
129	Negative	0.00	142.20	307.15	307.15	0.00	larger than 100	OK
130	Negative	0.00	207.54	353.54	353.54	0.00	larger than 100	OK
131	Negative	0.00	277.46	403.43	403.43	0.00	larger than 100	OK
132	Negative	0.00	351.53	455.87	455.87	0.00	larger than 100	OK
133	Negative	0.00	429.18	510.24	510.24	0.00	larger than 100	OK
134	Negative	-67.54	387.81	359.15	359.15	0.19	4.00	OK
135	Negative	-192.32	454.36	392.01	392.01	0.49	1.53	OK
1	Positive	0.00	540.55	817.56	817.56	0.00	larger than 100	OK
2	Positive	40.06	568.42	867.77	867.77	0.05	16.29	OK
3	Positive	127.83	596.64	916.12	916.12	0.14	5.39	OK
4	Positive	201.33	624.70	961.70	961.70	0.21	3.59	OK
5	Positive	261.23	651.71	1003.53	1003.53	0.26	2.89	OK
6	Positive	309.97	676.93	1040.69	1040.69	0.30	2.52	OK
7	Positive	347.88	698.42	1072.24	1072.24	0.32	2.32	OK
8	Positive	373.77	716.23	1097.27	1097.27	0.34	2.21	OK
9	Positive	388.32	729.25	1114.84	1114.84	0.35	2.16	OK
10	Positive	391.99	736.75	1124.07	1124.07	0.35	2.16	OK
11	Positive	392.62	736.10	1124.07	1124.07	0.35	2.15	OK
12	Positive	389.73	727.85	1114.84	1114.84	0.35	2.15	OK
13	Positive	375.68	714.30	1097.27	1097.27	0.34	2.20	OK
14	Positive	350.02	696.05	1072.24	1072.24	0.33	2.30	OK
15	Positive	311.90	674.37	1040.69	1040.69	0.30	2.51	OK
16	Positive	261.81	649.90	1003.53	1003.53	0.26	2.88	OK
17	Positive	200.39	623.74	961.70	961.70	0.21	3.61	OK
18	Positive	124.93	596.49	916.12	916.12	0.14	5.51	OK
19	Positive	34.91	569.00	867.77	867.77	0.04	18.69	OK
20	Positive	0.00	541.82	817.56	817.56	0.00	larger than 100	OK
21	Positive	0.00	515.55	766.47	766.47	0.00	larger than 100	OK

Elem	Positive/ Negative	Muy (tonf*m)	Mcr (tonf*m)	Mny (tonf*m)	PhiMny (tonf*m)	Ratio (Muy/PhiMny)	PhiMny/ min(1.33Muy,Mcr)	CHK
106	Positive	4.94	533.18	802.25	802.25	0.01	larger than 100	OK
107	Positive	177.16	548.27	837.74	837.74	0.21	3.56	OK
108	Positive	343.80	588.80	944.54	944.54	0.36	2.07	OK
109	Positive	471.12	622.17	1005.31	1005.31	0.47	1.62	OK
110	Positive	585.03	656.06	1064.31	1064.31	0.55	1.62	OK
111	Positive	685.85	690.02	1120.82	1120.82	0.61	1.62	OK
112	Positive	774.12	723.56	1174.16	1174.16	0.66	1.62	OK
113	Positive	850.39	756.26	1223.63	1223.63	0.70	1.62	OK
114	Positive	916.46	787.34	1268.62	1268.62	0.72	1.61	OK
115	Positive	971.09	816.06	1308.43	1308.43	0.74	1.60	OK
116	Positive	1015.11	841.26	1342.42	1342.42	0.76	1.60	OK
117	Positive	1048.65	862.36	1369.98	1369.98	0.77	1.59	OK
118	Positive	1072.06	878.57	1390.42	1390.42	0.77	1.58	OK
119	Positive	1085.33	889.48	1403.15	1403.15	0.77	1.58	OK
120	Positive	1089.33	894.03	1407.55	1407.55	0.77	1.57	OK
121	Positive	1086.98	888.25	1403.15	1403.15	0.77	1.58	OK
122	Positive	1075.55	876.65	1390.42	1390.42	0.77	1.59	OK
123	Positive	1053.72	859.83	1369.98	1369.98	0.77	1.59	OK
124	Positive	1021.71	838.14	1342.42	1342.42	0.76	1.60	OK
125	Positive	979.33	812.63	1308.43	1308.43	0.75	1.61	OK
126	Positive	924.44	784.69	1268.62	1268.62	0.73	1.62	OK
127	Positive	857.51	754.41	1223.63	1223.63	0.70	1.62	OK
128	Positive	780.01	722.47	1174.16	1174.16	0.66	1.63	OK
129	Positive	690.16	689.61	1120.82	1120.82	0.62	1.63	OK
130	Positive	587.40	656.32	1064.31	1064.31	0.55	1.62	OK
131	Positive	471.27	622.98	1005.31	1005.31	0.47	1.61	OK
132	Positive	341.57	589.91	944.54	944.54	0.36	2.08	OK
133	Positive	197.79	557.58	882.83	882.83	0.22	3.36	OK
134	Positive	0.00	534.92	802.25	802.25	0.00	larger than 100	OK
135	Positive	0.00	521.26	766.47	766.47	0.00	larger than 100	OK

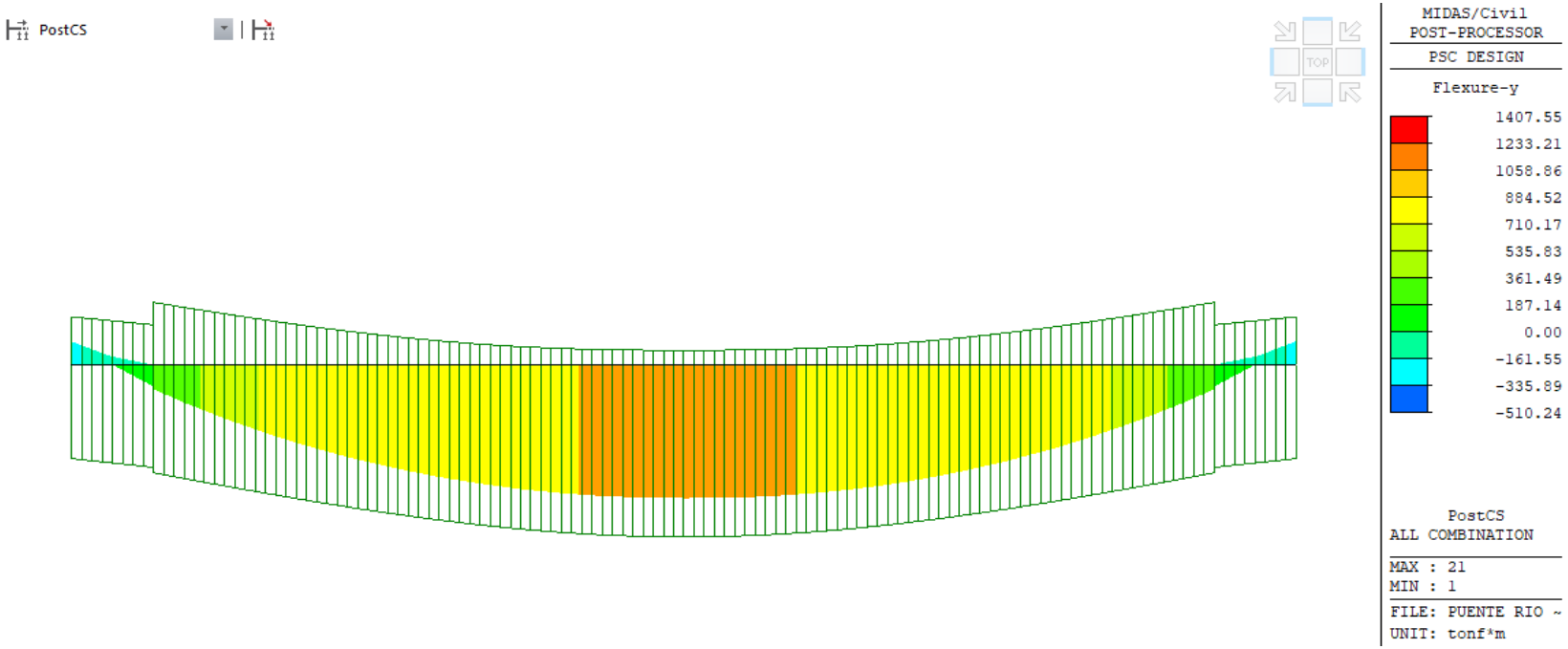


Figura 186. Verificación de capacidad a diseño de flexión positiva

3.4.24.2. VERIFICACIÓN POR FUERZA CORTANTE

Tabla 25. Verificación de capacidad de diseño a fuerza cortante

Elem	Max/Min	CHK	Vu (tonf)	Mu (tonf*m)	Vn (tonf)	Phi	Vc (tonf)	Vs (tonf)	Vp (tonf)	PhiVn (tonf)	de (m)	dv (m)	theta ([deg])	beta	Avs (m^2)	Ast (m^2)	Al (m^2)	bv (m)	Vri (tonf)	Vui (tonf)	Interface Shear(CHK)
106	Max	OK	-121.59	-63.90	234.50	0.90	24.15	189.75	20.60	211.05	0.94	1.38	38.27	1.61	0.00	0.00	0.00	0.22	22.93	-0.05	OK
106	Min	NG	-186.68	-44.40	160.06	0.90	13.19	126.27	20.60	144.05	0.94	1.38	49.86	0.88	0.00	0.00	0.00	0.22	22.93	-0.08	OK
107	Max	OK	-110.38	48.44	286.27	0.90	72.14	270.08	20.39	257.65	1.26	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.05	OK
107	Min	OK	-171.46	80.49	286.27	0.90	72.14	270.08	20.39	257.65	1.26	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.08	OK
108	Max	OK	-82.13	158.87	301.26	0.90	72.14	270.08	35.37	271.13	1.16	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.04	OK
108	Min	OK	-139.51	201.91	301.26	0.90	72.14	270.08	35.37	271.13	1.16	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.06	OK
109	Max	OK	-71.72	233.74	300.44	0.90	72.14	270.08	34.55	270.39	1.22	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.03	OK
109	Min	OK	-125.51	286.33	300.44	0.90	72.14	270.08	34.55	270.39	1.22	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.06	OK
110	Max	OK	-62.02	299.02	299.18	0.90	72.14	270.08	33.29	269.26	1.29	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.03	OK
110	Min	OK	-112.40	359.81	299.18	0.90	72.14	270.08	33.29	269.26	1.29	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.05	OK
111	Max	OK	-52.78	355.16	297.68	0.90	72.14	270.08	31.80	267.91	1.35	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.02	OK
111	Min	OK	-99.88	422.89	297.68	0.90	72.14	270.08	31.80	267.91	1.35	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.04	OK
112	Max	OK	-43.30	673.82	295.73	0.90	72.14	270.08	29.85	266.16	1.41	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.02	OK
112	Min	OK	-88.18	476.07	295.73	0.90	72.14	270.08	29.85	266.16	1.41	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.04	OK
113	Max	OK	-33.94	739.27	298.19	0.90	73.48	275.06	27.40	268.37	1.46	1.40	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.01	OK
113	Min	OK	-77.25	519.70	298.19	0.90	73.48	275.06	27.40	268.37	1.46	1.40	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.03	OK
114	Max	OK	-25.01	795.11	305.18	0.90	76.10	284.88	24.73	274.67	1.52	1.45	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.01	OK
114	Min	OK	-66.70	554.31	305.18	0.90	76.10	284.88	24.73	274.67	1.52	1.45	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.03	OK
115	Max	OK	-16.64	840.52	310.70	0.90	78.42	293.57	21.69	279.63	1.56	1.49	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.01	OK
115	Min	OK	-56.72	580.43	310.70	0.90	78.42	293.57	21.69	279.63	1.56	1.49	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.02	OK
116	Max	OK	-9.17	876.34	314.34	0.90	80.41	301.02	18.00	282.91	1.60	1.53	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
116	Min	OK	-47.50	598.90	314.34	0.90	80.41	301.02	18.00	282.91	1.60	1.53	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.02	OK
117	Max	OK	-1.88	902.80	316.44	0.90	82.02	307.05	14.16	284.80	1.63	1.56	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
117	Min	OK	-38.43	610.14	316.44	0.90	82.02	307.05	14.16	284.80	1.63	1.56	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.01	OK
118	Max	OK	5.41	1072.06	316.65	0.90	83.22	311.53	9.95	284.98	1.65	1.59	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
118	Min	OK	-29.75	614.76	316.65	0.90	83.22	311.53	9.95	284.98	1.65	1.59	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.01	OK
119	Max	OK	11.99	1085.33	314.45	0.90	83.96	314.32	5.01	283.01	1.67	1.60	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK

Elem	Max/Min	CHK	Vu (tonf)	Mu (tonf*m)	Vn (tonf)	Phi	Vc (tonf)	Vs (tonf)	Vp (tonf)	PhiVn (tonf)	de (m)	dv (m)	theta ([deg])	beta	Avs (m^2)	Ast (m^2)	Al (m^2)	bv (m)	Vri (tonf)	Vui (tonf)	Interface Shear(CHK)
119	Min	OK	-21.80	613.14	314.45	0.90	83.96	314.32	5.01	283.01	1.67	1.60	29.00	4.80	0.00	0.00	0.00	0.22	22.93	-0.01	OK
120	Max	OK	18.70	1089.33	310.39	0.90	84.22	315.29	0.00	279.35	1.67	1.61	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.01	OK
120	Min	OK	-12.58	606.28	310.39	0.90	84.22	315.29	0.00	279.35	1.67	1.61	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
121	Max	OK	21.36	1086.98	314.44	0.90	83.96	314.32	5.00	283.00	1.67	1.60	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.01	OK
121	Min	OK	-9.89	614.40	314.44	0.90	83.96	314.32	5.00	283.00	1.67	1.60	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
122	Max	OK	30.85	1075.55	316.62	0.90	83.22	311.53	9.92	284.96	1.65	1.59	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.01	OK
122	Min	OK	-2.85	617.66	316.62	0.90	83.22	311.53	9.92	284.96	1.65	1.59	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
123	Max	OK	39.87	1053.72	316.40	0.90	82.02	307.05	14.11	284.76	1.63	1.56	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.02	OK
123	Min	OK	4.27	568.83	316.40	0.90	82.02	307.05	14.11	284.76	1.63	1.56	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
124	Max	OK	49.07	1021.71	314.27	0.90	80.41	301.02	17.92	282.84	1.60	1.53	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.02	OK
124	Min	OK	11.61	561.09	314.27	0.90	80.41	301.02	17.92	282.84	1.60	1.53	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.00	OK
125	Max	OK	57.75	979.33	310.61	0.90	78.42	293.57	21.59	279.55	1.56	1.49	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.02	OK
125	Min	OK	18.53	546.65	310.61	0.90	78.42	293.57	21.59	279.55	1.56	1.49	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.01	OK
126	Max	OK	68.08	924.44	305.07	0.90	76.10	284.88	24.62	274.57	1.52	1.45	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.03	OK
126	Min	OK	27.04	523.10	305.07	0.90	76.10	284.88	24.62	274.57	1.52	1.45	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.01	OK
127	Max	OK	77.93	857.51	298.07	0.90	73.48	275.06	27.28	268.26	1.46	1.40	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.03	OK
127	Min	OK	35.29	490.75	298.07	0.90	73.48	275.06	27.28	268.26	1.46	1.40	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.02	OK
128	Max	OK	88.88	780.01	295.61	0.90	72.14	270.08	29.72	266.05	1.41	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.04	OK
128	Min	OK	44.26	408.58	295.61	0.90	72.14	270.08	29.72	266.05	1.41	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.02	OK
129	Max	OK	100.74	690.16	297.55	0.90	72.14	270.08	31.67	267.80	1.35	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.04	OK
129	Min	OK	52.90	359.80	297.55	0.90	72.14	270.08	31.67	267.80	1.35	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.02	OK
130	Max	OK	113.30	587.40	299.05	0.90	72.14	270.08	33.16	269.14	1.29	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.05	OK
130	Min	OK	62.16	302.15	299.05	0.90	72.14	270.08	33.16	269.14	1.29	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.03	OK
131	Max	OK	126.38	471.27	300.31	0.90	72.14	270.08	34.42	270.28	1.22	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.06	OK
131	Min	OK	71.82	235.27	300.31	0.90	72.14	270.08	34.42	270.28	1.22	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.03	OK
132	Max	OK	140.20	341.57	301.13	0.90	72.14	270.08	35.24	271.01	1.16	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.06	OK
132	Min	OK	82.12	158.79	301.13	0.90	72.14	270.08	35.24	271.01	1.16	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.04	OK
133	Max	OK	154.74	197.79	301.58	0.90	72.14	270.08	35.69	271.42	1.09	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.07	OK
133	Min	OK	93.02	72.14	301.58	0.90	72.14	270.08	35.69	271.42	1.09	1.38	29.00	4.80	0.00	0.00	0.00	0.22	22.93	0.04	OK
134	Max	NG	187.00	-2.30	199.10	0.90	18.04	160.57	20.49	179.19	0.94	1.38	43.00	1.20	0.00	0.00	0.00	0.22	22.93	0.08	OK
134	Min	OK	121.54	-67.54	286.38	0.90	49.26	247.51	20.49	257.74	0.94	1.38	31.17	3.28	0.00	0.00	0.00	0.22	22.93	0.05	OK
135	Max	NG	202.87	-192.32	184.32	0.90	16.01	147.79	20.52	165.89	1.01	1.38	45.37	1.06	0.00	0.00	0.00	0.22	22.93	0.09	OK
135	Min	OK	133.00	-192.32	261.56	0.90	30.41	210.63	20.52	235.41	1.01	1.38	35.40	2.02	0.00	0.00	0.00	0.22	22.93	0.06	OK

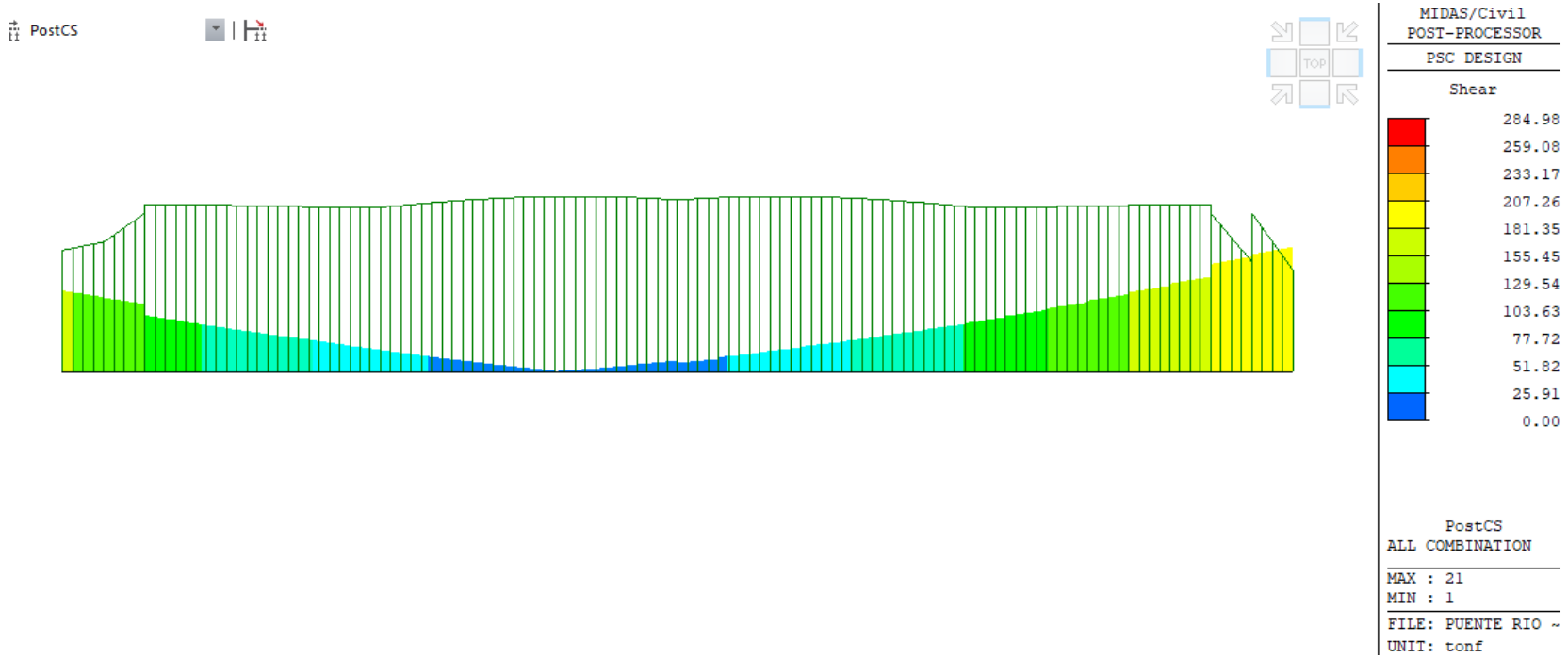


Figura 187. Verificación de capacidad a fuerza cortante

3.4.24.3. VERIFICACIÓN POR CORTANTE Y TORSIÓN COMBINADA

Tabla 26. Verificación de capacidad de diseño a cortante y torsión combinada

Elem	Max/Min	Vu (tonf)	Mu (tonf*m)	Tu (tonf*m)	Vn (tonf)	Tn (tonf*m)	Phi	Phi_t	Vc (tonf)	Vs (tonf)	Vp (tonf)	PhiVn (tonf)	Phi_tTn (tonf*m)	de (m)	dv (m)	theta ([deg])	beta	bv (m)	bv_min (m)	CHK
106	T-Max	-186.68	-44.40	-23.43	160.06	254.51	0.90	0.90	13.19	126.27	20.60	144.05	229.06	0.94	1.38	49.86	0.88	0.22	0.19	OK
106	V-Max	-121.59	-63.90	-10.44	234.50	382.47	0.90	0.90	24.15	189.75	20.60	211.05	344.22	0.94	1.38	38.27	1.61	0.22	0.13	OK
106	V-Min	-186.68	-44.40	-23.43	160.06	254.51	0.90	0.90	13.19	126.27	20.60	144.05	229.06	0.94	1.38	49.86	0.88	0.22	0.19	OK
107	T-Max	-122.89	177.16	4.14	286.27	544.38	0.90	0.90	72.14	270.08	20.39	257.65	489.94	1.26	1.38	29.00	4.80	0.22	0.13	OK
107	V-Max	-110.38	48.44	-0.47	286.27	544.38	0.90	0.90	72.14	270.08	20.39	257.65	489.94	1.26	1.38	29.00	4.80	0.22	0.12	OK
107	V-Min	-171.46	80.49	-3.96	286.27	544.38	0.90	0.90	72.14	270.08	20.39	257.65	489.94	1.26	1.38	29.00	4.80	0.22	0.17	OK
108	T-Max	-92.20	343.80	10.66	301.26	544.38	0.90	0.90	72.14	270.08	35.37	271.13	489.94	1.16	1.38	29.00	4.80	0.22	0.11	OK
108	V-Max	-82.13	158.87	2.86	301.26	544.38	0.90	0.90	72.14	270.08	35.37	271.13	489.94	1.16	1.38	29.00	4.80	0.22	0.10	OK
108	V-Min	-139.51	201.91	2.71	301.26	544.38	0.90	0.90	72.14	270.08	35.37	271.13	489.94	1.16	1.38	29.00	4.80	0.22	0.16	OK
109	T-Max	-79.48	471.12	11.71	300.44	544.38	0.90	0.90	72.14	270.08	34.55	270.39	489.94	1.22	1.38	29.00	4.80	0.22	0.10	OK
109	V-Max	-71.72	233.74	3.68	300.44	544.38	0.90	0.90	72.14	270.08	34.55	270.39	489.94	1.22	1.38	29.00	4.80	0.22	0.09	OK
109	V-Min	-125.51	286.33	4.13	300.44	544.38	0.90	0.90	72.14	270.08	34.55	270.39	489.94	1.22	1.38	29.00	4.80	0.22	0.14	OK
110	T-Max	-67.63	585.03	10.76	299.18	544.38	0.90	0.90	72.14	270.08	33.29	269.26	489.94	1.29	1.38	29.00	4.80	0.22	0.09	OK
110	V-Max	-62.02	299.02	3.58	299.18	544.38	0.90	0.90	72.14	270.08	33.29	269.26	489.94	1.29	1.38	29.00	4.80	0.22	0.08	OK
110	V-Min	-112.40	359.81	3.28	299.18	544.38	0.90	0.90	72.14	270.08	33.29	269.26	489.94	1.29	1.38	29.00	4.80	0.22	0.13	OK
111	T-Max	-56.37	685.85	9.34	297.68	544.38	0.90	0.90	72.14	270.08	31.80	267.91	489.94	1.35	1.38	29.00	4.80	0.22	0.08	OK
111	V-Max	-52.78	355.16	3.21	297.68	544.38	0.90	0.90	72.14	270.08	31.80	267.91	489.94	1.35	1.38	29.00	4.80	0.22	0.07	OK
111	V-Min	-99.88	422.89	1.91	297.68	544.38	0.90	0.90	72.14	270.08	31.80	267.91	489.94	1.35	1.38	29.00	4.80	0.22	0.12	OK
112	T-Max	-45.84	774.12	7.96	295.73	544.38	0.90	0.90	72.14	270.08	29.85	266.16	489.94	1.41	1.38	29.00	4.80	0.22	0.07	OK
112	V-Max	-43.30	673.82	7.19	295.73	544.38	0.90	0.90	72.14	270.08	29.85	266.16	489.94	1.41	1.38	29.00	4.80	0.22	0.06	OK

Elem	Max/Min	Vu (tonf)	Mu (tonf*m)	Tu (tonf*m)	Vn (tonf)	Tn (tonf*m)	Phi	Phi_t	Vc (tonf)	Vs (tonf)	Vp (tonf)	PhiVn (tonf)	Phi_tTn (tonf*m)	de (m)	dv (m)	theta ([deg])	beta	bv (m)	bv_min (m)	CHK
112	V-Min	-88.18	476.07	0.66	295.73	544.38	0.90	0.90	72.14	270.08	29.85	266.16	489.94	1.41	1.38	29.00	4.80	0.22	0.11	OK
113	T-Max	-36.05	850.39	6.89	298.19	544.38	0.90	0.90	73.48	275.06	27.40	268.37	489.94	1.46	1.40	29.00	4.80	0.22	0.05	OK
113	V-Max	-33.94	739.27	6.22	298.19	544.38	0.90	0.90	73.48	275.06	27.40	268.37	489.94	1.46	1.40	29.00	4.80	0.22	0.05	OK
113	V-Min	-77.25	519.70	-0.33	298.19	544.38	0.90	0.90	73.48	275.06	27.40	268.37	489.94	1.46	1.40	29.00	4.80	0.22	0.09	OK
114	T-Max	-26.66	916.46	6.21	305.18	544.38	0.90	0.90	76.10	284.88	24.73	274.67	489.94	1.52	1.45	29.00	4.80	0.22	0.04	OK
114	V-Max	-25.01	795.11	5.59	305.18	544.38	0.90	0.90	76.10	284.88	24.73	274.67	489.94	1.52	1.45	29.00	4.80	0.22	0.04	OK
114	V-Min	-66.70	554.31	-1.19	305.18	544.38	0.90	0.90	76.10	284.88	24.73	274.67	489.94	1.52	1.45	29.00	4.80	0.22	0.08	OK
115	T-Max	-17.82	971.09	5.71	310.70	544.38	0.90	0.90	78.42	293.57	21.69	279.63	489.94	1.56	1.49	29.00	4.80	0.22	0.03	OK
115	V-Max	-16.64	840.52	5.13	310.70	544.38	0.90	0.90	78.42	293.57	21.69	279.63	489.94	1.56	1.49	29.00	4.80	0.22	0.03	OK
115	V-Min	-56.72	580.43	-1.84	310.70	544.38	0.90	0.90	78.42	293.57	21.69	279.63	489.94	1.56	1.49	29.00	4.80	0.22	0.06	OK
116	T-Max	-9.84	1015.11	5.30	314.34	544.38	0.90	0.90	80.41	301.02	18.00	282.91	489.94	1.60	1.53	29.00	4.80	0.22	0.02	OK
116	V-Max	-9.17	876.34	4.74	314.34	544.38	0.90	0.90	80.41	301.02	18.00	282.91	489.94	1.60	1.53	29.00	4.80	0.22	0.02	OK
116	V-Min	-47.50	598.90	-2.33	314.34	544.38	0.90	0.90	80.41	301.02	18.00	282.91	489.94	1.60	1.53	29.00	4.80	0.22	0.05	OK
117	T-Max	-1.99	1048.65	4.92	316.44	544.38	0.90	0.90	82.02	307.05	14.16	284.80	489.94	1.63	1.56	29.00	4.80	0.22	0.01	OK
117	V-Max	-1.88	902.80	4.34	316.44	544.38	0.90	0.90	82.02	307.05	14.16	284.80	489.94	1.63	1.56	29.00	4.80	0.22	0.01	OK
117	V-Min	-38.43	610.14	-2.67	316.44	544.38	0.90	0.90	82.02	307.05	14.16	284.80	489.94	1.63	1.56	29.00	4.80	0.22	0.04	OK
118	T-Max	5.41	1072.06	4.53	316.65	544.38	0.90	0.90	83.22	311.53	9.95	284.98	489.94	1.65	1.59	29.00	4.80	0.22	0.00	OK
118	V-Max	5.41	1072.06	4.53	316.65	544.38	0.90	0.90	83.22	311.53	9.95	284.98	489.94	1.65	1.59	29.00	4.80	0.22	0.00	OK
118	V-Min	-29.75	614.76	-3.19	316.65	544.38	0.90	0.90	83.22	311.53	9.95	284.98	489.94	1.65	1.59	29.00	4.80	0.22	0.03	OK
119	T-Max	11.99	1085.33	4.50	314.45	544.38	0.90	0.90	83.96	314.32	5.01	283.01	489.94	1.67	1.60	29.00	4.80	0.22	0.01	OK
119	V-Max	11.99	1085.33	4.50	314.45	544.38	0.90	0.90	83.96	314.32	5.01	283.01	489.94	1.67	1.60	29.00	4.80	0.22	0.01	OK
119	V-Min	-21.80	613.14	-3.74	314.45	544.38	0.90	0.90	83.96	314.32	5.01	283.01	489.94	1.67	1.60	29.00	4.80	0.22	0.02	OK
120	T-Max	-12.58	606.28	-4.78	310.39	544.38	0.90	0.90	84.22	315.29	0.00	279.35	489.94	1.67	1.61	29.00	4.80	0.22	0.01	OK
120	V-Max	18.70	1089.33	4.40	310.39	544.38	0.90	0.90	84.22	315.29	0.00	279.35	489.94	1.67	1.61	29.00	4.80	0.22	0.01	OK
120	V-Min	-12.58	606.28	-4.78	310.39	544.38	0.90	0.90	84.22	315.29	0.00	279.35	489.94	1.67	1.61	29.00	4.80	0.22	0.01	OK
121	T-Max	-9.89	614.40	-4.68	314.44	544.38	0.90	0.90	83.96	314.32	5.00	283.00	489.94	1.67	1.60	29.00	4.80	0.22	0.01	OK
121	V-Max	21.36	1086.98	4.47	314.44	544.38	0.90	0.90	83.96	314.32	5.00	283.00	489.94	1.67	1.60	29.00	4.80	0.22	0.01	OK
121	V-Min	-9.89	614.40	-4.68	314.44	544.38	0.90	0.90	83.96	314.32	5.00	283.00	489.94	1.67	1.60	29.00	4.80	0.22	0.01	OK
122	T-Max	30.85	1075.55	4.48	316.62	544.38	0.90	0.90	83.22	311.53	9.92	284.96	489.94	1.65	1.59	29.00	4.80	0.22	0.02	OK

Elem	Max/Min	Vu (tonf)	Mu (tonf*m)	Tu (tonf*m)	Vn (tonf)	Tn (tonf*m)	Phi	Phi_t	Vc (tonf)	Vs (tonf)	Vp (tonf)	PhiVn (tonf)	Phi_tTn (tonf*m)	de (m)	dv (m)	theta ([deg])	beta	bv (m)	bv_min (m)	CHK
122	V-Max	30.85	1075.55	4.48	316.62	544.38	0.90	0.90	83.22	311.53	9.92	284.96	489.94	1.65	1.59	29.00	4.80	0.22	0.02	OK
122	V-Min	-2.85	617.66	-3.72	316.62	544.38	0.90	0.90	83.22	311.53	9.92	284.96	489.94	1.65	1.59	29.00	4.80	0.22	0.01	OK
123	T-Max	39.87	1053.72	6.98	316.40	544.38	0.90	0.90	82.02	307.05	14.11	284.76	489.94	1.63	1.56	29.00	4.80	0.22	0.02	OK
123	V-Max	39.87	1053.72	6.98	316.40	544.38	0.90	0.90	82.02	307.05	14.11	284.76	489.94	1.63	1.56	29.00	4.80	0.22	0.02	OK
123	V-Min	4.27	568.83	-0.21	316.40	544.38	0.90	0.90	82.02	307.05	14.11	284.76	489.94	1.63	1.56	29.00	4.80	0.22	0.01	OK
124	T-Max	49.07	1021.71	13.96	314.27	544.38	0.90	0.90	80.41	301.02	17.92	282.84	489.94	1.60	1.53	29.00	4.80	0.22	0.03	OK
124	V-Max	49.07	1021.71	13.96	314.27	544.38	0.90	0.90	80.41	301.02	17.92	282.84	489.94	1.60	1.53	29.00	4.80	0.22	0.03	OK
124	V-Min	11.61	561.09	6.81	314.27	544.38	0.90	0.90	80.41	301.02	17.92	282.84	489.94	1.60	1.53	29.00	4.80	0.22	0.00	OK
125	T-Max	57.75	979.33	32.94	310.61	544.38	0.90	0.90	78.42	293.57	21.59	279.55	489.94	1.56	1.49	29.00	4.80	0.22	0.03	OK
125	V-Max	57.75	979.33	32.94	310.61	544.38	0.90	0.90	78.42	293.57	21.59	279.55	489.94	1.56	1.49	29.00	4.80	0.22	0.03	OK
125	V-Min	18.53	546.65	25.82	310.61	544.38	0.90	0.90	78.42	293.57	21.59	279.55	489.94	1.56	1.49	29.00	4.80	0.22	0.00	OK
126	T-Max	29.23	561.66	-34.00	305.07	544.38	0.90	0.90	76.10	284.88	24.62	274.57	489.94	1.52	1.45	29.00	4.80	0.22	0.01	OK
126	V-Max	68.08	924.44	-26.44	305.07	544.38	0.90	0.90	76.10	284.88	24.62	274.57	489.94	1.52	1.45	29.00	4.80	0.22	0.04	OK
126	V-Min	27.04	523.10	-33.58	305.07	544.38	0.90	0.90	76.10	284.88	24.62	274.57	489.94	1.52	1.45	29.00	4.80	0.22	0.00	OK
127	T-Max	37.95	526.22	-15.31	298.07	544.38	0.90	0.90	73.48	275.06	27.28	268.26	489.94	1.46	1.40	29.00	4.80	0.22	0.01	OK
127	V-Max	77.93	857.51	-7.91	298.07	544.38	0.90	0.90	73.48	275.06	27.28	268.26	489.94	1.46	1.40	29.00	4.80	0.22	0.05	OK
127	V-Min	35.29	490.75	-14.83	298.07	544.38	0.90	0.90	73.48	275.06	27.28	268.26	489.94	1.46	1.40	29.00	4.80	0.22	0.01	OK
128	T-Max	47.75	481.36	-8.66	295.61	544.38	0.90	0.90	72.14	270.08	29.72	266.05	489.94	1.41	1.38	29.00	4.80	0.22	0.02	OK
128	V-Max	88.88	780.01	-1.43	295.61	544.38	0.90	0.90	72.14	270.08	29.72	266.05	489.94	1.41	1.38	29.00	4.80	0.22	0.06	OK
128	V-Min	44.26	408.58	-3.53	295.61	544.38	0.90	0.90	72.14	270.08	29.72	266.05	489.94	1.41	1.38	29.00	4.80	0.22	0.02	OK
129	T-Max	58.47	426.59	-6.93	297.55	544.38	0.90	0.90	72.14	270.08	31.67	267.80	489.94	1.35	1.38	29.00	4.80	0.22	0.03	OK
129	V-Max	100.74	690.16	0.38	297.55	544.38	0.90	0.90	72.14	270.08	31.67	267.80	489.94	1.35	1.38	29.00	4.80	0.22	0.07	OK
129	V-Min	52.90	359.80	-2.40	297.55	544.38	0.90	0.90	72.14	270.08	31.67	267.80	489.94	1.35	1.38	29.00	4.80	0.22	0.02	OK
130	T-Max	69.87	361.55	-7.37	299.05	544.38	0.90	0.90	72.14	270.08	33.16	269.14	489.94	1.29	1.38	29.00	4.80	0.22	0.04	OK
130	V-Max	113.30	587.40	0.07	299.05	544.38	0.90	0.90	72.14	270.08	33.16	269.14	489.94	1.29	1.38	29.00	4.80	0.22	0.08	OK
130	V-Min	62.16	302.15	-2.30	299.05	544.38	0.90	0.90	72.14	270.08	33.16	269.14	489.94	1.29	1.38	29.00	4.80	0.22	0.03	OK
131	T-Max	81.71	285.85	-8.64	300.31	544.38	0.90	0.90	72.14	270.08	34.42	270.28	489.94	1.22	1.38	29.00	4.80	0.22	0.05	OK
131	V-Max	126.38	471.27	-1.13	300.31	544.38	0.90	0.90	72.14	270.08	34.42	270.28	489.94	1.22	1.38	29.00	4.80	0.22	0.09	OK
131	V-Min	71.82	235.27	-2.59	300.31	544.38	0.90	0.90	72.14	270.08	34.42	270.28	489.94	1.22	1.38	29.00	4.80	0.22	0.04	OK

Elem	Max/Min	Vu (tonf)	Mu (tonf*m)	Tu (tonf*m)	Vn (tonf)	Tn (tonf*m)	Phi	Phi_t	Vc (tonf)	Vs (tonf)	Vp (tonf)	PhiVn (tonf)	Phi_tTn (tonf*m)	de (m)	dv (m)	theta ([deg])	beta	bv (m)	bv_min (m)	CHK
132	T-Max	94.26	199.06	-9.74	301.13	544.38	0.90	0.90	72.14	270.08	35.24	271.01	489.94	1.16	1.38	29.00	4.80	0.22	0.06	OK
132	V-Max	140.20	341.57	-2.13	301.13	544.38	0.90	0.90	72.14	270.08	35.24	271.01	489.94	1.16	1.38	29.00	4.80	0.22	0.10	OK
132	V-Min	82.12	158.79	-2.77	301.13	544.38	0.90	0.90	72.14	270.08	35.24	271.01	489.94	1.16	1.38	29.00	4.80	0.22	0.05	OK
133	T-Max	107.53	100.55	-8.93	301.58	544.38	0.90	0.90	72.14	270.08	35.69	271.42	489.94	1.09	1.38	29.00	4.80	0.22	0.07	OK
133	V-Max	154.74	197.79	-0.94	301.58	544.38	0.90	0.90	72.14	270.08	35.69	271.42	489.94	1.09	1.38	29.00	4.80	0.22	0.11	OK
133	V-Min	93.02	72.14	-2.07	301.58	544.38	0.90	0.90	72.14	270.08	35.69	271.42	489.94	1.09	1.38	29.00	4.80	0.22	0.06	OK
134	T-Max	187.00	-2.30	5.48	199.10	323.65	0.90	0.90	18.04	160.57	20.49	179.19	291.28	0.94	1.38	43.00	1.20	0.22	0.16	OK
134	V-Max	187.00	-2.30	5.48	199.10	323.65	0.90	0.90	18.04	160.57	20.49	179.19	291.28	0.94	1.38	43.00	1.20	0.22	0.16	OK
134	V-Min	121.54	-67.54	1.13	286.38	498.88	0.90	0.90	49.26	247.51	20.49	257.74	448.99	0.94	1.38	31.17	3.28	0.22	0.09	OK
135	T-Max	202.87	-192.32	24.66	184.32	297.90	0.90	0.90	16.01	147.79	20.52	165.89	268.11	1.01	1.38	45.37	1.06	0.22	0.17	OK
135	V-Max	202.87	-192.32	24.66	184.32	297.90	0.90	0.90	16.01	147.79	20.52	165.89	268.11	1.01	1.38	45.37	1.06	0.22	0.17	OK
135	V-Min	133.00	-192.32	10.96	261.56	424.55	0.90	0.90	30.41	210.63	20.52	235.41	382.10	1.01	1.38	35.40	2.02	0.22	0.11	OK

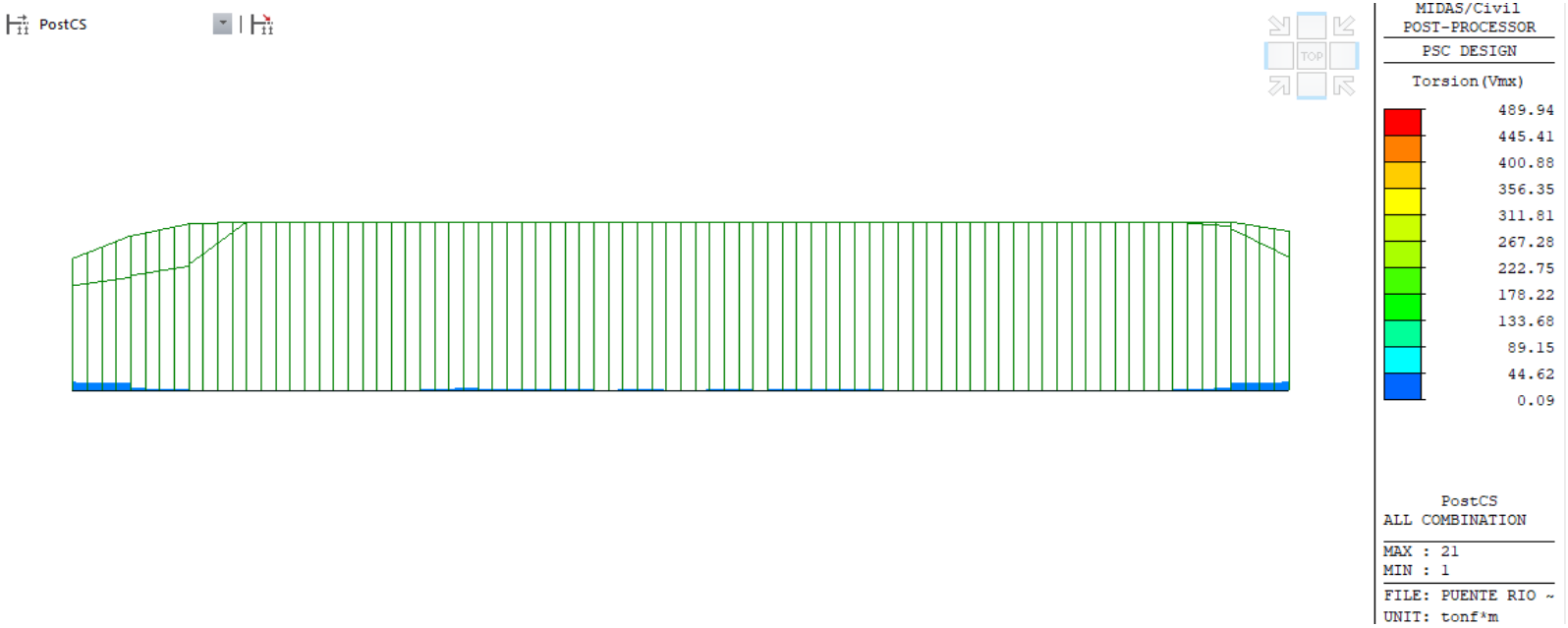


Figura 188. Verificación de capacidad de torsión por fuerza cortante (Vmx)

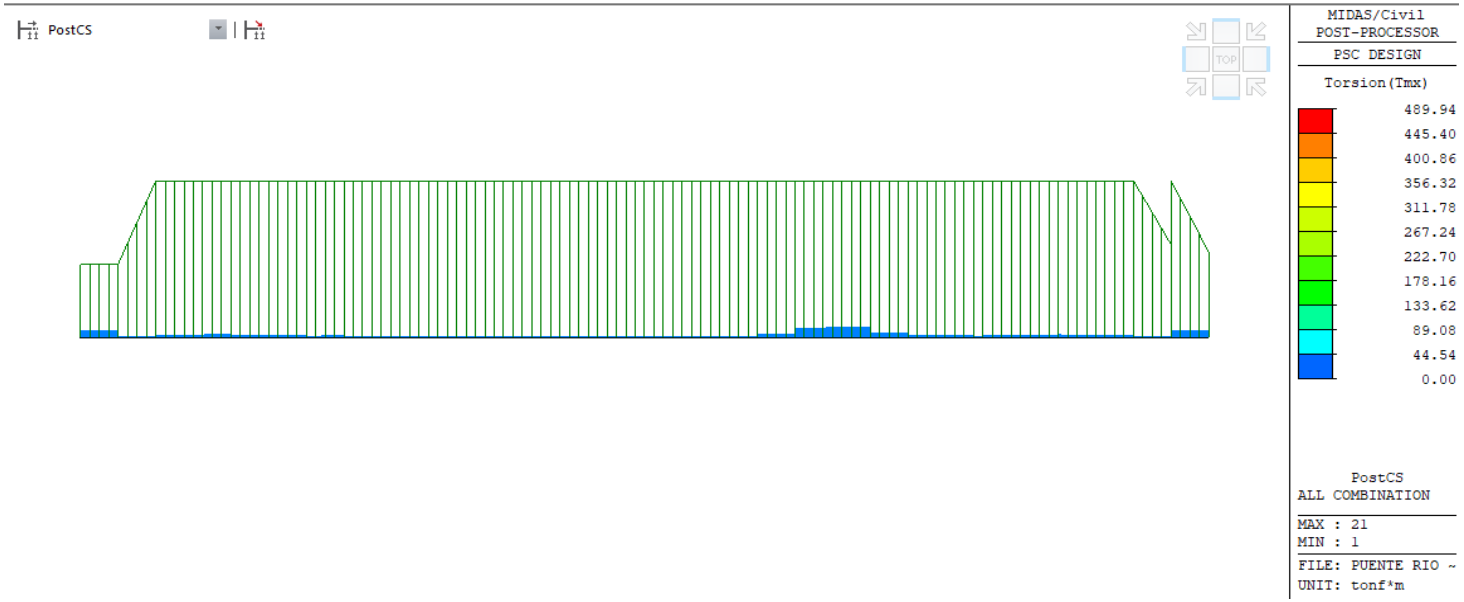


Figura 189. Verificación de capacidad de torsión combinada (Tmx)

3.4.24.4. VERIFICACIÓN POR ANCHO DE FISURA POR CARGAS DE SERVICIO

Elem	Top/Bottom	FT (kgf/cm^2)	FB (kgf/cm^2)	fss (kgf/cm^2)	s_use (cm)	s_max (cm)	CHK
106	Bottom	2.75	-10.47	72.03	15.25	1661.35	OK
107	Bottom	5.33	-20.54	141.40	15.25	841.40	OK
108	Bottom	7.81	-30.17	207.64	15.25	569.78	OK
109	Bottom	10.16	-39.29	270.45	15.25	435.12	OK
110	Bottom	12.36	-47.90	329.73	15.25	355.10	OK
111	Bottom	14.42	-55.94	385.05	15.25	302.65	OK
112	Bottom	16.32	-63.42	436.52	15.25	265.78	OK
113	Bottom	18.07	-70.38	484.50	15.25	238.47	OK
114	Bottom	19.72	-77.12	530.90	15.25	216.76	OK
115	Bottom	21.15	-83.27	573.28	15.25	199.99	OK
116	Bottom	22.31	-88.86	611.79	15.25	186.77	OK
117	Bottom	23.08	-93.90	646.58	15.25	176.19	OK
118	Bottom	22.76	-98.79	680.56	15.25	166.89	OK
119	Bottom	21.36	-103.43	712.94	15.25	158.86	OK
120	Bottom	18.63	-107.94	744.74	15.25	151.65	OK
121	Bottom	21.54	-103.41	712.75	15.25	158.90	OK
122	Bottom	23.00	-98.78	680.44	15.25	166.92	OK
123	Bottom	23.18	-94.00	647.24	15.25	176.00	OK
124	Bottom	22.34	-89.01	612.84	15.25	186.44	OK
125	Bottom	21.16	-83.44	574.40	15.25	199.58	OK
126	Bottom	19.71	-77.29	532.10	15.25	216.25	OK
127	Bottom	18.04	-70.56	485.75	15.25	237.84	OK
128	Bottom	16.26	-63.62	437.92	15.25	264.90	OK
129	Bottom	14.33	-56.17	386.64	15.25	301.36	OK
130	Bottom	12.24	-48.15	331.50	15.25	353.15	OK
131	Bottom	10.00	-39.57	272.40	15.25	431.94	OK
132	Bottom	7.64	-30.44	209.59	15.25	564.38	OK
133	Bottom	5.17	-20.81	143.26	15.25	830.32	OK
134	Bottom	2.63	-10.79	74.29	15.25	1610.53	OK
135	Bottom	0.00	0.00	0.00	15.25	639745508.49	OK

3.4.25. DISEÑO DEL TABLERO

3.4.25.1. ANCHO DE FRANJA EQUIVALENTE

Para el diseño del tablero se utilizó el método de análisis aproximado de las franjas, tal como se especifica en el numeral 4.6.2.1.1, en donde el tablero de subdivide en franjas perpendiculares a la dirección del tráfico

Se tomará una porción equivalente de tablero como muestra representativa para el diseño del tablero, por tanto, el ancho de franja equivalente se determinó con base a la tabla 4.6.2.1.3-1 de la CCP-14, que para el tablero de concreto vaciado en sitio se tiene lo siguiente:

Momento Positivo $(+\boldsymbol{M})$: $S_d\ M^+ = 0.66 + 0.55 * S = 0.66\ m + 0.55 * 2.84\ m = 2.22\ m$

Momento Negativo $(-\boldsymbol{M})$: $S_d\ M^- = 1.22 + 0.25 * S = 1.22\ m + 0.25 * 2.84 m = 1.93\ m$

Nota: para las franjas de diseño perpendiculares a la dirección del tráfico no existe límites en el ancho de la franja.
El diseño estructural del tablero aplica para los dos tramos, de luz de 21.0m y luz de 30.0m

3.4.25.2. FACTOR DE PRESENCIA MÚLTIPLE

La fuerza de carga viva para el diseño se determinó considerando las combinaciones posibles del número de carriles cargados afectados por un factor correspondiente de presencia múltiple, el cual tiene en cuenta la probabilidad de ocupación simultanea de los carriles de carga por la carga viva CC-14 completa. Se presenta los factores de presencia múltiple empleados con base a la tabla 3.6.1.1.2-1 de la CCP-14.

