7.4. DETAILED DESIGN OF ABUTMENTS

Notes on Detailed Design of Abutments

(1) General

Refer to Notes on Flexural Design and Notes on Shear Design – RC Columns.

(2) Strut and Tie Model

At the pile cap, the piles are located such that a strut-and-tie model may be used to determine internal forces.

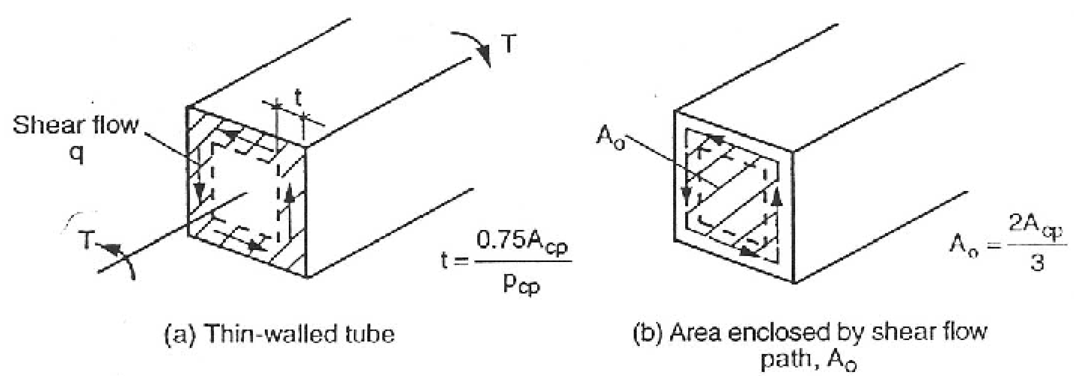
The provisions of AASHTO LRFD Article 5.6.3 will be followed in the use of strut-and-tie models.

(3) Torsion Design

The abutment designs that feature a central shear wall between the supporting abutment column sections will pick up substantial torsion moments under transverse earthquake loading. In addition, pile caps that do not feature piles in-line with the abutment column sections will be subject to torsion moments in transferring longitudinal plastic hinging effects.

The proper treatment of torsion forces and reinforcement against torsional effects is therefore critical to the design of the abutments.

For torsion design purposes the center portion of a solid beam can be conservatively neglected. Therefore the beam is idealized as a tube. Torsion is resisted through a constant shear flow q (force per unit length of wall centerline) acting around the centerline of the tube as shown below:



Thin-Wall Tube Analogy

From equilibrium of external torque T and internal stresses:

$$T = 2 \cdot A_O \cdot q = 2 \cdot A_O \cdot \tau \cdot t \quad \text{(Equation 1)}$$

Rearranging Equation 1 gives:

$$q = \tau \cdot t = \frac{T}{2 \cdot A_o} \quad \text{(Equation 2)}$$

where:

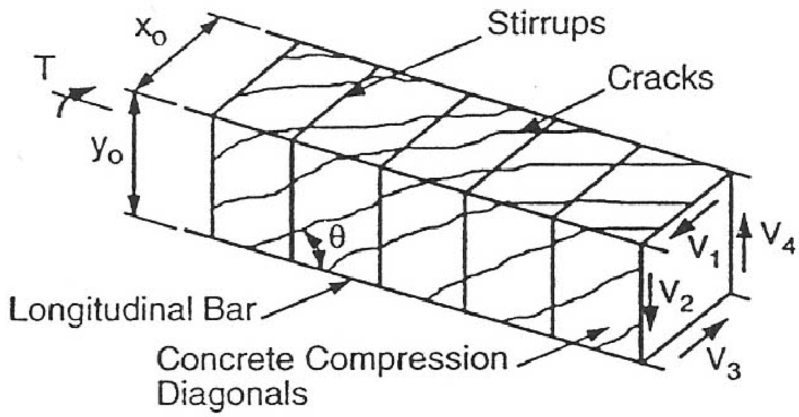
τ = shear stress across wall thickness

t = wall thickness

A_o = applied Torque

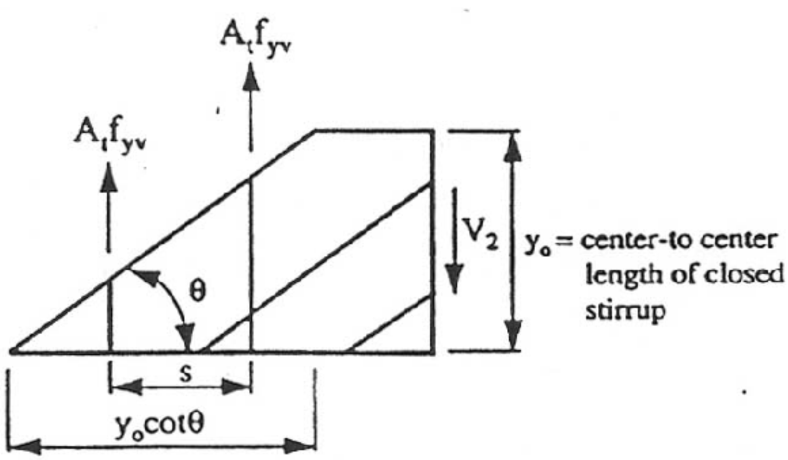
T = area enclosed within the tube centerline

When a concrete beam is subjected to torsion diagonal cracks form around the beam. Thus, after cracking, the tube is idealized as a space truss as shown below:



Space Truss Analogy

In this truss, diagonal members are inclined at an angle θ. Inclination of the diagonals in all tube walls is the same. The resultant of the shear flow in each tube wall induces forces in the truss members. The truss members that are in tension consist of steel reinforcement "tension ties". Truss diagonals and other members that are in compression consist of concrete "compression struts". Forces in the truss members can be determined from equilibrium conditions. The figure below depicts a free body extracted from the front vertical wall of the truss.



Free Body Diagram for Vertical Equilibrium

Shear force V_2 is equal to shear flow q (force per unit length) times the height of wall y_o. Stirrups are designed to yield when the maximum torque is reached. The number of stirrups intercepted is a function of stirrup spacing s and the horizontal projection $y_o cot\theta$ of the inclined surface. From vertical equilibrium:

$$V_2 = \frac{A_t \cdot f_{yv}}{s} \cdot \cot\theta$$ ……(Equation 3)

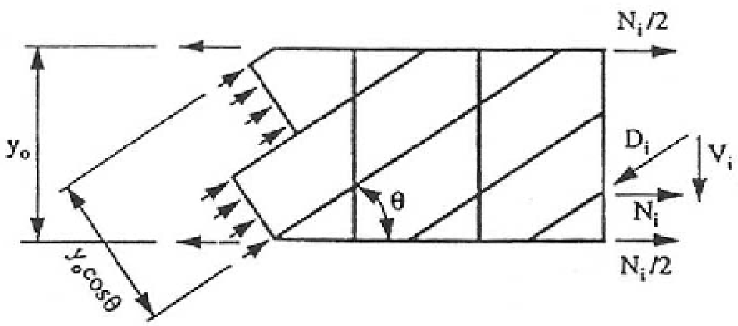
As the shear flow (force per unit length) is constant over the height of the wall,

$$V_2 = q \cdot y_o = \frac{T}{2 \cdot A_O} \cdot y_o$$ ……(Equation 4)

Substituting for V_2 in Equation 3 and 4 gives:

$$T = \frac{2 \cdot A_o \cdot A_t \cdot f_{yv}}{s} \cdot \cot\theta$$ ……AASHTO LRFD (5.8.3.6.2-1)...(Equation 5)

A free body diagram for horizontal equilibrium is shown below:



<u>Free Body Diagram for Horizontal Equilibrium</u>

The vertical shear force V_i in wall "i" is equal to the product of the shear flow q times the length of the wall y_i. Vector V_i can be resolved into two components: a diagonal component with an inclination θ equal to the angle of the truss diagonals, and a horizontal component equal to:

$$N_i = V_i \cdot \cot\theta$$ ……(Equation 6)

Force N_i is centered at the mid-height of the wall since q is constant along the side of the element. Each of the chords of the free body resists a force $N_i/2$. Summing the forces in the chords of all the space truss walls and assuming the longitudinal steel yields when the maximum torque is reached gives:

$$A_l \cdot f_{yl} = \sum N_i = \sum V_i \cdot \cot\theta = \sum q \cdot y_i \cdot \cot\theta = \sum \frac{T}{2 \cdot A_o} \cdot y_i \cdot \cot\theta = \frac{T}{2 \cdot A_o} \cdot \cot\theta \sum y_i$$ ………

……(Equation 7)

where:

$A_l \cdot f_{yl}$ = the yield force in all longitudinal reinforcement required for torsion

Rearranging Equation 7 gives:

$$T = \frac{2 \cdot A_o \cdot A_l \cdot f_{yl}}{2 \cdot (x_o + y_o) \cdot \cot\theta}$$ ……(Equation 8)

AASHTO LRFD allows the assumption of $\theta = 45\deg$ giving $\cot\theta = 1.0$ for reinforced concrete design.

Defining the perimeter, p_h, of the centerline of the closed transverse reinforcement as:

$p_h = 2 \cdot (x_o + y_o)$ ……………………………………………………………………(Equation 9)

Substituting Equation 9 into Equation 8 with $\cot\theta = 1.0$ and rearranging gives the equation below to solve directly for A_l, the longitudinal reinforcement required for torsion:

$A_l = \frac{p_h \cdot T}{2 \cdot A_o} \cdot \frac{1}{f_{yl}}$ ……………………………………………………………………(Equation 10)

Rearranging Equation 5 with $\cot\theta = 1.0$ gives directly the required spacing of transverse reinforcement, s, required to resist torsion:

$s = \frac{2 \cdot A_o \cdot A_t \cdot f_{yv}}{T}$ ……………………………………………………………………(Equation 11)

Torsional Moment Strength

The factored torsional resistance, T_r, shall be taken as:

$$T_r = \phi \cdot T_n$$

where in both cases:

$T_n = \frac{2 \cdot A_o \cdot A_t \cdot f_{yv}}{s} \cdot \cot\theta$ from Equation 5 above

ϕ = the strength reduction factor for shear (= 0.70)

f_{yv} = yield strength of transverse torsion reinforcement

A_o = area enclosed by the shear flow path

$$A_o = 0.85 \cdot A_{oh}$$

A_t = area of one leg of transverse torsion reinforcement

A_{oh} = area enclosed by centerline of exterior closed transverse torsion reinforcement

For normal density concrete, torsional effects shall be investigated where:

$$T_u > 0.25 \cdot \phi \cdot T_{cr}$$

in which for REINFORCED CONCRETE:

$$T_{cr} = 0.328 \cdot \sqrt{f_c} \cdot \frac{A_{cp}^{\ 2}}{p_c}$$

where:

T_u = factored torsional moment (N-mm)

T_{cr} = torsional cracking moment (N-mm)

ϕ = the strength reduction factor for shear (= 0.70)

f_c = compressive strength of concrete (MPa)

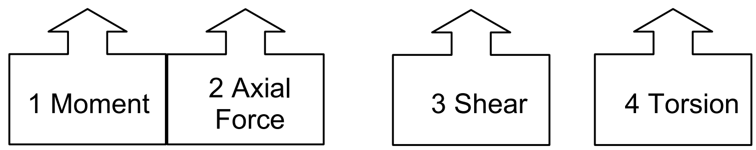
A_{cp} = total area enclosed by outside perimeter of the concrete section (mm)

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

Longitudinal Reinforcement

AASHTO LRFD Article 5.8.3.6.3 gives the provisions for longitudinal reinforcement. The , longitudinal reinforcement shall be proportioned such that, for REINFORCED CONCRETE:

$$A_s \cdot f_y \geq \frac{M_u}{\phi \cdot d_v} + \frac{0.5 \cdot N_u}{\phi} + \cot\theta \cdot \sqrt{\left(\frac{V_u}{\phi} - 0.5 \cdot V_s\right)^2 + \left(\frac{0.45 \cdot p_h \cdot T_u}{2 \cdot A_o \cdot \phi}\right)^2} \quad \text{........(Equation 12)}$$



The first two terms (1 Moment and 2 Axial Force) of Equation 12 above are taken into account in the flexural design of the section.

With regard to the third term (3 Shear), Article 5.8.3.5 specifies that the area of longitudinal reinforcement under combined moment and shear force need not exceed the area required to resist the maximum moment acting alone. Given that in all cases maximum shear force coincides with maximum moment in the abutment, the third term (3 Shear) will therefore not be considered.

Equation 12 is therefore simplified down to the following, with $\cot\theta = 1.0$:

$$A_s \geq \left(Design \cdot area \cdot from \cdot flexural \cdot design\right) + \frac{p_h \cdot T_u}{2 \cdot A_o \cdot \phi} \cdot \frac{1}{f_y} \quad \text{......................(Equation 13)}$$

Note that as a conservative approach the full value of additional reinforcement required for torsion is used.

ABUTMENTS 1

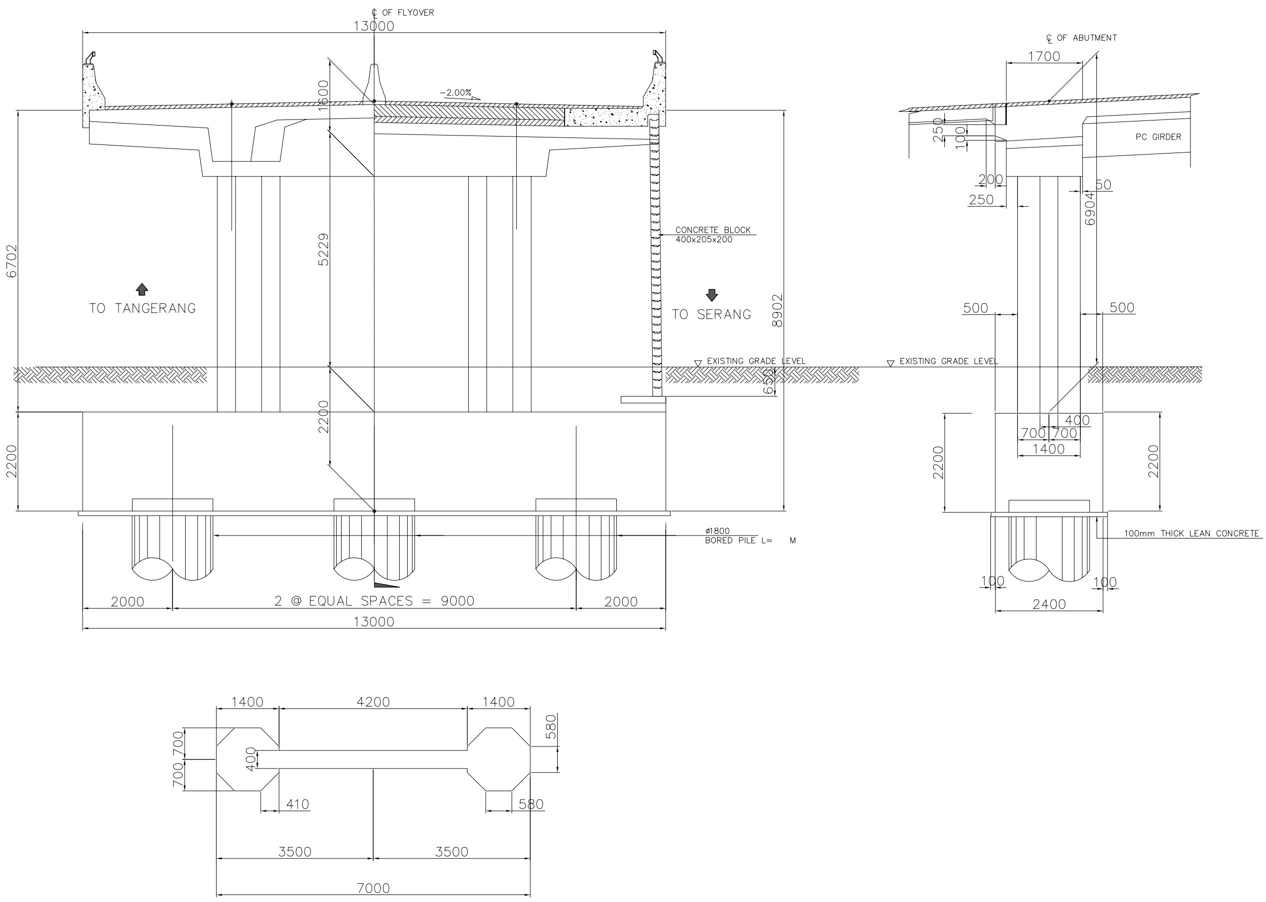
Design

KATAHIRA & ENGINEERS
INTERNATIONAL

Project: **Detailed Design Study of**
North Java Corridor Flyover Project

Calculation: **Detailed Design Substructure**
Balaraja Flyover
Abutment Design - A1

Layout



Initial Data

Compressive strength of concrete	$f_c := 30 \cdot MPa$
Yield strength of reinforcement	$f_y := 390 \cdot MPa$
Effective abutment width (section monlithic with deck)	$b := 7000mm$
Abutment wall thickness	$h := 400mm$
Abutment column size	$d := 1400mm$
Total area of section	$A_c := 4.927 \cdot m^2$
Resistance factor for bending	$\phi_b := 0.8$
Resistance factor for compression	$\phi_c := 0.7$
Resistance factor for shear and torsion (SPIRALS)	$\phi_{ss} := 0.7$
Resistance factor for shear and torsion (HOOPS)	$\phi_{sh} := 0.7$
Concrete cover	$cover := 40 \cdot mm$
Diameter of shear /torsion link	$\phi_{link} := 19mm$
Diameter of ties	$\phi_{tie} := 13mm$
Angle of crack for reinforced concrete	$\theta := 45deg$
Total area of concrete section - stem	$A_{cp} := 4.927 \cdot m^2$
Length of outside perimeter - stem	$p_c := 16877 \cdot mm$
Height of abutment stem - from top of pile cap to deck diaphragm beam	$H_{abut} := 5229mm$

Abutment Stem - Design for Service Loads

The integral abutment is subjected to large service moments given that the abutment stem carries all out of balance moments from dead load and live load from the deck.

Limiting tensile stresses in the abutment stem is therefore the governing condition in the design of the longitudinal reinforcement - in particular the case where the abutment is subjected to the effects of vertical traffic load only (with no overstress allowance) is the critical condition.

A summary of the service design result is presented below. The half traffic case is not presened given that this is not a critical condition for the abutment design.

See separate calculations for the detailed analysis of the sections under service loads.

Service Loads - Total for Abutment

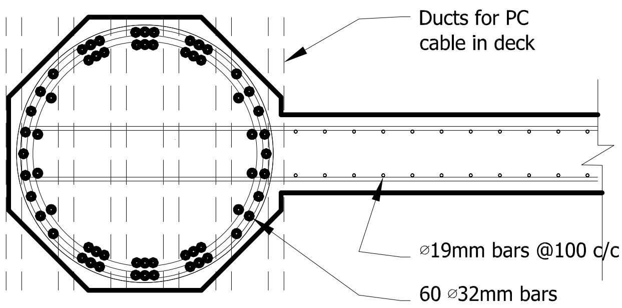
	Location	Case	Comb 1 - Full Live		Comb 1 - Traffic only	
			P AXIAL	M3 design moment longitudinal	P AXIAL	M3 design moment longitudinal
			KN	KN-m	KN	KN-m
Total for Abutment	Top	min	1472.9	1103.4	1569.0	424.2
		max	4832.1	10022.1	4735.9	9342.9
	Base	min	2104.2	383.8	2200.3	920.7
		max	5463.4	6474.9	5367.2	5938.0
Demand per column	Top	max	2416	5011	2368	4671
	Base	max	2732	3237	2684	2969

Note that for the above cases:

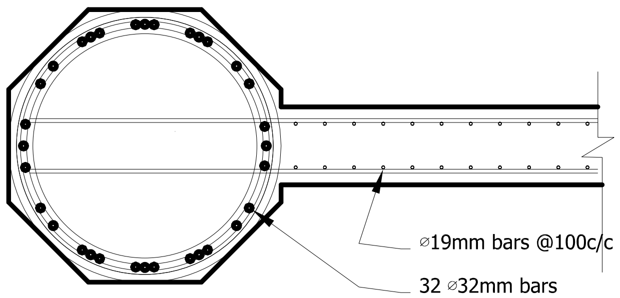
- maximum axial loads have been adopted fo the Combination 1 cases - given that as an end span, the maximum moment from traffic load will closely correspond with maximum axial load from traffic load
- longitudinal design moments only have been analyzed given that transverse moments under Combination 1 loading are relatively small and carried by the wide abutment section.

A reinforcement arrangement has been developed at both the base and at the top section of the abutment to limit tensile stresses in service. The layouts take into account the location of PC ducts in the deck and the layout of the pile cap reinforcement. The top section is more highly loaded than the base section and therefore requires greater number of reinforcing bars.

The reinforcement arrangements adopted at each section to limit tensile stresses in the reinforcement under longitudinal moments are presented below:



SECTION AT TOP



SECTION AT BASE

Abutment Stem - Flexural Design (AASHTO LRFD Section 5.7)

Ultimate Factored Loads - Total for Abutment

Case	Location	Comb 1 - Full Live		Comb 1 - 1/2 Live		Comb 5 EQX		Comb 5 EQY	
		P AXIAL	MD design moment	P AXIAL	MD design moment	P AXIAL	MD design moment	P AXIAL	MD design moment
		KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m
Case 1	TOP X	-7170.9	11953.8	-5058.3	8179.9	-1546.6	7860.5	-1878.5	5113.7
	TOP Y	-7170.9	1009.7	-5058.25	6049.6	-1546.6	1029.0	-1878.5	2340.2
Case 2	BASE X	-7991.6	7778.0	-5878.9	5831.0	-2177.9	5145.9	-2509.8	3246.6
	BASE Y	-7991.6	1006.1	-5878.9	6008.1	-2177.9	5287.0	-2509.8	12076.7

Note that for the above cases:

- maximum axial loads have been adopted fo the Combination 1 cases - given that as an end span, the maximum moment from traffic load will closely correspond with maximum axial load from traffic load
- minimum axial load from earthquake have been adopted fo the Combination 5 cases - the level of axial load applied given the column section will make minimum axial load cases always critical.

Defining axial load cases as follows:

P1	=	Axial load Combination 1 - Full Live Load
P11	=	Axial load Combination 1 - 1/2 Live Load
P5X	=	Axial load Combination 5 - EQX
P5Y	=	Axial load Combination 5 - EQY

Factored axial resistance

$$P_r := 0.1\phi_c f_c \cdot A_c \qquad P_r = 10347\,kN$$

$$\frac{P1}{P_r} = \begin{pmatrix} 0.693 \\ 0.693 \\ 0.772 \\ 0.772 \end{pmatrix} \qquad \frac{P11}{P_r} = \begin{pmatrix} 0.489 \\ 0.489 \\ 0.568 \\ 0.568 \end{pmatrix} \qquad \frac{P5X}{P_r} = \begin{pmatrix} 0.149 \\ 0.149 \\ 0.21 \\ 0.21 \end{pmatrix} \qquad \frac{P5Y}{P_r} = \begin{pmatrix} 0.182 \\ 0.182 \\ 0.243 \\ 0.243 \end{pmatrix}$$

With reference to AASHTO LRFD Article 5.7.4.5, Biaxial Flexure, if the factored axial load is less than $0.10\phi f_c A_g$ then the section shall satisfy: **$M_{ux}/M_{rx} + M_{uy}/M_{ry} \leq 1.0$** (i.e. the section shall be treated as a flexural member - not a compression member)

As can be seen from the ratios calculated above, the ratio of applied ultimate axial load to the resistance is always less than $0.10\phi f_c A_g$. Therefore the design will check the section for biaxial flexure as defined above.
To account for additional demand from torsion effects - a number of bars were excluded from the above layouts in the section analysis at both the top section and base section to reserve capacity for applied torsion. (refer below under Torsion and Shear Design).

The biaxial checks for each case are presented below. Refer to attached sheets for PCACOL results. The reinforcement arangement is accepted in each case - the low biaxial bending ratios are indicative of the impact service load considerations have had on the design .

Case	Location	Comb 1 - Full Live		Comb 1 - 1/2 Live		Comb 5 EQX		Comb 5 EQY	
		Mu applied moment	Mr resist moment	Mu applied moment	Mr resist moment	Mu applied moment	Mr resist moment	Mu applied moment	Mr resist moment
		KN-m	KN-m	KN-m	KN-m	KN-m	KN-m	KN-m	KN-m
Case 1	TOP X	11953.8	24083.6	8179.9	23597.4	7860.5	22690.5	5113.7	22777.4
	TOP Y	1009.7	158790.0	6049.6	153291.2	1029.0	144056.3	2340.2	144932.7
BIAXIAL CHECK		0.50		0.39		0.35		0.24	
Case 2	BASE X	7778.0	17311.6	5831.0	16778.1	5145.9	15786.2	3246.6	15880.3
	BASE Y	1006.1	114983.9	6008.1	109369.2	5287.0	99414.3	12076.7	100311.4
BIAXIAL CHECK		0.46		0.40		0.38		0.32	

Torsion Effects

Ultimate Factored Torsions

Case	Location	Comb 1 - Full Live	Comb 1 - 1/2 Live	Comb 5 - EQX	Comb 5 - EQY
		Torsion moment	Torsion moment	Torsion moment	Torsion moment
		KN-m	KN-m	KN-m	KN-m
Case 1	TOP X	41.0	318.1	822.3	2123.5
	TOP Y	41.0	318.1	822.3	2123.5
Case 2	BASE X	41.0	318.1	822.3	2123.5
	BASE Y	41.0	318.1	822.3	2123.5

$$T1 := T1 \cdot kN \cdot m \qquad T11 := T11 \cdot kN \cdot m \qquad T5X := T5X \cdot kN \cdot m \qquad T5Y := T5Y \cdot kN \cdot m$$

Note :
- Torsion moments have been modified using R=2 - adopting same appoach as for bending in walls
- X - Longitudinal Direction Y - Transverse Direction

Determine if torsional effects need to be investigated

Maximum ultimate torsion on abutment stem $T_u := \max(T1, T11, T5X, T5Y)$ $T_u = 2124\,kN\cdot m$

Torsional cracking moment $T_{cr} := 0.328\cdot\sqrt{\frac{f_c}{MPa}}\cdot\frac{A_{cp}^2}{p_c}\cdot MPa$ $T_{cr} = 2584\,kN\cdot m$

For normal density concrete, torsional effects shall be investigated where - for hoop reinforcement: $T_u > 0.25\cdot\phi_{sh}\cdot T_{cr}$

$Torsion_{Check} :=$ "NECESSARY" if $T_u > 0.25\cdot\phi_{sh}\cdot T_{cr}$; "NOT Required" otherwise $Torsion_{Check} =$ "NECESSARY"

Determine longitudinal reinforcement required to resist torsion

Radius of shear link of column $R := \frac{d}{2} - cover - \frac{\phi_{link}}{2}$ $R = 651\,mm$

Area enclosed within centerline of transverse rebar $A_{oh} := \pi R^2$

Area enclosed by shear flow path $A_o := 0.85A_{oh}$

Perimeter of centerline of closed transverse torsion reinforcement $p_h := \pi\cdot R\cdot 2$ $p_h = 4087\,mm$

Additional area, A_{ts}, per column required to resist torsion, T/2 per abutment column section for hoops, given by: $A_{ts}(T) := \left(\frac{p_h\cdot\frac{T}{2}}{2\cdot A_o\cdot\phi_{ss}}\right)\frac{1}{f_y}$

Note that torsion reduced by 1/2 given that each column will resist half torsion effect

Summary of additional reinforcement area and number of additional ϕ32mm bars per column required for torsion:

Case	Location	Comb 1 - Full Live		Comb 1 - 1/2 Live		Comb 5 EQX		Comb 5 EQY	
		Area A_{ts}	No. of bars	Area A_{ts}	No. of bars	Area A_{ts}	No. of bars	Area A_{ts}	No. of bars
		mm2	n	mm2	n	mm2	n	mm2	n
Case 1	TOP X	135.6	0.2	1053.5	1.3	2723.8	3.4	7033.9	8.7
	TOP Y	135.6	0.2	1053.5	1.3	2723.8	3.4	7033.9	8.7
Case 2	BASE X	135.6	0.2	1053.5	1.3	2723.8	3.4	7033.9	8.7
	BASE Y	135.6	0.2	1053.5	1.3	2723.8	3.4	7033.9	8.7

Plastic Hinge Effects

Results from PCACOL for reinforcement arrangement adopted using axial load due to dead load and superimposed dead load, P_p , with load factor 1.0:

Top Section $P1_{up} := (1818.1 + 280.1)\cdot kN$ $P1_{up} = 2098\,kN$

Base Section $P2_{up} := (2449.4 + 280.1)\cdot kN$ $P2_{up} = 2730\,kN$

Reversible Plastic Hinge Moment - longitudinal case:

Top Section $M1_p := 22834.7\cdot 1.3\cdot kN\cdot m$

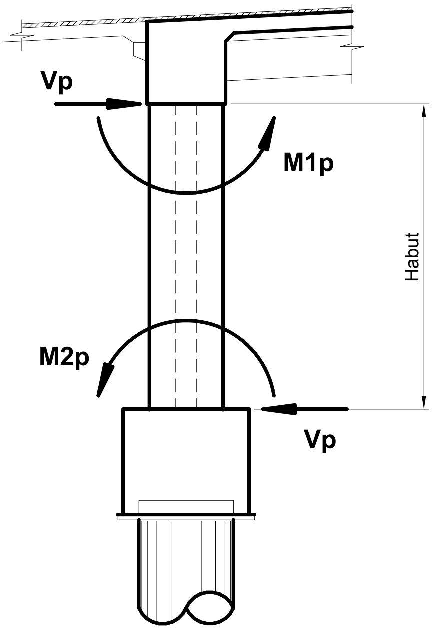
Base Section $M2_p := 15942.6\cdot 1.3\cdot kN\cdot m$

Height of abutment from base hinge to the deck diaphragm beam

$H_{abut} = 5229\,mm$

Reversible longitudinal shear force due to plastic hinging:

$$V_p := \frac{M1_p + M2_p}{H_{abut}}$$



$M1_p = 29685\,kN\cdot m$

$M2_p = 20725\,kN\cdot m$

$V_p = 9641\,kN$

Note that the transverse case has not been investigated. The wide section of the abutment in the transverse direction will generate plastic hinge effects that will be an order of magnitude greater than the elastic forces - therefore in all cases the elastic forces will be used in the design of transverse shear in the abument stem and for the design of foundations under transverse earthquake load, modified by Reponse Modification Factor.

Design Earthquake Effects for Shear, Deck Connections and Foundations

In the design of shear capacity, deck connections and in the design of the foundations, AASHTO LRFD allows the use either of the forces obtained from the elastic seismic analyis, modified by Response Modification Factor R, or the forces obtained from plastic hinging, whichever is the lowest. Response Modification Factor R=1.0 both for shear design, foundation design and deck connection design.

Given the amount of longitudinal reinforcement required in the abutment columns sections to control tensile stresses in service, the plastic hinge forces are very large. The forces obtained from the elastic seismic analysis with R=1.0 are presented below for the abutment for comparison:

$j := 1 .. 4$

	COMBINATION 5 - 1.0 EQX + 0.3 EQY (R=1.0)						
Location	***Case***	**P AXIAL**	**V2 SHEAR LONG**	**V3 SHEAR TRANS**	**T TORSION**	**M2 MOMENT TRANS**	**M3 MOMENT LONG**
		KN	KN	KN	KN-m	KN-m	KN-m
Base	Max	-1074.8	4181.1	1629.6	1644.1	10575.7	11466.1
Top	Max	-443.5	4181.1	1629.6	1644.1	2057.9	17035.4
Base	Min	-4384.2	-4673.5	-1630.2	-1643.1	-10579.1	-7494.5
Top	Min	-3752.9	-4673.5	-1630.2	-1643.1	-2057.9	-10489.4

$P_{EX} := -P_{EX} \cdot kN \qquad V2_{EX} := V2_{EX} \cdot kN \qquad V3_{EX} := V3_{EX} \cdot kN$

$T_{EX} := T_{EX} \cdot kN \cdot m \qquad M2_{EX} := M2_{EX} \cdot kN \cdot m \qquad M3_{EX} := M3_{EX} \cdot kN \cdot m$

	COMBINATION 5 - 0.3 EQX + 1.0 EQY (R=1.0)						
Location	***Case***	**P AXIAL**	**V2 SHEAR LONG**	**V3 SHEAR TRANS**	**T TORSION**	**M2 MOMENT TRANS**	**M3 MOMENT LONG**
		KN	KN	KN	KN-m	KN-m	KN-m
Base	Max	-2070.4	1527.5	3725.9	4246.5	24155.0	5768.3
Top	Max	-1439.1	1527.5	3725.9	4246.5	4680.4	8795.2
Base	Min	-3388.6	-2019.9	-3726.5	-4245.5	-24158.4	-1796.7
Top	Min	-2757.3	-2019.9	-3726.5	-4245.5	-4680.4	-2249.2

$P_{EY} := -P_{EY} \cdot kN \qquad V2_{EY} := V2_{EY} \cdot kN \qquad V3_{EY} := V3_{EY} \cdot kN$

$T_{EY} := T_{EY} \cdot kN \cdot m \qquad M2_{EY} := M2_{EY} \cdot kN \cdot m \qquad M3_{EY} := M3_{EY} \cdot kN \cdot m$

From inspection above it can be seen that the forces from longitudinal plastic hinging are substantially greater than the elastic forces from the seismic analysis with R=1.0.

Use the lowest forces for the shear and torsion design of the abutment stem, deck connection design and foundation design.

For the longitudinal case:

Design ultimate shear force $Vx_u := \min\left(V_p, \max\left(\sqrt{V2_{EX}^2}\right)\right)$ $Vx_u = 4673\,kN$

Design ultimate moment - deck $MD_u := \min\left[M1_p, \max\left[\sqrt{\left(M3_{EX_2}\right)^2}, \sqrt{\left(M3_{EX_4}\right)^2}\right]\right]$ $MD_u = 17035\,kN \cdot m$

Design ultimate moment - base $MB_u := \min\left[M2_p, \max\left[\sqrt{\left(M3_{EX_1}\right)^2}, \sqrt{\left(M3_{EX_3}\right)^2}\right]\right]$ $MB_u = 11466\,kN \cdot m$

Design for Shear and Torsion (AASHTO LRFD Section 5.8)

Longitudinal Shear - Plastic Hinge Zone

Within the plasic hinge zone, ignore strength of concrete in shear and carry entire shear by the reinforcement .

The required amount of tie steel is as follows:

Area of tie steel provided

$\phi_{link} = 19\,mm$ $\qquad A_v := \pi \cdot \frac{\phi_{link}^2}{4} \cdot 2$ $\qquad A_v = 567\,mm^2$

Effective depth of section $\qquad d_e := \frac{d}{2} + \frac{d - 2(cover + \phi_{link}) - 16mm}{\pi}$ $\qquad d_e = 1103\,mm$

Effective shear depth $\qquad d_v := 0.9 \cdot d_e$ $\qquad d_v = 993\,mm$

Spacing of tie steel required $\qquad s_t := A_v \cdot \left(\frac{\frac{Vx_u}{2}}{\phi_{ss}} \cdot \frac{1}{f_y \cdot d_v} \right)^{-1}$ $\qquad$ Note that plastic hinge shear reduced by 1/2 given that each column will resist half shear effect

$s_t = 66\,mm$

<u>Provide 19mmϕ spirals with a spacing between ties of 60mm</u>. Note that the maximum allowable spacing in a plastic hinge zone is 150mm.

$s_t := 60 \cdot mm$

Calculate the volumetric ratio of spiral reinforcement:

$$\rho := \frac{A_v}{2}\left(d - cover \cdot 2 - \phi_{link}\right)\frac{4}{s_t \cdot d^2} \qquad \rho = 0.012547$$

<u>Check transverse reinforcement for confinement:</u>

Area of column core $\qquad A_c := \pi \frac{(d - 2 \cdot cover)^2}{4}$

Gross area of concrete $\qquad A_g := \pi \frac{d^2}{4}$

For a circular column, the the volumetric ratio of spiral reinforcement shall be greater than either ρ_{s1} or ρ_{s2} as defined below:

$\rho_{s1} := 0.45 \cdot \frac{f_c}{f_y} \cdot \left(\frac{A_g}{A_c} - 1 \right)$ $\qquad \rho_{s1} = 0.0043$

$\rho_{s2} := 0.12 \cdot \frac{f_c}{f_y}$ $\qquad \rho_{s2} = 0.0092$

Given that ρ is greater than either ρ_{s1} and ρ_{s2} as defined above, accept design to satisfy confinement requirements

Longitudinal Shear - Abutment Stem

Effective shear depth:

$$d_v = 993\,mm$$

Calculate nominal shear resistance of concrete section - two column sections:

$$V_c := 0.166 \cdot \sqrt{\frac{f_c}{MPa}} \cdot d \cdot d_v \cdot 2 \cdot MPa \qquad V_c = 2527\,kN$$

Required nominal shear resistance of transverse reinforcement under plastic hinging:

$$V_s := \frac{Vx_u}{\phi_{ss}} - V_c \qquad V_s = 4149\,kN$$

Check that required total required nominal strength does not exceed limit, V_n:

$$V_n := 0.25 \cdot f_c \cdot b \cdot d_v \qquad V_n = 52116\,kN$$

$$Shear_{Limit} := \begin{cases} \text{"OK"} & \text{if } V_s + V_c \le V_n \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases} \qquad Shear_{Limit} = \text{"OK"}$$

Area of tie steel provided

$$\phi_{link} = 19\,mm \qquad A_v := \pi \cdot \frac{\phi_{link}^{\,2}}{4} \cdot 2 \qquad A_v = 567\,mm^2$$

Determine required spacing of transverse reinforcement

$$s_t := A_v \cdot \left(\frac{\frac{V_s}{2}}{f_y \cdot d_v} \right)^{-1} \qquad s_t = 106\,mm$$

Note that required shear reduced by 1/2 given that each column will resist half shear effect

Provide 19mmϕ spirals with a spacing between ties of 100mm. Note that the maximum allowable spacing of spiral transverse reinforcement is 100mm.

