8. DETAILED DESIGN OF COPING

Notes on General Design and Location Features

(1) Maintainability (AASHTO LRFD Section 2.5.2.3)

Areas around bearing seats and under deck joints shall be designed to facilitate jacking, repair and replacement of bearings and joints.

Jacking points shall be indicated on the plans and the structure shall be designed for jacking forces.

The design forces for jacking in service shall not be less than 1.3 times the permanent load reaction at the bearing adjacent to the point of jacking (AASHTO Article 3.4.3). The bridge shall be assumed to be closed to traffic during jacking operations.

(2) Criteria for Deflection (AASHTO LRFD Section 2.5.2.6.2)

The following deflection limits shall be considered:

- Vehicular load on cantilever arms……………………………………….Span/300

(3) Load Factors for Construction Loads (AASHTO LRFD Section 3.4.2)

Load factors for the weight of the structure and appurtenances shall not be taken as less than 1.25.

The load factor for construction loads and for dynamic effects shall not be less than 1.5.

Notes on Structural Analysis and Application of Design Vehicular Loads

(1) Equivalent width of Deck Overhang (AASHTO LRFD Table 4.6.2.1.3-1)

The width of the equivalent strip of deck overhangs (cantilever slabs) shall be as follows:

Width of primary strip (mm) = 1140 + 0.833X …………………………(Equation 1)

where:

X = distance from load to point of support

Given that the deck overhang on the coping is not a continuous slab, Equation 1 above shall be modified to account for edge loading in accordance with AASHTO LRFD Article 4.6.2.1.4 as follows:

Width of primary strip (mm) = 570 + 0.416X …………………………..(Equation 1A)

(2) Application of Load (AASHTO LRFD Article 3.6.1.3)

The design truck shall be positioned transversely such that the center of any wheel is not closer than:

- For the design of deck overhang – 300mm from the face of the curb or railing

Notes on Flexural Design

(1) Loads and Load Combinations

The loads and load combinations are taken from Section 6 and summarized in the following pages. Both ultimate limit state and serviceability limit state combinations were included in the design in accordance with the Design Criteria.

(2) Ultimate Moment Capacity (AASHTO LRFD Section 5.7)

Ultimate moment capacity of reinforced concrete beams is determined in accordance with AASHTO LRFD Article 5.7.3.2.3 as follows.

The nominal flexural resistance of a singly reinforced beam without compression reinforcement is given as:

$$M_n = A_S \cdot f_y \cdot \left(d_s - \frac{a}{2} \right) \text{..(Equation 1)}$$

where:

A_s = area of non prestressed tensile reinforcement

f_y = yield strength of reinforcing bars

d_s = distance from extreme compression fiber to the centroid of non prestressed tensile reinforcement

a = depth of equivalent stress block, $c \cdot \beta_1$ (Equation 2)

c = distance from extreme compression fiber to the neutral axis

β_1 = stress block factor

Assuming rectangular section behavior and yielding of reinforcement:

$$c = \frac{A_S \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} \text{..(Equation 3)}$$

where:

b = width of rectangular section

f_c = compressive strength of concrete

Substituting Equation 2 and 3 into Equation 1 gives:

$$M_n = A_S \cdot f_y \cdot \left(d_s - \frac{1}{2} \cdot \frac{A_S \cdot f_y}{0.85 \cdot f_c \cdot b} \right) \text{..(Equation 4)}$$

Rearranging Equation 4 and dividing all terms by ($d_s^2 \cdot b$) gives:

$$\left(\frac{A_S^{\,2} \cdot f_y^2}{2 \cdot 0.85 \cdot f_c \cdot d_s^2 b^2} \right) - \frac{A_S \cdot f_y}{d_s \cdot b} + \frac{M_n}{d_s^2 \cdot b} = 0.0 \text{.....................................(Equation 5)}$$

Defining terms:

$$\rho = \frac{A_S}{d_s \cdot b}$$

$$M = \frac{0.85 \cdot f_c}{f_y}$$

$$M_n = \frac{M_U}{\phi}$$

where:

M_U = applied ultimate moment from factored loads

ϕ = strength reduction factor for flexure

Substituting defined terms into Equation 5 and dividing through by f_y gives:

$$\left(\frac{\rho^2}{2 \cdot M}\right) - \rho + \frac{M_U}{\phi \cdot d_s^2 \cdot b \cdot f_y} = 0.0 \text{ ....(Equation 6)}$$

Defining terms:

$$R = \frac{M_U}{\phi \cdot d_s^2 \cdot b \cdot f_y}$$

Substituting defined terms into Equation 6 gives:

$$\left(\frac{\rho^2}{2 \cdot M}\right) - \rho + R = 0.0 \text{ ....(Equation 7)}$$

Solving the quadratic Equation 7 gives:

$$\rho = M \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R}{M}}\right) \text{ ....(Equation 8)}$$

Equation 8 gives directly the percentage of reinforcement required to resist the applied factored loads.

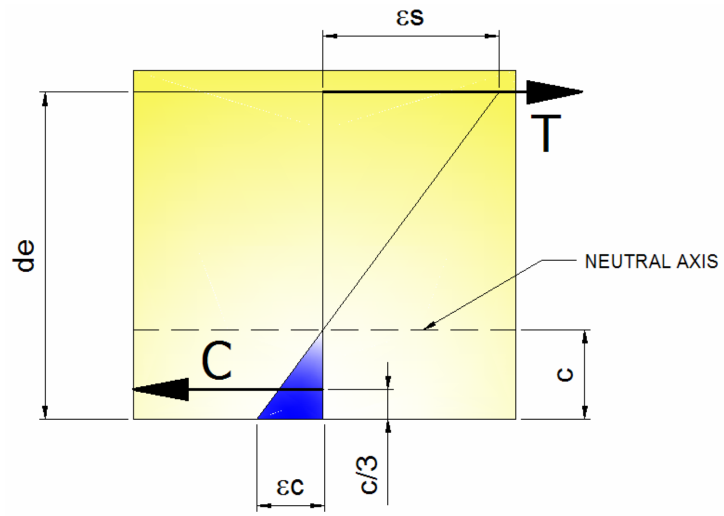
The Ultimate Moment Capacity of reinforced concrete columns is determined using the computer program PCACOL. This is based on ACI-95 and is consistent with the requirements of AASHTO LRFD.

(3) Service Moment Capacity (AASHTO LRFD Section 5.7)

The solution for the analysis of reinforced concrete sections in flexure under no axial loading and no compression reinforcement is derived from:

- Linear stress-stress relations
- Plane sections remain plane under flexure
- Equilibrium of internal forces (compressive force in concrete is equal the tensile force in the reinforcement)

With reference to the illustration below for a rectangular section:



Defining terms:

C = Compressive force in concrete

T = Tensile force in reinforcing steel

d_e = Effective depth from extreme compressive fiber to centroid of tensile reinforcement

c = Depth of concrete in compression

b = Width of rectangular section

ε_c = Compressive strain in concrete at extreme compressive fiber

ε_s = Tensile strain in reinforcement

E_c = Modulus of elasticity of concrete

E_s = Modulus of elasticity of reinforcement

α = Modular ratio, $\alpha = \frac{E_S}{E_C}$

Establishing equilibrium of forces gives the following:

$T = C$ ……………………………………………………………………….(Equation 1)

$T = A_S \cdot \varepsilon_S \cdot E_S$ ……………………………………………………………….(Equation 2)

$C = \frac{b \cdot c}{2} \cdot \varepsilon_c \cdot E_c$ ……………………………………………………………(Equation 3)

From considerations of compatibility of strains:

$\frac{\varepsilon_C}{c} = \frac{\varepsilon_s}{d_e - c}$ ……………………………………………………………….(Equation 4)

$\alpha = \frac{E_S}{E_C}$ ………………………………………………………………………(Equation 5)

Substituting Equation 2 to Equation 5 into Equation 1 gives:

$$A_S \cdot \varepsilon_S \cdot E_S = \frac{b \cdot c}{2} \cdot \frac{\varepsilon_S}{d_e - c} \cdot c \cdot \frac{E_S}{\alpha}$$ ………………………………………….(Equation 6)

Dividing all terms of Equation 6 by $(\varepsilon_S \cdot E_S)$ and rearranging gives:

$$\frac{b \cdot c^2}{2 \cdot \alpha} + A_S \cdot c - A_S \cdot d_e = 0.0$$ ……………………………………………..(Equation 7)

Solving the quadratic Equation 7 gives:

$$c = \frac{\left(\sqrt{A_S^2 + \frac{2 \cdot b \cdot A_S \cdot d_e}{\alpha}} - A_S\right)}{b} \cdot \alpha$$ …………………………………………(Equation 8)

Equation 8 gives directly the depth of concrete in compression, c, for a given area of reinforcing steel A_S.

The lever arm of the reinforcing steel, z, with respect to the centroid of the compressive force in the concrete is then obtained from:

$$z = d_e - \frac{c}{3}$$ ………………………………………………………………..(Equation 9)

The stress in the reinforcement, f_s, can then be determined from:

$$f_s = \frac{M_S}{A_S \cdot z}$$ ……………………………………………………………..(Equation 10)

where:

M_S = applied serviceability limit state moment from factored loads.

For beams with multiple layers of tensile reinforcement, $As1$, $As2$, $As3$, ...Asn, located at effective depths $d1$, $d2$, $d3$, ...dn, Equation 8 is modified as follows:

$$c = \frac{\left(\sqrt{A_T^2 + \frac{2 \cdot b \cdot B}{\alpha}} - A_T\right)}{b} \cdot \alpha$$ ……………………………………………..(Equation 8A)

where:

A_T = Total area of tensile reinforcement, $A_T = A_{S1} + A_{S2} + A_{S3} + \ldots\ldots\ldots\ldots A_{Sn}$

B = $A_{S1} \cdot d1 + A_{S2} \cdot d2 + A_{S3} \cdot d3 + \ldots\ldots\ldots\ldots A_{Sn} \cdot dn$

AASHTO Article 5.7.3.4 requires that components shall be so proportioned that the tensile stress in the mild steel reinforcement at the service limit state does not exceed f_{sa}, determined as:

$$f_{sa} = \frac{Z}{(d_c \cdot A)^{\frac{1}{3}}} \leq 0.6 \cdot f_y$$

where:

d_c= depth of concrete from extreme tension fiber to center of bar (mm)

A = area of concrete having the same centroid as the principal tensile reinforcement and bounded by the surfaces of the cross-section and a straight line parallel to the neutral axis, divided by the number of bars (mm^2)

Z = crack width parameter (N/mm)

For moderate exposure conditions, Z shall not exceed 30000 N/mm.

The Design Criteria established for the project (based on current Indonesian Standards and BMS) specifies that the allowable stress of reinforcing bars in tension shall be 0.5fy or 170MPa, whichever is smaller. (Design Criteria Table 2.4.2-2)

Given that for Grade 40 reinforcement fy = 390MPa, the 170MPa allowable stress implies a limit of 0.43fy.

The Design Criteria are therefore considered more onerous and will be applied for the serviceability checks on the coping.

(4) Limits for Reinforcement (AASHTO LRFD Article 5.7.3.3)

Maximum Reinforcement

The maximum area of longitudinal reinforcement for RC columns shall be such that:

$$\frac{c}{d_e} \leq 0.42$$

where:

c = distance from extreme compression fiber to the neutral axis

$$c = \frac{A_S \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b}$$

d_e = distance from extreme compression fiber to the centroid of non prestressed tensile reinforcement

β_1= stress block factor

f_y = yield strength of reinforcing bars

f_c = compressive strength of concrete

Minimum Reinforcement

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, at least equal to the lesser of:

- 1.2 times the cracking moment, M_{CR} , determined on the basis of elastic stress distribution and the modulus of rupture, f_r, of the concrete:

  $f_r = 0.63 \cdot \sqrt{f_c}$ in MPa

  For monolithic construction:

  $M_{CR} = S_C \cdot f_r$

  where:

  S_c = section modulus for the extreme fiber of the section where tensile stress is caused

- 1.33 times the factored moment required by the applicable ultimate load combinations

Notes on Shear Design – Reinforced Concrete Coping

(1) General

Refer to Notes on Shear Design – Reinforced Concrete for the design approach.

(2) Column Connections (AASHTO LRFD Article 5.10.11.4.3)

The nominal shear resistance, V_n, provided by the concrete in the joint of a frame or bent in the direction under consideration, shall satisfy:

$$V_n \leq 1.0 \cdot b \cdot d \cdot \sqrt{f_c}$$

(3) Brackets and Corbels (AASHTO LRFD Article 5.13.2.4)

The requirements of AASHTO are satisfied in the design of the components of the pier coping that can be considered as brackets and corbels.

(4) Beam Ledges (AASHTO LRFD Article 5.13.2.5)

The requirements of AASHTO are satisfied in the design of the components of the pier coping that can be considered as beam ledges.

Notes on Joint Conection Design

References :

R1. SEISMIC DESIGN AND RETROFIT OF BRIDGES – M.J.N Priestley, F. Sieble, G.M. Calvi.
R2. SEISMIC DESIGN OF REINFORCED CONCRETE BRIDGES – Yan Xiao

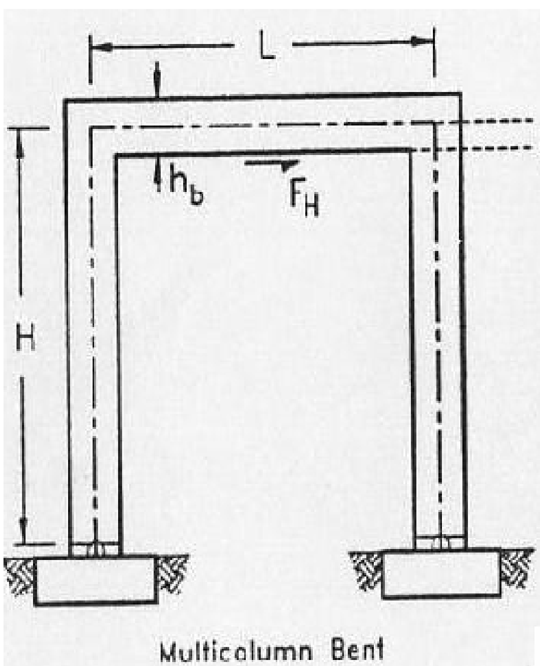
Moment-Resisting Conection Between Column and Beam

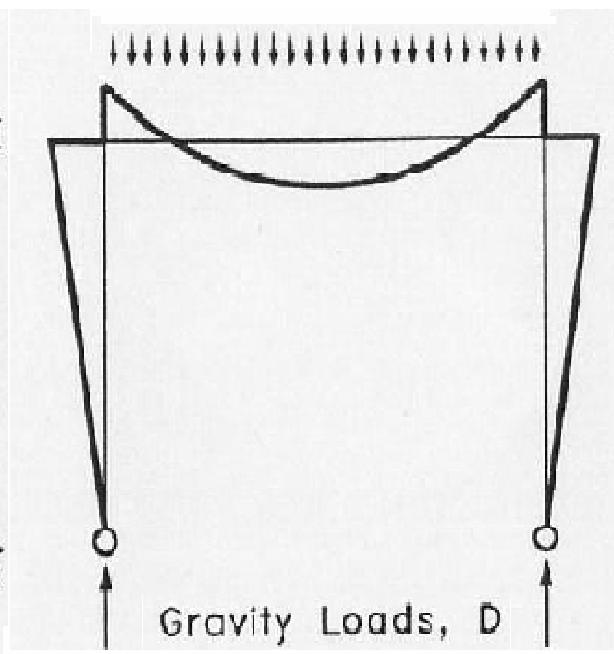
Connections are key elements that maintain the integrity of overall structure, they should be designed carefully to ensure the full transfer of seismic forces and moments. Because of their importance, complexity, and difficulty of repair if damaged, connections are typically provided with a higher degree of safety and conservativeness than column or beam members. Current AASHTO-LRFD Code do not provide specific design requirements for joint, except requiring the lateral reinforcement for columns to be extended into column/footing or column cap beam joint. A new design approach recently developed by Priestly & Calvi guidelines design is summarized bellow:

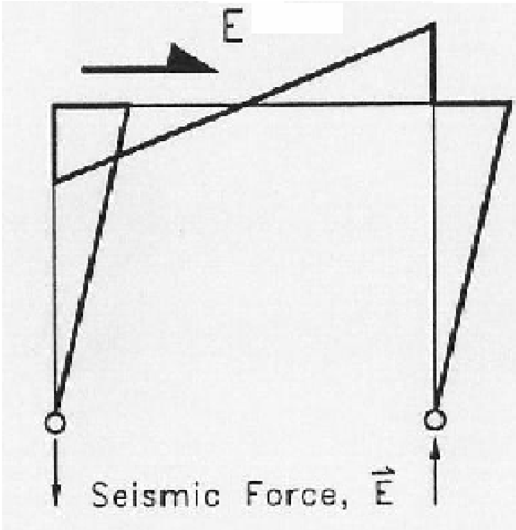
Design Force

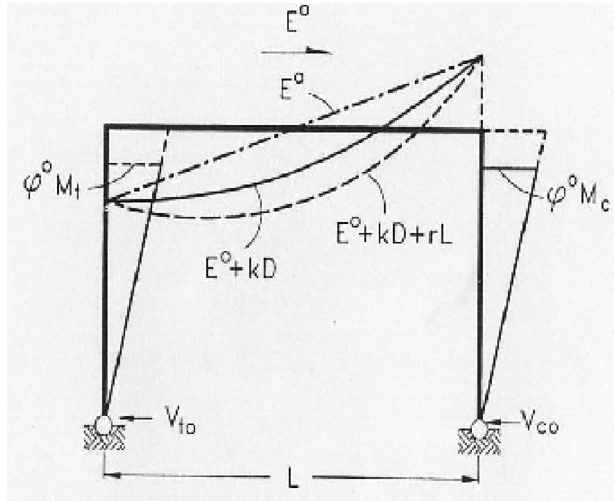
In moment-resisting frame structures, the force transfer typically results in sudden changes (magnitude and direction) of moments at connections. Sudden moment change cause significant shear forces. Thus, joint shear design is the major concern of column and beam connection, as well as that longitudinal reinforcement of beam and column are to be properly anchored or continued through the joint to transmit. For seismic design, joint shear force can be calculated based on the equilibrium condition using force generated by maximum plastic moment acting on the boundary of connections.

Momen Redistribution of Design Action





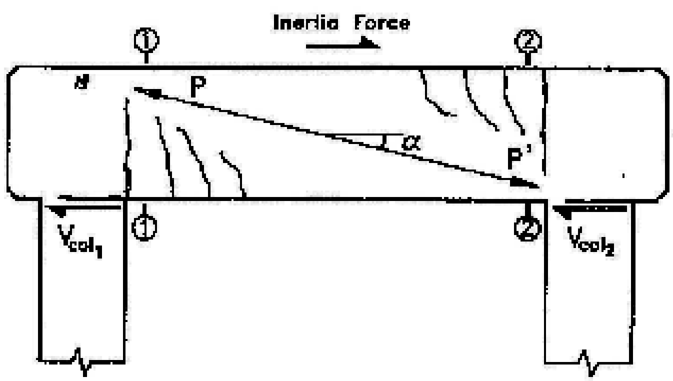




Cap beam design moments.

Beam Design for Flexure and Shear for beam are follow AASHTO LRFD Section 5.7 Flexure Design and AASHTO LRFD Section 5.8 Shear Design

Truss mechanism contribution for beam is illustrated by twin-column bent of figure bellow :



At section 1-1

$P := F_p - V_{col.1}$ hence

$V_p := 0.85\left(F_p - V_{col.1}\right)\tan(\alpha)$ ……………………………………….5.78a(R1)

At section 2-2

$P' := F_p - V_{col.2}$ hence

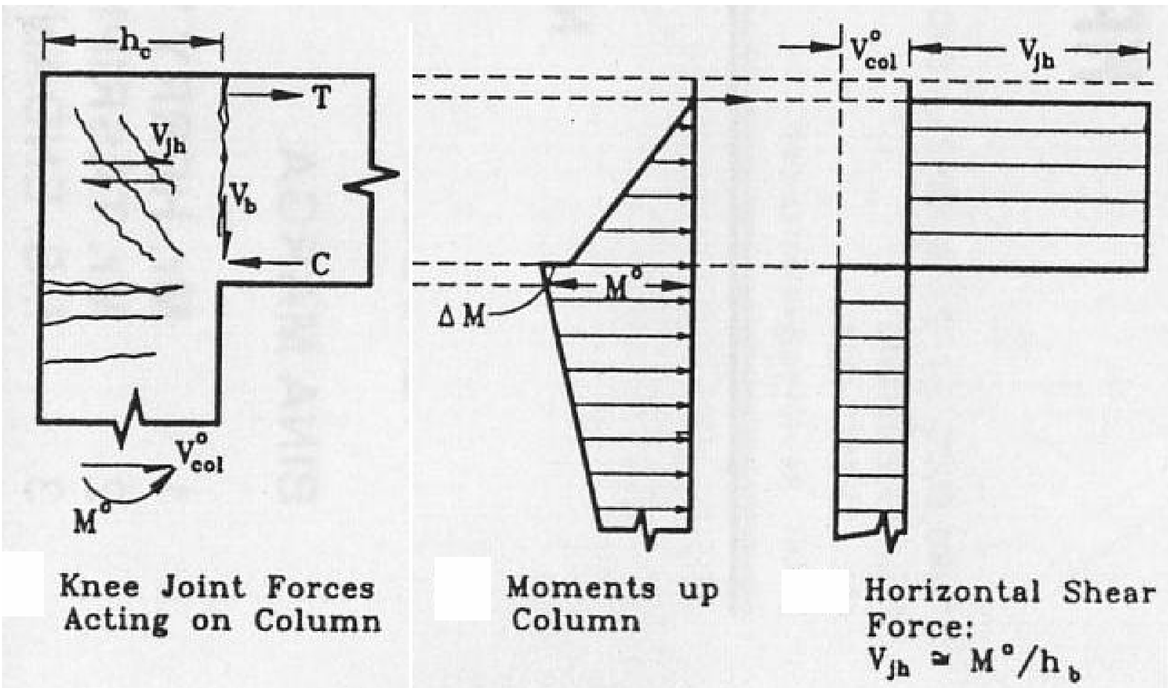
$V_p := 0.85\left(F_p - V_{col.2}\right)\tan(\alpha)$ ……………………………………….5.78b(R1)

Design of Beam-Column Joint

Shear Force in Beam-Column Joint

The traditional approach for investigating force transfer in beam-column joint has been based on an assessment of the joint shear force developed from equilibrium considerations of the member force acting at the joint boundary.

Consider knee joint in twin column P6 where column moment is overstrength, corresponding to plastic moment capacity in accordance with section 7 Column Design the column is considered as an independent member extending to the top of joint, with the influence of the beam or beams represented by force T, C, and V_B applied to this independent member.



The overstrength moment M^o continues to increase above the level of the beam soffit until the line of action of the beam force C. The moment slope reverse under this force, decreasing to zero at the height of the upper stress resultant T. Note that an incremental moment decrease ΔM is shown at the beam lower stress resultant due to moment provided by the beam shear.

$$\Delta M := V_b \cdot \frac{h_c}{2} \qquad \text{……5.81a(R1)}$$

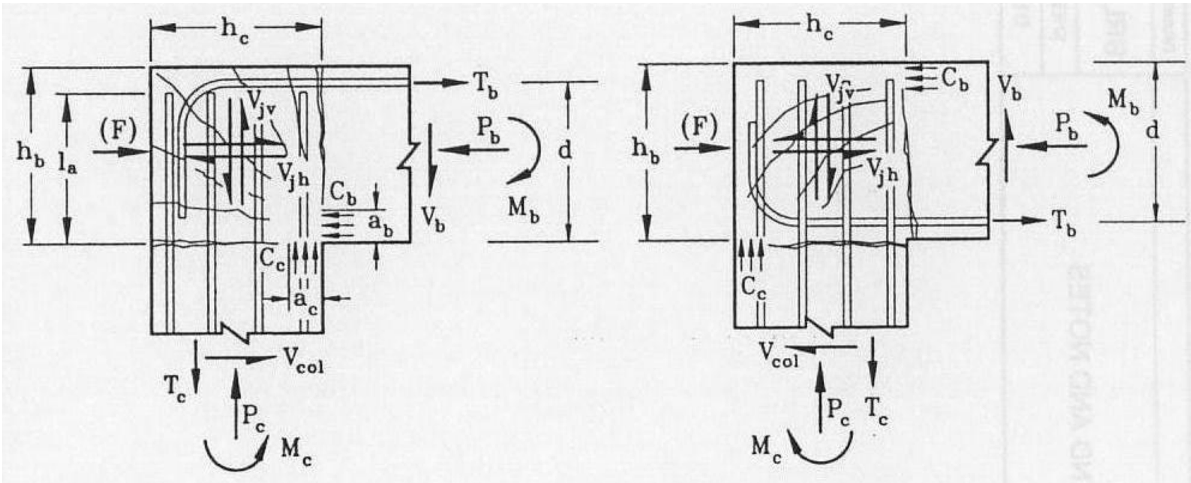
$$V_{jh} := \frac{Mo}{h_b} \qquad \text{……5.83(R1)}$$

In similar fashion, the force acting on beam, considered to extend through the joint given

$$V_{jv} := \frac{V_{jh} \cdot h_b}{h_c} \qquad \text{……5.83(R1)}$$

Knee Joints

Knee joints are the most common type of joint occurring in multi column bridge bents when transverse response is considered. Equilibrium under closing and opening moment are represented in Figures bellow :



In these figures, the beam tensile, compressive and shear stress resultants are indicate by T_b, C_b, and V_b, with T_c, C_c, and V_{col} being corresponding force for column. Axial forces P_c and P_b are present in the column and beam, respectively, and prestress force F is shown, which will, of course, be zero if the cap bem is reinforced conventionally. Moment M_b and M_c on the joint boundaries induce the flexural stress resultant note above. Equilibrium equations governing the relationships between the various force are summarized (R1) bellow :

Action	Closing Joint	Opening Joint	
Beam moment	$M_b = T_b\left(d - \frac{a_b}{2}\right) + P_b\left(\frac{h_b}{2} - \frac{a_b}{2}\right)$	(same)	(5.85*a*)
Beam axial force	$P_b = (F) + V_{col}$	$P_b = (F) - V_{col}$	(5.85*b*)
Beam compressive force	$C_b = T_b + P_b$	(same)	(5.85*c*)
Column moment	$M_c = M_b + V_b\frac{h_c}{2} - V_{col}\frac{h_b}{2}$	(same)	(5.85*d*)
	$\approx T_c\left(0.7h_c - \frac{a_c}{2}\right) + P_c\left(\frac{h_c}{2} - \frac{a_c}{2}\right)$	(same)	(5.85*e*)

Column compressive force	$C_c = T_c + P_c$	(same)	(5.85f)
Horizontal joint shear force	$V_{jh} = T_b(+0.5F)$	$V_{jh} = C_b(-0.5F)$	(5.85g)
Vertical joint shear force	$V_{jv} \approx \frac{V_{jh} h_b}{h_c}$	(same)	(5.85h)

Nominal Shear Stress

The nominal shear stress in beam-column joints can be found directly from the joint shear force as :

$$v_{jh} := \frac{V_{jh}}{b_{je} \cdot h_c} \qquad \text{……………………………………………………5.87(R1)}$$

$$v_{jv} := \frac{V_{jv}}{b_{je} \cdot h_b}$$

Where b_{je} is the effective width of the joint.

Design of Uncracked Joint

Joint can be conservatively designed based on elastic theory for not permitting cracks. In this approach, the principal tensile stress within a connection is calculated and compared with allowable tensile strength . The principal tensile stress, p_t and principal compression stress, p_c with a simple Mohr's circle analysis for stress shows that nominal principal stresses in joint region are given :

$$p_c := \frac{f_v + f_h}{2} + \sqrt{\left(\frac{f_v - f_h}{2}\right)^2 + v_j^2} \qquad \le 0.3\ f'c$$

……………………………………5.89(R1,2)

$$p_t := \frac{f_v + f_h}{2} - \sqrt{\left(\frac{f_v - f_h}{2}\right)^2 + v_j^2} \qquad f_t \le 0.29\sqrt{f'c} \cdot MPa$$

where :

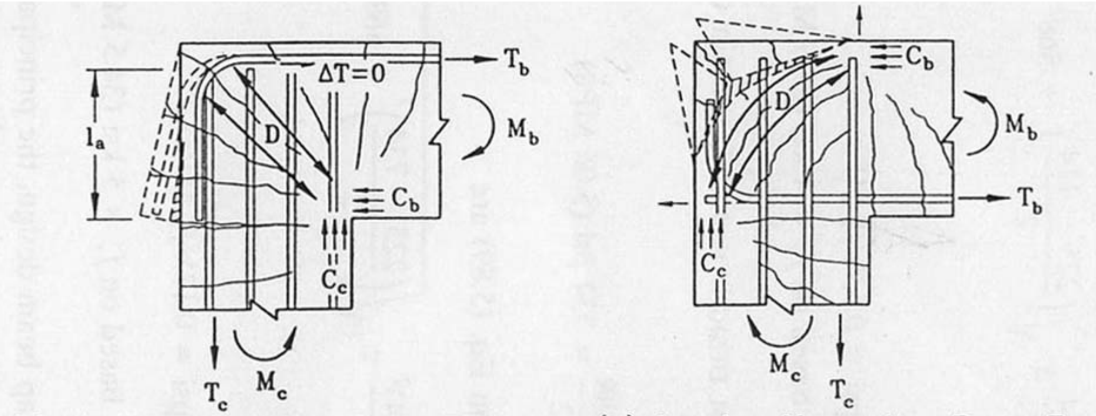
$$f_h := \frac{P_b}{b_b \cdot h_b}$$

and ………………………………………………………………5.90(R1)

$$f_v := \frac{P_c}{b_{je} \cdot (h_c + 0.5 \cdot h_b)}$$

Mechanism of Force Tranfer in Cracked Joints

When principal tension stresses excided the joint tension strength, cracking occurs and the force transfer from beam to column implied by equilibrium considerations can no longer be based on assumptions of isotropic material performance. Typical pattern of cracks developed in knee joint under closing and opening moments.