- Fpm = 1.20 Un (1) carril cargado
- Fpm = 1.00 dos (2) carriles cargados

3.4.25.3. CARGAS VIVAS APLICADAS

Para estudiar la influencia tanto para momento positivo como negativo de la carga viva en el tablero, se evaluaron los esfuerzos causados por uno (1) y dos (2) camiones y actuando simultáneamente con uno (1) y dos (2) carriles, tal como se muestra a continuación:

La carga de camión por metro de tablero para uno (1) y dos (2) carriles cargados está dado por:

Momento Positivo

$$P_1 = \frac{E_C F_{pm}(1+IM)}{S_d M^+} = \frac{80\ kN * \ 1.2 * \ (1+0.33)}{2.22\ m} = 57.51 \frac{kN}{m}$$

$$P_2 = \frac{E_C F_{pm}(1+IM)}{S_d\ M^+} = \frac{80\ kN * \ 1.0 * \ (1+0.33)}{2.22\ m} = 47.92 \frac{kN}{m}$$

Momento Negativo

$$P_1 = \frac{E_C F_{pm}(1+IM)}{S_d M^+} = \frac{80\ kN * \ 1.2 * \ (1+0.33)}{1.93\ m} = 66.15 \frac{kN}{m}$$

$$P_2 = \frac{E_C F_{pm}(1+IM)}{S_d\ M^+} = \frac{80\ kN * \ 1.0 * \ (1+0.33)}{1.93\ m} = 55.13 \frac{kN}{m}$$

La carga de carril por metro de tablero para uno (1) y dos (2) carriles cargados está dado por:

$$Wc_1 = \frac{W_C F_{pm}}{3.0} = \frac{10.3\ \frac{kN}{m} * \ 1.2}{3.00\ m} = 4.12\ \frac{kN}{m^2}$$

$$Wc_2 = \frac{W_C F_{pm}}{3.0} = \frac{10.3\ \frac{kN}{m} * \ 1.0}{3.00\ m} = 3.44\ \frac{kN}{m^2}$$

Se tomarán las condiciones más desfavorables para el diseño del tablero, es decir, la condición para momento negativo con un carril

3.4.25.4. MODELO ESTRUCTURAL

Se modeló el tablero estructural en| MIDAS CIVIL como un elemento de ancho unitario y espesor 0.25 m correspondiente al espesor seleccionado del tablero, los apoyos del tablero se consideraron como articulaciones en las vigas longitudinales, tal como se muestra a continuación.



Figura 190. Modelo del tablero en MIDAS CIVIL

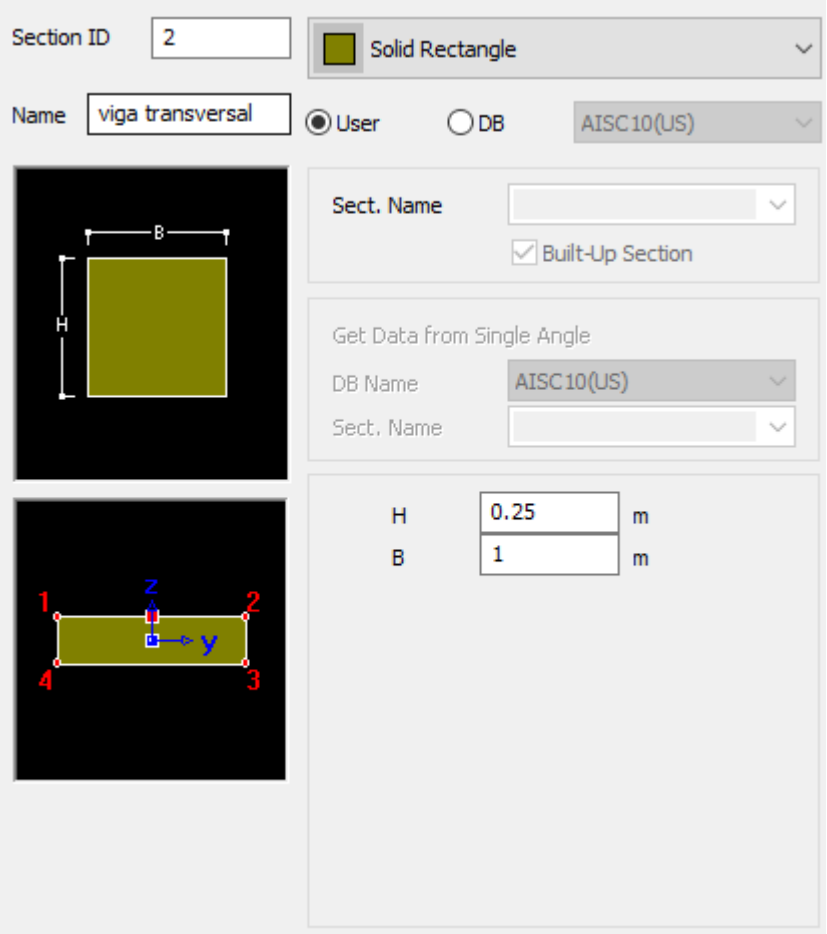


Figura 191. Sección transversal del modelo del tablero

CARGAS APLICADAS

Carga Muerta

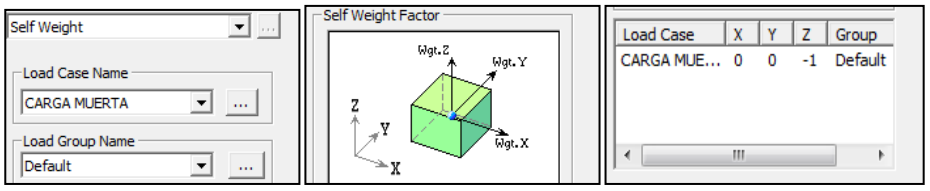


Figura 192. Asignación de la carga muerta en el modelo del tablero

Bordillo: 0.15 t/m

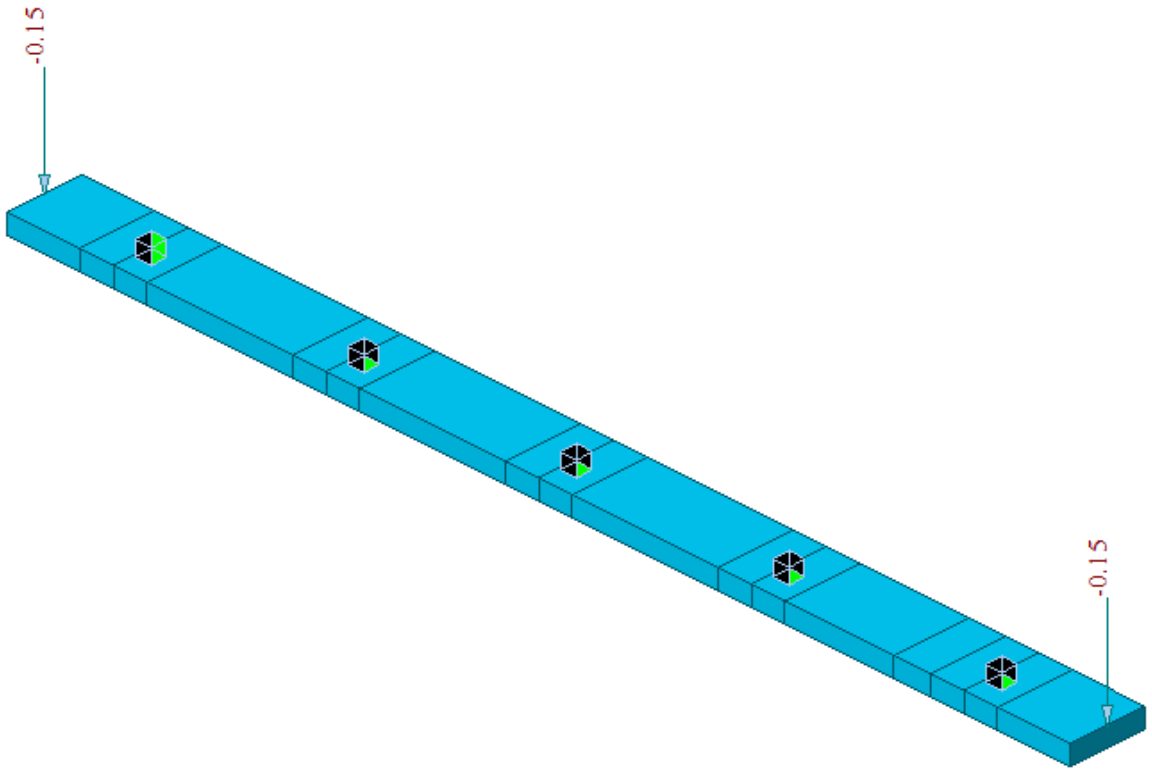


Figura 193. Asignación de la carga de bordillo en el modelo del tablero

Baranda: 0.10 t/m

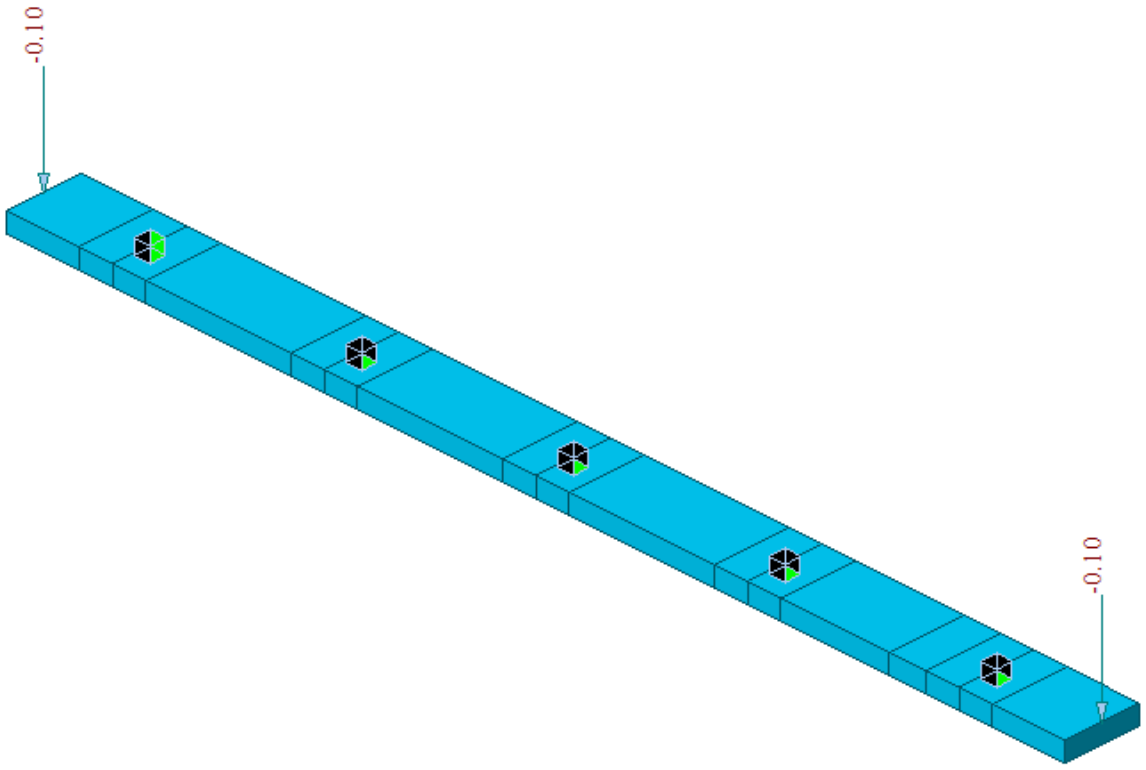


Figura 194. Asignación de la carga de baranda en el modelo del tablero

New Jersey: 0.10 t/m

La carga de barrera New Jersey se aplica a 0.55 m del eje de la viga externa izquierda y a 0.58m del eje de la viga interna

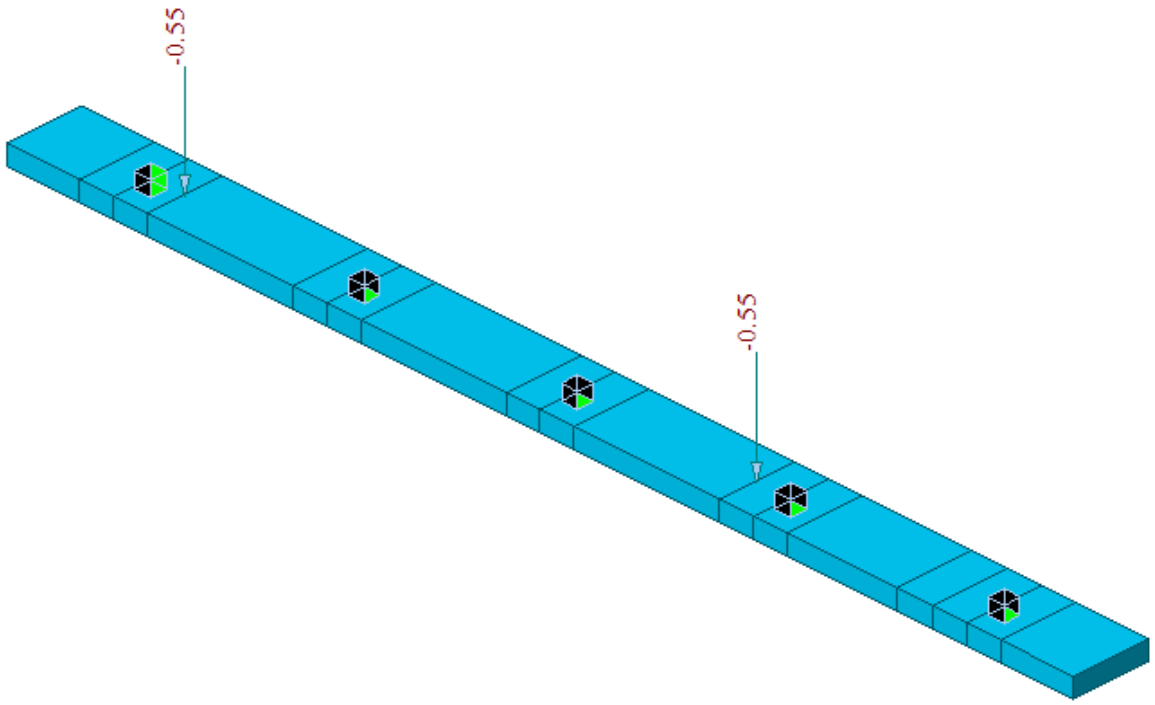


Figura 195. Asignación de la carga de New Jersey en el modelo del tablero

Andén:

Andén derecho: (1.60m + 2.60m + 0.30m) * 0.22m * 2.40 ton/m3 = 2.40 ton/m
Andén izquierdo: 1.60m * 0.22m * 2.40 ton/m3 = 0.85 ton/m

Para el andén derecho, se tiene un ancho de pasarela de 4.50m y espesor de 0.22m; se aplica como carga distribuida con una excentricidad de 0.98m medidas desde el eje de la viga externa derecha del puente.

Para el andén izquierdo, se tiene un ancho de pasarela de 1.60m y espesor de 0.22m; se aplica como carga distribuida con una excentricidad de 0.47m medidas desde el eje de la viga externa izquierda del puente.

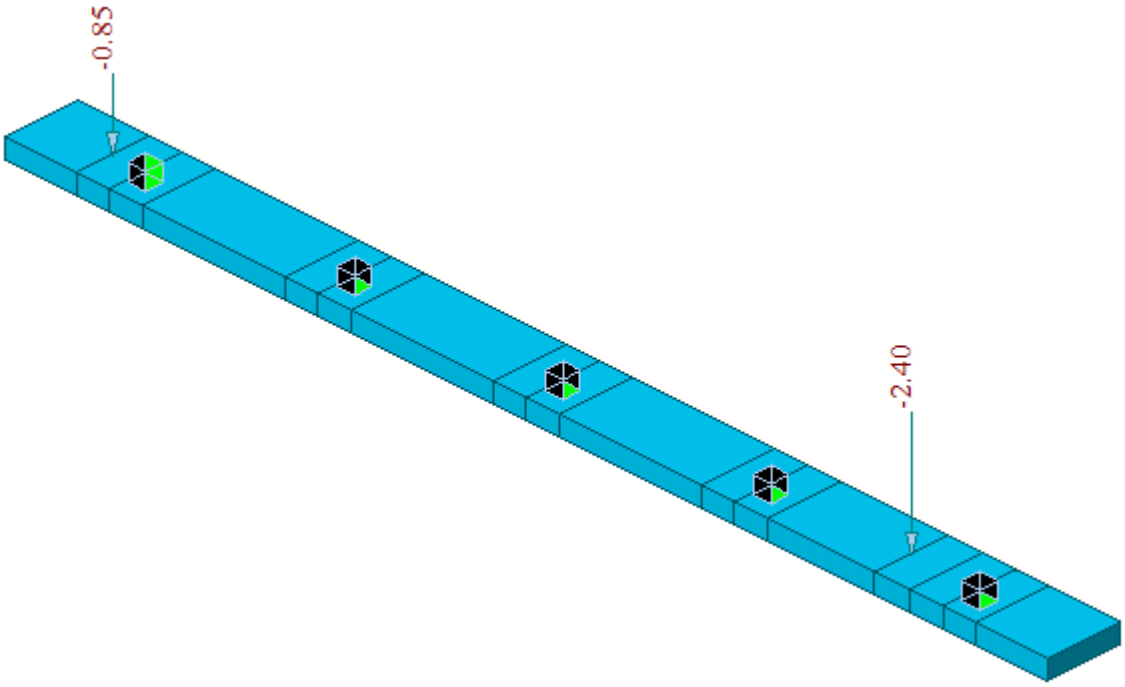


Figura 196. Asignación de la carga de andén en el modelo del tablero

Pavimento:

Espesor pavimento más repavimentación e=0.075m
Ancho de doble calzada 7.0m
Densidad pavimento γpav = 2.25 ton/m3

DW = 0.075m * 7.0m * 2.25 ton/m3 = 1.20 ton/m

La carga de pavimento se aplica sobre dos vigas internas, es decir:

DW = 1.20 ton/m / 2 vigas = 0.60 ton/m por cada viga

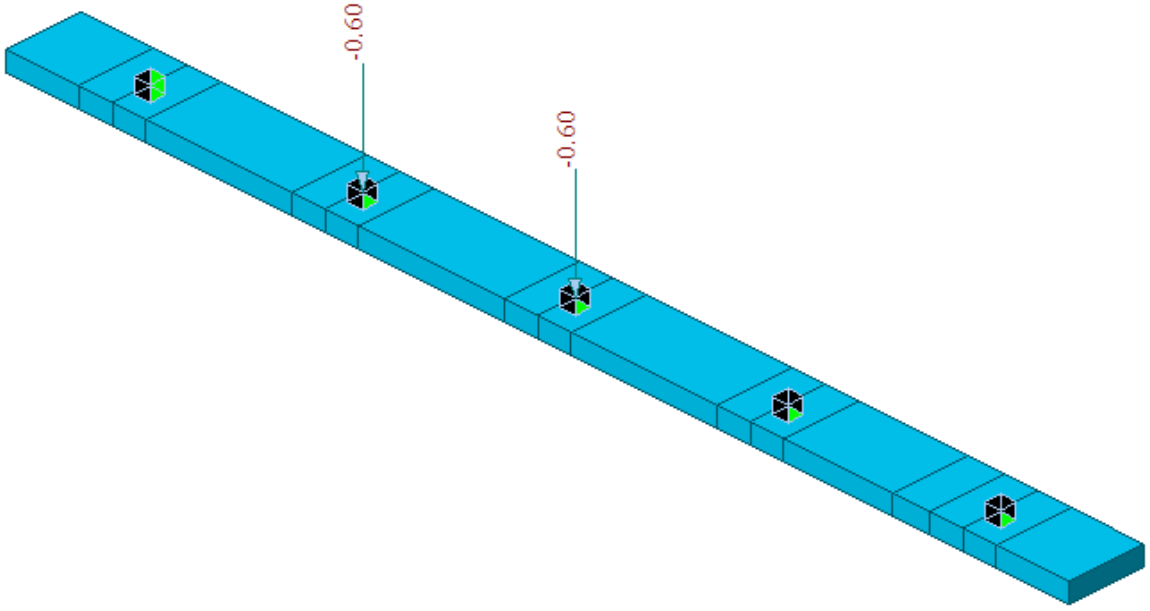


Figura 197. Asignación de la carga de pavimento en el modelo del tablero

Peatonal:

Es decir, la carga aplicada es de 90 libras por pie cuadrado (1 psf es equivalente a 0.0478803 kPa) aproximadamente 4.40 kN/m^2 (0.45 ton/m2), con el fin de calcular los efectos torsionales producidos por la carga parcial de la pasarela, la carga viva peatonal y ciclo-ruta está determinada por

$$PL = 0.45\ {}^{ton}/_{m^2} * 1.60\ m = 0.75\ {}^{ton}/_{m}$$

La carga peatonal del costado izquierdo, se aplica con una excentricidad de 0.47m

$$PL = 0.45\ {}^{ton}/_{m^2} * 4.50\ m = 2.05\ {}^{ton}/_{m}$$

La carga peatonal del costado derecho, se aplica con una excentricidad de 0.98m

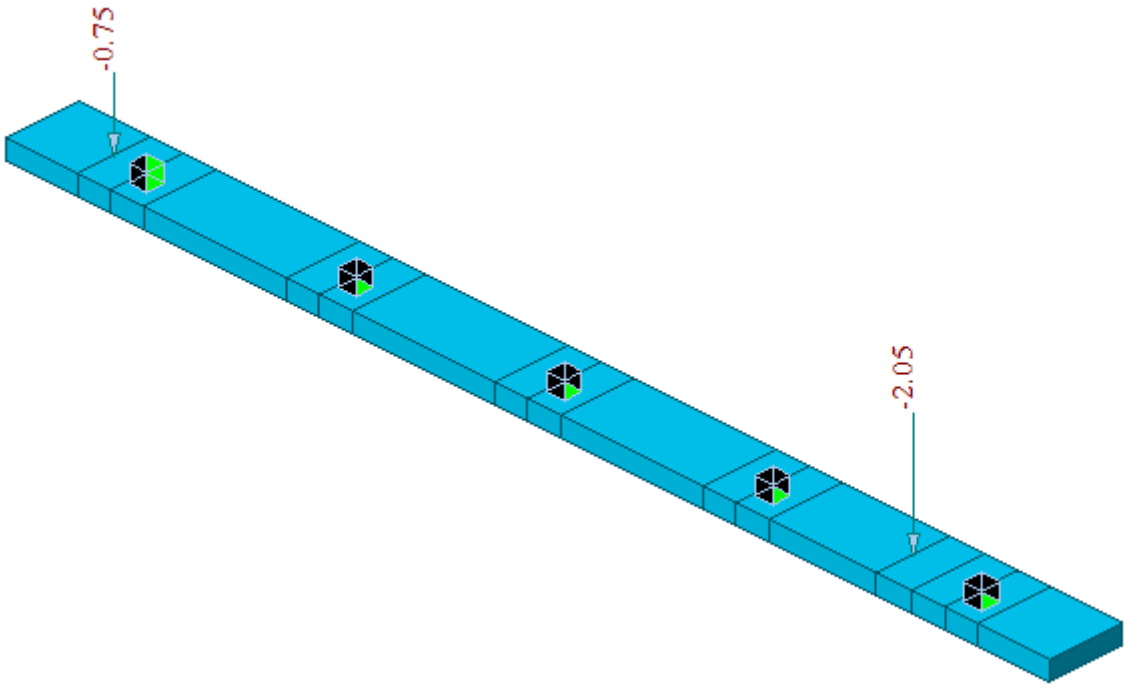


Figura 198. Asignación de la carga de peatonal en el modelo del tablero

Carga Viva

Se generó un Lane Transversal para cargar el eje trasero del camión de diseño

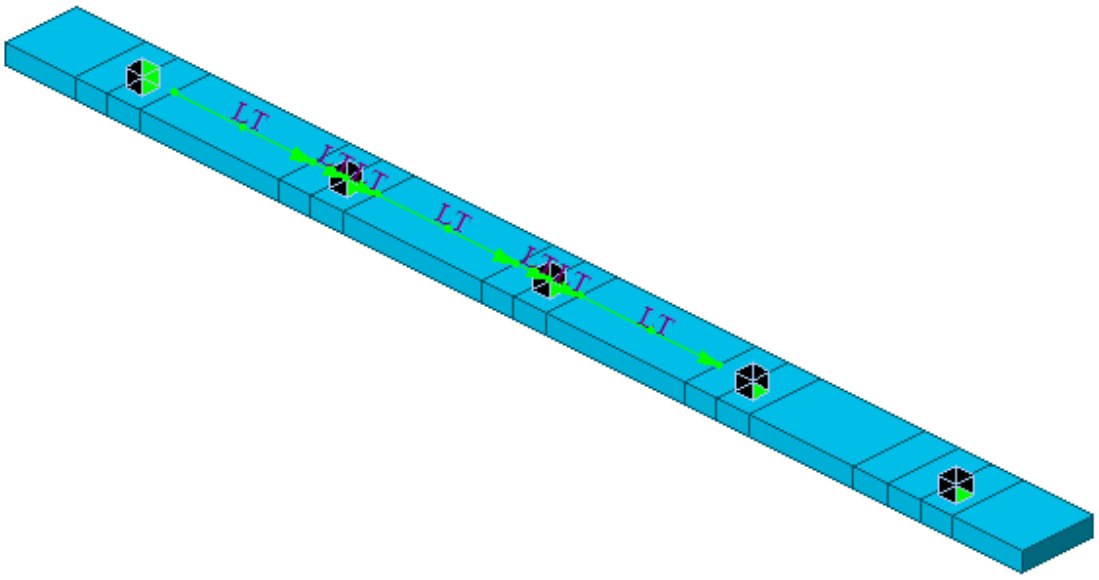


Figura 199. Lane Transversal para la carga viva en el modelo del tablero

Eje Camión:

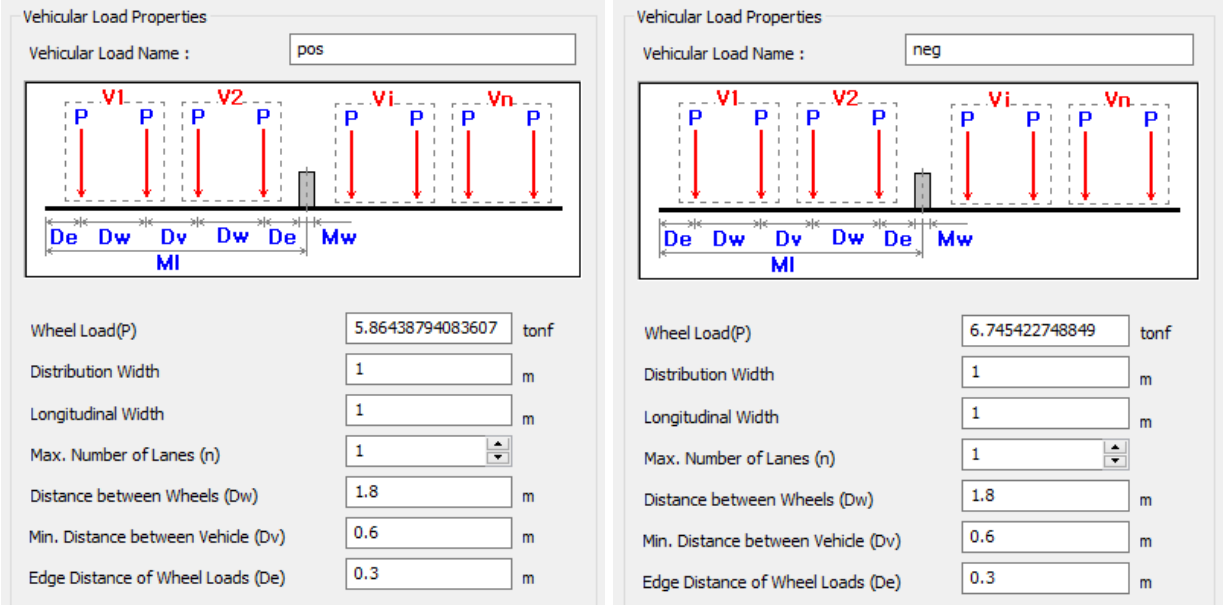


Figura 200. Carga viva de 1 carril para momento positivo y negativo

3.4.25.5. SOLICITACIONES DE DISEÑO MOMENTO POSITIVO Y NEGATIVO

Se presentan los momentos últimos de los respectivos modelos estructurales del tablero.

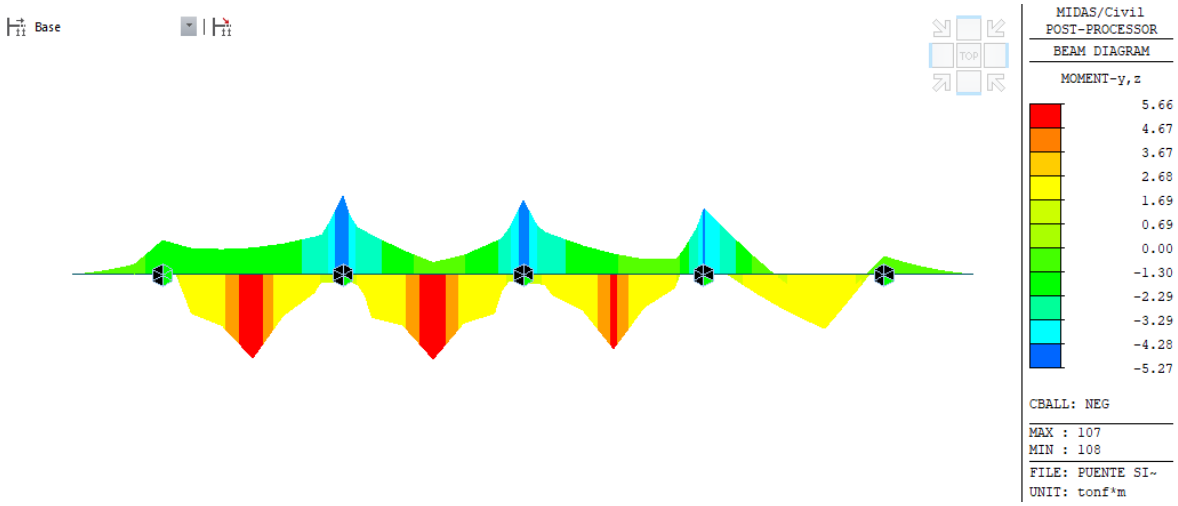


Figura 201. Momentos máximos positivo y negativo

MOMENTOS DE DISEÑO

$$M^{+} = 5.66\ t.m$$
$$M^{-} = 5.27\ t.m$$

1. Design Information

Design Code : AASHTO-LRFD16

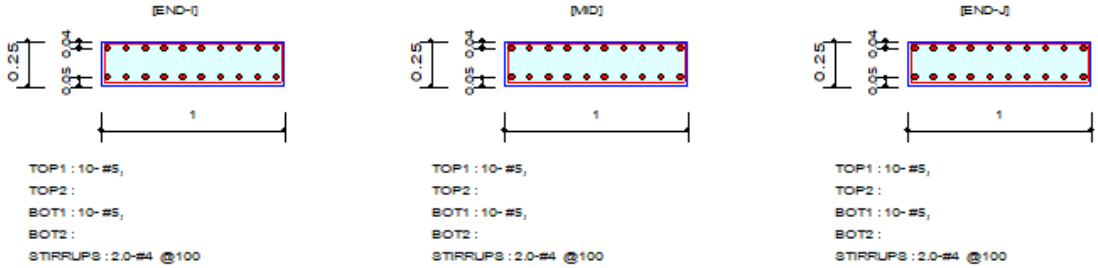
Unit System : tonf, m

Material Data : fc = 2812.28, fy = 42184.2, fys = 42184.2 tonf/m^2

Beam Span : 1.455 m

Section Property : viga transversal (No : 2)

2. Section Diagram



3. Bending Moment Capacity

	END-I	MID	END-J
Negative Moment (Mu)	5.27	4.33	5.27
(-) Load Combination No.	2-	2-	2-
Factored Strength (Mr)	14.58	14.58	14.58
Check Ratio (Mu/Mr)	0.3618	0.2971	0.3618
Positive Moment (Mu)	3.63	5.66	3.63
(+) Load Combination No.	2+	2+	2+
Factored Strength (Mr)	13.82	13.82	13.82
Check Ratio (Mu/Mr)	0.2629	0.4098	0.2629
Using Rebar Top (As_top)	0.0020	0.0020	0.0020
Using Rebar Bot (As_bot)	0.0020	0.0020	0.0020

4. Shear Capacity

	END-I	MID	END-J
Load Combination No.	2-	2-	2+
Factored Shear Force (Vu)	17.47	15.80	16.94
Shear Strength by Conc.(PhiV)	20.23	22.86	21.57
Shear Strength by Rebar.(PhiVs)	29.19	30.31	28.70
Using Shear Reinf. (Av)	0.0001	0.0001	0.0001
Using Stirrups Spacing	2.0-#4 @100	2.0-#4 @100	2.0-#4 @100
Check Ratio	0.3536	0.2971	0.3370

5. Torsion Capacity

	END-I	MID	END-J
Load Combination No.	2+	2+	2+
Factored Torsion (Tu)	0.00	0.00	0.00
Torsion Resistance (Tn)	27.64	27.64	27.25
Required Stirrups Spacing	0.00	0.00	0.00
Check Ratio	0.0000	0.0000	0.0000

```
================================================================================================
 MIDAS/Civil - RC-Beam Checking   [ AASHTO-LRFD16 ]                              Civil 2019
================================================================================================

      *.MIDAS/Civil - RC-BEAM Analysis/Design Program.

      *.PROJECT       :
      *.DESIGN CODE   : AASHTO-LRFD16,    *.UNIT SYSTEM : tonf, m
      *.MEMBER        : Member Type = BEAM,   MEMB =   101

      *.DESCRIPTION OF BEAM DATA (iSEC =     2) : viga transversal
        Section Type : Rectangle (RECT)
        Beam Length (Span)             =         0.450 m.
        Section Depth (Hc)             =         0.250 m.
        Section Width (Bc)             =         1.000 m.
        Concrete Strength (fc')        =      2812.279 tonf/m^2.
        Main Rebar Strength (fy)       =     42184.188 tonf/m^2.
        Stirrups Strength (fys)        =     42184.188 tonf/m^2.
        Modulus of Elasticity (Es)     =  20389024.157 tonf/m^2.

      *.FORCES AND MOMENTS AT CHECK POINT <I> :
        Positive Bending Moment   P-Mu =        0.52 tonf-m.,  LCB =  2+
        Negative Bending Moment   N-Mu =        5.27 tonf-m.,  LCB =  2-
        Shear Force               Vu   =       17.47 tonf.  ,  LCB =  2-

      *.REINFORCEMENT PATTERN :
        ------------------------------------------------------
           Location    i     di( m.)       Rebar    Asi( m^2.)
        ------------------------------------------------------
           Top         1      0.040       10- #5      0.00200
           Bottom      1      0.050       10- #5      0.00200
        ------------------------------------------------------


================================================================================
   [[[*]]]   ANALYZE NEGATIVE BENDING MOMENT CAPACITY.
================================================================================

      ( ). Compute parameter.
           -. phi   =   0.90
           -. Alpha =   0.85
           -. Beta  =   0.85
           -. d     =     0.2100 m.
           -. ecu   =  0.0030

      ( ). Compute maximum and minimum reinforcement.
           -. Rhomin1 = (1.2)*Mcr/[ phi*fy*b*d*(d-a/2) ]      =   0.0026
           -. Rhomin2 = 1.33*Mu/[ phi*fy*b*d*(d-a/2) ]        =   0.0044
           -. Rhomin  = MIN[ Rhomin1, Rhomin2 ]               =   0.0026
           -. As_min =  Rhomin * Ag  =     0.0006 m^2.

      ( ). Check tensile reinforcement.
           -. As     =     0.0020 m^2.
           -. As_min < As   --->  O.K !
```

```
     ( ). Check moment capacity.
          -. c          =     0.0413 m.
          -. Cc         =         84.00 tonf.
          -. Ts         =         84.37 tonf.
          -. Mr         =         14.58 tonf-m.
          -. Mu/Mr      =   0.362  --->  O.K !

================================================================================
   [[[*]]]   ANALYZE POSITIVE BENDING MOMENT CAPACITY.
================================================================================

     ( ). Compute parameter.
          -. phi   =   0.90
          -. Alpha =   0.85
          -. Beta  =   0.85
          -. d     =     0.2000 m.
          -. ecu   =  0.0030

     ( ). Compute maximum and minimum reinforcement.
          -. Rhomin1 = (1.2)*Mcr/[ phi*fy*b*d*(d-a/2) ]      =   0.0028
          -. Rhomin2 = 1.33*Mu/[ phi*fy*b*d*(d-a/2) ]        =   0.0005
          -. Rhomin  = MIN[ Rhomin1, Rhomin2 ]               =   0.0005
          -. As_min =  Rhomin * Ag  =     0.0001 m^2.

     ( ). Check tensile reinforcement.
          -. As     =     0.0020 m^2.
          -. As_min < As   --->  O.K !

     ( ). Check moment capacity.
          -. c          =     0.0413 m.
          -. Cc         =         84.00 tonf.
          -. Ts         =         84.37 tonf.
          -. Mr         =         13.82 tonf-m.
          -. Mu/Mr      =   0.038  --->  O.K !

================================================================================
   [[[*]]]   ANALYZE SHEAR CAPACITY.
================================================================================

     ( ). Compute shear parameter.
          -. phi   =   0.90
          -. Av    =     0.0003 m^2.
          -. bv    =   1.00 m.
          -. dv    =  MAX[ dv, 0.9*d, 0.72*Hc ]  =   0.19 m.
          -. theta =  32.85 Deg.   [Clause 5.8.3.4.2]
          -. beta  =   2.63

     ( ). Compute shear strength of concrete.
          -. Vu     =       17.47 tonf.
          -. Vc     = 0.0316*beta*SQRT[fc']*bv*dv =       22.47 tonf.
          -. phiVc  = phi * Vc =       20.23 tonf.
          -. Vn_lim = 0.25*fc'*bv*dv =      135.24 tonf.
```

```
     ( ). Compute stirrup spacing.
          -. Maximum spacing smax = MIN[ 0.8*dv, 24 in ] =    0.154 m.
          -. phiVc/2 < Vu < phiVc  --->  Required minimum shear reinforcement.
          -.                  smax = MIN[ smax, Av/(0.0316*SQRT(fc')*bv/fys) ] =    0.154 m.
          -. Applied spacing s    =    0.100 m.

     ( ). Compute shear strength of reinforcement.
          -. Vs      = Av*fys*dv*cot(theta) / s =       32.43 tonf.
          -. Vs_lim  = 0.25*fc'*bv*dv - Vc =      112.77 tonf.
          -. Vs      = MIN[ Vs, Vs_lim ] =       32.43 tonf.
          -. phiVs   = phi*Vs =       29.19 tonf.

          -. phiVs > (Vu-phiVc)  --->  O.K !
          -. Using Av/s  =  Vs / (fys*dv*cot(theta)) =   0.0026 m^2/m.

     ( ). Check tension force in the longitudinal reinforcement caused by shear.
          -. phib    =   0.90
          -. phiv    =   0.90
          -. Vsl     =  MIN[ Vs, Vu/phiv ] =       19.42 tonf.
          -. As_req  =  [ Mu/(phib*dv) + (Vu/phiv - 0.5*Vsl)*cot(theta) ] / fy =   0.0011 m^2.
          -. As      =   0.0020 m^2.
          -. As_req < As  --->  O.K !

================================================================================
   [[[*]]]   ANALYZE TORSION CAPACITY.
================================================================================

     ( ). Compute torsion parameter.
          -. phi   =   0.90
          -. Acp   =       0.25 m^2.
          -. Pc    =       2.50 m.
          -. Aoh   =       0.19 m^2.
          -. Ao    =  0.85*Aoh =      0.16 m^2.
          -. Ph    =       2.25 m.
          -. Tu    =          0.00 tonf-m.
          -. theta =  32.02 Deg.
          -. Tcr   =  0.125*SQRT(fc')*Acp^2/Pc =         4.39 tonf-m.

          -. Tu < 0.25*phi*Tcr =         0.99 tonf-m.
              ---> Torsion check is not required.
```

```
================================================================================
   [[[*]]]   ANALYZE CRACK.
================================================================================

     ( ). Check crack control of top reinforcement. ( LCB =  14- )
          -. Mu      =      4.3581 tonf-m.
          -. h       =      0.2500 m.
          -. dc      =      0.0400 m.
          -. Beta_s  = 1+dc/(0.7*(h-dc)) = 1.27 m.
          -. Gamma_e = 1.00
          -. c_cr    =      0.0649 m.
          -. Icr     =      0.0004 m^4.
          -. fss     =     11713.862 tonf/m^2.
          -. s       =       0.0976 m.
          -. sa      = 700*Gamma_e/(Beta_s*fss)-2*dc =      0.7589 m.
          -. s < sa --->  O.K !

     ( ). Check crack control of bottom reinforcement. ( LCB =  14+ )
          -. Mu      =      0.5960 tonf-m.
          -. h       =      0.2500 m.
          -. dc      =      0.0500 m.
          -. Beta_s  = 1+dc/(0.7*(h-dc)) = 1.36 m.
          -. Gamma_e = 1.00
          -. c_cr    =      0.0617 m.
          -. Icr     =      0.0004 m^4.
          -. fss     =      1683.609 tonf/m^2.
          -. s       =       0.0954 m.
          -. sa      = 700*Gamma_e/(Beta_s*fss)-2*dc =      5.3710 m.
          -. s < sa --->  O.K !
```

3.4.25.6. DISEÑO REFUERZO DE DISTRIBUCIÓN

Para garantizar la distribución de las cargas concentradas, se debe colocar longitudinalmente, en la cara del refuerzo principal del tablero, un porcentaje del refuerzo requerido para el momento de diseño, el porcentaje de distribución está dado por (9.7.3.2):

$$\%d_{ist} = \frac{3840}{\sqrt{S}} \leq 67\%; = \frac{3840}{\sqrt{2840}} = 72.05\%, Se\ elige\ \%d_{ist} = 67\%$$

$$A_s = 0.67 * 19.90\ cm^2 = 13.33\ cm^2$$

Empleando barras No.5 cada 0.15

$$A_s sumin = 1.99 cm^2 * 7\ barras = 13.93\ cm^2 \qquad A_s sumin > A_s\ OK, cumple$$

Por lo tanto, se empleará barras No.5 cada 0.15 en el sentido longitudinal y barras No.5 cada 0.10 en el sentido transversal, es decir en la cara inferior del tablero

3.4.25.7. DISEÑO REFUERZO RETRACCIÓN Y TEMPERATURA

El refuerzo dispuesto por retracción y temperatura debe cumplir lo siguiente (C5.10.8):

$$A_s \geq 0.0018 * b * h = 0.0018 * 18\ cm * 100\ cm = 3.24\ cm^2$$

Los cuales se deben distribuir en un ancho de 1.0 m, y serán atendidos con barras N4 cada 0.30 m.

3.5. DISEÑO DE VIGAS DIAFRAGMA

Para el diseño estructural de las vigas diafragma se plantea como elementos tipo Frame, los cuales sirven para poder redistribuir cargas y evitar la torsión y pandeo de las vigas postensadas.

Se presenta la geometría de las vigas diafragma extremas e intermedias.