Calculate the volumetric ratio of spiral reinforcement:

$$\rho := A_v \left(d - cover \cdot 2 - \phi_{link} \right) \frac{2}{s_t \cdot d^2} \qquad \rho = 0.0071$$

Check minimum transverse reinforcement:

The ratio of tie reinforcement to total volume of concrete core, ρ, shall be greater than that defined below:

$$\rho_{s1} := 0.45 \cdot \frac{f_c}{f_y} \cdot \left(\frac{A_g}{A_c} - 1 \right) \qquad \rho_{s1} = 0.00432$$

Given that ρ is greater than either ρ_{s1} as defined above, accept design to satisfy confinement requirements

Ties across Infill Wall

Maximum vertical spacing of ties shall be 300mm.

Maximum transverse spacing of ties shall be such that no bar is further than 610mm center-to-center on each side of a laterally supported bar.

Provide 13mm ties at 300mm c/c vertically and 1000mm c/c horizontally

Torsion Check

The area of longitudinal reinforcement to resist torsion in the stem has been taken account in the flexural design of the abutment stem. Transverse reinfocement will be included in the transverse shear and torsion check below.

Transverse Shear - Abutment Stem

Maximum transverse shear on abutment from elastic seismic analysis, R=1:

$$Vy_u := \max\left(\sqrt{V3_{EY}^2}\right) \qquad Vy_u = 3727\,kN$$

Effective shear depth:

$$d_v := 0.9\cdot\left(b - \frac{d}{2}\right)$$

$$d_v = 5670\,mm$$

Calculate nominal shear resistance of concrete section assuming beam section - taken through 400mm infill wall:

$$V_c := 0.166\cdot\sqrt{\frac{f_c}{MPa}}\cdot h\cdot d_v\cdot MPa$$

$$V_c = 2062\,kN$$

Required nominal shear resistance of transverse reinforcement - assuming hoop reinforcement:

$$V_s := \frac{Vy_u}{\phi_{sh}} - V_c \qquad V_s = 3261\,kN$$

Provide 19mmϕ shear links

$$A_v := \pi\frac{\phi_{link}^2}{4}\cdot 2 \qquad A_v = 567\,mm^2$$

Determine required spacing of transverse reinforcement

$$s_{t1} := \frac{A_v\cdot f_y\cdot d_v}{V_s} \qquad s_{t1} = 384\,mm$$

Maximum torsion moment on abutment - assuming same response modification factor for bending moment on walls, R=2:

$$T_u := \max\left(\sqrt{T_{EY}^2}\right)\cdot\frac{1}{2} \qquad T_u = 2123\,kN\cdot m$$

Area enclosed with centerline of transverse rebar

$$A_{oh} := \left(h - 2\cdot cover - \phi_{link}\right)\cdot\left(b - 2\cdot cover - \phi_{link}\right)$$

Area enclosed by shear flow path

$$A_o := 0.85A_{oh}$$

Determine required transverse reinforcement for torsion:

Area of one leg of transverse torsion reinforcement $\quad A_t := \pi\frac{\phi_{link}^{\;2}}{4} \qquad A_t = 284\,mm^2$

Required spacing of torsional reinforcement $\quad s_{t2} := \frac{2\cdot A_o\cdot A_t\cdot f_y\cdot \cot(\theta)}{T_u}\cdot\phi_{sh} \qquad s_{t2} = 128.731\,mm$

Required combined spacing of transverse reinforcement for shear and torsion:

$$s_t := \left(\frac{1}{s_{t1}} + \frac{1}{s_{t2}}\right)^{-1} \qquad s_t = 96\,mm$$

Provide transverse reinforcement at 90mm c/c

$$s_t := 90mm$$

<u>Check requirements for wall type piers (AASHTO LRFD Article 5.10.11.4.2)</u>

The minimum reinforcement ratio, both horizontally and vertically shall not be less than 0.0025. Spacing shall not exceed 450mm.

Calculate reinforcement ratio horizontal: $\quad \rho_h := \frac{A_v}{s_t\cdot h} \qquad \rho_h = 0.01575$

Calculate reinforcement ratio vertical given ϕ19mm bars at 100mmc/c (see flexural design): $\quad \rho_v := \frac{\pi\cdot(9.5mm)^2\cdot 2}{100mm\cdot h} \qquad \rho_v = 0.01418$

Accept proposed reinforcement arrangement to satisfy design requirements

Check factored shear resistance for wall type pier, V_r:

$$V_n := \left(0.165\cdot\sqrt{\frac{f_c}{MPa}}\cdot MPa + \rho_h\cdot f_y\right)\cdot b\cdot h \qquad V_n = 52116\,kN$$

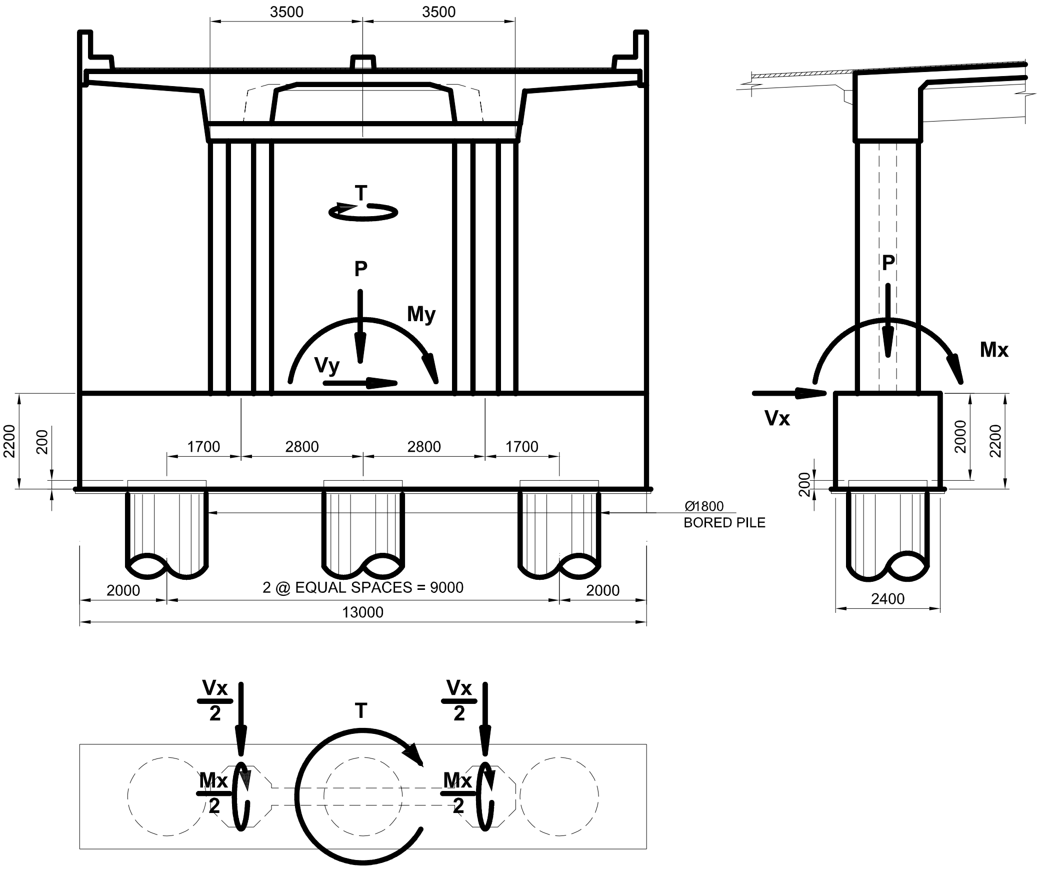
$$V_r := \begin{vmatrix} V_r \leftarrow 0.66\cdot\sqrt{\frac{f_c}{MPa}}\cdot MPa\cdot b\cdot h \\ V_r \text{ if } V_r \le \phi_{sh}\cdot V_n \\ \phi_{sh}\cdot V_n \text{ otherwise} \end{vmatrix} \qquad V_r = 10122\,kN$$

$$Shear_{Capacity} := \begin{vmatrix} \text{"OK" if } V_r \ge Vy_u \\ \text{"INADEQUATE" otherwise} \end{vmatrix} \qquad Shear_{Capacity} = \text{"OK"}$$

Accept proposed wall arrangement to satisfy design requirements

requirements

Pile Cap Design



The critical load cases for the design of the pile cap are:

1. Transverse elastic earthquake effects with R=1.0 - creating maximum demand Vy My and torsion Ty

2. The lowest of either longitudinal plastic hinging effects or longitudinal elastic earthquake effects with R=1.0 - creating maximum demand Vx Mx and torsion Tx

1. Determine Maximum Axial Load and Shear Force on Pile

Effective depth of pile cap $d_{cap} := 2000mm$

Width of pile cap $b_{cap} := 2400mm$

Length of pile cap $L_{cap} := 13000mm$

Pile spacing $s_p := 1.8m \cdot 2.5$ $s_p = 4.5\ m$

Number of piles $n_{piles} := 3$

Weight of pile cap $W_{cap} := L_{cap} \cdot (d_{cap} + 200mm) \cdot b_{cap} \cdot 24.5 \frac{kN}{m^3}$ $W_{cap} = 1682\ kN$

Applied ultimate loads - : **transverse elastic earthquake** effects with R=1.0 - Combinaton 5 (0.3EQX + 1.0EQY)

Design max axial force - base $PYmax_u := max(P_{EY_1}, P_{EY_3})$ $PYmax_u = 3389\ kN$

Design min axial force - base $PYmin_u := min(P_{EY_1}, P_{EY_3})$ $PYmin_u = 2070\ kN$

Design ultimate shear force - base (transverse) $Vy_u := max\left(\sqrt{V3_{EY}^2}\right)$ $Vy_u = 3727\ kN$

Design ultimate shear force - base (longitudinal) $Vyx_u := max\left(\sqrt{V2_{EY}^2}\right)$ $Vyx_u = 2020\ kN$

Design ultimate moment - base (transverse) $MBy_u := max\left[\sqrt{(M2_{EY_1})^2}, \sqrt{(M2_{EY_3})^2}\right]$ $MBy_u = 24158\ kN \cdot m$

Design ultimate torsion - base $Ty_u := max\left[\sqrt{(T_{EY_1})^2}, \sqrt{(T_{EY_3})^2}\right]$ $Ty_u = 4247\ kN \cdot m$

Applied ultimate loads - : **longitudinal elastic earthquake** effects with R=1.0 - Combinaton 5 (1.0EQX + 0.3EQY)

Design max axial force - base $PXmax_u := max(P_{EX_1}, P_{EX_3})$ $PXmax_u = 4384\ kN$

Design min axial force - base $PXmin_u := min(P_{EX_1}, P_{EX_3})$ $PXmin_u = 1075\ kN$

Design ultimate shear force - base (longitudinal) $Vx_u := max\left(\sqrt{V2_{EX}^2}\right)$ $Vx_u = 4673\ kN$

Design ultimate shear force - base (transverse) $Vxy_u := max\left(\sqrt{V3_{EX}^2}\right)$ $Vxy_u = 1630\ kN$

Design ultimate moment - base (transverse) $MBx_u := max\left[\sqrt{(M2_{EX_1})^2}, \sqrt{(M2_{EX_3})^2}\right]$ $MBx_u = 10579\ kN \cdot m$

Design ultimate torsion - base $Tx_u := max\left[\sqrt{(T_{EX_1})^2}, \sqrt{(T_{EX_3})^2}\right]$ $Tx_u = 1644\ kN \cdot m$

Determine Maximum/Minimum axial load on piles - from transverse EQY case

Maximum pile load $PY_{max} := \frac{W_{cap}}{n_{piles}} + \frac{PYmax_u}{n_{piles}} + \frac{MBy_u}{s_p \cdot (n_{piles} - 1)} + \frac{Vy_u \cdot d_{cap}}{s_p \cdot (n_{piles} - 1)}$ $PY_{max} = 5202\,kN$

Minimum pile load $PY_{min} := \frac{W_{cap}}{n_{piles}} + \frac{PYmin_u}{n_{piles}} - \frac{MBy_u}{s_p \cdot (n_{piles} - 1)} - \frac{Vy_u \cdot d_{cap}}{s_p \cdot (n_{piles} - 1)}$ $PY_{min} = -2262\,kN$

Determine Maximum/Minimum axial load on piles - from longitudinal EQX case

Maximum pile load $PX_{max} := \frac{W_{cap}}{n_{piles}} + \frac{PXmax_u}{n_{piles}} + \frac{MBx_u}{s_p \cdot (n_{piles} - 1)} + \frac{Vxy_u \cdot d_{cap}}{s_p \cdot (n_{piles} - 1)}$ $PX_{max} = 3560\,kN$

Minimum pile load $PX_{min} := \frac{W_{cap}}{n_{piles}} + \frac{PXmin_u}{n_{piles}} - \frac{MBx_u}{s_p \cdot (n_{piles} - 1)} - \frac{Vxy_u \cdot d_{cap}}{s_p \cdot (n_{piles} - 1)}$ $PX_{min} = -619\,kN$

Determine maximum shear force on pile including longitudinal and torsional effects - transverse EQY case

Longitudinal shear force per pile $Vyx_u := \frac{Ty_u}{s_p \cdot (n_{piles} - 1)} + \frac{Vyx_u}{n_{piles}}$ $Vyx_u = 1145\,kN$

Total shear force on pile - transverse earthquake case $Vyt_u := \sqrt{\left(\frac{Vy_u}{n_{piles}}\right)^2 + Vyx_u^2}$ $Vyt_u = 1689\,kN$

Determine maximum shear force on pile including transverse and torsional effects - longitudinal EQX case

Longitudinal shear force per pile $Vxx_u := \frac{Tx_u}{s_p \cdot (n_{piles} - 1)} + \frac{Vx_u}{n_{piles}}$ $Vxx_u = 1741\,kN$

Total shear force on pile - longitudinal earthquake case $Vxt_u := \sqrt{\left(\frac{Vxy_u}{n_{piles}}\right)^2 + Vxx_u^2}$ $Vxt_u = 1823\,kN$

SUMMARY - PILE AXIAL LOAD AND SHEAR FORCE

Maximum axial load on pile $P_{max} := \max(PX_{max}, PY_{max})$ $P_{max} = 5202\,kN$

Minimum axial load on pile $P_{min} := \min(PX_{min}, PY_{min})$ $P_{min} = -2262\,kN$

Maximum shear on pile $V_{max} := \max(Vxt_u, Vyt_u)$ $V_{max} = 1823\,kN$

2. Design for Longitudinal Case

Applied ultimate loads:

Moment	$Mx := MB_u$	$Mx = 11466\,kN\cdot m$
Shear	$Vx := Vx_u$	$Vx = 4673\,kN$
Torsion	$Tx := Tx_u$	$Tx = 1644\,kN\cdot m$

Cover to pile cap rebar

$$cover_{cap} := 75mm$$

Longitudinal effects will create pile cap torsions and shear forces in transferring loads to the piles as follows:

$$V_{cap} := \left[\frac{Vx}{n_{piles}} + \frac{Tx}{s_p\cdot(n_{piles} - 1)}\right] \qquad V_{cap} = 1741\,kN$$

$$T_{cap} := \left(\frac{Mx}{2}\right)\cdot\frac{2800}{2800 + 1700} + V_{cap}\cdot\frac{d_{cap}}{2} \qquad T_{cap} = 5308\,kN\cdot m$$

Determine longitudinal reinforcement required to resist torsion

Radius of shear link $\phi_{link} := 25mm$

Area enclosed within centerline of transverse rebar

$$A_{oh} := \left[d_{cap} - (cover_{cap}\cdot 2) - \phi_{link}\right]\cdot\left[b_{cap} - (cover_{cap}\cdot 2) - \phi_{link}\right] \qquad A_{oh} = 4.061\ m^2$$

Area enclosed by shear flow path $A_o := 0.85A_{oh}$ $A_o = 3.452\ m^2$

Perimeter of centerline of closed transverse torsion reinforcement

$$p_h := \left[d_{cap} - (cover_{cap}\cdot 2) - \phi_{link}\right]\cdot 2 + \left[b_{cap} - (cover_{cap}\cdot 2) - \phi_{link}\right]\cdot 2$$

$$p_h = 8100\,mm$$

Area of longitudinal reinforcement, A_{ts}, required to resist torsion, T, given by:

$$A_{ts} := \left(\frac{p_h\cdot T_{cap}}{2\cdot A_o\cdot\phi_{sh}}\right)\frac{1}{f_y} \qquad A_{ts} = 22813\,mm^2$$

Using 32mmϕ bars gives total number of bars to be distributed around section:

$$n_{bars} := \frac{A_{ts}}{802\cdot mm^2} \qquad n_{bars} = 28.4$$

Provide 36 bars around perimeter of pile cap

$$n_{bars} := 36$$

Determine longitudinal reinforcement required to resist flexure in pile cap due to shear forces

Assuming simply supported beams spanning between piles, the maximum moment on the pile cap from the column shear forces is as follows:

$M_{cap} := V_{cap} \cdot 1.7m$ $\qquad M_{cap} = 2959\,kN \cdot m$

Effective depth of section across the cap:

$d_e := \left(b_{cap} - cover_{cap} - \phi_{link} - \frac{32mm}{2} \right)$ $\qquad d_e = 2284\,mm$

Determine area of additonal reinforcement, A_f, in the face of the cap to resist flexure:

$R := \frac{M_{cap}}{\phi_b \cdot d_{cap} \cdot d_e^2 \cdot f_y}$ $\qquad R = 0.0009$ $\qquad M := \frac{0.85 \cdot f_c}{f_y}$ $\qquad M = 0.0654$

$\rho := M \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R}{M}} \right)$ $\qquad \rho = 0.000915$

$A_f := \rho \cdot d_e \cdot d_{cap}$

$A_f = 4181.4\,mm^2$

Using 32mmϕ bars gives total number of additional bars to be distributed down the face of the pile cap:

$n_{bf} := \frac{A_f}{802 \cdot mm^2}$ $\qquad n_{bf} = 5.2$

Total number of ϕ32mm bars on vertical face of pile cap:

$n_v := n_{bars} \cdot \frac{2}{8.8} + n_{bf}$ $\qquad n_v = 13.396$

<u>Provide 14 no. ϕ32mm bars @125mm c/c each vertical face</u>

Total number of ϕ32mm bars on horizontal face of pile cap:

$n_h := n_{bars} \cdot \frac{2.4}{8.8}$ $\qquad n_h = 9.818$

<u>Provide 10 no. ϕ32mm bars each horizontal face</u>

Transverse Reinforcement for Shear and Torsion

Shear on pile cap from longitudinal load effects:

$$V_{cap} = 1741\,kN$$

Effective shear depth:

$$d_v := 0.9 \cdot d_e \qquad d_e = 2284\,mm$$

$$d_v = 2056\,mm$$

Calculate nominal shear resistance of concrete section assuming beam section:

$$V_c := 0.166 \cdot \sqrt{\frac{f_c}{MPa}} \cdot d_{cap} \cdot d_v \cdot MPa$$

$$V_c = 3738\,kN$$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \begin{vmatrix} V_s \leftarrow \dfrac{V_{cap}}{\phi_{sh}} - V_c \\ V_s \ \text{ if } \ V_s > 0kN \\ 0kN \ \text{ otherwise} \end{vmatrix}$$

$$V_s = 0\,kN$$

Provide 25mmϕ shear links

$$\phi_{link} := 25mm$$

$$A_v := \pi \frac{\phi_{link}^{\;2}}{4} \cdot 2 \qquad A_v = 982\,mm^2$$

Determine required spacing of transverse reinforcement

$$s_{t1} := \begin{vmatrix} \dfrac{A_v \cdot f_y \cdot d_v}{V_s} \ \text{ if } \ V_s > 0kN \\ \dfrac{A_v \cdot f_y}{0.083 \cdot \sqrt{\dfrac{f_c}{MPa}} \cdot MPa \cdot d_{cap}} \ \text{ otherwise} \end{vmatrix}$$

$$s_{t1} = 421\,mm$$

Note that the maximum spacing of transverse reinforcement for shear is 600mm - however the calculated value is carried forward to determine the combined shear and torsion spacing

Maximum torsion moment on abutment pile cap:

$$T_{cap} = 5308\,kN \cdot m$$

Provide 19ϕ shear links

$$\phi_{link} := 25mm$$

Area enclosed with centerline of transverse rebar

$$A_{oh} = 4.061\ m^2$$

Area enclosed by shear flow path

$$A_o := 0.85A_{oh}$$

Determine required transverse reinforcement for torsion:

Area of one leg of transverse torsion reinforcement $A_t := \pi \frac{\phi_{link}^2}{4}$ $A_t = 491\,mm^2$ $\phi_{link} = 25\,mm$

Required spacing of torsional reinforcement $s_{t2} := \frac{2 \cdot A_o \cdot A_t \cdot f_y \cdot \cot(\theta)}{T_{cap}} \cdot \phi_{sh}$ $s_{t2} = 174\,mm$

Calculate combined spacing of shear and torsion transverse reinforcement:

$$s_t := \begin{array}{|l} s_{t1} \leftarrow \left(\frac{1}{s_{t1}} + \frac{1}{s_{t2}}\right)^{-1} \\ s_{t1} \ \text{ if } \ V_s > 0\ kN \\ s_{t2} \ \text{ if } \ V_s = 0\ kN \\ 600mm \ \text{ if } \ (s_{t2} > 600mm) \cdot (V_s = 0\ kN) \\ 600mm \ \text{ if } \ (s_{t1} > 600mm) \cdot (V_s > 0\ kN) \end{array}$$

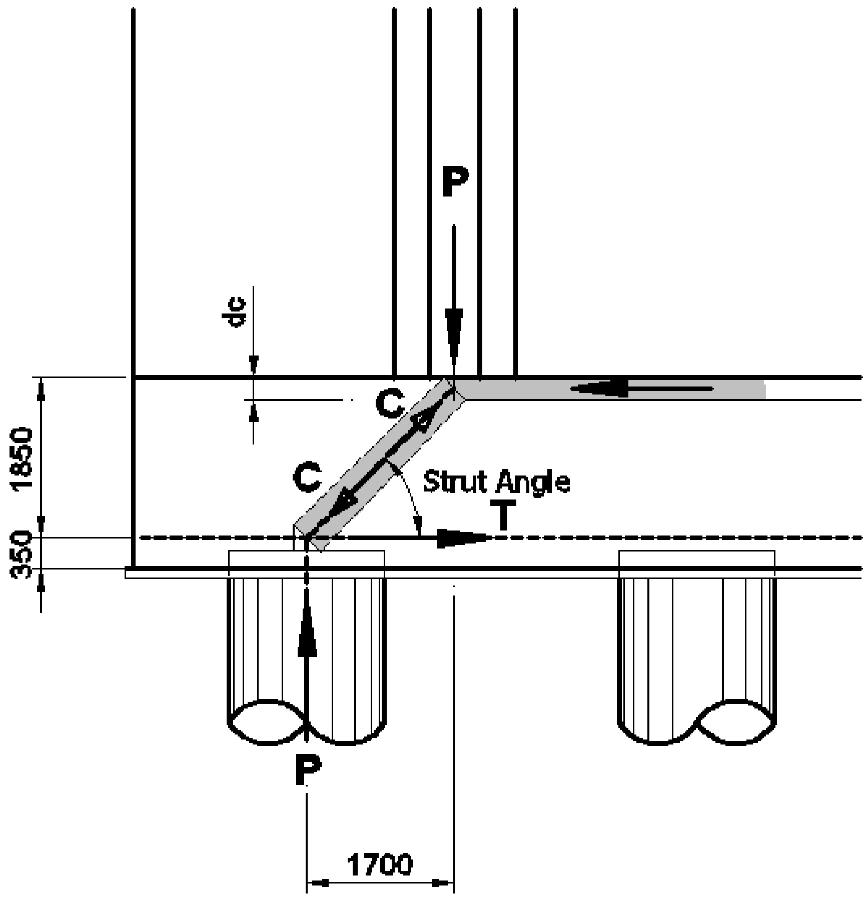
$$s_t = 174\,mm$$

Provide 19mm dia transverse reinforcement at 150mm c/c or closer spacing to suit pile rebar layout

3. Design Tie Steel in Pile Cap to Carry Maximum/Minimum Pile Loads

Determine required pile cap tension tie steel to support maximum/minimum reactions in pile

Using a strut and tie model in accordance with AASHTO LRFD Article 5.6.3:



Maximum pile load $P_{max} = 5202\,kN$

Minimum pile load $P_{min} = -2262\,kN$

Assume depth of horizonal concrete compression strut: $d_c := 200 \cdot mm$

Angle between compressive strut and tension tie: $\alpha_s := atan\left(\frac{1850mm - \frac{d_c}{2}}{1700mm}\right)$ $\alpha_s = 45.83\,deg$

Resistance factor for compression: (in strut and tie models) $\phi_c := 0.70$

Resistance factor for tension: $\phi_t := 0.8$

Area of tension reinforcement required in tensile tie:

Tensile tie force required:

$$T := \frac{\left(P_{max} - \frac{W_{cap}}{3}\right)\cdot\frac{1}{\tan(\alpha_s)}}{\phi_t} \qquad T = 5637\,kN$$

$$A_{st} := \frac{T}{f_y} \qquad A_{st} = 14453\,mm^2$$

Number of ϕ32mm bars required in tie:

$$n_{bt} := \frac{A_{st}}{804\cdot mm^2} \qquad n_{bt} = 18.0$$

Provide 18 number ϕ32mm bars in tension tie in central section + 2 number ϕ32mm bars at each side

20 **number ϕ32mm bars total over width of 1800 pile**

Strength of compressive strut:

The tensile strain in the concrete in the direction of the tension tie:

$\varepsilon_s := 0.002$ Limiting tensile strain of reinforcement

Design tensile strain across concrete strut:

$$\varepsilon_1 := \varepsilon_s + (\varepsilon_s + .002)\cdot\frac{1}{\tan(\alpha_s)^2} \qquad \varepsilon_1 = 0.005775$$

The limiting compressive stress of the concrete strut is then given by:

$$f_{cu} := \left| \begin{array}{l} f_{cu} \leftarrow \frac{f_c}{0.8 + 170\cdot\varepsilon_1} \\ f_{cu} \ \text{if} \ f_{cu} \le 0.85\cdot f_c \\ 0.85\cdot f_c \ \text{otherwise} \end{array} \right.$$

$$f_{cu} = 16.8\,MPa$$

Width of concrete strut at base limited to width of pile section at the base node:

$b_{cs} := 1800mm$

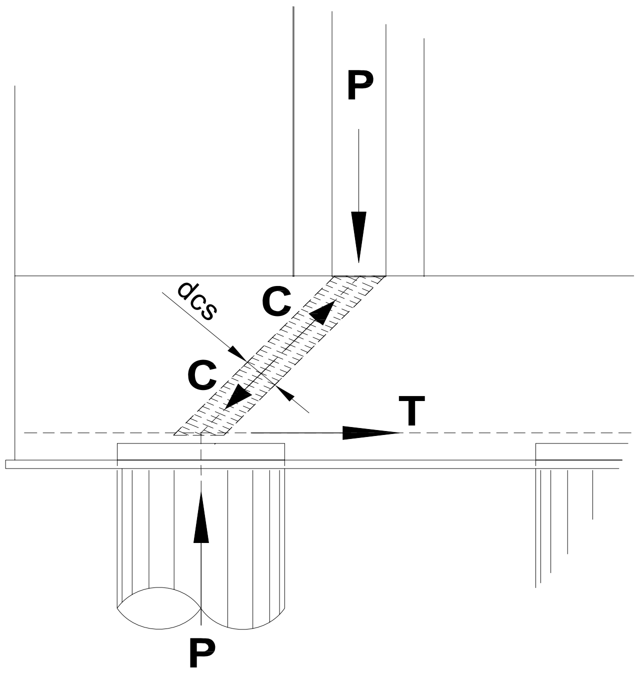
Compressive resistance force required in concrete strut:

$$C := \frac{P_{max}\cdot\frac{1}{\sin(\alpha_s)}}{\phi_c} \qquad C = 10362\,kN$$

Depth of concrete strut required:

$$d_{cs} := \frac{C}{f_{cu}} \frac{1}{b_{cs}}$$

$$d_{cs} = 342\,mm$$



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Conclude that the compressive strut can be comfortably accommodated within the available space provided by the pile cap.

Check depth of horizonal concrete compression strut:

Horizontal compressive force $C_H := C \cdot \cos(\alpha_s)$ $C_H = 7220\,kN$

Effective width of strut is the abutment column section width at the top node $b_{cs} := 1400mm$

Depth of concrete strut required:

$$d_{cs} := \frac{C_H}{0.85 \cdot f_c} \frac{1}{b_{cs}} \qquad d_{cs} = 202\,mm$$

Accept assumed depth

Area of tension reinforcement required in tensile tie for minimum tensile load - (reverse case):

Resistance tensile force required:

$$T := \frac{-\left(P_{min} \cdot \frac{1}{\tan(\alpha_s)} - \frac{W_{cap}}{3}\right)}{\phi_t} \qquad T = 3447\,kN$$

$$A_{st} := \frac{T}{f_y} \qquad A_{st} = 8839\,mm^2$$

Number of ϕ32mm bars required in tie:

$$n_{bt} := \frac{A_{st}}{802 \cdot mm^2} \qquad n_{bt} = 11$$

Provide 12 number ϕ32mm bars in tension tie in top layer

ABUTMENTS 2

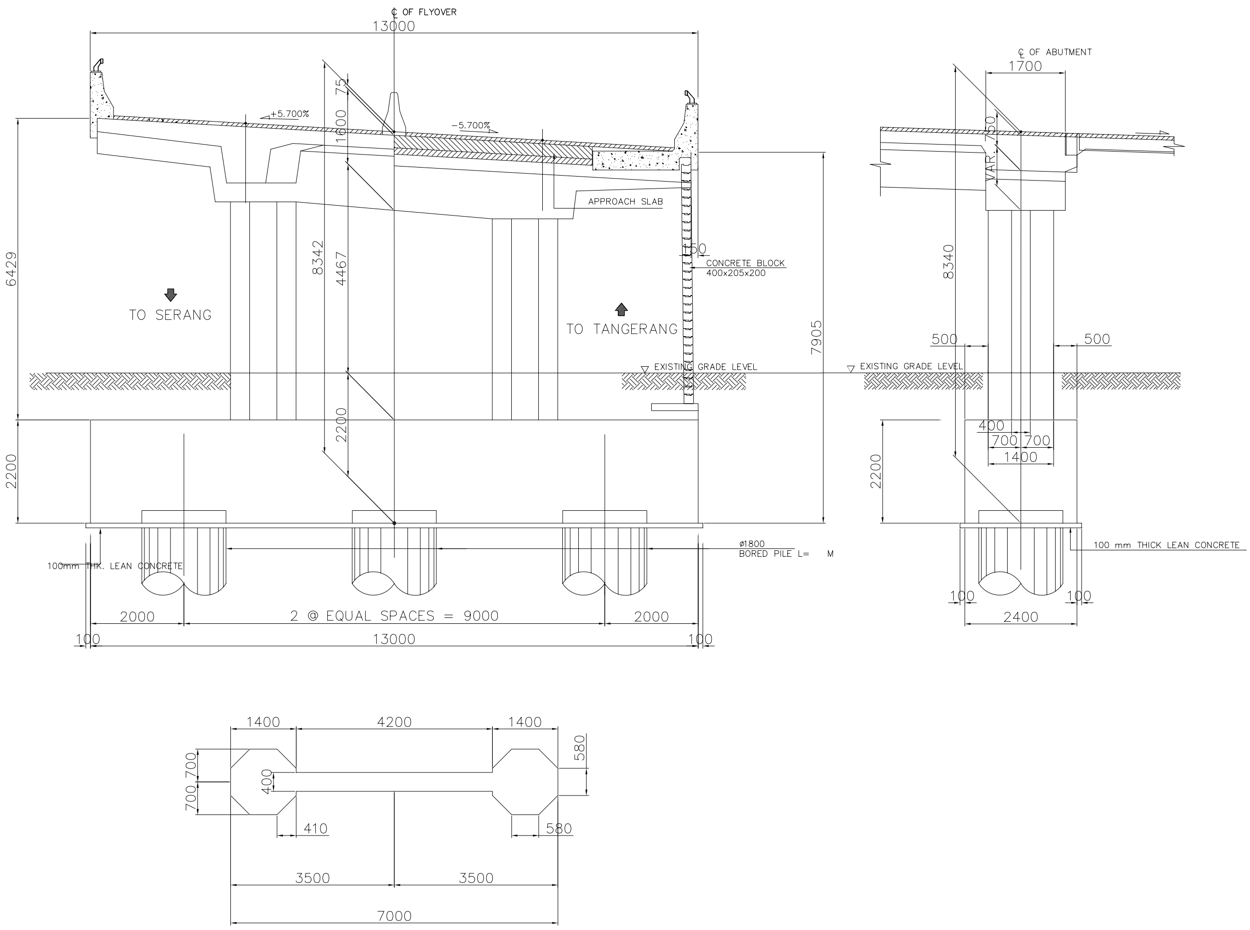
Design

KATAHIRA & ENGINEERS
INTERNATIONAL

Project: **Detailed Design Study of**
North Java Corridor Flyover Project

Calculation: **Detailed Design Substructure**
Balaraja Flyover
Abutment Design - A2

Layout



Initial Data

Compressive strength of concrete	$f_c := 30 \cdot MPa$
Yield strength of reinforcement	$f_y := 390 \cdot MPa$
Effective abutment width (section monlithic with deck)	$b := 7000mm$
Abutment wall thickness	$h := 400mm$
Abutment column size	$d := 1400mm$
Total area of section	$A_c := 4.927 \cdot m^2$
Resistance factor for bending	$\phi_b := 0.8$
Resistance factor for compression	$\phi_c := 0.7$
Resistance factor for shear and torsion (SPIRALS)	$\phi_{ss} := 0.7$
Resistance factor for shear and torsion (HOOPS)	$\phi_{sh} := 0.7$
Concrete cover	$cover := 40 \cdot mm$
Diameter of shear /torsion link	$\phi_{link} := 19mm$
Diameter of ties	$\phi_{tie} := 13mm$
Angle of crack for reinforced concrete	$\theta := 45deg$
Total area of concrete section - stem	$A_{cp} := 4.927 \cdot m^2$
Length of outside perimeter - stem	$p_c := 16877 \cdot mm$
Height of abutment stem - from top of pile cap to deck diaphragm beam	$H_{abut} := 4467mm$

Abutment Stem - Design for Service Loads

The integral abutment is subjected to large service moments given that the abutment stem carries all out of balance moments from dead load and live load from the deck.

Limiting tensile stresses in the abutment stem is therefore the governing condition in the design of the longitudinal reinforcement - in particular the case where the abutment is subjected to the effects of vertical traffic load only (with no overstress allowance) is the critical condition.

A summary of the service design result is presented below. The half traffic case is not presened given that this is not a critical condition for the abutment design.

See separate calculations for the detailed analysis of the sections under service loads.

Service Loads - Total for Abutment

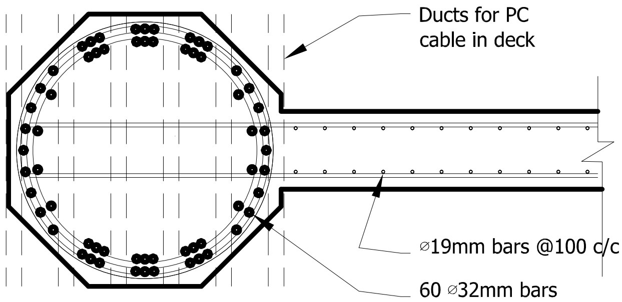
	Location	Case	Comb 1 - Full Live		Comb 1 - Traffic only	
			P AXIAL	M3 design moment longitudinal	P AXIAL	M3 design moment longitudinal
			KN	KN-m	KN	KN-m
Total for Abutment	Top	min	1355.6	1708.0	1480.5	878.7
		max	4824.3	10313.1	4699.4	9483.8
	Base	min	1894.9	598.8	2019.8	1191.9
		max	5363.6	7629.8	5238.7	7036.7
Demand per column	Top	max	2412	5157	2350	4742
	Base	max	2682	3815	2619	3518

Note that for the above cases:

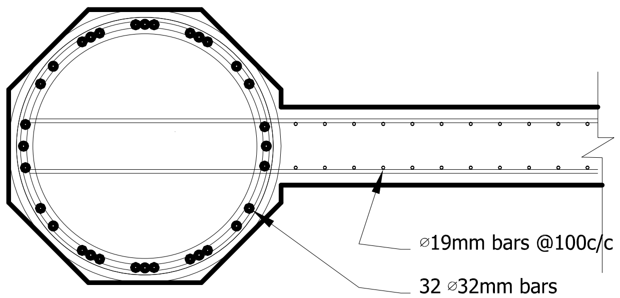
- maximum axial loads have been adopted fo the Combination 1 cases - given that as an end span, the maximum moment from traffic load will closely correspond with maximum axial load from traffic load
- longitudinal design moments only have been analyzed given that transverse moments under Combination 1 loading are relatively small and carried by the wide abutment section.

A reinforcement arrangement has been developed at both the base and at the top section of the abutment to limit tensile stresses in service. The layouts take into account the location of PC ducts in the deck and the layout of the pile cap reinforcement. The top section is more highly loaded than the base section and therefore requires greater number of reinforcing bars.

The reinforcement arrangements adopted at each section to limit tensile stresses in the reinforcement under longitudinal moments are presented below:



SECTION AT TOP



SECTION AT BASE

Abutment Stem - Flexural Design (AASHTO LRFD Section 5.7)

Ultimate Factored Loads - Total for Abutment

Case	Location	Comb 1 - Full Live		Comb 1 - 1/2 Live		Comb 5 EQX		Comb 5 EQY	
		P AXIAL	MD design moment	P AXIAL	MD design moment	P AXIAL	MD design moment	P AXIAL	MD design moment
		KN	KN-m	KN	KN-m	KN	KN-m	KN	KN-m
Case 1	TOP X	-7173.6	-12504.0	-5054.9	-9019.6	-1519.2	-8132.4	-1865.8	-5445.2
	TOP Y	-7173.6	-644.6	-5054.89	-5773.7	-1519.2	-1263.1	-1865.8	-3447.9
Case 2	BASE X	-7874.7	-9084.5	-5756.0	-7112.9	-2058.5	-5192.7	-2405.1	-3536.6
	BASE Y	-7874.7	-1619.3	-5756.0	-6651.1	-2058.5	-7402.3	-2405.1	-15694.3

Note that for the above cases:

- maximum axial loads have been adopted fo the Combination 1 cases - given that as an end span, the maximum moment from traffic load will closely correspond with maximum axial load from traffic load
- minimum axial load from earthquake have been adopted fo the Combination 5 cases - the level of axial load applied given the column section will make minimum axial load cases always critical.