(a) Closing Moments. Beam Bottom Steel Omitted for Clarity
(b) Opening Moments. Beam Top Steel Omitted for Clarity

a. Closing Moment

The amount of transverse hoop reinforcement required to provide anchorage of column reinforcement after splitting crack develop can be calculated by a shear friction approach.The hoop reinforcement should not excided that corresponding to a strain εσ=0.0015, since higher strains appear to result in excessive dilation of circumferential crack with reduced efficiency of the shear friction mechanism. A required volumetric ratio of transverse reinforcement to avoid anchorage failure is :

$$\rho_{s1} := \frac{0.46 \cdot A_{sc} \cdot fo_{yc}}{D_r \cdot l_a \cdot f_{sh}} \qquad \text{...5.92(R1)}$$

A_{sc} = Total area of column longitudinal reinforcement.

fo_{yc} = is overstrength stress in the column reinforcement, including strain hardening and yield overstrength.
= 1.4 fy

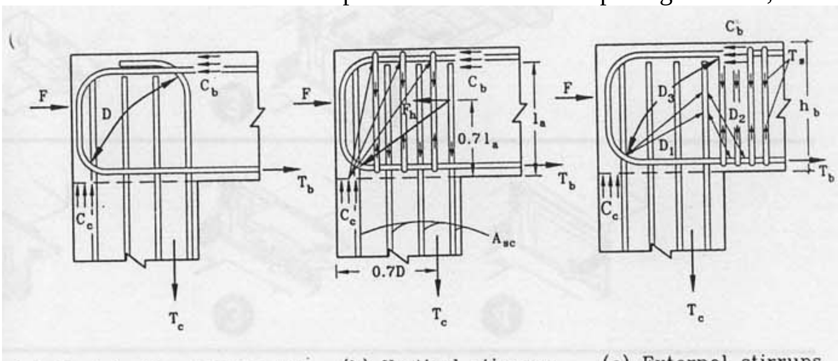
f_{sh} = 0.0015 Es

again, the minimum requirement should be satisfied :

$$\rho_{smin} := \frac{0.29 \cdot \sqrt{\frac{f_c}{MPa}} \cdot MPa}{f_y} \qquad \text{...5.96(R1)}$$

b. Opening Moment

Three mechanisms to avoid the potential failure under opening moment, as show



(a) Bent inside column bar (b) Vertical stirrups (c) External stirrups

The solution (a) and (b) is likely to cause unacceptable congestion and would require each of the tails of the column bars to be anchored with the resisting force of not less than $0.0033A_b\ f_y$.
Total area of vertical stirrup reinforcement required is

$$A_{jv} := 0.25A_{sc} \cdot \frac{fo_{yc}}{f_{yv}} \qquad \text{……………………………………………….5.97(R1)}$$

If $fo_{yc} = 1.4\ f_{yc}$ for grade 60 rebar design , placing this amount vertical reinforcement can be difficult.

Horizontal hoops are needed, that amount of hoop reinforcement is given by

$$\rho_s := \frac{0.6A_{sc} \cdot fo_{yc}}{l_a^{\,2} \cdot f_{yh}} \qquad \text{………………………………………………….5.99(R1)}$$

The mechanism (c) required amount of vertical beam stirrups reinforcement is

$$A_{jv} := 0.125A_{sc} \cdot \frac{fo_{yc}}{f_{yv}} \qquad \text{……………………….……………………5.100(R1)}$$

And the additional area of beam bottom reinforcement required is thus :

$$\Delta A_{sb} := 0.0625A_{sc} \cdot \frac{fo_{yc}}{f_{yb}} \qquad \text{……………………………………………….5.101(R1)}$$

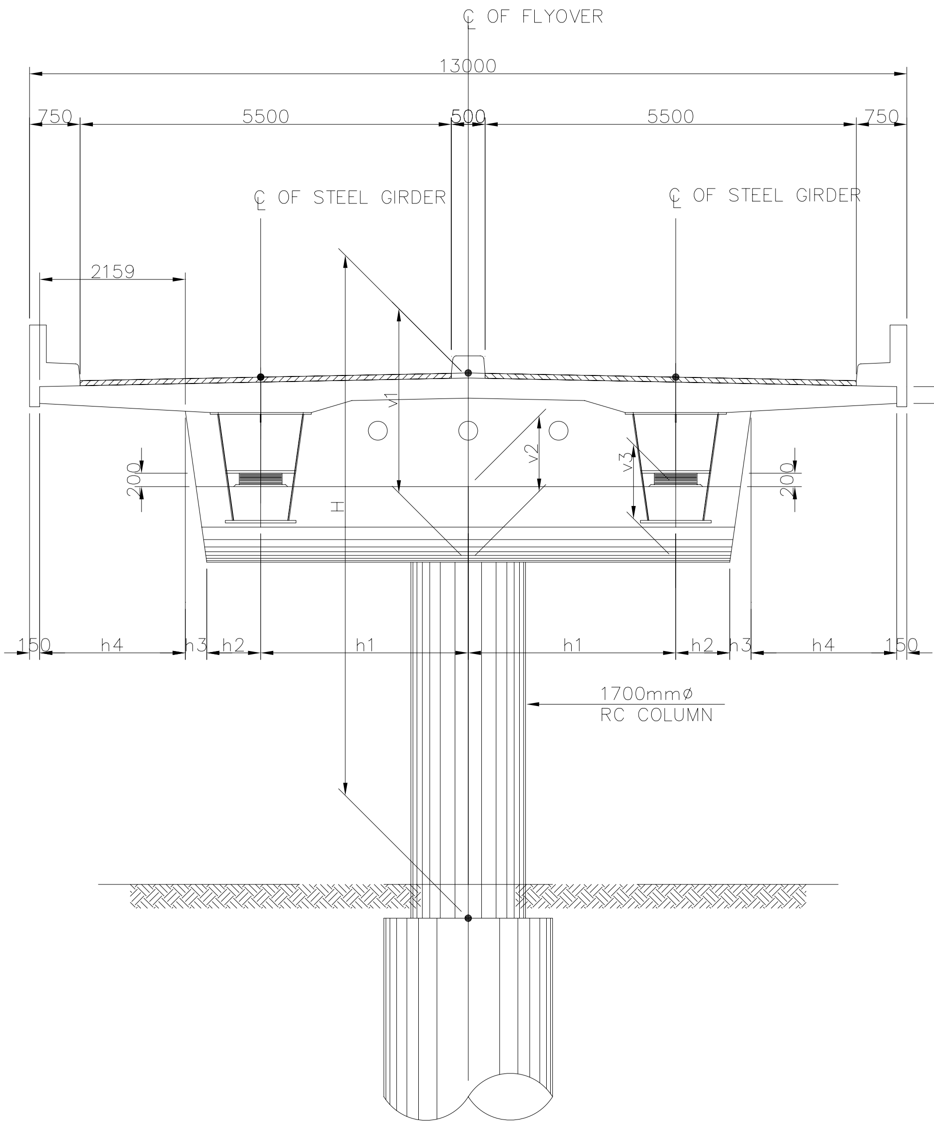
P3 Expansion Coping

KATAHIRA & ENGINEERS INTERNATIONAL

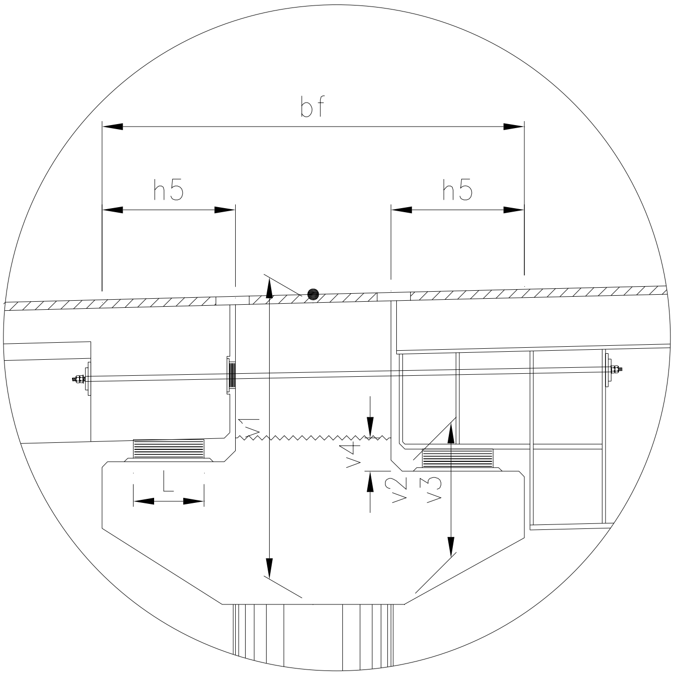
Project: **Detailed Design Study of North Java Corridor Flyover Project**

Calculation: **Detailed Design Substructure**
Balaraja Flyover
Coping Design - Pier P3

Layout



Coping Cross Section



Pier Coping Data

Overall width of deck	**B**	13000	mm
Offset to bearing - PC deck	**$h1_c$**	3175	mm
Offset to bearing - Steel deck	**$h1_s$**	3075	mm
Edge distance	**h2**	800	mm
Side slope width	**h3**	316	mm
Cantilever length	**h4**	2160	mm
Beam ledge width	**h5**	975	mm
Beam ledge soffit width	**h6**	950	mm
Width of coping	**b_f**	3350	mm
Total depth at coping	**v1**	2732	mm
Beam ledge height at column	**v2**	1200	mm
Beam ledge height at bearing	**v3**	1200	mm
Upstand to const. joint	**v4**	300	mm
Bearing width	**W**	800	mm
Bearing length	**L**	620	mm
Column Diameter	**D**	1700	mm
Concrete Comp Strength	**f_c**	30	MPa
Rebar Yield Strength	**f_y**	390	MPa
Strength Reduction Factor - Bending		0.8	
Strength Reduction Factor - Shear		0.7	
Mod. Elasticity - Concrete	**E_c**	27628	MPa
Mod. Elasticity - Steel	**E_s**	200000	MPa
Modular Ratio		7.24	
Height of piers supporting deck	**H**	9912	mm
Length of deck btwn. Joints	**L_d**	73.6	m
Deck skew	**S_k**	0	Deg

$B := B\cdot mm$

$h1_c := h1_c\cdot mm$

$h1_s := h1_s\cdot mm$

$h2 := h2\cdot mm$

$h3 := h3\cdot mm$

$h4 := h4\cdot mm$

$h5 := h5\cdot mm$

$h6 := h6\cdot mm$

$b_f := b_f\cdot mm$

$v1 := v1\cdot mm$

$v2 := v2\cdot mm$

$v3 := v3\cdot mm$

$v4 := v4\cdot mm$

$D := D\cdot mm$

$L := L\cdot mm$

$W := W\cdot mm$

$f_c := f_c\cdot MPa$

$f_y := f_y\cdot MPa$

$E_c := E_c\cdot MPa$

$E_s := E_s\cdot MPa$

Analysis Output

The analysis output for **deck reactions** at the expansion piers, obtained from the SAP 3D model, is presented below for:

- Nominal deck dead load case - used for erection case
- Nominal superimposed dead load case
- ULS Combination 1 - live load
- SLS Combination 1 - live load
- Nominal earthquake effects (R=1.0)

The frame elements selected are as follows:

- D34 - end span frame in span 3 (PC deck) adjacent to pier 3
- D41 - end span frame in span 3 (Steel Deck) adjacent to pier 3

NOMINAL DECK DEAD LOAD

TABLE: Element Forces - Deck Dead Load Reactions				
Frame	**Station**	**V2 SHEAR VERT**	**V3 SHEAR TRANS**	**T TORSION**
Text	m	KN	KN	KN-m
DECK34	Min	1342.9	-0.4	-0.6
DECK34	Max	1342.9	-0.4	-0.6
DECK41	Min	-1505.0	-1.1	2.2
DECK41	Max	-1505.0	-1.1	2.2

$V2_{dl} := V2_{dl} \cdot kN$ $\quad V3_{dl} := V3_{dl} \cdot kN$ $\quad T_{dl} := T_{dl} \cdot kN \cdot m$

NOMINAL SUPERIMPOSED DEAD LOAD

TABLE: Element Forces -Superimposed Load Reaction				
Frame	**Station**	**V2 SHEAR VERT**	**V3 SHEAR TRANS**	**T TORSION**
Text	m	KN	KN	KN-m
DECK34	Min	244.6	0.0	0.0
DECK34	Max	244.6	0.0	0.0
DECK41	Min	-289.9	-0.2	0.4
DECK41	Max	-289.9	-0.2	0.4

$V2_{sdl} := V2_{sdl} \cdot kN$ $\quad V3_{sdl} := V3_{sdl} \cdot kN$ $\quad T_{sdl} := T_{sdl} \cdot kN \cdot m$

ULS COMBINATION 1 - LIVE LOAD

TABLE: Element Forces - Deck COMB1 ULS Reactions				
Frame	Step Type	V2 SHEAR VERT	V3 SHEAR TRANS	T TORSION
Text	Text	KN	KN	KN-m
DECK34	Min	1911.7	-39.1	-900.2
DECK34	Max	6180.6	36.3	896.3
DECK41	Min	-6701.7	-68.4	-951.3
DECK41	Max	-2076.1	59.8	969.1

$V2_u := V2_u \cdot kN$ $\quad V3_u := V3_u \cdot kN$ $\quad T_u := T_u \cdot kN \cdot m$

SLS COMBINATION 1 - LIVE LOAD

TABLE: Element Forces - Deck COMB1 SLS Reactions				
Frame	Step Type	V2 SHEAR VERT	V3 SHEAR TRANS	T TORSION
Text	Text	KN	KN	KN-m
DECK34	Min	1289.6	-22.8	-501.6
DECK34	Max	3974.7	20.0	497.7
DECK41	Min	-4463.8	-41.1	-527.8
DECK41	Max	-1344.5	32.8	544.9

$V2_s := V2_s \cdot kN$ $\quad V3_s := V3_s \cdot kN$ $\quad T_s := T_s \cdot kN \cdot m$

NOMINAL EARTHQUAKE LOAD - EQX (R=1)

TABLE: Element Forces - EQX				
Frame	**Step Type**	**V2 SHEAR VERT**	**V3 SHEAR TRANS**	**T TORSION**
Text	Text	KN	KN	KN-m
DECK34	Min	1147.0	-1395.6	-4245.8
DECK34	Max	2028.0	1394.8	4244.6
DECK41	Min	-2437.3	-2453.4	-1369.2
DECK41	Max	-1152.6	2450.8	1374.4

$V2_{eqx} := V2_{eqx} \cdot kN$ $V3_{eqx} := V3_{eqx} \cdot kN$ $T_{eqx} := T_{eqx} \cdot kN \cdot m$

NOMINAL EARTHQUAKE LOAD - EQY (R=1)

TABLE: Element Forces - EQY				
Frame	**Step Type**	**V2 SHEAR VERT**	**V3 SHEAR TRANS**	**T TORSION**
Text	Text	KN	KN	KN-m
DECK34	Min	1291.4	-2022.4	-6090.3
DECK34	Max	1883.6	2021.6	6089.1
DECK41	Min	-2203.7	-3501.8	-1914.9
DECK41	Max	-1386.1	3499.2	1920.1

$V2_{eqy} := V2_{eqy} \cdot kN$ $V3_{eqy} := V3_{eqy} \cdot kN$ $T_{eqy} := T_{eqy} \cdot kN \cdot m$

MAXIMUM SHEAR FORCE DUE TO PLASTIC HINGING

The maximum shear force that the bearings will be subject to in the tranverse direction is the shear force due to plastic hinging at the base of the column.

The shear force due to plastic hinging at the base of the column $V_P := 2612 \cdot kN$

This shear force should be carried by the transverse shear key on each bearing shelf or carried by the bearings directly on on bearing shelf. The shear forces given above for earthquake loading should be used in the design if they are smaller than the plastic hinging effects.

Minimum Displacement Requirements (AASHTO LRFD Article 4.7.4.4)

Bridge seat widths at expansion bearings without restrainers, STU's or dampers shall either accommodate the greater of the maximum displacement calculated from the seismic analysis or a percentage of the emprical seat width, *N*, specified below.

The percentage of *N* applicable to the bridge seismic zone shall be 150%.

The length of the bridge deck to the adjacent expansion joint $L_d := L_d \cdot m$ $L_d = 73.6\ m$

The height of the columns supporting the deck $H := H \cdot mm$ $H = 9912\,mm$

Skew of the support measured from line normal to span $S_k = 0$

The empirical seat width shall be taken as:

$$N := \left(200 + 0.0017 \cdot \frac{L}{mm} + 0.0067 \cdot \frac{H}{mm}\right) \cdot \left(1 + 0.000125 \cdot S_k^{\ 2}\right) \cdot mm$$

$N = 267\,mm$

Check that the seat width provided is greater than 150% of *N*:

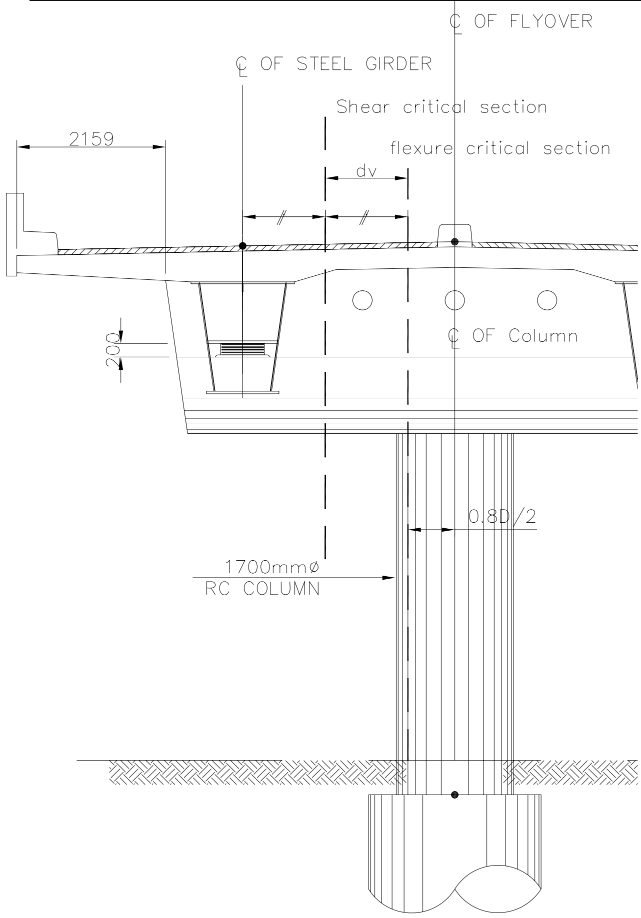
Seat width $h5 = 975\,mm$

$150\% \cdot N = 401\,mm$

$$Seat_{Width} := \begin{cases} \text{"OK"} & \text{if } h5 \geq N \cdot 150\% \\ \text{"INADEQUATE"} & \text{otherwise} \end{cases}$$

$Seat_{Width} = \text{"OK"}$

Critical Sections for Design of Pier Coping



Design for Erection Case

During erection the deck dead loads are supported by the partially constructed pier coping.

Partial construction is required to accommodate the prestressing jacks of the PC deck.

To take account of construction equipment on the deck a construction load of $2kN/m^2$ is applied over the full deck area in addition to the deck dead load.

Cross sectional area of coping during erection:

$$A_{ce} := v3 \cdot b_f + (v2 - v3) \cdot D + 113mm \cdot h5 + (v1 - v2) \cdot \left(b_f - 2 \cdot h5\right) \qquad A_{ce} = 6.275\ m^2$$

Coping self weight during erection:

$$w_{ce} := A_{ce} \cdot 24.5 \cdot \frac{kN}{m^3} \qquad w_{ce} = 153.7\ \frac{kN}{m}$$

PC Deck Dead Load Reaction - max at the bearing

$$V2_{pc} := \left|\frac{V2_{dl_2}}{2}\right| + \frac{\left|T_{dl_2}\right|}{h1_c \cdot 2} \qquad V2_{pc} = 671.5\ kN$$

Steel Deck Dead Load Reaction - max at the bearing

$$V2_{st} := \left|\frac{V2_{dl_3}}{2}\right| + \frac{\left|T_{dl_3}\right|}{h1_s \cdot 2} \qquad V2_{st} = 752.9\ kN$$

Erection Load Reaction - **20m PC deck** - per bearing - assuming 45% total reaction into bearing

$$V2_{ERpc} := 2\cdot\frac{kN}{m^2}\cdot B\cdot(20m)\cdot 45\%\cdot\frac{1}{2} \quad V2_{ERpc} = 117.0\,kN$$

Erection Load Reaction - **31m Steel deck** - per bearing - assuming 45% total reaction into bearing

$$V2_{ERst} := 2\cdot\frac{kN}{m^2}\cdot B\cdot(31m)\cdot 45\%\cdot\frac{1}{2} \quad V2_{ERst} = 181.3\,kN$$

Design for Flexure (AASHTO LRFD Section 5.7)

Bending moment in coping at face of column during erection

Coping self weight
$$M_{ce} := w_{ce}\cdot\frac{\left(\max\left(h1_c, h1_s\right) + h2 + \frac{h3}{2} - \frac{D}{2}\cdot.8\right)^2}{2} \qquad M_{ce} = 916.5\,kN\cdot m$$

PC deck reaction
$$M_{pc} := V2_{pc}\cdot\left(h1_c - \frac{D}{2}\cdot.8\right) \qquad M_{pc} = 1675.5\,kN\cdot m$$

Steel deck reaction
$$M_{st} := V2_{st}\cdot\left(h1_s - \frac{D}{2}\cdot.8\right) \qquad M_{st} = 1803.1\,kN\cdot m$$

Erection load reaction
$$M_{ER} := V2_{ERpc}\cdot\left(h1_c - \frac{D\cdot.8}{2}\right) + V2_{ERst}\cdot\left(h1_s - \frac{D\cdot.8}{2}\right) \qquad M_{ER} = 726.2\,kN\cdot m$$

Total bending moment in coping at face of column during erection - Ultimate Limit State

$$M_{EU} := \left(M_{ce} + M_{pc} + M_{st}\right)\cdot 1.25 + M_{ER}\cdot 1.5$$

$$M_{EU} = 6583.3\,kN\cdot m$$

Depth of section:

$$h_s := v2 + v4 \qquad h_s = 1500\,mm$$

Effective depth of coping during erection:

$$d_e := \left(h_s - 100mm\right) \qquad d_e = 1400\,mm$$

Width of coping at column:

$$b := b_f - h6\cdot 2 \qquad b = 1450\,mm$$

Strength reduction factor for flexure:

$$\Phi = 0.8$$

Determine area of reinforcement, A_f, in the coping to resist flexure
(ref AASHTO LFRD Article 5.7.3.2.3 -see notes on flexural design for rearrangement of terms):

$$R := \frac{M_{EU}}{\Phi\cdot b\cdot d_e^{\,2}\cdot f_y} \quad R = 0.0074 \qquad M := \frac{0.85\cdot f_c}{f_y} \quad M = 0.0654$$

$$\rho := M\cdot\left(1 - \sqrt{1 - \frac{2\cdot R}{M}}\right) \qquad \rho = 0.007902$$

$$A_f := \rho \cdot d_e \cdot b$$

$$A_f = 16040.9\,\text{mm}^2$$

Using 32mmϕ bars gives total number of bars to be distributed across the coping:

$$n_{bf} := \frac{A_f}{804 \cdot \text{mm}^2} \qquad n_{bf} = 20$$

Provide 30 No 32mm ϕ bars in two layers

$$A_f := 30 \cdot 804 \cdot \text{mm}^2$$

Calculate the stress block factor, β_1:

$$\beta_1 := \left| \begin{array}{l} \beta_1 \leftarrow 0.85 \\ \beta_1 \leftarrow \beta_1 - 0.05 \cdot \dfrac{f_c - 28\text{MPa}}{7.\text{MPa}} \ \text{ if } f_c > 28\text{MPa} \\ 0.65 \ \text{ if } \beta_1 < 0.65 \end{array} \right.$$

$$\beta_1 = 0.836$$

For rectangular sections, the depth of concrete in compression, c, is given by:

$$c := \frac{A_f \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} \qquad c = 304\,\text{mm}$$

Check that the required amount of reinforcement does not exceed the maximum allowed by the code:

$$\frac{c}{d_e} = 0.22$$

$$Max_{Limit} := \left| \begin{array}{l} \text{"OK"} \ \text{ if } \dfrac{c}{d_e} \le 0.42 \\ \text{"EXCEEDED"} \ \text{ otherwise} \end{array} \right. \qquad Max_{Limit} = \text{"OK"}$$

Calculate the modulus of rupture, f_r ,of the concrete:

$$f_r := 0.63 \cdot \sqrt{\frac{f_c}{\text{MPa}}} \cdot \text{MPa} \qquad f_r = 3.451\,\text{MPa}$$

Section modulus for the extreme fiber of the section, assuming rectangular section of width b:

$$S_c := \frac{b \cdot h_s^2}{6} \qquad S_c = 0.544\,\text{m}^3$$

Moment resisting by reinforcemen provided

$$M_{EUR} := \Phi \cdot c \cdot b \cdot \left(d_e - \frac{c}{2} \right) \cdot 0.85 \cdot f_c$$

$$M_{EUR} = 11236.1\,\text{kN} \cdot \text{m}$$

The cracking moment, M_{cr} ,is the given by:

$$M_{cr} := S_c \cdot f_r \qquad M_{cr} = 1876\,\text{kN} \cdot \text{m}$$

Check that the reinforcement can develop a resistance moment Mr at least equal to the lesser of:

- 1.2 times the cracking moment Mcr
- 1.33 times the factored moment required by the applicable strength load combination

$$Minimum_{Steel} := \left| \begin{array}{l} M_r \leftarrow \dfrac{M_{EUR}}{\Phi} \\ M \leftarrow \min\left(1.2M_{cr}, 1.33 \cdot M_{EU}\right) \\ \text{"OK"} \quad \text{if } M_r \geq M \\ \text{"NOT SATISFIED"} \quad \text{otherwise} \end{array} \right. \qquad Minimum_{Steel} = \text{"OK"}$$

Design for Shear (AASHTO LRFD Section 5.8)

Critical section for shear shall be taken as d_v from the internal face of the support (AASHTO LRFD Article 5.8.3.2)

$$d_v := \left| \begin{array}{l} 0.9 \cdot d_e \quad \text{if } 0.9 \cdot d_e > 0.72 \cdot h_s \\ 0.72 \cdot h_s \quad \text{otherwise} \end{array} \right.$$

$$d_v = 1260\,mm$$

Shear in coping at critical section during erection

Coping self weight $\quad V_{ce} := w_{ce} \cdot \left(\max\left(h1_c, h1_s\right) + h2 + \dfrac{h3}{2} - \dfrac{D}{2} \cdot .8 - d_v \right) \qquad V_{ce} = 337.1\,kN$

PC deck reaction $\quad V_{pc} := V2_{pc} \qquad V_{pc} = 671.5\,kN$

Steel deck reaction $\quad V_{st} := V2_{st} \qquad V_{st} = 752.9\,kN$

Erection load reaction $\quad V_{ER} := V2_{ERpc} + V2_{ERst} \qquad V_{ER} = 298.4\,kN$

Total shear force in coping at face of column during erection - Ultimate Limit State

$$V_{EU} := \left(V_{ce} + V_{pc} + V_{st}\right) \cdot 1.3 + V_{ER} \cdot 1.3$$

$$V_{EU} = 2677.9\,kN$$

Calculate nominal shear resistance of concrete section assuming beam section:

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

$$V_c = 1661\,kN$$

Strength reduction factor for shear:

$$\Phi_s = 0.7$$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \left| \begin{array}{l} V_s \leftarrow \dfrac{V_{EU}}{\Phi_s} - V_c \\ V_s \quad \text{if } V_s > 0kN \\ 0kN \quad \text{otherwise} \end{array} \right. \qquad V_s = 2164\,kN$$