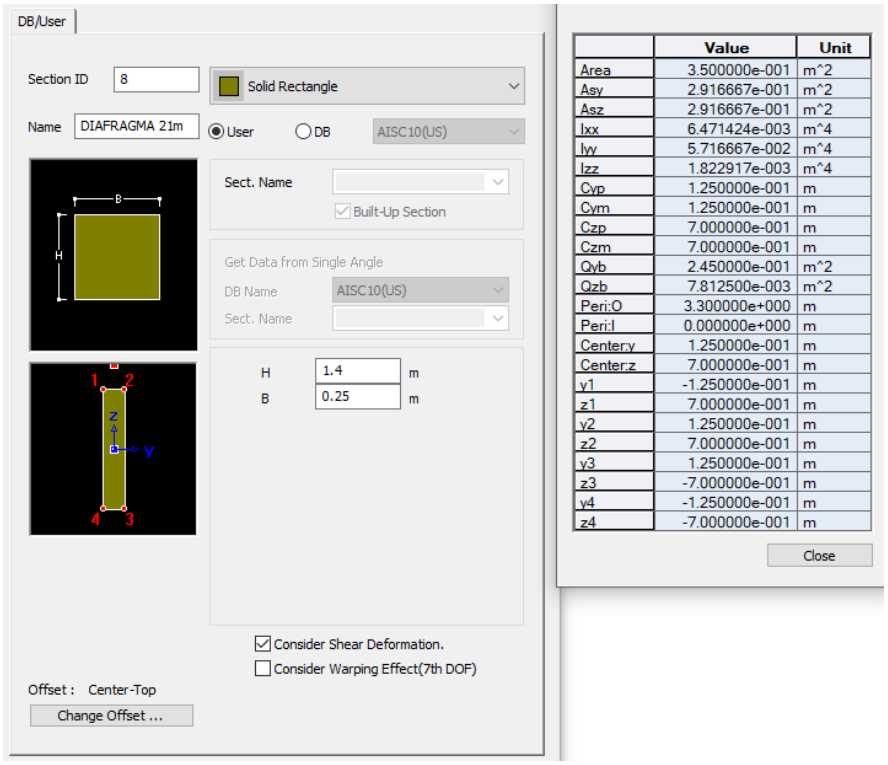


Figura 202. Viga diafragma viga 21.0m y 30.0m

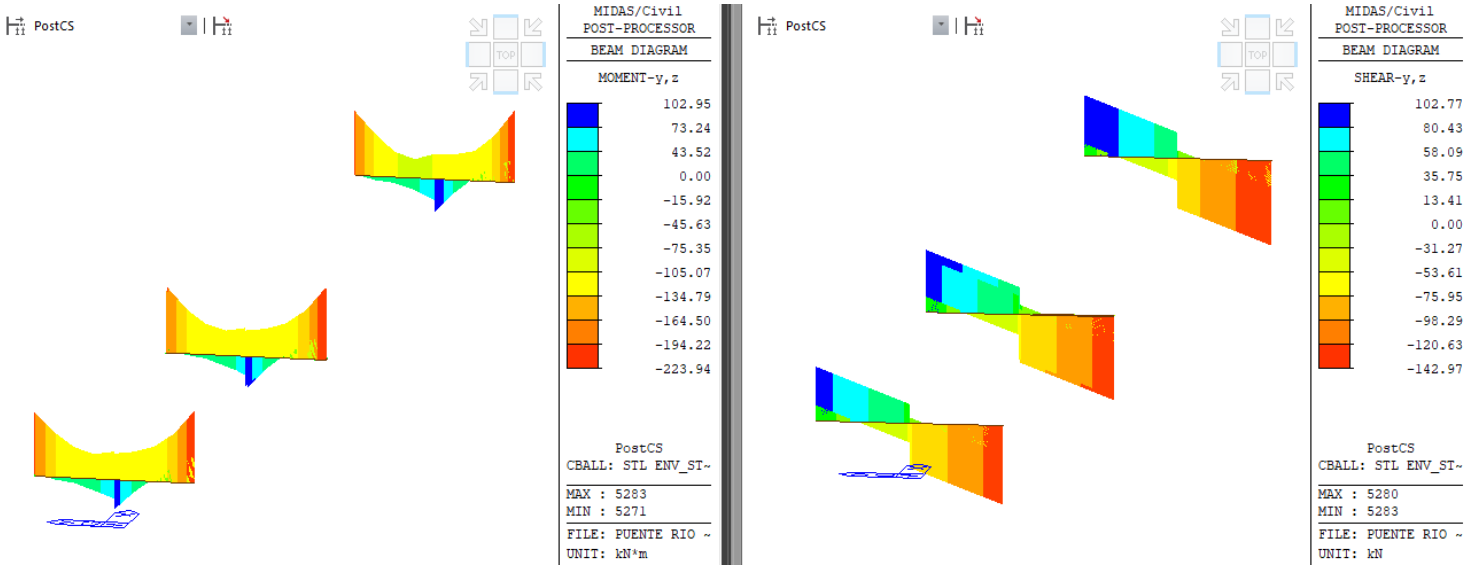


Figura 203. Momento flector y fuerza cortante para vigas diafragma

SOLICITACIONES DE DISEÑO ESTRUCTURAL

$M_U{}^{(+)} = 102.95\ kN.m$ $\quad V_U{}^{(+)} = 102.77\ kN$

$M_U{}^{(-)} = -223.94\ kN.m$ $\quad V_U{}^{(-)} = -142.97\ kN$

DISEÑO A FLEXIÓN POSITIVA	
Resistencia del concreto f'c	280 kg/cm²
Fluencia del acero de refuerzo Fy	4200 kg/cm²
Momento de diseño Mu	102.95 kN-m/m
Momento de diseño Mu	10.51 ton-m/m

SECCIÓN	
$\phi_{FLUENCIA}$	0.9
Ancho b (cm)	25.0 cm
Altura h (cm)	141.0 cm
Recubrimiento (cm)	5.0 cm
Profundidad efectiva d (cm)	136.0 cm
Cuantía Calculada ρ_{CALC}	0.00060
Factor del bloque a comp. β1	0.85
Cuantía Balanceada ρ_b	0.02856
Cuantía Mínima $\rho_{Mínima}$	0.00333
Verificación Cuantía Máxima	MENOR QUE LA BALANCEADA
Verificación Cuantía Mínima	MENOR QUE LA MÍNIMA
Cuantía Seleccionada $\rho_{Seleccionada}$	0.00060

As a Utilizar (cm²)	2.05 cm²
As a Colocado (cm²)	1.61 cm²
Cuantía calculada	**0.0005**

Prof. Bl. de Compresión a (cm)	1.45 cm
Prof. Eje Neutro C (cm)	1.71 cm
Resistencia a la flexión refuerzo utilizado ϕMn (Ton-m)	8.25 ton-m
VERIFICACIÓN RESISTENCIA	NO CUMPLE

REFUERZO A FLEXION	
Tipo de Barra a usar	N4
Área de la barra (cm^2)	1.29 cm^2
Separación (m)	0.157 m
Separación Seleccionada (m)	0.200 m
Numero de barras	1.593
Numero de barras seleccionada	2

Por lo tanto, para momento positivo se emplean dos barras No.4 en la cara inferior de la viga diafragma.

DISEÑO A FLEXIÓN NEGATIVA	
Resistencia del concreto f'c	280 kg/cm^2
Fluencia del acero de refuerzo Fy	4200 kg/cm^2
Momento de diseño Mu	223.94 kN-m/m
Momento de diseño Mu	22.85 ton-m/m

SECCIÓN	
$\phi_{FLUENCIA}$	0.9
Ancho b (cm)	25.0 cm
Altura h (cm)	141.0 cm
Recubrimiento (cm)	5.0 cm
Profundidad efectiva d (cm)	136.0 cm
Cuantía Calculada ρ_{CALC}	0.00132
Factor del bloque a comp. β1	0.85
Cuantía Balanceada ρ_b	0.02856
Cuantía Mínima $\rho_{Mínima}$	0.00333
Verificación Cuantía Máxima	MENOR QUE LA BALANCEADA
Verificación Cuantía Mínima	MENOR QUE LA MÍNIMA
Cuantía Seleccionada $\rho_{Seleccionada}$	0.00132

As a Utilizar (cm^2)	4.50 cm^2
As a Colocado (cm^2)	5.00 cm^2
Cuantía calculada	**0.0015**
Prof. Bl. de Compresión a (cm)	3.17 cm
Prof. Eje Neutro C (cm)	3.74 cm
Resistencia a la flexión refuerzo utilizado ϕMn (Ton-m)	25.40 ton-m
VERIFICACIÓN RESISTENCIA	CUMPLE

REFUERZO A FLEXION	
Tipo de Barra a usar	N5

Área de la barra (cm^2)	2.00 cm²
Separación (m)	0.111 m
Separación Seleccionada (m)	0.100 m
Numero de barras	2.249
Numero de barras seleccionada	3

Por lo tanto, para momento negativo se emplean tres barras No.5 en la cara superior de la viga diafragma.

Se incluye refuerzo de piel con barras No.4 cada 0.20 en las paredes laterales de la viga diafragma

Diseño a Cortante

La resistencia nominal a cortante se tomará como:

$$V_n = V_c + V_s$$

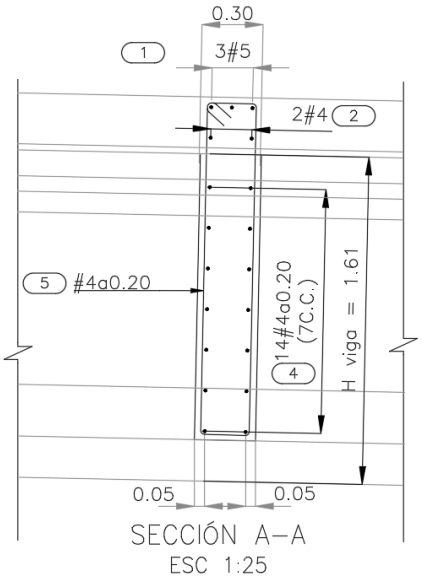
La cortante última es: $V_U = 142.97\ kN$) y la resistencia al corte del concreto en la garganta (H = 1.41 m y dv = 1.36 m) se calcula como:

$$V_C = \emptyset * 0.083 * \beta * \sqrt{f'c} * d_v * b_v$$

$$V_C = 0.083 * 2 * \sqrt{28\ MPa} * 250\ mm * 1360\ mm = 298652\ N = 298\ kN$$

$$Vu < \emptyset Vn$$
$$Vu < \emptyset\ (Vc + Vs)$$
$$Vu < 0.90(298 + 0)$$
$$Vu = 142.97\ kN < \emptyset Vn = 268\ kN \quad OK, CUMPLE$$

Por lo tanto, no se requiere el uso de estribos de cortante, el concreto es capaz de soportar todas las solicitaciones a fuerza cortante, sin embargo por temas de confinamiento, se empleará estribos No.4 cada 0.20, equivalente a dos ramas de acero transversal.



3.6. DISEÑO DE APOYOS ELASTOMÉRICOS

Se define la mayor carga que desciende por cada viga de concreto reforzado proveniente de la superestructura, tanto para las vigas postensadas de luz 21.0m (izquierda) y luz de 30.0m (derecha) respectivamente

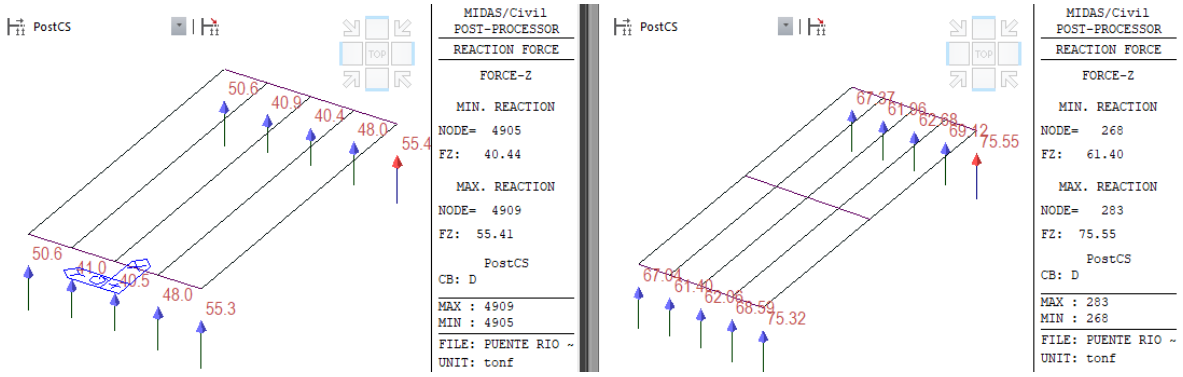


Figura 204. Reacciones debidas a cargas muertas en servicio

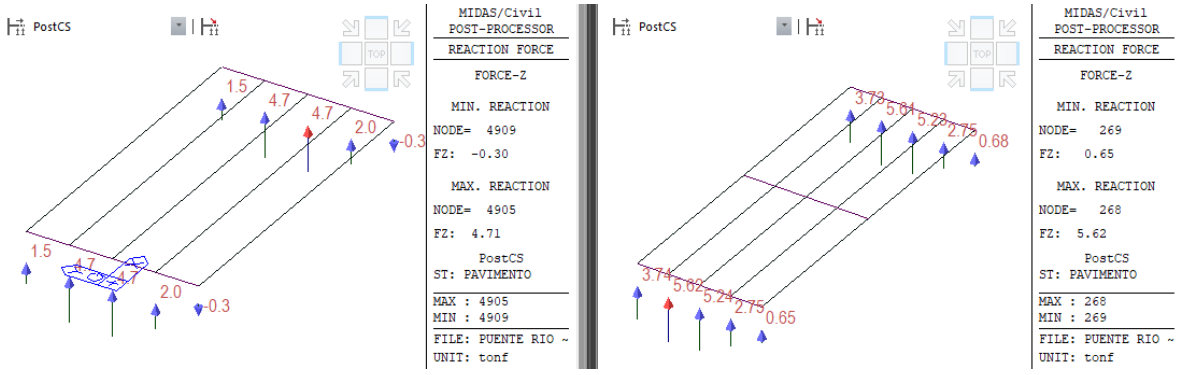


Figura 205. Reacciones debidas a cargas de pavimento en servicio

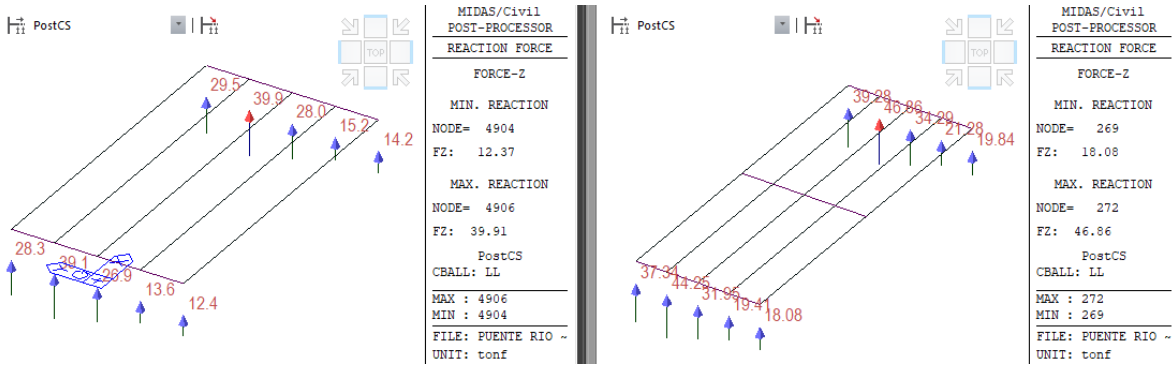


Figura 206. Reacciones debidas a cargas vivas en servicio

ton	APOYO INTERMEDIO 3 - VIGA 21.0m					Máxima Carga	total
CARGA MUERTA	50.60	41.00	40.50	48.00	55.30	55.30	235.40
PAVIMENTO	1.50	4.70	4.70	2.00	-0.30	4.70	12.60
CARGA VIVA	28.30	39.10	26.90	13.60	12.40	39.10	120.30

ton	APOYO INTERMEDIO 4 - VIGA 21.0m					Máxima Carga	total
CARGA MUERTA	50.60	40.90	40.40	48.00	55.40	55.40	235.30
PAVIMENTO	1.50	4.70	4.70	2.00	-0.30	4.70	12.60
CARGA VIVA	29.50	39.90	28.00	15.20	14.20	39.90	126.80

ton	APOYO INTERMEDIO 4 - VIGA 30.0m					Máxima Carga	total
CARGA MUERTA	67.04	61.40	62.06	68.59	75.32	75.32	334.41
PAVIMENTO	3.74	5.62	5.24	2.75	0.65	5.62	18.00
CARGA VIVA	37.34	44.25	31.95	19.41	18.08	44.25	151.03

ton	ESTRIBO 2 VIGA 30.0m					Máxima Carga	total
CARGA MUERTA	67.37	61.96	62.68	69.12	75.55	75.55	336.68
PAVIMENTO	3.73	5.61	5.28	2.75	0.68	5.61	18.05
CARGA VIVA	39.28	46.86	34.29	21.28	19.84	46.86	161.55

En resumen, se realiza el diseño estructural de los apoyos de neopreno para las mayores cargas

Vigas Postensadas 21.0m	Máxima Carga (ton)		Vigas Postensadas 30.0m	Máxima Carga (ton)
CARGA MUERTA	55.40		CARGA MUERTA	75.55
PAVIMENTO	4.70		PAVIMENTO	5.61
CARGA VIVA	39.90		CARGA VIVA	46.86

Se diseña para la condición más crítica, es decir para las solicitaciones provenientes de la luz de 30.0m, debido a que se tiene la misma sección transversal y peralte de la viga postensada y que se utiliza para las luces de 21.0m y 30.0m.

1. DATOS DE ENTRADA

carga muerta	DC	75.55	ton
pavimento	DW	5.61	ton
carga viva sin impacto	LL	46.86	ton
Luz del puente	L	30	m
carga total	PT	128.02	ton

Nota: sin impacto según 14.4.1

Estado límite de servicio 1
neopreno dureza 50 en escala Shore A
Metodo diseño tipo A (14.7.6)

max temp	22	°C	
mín temp	-10	°C	
temp instalación	23	°C	
ΔT	32	°C	
acortamiento por postensado	Δpost	10	mm
por retracción de fragua	Δfragua	9	mm

Coef. Dilatación concreto	α	1.08E-05	1/°C
defor. Unitaria por flujo plástico	εp	0.0003	
Acero A36 Fy	250	Mpa	

2. DEFORM. CORTANTE DEL APOYO NEOPRENO

La altura total de las capas de neopreno hrt exteriores e interiores, que conforman el apoyo debe cumplir con la siguiente condición

hrt	2*γTu*Δs		14.7.6.3.4-1
Δtemp	α*L*ΔT		
Δtemp	10.368	mm	
γTu	1.2		
Δs	29.37		
Δs	35.24	mm	

Espesor mínimmo requerido de las capas de elastomérico

De acuerdo con tabla 3.4.1-1 y para el estado límite de servicio se tiene

Sutituyendo valores en 14.7.6.3.4-1

hrt 70.4832 mm
hrt 71 mm

Tabla 14.7.6.2-1 — Propiedades Físicas Correlacionadas

	Dureza (Shore A)		
	50	60	70[1]
Módulo de Cortante a 23°C (MPa)	0.66-0.90	0.90-1.38	1.38-2.07
Deflexión de flujo plástico a 25 años dividida por la deflexión inicial	0.25	0.35	0.45

[1] Sólo para PEP, FGP, y apoyos elastoméricos reforzados con acero con un deslizador de PTFE o equivalente encima del apoyo.

módulo de elasticidad a cortante
dureza 60 G 0.9 Mpa
dureza 60 G 9.14 kg/m2

σs req PT/A 45.72 kg/cm2

3. ESFUERZOS Y RELACIONES GEOMÉTRICAS DEL APOYO

de acuerdo con 14.7.6.3.2, los esfuerzos sobre el apoyo de neopreno debe cumplir lo siguiente

$\sigma_s \leq 1.25GS_i$ y (14.7.6.3.2-7)

$\sigma_s \leq 8.6MPa$ (14.7.6.3.2-8)

σs 8.6 Mpa
σs 87.72 kg/cm2

determinación del ancho W (normal al eje del puente) del apoyo de neopreno

Areq PT/σs
Areq 1459.416 cm2

W selec. 70 cm
W es la superficie de contacto con neopreno

determinación de la longitud L (a lo largo del eje del puente) del apoyo de neopreno

$$L = \frac{DC + DW + LL}{\sigma s * W}$$

L	20.8488	cm
L	21	cm

se toma longitud de apoyo de neopreno tal que se tenga en cuenta el cumplimiento de requisitos adicionales dimensionales

L selec	40	cm
Area	2800	cm2

OK CUMPLE

determinación del factor de forma

$$S_i \geq \frac{\sigma_s}{1.25\,G} = \text{min} = \quad 4.00$$

Grosor de una capa interna del elastómero

$$S_i \geq \frac{LW}{2h_n(L+W)} \rightarrow h_{ri} \leq \frac{LW}{2S_i(L+W)} \qquad (14.7.5.1\text{-}1)$$

hri	3.18	cm
hri	31.79	mm
hri selec	20	mm

hri selec debe ser menor que hri

Si	6.36

OK CUMPLE

	σs req		σs máx	
si	4.57 Mpa	<	8.6 Mpa	**OK CUMPLE**

grosor espesor capas externas

$h_{re} \leq 0.7\ h_{ri}$ (Art. 14.7.6.1)

	hre	14	mm
	hre selec	14	mm

factor de forma externo

Se	9.09	

numero de capas interiores

Siendo: $h_{rt} = nh_{ri} + 2h_{re}$

n	2.15	
n selec	3	

Se verifica además:

$$\frac{S_i^2}{n} < 20, \text{ para dispositivos rectangulares con } n \geq 3 \qquad (C14.7.6.1)$$

$\frac{S_i^2}{n}$	=	10.12	<	20	OK CUMPLE

Espesor total elastómero

$h_{rt} = nh_{ri} + 2h_{re}$	hrt	88	mm

Espesor placas de refuerzo

- En el estado límite de servicio:

$$h_s \geq \frac{3h_{ri}\sigma_s}{F_y} \qquad (14.7.5.3.5\text{-}1)$$

$\frac{3h_{ri}\sigma_s}{F_y}$	=	1.10	mm	<	20 mm

OK cumple

- En el estado límite de fatiga:

$$h_s \geq \frac{2h_{ri}\sigma_L}{\Delta F_{TH}} \qquad (14.7.5.3.5\text{-}2)$$

Umbral de fatiga 6.6.1.2.5-3 ΔFTH 165 Mpa

$\frac{2h_{ri}\sigma_L}{\Delta F_{TH}}$ = 0.41 mm < 20 mm

σL PLL/A 16.73571 kg/cm2

OK cumple

hs 1.10 mm
hs selex 3 mm

altura total elastómero

$H = h_{rt} + (n+1)h_s$ H 100 mm

verificaciones por estabilidad

el espesor total es H= 100mm y no debe exceder el menor valor entre

L/3 y W/3

L/3= 133.33 mm > 100 mm

OK CUMPLE

W/3= 233.33 mm > 100 mm

OK CUMPLE

Verificación de la condición requerida 14.7.6.1

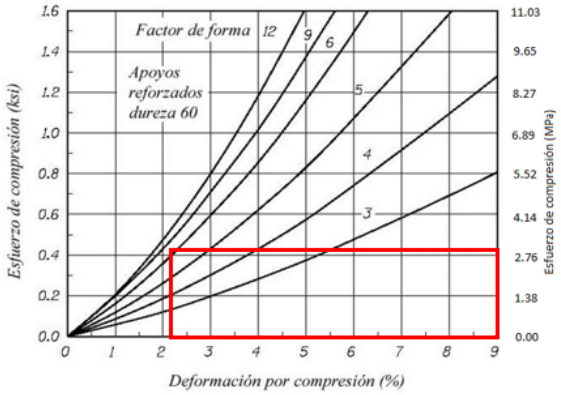
- Apoyos elastoméricos reforzados con acero en los cuales $S_i^2/n < 22$, y para los cuales la rotación principal es alrededor del eje paralelo al eje transversal del puente; y

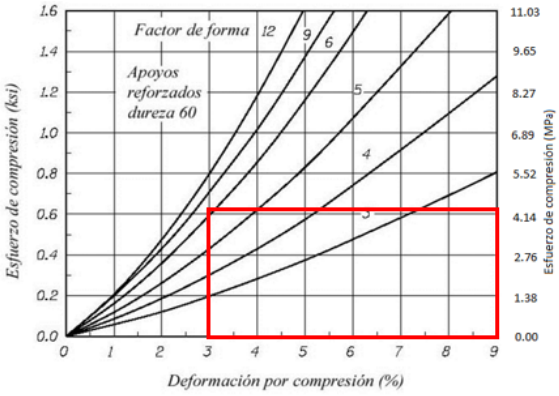
Si*Si/n	13.50	<	22

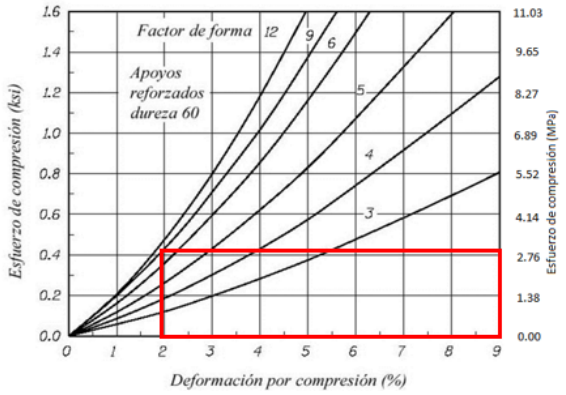
OK CUMPLE METODO A

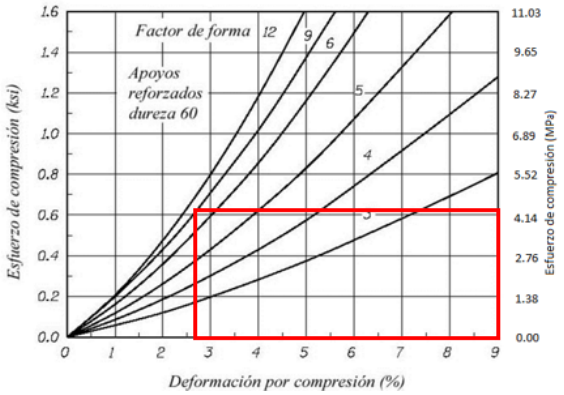
4. DEFORMACIONES DEL APOYO DE NEOPRENO

CAPA	CARGA	S	σ (Mpa)	ε unitaria %
INTERIOR	MUERTA	6.36	2.84	2.2%
	TOTAL	6.36	4.57	3.0%
EXTERIOR	MUERTA	9.09	2.84	2.0%
	TOTAL	9.09	4.57	2.6%









deflexión por compresión inicial del dispositivo

Deflexión por compresión inicial del dispositivo (14.7.5.3.6-1 y 14.7.5.3.6-2)

$$\delta = nh_{ri}\varepsilon_{int} + 2h_{re}\varepsilon_{ext}$$

δ= 2.528 mm

Esfuerzo promedio sobre el apoyo de neopreno debido a la carga permanente de servicio

$$\sigma = \frac{DC + DW}{W * L}$$

σ 289.86 ton/m2
σ 2.84 Mpa

Deflexión por compresión inicial debido a la carga muerta (14.7.5.3.6-2)

$$\delta_{DC} = nh_{ri}\varepsilon_{D\,int} + 2h_{re}\varepsilon_{D\,ext}$$

δdc= 1.880 mm

Deflexión por compresión debido a la carga viva

$$\delta_{LL} = \delta - \beta_{DC}$$

δLL= 0.648 mm

La deflexión de largo plazo de carga muerta, incluyendo los efectos de flujo plástico, debe tomarse así:

$$\delta_{lt} = \delta_d + \alpha_{cr}\delta_d \qquad (14.7.5.3.6\text{-}3)$$

deflexión por flujo plástico αcr= 0.35

δcreep= 0.658 mm

δcreep+LL= 1.306 mm

De acuerdo con 14.7.6.3.3, la deformación por compresión debe ser menor que 0.09*hri

Deflexión por compresión inicial de una capa interior de elastómero

$$\delta_i = \varepsilon_{int} h_{ri} < 0.09 h_{ri} \qquad (14.7.6.3.3)$$

$$\varepsilon_{int} < 0.09$$

0.09*hri=	1.8	mm
hri*ε unitaria int=	0.60	
0.60	<	1.8

OK CUMPLE

5. FUERZA CORTANTE GENERADA EN EL APOYO DEBIDA A LOS MOVIMIENTOS POR TEMPERATURA

La fuerza debida a la deformación de un elemento elastomérico debe tomarse así:

$$H_{bu} = GA\frac{\Delta_u}{h_{rt}} \qquad (14.6.3.1\text{-}2)$$

donde:

G = módulo de cortante del elastómero (MPa)
A = área en planta del elemento elastomérico o del apoyo (mm^2)
Δ_u = deformación de cortante, obtenida de las combinaciones de carga de resistencia aplicables y evento extremo de la Tabla 3.4.1-1 (mm)
h_{rt} = espesor total del elastómero (mm)

14.7.5.2 — *Propiedades Físicas* — El módulo de cortante del elastómero a 23°C debe usarse como base para el diseño.

El elastómero debe tener un módulo de cortante especificado entre 0.55 MPa y 1.21| MPa.

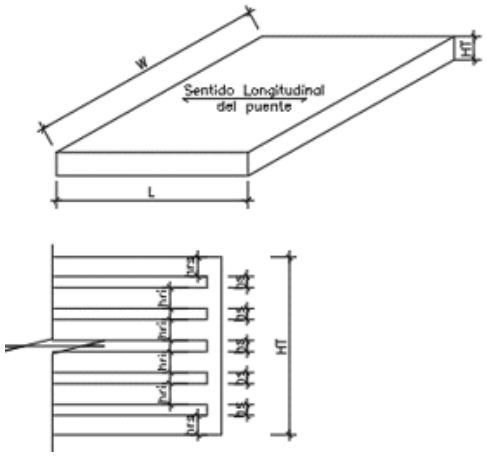
G	0.9	Mpa
G	9.144	kg/cm2
Δu	1.2*Δs	
Δu	35.2416	mm
Hu	G*W*L*Δu/hrt	
Hu	0.101	MN
Hu	10.440	ton
con u=	0.200	
Ff=	u*Pdc	
Ff	15.1	ton
como Hu	< Ft	

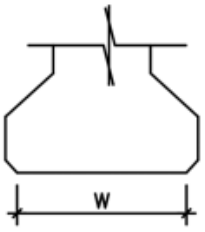
OK CUMPLE

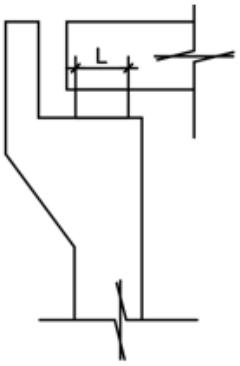
NO REQUIERE ANCLAJES

Resumen

W=	700	mm		
L=	400	mm		
HT	100	mm		
capas externas - hrs	2	und	14 mm	cada una
capas internas - hri	3	und	20 mm	cada una
láminas de acero - hs	4	und	3 mm	cada una







CALCULO DE RIGIDEZ NEOPRENO

se describe la geometría del apoyo elastómero a considerar

Base Elastómero W=	700	mm
Largo Elastómero L=	400	mm
Altura total Elastómero HT=	100	mm

La rigidez K de un elemento lineal elástico, se puede definir como la relación entre la fuerza P aplicada en el elemento y al deformación δ resultante:

$$P = K * \delta$$

Despejando K se tiene

$$K = P/\delta$$

Así mismo, teniendo en cuenta la resistencia de materiales de los elementos, el desplazamiento está definido como

$$\delta = \frac{P*L}{A*L}$$

despejando la variable δ, se tiene lo siguiente.

$$K = \frac{A*E}{L}$$

A es el área en planta del elastómero
L es la longitud del elemento, tomado como el espesor HT del neopreno
E es el módulo de elasticidad del material, se tomará como módulo efectivo a compresión del apoyo Ec, acorde con el C.14.6.3.2-1, el cual está definido

Ec=4.8 * G * S * S (Mpa)

A=	280000	mm2	0.28	m2
L=	100	mm	0.1	m
			0.07	
G=	0.9	Mpa	91.776	ton/m2
S=	6.36			

Ec= 174.94 Mpa
17839.36 Ton/m2

La rigidez vertical del elemento está definida como

K= 71357.4 ton/m

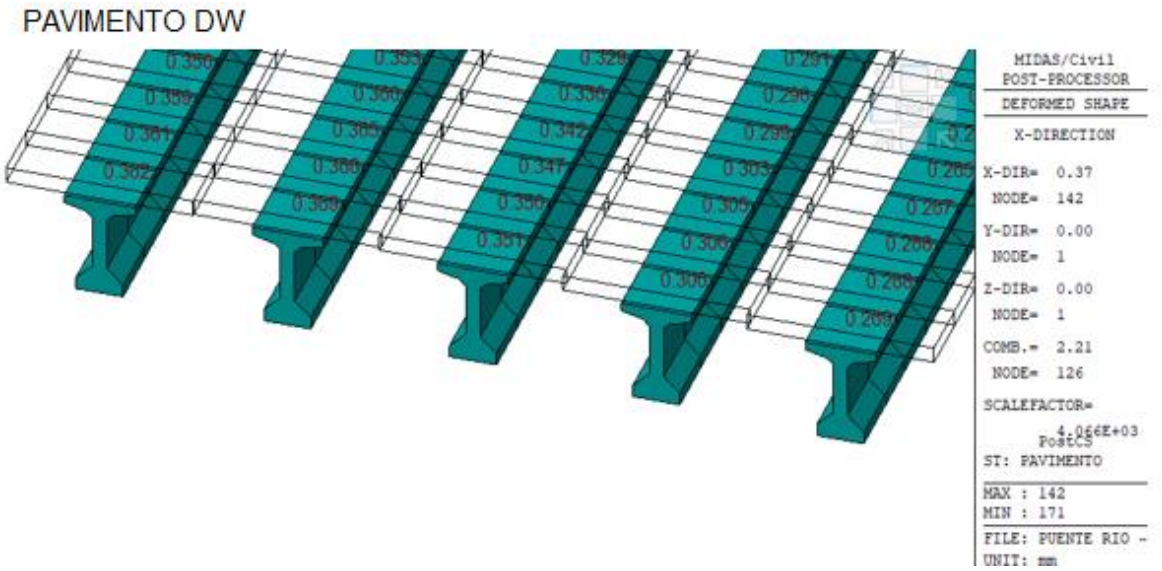
La rigidez horizontal, acorde con el numeral 14.6.3.1-2, se define como

$$Kel = \frac{G * A}{hrt}$$

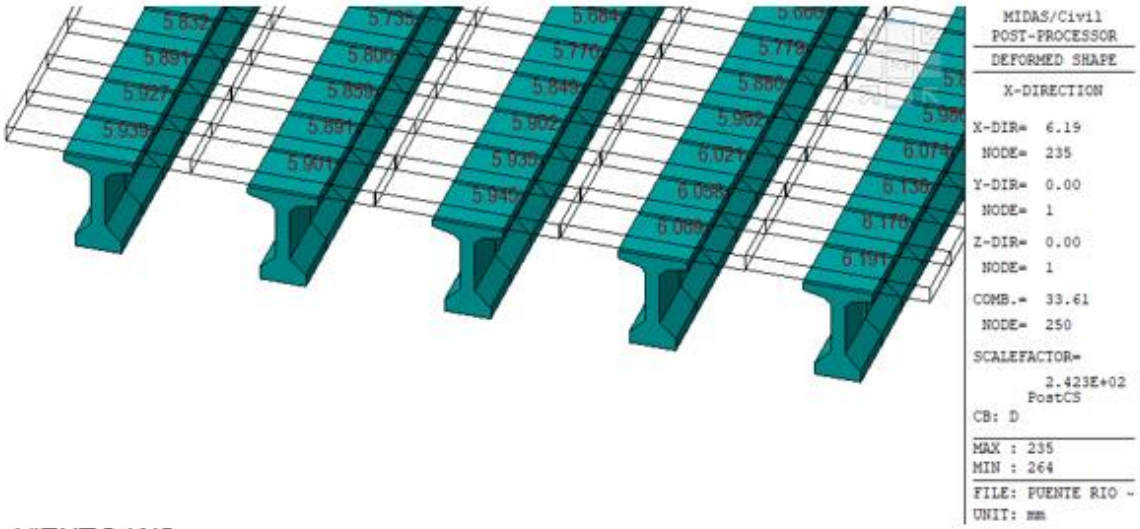
Kel= 257.0 ton/m

3.7. DISEÑO DE JUNTAS DE EXPANSIÓN

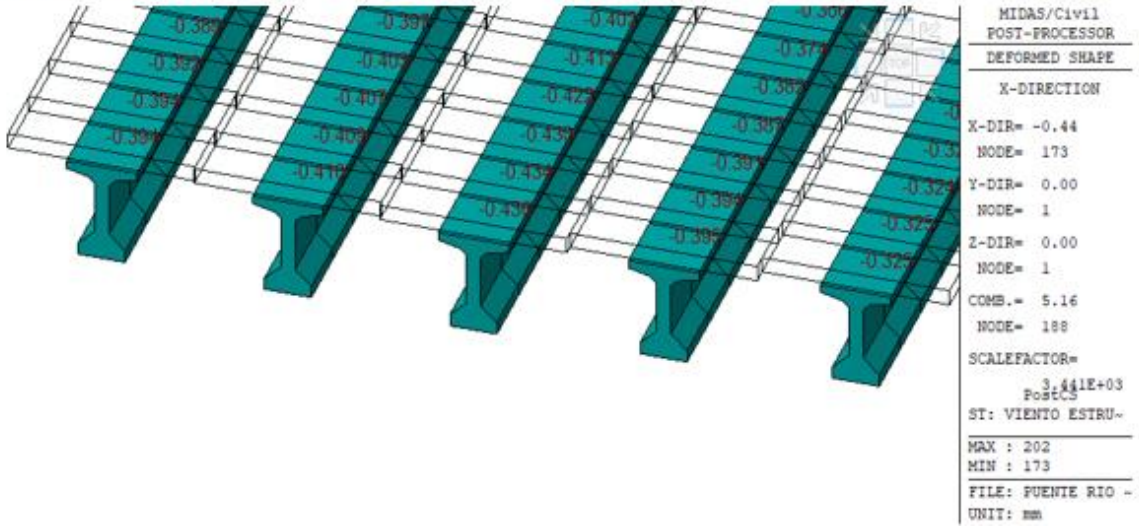
Se diseña para la condición más crítica, es decir para las solicitaciones provenientes de la luz de 30.0m, debido a que se tiene la misma sección transversal y peralte de la viga postensada y que se utiliza para las luces de 21.0m y 30.0m.



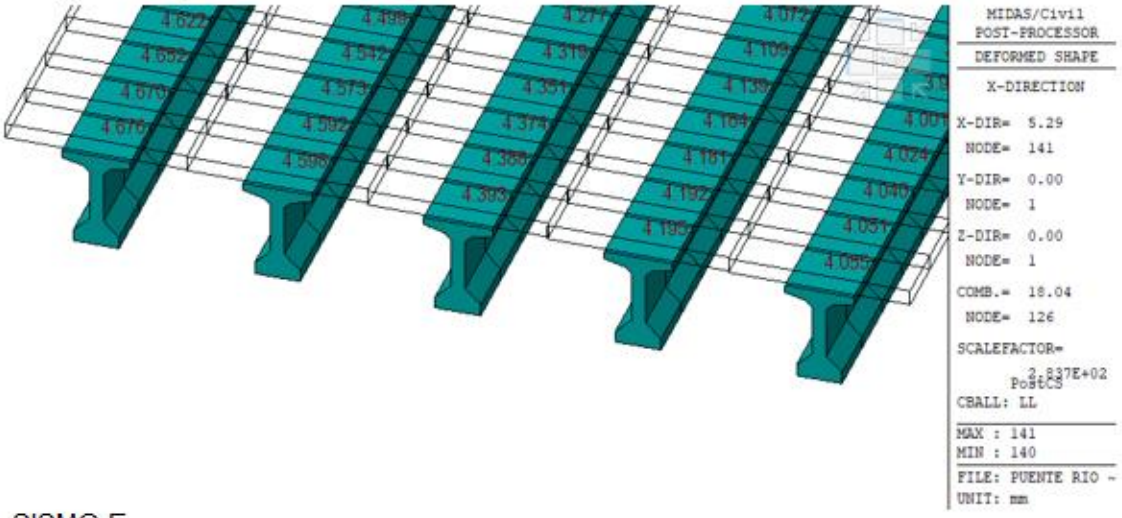
CARGA MUERTA DC



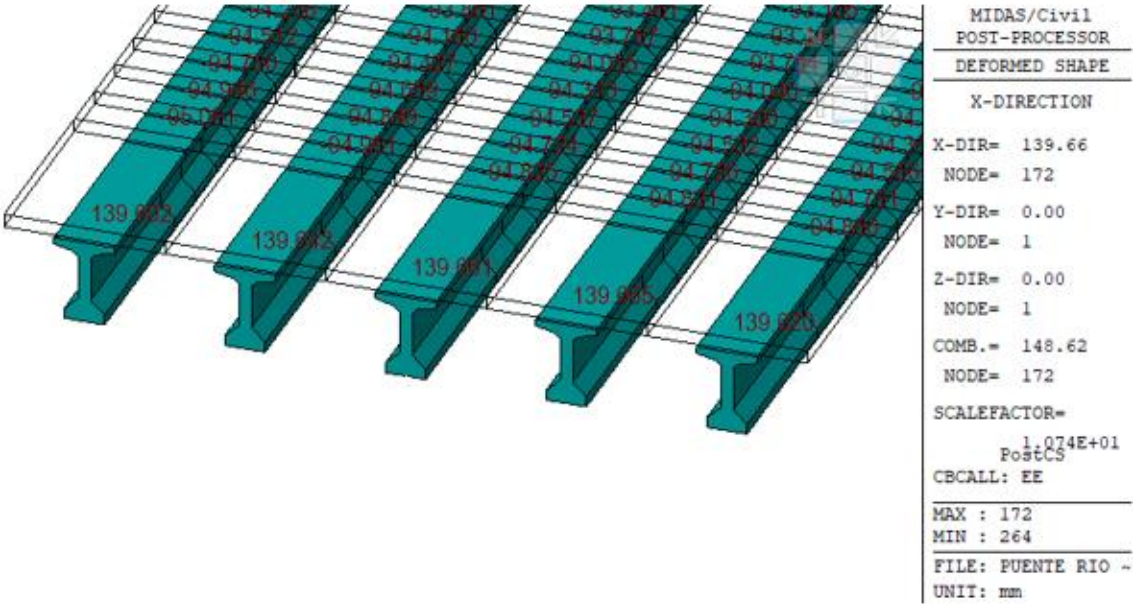
VIENTO WS



Carga viva + peatonal



SISMO E



JUNTAS DE EXPANSIÓN TÉRMICA

Las juntas de expansión serán diseñadas para acomodar los desplazamientos que se derivan de condiciones especificas

1. Máximo desplazamiento debido a combinaciones de estado límite de servicio que se obtienen del modelo estructural, más los desplazamientos reológicos del concreto (temperatura, contracción y flujo plástico), más un porcentaje del desplazamiento máximo producto del sismo de diseño.
2. Máximo desplazamiento debido a las combinaciones de estado límite de resistencia que se obtiene del modelo estructural, más el desplazamiento por el acortamiento del concreto, más 1.2 veces el desplazamiento debido al cambio uniforme de temperatura.

Δ DW=	0.37	mm
Δ DC=	6.19	mm
Δ WS=	0.44	mm
Δ LL=	5.29	mm
Δ E=	139.6	mm
% sismo	30%	
Δ sismo diseño	41.88	mm

Según el numeral 5.4.2.3.2 de la CCP-14, el coeficiente del flujo plástico puede calcularse como:

$$\psi = 1.9 K_s K_{hc} K_f K_{td} t_i^{-0.118}$$

Edad al inicio de carga, t_i	28	dias
Tiempo final de cálculo, t_i	4000	dias
Resistencia del concreto f'c	35	MPa
% Resistencia al momento de carga f'ci	80%	
Resistencia al momento de carga f'ci	28	MPa
Humedad relativa, H	75%	
F. relación de volumen, K_s	1.00	(para vigas presforzadas)
F. humedad, K_{hc}	0.96	
F. resistencia concreto, K_f	0.83	
F. desarrollo de tiempo, K_{td}	0.99	
Coeficiente flujo Plástico, ψ	0.565	

El movimiento provocado por el flujo plástico, se calcula multiplicando los desplazamientos elásticos debidos a las cargas muertas por el coeficiente de flujo plástico

Δ Creep = 3.706 mm

Para concreto no restringido, la deformación por contracción máxima despues de la instalación, β, puede suponerse igual a 0.0002 (WSDOT,2011). El acortamiento del tablero, Δ Shrink, se calcula como:

$$\Delta_{SHRINK} = \beta(\mu)L_{trib}$$

Deformación unitaria máxima, β=	0.0002	
F. efecto de restricción, μ=	0.5	(para vigas de concreto presforzadas)
Longitud tributaria Ltrib=	30	m
Δ Shrink =	3	mm

El desplazamiento debido al cambio de temperatura uniforme se calcula:

$$\Delta_{temp} = \alpha(L_{trib})\delta T$$

Coef. De expansión térmica, α=	9.00E-06	
Variación de temperatura, δT=	10	°C
Δ temp =	2.70	mm

El desplazamiento total se determina:

Caso 1 =	63.27 mm
Caso 2 =	27.50 mm

gobierna el mayor desplazamiento
63.27 mm

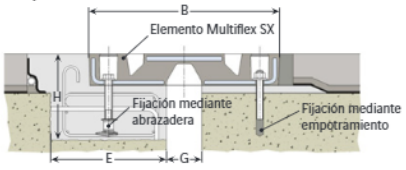
Se presentan catálogos comerciales de juntas de dilatación elastomérica, queda a criterio del constructor definir la marca y especificación técnica de cada una de ellas; se recomienda que el GAP mínimo de desplazamiento longitudinal no sea menor de 100mm±50mm.

Según el cálculo, se registra un desplazamiento máximo de 63.27 mm → CUMPLE OK

- Catalogo FREYSSINET

Ficha técnica

Existen dos diseños diferentes: la junta de módulo simple y la junta puenteada. Estos diseños se derivan del movimiento admisible de la junta.