Defining axial load cases as follows:

P1	=	Axial load Combination 1 - Full Live Load
P11	=	Axial load Combination 1 - 1/2 Live Load
P5X	=	Axial load Combination 5 - EQX
P5Y	=	Axial load Combination 5 - EQY

Factored axial resistance

$P_r := 0.1\phi_c f_c \cdot A_c \qquad P_r = 10347\,\text{kN}$

$$\frac{P1}{P_r} = \begin{pmatrix} 0.693 \\ 0.693 \\ 0.761 \\ 0.761 \end{pmatrix} \qquad \frac{P11}{P_r} = \begin{pmatrix} 0.489 \\ 0.489 \\ 0.556 \\ 0.556 \end{pmatrix} \qquad \frac{P5X}{P_r} = \begin{pmatrix} 0.147 \\ 0.147 \\ 0.199 \\ 0.199 \end{pmatrix} \qquad \frac{P5Y}{P_r} = \begin{pmatrix} 0.18 \\ 0.18 \\ 0.232 \\ 0.232 \end{pmatrix}$$

With reference to AASHTO LRFD Article 5.7.4.5, Biaxial Flexure, if the factored axial load is less than $0.10\phi f_c A_g$ then the section shall satisfy: **$M_{ux}/M_{rx} + M_{uy}/M_{ry} \leq 1.0$** (i.e. the section shall be treated as a flexural member - not a compression member)

As can be seen from the ratios calculated above, the ratio of applied ultimate axial load to the resistance is always less than $0.10\phi f_c A_g$. Therefore the design will check the section for biaxial flexure as defined above.

To account for additional demand from torsion effects - a number of bars were excluded from the above layouts in the section analysis at both the top section and base section to reserve capacity for applied torsion. (refer below under Torsion and Shear Design).

The biaxial checks for each case are presented below. Refer to attached sheets for PCACOL results. The reinforcement arangement is accepted in each case - the low biaxial bending ratios are indicative of the impact service load considerations have had on the design .

Case	Location	Comb 1 - Full Live		Comb 1 - 1/2 Live		Comb 5 EQX		Comb 5 EQY	
		Mu applied moment	Mr resist moment	Mu applied moment	Mr resist moment	Mu applied moment	Mr resist moment	Mu applied moment	Mr resist moment
		KN-m	KN-m	KN-m	KN-m	KN-m	KN-m	KN-m	KN-m
Case 1	TOP X	-12504.0	24084.1	-9019.6	23596.5	-8132.4	22683.3	-5445.2	22774.0
	TOP Y	-644.6	158796.9	-5773.7	153282.4	-1263.1	143984.0	-3447.9	144899.3
BIAXIAL CHECK		-0.52		-0.42		-0.37		-0.26	
Case 2	BASE X	-9084.5	17282.4	-7112.9	16746.7	-5192.7	15752.2	-3536.6	15850.6
	BASE Y	-1619.3	114674.8	-6651.1	109040.6	-7402.3	99091.3	-15694.3	100028.5
BIAXIAL CHECK		-0.54		-0.49		-0.40		-0.38	

Torsion Effects

Ultimate Factored Torsions

Case	Location	Comb 1 - Full Live	Comb 1 - 1/2 Live	Comb 5 - EQX	Comb 5 - EQY
		Torsion moment	Torsion moment	Torsion moment	Torsion moment
		KN-m	KN-m	KN-m	KN-m
Case 1	TOP X	16.6	40.3	810.3	1224.4
	TOP Y	16.6	40.3	810.3	1224.4
Case 2	BASE X	16.6	40.3	810.3	1224.4
	BASE Y	16.6	40.3	810.3	1224.4

$T1 := T1\cdot kN\cdot m \qquad T11 := T11\cdot kN\cdot m \qquad T5X := T5X\cdot kN\cdot m \qquad T5Y := T5Y\cdot kN\cdot m$

Note :

- Torsion moments have been modified using R=2 - adopting same appoach as for bending in walls
- X - Longitudinal Direction Y - Transverse Direction

Determine if torsional effects need to be investigated

Maximum ultimate torsion on abutment stem $T_u := \max(T1, T11, T5X, T5Y)$ $T_u = 1224\,kN\cdot m$

Torsional cracking moment $T_{cr} := 0.328\cdot\sqrt{\frac{f_c}{MPa}}\cdot\frac{A_{cp}^2}{p_c}\cdot MPa$ $T_{cr} = 2584\,kN\cdot m$

For normal density concrete, torsional effects shall be investigated where - for hoop reinforcement: $T_u > 0.25\cdot\phi_{sh}\cdot T_{cr}$

$Torsion_{Check} := \begin{cases} \text{"NECESSARY"} & \text{if } T_u > 0.25\cdot\phi_{sh}\cdot T_{cr} \\ \text{"NOT Required"} & \text{otherwise} \end{cases}$ $Torsion_{Check} = \text{"NECESSARY"}$

Determine longitudinal reinforcement required to resist torsion

Radius of shear link of column $R := \frac{d}{2} - cover - \frac{\phi_{link}}{2}$ $R = 651\,mm$

Area enclosed within centerline of transverse rebar $A_{oh} := \pi R^2$

Area enclosed by shear flow path $A_o := 0.85A_{oh}$

Perimeter of centerline of closed transverse torsion reinforcement $p_h := \pi\cdot R\cdot 2$ $p_h = 4087\,mm$

Additional area, A_{ts}, per column required to resist torsion, T/2 per abutment column section for hoops, given by: $A_{ts}(T) := \left(\frac{p_h\cdot\frac{T}{2}}{2\cdot A_o\cdot\phi_{ss}}\right)\frac{1}{f_y}$

Note that torsion reduced by 1/2 given that each column will resist half torsion effect

Summary of additional reinforcement area and number of additional ϕ32mm bars per column required for torsion:

Case	Location	Comb 1 - Full Live		Comb 1 - 1/2 Live		Comb 5 EQX		Comb 5 EQY	
		Area A_{ts}	No. of bars	Area A_{ts}	No. of bars	Area A_{ts}	No. of bars	Area A_{ts}	No. of bars
		mm2	n	mm2	n	mm2	n	mm2	n
Case 1	TOP X	54.9	0.1	133.3	0.2	2684.1	3.3	4055.6	5.0
	TOP Y	54.9	0.1	133.3	0.2	2684.1	3.3	4055.6	5.0
Case 2	BASE X	54.9	0.1	133.3	0.2	2684.1	3.3	4055.6	5.0
	BASE Y	54.9	0.1	133.3	0.2	2684.1	3.3	4055.6	5.0

Plastic Hinge Effects

Results from PCACOL for reinforcement arrangement adopted using axial load due to dead load and superimposed dead load, P_p , with load factor 1.0:

Top Section $P1_{up} := (1825.6 + 281.4)\cdot kN$ $P1_{up} = 2107\,kN$

Base Section $P2_{up} := (2364.9 + 281.4)\cdot kN$ $P2_{up} = 2646\,kN$

Reversible Plastic Hinge Moment - longitudinal case:

Top Section $M1_p := 22837\cdot 1.3\cdot kN\cdot m$

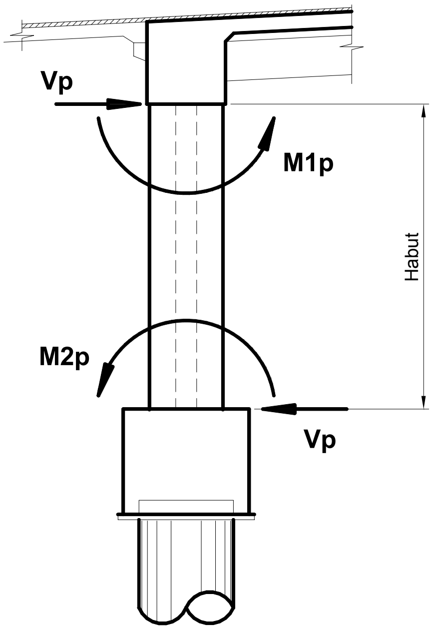
Base Section $M2_p := 15918.9\cdot 1.3\cdot kN\cdot m$

Height of abutment from base hinge to the deck diaphragm beam

$$H_{abut} = 4467\,mm$$

Reversible longitudinal shear force due to plastic hinging:

$$V_p := \frac{M1_p + M2_p}{H_{abut}}$$



$M1_p = 29688\,kN\cdot m$

$M2_p = 20695\,kN\cdot m$

$V_p = 11279\,kN$

Note that the transverse case has not been investigated. The wide section of the abutment in the transverse direction will generate plastic hinge effects that will be an order of magnitude greater than the elastic forces - therefore in all cases the elastic forces will be used in the design of transverse shear in the abument stem and for the design of foundations under transverse earthquake load, modified by Reponse Modification Factor.

Design Earthquake Effects for Shear, Deck Connections and Foundations

In the design of shear capacity, deck connections and in the design of the foundations, AASHTO LRFD allows the use either of the forces obtained from the elastic seismic analyis, modified by Response Modification Factor R, or the forces obtained from plastic hinging, whichever is the lowest. Response Modification Factor R=1.0 both for shear design, foundation design and deck connection design.

Given the amount of longitudinal reinforcement required in the abutment columns sections to control tensile stresses in service, the plastic hinge forces are very large. The forces obtained from the elastic seismic analysis with R=1.0 are presented below for the abutment for comparison:

$j := 1..4$

	COMBINATION 5 - 1.0 EQX + 0.3 EQY (R=1.0)						
Location	Case	P AXIAL	V2 SHEAR LONG	V3 SHEAR TRANS	T TORSION	M2 MOMENT TRANS	M3 MOMENT LONG
		KN	KN	KN	KN-m	KN-m	KN-m
Base	Max	-882.9	5393.3	2933.4	1580.1	16033.9	6750.5
Top	Max	-343.6	5393.3	2933.4	1580.1	3159.0	10570.3
Base	Min	-4409.7	-4833.7	-2844.4	-1498.9	-15214.3	-11164.3
Top	Min	-3870.4	-4833.7	-2844.4	-1498.9	-2737.2	-17483.7

$P_{EX} := -P_{EX} \cdot kN \qquad V2_{EX} := V2_{EX} \cdot kN \qquad V3_{EX} := V3_{EX} \cdot kN$

$T_{EX} := T_{EX} \cdot kN \cdot m \qquad M2_{EX} := M2_{EX} \cdot kN \cdot m \qquad M3_{EX} := M3_{EX} \cdot kN \cdot m$

	COMBINATION 5 - 0.3 EQX + 1.0 EQY (R=1.0)						
Location	Case	P AXIAL	V2 SHEAR LONG	V3 SHEAR TRANS	T TORSION	M2 MOMENT TRANS	M3 MOMENT LONG
		KN	KN	KN	KN-m	KN-m	KN-m
Base	Max	-1922.8	2482.7	5633.2	2408.2	32618.0	1782.2
Top	Max	-1383.5	2482.7	5633.2	2408.2	7528.6	2508.8
Base	Min	-3369.8	-1923.1	-5544.2	-2327.0	-31798.4	-6196.0
Top	Min	-2830.5	-1923.1	-5544.2	-2327.0	-7106.8	-9422.2

$P_{EY} := -P_{EY} \cdot kN \qquad V2_{EY} := V2_{EY} \cdot kN \qquad V3_{EY} := V3_{EY} \cdot kN$

$T_{EY} := T_{EY} \cdot kN \cdot m \qquad M2_{EY} := M2_{EY} \cdot kN \cdot m \qquad M3_{EY} := M3_{EY} \cdot kN \cdot m$

From inspection above it can be seen that the forces from longitudinal plastic hinging are substantially greater than the elastic forces from the seismic analysis with R=1.0.

Use the lowest forces for the shear and torsion design of the abutment stem, deck connection design and foundation design.

For the longitudinal case:

Design ultimate shear force $Vx_u := \min\left(V_p, \max\left(\sqrt{V2_{EX}^2}\right)\right)$ $\qquad Vx_u = 5393\,kN$

Design ultimate moment - deck $MD_u := \min\left[M1_p, \max\left[\sqrt{\left(M3_{EX_2}\right)^2}, \sqrt{\left(M3_{EX_4}\right)^2}\right]\right]$ $\qquad MD_u = 17484\,kN \cdot m$

Design ultimate moment - base $MB_u := \min\left[M2_p, \max\left[\sqrt{\left(M3_{EX_1}\right)^2}, \sqrt{\left(M3_{EX_3}\right)^2}\right]\right]$ $\qquad MB_u = 11164\,kN \cdot m$

Design for Shear and Torsion (AASHTO LRFD Section 5.8)

Longitudinal Shear - Plastic Hinge Zone

Within the plasic hinge zone, ignore strength of concrete in shear and carry entire shear by the reinforcement .

The required amount of tie steel is as follows:

Area of tie steel provided

$\phi_{link} = 19\,mm$ $\quad A_v := \pi \cdot \frac{\phi_{link}^2}{4} \cdot 2$ $\quad A_v = 567\,mm^2$

Effective depth of section $\quad d_e := \frac{d}{2} + \frac{d - 2(cover + \phi_{link}) - 16mm}{\pi}$ $\quad d_e = 1103\,mm$

Effective shear depth $\quad d_v := 0.9 \cdot d_e$ $\quad d_v = 993\,mm$

Spacing of tie steel required $\quad s_t := A_v \cdot \left(\frac{\frac{Vx_u}{2}}{\phi_{ss}} \cdot \frac{1}{f_y \cdot d_v} \right)^{-1}$ $\quad$ Note that plastic hinge shear reduced by 1/2 given that each column will resist half shear effect

$s_t = 57\,mm$

Provide 19mmϕ spirals with a spacing between ties of 50mm. Note that the maximum allowable spacing in a plastic hinge zone is 100mm.

$s_t := 50 \cdot mm$

Calculate the volumetric ratio of spiral reinforcement:

$$\rho := \frac{A_v}{2} \left(d - cover \cdot 2 - \phi_{link} \right) \frac{4}{s_t \cdot d^2} \qquad \rho = 0.015056$$

Check transverse reinforcement for confinement:

Area of column core $\quad A_c := \pi \frac{(d - 2 \cdot cover)^2}{4}$

Gross area of concrete $\quad A_g := \pi \frac{d^2}{4}$

For a circular column, the the volumetric ratio of spiral reinforcement shall be greater than either ρ_{s1} or ρ_{s2} as defined below:

$$\rho_{s1} := 0.45 \cdot \frac{f_c}{f_y} \cdot \left(\frac{A_g}{A_c} - 1 \right) \qquad \rho_{s1} = 0.0043$$

$$\rho_{s2} := 0.12 \cdot \frac{f_c}{f_y} \qquad \rho_{s2} = 0.0092$$

Given that ρ is greater than either ρ_{s1} and ρ_{s2} as defined above, accept design to satisfy confinement requirements

Longitudinal Shear - Abutment Stem

Effective shear depth:

$$d_v = 993\,mm$$

Calculate nominal shear resistance of concrete section - two column sections:

$$V_c := 0.166\cdot\sqrt{\frac{f_c}{MPa}}\cdot d\cdot d_v\cdot 2\cdot MPa \qquad V_c = 2527\,kN$$

Required nominal shear resistance of transverse reinforcement under plastic hinging:

$$V_s := \frac{Vx_u}{\phi_{ss}} - V_c \qquad V_s = 5178\,kN$$

Check that required total required nominal strength does not exceed limit, V_n:

$$V_n := 0.25\cdot f_c\cdot b\cdot d_v \qquad V_n = 52116\,kN$$

$$Shear_{Limit} := \begin{cases} \text{"OK"} & \text{if } V_s + V_c \le V_n \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases} \qquad Shear_{Limit} = \text{"OK"}$$

Area of tie steel provided

$$\phi_{link} = 19\,mm \qquad A_v := \pi\cdot\frac{\phi_{link}^{\,2}}{4}\cdot 2 \qquad A_v = 567\,mm^2$$

Determine required spacing of transverse reinforcement

$$s_t := A_v\cdot\left(\frac{\frac{V_s}{2}}{f_y\cdot d_v}\right)^{-1} \qquad s_t = 85\,mm$$

Note that required shear reduced by 1/2 given that each column will resist half shear effect

Provide 19mmϕ spirals with a spacing between ties of 75mm. Note that the maximum allowable spacing of spiral transverse reinforcement is 75mm.

Calculate the volumetric ratio of spiral reinforcement:

$$\rho := A_v\left(d - cover\cdot 2 - \phi_{link}\right)\frac{2}{s_t\cdot d^2} \qquad \rho = 0.0089$$

Check minimum transverse reinforcement:

The ratio of tie reinforcement to total volume of concrete core, ρ, shall be greater than that defined below:

$$\rho_{s1} := 0.45\cdot\frac{f_c}{f_y}\cdot\left(\frac{A_g}{A_c} - 1\right) \qquad \rho_{s1} = 0.00432$$

Given that ρ is greater than either ρ_{s1} as defined above, accept design to satisfy confinement requirements

Ties across Infill Wall

Maximum vertical spacing of ties shall be 300mm.

Maximum transverse spacing of ties shall be such that no bar is further than 610mm center-to-center on each side of a laterally supported bar.

Provide 13mm ties at 300mm c/c vertically and 1000mm c/c horizontally

Torsion Check

The area of longitudinal reinforcement to resist torsion in the stem has been taken account in the flexural design of the abutment stem. Transverse reinfocement will be included in the transverse shear and torsion check below.

Transverse Shear - Abutment Stem

Maximum transverse shear on abutment from elastic seismic analysis, R=1:

$$Vy_u := \max\left(\sqrt{V3_{EY}^2}\right) \qquad Vy_u = 5633\,kN$$

Effective shear depth:

$$d_v := 0.9\cdot\left(b - \frac{d}{2}\right)$$

$$d_v = 5670\,mm$$

Calculate nominal shear resistance of concrete section assuming beam section - taken through 400mm infill wall:

$$V_c := 0.166\cdot\sqrt{\frac{f_c}{MPa}}\cdot h\cdot d_v\cdot MPa$$

$$V_c = 2062\,kN$$

Required nominal shear resistance of transverse reinforcement - assuming hoop reinforcement:

$$V_s := \frac{Vy_u}{\phi_{sh}} - V_c \qquad V_s = 5985\,kN$$

Provide 19mmϕ shear links

$$A_v := \pi\frac{\phi_{link}^2}{4}\cdot 2 \qquad A_v = 567\,mm^2$$

Determine required spacing of transverse reinforcement

$$s_{t1} := \frac{A_v\cdot f_y\cdot d_v}{V_s} \qquad s_{t1} = 210\,mm$$

Maximum torsion moment on abutment - assuming same response modification factor for bending moment on walls, R=2:

$$T_u := \max\left(\sqrt{T_{EY}^2}\right)\cdot\frac{1}{2} \qquad T_u = 1204\,kN\cdot m$$

Area enclosed with centerline of transverse rebar

$$A_{oh} := \left(h - 2\cdot cover - \phi_{link}\right)\cdot\left(b - 2\cdot cover - \phi_{link}\right)$$

Area enclosed by shear flow path

$$A_o := 0.85A_{oh}$$

Determine required transverse reinforcement for torsion:

Area of one leg of transverse torsion reinforcement $A_t := \pi\frac{\phi_{link}^2}{4}$ $A_t = 284\,mm^2$

Required spacing of torsional reinforcement $s_{t2} := \frac{2\cdot A_o\cdot A_t\cdot f_y\cdot cot(\theta)}{T_u}\cdot\phi_{sh}$ $s_{t2} = 227\,mm$

Required combined spacing of transverse reinforcement for shear and torsion:

$$s_t := \left(\frac{1}{s_{t1}} + \frac{1}{s_{t2}}\right)^{-1} \qquad s_t = 109\,mm$$

Provide transverse reinforcement at 100mm c/c

$$s_t := 100mm$$

<u>Check requirements for wall type piers (AASHTO LRFD Article 5.10.11.4.2)</u>

The minimum reinforcement ratio, both horizontally and vertically shall not be less than 0.0025. Spacing shall not exceed 450mm.

Calculate reinforcement ratio horizontal: $\rho_h := \frac{A_v}{s_t\cdot h}$ $\rho_h = 0.01418$

Calculate reinforcement ratio vertical given ϕ19mm bars at 100mmc/c (see flexural design): $\rho_v := \frac{\pi\cdot(9.5mm)^2\cdot 2}{100mm\cdot h}$ $\rho_v = 0.01418$

Accept proposed reinforcement arrangement to satisfy design requirements

Check factored shear resistance for wall type pier, V_r:

$$V_n := \left(0.165\cdot\sqrt{\frac{f_c}{MPa}}\cdot MPa + \rho_h\cdot f_y\right)\cdot b\cdot h \qquad V_n = 52116\,kN$$

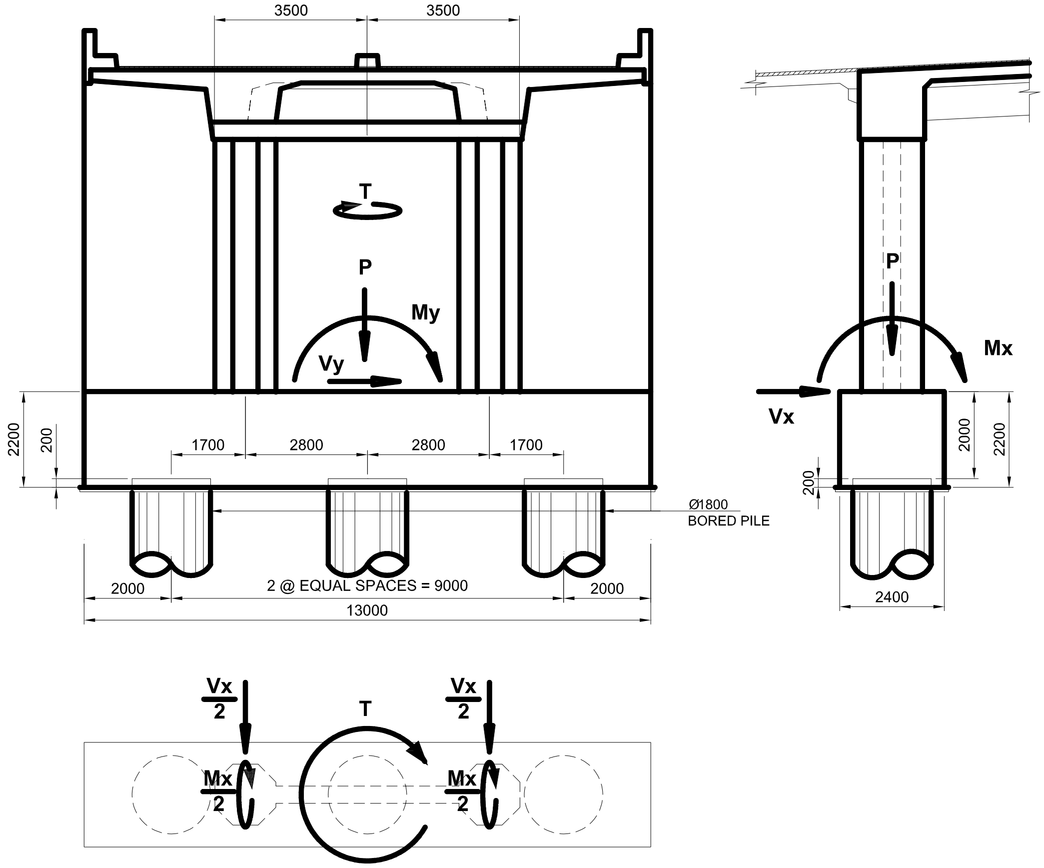
$$V_r := \begin{array}{|l} V_r \leftarrow 0.66\cdot\sqrt{\frac{f_c}{MPa}}\cdot MPa\cdot b\cdot h \\ V_r \ \text{ if } \ V_r \le \phi_{sh}\cdot V_n \\ \phi_{sh}\cdot V_n \ \text{ otherwise} \end{array} \qquad V_r = 10122\,kN$$

$$Shear_{Capacity} := \begin{array}{|l} \text{"OK"} \ \text{ if } \ V_r \ge Vy_u \\ \text{"INADEQUATE"} \ \text{ otherwise} \end{array} \qquad Shear_{Capacity} = \text{"OK"}$$

Accept proposed wall arrangement to satisfy design requirements

requirements

Pile Cap Design



The critical load cases for the design of the pile cap are:

1. Transverse elastic earthquake effects with R=1.0 - creating maximum demand Vy My and torsion Ty

2. The lowest of either longitudinal plastic hinging effects or longitudinal elastic earthquake effects with R=1.0 - creating maximum demand Vx Mx and torsion Tx

1. Determine Maximum Axial Load and Shear Force on Pile

Effective depth of pile cap $d_{cap} := 2000mm$

Width of pile cap $b_{cap} := 2400mm$

Length of pile cap $L_{cap} := 13000mm$

Pile spacing $s_p := 1.8m \cdot 2.5$ $\quad s_p = 4.5\ m$

Number of piles $n_{piles} := 3$

Weight of pile cap $W_{cap} := L_{cap} \cdot (d_{cap} + 200mm) \cdot b_{cap} \cdot 24.5 \frac{kN}{m^3}$ $\quad W_{cap} = 1682\ kN$

Applied ultimate loads - : **transverse elastic earthquake** effects with R=1.0 - Combinaton 5 (0.3EQX + 1.0EQY)

Design max axial force - base $PYmax_u := max(P_{EY_1}, P_{EY_3})$ $\quad PYmax_u = 3370\ kN$

Design min axial force - base $PYmin_u := min(P_{EY_1}, P_{EY_3})$ $\quad PYmin_u = 1923\ kN$

Design ultimate shear force - base (transverse) $Vy_u := max\left(\sqrt{V3_{EY}^2}\right)$ $\quad Vy_u = 5633\ kN$

Design ultimate shear force - base (longitudinal) $Vyx_u := max\left(\sqrt{V2_{EY}^2}\right)$ $\quad Vyx_u = 2483\ kN$

Design ultimate moment - base (transverse) $MBy_u := max\left[\sqrt{\left(M2_{EY_1}\right)^2}, \sqrt{\left(M2_{EY_3}\right)^2}\right]$ $\quad MBy_u = 32618\ kN \cdot m$

Design ultimate torsion - base $Ty_u := max\left[\sqrt{\left(T_{EY_1}\right)^2}, \sqrt{\left(T_{EY_3}\right)^2}\right]$ $\quad Ty_u = 2408\ kN \cdot m$

Applied ultimate loads - : **longitudinal elastic earthquake** effects with R=1.0 - Combinaton 5 (1.0EQX + 0.3EQY)

Design max axial force - base $PXmax_u := max(P_{EX_1}, P_{EX_3})$ $\quad PXmax_u = 4410\ kN$

Design min axial force - base $PXmin_u := min(P_{EX_1}, P_{EX_3})$ $\quad PXmin_u = 883\ kN$

Design ultimate shear force - base (longitudinal) $Vx_u := max\left(\sqrt{V2_{EX}^2}\right)$ $\quad Vx_u = 5393\ kN$

Design ultimate shear force - base (transverse) $Vxy_u := max\left(\sqrt{V3_{EX}^2}\right)$ $\quad Vxy_u = 2933\ kN$

Design ultimate moment - base (transverse) $MBx_u := max\left[\sqrt{\left(M2_{EX_1}\right)^2}, \sqrt{\left(M2_{EX_3}\right)^2}\right]$ $\quad MBx_u = 16034\ kN \cdot m$

Design ultimate torsion - base $Tx_u := max\left[\sqrt{\left(T_{EX_1}\right)^2}, \sqrt{\left(T_{EX_3}\right)^2}\right]$ $\quad Tx_u = 1580\ kN \cdot m$

Determine Maximum/Minimum axial load on piles - from transverse EQY case

Maximum pile load $PY_{max} := \frac{W_{cap}}{n_{piles}} + \frac{PYmax_u}{n_{piles}} + \frac{MBy_u}{s_p \cdot (n_{piles} - 1)} + \frac{Vy_u \cdot d_{cap}}{s_p \cdot (n_{piles} - 1)}$ $PY_{max} = 6560\,kN$

Minimum pile load $PY_{min} := \frac{W_{cap}}{n_{piles}} + \frac{PYmin_u}{n_{piles}} - \frac{MBy_u}{s_p \cdot (n_{piles} - 1)} - \frac{Vy_u \cdot d_{cap}}{s_p \cdot (n_{piles} - 1)}$ $PY_{min} = -3675\,kN$

Determine Maximum/Minimum axial load on piles - from longitudinal EQX case

Maximum pile load $PX_{max} := \frac{W_{cap}}{n_{piles}} + \frac{PXmax_u}{n_{piles}} + \frac{MBx_u}{s_p \cdot (n_{piles} - 1)} + \frac{Vxy_u \cdot d_{cap}}{s_p \cdot (n_{piles} - 1)}$ $PX_{max} = 4464\,kN$

Minimum pile load $PX_{min} := \frac{W_{cap}}{n_{piles}} + \frac{PXmin_u}{n_{piles}} - \frac{MBx_u}{s_p \cdot (n_{piles} - 1)} - \frac{Vxy_u \cdot d_{cap}}{s_p \cdot (n_{piles} - 1)}$ $PX_{min} = -1579\,kN$

Determine maximum shear force on pile including longitudinal and torsional effects - transverse EQY case

Longitudinal shear force per pile $Vyx_u := \frac{Ty_u}{s_p \cdot (n_{piles} - 1)} + \frac{Vyx_u}{n_{piles}}$ $Vyx_u = 1095\,kN$

Total shear force on pile - transverse earthquake case $Vyt_u := \sqrt{\left(\frac{Vy_u}{n_{piles}}\right)^2 + Vyx_u^2}$ $Vyt_u = 2174\,kN$

Determine maximum shear force on pile including transverse and torsional effects - longitudinal EQX case

Longitudinal shear force per pile $Vxx_u := \frac{Tx_u}{s_p \cdot (n_{piles} - 1)} + \frac{Vx_u}{n_{piles}}$ $Vxx_u = 1973\,kN$

Total shear force on pile - longitudinal earthquake case $Vxt_u := \sqrt{\left(\frac{Vxy_u}{n_{piles}}\right)^2 + Vxx_u^2}$ $Vxt_u = 2202\,kN$

SUMMARY - PILE AXIAL LOAD AND SHEAR FORCE

Maximum axial load on pile $P_{max} := \max(PX_{max}, PY_{max})$ $P_{max} = 6560\,kN$

Minimum axial load on pile $P_{min} := \min(PX_{min}, PY_{min})$ $P_{min} = -3675\,kN$

Maximum shear on pile $V_{max} := \max(Vxt_u, Vyt_u)$ $V_{max} = 2202\,kN$

2. Design for Longitudinal Case

Applied ultimate loads:

Moment	$Mx := MB_u$	$Mx = 11164\,kN \cdot m$
Shear	$Vx := Vx_u$	$Vx = 5393\,kN$
Torsion	$Tx := Tx_u$	$Tx = 1580\,kN \cdot m$

Cover to pile cap rebar

$$cover_{cap} := 75mm$$

Longitudinal effects will create pile cap torsions and shear forces in transferring loads to the piles as follows:

$$V_{cap} := \left[\frac{Vx}{n_{piles}} + \frac{Tx}{s_p \cdot (n_{piles} - 1)}\right] \qquad V_{cap} = 1973\,kN$$

$$T_{cap} := \left(\frac{Mx}{2}\right) \cdot \frac{2800}{2800 + 1700} + V_{cap} \cdot \frac{d_{cap}}{2} \qquad T_{cap} = 5447\,kN \cdot m$$

Determine longitudinal reinforcement required to resist torsion

Radius of shear link $\phi_{link} := 25mm$

Area enclosed within centerline of transverse rebar

$$A_{oh} := \left[d_{cap} - (cover_{cap} \cdot 2) - \phi_{link}\right] \cdot \left[b_{cap} - (cover_{cap} \cdot 2) - \phi_{link}\right] \qquad A_{oh} = 4.061\ m^2$$

Area enclosed by shear flow path $A_o := 0.85 A_{oh}$ $\qquad A_o = 3.452\ m^2$

Perimeter of centerline of closed transverse torsion reinforcement

$$p_h := \left[d_{cap} - (cover_{cap} \cdot 2) - \phi_{link}\right] \cdot 2 + \left[b_{cap} - (cover_{cap} \cdot 2) - \phi_{link}\right] \cdot 2$$

$$p_h = 8100\,mm$$

Area of longitudinal reinforcement, A_{ts}, required to resist torsion, T, given by:

$$A_{ts} := \left(\frac{p_h \cdot T_{cap}}{2 \cdot A_o \cdot \phi_{sh}}\right) \frac{1}{f_y} \qquad A_{ts} = 23411\,mm^2$$

Using 32mmϕ bars gives total number of bars to be distributed around section:

$$n_{bars} := \frac{A_{ts}}{802 \cdot mm^2} \qquad n_{bars} = 29.2$$

Provide 30 bars around perimeter of pile cap

$$n_{bars} := 30$$

Determine longitudinal reinforcement required to resist flexure in pile cap due to shear forces

Assuming simply supported beams spanning between piles, the maximum moment on the pile cap from the column shear forces is as follows:

$$M_{cap} := V_{cap} \cdot 1.7m \qquad M_{cap} = 3355\,kN \cdot m$$

Effective depth of section across the cap:

$$d_e := \left(b_{cap} - cover_{cap} - \phi_{link} - \frac{32mm}{2}\right) \qquad d_e = 2284\,mm$$

Determine area of additonal reinforcement, A_f, in the face of the cap to resist flexure:

$$R := \frac{M_{cap}}{\phi_b \cdot d_{cap} \cdot d_e^{\,2} \cdot f_y} \qquad R = 0.0010 \qquad M := \frac{0.85 \cdot f_c}{f_y} \qquad M = 0.0654$$

$$\rho := M \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R}{M}}\right) \qquad \rho = 0.001039$$

$$A_f := \rho \cdot d_e \cdot d_{cap}$$

$$A_f = 4745.3\,mm^2$$

Using 32mmϕ bars gives total number of additional bars to be distributed down the face of the pile cap:

$$n_{bf} := \frac{A_f}{802 \cdot mm^2} \qquad n_{bf} = 5.9$$

Total number of ϕ32mm bars on vertical face of pile cap:

$$n_v := n_{bars} \cdot \frac{2}{8.8} + n_{bf} \qquad n_v = 12.735$$

Provide 14 no. ϕ32mm bars @125mm c/c each vertical face

Total number of ϕ32mm bars on horizontal face of pile cap:

$$n_h := n_{bars} \cdot \frac{2.4}{8.8} \qquad n_h = 8.182$$

Provide 10 no. ϕ32mm bars each horizontal face

Transverse Reinforcement for Shear and Torsion

Shear on pile cap from longitudinal load effects:

$$V_{cap} = 1973\,kN$$

Effective shear depth:

$$d_v := 0.9 \cdot d_e \qquad d_e = 2284\,mm$$

$$d_v = 2056\,mm$$

Calculate nominal shear resistance of concrete section assuming beam section:

$$V_c := 0.166 \cdot \sqrt{\frac{f_c}{MPa}} \cdot d_{cap} \cdot d_v \cdot MPa$$

$$V_c = 3738\,kN$$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \begin{vmatrix} V_s \leftarrow \dfrac{V_{cap}}{\phi_{sh}} - V_c \\ V_s \;\text{ if }\; V_s > 0kN \\ 0kN \;\text{ otherwise} \end{vmatrix}$$

$$V_s = 0\,kN$$

Provide 25mmϕ shear links

$$\phi_{link} := 25mm$$

$$A_v := \pi \frac{\phi_{link}^{\,2}}{4} \cdot 2 \qquad A_v = 982\,mm^2$$

Determine required spacing of transverse reinforcement

$$s_{t1} := \begin{vmatrix} \dfrac{A_v \cdot f_y \cdot d_v}{V_s} \;\text{ if }\; V_s > 0kN \\ \dfrac{A_v \cdot f_y}{0.083 \cdot \sqrt{\dfrac{f_c}{MPa}} \cdot MPa \cdot d_{cap}} \;\text{ otherwise} \end{vmatrix}$$

$$s_{t1} = 421\,mm$$

Note that the maximum spacing of transverse reinforcement for shear is 600mm - however the calculated value is carried forward to determine the combined shear and torsion spacing

Maximum torsion moment on abutment pile cap:

$$T_{cap} = 5447\,kN \cdot m$$

Provide 19ϕ shear links

$\phi_{link} := 25mm$

Area enclosed with centerline of transverse rebar

$A_{oh} = 4.061\ m^2$

Area enclosed by shear flow path

$A_o := 0.85A_{oh}$

Determine required transverse reinforcement for torsion:

Area of one leg of transverse torsion reinforcement $A_t := \pi \frac{\phi_{link}^2}{4}$ $A_t = 491\ mm^2$ $\phi_{link} = 25\ mm$

Required spacing of torsional reinforcement $s_{t2} := \frac{2 \cdot A_o \cdot A_t \cdot f_y \cdot \cot(\theta)}{T_{cap}} \cdot \phi_{sh}$ $s_{t2} = 170\ mm$

Calculate combined spacing of shear and torsion transverse reinforcement:

$$s_t := \begin{vmatrix} s_{t1} \leftarrow \left(\frac{1}{s_{t1}} + \frac{1}{s_{t2}}\right)^{-1} \\ s_{t1} \text{ if } V_s > 0\ kN \\ s_{t2} \text{ if } V_s = 0\ kN \\ 600mm \text{ if } (s_{t2} > 600mm) \cdot (V_s = 0\ kN) \\ 600mm \text{ if } (s_{t1} > 600mm) \cdot (V_s > 0\ kN) \end{vmatrix}$$