Check that total required nominal shear resistance does not exceed limit:

$$V_{nlimit} := 0.25 \cdot f_c \cdot b \cdot d_v \qquad V_{nlimit} = 13702.5\,\text{kN}$$

$$CHECK := \begin{cases} \text{"OK"} & \text{if } V_c + V_s \le V_{nlimit} \\ \text{"FAIL"} & \text{otherwise} \end{cases} \qquad CHECK = \text{"OK"}$$

Provide 19mmϕ shear links wth **4 legs** across the section:

$$\phi_{link} := 19\text{mm}$$

$$A_v := \pi \frac{\phi_{link}^2}{4} \cdot 4 \qquad A_v = 1134\,\text{mm}^2$$

Determine required spacing of transverse reinforcement

$$s_t := \left| \begin{array}{l} s_{t1} \leftarrow \dfrac{A_v \cdot f_y}{0.083 \cdot \sqrt{\dfrac{f_c}{MPa}} \cdot MPa \cdot b} \\ s_{t2} \leftarrow \dfrac{A_v \cdot f_y \cdot d_v}{V_s} \quad \text{if } V_s > 0kN \\ s_{t2} \quad \text{if } (V_s > 0kN) \cdot (s_{t2} \le s_{t1}) \\ s_{t1} \quad \text{otherwise} \end{array} \right. \qquad s_t = 257\,\text{mm}$$

Calculate shear stress in order to determine maximum spacing of transverse reinforcement to resist shear:

$$v_u := \frac{V_{EU}}{\Phi_s \cdot (b \cdot d_v)} \qquad v_u = 2.094\,\text{MPa}$$

Calculate spacing taking into account maximum spacing requirements:

$$s_{max} := \begin{cases} \min(600mm, 0.8d_v) & \text{if } v_u < 0.125 f_c \\ \min(300mm, 0.4d_v) & \text{otherwise} \end{cases}$$

$$s_{max} = 600\,\text{mm}$$

Determine maximum required spacing of transverse reinforcement:

$$s_{t.erection} := \begin{cases} s_{max} & \text{if } s_{max} \le s_t \\ s_t & \text{otherwise} \end{cases}$$

$$s_{t.erection} = 257\,\text{mm}$$

Design for Permanent Condition

Cross sectional area of coping at support:

$$A_c := v3 \cdot b_f + (v2 - v3) \cdot D + \left(b_f - h5 \cdot 2\right) \cdot (v1 - v2) + 113mm \cdot h5 \qquad A_c = 6.275\ m^2$$

Coping self weight - main body:

$$w_c := A_c \cdot 24.5 \cdot \frac{kN}{m^3} \qquad w_c = 153.7\ \frac{kN}{m}$$

Coping self weight - cantilever wings:

$$w_{cw} := \frac{0.25m + 0.45m}{2} \cdot \left(b_f - h5 \cdot 2\right) 24.5 \cdot \frac{kN}{m^3} \qquad w_{cw} = 12.0\ \frac{kN}{m}$$

Superimposed dead load:

$$w_{sdl} := 0.125m \cdot \left(b_f - h5 \cdot 2\right) 22.5 \cdot \frac{kN}{m^3} \qquad w_{sdl} = 3.9\ \frac{kN}{m}$$

Railing dead load - each side:

$$W_r := \frac{0.433}{2} m^2 \cdot \left(b_f - h5 \cdot 2\right) 24.5 \cdot \frac{kN}{m^3} \qquad W_r = 7.4\ kN$$

D live loading on coping:

$$w_d := 9.0 \frac{kN}{m^2} \cdot \left(b_f - h5 \cdot 2\right) \qquad w_d = 12.6\ \frac{kN}{m}$$

PC Deck Dead Load Reaction - max at the bearing - ULS Comb1

$$V2_{upc} := \left| \frac{V2_{u_2}}{2} \right| + \frac{\left|T_{u_2}\right|}{h1_c \cdot 2} \qquad V2_{upc} = 3231.4\ kN$$

Steel Deck Dead Load Reaction - max at the bearing - ULS Comb1

$$V2_{ust} := \left| \frac{V2_{u_3}}{2} \right| + \frac{\left|T_{u_3}\right|}{h1_s \cdot 2} \qquad V2_{ust} = 3505.5\ kN$$

<u>Design for Flexure (AASHTO LRFD Section 5.7)</u>

Nominal bending moment in coping at face of column

Coping main body self weight

$$M_{cb} := w_c \cdot \frac{\left(\max(h1_c, h1_s) + h2 + \frac{h3}{2} - \frac{D}{2} \cdot .8\right)^2}{2} \qquad M_{cb} = 916.5\,kN \cdot m$$

Coping cantilever self weight

$$M_{cw} := w_{cw} \cdot h4 \cdot \left(\max(h1_c, h1_s) + h2 + h3 + \frac{h4}{2} - \frac{D}{2} \cdot .8\right) \qquad M_{cw} = 121.6\,kN \cdot m$$

Railing weight

$$M_r := W_r \cdot \left(\max(h1_c, h1_s) + h2 + h3 + h4 - \frac{D}{2} \cdot .8\right) \qquad M_r = 42.9\,kN \cdot m$$

Superimposed dead load on coping

$$M_{sdl} := w_{sdl} \cdot \frac{\left(5.75m - \frac{D \cdot .8}{2}\right)^2}{2} \qquad M_{sdl} = 50.6\,kN \cdot m$$

D live load on coping

$$M_d := w_d \cdot \frac{\left(5.75m - \frac{D \cdot .8}{2}\right)^2}{2} \qquad M_d = 161.9\,kN \cdot m$$

Ultimate bending moment in coping from max bearing reactions at face of column:

PC deck reaction

$$M_{upc} := V2_{upc} \cdot \left(h1_c - \frac{D}{2} \cdot .8\right) \qquad M_{upc} = 8062.4\,kN \cdot m$$

Steel deck reaction

$$M_{ust} := V2_{ust} \cdot \left(h1_s - \frac{D}{2} \cdot .8\right) \qquad M_{ust} = 8395.7\,kN \cdot m$$

Total bending moment in coping at face of column - Ultimate Limit State

Moment from loads on coping body

$$M_{CU} := \left(M_{cb} + M_{cw} + M_r\right) \cdot 1.3 + M_{sdl} \cdot 2.0 + M_d \cdot 1.8$$

$$M_{CU} = 1798.0\,kN \cdot m$$

Moment from max loads on bearings

$$M_{BU} := M_{upc} + M_{ust}$$

$$M_{BU} = 16458.1\,kN \cdot m$$

Total ULS moment

$$M_U := M_{CU} + M_{BU}$$

$$M_U = 18256.1\,kN \cdot m$$

Depth of section:

$$h_s := v1$$

Effective depth of coping:

$$d_e := \left(h_s - 150mm\right) \qquad d_e = 2582\,mm$$

Width of coping at column:

$$b := b_f - h6 \cdot 2$$

Strength reduction factor for flexure: $\Phi = 0.8$

Determine area of reinforcement, A_f, in the coping to resist flexure
(ref AASHTO LFRD Article 5.7.3.2.3 - see notes on flexural design for rearrangement of terms):

$$R := \frac{M_U}{\Phi \cdot b \cdot d_e^{\,2} \cdot f_y} \qquad R = 0.0061 \qquad M := \frac{0.85 \cdot f_c}{f_y} \qquad M = 0.0654$$

$$\rho := M \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R}{M}}\right) \qquad \rho = 0.006363$$

$$A_f := \rho \cdot d_e \cdot b$$

$$A_f = 23821\,\text{mm}^2$$

Using 32mmϕ bars gives total number of bars to be distributed across the coping:

$$n_{bf} := \frac{A_f}{804 \cdot \text{mm}^2} \qquad n_{bf} = 29.6$$

<u>Provide 36 No 32mmϕ bars</u>

$$n_{bars} := 36 \qquad A_f := n_{bars} \cdot 804 \cdot \text{mm}^2$$

Calculate the stress block factor, β_1:

$$\beta_1 := \left| \begin{array}{l} \beta_1 \leftarrow 0.85 \\ \beta_1 \leftarrow \beta_1 - 0.05 \cdot \dfrac{f_c - 28\text{MPa}}{7.\text{MPa}} \ \text{ if } \ f_c > 28\text{MPa} \\ 0.65 \ \text{ if } \ \beta_1 < 0.65 \end{array} \right.$$

$$\beta_1 = 0.836$$

For rectangular sections, the depth of concrete in compression, c, is given by:

$$c := \frac{A_f \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} \qquad c = 365\,\text{mm}$$

Check that the required amount of reinforcement does not exceed the maximum allowed by the code:

$$\frac{c}{d_e} = 0.14$$

$$Max_{Limit} := \left| \begin{array}{l} \text{"OK"} \ \text{ if } \ \dfrac{c}{d_e} \le 0.42 \\ \text{"EXCEEDED"} \ \text{ otherwise} \end{array} \right. \qquad Max_{Limit} = \text{"OK"}$$

Calculate the modulus of rupture, f_r ,of the concrete:

$$f_r := 0.63 \cdot \sqrt{\frac{f_c}{\text{MPa}}} \cdot \text{MPa} \qquad f_r = 3.451\,\text{MPa}$$

Section modulus for the extreme fiber of the section, assuming rectangular section of width b:

$$S_c := \frac{b \cdot h_s^2}{6} \qquad S_c = 1.804\,m^3$$

Moment resisting by reinforcemen provided

$$M_{EUR} := \Phi \cdot c \cdot b \cdot \left(d_e - \frac{c}{2}\right) \cdot 0.85 \cdot f_c$$

$$M_{EUR} = 25926.8\,kN \cdot m$$

The cracking moment, M_{cr} ,is the given by:

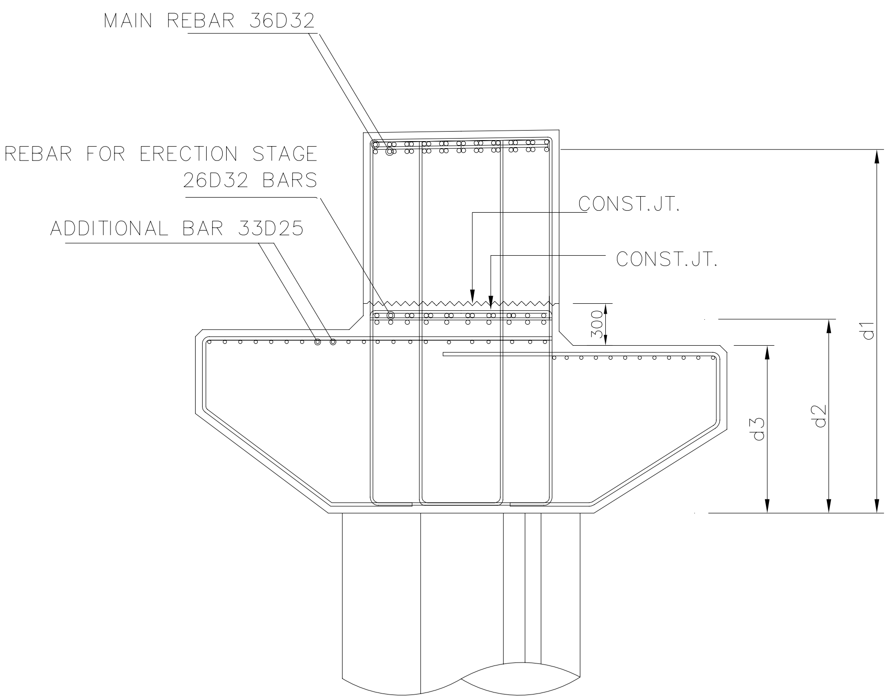
$$M_{cr} := S_c \cdot f_r \qquad M_{cr} = 6224\,kN \cdot m$$

Check that the reinforcement can develop a resistance moment Mr at least equal to the lesser of:

- 1.2 times the cracking momen Mcr
- 1.33 times the factored moment required by the applicable strength load combination

$$Minimum_{Steel} := \left| \begin{array}{l} M_r \leftarrow \dfrac{M_{EUR}}{\Phi} \\ M \leftarrow \min\left(1.2M_{cr}, 1.33 \cdot M_{EU}\right) \\ \text{"OK"} \quad \text{if } M_r \ge M \\ \text{"NOT SATISFIED"} \quad \text{otherwise} \end{array} \right. \qquad Minimum_{Steel} = \text{"OK"}$$

FINAL LAYOUT OF LONGITUDINAL REBAR



CHECK STRESSES AT SLS *(AASHTO LRFD Article 5.7.3.4)*

With reference to the illustration above - the final layout of reinforcement features three layers of rebar as follows:

Layer 1 - 36 No. 32mmϕ bars $A_{f1} := 36 \cdot 804 mm^2$ $d_1 := v1 - 150mm$

$A_{f1} = 28944\,mm^2$ $d_1 = 2582\,mm$

Layer 2 - 28 No. 32mmϕ bars $A_{f2} := 26 \cdot 490 mm^2$ $d_2 := v2 + v4 - 100mm$

$A_{f2} = 12740\,mm^2$ $d_2 = 1400\,mm$

Layer 3 - 20 No. 25mmϕ bars $A_{f3} := 20 \cdot 491 mm^2$ $d_3 := v2$

$A_{f3} = 9820\,mm^2$ $d_3 = 1200\,mm$

Total area $A_T := A_{f1} + A_{f2} + A_{f3}$

$A_T = 51504\,mm^2$

Lever arm factor $B := A_{f1} \cdot d_1 + A_{f2} \cdot d_2 + A_{f3} \cdot d_3$

The depth of concrete in compression, C, at the serviceability limit state, assuming three levels of rebar, is given by:

$$C := \frac{\sqrt{A_T^2 + \frac{2 \cdot b \cdot B}{\alpha}} - A_T}{b} \cdot \alpha \qquad C = 796\,mm$$

PC Deck Dead Load Reaction - max at the bearing - SLS Comb1

$$V2_{spc} := \left| \frac{V2_{s_2}}{2} \right| + \frac{\left| T_{s_2} \right|}{h1_c \cdot 2} \qquad V2_{spc} = 2065.7\,kN$$

Steel Deck Dead Load Reaction - max at the bearing - SLS Comb1

$$V2_{sst} := \left| \frac{V2_{s_3}}{2} \right| + \frac{\left| T_{s_3} \right|}{h1_s \cdot 2} \qquad V2_{sst} = 2317.7\,kN$$

Service bending moment in coping from max bearing reactions at face of column:

PC deck reaction $M_{spc} := V2_{spc} \cdot \left(h1_c - \frac{D}{2} \cdot .8 \right)$ $M_{spc} = 5154.0\,kN \cdot m$

Steel deck reaction $M_{sst} := V2_{sst} \cdot \left(h1_s - \frac{D}{2} \cdot .8 \right)$ $M_{sst} = 5550.9\,kN \cdot m$

Total bending moment in coping at face of column - Service Limit State

Moment from loads on coping body $M_{CS} := \left(M_{cb} + M_{cw} + M_r \right) \cdot 1.0 + M_{sdl} \cdot 1.0 + M_d \cdot 1.0$

$M_{CS} = 1293.6\,kN \cdot m$

Moment from max loads on bearings $M_{BS} := M_{spc} + M_{sst}$ $M_{BS} = 10704.9\,kN \cdot m$

Total SLS moment $M_S := M_{CS} + M_{BS} - 0.5 \cdot (M_{ce} + M_{pc} + M_{st})$

$M_S = 9800.9\,kN \cdot m$

NOTE!: The maximum service moment has been reduced to account for the staged construction of the coping. 50% of the erection stage moments - carried by the lower reinforcement - have been subtracted. The full amount of the erection stage moments have not been subtrated given that creep effects will transfer loads to the permanent stage.

Calculate the maximum stress in the reinforcement (IN LAYER 1) at SLS:

$$f_s := \frac{M_S}{A_{f1} \cdot \left(d_1 - \frac{C}{3}\right) + A_{f2} \cdot \frac{d_2 - C}{d_1 - C} \cdot \left(d_2 - \frac{C}{3}\right) + A_{f3} \cdot \frac{d_3 - C}{d_1 - C} \cdot \left(d_3 - \frac{C}{3}\right)}$$

$f_s = 132\,MPa$

Check that stress in reinforcement does not exceed limit, f_{sa}:

Crack width parameter $Z := 30000 \cdot \frac{N}{mm}$

Depth of concrete from extreme tensile fiber to center of bar $d_c := 150mm$

Area of concrete with same centoid per bar $A := \frac{(b_f - 2 \cdot h5) \cdot d_c \cdot 2}{n_{bars}}$

$$f_{sa} := \begin{array}{|l} f_{sa} \leftarrow \frac{Z}{(d_c \cdot A)^{\frac{1}{3}}} \\ f_{sa} \text{ if } f_{sa} < 170MPa \\ 170 \cdot MPa \text{ otherwise} \end{array}$$

$f_{sa} = 170\,MPa$

$$Stress_{Check} := \begin{array}{|l} \text{"OK" if } f_s \le f_{sa} \\ \text{"NOT SATISFIED" otherwise} \end{array}$$

$Stress_{Check} = \text{"OK"}$

Calculate forces in section to check calculation result:

Total force in rebar

$$T_S := \left(A_{f1} + A_{f2} \cdot \frac{d_2 - C}{d_1 - C} + A_{f3} \cdot \frac{d_3 - C}{d_1 - C}\right) \cdot f_s$$

$T_S = 4697\,kN$

Stress in concrete

$$f_{sc} := f_s \cdot \frac{C}{d_1 - C} \cdot \frac{1}{\alpha}$$

$f_{sc} = 8.1\,MPa$

Force in concrete

$$C_C := f_{sc} \cdot \frac{b \cdot C}{2}$$

$C_C = 4697\,kN$

Design for Shear (AASHTO LRFD Section 5.8)

Critical section for shear shall be taken as d_v from the internal face of the support (AASHTO LRFD Article 5.8.3.2)

$$d_v := \begin{cases} 0.9 \cdot d_e & \text{if } 0.9 \cdot d_e > 0.72 \cdot h_s \\ 0.72 \cdot h_s & \text{otherwise} \end{cases}$$

$$d_v = 2323.8\,\text{mm}$$

Shear in coping at critical section due to loads applied on coping body

Coping self weight	$V_c := w_c \cdot \left(\max\left(h1_c, h1_s\right) + h2 + \frac{h3}{2} - \frac{D}{2} \cdot .8 - d_v \right)$	$V_c = 173.6\,\text{kN}$
Coping cantilever self weight	$V_{cw} := w_{cw} \cdot h4$	$V_{cw} = 25.9\,\text{kN}$
Railing weight	$V_r := W_r$	$V_r = 7.4\,\text{kN}$
Superimposed dead load on coping	$V_{sdl} := w_{sdl} \cdot \left(5.75\text{m} - \frac{D \cdot .8}{2} - d_v \right)$	$V_{sdl} = 10.8\,\text{kN}$
D live load on coping	$V_d := w_d \cdot \left(5.75\text{m} - \frac{D \cdot .8}{2} - d_v \right)$	$V_d = 34.6\,\text{kN}$

Ultimate shear force in coping from max bearing reactions at face of column:

PC deck reaction	$V_{pc} := V2_{upc}$	$V_{pc} = 3231.4\,\text{kN}$
Steel deck reaction	$V_{st} := V2_{ust}$	$V_{st} = 3505.5\,\text{kN}$

Total shear force in coping at face of column - Ultimate Limit State

Shear from loads on coping body

$$V_{CU} := \left(V_c + V_{cw} + V_r\right) \cdot 1.3 + V_{sdl} \cdot 2.0 + \left(V_d\right) \cdot 1.8$$

$$V_{CU} = 353.0\,\text{kN}$$

Shear from max loads on bearings

$$V_{BU} := V_{pc} + V_{st}$$

$$V_{BU} = 6736.9\,\text{kN}$$

Total shear force

$$V_U := V_{CU} + V_{BU}$$

$$V_U = 7090\,\text{kN}$$

Calculate nominal shear resistance of concrete section assuming beam section:

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

$$V_c = 3064\,\text{kN}$$

Strength reduction factor for shear:

$$\Phi_s = 0.7$$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \begin{vmatrix} V_s \leftarrow \dfrac{V_U}{\Phi_s} - V_c \\ V_s \ \text{ if } \ V_s > 0\text{kN} \\ 0\text{kN} \ \ \text{otherwise} \end{vmatrix}$$

$$V_s = 7065\,\text{kN}$$

Check that total required nominal shear resistance does not exceed limit:

$$V_{nlimit} := 0.25 \cdot f_c \cdot b \cdot d_v \qquad V_{nlimit} = 25271.3\,\text{kN}$$

$$\text{CHECK} := \begin{vmatrix} \text{"OK"} \ \text{ if } \ V_c + V_s \le V_{nlimit} \\ \text{"FAIL"} \ \ \text{otherwise} \end{vmatrix} \qquad \text{CHECK} = \text{"OK"}$$

Provide 19mmϕ shear links wth 4 legs across the section

$$\phi_{link} := 19\text{mm}$$

$$A_v := \pi \frac{\phi_{link}^{\,2}}{4} \cdot 4 \qquad A_v = 1134\,\text{mm}^2$$

Determine required spacing of transverse reinforcement

$$s_t := \begin{vmatrix} s_{t1} \leftarrow \dfrac{A_v \cdot f_y}{0.083 \cdot \sqrt{\dfrac{f_c}{\text{MPa}}} \cdot \text{MPa} \cdot b} \\ s_{t2} \leftarrow \dfrac{A_v \cdot f_y \cdot d_v}{V_s} \ \text{ if } \ V_s > 0\text{kN} \\ s_{t2} \ \text{ if } \ (V_s > 0\text{kN}) \cdot (s_{t2} \le s_{t1}) \\ s_{t1} \ \ \text{otherwise} \end{vmatrix} \qquad s_t = 145\,\text{mm}$$

Calculate shear stress in order to determine maximum spacing of transverse reinforcement to resist shear:

$$v_u := \frac{V_U}{\Phi_s \cdot (b \cdot d_v)} \qquad v_u = 3.006\,\text{MPa}$$

Calculate spacing taking into account maximum spacing requirements:

$$s_{max} := \begin{vmatrix} \min(600\text{mm}, 0.8 d_v) \ \text{ if } \ v_u < 0.125 f_c \\ \min(300\text{mm}, 0.4 d_v) \ \ \text{otherwise} \end{vmatrix}$$

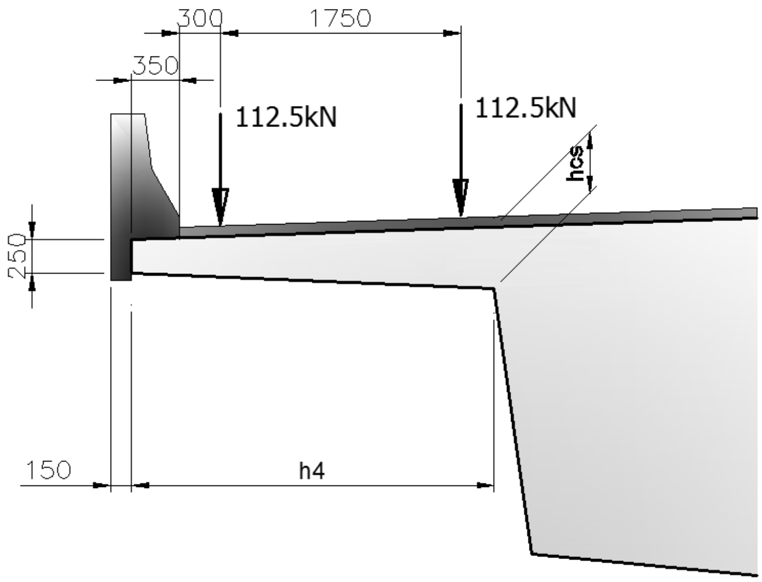
$$s_{max} = 600\,\text{mm}$$

Determine maximum required spacing of transverse reinforcement:

$$s_t := \begin{vmatrix} s_{max} \ \text{ if } \ s_{max} \le s_t \\ s_t \ \ \text{otherwise} \end{vmatrix} \qquad s_t = 145\,\text{mm}$$

<u>Provide 19mmϕ links at 100c/c in outer legs and 19mmϕ links at 200c/c in inner legs - giving 150mm c/c spacing overall</u>

Design of Cantilever Slab



Equivalent Width of Deck Overhang (AASHTO LRFD Table 4.6.2.1.3-1)

Width of coping slab at support:

$b := b_f - 2 \cdot h5$ $\qquad b = 1400\,\text{mm}$

Distance from outermost load to point of support:

$X := (h4 - 300\text{mm} - 350\text{mm})$ $\qquad X = 1510\,\text{mm}$

Equivalent width of deck overhang:

$$b_{ew} := \left| \begin{array}{l} b_{ew} \leftarrow 570\text{mm} + 0.416 \cdot X \\ b_{ew} \ \text{ if } \ b_{ew} \le b \\ b \ \text{ otherwise} \end{array} \right. \qquad b_{ew} = 1198\,\text{mm}$$

Check Deflection of Cantilever Slab (AASHTO LRFD Article 2.5.2.6.2)

Depth of cantilever slab in main deck section at support:

$h_{ds} := 450\text{mm}$

Span length of cantilever slab in main deck section:

$l_{ds} := 2645\text{mm}$

Depth of deck overhang at support to match main deck outline:

$$h_{cs1} := \left(h_{ds} - 250\text{mm}\right) \cdot \frac{h4}{l_{ds}} + 250\text{mm} \qquad h_{cs1} = 413\,\text{mm}$$

Depth of deck overhang at outermost load point:

$$h_{cs2} := \left(h_{ds} - 250\text{mm}\right) \cdot \frac{350\text{mm} + 300\text{mm}}{l_{ds}} + 250\text{mm} \qquad h_{cs2} = 299\,\text{mm}$$

Depth of deck overhang at innermost load point:

$$h_{cs3} := \left| \begin{array}{l} h_{cs3} \leftarrow (h_{ds} - 250mm)\cdot\dfrac{350mm + 300mm + 1750mm}{l_{ds}} + 250mm \\ h_{cs3} \ \text{ if } \ h_{cs3} \le h_{cs1} \\ h_{cs1} \ \text{ otherwise} \end{array} \right.$$

$h_{cs3} = 413\,mm$

Distance from innermost load to point of support:

$$X3 := \left| \begin{array}{l} X3 \leftarrow h4 - 300mm - 350mm - 1750mm \\ X3 \ \text{ if } \ X3 > 0 \\ 0m \ \text{ otherwise} \end{array} \right.$$

$X3 = 0\,mm$

Moment of inertia of equivalent width of deck overhang assuming cracked section:

at support $I_{ew1} := 40\cdot\%\cdot\dfrac{b_{ew}\cdot h_{cs1}^{3}}{12}$

at outer load point $I_{ew2} := 40\cdot\%\cdot\dfrac{b_{ew}\cdot h_{cs2}^{3}}{12}$

at innermost load point $I_{ew3} := 40\cdot\%\cdot\dfrac{b_{ew}\cdot h_{cs3}^{3}}{12}$

The deflection under the applied wheel loads is then given by (increaseed by 30% to account for dynamic loading):

$$\delta_T := \frac{112.5\cdot kN\cdot 130\%}{E_c}\cdot\left[\int_0^X \left[\frac{x^2}{I_{ew2} + (I_{ew1} - I_{ew2})\cdot\frac{x}{X}}\right] dx + \int_0^{X3} \left[\frac{x^2}{I_{ew3} + (I_{ew1} - I_{ew3})\cdot\frac{x}{X3}}\right] dx\right]$$

$\delta_T = 2.61\,mm$

Check that the deflection does not exceed the limit for vehicular load on cantilever arms:

$\delta_{LIMIT} := \dfrac{X}{300}$ $\qquad \delta_{LIMIT} = 5.03\,mm$

$$DEFLECTION_{CHECK} := \left| \begin{array}{l} \text{"OK"} \ \text{ if } \ \delta_T \le \delta_{LIMIT} \\ \text{"FAIL"} \ \text{ otherwise} \end{array} \right.$$

$DEFLECTION_{CHECK} = \text{"OK"}$

Design for Flexure (AASHTO LRFD Section 5.7)

Nominal bending moment in slab at support

Coping cantilever self weight $M_{cw} := w_{cw}\cdot\dfrac{h4^2}{2}$ $\qquad M_{cw} = 28.0\,kN\cdot m$

Railing weight $M_r := W_r \cdot (h4 + 0.15m - 0.25m)$ $M_r = 15.3\,kN\cdot m$

Superimposed dead load on coping $M_{sdl} := w_{sdl} \cdot \frac{(h4 - 350mm)^2}{2}$ $M_{sdl} = 6.4\,kN\cdot m$

T live load on coping $M_t := 112.5 \cdot kN \cdot (X + X3)$ $M_t = 169.9\,kN\cdot m$

Total bending moment in slab at face of support - Ultimate Limit State

$$M_{SU} := (M_{cw} + M_r)\cdot 1.3 + M_{sdl}\cdot 2.0 + (M_t \cdot 1.3)\cdot 1.8$$

$$M_{SU} = 466.7\,kN\cdot m$$

Depth of section:

$h_{cs} := h_{cs1}$ $h_{cs} = 413\,mm$

Effective depth of slab:

$d_e := (h_{cs} - 90mm)$ $d_e = 323\,mm$

Effective width of coping slab at support:

$b_{ew} = 1198\,mm$

Strength reduction factor for flexure: $\Phi = 0.8$

Determine area of reinforcement, A_f, in the coping to resist flexure
(ref AASHTO LFRD Article 5.7.3.2.3 - see notes on flexural design for rearrangement of terms):

$R := \frac{M_{SU}}{\Phi \cdot b_{ew} \cdot d_e^2 \cdot f_y}$ $R = 0.0119$ $M := \frac{0.85 \cdot f_c}{f_y}$ $M = 0.0654$

$\rho := M \cdot \left(1 - \sqrt{1 - \frac{2\cdot R}{M}}\right)$ $\rho = 0.013294$

$A_f := \rho \cdot d_e \cdot b$ $b = 1400\,mm$

$A_f = 6017.5\,mm^2$

Using 32mmϕ bars gives total number of bars to be distributed across the coping:

$n_{bf} := \frac{A_f}{804 \cdot mm^2}$ $n_{bf} = 7.5$

Provide 10 No 32mmϕ bars

$n_{bars} := 10$ $A_f := n_{bars} \cdot 804 \cdot mm^2$ $A_f = 8040\,mm^2$

Calculate the stress block factor, β_1:

$$\beta_1 := \begin{vmatrix} \beta_1 \leftarrow 0.85 \\ \beta_1 \leftarrow \beta_1 - 0.05 \cdot \dfrac{f_c - 28\text{MPa}}{7.\text{MPa}} \quad \text{if } f_c > 28\text{MPa} \\ 0.65 \quad \text{if } \beta_1 < 0.65 \end{vmatrix}$$

$$\beta_1 = 0.836$$

For rectangular sections, the depth of concrete in compression, c, is given by:

$$c := \frac{A_f \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} \qquad c = 105\,\text{mm}$$

Check that the required amount of reinforcement does not exceed the maximum allowed by the code:

$$\frac{c}{d_e} = 0.33$$

$$Max_{Limit} := \begin{vmatrix} \text{"OK"} \quad \text{if } \dfrac{c}{d_e} \le 0.42 \\ \text{"EXCEEDED"} \quad \text{otherwise} \end{vmatrix} \qquad Max_{Limit} = \text{"OK"}$$

Calculate the modulus of rupture, f_r ,of the concrete:

$$f_r := 0.63 \cdot \sqrt{\frac{f_c}{\text{MPa}}} \cdot \text{MPa} \qquad f_r = 3.451\ \text{MPa}$$

Section modulus for the extreme fiber of the section, assuming rectangular section of width b:

$$S_c := \frac{b \cdot h_{cs}^{\ 2}}{6} \qquad S_c = 0.04\,\text{m}^3$$

The cracking moment, M_{cr} ,is the given by:

$$M_{cr} := S_c \cdot f_r \qquad M_{cr} = 138\,\text{kN}\cdot\text{m}$$

Check that the reinforcement can develop a resistance moment Mr at least equal to the lesser of:

- 1.2 times the cracking momen Mcr
- 1.33 times the factored moment required by the applicable strength load combination

$$Minimum_{Steel} := \begin{vmatrix} M_r \leftarrow \dfrac{M_{SU}}{\Phi} \\ M \leftarrow \min\left(1.2M_{cr}, 1.33 \cdot M_{EU}\right) \\ \text{"OK"} \quad \text{if } M_r \ge M \\ \text{"NOT SATISFIED"} \quad \text{otherwise} \end{vmatrix} \qquad Minimum_{Steel} = \text{"OK"}$$

CHECK STRESSES AT SLS (AASHTO LRFD Article 5.7.3.4)

The depth of concrete in compression, C, at the serviceability limit state, assuming one level of rebar, is given by:

$$C := \frac{\sqrt{A_f^2 + \frac{2 \cdot b \cdot A_f \cdot d_e}{\alpha}} - A_f}{b} \cdot \alpha \qquad C = 128\,\text{mm}$$

Calculate lever arm, z:

$$z := d_e - \frac{C}{3} \qquad z = 281\,\text{mm}$$

Total bending moment in slab at face of support - Service Limit State

$$M_{SS} := (M_{cw} + M_r) \cdot 1.0 + M_{sdl} \cdot 1.0 + (M_t \cdot 1.3) \cdot 1.0$$

$$M_{SS} = 270.6\,\text{kN} \cdot \text{m}$$

Calculate the maximum stress in the reinforcement at SLS:

$$f_s := \frac{M_{SS}}{A_f \cdot z} \qquad f_s = 120\,\text{MPa}$$

Check that stress in reinforcement does not exceed limit, f_{sa}:

Crack width parameter $\qquad Z := 30000 \cdot \frac{N}{mm}$

Depth of concrete from extreme tensile fiber to center of bar $\qquad d_c := 90mm$

Area of concrete with same centoid per bar $\qquad A := \frac{b \cdot d_c \cdot 2}{n_{bars}}$

$$f_{sa} := \left| \begin{array}{l} f_{sa} \leftarrow \dfrac{Z}{(d_c \cdot A)^{\frac{1}{3}}} \\ f_{sa} \ \text{ if } \ f_{sa} < 170MPa \\ 170 \cdot MPa \ \text{ otherwise} \end{array} \right. \qquad f_{sa} = 170\,\text{MPa}$$

$$Stress_{Check} := \left| \begin{array}{l} \text{"OK"} \ \text{ if } \ f_s \le f_{sa} \\ \text{"NOT SATISFIED"} \ \text{ otherwise} \end{array} \right. \qquad Stress_{Check} = \text{"OK"}$$

Calculate forces in section to check calculation result:

Total force in rebar

$$T_S := (A_f) \cdot f_s \qquad T_S = 964\,\text{kN}$$

Stress in concrete

$$f_{sc} := f_s \cdot \frac{C}{d_e - C} \cdot \frac{1}{\alpha} \qquad f_{sc} = 10.8\,\text{MPa}$$

Force in concrete

$$C_C := f_{sc} \cdot \frac{b \cdot C}{2} \qquad C_C = 964\,\text{kN}$$

Design for Shear (AASHTO LRFD Section 5.8)

Critical section for shear shall be taken as d_v from the internal face of the support (AASHTO LRFD Article 5.8.3.2)

$$d_v := \begin{cases} 0.9 \cdot d_e & \text{if } 0.9 \cdot d_e > 0.72 \cdot h_{cs} \\ 0.72 \cdot h_{cs} & \text{otherwise} \end{cases}$$

$$d_v = 298\,\text{mm}$$

Shear in coping slab at critical section due to loads applied on coping body

Coping cantilever self weight	$V_{cw} := w_{cw} \cdot (h4 - d_v)$	$V_{cw} = 22.4\,\text{kN}$
Railing weight	$V_r := W_r$	$V_r = 7.4\,\text{kN}$
Superimposed dead load on coping	$V_{sdl} := w_{sdl} \cdot (h4 - 600\text{mm} - d_v)$	$V_{sdl} = 5.0\,\text{kN}$
T live load on coping	$V_t := 112.5 \cdot \text{kN}$	$V_t = 112.5\,\text{kN}$

Total shear force in coping slab at face of support - Ultimate Limit State

Shear from loads on coping body

$$V_{SU} := (V_{cw} + V_r) \cdot 1.3 + V_{sdl} \cdot 2.0 + (V_t \cdot 1.4) \cdot 1.8$$

$$V_{SU} = 332.2\,\text{kN}$$

Calculate nominal shear resistance of concrete section assuming beam section:

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

$$V_c = 379\,\text{kN}$$

Strength reduction factor for shear:

$$\Phi_s = 0.7$$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \begin{vmatrix} V_s \leftarrow \dfrac{V_{SU}}{\Phi_s} - V_c \\ V_s \quad \text{if } V_s > 0\text{kN} \\ 0\text{kN} \quad \text{otherwise} \end{vmatrix} \qquad V_s = 96\,\text{kN}$$

Check that total required nominal shear resistance does not exceed limit:

$$V_{nlimit} := 0.25 \cdot f_c \cdot b \cdot d_v \qquad V_{nlimit} = 3124.8\,\text{kN}$$

$$\text{CHECK} := \begin{cases} \text{"OK"} & \text{if } V_c + V_s \le V_{nlimit} \\ \text{"FAIL"} & \text{otherwise} \end{cases} \qquad \text{CHECK} = \text{"OK"}$$

Provide 13mmϕ shear links with 4 legs across the section

$$\phi_{link} := 13mm$$

$$A_v := \pi \frac{\phi_{link}^2}{4} \cdot 4 \qquad A_v = 531\,mm^2$$

Determine required spacing of transverse reinforcement

$$s_t := \left| \begin{array}{l} s_{t1} \leftarrow \dfrac{A_v \cdot f_y}{0.083 \cdot \sqrt{\dfrac{f_c}{MPa}} \cdot MPa \cdot b} \\ s_{t2} \leftarrow \dfrac{A_v \cdot f_y \cdot d_v}{V_s} \quad \text{if } V_s > 0kN \\ s_{t2} \quad \text{if } (V_s > 0kN) \cdot (s_{t2} \le s_{t1}) \\ s_{t1} \quad \text{otherwise} \end{array} \right. \qquad s_t = 644\,mm$$

Calculate shear stress in order to determine maximum spacing of transverse reinforcement to resist shear:

$$v_u := \frac{V_{SU}}{\Phi_s \cdot (b \cdot d_v)} \qquad v_u = 1.139\,MPa$$

Calculate spacing taking into account maximum spacing requirements:

$$s_{max} := \left| \begin{array}{l} \min(600mm, 0.8d_v) \quad \text{if } v_u < 0.125 f_c \\ \min(300mm, 0.4d_v) \quad \text{otherwise} \end{array} \right.$$

$$s_{max} = 238\,mm$$

Determine maximum required spacing of transverse reinforcement:

$$s_t := \left| \begin{array}{l} s_{max} \quad \text{if } s_{max} \le s_t \\ s_t \quad \text{otherwise} \end{array} \right.$$

$$s_t = 238\,mm$$

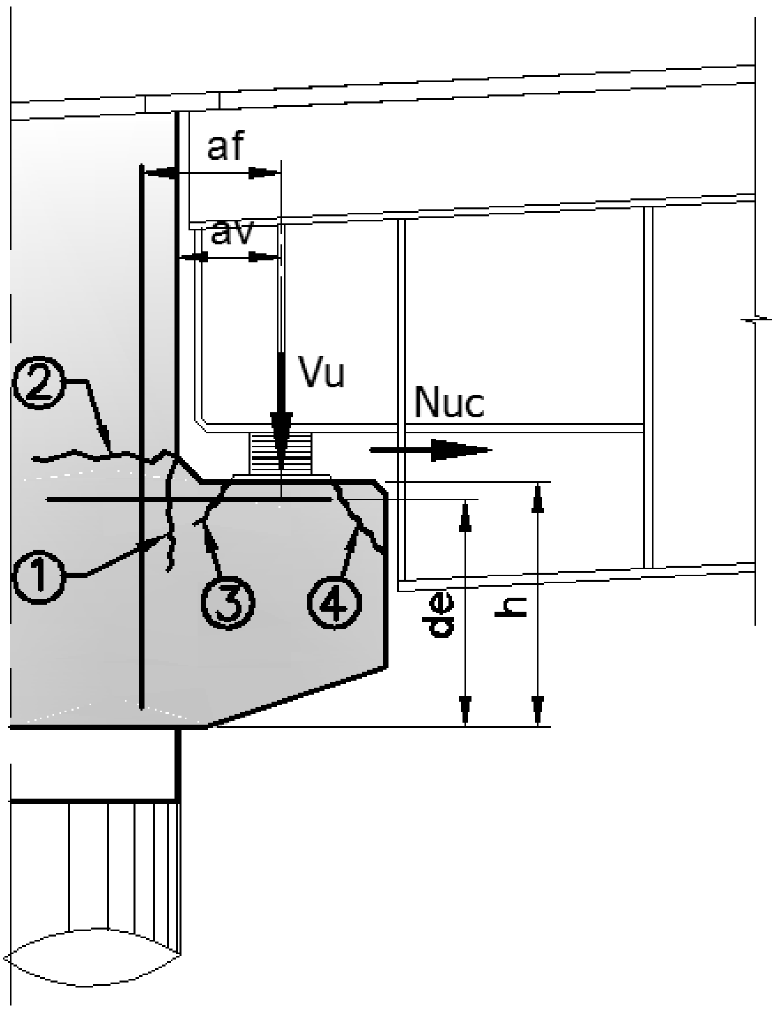
<u>Provide 13mmϕ links at 150c/c</u>

Beam Ledge Design AASHTO LRFD Article 5.13.2.5

General, Article 5.13.2.5.1

As illustrated below, beam ledges shall resist:

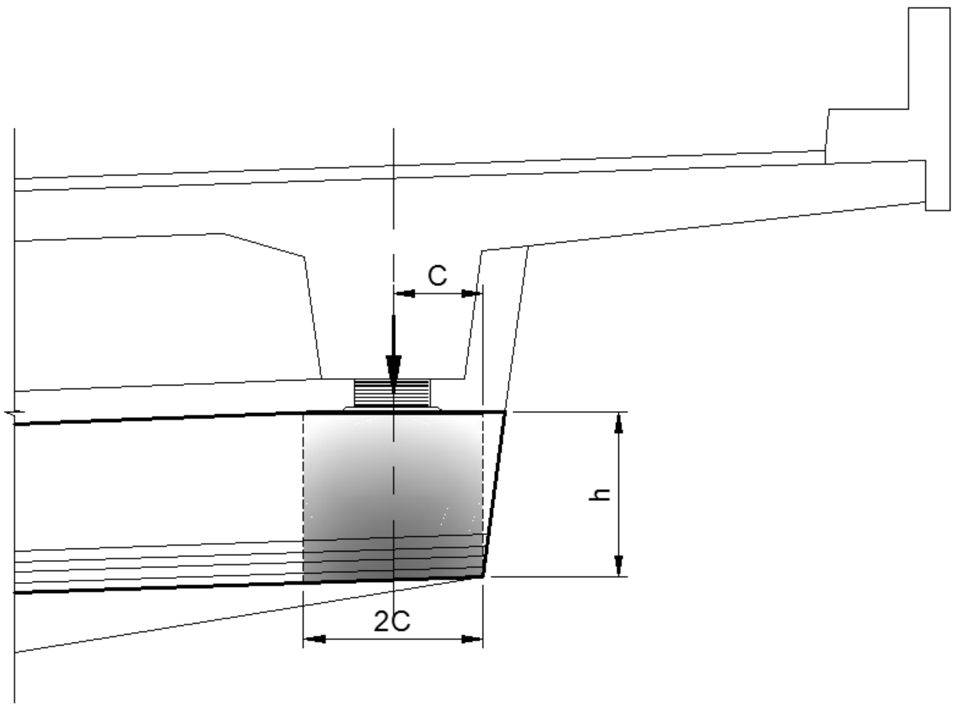
- Flexure, shear and horizontal forces at the loaction of Crack 1;
- Tension force in the supporting element at the location of Crack 2;
- Punching shear at points of loading at the location of Crack 3; and
- Bearing forces at the location of Crack 4



Design for Shear, Article 5.13.2.5.2

Design of beam ledges for shear shall be in accordance with the requirements of shear friction in Article 5.8.4.

The width of the concrete face assumed to participate shall not exceed the width ilustrated below:



Edge distance of bearing	$C := h2$	$C = 800\,mm$
Depth of beam ledge	$h := v3$	$h = 1200\,mm$
Effective depth	$d_e := h - 65mm$	$d_e = 1135\,mm$
Width of the interface	$b_v := 2 \cdot C$	$b_v = 1600\,mm$
Area of concrete resisting shear transfer	$A_{cv} := b_v \cdot d_e$	$A_{cv} = 1.816\ m^2$

Loads on Ledge

Ultimate shear force in coping from max bearing reactions at face of column:

PC deck reaction $V_{upc} := V2_{upc}$ $V_{upc} = 3231.4\,kN$

Steel deck reaction $V_{ust} := V2_{ust}$ $V_{ust} = 3505.5\,kN$

Maximum design shear force

$V_u := \max(V_{upc}, V_{ust})$ $V_u = 3506\,kN$

Calculate limiting nominal shear strength

$$V_{nlimit} := \begin{cases} 0.2 \cdot f_c \cdot A_{cv} & \text{if } 0.2 \cdot f_c \cdot A_{cv} < 5.5 \cdot \dfrac{A_{cv}}{mm^2} \cdot N \\ 5.5 \cdot \dfrac{A_{cv}}{mm^2} \cdot N & \text{otherwise} \end{cases}$$

$$V_{nlimit} = 9988\,kN$$

Calculate required nominal shear strength and check beam ledge depth

$$V_n := \frac{V_u}{\Phi_s} \qquad V_n = 5007.9\,kN$$

$$Beam_{Ledge} := \begin{cases} \text{"OK"} & \text{if } V_n \le V_{nlimit} \\ \text{"INADEQUATE"} & \text{otherwise} \end{cases} \qquad Beam_{Ledge} = \text{"OK"}$$

Calculate shear friction reinforcement, A_{vf}

Cohesion and friction values for monolithically cast concrete

$$c := 1.0MPa$$

$$\lambda := 1.00$$

$$\mu := 1.4 \cdot \lambda$$

Shear reinforcement is then given by:

$$A_{vf} := \left| \begin{array}{l} A_{vf1} \leftarrow \dfrac{V_n - c \cdot A_{cv}}{f_y \cdot \mu} \\ A_{vf2} \leftarrow 0.35 \cdot \dfrac{b_v}{mm} \cdot \dfrac{MPa}{f_y} \cdot mm^2 \\ A_{vf} \leftarrow A_{vf1} \ \text{ if } A_{vf1} > A_{vf2} \\ A_{vf} \leftarrow A_{vf2} \ \text{ if } A_{vf2} > A_{vf1} \\ 0 \ \text{ if } \dfrac{V_n}{A_{cv}} < 0.7MPa \end{array} \right. \qquad \frac{V_n}{A_{cv}} = 2.758\,MPa$$

$$A_{vf} = 5845.9\,mm^2$$

Design for Flexure and Horizontal Force, Article 5.13.2.5.3

The area of total primary tension reinforcement shall satisfy the requirements of Article 5.13.2.4.2.

The primary tension reinforcement shall be spaced uniformly with the region 2C.

The section at the face of the support shall be designed to resist simultaneously a factored shear force V_u, a factored moment M_u and a concurrent factored horizontal tensile force N_{uc}.

N_{uc} shall not be taken to be less than $0.2V_u$ and shall be regarded as a live load, even where it results from creep, shrinkage or temperature change.

These provisions apply to beam ledges:

- with a shear span-to-depth ratio av/d_e not greater than unity
- subject to a horizontal tensile force Nuc not larger than Vu

The depth at outside edge of bearing shall not be less than 0.5de, where d_e is effective depth.

Horizontal tensile force $N_{uc} := 0.2 \cdot V_u$ $N_{uc} = 701.1\ \mathrm{kN}$

Shear span $a_v := \dfrac{h5}{2}$ $a_v = 488\ \mathrm{mm}$

Design Moment $M_u := V_u \cdot a_v + N_{uc} \cdot (h - d_e)$ $M_u = 1754.5\ \mathrm{kN \cdot m}$

Design the primary tensile force reinforcement A_s:

Strength reduction factor for bending $\Phi = 0.8$

Width of section $b := 2 \cdot C$ $b = 1600\ \mathrm{mm}$

Determine area of primary reinforcement, A_s, to resist flexure
(ref AASHTO LFRD Article 5.7.3.2.3 - see notes on flexural design for rearrangement of terms):

$$R := \frac{M_u}{\Phi \cdot b \cdot d_e^{\,2} \cdot f_y} \qquad R = 0.0027 \qquad M := \frac{0.85 \cdot f_c}{f_y} \qquad M = 0.0654$$

$$\rho := M \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R}{M}}\right) \qquad \rho = 0.002788$$

$$A_s := \rho \cdot b \cdot d_e \qquad A_s = 5062.5\ \mathrm{mm^2}$$

Design the tensile force reinforcement A_n:

$$A_n := \frac{N_{uc}}{\Phi \cdot f_y} \qquad A_n = 2247\ \mathrm{mm^2}$$

Check that the Area of Primary Reinforcement, A_s, satisfies code requirements:

$$A_s := \begin{cases} A_s & \text{if } A_s > \frac{2}{3}\cdot A_{vf} + A_n \\ \frac{2}{3}\cdot A_{vf} + A_n & \text{otherwise} \end{cases}$$

$$A_s := \begin{cases} 0.04\cdot\frac{f_c}{f_y}\cdot b\cdot d_e & \text{if } A_s < 0.04\cdot\frac{f_c}{f_y}\cdot b\cdot d_e \\ A_s & \text{otherwise} \end{cases}$$

$b = 1.6\ m$

$A_s = 6144\,mm^2$

$\frac{A_s}{201mm^2} = 30.569$

$A_s := \frac{A_s}{b}$ $\qquad A_s = 3.84\,\frac{mm^2}{mm}$

check area of steel required

$\rho_{REQUIRED} := \frac{A_s\cdot m}{b\cdot d_e}\cdot 100 \qquad \rho_{REQUIRED} = 0.21$ PERCENT

Determine area of closed stirrups or ties, Ah:

$A_h := 0.5\cdot\left(A_s\cdot b - A_n\right) \qquad A_h = 1949\,mm^2$

This reinforcement shall be uniformly distributed within two-thirds of the effective depth of the beam ledge adjacent to As.

Anchorage of primary reinforcement:

At the front face of the beam ledge, the primary tension reinforcement, As, shall be anchored by one of the following:

a) a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength fy of As bars

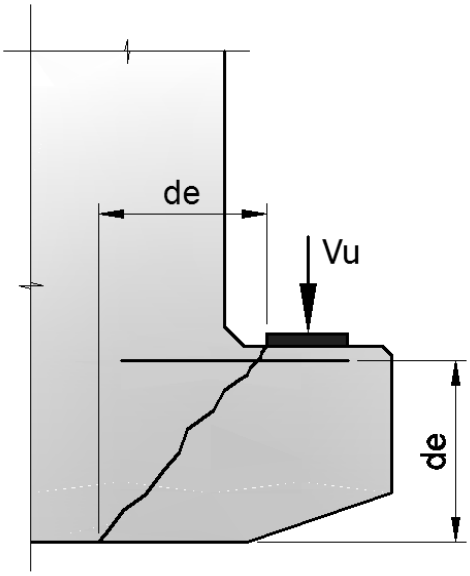
b) bending primary tension bars As back to form a horizontal loop

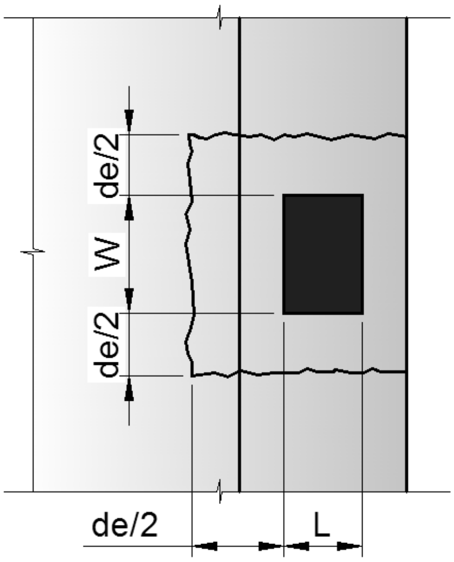
c) some other means of positive anchorage

The bearing area of load on the bracket or corbel shall not project beyond interior face of transverse anchor bar (if one is provided).

Design for Punching Shear, Article 5.13.2.5.4

The truncated pyramids assumed as failure surfaces for punching shear, as illustrated below, shall not overlap.





Applied ultimate reaction	$V_u = 3506\,kN$
Width of bearing	$L = 620\,mm$
Length of bearing	$W = 800\,mm$
Effective depth	$d_e = 1135\,mm$
Bearing pad spacing	$S := \min(h1_c, h1_s) \cdot 2$ $\quad S = 6150\,mm$

Nominal punching shear resistance, V_n, shall be taken as:

- At interior pads, or exterior pads, where the end distance C is greater than half the pad spacing S/2:

$$V_{n1} := 0.328 \cdot \sqrt{\frac{f_c}{MPa}} \cdot MPa \cdot (W + 2 \cdot L + 2 \cdot d_e) \cdot d_e \qquad V_{n1} = 8788\,kN$$

- At exterior pads where the end distance C is less than half the pad spacing S/2 and C-0.5W is less than d_e:

$$V_{n2} := 0.328 \cdot \sqrt{\frac{f_c}{MPa}} \cdot MPa \cdot (W + L + d_e) \cdot d_e \qquad V_{n2} = 5210\,kN$$

- At exterior pads where the end distance C is less than half the pad spacing S/2 but C-0.5W is greater than d_e:

$$V_{n3} := 0.328 \cdot \sqrt{\frac{f_c}{MPa}} \cdot MPa \cdot (0.5W + L + d_e + C) \cdot d_e \qquad V_{n3} = 6025\,kN$$

Determine the nominal punching resistance:

$$V_n := \begin{cases} V_{n1} & \text{if } C \geq \dfrac{S}{2} \\ V_{n2} & \text{if } \left(C < \dfrac{S}{2}\right) \cdot \left[(C - 0.5 \cdot W) \leq d_e\right] \\ V_{n3} & \text{otherwise} \end{cases}$$

$$V_n = 5210\,kN$$

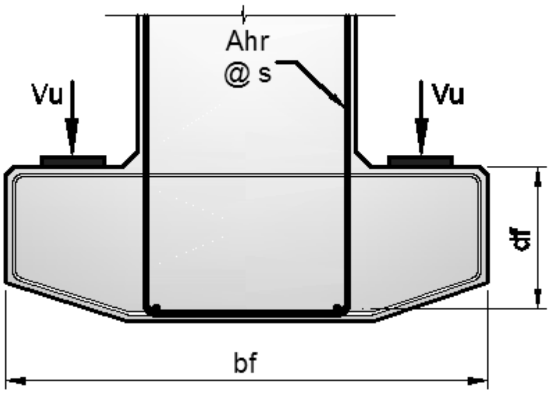
Check that the nominal punching resistance is adequate:

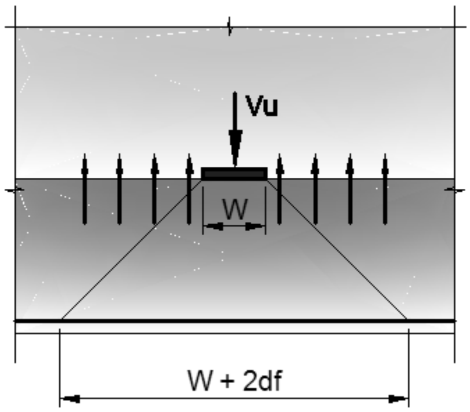
$$PunchingShear_{CHECK} := \begin{cases} \text{"SATISFIED"} & \text{if } V_n \cdot \Phi_s \geq V_u \\ \text{"FAIL"} & \text{otherwise} \end{cases}$$

$$PunchingShear_{CHECK} = \text{"SATISFIED"}$$

<u>Design for Hanger Reinforcement, Article 5.13.2.5.5</u>

Hanger reinforcement specified herein shall be provided in addition to the lesser shear reinforcement required on either side of the beam reaction being supported.





The distance from the top of the ledge to the compression reinforcement as illustrated above is d_f.