Elemento de módulo simple

Tipo	Módulo	B mín.	B máx.	G mín.	G máx.	E mín.	H mín.
SX80	simple	235	315	10	90	195	190
SX100	simple	305	405	10	110	225	190
SX120	simple	330	450	10	130	235	210
SX160	simple	390	550	10	170	265	230
SX180	simple	410	590	10	190	275	230

Dimensiones en mm

- Catalogo AGOM

AGOM
IDEAS, ENGINEERING AND MANUFACTURE

Technical Specifications: AGFLEXJ-series expansion joints

Model	Movement [mm]	Dimensions AxHxL [mm]	P [mm]	Weight [kg/m]
AGFLEXJ 30	± 15	269 X 32 X 2000	200	17
AGFLEXJ 50	± 25	272 X 41 X 2000	200	25
AGFLEXJ 80	± 40	358 X 46 X 2000	250	32
AGFLEXJ 100	± 50	388 X 53 X 2000	250	37
AGFLEXJ 120	± 60	427 X 69 X 1000	250	59
AGFLEXJ 140	± 70	465 X 80 X 2000	250	81
AGFLEXJ 160	± 80	498 X 84 X 2000	250	88
AGFLEXJ 200	± 100	802 X 71 X 2000	250	135
AGFLEXJ 250	± 125	882 X 78 X 2000	250	170
AGFLEXJ 330	± 165	1108 X 100 X 1000	250	285

- Catalogo E&M

E&M
AISLADORES APOYOS Y JUNTAS DE DILATACIÓN

JUNTAS DE DILATACIÓN ELASTOMÉRICAS

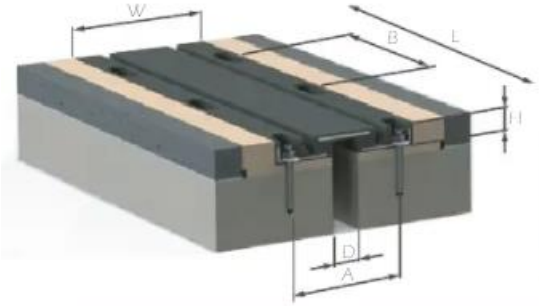
MODELO DE JUNTA	LONGITUD (mm)	ANCHURA DE TOPE (mm)	PROFUNDIDAD (mm)	DESP. LONGITUDINAL (mm)	DESP. TRANSVERSAL (mm)	APLICACIONES RECOMENDADAS
E&M 50 	2000±10	264±10	39±5	50(±25)	50(±25)	Losas de puentes, losas de parqueo, rampas, estribos, estructuras elevadas, sobrepasos, presas, estadios y juntas de alivio de presión.
E&M 64 	2000±10	343±10	45±5	64(±32)	64(±32)	
E&M 80 	2000±10	357±10	46±5	80(±40)	80(±40)	
E&M 100 	2000±10	390±10	53±5	100(±50)	100(±50)	Losas de puentes, estribos y estructuras elevadas.
E&M 140 	2000±10	440±5	54±5	140(±70)	140(±70)	
E&M 160 	2000±10	500±10	80±5	160(±80)	160(±80)	
E&M 180 	2000±10	480±5	66±5	180(±90)	180(±90)	
E&M 200 	2000±10	480±5	66±5	200(±100)	200(±100)	

- Catalogo HERFLEX SISTEMAS

Herflex Sistemas

JUNTAS DE DILATACIÓN PARA PUENTES

El sistema Herflex HF permite absorber grandes movimientos debido a su elemento elástico, de deformación multidireccional. **Se asegura un sellado eficaz, bajo mantenimiento y fácil reposición.**
Los modelos Herflex HF 50-180 cubren un rango de movimiento que va desde 50mm a 180mm y los modelos Herflex HF 195-380 desde 195mm a 380mm.



HF 50 - 180

APLICACIONES PRINCIPALES

Puentes con recorridos inferiores a 180mm
Estructuras de tamaño medío
Parkings
Viaductos y puentes

MODELO		MÓDULO							PERNO
	Recorrido (mm)	L (mm)	W (mm)	H (mm)	A (mm)	B (mm)	D* (mm)	Peso (kg)	M x c (mm)
HF-50	50 (±25)	1830	240	35	190	334	40	26	M14 x 170
HF-80	80 (±40)	1830	274	40	220	305	55	35	M14 x 170
HF-110	110 (±55)	1830	356	46	279	305	70	55	M16 x 170
HF-140	140 (±70)	1830	432	54	342	305	85	75	M18 x 196
HF-180	180 (±90)	1830	480	66	390	305	105	109	M20 x 196

* Máxima abertura estructural en el momento de la instalación a temperatura media.

3.8. DISEÑO DE BLOQUES SÍSMICOS

Se define la carga muerta que desciende de la superestructura, esta deberá ser soportada por los topes sísmicos ante algún movimiento telúrico. La masa gravitacional por la aceleración horizontal del espectro de aceleraciones, genera la fuerza horizontal que deberá soportar el bloque de cortante o tope sísmico.

Se diseña para la condición más crítica, es decir para la luz de 30.0m.

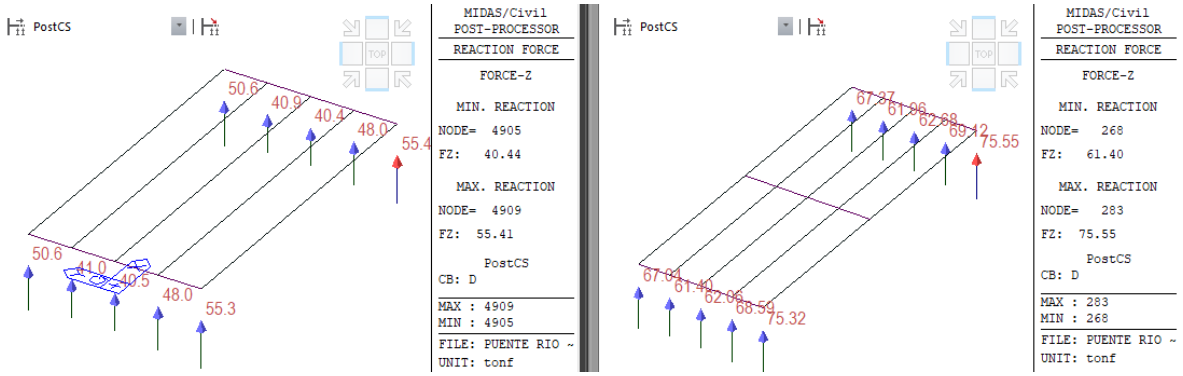


Figura 207. Reacciones debidas a cargas muertas en servicio

BLOQUES DE CORTANTE		
Fluencia del acero de refuerzo Fy	4200	kg/cm2
R: coef. de modificación de respuesta =	0.80	
P_{DE}: Carga muerta sobre el estribo (ton)	76.0 ton	
N_{blq} : numero de bloques de cortante =	1	
Coef. para presión lateral de tierra b_E =	1.30	
Máxima Aceleración Horizontal As	0.225	
Cs = As / R =	0.281	
$V_u = 1{,}0*b_E*P_{DE}*Cs/N_{blq}$ =	27.8 ton	(Por bloque de cortante)
Cortante por fricción : Av		
Coeficiente de fricción para concreto vaciado contra concreto endurecido con su superficie intencionalmente rugosa		
μ:	1.00	
f: coef. de reducción de resistencia =	0.90	
$A_v = Vu / (\phi*fy*\mu)$	7.35 cm²	
Tipo de Barra a usar	N6	
Área de la barra (cm²)	2.84 cm²	
Ramas N6	1.29	U
Se usaran	2.00	Ramas N6 por cada bloque

Por lo tanto, se requieren mínimo 2 ramas No.6 por cada bloque sísmico.

3.9. DISEÑO DE LOSA DE ACCESO

Parámetros de Diseño	
Peso específico Suelo Ys	2.0 t/m³
Peso específico Concreto Yc	2.4 t/m³
Ángulo de Fricción	33°
Fy	420 Mpa
f'c	28 Mpa

Ancho del estribo B_E =	15.00	m
Ancho losa de aproximación B_{losa} =	15.00	m
Longitud losa de aproximación L_{losa} =	3.00	m
Espesor losa de aproximación E_{losa} =	0.25	m
Espesor altura de pavimento $E_{pavimento}$ =	0.075	m
Altura lleno sobre losa h_{lleno} =	1.45	m

AVALUO DE CARGAS

1. Carga Muerta

$$P_D = E_{losa} * \gamma_{conc} * B_{losa} * L_{losa}/2$$

P_D = 13.50 t

2. Carga Viva

Numero de carriles n_c =	2	U	
Longitud entre ejes de mas carga L_{ejes} =	4.30	m	CCP14

	Carga t	Li m
Eje trasero P_1 =	16.00	3.00
Eje trasero P_2 =	0	0
Eje delantero P_3 =	0	0

$$P_L = ((P_1*L_1+P_2*L_2+P_3*L_3)/L_{losa})*n_c$$

$P_L =$ 32.00 t

Impacto I = 33% $P_{L+I} =$ 42.56 t

3. Lleno

$$P_{lleno} = h_{lleno}*\gamma_{suelo}*B_{losa}*L_{losa}/2$$

$P_{lleno} =$ 65.25 t

4. Pavimento

$$P_{pavimento} = h_{pavimento}*\gamma_{concreto}*B_{losa}*L_{los}$$

$P_{pavimento}$ 4.05 t

DIMENSIONAMIENTO

Cortante actuante

$$V_u = 1{,}25*P_{D\ total}+1.30*Plleno+1.50Ppavimento + 1.75*P_{L+I}$$

$V_u =$ 182.26 t

Se tiene además que : $V_u = \phi*0{,}85*f'_c*A_{mensula}$

$\phi =$ 0.85

$A_{mensula} =$ 900.91 cm^2

$A_{mensula} = b_{mensula} * L_{mensula}$; siendo $b_{mensula} =$ 1,500.00 cm

Longitud de apoyo o longitud de mensula $L_{mensula} =$ 0.60 cm

Se toma $L_{mensula} =$ 30.00 cm

ALTURA EN LA CARA DEL APOYO

Se tiene que : $V_u = \phi * V_n = \phi * 0{,}2 * f'_c * A_{c\,min}$

$\phi =$ 0.85

$A_{c\,min} =$ 3,828.89 cm^2

$A_{c\,min} = b_{mensula} * d_{c\,min}$; siendo $b_{mensula} =$ 1,500.00 cm

$d_{c\,min} =$ 2.55 cm

Se toma $d_{c\,min} =$ 30.00 cm

Altura constante de la mensula $h_1 =$ 30.00 cm

Altura constante de la mensula $h_2 =$ - cm

Altura total de la mensula $h_{t\,mensula} =$ 30.00 cm

DETERMINACION DEL REFUERZO

1. A_{vf} para cortante por fricción

Se tiene que : $V_u \leq \phi * V_n = \phi * \mu * f_y * A_{vf}$

Factor de fricción para construcción monolitica $\mu =$ 1.00

$\phi =$ 0.85

$A_{vf} =$ 51.05 cm^2

2. A_n para tracción directa

Se tiene que : $N_{uc} \leq \phi * A_n * f_y$ y $N_{uc} = 0.20 * V_{u\,actuante}$

$\phi =$ 0.85

$A_n =$ 10.21 cm^2

3. A_f para flexión

Se tiene que : $M_u = V_u * a + N_{uc} * (h-d)$

$a =$ 15.00 cm

$M_u =$ 2,733,825 kg-cm

Se tiene que : $M_u \leq \phi * f_y * ju * d * A_f$

$ju =$ 0.90
$\phi =$ 0.85
$A_f =$ 28.36 cm^2

4. Calculo de refuerzo principal

i. $A_f + A_n =$ 38.57 cm^2

ii. $(2/3)*A_{vf} + A_n =$ 44.24 cm^2

iii. $\rho_{min} = 1.4 / f_y =$ 0.00333

$A_{s\ min} =$ 150.00 cm^2

No.4@ 12.70 cm o No.5@ 19.90 cm

Se usara No.5@ 10.00 cm

P = 3840/(S)^0.5) < 67% P= 70.11%
escoger P= 67.00%
As min por metro lineal 10.00 cm^2/m
As distribución As min * P = 6.70 cm^2/m

No.4@ 0.19 cm o No.5@ 0.30 cm

Se usara No.5@ 0.1 cm

6. Calculo del refuerzo de retracción y temperatura

As retracción As min 2.196 cm^2/m

No.4@ 0.58 cm o No.5@ 0.91 cm

Se usara No.4 cada 0.30 cm

Pot lo tanto, se requieren barras No.5 a flexión positiva y barras No.4 a 0.30 para retracción y temperatura.

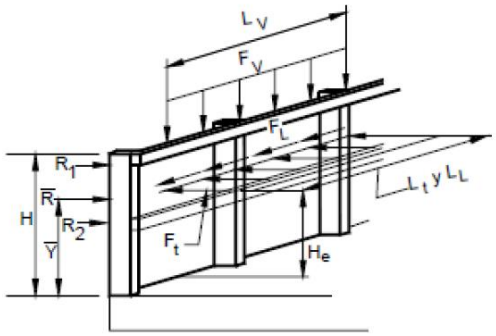
3.10. DISEÑO BARRERA DE PROTECCIÓN METÁLICA PEATONAL

Para el diseño estructural de las barandas metálicas vehiculares, Según la CCP14, en el capítulo 13, correspondientes a barandas, se especifica lo siguiente

Tabla A13.2-1 — Fuerzas de diseño para barandas vehiculares

Fuerzas de diseño y simbología	Niveles de Ensayo para las Barandas					
	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
Transversal F_t (N)	60.000	120.000	240.000	240.000	550.000	780.000
Longitudinal F_L (N)	20.000	40.000	80.000	80.000	183.000	260.000
Vertical descendente F_v (N)	20.000	20.000	20.000	80.000	355.000	355.000
L_t y L_L (mm)	1220	1220	1220	1070	2440	2440
L_v (mm)	5500	5500	5500	5500	12.200	12.200
H_e (mín.) (mm)	460	510	610	810	1070	1420
Mínima altura del riel H (mm)	685	685	685	810	1070	2290

Para el diseño de estas estructuras, según estudios, se contempla el uso de ensayo tipo 4 TL-4, que corresponde al uso para la mayoría de las aplicaciones en carreteras de alta velocidad, autovías, autopistas y carreteras interestatales en las cuales el tráfico incluye camiones y vehículos pesados.



La carga de colisión se aplica como carga distribuida en un ancho Lv, ya que es la longitud efectiva donde se desarrolla las mayores tensiones debido a la colisión frontal de un vehículo

Fuerza de diseño P = 240kN/5.50m =43.63 kN/m

DISEÑO BARRERA DE PROTECCIÓN METÁLICA

MATERIAL

Poste

Acero		A-36	
fy		250	MPa

Tubos longitudinales

Acero ASTM - A500 Grado C			
fy		345	MPa

DISEÑO TUBO LONGITUDINAL

DIMENSIONES

Separacion poste	Sp	1.50	m	
Fuerza diseño	P	43.63	kN/m	
Momento $M = \frac{P * Sp}{6}$	M	10.91	kN-m	
Modulo de sección	S	52.69	cm³	

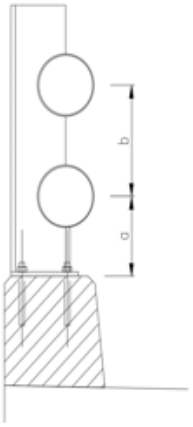
$$S = \frac{M}{0.6fy}$$

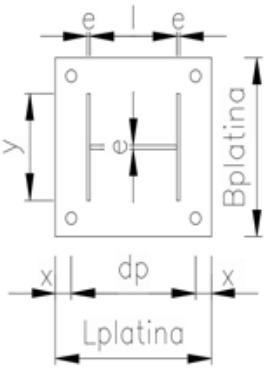
Tubo seleccionado

Diametro nominal		4	in	
Diametro exterior	d	11.35	cm	
Diametro Interior	dint	10.08	cm	
Espesor pared	e	6.35	mm	
Modulo de sección	S	54.25	cm³	OK!!!

DISEÑO POSTE

Distancia a		25	cm
Distancia b		25	cm
Momento poste	Mpost	1636.1	kN-cm
Modulo de sección	S	109.08	cm³
Diametro pernos		1	in
Distancia de borde al centro de	x	25	mm
Longitud platina	y	20	cm
Espesor platina	e	0.95	cm
Distancia l platina	l	18.1	cm
Ancho platina	Bplat	300	mm
Longitud platina	Lplat	300	mm
Inercia del poste	I	4E+03	cm4
Modulo de sección calculado	S	427.23	cm³





$$I = \sum I0 + \sum Adi^2$$

OK!!!

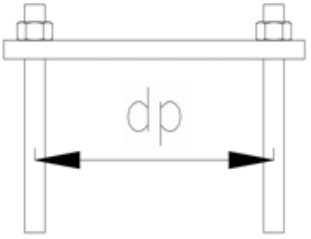
DISEÑO PERNOS DE ANCLAJE

Pernos Acero G-60	fy	420	N/mm2
Distancia entre pernos	dp	25.0	cm
Tension del perno	T	65.45	kN
Área perno requerida a tension	Asp	130	mm²
Área acero suministrado	Asp	510	mm²
Previendo fabricacion de rosca utilizar barilla		1 1/8	in

$$T = \frac{M}{dp}$$

$$Asp = \frac{T}{0.60fy}$$

OK!!!



3.11. DISEÑO BARRERA DE PROTECCIÓN VEHICULAR

DISEÑO DE LAS BARRERAS NEW JERSEY

El diseño de las barandas o barreras de seguridad se realiza acorde con lo especificado en los artículos 13.7; 13.8 y 13.10 de la Norma CCP-14, el diseño se realiza para un (1) metro lineal de barrera.

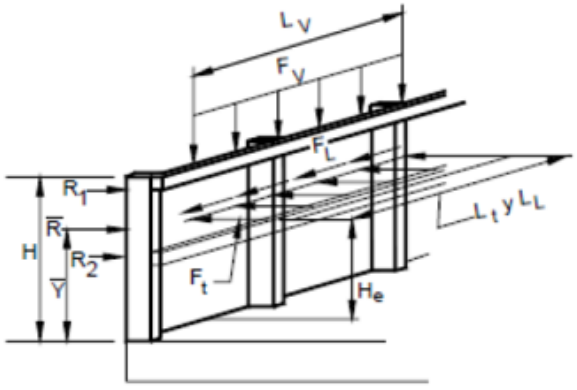
MATERIALES

$f'c$ = 28 MPa
Fy = 420 MPa

CARGAS DE DISEÑO

Para definir la fuerza de diseño se deberá determinar en primera instancia el Nivel de ensayo correspondiente a la baranda a diseñar.

Las fuerzas de diseño para la baranda vehícular son las presentadas en el apéndice A13.2 de la Norma, la cual presenta la siguiente configuración de cargas.



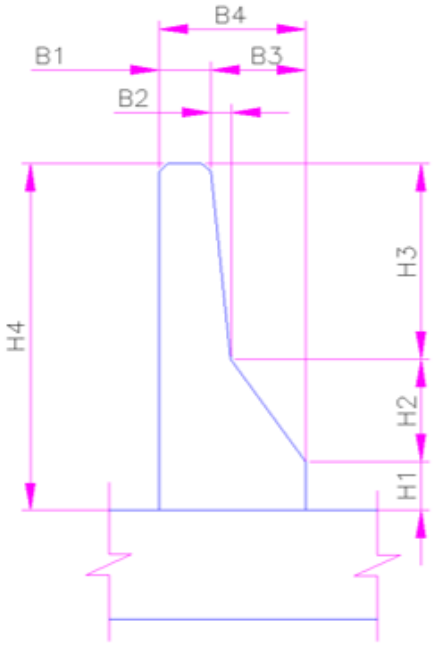
Zonas de alta velocidad, autovías, autopistas y carreteras interestatales en las cuales el tráfico incluye camiones y vehículos pesados.

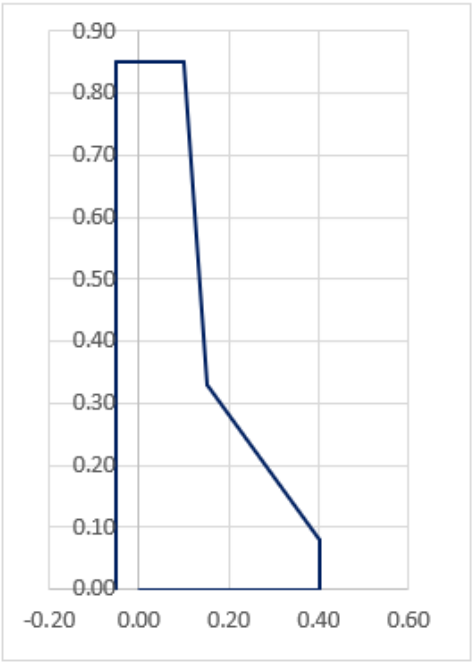
Nivel de ensayo =	TL-4
Fuerza Transversal, Ft =	24.0 t
Fuerza Longitudinal, FL =	8.0 t
Fuerza Vertical, Fv =	8.0 t
Longitud Transversal Lt =	1.070 m

Longitud Longitudinal L_L =	1.070 m	
Longitud Vertical, Lv =	5.500 m	
Altura de aplicación de carga, He =	0.810 m	
Altura mínima del parapeto, H =	0.810 m	Acorde con 13.7.3.2

GEOMETRÍA

Se describe la geometría de la barrera propuesta:





H1 = 0.08 m	B1 = 0.15 m	B4 = 0.40 m
H2 = 0.25 m	B2 = 0.05 m	H4 = 0.85 m
H3 = 0.52 m	B3 = 0.25 m	Ver H_{min}: OK

Se calcula el refuerzo requerido en la base de la barrera para soportar el momento generado por la carga de impacto, la cual se calcula como:

$$M_v = (P_t / L_t) \cdot H_e$$

$$M_v = 22.43 \text{ t/m} \cdot 0.81 \text{ m}$$

$$M_v = 18.17 \text{ t-m/m}$$

Ancho Barrera b = 1.000 m
Altura Barrera h = 0.400 m
Recubrimiento = 0.040 m
Profundidad efectiva d = 0.360 m

$f'c$ = 28 MPa
Fy = 420 MPa
$\varphi_{Fluencia}$ = 0.9

Cuantía Calculada ρ_{CALC} = 0.00379
Factor bloque a comp. B1 = 0.85
Cuantía Balanceada ρ_b = 0.02856 Menor que la balanceada
Cuantía Mínima $\rho_{Mínima}$ = 0.00333 Mayor que la mínima

Cuantía Seleccionada ρ_b = 0.00379

As a Utilizar = 13.64 cm²
Barra a usar = N6
Área de la barra = 2.84 cm²
Separación de barras calc = 0.21
Separación seleccionada = 0.20

As a Colocado = 14.20 cm²
Prof. Bl. de Compresión 2.51 cm
Prof. Eje Neutro c = 2.95 cm
Resistencia a flexión = 18.88 t-m
VERIFICACIÓN CUMPLE

Revisión a cortante

$\varphi_{Cortante}$ = 0.9

Se revisa la cortante resistente en la base de la barrera, en la cual se genera una interfaz de transferencia de cortante, cortante por fricción:

La resistencia nominal a cortante en el plano de interfaz debe tomarse como:

$$V_{ni} = cA_{cv} + \mu\left(A_{vf} f_y + P_c\right)$$

Donde:

A_{cv}: área de concreto que participa en la transferencia de cortante por fricción
A_{vf}: área de refuerzo a cortante de interfaz que cruza el plano de corte.
c: factor de cohesión.
μ: factor de fricción
Pc: Fuerza permanente neta a compresión perpendicular al plano de corte.

Para este caso se considera que la transferencia se da de la siguiente manera:

Concreto de densidad normal vaciado con superficie intencionalmente rugosa

Por lo cual,

$c = 170\ t/m^2$

$\mu = 1.00$

$K_1 = 0.25$

$K_2 = 1000\ t/m^2$

Cortante resistente, $\mathbf{V_{ni} = 114.9\ t}$ > Vu = 24.0 t OK

3.12. DISEÑO ESTRUCTURAL DE SUBESTRUCTURA

El diseño estructural de la subestructura del puente estará acondicionado por dos estribos extremos y cuatro (4) apoyos intermedios que soportan la superestructura; la subestructura del puente está construida en concreto reforzado de 28 MPa para caissons y vigas cabezales, mientras que las pilas principales tendrán la misma resistencia de 45 MPa.

El diseño estructural de la subestructura del puente estará acondicionado por el estribo 1 con una viga cabezal de 3.20m de ancho por 2.25m de alto, apoyada sobre tres (3) pilotes de diámetro de Ø2.0m separados cada 5.0m.

El apoyo intermedio 1 está constituido por una columna rectangular hueca de 7.10m de largo por 5.0m de ancho y 0.60 m de espesor, la altura de la columna es de 8.0 m que se conecta monolíticamente a un encepado de concreto reforzado de 21.0m de ancho por 21.0m de largo y 3.50m de altura, que a su vez estará apoyado sobre 16 caissons de diámetro Ø2.0m, a fin de poder distribuir equitativamente las cargas gravitacionales hacia el suelo de cimentación; los caissons se encuentran separados 6.0m centro a centro.

El apoyo intermedio 2 está constituido por una columna rectangular hueca de 7.10m de largo por 5.0m de ancho y 0.60 m de espesor, la altura de la columna es de 10.0 m que se conecta monolíticamente a un encepado de concreto reforzado de 21.0m de ancho por 21.0m de largo y 3.50m de altura, que a su vez estará apoyado sobre 16 caissons de diámetro Ø2.0m, a fin de poder distribuir equitativamente las cargas gravitacionales hacia el suelo de cimentación; los caissons se encuentran separados 6.0m centro a centro.

El apoyo intermedio 3 está constituido por una viga cabezal de 3.50m de ancho por 2.45m de alto que conecta monolíticamente a una columna rectangular hueca de 7.10m de largo por 3.50m de ancho y 0.60 m de espesor, la altura de la columna es de 5.08 m que se conecta monolíticamente a un encepado de concreto reforzado de 11.0m de ancho por 11.0m de largo y 3.50m de altura, que a su vez estará apoyado sobre cuatro (4) caissons de diámetro Ø2.50m, a fin de poder distribuir equitativamente las cargas gravitacionales hacia el suelo de cimentación; los caissons se encuentran separados 7.50m centro a centro.

El apoyo intermedio 4 está constituido por una viga cabezal de 1.90m de ancho por 1.50m de alto que conecta monolíticamente a cuatro columnas circulares de diámetro Ø1.50m, separadas equitativamente 4.0m centro a centro; estas columnas se convierten en pilotes de cimentación cuando entra en contacto con el suelo de cimentación.

El estribo 2 está constituido por una viga cabezal de 1.60m de ancho por 1.50m de alto, que conecta monolíticamente a cuatro columnas circulares de diámetro Ø1.50m, separadas equitativamente 4.0m centro a centro; estas columnas se convierten en pilotes de cimentación cuando entra en contacto con el suelo de cimentación.

Teniendo en cuenta toda la información presentada, se realiza el modelo digital basándose en la geometría estructural de los apoyos intermedios, estribos y vigas postensadas y dovelas sucesivas.

3.12.1. **PARAMETROS DE DISEÑO**

RESISTENCIA DE LOS MATERIALES

CONCRETOS

Resistencia especifica en estribos, apoyos intermedios y aletas f'c = 28 MPa
Resistencia especifica en ménsulas, topes sísmicos, bordillos f'c = 28 MPa
Resistencia especifica en encepados, pilotes y caissons f'c = 28 MPa
Resistencia especifica columnas rectangulares principales f'c = 45 MPa

Módulo de elasticidad Ec =0.043K1wc1.5 √f'c MPa
Coeficiente de expansión térmica α = 1.08 * 10-5 /ºC

ACERO DE REFUERZO

NTC 2289 (ASTM A706)
Resistencia a la fluencia del acero fy = 4200 kg/cm2

COEFICIENTES DE RESISTENCIA

Se considerarán los siguientes coeficientes de resistencia dados en el titulo 5.5.4.2.1.

Flexión y tracción en concreto reforzado	ϕ = 0.90
Cortante y torsión en concreto de densidad normal	ϕ = 0.90
Apoyo en concreto	ϕ = 0.70
Compresión en modelos de puntal - tirantes	ϕ = 0.70
Compresión en zonas de anclaje para concreto de densidad normal	ϕ = 0.80
Tracción en el acero en zonas de anclaje	ϕ = 1.00

3.12.2. **MODELO ESTRUCTURAL**

Para el modelo estructural de la subestructura del puente fue implementado el software de diseño estructural MIDAS CIVIL, donde se incluye las diferentes secciones empleadas para el modelado estructural, las diferentes propiedades mecánicas de los materiales, condiciones

de contorno para los apoyos y las diferentes cargas provenientes y aplicadas a la superestructura.

En las siguientes figuras se presentan el modelo estructural creado en el software MIDAS CIVIL, conformado por una estructura tipo puente en vigas postensadas y cimentación profunda.

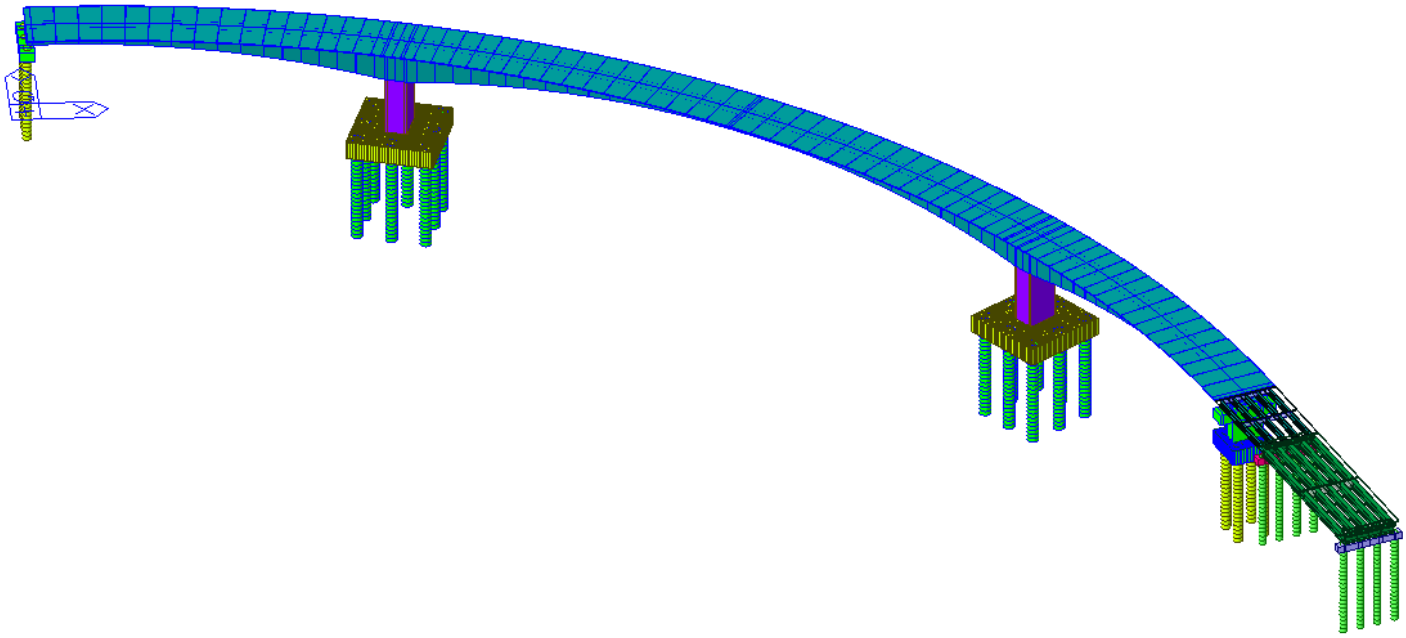


Figura 208. Modelo estructural software MIDAS CIVIL (1/2)

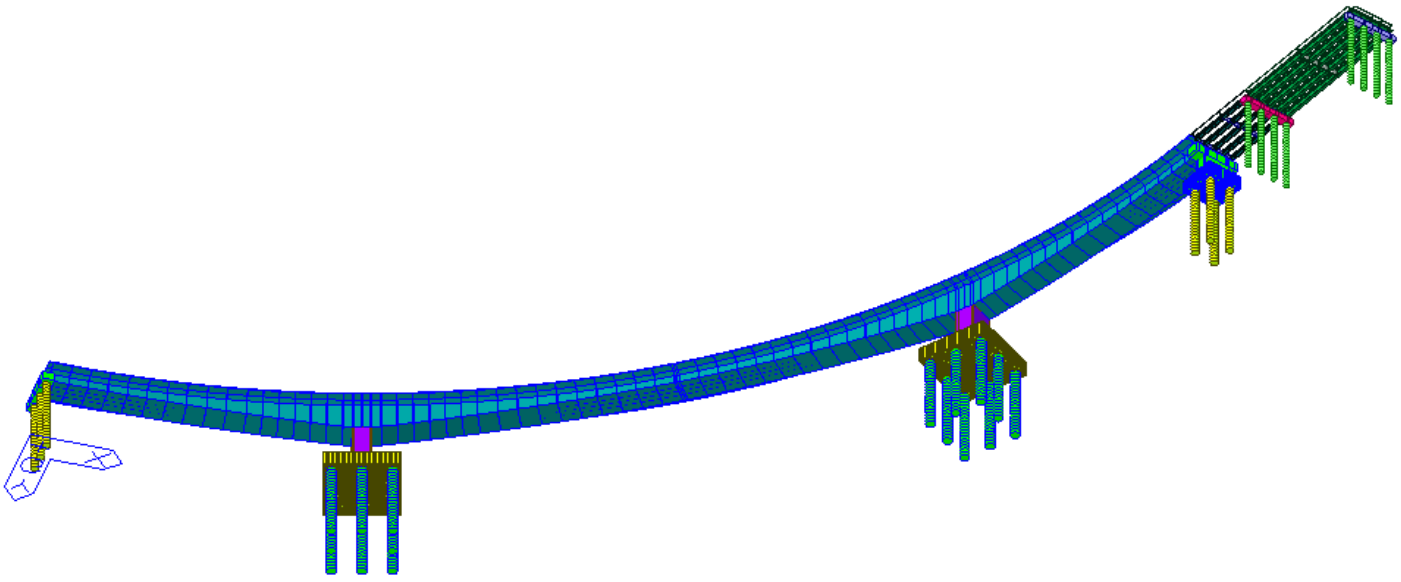


Figura 209. Modelo estructural software MIDAS CIVIL (2/2)

3.12.3. **PROPIEDADES MECÁNICAS DE LOS MATERIALES**

Se definen las propiedades mecánicas de los materiales empleados para el modelo estructural en la siguiente figura.

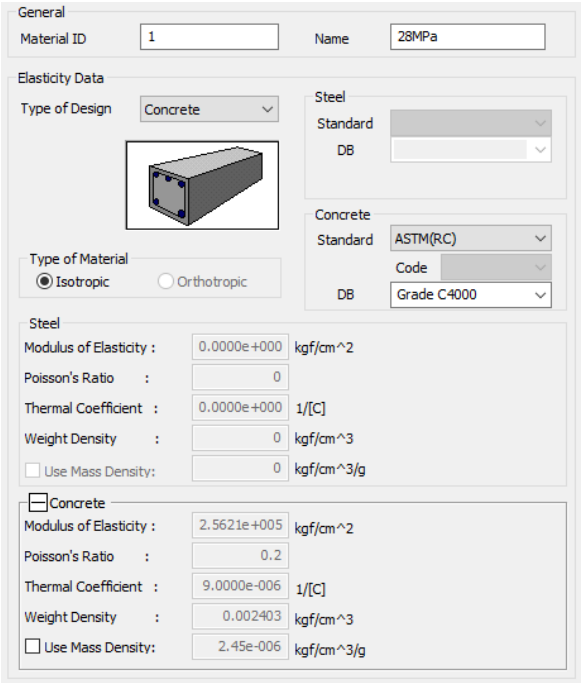


Figura 210. Propiedades mecánicas para los concretos de 28 MPa (280 kg/cm2)

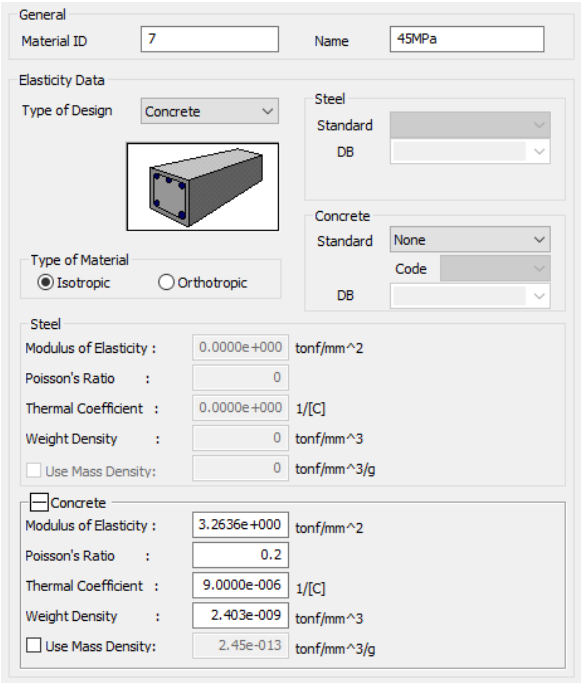


Figura 211. Propiedades para concreto de 45 MPa (450 kg/cm2)

3.12.4. **PROPIEDADES GEOMÉTRICAS DE LOS ELEMENTOS**

Definidas las diferentes características de los materiales, se definen las secciones y elementos estructurales con sus respectivas propiedades geométricas empleadas en el modelo estructural.

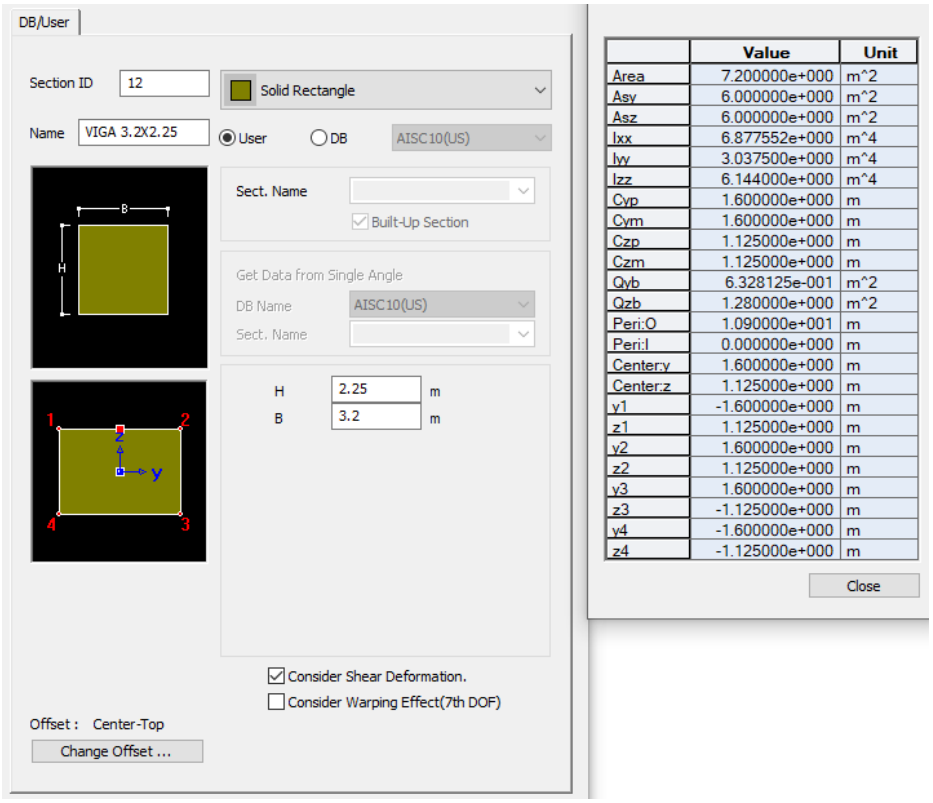


Figura 212. Viga cabezal estribo 1

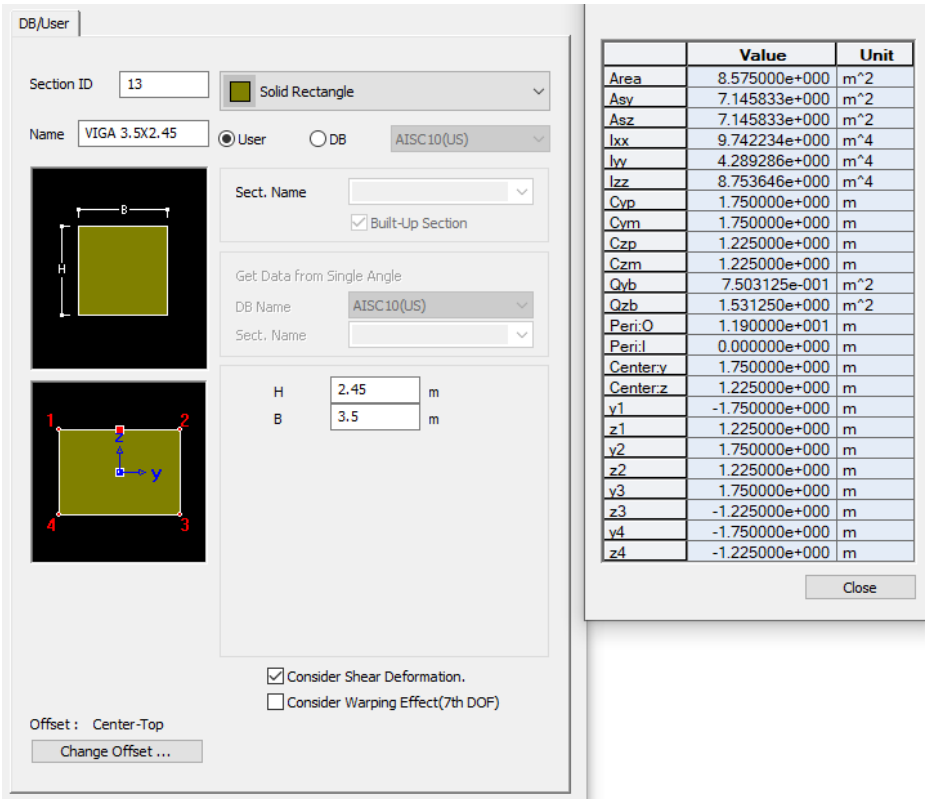


Figura 213. Viga cabezal apoyo 3

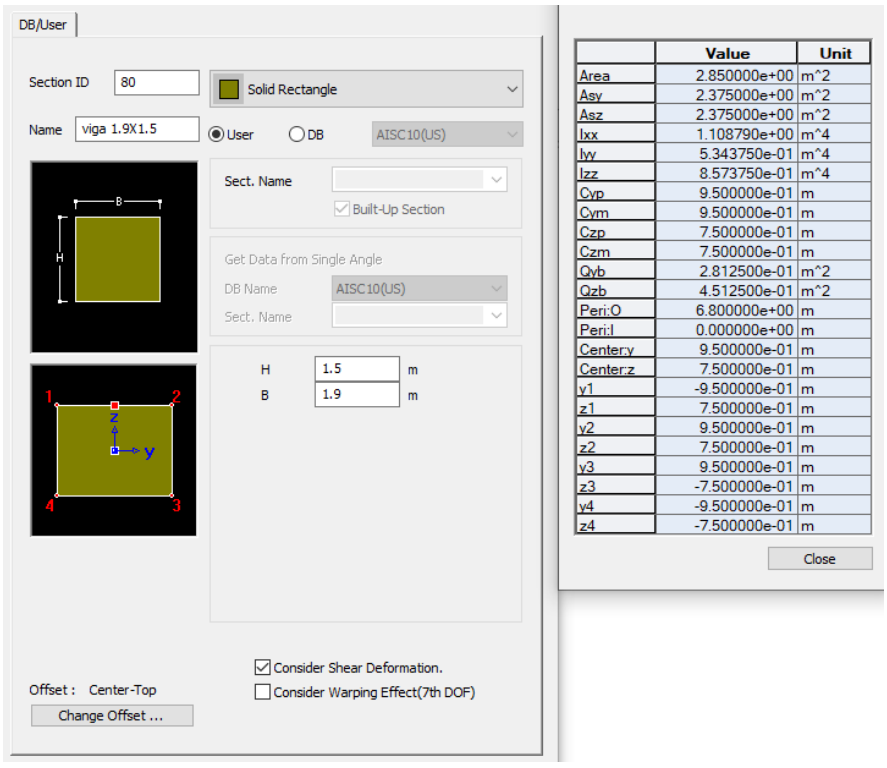
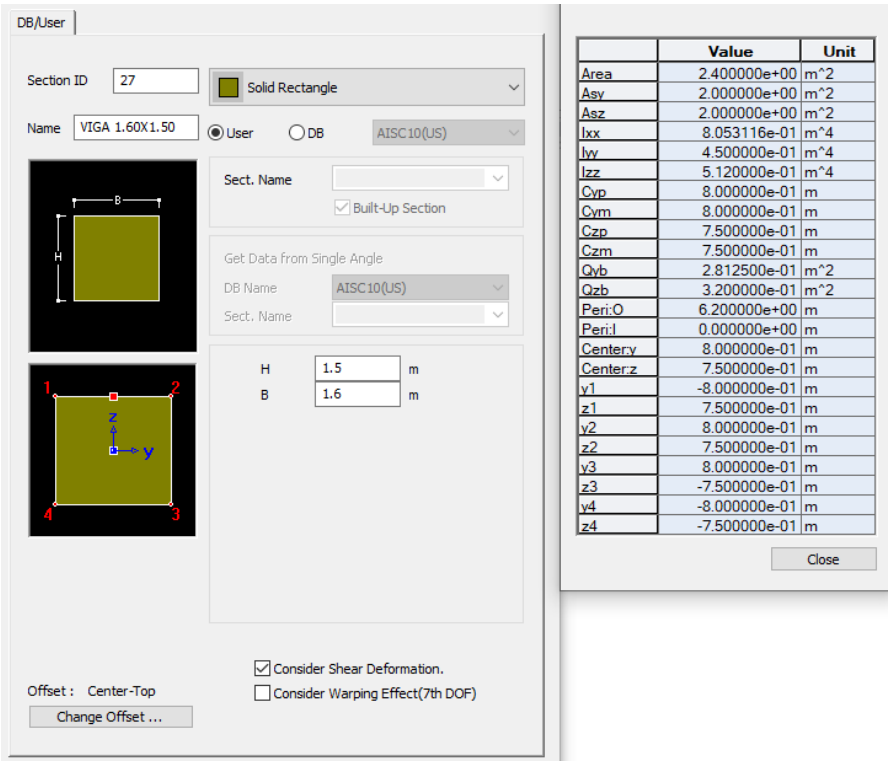
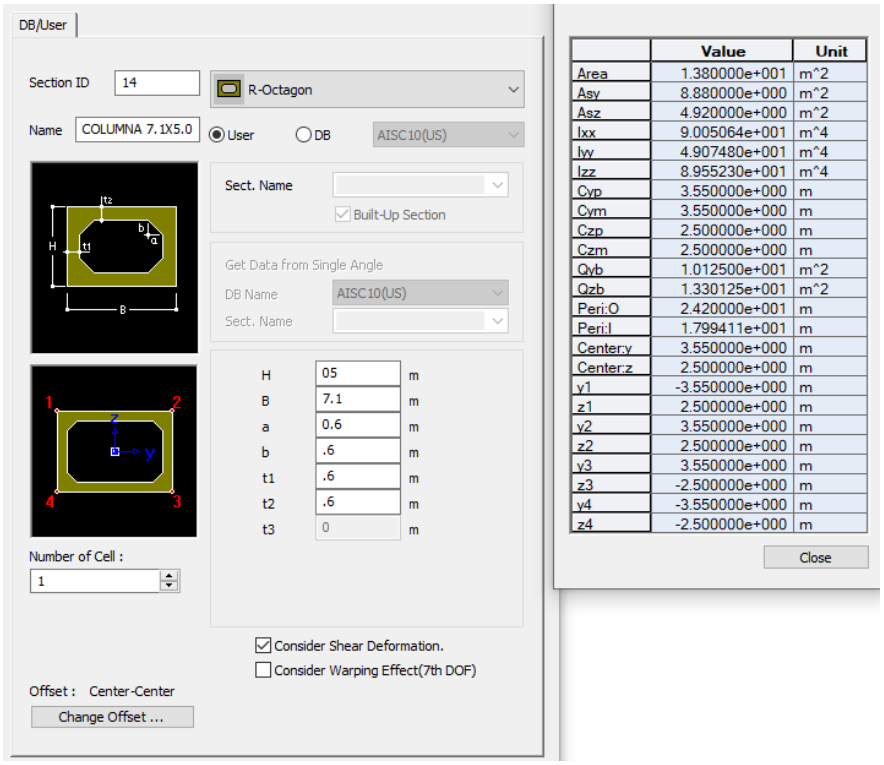