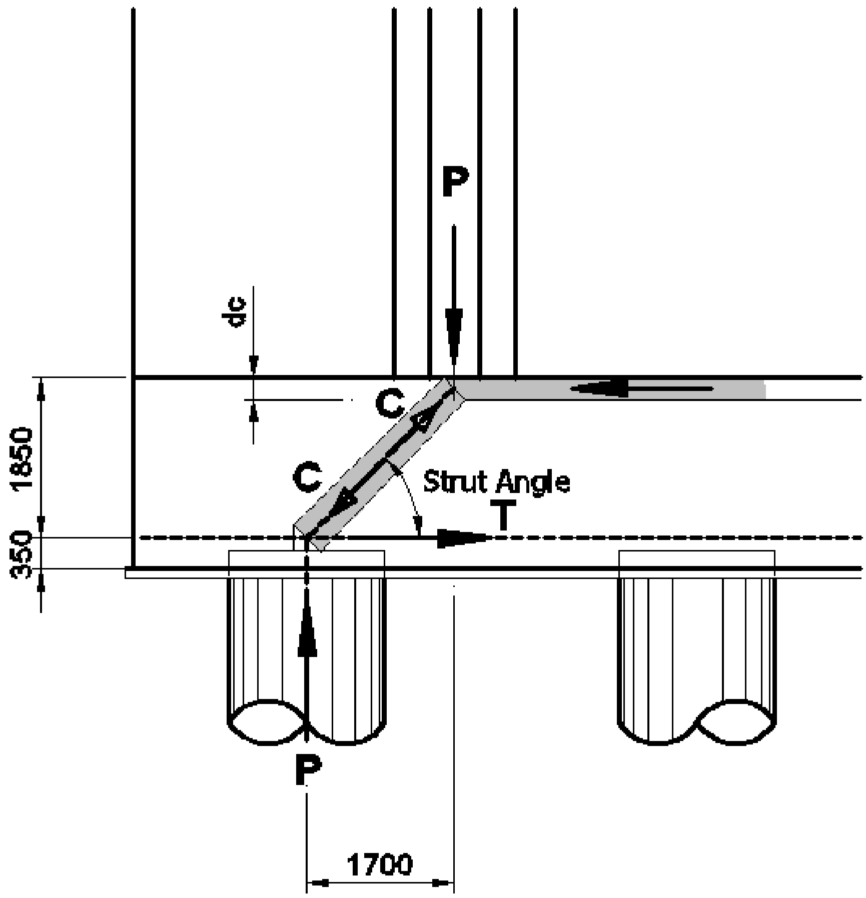
$s_t = 170\ mm$

<u>Provide 19mm dia transverse reinforcement at 150mm c/c or closer spacing to suit pile rebar layout</u>

3. Design Tie Steel in Pile Cap to Carry Maximum/Minimum Pile Loads

Determine required pile cap tension tie steel to support maximum/minimum reactions in pile

Using a strut and tie model in accordance with AASHTO LRFD Article 5.6.3:



Maximum pile load $P_{max} = 6560\,kN$

Minimum pile load $P_{min} = -3675\,kN$

Assume depth of horizonal concrete compression strut: $d_c := 200 \cdot mm$

Angle between compressive strut and tension tie: $\alpha_s := atan\left(\frac{1850mm - \frac{d_c}{2}}{1700mm}\right)$ $\alpha_s = 45.83\,deg$

Resistance factor for compression: $\phi_c := 0.70$
(in strut and tie models)

Resistance factor for tension: $\phi_t := 0.8$

Area of tension reinforcement required in tensile tie:

Tensile tie force required:

$$T := \frac{\left(P_{max} - \frac{W_{cap}}{3}\right)\cdot\frac{1}{\tan(\alpha_s)}}{\phi_t} \qquad T = 7285\,kN$$

$$A_{st} := \frac{T}{f_y} \qquad A_{st} = 18679\,mm^2$$

Number of ϕ32mm bars required in tie:

$$n_{bt} := \frac{A_{st}}{804\cdot mm^2} \qquad n_{bt} = 23.2$$

Provide 24 number ϕ32mm bars in tension tie in central section + 2 number ϕ32mm bars at each side

26 **number ϕ32mm bars total over width of 1800 pile**

Strength of compressive strut:

The tensile strain in the concrete in the direction of the tension tie:

$\varepsilon_s := 0.002$ Limiting tensile strain of reinforcement

Design tensile strain across concrete strut:

$$\varepsilon_1 := \varepsilon_s + (\varepsilon_s + .002)\cdot\frac{1}{\tan(\alpha_s)^2} \qquad \varepsilon_1 = 0.005775$$

The limiting compressive stress of the concrete strut is then given by:

$$f_{cu} := \begin{array}{|l} f_{cu} \leftarrow \frac{f_c}{0.8 + 170\cdot\varepsilon_1} \\ f_{cu} \ \text{if} \ f_{cu} \le 0.85\cdot f_c \\ 0.85\cdot f_c \ \text{otherwise} \end{array}$$

$f_{cu} = 16.8\,MPa$

Width of concrete strut at base limited to width of pile section at the base node:

$b_{cs} := 1800mm$

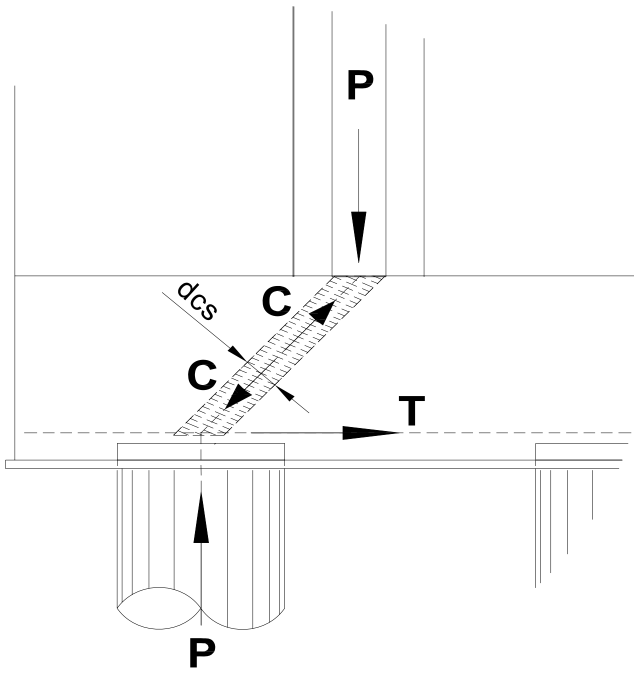
Compressive resistance force required in concrete strut:

$$C := \frac{P_{max}\cdot\frac{1}{\sin(\alpha_s)}}{\phi_c} \qquad C = 13065\,kN$$

Depth of concrete strut required:

$$d_{cs} := \frac{C}{f_{cu}} \frac{1}{b_{cs}}$$

$$d_{cs} = 431\,mm$$



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Conclude that the compressive strut can be comfortably accommodated within the available space provided by the pile cap.

Check depth of horizonal concrete compression strut:

Horizontal compressive force $C_H := C \cdot \cos(\alpha_s)$ $C_H = 9103\,kN$

Effective width of strut is the abutment column section width at the top node $b_{cs} := 1400mm$

Depth of concrete strut required:

$$d_{cs} := \frac{C_H}{0.85 \cdot f_c} \frac{1}{b_{cs}}$$ $d_{cs} = 255\,mm$ Accept assumed depth

<u>Area of tension reinforcement required in tensile tie for minimum tensile load - (reverse case)</u>:

Resistance tensile force required:

$$T := \frac{-\left(P_{min} \cdot \frac{1}{\tan(\alpha_s)} - \frac{W_{cap}}{3}\right)}{\phi_t} \qquad T = 5163\,kN$$

$$A_{st} := \frac{T}{f_y} \qquad A_{st} = 13238\,mm^2$$

Number of ϕ32mm bars required in tie:

$$n_{bt} := \frac{A_{st}}{802 \cdot mm^2} \qquad n_{bt} = 16.5$$

<u>Provide 18 number ϕ32mm bars in tension tie in top layer</u>

Serviceability Check

BASE

Serviceability Check

KATAHIRA & ENGINEERS INTERNATIONAL

Project: Detailed Design Study of North Java Corridor Flyover Project

Calculation: Balaraja Flyover
Serviceability Check - Full Live Load
1400mm Dia Circular RC Column - Base Section

Reference: Project Specific Design Criteria

Section Data

$MPa := 1000000 \cdot Pa$ $kN := 1000 \cdot N$

Input Item			
Concrete Compressive Strength	fc	30	MPa
Structural Steel Yield Strength	fys	250	MPa
Rebar Yield Strength	fy	390	MPa
Diameter of reinforced concrete section	D	1400	mm
Thickness of CHS section	t	0	mm
Diameter of rebar - layer 1	dia1	32	mm
Diameter of rebar - layer 2	dia2	32	mm
Number bars - layer 1 (max 100)	n1	36	
Number bars - layer 2 (max 100)	n2	10	
Cover from face of section - layer 1	cov1	60	mm
Cover from face of section - layer 2	cov2	115	mm

Load Data

Ref	Pier	Load Case	P	M	Stress
			kN	kNm	Allowance
1	A1	Combination 1 - P + Setlement + Traffic + Temp + Shcr	2732.0	3237.0	140%
2	A2	Combination 1 - P + Setlement + Traffic + Temp + Shcr	2681.0	3793.0	140%

$f_c := f_c \cdot MPa$ $f_{ys} := f_{ys} \cdot MPa$ $f_y := f_y \cdot MPa$ $D := D \cdot mm$ $ts := ts \cdot mm$

$dia1 := dia1 \cdot mm$ $dia2 := dia2 \cdot mm$ $cov1 := cov1 \cdot mm$ $cov2 := cov2 \cdot mm$

$P := P \cdot kN$ $M := M \cdot kN \cdot m$

$E_S := 200000 \cdot MPa$ $E_C := 4700\sqrt{\frac{f_c}{MPa}} \cdot MPa$ Modular ratio $\alpha := \begin{cases} \frac{E_S}{E_C} & \text{if } E_C > 0 \\ 1 & \text{otherwise} \end{cases}$ $\alpha = 7.77$

$E_C = 25743\,MPa$

Calculate Basic Allowable Stresses

Calculate rupture stress: $\sigma_{ct} := 0.5 \cdot \left(\frac{f_c}{MPa}\right)^{\frac{2}{3}} \cdot MPa$ $\sigma_{ct} = 4.8\,MPa$

Calculate basic allowable stress of concrete $\sigma_{cc} := 1.0 \cdot f_c$ $\sigma_{cc} = 30.0\,MPa$

Calculate basic allowable tensile stress of rebar $\sigma_{rs} := \begin{cases} 0.5 \cdot f_y & \text{if } 0.5 \cdot f_y \le 170MPa \\ 170MPa & \text{otherwise} \end{cases}$ $\sigma_{rs} = 170\,MPa$

Calculate basic allowable compressive stress of rebar $\sigma_{rc} := \begin{cases} 0.5 \cdot f_y & \text{if } 0.5 \cdot f_y \le 110MPa \\ f_y & \text{otherwise} \end{cases}$ $\sigma_{rc} = 390\,MPa$

Calculate basic allowable stress of structural steel $\sigma_{ts} := -0.6 f_{ys}$ $\sigma_{ts} = -150\,MPa$

$\sigma_{tc} := 1 f_{ys}$ $\sigma_{tc} = 250\,MPa$

Limiting strain of rebar $\varepsilon_{rs} := -\frac{\sigma_{rs}}{E_S}$ $\varepsilon_{rs} = -0.000850$

$\varepsilon_{rc} := \frac{\sigma_{rc}}{E_S}$ $\varepsilon_{rc} = 0.001950$

Limiting strain of structural steel $\varepsilon_{ts} := \frac{\sigma_{ts}}{E_S}$ $\varepsilon_{ts} = -0.000750$

$\varepsilon_{tc} := \frac{\sigma_{tc}}{E_S}$ $\varepsilon_{tc} = 0.001250$

Concrete Cross Section Data - generated

$n := 50$ Number of Points - 50 points maximum

$i := 1 .. n + 1$ Range from 1 to n+1

Ref.	X	Y	Ref.	X	Y
	mm	mm		mm	mm
1	0	-700	**26**	0	700
2	-88	-694	**27**	88	694
3	-174	-678	**28**	174	678
4	-258	-651	**29**	258	651
5	-337	-613	**30**	337	613
6	-411	-566	**31**	411	566
7	-479	-510	**32**	479	510
8	-539	-446	**33**	539	446
9	-591	-375	**34**	591	375
10	-633	-298	**35**	633	298
11	-666	-216	**36**	666	216
12	-688	-131	**37**	688	131
13	-699	-44	**38**	699	44
14	-699	44	**39**	699	-44
15	-688	131	**40**	688	-131
16	-666	216	**41**	666	-216
17	-633	298	**42**	633	-298
18	-591	375	**43**	591	-375
19	-539	446	**44**	539	-446
20	-479	510	**45**	479	-510
21	-411	566	**46**	411	-566
22	-337	613	**47**	337	-613
23	-258	651	**48**	258	-651
24	-174	678	**49**	174	-678
25	-88	694	**50**	88	-694

$k := 1 .. 25$ $XS1 := XS1 \cdot mm$ $XS2 := XS2 \cdot mm$ $YS1 := YS1 \cdot mm$ $YS2 := YS2 \cdot mm$

$x_k := XS1_k$ $y_k := YS1_k$ $x_{k+25} := XS2_k$ $y_{k+25} := YS2_k$ $x_{n+1} := XS1_1$ $y_{n+1} := YS1_1$

Calculate Section Properties of Concrete Section

$$A_C := -\sum_{i=1}^{n}\left[\left(y_{i+1} - y_i\right)\cdot\frac{x_{i+1} + x_i}{2}\right]$$ $A_C = 1.53533\ m^2$

$$x_C := -\frac{1}{A_C}\cdot\sum_{i=1}^{n}\left[\frac{y_{i+1} - y_i}{8}\cdot\left[\left(x_{i+1} + x_i\right)^2 + \frac{\left(x_{i+1} - x_i\right)^2}{3}\right]\right]$$ $x_C = 0\ m$

$$y_C := \frac{1}{A_C}\cdot\sum_{i=1}^{n}\left[\frac{x_{i+1} - x_i}{8}\cdot\left[\left(y_{i+1} + y_i\right)^2 + \frac{\left(y_{i+1} - y_i\right)^2}{3}\right]\right]$$ $y_C = 0\ m$

$$I_x := \sum_{i=1}^{n}\left[\left[\left(x_{i+1} - x_i\right)\cdot\frac{y_{i+1} + y_i}{24}\right]\cdot\left[\left(y_{i+1} + y_i\right)^2 + \left(y_{i+1} - y_i\right)^2\right]\right]$$ $I_x = 0.18758\ m^4$

$$I_y := -\sum_{i=1}^{n}\left[\left[\left(y_{i+1} - y_i\right)\cdot\frac{x_{i+1} + x_i}{24}\right]\cdot\left[\left(x_{i+1} + x_i\right)^2 + \left(x_{i+1} - x_i\right)^2\right]\right]$$ $I_y = 0.18758\ m^4$

$$I_{xC} := I_x - A_C\cdot x_C^{\,2}$$ $I_{xC} = 0.18758\ m^4$

$$I_{yC} := I_y - A_C\cdot y_C^{\,2}$$ $I_{yC} = 0.18758\ m^4$

Steel Tube Cross Section Data - generated from input

$ns := 50$ Number of Points - 50 points maximum

$ps := 1..ns+1$ Range from 1 to ns+1

Ref.	X	Y	Ref.	X	Y
	mm	mm		mm	mm
1	0	-700	**26**	0	-700
2	-181	-676	**27**	181	-676
3	-350	-606	**28**	350	-606
4	-495	-495	**29**	495	-495
5	-606	-350	**30**	606	-350
6	-676	-181	**31**	676	-181
7	-700	0	**32**	700	0
8	-676	181	**33**	676	181
9	-606	350	**34**	606	350
10	-495	495	**35**	495	495
11	-350	606	**36**	350	606
12	-181	676	**37**	181	676
13	0	700	**38**	0	700
14	181	676	**39**	-181	676
15	350	606	**40**	-350	606
16	495	495	**41**	-495	495
17	606	350	**42**	-606	350
18	676	181	**43**	-676	181
19	700	0	**44**	-700	0
20	676	-181	**45**	-676	-181
21	606	-350	**46**	-606	-350
22	495	-495	**47**	-495	-495
23	350	-606	**48**	-350	-606
24	181	-676	**49**	-181	-676
25	0	-700	**50**	0	-700

$XSS1 := XSS1 \cdot mm$ $XSS2 := XSS2 \cdot mm$ $YSS1 := YSS1 \cdot mm$ $YSS2 := YSS2 \cdot mm$

$z := 1..25$ $xs_z := XSS1_z$ $ys_z := YSS1_z$

$z := 26..50$ $xs_z := XSS2_{z-25}$ $ys_z := YSS2_{z-25}$

$xs_{ns+1} := XSS1_1$ $ys_{ns+1} := YSS1_1$

Calculate Section Properties of Steel Tube Section

$$A_{ST} := -\sum_{ps=1}^{ns}\left[\left(ys_{ps+1} - ys_{ps}\right)\cdot\frac{xs_{ps+1} + xs_{ps}}{2}\right]$$ $A_{ST} = 0\ m^2$

$$x_{ST} := -\frac{1}{A_{ST}}\cdot\sum_{ps=1}^{ns}\left[\frac{ys_{ps+1} - ys_{ps}}{8}\cdot\left[\left(xs_{ps+1} + xs_{ps}\right)^2 + \frac{\left(xs_{ps+1} - xs_{ps}\right)^2}{3}\right]\right]$$ $x_{ST} = -1.0\ m$

$$y_{ST} := \frac{1}{A_{ST}}\cdot\sum_{ps=1}^{ns}\left[\frac{xs_{ps+1} - xs_{ps}}{8}\cdot\left[\left(ys_{ps+1} + ys_{ps}\right)^2 + \frac{\left(ys_{ps+1} - ys_{ps}\right)^2}{3}\right]\right]$$ $y_{ST} = -0.499\ m$

$$I_{xS} := \sum_{ps=1}^{ns}\left[\left[\left(xs_{ps+1} - xs_{ps}\right)\cdot\frac{ys_{ps+1} + ys_{ps}}{24}\right]\cdot\left[\left(ys_{ps+1} + ys_{ps}\right)^2 + \left(ys_{ps+1} - ys_{ps}\right)^2\right]\right]$$

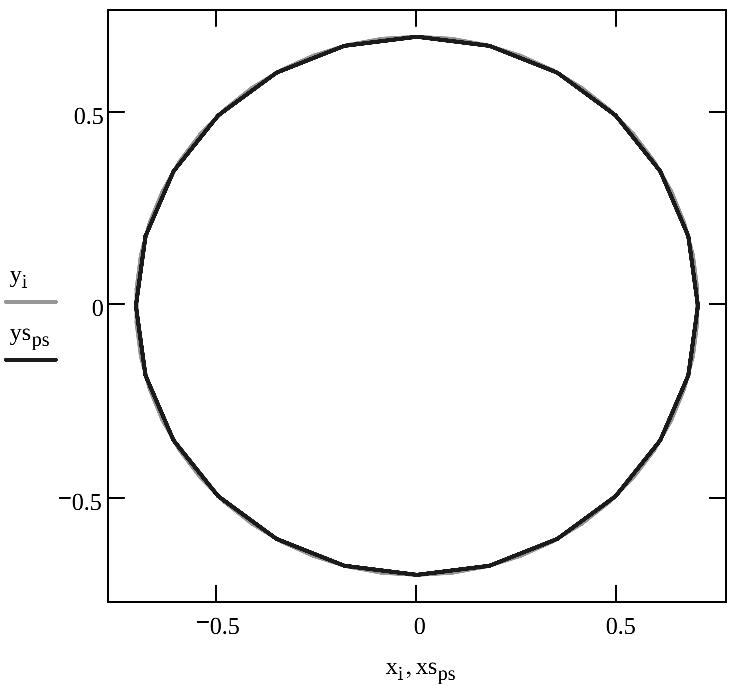
$I_{xS} = 0\ m^4$

$$I_{yS} := -\sum_{ps=1}^{ns}\left[\left[\left(ys_{ps+1} - ys_{ps}\right)\cdot\frac{xs_{ps+1} + xs_{ps}}{24}\right]\cdot\left[\left(xs_{ps+1} + xs_{ps}\right)^2 + \left(xs_{ps+1} - xs_{ps}\right)^2\right]\right]$$

$I_{yS} = 0\ m^4$

$I_{xS} := I_{xS} - A_{ST}\cdot x_{ST}^2$ $I_{xS} = 0\ m^4$

$I_{yS} := I_{yS} - A_{ST}\cdot y_{ST}^2$ $I_{yS} = 0.00000\ m^4$



Rebar Data Layer 1 - generated from input

Ref	Area mm2	X mm	Y mm	Ref	Area mm2	X mm	Y mm
1	804	0	625	51	0	0	0
2	804	0	-625	52	0	0	0
3	804	625	0	53	0	0	0
4	804	-625	0	54	0	0	0
5	804	48	623	55	0	0	0
6	804	48	-623	56	0	0	0
7	804	-48	623	57	0	0	0
8	804	-48	-623	58	0	0	0
9	804	232	580	59	0	0	0
10	804	232	-580	60	0	0	0
11	804	-232	580	61	0	0	0
12	804	-232	-580	62	0	0	0
13	804	282	558	63	0	0	0
14	804	282	-558	64	0	0	0
15	804	-282	558	65	0	0	0
16	804	-282	-558	66	0	0	0
17	804	330	530	67	0	0	0
18	804	330	-530	68	0	0	0
19	804	-330	530	69	0	0	0
20	804	-330	-530	70	0	0	0
21	804	447	403	71	0	0	0
22	804	447	-403	72	0	0	0
23	804	-447	403	73	0	0	0
24	804	-447	-403	74	0	0	0
25	804	540	313	75	0	0	0
26	804	540	-313	76	0	0	0
27	804	-540	313	77	0	0	0
28	804	-540	-313	78	0	0	0
29	804	587	215	79	0	0	0
30	804	587	-215	80	0	0	0
31	804	-587	215	81	0	0	0
32	804	-587	-215	82	0	0	0
33	804	615	110	83	0	0	0
34	804	615	-110	84	0	0	0
35	804	-615	110	85	0	0	0
36	804	-615	-110	86	0	0	0
37	0	0	0	87	0	0	0
38	0	0	0	88	0	0	0
39	0	0	0	89	0	0	0
40	0	0	0	90	0	0	0
41	0	0	0	91	0	0	0
42	0	0	0	92	0	0	0
43	0	0	0	93	0	0	0
44	0	0	0	94	0	0	0
45	0	0	0	95	0	0	0
46	0	0	0	96	0	0	0
47	0	0	0	97	0	0	0
48	0	0	0	98	0	0	0
49	0	0	0	99	0	0	0
50	0	0	0	100	0	0	0

Rebar Data Layer 2 - generated from input

Ref	Area mm2	X mm	Y mm	Ref	Area mm2	X mm	Y mm
1	804	0	574	51	0	0	0
2	804	0	-574	52	0	0	0
3				53	0	0	0
4				54	0	0	0
5				55	0	0	0
6				56	0	0	0
7				57	0	0	0
8				58	0	0	0
9				59	0	0	0
10				60	0	0	0
11	804	258	512	61	0	0	0
12	804	258	-512	62	0	0	0
13	804	-258	512	63	0	0	0
14	804	-258	-512	64	0	0	0
15				65	0	0	0
16				66	0	0	0
17				67	0	0	0
18				68	0	0	0
19	804	497	285	69	0	0	0
20	804	497	-285	70	0	0	0
21	804	-497	285	71	0	0	0
22	804	-497	-285	72	0	0	0
23				73	0	0	0
24				74	0	0	0
25				75	0	0	0
26				76	0	0	0
27	0	0	0	77	0	0	0
28	8040	0	88	78	8040	0	-88
29	8040	0	56	79	8040	0	-56
30	8040	0	24	80	8040	0	-24
31	0	0	0	81	0	0	0
32	0	0	0	82	0	0	0
33	283	0	120	83	283	0	-120
34	283	0	120	84	283	0	-120
35	283	0	120	85	283	0	-120
36	283	0	120	86	283	0	-120
37	283	0	120	87	283	0	-120
38	283	0	120	88	283	0	-120
39	283	0	120	89	283	0	-120
40	283	0	120	90	283	0	-120
41	283	0	120	91	283	0	-120
42	283	0	120	92	283	0	-120
43	283	0	120	93	283	0	-120
44	283	0	120	94	283	0	-120
45	283	0	120	95	283	0	-120
46	283	0	120	96	283	0	-120
47	283	0	120	97	283	0	-120
48	283	0	120	98	283	0	-120
49	283	0	120	99	283	0	-120
50	0	0	0	100	0	0	0

$A1 := A1 \cdot mm^2$ $A2 := A2 \cdot mm^2$ $A3 := A3 \cdot mm^2$ $A4 := A4 \cdot mm^2$

$X1 := X1 \cdot mm$ $X2 := X2 \cdot mm$ $X3 := X3 \cdot mm$ $X4 := X4 \cdot mm$

$Y1 := Y1 \cdot mm$ $Y2 := Y2 \cdot mm$ $Y3 := Y3 \cdot mm$ $Y4 := Y4 \cdot mm$

$k := 1..50$ $A_{bar_k} := A1_k$ $x_{bar_k} := X1_k$ $y_{bar_k} := Y1_k$

$A_{bar_{k+50}} := A2_k$ $x_{bar_{k+50}} := X2_k$ $y_{bar_{k+50}} := Y2_k$

$A_{bar_{k+100}} := A3_k$ $x_{bar_{k+100}} := X3_k$ $y_{bar_{k+100}} := Y3_k$

$A_{bar_{k+150}} := A4_k$ $x_{bar_{k+150}} := X4_k$ $y_{bar_{k+150}} := Y4_k$

Calculate Section Properties of Reinforcement

$$A_{BAR} := \sum_{j=1}^{200} A_{bar_j}$$ $A_{BAR} = 94855\ mm^2$

$$\rho := \frac{A_{BAR}}{A_C}$$ $\rho = 0.0618$

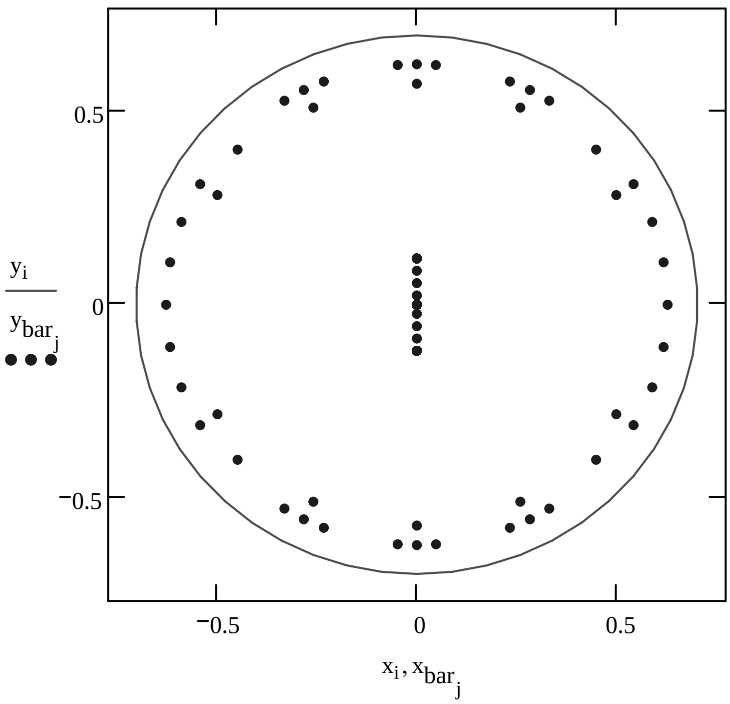
$$x_b := \begin{cases} \left[\sum_{j=1}^{200} \left(A_{bar_j} \cdot x_{bar_j}\right)\right] \cdot \frac{1}{A_{BAR}} & \text{if } A_{BAR} > 0 \\ 0m & \text{otherwise} \end{cases}$$ $x_b = -4 \times 10^{-6}\ m$

$$y_b := \begin{cases} \left[\sum_{j=1}^{200} \left(A_{bar_j} \cdot y_{bar_j}\right)\right] \cdot \frac{1}{A_{BAR}} & \text{if } A_{BAR} > 0 \\ 0m & \text{otherwise} \end{cases}$$ $y_b = 0\ m$

$$I_{xb} := \sum_{j=1}^{200} \left[A_{bar_j} \cdot \left(x_{bar_j}\right)^2\right] + A_{BAR} \cdot x_b^2$$ $I_{xb} = 0.00633\ m^4$

$$I_{yb} := \sum_{j=1}^{200} \left[A_{bar_j} \cdot \left(y_{bar_j}\right)^2\right] + A_{BAR} \cdot y_b^2$$ $I_{yb} = 0.00785\ m^4$

$j := 1 .. 200$



Calculate Composite Section Properties (before cracking)

Effective area $A_E := A_C \cdot [1 + \rho \cdot (\alpha - 1)] + A_{ST} \cdot \alpha$ $\qquad A_E = 2177416\ mm^2$

Effective centroid $x_E := \dfrac{A_C \cdot [(1-\rho) \cdot x_C + \rho \cdot x_b \cdot \alpha] + A_{ST} \cdot \alpha \cdot x_{ST}}{A_E}$ $\qquad x_E = -0.000\ m$

$y_E := \dfrac{A_C \cdot [(1-\rho) \cdot y_C + \rho \cdot y_b \cdot \alpha] + A_{ST} \cdot \alpha \cdot x_{ST}}{A_E}$ $\qquad y_E = 0.000\ m$

Effective stiffness $I_{EX} := I_{xC} + I_{xb} \cdot (\alpha - 1) + A_C \cdot \left[(1-\rho) \cdot x_C^2 + \rho \cdot x_b^2 \cdot \alpha\right] + \left(I_{xS} + A_{ST} \cdot x_{ST}^2\right) \cdot \alpha$

$I_{EX} = 0\ m^4$

$I_{EY} := I_{yC} + I_{yb} \cdot (\alpha - 1) + A_C \cdot \left[(1-\rho) \cdot y_C^2 + \rho \cdot y_b^2 \cdot \alpha\right] + \left(I_{yS} + A_{ST} \cdot y_{ST}^2\right) \cdot \alpha$

$I_{EY} = 0\ m^4$

Distance from extreme concrete fiber to centroid

$xF_{pos} := \max(x - x_E)$ $\qquad xF_{neg} := \min(x - x_E)$

$yF_{pos} := \max(y - y_E)$ $\qquad yF_{neg} := \min(y - y_E)$

Total depth of concrete section

$H_{CX} := xF_{pos} - xF_{neg}$ $\qquad H_{CX} = 1\ m$

$H_{CY} := yF_{pos} - yF_{neg}$ $\qquad H_{CY} = 1\ m$

Section modulus

$Z_{Xpos} := \dfrac{I_{EX}}{xF_{pos}}$ $\quad Z_{Xneg} := \dfrac{I_{EX}}{xF_{neg}}$

$Z_{Ypos} := \dfrac{I_{EY}}{yF_{pos}}$ $\quad Z_{Yneg} := \dfrac{I_{EY}}{yF_{neg}}$

Thickness of steel tube:

$ts := y_1 - ys_1$ $\quad ts = 0\,mm$

Establish Section Dimensions

Positive case - determine coord of extreme concrete fiber $\quad y_{Epos} := \max(y)$ $\quad y_{Epos} = 700\,mm$

Negative case - determine coord of extreme concrete fiber $\quad y_{Eneg} := \min(y)$ $\quad y_{Eneg} = -700\,mm$

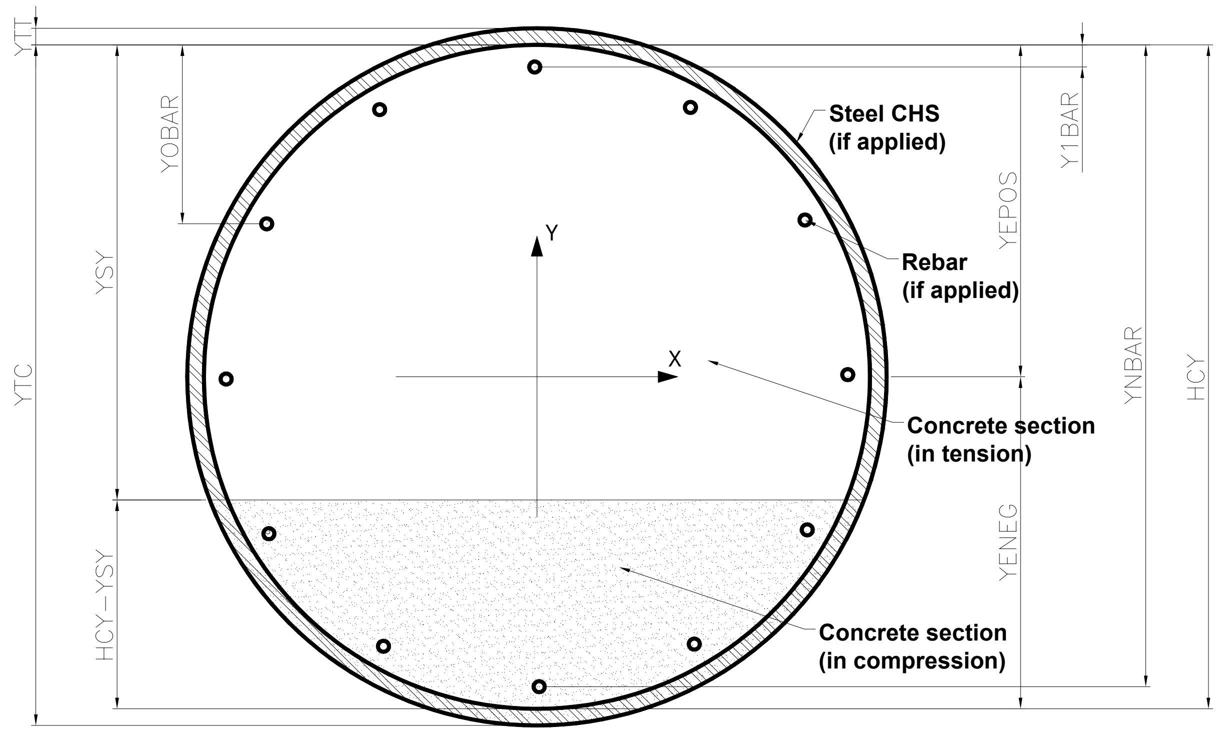
Offsets of rebar from extreme fiber $\quad y_{Obar} := y_{Epos} - y_{bar}$

Determine most extreme rebar (minimum offset) $\quad y_{1bar} := \min(y_{Epos} - y_{bar})$ $\quad y_{1bar} = 75\,mm$

Determine most extreme rebar (maximum offset) $\quad y_{nbar} := \max(y_{Epos} - y_{bar})$ $\quad y_{nbar} = 1325\,mm$

Offsets of extreme steel tube fiber from extreme concrete fiber $\quad y_{tt} := ts$ $\quad y_{tt} = 0\,mm$

$y_{tc} := H_{CY} + ts$ $\quad y_{tc} = 1400\,mm$



ASSIGN NEUTRAL AXIS VALUES

Number of sections to analysed $ns := 500$

$q := 2 .. ns$

Distance of neutral axis from extreme fiber in tension $y_{SY_q} := H_{CY} \cdot \frac{q}{ns + 1}$

Calculate stresses and strains in reinforcement and concrete at extreme fibers

Calculate strain at extreme compression fiber assuming max allowable stress in concrete:

Trial value of concrete strain

$\varepsilon cc := \frac{\sigma_{cc}}{E_C} \cdot 2$ $\frac{\sigma_{cc}}{E_C} = 0.001165$

Given

$$\sigma_{cc} = \varepsilon cc \cdot \left(4700 \sqrt{\frac{f_c \cdot 2}{MPa}} - \frac{4700^2}{2.68} \cdot \varepsilon cc\right) \cdot MPa$$

$\varepsilon_{cc} := Find(\varepsilon cc)$ $\varepsilon_{cc} := 0.002$

$$\varepsilon_{cc} := \begin{cases} \varepsilon_{tc} & \text{if } (f_c = 0) \cdot (A_{BAR} = 0) \\ \varepsilon_{rc} & \text{if } (f_c = 0) \cdot (ts = 0) \\ \varepsilon_{cc} & \text{otherwise} \end{cases}$$

$\varepsilon_{cc} = 0.002000$

Strain at other stresses taken to be linear:

$$\varepsilon cc(f_c, \sigma_{cd}) := \begin{cases} \frac{\sigma_{cd}}{\sigma_{tc}} \cdot \varepsilon_{tc} & \text{if } (f_c = 0) \cdot (A_{BAR} = 0) \\ \frac{\sigma_{cd}}{\sigma_{rc}} \cdot \varepsilon_{rc} & \text{if } (f_c = 0) \cdot (ts = 0) \\ \frac{\sigma_{cd}}{\sigma_{cc}} \cdot \varepsilon_{cc} & \text{otherwise} \end{cases}$$

Calculate strain in steel tube assuming max allowable stress in concrete:

In compression $\varepsilon_{tcc_q} := \varepsilon_{cc} \cdot \frac{y_{tc} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

In tension $\varepsilon_{tct_q} := \varepsilon_{cc} \cdot \frac{-\left(y_{SY_q} + y_{tt}\right)}{H_{CY} - y_{SY_q}}$

Calculate strain in rebar assuming max allowable stress in concrete:

In compression $\varepsilon_{rcc_q} := \varepsilon_{cc} \cdot \dfrac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

In tension $\varepsilon_{rct_q} := \varepsilon_{cc} \cdot \dfrac{y_{1bar} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