$b_f = 3350\,\text{mm}$ $\quad d_f := d_e - 40\text{mm}$ $\quad d_f = 1095\,\text{mm}$

The nominal shear resistance of ledges of inverted T-beams shall be the lessor of the following:

$$V_{N1}(A_{hr}, s) := \frac{A_{hr} \cdot f_y}{s} \cdot S \qquad \text{(Equation 1)}$$

$$V_{N2}(A_{hr}, s) := 0.165 \cdot \sqrt{\frac{f_c}{\text{MPa}}} \cdot \text{MPa} \cdot b_f \cdot d_f + \frac{A_{hr} \cdot f_y}{s} \cdot (W + 2d_f) \qquad \text{(Equation 2)}$$

In the case of the applied design, the edge distance between the exterior bearing pad and the end of the shelf is less than d_f. Equation 2 above is therefore modified as shown below:

$$V_{N2}(A_{hr}, s) := 0.165 \cdot \sqrt{\frac{f_c}{\text{MPa}}} \cdot \text{MPa} \cdot b_f \cdot d_f + \frac{A_{hr} \cdot f_y}{s} \cdot (2 \cdot C) \qquad \text{(Equation 3)}$$

Try 25mm ϕ rebar at 100mm c/c

Area of one leg of hanger reinforcement A_{hr} and spacing s are then:

$A_{hr} := \frac{\pi \cdot (25\text{mm})^2}{4}$ $\quad s := 100\text{mm}$

Total number of bars required:

$n_{bars} := \frac{2 \cdot C}{s}$ $\quad n_{bars} = 16.0$

This gives:

$$V_{N1}(A_{hr}, s) = 11774\,\text{kN}$$

$$V_{N2}(A_{hr}, s) = 6378\,\text{kN}$$

The minimum nominal shear resistance is then given by:

$$V_n := \min\left(V_{N1}(A_{hr}, s), V_{N2}(A_{hr}, s)\right)$$

$$V_n = 6378\,\text{kN}$$

Max shear force from the bearings:

$$V_u = 3506\,kN$$

Check that the nominal shear resistance is adequate

$$Hanger_{CHECK} := \begin{cases} \text{"SATISFIED"} & \text{if } V_n \cdot \Phi_s \geq V_u \\ \text{"FAIL"} & \text{otherwise} \end{cases}$$

$$Hanger_{CHECK} = \text{"SATISFIED"}$$

<u>Check Torsional Requirements (AASHTO LRFD Section 5.8)</u>

Check the torsional moment requirements of the beam ledge assuming the bearings (on the steel deck side) are fully loaded and the opposite bearings (on the PC deck side) are loaded only with permanent load:

PC Deck Dead Load Reaction - max at the bearing

$$V2_{pc} = 672\,kN$$

Superimposed Dead Load PC Deck Reaction - max at the bearing

$$V2_{sdl_2} = 245\,kN$$

Ultimate limit state reaction in bearing under permanent load:

$$V_{upc} := V2_{pc} \cdot 1.3 + V2_{sdl_2} \cdot 2.0$$

$$V_{upc} = 1362\,kN$$

Calculate torsion in the coping:

$$T_c := \left(V_u - V_{upc}\right)\left(\frac{b_f}{2} - \frac{h5}{2}\right)$$

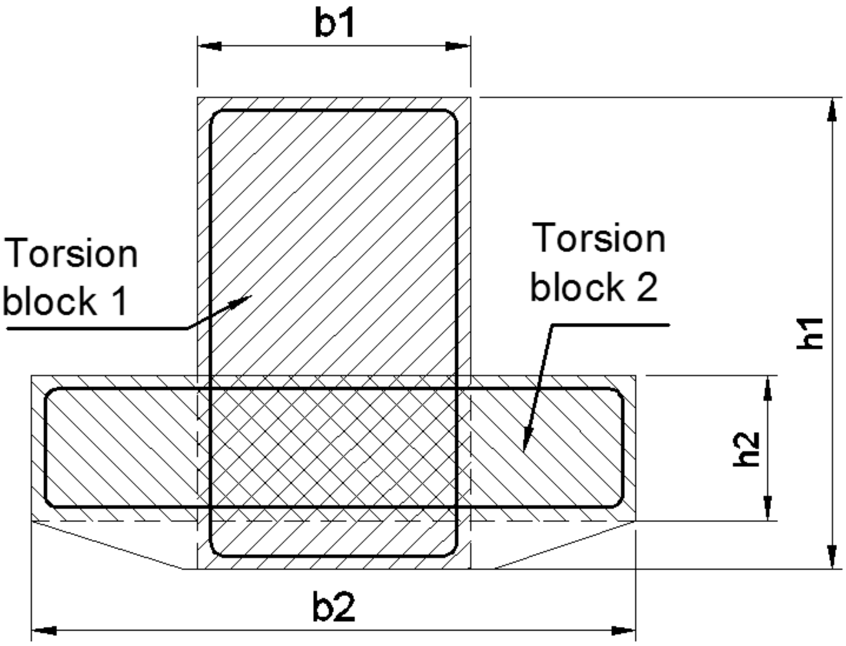
$$T_c = 2545\,kN \cdot m$$

Associated shear force:

$$V_{TU} := V_{CU} + V_{ust} + V_{upc}$$

$$V_{TU} = 5221\,kN$$

The coping will resist torsion moment with two torson blocks as illustrated below. The torsion moment will be distributed into each torsion block in accordance with the relative area of each block.



Dimensions of the torsion blocks are as follows:

Torsion block 1	$h1 := v1$	$h1 = 2732\,mm$
	$b1 := b_f - h5 \cdot 2$	$b1 = 1400\,mm$
Torsion block 1	$h2 := \frac{2}{3} \cdot v3$	$h2 = 800\,mm$
	$b2 := b_f$	$b2 = 3350\,mm$

Area enclosed with centerline of transverse rebar

Torsion block 1 $A_{oh1} := (b1 - 40mm \cdot 2 - 19mm) \cdot (h1 - 40mm \cdot 2 - 19mm)$

Torsion block 2 $A_{oh2} := (b2 - 40mm \cdot 2 - 16mm) \cdot (h2 - 40mm \cdot 2 - 16mm)$

Area enclosed by shear flow path

Torsion block 1 $A_{o1} := 0.85 A_{oh1}$

Torsion block 2 $A_{o2} := 0.85 A_{oh2}$

Calculate torsion moments carried by each block:

Torsion block 1 $T_{c1} := \frac{A_{o1}}{A_{o1} + A_{o2}} \cdot T_c$ $\quad T_{c1} = 1525\,kN \cdot m$

Torsion block 2 $T_{c2} := \frac{A_{o2}}{A_{o1} + A_{o2}} \cdot T_c$ $\quad T_{c2} = 1020\,kN \cdot m$

Transverse Reinforcement for Shear and Torsion - Torsion Block 1

Depth of section:

$$h_s := v1$$

Effective depth of coping:

$$d_e := \left(h_s - 150\text{mm}\right) \qquad d_e = 2582\,\text{mm}$$

Width of coping:

$$b := b_f - h5\cdot 2 \qquad b = 1400\,\text{mm}$$

Critical section for shear shall be taken as d_v from the internal face of the support (AASHTO LRFD Article 5.8.3.2)

$$d_v := \begin{cases} 0.9\cdot d_e & \text{if } 0.9\cdot d_e > 0.72\cdot h_s \\ 0.72\cdot h_s & \text{otherwise} \end{cases}$$

$$d_v = 2323.8\,\text{mm}$$

Calculate nominal shear resistance of concrete section assuming beam section:

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

$$V_c = 2958\,\text{kN}$$

Strength reduction factor for shear:

$$\Phi_s = 0.7$$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \begin{vmatrix} V_s \leftarrow \dfrac{V_{TU}}{\Phi_s} - V_c \\ V_s \quad \text{if } V_s > 0\text{kN} \\ 0\text{kN} \quad \text{otherwise} \end{vmatrix}$$

$$V_s = 4500\,\text{kN}$$

Provide 19mmϕ shear links wth 4 legs across the section

$$\phi_{link} := 19\text{mm}$$

$$A_v := \pi\,\frac{\phi_{link}^{\,2}}{4}\cdot 4 \qquad A_v = 1134\,\text{mm}^2$$

Determine required spacing of transverse reinforcement

$$s_{t1} := \begin{vmatrix} s_{t1} \leftarrow \dfrac{A_v\cdot f_y}{0.083\cdot\sqrt{\dfrac{f_c}{\text{MPa}}}\cdot\text{MPa}\cdot b} \\ s_{t2} \leftarrow \dfrac{A_v\cdot f_y\cdot d_v}{V_s} \quad \text{if } V_s > 0\text{kN} \\ s_{t2} \quad \text{if } \left(V_s > 0\text{kN}\right)\cdot\left(s_{t2} \le s_{t1}\right) \\ s_{t1} \quad \text{otherwise} \end{vmatrix} \qquad s_{t1} = 228\,\text{mm}$$

For reinforced concrete the angle of inclination of diagonal stresses, θ, can be assumed to be 45 degrees:

$$\theta := 45\text{deg}$$

Determine required transverse reinforcement for torsion:

Area of one leg of transverse torsion reinforcement $A_t := \pi \dfrac{\phi_{link}^2}{4}$ $A_t = 284\,\text{mm}^2$ $\phi_{link} = 19\,\text{mm}$

Required spacing of torsional reinforcement $s_{t2} := \dfrac{2 \cdot A_{o1} \cdot A_t \cdot f_y \cdot \cot(\theta)}{T_{c1}} \cdot \Phi_s$ $s_{t2} = 296\,\text{mm}$

Calculate combined spacing of shear and torsion transverse reinforcement:

$$\left(\frac{1}{s_{t1}} + \frac{1}{s_{t2}}\right)^{-1} = 128.833\,\text{mm}$$

Provide 19mm dia transverse reinforcement at 100mm c/c in main body of coping

Transverse Reinforcement for Shear and Torsion - Torsion Block 2

Provide 16mmϕ shear links wth 2 legs across the section

$$\phi_{link} := 16\text{mm}$$

$$A_v := \pi \frac{\phi_{link}^2}{4} \cdot 2 \qquad A_v = 402\,\text{mm}^2$$

Determine required transverse reinforcement for torsion:

Area of one leg of transverse torsion reinforcement $A_t := \pi \dfrac{\phi_{link}^2}{4}$ $A_t = 201\,\text{mm}^2$ $\phi_{link} = 16\,\text{mm}$

Required spacing of torsional reinforcement $s_{t2} := \dfrac{2 \cdot A_{o2} \cdot A_t \cdot f_y \cdot \cot(\theta)}{T_{c2}} \cdot \Phi_s$ $s_{t2} = 210\,\text{mm}$

Provide 16mm dia transverse reinforcement at 200mm c/c in beam ledges

Design for Loads from Deck Jacking

AASHTO LRFD requires that the beam ledge is designed to resist deck jacking forces.

The deck jacking loads shall not be less than 1.3 times the permanent load reaction at the bearing adjacent to the point of jacking.

PC Deck Dead Load Reaction - max at the bearing

$$Vdl_{pc} := \left|\frac{V2_{dl_2}}{2}\right| + \frac{\left|T_{dl_2}\right|}{h1_c \cdot 2} \qquad Vdl_{pc} = 671.5\,\text{kN}$$

Steel Deck Dead Load Reaction - max at the bearing

$$Vdl_{st} := \left|\frac{V2_{dl_3}}{2}\right| + \frac{\left|T_{dl_3}\right|}{h1_s \cdot 2} \qquad Vdl_{st} = 752.9\,\text{kN}$$

PC Deck Superimposed Dead Load Reaction - max at the bearing

$$Vsdl_{pc} := \left|\frac{V2_{sdl_2}}{2}\right| + \frac{\left|T_{sdl_2}\right|}{h1_c \cdot 2} \qquad Vsdl_{pc} = 122.3\,\text{kN}$$

Steel Deck Superimposed Dead Load Reaction - max at the bearing

$$Vsdl_{st} := \left|\frac{V2_{sdl_3}}{2}\right| + \frac{\left|T_{sdl_3}\right|}{h1_s \cdot 2} \qquad Vsdl_{st} = 145.0\,\text{kN}$$

Maximum design load reaction due to deck jacking:

$$V_{Jack} := 1.3\max\left(Vdl_{pc} + Vsdl_{pc}, Vdl_{st} + Vsdl_{st}\right)$$

$$V_{Jack} = 1167\,\text{kN}$$

Assuming the worst case for positioning of the jack and ignoring width of loaded area from jack gives:

Horizontal tensile force	$N_{uc} := 0.2 \cdot V_{Jack}$	$N_{uc} = 233.4\,\text{kN}$
Shear span	$a_v := h5$	$a_v = 975\,\text{mm}$
Design Moment	$M_u := V_{Jack} \cdot a_v + N_{uc} \cdot \left(h - d_e\right)$	$M_u = 815.4\,\text{kN} \cdot \text{m}$
Loaded width during jacking	$w := 1000\text{mm}$	$w = 1000\,\text{mm}$
Depth of beam ledge	$h := v3$	$h = 1200\,\text{mm}$
Effective depth	$d_e := h - 65\text{mm}$	$d_e = 1135\,\text{mm}$
Width of the interface	$b_v := w$	$b_v = 1000\,\text{mm}$
Area of concrete resisting shear transfer	$A_{cv} := b_v \cdot d_e$	$A_{cv} = 1.135\ \text{m}^2$

Calculate required nominal shear strength

$$V_n := \frac{V_{Jack}}{\Phi_s} \qquad V_n = 1667.5\,\text{kN}$$

Calculate shear friction reinforcement, A_{vf}

Cohesion and friction values for monolithically cast concrete

$$c := 1.0\text{MPa}$$

$$\lambda := 1.00$$

$$\mu := 1.4\cdot\lambda$$

Shear reinforcement is then given by:

$$A_{vf} := \left| \begin{array}{l} A_{vf1} \leftarrow \dfrac{V_n - c\cdot A_{cv}}{f_y\cdot\mu} \\ A_{vf2} \leftarrow 0.35\cdot\dfrac{b_v}{mm}\cdot\dfrac{MPa}{f_y}\cdot mm^2 \\ A_{vf} \leftarrow A_{vf1} \text{ if } A_{vf1} > A_{vf2} \\ A_{vf} \leftarrow A_{vf2} \text{ if } A_{vf2} > A_{vf1} \\ 0 \text{ if } \dfrac{V_n}{A_{cv}} < 0.7\text{MPa} \end{array} \right. \qquad \frac{V_n}{A_{cv}} = 1.469\,\text{MPa}$$

$$A_{vf} = 975.2\,\text{mm}^2$$

Design the primary tensile force reinforcement A_s:

Strength reduction factor for bending $\Phi = 0.8$

Width of section $b := 1000\text{mm}$

Determine area of primary reinforcement, A_s, to resist flexure
(ref AASHTO LFRD Article 5.7.3.2.3 - see notes on flexural design for rearrangement of terms):

$$R := \frac{M_u}{\Phi\cdot b\cdot d_e^2\cdot f_y} \qquad R = 0.0020 \qquad M := \frac{0.85\cdot f_c}{f_y} \qquad M = 0.0654$$

$$\rho := M\cdot\left(1 - \sqrt{1 - \frac{2\cdot R}{M}}\right) \qquad \rho = 0.002061$$

$$A_s := \rho\cdot b\cdot d_e \qquad A_s = 2339.6\,\text{mm}^2$$

Design the tensile force reinforcement A_n:

$$A_n := \frac{N_{uc}}{\Phi\cdot f_y} \qquad A_n = 748\,\text{mm}^2$$

Check that the Area of Primary Reinforcement, A_s, satisfies code requirements:

$$A_s := \begin{cases} A_s & \text{if } A_s > \frac{2}{3}\cdot A_{vf} + A_n \\ \frac{2}{3}\cdot A_{vf} + A_n & \text{otherwise} \end{cases}$$

$$A_s := \begin{cases} 0.04\cdot\frac{f_c}{f_y}\cdot b\cdot d_e & \text{if } A_s < 0.04\cdot\frac{f_c}{f_y}\cdot b\cdot d_e \\ A_s & \text{otherwise} \end{cases}$$

$$A_s = 3492\,mm^2$$

$$A_s := \frac{A_s}{b} \qquad A_s = 3492\,\frac{mm^2}{m}$$

check area of steel required

$$\rho_{REQUIRED} := \frac{A_s\cdot m}{b\cdot d_e}\cdot 100 \qquad \rho_{REQUIRED} = 0.31 \quad \text{PERCENT}$$

Determine area of closed stirrups or ties, Ah:

$$A_h := 0.5\cdot\left(A_s\cdot b - A_n\right) \qquad A_h = 1372\,mm^2$$

This reinforcement shall be uniformly distributed within two-thirds of the effective depth of the beam ledge adjacent to As.

<u>Design for Hanger Reinforcement</u>

The nominal shear resistance of ledges of inverted T-beams shall be the lessor of the following:

$$V_{N1}(A_{hr}, s) := \frac{A_{hr}\cdot f_y}{s}\cdot S$$

$$V_{N2}(A_{hr}, s) := 0.165\cdot\sqrt{\frac{f_c}{MPa}}\cdot MPa\cdot b_f\cdot d_f + \frac{A_{hr}\cdot f_y}{s}\cdot b$$

Try 19mm ϕ rebar at 100mm c/c

Area of one leg of hanger reinforcement A_{hr} and spacing s are then:

$$A_{hr} := \frac{\pi\cdot(19mm)^2}{4} \qquad s := 100mm$$

Total number of bars required:

$$n_{bars} := \frac{b}{s} \qquad n_{bars} = 10.0$$

This gives:

$$V_{N1}(A_{hr}, s) = 6800\,kN$$

$$V_{N2}(A_{hr}, s) = 4421\,kN$$

The minimum nominal shear resistance is then given by:

$$V_n := \min\left(V_{N1}\left(A_{hr}, s\right), V_{N2}\left(A_{hr}, s\right)\right)$$

$$V_n = 4421\,kN$$

Max shear force from the jack:

$$V_{Jack} = 1167\,kN$$

Check that the nominal shear resistance is adequate

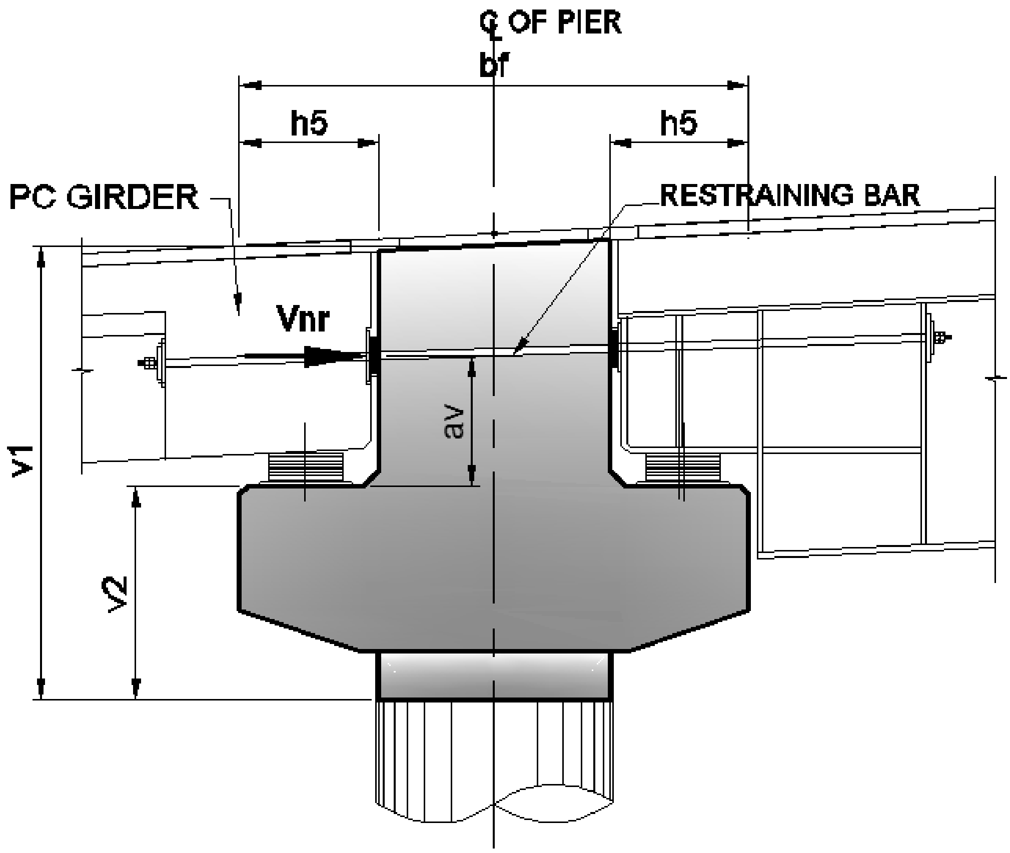
$$Hanger_{CHECK} := \begin{cases} \text{"SATISFIED"} & \text{if } V_n \cdot \Phi_s \geq V_{Jack} \\ \text{"FAIL"} & \text{otherwise} \end{cases}$$

$$Hanger_{CHECK} = \text{"SATISFIED"}$$

CONCLUSION - PROVIDE SAME REINFORCEMENT FOR MAIN BEAM LEDGE DESIGN ACROSS ENTIRE BEAM LEDGE WIDTH - EXCEPT OVER WIDTH OF COLUMN.

JACKING POINTS SHALL NOT BE CLOSER THAN 500mm FROM FACE OF COLUMN.

Design for Loads from Longitudinal Restrainers, Article 3.10.9.5



This calculation note is intended for the design of the reinforced concrete elements of the pier coping supporting the restrainers.

For the design of the restrainers themselves, inlcuding any local bursting reinforcement required, refer to a separatee calculation.

Restrainers shall be designed for a force calculated as the acceleration coefficient times the permanent load of the lighter of the two adjoining spans.

Acceleration coefficient

$$A := 0.40$$

Permanent load of the lighter of the two spans assuming, conservatively that only 40% of the total load appears as a reaction at the pier coping:

$$PERM_{LOAD} := \left(V2_{pc} + V2_{sdl_2}\right)\frac{1}{40\%} \qquad PERM_{LOAD} = 2290\,kN$$

Design load in restrainer

$$REST_{LOAD} := PERM_{LOAD} \cdot A \qquad REST_{LOAD} = 916\,kN$$

Required nominal shear strength of pier coping supporting the restrainers:

$$V_{nr} := \frac{REST_{LOAD}}{\Phi_s} \qquad V_{nr} = 1309\,kN$$

Assume conservatively that this load is carried at a single point with a 150mm x 150mm bearing plate located at mid height of the PC deck. Given that the applied load will be supported by a coping width that is at least equal to the shear span, a_v, of the load - design shear friction reinforcement in accordance with Article 5.13.2.5.2 and Article 5.8.4.

Shear span of the load $a_v := v1 - v2 - \frac{1200mm}{2}$ $a_v = 932\,mm$

The width of the concrete face assumed to participate in resistance to shear is as defined below:

$$b_v := \begin{array}{|l} S \leftarrow \min(2\cdot h1_c, 2\cdot h1_s) \\ d_w \leftarrow 150mm + 4\cdot a_v \\ d_w \;\; \text{if} \;\; d_w < S \\ S \;\; \text{otherwise} \end{array} \qquad b_v = 3878\,mm$$

Depth of support $h := b_f - h5\cdot 2$ $h = 1400\,mm$

Effective depth $d_e := h - 65mm$ $d_e = 1335\,mm$

Area of concrete resisting shear transfer $A_{cv} := b_v \cdot d_e$ $A_{cv} = 5.177\ m^2$

Calculate limiting nominal shear strength

$$V_{nlimit} := \begin{array}{|l} 0.2\cdot f_c\cdot A_{cv} \;\; \text{if} \;\; 0.2\cdot f_c\cdot A_{cv} < 5.5\cdot\frac{A_{cv}}{mm^2}\cdot N \\ 5.5\cdot\frac{A_{cv}}{mm^2}\cdot N \;\; \text{otherwise} \end{array} \qquad V_{nlimit} = 28474.2\,kN$$

Check that the required nominal shear strength is less than that provided

$$PierCoping_{Capacity} := \begin{array}{|l} \text{"OK"} \;\; \text{if} \;\; V_{nr} \le V_{nlimit} \\ \text{"INADEQUATE"} \;\; \text{otherwise} \end{array} \qquad PierCoping_{Capacity} = \text{"OK"}$$

Calculate shear friction reinforcement, A_{vf}

Cohesion and friction values for monolithically cast concrete

$c := 1.0MPa$ $\lambda := 1.00$ $\mu := 1.4\cdot\lambda$

Shear reinforcement is then given by:

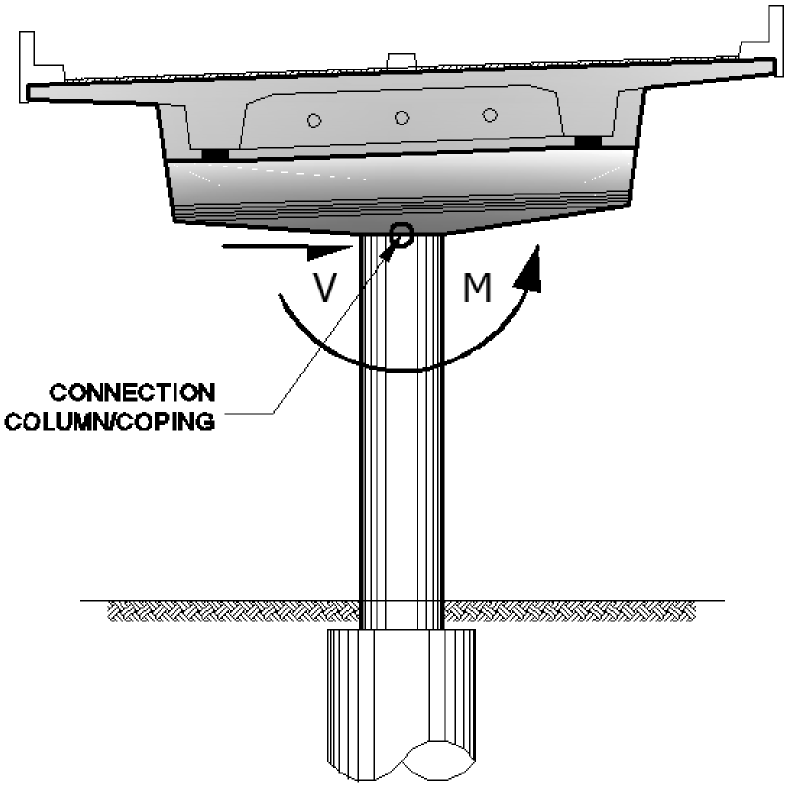
$$A_{vf} := \begin{array}{|l} A_{vf1} \leftarrow \frac{V_{nr} - c\cdot A_{cv}}{f_y\cdot\mu} \\ A_{vf2} \leftarrow 0.35\cdot\frac{b_v}{mm}\cdot\frac{MPa}{f_y}\cdot mm^2 \\ A_{vf} \leftarrow A_{vf1} \;\; \text{if} \;\; A_{vf1} > A_{vf2} \\ A_{vf} \leftarrow A_{vf2} \;\; \text{if} \;\; A_{vf2} > A_{vf1} \\ 0 \;\; \text{if} \;\; \frac{V_{nr}}{A_{cv}} < 0.7MPa \end{array} \qquad \frac{V_{nr}}{A_{cv}} = 0.253\,MPa$$

$A_{vf} = 0\,mm^2$

Conclude that the concrete section alone has adequate capacity to carry the loads from the longitudinal restrainers without the need for addiional shear reinforcement.

Design the Pier Column/Coping Connection for Flexure and Shear

The pier column/coping conection will be designed to resist the maximum moments and shear forces applied from live loads and earthquake loads.



Analysis Output

The analysis output for **load effects** at the top of the expansion piers, obtained from the SAP 3D model, is presented below.

The half live load case is with live load only occupying the lanes in one carriageway to create maximum transverse moment at the top of the column.

ULTIMATE FLEXURAL DEMAND AT THE TOP OF THE COLUMN

		KN	KN-m	KN-m	KN-m
COMBINATION 1	max	-5640.0	742.8	0.0	742.8
Full live Load	min	-10523.7	-752.6	0.0	-752.6
COMBINATION 1	max	-5840.4	5159.7	0.0	5159.7
Half Live Load	min	-8317.3	-5171.0	0.0	-5171.0
COMBINATION 5	max	-4216.6	390.4	1180.1	1243.0
1.0 EQX+ 0.3 EQY	min	-4593.0	-393.2	-1180.1	1243.9
COMBINATION 5	max	-4340.9	920.2	465.2	1031.1
0.3 EQX+ 1.0 EQY	min	-4468.7	-923.0	-465.2	1033.6

$$P := -P \cdot kN \qquad M_d := \sqrt{{M_d}^2} \cdot kN \cdot m$$

Design for Flexure in Column (AASHTO LRFD Section 5.7)

The maximum moment to be used in the fexural design of the connection is given below (maximum of moments obtained from global analysis and longitudinal moment generated by loading only one deck span):

$$M_{max} := \begin{array}{|l} M1 \leftarrow \max(M_d) \\ M2 \leftarrow T_c \cdot 2 \\ \max(M1, M2) \end{array} \qquad M_{max} = 5171\,kN\cdot m$$

From inspection above the critical case is due to transverse live load occupying lanes on one side of the deck only. The associated axial load with this case, P, therefore will be intermediate between the maximum and minimm axial loads identified above for the load configuration.

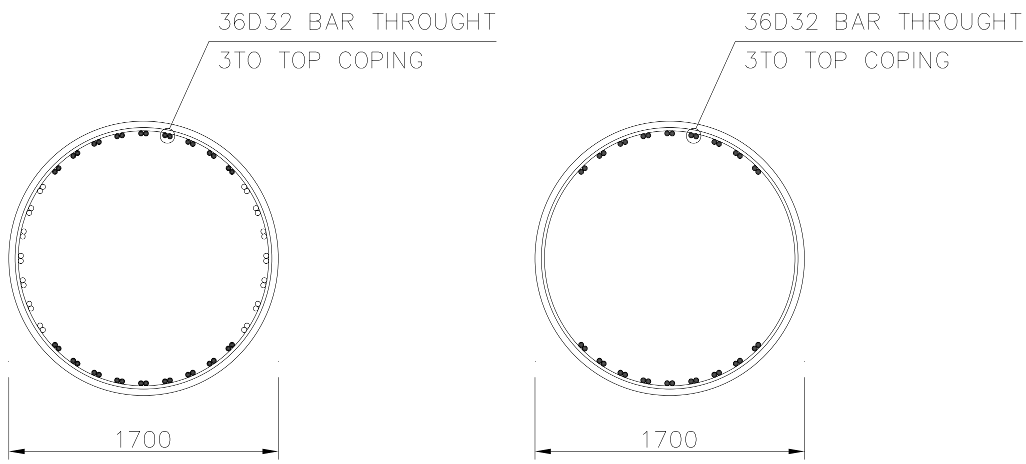
$$P := \frac{P_3 + P_4}{2} \qquad P = 7079\,kN$$

PCACol has been used to design the reinforcement concrete column.