Figura 214. Viga cabezal apoyo 4



	Value	Unit
Area	2.400000e+00	m^2
Asy	2.000000e+00	m^2
Asz	2.000000e+00	m^2
Ixx	8.053116e-01	m^4
Iyy	4.500000e-01	m^4
Izz	5.120000e-01	m^4
Cyp	8.000000e-01	m
Cym	8.000000e-01	m
Czp	7.500000e-01	m
Czm	7.500000e-01	m
Qyb	2.812500e-01	m^2
Qzb	3.200000e-01	m^2
Peri:O	6.200000e+00	m
Peri:I	0.000000e+00	m
Center:y	8.000000e-01	m
Center:z	7.500000e-01	m
y1	-8.000000e-01	m
z1	7.500000e-01	m
y2	8.000000e-01	m
z2	7.500000e-01	m
y3	8.000000e-01	m
z3	-7.500000e-01	m
y4	-8.000000e-01	m
z4	-7.500000e-01	m

Figura 215. Viga cabezal Estribo 2



	Value	Unit
Area	1.380000e+001	m^2
Asy	8.880000e+000	m^2
Asz	4.920000e+000	m^2
Ixx	9.005064e+001	m^4
Iyy	4.907480e+001	m^4
Izz	8.955230e+001	m^4
Cyp	3.550000e+000	m
Cym	3.550000e+000	m
Czp	2.500000e+000	m
Czm	2.500000e+000	m
Qyb	1.012500e+001	m^2
Qzb	1.330125e+001	m^2
Peri:O	2.420000e+001	m
Peri:I	1.799411e+001	m
Center:y	3.550000e+000	m
Center:z	2.500000e+000	m
y1	-3.550000e+000	m
z1	2.500000e+000	m
y2	3.550000e+000	m
z2	2.500000e+000	m
y3	3.550000e+000	m
z3	-2.500000e+000	m
y4	-3.550000e+000	m
z4	-2.500000e+000	m

Figura 216. Pila - Columna apoyo intermedio 1 y 2

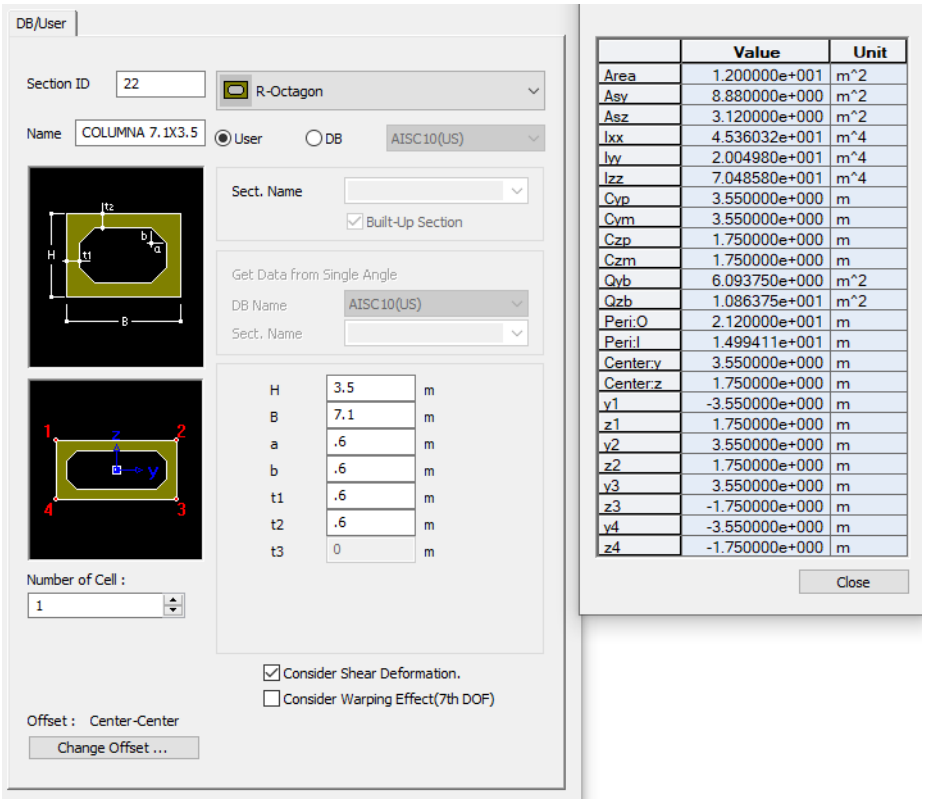


Figura 217. Pila - Columna apoyo intermedio 3

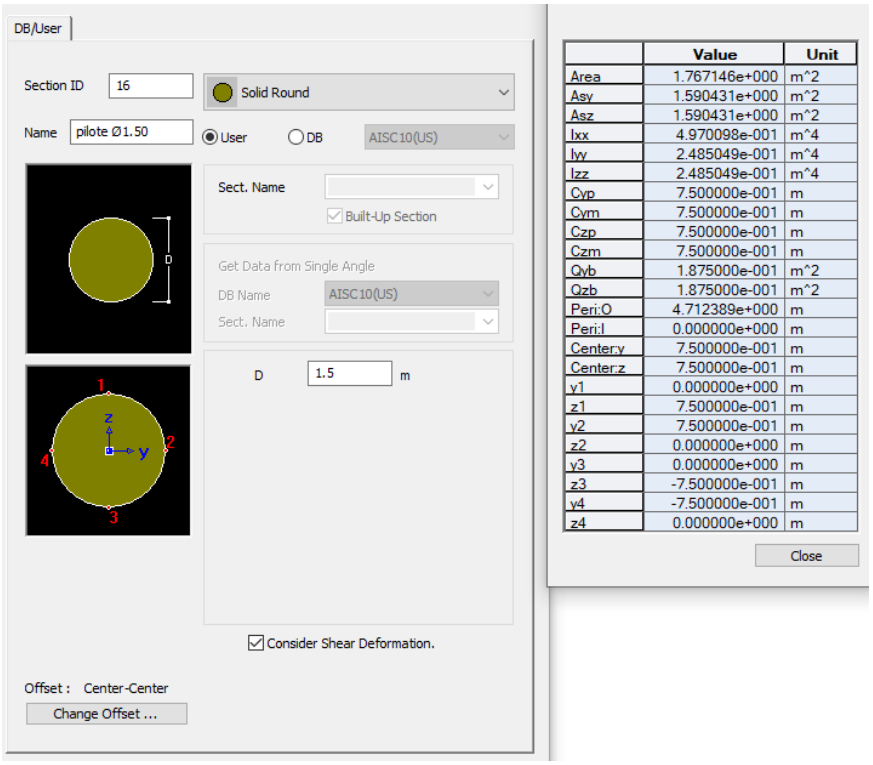
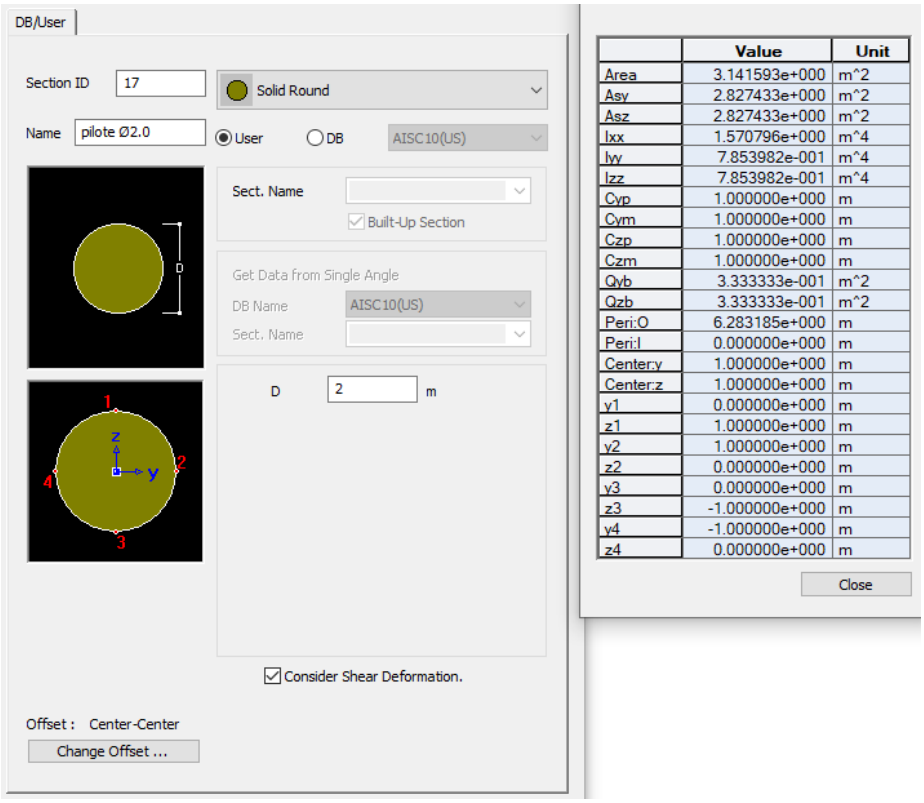
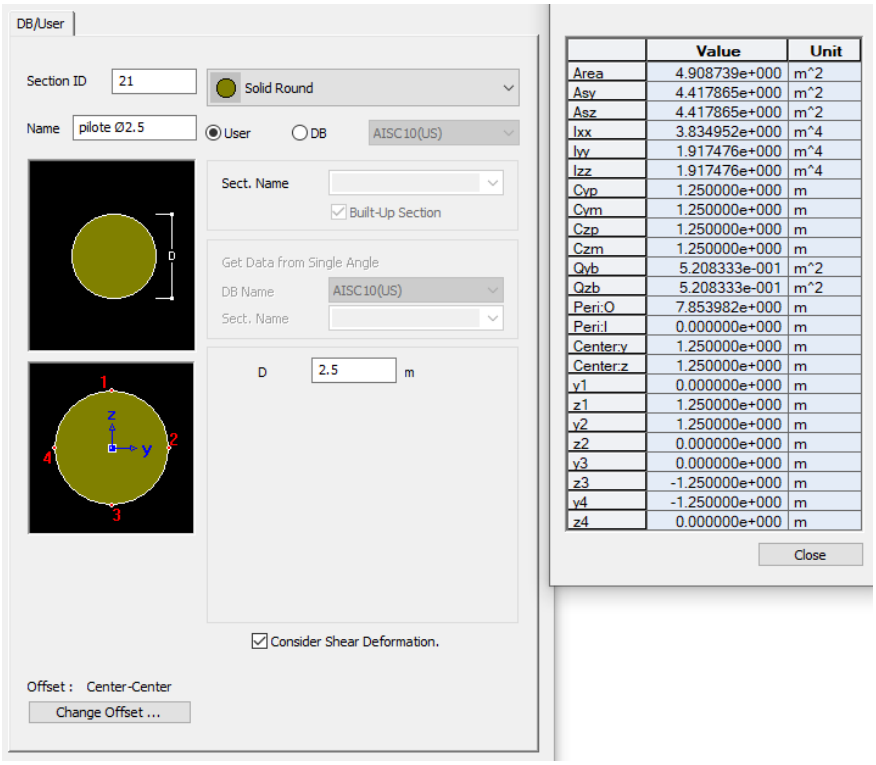


Figura 218. Pilote Ø1.50m



	Value	Unit
Area	3.141593e+000	m^2
Asy	2.827433e+000	m^2
Asz	2.827433e+000	m^2
Ixx	1.570796e+000	m^4
Iyy	7.853982e-001	m^4
Izz	7.853982e-001	m^4
Cyp	1.000000e+000	m
Cym	1.000000e+000	m
Czp	1.000000e+000	m
Czm	1.000000e+000	m
Qyb	3.333333e-001	m^2
Qzb	3.333333e-001	m^2
Peri:O	6.283185e+000	m
Peri:I	0.000000e+000	m
Center:y	1.000000e+000	m
Center:z	1.000000e+000	m
y1	0.000000e+000	m
z1	1.000000e+000	m
y2	1.000000e+000	m
z2	0.000000e+000	m
y3	0.000000e+000	m
z3	-1.000000e+000	m
y4	-1.000000e+000	m
z4	0.000000e+000	m

Figura 219. Pilote Ø2.0m



	Value	Unit
Area	4.908739e+000	m^2
Asy	4.417865e+000	m^2
Asz	4.417865e+000	m^2
Ixx	3.834952e+000	m^4
Iyy	1.917476e+000	m^4
Izz	1.917476e+000	m^4
Cyp	1.250000e+000	m
Cym	1.250000e+000	m
Czp	1.250000e+000	m
Czm	1.250000e+000	m
Qyb	5.208333e-001	m^2
Qzb	5.208333e-001	m^2
Peri:O	7.853982e+000	m
Peri:I	0.000000e+000	m
Center:y	1.250000e+000	m
Center:z	1.250000e+000	m
y1	0.000000e+000	m
z1	1.250000e+000	m
y2	1.250000e+000	m
z2	0.000000e+000	m
y3	0.000000e+000	m
z3	-1.250000e+000	m
y4	-1.250000e+000	m
z4	0.000000e+000	m

Figura 220. Pilote Ø2.50m

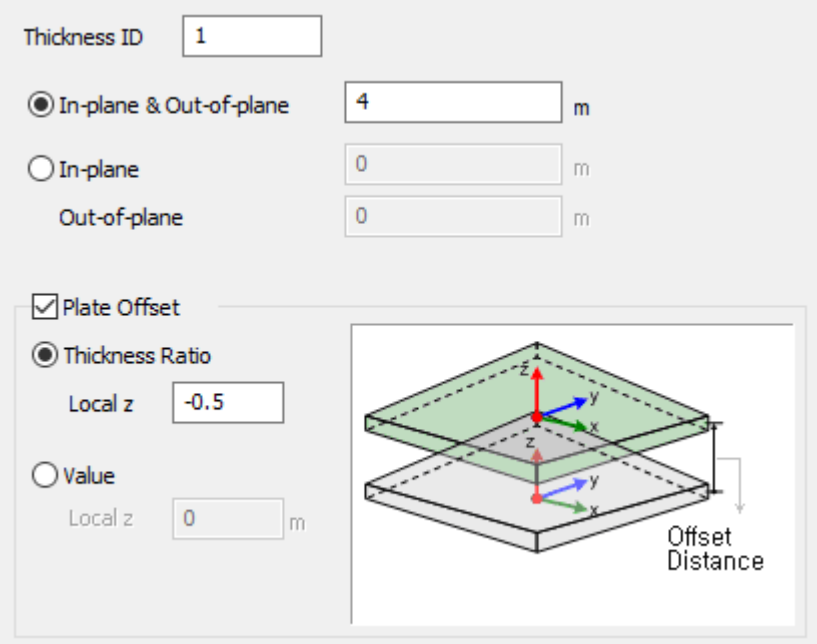


Figura 221. Espesor encepado apoyos intermedios 1 y 2

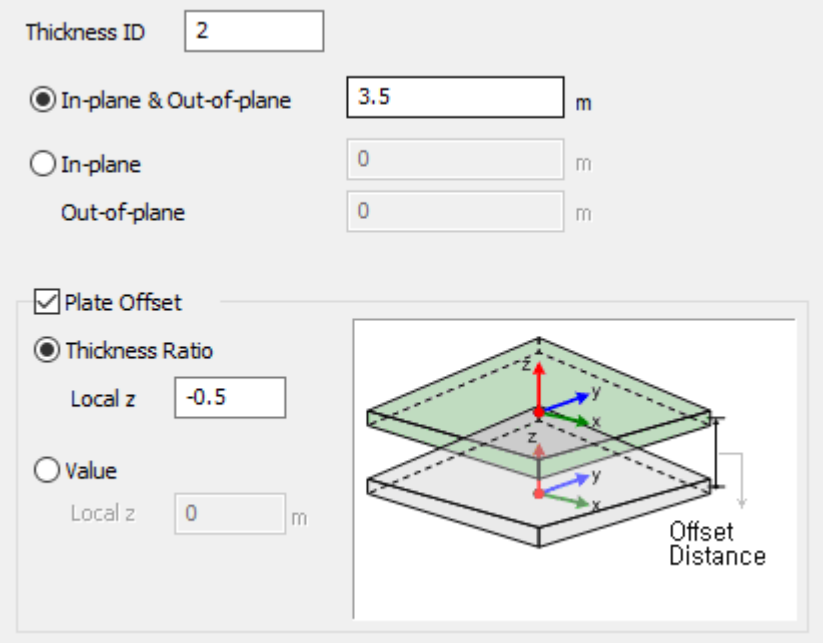


Figura 222. Espesor encepado apoyo intermedio 3

3.12.5. **CONDICIONES DE CONTORNO**

Para las diferentes condiciones de contorno, fue utilizada un único tipo de condición:

1. Apoyos móviles, con un (1) grado de restricción al desplazamiento vertical.

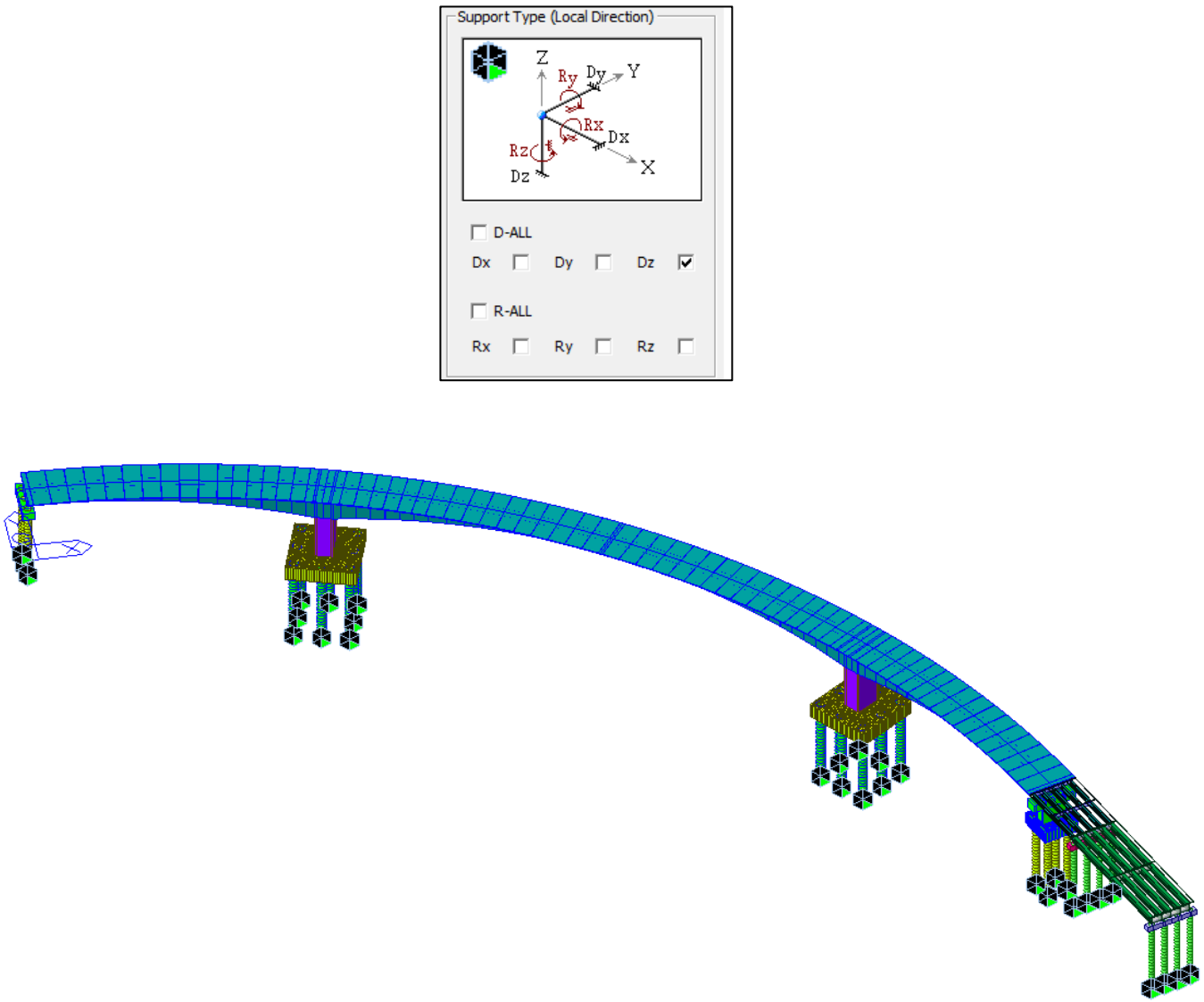


Figura 223. Condiciones de contorno

En cuanto al diseño estructural, este se realiza utilizando los parámetros e información definidas por La Norma Colombiana De Diseño De Puentes LRFD CCP-14 y norma americana AASHTO LRFD, donde se determina las combinaciones de carga que serán impuestas al modelo estructural para determinar de manera más aproximada las solicitaciones actuantes, a fin de poder diseñar todos y cada uno de los elementos estructurales. El diseño se realiza bajo las combinaciones de carga para los siguientes grupos:

Load Combination List

No	Name	Description
1	cLCB1	Strength-I:1.25DC+1.5DW+1.75M[1]+1.75CF+1.75BRK[1]+1.75LS+0.5T[1]+1.25(cD)+1…
2	cLCB2	Strength-I:1.25DC+1.5DW+1.75M[2]+1.75CF+1.75BRK[1]+1.75LS+0.5T[1]+1.25(cD)+1…
3	cLCB3	Strength-II:1.25DC+1.5DW+1.35M[1]+1.35CF+1.35BRK[1]+1.35LS+0.5T[1]+1.25(cD)+…
4	cLCB4	Strength-II:1.25DC+1.5DW+1.35M[2]+1.35CF+1.35BRK[1]+1.35LS+0.5T[1]+1.25(cD)+…
5	cLCB5	Strength-III:1.25DC+1.5DW+1.0W[1]+0.5T[1]+1.25(cD)+1.25(cEL1)+1.25(cEL2)+1.0…
6	cLCB6	Strength-III:1.25DC+1.5DW-1.0W[1]+0.5T[1]+1.25(cD)+1.25(cEL1)+1.25(cEL2)+1.0…
7	cLCB7	Strength-IV:1.5DC+1.5DW+0.5T[1]+1.5(cD)+1.5(cEL1)+1.5(cEL2)+1.0(cTs)+0.5(cCR…
8	cLCB8	Strength-V:1.25DC+1.5DW+1.35M[1]+1.35CF+1.35BRK[1]+1.35LS+1.0W[1]+1.0WL[1]+0…
9	cLCB9	Strength-V:1.25DC+1.5DW+1.35M[1]+1.35CF+1.35BRK[1]+1.35LS+1.0W[1]-1.0WL[1]+0…
10	cLCB10	Strength-V:1.25DC+1.5DW+1.35M[1]+1.35CF+1.35BRK[1]+1.35LS-1.0W[1]+1.0WL[1]+0…
11	cLCB11	Strength-V:1.25DC+1.5DW+1.35M[1]+1.35CF+1.35BRK[1]+1.35LS-1.0W[1]-1.0WL[1]+0…
12	cLCB12	Strength-V:1.25DC+1.5DW+1.35M[2]+1.35CF+1.35BRK[1]+1.35LS+1.0W[1]+1.0WL[1]+0…
13	cLCB13	Strength-V:1.25DC+1.5DW+1.35M[2]+1.35CF+1.35BRK[1]+1.35LS+1.0W[1]-1.0WL[1]+0…
14	cLCB14	Strength-V:1.25DC+1.5DW+1.35M[2]+1.35CF+1.35BRK[1]+1.35LS-1.0W[1]+1.0WL[1]+0…
15	cLCB15	Strength-V:1.25DC+1.5DW+1.35M[2]+1.35CF+1.35BRK[1]+1.35LS-1.0W[1]-1.0WL[1]+0…
16	cLCB16	Extreme-I::1.0DC+1.0DW+0.5M[1]+0.5CF+0.5BRK[1]+0.5LS+1.0(cD)+1.0(cEL1)+1.0(c…
17	cLCB17	Extreme-I::1.0DC+1.0DW+0.5M[2]+0.5CF+0.5BRK[1]+0.5LS+1.0(cD)+1.0(cEL1)+1.0(c…
18	cLCB18	Extreme-I::1.0DC+1.0DW+0.5M[1]+0.5CF+0.5BRK[1]+0.5LS+1.0ESP[1]+1.0(cD)+1.0(c…
19	cLCB19	Extreme-I::1.0DC+1.0DW+0.5M[1]+0.5CF+0.5BRK[1]+0.5LS-1.0ESP[1]+1.0(cD)+1.0(c…
20	cLCB20	Extreme-I::1.0DC+1.0DW+0.5M[1]+0.5CF+0.5BRK[1]+0.5LS+1.0ESP[2]+1.0(cD)+1.0(c…
21	cLCB21	Extreme-I::1.0DC+1.0DW+0.5M[1]+0.5CF+0.5BRK[1]+0.5LS-1.0ESP[2]+1.0(cD)+1.0(c…
22	cLCB22	Extreme-I::1.0DC+1.0DW+0.5M[2]+0.5CF+0.5BRK[1]+0.5LS+1.0ESP[1]+1.0(cD)+1.0(c…
23	cLCB23	Extreme-I::1.0DC+1.0DW+0.5M[2]+0.5CF+0.5BRK[1]+0.5LS-1.0ESP[1]+1.0(cD)+1.0(c…
24	cLCB24	Extreme-I::1.0DC+1.0DW+0.5M[2]+0.5CF+0.5BRK[1]+0.5LS+1.0ESP[2]+1.0(cD)+1.0(c…
25	cLCB25	Extreme-I::1.0DC+1.0DW+0.5M[2]+0.5CF+0.5BRK[1]+0.5LS-1.0ESP[2]+1.0(cD)+1.0(c…
26	cLCB26	Service-I:1.0DC+1.0DW+1.0M[1]+1.0CF+1.0BRK[1]+1.0LS+1.0W[1]+1.0WL[1]+1.0T[1]…
27	cLCB27	Service-I:1.0DC+1.0DW+1.0M[1]+1.0CF+1.0BRK[1]+1.0LS+1.0W[1]-1.0WL[1]+1.0T[1]…
28	cLCB28	Service-I:1.0DC+1.0DW+1.0M[1]+1.0CF+1.0BRK[1]+1.0LS-1.0W[1]+1.0WL[1]+1.0T[1]…
29	cLCB29	Service-I:1.0DC+1.0DW+1.0M[1]+1.0CF+1.0BRK[1]+1.0LS-1.0W[1]-1.0WL[1]+1.0T[1]…
30	cLCB30	Service-I:1.0DC+1.0DW+1.0M[2]+1.0CF+1.0BRK[1]+1.0LS+1.0W[1]+1.0WL[1]+1.0T[1]…
31	cLCB31	Service-I:1.0DC+1.0DW+1.0M[2]+1.0CF+1.0BRK[1]+1.0LS+1.0W[1]-1.0WL[1]+1.0T[1]…
32	cLCB32	Service-I:1.0DC+1.0DW+1.0M[2]+1.0CF+1.0BRK[1]+1.0LS-1.0W[1]+1.0WL[1]+1.0T[1]…
33	cLCB33	Service-I:1.0DC+1.0DW+1.0M[2]+1.0CF+1.0BRK[1]+1.0LS-1.0W[1]-1.0WL[1]+1.0T[1]…
34	cLCB34	Service-II:1.0DC+1.0DW+1.3M[1]+1.3CF+1.3BRK[1]+1.3LS+1.0T[1]+1.0(cD)+1.0(cEL…
35	cLCB35	Service-II:1.0DC+1.0DW+1.3M[2]+1.3CF+1.3BRK[1]+1.3LS+1.0T[1]+1.0(cD)+1.0(cEL…
36	cLCB36	Service-III:1.0DC+1.0DW+0.8M[1]+0.8CF+0.8BRK[1]+0.8LS+1.0T[1]+1.0(cD)+1.0(cE…
37	cLCB37	Service-III:1.0DC+1.0DW+0.8M[2]+0.8CF+0.8BRK[1]+0.8LS+1.0T[1]+1.0(cD)+1.0(cE…
38	cLCB38	Service-IV:1.0DC+1.0DW+1.0W[1]+1.0WL[1]+1.0T[1]+1.0(cD)+1.0(cEL1)+1.0(cEL2)+…
39	cLCB39	Service-IV:1.0DC+1.0DW+1.0W[1]-1.0WL[1]+1.0T[1]+1.0(cD)+1.0(cEL1)+1.0(cEL2)+…
40	cLCB40	Service-IV:1.0DC+1.0DW-1.0W[1]+1.0WL[1]+1.0T[1]+1.0(cD)+1.0(cEL1)+1.0(cEL2)+…
41	cLCB41	Service-IV:1.0DC+1.0DW-1.0W[1]-1.0WL[1]+1.0T[1]+1.0(cD)+1.0(cEL1)+1.0(cEL2)+…
42	cLCB42	Fatigue-I:1.75CF
43	cLCB43	Fatigue-I:1.75M[1]+1.75CF
44	cLCB44	Fatigue-I:1.75M[2]+1.75CF

Load Cases and Factors

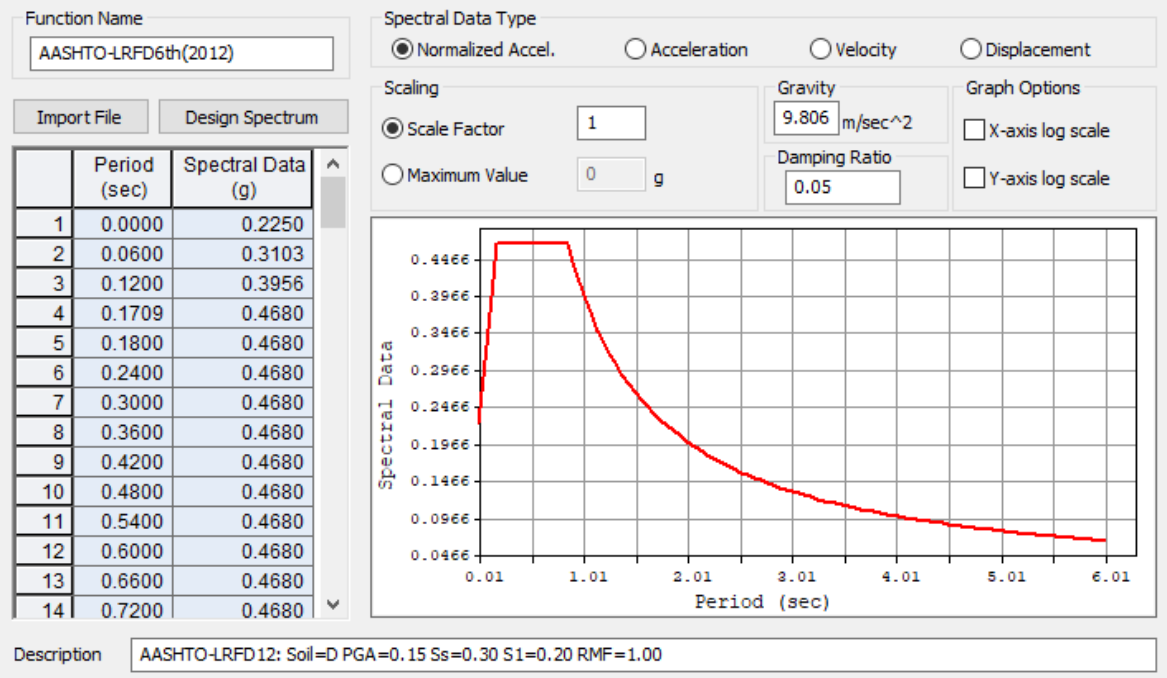
LoadCase	Factor
BARRERA NJ(ST)	1.2500
BARANDA(ST)	1.2500
BORDILLO(ST)	1.2500
ANDEN(ST)	1.2500
PAVIMENTO(ST)	1.5000
3C_I_C_D(MV)	1.7500
FUERZA CENTRIFUGA(ST)	1.7500
FRENADO(ST)	1.7500
PEATONAL(ST)	1.7500
TEMPERATURA(ST)	0.5000
Dead Load(CS)	1.2500
Wet Concrete(CS)	1.2500
Form Traveler(CS)	1.2500
Tendon Secondary(CS)	1.0000
Creep Secondary(CS)	1.2500
Shrinkage Secondary(CS)	1.2500

El análisis sísmico y cálculo de las cargas dinámicas se realizará de acuerdo al Título 3.10 del CCP-14; adicionalmente considerando el tipo de estructuras teniendo en cuenta lo expuesto en el titulo 3.10.5 – Clasificación Operacional este puede clasificarse como puente esencial.

El espectro de diseño, se realizará teniendo en cuenta los parámetros sísmicos dados en el estudio geotécnico y se tendrán en cuenta los efectos ortogonales del sismo, donde se considerará el 100% actuando en la dirección longitudinal más el 30% del sismo en la dirección transversal y posteriormente el sismo actuando 100% en la dirección transversal y 30% en la dirección longitudinal.

El modelo estructural debe ser afectado por las fuerzas sísmicas generadas por el espectro de aceleraciones de diseño, grafica que dependen de los siguientes parámetros sísmicos:

Coeficiente de aceleración pico efectiva PGA	Región 3 = 0.15
Coeficiente de Aceleración Espectral Horizontal Ss	Región 3 = 0.30
Coeficiente de aceleración de respuesta espectral horizontal S1	Región 4 = 0.20



	Period (sec)	Spectral Data (g)
1	0.0000	0.2250
2	0.0600	0.3103
3	0.1200	0.3956
4	0.1709	0.4680
5	0.1800	0.4680
6	0.2400	0.4680
7	0.3000	0.4680
8	0.3600	0.4680
9	0.4200	0.4680
10	0.4800	0.4680
11	0.5400	0.4680
12	0.6000	0.4680
13	0.6600	0.4680
14	0.7200	0.4680

Figura 224. Espectro de aceleraciones de diseño CCP14

3.13. DISEÑO ESTRUCTURAL ESTRIBO 1

3.13.1. DISEÑO ESTRUCTURAL VIGA CABEZAL

Para el diseño estructural de la viga cabezal, se plantea un modelo estructural desarrollado en el software MIDAS CIVIL 2022, que cumple con el diseño geométrico para determinar las solicitaciones tanto para momento flector, como para fuerza axial y cortante, y así, determinar el refuerzo requerido que se dispondrá tanto en la sección transversal propuesta del estribo 1. En la Figura 225 está representado el modelo estructural que será empleado en el software para el diseño estructural estribo 1.

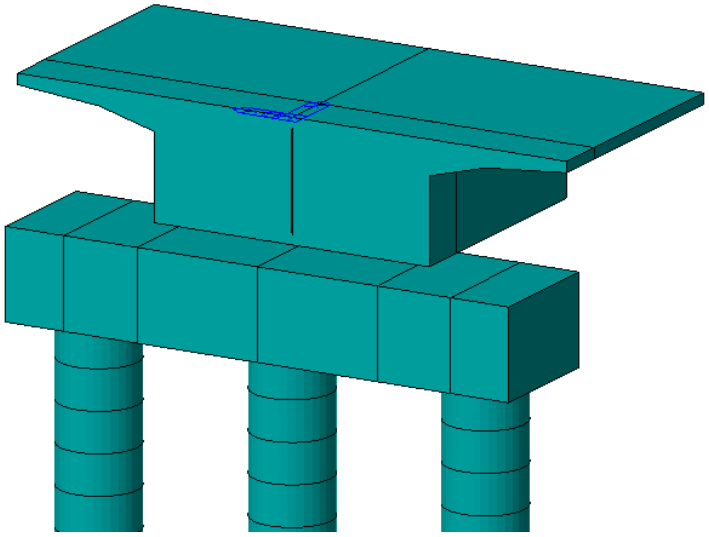


Figura 225. Modelo digital en software MIDAS CIVIL 2022

Para determinar las respectivas solicitaciones, se realiza el modelamiento estructural completo, integrando la superestructura y subestructura por medio de links elásticos que simulan la rigidez vertical y torsional de los neoprenos, a fin de poder transmitir las cargas de la superestructura a la viga cabezal, siendo estas zonas las que se verán más afectadas a momento flector tanto positivo como negativo, valores con los cuales se podrá determinar el refuerzo necesario para suplir estas solicitaciones.

Se realiza el diseño estructural y se aprecia las solicitaciones máximas para momento flector positivo y negativo, además de la fuerza cortante respectivamente, que está siendo sometida la viga cabezal de los estribos a las diferentes combinaciones de carga impuestas en el modelo estructural.

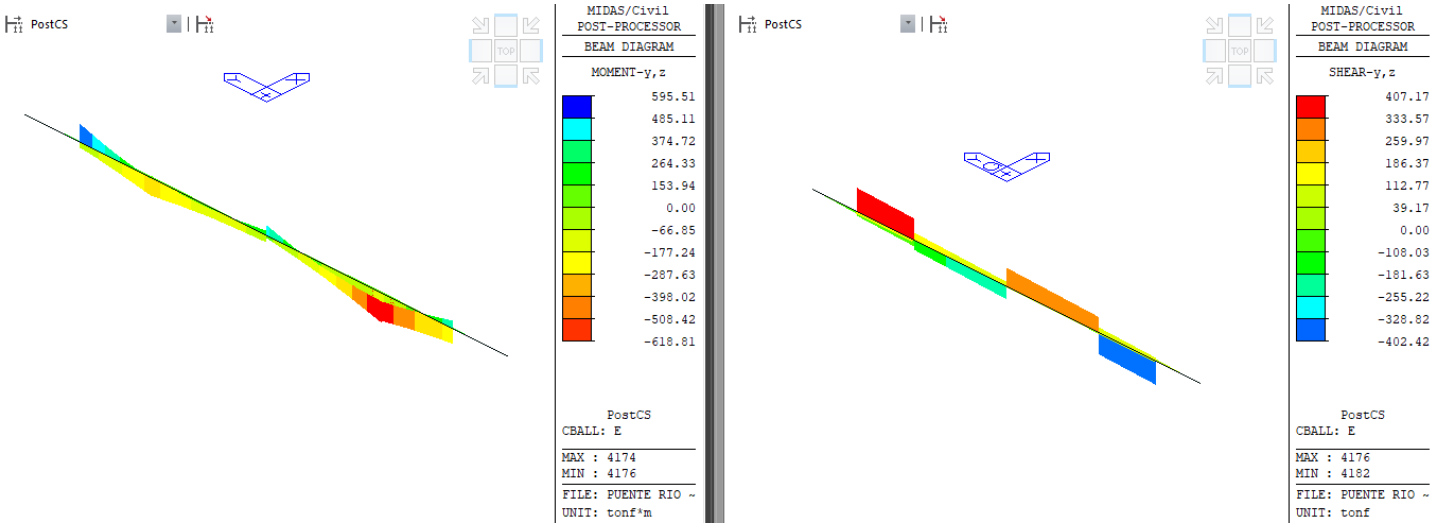


Figura 226. Máximo momento flector y fuerza cortante

SOLICITACIONES DE DISEÑO ESTRUCTURAL

$$M_U{}^{(+)} = 595.51\ ton.m \qquad V_U{}^{(+)} = 407.17\ ton$$
$$M_U{}^{(-)} = -618.81\ ton.m \qquad V_U{}^{(-)} = -402.42\ ton$$

MEMB / SECT / Span	SEL	Section Bc / bf	Hc / hf	fc fy / fys	POS	CHK	AsTop	AsBot	N(-) Mu	N(-) Mr	LCB	Rat-N	P(+) Mu	P(+) Mr	LCB	Rat-P	Vu	pV	LCB	Rat-V
4173		VIGA 3.2X2.2		2812.28	I	OK	0.0107	0.0107	7.03897	868.127	25+	0.008	5.77369	868.127	20-	0.007	9.73122	2009.10	7	0.005
120	☐	3.200	2.250	42184.2	M	OK	0.0107	0.0107	16.8679	868.127	25+	0.019	4.60616	868.127	20-	0.005	29.1937	1852.09	7	0.016
1.5000		0.000	0.000	42184.2	J	OK	0.0107	0.0107	29.1937	868.127	7	0.034	0.00000	868.127	2+	0.000	38.9249	1784.75	7	0.022
4174		VIGA 3.2X2.2		2812.28	I	OK...-	0.0107	0.0107	0.00000	868.127	2+	0.000	621.905	868.127	1+	0.716	282.051	996.109	20+	0.283
120	☐	3.200	2.250	42184.2	M	OK	0.0107	0.0107	106.292	868.127	21-	0.122	459.577	868.127	1+	0.529	308.865	953.426	20+	0.324
3.1000		0.000	0.000	42184.2	J	OK	0.0107	0.0107	336.515	868.127	21-	0.388	202.645	868.127	20+	0.233	322.273	933.421	20+	0.345
4175		VIGA 3.2X2.2		2812.28	I	OK	0.0107	0.0107	234.078	868.127	21-	0.270	274.264	868.127	20+	0.316	216.850	1137.44	21-	0.191
120	☐	3.200	2.250	42184.2	M	OK	0.0107	0.0107	91.2399	868.127	21-	0.105	291.909	868.127	1+	0.336	203.442	1167.90	21-	0.174
3.1000		0.000	0.000	42184.2	J	OK	0.0107	0.0107	67.2300	868.127	21-	0.077	364.668	868.127	20+	0.420	176.627	1234.70	21-	0.143
4176		VIGA 3.2X2.2		2812.28	I	OK	0.0107	0.0107	131.596	868.127	21-	0.152	378.236	868.127	1+	0.436	382.515	847.033	20+	0.452
120	☐	3.200	2.250	42184.2	M	OK	0.0107	0.0107	446.041	868.127	21-	0.514	359.456	868.127	20+	0.414	398.950	827.442	20+	0.482
1.9000		0.000	0.000	42184.2	J	OK	0.0107	0.0107	618.808	868.127	21-	0.713	376.839	868.127	20+	0.434	407.167	817.948	20+	0.498
4181		VIGA 3.2X2.2		2812.28	I	OK	0.0107	0.0107	29.1937	868.127	7	0.034	0.00000	868.127	2+	0.000	38.9249	1784.75	7	0.022
120	☐	3.200	2.250	42184.2	M	OK	0.0107	0.0107	16.8728	868.127	25+	0.019	4.60739	868.127	20-	0.005	29.1937	1852.09	7	0.016
1.5000		0.000	0.000	42184.2	J	OK	0.0107	0.0107	7.04019	868.127	25+	0.008	5.77312	868.127	20-	0.007	9.73122	2009.10	7	0.005
4182		VIGA 3.2X2.2		2812.28	I	OK...-	0.0107	0.0107	430.864	868.127	21-	0.496	563.632	868.127	20+	0.649	402.422	844.780	21-	0.476
120	☐	3.200	2.250	42184.2	M	OK	0.0107	0.0107	256.305	868.127	21-	0.295	547.525	868.127	20+	0.631	394.205	854.844	21-	0.461
1.9000		0.000	0.000	42184.2	J	OK	0.0107	0.0107	0.00000	868.127	2+	0.000	600.532	868.127	1+	0.692	377.770	875.644	21-	0.431

1. Design Information

Design Code : AASHTO-LRFD16

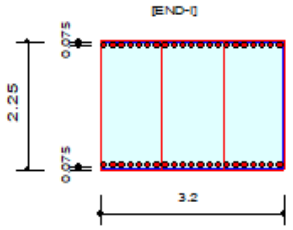
Unit System : tonf, m

Material Data : fc = 2812.28, fy = 42184.2, fys = 42184.2 tonf/m^2

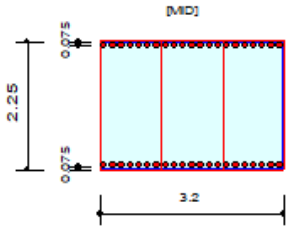
Beam Span : 1.9 m

Section Property : VIGA 3.2X2.25 (No : 120)

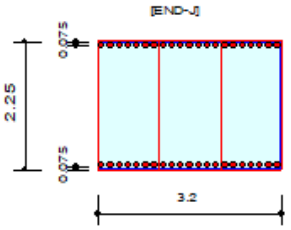
2. Section Diagram



TOP1 : 21- #8,
TOP2 :
BOT1 : 21- #8,
BOT2 :
STIRRUPS : 4.0-#5 @0



TOP1 : 21- #8,
TOP2 :
BOT1 : 21- #8,
BOT2 :
STIRRUPS : 4.0-#5 @0



TOP1 : 21- #8,
TOP2 :
BOT1 : 21- #8,
BOT2 :
STIRRUPS : 4.0-#5 @0

3. Bending Moment Capacity

	END-I	MID	END-J
Negative Moment (Mu)	430.86	446.04	618.81
(-) Load Combination No.	21-	21-	21-
Factored Strength (Mr)	868.13	868.13	868.13
Check Ratio (Mu/Mr)	0.4963	0.5138	0.7128
Positive Moment (Mu)	621.90	547.53	600.53
(+) Load Combination No.	1+	20+	1+
Factored Strength (Mr)	868.13	868.13	868.13
Check Ratio (Mu/Mr)	0.7164	0.6307	0.6918
Using Rebar Top (As_top)	0.0107	0.0107	0.0107
Using Rebar Bot (As_bot)	0.0107	0.0107	0.0107

4. Shear Capacity

	END-I	MID	END-J
Load Combination No.	21-	20+	20+
Factored Shear Force (Vu)	402.42	398.95	407.17
Shear Strength by Conc.(PhiV)	355.79	346.42	341.35
Shear Strength by Rebar.(PhiVs)	488.99	481.02	476.60
Using Shear Reinf. (Av)	0.0001	0.0001	0.0001
Using Stirrups Spacing	4.0-#5 @0	4.0-#5 @0	4.0-#5 @0
Check Ratio	0.4764	0.4821	0.4978
Required long. rebar	*****	*****	*****

5. Torsion Capacity

	END-I	MID	END-J
Load Combination No.	18-	18+	18+
Factored Torsion (Tu)	54.73	55.21	55.21
Torsion Resistance (Tn)	896.04	892.73	878.46
Required Stirrups Spacing	0.00	0.00	0.00
Check Ratio	0.0611	0.0618	0.0628
Required long. rebar	*****	*****	*****

MIDAS/Civil — RC Beam Checking Result

PROJECT TITLE :