Calculate design max stress in compression taking account of other limits:

$$\sigma cd(\varepsilon_{tcc}, q) := \left| \begin{array}{l} \sigma_{cd} \leftarrow \sigma_{cc} \ \text{ if } f_c > 0 \\ \sigma_{cd} \leftarrow \sigma_{tc} \ \text{ if } (f_c = 0)\cdot(A_{BAR} = 0) \\ \sigma_{cd} \leftarrow \sigma_{rc} \ \text{ if } (f_c = 0)\cdot(ts = 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{tc}}{\varepsilon_{tcc}} \ \text{ if } (\varepsilon_{tcc} > \varepsilon_{tc})\cdot(ts > 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{rc}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}}} \ \text{ if } \left(\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}} > \varepsilon_{rc} \right)\cdot(A_{BAR} > 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{ts}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} + y_{tt})}{H_{CY} - y_{SY_q}}} \ \text{ if } \left[\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} + y_{tt})}{H_{CY} - y_{SY_q}} < \varepsilon_{ts} \right]\cdot(ts > 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{rs}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} - y_{1bar})}{H_{CY} - y_{SY_q}}} \ \text{ if } \left[\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} - y_{1bar})}{H_{CY} - y_{SY_q}} < \varepsilon_{rs} \right]\cdot(A_{BAR} > 0) \\ \sigma_{cc} \ \text{ otherwise} \end{array} \right.$$

$$\sigma_{cd_q} := \sigma cd(\varepsilon_{tcc_q}, q)$$

CALCULATE FORCES AND MOMENTS AT EACH NEUTRAL AXIS LOCATION

Calculate force in concrete:

$$F_{C_q} := \begin{cases} \displaystyle\int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot(1-\rho)\cdot\left[\frac{\sigma_{cd_q}\cdot\left[y+\left(\frac{H_{CY}}{2}-y_{SY_q}\right)\right]}{H_{CY}-y_{SY_q}}\right]dy & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from concrete about column centroid:

$$M_{C_q} := \begin{cases} \displaystyle\int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot(1-\rho)\cdot\left[\frac{\sigma_{cd_q}\cdot\left[y+\left(\frac{H_{CY}}{2}-y_{SY_q}\right)\right]}{H_{CY}-y_{SY_q}}\right]\cdot y\,dy & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate strain in rebar assuming design max stress in concrete:

$$y_{SY_q} := \begin{cases} y_{nbar}\cdot\frac{q}{ns+1} & \text{if } (f_c = 0)\cdot(A_{BAR} > 0) \\ y_{SY_q} & \text{otherwise} \end{cases}$$

$$\varepsilon_{S_{j,q}} := \begin{cases} -\frac{y_{SY_q} - y_{Obar_j}}{y_{nbar} - y_{SY_q}}\cdot\varepsilon cc\left(f_c,\sigma_{cd_q}\right) & \text{if } f_c = 0 \\ -\frac{y_{SY_q} - y_{Obar_j}}{H_{CY} - y_{SY_q}}\cdot\varepsilon cc\left(f_c,\sigma_{cd_q}\right) & \text{otherwise} \end{cases}$$

Calculate force in each rebar:

$$F_{S_{j,q}} := \begin{cases} \varepsilon_{S_{j,q}} E_S\cdot A_{bar_j} & \text{if } A_{BAR} > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate total force in reinforcement:

$$F_{R_q} := \sum_j F_{S_{j,q}}$$

Calculate moment from reinforcement about section centroid:

$$M_{R_q} := \begin{cases} \sum_j -\left(\varepsilon_{S_{j,q}} E_S \cdot A_{bar_j} \cdot y_{bar_j}\right) & \text{if } A_{BAR} > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate strain in steel tube at extreme tension fiber:

$$\varepsilon_{tds_q} := \frac{-\left(y_{SY_q} + y_{tt}\right)}{H_{CY} - y_{SY_q}} \varepsilon cc\left(f_c, \sigma_{cd_q}\right)$$

Calculate strain in steel tube at extreme compression fiber:

$$\varepsilon_{tdc_q} := \frac{y_{tc} - y_{SY_q}}{H_{CY} - y_{SY_q}} \varepsilon cc\left(f_c, \sigma_{cd_q}\right)$$

Calculate tensile force in steel tube:

$$F_{TS1_q} := \int_{\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2} + y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tds_q} \cdot E_S \cdot \left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TS2_q} := \int_{\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tds_q} \cdot E_S \cdot \left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TS} := \begin{cases} F_{TS1} - F_{TS2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate compressive force in steel tube:

$$F_{TC1_q} := \int_{-\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2} + y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tdc_q} \cdot E_S \cdot \left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TC2_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tdc_q}\cdot E_S\cdot\left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TC} := \begin{cases} F_{TC1} - F_{TC2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from tensile force in steel tube:

$$M_{TS1_q} := \int_{\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}+y_{tt}} -2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tds_q}\cdot E_S\cdot\left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right]\cdot y\, dy$$

$$M_{TS2_q} := \int_{\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} -2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tds_q}\cdot E_S\cdot\left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right]\cdot y\, dy$$

$$M_{TS} := \begin{cases} M_{TS1} - M_{TS2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from compressive force in steel tube:

$$M_{TC1_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}+y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tdc_q}\cdot E_S\cdot\left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right]\cdot y\, dy$$

$$M_{TC2_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tdc_q}\cdot E_S\cdot\left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right]\cdot y\, dy$$

$$M_{TC} := \begin{cases} M_{TC1} - M_{TC2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calate total axial response from section:

$$F_T := F_C + F_R + F_{TS} + F_{TC} \qquad F_{TC} := F_C$$

Calculate total moment response from section:

$$M_T := M_C + M_R + M_{TS} + M_{TC} \qquad M_{TC} := M_C$$

CALCULATE MAXIMUM ALLOWABLE AXIAL FORCE IN SECTION

Limiting strain in axial compression:

$$\varepsilon_{cL} := \begin{cases} \min(\varepsilon_{cc}, \varepsilon_{tc}) & \text{if } (A_{BAR} = 0)\cdot(ts \neq 0)\cdot(f_c \neq 0) \\ \min(\varepsilon_{cc}, \varepsilon_{rc}) & \text{if } (ts = 0)\cdot(A_{BAR} \neq 0)\cdot(f_c \neq 0) \\ \varepsilon_{tc} & \text{if } (A_{BAR} = 0)\cdot(f_c = 0) \\ \varepsilon_{rc} & \text{if } (ts = 0)\cdot(f_c = 0) \\ \min(\varepsilon_{cc}, \varepsilon_{rc}, \varepsilon_{tc}) & \text{otherwise} \end{cases}$$

$\varepsilon_{cL} = 0.001950$

Limiting concrete stress in axial compression

$$\sigma_{cL} := \begin{cases} \sigma_{cd_2} & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases}$$

$\sigma_{cL} = 30\,\text{MPa}$

$$P_{MAX} := \sigma_{cL}\cdot A_C(1-\rho) + \varepsilon_{cL}\cdot E_S(A_{BAR} + A_{ST})$$

$P_{MAX} = 80207.7\,\text{kN}$ $\quad F_{T_1} := P_{MAX}$ $\quad M_{T_1} := 0\cdot\text{kN}\cdot\text{m}$

$$P_{MAXC} := \sigma_{cL}\cdot A_C\cdot(1-\rho)$$

$P_{MAXC} = 43214.3\,\text{kN}$ $\quad F_{TC_1} := P_{MAXC}$ $\quad M_{TC_1} := 0\cdot\text{kN}\cdot\text{m}$

CALCULATE MINIMUM ALLOWABLE AXIAL FORCE IN SECTION

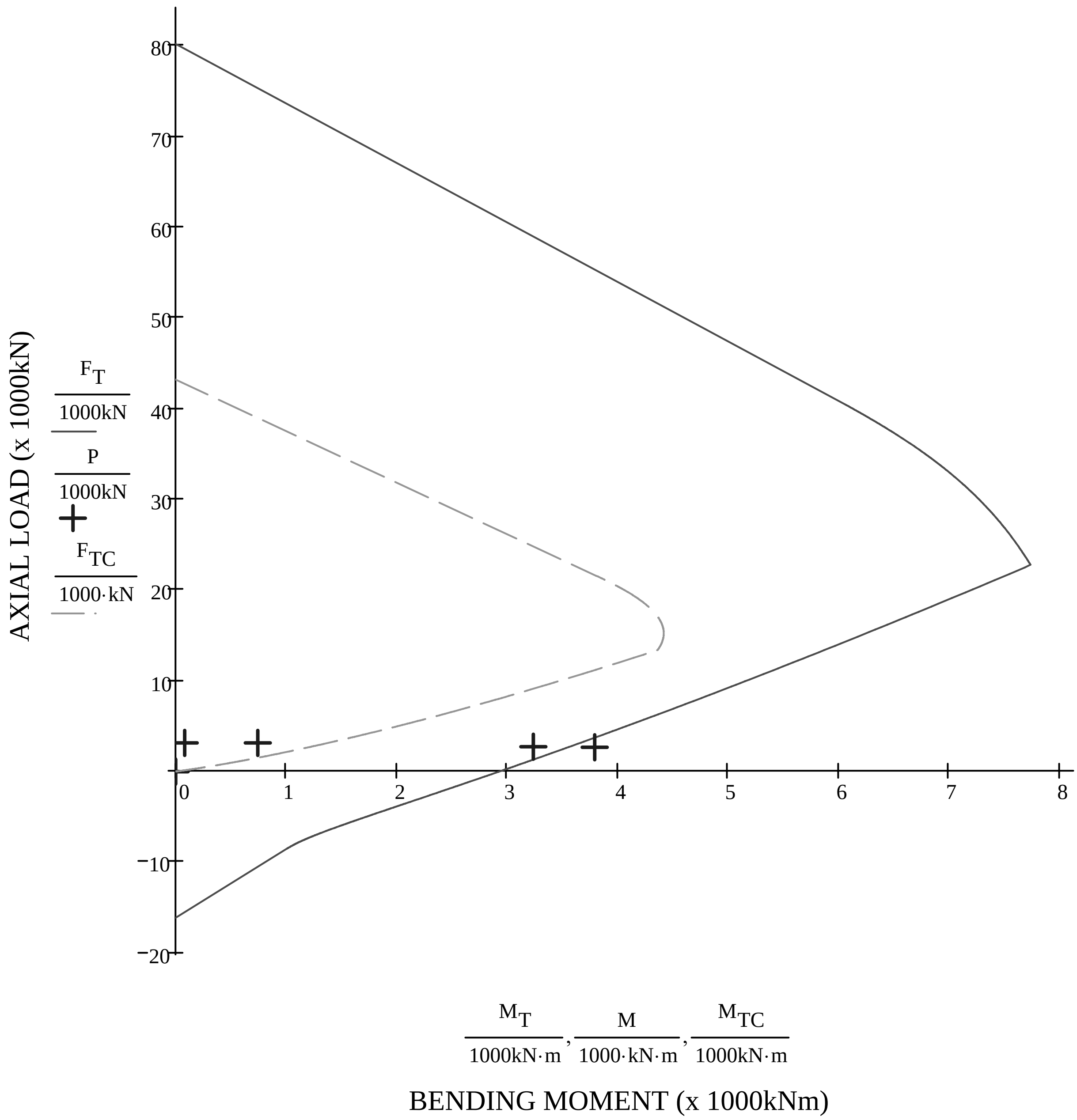
$$P_{MIN} := \begin{cases} \varepsilon_{rs}\cdot E_S(A_{BAR}) & \text{if } ts = 0 \\ \varepsilon_{ts}\cdot E_S(A_{ST}) & \text{if } A_{BAR} = 0 \\ \max(\varepsilon_{ts}, \varepsilon_{rs})\cdot E_S(A_{BAR} + A_{ST}) & \text{otherwise} \end{cases}$$

$P_{MIN} = -16125.3\,\text{kN}$ $\quad F_{T_{ns+1}} := P_{MIN}$ $\quad M_{T_{ns+1}} := 0\cdot\text{kN}\cdot\text{m}$

$$\text{Limit} := \begin{cases} \min(P, F_T)\cdot 0.75 & \text{if } \min(P) > 0 \\ \min(P, F_T)\cdot 1.25 & \text{otherwise} \end{cases}$$

$P_{MINC} := 0\text{kN}$ $\quad M_{TC_{ns+1}} := 0\cdot\text{kN}\cdot\text{m}$

Diameter of Column	$D = 1400\,mm$	Characteristic strength of concrete	$f_c = 30\,MPa$
Percentage reinforcement	$\rho = 6.18\,\%$	Yield Strength of Rebar	$f_y = 390\,MPa$
Thickness of CHS	$ts = 0\,mm$	Yield Strength of CHS	$f_{ys} = 250\,MPa$



INTERACTION CURVE AT SERVICEABILITY LIMIT STATE

Equation of interaction line - upper region (between 1 and 2 calculation points)

$$m1 := \frac{M_{T_2} - M_{T_1}}{F_{T_2} - F_{T_1}} \qquad c1 := F_{T_1}$$

Equation of interaction line - lower region (between ns and ns+1 calculation points)

$$m2 := \frac{M_{T_{ns}} - M_{T_{ns+1}}}{F_{T_{ns}} - F_{T_{ns+1}}} \qquad c2 := F_{T_{ns+1}}$$

$r := 1..8$

$$M_{SLS_r} := \begin{cases} 0.000000000000000001 \cdot kN \cdot m & \text{if } P_r > F_{T_1} \\ 0.000000000000000001 \cdot kN \cdot m & \text{if } P_r < F_{T_{ns+1}} \\ (P_r - c1) \cdot m1 & \text{if } (P_r > F_{T_2}) \cdot (P_r \le F_{T_1}) \\ (P_r - c2) \cdot m2 & \text{if } (P_r \ge F_{T_{ns+1}}) \cdot (P_r < F_{T_{ns}}) \\ \text{otherwise} & \left| \begin{array}{l} j \leftarrow 1 \\ \text{while } F_{T_j} > P_r \\ \quad j \leftarrow j + 1 \\ M_{T_j} \end{array} \right. \end{cases}$$

$$StressFactor_r := \begin{cases} \text{"No Result"} & \text{if } M_{SLS_r} < 0.00000000000000001 \cdot kN \cdot m \\ \dfrac{M_r}{M_{SLS_r}} & \text{otherwise} \end{cases}$$

$$P = \begin{pmatrix} 2732 \\ 2681 \\ 3159 \\ 3159 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kN \qquad M = \begin{pmatrix} 3237 \\ 3793 \\ 84 \\ 745 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kN \cdot m \qquad M_{SLS} = \begin{pmatrix} 3559.2 \\ 3532.8 \\ 3639.7 \\ 3639.7 \\ 2921.1 \\ 2921.1 \\ 2921.1 \\ 2921.1 \end{pmatrix} kN \cdot m \qquad StressFactor = \begin{pmatrix} 0.909 \\ 1.074 \\ 0.023 \\ 0.205 \\ 0.000 \\ 0.000 \\ 0.000 \\ 0.000 \end{pmatrix}$$

RESULTS SUMMARY
SERVICEABILITY LIMIT STATE ANALYSIS OF CIRCULAR BEAM COLUMN

Diameter of Column			1400	mm			
Percentage of rebar			6.18	%			
Load Case Ref	**Pier**	**Applied Service Axial Load kN**	**Applied Service Bending Moment kNm**	**Service Limit State Bending Moment kNm**	**Allowable Stress Factor**	**Applied Stress Factor**	**Serviceability Limit State Design Result**
1	A1	2732	3237	3559.2	140%	91%	OK
2	A2	2681	3793	3532.8	140%	107%	OK

Serviceability Check - Traffic Load Only

KATAHIRA & ENGINEERS INTERNATIONAL

Project: **Detailed Design Study of North Java Corridor Flyover Project**

Calculation: **Balaraja Flyover**
Serviceability Check - Traffic Load Only
1400mm Dia Circular RC Column - Base Section

Reference: **Project Specific Design Criteria**

Section Data

$MPa := 1000000 \cdot Pa$ $kN := 1000 \cdot N$

Input Item			
Concrete Compressive Strength	fc	30	MPa
Structural Steel Yield Strength	fys	250	MPa
Rebar Yield Strength	fy	390	MPa
Diameter of reinforced concrete section	D	1400	mm
Thickness of CHS section	t	0	mm
Diameter of rebar - layer 1	dia1	32	mm
Diameter of rebar - layer 2	dia2	32	mm
Number bars - layer 1 (max 100)	n1	36	
Number bars - layer 2 (max 100)	n2	10	
Cover from face of section - layer 1	cov1	60	mm
Cover from face of section - layer 2	cov2	115	mm

Load Data

Ref	Pier	Load Case	P	M	Stress
			kN	kNm	Allowance
1	A1	Combination 1 - P + Traffic Load Only	2684.0	2969.0	100%
2	A2	Combination 1 - P + Traffic Load Only	2619.0	3499.0	100%

$f_c := f_c \cdot MPa$ $\quad f_{ys} := f_{ys} \cdot MPa$ $\quad f_y := f_y \cdot MPa$ $\quad D := D \cdot mm$ $\quad ts := ts \cdot mm$

$dia1 := dia1 \cdot mm$ $\quad dia2 := dia2 \cdot mm$ $\quad cov1 := cov1 \cdot mm$ $\quad cov2 := cov2 \cdot mm$

$P := P \cdot kN$ $\quad M := M \cdot kN \cdot m$

$E_S := 200000 \cdot MPa$ $\quad E_C := 4700 \sqrt{\frac{f_c}{MPa}} \cdot MPa$ Modular ratio $\quad \alpha := \begin{cases} \frac{E_S}{E_C} & \text{if } E_C > 0 \\ 1 & \text{otherwise} \end{cases}$ $\quad \alpha = 7.77$

$E_C = 25743\,MPa$

Calculate Basic Allowable Stresses

Calculate rupture stress: $\quad \sigma_{ct} := 0.5 \cdot \left(\frac{f_c}{MPa}\right)^{\frac{2}{3}} \cdot MPa$ $\quad \sigma_{ct} = 4.8\,MPa$

Calculate basic allowable stress of concrete $\quad \sigma_{cc} := 1.0 \cdot f_c$ $\quad \sigma_{cc} = 30.0\,MPa$

Calculate basic allowable tensile stress of rebar $\quad \sigma_{rs} := \begin{cases} 0.5 \cdot f_y & \text{if } 0.5 \cdot f_y \le 170MPa \\ 170MPa & \text{otherwise} \end{cases}$ $\quad \sigma_{rs} = 170\,MPa$

Calculate basic allowable compressive stress of rebar $\quad \sigma_{rc} := \begin{cases} 0.5 \cdot f_y & \text{if } 0.5 \cdot f_y \le 110MPa \\ f_y & \text{otherwise} \end{cases}$ $\quad \sigma_{rc} = 390\,MPa$

Calculate basic allowable stress of structural steel $\quad \sigma_{ts} := -0.6 f_{ys}$ $\quad \sigma_{ts} = -150\,MPa$

$\sigma_{tc} := 1 f_{ys}$ $\quad \sigma_{tc} = 250\,MPa$

Limiting strain of rebar $\quad \varepsilon_{rs} := -\frac{\sigma_{rs}}{E_S}$ $\quad \varepsilon_{rs} = -0.000850$

$\varepsilon_{rc} := \frac{\sigma_{rc}}{E_S}$ $\quad \varepsilon_{rc} = 0.001950$

Limiting strain of structural steel $\quad \varepsilon_{ts} := \frac{\sigma_{ts}}{E_S}$ $\quad \varepsilon_{ts} = -0.000750$

$\varepsilon_{tc} := \frac{\sigma_{tc}}{E_S}$ $\quad \varepsilon_{tc} = 0.001250$

Concrete Cross Section Data - generated

$n := 50$ Number of Points - 50 points maximum

$i := 1 .. n + 1$ Range from 1 to n+1

Ref.	X	Y	Ref.	X	Y
	mm	mm		mm	mm
1	0	-700	**26**	0	700
2	-88	-694	**27**	88	694
3	-174	-678	**28**	174	678
4	-258	-651	**29**	258	651
5	-337	-613	**30**	337	613
6	-411	-566	**31**	411	566
7	-479	-510	**32**	479	510
8	-539	-446	**33**	539	446
9	-591	-375	**34**	591	375
10	-633	-298	**35**	633	298
11	-666	-216	**36**	666	216
12	-688	-131	**37**	688	131
13	-699	-44	**38**	699	44
14	-699	44	**39**	699	-44
15	-688	131	**40**	688	-131
16	-666	216	**41**	666	-216
17	-633	298	**42**	633	-298
18	-591	375	**43**	591	-375
19	-539	446	**44**	539	-446
20	-479	510	**45**	479	-510
21	-411	566	**46**	411	-566
22	-337	613	**47**	337	-613
23	-258	651	**48**	258	-651
24	-174	678	**49**	174	-678
25	-88	694	**50**	88	-694

$k := 1 .. 25$ $XS1 := XS1 \cdot mm$ $XS2 := XS2 \cdot mm$ $YS1 := YS1 \cdot mm$ $YS2 := YS2 \cdot mm$

$x_k := XS1_k$ $y_k := YS1_k$ $x_{k+25} := XS2_k$ $y_{k+25} := YS2_k$ $x_{n+1} := XS1_1$ $y_{n+1} := YS1_1$

Calculate Section Properties of Concrete Section

$$A_C := -\sum_{i=1}^{n}\left[\left(y_{i+1}-y_i\right)\cdot\frac{x_{i+1}+x_i}{2}\right]$$

$A_C = 1.53533\ m^2$

$$x_C := -\frac{1}{A_C}\cdot\sum_{i=1}^{n}\left[\frac{y_{i+1}-y_i}{8}\cdot\left[\left(x_{i+1}+x_i\right)^2+\frac{\left(x_{i+1}-x_i\right)^2}{3}\right]\right]$$

$x_C = 0\ m$

$$y_C := \frac{1}{A_C}\cdot\sum_{i=1}^{n}\left[\frac{x_{i+1}-x_i}{8}\cdot\left[\left(y_{i+1}+y_i\right)^2+\frac{\left(y_{i+1}-y_i\right)^2}{3}\right]\right]$$

$y_C = 0\ m$

$$I_x := \sum_{i=1}^{n}\left[\left[\left(x_{i+1}-x_i\right)\cdot\frac{y_{i+1}+y_i}{24}\right]\cdot\left[\left(y_{i+1}+y_i\right)^2+\left(y_{i+1}-y_i\right)^2\right]\right]$$

$I_x = 0.18758\ m^4$

$$I_y := -\sum_{i=1}^{n}\left[\left[\left(y_{i+1}-y_i\right)\cdot\frac{x_{i+1}+x_i}{24}\right]\cdot\left[\left(x_{i+1}+x_i\right)^2+\left(x_{i+1}-x_i\right)^2\right]\right]$$

$I_y = 0.18758\ m^4$

$$I_{xC} := I_x - A_C\cdot x_C^{\,2}$$

$I_{xC} = 0.18758\ m^4$

$$I_{yC} := I_y - A_C\cdot y_C^{\,2}$$

$I_{yC} = 0.18758\ m^4$

Steel Tube Cross Section Data - generated from input

$ns := 50$ Number of Points - 50 points maximum

$ps := 1 .. ns + 1$ Range from 1 to ns+1

Ref.	X	Y	Ref.	X	Y
	mm	mm		mm	mm
1	0	-700	**26**	0	-700
2	-181	-676	**27**	181	-676
3	-350	-606	**28**	350	-606
4	-495	-495	**29**	495	-495
5	-606	-350	**30**	606	-350
6	-676	-181	**31**	676	-181
7	-700	0	**32**	700	0
8	-676	181	**33**	676	181
9	-606	350	**34**	606	350
10	-495	495	**35**	495	495
11	-350	606	**36**	350	606
12	-181	676	**37**	181	676
13	0	700	**38**	0	700
14	181	676	**39**	-181	676
15	350	606	**40**	-350	606
16	495	495	**41**	-495	495
17	606	350	**42**	-606	350
18	676	181	**43**	-676	181
19	700	0	**44**	-700	0
20	676	-181	**45**	-676	-181
21	606	-350	**46**	-606	-350
22	495	-495	**47**	-495	-495
23	350	-606	**48**	-350	-606
24	181	-676	**49**	-181	-676
25	0	-700	**50**	0	-700

$XSS1 := XSS1\cdot mm$ $XSS2 := XSS2\cdot mm$ $YSS1 := YSS1\cdot mm$ $YSS2 := YSS2\cdot mm$

$z := 1 .. 25$ $xs_z := XSS1_z$ $ys_z := YSS1_z$

$z := 26 .. 50$ $xs_z := XSS2_{z-25}$ $ys_z := YSS2_{z-25}$

$xs_{ns+1} := XSS1_1$ $ys_{ns+1} := YSS1_1$

Calculate Section Properties of Steel Tube Section

$$A_{ST} := -\sum_{ps=1}^{ns}\left[\left(ys_{ps+1} - ys_{ps}\right)\cdot\frac{xs_{ps+1} + xs_{ps}}{2}\right]$$ $A_{ST} = 0\ m^2$

$$x_{ST} := -\frac{1}{A_{ST}}\cdot\sum_{ps=1}^{ns}\left[\frac{ys_{ps+1} - ys_{ps}}{8}\cdot\left[\left(xs_{ps+1} + xs_{ps}\right)^2 + \frac{\left(xs_{ps+1} - xs_{ps}\right)^2}{3}\right]\right]$$ $x_{ST} = -1.0\ m$

$$y_{ST} := \frac{1}{A_{ST}}\cdot\sum_{ps=1}^{ns}\left[\frac{xs_{ps+1} - xs_{ps}}{8}\cdot\left[\left(ys_{ps+1} + ys_{ps}\right)^2 + \frac{\left(ys_{ps+1} - ys_{ps}\right)^2}{3}\right]\right]$$ $y_{ST} = -0.499\ m$

$$I_{xS} := \sum_{ps=1}^{ns}\left[\left[\left(xs_{ps+1} - xs_{ps}\right)\cdot\frac{ys_{ps+1} + ys_{ps}}{24}\right]\cdot\left[\left(ys_{ps+1} + ys_{ps}\right)^2 + \left(ys_{ps+1} - ys_{ps}\right)^2\right]\right]$$

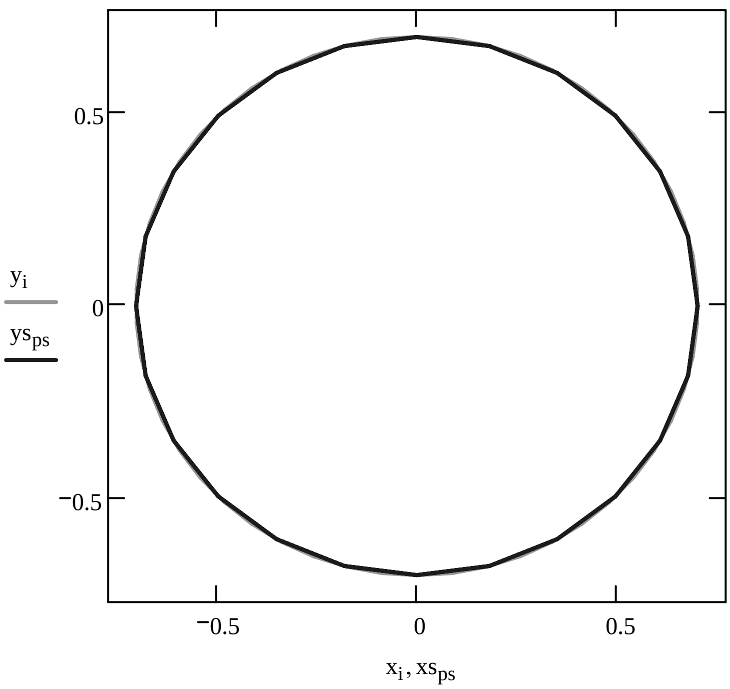
$I_{xS} = 0\ m^4$

$$I_{yS} := -\sum_{ps=1}^{ns}\left[\left[\left(ys_{ps+1} - ys_{ps}\right)\cdot\frac{xs_{ps+1} + xs_{ps}}{24}\right]\cdot\left[\left(xs_{ps+1} + xs_{ps}\right)^2 + \left(xs_{ps+1} - xs_{ps}\right)^2\right]\right]$$

$I_{yS} = 0\ m^4$

$I_{xS} := I_{xS} - A_{ST}\cdot x_{ST}^2$ $I_{xS} = 0\ m^4$

$I_{yS} := I_{yS} - A_{ST}\cdot y_{ST}^2$ $I_{yS} = 0.00000\ m^4$



Rebar Data Layer 1 - generated from input

Ref	Area mm2	X mm	Y mm	Ref	Area mm2	X mm	Y mm
1	804	0	625	51	0	0	0
2	804	0	-625	52	0	0	0
3	804	625	0	53	0	0	0
4	804	-625	0	54	0	0	0
5	804	48	623	55	0	0	0
6	804	48	-623	56	0	0	0
7	804	-48	623	57	0	0	0
8	804	-48	-623	58	0	0	0
9	804	232	580	59	0	0	0
10	804	232	-580	60	0	0	0
11	804	-232	580	61	0	0	0
12	804	-232	-580	62	0	0	0
13	804	282	558	63	0	0	0
14	804	282	-558	64	0	0	0
15	804	-282	558	65	0	0	0
16	804	-282	-558	66	0	0	0
17	804	330	530	67	0	0	0
18	804	330	-530	68	0	0	0
19	804	-330	530	69	0	0	0
20	804	-330	-530	70	0	0	0
21	804	447	403	71	0	0	0
22	804	447	-403	72	0	0	0
23	804	-447	403	73	0	0	0
24	804	-447	-403	74	0	0	0
25	804	540	313	75	0	0	0
26	804	540	-313	76	0	0	0
27	804	-540	313	77	0	0	0
28	804	-540	-313	78	0	0	0
29	804	587	215	79	0	0	0
30	804	587	-215	80	0	0	0
31	804	-587	215	81	0	0	0
32	804	-587	-215	82	0	0	0
33	804	615	110	83	0	0	0
34	804	615	-110	84	0	0	0
35	804	-615	110	85	0	0	0
36	804	-615	-110	86	0	0	0
37	0	0	0	87	0	0	0
38	0	0	0	88	0	0	0
39	0	0	0	89	0	0	0
40	0	0	0	90	0	0	0
41	0	0	0	91	0	0	0
42	0	0	0	92	0	0	0
43	0	0	0	93	0	0	0
44	0	0	0	94	0	0	0
45	0	0	0	95	0	0	0
46	0	0	0	96	0	0	0
47	0	0	0	97	0	0	0
48	0	0	0	98	0	0	0
49	0	0	0	99	0	0	0
50	0	0	0	100	0	0	0

Rebar Data Layer 2 - generated from input

Ref	Area mm2	X mm	Y mm	Ref	Area mm2	X mm	Y mm
1	804	0	574	51	0	0	0
2	804	0	-574	52	0	0	0
3				53	0	0	0
4				54	0	0	0
5				55	0	0	0
6				56	0	0	0
7				57	0	0	0
8				58	0	0	0
9				59	0	0	0
10				60	0	0	0
11	804	258	512	61	0	0	0
12	804	258	-512	62	0	0	0
13	804	-258	512	63	0	0	0
14	804	-258	-512	64	0	0	0
15				65	0	0	0
16				66	0	0	0
17				67	0	0	0
18				68	0	0	0
19	804	497	285	69	0	0	0
20	804	497	-285	70	0	0	0
21	804	-497	285	71	0	0	0
22	804	-497	-285	72	0	0	0
23				73	0	0	0
24				74	0	0	0
25				75	0	0	0
26				76	0	0	0
27				77	0	0	0
28	8040	0	88	78	8040	0	-88
29	8040	0	56	79	8040	0	-56
30	8040	0	24	80	8040	0	-24
31	0	0	0	81	0	0	0
32	0	0	0	82	0	0	0
33	283	0	120	83	283	0	-120
34	283	0	120	84	283	0	-120
35	283	0	120	85	283	0	-120
36	283	0	120	86	283	0	-120
37	283	0	120	87	283	0	-120
38	283	0	120	88	283	0	-120
39	283	0	120	89	283	0	-120
40	283	0	120	90	283	0	-120
41	283	0	120	91	283	0	-120
42	283	0	120	92	283	0	-120
43	283	0	120	93	283	0	-120
44	283	0	120	94	283	0	-120
45	283	0	120	95	283	0	-120
46	283	0	120	96	283	0	-120
47	283	0	120	97	283	0	-120
48	283	0	120	98	283	0	-120
49	283	0	120	99	283	0	-120
50	0	0	0	100	0	0	0

$A1 := A1 \cdot mm^2$ $A2 := A2 \cdot mm^2$ $A3 := A3 \cdot mm^2$ $A4 := A4 \cdot mm^2$

$X1 := X1 \cdot mm$ $X2 := X2 \cdot mm$ $X3 := X3 \cdot mm$ $X4 := X4 \cdot mm$

$Y1 := Y1 \cdot mm$ $Y2 := Y2 \cdot mm$ $Y3 := Y3 \cdot mm$ $Y4 := Y4 \cdot mm$

$k := 1..50$ $A_{bar_k} := A1_k$ $x_{bar_k} := X1_k$ $y_{bar_k} := Y1_k$

$A_{bar_{k+50}} := A2_k$ $x_{bar_{k+50}} := X2_k$ $y_{bar_{k+50}} := Y2_k$

$A_{bar_{k+100}} := A3_k$ $x_{bar_{k+100}} := X3_k$ $y_{bar_{k+100}} := Y3_k$

$A_{bar_{k+150}} := A4_k$ $x_{bar_{k+150}} := X4_k$ $y_{bar_{k+150}} := Y4_k$

Calculate Section Properties of Reinforcement

$$A_{BAR} := \sum_{j=1}^{200} A_{bar_j}$$ $A_{BAR} = 94855\ mm^2$

$$\rho := \frac{A_{BAR}}{A_C}$$ $\rho = 0.0618$

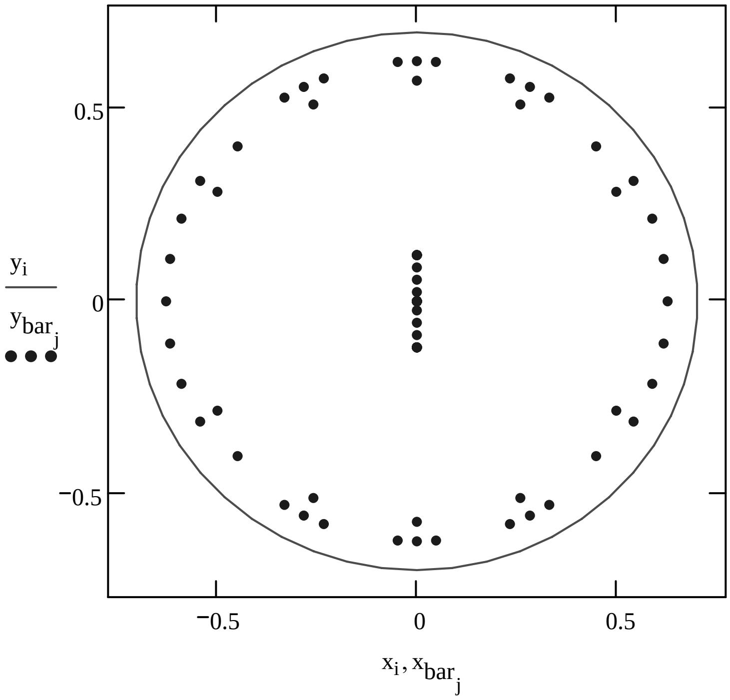
$$x_b := \begin{cases} \left[\sum_{j=1}^{200} \left(A_{bar_j} \cdot x_{bar_j}\right)\right] \cdot \frac{1}{A_{BAR}} & \text{if } A_{BAR} > 0 \\ 0m & \text{otherwise} \end{cases}$$ $x_b = -4 \times 10^{-6}\ m$

$$y_b := \begin{cases} \left[\sum_{j=1}^{200} \left(A_{bar_j} \cdot y_{bar_j}\right)\right] \cdot \frac{1}{A_{BAR}} & \text{if } A_{BAR} > 0 \\ 0m & \text{otherwise} \end{cases}$$ $y_b = 0\ m$

$$I_{xb} := \sum_{j=1}^{200} \left[A_{bar_j} \cdot \left(x_{bar_j}\right)^2\right] + A_{BAR} \cdot x_b^2$$ $I_{xb} = 0.00633\ m^4$

$$I_{yb} := \sum_{j=1}^{200} \left[A_{bar_j} \cdot \left(y_{bar_j}\right)^2\right] + A_{BAR} \cdot y_b^2$$ $I_{yb} = 0.00785\ m^4$

$j := 1 .. 200$



Calculate Composite Section Properties (before cracking)

Effective area $A_E := A_C \cdot [1 + \rho \cdot (\alpha - 1)] + A_{ST} \cdot \alpha$ $\quad A_E = 2177416\ mm^2$

Effective centroid $x_E := \frac{A_C \cdot [(1-\rho) \cdot x_C + \rho \cdot x_b \cdot \alpha] + A_{ST} \cdot \alpha \cdot x_{ST}}{A_E}$ $\quad x_E = -0.000\ m$

$y_E := \frac{A_C \cdot [(1-\rho) \cdot y_C + \rho \cdot y_b \cdot \alpha] + A_{ST} \cdot \alpha \cdot x_{ST}}{A_E}$ $\quad y_E = 0.000\ m$

Effective stiffness $I_{EX} := I_{xC} + I_{xb} \cdot (\alpha - 1) + A_C \cdot \left[(1-\rho) \cdot x_C^2 + \rho \cdot x_b^2 \cdot \alpha\right] + \left(I_{xS} + A_{ST} \cdot x_{ST}^2\right) \cdot \alpha$

$I_{EX} = 2 \times 10^7\ cm^4$

$I_{EY} := I_{yC} + I_{yb} \cdot (\alpha - 1) + A_C \cdot \left[(1-\rho) \cdot y_C^2 + \rho \cdot y_b^2 \cdot \alpha\right] + \left(I_{yS} + A_{ST} \cdot y_{ST}^2\right) \cdot \alpha$

$I_{EY} = 2 \times 10^7\ cm^4$

Distance from extreme concrete fiber to centroid

$xF_{pos} := \max(x - x_E)$ $\quad xF_{neg} := \min(x - x_E)$

$yF_{pos} := \max(y - y_E)$ $\quad yF_{neg} := \min(y - y_E)$

Total depth of concrete section

$H_{CX} := xF_{pos} - xF_{neg}$ $\quad H_{CX} = 1\ m$

$H_{CY} := yF_{pos} - yF_{neg}$ $\quad H_{CY} = 1\ m$

Section modulus

$$Z_{Xpos} := \frac{I_{EX}}{xF_{pos}} \qquad Z_{Xneg} := \frac{I_{EX}}{xF_{neg}}$$

$$Z_{Ypos} := \frac{I_{EY}}{yF_{pos}} \qquad Z_{Yneg} := \frac{I_{EY}}{yF_{neg}}$$

Thickness of steel tube:

$ts := y_1 - ys_1$ $\qquad ts = 0\,mm$

Establish Section Dimensions

Positive case - determine coord of extreme concrete fiber $\quad y_{Epos} := max(y) \quad y_{Epos} = 700\,mm$

Negative case - determine coord of extreme concrete fiber $\quad y_{Eneg} := min(y) \quad y_{Eneg} = -700\,mm$

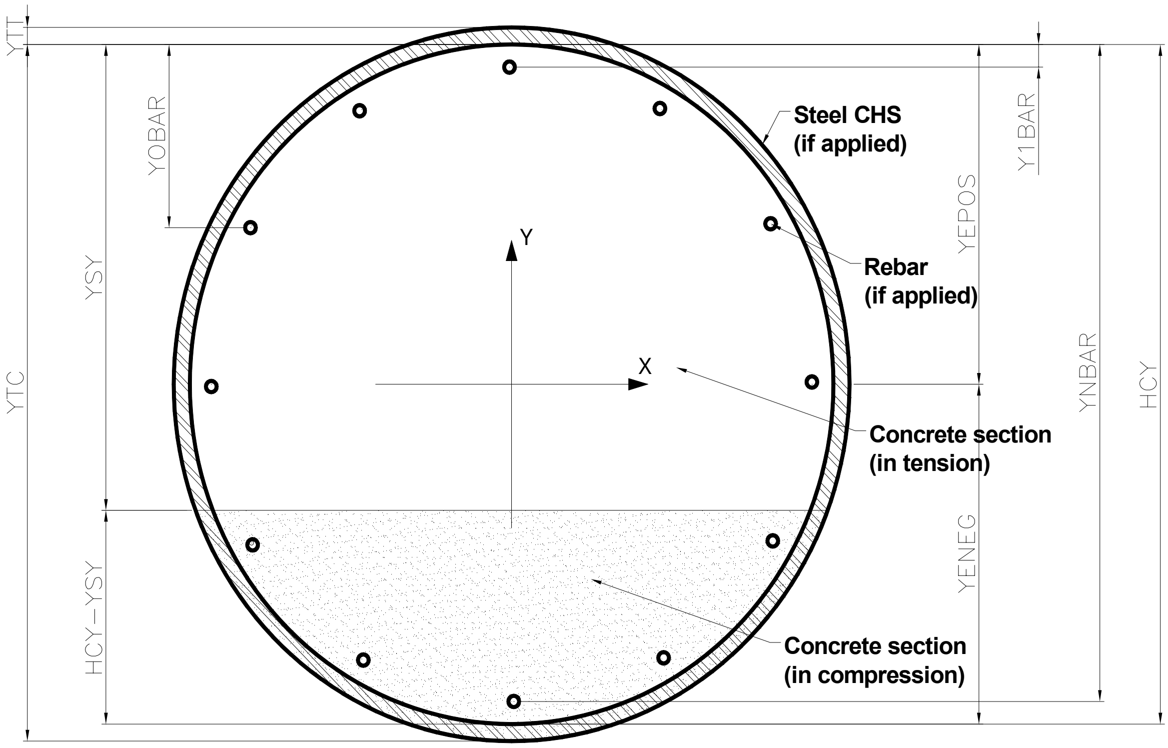
Offsets of rebar from extreme fiber $\quad y_{Obar} := y_{Epos} - y_{bar}$

Determine most extreme rebar (minimum offset) $\quad y_{1bar} := min(y_{Epos} - y_{bar}) \quad y_{1bar} = 75\,mm$

Determine most extreme rebar (maximum offset) $\quad y_{nbar} := max(y_{Epos} - y_{bar}) \quad y_{nbar} = 1325\,mm$

Offsets of extreme steel tube fiber from extreme concrete fiber $\quad y_{tt} := ts \quad y_{tt} = 0\,mm$

$y_{tc} := H_{CY} + ts \quad y_{tc} = 1400\,mm$



ASSIGN NEUTRAL AXIS VALUES

Number of sections to analysed $ns := 500$

$q := 2 .. ns$

Distance of neutral axis from extreme fiber in tension $y_{SY_q} := H_{CY} \cdot \frac{q}{ns + 1}$

Calculate stresses and strains in reinforcement and concrete at extreme fibers

Calculate strain at extreme compression fiber assuming max allowable stress in concrete:

Trial value of concrete strain

$$\varepsilon cc := \frac{\sigma_{cc}}{E_C} \cdot 2 \qquad \frac{\sigma_{cc}}{E_C} = 0.001165$$

Given

$$\sigma_{cc} = \varepsilon cc \cdot \left(4700 \sqrt{\frac{f_c \cdot 2}{MPa}} - \frac{4700^2}{2.68} \cdot \varepsilon cc\right) \cdot MPa$$

$$\varepsilon_{cc} := Find(\varepsilon cc) \qquad \varepsilon_{cc} := 0.002$$

$$\varepsilon_{cc} := \begin{cases} \varepsilon_{tc} & \text{if } (f_c = 0) \cdot (A_{BAR} = 0) \\ \varepsilon_{rc} & \text{if } (f_c = 0) \cdot (ts = 0) \\ \varepsilon_{cc} & \text{otherwise} \end{cases} \qquad \varepsilon_{cc} = 0.002000$$

Strain at other stresses taken to be linear:

$$\varepsilon cc(f_c, \sigma_{cd}) := \begin{cases} \frac{\sigma_{cd}}{\sigma_{tc}} \cdot \varepsilon_{tc} & \text{if } (f_c = 0) \cdot (A_{BAR} = 0) \\ \frac{\sigma_{cd}}{\sigma_{rc}} \cdot \varepsilon_{rc} & \text{if } (f_c = 0) \cdot (ts = 0) \\ \frac{\sigma_{cd}}{\sigma_{cc}} \cdot \varepsilon_{cc} & \text{otherwise} \end{cases}$$

Calculate strain in steel tube assuming max allowable stress in concrete:

In compression $\varepsilon_{tcc_q} := \varepsilon_{cc} \cdot \frac{y_{tc} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

In tension $\varepsilon_{tct_q} := \varepsilon_{cc} \cdot \frac{-\left(y_{SY_q} + y_{tt}\right)}{H_{CY} - y_{SY_q}}$

Calculate strain in rebar assuming max allowable stress in concrete:

In compression $\varepsilon_{rcc_q} := \varepsilon_{cc} \cdot \dfrac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

In tension $\varepsilon_{rct_q} := \varepsilon_{cc} \cdot \dfrac{y_{1bar} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

Calculate design max stress in compression taking account of other limits:

$$
\sigma cd(\varepsilon_{tcc}, q) := \left|
\begin{array}{l}
\sigma_{cd} \leftarrow \sigma_{cc} \text{ if } f_c > 0 \\
\sigma_{cd} \leftarrow \sigma_{tc} \text{ if } (f_c = 0)\cdot(A_{BAR} = 0) \\
\sigma_{cd} \leftarrow \sigma_{rc} \text{ if } (f_c = 0)\cdot(ts = 0) \\
\sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{tc}}{\varepsilon_{tcc}} \text{ if } (\varepsilon_{tcc} > \varepsilon_{tc})\cdot(ts > 0) \\
\sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{rc}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}}} \text{ if } \left(\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}} > \varepsilon_{rc}\right)\cdot(A_{BAR} > 0) \\
\sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{ts}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} + y_{tt})}{H_{CY} - y_{SY_q}}} \text{ if } \left[\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} + y_{tt})}{H_{CY} - y_{SY_q}} < \varepsilon_{ts}\right]\cdot(ts > 0) \\
\sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{rs}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} - y_{1bar})}{H_{CY} - y_{SY_q}}} \text{ if } \left[\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} - y_{1bar})}{H_{CY} - y_{SY_q}} < \varepsilon_{rs}\right]\cdot(A_{BAR} > 0) \\
\sigma_{cc} \text{ otherwise}
\end{array}
\right.
$$

$$\sigma_{cd_q} := \sigma cd(\varepsilon_{tcc_q}, q)$$

CALCULATE FORCES AND MOMENTS AT EACH NEUTRAL AXIS LOCATION

Calculate force in concrete:

$$F_{C_q} := \begin{cases} \displaystyle\int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot(1-\rho)\cdot\left[\frac{\sigma_{cd_q}\cdot\left[y+\left(\frac{H_{CY}}{2}-y_{SY_q}\right)\right]}{H_{CY}-y_{SY_q}}\right] dy & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from concrete about column centroid:

$$M_{C_q} := \begin{cases} \displaystyle\int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot(1-\rho)\cdot\left[\frac{\sigma_{cd_q}\cdot\left[y+\left(\frac{H_{CY}}{2}-y_{SY_q}\right)\right]}{H_{CY}-y_{SY_q}}\right]\cdot y\, dy & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate strain in rebar assuming design max stress in concrete:

$$y_{SY_q} := \begin{cases} y_{nbar}\cdot\dfrac{q}{ns+1} & \text{if } (f_c = 0)\cdot(A_{BAR} > 0) \\ y_{SY_q} & \text{otherwise} \end{cases}$$

$$\varepsilon_{S_{j,q}} := \begin{cases} -\dfrac{y_{SY_q} - y_{Obar_j}}{y_{nbar} - y_{SY_q}}\cdot\varepsilon cc\left(f_c, \sigma_{cd_q}\right) & \text{if } f_c = 0 \\ -\dfrac{y_{SY_q} - y_{Obar_j}}{H_{CY} - y_{SY_q}}\cdot\varepsilon cc\left(f_c, \sigma_{cd_q}\right) & \text{otherwise} \end{cases}$$

Calculate force in each rebar:

$$F_{S_{j,q}} := \begin{cases} \varepsilon_{S_{j,q}} E_S\cdot A_{bar_j} & \text{if } A_{BAR} > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate total force in reinforcement:

$$F_{R_q} := \sum_j F_{S_{j,q}}$$

Calculate moment from reinforcement about section centroid:

$$M_{R_q} := \begin{cases} \sum_j -\left(\varepsilon_{S_{j,q}} E_S \cdot A_{bar_j} \cdot y_{bar_j}\right) & \text{if } A_{BAR} > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate strain in steel tube at extreme tension fiber:

$$\varepsilon_{tds_q} := \frac{-\left(y_{SY_q} + y_{tt}\right)}{H_{CY} - y_{SY_q}} \varepsilon cc\left(f_c, \sigma_{cd_q}\right)$$

Calculate strain in steel tube at extreme compression fiber:

$$\varepsilon_{tdc_q} := \frac{y_{tc} - y_{SY_q}}{H_{CY} - y_{SY_q}} \varepsilon cc\left(f_c, \sigma_{cd_q}\right)$$

Calculate tensile force in steel tube:

$$F_{TS1_q} := \int_{\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2} + y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tds_q} \cdot E_S \cdot \left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TS2_q} := \int_{\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tds_q} \cdot E_S \cdot \left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TS} := \begin{cases} F_{TS1} - F_{TS2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate compressive force in steel tube:

$$F_{TC1_q} := \int_{-\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2} + y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tdc_q} \cdot E_S \cdot \left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TC2_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tdc_q}\cdot E_S\cdot\left[y+\left(\frac{H_{CY}}{2}-y_{SY_q}\right)\right]}{H_{CY}-y_{SY_q}+y_{tt}}\right]dy$$

$$F_{TC} := \begin{cases} F_{TC1} - F_{TC2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from tensile force in steel tube:

$$M_{TS1_q} := \int_{\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}+y_{tt}} -2\sqrt{\left(\frac{H_{CY}}{2}+y_{tt}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tds_q}\cdot E_S\cdot\left[y-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)\right]}{y_{SY_q}+y_{tt}}\right]\cdot y\,dy$$

$$M_{TS2_q} := \int_{\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} -2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tds_q}\cdot E_S\cdot\left[y-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)\right]}{y_{SY_q}+y_{tt}}\right]\cdot y\,dy$$

$$M_{TS} := \begin{cases} M_{TS1} - M_{TS2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from compressive force in steel tube:

$$M_{TC1_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}+y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2}+y_{tt}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tdc_q}\cdot E_S\cdot\left[y+\left(\frac{H_{CY}}{2}-y_{SY_q}\right)\right]}{H_{CY}-y_{SY_q}+y_{tt}}\right]\cdot y\,dy$$

$$M_{TC2_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tdc_q}\cdot E_S\cdot\left[y+\left(\frac{H_{CY}}{2}-y_{SY_q}\right)\right]}{H_{CY}-y_{SY_q}+y_{tt}}\right]\cdot y\,dy$$

$$M_{TC} := \begin{cases} M_{TC1} - M_{TC2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calate total axial response from section:

$$F_T := F_C + F_R + F_{TS} + F_{TC} \qquad F_{TC} := F_C$$

Calculate total moment response from section:

$$M_T := M_C + M_R + M_{TS} + M_{TC} \qquad M_{TC} := M_C$$

CALCULATE MAXIMUM ALLOWABLE AXIAL FORCE IN SECTION

Limiting strain in axial compression:

$$\varepsilon_{cL} := \begin{cases} \min(\varepsilon_{cc},\varepsilon_{tc}) & \text{if } (A_{BAR} = 0)\cdot(ts \neq 0)\cdot(f_c \neq 0) \\ \min(\varepsilon_{cc},\varepsilon_{rc}) & \text{if } (ts = 0)\cdot(A_{BAR} \neq 0)\cdot(f_c \neq 0) \\ \varepsilon_{tc} & \text{if } (A_{BAR} = 0)\cdot(f_c = 0) \\ \varepsilon_{rc} & \text{if } (ts = 0)\cdot(f_c = 0) \\ \min(\varepsilon_{cc},\varepsilon_{rc},\varepsilon_{tc}) & \text{otherwise} \end{cases} \qquad \varepsilon_{cL} = 0.001950$$

Limiting concrete stress in axial compression

$$\sigma_{cL} := \begin{cases} \sigma_{cd_2} & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases} \qquad \sigma_{cL} = 30\,\text{MPa}$$

$$P_{MAX} := \sigma_{cL}\cdot A_C(1-\rho) + \varepsilon_{cL}\cdot E_S(A_{BAR} + A_{ST})$$

$P_{MAX} = 80207.7\,\text{kN}$ $\qquad F_{T_1} := P_{MAX}$ $\qquad M_{T_1} := 0\cdot\text{kN}\cdot\text{m}$

$$P_{MAXC} := \sigma_{cL}\cdot A_C\cdot(1-\rho)$$

$P_{MAXC} = 43214.3\,\text{kN}$ $\qquad F_{TC_1} := P_{MAXC}$ $\qquad M_{TC_1} := 0\cdot\text{kN}\cdot\text{m}$

CALCULATE MINIMUM ALLOWABLE AXIAL FORCE IN SECTION

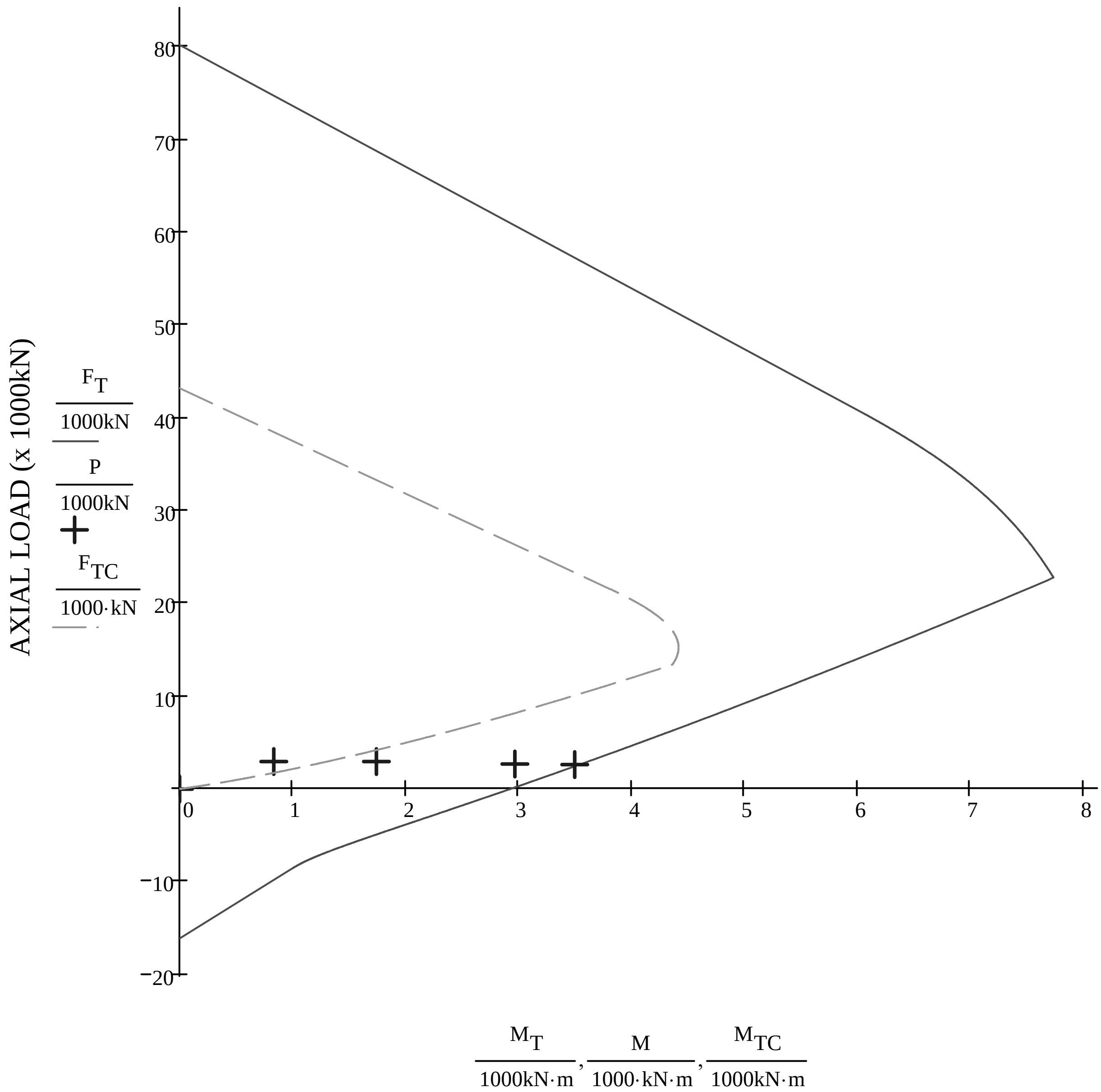
$$P_{MIN} := \begin{cases} \varepsilon_{rs}\cdot E_S(A_{BAR}) & \text{if } ts = 0 \\ \varepsilon_{ts}\cdot E_S(A_{ST}) & \text{if } A_{BAR} = 0 \\ \max(\varepsilon_{ts},\varepsilon_{rs})\cdot E_S(A_{BAR} + A_{ST}) & \text{otherwise} \end{cases}$$

$P_{MIN} = -16125.3\,\text{kN}$ $\qquad F_{T_{ns+1}} := P_{MIN}$ $\qquad M_{T_{ns+1}} := 0\cdot\text{kN}\cdot\text{m}$

$$\text{Limit} := \begin{cases} \min(P, F_T)\cdot 0.75 & \text{if } \min(P) > 0 \\ \min(P, F_T)\cdot 1.25 & \text{otherwise} \end{cases}$$

$P_{MINC} := 0\text{kN}$ $\qquad M_{TC_{ns+1}} := 0\cdot\text{kN}\cdot\text{m}$

Diameter of Column	$D = 1400\,mm$	Characteristic strength of concrete	$f_c = 30\,MPa$
Percentage reinforcement	$\rho = 6.18\,\%$	Yield Strength of Rebar	$f_y = 390\,MPa$
Thickness of CHS	$ts = 0\,mm$	Yield Strength of CHS	$f_{ys} = 250\,MPa$



INTERACTION CURVE AT SERVICEABILITY LIMIT STATE

Equation of interaction line - upper region (between 1 and 2 calculation points)

$$m1 := \frac{M_{T_2} - M_{T_1}}{F_{T_2} - F_{T_1}} \qquad c1 := F_{T_1}$$

Equation of interaction line - lower region (between ns and ns+1 calculation points)

$$m2 := \frac{M_{T_{ns}} - M_{T_{ns+1}}}{F_{T_{ns}} - F_{T_{ns+1}}} \qquad c2 := F_{T_{ns+1}}$$

$r := 1 .. 8$

$$M_{SLS_r} := \begin{cases} 0.000000000000000001 \cdot kN \cdot m & \text{if } P_r > F_{T_1} \\ 0.000000000000000001 \cdot kN \cdot m & \text{if } P_r < F_{T_{ns+1}} \\ (P_r - c1) \cdot m1 & \text{if } (P_r > F_{T_2}) \cdot (P_r \le F_{T_1}) \\ (P_r - c2) \cdot m2 & \text{if } (P_r \ge F_{T_{ns+1}}) \cdot (P_r < F_{T_{ns}}) \\ \text{otherwise} & \begin{array}{l} j \leftarrow 1 \\ \text{while } F_{T_j} > P_r \\ \quad j \leftarrow j + 1 \\ M_{T_j} \end{array} \end{cases}$$

$$StressFactor_r := \begin{cases} \text{"No Result"} & \text{if } M_{SLS_r} < 0.00000000000000001 \cdot kN \cdot m \\ \dfrac{M_r}{M_{SLS_r}} & \text{otherwise} \end{cases}$$

$$P = \begin{pmatrix} 2684 \\ 2619 \\ 2952 \\ 2952 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kN \qquad M = \begin{pmatrix} 2969 \\ 3499 \\ 836 \\ 1744 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kN \cdot m \qquad M_{SLS} = \begin{pmatrix} 3532.8 \\ 3532.8 \\ 3612.6 \\ 3612.6 \\ 2921.1 \\ 2921.1 \\ 2921.1 \\ 2921.1 \end{pmatrix} kN \cdot m \qquad StressFactor = \begin{pmatrix} 0.840 \\ 0.990 \\ 0.232 \\ 0.483 \\ 0.000 \\ 0.000 \\ 0.000 \\ 0.000 \end{pmatrix}$$

RESULTS SUMMARY
SERVICEABILITY LIMIT STATE ANALYSIS OF CIRCULAR BEAM COLUMN

Diameter of Column			1400	mm			
Percentage of rebar			6.18	%			
Load Case Ref	**Pier**	**Applied Service Axial Load kN**	**Applied Service Bending Moment kNm**	**Service Limit State Bending Moment kNm**	**Allowable Stress Factor**	**Applied Stress Factor**	**Serviceability Limit State Design Result**
1	A1	2684	2969	3532.8	100%	84%	OK
2	A2	2619	3499	3532.8	100%	99%	OK

TOP

Serviceability Check - Full Live Load

KATAHIRA & ENGINEERS INTERNATIONAL

Project: **Detailed Design Study of North Java Corridor Flyover Project**

Calculation: **Balaraja Flyover**
Serviceability Check - Full Live Load
1400mm Dia Circular RC Column - Top Section

Reference: **Project Specific Design Criteria**

Section Data

$MPa := 1000000 \cdot Pa$ $kN := 1000 \cdot N$

Input Item			
Concrete Compressive Strength	fc	30	MPa
Structural Steel Yield Strength	fys	250	MPa
Rebar Yield Strength	fy	390	MPa
Diameter of reinforced concrete section	D	1400	mm
Thickness of CHS section	t	0	mm
Diameter of rebar - layer 1	dia1	32	mm
Diameter of rebar - layer 2	dia2	32	mm
Number bars - layer 1 (max 100)	n1	36	
Number bars - layer 2 (max 100)	n2	26	
Cover from face of section - layer 1	cov1	60	mm
Cover from face of section - layer 2	cov2	115	mm

Load Data

Ref	Pier	Load Case	P	M	Stress
			kN	kNm	Allowance
1	A1	Combination 1 - P + Setlement + Traffic + Temp + Shcr	2416.0	5011.0	140%
2	A2	Combination 1 - P + Setlement + Traffic + Temp + Shcr	2412.0	5157.0	140%

$f_c := f_c \cdot MPa \quad f_{ys} := f_{ys} \cdot MPa \quad f_y := f_y \cdot MPa \qquad D := D \cdot mm \qquad ts := ts \cdot mm$

$dia1 := dia1 \cdot mm \qquad dia2 := dia2 \cdot mm \qquad cov1 := cov1 \cdot mm \qquad cov2 := cov2 \cdot mm$

$P := P \cdot kN \qquad M := M \cdot kN \cdot m$

$E_S := 200000 \cdot MPa \qquad E_C := 4700\sqrt{\frac{f_c}{MPa}} \cdot MPa$ Modular ratio $\alpha := \begin{cases} \frac{E_S}{E_C} & \text{if } E_C > 0 \\ 1 & \text{otherwise} \end{cases} \qquad \alpha = 7.77$

$E_C = 25743\,MPa$

Calculate Basic Allowable Stresses

Calculate rupture stress: $\sigma_{ct} := 0.5 \cdot \left(\frac{f_c}{MPa}\right)^{\frac{2}{3}} \cdot MPa \qquad \sigma_{ct} = 4.8\,MPa$

Calculate basic allowable stress of concrete $\sigma_{cc} := 1.0 \cdot f_c \qquad \sigma_{cc} = 30.0\,MPa$

Calculate basic allowable tensile stress of rebar $\sigma_{rs} := \begin{cases} 0.5 \cdot f_y & \text{if } 0.5 \cdot f_y \le 170MPa \\ 170MPa & \text{otherwise} \end{cases} \qquad \sigma_{rs} = 170\,MPa$

Calculate basic allowable compressive stress of rebar $\sigma_{rc} := \begin{cases} 0.5 \cdot f_y & \text{if } 0.5 \cdot f_y \le 110MPa \\ f_y & \text{otherwise} \end{cases} \qquad \sigma_{rc} = 390\,MPa$

Calculate basic allowable stress of structural steel $\sigma_{ts} := -0.6 f_{ys} \qquad \sigma_{ts} = -150\,MPa$

$\sigma_{tc} := 1 f_{ys} \qquad \sigma_{tc} = 250\,MPa$

Limiting strain of rebar $\varepsilon_{rs} := -\frac{\sigma_{rs}}{E_S} \qquad \varepsilon_{rs} = -0.000850$

$\varepsilon_{rc} := \frac{\sigma_{rc}}{E_S} \qquad \varepsilon_{rc} = 0.001950$

Limiting strain of structural steel $\varepsilon_{ts} := \frac{\sigma_{ts}}{E_S} \qquad \varepsilon_{ts} = -0.000750$

$\varepsilon_{tc} := \frac{\sigma_{tc}}{E_S} \qquad \varepsilon_{tc} = 0.001250$

Concrete Cross Section Data - generated

$n := 50$ Number of Points - 50 points maximum

$i := 1 .. n + 1$ Range from 1 to n+1

Ref.	X	Y	Ref.	X	Y
	mm	mm		mm	mm
1	0	-700	**26**	0	700
2	-88	-694	**27**	88	694
3	-174	-678	**28**	174	678
4	-258	-651	**29**	258	651
5	-337	-613	**30**	337	613
6	-411	-566	**31**	411	566
7	-479	-510	**32**	479	510
8	-539	-446	**33**	539	446
9	-591	-375	**34**	591	375
10	-633	-298	**35**	633	298
11	-666	-216	**36**	666	216
12	-688	-131	**37**	688	131
13	-699	-44	**38**	699	44
14	-699	44	**39**	699	-44
15	-688	131	**40**	688	-131
16	-666	216	**41**	666	-216
17	-633	298	**42**	633	-298
18	-591	375	**43**	591	-375
19	-539	446	**44**	539	-446
20	-479	510	**45**	479	-510
21	-411	566	**46**	411	-566
22	-337	613	**47**	337	-613
23	-258	651	**48**	258	-651
24	-174	678	**49**	174	-678
25	-88	694	**50**	88	-694

$k := 1 .. 25$ $XS1 := XS1 \cdot mm$ $XS2 := XS2 \cdot mm$ $YS1 := YS1 \cdot mm$ $YS2 := YS2 \cdot mm$

$x_k := XS1_k$ $y_k := YS1_k$ $x_{k+25} := XS2_k$ $y_{k+25} := YS2_k$ $x_{n+1} := XS1_1$ $y_{n+1} := YS1_1$

Calculate Section Properties of Concrete Section

$$A_C := -\sum_{i=1}^{n} \left[\left(y_{i+1} - y_i\right) \cdot \frac{x_{i+1} + x_i}{2} \right]$$

$A_C = 1.53533\ m^2$

$$x_C := -\frac{1}{A_C} \cdot \sum_{i=1}^{n} \left[\frac{y_{i+1} - y_i}{8} \cdot \left[\left(x_{i+1} + x_i\right)^2 + \frac{\left(x_{i+1} - x_i\right)^2}{3} \right] \right]$$

$x_C = 0\ m$

$$y_C := \frac{1}{A_C} \cdot \sum_{i=1}^{n} \left[\frac{x_{i+1} - x_i}{8} \cdot \left[\left(y_{i+1} + y_i\right)^2 + \frac{\left(y_{i+1} - y_i\right)^2}{3} \right] \right]$$

$y_C = 0\ m$

$$I_x := \sum_{i=1}^{n} \left[\left[\left(x_{i+1} - x_i\right) \cdot \frac{y_{i+1} + y_i}{24} \right] \cdot \left[\left(y_{i+1} + y_i\right)^2 + \left(y_{i+1} - y_i\right)^2 \right] \right]$$

$I_x = 0.18758\ m^4$

$$I_y := -\sum_{i=1}^{n} \left[\left[\left(y_{i+1} - y_i\right) \cdot \frac{x_{i+1} + x_i}{24} \right] \cdot \left[\left(x_{i+1} + x_i\right)^2 + \left(x_{i+1} - x_i\right)^2 \right] \right]$$

$I_y = 0.18758\ m^4$

$$I_{xC} := I_x - A_C \cdot x_C^{\,2}$$

$I_{xC} = 0.18758\ m^4$

$$I_{yC} := I_y - A_C \cdot y_C^{\,2}$$

$I_{yC} = 0.18758\ m^4$

Steel Tube Cross Section Data - generated from input

$ns := 50$ Number of Points - 50 points maximum

$ps := 1..ns+1$ Range from 1 to ns+1

Ref.	X	Y	Ref.	X	Y
	mm	mm		mm	mm
1	0	-700	**26**	0	-700
2	-181	-676	**27**	181	-676
3	-350	-606	**28**	350	-606
4	-495	-495	**29**	495	-495
5	-606	-350	**30**	606	-350
6	-676	-181	**31**	676	-181
7	-700	0	**32**	700	0
8	-676	181	**33**	676	181
9	-606	350	**34**	606	350
10	-495	495	**35**	495	495
11	-350	606	**36**	350	606
12	-181	676	**37**	181	676
13	0	700	**38**	0	700
14	181	676	**39**	-181	676
15	350	606	**40**	-350	606
16	495	495	**41**	-495	495
17	606	350	**42**	-606	350
18	676	181	**43**	-676	181
19	700	0	**44**	-700	0
20	676	-181	**45**	-676	-181
21	606	-350	**46**	-606	-350
22	495	-495	**47**	-495	-495
23	350	-606	**48**	-350	-606
24	181	-676	**49**	-181	-676
25	0	-700	**50**	0	-700

$XSS1 := XSS1 \cdot mm$ $XSS2 := XSS2 \cdot mm$ $YSS1 := YSS1 \cdot mm$ $YSS2 := YSS2 \cdot mm$

$z := 1..25$ $xs_z := XSS1_z$ $ys_z := YSS1_z$

$z := 26..50$ $xs_z := XSS2_{z-25}$ $ys_z := YSS2_{z-25}$

$xs_{ns+1} := XSS1_1$ $ys_{ns+1} := YSS1_1$

Calculate Section Properties of Steel Tube Section

$$A_{ST} := -\sum_{ps=1}^{ns}\left[\left(ys_{ps+1} - ys_{ps}\right)\cdot\frac{xs_{ps+1} + xs_{ps}}{2}\right]$$ $A_{ST} = 0\ m^2$

$$x_{ST} := -\frac{1}{A_{ST}}\cdot\sum_{ps=1}^{ns}\left[\frac{ys_{ps+1} - ys_{ps}}{8}\cdot\left[\left(xs_{ps+1} + xs_{ps}\right)^2 + \frac{\left(xs_{ps+1} - xs_{ps}\right)^2}{3}\right]\right]$$ $x_{ST} = -1.0\ m$

$$y_{ST} := \frac{1}{A_{ST}}\cdot\sum_{ps=1}^{ns}\left[\frac{xs_{ps+1} - xs_{ps}}{8}\cdot\left[\left(ys_{ps+1} + ys_{ps}\right)^2 + \frac{\left(ys_{ps+1} - ys_{ps}\right)^2}{3}\right]\right]$$ $y_{ST} = -0.499\ m$

$$I_{xS} := \sum_{ps=1}^{ns}\left[\left[\left(xs_{ps+1} - xs_{ps}\right)\cdot\frac{ys_{ps+1} + ys_{ps}}{24}\right]\cdot\left[\left(ys_{ps+1} + ys_{ps}\right)^2 + \left(ys_{ps+1} - ys_{ps}\right)^2\right]\right]$$

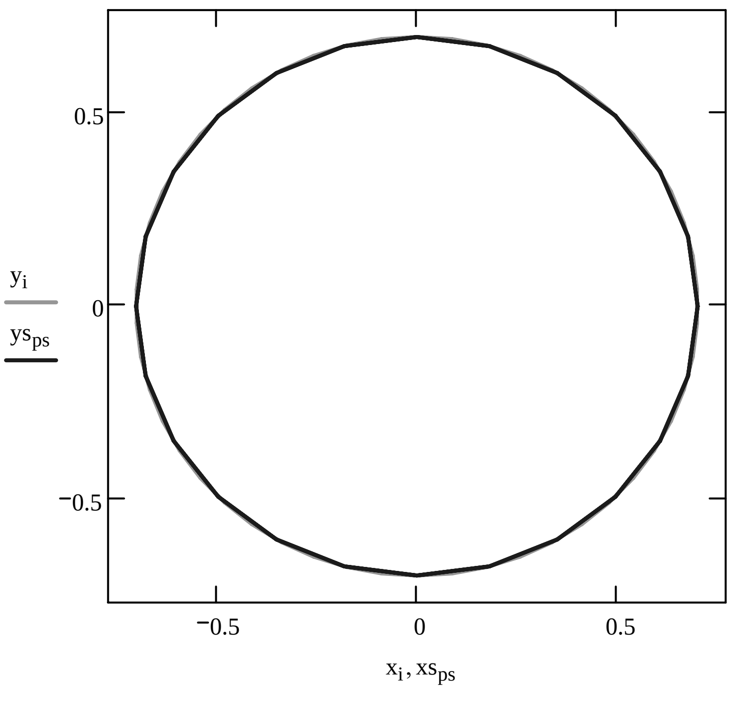
$I_{xS} = 0\ m^4$

$$I_{yS} := -\sum_{ps=1}^{ns}\left[\left[\left(ys_{ps+1} - ys_{ps}\right)\cdot\frac{xs_{ps+1} + xs_{ps}}{24}\right]\cdot\left[\left(xs_{ps+1} + xs_{ps}\right)^2 + \left(xs_{ps+1} - xs_{ps}\right)^2\right]\right]$$

$I_{yS} = 0\ m^4$

$I_{xS} := I_{xS} - A_{ST}\cdot x_{ST}^2$ $I_{xS} = 0\ m^4$

$I_{yS} := I_{yS} - A_{ST}\cdot y_{ST}^2$ $I_{yS} = 0.00000\ m^4$



Rebar Data Layer 1 - generated from input

Ref	Area mm2	X mm	Y mm	Ref	Area mm2	X mm	Y mm
1	804	0	625	51	0	0	0
2	804	0	-625	52	0	0	0
3	804	625	0	53	0	0	0
4	804	-625	0	54	0	0	0
5	804	48	623	55	0	0	0
6	804	48	-623	56	0	0	0
7	804	-48	623	57	0	0	0
8	804	-48	-623	58	0	0	0
9	804	232	580	59	0	0	0
10	804	232	-580	60	0	0	0
11	804	-232	580	61	0	0	0
12	804	-232	-580	62	0	0	0
13	804	282	558	63	0	0	0
14	804	282	-558	64	0	0	0
15	804	-282	558	65	0	0	0
16	804	-282	-558	66	0	0	0
17	804	330	530	67	0	0	0
18	804	330	-530	68	0	0	0
19	804	-330	530	69	0	0	0
20	804	-330	-530	70	0	0	0
21	804	447	403	71	0	0	0
22	804	447	-403	72	0	0	0
23	804	-447	403	73	0	0	0
24	804	-447	-403	74	0	0	0
25	804	540	313	75	0	0	0
26	804	540	-313	76	0	0	0
27	804	-540	313	77	0	0	0
28	804	-540	-313	78	0	0	0
29	804	587	215	79	0	0	0
30	804	587	-215	80	0	0	0
31	804	-587	215	81	0	0	0
32	804	-587	-215	82	0	0	0
33	804	615	110	83	0	0	0
34	804	615	-110	84	0	0	0
35	804	-615	110	85	0	0	0
36	804	-615	-110	86	0	0	0
37	0	0	0	87	0	0	0
38	0	0	0	88	0	0	0
39	0	0	0	89	0	0	0
40	0	0	0	90	0	0	0
41	0	0	0	91	0	0	0
42	0	0	0	92	0	0	0
43	0	0	0	93	0	0	0
44	0	0	0	94	0	0	0
45	0	0	0	95	0	0	0
46	0	0	0	96	0	0	0
47	0	0	0	97	0	0	0
48	0	0	0	98	0	0	0
49	0	0	0	99	0	0	0
50	0	0	0	100	0	0	0