Results of the PCACol design are as follows:

Use 36 number 32mmϕ bars in two bar bundles in a single layer ajaccent to top coping

To aid with reinforcement fixing the bars are placed to give a cover of 60 mm longitudinal bars.This will allow the placing of horizontal shear reinforcement in the faces of the coping beam adjacent to the column rebar. The bar layout is shown below:



Design for Shear in Column (AASHTO LRFD Section 5.8)

Depth of section $h_s := D$ $h_s = 1700\,mm$

Width of section $b := D$ $b = 1700\,mm$

Diameter of circle passing through centers of longitudinal rebar $D_r := h_s - (2\cdot 80 + 32)\cdot mm$

Effective depth $d_e := \frac{h_s}{2} + \frac{D_r}{\pi}$

Critical section for shear shall be taken as d_v from the internal face of the support (AASHTO LRFD Article 5.8.3.2)

$$d_v := \begin{cases} 0.9\cdot d_e & \text{if } 0.9\cdot d_e > 0.72\cdot h_s \\ 0.72\cdot h_s & \text{otherwise} \end{cases}$$

$d_v = 1224\,mm$

Maximum shear force (from plastic hinging at base of column)

$V_P = 2612\,kN$

Calculate nominal shear resistance of concrete section assuming beam section:

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

$V_c = 1892\,kN$

Strength reduction factor for shear:

$\Phi_s = 0.7$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \begin{array}{|l} V_s \leftarrow \frac{V_P}{\Phi_s} - V_c \\ V_s \text{ if } V_s > 0kN \\ 0kN \text{ otherwise} \end{array}$$

$V_s = 1840\,kN$

Check that total required nominal shear resistance does not exceed limit:

$V_{nlimit} := 0.25\cdot f_c\cdot b\cdot d_v$ $V_{nlimit} = 15606.0\,kN$

$$CHECK := \begin{cases} \text{"OK"} & \text{if } V_c + V_s \le V_{nlimit} \\ \text{"FAIL"} & \text{otherwise} \end{cases}$$ $CHECK = \text{"OK"}$

Provide 19mmϕ spirals

$\phi_{link} := 19mm$

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

Determine required spacing of transverse reinforcement

$$s_t := \begin{array}{|l} s_{t1} \leftarrow \dfrac{A_v \cdot f_y}{0.083 \cdot \sqrt{\dfrac{f_c}{MPa}} \cdot MPa \cdot b} \\ s_{t2} \leftarrow \dfrac{A_v \cdot f_y \cdot d_v}{V_s} \ \text{ if } \ V_s > 0kN \\ s_{t2} \ \text{ if } \ (V_s > 0kN) \cdot (s_{t2} \le s_{t1}) \\ s_{t1} \ \text{ otherwise} \end{array} \qquad s_t = 147\,mm$$

Calculate shear stress in order to determine maximum spacing of transverse reinforcement to resist shear:

$$v_u := \frac{V_P}{\Phi_s \cdot (b \cdot d_v)} \qquad v_u = 1.793\,MPa$$

Calculate spacing taking into account maximum spacing requirements:

$$s_{max} := \begin{array}{|l} \min(600mm, 0.8d_v) \ \text{ if } \ v_u < 0.125f_c \\ \min(300mm, 0.4d_v) \ \text{ otherwise} \end{array}$$

$$s_{max} = 600\,mm$$

Determine maximum required spacing of transverse reinforcement:

$$s_t := \begin{array}{|l} s_{max} \ \text{ if } \ s_{max} \le s_t \\ s_t \ \text{ otherwise} \end{array}$$

$$s_t = 147\,mm$$

Provide 19mmϕ spirals at 100mm c/c

Note that although this section will not be subject to plastic hinging, the detailing of the reinforcement will follow the requirements at plastic hinge zones:

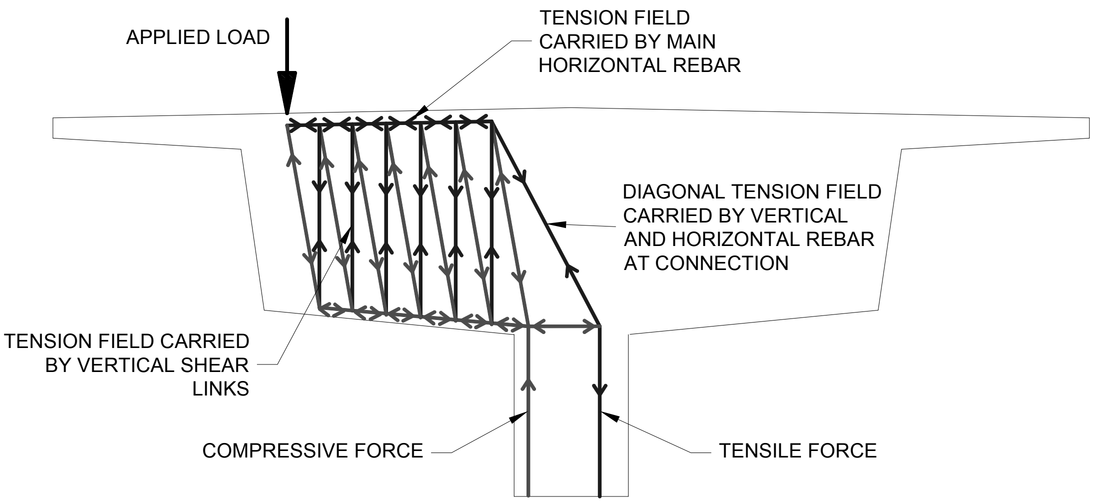
- maximum spacing 100mm
- shear links extended into the support for a distance not less than one half the column diameter column
- the development length of the longitidinal steel shall be 1.25 times that required for the full yield of the reinforcing bar

Design for Shear at Connection (AASHTO LRFD Section 5.8)

The applied loading on the coping will be carried by shear forces in the connection generating tension and compression fields as shown below.

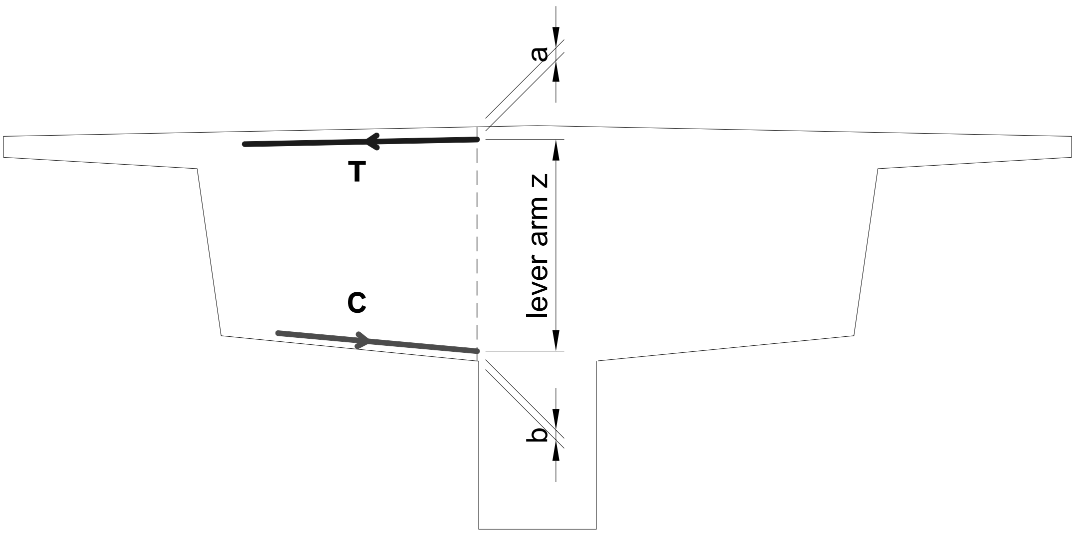
The vertical component of the tension field in the connection will be carried by the main column reinforcement, extended to the top of the connection.

The horizontal component will be carried by additional reinforcement designed to resist the horizontal shear force generated at the connection.



Defining offset "a" to the main rebar in the upper coping beam and offset "b" to cenroid of compression zone in the lower coping beam resisting the applied moment:

$a := 150\text{mm} \qquad b := 100\text{mm}$



Lever arm in the coping at the face of the conection $\quad z := v1 - a - b \qquad z = 2482\,\text{mm}$

Calculate the tension T and compression C forces at the connection under maximum applied moment:

$$T := -\frac{M_{max}}{z} \qquad T = -2083\,\text{kN} \qquad C := \frac{M_{max}}{z} \qquad C = 2083\,\text{kN}$$

Check the assumed depth of concrete in compression:

width of coping $b_w := b_f - 2 \cdot h5$ $\quad b_w = 1400\,mm$

depth of concrete in compression $c := \dfrac{C}{b_w \cdot 0.85 \cdot f_c}$ $\quad c = 58\,mm$

Given that the depth of concrete in compression is less than the assumed offset to the centroid, accept the assumed values as conservative.

Depth of section $h_s := D$ $\quad h_s = 1700\,mm$

Width of section $b := D$ $\quad b = 1700\,mm$

Diameter of circle passing through centers of longitudinal rebar in column $D_r := h_s - (2 \cdot 80 + 32) \cdot mm$

Effective depth $d_e := \dfrac{h_s}{2} + \dfrac{D_r}{\pi}$ $\quad d_e = 1330\,mm$

Critical section for shear shall be taken as d_v from the internal face of the support (AASHTO LRFD Article 5.8.3.2)

$$d_v := \begin{array}{|l} 0.9 \cdot d_e \quad \text{if } 0.9 \cdot d_e > 0.72 \cdot h_s \\ 0.72 \cdot h_s \quad \text{otherwise} \end{array}$$

$d_v = 1224\,mm$

Maximum shear force (from maximum moment applied at the connection)

$V_U := C$ $\quad V_U = 2083\,kN$

Calculate nominal shear resistance of concrete section at the connection (refer ASHTO LRFD Article 5.10.11.4.3):

$$V_c := 1.0 \cdot \sqrt{\frac{f_c}{MPa}} \cdot b \cdot d_v \cdot MPa$$

$V_c = 11397\,kN$

Strength reduction factor for shear:

$\Phi_s = 0.7$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \begin{array}{|l} V_s \leftarrow \dfrac{V_U}{\Phi_s} - V_c \\ V_s \quad \text{if } V_s > 0kN \\ 0kN \quad \text{otherwise} \end{array}$$

$V_s = 0\,kN$

Check that total required nominal shear resistance does not exceed limit:

$V_{nlimit} := 0.25 \cdot f_c \cdot b \cdot d_v$ $\quad V_{nlimit} = 15606.0\,kN$

$$CHECK := \begin{array}{|l} \text{"OK"} \quad \text{if } V_c + V_s \le V_{nlimit} \\ \text{"FAIL"} \quad \text{otherwise} \end{array}$$

$CHECK = \text{"OK"}$

Provide 19mmϕ shear links wth 2 legs across the section - one each face

$$\phi_{link} := 19\text{mm}$$

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

Determine required spacing of transverse reinforcement

$$s_t := \left| \begin{array}{l} s_{t1} \leftarrow \dfrac{A_v \cdot f_y}{0.083 \cdot \sqrt{\dfrac{f_c}{\text{MPa}}} \cdot \text{MPa} \cdot b} \\ s_{t2} \leftarrow \dfrac{A_v \cdot f_y \cdot d_v}{V_s} \ \text{ if } V_s > 0\text{kN} \\ s_{t2} \ \text{ if } (V_s > 0\text{kN}) \cdot (s_{t2} \le s_{t1}) \\ s_{t1} \ \text{ otherwise} \end{array} \right. \qquad s_t = 286\,\text{mm}$$

Calculate shear stress in order to determine maximum spacing of transverse reinforcement to resist shear:

$$v_u := \frac{V_P}{\Phi_s \cdot (b \cdot d_v)} \qquad v_u = 1.793\,\text{MPa}$$

Calculate spacing taking into account maximum spacing requirements:

$$s_{max} := \left| \begin{array}{l} \min(600\text{mm}, 0.8d_v) \ \text{ if } v_u < 0.125f_c \\ \min(300\text{mm}, 0.4d_v) \ \text{ otherwise} \end{array} \right.$$

$$s_{max} = 600\,\text{mm}$$

Determine maximum required spacing of transverse reinforcement:

$$s_t := \left| \begin{array}{l} s_{max} \ \text{ if } s_{max} \le s_t \\ s_t \ \text{ otherwise} \end{array} \right.$$

$$s_t = 286\,\text{mm}$$

Provide 19mmϕ bar at 200mm c/c each face

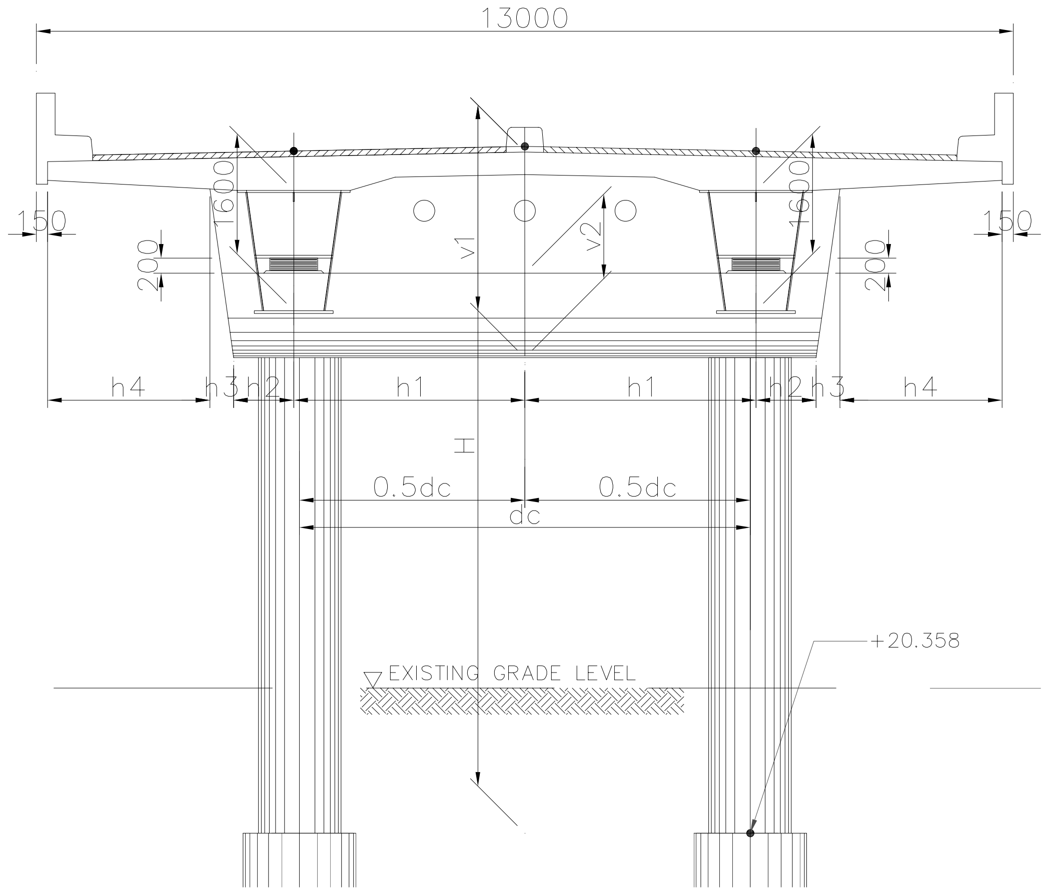
P6 Expansion Coping

KATAHIRA & ENGINEERS
INTERNATIONAL

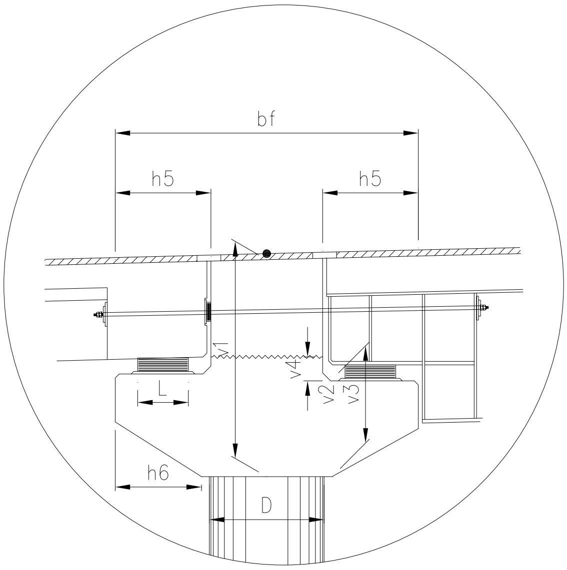
Project: **Detailed Design Study of**
North Java Corridor Flyover Project

Calculation: **Detailed Design Substructure**
Balaraja Flyover
Coping Design - Pier P6

Layout



Coping Cross Section



Pier Coping Data

Overall width of deck	**B**	13000	mm
Offset to bearing - PC deck	**$h1_c$**	3175	mm
Offset to bearing - Steel deck	**$h1_s$**	3075	mm
Edge distance	**h2**	800	mm
Side slope width	**h3**	316	mm
Cantilever length	**h4**	2160	mm
Beam ledge width	**h5**	1200	mm
Beam ledge soffit width	**h6**	950	mm
Width of coping	**b_f**	3800	mm
Total depth at coping	**v1**	2732	mm
Beam ledge height at column	**v2**	1200	mm
Beam ledge height at bearing	**v3**	1200	mm
Upstand to const. joint	**v4**	300	mm
Bearing width	**W**	800	mm
Bearing length	**L**	800	mm
Column Diameter	**D**	1100	mm
Distance Column to Column	**dc**	6000	mm
Concrete Comp Strength	**f_c**	30	MPa
Rebar Yield Strength	**f_y**	390	MPa
Strength Reduction Factor - Bending		0.8	
Strength Reduction Factor - Shear		0.7	
Mod. Elasticity - Concrete	**E_c**	27628	MPa
Mod. Elasticity - Steel	**E_s**	200000	MPa
Modular Ratio		7.24	
Height of piers supporting deck	**H**	9912	mm
Length of deck btwn. Joints	**L_d**	73.6	m
Deck skew	**S_k**	0	Deg

$B := B \cdot mm$

$h1_c := h1_c \cdot mm$

$h1_s := h1_s \cdot mm$

$h2 := h2 \cdot mm$

$h3 := h3 \cdot mm$

$h4 := h4 \cdot mm$

$h5 := h5 \cdot mm$

$h6 := h6 \cdot mm$

$b_f := b_f \cdot mm$

$v1 := v1 \cdot mm$

$v2 := v2 \cdot mm$

$v3 := v3 \cdot mm$

$v4 := v4 \cdot mm$

$D := D \cdot mm$

$d_c := d_c \cdot mm$

$L := L \cdot mm$

$W := W \cdot mm$

$H := H \cdot mm$

$f_c := f_c \cdot MPa$

$f_y := f_y \cdot MPa$

$E_c := E_c \cdot MPa$

$E_s := E_s \cdot MPa$

Analysis Output

The analysis output for **deck reactions** at the expansion piers, obtained from the SAP 3D model, is presented below for:

- Nominal deck dead load case - used for erection case
- Nominal superimposed dead load case
- ULS Combination 1 - live load
- SLS Combination 1 - live load

- Nominal earthquake effects (R=1.0)
- The frame elements selected are as follows:
- D34 - end span frame in span 3 (PC deck) adjacent to pier 3
- D41 - end span frame in span 3 (Steel Deck) adjacent to pier 3

NOMINAL COPING DEAD LOAD

TABLE: Element Forces - Frames						
Frame	Station	OutputCase	StepType	P	V2	M3
Text	m	Text	Text	KN	KN	KN-m
C Left	0	DEAD-COPING	Max	2.7	459.4	20.6
C Left	3	DEAD-COPING	Min	2.7	-2.3	-665
C Right	0	DEAD-COPING	Max	2.6	2.4	-664.9
C Right	3	DEAD-COPING	Min	2.6	-459.3	20.4

Shear V2 Moment M3

$VCop := V2 \cdot kN$ $MCop := M3 \cdot kN \cdot m$

NOMINAL DECK DEAD LOAD

TABLE: Element Forces - Deck Dead Load Reactions				
Frame	Station	V2 SHEAR VERT	V3 SHEAR TRANS	T TORSION
Text	m	KN	KN	KN-m
DECK64	Min	1504.6	1.8	-4.1
DECK64	Max	1504.6	1.8	-4.1
DECK71	Min	-1331.7	8.5	-0.6
DECK71	Max	-1331.7	8.5	-0.6

$V2_{dl} := V2_{dl} \cdot kN$ $V3_{dl} := V3_{dl} \cdot kN$ $T_{dl} := T_{dl} \cdot kN \cdot m$

NOMINAL SUPERIMPOSED DEAD LOAD

TABLE: Element Forces -Superimposed Load Reactions				
Frame	Station	V2 SHEAR VERT	V3 SHEAR TRANS	T TORSION
Text	m	KN	KN	KN-m
DECK64	Min	289.5	0.3	-0.7
DECK64	Max	289.5	0.3	-0.7
DECK71	Min	-242.4	1.5	0.0
DECK71	Max	-242.4	1.5	0.0

$V2_{sdl} := V2_{sdl} \cdot kN$ $V3_{sdl} := V3_{sdl} \cdot kN$ $T_{sdl} := T_{sdl} \cdot kN \cdot m$

ULS COMBINATION 1 - LIVE LOAD

TABLE: Element Forces - Deck COMB1 ULS Reactions				
Frame	Step Type	V2 SHEAR VERT	V3 SHEAR TRANS	T TORSION
Text	Text	KN	KN	KN-m
DECK64	Min	2071.6	-61.2	-1084.6
DECK64	Max	6693.9	75.4	1052.6
DECK71	Min	-6138.7	-39.2	-1004.9
DECK71	Max	-1877.1	107.8	1004.4

$V2_u := V2_u \cdot kN$ $\quad V3_u := V3_u \cdot kN$ $\quad T_u := T_u \cdot kN \cdot m$

SLS COMBINATION 1 - LIVE LOAD

TABLE: Element Forces - Deck COMB1 SLS Reactions				
Frame	Step Type	V2 SHEAR VERT	V3 SHEAR TRANS	T TORSION
Text	Text	KN	KN	KN-m
DECK64	Min	1341.6	-33.4	-613.2
DECK64	Max	4457.6	47.0	583.5
DECK71	Min	-3942.9	-19.0	-558.6
DECK71	Max	-1255.0	84.7	559.2

$V2_s := V2_s \cdot kN$ $\quad V3_s := V3_s \cdot kN$ $\quad T_s := T_s \cdot kN \cdot m$

NOMINAL EARTHQUAKE LOAD - EQX (R=1)

TABLE: Element Forces - EQX				
Frame	Step Type	V2 SHEAR VERT	V3 SHEAR TRANS	T TORSION
Text	Text	KN	KN	KN-m
DECK64	Min	1142.0	-2096.0	-3784.7
DECK64	Max	2446.2	2100.2	3775.1
DECK71	Min	-2023.2	-1463.9	-269.5
DECK71	Max	-1125.0	1483.9	268.3

$V2_{eqx} := V2_{eqx} \cdot kN$ $\quad V3_{eqx} := V3_{eqx} \cdot kN$ $\quad T_{eqx} := T_{eqx} \cdot kN \cdot m$

NOMINAL EARTHQUAKE LOAD - EQY (R=1)

TABLE: Element Forces - EQY				
Frame	Step Type	V2 SHEAR VERT	V3 SHEAR TRANS	T TORSION
Text	Text	KN	KN	KN-m
DECK64	Min	1379.1	-3370.1	-5573.0
DECK64	Max	2209.1	3374.3	5563.4
DECK71	Min	-1823.9	-2562.4	-279.2
DECK71	Max	-1324.3	2582.4	278.0

$V2_{eqy} := V2_{eqy} \cdot kN$ $\quad V3_{eqy} := V3_{eqy} \cdot kN$ $\quad T_{eqy} := T_{eqy} \cdot kN \cdot m$

MAXIMUM SHEAR FORCE DUE TO PLASTIC HINGING

The maximum shear force that the bearings will be subject to in the tranverse direction is the shear force due to plastic hinging at the base of the column.

The shear force due to plastic hinging at the top of the column tranverse direction $V_P := 1197 \cdot kN$

This shear force should be carried by the transverse shear key on each bearing shelf or carried by the bearings directly on on bearing shelf. The shear forces given above for earthquake loading should be used in the design if they are smaller than the plastic hinging effects.

MAXIMUM & MINIMUM VERTICAL ORRENPONDING PLASTIC HINGING EFECT

$$P_{max} := 5542kN$$

$$P_{min} := -1052kN$$

Minimum Displacement Requirements (AASHTO LRFD Article 4.7.4.4)

Bridge seat widths at expansion bearings without restrainers, STU's or dampers shall either accommodate the greater of the maximum displacement calculated from the seismic analysis or a percentage of the emprical seat width, *N*, specified below.

The percentage of *N* applicable to the bridge seismic zone shall be 150%.

The length of the bridge deck to the adjacent expansion joint $L_d := L_d \cdot m$ $\quad L_d = 73.6\ m$

The height of the columns supporting the deck $H = 9.912\ m$

Skew of the support measured from line normal to span $S_k = 0$

The empirical seat width shall be taken as:

$$N := \left(200 + 0.0017 \cdot \frac{L}{mm} + 0.0067 \cdot \frac{H}{mm}\right) \cdot \left(1 + 0.000125 \cdot S_k^{\,2}\right) \cdot mm$$

$$N = 268\,mm$$

Check that the seat width provided is greater than 150% of *N*:

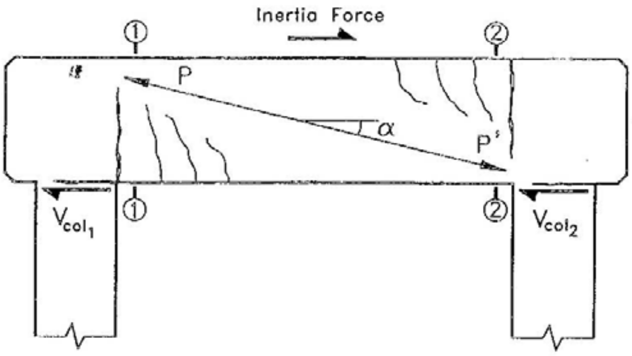
Seat width $h5 = 1200\,mm$

$$150\% \cdot N = 402\,mm$$

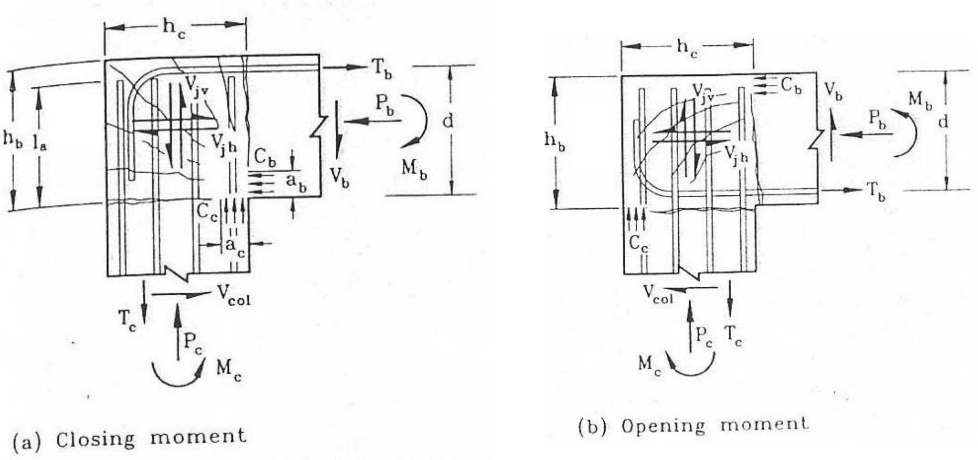
$$Seat_{Width} := \begin{vmatrix} \text{"OK"} & \text{if } h5 \ge N \cdot 150\% \\ \text{"INADEQUATE"} & \text{otherwise} \end{vmatrix}$$

$$Seat_{Width} = \text{"OK"}$$

Design of Pier Coping based on plastic Hinge Force



Beam Column Joint



Bending moment due to plastic hinges force

Plastic Hinge Moment

$M_P := 5542\text{kN}\cdot\text{m}$

Knee joints are the most common type of joint occuring in multicolumn bridge bents when tranverse response is considered. Equilibriun conditions under openinng and closing momentt are represented in figs. (a) and (b) above respectively. In these figures, the beam tensile, compressive, and shear stress resultants are indicated by T_b, C_b,
and V_b, with T_c C_c, and V_{col} being the corresponding force for the column. Axial forces P_c and P_b are present in the colun and beam, respectively. Moments M_b and M_c on joint boundaries induce the flexural stress resultants noted above.