	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	puente rio sinú COMPLETO geotecnia.rcs

```
==========================================================================================
 MIDAS/Civil - RC-Beam Checking    [ AASHTO-LRFD16 ]                          Civil 2022
==========================================================================================

      *.MIDAS/Civil - RC-BEAM Analysis/Design Program.

      *.PROJECT       :
      *.DESIGN CODE   : AASHTO-LRFD16,     *.UNIT SYSTEM : tonf, m
      *.MEMBER        : Member Type = BEAM,    MEMB =  4176

      *.DESCRIPTION OF BEAM DATA (iSEC =   120) : VIGA 3.2X2.25
        Section Type : Rectangle (RECT)
        Beam Length (Span)              =         1.900 m.
        Section Depth (Hc)              =         2.250 m.
        Section Width (Bc)              =         3.200 m.
        Concrete Strength (fc')         =      2812.279 tonf/m^2.
        Main Rebar Strength (fy)        =     42184.188 tonf/m^2.
        Stirrups Strength (fys)         =     42184.188 tonf/m^2.
        Modulus of Elasticity (Es)      =  20389024.157 tonf/m^2.

      *.FORCES AND MOMENTS AT CHECK POINT <I> :
        Positive Bending Moment   P-Mu =       378.24 tonf-m.,   LCB =  1+
        Negative Bending Moment   N-Mu =       131.60 tonf-m.,   LCB = 21-
        Shear Force               Vu   =       382.51 tonf.  ,   LCB = 20+

      *.REINFORCEMENT PATTERN :
        ------------------------------------------------------
           Location    i     di( m.)       Rebar    Asi( m^2.)
        ------------------------------------------------------
           Top         1       0.075       21- #8       0.01070
           Bottom      1       0.075       21- #8       0.01070
        ------------------------------------------------------
           Stirrups : 4.0-#5  @150

 ==========================================================================
    [[[*]]]   ANALYZE NEGATIVE BENDING MOMENT CAPACITY.
 ==========================================================================

      ( ). Compute parameter.
           -. phi    =   0.90
           -. Alpha  =   0.85
           -. Beta   =   0.85
           -. d      =     2.1750 m.
           -. ecu    =  0.0030

      ( ). Compute maximum and minimum reinforcement.
           -. Iyy      =       3.0375 m^4.
           -. zbar     =       1.1250 m.
           -. fr       = 0.24*SQRT[fc'] =      337.47 tonf/m^2.
           -. Sc       = Iyy / zbar     =      2.7000 m^3.
           -. Mcr      = 1.2*Sc*fr      =     1093.41 tonf-m.
           -. Rhomin1 = Mcr/[ phi*fy*b*d*(d-a/2) ]     =   0.0019
           -. Rhomin2 = 1.33*Mu/[ phi*fy*b*d*(d-a/2) ] =   0.0003
           -. Rhomin  = MIN[ Rhomin1, Rhomin2 ]        =   0.0003
           -. As_min =  Rhomin * Ag  =     0.0022 m^2.

      ( ). Check tensile reinforcement.
           -. As     =     0.0107 m^2.
           -. As_min < As    --->  O.K !

      ( ). Check moment capacity.
           -. c          =     0.0688 m.
           -. Cc         =        447.65 tonf.
           -. Ts         =        451.51 tonf.
```

MIDAS/Civil — RC Beam Checking Result

PROJECT TITLE :

	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	puente rio sinú COMPLETO geotecnia.rcs

```
          -. Mr         =        868.13 tonf-m.
          -. Mu/Mr      =   0.152  --->  O.K !

================================================================================
   [[[*]]]   ANALYZE POSITIVE BENDING MOMENT CAPACITY.
================================================================================

     ( ). Compute parameter.
          -. phi   =   0.90
          -. Alpha =   0.85
          -. Beta  =   0.85
          -. d     =     2.1750 m.
          -. ecu   =  0.0030

     ( ). Compute maximum and minimum reinforcement.
          -. Iyy     =       3.0375 m^4.
          -. ybar    =       1.1250 m.
          -. fr      = 0.24*SQRT[fc'] =      337.47 tonf/m^2.
          -. Sc      = Iyy / ybar     =      2.7000 m^3.
          -. Mcr     = 1.2*Sc*fr      =     1093.41 tonf-m.
          -. Rhomin1 = Mcr/[ phi*fy*b*d*(d-a/2) ]     =   0.0019
          -. Rhomin2 = 1.33*Mu/[ phi*fy*b*d*(d-a/2) ] =   0.0009
          -. Rhomin  = MIN[ Rhomin1, Rhomin2 ]        =   0.0009
          -. As_min =  Rhomin * Ag  =     0.0064 m^2.

     ( ). Check tensile reinforcement.
          -. As     =     0.0107 m^2.
          -. As_min < As   --->  O.K !

     ( ). Check moment capacity.
          -. c          =     0.0688 m.
          -. Cc         =        447.65 tonf.
          -. Ts         =        451.51 tonf.
          -. Mr         =        868.13 tonf-m.
          -. Mu/Mr      =   0.436  --->  O.K !

================================================================================
   [[[*]]]   ANALYZE SHEAR CAPACITY.
================================================================================

     ( ). Compute shear parameter.
          -. phi   =   0.90
          -. Av    =     0.0008 m^2.
          -. bv    =   3.20 m.
          -. dv    =  MAX[ dv, 0.9*d, 0.72*Hc ]  =   2.15 m.
          -. theta =  41.56 Deg.          [Clause 5.8.3.4.2]
          -. beta  =   1.30

     ( ). Compute shear strength of concrete.
          -. Vu     =      382.51 tonf.
          -. Vc     = 0.0316*beta*SQRT[fc']*bv*dv =      396.69 tonf.
          -. phiVc  = phi * Vc =      357.02 tonf.
          -. Vn_lim = 0.25*fc'*bv*dv =     4827.19 tonf.

     ( ). Compute stirrup spacing.
          -. Maximum spacing smax = MIN[ 0.8*dv, 24 in ] =    0.610 m.
          -. Vu > phiVc  --->  Required shear reinforcement.
          -.                   smax = MIN[ smax, Av/(0.0316*SQRT(fc')*bv/fys) ] =    0.237 m.
          -. Applied spacing s    =    0.150 m.

     ( ). Compute shear strength of reinforcement.
          -. Vs      = Av*fys*dv*cot(theta) / s =      544.46 tonf.
          -. Vs_lim  = 0.25*fc'*bv*dv - Vc =     4430.50 tonf.
          -. Vs      = MIN[ Vs, Vs_lim ] =      544.46 tonf.
```

MIDAS/Civil — RC Beam Checking Result

PROJECT TITLE :

MIDAS	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	puente rio sinú COMPLETO geotecnia.rcs

```
          -. phiVs   = phi*Vs =      490.01 tonf.

          -. phiVs > (Vu-phiVc)  --->  O.K !
          -. Using Av/s  =  Vs / (fys*dv*cot(theta)) =   0.0053 m^2/m.

     ( ). Check tension force in the longitudinal reinforcement caused by shear.
          -. phib    =   0.90
          -. phiv    =   0.90
          -. Vsl     =  MIN[ Vs, Vu/phiv ] =      425.02 tonf.
          -. As_req  =  [ Mu/(phib*dv) + (Vu/phiv - 0.5*Vsl)*cot(theta) ] / fy =   0.0098 m^2.

================================================================================
   [[[*]]]   ANALYZE TORSION CAPACITY.
================================================================================

     ( ). Compute torsion parameter.
          -. phi   =   0.90
          -. Acp   =       7.20 m^2.
          -. Pc    =      10.90 m.
          -. Aoh   =       6.74 m^2.
          -. Ao    =  0.85*Aoh =      5.73 m^2.
          -. Ph    =      10.56 m.
          -. Tu    =       -52.61 tonf-m.
          -. theta =  30.07 Deg.

          -. Tcr   =  0.125*SQRT(fc')*Acp^2/Pc =       835.94 tonf-m.

          -. Tu < 0.25*phi*Tcr =       188.09 tonf-m.
              ---> Torsion check is not required.

================================================================================
   [[[*]]]   ANALYZE CRACK.
================================================================================

     ( ). Check crack control of top reinforcement. ( LCB =  44- )
          -. Mu      =      0.7736 tonf-m.
          -. h       =      2.2500 m.
          -. dc      =      0.0750 m.
          -. Beta_s  = 1+dc/(0.7*(h-dc)) = 1.05 m.
          -. Gamma_e = 1.00
          -. c_cr    =      0.2991 m.
          -. Icr     =      0.3324 m^4.
          -. fss     =        34.785 tonf/m^2.
          -. s       =       0.1496 m.
          -. sa      = 700*Gamma_e/(Beta_s*fss)-2*dc =      0.4572 m.
          -. s < sa --->  O.K !

     ( ). Check crack control of bottom reinforcement. ( LCB =  29+ )
          -. Mu      =    295.7091 tonf-m.
          -. h       =      2.2500 m.
          -. dc      =      0.0750 m.
          -. Beta_s  = 1+dc/(0.7*(h-dc)) = 1.05 m.
          -. Gamma_e = 1.00
          -. c_cr    =      0.2991 m.
          -. Icr     =      0.3324 m^4.
          -. fss     =     13296.925 tonf/m^2.
          -. s       =       0.1496 m.
          -. sa      = 700*Gamma_e/(Beta_s*fss)-2*dc =      0.4572 m.
          -. s < sa --->  O.K !
```

3.13.2. **DISEÑO DE CAISSONS CIRCULARES Ø = 2.0 m**

Para el diseño estructural de la estructura tipo caisson circular de concreto reforzado, se hace necesario utilizar el software MIDAS y determinar el refuerzo necesario para suplir todas las solicitaciones impuestas y que se encuentran generadas por el modelo, de esta manera, los pilotes se encuentran subdivididos en segmentos de 1.0 m de altura.

Del modelo estructural, se requiere determinar la carga gravitacional que desciende de la viga cabezal a los caissons de los estribos y posteriormente al suelo de cimentación.

PK	**0+138.00**	**0+218.00**	**0+378.00**	**0+458.00**	**0+479.50**	**0+509.60**
ENVOLVENTE DE RESISTENCIA ÚLTIMA						
Øcaisson	**2.00**	**2.00**	**2.00**	**2.50**	**1.50**	**1.50**
Sep. Caisson C.A.C.	**5.00**	**6.00**	**6.00**	**7.50**	**4.00**	**4.00**
cargas en ton	**ESTRIBO 1**	**APOYO 1**	**APOYO 2**	**APOYO 3**	**APOYO 4**	**ESTRIBO 2**
P1	325.00	700.00	873.00	894.00	667.00	424.00
P2	400.00	1003.00	1170.00	637.00	449.00	289.00
P3	347.00	1160.00	1334.00	888.00	357.00	256.00
P4		1105.00	1283.00	645.00	236.00	132.00
P5		845.00	893.00			
P6		1169.00	1189.00			
P7		1384.00	1425.00			
P8		1336.00	1412.00			
P9		896.00	800.00			
P10		1199.00	1125.00			
P11		1418.00	1365.00			
P12		1371.00	1366.00			
P13		859.00	632.00			
P14		1163.00	933.00			
P15		1312.00	**1115.00**			
P16		1222.00	1110.00			
Long.	**34.00**	**50.00**	**40.00**	**30.00**	**35.00**	**20.00**
PROMEDIO PILOTE ton	357.33	1133.88	1126.56	766.00	427.25	275.25
PROMEDIO PILOTE kN	3573.33	11338.75	11265.63	7660.00	4272.50	2752.50
TOTAL ton	**1072.00**	**18142.00**	**18025.00**	**3064.00**	**1709.00**	**1101.00**
TOTAL kN	**10720.00**	**181420.00**	**180250.00**	**30640.00**	**17090.00**	**11010.00**

ENVOLVENTE DE EVENTO EXTREMO						
Øcaisson	**2.00**	**2.00**	**2.00**	**2.50**	**1.50**	**1.50**
Sep. Caisson C.A.C.	**5.00**	**6.00**	**6.00**	**7.50**	**4.00**	**4.00**
cargas en ton	**ESTRIBO 1**	**APOYO 1**	**APOYO 2**	**APOYO 3**	**APOYO 4**	**ESTRIBO 2**
P1	437.00	1073.00	1289.00	1167.00	583.00	38.00
P2	303.00	1100.00	1338.00	1005.00	314.00	186.00
P3	467.00	1233.00	1436.00	1132.00	287.00	186.00
P4		1298.00	1525.00	992.00	333.00	210.00
P5		1203.00	1189.00			
P6		1097.00	1167.00			
P7		1235.00	**1309.00**			
P8		1544.00	1516.00			
P9		1292.00	1057.00			
P10		1174.00	1031.00			
P11		1344.00	1213.00			
P12		1617.00	1457.00			
P13		1281.00	969.00			
P14		1384.00	1023.00			
P15		1465.00	**1156.00**			
P16		1569.00	1308.00			
Long.	**34.00**	**50.00**	**40.00**	**30.00**	**35.00**	**20.00**
PROMEDIO PILOTE ton	402.33	1306.81	1248.94	1074.00	379.25	155.00
PROMEDIO PILOTE kN	4023.33	13068.13	12489.38	10740.00	3792.50	1550.00
TOTAL ton	**1207.00**	**20909.00**	**19983.00**	**4296.00**	**1517.00**	**620.00**
TOTAL kN	**12070.00**	**209090.00**	**199830.00**	**42960.00**	**15170.00**	**6200.00**

ENVOLVENTE DE SERVICIO						
Øcaisson	**2.00**	**2.00**	**2.00**	**2.50**	**1.50**	**1.50**
Sep. Caisson C.A.C.	**5.00**	**6.00**	**6.00**	**7.50**	**4.00**	**4.00**
cargas en ton	**ESTRIBO 1**	**APOYO 1**	**APOYO 2**	**APOYO 3**	**APOYO 4**	**ESTRIBO 2**
P1	260.00	436.00	711.00	715.00	518.00	329.00
P2	327.00	634.00	931.00	517.00	344.00	220.00
P3	292.00	780.00	1052.00	704.00	284.00	200.00
P4		732.00	1004.00	524.00	199.00	111.00
P5		650.00	739.00			
P6		859.00	960.00			
P7		1052.00	**1151.00**			
P8		1016.00	1127.00			
P9		740.00	642.00			
P10		969.00	838.00			
P11		1149.00	1037.00			
P12		1098.00	1034.00			
P13		699.00	437.00			
P14		928.00	624.00			
P15		1040.00	**775.00**			
P16		962.00	760.00			
Long.	**34.00**	**50.00**	**40.00**	**30.00**	**35.00**	**20.00**
PROMEDIO PILOTE ton	293.00	859.00	863.88	615.00	336.25	215.00
PROMEDIO PILOTE kN	2930.00	8590.00	8638.75	6150.00	3362.50	2150.00
TOTAL ton	**879.00**	**13744.00**	**13822.00**	**2460.00**	**1345.00**	**860.00**
TOTAL kN	**8790.00**	**137440.00**	**138220.00**	**24600.00**	**13450.00**	**8600.00**

Estribo 1 – PK0+138

Según el **Volumen IV. ESTUDIO DE SUELOS PARA DISEÑO DE FUNDACIONES Y OBRAS COMPLEMENTARIAS**, para transmitir la carga vertical adecuadamente al suelo de cimentación, se requiere tener un diámetro de caisson de Ø=2.0 m hasta completar una longitud total de 34.0 m según recomendaciones del estudio de suelos y geotecnia.

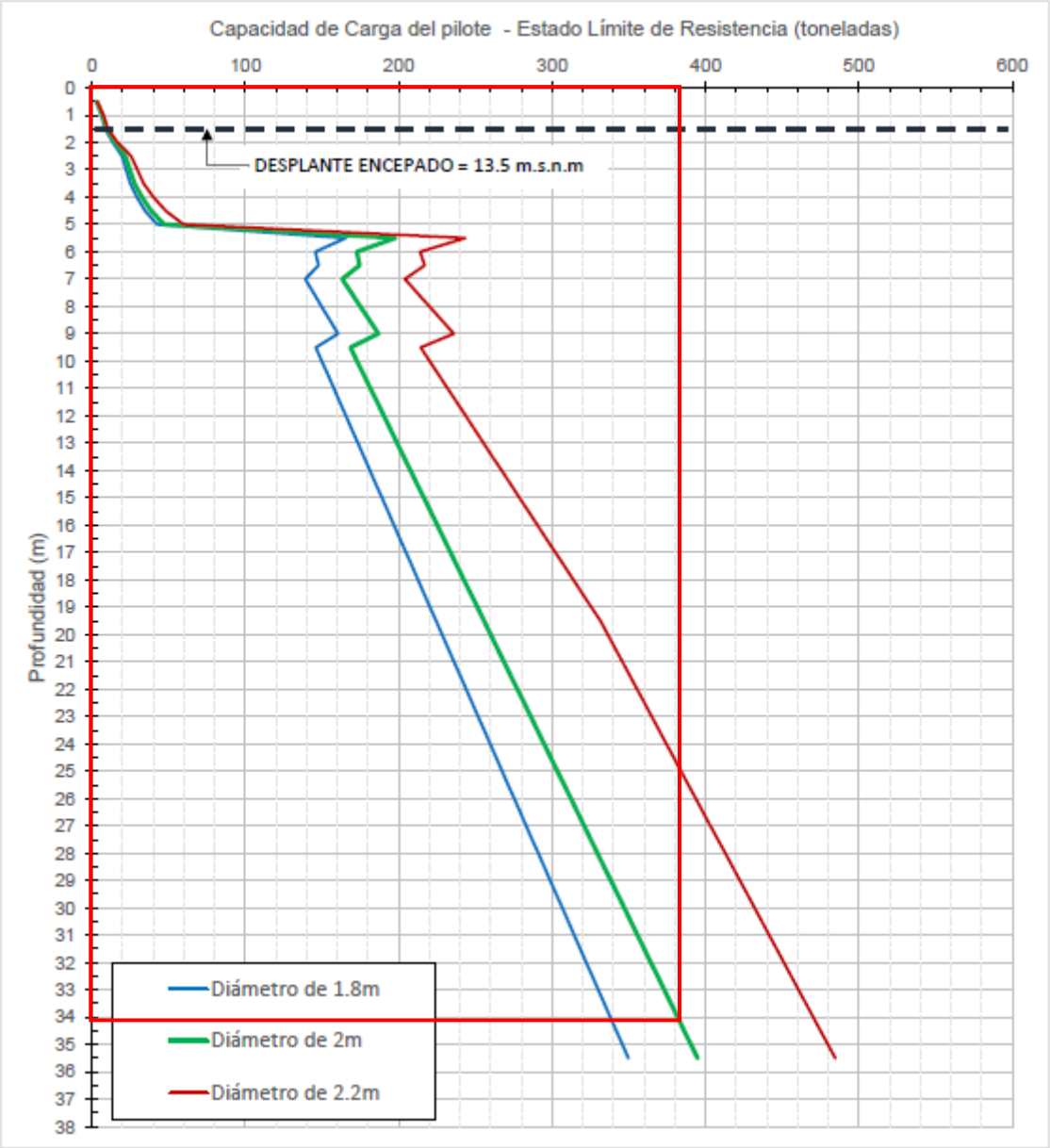


Figura 227. Resistencia Reducida Caissons Estribo 1

De la anterior gráfica, se puede estimar una carga por resistencia última de 381 ton por caissons, y una carga promedio de 358 ton por caissons, con factor de seguridad de Demanda/capacidad FS=381 ton/ 358 ton = 1.06

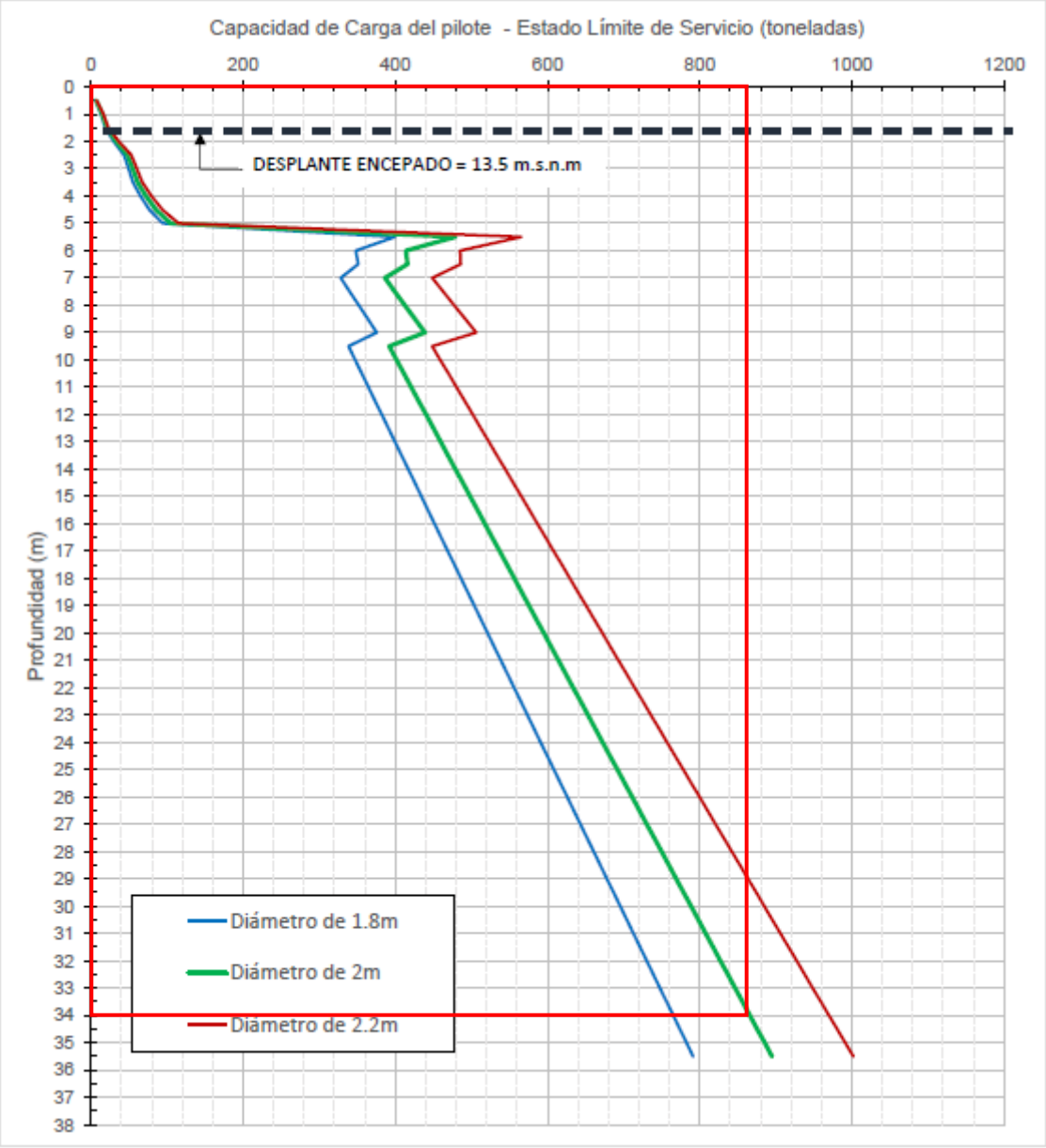


Figura 228. Resistencia evento extremo Caissons Estribo 1

De la anterior gráfica, se puede estimar una carga por evento extremo de 866 ton por caisson, y una carga promedio de 402 ton por caisson, con factor de seguridad de Demanda/capacidad FS = 866 ton / 402 ton = 2.15.

Del mismo modo, del **Volumen IV. ESTUDIO DE SUELOS PARA DISEÑO DE FUNDACIONES Y OBRAS COMPLEMENTARIAS** se extrajeron los módulos de

reacción horizontal del suelo, los cuales se utilizaron para determinar los resortes en el modelo estructural de los pilotes, a fin de poder determinar la interacción suelo-estructura ante las solicitaciones presentadas.

VARIACIÓN DE MODULO DE BALASTO								
	Inicio	Fin	Diámetro	Balasto	Diámetro	Balasto	Diámetro	Balasto
(#)	(m)	(m)	(m)	kh (kg/cm3)	(m)	kh (kg/cm3)	(m)	kh (kg/cm3)
1	0	3	1.8	0.1808	2	0.1605	2.2	0.1459
2	3	5	1.8	0.2917	2	0.2626	2.2	0.2387
3	5	9	1.8	0.6178	2	0.5560	2.2	0.5055
4	9	30	1.8	0.5370	2	0.4833	2.2	0.4393

ESTRIBO 1				
Diámetro de pilote (m) Ø		**2.00**	**Altura elemento h=1.0m**	
Profundidades (m)		**Módulo de reacción**	**Módulo de reacción**	**K_H**
De	**A**	**(kg/cm^3)**	**ton/m3**	**(ton/m)**
0.00	3.00	0.1605	160.50	321
3.00	5.00	0.2626	262.60	525
5.00	9.00	0.5560	556.00	1112
9.00	30.00	0.4833	483.30	967

Se presenta la interacción suelo-estructura debidamente incluida en el software MIDAS CIVIL

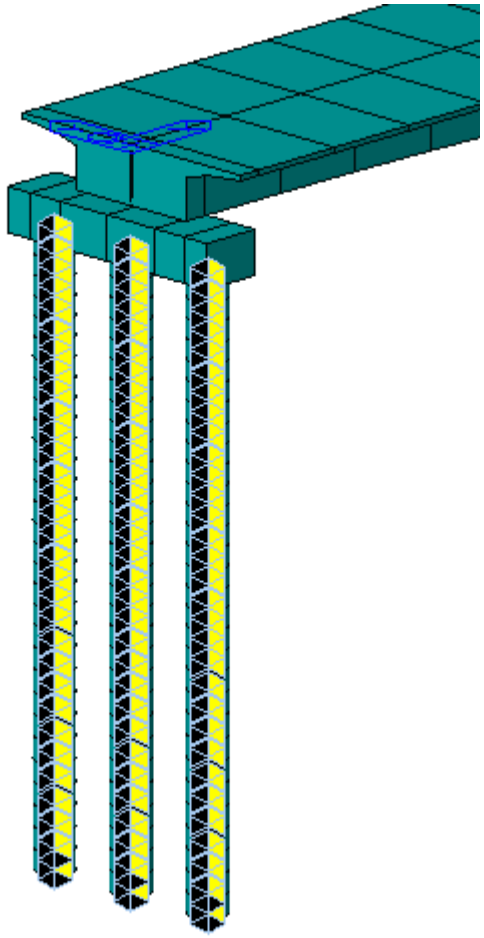


Figura 229. Módulos de reacción horizontal Kh Estribo 1

Para el diseño estructural de elementos sometidos a flexo-compresión se debe realizar de manera iterativa hasta determinar qué cantidad de refuerzo cumple con la cuantía requerida y con la cuantía mínima, en este orden de ideas, se plantea una alternativa para los caissons del estribo 1.

Con ayuda del software MIDAS CIVIL se pudo determinar la cuantía requerida para atender las solicitaciones a momento flector y fuerza cortante generada por la variación entre combinaciones de carga establecidas por el código CCP-14.

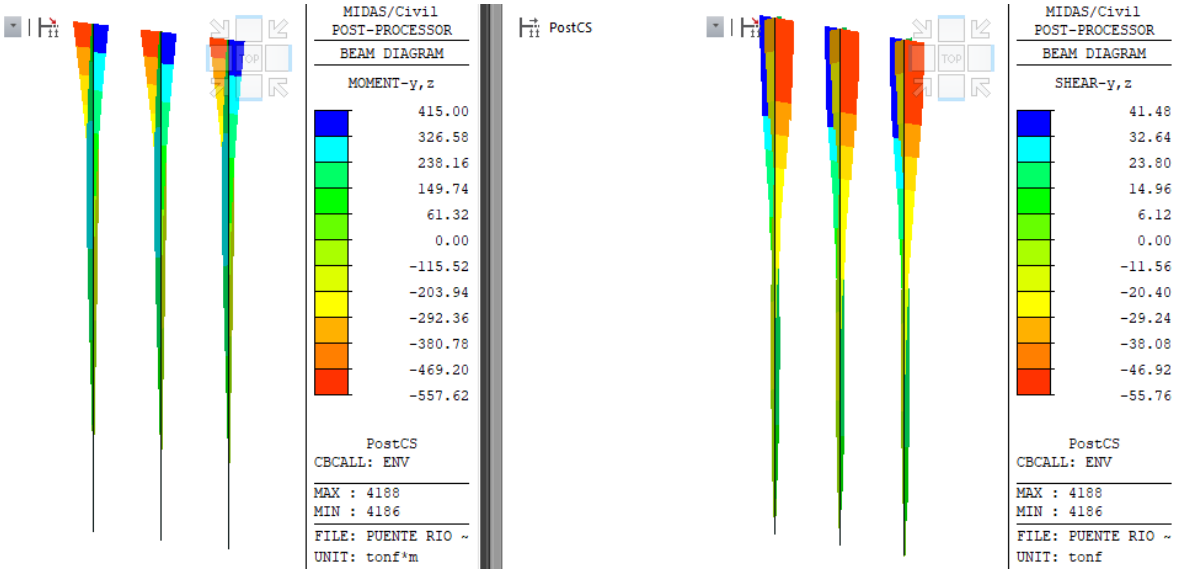


Figura 230. Momento flector y fuerza cortante para la envolvente de carga Estribo 1

SOLICITACIONES DE DISEÑO ESTRUCTURAL

$M_U{}^{(+)} = 415.00\ ton.m$ $\quad V_U{}^{(+)} = 41.48\ ton$

$M_U{}^{(-)} = -557.62\ ton.m$ $\quad V_U{}^{(-)} = -55.76\ ton$

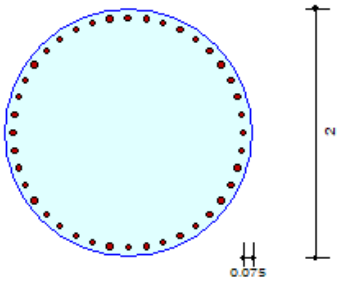
Diseño a Flexo-compresión

1. Design Condition

Design Code	AASHTO-LRFD16
Unit System	tonf, m
Member Number	4188 (PM), 4188 (Shear)
Material Data	fc = 2812.28, fy = 42184.2, fys = 42184.2 tonf/m^2
Column Height	1.05 m
Section Property	pilote Ø2.0 estribos (No : 170)
Rebar Pattern	

	Pos 1	Pos 2	Pos 3
Layer 1	40-#10	--	--

Total Rebar Area Ast = 0.0327741 m^2 (Rhost = 0.0104)



2. Applied Loads

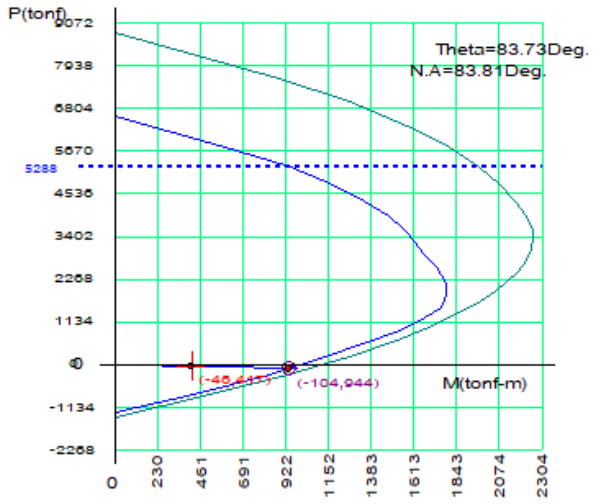
Load Combination 20+ AT (I) Point

Pu = -45.737 tonf, Mcy = 45.0422, Mcz = 415.001, Mc = 417.438 tonf-m

3. Axial Forces and Moments Capacity Check

Concentric Max. Axial Load	Pr-max	= 5288.39 tonf	
Axial Load Ratio	Pu/Pr	= -45.737 / -104.27	= 0.439 < 1.000 ..... O.K
Moment Ratio	Mcy/Mry	= 45.0422 / 103.079	= 0.437 < 1.000 ..... O.K
	Mcz/Mrz	= 415.001 / 938.295	= 0.442 < 1.000 ..... O.K
	Mc/Mr	= 417.438 / 943.940	= 0.442 < 1.000 ..... O.K

4. P-M Interaction Diagram



Pr(tonf)	Mr(tonf-m)
6610.49	0.00
5238.57	950.92
4338.02	1351.82
3574.28	1563.48
2964.52	1665.13
2569.61	1743.22
2277.31	1778.70
2003.93	1789.63
1695.62	1778.85
1248.27	1675.68
588.17	1369.73
-253.98	837.28
-1244.29	0.00

5. Shear Force Capacity Check

Applied Shear Force	Vu	= 56.6449 tonf (Load Combination 25-)	
Shear Strength by Conc	PhiV	= 492.718 tonf	
Shear Strength by Rebar	PhiVs	= 315.321 tonf (2.0-#5 @0)	
Shear Ratio	Vu/Vr	= 56.6449 / 808.039	= 0.070 < 1.000 ..... O.K

MIDAS/Civil — RC Column Checking Result

PROJECT TITLE :

MIDAS	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	e rio sinú COMPLETO geotecnia hzapata=3.

```
==========================================================================================
 MIDAS/Civil - RC-Column Checking [ AASHTO-LRFD16 ]                          Civil 2022
==========================================================================================

      *.MIDAS/Civil - RC-COLUMN  Analysis/Design Program.
      *.DESIGN CODE    : AASHTO-LRFD16,     *.UNIT SYSTEM : tonf, m
      *.MEMBER         : Member Type = COLUMN ,  MEMB =  4188,  LCB =  20+,  POS =    I

      *.DESCRIPTION OF COLUMN  DATA (iSEC =   170) : pilote Ø2.0 estribos
        Column Height (L)                =         1.000 m.

        Section Type : SOLID ROUND (SR)
        Section Diameter  (D)             =         2.000 m.
        Concrete Strength (fc)            =      2812.279 tonf/m^2.
        Main Rebar Strength (fy)          =     42184.188 tonf/m^2.
        Ties/Spirals Strength (fys)       =     42184.188 tonf/m^2.
        Modulus of Elasticity (Es)        =  20389024.157 tonf/m^2.

      *.REINFORCEMENT PATTERN :
        ------------------------------------------------------------------------------------
          Layer         Pos1    As( m^2.)         Pos2    As( m^2.)         Pos3    As( m^2.)
        ------------------------------------------------------------------------------------
            1         40-#10 (    0.0328)           -                         -
        ------------------------------------------------------------------------------------

      *.Ties :  2.0-#5  @0

 ==========================================================================
    [[[*]]]   CALCULATE SLENDERNESS RATIOS, MAGNIFIED FORCES/MOMENTS.
 ==========================================================================

      ( ). Factored forces/moments caused by unit load case.  Unit : tonf., m.
           *.Load combination ID =  20+
           ------------------------------------------------------------------------------
            Load Case          Pu_max          Myi          Myj          Mzi          Mzj
           ------------------------------------------------------------------------------
            DL                190.10        14.96        20.04       -60.93       -54.24
            LL                 -3.87         0.18         0.27         2.71         2.53
            DL+LL             186.23        15.15        20.31       -58.22       -51.71
            Others           -231.97        29.90        39.19       473.22       425.23
           ------------------------------------------------------------------------------
            DL+LL+Others      -45.74        45.04        59.50       415.00       373.52
           ------------------------------------------------------------------------------

 ==========================================================================
    [[[*]]]   ANALYZE CAPACITY OF BIAXIALLY LOADED RC-COLUMN.
 ==========================================================================

      ( ). Compute design parameter.
           -. phi   =   0.90
           -. Alpha =   0.85
           -. Beta  =   0.85
           -. ecu   =  0.0030

      ( ). Compute maximum and minimum reinforcement.
           -. Rhomaxl =   0.0300 (User Input)
           -. Rhomax  = MIN[ Rhomaxl, 0.08 ] =   0.0300
           -. Rhomin  = 0.135*fc'/fy =   0.0090

      ( ). Check tensile reinforcement.
           -. Rho     =     0.0104
           -. Rho_min < Rho < Rho_max  --->  O.K !

      ( ). Check moment capacity.
```

```
        -. Pr_max     =      5288.391 tonf.
        -. Pr         =      -104.273 tonf.
        -. Mry        =       103.079 tonf-m.
        -. Mrz        =       938.295 tonf-m.
        -. Mr         =       943.940 tonf-m.
        -. Pu/Pr      =   0.439  --->  O.K !
        -. Muy/Mry    =   0.437  --->  O.K !
        -. Muz/Mrz    =   0.442  --->  O.K !
        -. Mu/Mr      =   0.442  --->  O.K !
        -. Ratio      =   0.442  --->  O.K !

================================================================================
   [[[*]]]   ANALYZE SHEAR CAPACITY OF BIAXIALLY LOADED RC-COLUMN.
================================================================================

    ( ). Compute shear parameter. ( LCB = 25, POS = I )
        -. phi    =   0.90
        -. Av     =     0.0004 m^2.
        -. Pu     =        448.83 tonf.
        -. Vuz    =         14.89 tonf.
        -. Muy    =        -14.54 tonf-m.
        -. bvz    =   2.00 m.
        -. dvz    =  MAX[ dvz calc, 0.9*dz, 0.72*Hc ] =   1.72 m.
        -. thetaz =  30.58 Deg.          [Clause 5.8.3.4.2]
        -. betaz  =   3.59
        -. Vuy    =         54.65 tonf.
        -. Muz    =       -533.97 tonf-m.
        -. bvy    =   2.00 m.
        -. dvy    =  MAX[ dvy calc, 0.9*dy, 0.72*Bc ] =   1.72 m.
        -. thetay =  30.58 Deg.          [Clause 5.8.3.4.2]
        -. betay  =   3.59

    ( ). Compute shear strength of concrete.
        -. Vuz       =       14.89 tonf.
        -. Vcz       = 0.0316*betaz*SQRT[fc']*bvz*dvz =      547.46 tonf.
        -. phiVcz    = phi * Vcz =      492.72 tonf.

        -. Vuy       =       54.65 tonf.
        -. Vcy       = 0.0316*betay*SQRT[fc']*bvy*dvy =      547.46 tonf.
        -. phiVcy    = phi * Vcy =      492.72 tonf.

    ( ). Compute maximum spacing of ties.
        -. Maximum spacing smax z  = MIN[ 0.8*dvz, 24 in ] =    0.610 m.
        -. Vuz < phiVcz/2  --->  Not required shear reinforcement.
        -. Applied spacing s_z   =    0.140 m.

        -. Maximum spacing smax y  = MIN[ 0.8*dvy, 24 in ] =    0.610 m.
        -. Vuy < phiVcy/2  --->  Not required shear reinforcement.
        -. Applied spacing s_y   =    0.140 m.

        -. Applied spacing s     = MIN[ s_y, s_z ] =    0.140 m.

    ( ). Compute shear strength of reinforcement.
        -. Vsz       = Av*fys*dvz*cot(thetaz) / s =      350.36 tonf.
        -. Vsz lim   = 0.25*fc'*bvz*dvz - Vcz =     1867.87 tonf.
        -. Vsz       = MIN[ Vsz, Vsz lim ] =      350.36 tonf.
        -. phiVsz    = phi*Vsz =      315.32 tonf.
        -. phiVsz > (Vuz-phiVcz)   --->  O.K !
        -. Using Avz/s  =  Vsz / (fys*dvz*cot(thetaz)) =   0.0029 m^2/m.

        -. Vsy       = Av*fys*dvy*cot(thetay) / s =      350.36 tonf.
        -. Vsy lim   = 0.25*fc'*bvy*dvy - Vcy =     1867.87 tonf.
        -. Vsy       = MIN[ Vsy, Vsy_lim ] =      350.36 tonf.
        -. phiVsy    = phi*Vsy =      315.32 tonf.
```

```
          -. phiVsy > (Vuy-phiVcy)   --->  O.K !
          -. Using Avy/s  =  Vsy / (fys*dvy*cot(thetay)) =   0.0029 m^2/m.

          -. Vu       =      56.645 tonf.
          -. Vr       =     808.039 tonf.
          -. Ratio    = Vu / Vr = 0.070   --->  O.K !

     ( ). Compute minimum longitudinal reinforcement by shear.
          -. phia     =   0.75
          -. phib     =   0.90
          -. phiv     =   0.90
          -. Vsly     =  MIN[ Vsy, Vuy/phiv ] =       62.94 tonf.
          -. Vslx     =  MIN[ Vsx, Vux/phiv ] =       62.94 tonf.
          -. Asy_req =   [ Mux/(phib*dvy)   - 0.5*Pu/phia + (Vuy/phiv - 0.5*Vsly)*cot(thetay) ] / fy
=   0.0024 m^2.
          -. Asx_req =   [ Muy/(phib*dvx)   - 0.5*Pu/phia + (Vux/phiv - 0.5*Vslx)*cot(thetax) ] / fy
=   0.0024 m^2.
          -. As_req  =  MAX[ Asy_req, Asx_req ] =   0.0024 m^2.
          -. As      =   0.0328 m^2.
          -. As_req < As   --->  O.K !
```

3.14. DISEÑO ESTRUCTURAL APOYO INTERMEDIO 1

3.14.1. DISEÑO ESTRUCTURAL COLUMNA RECTANGULAR

Para el diseño estructural de la columna rectangular, se plantea un modelo estructural desarrollado en el software MIDAS CIVIL 2022, que cumple con el diseño geométrico para determinar las solicitaciones tanto para momento flector, como para fuerza axial, cortante y torsión y así, determinar el refuerzo requerido que se dispondrá tanto en la sección transversal propuesta del estribo 1. En la siguiente figura está representado el modelo estructural que será empleado en el software para el diseño estructural del apoyo intermedio 1.

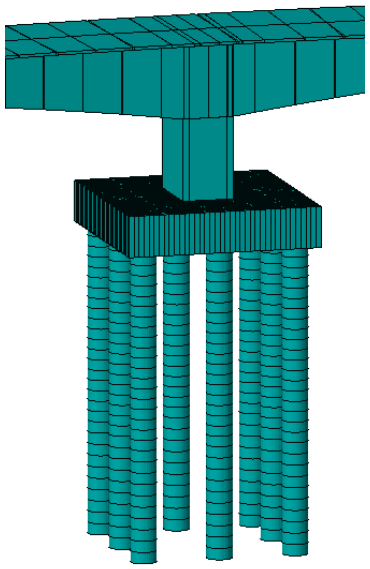


Figura 231. Modelo digital en software MIDAS CIVIL 2022

Para determinar las respectivas solicitaciones, se realiza el modelamiento estructural completo, integrando la superestructura y subestructura, a fin de poder transmitir las cargas de la superestructura a la columna rectangular, siendo estas zonas las que se verán más afectadas a momento flector tanto positivo como negativo, valores con los cuales se podrá determinar el refuerzo necesario para suplir estas solicitaciones.

Se realiza el diseño estructural y se aprecia las solicitaciones máximas para momento flector positivo y negativo, además de la fuerza cortante respectivamente, que está siendo sometida la viga cabezal de los estribos a las diferentes combinaciones de carga impuestas en el modelo estructural.