Rebar Data Layer 2 - generated from input

Ref	Area mm2	X mm	Y mm	Ref	Area mm2	X mm	Y mm
1	804	0	574	51	0	0	0
2	804	0	-574	52	0	0	0
3	804	47	572	53	0	0	0
4	804	47	-572	54	0	0	0
5	804	-47	572	55	0	0	0
6	804	-47	-572	56	0	0	0
7	804	212	533	57	0	0	0
8	804	212	-533	58	0	0	0
9	804	-212	533	59	0	0	0
10	804	-212	-533	60	0	0	0
11	804	258	512	61	0	0	0
12	804	258	-512	62	0	0	0
13	804	-258	512	63	0	0	0
14	804	-258	-512	64	0	0	0
15	804	497	285	65	0	0	0
16	804	497	-285	66	0	0	0
17	804	-497	285	67	0	0	0
18	804	-497	-285	68	0	0	0
19	804	563	107	69	0	0	0
20	804	563	-107	70	0	0	0
21	804	-563	107	71	0	0	0
22	804	-563	-107	72	0	0	0
23	804	302	488	73	0	0	0
24	804	302	-488	74	0	0	0
25	804	-302	488	75	0	0	0
26	804	-302	-488	76	0	0	0
27	0	0	0	77	0	0	0
28	8040	0	88	78	8040	0	-88
29	8040	0	56	79	8040	0	-56
30	8040	0	24	80	8040	0	-24
31	0	0	0	81	0	0	0
32	0	0	0	82	0	0	0
33	283	0	120	83	283	0	-120
34	283	0	120	84	283	0	-120
35	283	0	120	85	283	0	-120
36	283	0	120	86	283	0	-120
37	283	0	120	87	283	0	-120
38	283	0	120	88	283	0	-120
39	283	0	120	89	283	0	-120
40	283	0	120	90	283	0	-120
41	283	0	120	91	283	0	-120
42	283	0	120	92	283	0	-120
43	283	0	120	93	283	0	-120
44	283	0	120	94	283	0	-120
45	283	0	120	95	283	0	-120
46	283	0	120	96	283	0	-120
47	283	0	120	97	283	0	-120
48	283	0	120	98	283	0	-120
49	283	0	120	99	283	0	-120
50	0	0	0	100	0	0	0

$A1 := A1 \cdot mm^2 \quad A2 := A2 \cdot mm^2 \quad A3 := A3 \cdot mm^2 \quad A4 := A4 \cdot mm^2$

$X1 := X1 \cdot mm \quad X2 := X2 \cdot mm \quad X3 := X3 \cdot mm \quad X4 := X4 \cdot mm$

$Y1 := Y1 \cdot mm \quad Y2 := Y2 \cdot mm \quad Y3 := Y3 \cdot mm \quad Y4 := Y4 \cdot mm$

$k := 1 .. 50$

$A_{bar_k} := A1_k \quad x_{bar_k} := X1_k \quad y_{bar_k} := Y1_k$

$A_{bar_{k+50}} := A2_k \quad x_{bar_{k+50}} := X2_k \quad y_{bar_{k+50}} := Y2_k$

$A_{bar_{k+100}} := A3_k \quad x_{bar_{k+100}} := X3_k \quad y_{bar_{k+100}} := Y3_k$

$A_{bar_{k+150}} := A4_k \quad x_{bar_{k+150}} := X4_k \quad y_{bar_{k+150}} := Y4_k$

Calculate Section Properties of Reinforcement

$$A_{BAR} := \sum_{j=1}^{200} A_{bar_j}$$

$A_{BAR} = 107725\,mm^2$

$$\rho := \frac{A_{BAR}}{A_C}$$

$\rho = 0.0702$

$$x_b := \begin{cases} \left[\sum_{j=1}^{200} \left(A_{bar_j} \cdot x_{bar_j}\right)\right] \cdot \frac{1}{A_{BAR}} & \text{if } A_{BAR} > 0 \\ 0m & \text{otherwise} \end{cases}$$

$x_b = -4 \times 10^{-6}\ m$

$$y_b := \begin{cases} \left[\sum_{j=1}^{200} \left(A_{bar_j} \cdot y_{bar_j}\right)\right] \cdot \frac{1}{A_{BAR}} & \text{if } A_{BAR} > 0 \\ 0m & \text{otherwise} \end{cases}$$

$y_b = 0\ m$

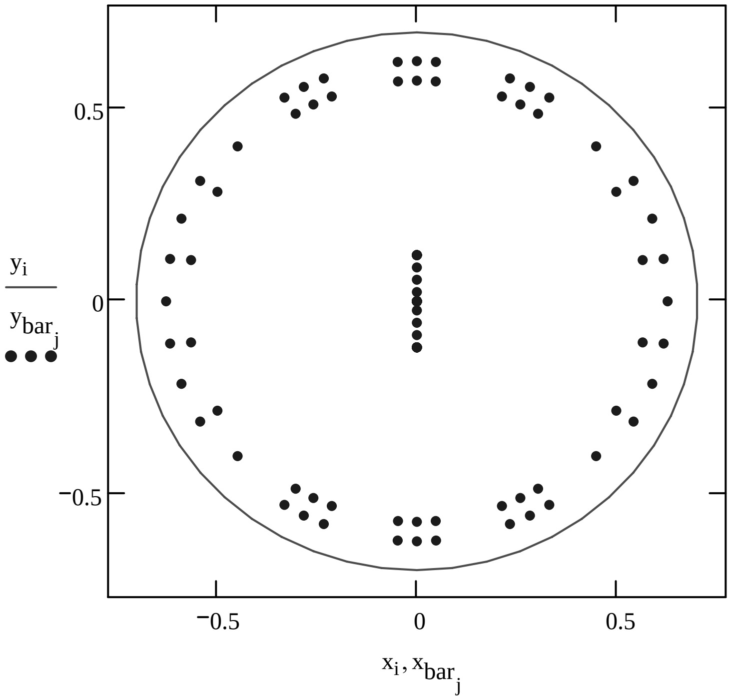
$$I_{xb} := \sum_{j=1}^{200} \left[A_{bar_j} \cdot \left(x_{bar_j}\right)^2\right] + A_{BAR} \cdot x_b^2$$

$I_{xb} = 0.00779\,m^4$

$$I_{yb} := \sum_{j=1}^{200} \left[A_{bar_j} \cdot \left(y_{bar_j}\right)^2\right] + A_{BAR} \cdot y_b^2$$

$I_{yb} = 0.01062\,m^4$

$j := 1 .. 200$



Calculate Composite Section Properties (before cracking)

Effective area $A_E := A_C \cdot [1 + \rho \cdot (\alpha - 1)] + A_{ST} \cdot \alpha$ $\quad A_E = 2264537\ mm^2$

Effective centroid $x_E := \dfrac{A_C \cdot [(1-\rho) \cdot x_C + \rho \cdot x_b \cdot \alpha] + A_{ST} \cdot \alpha \cdot x_{ST}}{A_E}$ $\quad x_E = -0.000\ m$

$y_E := \dfrac{A_C \cdot [(1-\rho) \cdot y_C + \rho \cdot y_b \cdot \alpha] + A_{ST} \cdot \alpha \cdot x_{ST}}{A_E}$ $\quad y_E = 0.000\ m$

Effective stiffness $I_{EX} := I_{xC} + I_{xb} \cdot (\alpha - 1) + A_C \cdot \left[(1-\rho) \cdot x_C^2 + \rho \cdot x_b^2 \cdot \alpha\right] + \left(I_{xS} + A_{ST} \cdot x_{ST}^2\right) \cdot \alpha$

$I_{EX} = 2 \times 10^7\ cm^4$

$I_{EY} := I_{yC} + I_{yb} \cdot (\alpha - 1) + A_C \cdot \left[(1-\rho) \cdot y_C^2 + \rho \cdot y_b^2 \cdot \alpha\right] + \left(I_{yS} + A_{ST} \cdot y_{ST}^2\right) \cdot \alpha$

$I_{EY} = 3 \times 10^7\ cm^4$

Distance from extreme concrete fiber to centroid

$xF_{pos} := \max(x - x_E)$ $\quad xF_{neg} := \min(x - x_E)$

$yF_{pos} := \max(y - y_E)$ $\quad yF_{neg} := \min(y - y_E)$

Total depth of concrete section

$H_{CX} := xF_{pos} - xF_{neg}$ $\quad H_{CX} = 1\ m$

$H_{CY} := yF_{pos} - yF_{neg}$ $\quad H_{CY} = 1\ m$

Section modulus

$$Z_{Xpos} := \frac{I_{EX}}{xF_{pos}} \qquad Z_{Xneg} := \frac{I_{EX}}{xF_{neg}}$$

$$Z_{Ypos} := \frac{I_{EY}}{yF_{pos}} \qquad Z_{Yneg} := \frac{I_{EY}}{yF_{neg}}$$

Thickness of steel tube:

$$ts := y_1 - ys_1 \qquad ts = 0\,mm$$

Establish Section Dimensions

Positive case - determine coord of extreme concrete fiber $y_{Epos} := \max(y)$ $y_{Epos} = 700\,mm$

Negative case - determine coord of extreme concrete fiber $y_{Eneg} := \min(y)$ $y_{Eneg} = -700\,mm$

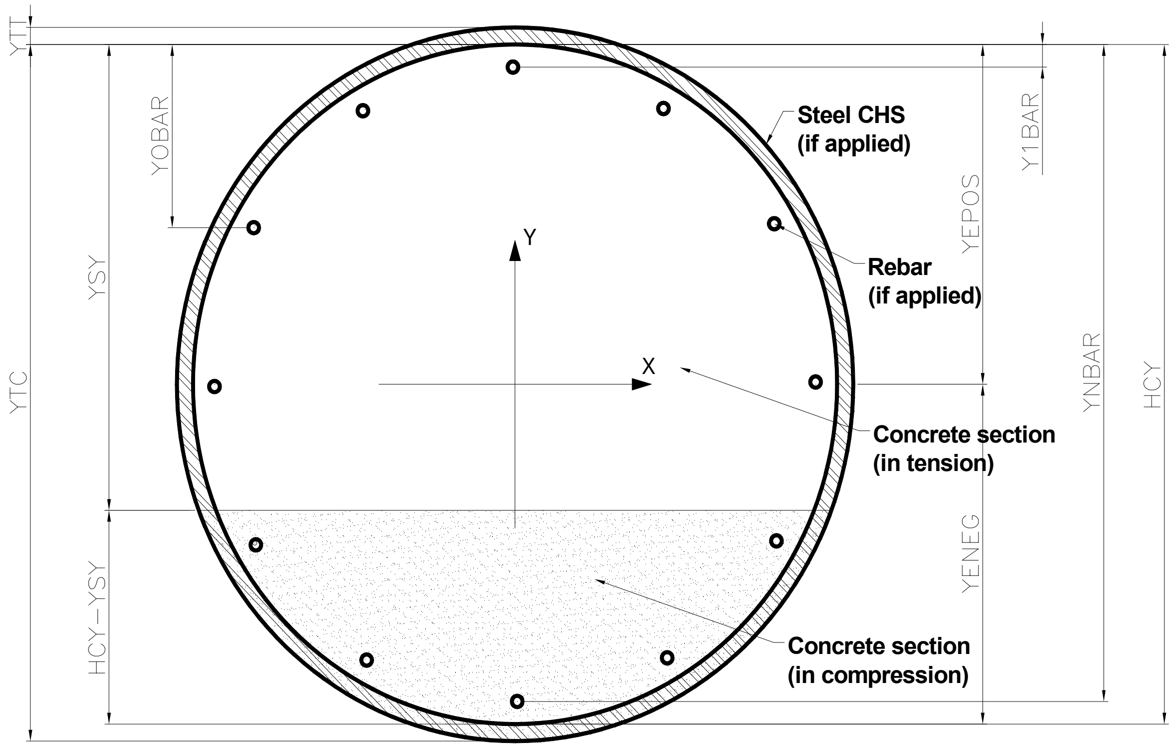
Offsets of rebar from extreme fiber $y_{Obar} := y_{Epos} - y_{bar}$

Determine most extreme rebar (minimum offset) $y_{1bar} := \min(y_{Epos} - y_{bar})$ $y_{1bar} = 75\,mm$

Determine most extreme rebar (maximum offset) $y_{nbar} := \max(y_{Epos} - y_{bar})$ $y_{nbar} = 1325\,mm$

Offsets of extreme steel tube fiber from extreme concrete fiber $y_{tt} := ts$ $y_{tt} = 0\,mm$

$y_{tc} := H_{CY} + ts$ $y_{tc} = 1400\,mm$



ASSIGN NEUTRAL AXIS VALUES

Number of sections to analysed $ns := 500$

$q := 2 .. ns$

Distance of neutral axis from extreme fiber in tension $y_{SY_q} := H_{CY} \cdot \frac{q}{ns + 1}$

Calculate stresses and strains in reinforcement and concrete at extreme fibers

Calculate strain at extreme compression fiber assuming max allowable stress in concrete:

Trial value of concrete strain

$$\varepsilon cc := \frac{\sigma_{cc}}{E_C} \cdot 2 \qquad \frac{\sigma_{cc}}{E_C} = 0.001165$$

Given

$$\sigma_{cc} = \varepsilon cc \cdot \left(4700 \sqrt{\frac{f_c \cdot 2}{MPa}} - \frac{4700^2}{2.68} \cdot \varepsilon cc\right) \cdot MPa$$

$$\varepsilon_{cc} := Find(\varepsilon cc) \qquad \varepsilon_{cc} := 0.002$$

$$\varepsilon_{cc} := \begin{cases} \varepsilon_{tc} & \text{if } (f_c = 0) \cdot (A_{BAR} = 0) \\ \varepsilon_{rc} & \text{if } (f_c = 0) \cdot (ts = 0) \\ \varepsilon_{cc} & \text{otherwise} \end{cases} \qquad \varepsilon_{cc} = 0.002000$$

Strain at other stresses taken to be linear:

$$\varepsilon cc(f_c, \sigma_{cd}) := \begin{cases} \frac{\sigma_{cd}}{\sigma_{tc}} \cdot \varepsilon_{tc} & \text{if } (f_c = 0) \cdot (A_{BAR} = 0) \\ \frac{\sigma_{cd}}{\sigma_{rc}} \cdot \varepsilon_{rc} & \text{if } (f_c = 0) \cdot (ts = 0) \\ \frac{\sigma_{cd}}{\sigma_{cc}} \cdot \varepsilon_{cc} & \text{otherwise} \end{cases}$$

Calculate strain in steel tube assuming max allowable stress in concrete:

In compression $\varepsilon_{tcc_q} := \varepsilon_{cc} \cdot \frac{y_{tc} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

In tension $\varepsilon_{tct_q} := \varepsilon_{cc} \cdot \frac{-(y_{SY_q} + y_{tt})}{H_{CY} - y_{SY_q}}$

Calculate strain in rebar assuming max allowable stress in concrete:

In compression $\varepsilon_{rcc_q} := \varepsilon_{cc} \cdot \frac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

In tension $\varepsilon_{rct_q} := \varepsilon_{cc} \cdot \frac{y_{1bar} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

Calculate design max stress in compression taking account of other limits:

$$\sigma cd(\varepsilon_{tcc}, q) := \begin{array}{l} \sigma_{cd} \leftarrow \sigma_{cc} \text{ if } f_c > 0 \\ \sigma_{cd} \leftarrow \sigma_{tc} \text{ if } (f_c = 0)\cdot(A_{BAR} = 0) \\ \sigma_{cd} \leftarrow \sigma_{rc} \text{ if } (f_c = 0)\cdot(ts = 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \frac{\varepsilon_{tc}}{\varepsilon_{tcc}} \text{ if } (\varepsilon_{tcc} > \varepsilon_{tc})\cdot(ts > 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \frac{\varepsilon_{rc}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \frac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}}} \text{ if } \left(\varepsilon cc(f_c, \sigma_{cd}) \cdot \frac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}} > \varepsilon_{rc}\right)\cdot(A_{BAR} > 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \frac{\varepsilon_{ts}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \frac{-(y_{SY_q} + y_{tt})}{H_{CY} - y_{SY_q}}} \text{ if } \left[\varepsilon cc(f_c, \sigma_{cd}) \cdot \frac{-(y_{SY_q} + y_{tt})}{H_{CY} - y_{SY_q}} < \varepsilon_{ts}\right]\cdot(ts > 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \frac{\varepsilon_{rs}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \frac{-(y_{SY_q} - y_{1bar})}{H_{CY} - y_{SY_q}}} \text{ if } \left[\varepsilon cc(f_c, \sigma_{cd}) \cdot \frac{-(y_{SY_q} - y_{1bar})}{H_{CY} - y_{SY_q}} < \varepsilon_{rs}\right]\cdot(A_{BAR} > 0) \\ \sigma_{cc} \text{ otherwise} \end{array}$$

$$\sigma_{cd_q} := \sigma cd(\varepsilon_{tcc_q}, q)$$

CALCULATE FORCES AND MOMENTS AT EACH NEUTRAL AXIS LOCATION

Calculate force in concrete:

$$F_{C_q} := \begin{cases} \displaystyle\int_{-\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot(1-\rho)\cdot\left[\frac{\sigma_{cd_q}\cdot\left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q}}\right] dy & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from concrete about column centroid:

$$M_{C_q} := \begin{cases} \displaystyle\int_{-\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot(1-\rho)\cdot\left[\frac{\sigma_{cd_q}\cdot\left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q}}\right]\cdot y\, dy & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate strain in rebar assuming design max stress in concrete:

$$y_{SY_q} := \begin{cases} y_{nbar}\cdot\frac{q}{ns+1} & \text{if } (f_c = 0)\cdot(A_{BAR} > 0) \\ y_{SY_q} & \text{otherwise} \end{cases}$$

$$\varepsilon_{S_{j,q}} := \begin{cases} -\frac{y_{SY_q} - y_{Obar_j}}{y_{nbar} - y_{SY_q}}\cdot\varepsilon cc\left(f_c, \sigma_{cd_q}\right) & \text{if } f_c = 0 \\ -\frac{y_{SY_q} - y_{Obar_j}}{H_{CY} - y_{SY_q}}\cdot\varepsilon cc\left(f_c, \sigma_{cd_q}\right) & \text{otherwise} \end{cases}$$

Calculate force in each rebar:

$$F_{S_{j,q}} := \begin{cases} \varepsilon_{S_{j,q}} E_S\cdot A_{bar_j} & \text{if } A_{BAR} > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate total force in reinforcement:

$$F_{R_q} := \sum_j F_{S_{j,q}}$$

Calculate moment from reinforcement about section centroid:

$$M_{R_q} := \begin{cases} \sum_j -\left(\varepsilon_{S_{j,q}} E_S \cdot A_{bar_j} \cdot y_{bar_j}\right) & \text{if } A_{BAR} > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate strain in steel tube at extreme tension fiber:

$$\varepsilon_{tds_q} := \frac{-\left(y_{SY_q} + y_{tt}\right)}{H_{CY} - y_{SY_q}} \varepsilon cc\left(f_c, \sigma_{cd_q}\right)$$

Calculate strain in steel tube at extreme compression fiber:

$$\varepsilon_{tdc_q} := \frac{y_{tc} - y_{SY_q}}{H_{CY} - y_{SY_q}} \varepsilon cc\left(f_c, \sigma_{cd_q}\right)$$

Calculate tensile force in steel tube:

$$F_{TS1_q} := \int_{\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2} + y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tds_q} \cdot E_S \cdot \left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TS2_q} := \int_{\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tds_q} \cdot E_S \cdot \left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TS} := \begin{cases} F_{TS1} - F_{TS2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate compressive force in steel tube:

$$F_{TC1_q} := \int_{-\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2} + y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tdc_q} \cdot E_S \cdot \left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TC2_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tdc_q}\cdot E_S\cdot\left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TC} := \begin{cases} F_{TC1} - F_{TC2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from tensile force in steel tube:

$$M_{TS1_q} := \int_{\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}+y_{tt}} -2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tds_q}\cdot E_S\cdot\left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right]\cdot y\, dy$$

$$M_{TS2_q} := \int_{\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} -2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tds_q}\cdot E_S\cdot\left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right]\cdot y\, dy$$

$$M_{TS} := \begin{cases} M_{TS1} - M_{TS2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from compressive force in steel tube:

$$M_{TC1_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}+y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tdc_q}\cdot E_S\cdot\left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right]\cdot y\, dy$$

$$M_{TC2_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot\left[\frac{\varepsilon_{tdc_q}\cdot E_S\cdot\left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right]\cdot y\, dy$$

$$M_{TC} := \begin{cases} M_{TC1} - M_{TC2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calate total axial response from section:

$F_T := F_C + F_R + F_{TS} + F_{TC}$ $\qquad$ $F_{TC} := F_C$

Calculate total moment response from section:

$M_T := M_C + M_R + M_{TS} + M_{TC}$ $\qquad$ $M_{TC} := M_C$

CALCULATE MAXIMUM ALLOWABLE AXIAL FORCE IN SECTION

Limiting strain in axial compression:

$$\varepsilon_{cL} := \begin{cases} \min(\varepsilon_{cc}, \varepsilon_{tc}) & \text{if } (A_{BAR} = 0)\cdot(ts \neq 0)\cdot(f_c \neq 0) \\ \min(\varepsilon_{cc}, \varepsilon_{rc}) & \text{if } (ts = 0)\cdot(A_{BAR} \neq 0)\cdot(f_c \neq 0) \\ \varepsilon_{tc} & \text{if } (A_{BAR} = 0)\cdot(f_c = 0) \\ \varepsilon_{rc} & \text{if } (ts = 0)\cdot(f_c = 0) \\ \min(\varepsilon_{cc}, \varepsilon_{rc}, \varepsilon_{tc}) & \text{otherwise} \end{cases}$$

$\varepsilon_{cL} = 0.001950$

Limiting concrete stress in axial compression

$$\sigma_{cL} := \begin{cases} \sigma_{cd_2} & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases}$$

$\sigma_{cL} = 30\,\text{MPa}$

$$P_{MAX} := \sigma_{cL}\cdot A_C(1-\rho) + \varepsilon_{cL}\cdot E_S(A_{BAR} + A_{ST})$$

$P_{MAX} = 84841.1\,\text{kN}$ $\quad F_{T_1} := P_{MAX}$ $\quad M_{T_1} := 0\cdot\text{kN}\cdot\text{m}$

$$P_{MAXC} := \sigma_{cL}\cdot A_C\cdot(1-\rho)$$

$P_{MAXC} = 42828.2\,\text{kN}$ $\quad F_{TC_1} := P_{MAXC}$ $\quad M_{TC_1} := 0\cdot\text{kN}\cdot\text{m}$

CALCULATE MINIMUM ALLOWABLE AXIAL FORCE IN SECTION

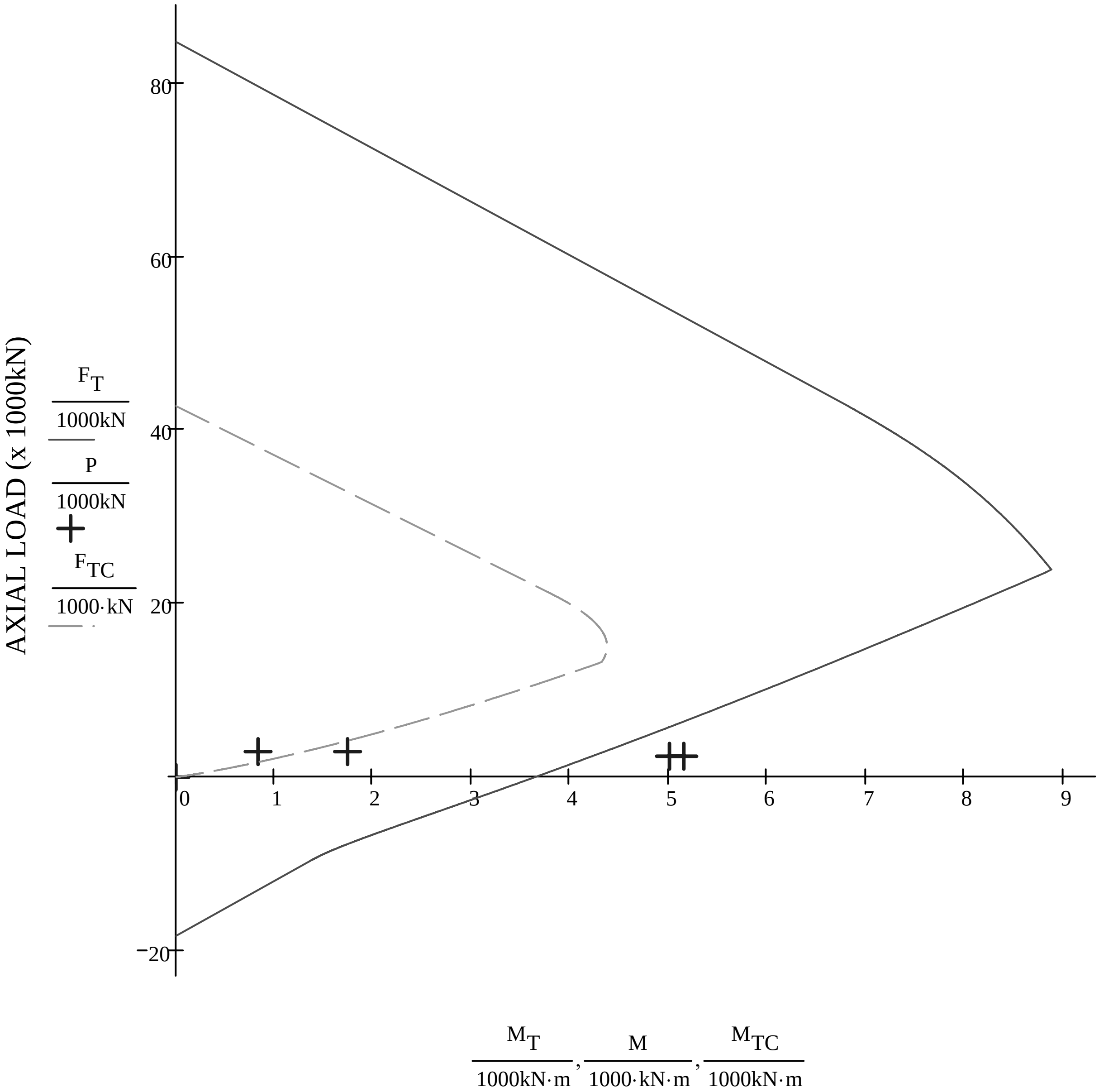
$$P_{MIN} := \begin{cases} \varepsilon_{rs}\cdot E_S(A_{BAR}) & \text{if } ts = 0 \\ \varepsilon_{ts}\cdot E_S(A_{ST}) & \text{if } A_{BAR} = 0 \\ \max(\varepsilon_{ts}, \varepsilon_{rs})\cdot E_S(A_{BAR} + A_{ST}) & \text{otherwise} \end{cases}$$

$P_{MIN} = -18313.3\,\text{kN}$ $\quad F_{T_{ns+1}} := P_{MIN}$ $\quad M_{T_{ns+1}} := 0\cdot\text{kN}\cdot\text{m}$

$$\text{Limit} := \begin{cases} \min(P, F_T)\cdot 0.75 & \text{if } \min(P) > 0 \\ \min(P, F_T)\cdot 1.25 & \text{otherwise} \end{cases}$$

$P_{MINC} := 0\text{kN}$ $\quad M_{TC_{ns+1}} := 0\cdot\text{kN}\cdot\text{m}$

Diameter of Column	$D = 1400\,mm$	Characteristic strength of concrete	$f_c = 30\,MPa$
Percentage reinforcement	$\rho = 7.02\,\%$	Yield Strength of Rebar	$f_y = 390\,MPa$
Thickness of CHS	$ts = 0\,mm$	Yield Strength of CHS	$f_{ys} = 250\,MPa$



INTERACTION CURVE AT SERVICEABILITY LIMIT STATE

Equation of interaction line - upper region (between 1 and 2 calculation points)

$$m1 := \frac{M_{T_2} - M_{T_1}}{F_{T_2} - F_{T_1}} \qquad c1 := F_{T_1}$$

Equation of interaction line - lower region (between ns and ns+1 calculation points)

$$m2 := \frac{M_{T_{ns}} - M_{T_{ns+1}}}{F_{T_{ns}} - F_{T_{ns+1}}} \qquad c2 := F_{T_{ns+1}}$$

$r := 1 .. 8$

$$M_{SLS_r} := \begin{cases} 0.000000000000000001 \cdot kN \cdot m & \text{if } P_r > F_{T_1} \\ 0.000000000000000001 \cdot kN \cdot m & \text{if } P_r < F_{T_{ns+1}} \\ (P_r - c1) \cdot m1 & \text{if } (P_r > F_{T_2}) \cdot (P_r \le F_{T_1}) \\ (P_r - c2) \cdot m2 & \text{if } (P_r \ge F_{T_{ns+1}}) \cdot (P_r < F_{T_{ns}}) \\ \text{otherwise} & \left| \begin{array}{l} j \leftarrow 1 \\ \text{while } F_{T_j} > P_r \\ \quad j \leftarrow j + 1 \\ M_{T_j} \end{array} \right. \end{cases}$$

$$StressFactor_r := \begin{cases} \text{"No Result"} & \text{if } M_{SLS_r} < 0.00000000000000001 \cdot kN \cdot m \\ \dfrac{M_r}{M_{SLS_r}} & \text{otherwise} \end{cases}$$

$$P = \begin{pmatrix} 2416 \\ 2412 \\ 2952 \\ 2952 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kN \qquad M = \begin{pmatrix} 5011 \\ 5157 \\ 836 \\ 1744 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kN \cdot m \qquad M_{SLS} = \begin{pmatrix} 4197.7 \\ 4197.7 \\ 4345.0 \\ 4345.0 \\ 3643.8 \\ 3643.8 \\ 3643.8 \\ 3643.8 \end{pmatrix} kN \cdot m \qquad StressFactor = \begin{pmatrix} 1.194 \\ 1.229 \\ 0.192 \\ 0.401 \\ 0.000 \\ 0.000 \\ 0.000 \\ 0.000 \end{pmatrix}$$

RESULTS SUMMARY
SERVICEABILITY LIMIT STATE ANALYSIS OF CIRCULAR BEAM COLUMN

Diameter of Column			1400	mm			
Percentage of rebar			7.02	%			
Load Case Ref	**Pier**	**Applied Service Axial Load kN**	**Applied Service Bending Moment kNm**	**Service Limit State Bending Moment kNm**	**Allowable Stress Factor**	**Applied Stress Factor**	**Serviceability Limit State Design Result**
1	A1	2416	5011	4197.7	140%	119%	OK
2	A2	2412	5157	4197.7	140%	123%	OK

Serviceability Check - Traffic Load Only

KATAHIRA & ENGINEERS INTERNATIONAL

Project: Detailed Design Study of North Java Corridor Flyover Project

Calculation: Balaraja Flyover
Serviceability Check - Traffic Load Only
1400mm Dia Circular RC Column - Top Section

Reference: Project Specific Design Criteria

Section Data

$MPa := 1000000 \cdot Pa$ $kN := 1000 \cdot N$

Input Item			
Concrete Compressive Strength	fc	30	MPa
Structural Steel Yield Strength	fys	250	MPa
Rebar Yield Strength	fy	390	MPa
Diameter of reinforced concrete section	D	1400	mm
Thickness of CHS section	t	0	mm
Diameter of rebar - layer 1	dia1	32	mm
Diameter of rebar - layer 2	dia2	32	mm
Number bars - layer 1 (max 100)	n1	36	
Number bars - layer 2 (max 100)	n2	26	
Cover from face of section - layer 1	cov1	60	mm
Cover from face of section - layer 2	cov2	115	mm

Load Data

Ref	Pier	Load Case	P	M	Stress
			kN	kNm	Allowance
1	A1	Combination 1 - P + Traffic Load Only	2368.0	4671.0	100%
2	A2	Combination 1 - P + Traffic Load Only	2350.0	4742.0	100%

$f_c := f_c \cdot MPa$ $f_{ys} := f_{ys} \cdot MPa$ $f_y := f_y \cdot MPa$ $D := D \cdot mm$ $ts := ts \cdot mm$

$dia1 := dia1 \cdot mm$ $dia2 := dia2 \cdot mm$ $cov1 := cov1 \cdot mm$ $cov2 := cov2 \cdot mm$

$P := P \cdot kN$ $M := M \cdot kN \cdot m$

$E_S := 200000 \cdot MPa$ $E_C := 4700\sqrt{\frac{f_c}{MPa}} \cdot MPa$ Modular ratio $\alpha := \begin{cases} \frac{E_S}{E_C} & \text{if } E_C > 0 \\ 1 & \text{otherwise} \end{cases}$ $\alpha = 7.77$

$E_C = 25743\, MPa$

Calculate Basic Allowable Stresses

Calculate rupture stress: $\sigma_{ct} := 0.5 \cdot \left(\frac{f_c}{MPa}\right)^{\frac{2}{3}} \cdot MPa$ $\sigma_{ct} = 4.8\, MPa$

Calculate basic allowable stress of concrete $\sigma_{cc} := 1.0 \cdot f_c$ $\sigma_{cc} = 30.0\, MPa$

Calculate basic allowable tensile stress of rebar $\sigma_{rs} := \begin{cases} 0.5 \cdot f_y & \text{if } 0.5 \cdot f_y \le 170MPa \\ 170MPa & \text{otherwise} \end{cases}$ $\sigma_{rs} = 170\, MPa$

Calculate basic allowable compressive stress of rebar $\sigma_{rc} := \begin{cases} 0.5 \cdot f_y & \text{if } 0.5 \cdot f_y \le 110MPa \\ f_y & \text{otherwise} \end{cases}$ $\sigma_{rc} = 390\, MPa$

Calculate basic allowable stress of structural steel $\sigma_{ts} := -0.6 f_{ys}$ $\sigma_{ts} = -150\, MPa$

$\sigma_{tc} := 1 f_{ys}$ $\sigma_{tc} = 250\, MPa$

Limiting strain of rebar $\varepsilon_{rs} := -\frac{\sigma_{rs}}{E_S}$ $\varepsilon_{rs} = -0.000850$

$\varepsilon_{rc} := \frac{\sigma_{rc}}{E_S}$ $\varepsilon_{rc} = 0.001950$

Limiting strain of structural steel $\varepsilon_{ts} := \frac{\sigma_{ts}}{E_S}$ $\varepsilon_{ts} = -0.000750$

$\varepsilon_{tc} := \frac{\sigma_{tc}}{E_S}$ $\varepsilon_{tc} = 0.001250$

Concrete Cross Section Data - generated

$n := 50$ Number of Points - 50 points maximum

$i := 1 .. n + 1$ Range from 1 to n+1

Ref.	X	Y	Ref.	X	Y
	mm	mm		mm	mm
1	0	-700	**26**	0	700
2	-88	-694	**27**	88	694
3	-174	-678	**28**	174	678
4	-258	-651	**29**	258	651
5	-337	-613	**30**	337	613
6	-411	-566	**31**	411	566
7	-479	-510	**32**	479	510
8	-539	-446	**33**	539	446
9	-591	-375	**34**	591	375
10	-633	-298	**35**	633	298
11	-666	-216	**36**	666	216
12	-688	-131	**37**	688	131
13	-699	-44	**38**	699	44
14	-699	44	**39**	699	-44
15	-688	131	**40**	688	-131
16	-666	216	**41**	666	-216
17	-633	298	**42**	633	-298
18	-591	375	**43**	591	-375
19	-539	446	**44**	539	-446
20	-479	510	**45**	479	-510
21	-411	566	**46**	411	-566
22	-337	613	**47**	337	-613
23	-258	651	**48**	258	-651
24	-174	678	**49**	174	-678
25	-88	694	**50**	88	-694

$k := 1 .. 25$ $XS1 := XS1 \cdot mm$ $XS2 := XS2 \cdot mm$ $YS1 := YS1 \cdot mm$ $YS2 := YS2 \cdot mm$

$x_k := XS1_k$ $y_k := YS1_k$ $x_{k+25} := XS2_k$ $y_{k+25} := YS2_k$ $x_{n+1} := XS1_1$ $y_{n+1} := YS1_1$

Calculate Section Properties of Concrete Section

$$A_C := -\sum_{i=1}^{n} \left[\left(y_{i+1} - y_i\right) \cdot \frac{x_{i+1} + x_i}{2} \right]$$ $A_C = 1.53533\ m^2$

$$x_C := -\frac{1}{A_C} \cdot \sum_{i=1}^{n} \left[\frac{y_{i+1} - y_i}{8} \cdot \left[\left(x_{i+1} + x_i\right)^2 + \frac{\left(x_{i+1} - x_i\right)^2}{3} \right] \right]$$ $x_C = 0\ m$

$$y_C := \frac{1}{A_C} \cdot \sum_{i=1}^{n} \left[\frac{x_{i+1} - x_i}{8} \cdot \left[\left(y_{i+1} + y_i\right)^2 + \frac{\left(y_{i+1} - y_i\right)^2}{3} \right] \right]$$ $y_C = 0\ m$

$$I_x := \sum_{i=1}^{n} \left[\left[\left(x_{i+1} - x_i\right) \cdot \frac{y_{i+1} + y_i}{24} \right] \cdot \left[\left(y_{i+1} + y_i\right)^2 + \left(y_{i+1} - y_i\right)^2 \right] \right]$$ $I_x = 0.18758\ m^4$

$$I_y := -\sum_{i=1}^{n} \left[\left[\left(y_{i+1} - y_i\right) \cdot \frac{x_{i+1} + x_i}{24} \right] \cdot \left[\left(x_{i+1} + x_i\right)^2 + \left(x_{i+1} - x_i\right)^2 \right] \right]$$ $I_y = 0.18758\ m^4$

$$I_{xC} := I_x - A_C \cdot x_C^{\,2}$$ $I_{xC} = 0.18758\ m^4$

$$I_{yC} := I_y - A_C \cdot y_C^{\,2}$$ $I_{yC} = 0.18758\ m^4$

Steel Tube Cross Section Data - generated from input

$ns := 50$ Number of Points - 50 points maximum

$ps := 1..ns + 1$ Range from 1 to ns+1

Ref.	X	Y	Ref.	X	Y
	mm	mm		mm	mm
1	0	-700	**26**	0	-700
2	-181	-676	**27**	181	-676
3	-350	-606	**28**	350	-606
4	-495	-495	**29**	495	-495
5	-606	-350	**30**	606	-350
6	-676	-181	**31**	676	-181
7	-700	0	**32**	700	0
8	-676	181	**33**	676	181
9	-606	350	**34**	606	350
10	-495	495	**35**	495	495
11	-350	606	**36**	350	606
12	-181	676	**37**	181	676
13	0	700	**38**	0	700
14	181	676	**39**	-181	676
15	350	606	**40**	-350	606
16	495	495	**41**	-495	495
17	606	350	**42**	-606	350
18	676	181	**43**	-676	181
19	700	0	**44**	-700	0
20	676	-181	**45**	-676	-181
21	606	-350	**46**	-606	-350
22	495	-495	**47**	-495	-495
23	350	-606	**48**	-350	-606
24	181	-676	**49**	-181	-676
25	0	-700	**50**	0	-700