Dimension :

Beam Coping

$h_b := v1$

assume beam coping are rectangular for flexural design

$b := b_f - 2h5$ $\quad$ $d_e := h_b - 100\text{mm}$

Column

$h_c := D$

Design for Flexure (AASHTO LRFD Section 5.7)

(a) Closing moment

Coping Dead Load at joint

$$Mdl := MCop_1$$

$$Mdl = 20.6\,kN\cdot m$$

$$M_{EU} := M_P + V_P\cdot\frac{h_b}{2} + Mdl$$

$$M_{EU} = 7197.7\,kN\cdot m$$

Strength reduction factor for flexure:

$$\Phi = 0.8$$

Determine area of reinforcement, A_f, in the coping to resist flexure
(ref AASHTO LFRD Article 5.7.3.2.3 -see notes on flexural design for rearrangement of terms):

$$R := \frac{M_{EU}}{\Phi\cdot b\cdot d_e^{\,2}\cdot f_y} \qquad R = 0.0024 \qquad M := \frac{0.85\cdot f_c}{f_y} \qquad M = 0.0654$$

$$\rho := M\cdot\left(1 - \sqrt{1 - \frac{2\cdot R}{M}}\right) \qquad \rho = 0.002424$$

$$A_f := \rho\cdot d_e\cdot b$$

$$A_f = 8930.5\,mm^2$$

Using 32mmϕ bars gives total number of bars to be distributed across the coping:

$$n_{bf} := \frac{A_f}{804\cdot mm^2} \qquad n_{bf} = 11.1$$

Provide 18 No 32mm ϕ bars in two layers $\quad n_p := 18$

$$A_f := n_p\cdot 804\cdot mm^2$$

Calculate the stress block factor, β_1:

$$\beta_1 := \left| \begin{array}{l} \beta_1 \leftarrow 0.85 \\ \beta_1 \leftarrow \beta_1 - 0.05\cdot\dfrac{f_c - 28MPa}{7.MPa} \quad \text{if } f_c > 28MPa \\ 0.65 \quad \text{if } \beta_1 < 0.65 \end{array} \right.$$

$$\beta_1 = 0.836$$

For rectangular sections, the depth of concrete in compression, c, is given by:

$$c := \frac{A_f\cdot f_y}{0.85\cdot f_c\cdot \beta_1\cdot b} \qquad c = 189\,mm$$

Check that the required amount of reinforcement does not exceed the maximum allowed by the code:

$$\frac{c}{d_e} = 0.07$$

$$Max_{Limit} := \begin{vmatrix} \text{"OK"} \quad \text{if } \frac{c}{d_e} \le 0.42 \\ \text{"EXCEEDED"} \quad \text{otherwise} \end{vmatrix} \qquad Max_{Limit} = \text{"OK"}$$

Calculate the modulus of rupture, f_r ,of the concrete:

$$f_r := 0.63 \cdot \sqrt{\frac{f_c}{MPa}} \cdot MPa \qquad f_r = 3.451\,MPa$$

Section modulus for the extreme fiber of the section, assuming rectangular section of width b:

$$S_c := \frac{b \cdot d_e^{\,2}}{6} \qquad S_c = 1.616\,m^3$$

The cracking moment, M_{cr} ,is the given by:

$$M_{cr} := S_c \cdot f_r \qquad M_{cr} = 5578\,kN \cdot m$$

Check that the reinforcement can develop a resistance moment Mr at least equal to the lesser of:

- 1.2 times the cracking moment Mcr
- 1.33 times the factored moment required by the applicable strength load combination

$$Minimum_{Steel} := \begin{vmatrix} M_r \leftarrow \frac{M_{EU}}{\Phi} \\ M \leftarrow \min\left(1.2M_{cr}, 1.33 \cdot M_{EU}\right) \\ \text{"OK"} \quad \text{if } M_r \ge M \\ \text{"NOT SATISFIED"} \quad \text{otherwise} \end{vmatrix} \qquad Minimum_{Steel} = \text{"OK"}$$

Design for Shear (AASHTO LRFD Section 5.8)

Critical section for shear shall be taken as d_v from the internal face of the support (AASHTO LRFD Article 5.8.3.2)

$$h_s := v1$$

$$d_v := \begin{vmatrix} 0.9 \cdot d_e \quad \text{if } 0.9 \cdot d_e > 0.72 \cdot h_s \\ 0.72 \cdot h_s \quad \text{otherwise} \end{vmatrix}$$

$$d_v = 2368.8\,mm$$

Shear in coping due to plastic hinge force

$$V_D := \frac{2 \cdot M_P}{d_c} + \frac{V_P \cdot \frac{v1}{2}}{d_c} + P_{max} \qquad P_{max} = 5542\,kN$$

$$V_D = 7662\,kN$$

Total shear force in coping at face of column during erection - Ultimate Limit State

$$V_{EU} := V_D + VCop_1$$

$$V_{EU} = 8121.3\,\mathrm{kN}$$

Section 1 range of 0 ~ 1 m from cl of column

$$V_{EU} = 8121\,\mathrm{kN}$$

Calculate nominal shear resistance of concrete section assuming beam section:

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

$$V_c = 3015\,\mathrm{kN}$$

Strength reduction factor for shear:

$$\Phi_s = 0.7$$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \left| \begin{array}{l} V_s \leftarrow \dfrac{V_{EU}}{\Phi_s} - V_c \\ V_s \ \text{ if } V_s > 0\mathrm{kN} \\ 0\mathrm{kN} \ \text{ otherwise} \end{array} \right. \qquad V_s = 8587\,\mathrm{kN}$$

Check that total required nominal shear resistance does not exceed limit:

$$V_{nlimit} := 0.25\cdot f_c\cdot b\cdot d_v \qquad V_{nlimit} = 24872.4\,\mathrm{kN}$$

$$CHECK := \left| \begin{array}{l} \text{"OK"} \ \text{ if } V_c + V_s \le V_{nlimit} \\ \text{"FAIL"} \ \text{ otherwise} \end{array} \right. \qquad CHECK = \text{"OK"}$$

Provide 25mmϕ shear links wth 4 **legs** across the section:

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

$$A_v := \pi\frac{\phi_{link}^{\;2}}{4}\cdot 4 \qquad A_v = 1963\,\mathrm{mm}^2$$

Determine required spacing of transverse reinforcement

$$s_t := \left| \begin{array}{l} s_{t1} \leftarrow \dfrac{A_v\cdot f_y}{0.0083\cdot\sqrt{\dfrac{f_c}{MPa}}\cdot MPa\cdot b} \\ s_{t2} \leftarrow \dfrac{A_v\cdot f_y\cdot d_v}{V_s} \ \text{ if } V_s > 0\mathrm{kN} \\ s_{t2} \ \text{ if } (V_s > 0\mathrm{kN})\cdot(s_{t2} \le s_{t1}) \\ s_{t1} \ \text{ otherwise} \end{array} \right. \qquad s_t = 211\,\mathrm{mm}$$

Calculate shear stress in order to determine maximum spacing of transverse reinforcement to resist shear:

$$v_u := \frac{V_{EU}}{\Phi_s \cdot (b \cdot d_v)} \qquad v_u = 3.498\,\text{MPa}$$

Calculate spacing taking into account maximum spacing requirements:

$$s_{max} := \begin{cases} \min(600\text{mm}, 0.8 d_v) & \text{if } v_u < 0.125 f_c \\ \min(300\text{mm}, 0.4 d_v) & \text{otherwise} \end{cases}$$

$$s_{max} = 600\,\text{mm}$$

Determine maximum required spacing of transverse reinforcement:

$$s_t := \begin{cases} s_{max} & \text{if } s_{max} \le s_t \\ s_t & \text{otherwise} \end{cases}$$

$$s_t = 211\,\text{mm}$$

Section 2 range of 1 ~ 2 m from cl of column

$$V_D := \frac{2 \cdot M_P}{d_c} + \frac{V_P \cdot \frac{v1}{2}}{d_c} + \frac{5}{6} P_{max}$$

$$V_D = 6738\,\text{kN}$$

Total shear force in coping at face of column during erection - Ultimate Limit State

$$V_{EU} := V_D + VCop_2$$

$$V_{EU} = 6735.9\,\text{kN}$$

Calculate nominal shear resistance of concrete section assuming beam section:

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

$$V_c = 3015\,\text{kN}$$

Strength reduction factor for shear:

$$\Phi_s = 0.7$$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \begin{array}{|l} V_s \leftarrow \frac{V_{EU}}{\Phi_s} - V_c \\ V_s \text{ if } V_s > 0\text{kN} \\ 0\text{kN} \text{ otherwise} \end{array} \qquad V_s = 6607\,\text{kN}$$

Check that total required nominal shear resistance does not exceed limit:

$$V_{nlimit} := 0.25 \cdot f_c \cdot b \cdot d_v \qquad V_{nlimit} = 24872.4\,kN$$

$$CHECK := \begin{cases} \text{"OK"} & \text{if } V_c + V_s \le V_{nlimit} \\ \text{"FAIL"} & \text{otherwise} \end{cases} \qquad CHECK = \text{"OK"}$$

Provide 25mmϕ shear links wth **4 legs** across the section:

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

$$A_v := \pi \frac{\phi_{link}^2}{4} \cdot 4 \qquad A_v = 1963\,mm^2$$

Determine required spacing of transverse reinforcement

$$s_t := \begin{vmatrix} s_{t1} \leftarrow \dfrac{A_v \cdot f_y}{0.0083 \cdot \sqrt{\dfrac{f_c}{MPa}} \cdot MPa \cdot b} \\ s_{t2} \leftarrow \dfrac{A_v \cdot f_y \cdot d_v}{V_s} \quad \text{if } V_s > 0kN \\ s_{t2} \quad \text{if } (V_s > 0kN) \cdot (s_{t2} \le s_{t1}) \\ s_{t1} \quad \text{otherwise} \end{vmatrix} \qquad s_t = 275\,mm$$

Calculate shear stress in order to determine maximum spacing of transverse reinforcement to resist shear:

$$v_u := \frac{V_{EU}}{\Phi_s \cdot (b \cdot d_v)} \qquad v_u = 2.902\,MPa$$

Calculate spacing taking into account maximum spacing requirements:

$$s_{max} := \begin{cases} \min(600mm, 0.8d_v) & \text{if } v_u < 0.125 f_c \\ \min(300mm, 0.4d_v) & \text{otherwise} \end{cases}$$

$$s_{max} = 600\,mm$$

Determine maximum required spacing of transverse reinforcement:

$$s_t := \begin{cases} s_{max} & \text{if } s_{max} \le s_t \\ s_t & \text{otherwise} \end{cases}$$

$$s_t = 275\,mm$$

Section 3 range of 2 ~ 3 m from cl of column

$$V_D := \frac{2 \cdot M_P}{d_c} + \frac{V_P \cdot \frac{v1}{2}}{d_c} + \frac{4}{6} P_{max}$$

$$V_D = 5815\,kN$$

Total shear force in coping at face of column during erection - Ultimate Limit State

$$V_{EU} := V_D + VCop_3$$

$$V_{EU} = 5816.9\,kN$$

Calculate nominal shear resistance of concrete section assuming beam section:

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

$$V_c = 3015\,kN$$

Strength reduction factor for shear:

$$\Phi_s = 0.7$$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \begin{vmatrix} V_s \leftarrow \dfrac{V_{EU}}{\Phi_s} - V_c \\ V_s \ \text{ if } \ V_s > 0kN \\ 0kN \ \text{ otherwise} \end{vmatrix} \qquad V_s = 5295\,kN$$

Check that total required nominal shear resistance does not exceed limit:

$$V_{nlimit} := 0.25\cdot f_c\cdot b\cdot d_v \qquad V_{nlimit} = 24872.4\,kN$$

$$CHECK := \begin{vmatrix} \text{"OK"} \ \text{ if } \ V_c + V_s \le V_{nlimit} \\ \text{"FAIL"} \ \text{ otherwise} \end{vmatrix} \qquad CHECK = \text{"OK"}$$

Provide 19mmϕ shear links wth 4 **legs** across the section: $\phi_{link} := 25mm$

$$A_v := \pi\frac{\phi_{link}^{\ 2}}{4}\cdot 4 \qquad A_v = 1963\,mm^2$$

Determine required spacing of transverse reinforcement

$$s_t := \begin{vmatrix} s_{t1} \leftarrow \dfrac{A_v\cdot f_y}{0.0083\cdot\sqrt{\dfrac{f_c}{MPa}}\cdot MPa\cdot b} \\ s_{t2} \leftarrow \dfrac{A_v\cdot f_y\cdot d_v}{V_s} \ \text{ if } \ V_s > 0kN \\ s_{t2} \ \text{ if } \ (V_s > 0kN)\cdot(s_{t2} \le s_{t1}) \\ s_{t1} \ \text{ otherwise} \end{vmatrix} \qquad s_t = 343\,mm$$

Calculate shear stress in order to determine maximum spacing of transverse reinforcement to resist shear:

$$v_u := \frac{V_{EU}}{\Phi_s\cdot(b\cdot d_v)} \qquad v_u = 2.506\,MPa$$

Calculate spacing taking into account maximum spacing requirements:

$$s_{max} := \begin{cases} \min(600mm, 0.8d_v) & \text{if } v_u < 0.125f_c \\ \min(300mm, 0.4d_v) & \text{otherwise} \end{cases}$$

$s_{max} = 600\,mm$

Determine maximum required spacing of transverse reinforcement:

$$s_t := \begin{cases} s_{max} & \text{if } s_{max} \le s_t \\ s_t & \text{otherwise} \end{cases}$$

$s_t = 343\,mm$

Design for Opening Moment and Closing Moment

$Mo_C := M_P$ Plastic moment capacity

$Vo_{col} := V_P$ Plastic hinge shear force

$h_b := v1$ Depth of beam

$b_{je} := (b_f - 2\cdot h6)$ Effective beam width

$h_c := D$ Column width

Calculate Horizontal joint shear force and joint shear stress

Joint shear force

$$V_{jh} := \frac{Mo_C}{h_b}$$

$V_{jh} = 2028.6\,kN$

Joint shear stress

$$v_j := \frac{V_{jh}}{b_{je}\cdot h_c}$$

$v_j = 0.971\,MPa$

CONVENTIONAL CAP BEAM DESIGN

$P_c := \max(|P_{max}|, |P_{min}|)$ Max axial force corresponding plastic hinge effect

$P_c = 5542\,kN$

$P_b := Vo_{col}$

The vertical and horizontal axial stress in the joint, allowing 45 degree spread into the cap beam , is

Vertical

$$f_v := \frac{P_c}{b_{je}\cdot(h_c + 0.5\cdot h_b)}$$

$f_v = 1.183\,MPa$

Horizontal

$$f_h := \frac{P_b}{b_{je}\cdot h_b}$$

$f_h = 0.231\,MPa$

Principal nominal stress

Compresion

$$p_c := \frac{f_v + f_h}{2} + \sqrt{\left(\frac{f_v - f_h}{2}\right)^2 + v_j^2}$$

$p_c = 1.788\,\text{MPa}$

Tension

$$p_t := \frac{f_v + f_h}{2} - \sqrt{\left(\frac{f_v - f_h}{2}\right)^2 + v_j^2}$$

$p_t = -0.374\,\text{MPa}$

$\left|p_t\right| = 0.374\,\text{MPa}$

The joint principal compression stress recommended limited to

$$plimit_c := 0.3f_c$$

$$CHECK_1 := \begin{cases} \text{"OK"} & \text{if } p_c \le plimit_c \\ \text{"DESIGN WELL-CONFINE JOINT"} & \text{otherwise} \end{cases}$$

$CHECK_1 = \text{"OK"}$

The joint principal tension stress recommended limited to

$$plimit_t := 0.29\cdot\sqrt{\frac{f_c}{\text{MPa}}}\cdot\text{MPa}$$

$$CHECK_2 := \begin{cases} \text{"OK"} & \text{if } \left|p_t\right| \le plimit_t \\ \text{"DESIGN WELL-CONFINE JOINT"} & \text{otherwise} \end{cases}$$

$CHECK_2 = \text{"OK"}$

Conclution notes : If principal tension stress is less than $plimit_t = 1.588\,\text{MPa}$, no vertical joint reinforcement is needed, and only nominal tranverse hoop reinforcement is required.

Design for Joint Reinforcement

Closing moment

$d_c := 60\text{mm}$ Clear cover to longitudinal bar

$n_c := 24$ Number of column longitudinal reinforcement

$\phi_{bar} := 32\text{mm}$ Bar diameter

$f_y = 390\,\text{MPa}$ Joint reinforcement nominal yield

$f_{ye} := 1.1f_y$ Steel design yield strength

$fo_{yc} := 1.4\cdot f_y$ Steel yield overstress

$A_{sc} := n_c\cdot\phi_{bar}^{\ 2}\cdot\frac{\pi}{4}$ Area of column reinforcement

$l_a := 2350\text{mm}$ Main column reinforcement ajjacent to beam

$$D_r := D - 2\cdot\left(d_c + \frac{\phi_{bar}}{2}\right)$$

$$f_{sh} := 0.0015\cdot E_s$$

Volumetric ratio of tranverse hoop reinforcement for Low principal tension stress

$$\rho_{s1} := \frac{0.46 \cdot A_{sc} \cdot fo_{yc}}{D_r \cdot l_a \cdot f_{sh}}$$

$$\rho_{s1} = 0.0072536$$

Minimum tranverse hoop reinforcement

$$\rho_{smin} := \frac{0.29 \cdot \sqrt{\frac{f_c}{MPa}} \cdot MPa}{f_y}$$

$$\rho_{smin} = 0.00407$$

$$\rho_s := \max(\rho_{s1}, \rho_{smin})$$

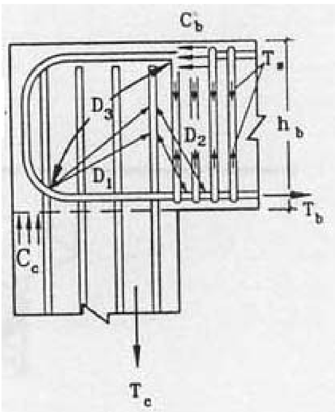
Calculate hoops reinforcement

$\phi_h := 19mm$ hoop reinforcement diameter

$$A_h := \frac{\pi \cdot \phi_h^2}{4}$$

$$s := \frac{4 \cdot A_h}{\rho_s \cdot D_r} \qquad s = 164.928\,mm$$

Opening joint



Anchorage to column bars closest to the cap beam is provided by struts D_1, directed toward the column compression resultant C_c, nd D_2 directed outward the column into the beam. The vertical component D_2, namely T_s, is provided by stirrups close to joint. Tranfers of this tension force to the top of the beam provides the neccesary force to incline the beam compression force C_b into major compression arch D_3. Horizontal component of D_1 and D_2 approximately balance each other, reducing the need for hoop reinforcement.

It is recommended that with this design, 50% of T_c (that portion closest to the inner face of the column) be tranferred by this mechanism. Allowing 50% of this force to be tranferred into to the joint via D_2, the tension force Ts in external beam stirrups is $T_s = 0.25T_c$. Again, approximating $T_c = 0.5A_{sc}Fo_{yc}$, the required amount of vertical beam stirrup reinforcement is

$$f_{yv} := f_y$$

$$A_{jv} := 0.25 \cdot A_{sc} \cdot \frac{fo_{yc}}{f_{yv}}$$

$$A_{jv} = 6755.68\,mm^2$$

Amount vertical reinforcement $A_{jv} = 6755.68\,mm^2$ to be placed over a length not greater than $0.5 \cdot h_b = 1366\,mm$ this is can provide :

$$\phi_v := 25mm$$

$$A_v := \frac{\pi \cdot \phi_v^{\,2}}{4}$$

$$n_v := \frac{A_{jv}}{A_v}$$

$$n_v = 13.763$$

This can be provided by 14 D25 in 7 layer of 2 stirrup leg each.

The strut D2 imposes additional tension force in the beam bottom flrxural reinforcement, as is apparent from equillibrium of force under D2 an Ts. Assuming the special vertical reinforcement to be place over length 0.5 hb from face of column as calculated above, the additional horizontal force to be resisted by the bottom beam reinforcement will be approximatelly).5Ts, the additionall area of beam bottom reinforcement required is thus

$$\Delta A_{sb} := 0.0625 A_{sc} \cdot \frac{fo_{yc}}{f_{yb}} \qquad \text{where :} \qquad f_{yb} := f_y$$

$$\Delta A_{sb} = 1688.9\,mm^2$$

$$n_{sb} := \frac{\Delta A_{sb}}{A_v}$$

$$n_{sb} = 3.441$$

To provide assistance in bond tranfer on top reinforcement and to avoid the total beam tension force being tranferred across to the hook, it is recommended that vertical stirrups, inside the column core, be provided for a vertical resisttance equal to 0.5 $T_{s.}$ this requires an internal vertical joint stirrup area of:

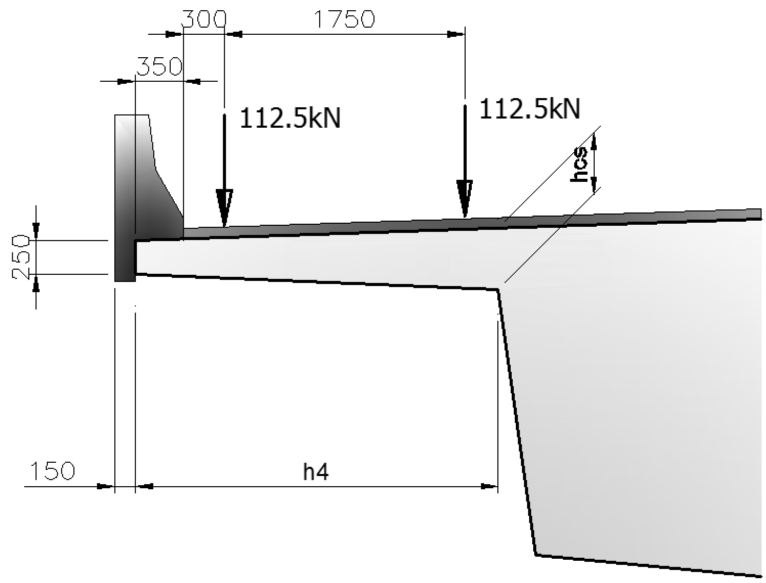
$$A_{vi} := 0.0625 A_{sc} \cdot \frac{fo_{yc}}{f_{yb}}$$

$$A_{vi} = 1689\,mm^2$$

$$n_{vi} := \frac{A_{vi}}{A_v}$$

$$n_{vi} = 3.441$$

Design of Cantilever Slab



Equivalent Width of Deck Overhang (AASHTO LRFD Table 4.6.2.1.3-1)

Width of coping slab at support:

$b := b_f - 2 \cdot h5$ $\qquad b = 1400\,mm$

Distance from outermost load to point of support:

$X := (h4 - 300mm - 350mm)$ $\qquad X = 1510\,mm$

Equivalent width of deck overhang:

$$b_{ew} := \left| \begin{array}{l} b_{ew} \leftarrow 570mm + 0.416 \cdot X \\ b_{ew} \ \text{ if } \ b_{ew} \le b \\ b \ \text{ otherwise} \end{array} \right. \qquad b_{ew} = 1198\,mm$$

Check Deflection of Cantilever Slab (AASHTO LRFD Article 2.5.2.6.2)

Depth of cantilever slab in main deck section at support:

$h_{ds} := 450mm$

Span length of cantilever slab in main deck section:

$l_{ds} := 2645mm$

Depth of deck overhang at support to match main deck outline:

$$h_{cs1} := \left(h_{ds} - 250mm\right) \cdot \frac{h4}{l_{ds}} + 250mm \qquad h_{cs1} = 413\,mm$$

Depth of deck overhang at outermost load point:

$$h_{cs2} := \left(h_{ds} - 250mm\right) \cdot \frac{350mm + 300mm}{l_{ds}} + 250mm \qquad h_{cs2} = 299\,mm$$

Depth of deck overhang at innermost load point:

$$h_{cs3} := \begin{vmatrix} h_{cs3} \leftarrow (h_{ds} - 250mm) \cdot \dfrac{350mm + 300mm + 1750mm}{l_{ds}} + 250mm \\ h_{cs3} \ \text{ if } \ h_{cs3} \le h_{cs1} \\ h_{cs1} \ \text{ otherwise} \end{vmatrix}$$

$$h_{cs3} = 413\,mm$$

Distance from innermost load to point of support:

$$X3 := \begin{vmatrix} X3 \leftarrow h4 - 300mm - 350mm - 1750mm \\ X3 \ \text{ if } \ X3 > 0 \\ 0m \ \text{ otherwise} \end{vmatrix} \qquad X3 = 0\,mm$$

Moment of inertia of equivalent width of deck overhang assuming cracked section:

at support $I_{ew1} := 40 \cdot \% \cdot \dfrac{b_{ew} \cdot h_{cs1}^{\,3}}{12}$

at outer load point $I_{ew2} := 40 \cdot \% \cdot \dfrac{b_{ew} \cdot h_{cs2}^{\,3}}{12}$

at innermost load point $I_{ew3} := 40 \cdot \% \cdot \dfrac{b_{ew} \cdot h_{cs3}^{\,3}}{12}$

The deflection under the applied wheel loads is then given by (increaseed by 30% to account for dynamic loading):

$$\delta_T := \frac{112.5 \cdot kN \cdot 130\%}{E_c} \cdot \left[\int_0^X \left[\frac{x^2}{I_{ew2} + (I_{ew1} - I_{ew2}) \cdot \dfrac{x}{X}} \right] dx + \int_0^{X3} \left[\frac{x^2}{I_{ew3} + (I_{ew1} - I_{ew3}) \cdot \dfrac{x}{X3}} \right] dx \right]$$

$$\delta_T = 2.61\,mm$$

Check that the deflection does not exceed the limit for vehicular load on cantilever arms:

$$\delta_{LIMIT} := \frac{X}{300} \qquad \delta_{LIMIT} = 5.03\,mm$$

$$DEFLECTION_{CHECK} := \begin{vmatrix} \text{"OK"} \ \text{ if } \ \delta_T \le \delta_{LIMIT} \\ \text{"FAIL"} \ \text{ otherwise} \end{vmatrix}$$

$$DEFLECTION_{CHECK} = \text{"OK"}$$

Design for Flexure (AASHTO LRFD Section 5.7)

Nominal bending moment in slab at support

Coping self weight - cantilever wings:

$$w_{cw} := \frac{0.25m + 0.45m}{2} \cdot \left(b_f - h5 \cdot 2\right) 24.5 \cdot \frac{kN}{m^3} \qquad w_{cw} = 12.0 \frac{kN}{m}$$

$$M_{cw} := w_{cw} \cdot \frac{h4^2}{2} \qquad M_{cw} = 28.0\, kN \cdot m$$

Railing dead load - each side:

$$W_r := \frac{0.433}{2} m^2 \cdot \left(b_f - h5 \cdot 2\right) 24.5 \cdot \frac{kN}{m^3} \qquad W_r = 7.4\, kN$$

Railing weight

$$M_r := W_r \cdot (h4 + 0.15m - 0.25m) \qquad M_r = 15.3\, kN \cdot m$$

Superimposed dead load on coping

$$w_{sdl} := 0.125m \cdot \left(b_f - h5 \cdot 2\right) 22.5 \cdot \frac{kN}{m^3} \qquad w_{sdl} = 3.9 \frac{kN}{m}$$

$$M_{sdl} := w_{sdl} \cdot \frac{(h4 - 350mm)^2}{2} \qquad M_{sdl} = 6.4\, kN \cdot m$$