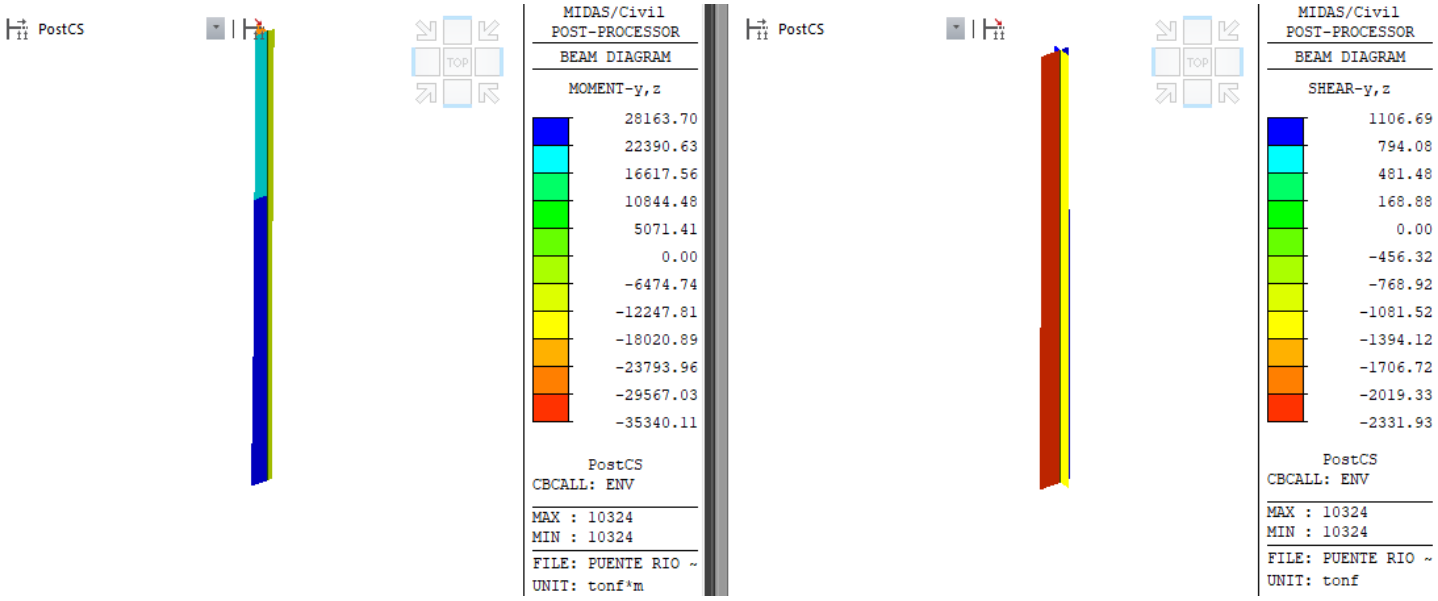


Figura 232. Máximo momento flector y fuerza cortante

SOLICITACIONES DE DISEÑO ESTRUCTURAL

$$M_U{}^{(+)} = 28163.70\ ton.m \qquad V_U{}^{(+)} = 1106.69\ ton$$
$$M_U{}^{(-)} = -35340.11\ ton.m \qquad V_U{}^{(-)} = -2331.93\ ton$$

MEMB / SECT	SEL	Section / t	Bc / a	Hc / b	fc / Height	fy / fys	CHK	LCB	POS	Ast / Rho	Pr-max	Pu / Mc	Pr / Mr	Rat-P / Rat-M	LCB	Asvy / Asvz	Vu / Rat-V
5059	☐	COLUMN	7.100	5.000	4588.72	42184.2	OK	1+	J	0.2589	36557.8	10248.4	15879.9	0.645	19-	0.0001	2287.40
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		30044.4	46893.9	0.650		0.0001	0.787
10318	☐	COLUMN	7.100	5.000	4588.72	42184.2	OK	1+	J	0.2589	36557.8	10287.7	15906.3	0.647	19-	0.0001	2294.11
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		29997.2	46775.6	0.652		0.0001	0.789
10319	☐	COLUMN	7.100	5.000	4588.72	42184.2	OK	1+	J	0.2589	36557.8	10327.0	15933.1	0.648	19-	0.0001	2300.70
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		29953.5	46650.4	0.653		0.0001	0.791
10320	☐	COLUMN	7.100	5.000	4588.72	42184.2	OK	1+	J	0.2589	36557.8	10366.2	15962.6	0.649	19-	0.0001	2307.18
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		29912.7	46526.5	0.655		0.0001	0.793
10321	☐	COLUMN	7.100	5.000	4588.72	42184.2	OK	1+	J	0.2589	36557.8	10405.5	16125.7	0.645	19-	0.0001	2313.53
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		29874.1	46304.4	0.656		0.0001	0.795
10322	☐	COLUMN	7.100	5.000	4588.72	42184.2	OK	1+	J	0.2589	36557.8	10444.8	16161.6	0.646	19-	0.0001	2319.77
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		29838.1	46182.0	0.657		0.0001	0.796
10323	☐	COLUMN	7.100	5.000	4588.72	42184.2	OK	1+	J	0.2589	36557.8	10484.1	16203.5	0.647	19-	0.0001	2325.90
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		29802.9	46068.9	0.658		0.0001	0.798
10324	☐	COLUMN	7.100	5.000	4588.72	42184.2	OK	11+	J	0.2589	36557.8	10565.1	15936.0	0.663	19-	0.0001	2331.93
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		31171.1	47393.5	0.661		0.0001	0.800

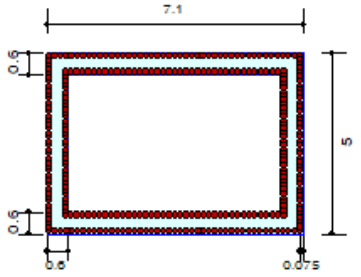
1. Design Condition

Design Code	AASHTO-LRFD16
Unit System	tonf, m
Member Number	10324 (PM), 10324 (Shear)
Material Data	fc = 4588.72, fy = 42184.2, fys = 42184.2 tonf/m^2
Column Height	1 m
Section Property	COLUMNA 7.1X5.0 (No : 79)

Rebar Pattern

	Pos 1	Pos 2	Pos 3
Layer 1	45-#10	40-#10	--
Layer 2	39-#10	34-#10	--

Total Rebar Area Ast = 0.258916 m^2 (Rhost = 0.0198)



2. Applied Loads

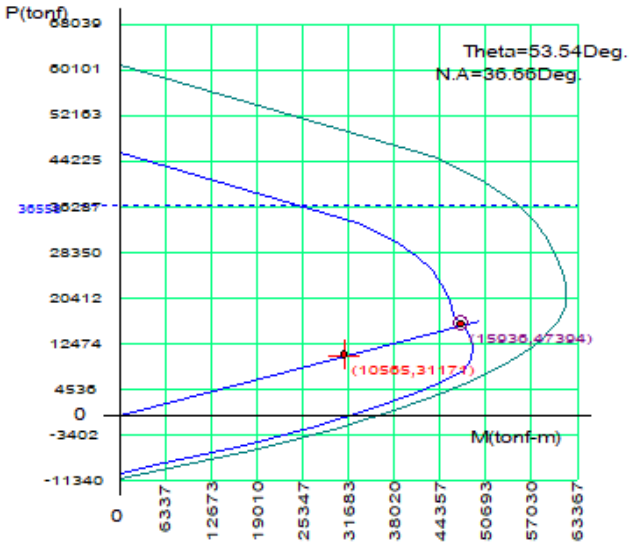
Load Combination 11+ AT (J) Point

Pu = 10565.1 tonf, Mcy = 18615.7, Mcz = -2.5e+04, Mc = 31171.1 tonf-m

3. Axial Forces and Moments Capacity Check

Concentric Max. Axial Load	Pr-max	= 36557.8 tonf	
Axial Load Ratio	Pu/Pr	= 10565.1 / 15936.0	= 0.663 < 1.000 ..... O.K
Moment Ratio	Mcy/Mry	= 18615.7 / 28165.6	= 0.661 < 1.000 ..... O.K
	Mcz/Mrz	= -2.5e+04 / 38116.2	= 0.656 < 1.000 ..... O.K
	Mc/Mr	= 31171.1 / 47393.5	= 0.658 < 1.000 ..... O.K

4. P-M Interaction Diagram



Pr(tonf)	Mr(tonf-m)
45697.27	0.00
30554.28	37995.96
25244.16	43362.81
20708.95	45464.81
17238.23	46348.39
14997.24	48007.57
13092.24	48782.14
11257.29	48928.97
9117.43	48409.63
5894.97	44537.02
1234.61	34887.44
-5017.65	17344.47
-9829.93	0.00

5. Shear Force Capacity Check

Applied Shear Force	Vuy	= 884.408,	Vuz	= 2331.93 tonf (Load Combination 19-)
Shear Strength (Y-Direction)	PhiVy	= 1601.01,	PhiVsy	= 2356.95 tonf (4.0-#5 @0)
Shear Strength (Z-Direction)	PhiVz	= 1237.06,	PhiVsz	= 1678.68 tonf (4.0-#5 @0)
Shear Ratio	Vu/Vr	= 2331.93 / 2915.74		= 0.800 < 1.000 ..... O.K

MIDAS/Civil — RC Column Checking Result

PROJECT TITLE :

	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	puente rio sinú COMPLETO geotecnia.rcs

```
==========================================================================================
 MIDAS/Civil - RC-Column Checking [ AASHTO-LRFD16 ]                          Civil 2022
==========================================================================================
      *.MIDAS/Civil - RC-COLUMN  Analysis/Design Program.
      *.DESIGN CODE    : AASHTO-LRFD16,     *.UNIT SYSTEM : tonf, m
      *.MEMBER         : Member Type = COLUMN ,  MEMB = 10324,  LCB =  11+,  POS =    J

      *.DESCRIPTION OF COLUMN  DATA (iSEC =    79) : COLUMNA 7.1X5.0
        Column Height (L)                =         1.000 m.

        Section Type : BOX (B)
        Section Depth  (Hc)              =         5.000 m.
        Section Width  (Bc)              =         7.100 m.
        Section Thick  (t)               =         0.600 m.
        Concrete Strength (fc)           =      4588.723 tonf/m^2.
        Main Rebar Strength (fy)         =     42184.188 tonf/m^2.
        Ties/Spirals Strength (fys)      =     42184.188 tonf/m^2.
        Modulus of Elasticity (Es)       =  20389024.157 tonf/m^2.

      *.REINFORCEMENT PATTERN :
        ------------------------------------------------------------------------------------
          Layer         Pos1    As( m^2.)         Pos2    As( m^2.)         Pos3    As( m^2.)
        ------------------------------------------------------------------------------------
            1         45-#10 (    0.0369)       40-#10 (    0.0328)           -
            2         39-#10 (    0.0320)       34-#10 (    0.0279)           -
        ------------------------------------------------------------------------------------

      *.Ties :  4.0-#5  @0.15

 ============================================================================
    [[[*]]]   CALCULATE SLENDERNESS RATIOS, MAGNIFIED FORCES/MOMENTS.
 ============================================================================
      ( ). Factored forces/moments caused by unit load case.  Unit : tonf., m.
           *.Load combination ID =  11+
           ---------------------------------------------------------------------------
            Load Case          Pu max          Myi          Myj          Msi          Msj
           ---------------------------------------------------------------------------
            DL               9862.81     10337.76     11080.93    -21353.32    -21219.29
            LL                -15.36      6370.63      6167.41       851.90       885.82
            DL+LL            9847.45     16708.39     17248.34    -20501.42    -20333.48
            Others            717.60      1687.22      1367.39     -4502.56     -4668.36
           ---------------------------------------------------------------------------
            DL+LL+Others    10565.05     18395.61     18615.73    -25003.99    -25001.84
           ---------------------------------------------------------------------------

      ( ). Check slenderness ratios of BRACED/UNBRACED frame.
           -. End Moments (Myl) =    18395.61 tonf-m.
           -. End Moments (My2) =    18615.73 tonf-m.
           -. Slenderness ratio limits.
              SRy(Unbraced) =  22.00
           -. Radii of gyration (Roy)       =     1.895 m.
           -. Unbraced lengths  (Ly)        =     1.000 m.
           -. Effective length factors (Ky) =     1.000
           -. SLENy  =  Ky*Ly/Roy  =     0.528  <  SRy  --->  NOT SLENDER.

      ( ). Compute moment magnification factors for major axis(DBy,DSy).
           -. DBy    =     1.00 (Default value)

      ( ). Compute magnified moments.
           -. No sidesway moments.
              QMb_y  =  My_G   =    17248.34 tonf-m.
           -. Sidesway moments.
              QMs_y  =  My_S   =     1367.39 tonf-m.
           -. Compute magnified moments.
```

```
            Mcy              =  DBy*QMb_y + DSy*QMs_y             =    18615.73 tonf-m.

     ( ). Check slenderness ratios of BRACED/UNBRACED frame.
          -. End Moments (Ms1) =    25001.84 tonf-m.
          -. End Moments (Ms2) =    25003.99 tonf-m.
          -. Slenderness ratio limits.
             SRs(Unbraced) =  22.00
          -. Radii of gyration (Ros)        =    2.536 m.
          -. Unbraced lengths  (Ls)         =    1.000 m.
          -. Effective length factors (Ks)  =    1.000
          -. SLENs  =  Ks*Ls/Ros  =    0.394  <  SRs  --->  NOT SLENDER.

     ( ). Compute moment magnification factors for minor axis(DBs,DSs).
          -. DBs    =     1.00 (Default value)

     ( ). Compute magnified moments.
          -. No sidesway moments.
             QMb_s  =  Ms_G   =   -20333.48 tonf-m.
          -. Sidesway moments.
             QMs_s  =  Ms_S   =    -4668.36 tonf-m.
          -. Compute magnified moments.
             Mcs              =  DBs*QMb_s + DSs*QMs_s             =   -25001.84 tonf-m.

================================================================================
   [[[*]]]   ANALYZE CAPACITY OF BIAXIALLY LOADED RC-COLUMN.
================================================================================
     ( ). Compute design parameter.
          -. phi   =   0.90
          -. Alpha =   0.85
          -. Beta  =   0.72
          -. ecu   =  0.0030

     ( ). Compute maximum and minimum reinforcement.
          -. Rhomax1 =   0.0300 (User Input)
          -. Rhomax  = MIN[ Rhomax1, 0.08 ] =   0.0300
          -. Rhomin  = 0.135*fc'/fy =   0.0147

     ( ). Check tensile reinforcement.
          -. Rho     =     0.0198
          -. Rho_min < Rho < Rho_max  --->  O.K !

     ( ). Check moment capacity.
          -. Pr max     =     36557.813 tonf.
          -. Pr         =     15935.966 tonf.
          -. Mry        =     28165.593 tonf-m.
          -. Mrs        =     38116.216 tonf-m.
          -. Mr         =     47393.529 tonf-m.
          -. Pu/Pr      =   0.663  --->  O.K !
          -. Muy/Mry    =   0.661  --->  O.K !
          -. Mus/Mrs    =   0.656  --->  O.K !
          -. Mu/Mr      =   0.663  --->  O.K !
          -. Ratio      =   0.663  --->  O.K !

================================================================================
   [[[*]]]   ANALYZE SHEAR CAPACITY OF BIAXIALLY LOADED RC-COLUMN.
================================================================================
     ( ). Compute shear parameter. ( LCB = 19, POS = I )
          -. phi    =   0.90
          -. Av     =     0.0008 m^2.
          -. Pu     =       8497.59 tonf.
          -. Vus    =       2331.93 tonf.
          -. Muy    =      -7534.18 tonf-m.
          -. bvs    =   1.20 m.
          -. dvs    =  MAX[ dvs_calc, 0.9*ds, 0.72*Hc ] =   4.70 m.
```

MIDAS/Civil — RC Column Checking Result

PROJECT TITLE :

MIDAS	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	puente rio sinú COMPLETO geotecnia.rcs

```
          -. thetaz =  29.55 Deg.          [Clause 5.8.3.4.2]
          -. betaz  =   4.29

          -. Vuy    =        884.41 tonf.
          -. Muz    =     -28392.33 tonf-m.
          -. bvy    =   1.20 m.
          -. dvy    =  MAX[ dvy_calc, 0.9*dy, 0.72*Bc ] =   6.75 m.
          -. thetay =  30.13 Deg.          [Clause 5.8.3.4.2]
          -. betay  =   3.87

     ( ). Compute shear strength of concrete.
          -. Vuz       =     2331.93 tonf.
          -. Vcz       = 0.0316*betaz*SQRT[fc']*bvz*dvz =     1374.51 tonf.
          -. phiVcz    = phi * Vcz =     1237.06 tonf.

          -. Vuy       =      884.41 tonf.
          -. Vcy       = 0.0316*betay*SQRT[fc']*bvy*dvy =     1778.89 tonf.
          -. phiVcy    = phi * Vcy =     1601.01 tonf.

     ( ). Compute maximum spacing of ties.
          -. Maximum spacing smax z  = MIN[ 0.8*dvz, 24 in ] =    0.610 m.
          -. Vuz > phiVcz  --->  Required shear reinforcement.
          -.                 smax z  = MIN[ smax_z, Av/(0.0316*SQRT(fc')*bvz/fys) ] =    0.495 m.
          -. Applied spacing s_z   =    0.150 m.

          -. Maximum spacing smax y  = MIN[ 0.8*dvy, 24 in ] =    0.610 m.
          -. phiVcy/2 < Vuy < phiVcy  --->  Required minimum shear reinforcement.
          -.                 smax y  = MIN[ smax_y, Av/(0.0316*SQRT(fc')*bvy/fys) ] =    0.495 m.
          -. Applied spacing s_y   =    0.150 m.

          -. Applied spacing s     = MIN[ s_y, s_z ] =    0.150 m.

     ( ). Compute shear strength of reinforcement.
          -. Vsz       = Av*fys*dvz*cot(thetaz) / s =     1865.20 tonf.
          -. Vsz_lim   = 0.25*fc'*bvz*dvz - Vcz =     5095.77 tonf.
          -. Vsz       = MIN[ Vsz, Vsz lim ] =     1865.20 tonf.
          -. phiVsz    = phi*Vsz =     1678.68 tonf.
          -. phiVsz > (Vuz-phiVcz)  --->  O.K !
          -. Using Avz/s  =  Vsz / (fys*dvz*cot(thetaz)) =   0.0053 m^2/m.

          -. Vsy       = Av*fys*dvy*cot(thetay) / s =     2618.84 tonf.
          -. Vsy lim   = 0.25*fc'*bvy*dvy - Vcy =     7519.90 tonf.
          -. Vsy       = MIN[ Vsy, Vsy lim ] =     2618.84 tonf.
          -. phiVsy    = phi*Vsy =     2356.95 tonf.
          -. phiVsy > (Vuy-phiVcy)  --->  O.K !
          -. Using Avy/s  =  Vsy / (fys*dvy*cot(thetay)) =   0.0053 m^2/m.

          -. Vu        =    2331.927 tonf.
          -. Vr        =    2915.738 tonf.
          -. Ratio     = Vu / Vr = 0.800  --->  O.K !

     ( ). Compute minimum longitudinal reinforcement by shear.
          -. phia      =   0.75
          -. phib      =   0.90
          -. phiv      =   0.90
          -. Vsly      =  MIN[ Vsy, Vuy/phiv ] =      982.68 tonf.
          -. Vslz      =  MIN[ Vsz, Vuz/phiv ] =     1865.20 tonf.
          -. Asy_req =  [ Muz/(phib*dvy)  - 0.5*Pu/phia + (Vuy/phiv - 0.5*Vsly)*cot(thetay) ] / fy
=  -0.0035 m^2.
          -. Asz_req =  [ Muy/(phib*dvz)  - 0.5*Pu/phia + (Vuz/phiv - 0.5*Vslz)*cot(thetaz) ] / fy
=  -0.0227 m^2.
          -. As req  =  MAX[ Asy req, Asz_req ] =  -0.0035 m^2.
          -. As      =   0.2589 m^2.
          -. As_req < As  --->  O.K !
```

3.14.2. **DISEÑO ENCEPADO APOYO INTERMEDIO 1 - H=4.0m**

Para el diseño estructural de los encepados cuadrados de concreto reforzado, se hace necesario utilizar el software MIDAS CIVIL y determinar el refuerzo necesario para suplir todas las solicitaciones impuestas por la transmisión de cargas provenientes de la superestructura a los Caissons de cimentación y que se encuentran generadas por el modelo.

El encepado del apoyo intermedio 1 corresponde a la geometría cuadrada; Se dispone de una zapata rectangular de 21.0 m de ancho por 21.0 m de largo y 3.50 m de espesor, el cual se apoya directamente sobre 16 caisson de 2.0 m de diámetro.

En la siguiente figura está representado el modelo estructural que será empleado en el software para el encepado.

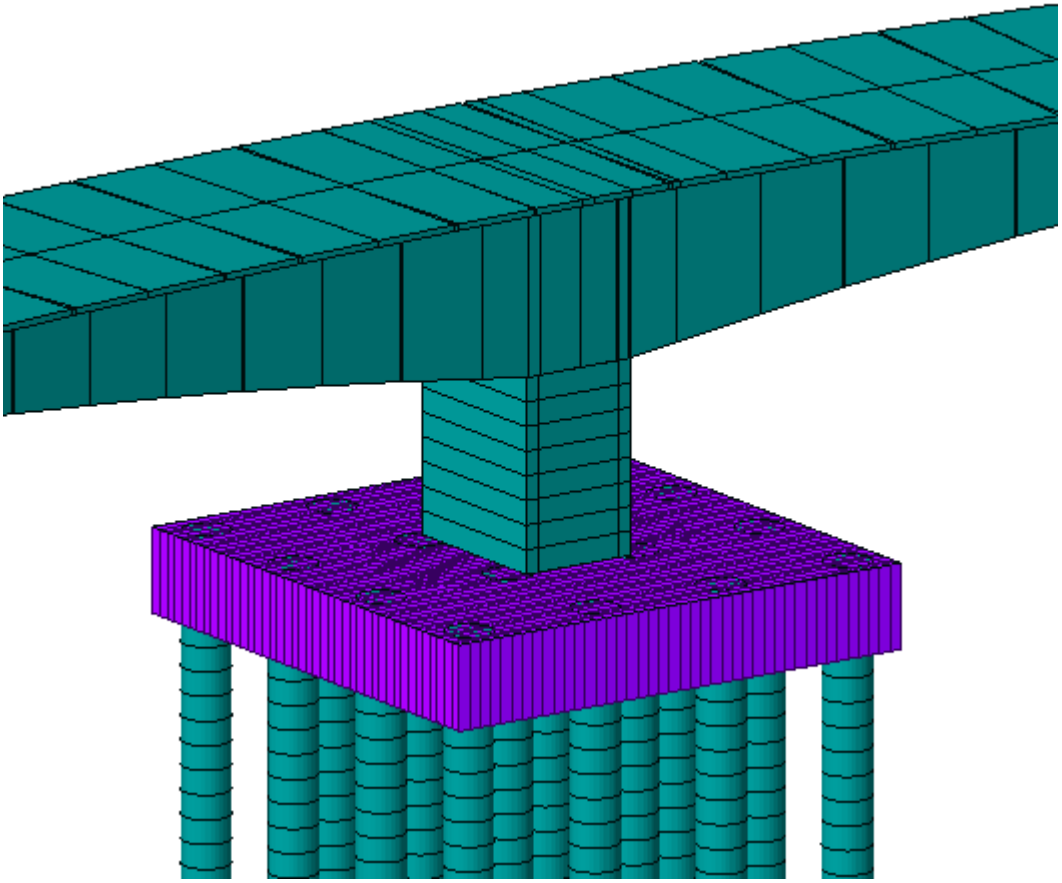


Figura 233. Modelo digital en software MIDAS CIVIL

Se realiza el diseño estructural y se aprecia las solicitaciones máximas para momento flector positivo y negativo, además de la fuerza cortante respectivamente, que está siendo sometida

el encepado del apoyo intermedio 1 a las diferentes combinaciones de carga impuestas en el modelo estructural, conforme la siguiente figura.

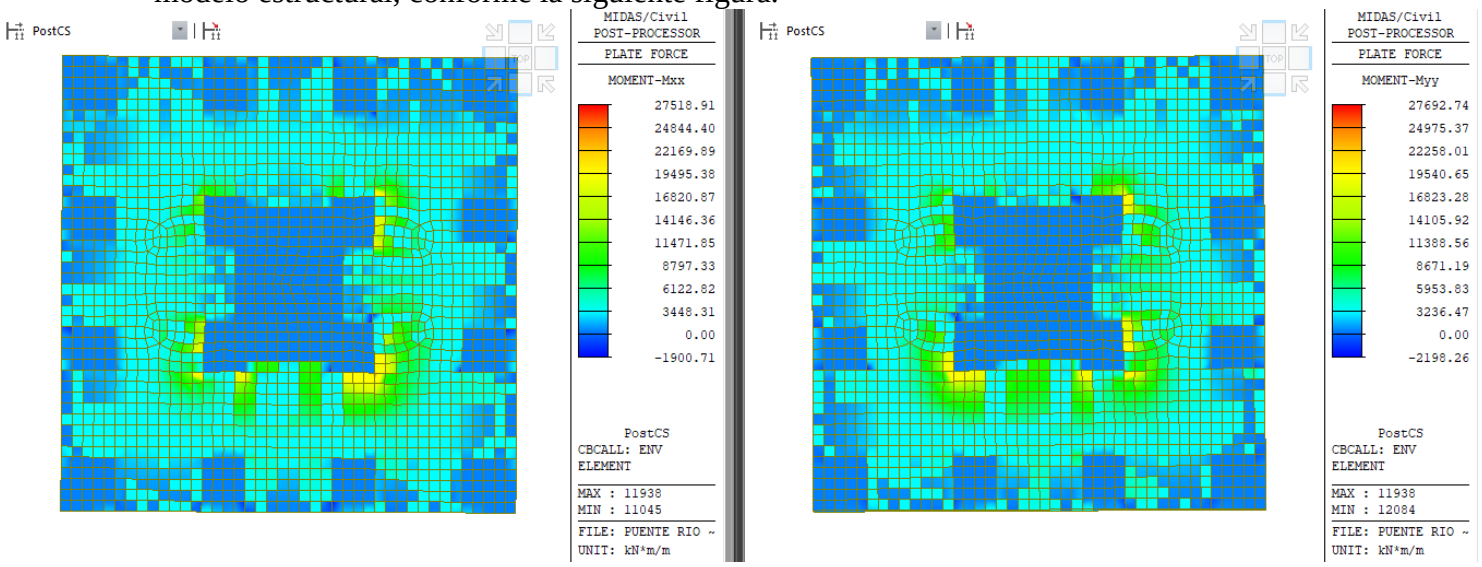


Figura 234. Máximo momento X y momento Y - Apoyo intermedio 1

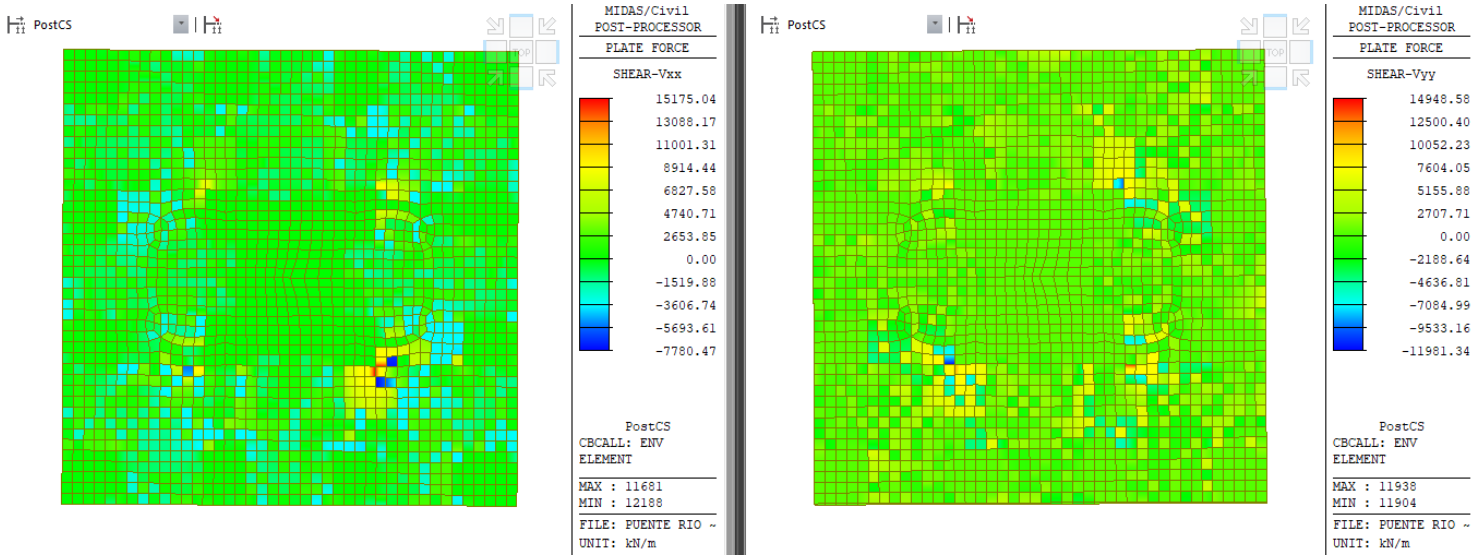


Figura 235. Máxima fuerza cortante X y cortante Y - Apoyo intermedio 1

Se define en dos direcciones principales:

- La dirección 1 en el sentido longitudinal del puente, es decir, el eje cartesiano X
- La dirección 2 en el sentido transversal del puente, es decir, el eje cartesiano Y

DIRECCIÓN 1 – SENTIDO LONGITUDINAL AL PUENTE, EJE CARTESIANO X

1. Design Condition

Design Type : Plate Beam (1D)
Sub-Domain : z1
Design Code : AASHTO-LRFD16
Unit System : kN, m, / m
Material Data : fc = 27579, fy = 413686, fys = 413686 KPa
Thickness : 3.5 m

2. Section Diagram



Element No : 11938

	Top(Negative)	Bottom(Positive)
Layer 1	#10@0.10	#10@0.10
Layer 2	--	#10@0.10
Layer 3	--	#10@0.10
Layer 4	--	#10@0.10

3. Bending Moment Capacity

	Top(Negative)	Bottom(Positive)
Mu	0.00	27518.91
Element No.	11938	11938
Load Combination	cLCB1	cLCB20
Mr	10108.60	35125.29
Check Ratio (Mu/Mr)	0.0000	0.7835
Using Rebar(As)	0.0082	0.0307

4. Shear Capacity

Element No.	11938		
Load Combination	cLCB11		
Applied Shear Force	Vu	= 15095.3	
Shear Strength (Out of plane)	Vr	= 19093.1	
Shear Ratio	Vu/Vr	= 15095.3 / 19093.1	= 0.791 < 1.000 ..... O.K

5. Crack Spacing Resistance Check

	Top(Negative)	Bottom(Positive)
Element No.	11938	11938
Load Combination	cLCB42	cLCB29
Crack Width(w)	0.1000	0.0800
Allowable Crack Width(wa)	0.4572	0.3087
Check Ratio	0.2187	0.2592

MIDAS/Civil — RC Beam Design Result

PROJECT TITLE:

MIDAS	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	rio sinú COMPLETO geotecnia hzapata=

```
================================================================================
 MIDAS/Civil - RC-Plate Beam Checking  [ AASHTO-LRFD16 ]                Civil 2022
================================================================================
      *.MIDAS/Civil - RC- PLATE BEAM Analysis/Design Program.
      *.DESIGN CODE    : AASHTO-LRFD16,     *.UNIT SYSTEM : kN, m, /m
      *.SUB-DOMAIN        : zl     Member Type = PLATE BEAM(1D),   Dir  = 1   )

      *.DESCRIPTION OF PLATE BEAM DATA :(Elem : 11938)
        Thickness              =          3.500 m.
        Unit Width             =              1 m.
        Concrete Strength (fc)         =     27579.038 KPa.
        Main Rebar Strength (fy)       =    413685.566 KPa.
        Stirrups Strength (fys)        =    413685.566 KPa.
        Modulus of Elasticity (Es)     = 199948023.746 KPa.

       < Selected Elements >
         11938
       < Positive Bending Moment >
          P-Mu =    27518.91 kN-m/m., ELEM = 11938,   LCB =  20+FX,   NODE =  18506
       < Negative Bending Moment >
          N-Mu =    0.00e+00 kN-m/m., ELEM = 11938,   LCB =   1+FX,   NODE = Center
       < Shear Force >
          Vu   =    15095.34 kN/m.  , ELEM = 11938,   LCB =  11+FX,   NODE =  18691


                  *.REINFORCEMENT PATTERN :
                    ---------------------------------------------------------
                       Location    i      di( m.)       Rebar  Asi( m^2/m.)
                    ---------------------------------------------------------
                       Top         1       0.032      #10@0.10       0.00819
                       Bottom      1       0.032      #10@0.10       0.00819
                       Bottom      2       0.032      #10@0.10       0.00819
                       Bottom      3       0.032      #10@0.10       0.00819
                       Bottom      4       0.032      #10@0.10       0.00819
                    ---------------------------------------------------------
================================================================================
   [[[*]]]   ANALYZE POSITIVE BENDING MOMENT CAPACITY.
================================================================================
      ( ). Compute parameter.
           -. phi   =   0.90
           -. Alpha =   0.85
           -. Beta  =   0.85
           -. d     =     3.3500 m.
           -. ecu   =  0.0030

      ( ). Compute maximum and minimum reinforcement.
           -. Rhomin1 = (1.2)*Mcr/[ phi*fy*b*d*(d-a/2) ]      =   0.0020
           -. Rhomin2 = 1.33*Mu/[ phi*fy*b*d*(d-a/2) ]        =   0.0096
           -. Rhomin  = MIN[ Rhomin1, Rhomin2 ]               =   0.0020
           -. As_min =  Rhomin * Ag  =     0.0069 m^2/m.

      ( ). Check tensile reinforcement.
           -. As     =     0.0307 m^2/m.
           -. As_min < As    --->  O.K !

      ( ). Check moment capacity.
           -. c          =     0.6335 m.
           -. Cc         =   12622.30 kN/m.
           -. Ts         =   12710.80 kN/m.
           -. Mr         =   35125.29 kN-m/m.
           -. Mu/Mr      =      0.783  --->  O.K !

================================================================================
   [[[*]]]   ANALYZE NEGATIVE BENDING MOMENT CAPACITY.
================================================================================
```

MIDAS/Civil — RC Beam Design Result

PROJECT TITLE:

	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	rio sinú COMPLETO geotecnia hzapata

```
     ( ). Compute parameter.
          -. phi    =   0.90
          -. Alpha  =   0.85
          -. Beta   =   0.85
          -. d      =     3.4000 m.
          -. ecu    =  0.0030

     ( ). Compute maximum and minimum reinforcement.
          -. Rhomin1 = (1.2)*Mcr/[ phi*fy*b*d*(d-a/2) ]      =    0.0019
          -. Rhomin2 = 1.33*Mu/[ phi*fy*b*d*(d-a/2) ]        =0.0000e+00
          -. Rhomin  = MIN[ Rhomin1, Rhomin2 ]               =0.0000e+00
          -. As_min =  Rhomin * Ag  =  0.0000e+00 m^2/m.

     ( ). Check tensile reinforcement.
          -. As     =     0.0082 m^2/m.
          -. As_min < As   --->  O.K !

     ( ). Check moment capacity.
          -. c          =     0.1686 m.
          -. Cc         =    3359.89 kN/m.
          -. Ts         =    3389.55 kN/m.
          -. Mr         =   10108.60 kN-m/m.
          -. Mu/Mr      =  0.000e+00  --->  O.K !

================================================================
   [[[*]]]   ANALYZE SHEAR CAPACITY.
================================================================
     ( ). Compute shear parameter.
          -. phi    =   0.90
          -. Av     =   0.0066 m^2/m.
          -. bv     =   0.03 m.
          -. dv     =   3.0769 m.  (for Beta Calculation)
          -. theta  =   0.709 Rad.
          -. Epsilon_s = MIN[ (|Mu|/dv + 0.5*Nu + |Vu|)/(Es*As), 0.006 ] =  0.0033
          -. beta   =  4.8/(1+750*Epsilon_s) =  1.3742

     ( ). Compute shear strength of concrete.
          -. dv     =  MAX[ dv, 0.9*d, 0.72*Hc ]  =     3.08 m.
          -. Vu     =1.51e+04 kN/m.
          -. Vc     = 0.0316*beta*SQRT[fc']*bv*dv =1842.43 kN/m.
          -. phiVc  = phi * Vc =1658.19 kN/m.
          -. Vn_lim = 0.25*fc'*bv*dv =2.12e+04 kN/m.

     ( ). Compute stirrup spacing.
          -. Maximum spacing smax   = MIN[ 0.4*dv, 12 in ] =    0.305 m.
          -. Vu > phiVc  --->  Required shear reinforcement.
          -. Applied spacing s    =    0.200 m.

     ( ). Compute shear strength of reinforcement.
          -. Vs      = Av*fys*dv*cot(theta) / s = 48664.57 kN/m.
          -. Vs lim  = 0.25*fc'*bv*dv - Vc = 19372.08 kN/m.
          -. Vs      = MIN[ Vs, Vs lim ] = 19372.08 kN/m.
          -. phiVs   = phi*Vs = 17434.87 kN/m.

          -. phiVs > (Vu-phiVc)  --->  O.K !
          -. Using Av  =  Vs / (fys*d)  =   0.0003 m^2/m.

================================================================
   [[[*]]]   ANALYZE CRACK.
================================================================
     ( ). Check crack control of Bottom reinforcement. ( LCB =   29+EX )
          -. Mu     =  21105.6099 kN-m/m.
          -. h      =      3.5000 m.
          -. dc     =      0.1000 m.
```

MIDAS/Civil — RC Beam Design Result

PROJECT TITLE:

MIDAS	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	rio sinú COMPLETO geotecnia hzapata=

```
      -. Beta s  = 1+dc/(0.7*(h-dc)) = 1.04
      -. Gamma e = 1.00
      -. c cr    =      1.0186 m.
      -. Icr     =      1.7314 m^4/m.
      -. fss     =    231275.428 KPa.
      -. s       =       0.0800 m.
      -. sa      = 700*Gamma_e/(Beta_s*fss)-2*dc =      0.3087 m.
      -. s < sa --->  O.K !

   ( ). Check crack control of Bottom reinforcement requirements.
      -. As req                =      0.0006 m^2/m.
      -. As(Outermost laver) =      0.0102 m^2/m.
      -. As req < As --->  O.K !
      -. Max. spacing smax   =      0.4572 m.
      -. s(Outermost laver)  =      0.0800 m.
      -. s < smax --->  O.K !

   ( ). Check crack control of Top reinforcement. ( LCB =   42 )
      -. Mu      =     36.9430 kN-m/m.
      -. h       =      3.5000 m.
      -. dc      =      0.1000 m.
      -. Beta s  = 1+dc/(0.7*(h-dc)) = 1.04
      -. Gamma e = 1.00
      -. c cr    =      0.4862 m.
      -. Icr     =      0.6171 m^4/m.
      -. fss     =      1389.716 KPa.
      -. s       =       0.1000 m.
      -. sa      = 700*Gamma_e/(Beta_s*fss)-2*dc =      0.4572 m.
      -. s < sa --->  O.K !

   ( ). Check crack control of Top reinforcement requirements.
      -. As req                =      0.0004 m^2/m.
      -. As(Outermost laver) =      0.0082 m^2/m.
      -. As req < As --->  O.K !
      -. Max. spacing smax   =      0.4572 m.
      -. s(Outermost laver)  =      0.1000 m.
      -. s < smax --->  O.K !

================================================================
   [[[*]]]   SHRINKAGE AND TEMPERATURE REINFORCEMENT.
================================================================
               *.REINFORCEMENT PATTERN :
                 ---------------------------------------------------
                    Location    i     di( m.)       Rebar  Asi( m^2/m.)
                 ---------------------------------------------------
                    Top         1      0.032     #10@0.10       0.00819
                    Bottom      1      0.032     #10@0.10       0.00819
                    Bottom      2      0.032     #10@0.10       0.00819
                    Bottom      3      0.032     #10@0.10       0.00819
                    Bottom      4      0.032     #10@0.10       0.00819
                 ---------------------------------------------------

   ( ). Check Bottom Shrinkage and Temperature Reinforcement.
      -. As Req   = 1.3*b*h/(2*(b+h)*fys) =      0.0003 m^2/m.
      -. As       =      0.0307 m^2/m.
      -. As_Req < As --->  O.K !

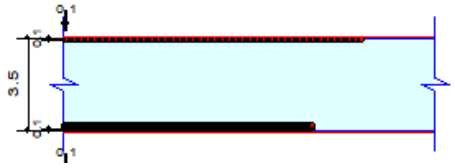
   ( ). Check Top Shrinkage and Temperature Reinforcement.
      -. As Req   = 1.3*b*h/(2*(b+h)*fys) =      0.0003 m^2/m.
      -. As       =      0.0246 m^2/m.
      -. As_Req < As --->  O.K !
```

DIRECCIÓN 2 – SENTIDO TRANSVERSAL AL PUENTE, EJE CARTESIANO Y

1. Design Condition

Design Type : Plate Beam (1D)
Sub-Domain : z1
Design Code : AASHTO-LRFD16
Unit System : kN, m, / m
Material Data : fc = 27579, fy = 413686, fys = 413686 KPa
Thickness : 3.5 m

2. Section Diagram



Element No : 11938

	Top(Negative)	Bottom(Positive)
Layer 1	#10@0.10	#10@0.10
Layer 2	--	#10@0.10
Layer 3	--	#10@0.10
Layer 4	--	#10@0.10

3. Bending Moment Capacity

	Top(Negative)	Bottom(Positive)
Mu	0.00	27692.74
Element No.	11938	11938
Load Combination	cLCB1	cLCB11
Mr	10108.60	35125.29
Check Ratio (Mu/Mr)	0.0000	0.7884
Using Rebar(As)	0.0082	0.0307

4. Shear Capacity

Element No.	11938		
Load Combination	cLCB11		
Applied Shear Force	Vu	= 14948.6	
Shear Strength (Out of plane)	Vr	= 19093.1	
Shear Ratio	Vu/Vr	= 14948.6 / 19093.1	= 0.783 < 1.000 ..... O.K

5. Crack Spacing Resistance Check

	Top(Negative)	Bottom(Positive)
Element No.	11938	11938
Load Combination	cLCB42	cLCB29
Crack Width(w)	0.1000	0.0800
Allowable Crack Width(wa)	0.4572	0.3099
Check Ratio	0.2187	0.2581

MIDAS/Civil — RC Beam Design Result

PROJECT TITLE:

MIDAS	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	rio sinú COMPLETO geotecnia hzapata

```
================================================================================
MIDAS/Civil - RC-Plate Beam Checking [ AASHTO-LRFD16 ]                  Civil 2022
================================================================================

*.MIDAS/Civil - RC- PLATE BEAM Analysis/Design Program.
*.DESIGN CODE    : AASHTO-LRFD16.     *.UNIT SYSTEM : kN. m. /m
*.SUB-DOMAIN        : zl     Member Type = PLATE BEAM(1D),   Dir  = 2  )

*.DESCRIPTION OF PLATE BEAM DATA :(Elem : 11938)
  Thickness              =          3.500 m.
  Unit Width             =              1 m.
  Concrete Strength (fc)         =     27579.038 KPa.
  Main Rebar Strength (fy)       =    413685.566 KPa.
  Stirrups Strength (fys)        =    413685.566 KPa.
  Modulus of Elasticity (Es)     = 199948023.746 KPa.

 < Selected Elements >
   11938
 < Positive Bending Moment >
    P-Mu =    27692.74 kN-m/m., ELEM = 11938,   LCB =  11+FX,   NODE =  18506
 < Negative Bending Moment >
    N-Mu =    0.00e+00 kN-m/m., ELEM = 11938,   LCB =   1+FX,   NODE = Center
 < Shear Force >
    Vu   =    14948.58 kN/m.  , ELEM = 11938,   LCB =  11+FX,   NODE =  18706


*.REINFORCEMENT PATTERN :
  ------------------------------------------------------------
     Location     i      di( m.)        Rebar  Asi( m^2/m.)
  ------------------------------------------------------------
     Top          1       0.032      #10@0.10       0.00819
     Bottom       1       0.032      #10@0.10       0.00819
     Bottom       2       0.032      #10@0.10       0.00819
     Bottom       3       0.032      #10@0.10       0.00819
     Bottom       4       0.032      #10@0.10       0.00819
  ------------------------------------------------------------
```

```
================================================================
   [[[*]]]   ANALYZE POSITIVE BENDING MOMENT CAPACITY.
================================================================
     ( ). Compute parameter.
          -. phi   =   0.90
          -. Alpha =   0.85
          -. Beta  =   0.85
          -. d     =     3.3500 m.
          -. ecu   =  0.0030

     ( ). Compute maximum and minimum reinforcement.
          -. Rhomin1 = (1.2)*Mcr/[ phi*fy*b*d*(d-a/2) ]      =   0.0020
          -. Rhomin2 = 1.33*Mu/[ phi*fy*b*d*(d-a/2) ]        =   0.0096
          -. Rhomin  = MIN[ Rhomin1, Rhomin2 ]               =   0.0020
          -. As_min =  Rhomin * Ag  =     0.0069 m^2/m.

     ( ). Check tensile reinforcement.
          -. As     =     0.0307 m^2/m.
          -. As_min < As   --->  O.K !

     ( ). Check moment capacity.
          -. c          =     0.6335 m.
          -. Cc         =   12622.30 kN/m.
          -. Ts         =   12710.80 kN/m.
          -. Mr         =   35125.29 kN-m/m.
          -. Mu/Mr      =      0.788  --->  O.K !

================================================================
   [[[*]]]   ANALYZE NEGATIVE BENDING MOMENT CAPACITY.
```

MIDAS/Civil — RC Beam Design Result

PROJECT TITLE:

MIDAS	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	rio sinú COMPLETO geotecnia hzapata

```
==========================================================
    ( ). Compute parameter.
         -. phi   =   0.90
         -. Alpha =   0.85
         -. Beta  =   0.85
         -. d     =     3.4000 m.
         -. ecu   =  0.0030

    ( ). Compute maximum and minimum reinforcement.
         -. Rhomin1 = (1.2)*Mcr/[ phi*fy*b*d*(d-a/2) ]      =   0.0019
         -. Rhomin2 = 1.33*Mu/[ phi*fy*b*d*(d-a/2) ]        =0.0000e+00
         -. Rhomin  = MIN[ Rhomin1, Rhomin2 ]               =0.0000e+00
         -. As_min =  Rhomin * Ag  =  0.0000e+00 m^2/m.

    ( ). Check tensile reinforcement.
         -. As     =     0.0082 m^2/m.
         -. As_min < As   --->  O.K !

    ( ). Check moment capacity.
         -. c          =     0.1686 m.
         -. Cc         =    3359.89 kN/m.
         -. Ts         =    3389.55 kN/m.
         -. Mr         =   10108.60 kN-m/m.
         -. Mu/Mr      =  0.000e+00  --->  O.K !

==========================================================
  [[[*]]]   ANALYZE SHEAR CAPACITY.
==========================================================
    ( ). Compute shear parameter.
         -. phi   =   0.90
         -. Av    =   0.0066 m^2/m.
         -. bv    =   0.03 m.
         -. dv    =   3.0769 m.  (for Beta Calculation)
         -. theta =   0.702 Rad.
         -. Epsilon_s = MIN[ (|Mu|/dv + 0.5*Nu + |Vu|)/(Es*As), 0.006 ] =  0.0032
         -. beta  =  4.8/(1+750*Epsilon_s) =  1.4090

    ( ). Compute shear strength of concrete.
         -. dv     =  MAX[ dv, 0.9*d, 0.72*Hc ]  =     3.08 m.
         -. Vu     =1.49e+04 kN/m.
         -. Vc     = 0.0316*beta*SQRT[fc']*bv*dv =1889.17 kN/m.
         -. phiVc  = phi * Vc =1700.26 kN/m.
         -. Vn_lim = 0.25*fc'*bv*dv =2.12e+04 kN/m.

    ( ). Compute stirrup spacing.
         -. Maximum spacing smax   = MIN[ 0.4*dv, 12 in ] =    0.305 m.
         -. Vu > phiVc  --->  Required shear reinforcement.
         -. Applied spacing s    =    0.200 m.

    ( ). Compute shear strength of reinforcement.
         -. Vs      = Av*fvs*dv*cot(theta) / s = 49363.47 kN/m.
         -. Vs lim  = 0.25*fc'*bv*dv - Vc = 19325.34 kN/m.
         -. Vs      = MIN[ Vs, Vs lim ] = 19325.34 kN/m.
         -. phiVs   = phi*Vs = 17392.81 kN/m.

         -. phiVs > (Vu-phiVc)  --->  O.K !
         -. Using Av  =  Vs / (fys*d)  =   0.0003 m^2/m.

==========================================================
  [[[*]]]   ANALYZE CRACK.
==========================================================
    ( ). Check crack control of Bottom reinforcement. ( LCB =   29+EX )
         -. Mu     =  21053.8826 kN-m/m.
         -. h      =     3.5000 m.
```

MIDAS/Civil — RC Beam Design Result

PROJECT TITLE:

	Company	JORGE FANDIÑO SAS	Client	
MIDAS	Author	rojaswalter3333@gmail.com	File Name	rio sinú COMPLETO geotecnia hzapata

```
      -. dc      =      0.1000 m.
      -. Beta_s  = 1+dc/(0.7*(h-dc)) = 1.04
      -. Gamma_e = 1.00
      -. c_cr    =      1.0186 m.
      -. Icr     =      1.7314 m^4/m.
      -. fss     =    230708.600 KPa.
      -. s       =      0.0800 m.
      -. sa      = 700*Gamma_e/(Beta_s*fss)-2*dc =      0.3099 m.
      -. s < sa --->  O.K !

   ( ). Check crack control of Bottom reinforcement requirements.
      -. As req'd              =      0.0006 m^2/m.
      -. As(Outermost layer) =      0.0102 m^2/m.
      -. As req'd < As --->  O.K !
      -. Max. spacing smax   =      0.4572 m.
      -. s(Outermost layer)  =      0.0800 m.
      -. s < smax --->  O.K !

   ( ). Check crack control of Top reinforcement. ( LCB =   42 )
      -. Mu      =     19.0016 kN-m/m.
      -. h       =      3.5000 m.
      -. dc      =      0.1000 m.
      -. Beta_s  = 1+dc/(0.7*(h-dc)) = 1.04
      -. Gamma_e = 1.00
      -. c_cr    =      0.4862 m.
      -. Icr     =      0.6171 m^4/m.
      -. fss     =       714.799 KPa.
      -. s       =       0.1000 m.
      -. sa      = 700*Gamma_e/(Beta_s*fss)-2*dc =      0.4572 m.
      -. s < sa --->  O.K !

   ( ). Check crack control of Top reinforcement requirements.
      -. As req'd              =      0.0004 m^2/m.
      -. As(Outermost layer) =      0.0082 m^2/m.
      -. As req'd < As --->  O.K !
      -. Max. spacing smax   =      0.4572 m.
      -. s(Outermost layer)  =      0.1000 m.
      -. s < smax --->  O.K !

================================================================
   [[[*]]]   SHRINKAGE AND TEMPERATURE REINFORCEMENT.
================================================================

      *.REINFORCEMENT PATTERN :
      ------------------------------------------------------------
         Location    i     di( m.)       Rebar  Asi( m^2/m.)
      ------------------------------------------------------------
         Top         1      0.032      #10@0.10       0.00819
         Bottom      1      0.032      #10@0.10       0.00819
         Bottom      2      0.032      #10@0.10       0.00819
         Bottom      3      0.032      #10@0.10       0.00819
         Bottom      4      0.032      #10@0.10       0.00819
      ------------------------------------------------------------

   ( ). Check Bottom Shrinkage and Temperature Reinforcement.
      -. As Req'd  = 1.3*b*h/(2*(b+h)*fys) =      0.0003 m^2/m.
      -. As        =      0.0307 m^2/m.
      -. As_Req < As --->  O.K !

   ( ). Check Top Shrinkage and Temperature Reinforcement.
      -. As Req'd  = 1.3*b*h/(2*(b+h)*fys) =      0.0003 m^2/m.
      -. As        =      0.0246 m^2/m.
      -. As_Req < As --->  O.K !
```

3.14.3. **DISEÑO DE CAISSONS CIRCULARES Ø = 2.0 m**

Para el diseño estructural de la estructura tipo caisson circular de concreto reforzado, se hace necesario utilizar el software MIDAS y determinar el refuerzo necesario para suplir todas las solicitaciones impuestas y que se encuentran generadas por el modelo, de esta manera, los pilotes se encuentran subdivididos en segmentos de 1.0 m de altura.