$XSS1 := XSS1 \cdot mm$ $XSS2 := XSS2 \cdot mm$ $YSS1 := YSS1 \cdot mm$ $YSS2 := YSS2 \cdot mm$

$z := 1..25$ $xs_z := XSS1_z$ $ys_z := YSS1_z$

$z := 26..50$ $xs_z := XSS2_{z-25}$ $ys_z := YSS2_{z-25}$

$xs_{ns+1} := XSS1_1$ $ys_{ns+1} := YSS1_1$

Calculate Section Properties of Steel Tube Section

$$A_{ST} := -\sum_{ps=1}^{ns}\left[\left(ys_{ps+1} - ys_{ps}\right)\cdot\frac{xs_{ps+1} + xs_{ps}}{2}\right]$$

$A_{ST} = 0\ m^2$

$$x_{ST} := -\frac{1}{A_{ST}}\cdot\sum_{ps=1}^{ns}\left[\frac{ys_{ps+1} - ys_{ps}}{8}\cdot\left[\left(xs_{ps+1} + xs_{ps}\right)^2 + \frac{\left(xs_{ps+1} - xs_{ps}\right)^2}{3}\right]\right]$$

$x_{ST} = -1.0\ m$

$$y_{ST} := \frac{1}{A_{ST}}\cdot\sum_{ps=1}^{ns}\left[\frac{xs_{ps+1} - xs_{ps}}{8}\cdot\left[\left(ys_{ps+1} + ys_{ps}\right)^2 + \frac{\left(ys_{ps+1} - ys_{ps}\right)^2}{3}\right]\right]$$

$y_{ST} = -0.499\ m$

$$I_{xS} := \sum_{ps=1}^{ns}\left[\left[\left(xs_{ps+1} - xs_{ps}\right)\cdot\frac{ys_{ps+1} + ys_{ps}}{24}\right]\cdot\left[\left(ys_{ps+1} + ys_{ps}\right)^2 + \left(ys_{ps+1} - ys_{ps}\right)^2\right]\right]$$

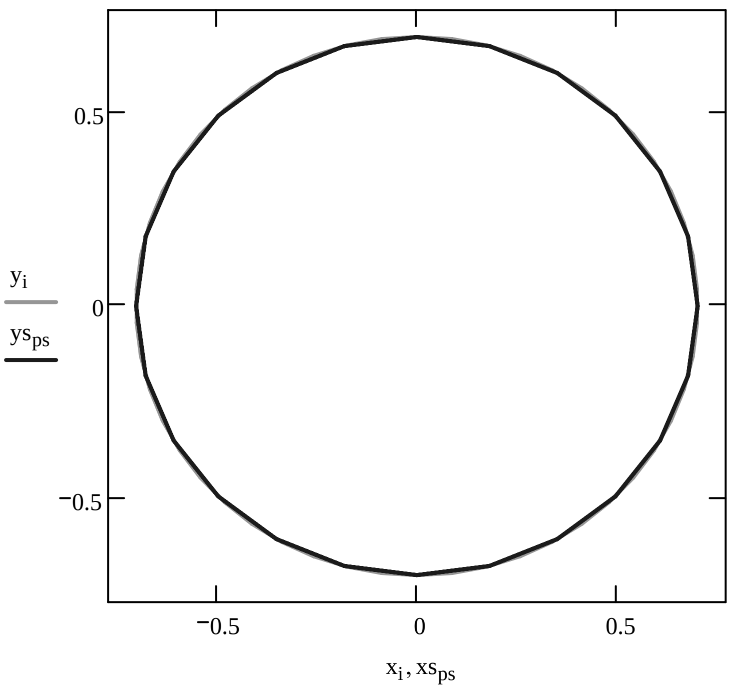
$I_{xS} = 0\ m^4$

$$I_{yS} := -\sum_{ps=1}^{ns}\left[\left[\left(ys_{ps+1} - ys_{ps}\right)\cdot\frac{xs_{ps+1} + xs_{ps}}{24}\right]\cdot\left[\left(xs_{ps+1} + xs_{ps}\right)^2 + \left(xs_{ps+1} - xs_{ps}\right)^2\right]\right]$$

$I_{yS} = 0\ m^4$

$I_{xS} := I_{xS} - A_{ST}\cdot x_{ST}^2$ $\qquad I_{xS} = 0\ m^4$

$I_{yS} := I_{yS} - A_{ST}\cdot y_{ST}^2$ $\qquad I_{yS} = 0.00000\ m^4$



Rebar Data Layer 1 - generated from input

Ref	Area mm2	X mm	Y mm	Ref	Area mm2	X mm	Y mm
1	804	0	625	51	0	0	0
2	804	0	-625	52	0	0	0
3	804	625	0	53	0	0	0
4	804	-625	0	54	0	0	0
5	804	48	623	55	0	0	0
6	804	48	-623	56	0	0	0
7	804	-48	623	57	0	0	0
8	804	-48	-623	58	0	0	0
9	804	232	580	59	0	0	0
10	804	232	-580	60	0	0	0
11	804	-232	580	61	0	0	0
12	804	-232	-580	62	0	0	0
13	804	282	558	63	0	0	0
14	804	282	-558	64	0	0	0
15	804	-282	558	65	0	0	0
16	804	-282	-558	66	0	0	0
17	804	330	530	67	0	0	0
18	804	330	-530	68	0	0	0
19	804	-330	530	69	0	0	0
20	804	-330	-530	70	0	0	0
21	804	447	403	71	0	0	0
22	804	447	-403	72	0	0	0
23	804	-447	403	73	0	0	0
24	804	-447	-403	74	0	0	0
25	804	540	313	75	0	0	0
26	804	540	-313	76	0	0	0
27	804	-540	313	77	0	0	0
28	804	-540	-313	78	0	0	0
29	804	587	215	79	0	0	0
30	804	587	-215	80	0	0	0
31	804	-587	215	81	0	0	0
32	804	-587	-215	82	0	0	0
33	804	615	110	83	0	0	0
34	804	615	-110	84	0	0	0
35	804	-615	110	85	0	0	0
36	804	-615	-110	86	0	0	0
37	0	0	0	87	0	0	0
38	0	0	0	88	0	0	0
39	0	0	0	89	0	0	0
40	0	0	0	90	0	0	0
41	0	0	0	91	0	0	0
42	0	0	0	92	0	0	0
43	0	0	0	93	0	0	0
44	0	0	0	94	0	0	0
45	0	0	0	95	0	0	0
46	0	0	0	96	0	0	0
47	0	0	0	97	0	0	0
48	0	0	0	98	0	0	0
49	0	0	0	99	0	0	0
50	0	0	0	100	0	0	0

Rebar Data Layer 2 - generated from input

Ref	Area mm2	X mm	Y mm	Ref	Area mm2	X mm	Y mm
1	804	0	574	51	0	0	0
2	804	0	-574	52	0	0	0
3	804	47	572	53	0	0	0
4	804	47	-572	54	0	0	0
5	804	-47	572	55	0	0	0
6	804	-47	-572	56	0	0	0
7	804	212	533	57	0	0	0
8	804	212	-533	58	0	0	0
9	804	-212	533	59	0	0	0
10	804	-212	-533	60	0	0	0
11	804	258	512	61	0	0	0
12	804	258	-512	62	0	0	0
13	804	-258	512	63	0	0	0
14	804	-258	-512	64	0	0	0
15	804	497	285	65	0	0	0
16	804	497	-285	66	0	0	0
17	804	-497	285	67	0	0	0
18	804	-497	-285	68	0	0	0
19	804	563	107	69	0	0	0
20	804	563	-107	70	0	0	0
21	804	-563	107	71	0	0	0
22	804	-563	-107	72	0	0	0
23	804	302	488	73	0	0	0
24	804	302	-488	74	0	0	0
25	804	-302	488	75	0	0	0
26	804	-302	-488	76	0	0	0
27	0	0	0	77	0	0	0
28	8040	0	88	78	8040	0	-88
29	8040	0	56	79	8040	0	-56
30	8040	0	24	80	8040	0	-24
31	0	0	0	81	0	0	0
32	0	0	0	82	0	0	0
33	283	0	120	83	283	0	-120
34	283	0	120	84	283	0	-120
35	283	0	120	85	283	0	-120
36	283	0	120	86	283	0	-120
37	283	0	120	87	283	0	-120
38	283	0	120	88	283	0	-120
39	283	0	120	89	283	0	-120
40	283	0	120	90	283	0	-120
41	283	0	120	91	283	0	-120
42	283	0	120	92	283	0	-120
43	283	0	120	93	283	0	-120
44	283	0	120	94	283	0	-120
45	283	0	120	95	283	0	-120
46	283	0	120	96	283	0	-120
47	283	0	120	97	283	0	-120
48	283	0	120	98	283	0	-120
49	283	0	120	99	283	0	-120
50	0	0	0	100	0	0	0

$A1 := A1 \cdot mm^2$ $\quad A2 := A2 \cdot mm^2$ $\quad A3 := A3 \cdot mm^2$ $\quad A4 := A4 \cdot mm^2$

$X1 := X1 \cdot mm$ $\quad X2 := X2 \cdot mm$ $\quad X3 := X3 \cdot mm$ $\quad X4 := X4 \cdot mm$

$Y1 := Y1 \cdot mm$ $\quad Y2 := Y2 \cdot mm$ $\quad Y3 := Y3 \cdot mm$ $\quad Y4 := Y4 \cdot mm$

$k := 1 .. 50$ $\quad A_{bar_k} := A1_k$ $\quad x_{bar_k} := X1_k$ $\quad y_{bar_k} := Y1_k$

$A_{bar_{k+50}} := A2_k$ $\quad x_{bar_{k+50}} := X2_k$ $\quad y_{bar_{k+50}} := Y2_k$

$A_{bar_{k+100}} := A3_k$ $\quad x_{bar_{k+100}} := X3_k$ $\quad y_{bar_{k+100}} := Y3_k$

$A_{bar_{k+150}} := A4_k$ $\quad x_{bar_{k+150}} := X4_k$ $\quad y_{bar_{k+150}} := Y4_k$

Calculate Section Properties of Reinforcement

$$A_{BAR} := \sum_{j=1}^{200} A_{bar_j} \qquad A_{BAR} = 107725\,mm^2$$

$$\rho := \frac{A_{BAR}}{A_C} \qquad \rho = 0.0702$$

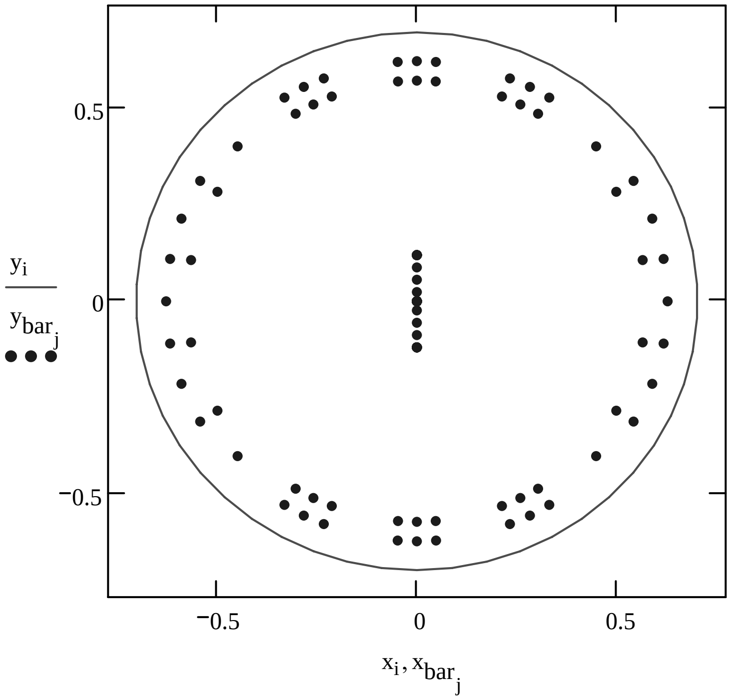
$$x_b := \begin{cases} \left[\sum_{j=1}^{200} \left(A_{bar_j} \cdot x_{bar_j}\right)\right] \cdot \frac{1}{A_{BAR}} & \text{if } A_{BAR} > 0 \\ 0m & \text{otherwise} \end{cases} \qquad x_b = -4 \times 10^{-6}\ m$$

$$y_b := \begin{cases} \left[\sum_{j=1}^{200} \left(A_{bar_j} \cdot y_{bar_j}\right)\right] \cdot \frac{1}{A_{BAR}} & \text{if } A_{BAR} > 0 \\ 0m & \text{otherwise} \end{cases} \qquad y_b = 0\ m$$

$$I_{xb} := \sum_{j=1}^{200} \left[A_{bar_j} \cdot \left(x_{bar_j}\right)^2\right] + A_{BAR} \cdot x_b^2 \qquad I_{xb} = 0.00779\,m^4$$

$$I_{yb} := \sum_{j=1}^{200} \left[A_{bar_j} \cdot \left(y_{bar_j}\right)^2\right] + A_{BAR} \cdot y_b^2 \qquad I_{yb} = 0.01062\,m^4$$

$j := 1..200$



Calculate Composite Section Properties (before cracking)

Effective area $A_E := A_C \cdot [1 + \rho \cdot (\alpha - 1)] + A_{ST} \cdot \alpha$ $\quad A_E = 2264537\ mm^2$

Effective centroid $x_E := \dfrac{A_C \cdot [(1-\rho) \cdot x_C + \rho \cdot x_b \cdot \alpha] + A_{ST} \cdot \alpha \cdot x_{ST}}{A_E}$ $\quad x_E = -0.000\ m$

$y_E := \dfrac{A_C \cdot [(1-\rho) \cdot y_C + \rho \cdot y_b \cdot \alpha] + A_{ST} \cdot \alpha \cdot x_{ST}}{A_E}$ $\quad y_E = 0.000\ m$

Effective stiffness $I_{EX} := I_{xC} + I_{xb} \cdot (\alpha - 1) + A_C \cdot \left[(1-\rho) \cdot x_C^2 + \rho \cdot x_b^2 \cdot \alpha\right] + \left(I_{xS} + A_{ST} \cdot x_{ST}^2\right) \cdot \alpha$

$I_{EX} = 2 \times 10^7\ cm^4$

$I_{EY} := I_{yC} + I_{yb} \cdot (\alpha - 1) + A_C \cdot \left[(1-\rho) \cdot y_C^2 + \rho \cdot y_b^2 \cdot \alpha\right] + \left(I_{yS} + A_{ST} \cdot y_{ST}^2\right) \cdot \alpha$

$I_{EY} = 3 \times 10^7\ cm^4$

Distance from extreme concrete fiber to centroid

$xF_{pos} := \max(x - x_E)$ $\quad xF_{neg} := \min(x - x_E)$

$yF_{pos} := \max(y - y_E)$ $\quad yF_{neg} := \min(y - y_E)$

Total depth of concrete section

$H_{CX} := xF_{pos} - xF_{neg}$ $\quad H_{CX} = 1\ m$

$H_{CY} := yF_{pos} - yF_{neg}$ $\quad H_{CY} = 1\ m$

Section modulus

$$Z_{Xpos} := \frac{I_{EX}}{xF_{pos}} \qquad Z_{Xneg} := \frac{I_{EX}}{xF_{neg}}$$

$$Z_{Ypos} := \frac{I_{EY}}{yF_{pos}} \qquad Z_{Yneg} := \frac{I_{EY}}{yF_{neg}}$$

Thickness of steel tube:

$ts := y_1 - ys_1 \qquad ts = 0\,mm$

Establish Section Dimensions

Positive case - determine coord of extreme concrete fiber $y_{Epos} := \max(y)$ $y_{Epos} = 700\,mm$

Negative case - determine coord of extreme concrete fiber $y_{Eneg} := \min(y)$ $y_{Eneg} = -700\,mm$

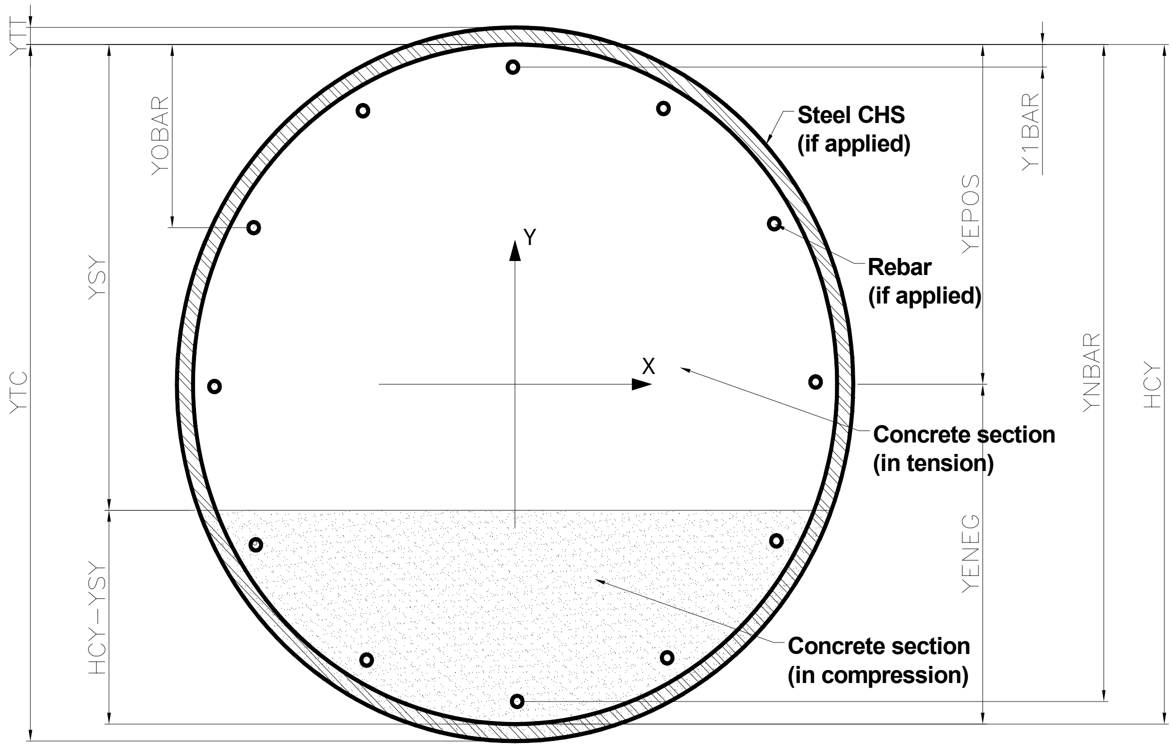
Offsets of rebar from extreme fiber $y_{Obar} := y_{Epos} - y_{bar}$

Determine most extreme rebar (minimum offset) $y_{1bar} := \min(y_{Epos} - y_{bar})$ $y_{1bar} = 75\,mm$

Determine most extreme rebar (maximum offset) $y_{nbar} := \max(y_{Epos} - y_{bar})$ $y_{nbar} = 1325\,mm$

Offsets of extreme steel tube fiber from extreme concrete fiber $y_{tt} := ts$ $y_{tt} = 0\,mm$

$y_{tc} := H_{CY} + ts$ $y_{tc} = 1400\,mm$



ASSIGN NEUTRAL AXIS VALUES

Number of sections to analysed $ns := 500$

$q := 2 .. ns$

Distance of neutral axis from extreme fiber in tension $y_{SY_q} := H_{CY} \cdot \frac{q}{ns + 1}$

Calculate stresses and strains in reinforcement and concrete at extreme fibers

Calculate strain at extreme compression fiber assuming max allowable stress in concrete:

Trial value of concrete strain

$\varepsilon cc := \frac{\sigma_{cc}}{E_C} \cdot 2$ $\frac{\sigma_{cc}}{E_C} = 0.001165$

Given

$$\sigma_{cc} = \varepsilon cc \cdot \left(4700 \sqrt{\frac{f_c \cdot 2}{MPa}} - \frac{4700^2}{2.68} \cdot \varepsilon cc \right) \cdot MPa$$

$\varepsilon_{cc} := Find(\varepsilon cc)$ $\varepsilon_{cc} := 0.002$

$$\varepsilon_{cc} := \begin{cases} \varepsilon_{tc} & \text{if } (f_c = 0) \cdot (A_{BAR} = 0) \\ \varepsilon_{rc} & \text{if } (f_c = 0) \cdot (ts = 0) \\ \varepsilon_{cc} & \text{otherwise} \end{cases}$$

$\varepsilon_{cc} = 0.002000$

Strain at other stresses taken to be linear:

$$\varepsilon cc(f_c, \sigma_{cd}) := \begin{cases} \frac{\sigma_{cd}}{\sigma_{tc}} \cdot \varepsilon_{tc} & \text{if } (f_c = 0) \cdot (A_{BAR} = 0) \\ \frac{\sigma_{cd}}{\sigma_{rc}} \cdot \varepsilon_{rc} & \text{if } (f_c = 0) \cdot (ts = 0) \\ \frac{\sigma_{cd}}{\sigma_{cc}} \cdot \varepsilon_{cc} & \text{otherwise} \end{cases}$$

Calculate strain in steel tube assuming max allowable stress in concrete:

In compression $\varepsilon_{tcc_q} := \varepsilon_{cc} \cdot \frac{y_{tc} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

In tension $\varepsilon_{tct_q} := \varepsilon_{cc} \cdot \frac{-\left(y_{SY_q} + y_{tt}\right)}{H_{CY} - y_{SY_q}}$

Calculate strain in rebar assuming max allowable stress in concrete:

In compression $\varepsilon_{rcc_q} := \varepsilon_{cc} \cdot \dfrac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

In tension $\varepsilon_{rct_q} := \varepsilon_{cc} \cdot \dfrac{y_{1bar} - y_{SY_q}}{H_{CY} - y_{SY_q}}$

Calculate design max stress in compression taking account of other limits:

$$\sigma cd(\varepsilon_{tcc}, q) := \begin{array}{|l} \sigma_{cd} \leftarrow \sigma_{cc} \text{ if } f_c > 0 \\ \sigma_{cd} \leftarrow \sigma_{tc} \text{ if } (f_c = 0)\cdot(A_{BAR} = 0) \\ \sigma_{cd} \leftarrow \sigma_{rc} \text{ if } (f_c = 0)\cdot(ts = 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{tc}}{\varepsilon_{tcc}} \text{ if } (\varepsilon_{tcc} > \varepsilon_{tc})\cdot(ts > 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{rc}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}}} \text{ if } \left(\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{y_{nbar} - y_{SY_q}}{H_{CY} - y_{SY_q}} > \varepsilon_{rc}\right)\cdot(A_{BAR} > 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{ts}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} + y_{tt})}{H_{CY} - y_{SY_q}}} \text{ if } \left[\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} + y_{tt})}{H_{CY} - y_{SY_q}} < \varepsilon_{ts}\right]\cdot(ts > 0) \\ \sigma_{cd} \leftarrow \sigma_{cd} \cdot \dfrac{\varepsilon_{rs}}{\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} - y_{1bar})}{H_{CY} - y_{SY_q}}} \text{ if } \left[\varepsilon cc(f_c, \sigma_{cd}) \cdot \dfrac{-(y_{SY_q} - y_{1bar})}{H_{CY} - y_{SY_q}} < \varepsilon_{rs}\right]\cdot(A_{BAR} > 0) \\ \sigma_{cc} \text{ otherwise} \end{array}$$

$$\sigma_{cd_q} := \sigma cd(\varepsilon_{tcc_q}, q)$$

CALCULATE FORCES AND MOMENTS AT EACH NEUTRAL AXIS LOCATION

Calculate force in concrete:

$$F_{C_q} := \begin{cases} \displaystyle\int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot(1-\rho)\cdot\left[\frac{\sigma_{cd_q}\cdot\left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q}}\right] dy & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from concrete about column centroid:

$$M_{C_q} := \begin{cases} \displaystyle\int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2}\cdot(1-\rho)\cdot\left[\frac{\sigma_{cd_q}\cdot\left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q}}\right]\cdot y\, dy & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate strain in rebar assuming design max stress in concrete:

$$y_{SY_q} := \begin{cases} y_{nbar}\cdot\frac{q}{ns+1} & \text{if } (f_c = 0)\cdot(A_{BAR} > 0) \\ y_{SY_q} & \text{otherwise} \end{cases}$$

$$\varepsilon_{S_{j,q}} := \begin{cases} -\frac{y_{SY_q} - y_{Obar_j}}{y_{nbar} - y_{SY_q}}\cdot\varepsilon cc\left(f_c, \sigma_{cd_q}\right) & \text{if } f_c = 0 \\ -\frac{y_{SY_q} - y_{Obar_j}}{H_{CY} - y_{SY_q}}\cdot\varepsilon cc\left(f_c, \sigma_{cd_q}\right) & \text{otherwise} \end{cases}$$

Calculate force in each rebar:

$$F_{S_{j,q}} := \begin{cases} \varepsilon_{S_{j,q}} E_S\cdot A_{bar_j} & \text{if } A_{BAR} > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate total force in reinforcement:

$$F_{R_q} := \sum_j F_{S_{j,q}}$$

Calculate moment from reinforcement about section centroid:

$$M_{R_q} := \begin{cases} \sum_j -\left(\varepsilon_{S_{j,q}} E_S \cdot A_{bar_j} \cdot y_{bar_j}\right) & \text{if } A_{BAR} > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate strain in steel tube at extreme tension fiber:

$$\varepsilon_{tds_q} := \frac{-\left(y_{SY_q} + y_{tt}\right)}{H_{CY} - y_{SY_q}} \varepsilon cc\left(f_c, \sigma_{cd_q}\right)$$

Calculate strain in steel tube at extreme compression fiber:

$$\varepsilon_{tdc_q} := \frac{y_{tc} - y_{SY_q}}{H_{CY} - y_{SY_q}} \varepsilon cc\left(f_c, \sigma_{cd_q}\right)$$

Calculate tensile force in steel tube:

$$F_{TS1_q} := \int_{\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2} + y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tds_q} \cdot E_S \cdot \left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TS2_q} := \int_{\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tds_q} \cdot E_S \cdot \left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TS} := \begin{cases} F_{TS1} - F_{TS2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate compressive force in steel tube:

$$F_{TC1_q} := \int_{-\left(\frac{H_{CY}}{2} - y_{SY_q}\right)}^{\frac{H_{CY}}{2} + y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tdc_q} \cdot E_S \cdot \left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TC2_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tdc_q} \cdot E_S \cdot \left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right] dy$$

$$F_{TC} := \begin{cases} F_{TC1} - F_{TC2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from tensile force in steel tube:

$$M_{TS1_q} := \int_{\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}+y_{tt}} -2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tds_q} \cdot E_S \cdot \left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right] \cdot y \, dy$$

$$M_{TS2_q} := \int_{\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} -2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tds_q} \cdot E_S \cdot \left[y - \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{y_{SY_q} + y_{tt}}\right] \cdot y \, dy$$

$$M_{TS} := \begin{cases} M_{TS1} - M_{TS2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calculate moment from compressive force in steel tube:

$$M_{TC1_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}+y_{tt}} 2\sqrt{\left(\frac{H_{CY}}{2} + y_{tt}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tdc_q} \cdot E_S \cdot \left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right] \cdot y \, dy$$

$$M_{TC2_q} := \int_{-\left(\frac{H_{CY}}{2}-y_{SY_q}\right)}^{\frac{H_{CY}}{2}} 2\sqrt{\left(\frac{H_{CY}}{2}\right)^2 - y^2} \cdot \left[\frac{\varepsilon_{tdc_q} \cdot E_S \cdot \left[y + \left(\frac{H_{CY}}{2} - y_{SY_q}\right)\right]}{H_{CY} - y_{SY_q} + y_{tt}}\right] \cdot y \, dy$$

$$M_{TC} := \begin{cases} M_{TC1} - M_{TC2} & \text{if } ts > 0 \\ 0 & \text{otherwise} \end{cases}$$

Calate total axial response from section:

$$F_T := F_C + F_R + F_{TS} + F_{TC}$$

$$F_{TC} := F_C$$

Calculate total moment response from section:

$$M_T := M_C + M_R + M_{TS} + M_{TC}$$

$$M_{TC} := M_C$$

CALCULATE MAXIMUM ALLOWABLE AXIAL FORCE IN SECTION

Limiting strain in axial compression:

$$\varepsilon_{cL} := \begin{cases} \min(\varepsilon_{cc}, \varepsilon_{tc}) & \text{if } (A_{BAR} = 0)\cdot(ts \neq 0)\cdot(f_c \neq 0) \\ \min(\varepsilon_{cc}, \varepsilon_{rc}) & \text{if } (ts = 0)\cdot(A_{BAR} \neq 0)\cdot(f_c \neq 0) \\ \varepsilon_{tc} & \text{if } (A_{BAR} = 0)\cdot(f_c = 0) \\ \varepsilon_{rc} & \text{if } (ts = 0)\cdot(f_c = 0) \\ \min(\varepsilon_{cc}, \varepsilon_{rc}, \varepsilon_{tc}) & \text{otherwise} \end{cases} \qquad \varepsilon_{cL} = 0.001950$$

Limiting concrete stress in axial compression

$$\sigma_{cL} := \begin{cases} \sigma_{cd_2} & \text{if } f_c > 0 \\ 0 & \text{otherwise} \end{cases} \qquad \sigma_{cL} = 30\,\text{MPa}$$

$$P_{MAX} := \sigma_{cL}\cdot A_C(1-\rho) + \varepsilon_{cL}\cdot E_S(A_{BAR} + A_{ST})$$

$P_{MAX} = 84841.1\,\text{kN}$ $\quad F_{T_1} := P_{MAX}$ $\quad M_{T_1} := 0\cdot\text{kN}\cdot\text{m}$

$$P_{MAXC} := \sigma_{cL}\cdot A_C\cdot(1-\rho)$$

$P_{MAXC} = 42828.2\,\text{kN}$ $\quad F_{TC_1} := P_{MAXC}$ $\quad M_{TC_1} := 0\cdot\text{kN}\cdot\text{m}$

CALCULATE MINIMUM ALLOWABLE AXIAL FORCE IN SECTION

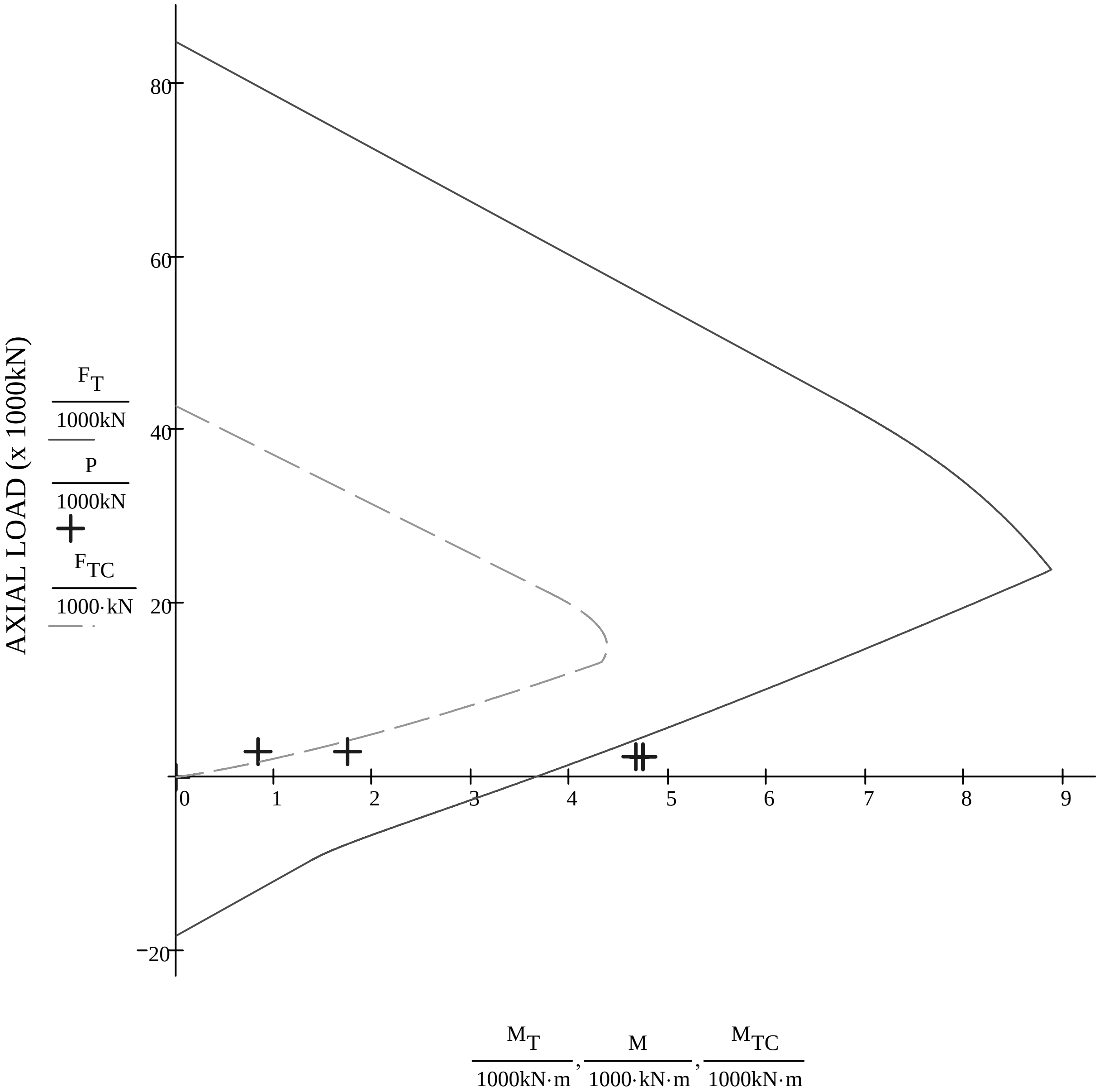
$$P_{MIN} := \begin{cases} \varepsilon_{rs}\cdot E_S(A_{BAR}) & \text{if } ts = 0 \\ \varepsilon_{ts}\cdot E_S(A_{ST}) & \text{if } A_{BAR} = 0 \\ \max(\varepsilon_{ts}, \varepsilon_{rs})\cdot E_S(A_{BAR} + A_{ST}) & \text{otherwise} \end{cases}$$

$P_{MIN} = -18313.3\,\text{kN}$ $\quad F_{T_{ns+1}} := P_{MIN}$ $\quad M_{T_{ns+1}} := 0\cdot\text{kN}\cdot\text{m}$

$$\text{Limit} := \begin{cases} \min(P, F_T)\cdot 0.75 & \text{if } \min(P) > 0 \\ \min(P, F_T)\cdot 1.25 & \text{otherwise} \end{cases}$$

$P_{MINC} := 0\text{kN}$ $\quad M_{TC_{ns+1}} := 0\cdot\text{kN}\cdot\text{m}$

Diameter of Column	$D = 1400\,mm$	Characteristic strength of concrete	$f_c = 30\,MPa$
Percentage reinforcement	$\rho = 7.02\,\%$	Yield Strength of Rebar	$f_y = 390\,MPa$
Thickness of CHS	$ts = 0\,mm$	Yield Strength of CHS	$f_{ys} = 250\,MPa$



INTERACTION CURVE AT SERVICEABILITY LIMIT STATE

Equation of interaction line - upper region (between 1 and 2 calculation points)

$$m1 := \frac{M_{T_2} - M_{T_1}}{F_{T_2} - F_{T_1}} \qquad c1 := F_{T_1}$$

Equation of interaction line - lower region (between ns and ns+1 calculation points)

$$m2 := \frac{M_{T_{ns}} - M_{T_{ns+1}}}{F_{T_{ns}} - F_{T_{ns+1}}} \qquad c2 := F_{T_{ns+1}}$$

$r := 1 .. 8$

$$M_{SLS_r} := \begin{cases} 0.000000000000000001 \cdot kN \cdot m & \text{if } P_r > F_{T_1} \\ 0.000000000000000001 \cdot kN \cdot m & \text{if } P_r < F_{T_{ns+1}} \\ (P_r - c1) \cdot m1 & \text{if } (P_r > F_{T_2}) \cdot (P_r \le F_{T_1}) \\ (P_r - c2) \cdot m2 & \text{if } (P_r \ge F_{T_{ns+1}}) \cdot (P_r < F_{T_{ns}}) \\ \text{otherwise} & \begin{array}{l} j \leftarrow 1 \\ \text{while } F_{T_j} > P_r \\ \quad j \leftarrow j + 1 \\ M_{T_j} \end{array} \end{cases}$$

$$StressFactor_r := \begin{cases} \text{"No Result"} & \text{if } M_{SLS_r} < 0.00000000000000001 \cdot kN \cdot m \\ \dfrac{M_r}{M_{SLS_r}} & \text{otherwise} \end{cases}$$

$$P = \begin{pmatrix} 2368 \\ 2350 \\ 2952 \\ 2952 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kN \quad M = \begin{pmatrix} 4671 \\ 4742 \\ 836 \\ 1744 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} kN \cdot m \quad M_{SLS} = \begin{pmatrix} 4197.7 \\ 4197.7 \\ 4345.0 \\ 4345.0 \\ 3643.8 \\ 3643.8 \\ 3643.8 \\ 3643.8 \end{pmatrix} kN \cdot m \quad StressFactor = \begin{pmatrix} 1.113 \\ 1.130 \\ 0.192 \\ 0.401 \\ 0.000 \\ 0.000 \\ 0.000 \\ 0.000 \end{pmatrix}$$

RESULTS SUMMARY
SERVICEABILITY LIMIT STATE ANALYSIS OF CIRCULAR BEAM COLUMN

	Diameter of Column		1400	mm			
	Percentage of rebar		7.02	%			
Load Case Ref	**Pier**	**Applied Service Axial Load kN**	**Applied Service Bending Moment kNm**	**Service Limit State Bending Moment kNm**	**Allowable Stress Factor**	**Applied Stress Factor**	**Serviceability Limit State Design Result**
1	A1	2368	4671	4197.7	100%	111%	Section Overstressed
2	A2	2350	4742	4197.7	100%	113%	Section Overstressed