Del modelo estructural, se requiere determinar la carga gravitacional que desciende de la columna rectangular al encepado de concreto reforzado y posteriormente a los caissons

Se presenta las cargas actuantes para los diferentes estados de carga.

PK	**0+138.00**	**0+218.00**	**0+378.00**	**0+458.00**	**0+479.50**	**0+509.60**
ENVOLVENTE DE RESISTENCIA ÚLTIMA						
Øcaisson	**2.00**	**2.00**	**2.00**	**2.50**	**1.50**	**1.50**
Sep. Caisson C.A.C.	**5.00**	**6.00**	**6.00**	**7.50**	**4.00**	**4.00**
cargas en ton	**ESTRIBO 1**	**APOYO 1**	**APOYO 2**	**APOYO 3**	**APOYO 4**	**ESTRIBO 2**
P1	325.00	700.00	873.00	894.00	667.00	424.00
P2	400.00	1003.00	1170.00	637.00	449.00	289.00
P3	347.00	1160.00	1334.00	888.00	357.00	256.00
P4		1105.00	1283.00	645.00	236.00	132.00
P5		845.00	893.00			
P6		1169.00	1189.00			
P7		1384.00	1425.00			
P8		1336.00	1412.00			
P9		896.00	800.00			
P10		1199.00	1125.00			
P11		1418.00	1365.00			
P12		1371.00	1366.00			
P13		859.00	632.00			
P14		1163.00	933.00			
P15		1312.00	**1115.00**			
P16		1222.00	1110.00			
Long.	**34.00**	**50.00**	**40.00**	**30.00**	**35.00**	**20.00**
PROMEDIO PILOTE ton	357.33	1133.88	1126.56	766.00	427.25	275.25
PROMEDIO PILOTE kN	3573.33	11338.75	11265.63	7660.00	4272.50	2752.50
TOTAL ton	**1072.00**	**18142.00**	**18025.00**	**3064.00**	**1709.00**	**1101.00**
TOTAL kN	**10720.00**	**181420.00**	**180250.00**	**30640.00**	**17090.00**	**11010.00**

ENVOLVENTE DE EVENTO EXTREMO						
Øcaisson	**2.00**	**2.00**	**2.00**	**2.50**	**1.50**	**1.50**
Sep. Caisson C.A.C.	**5.00**	**6.00**	**6.00**	**7.50**	**4.00**	**4.00**
cargas en ton	**ESTRIBO 1**	**APOYO 1**	**APOYO 2**	**APOYO 3**	**APOYO 4**	**ESTRIBO 2**
P1	437.00	1073.00	1289.00	1167.00	583.00	38.00
P2	303.00	1100.00	1338.00	1005.00	314.00	186.00
P3	467.00	1233.00	1436.00	1132.00	287.00	186.00
P4		1298.00	1525.00	992.00	333.00	210.00
P5		1203.00	1189.00			
P6		1097.00	1167.00			
P7		1235.00	**1309.00**			
P8		1544.00	1516.00			
P9		1292.00	1057.00			
P10		1174.00	1031.00			
P11		1344.00	1213.00			
P12		1617.00	1457.00			
P13		1281.00	969.00			
P14		1384.00	1023.00			
P15		1465.00	**1156.00**			
P16		1569.00	1308.00			
Long.	**34.00**	**50.00**	**40.00**	**30.00**	**35.00**	**20.00**
PROMEDIO PILOTE ton	402.33	1306.81	1248.94	1074.00	379.25	155.00
PROMEDIO PILOTE kN	4023.33	13068.13	12489.38	10740.00	3792.50	1550.00
TOTAL ton	**1207.00**	**20909.00**	**19983.00**	**4296.00**	**1517.00**	**620.00**
TOTAL kN	**12070.00**	**209090.00**	**199830.00**	**42960.00**	**15170.00**	**6200.00**

ENVOLVENTE DE SERVICIO						
Øcaisson	**2.00**	**2.00**	**2.00**	**2.50**	**1.50**	**1.50**
Sep. Caisson C.A.C.	**5.00**	**6.00**	**6.00**	**7.50**	**4.00**	**4.00**
cargas en ton	**ESTRIBO 1**	**APOYO 1**	**APOYO 2**	**APOYO 3**	**APOYO 4**	**ESTRIBO 2**
P1	260.00	436.00	711.00	715.00	518.00	329.00
P2	327.00	634.00	931.00	517.00	344.00	220.00
P3	292.00	780.00	1052.00	704.00	284.00	200.00
P4		732.00	1004.00	524.00	199.00	111.00
P5		650.00	739.00			
P6		859.00	960.00			
P7		1052.00	**1151.00**			
P8		1016.00	1127.00			
P9		740.00	642.00			
P10		969.00	838.00			
P11		1149.00	1037.00			
P12		1098.00	1034.00			
P13		699.00	437.00			
P14		928.00	624.00			
P15		1040.00	**775.00**			
P16		962.00	760.00			
Long.	**34.00**	**50.00**	**40.00**	**30.00**	**35.00**	**20.00**
PROMEDIO PILOTE ton	293.00	859.00	863.88	615.00	336.25	215.00
PROMEDIO PILOTE kN	2930.00	8590.00	8638.75	6150.00	3362.50	2150.00
TOTAL ton	**879.00**	**13744.00**	**13822.00**	**2460.00**	**1345.00**	**860.00**
TOTAL kN	**8790.00**	**137440.00**	**138220.00**	**24600.00**	**13450.00**	**8600.00**

Apoyo Intermedio 1 – PK0+218

Según el **Volumen IV. ESTUDIO DE SUELOS PARA DISEÑO DE FUNDACIONES Y OBRAS COMPLEMENTARIAS**, para transmitir la carga vertical adecuadamente al suelo de cimentación, se requiere tener un diámetro de caisson de Ø=2.0 m hasta completar una longitud total de 50.0 m según recomendaciones del estudio de suelos y geotecnia.

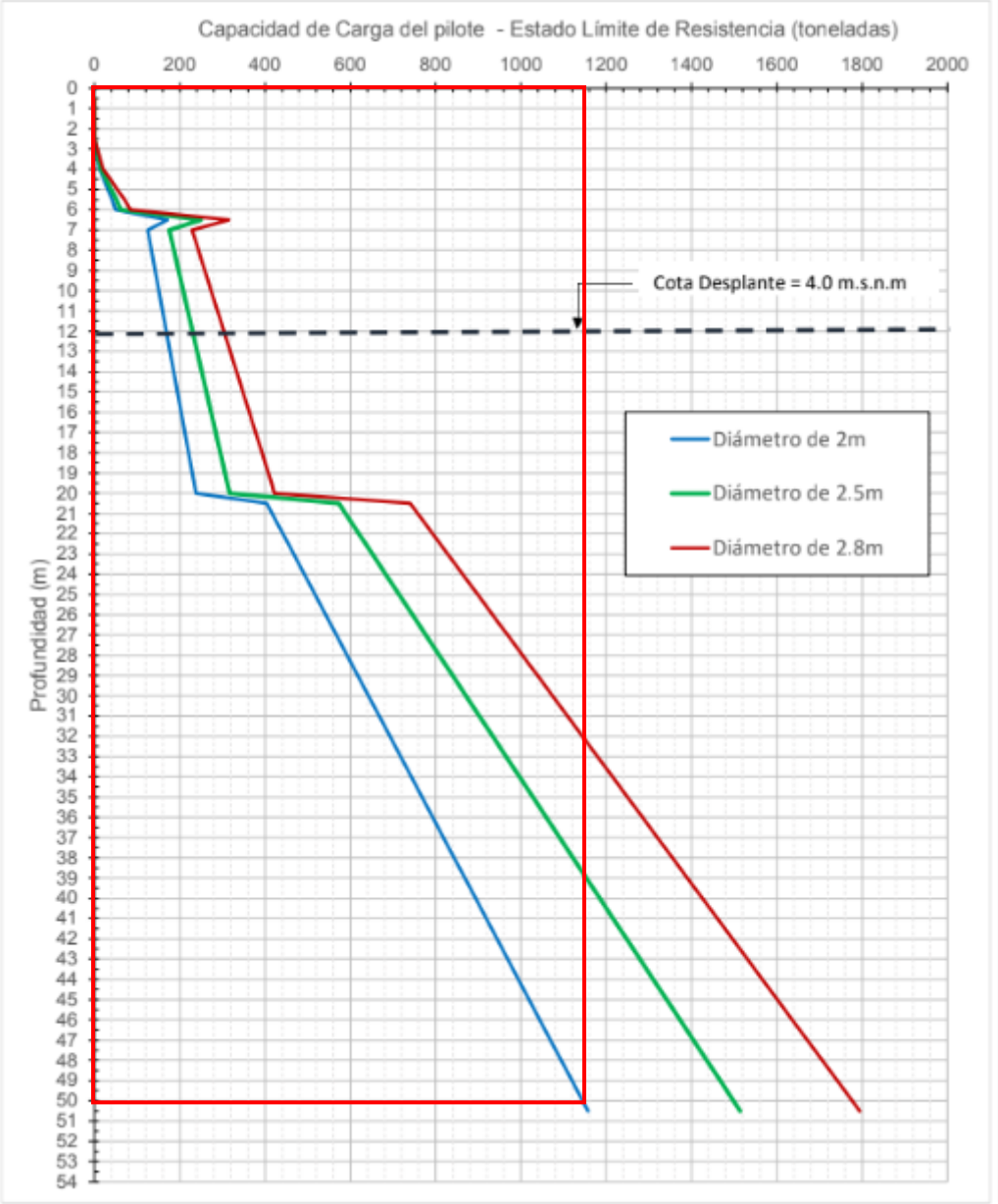


Figura 236. Resistencia Reducida Caissons Apoyo intermedio 1

De la anterior gráfica, se puede estimar una carga por resistencia última de 1200 ton por caissons, y una carga promedio de 1134 ton por caissons, con factor de seguridad de Demanda/capacidad FS=1200 ton / 1134 ton = 1.06

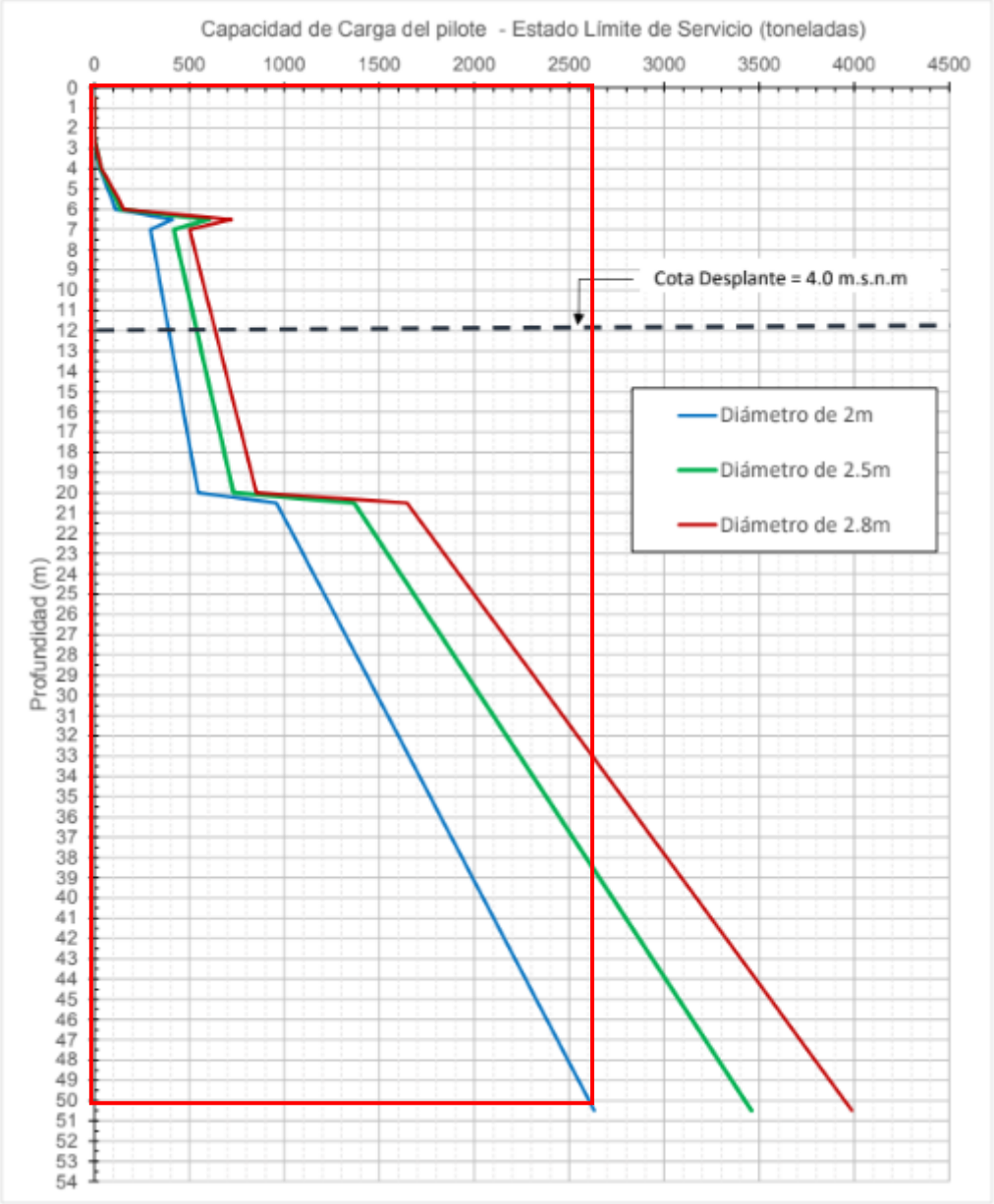


Figura 237. Resistencia evento extremo Caissons Estribo 1

De la anterior gráfica, se puede estimar una carga por evento extremo de 2700 ton por caisson, y una carga promedio de 1307 ton por caisson, con factor de seguridad de Demanda/capacidad FS = 2700 ton / 1307 ton = 2.06.

Del mismo modo, del **Volumen IV. ESTUDIO DE SUELOS PARA DISEÑO DE FUNDACIONES Y OBRAS COMPLEMENTARIAS** se extrajeron los módulos de

reacción horizontal del suelo, los cuales se utilizaron para determinar los resortes en el modelo estructural de los pilotes, a fin de poder determinar la interacción suelo-estructura ante las solicitaciones presentadas.

VARIACIÓN DE MODULO DE BALASTO								
(#)	Inicio (m)	Fin (m)	Diámetro (m)	Balasto kh (kg/cm3)	Diámetro (m)	Balasto kh (kg/cm3)	Diámetro (m)	Balasto kh (kg/cm3)
1	0	3.5	2	0.1359	2.5	0.1087	2.8	0.0971
2	3.5	6	2	0.2328	2.5	0.1709	2.8	0.1663
3	6	20	2	0.4818	2.5	0.3855	2.8	0.3442
4	20	50	2	0.8068	2.5	0.6454	2.8	0.5763

APOYO 1				
Diámetro de pilote (m) Ø		**2.00**	**Altura elemento h=1.0m**	
Profundidades (m)		**Módulo de reacción**	**Módulo de reacción**	**K_H**
De	**A**	**(kg/cm^3)**	**ton/m3**	**(ton/m)**
0.00	3.50	0.1359	135.90	272
3.50	6.00	0.2328	232.80	466
6.00	20.00	0.4818	481.80	964
20.00	50.00	0.8068	806.80	1614

Se presenta la interacción suelo-estructura debidamente incluida en el software MIDAS CIVIL

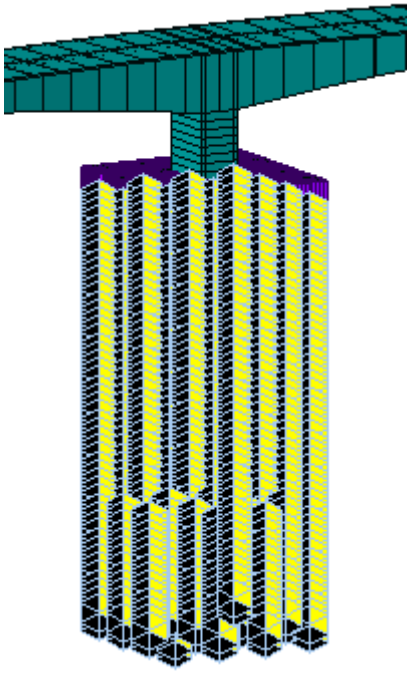


Figura 238. Módulos de reacción horizontal Kh Apoyo intermedio 1

Para el diseño estructural de elementos sometidos a flexo-compresión se debe realizar de manera iterativa hasta determinar qué cantidad de refuerzo cumple con la cuantía requerida y con la cuantía mínima, en este orden de ideas, se plantea una alternativa para los caissons del apoyo intermedio 1.

Con ayuda del software MIDAS CIVIL se pudo determinar la cuantía requerida para atender las solicitaciones a momento flector y fuerza cortante generada por la variación entre combinaciones de carga establecidas por el código CCP-14.

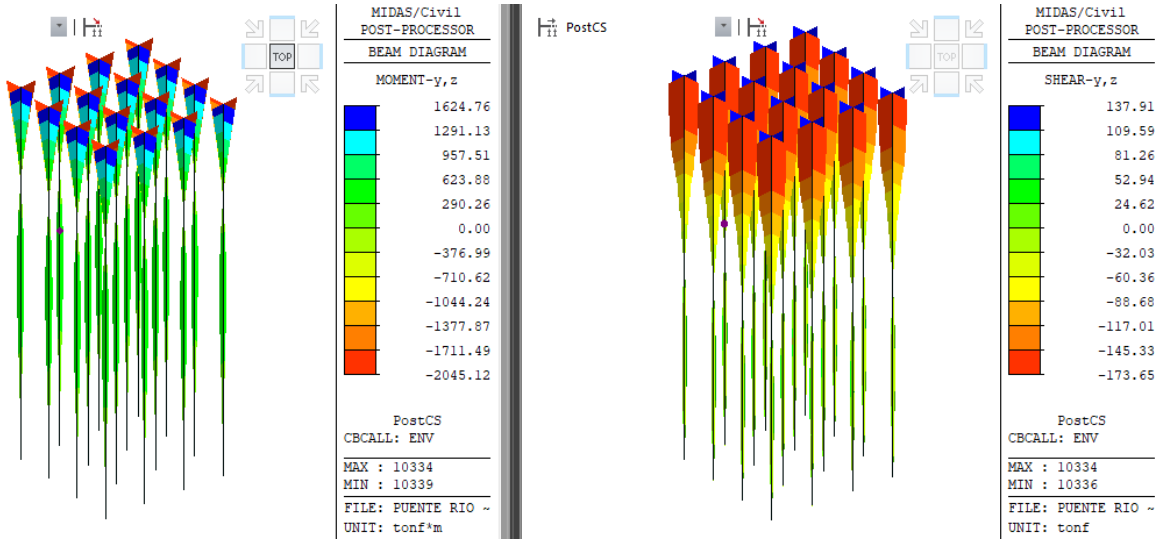


Figura 239. Momento flector y fuerza cortante para la envolvente de carga Apoyo intermedio 1

SOLICITACIONES DE DISEÑO ESTRUCTURAL

$$M_U{}^{(+)} = 1624.76\ ton.m \qquad V_U{}^{(+)} = 137.91\ ton$$
$$M_U{}^{(-)} = -2045.12\ ton.m \qquad V_U{}^{(-)} = -173.65\ ton$$

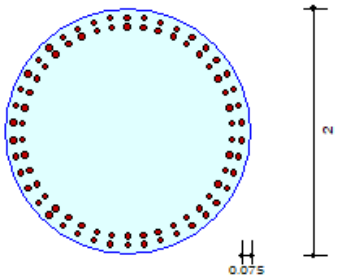
Diseño a Flexo-compresión

1. Design Condition

Design Code	AASHTO-LRFD16
Unit System	tonf, m
Member Number	10336 (PM), 10336 (Shear)
Material Data	fc = 2812.28, fy = 42184.2, fys = 42184.2 tonf/m^2
Column Height	1 m
Section Property	pilote Ø2.0 (No : 82)
Rebar Pattern	

	Pos 1	Pos 2	Pos 3
Layer 1	42-#10	--	--
Layer 2	42-#10	--	--

Total Rebar Area Ast = 0.0688257 m^2 (Rhost = 0.0219)



2. Applied Loads

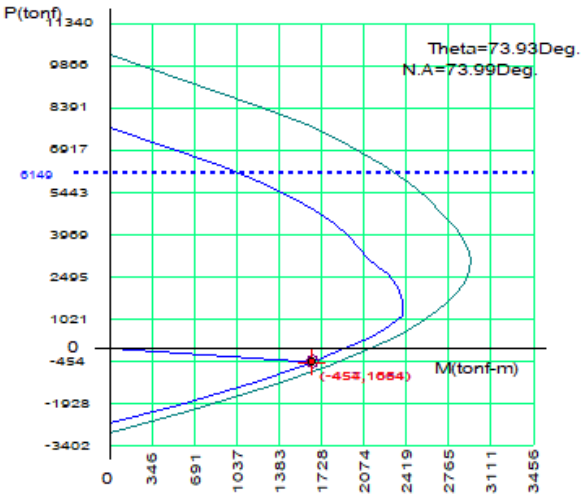
Load Combination 20+ AT (I) Point

Pu = -454.25 tonf, Mcy = 456.228, Mcz = 1589.79, Mc = 1653.96 tonf-m

3. Axial Forces and Moments Capacity Check

Concentric Max. Axial Load	Pr-max	= 6149.17 tonf	
Axial Load Ratio	Pu/Pr	= -454.25 / -456.90	= 0.994 < 1.000 ..... O.K
Moment Ratio	Mcy/Mry	= 456.228 / 459.877	= 0.992 < 1.000 ..... O.K
	Mcz/Mrz	= 1589.79 / 1596.32	= 0.996 < 1.000 ..... O.K
	Mc/Mr	= 1653.96 / 1661.24	= 0.996 < 1.000 ..... O.K

4. P-M Interaction Diagram



Pr(tonf)	Mr(tonf-m)
7686.46	0.00
5858.23	1204.01
4813.30	1671.63
3898.58	1951.38
3138.32	2119.14
2606.33	2267.36
2216.83	2342.40
1853.26	2381.03
1437.82	2392.68
847.88	2290.89
-6.99	1913.20
-1175.65	1187.10
-2613.02	0.00

5. Shear Force Capacity Check

Applied Shear Force	Vu	= 190.983 tonf (Load Combination 19-)	
Shear Strength by Conc	PhiV	= 251.096 tonf	
Shear Strength by Rebar	PhiVs	= 235.824 tonf (2.0-#5 @0)	
Shear Ratio	Vu/Vr	= 190.983 / 486.919	= 0.392 < 1.000 ..... O.K

MIDAS/Civil — RC Column Checking Result

PROJECT TITLE :

MIDAS	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	e rio sinú COMPLETO geotecnia hzapata=3.

```
================================================================================
 MIDAS/Civil - RC-Column Checking [ AASHTO-LRFD16 ]                  Civil 2022
================================================================================

      *.MIDAS/Civil - RC-COLUMN  Analysis/Design Program.
      *.DESIGN CODE    : AASHTO-LRFD16,     *.UNIT SYSTEM : tonf, m
      *.MEMBER         : Member Type = COLUMN ,  MEMB = 10336,  LCB =  20+,  POS =    I

      *.DESCRIPTION OF COLUMN  DATA (iSEC =    82) : pilote Ø2.0
        Column Height (L)                =         3.500 m.

        Section Type : SOLID ROUND (SR)
        Section Diameter  (D)             =         2.000 m.
        Concrete Strength (fc)            =      2812.279 tonf/m^2.
        Main Rebar Strength (fy)          =     42184.188 tonf/m^2.
        Ties/Spirals Strength (fys)       =     42184.188 tonf/m^2.
        Modulus of Elasticity (Es)        =  20389024.157 tonf/m^2.

      *.REINFORCEMENT PATTERN :
        ------------------------------------------------------------------------------------------
          Layer         Pos1     As( m^2.)         Pos2     As( m^2.)         Pos3     As( m^2.)
        ------------------------------------------------------------------------------------------
            1         42-#10 (    0.0344)            -                           -
            2         42-#10 (    0.0344)            -                           -
        ------------------------------------------------------------------------------------------

      *.Ties :  2.0-#5  @0.14

 ============================================================================
    [[[*]]]   CALCULATE SLENDERNESS RATIOS, MAGNIFIED FORCES/MOMENTS.
 ============================================================================

      ( ). Factored forces/moments caused by unit load case.  Unit : tonf., m.
           *.Load combination ID =  20+
           ------------------------------------------------------------------------------
            Load Case          Pu max          Myi          Myj          Mzi          Mzj
           ------------------------------------------------------------------------------
            DL                319.40      -254.54      -175.29      -157.62      -117.94
            LL                -23.82        18.14        13.18        2.36         1.64
            DL+LL             296.59      -236.40      -162.10      -155.26      -116.30
            Others           -750.84       692.63       487.16      1745.05      1225.04
           ------------------------------------------------------------------------------
            DL+LL+Others      -454.25       456.23       325.05      1589.79      1108.74
           ------------------------------------------------------------------------------

 ============================================================================
    [[[*]]]   ANALYZE CAPACITY OF BIAXIALLY LOADED RC-COLUMN.
 ============================================================================

      ( ). Compute design parameter.
           -. phi   =   0.90
           -. Alpha =   0.85
           -. Beta  =   0.85
           -. ecu   =  0.0030

      ( ). Compute maximum and minimum reinforcement.
           -. Rhomax1 =   0.0300 (User Input)
           -. Rhomax  = MIN[ Rhomax1, 0.08 ] =   0.0300
           -. Rhomin  = 0.135*fc'/fy =   0.0090

      ( ). Check tensile reinforcement.
           -. Rho     =     0.0219
           -. Rho_min < Rho < Rho_max  ---> O.K !
```

MIDAS/Civil — RC Column Checking Result

PROJECT TITLE :

MIDAS	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	e rio sinú COMPLETO geotecnia hzapata=3.

```
      ( ). Check moment capacity.
           -. Pr max      =      6149.167 tonf.
           -. Pr          =      -456.899 tonf.
           -. Mry         =       459.877 tonf-m.
           -. Mrz         =      1596.321 tonf-m.
           -. Mr          =      1661.242 tonf-m.
           -. Pu/Pr       =   0.994  --->  O.K !
           -. Muy/Mry     =   0.992  --->  O.K !
           -. Muz/Mrz     =   0.996  --->  O.K !
           -. Mu/Mr       =   0.996  --->  O.K !
           -. Ratio       =   0.996  --->  O.K !

=============================================================================
   [[[*]]]   ANALYZE SHEAR CAPACITY OF BIAXIALLY LOADED RC-COLUMN.
=============================================================================

      ( ). Compute shear parameter. ( LCB = 19, POS = I )
           -. phi    =   0.90
           -. Av     =     0.0004 m^2.
           -. Pu     =        882.56 tonf.
           -. Vuz    =        173.65 tonf.
           -. Muy    =      -2032.18 tonf-m.
           -. bvz    =   2.00 m.
           -. dvz    =  MAX[ dvz_calc, 0.9*dz, 0.72*Hc ] =   1.47 m.
           -. thetaz =  35.40 Deg.          [Clause 5.8.3.4.2]
           -. betaz  =   2.02
           -. Vuy    =         79.49 tonf.
           -. Muz    =       -959.04 tonf-m.
           -. bvy    =   2.00 m.
           -. dvy    =  MAX[ dvy_calc, 0.9*dy, 0.72*Bc ] =   1.54 m.
           -. thetay =  35.33 Deg.          [Clause 5.8.3.4.2]
           -. betay  =   2.04

      ( ). Compute shear strength of concrete.
           -. Vuz       =      173.65 tonf.
           -. Vcz       = 0.0316*betaz*SQRT[fc']*bvz*dvz =      263.84 tonf.
           -. phiVcz    = phi * Vcz =      237.46 tonf.

           -. Vuy       =       79.49 tonf.
           -. Vcy       = 0.0316*betay*SQRT[fc']*bvy*dvy =      279.00 tonf.
           -. phiVcy    = phi * Vcy =      251.10 tonf.

      ( ). Compute maximum spacing of ties.
           -. Maximum spacing smax z  = MIN[ 0.8*dvz, 24 in ] =    0.610 m.
           -. phiVcz/2 < Vuz < phiVcz  --->  Required minimum shear reinforcement.
           -.                  smax_z  = MIN[ smax_z, Av/(0.0316*SQRT(fc')*bvz/fys) ] =    0.190 m.
           -. Applied spacing s_z   =    0.140 m.

           -. Maximum spacing smax y  = MIN[ 0.8*dvy, 24 in ] =    0.610 m.
           -. phiVcy/2 < Vuy < phiVcy  --->  Required minimum shear reinforcement.
           -.                  smax y  = MIN[ smax_y, Av/(0.0316*SQRT(fc')*bvy/fys) ] =    0.190 m.
           -. Applied spacing s_y   =    0.140 m.

           -. Applied spacing s     = MIN[ s_y, s_z ] =    0.140 m.

      ( ). Compute shear strength of reinforcement.
           -. Vsz       = Av*fys*dvz*cot(thetaz) / s =      248.71 tonf.
           -. Vsz_lim   = 0.25*fc'*bvz*dvz - Vcz =     1797.95 tonf.
           -. Vsz       = MIN[ Vsz, Vsz lim ] =      248.71 tonf.
           -. phiVsz    = phi*Vsz =      223.84 tonf.
           -. phiVsz > (Vuz-phiVcz)  --->  O.K !
           -. Using Avz/s  =  Vsz / (fys*dvz*cot(thetaz)) =   0.0029 m^2/m.

           -. Vsy       = Av*fys*dvy*cot(thetay) / s =      262.03 tonf.
```

```
          -. Vsy_lim  = 0.25*fc'*bvy*dvy - Vcy =     1887.67 tonf.
          -. Vsy      = MIN[ Vsy, Vsy lim ] =      262.03 tonf.
          -. phiVsy   = phi*Vsy =      235.82 tonf.
          -. phiVsy > (Vuy-phiVcy)   --->  O.K !
          -. Using Avy/s  =  Vsy / (fys*dvy*cot(thetay)) =   0.0029 m^2/m.

          -. Vu       =     190.983 tonf.
          -. Vr       =     486.919 tonf.
          -. Ratio    = Vu / Vr = 0.392  --->  O.K !

     ( ). Compute minimum longitudinal reinforcement by shear.
          -. phia     =   0.75
          -. phib     =   0.90
          -. phiv     =   0.90
          -. Vsly     =  MIN[ Vsy, Vuy/phiv ] =      212.20 tonf.
          -. Vslz     =  MIN[ Vsz, Vuz/phiv ] =      212.20 tonf.
          -. Asy_req =  [ Muz/(phib*dvy)  - 0.5*Pu/phia + (Vuy/phiv - 0.5*Vsly)*cot(thetay) ] / fy
=   0.0280 m^2.
          -. Asz_req =  [ Muy/(phib*dvz)  - 0.5*Pu/phia + (Vuz/phiv - 0.5*Vslz)*cot(thetaz) ] / fy
=   0.0300 m^2.
          -. As req  =  MAX[ Asy req, Asz_req ] =   0.0300 m^2.
          -. As      =   0.0688 m^2.
          -. As_req < As  --->  O.K !
```

3.15. DISEÑO ESTRUCTURAL APOYO INTERMEDIO 2

3.15.1. DISEÑO ESTRUCTURAL COLUMNA RECTANGULAR

Para el diseño estructural de la columna rectangular, se plantea un modelo estructural desarrollado en el software MIDAS CIVIL 2022, que cumple con el diseño geométrico para determinar las solicitaciones tanto para momento flector, como para fuerza axial, cortante y torsión y así, determinar el refuerzo requerido que se dispondrá tanto en la sección transversal propuesta del estribo 1. En la siguiente figura está representado el modelo estructural que será empleado en el software para el diseño estructural del apoyo intermedio 2.

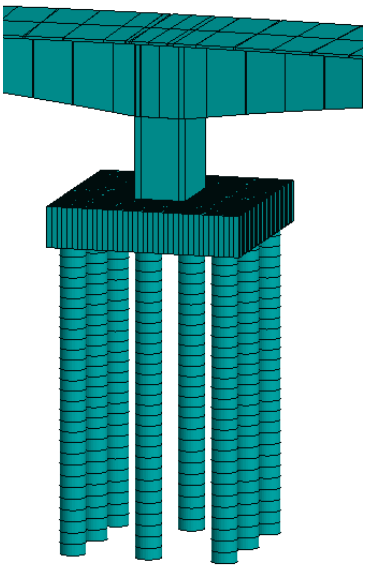


Figura 240. Modelo digital en software MIDAS CIVIL 2022

Para determinar las respectivas solicitaciones, se realiza el modelamiento estructural completo, integrando la superestructura y subestructura, a fin de poder transmitir las cargas de la superestructura a la columna rectangular, siendo estas zonas las que se verán más afectadas a momento flector tanto positivo como negativo, valores con los cuales se podrá determinar el refuerzo necesario para suplir estas solicitaciones.

Se realiza el diseño estructural y se aprecia las solicitaciones máximas para momento flector positivo y negativo, además de la fuerza cortante respectivamente, que está siendo sometida la viga cabezal de los estribos a las diferentes combinaciones de carga impuestas en el modelo estructural.

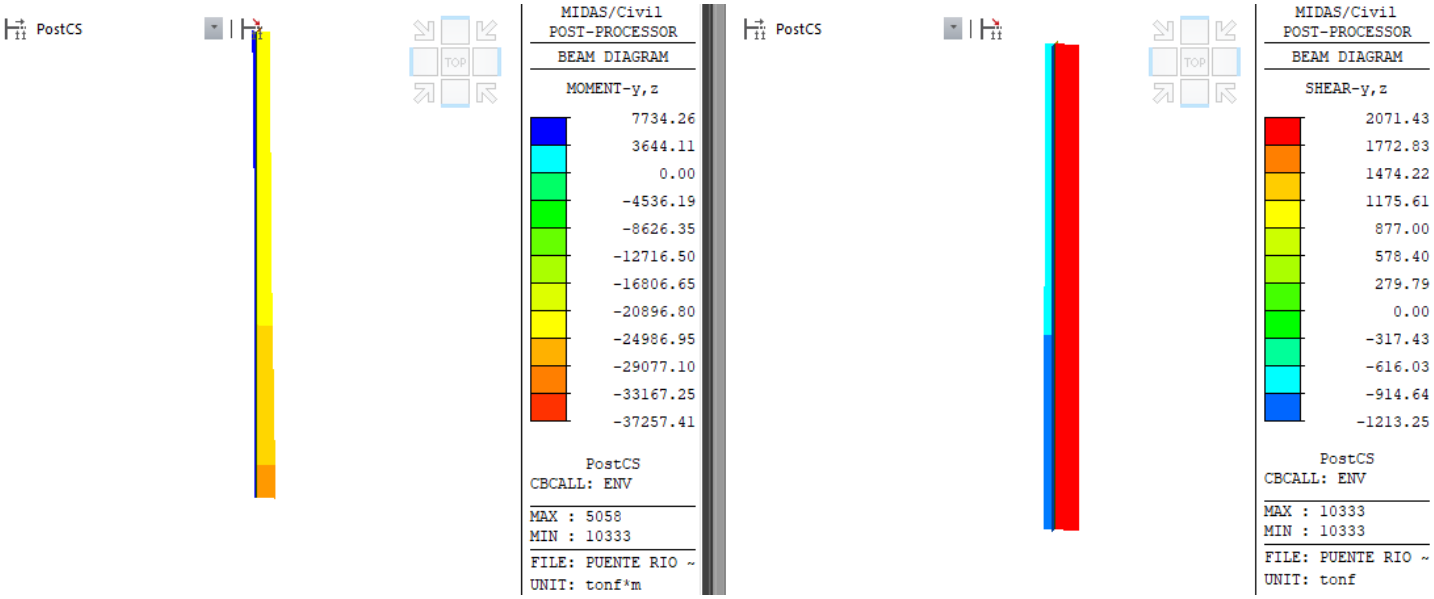


Figura 241. Máximo momento flector y fuerza cortante

SOLICITACIONES DE DISEÑO ESTRUCTURAL

$$M_U{}^{(+)} = 7734.26\ ton.m \qquad V_U{}^{(+)} = 2071.43\ ton$$
$$M_U{}^{(-)} = -37257.41\ ton.m \qquad V_U{}^{(-)} = -1213.25\ ton$$

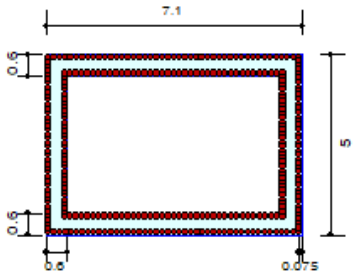
MEMB / SECT	SEL	Section / t	Bc / a	Hc / b	fc / Height	fy / fys	CHK	LCB	POS	Ast / Rho	Pr-max	Pu / Mc	Pr / Mr	Rat-P / Rat-M	LCB	Asvy / Asvz	Vu / Rat-V
5058		COLUMN	7.100	5.000	4588.72	42184.2	OK	21-	I	0.2589	36557.8	8493.96	11673.2	0.728	20+	0.0001	2021.73
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		35181.4	47996.7	0.747		0.0001	0.686
10325		COLUMN	7.100	5.000	4588.72	42184.2	OK	21-	J	0.2589	36557.8	8556.86	11488.8	0.745	20+	0.0001	2027.61
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		36682.2	48929.2	0.773		0.0001	0.687
10326		COLUMN	7.100	5.000	4588.72	42184.2	OK	21-	J	0.2589	36557.8	8588.33	11124.8	0.772	20+	0.0001	2033.41
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		37564.4	49009.9	0.788		0.0001	0.689
10327		COLUMN	7.100	5.000	4588.72	42184.2	OK	21-	J	0.2589	36557.8	8619.80	10994.1	0.784	20+	0.0001	2039.11
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		38539.9	49066.5	0.805		0.0001	0.690
10328		COLUMN	7.100	5.000	4588.72	42184.2	OK	21-	J	0.2589	36557.8	8651.27	10743.7	0.805	20+	0.0001	2044.72
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		39615.8	49068.7	0.826		0.0001	0.692
10329		COLUMN	7.100	5.000	4588.72	42184.2	OK	21-	J	0.2589	36557.8	8682.73	10490.5	0.828	20+	0.0001	2050.25
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		40796.1	49029.2	0.852		0.0001	0.693
10330		COLUMN	7.100	5.000	4588.72	42184.2	OK	21-	J	0.2589	36557.8	8714.20	10097.6	0.863	20+	0.0001	2055.68
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		42082.3	48912.2	0.883		0.0001	0.694
10331		COLUMN	7.100	5.000	4588.72	42184.2	OK	21-	J	0.2589	36557.8	8745.66	9823.10	0.890	20+	0.0001	2061.01
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		43469.4	48793.0	0.918		0.0001	0.696
10332		COLUMN	7.100	5.000	4588.72	42184.2	OK	21-	J	0.2589	36557.8	8777.13	9433.73	0.930	20+	0.0001	2066.27
79		0.6000	0.000	0.000	1.0000	42184.2				0.0198		44952.4	48229.5	0.932		0.0001	0.697
10333		COLUMN	7.100	5.000	4588.72	42184.2	OK	21-	J	0.2589	36557.8	8808.59	9054.49	0.973	20+	0.0001	2071.43

1. Design Condition

Design Code	AASHTO-LRFD16
Unit System	tonf, m
Member Number	10333 (PM), 10333 (Shear)
Material Data	fc = 4588.72, fy = 42184.2, fys = 42184.2 tonf/m^2
Column Height	1 m
Section Property	COLUMNA 7.1X5.0 (No : 79)
Rebar Pattern	

	Pos 1	Pos 2	Pos 3
Layer 1	45-#10	40-#10	--
Layer 2	39-#10	34-#10	--

Total Rebar Area Ast = 0.258916 m^2 (Rhost = 0.0198)



2. Applied Loads

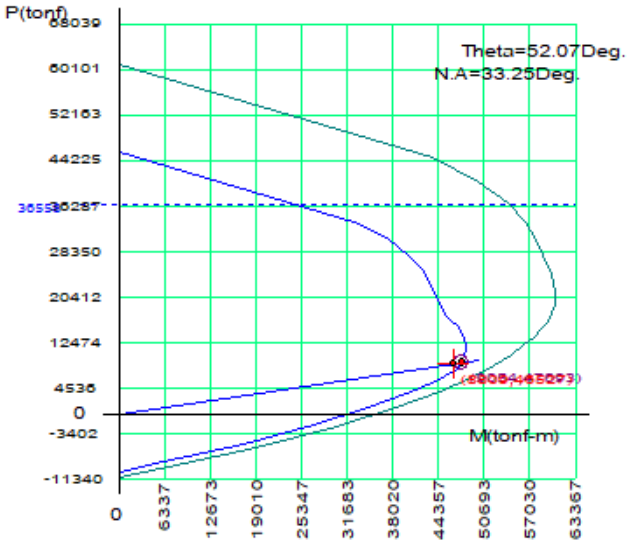
Load Combination 21- AT (J) Point

Pu = 8808.59 tonf, Mcy = -2.8e+04, Mcz = -3.7e+04, Mc = 46526.9 tonf-m

3. Axial Forces and Moments Capacity Check

Concentric Max. Axial Load	Pr-max	= 36557.8 tonf	
Axial Load Ratio	Pu/Pr	= 8808.59 / 9054.49	= 0.973 < 1.000 ..... O.K
Moment Ratio	Mcy/Mry	= -2.8e+04 / 29316.4	= 0.951 < 1.000 ..... O.K
	Mcz/Mrz	= -3.7e+04 / 37619.2	= 0.990 < 1.000 ..... O.K
	Mc/Mr	= 46526.9 / 47693.3	= 0.976 < 1.000 ..... O.K

4. P-M Interaction Diagram



Pr(tonf)	Mr(tonf-m)
45697.27	0.00
30440.88	37453.43
25080.93	42391.73
20865.27	44244.02
17398.74	45348.27
15139.45	47115.68
13223.24	47978.32
11372.79	48197.63
9218.18	47752.83
5975.39	44021.57
1282.03	34633.41
-5023.81	17233.77
-9829.93	0.00

5. Shear Force Capacity Check

Applied Shear Force	Vuy	= 1019.24,	Vuz	= 2071.43 tonf (Load Combination 20+)
Shear Strength (Y-Direction)	PhiVy	= 1987.52,	PhiVsy	= 2467.47 tonf (4.0-#5 @0)
Shear Strength (Z-Direction)	PhiVz	= 1276.78,	PhiVsz	= 1689.82 tonf (4.0-#5 @0)
Shear Ratio	Vu/Vr	= 2071.43 / 2966.60		= 0.698 < 1.000 ..... O.K

MIDAS/Civil — RC Column Checking Result

PROJECT TITLE :

MIDAS	Company	JORGE FANDIÑO SAS	Client	
	Author	rojaswalter3333@gmail.com	File Name	puente rio sinú COMPLETO geotecnia.rcs

```
================================================================================
 MIDAS/Civil - RC-Column Checking [ AASHTO-LRFD16 ]                    Civil 2022
================================================================================
      *.MIDAS/Civil - RC-COLUMN  Analysis/Design Program.
      *.DESIGN CODE   : AASHTO-LRFD16,     *.UNIT SYSTEM : tonf, m
      *.MEMBER        : Member Type = COLUMN ,  MEMB = 10333,  LCB =  21-,  POS =    J

      *.DESCRIPTION OF COLUMN  DATA (iSEC =    79) : COLUMNA 7.1X5.0
        Column Height (L)               =         1.000 m.

        Section Type : BOX (B)
        Section Depth  (Hc)             =         5.000 m.
        Section Width  (Bc)             =         7.100 m.
        Section Thick  (t)              =         0.600 m.
        Concrete Strength (fc)          =      4588.723 tonf/m^2.
        Main Rebar Strength (fy)        =     42184.188 tonf/m^2.
        Ties/Spirals Strength (fys)     =     42184.188 tonf/m^2.
        Modulus of Elasticity (Es)      =  20389024.157 tonf/m^2.

      *.REINFORCEMENT PATTERN :
        ------------------------------------------------------------------------------------
          Layer         Pos1    As( m^2.)         Pos2    As( m^2.)         Pos3    As( m^2.)
        ------------------------------------------------------------------------------------
            1         45-#10 (    0.0369)       40-#10 (    0.0328)           -
            2         39-#10 (    0.0320)       34-#10 (    0.0279)           -
        ------------------------------------------------------------------------------------

      *.Ties :  4.0-#5  @0.15

 ============================================================================
    [[[*]]]   CALCULATE SLENDERNESS RATIOS, MAGNIFIED FORCES/MOMENTS.
 ============================================================================
      ( ). Factored forces/moments caused by unit load case.  Unit : tonf., m.
           *.Load combination ID =  21-
           ------------------------------------------------------------------------------
            Load Case          Pu max          Myi          Myj          Mzi          Mzj
           ------------------------------------------------------------------------------
            DL               7968.09    -11206.92    -11851.96    -17974.42    -17903.79
            LL                217.04     -2023.60     -1955.25      -182.47      -185.06
            DL+LL            8185.13    -13230.52    -13807.21    -18156.89    -18088.86
            Others            623.45    -13350.17    -14060.98    -18094.80    -19168.55
           ------------------------------------------------------------------------------
            DL+LL+Others     8808.59    -26580.69    -27868.20    -36251.69    -37257.41
           ------------------------------------------------------------------------------

      ( ). Check slenderness ratios of BRACED/UNBRACED frame.
           -. End Moments (My1) =    26580.69 tonf-m.
           -. End Moments (My2) =    27868.20 tonf-m.
           -. Slenderness ratio limits.
              SRy(Unbraced) =  22.00
           -. Radii of gyration (Roy)        =    1.895 m.
           -. Unbraced lengths  (Ly)         =    1.000 m.
           -. Effective length factors (Ky)  =    1.000
           -. SLENy  =  Ky*Ly/Roy  =    0.528  <  SRy  --->  NOT SLENDER.

      ( ). Compute moment magnification factors for major axis(DBy,DSy).
           -. DBy    =     1.00 (Default value)

      ( ). Compute magnified moments.
           -. No sidesway moments.
              QMb_y  =  My_G   =   -13807.21 tonf-m.
           -. Sidesway moments.
              QMs_y  =  My_S   =   -14060.98 tonf-m.
           -. Compute magnified moments.
```