T live load on coping

$$M_t := 112.5 \cdot kN \cdot (X + X3) \qquad M_t = 169.9\, kN \cdot m$$

Total bending moment in slab at face of support - Ultimate Limit State

$$M_{SU} := \left(M_{cw} + M_r\right) \cdot 1.3 + M_{sdl} \cdot 2.0 + \left(M_t \cdot 1.3\right) \cdot 1.8$$

$$M_{SU} = 466.7\, kN \cdot m$$

Depth of section:

$$h_{cs} := h_{cs1} \qquad h_{cs} = 413\, mm$$

Effective depth of slab:

$$d_e := \left(h_{cs} - 90mm\right) \qquad d_e = 323\, mm$$

Effective width of coping slab at support:

$$b_{ew} = 1198\, mm$$

Strength reduction factor for flexure: $\Phi = 0.8$

Determine area of reinforcement, A_f, in the coping to resist flexure
(ref AASHTO LFRD Article 5.7.3.2.3 - see notes on flexural design for rearrangement of terms):

$$R := \frac{M_{SU}}{\Phi \cdot b_{ew} \cdot d_e^{\,2} \cdot f_y} \quad R = 0.0119 \qquad M := \frac{0.85 \cdot f_c}{f_y} \quad M = 0.0654$$

$$\rho := M \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R}{M}}\right) \qquad \rho = 0.013294$$

$$A_f := \rho \cdot d_e \cdot b \qquad b = 1400\,\text{mm}$$

$$A_f = 6017.5\,\text{mm}^2$$

Using 32mmϕ bars gives total number of bars to be distributed across the coping:

$$n_{bf} := \frac{A_f}{804 \cdot \text{mm}^2} \qquad n_{bf} = 7.5$$

Provide 10 No 32mmϕ bars

$$n_{bars} := 10 \qquad A_f := n_{bars} \cdot 804 \cdot \text{mm}^2 \qquad A_f = 8040\,\text{mm}^2$$

Calculate the stress block factor, β_1:

$$\beta_1 := \left| \begin{array}{l} \beta_1 \leftarrow 0.85 \\ \beta_1 \leftarrow \beta_1 - 0.05 \cdot \dfrac{f_c - 28\text{MPa}}{7.\text{MPa}} \quad \text{if } f_c > 28\text{MPa} \\ 0.65 \quad \text{if } \beta_1 < 0.65 \end{array} \right.$$

$$\beta_1 = 0.836$$

For rectangular sections, the depth of concrete in compression, c, is given by:

$$c := \frac{A_f \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} \qquad c = 105\,\text{mm}$$

Check that the required amount of reinforcement does not exceed the maximum allowed by the code:

$$\frac{c}{d_e} = 0.33$$

$$Max_{Limit} := \left| \begin{array}{l} \text{"OK"} \quad \text{if } \dfrac{c}{d_e} \le 0.42 \\ \text{"EXCEEDED"} \quad \text{otherwise} \end{array} \right. \qquad Max_{Limit} = \text{"OK"}$$

Calculate the modulus of rupture, f_r ,of the concrete:

$$f_r := 0.63 \cdot \sqrt{\frac{f_c}{\text{MPa}}} \cdot \text{MPa} \qquad f_r = 3.451\,\text{MPa}$$

Section modulus for the extreme fiber of the section, assuming rectangular section of width b:

$$S_c := \frac{b \cdot h_{cs}^{\,2}}{6} \qquad S_c = 0.04\,\text{m}^3$$

The cracking moment, M_{cr} ,is the given by:

$$M_{cr} := S_c \cdot f_r \qquad M_{cr} = 138\,\text{kN} \cdot \text{m}$$

Check that the reinforcement can develop a resistance moment Mr at least equal to the lesser of:

- 1.2 times the cracking momen Mcr
- 1.33 times the factored moment required by the applicable strength load combination

$$Minimum_{Steel} := \begin{array}{|l} M_r \leftarrow \dfrac{M_{SU}}{\Phi} \\ M \leftarrow \min\left(1.2M_{cr}, 1.33 \cdot M_{EU}\right) \\ \text{"OK"} \quad \text{if } M_r \geq M \\ \text{"NOT SATISFIED"} \quad \text{otherwise} \end{array}$$

$$Minimum_{Steel} = \text{"OK"}$$

<u>CHECK STRESSES AT SLS (AASHTO LRFD Article 5.7.3.4)</u>

The depth of concrete in compression, C, at the serviceability limit state, assuming one level of rebar, is given by:

$$C := \frac{\sqrt{A_f^{\,2} + \dfrac{2 \cdot b \cdot A_f \cdot d_e}{\alpha}} - A_f}{b} \cdot \alpha \qquad C = 128\,\text{mm}$$

Calculate lever arm, z:

$$z := d_e - \frac{C}{3} \qquad z = 281\,\text{mm}$$

Total bending moment in slab at face of support - Service Limit State

$$M_{SS} := \left(M_{cw} + M_r\right) \cdot 1.0 + M_{sdl} \cdot 1.0 + \left(M_t \cdot 1.3\right) \cdot 1.0$$

$$M_{SS} = 270.6\,\text{kN} \cdot \text{m}$$

Calculate the maximum stress in the reinforcement at SLS:

$$f_s := \frac{M_{SS}}{A_f \cdot z} \qquad f_s = 120\,\text{MPa}$$

Check that stress in reinforcement does not exceed limit, f_{sa}:

Crack width parameter $\quad Z := 30000 \cdot \dfrac{N}{mm}$

Depth of concrete from extreme tensile fiber to center of bar $\quad d_c := 90mm$

Area of concrete with same centoid per bar $\quad A := \dfrac{b \cdot d_c \cdot 2}{n_{bars}}$

$$f_{sa} := \begin{array}{|l} f_{sa} \leftarrow \dfrac{Z}{\left(d_c \cdot A\right)^{\frac{1}{3}}} \\ f_{sa} \quad \text{if } f_{sa} < 170MPa \\ 170 \cdot MPa \quad \text{otherwise} \end{array} \qquad f_{sa} = 170\,\text{MPa}$$

$$Stress_{Check} := \begin{array}{|l} \text{"OK"} \quad \text{if } f_s \leq f_{sa} \\ \text{"NOT SATISFIED"} \quad \text{otherwise} \end{array} \qquad Stress_{Check} = \text{"OK"}$$

Calculate forces in section to check calculation result:

Total force in rebar

$T_S := (A_f) \cdot f_s \qquad T_S = 964\,\text{kN}$

Stress in concrete

$f_{sc} := f_s \cdot \dfrac{C}{d_e - C} \cdot \dfrac{1}{\alpha} \qquad f_{sc} = 10.8\,\text{MPa}$

Force in concrete

$C_C := f_{sc} \cdot \dfrac{b \cdot C}{2} \qquad C_C = 964\,\text{kN}$

<u>Design for Shear (AASHTO LRFD Section 5.8)</u>

Critical section for shear shall be taken as d_v from the internal face of the support (AASHTO LRFD Article 5.8.3.2)

$$d_v := \begin{cases} 0.9 \cdot d_e & \text{if } 0.9 \cdot d_e > 0.72 \cdot h_{cs} \\ 0.72 \cdot h_{cs} & \text{otherwise} \end{cases}$$

$d_v = 298\,\text{mm}$

Shear in coping slab at critical section due to loads applied on coping body

Coping cantilever self weight	$V_{cw} := w_{cw} \cdot (h4 - d_v)$	$V_{cw} = 22.4\,\text{kN}$
Railing weight	$V_r := W_r$	$V_r = 7.4\,\text{kN}$
Superimposed dead load on coping	$V_{sdl} := w_{sdl} \cdot (h4 - 600\text{mm} - d_v)$	$V_{sdl} = 5.0\,\text{kN}$
T live load on coping	$V_t := 112.5 \cdot \text{kN}$	$V_t = 112.5\,\text{kN}$

Total shear force in coping slab at face of support - Ultimate Limit State

Shear from loads on coping body

$V_{SU} := (V_{cw} + V_r) \cdot 1.3 + V_{sdl} \cdot 2.0 + (V_t \cdot 1.4) \cdot 1.8$

$V_{SU} = 332.2\,\text{kN}$

Calculate nominal shear resistance of concrete section assuming beam section:

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

$V_c = 379\,\text{kN}$

Strength reduction factor for shear:

$\Phi_s = 0.7$

Required nominal shear resistance of transverse reinforcement:

$$V_s := \begin{array}{|l} V_s \leftarrow \dfrac{V_{SU}}{\Phi_s} - V_c \\ V_s \ \text{ if } \ V_s > 0kN \\ 0kN \ \text{ otherwise} \end{array} \qquad V_s = 96\,kN$$

Check that total required nominal shear resistance does not exceed limit:

$$V_{nlimit} := 0.25 \cdot f_c \cdot b \cdot d_v \qquad V_{nlimit} = 3124.8\,kN$$

$$CHECK := \begin{array}{|l} \text{"OK"} \ \text{ if } \ V_c + V_s \le V_{nlimit} \\ \text{"FAIL"} \ \text{ otherwise} \end{array} \qquad CHECK = \text{"OK"}$$

Provide 13mmϕ shear links with 4 legs across the section

$$\phi_{link} := 13mm$$

$$A_v := \pi \frac{\phi_{link}^2}{4} \cdot 4 \qquad A_v = 531\,mm^2$$

Determine required spacing of transverse reinforcement

$$s_t := \begin{array}{|l} s_{t1} \leftarrow \dfrac{A_v \cdot f_y}{0.0083 \cdot \sqrt{\dfrac{f_c}{MPa}} \cdot MPa \cdot b} \\ s_{t2} \leftarrow \dfrac{A_v \cdot f_y \cdot d_v}{V_s} \ \text{ if } \ V_s > 0kN \\ s_{t2} \ \text{ if } \ (V_s > 0kN) \cdot (s_{t2} \le s_{t1}) \\ s_{t1} \ \text{ otherwise} \end{array} \qquad s_t = 644\,mm$$

Calculate shear stress in order to determine maximum spacing of transverse reinforcement to resist shear:

$$v_u := \frac{V_{SU}}{\Phi_s \cdot (b \cdot d_v)} \qquad v_u = 1.139\,MPa$$

Calculate spacing taking into account maximum spacing requirements:

$$s_{max} := \begin{array}{|l} \min(600mm, 0.8d_v) \ \text{ if } \ v_u < 0.125f_c \\ \min(300mm, 0.4d_v) \ \text{ otherwise} \end{array}$$

$$s_{max} = 238\,mm$$

Determine maximum required spacing of transverse reinforcement:

$$s_t := \begin{array}{|l} s_{max} \ \text{ if } \ s_{max} \le s_t \\ s_t \ \text{ otherwise} \end{array} \qquad s_t = 238\,mm$$

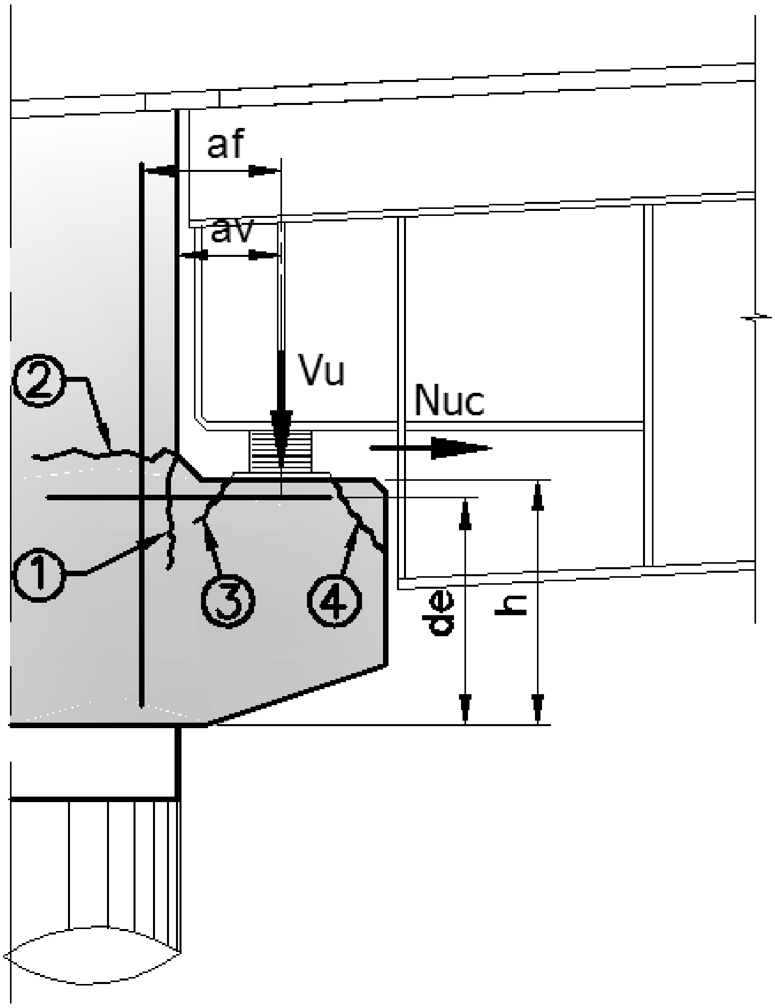
<u>**Provide 13mmϕ links at 150c/c**</u>

Beam Ledge/Corbel Design AASHTO LRFD Article 5.13.2.5

General, Article 5.13.2.5.1

As illustrated below, beam ledges shall resist:

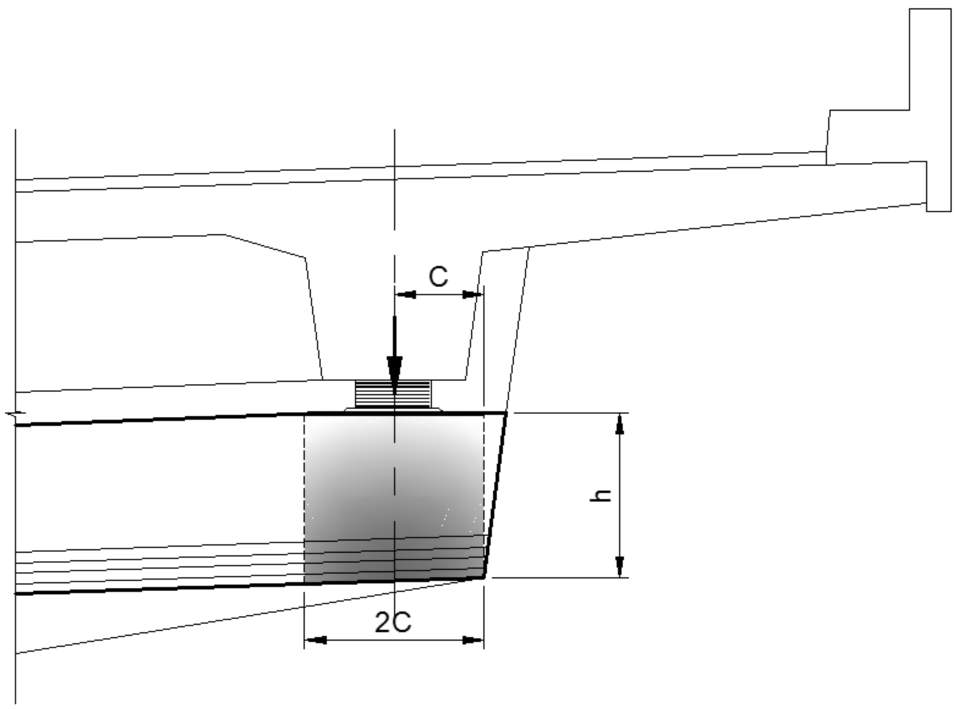
- Flexure, shear and horizontal forces at the loaction of Crack 1;
- Tension force in the supporting element at the location of Crack 2;
- Punching shear at points of loading at the location of Crack 3; and
- Bearing forces at the location of Crack 4



Design for Shear, Article 5.13.2.5.2

Design of beam ledges for shear shall be in accordance with the requirements of shear friction in Article 5.8.4.

The width of the concrete face assumed to participate shall not exceed the width ilustrated below:



Edge distance of bearing	$C := h2$	$C = 800\,mm$
Depth of beam ledge	$h := v3$	$h = 1200\,mm$
Effective depth	$d_e := h - 65mm$	$d_e = 1135\,mm$
Width of the interface	$b_v := 2 \cdot C$	$b_v = 1600\,mm$
Area of concrete resisting shear transfer	$A_{cv} := b_v \cdot d_e$	$A_{cv} = 1.816\ m^2$

Loads on Ledge

PC Deck Dead Load Reaction - max at the bearing - ULS Comb1

$$V2_{upc} := \left|\frac{V2_{u_2}}{2}\right| + \frac{\left|T_{u_2}\right|}{h1_c \cdot 2} \qquad V2_{upc} = 3512.7\,kN$$

Steel Deck Dead Load Reaction - max at the bearing - ULS Comb1

$$V2_{ust} := \left|\frac{V2_{u_3}}{2}\right| + \frac{\left|T_{u_3}\right|}{h1_s \cdot 2} \qquad V2_{ust} = 3232.7\,kN$$

Ultimate shear force in coping from max bearing reactions at face of column:

PC deck reaction $V_{upc} := V2_{upc}$ $V_{upc} = 3512.7\,kN$

Steel deck reaction $V_{ust} := V2_{ust}$ $V_{ust} = 3232.7\,kN$

Maximum design shear force

$V_u := \max(V_{upc}, V_{ust})$ $V_u = 3513\,kN$

Calculate limiting nominal shear strength

$$V_{nlimit} := \begin{cases} 0.2 \cdot f_c \cdot A_{cv} & \text{if } 0.2 \cdot f_c \cdot A_{cv} < 5.5 \cdot \frac{A_{cv}}{mm^2} \cdot N \\ 5.5 \cdot \frac{A_{cv}}{mm^2} \cdot N & \text{otherwise} \end{cases}$$

$V_{nlimit} = 9988\,kN$

Calculate required nominal shear strength and check beam ledge depth

$V_n := \frac{V_u}{\Phi_s}$ $V_n = 5018.2\,kN$

$$Beam_{Ledge} := \begin{cases} \text{"OK"} & \text{if } V_n \le V_{nlimit} \\ \text{"INADEQUATE"} & \text{otherwise} \end{cases}$$

$Beam_{Ledge} = \text{"OK"}$

Calculate shear friction reinforcement, A_{vf}

Cohesion and friction values for monolithically cast concrete

$c := 1.0MPa$ $\lambda := 1.00$ $\mu := 1.4 \cdot \lambda$

Shear reinforcement is then given by:

$$A_{vf} := \begin{array}{|l} A_{vf1} \leftarrow \frac{V_n - c \cdot A_{cv}}{f_y \cdot \mu} \\ A_{vf2} \leftarrow 0.35 \cdot \frac{b_v}{mm} \cdot \frac{MPa}{f_y} \cdot mm^2 \\ A_{vf} \leftarrow A_{vf1} \text{ if } A_{vf1} > A_{vf2} \\ A_{vf} \leftarrow A_{vf2} \text{ if } A_{vf2} > A_{vf1} \\ 0 \text{ if } \frac{V_n}{A_{cv}} < 0.7MPa \end{array}$$

$\frac{V_n}{A_{cv}} = 2.763\,MPa$

$A_{vf} = 5864.8\,mm^2$

Design for Flexure and Horizontal Force, Article 5.13.2.5.3

The area of total primary tension reinforcement shall satisfy the requirements of Article 5.13.2.4.2.

The primary tension reinforcement shall be spaced uniformly with the region 2C.

The section at the face of the support shall be designed to resist simultaneously a factored shear force V_u, a factored moment M_u and a concurrent factored horizontal tensile force N_{uc}.

N_{uc} shall not be taken to be less than $0.2V_u$ and shall be regarded as a live load, even where it results from creep, shrinkage or temperature change.

These provisions apply to beam ledges:

- with a shear span-to-depth ratio av/d_e not greater than unity
- subject to a horizontal tensile force Nuc not larger than Vu

The depth at outside edge of bearing shall not be less than 0.5de, where d_e is effective depth.

Horizontal tensile force $N_{uc} := 0.2 \cdot V_u$ $\quad N_{uc} = 702.5\,kN$

Shear span $a_v := \frac{h5}{2}$ $\quad a_v = 600\,mm$

Design Moment $M_u := V_u \cdot a_v + N_{uc} \cdot (h - d_e)$ $\quad M_u = 2153.3\,kN \cdot m$

Design the primary tensile force reinforcement A_s:

Strength reduction factor for bending $\Phi = 0.8$

Width of section $b := 2 \cdot C$ $\quad b = 1600\,mm$

Determine area of primary reinforcement, A_s, to resist flexure
(ref AASHTO LFRD Article 5.7.3.2.3 - see notes on flexural design for rearrangement of terms):

$$R := \frac{M_u}{\Phi \cdot b \cdot d_e^2 \cdot f_y} \quad R = 0.0033 \qquad M := \frac{0.85 \cdot f_c}{f_y} \quad M = 0.0654$$

$$\rho := M \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R}{M}}\right) \qquad \rho = 0.003439$$

$$A_s := \rho \cdot b \cdot d_e \qquad A_s = 6244.9\,mm^2$$

Design the tensile force reinforcement A_n:

$$A_n := \frac{N_{uc}}{\Phi \cdot f_y} \qquad A_n = 2252\,mm^2$$

Check that the Area of Primary Reinforcement, A_s, satisfies code requirements:

$$A_s := \begin{cases} A_s & \text{if } A_s > \frac{2}{3} \cdot A_{vf} + A_n \\ \frac{2}{3} \cdot A_{vf} + A_n & \text{otherwise} \end{cases}$$

$$A_s := \begin{cases} 0.04 \cdot \frac{f_c}{f_y} \cdot b \cdot d_e & \text{if } A_s < 0.04 \cdot \frac{f_c}{f_y} \cdot b \cdot d_e \\ A_s & \text{otherwise} \end{cases}$$

$$A_s = 6245\,mm^2$$

$$A_s := \frac{A_s}{b} \qquad A_s = 3903\,\frac{mm^2}{m} \qquad A_s = 3.903\,\frac{mm^2}{mm}$$

check area of steel required

$$\rho_{REQUIRED} := \frac{A_s \cdot m}{b \cdot d_e} \cdot 100 \qquad \rho_{REQUIRED} = 0.21$$ PERCENT

Determine area of closed stirrups or ties, Ah:

$$A_h := 0.5 \cdot \left(A_s \cdot b - A_n\right) \qquad A_h = 1997\,mm^2$$

This reinforcement shall be uniformly distributed within two-thirds of the effective depth of the beam ledge adjacent to As.

Anchorage of primary reinforcement:

At the front face of the beam ledge, the primary tension reinforcement, As, shall be anchored by one of the following:

a) a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength fy of As bars

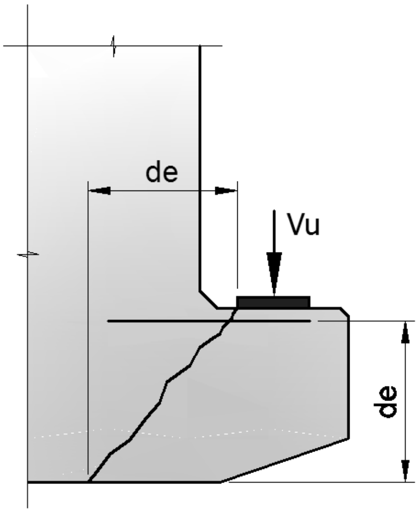
b) bending primary tension bars As back to form a horizontal loop

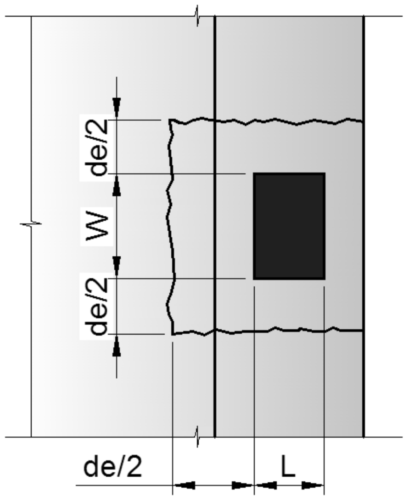
c) some other means of positive anchorage

The bearing area of load on the bracket or corbel shall not project beyond interior face of transverse anchor bar (if one is provided).

Design for Punching Shear, Article 5.13.2.5.4

The truncated pyramids assumed as failure surfaces for punching shear, as illustrated below, shall not overlap.





Applied ultimate reaction	$V_u = 3513\,kN$
Width of bearing	$L = 800\,mm$
Length of bearing	$W = 800\,mm$
Effective depth	$d_e = 1135\,mm$
Bearing pad spacing	$S := \min(h1_c, h1_s) \cdot 2 \qquad S = 6150\,mm$

Nominal punching shear resistance, V_n, shall be taken as:

- At interior pads, or exterior pads, where the end distance C is greater than half the pad spacing S/2:

$$V_{n1} := 0.328 \cdot \sqrt{\frac{f_c}{MPa}} \cdot MPa \cdot (W + 2 \cdot L + 2 \cdot d_e) \cdot d_e \qquad V_{n1} = 9522\,kN$$

- At exterior pads where the end distance C is less than half the pad spacing S/2 and C-0.5W is less than d_e:

$$V_{n2} := 0.328 \cdot \sqrt{\frac{f_c}{MPa}} \cdot MPa \cdot (W + L + d_e) \cdot d_e \qquad V_{n2} = 5577\,kN$$

- At exterior pads where the end distance C is less than half the pad spacing S/2 but C-0.5W is greater than d_e:

$$V_{n3} := 0.328 \cdot \sqrt{\frac{f_c}{MPa}} \cdot MPa \cdot (0.5W + L + d_e + C) \cdot d_e \qquad V_{n3} = 6392\,kN$$

Determine the nominal punching resistance:

$$V_n := \begin{cases} V_{n1} & \text{if } C \ge \dfrac{S}{2} \\ V_{n2} & \text{if } \left(C < \dfrac{S}{2}\right)\cdot\left[(C - 0.5\cdot W) \le d_e\right] \\ V_{n3} & \text{otherwise} \end{cases}$$

$$V_n = 5577\,\text{kN}$$

Check that the nominal punching resistance is adequate:

$$PunchingShear_{CHECK} := \begin{cases} \text{"SATISFIED"} & \text{if } V_n\cdot\Phi_s \ge V_u \\ \text{"FAIL"} & \text{otherwise} \end{cases}$$

$$PunchingShear_{CHECK} = \text{"SATISFIED"}$$

Check Torsional Requirements (AASHTO LRFD Section 5.8)

Check the torsional moment requirements of the beam ledge assuming the bearings (on the steel deck side) are fully loaded and the opposite bearings (on the PC deck side) are loaded only with permanent load:

PC Deck Dead Load Reaction - max at the bearing

$$V2_{pc} := \left|\frac{V2_{dl_2}}{2}\right| + \frac{\left|T_{dl_2}\right|}{h1_c\cdot 2}$$

$$V2_{pc} = 752.9\,\text{kN}$$

Superimposed Dead Load PC Deck Reaction - max at the bearing

$$V2_{st} := \left|\frac{V2_{dl_3}}{2}\right| + \frac{\left|T_{dl_3}\right|}{h1_s\cdot 2}$$

$$V2_{sdl_2} = 290\,\text{kN}$$

Ultimate limit state reaction in bearing under permanent load:

$$V_{upc} := V2_{pc}\cdot 1.3 + V2_{sdl_2}\cdot 2.0$$

$$V_{upc} = 1558\,\text{kN}$$

Calculate torsion in the coping:

$$T_c := \left(V_u - V_{upc}\right)\left(\frac{b_f}{2} - \frac{h5}{2}\right)$$

$$T_c = 2541\,\text{kN}\cdot\text{m}$$

Total shear force in coping at face of column - Ultimate Limit State

Shear from loads on coping body

$$V_{CU} := \left(V_c + V_{cw} + V_r\right)\cdot 1.3 + V_{sdl}\cdot 2.0$$

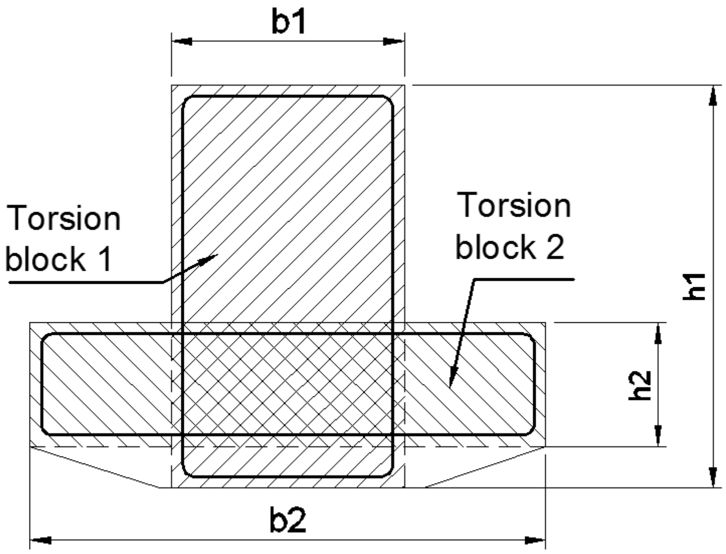
$$V_{CU} = 541.1\,\text{kN}$$

Associated shear force:

$$V_{TU} := V_{CU} + V_{ust} + V_{upc}$$

$$V_{TU} = 5332\,kN$$

The coping will resist torsion moment with two torson blocks as illustrated below. The torsion moment will be distributed into each torsion block in accordance with the relative area of each block.



Dimensions of the torsion blocks are as follows:

Torsion block 1 $\quad h1 := v1 \quad\quad h1 = 2732\,mm$

$b1 := b_f - h5\cdot 2 \quad\quad b1 = 1400\,mm$

Torsion block 1 $\quad h2 := \frac{2}{3}\cdot v3 \quad\quad h2 = 800\,mm$

$b2 := b_f \quad\quad b2 = 3800\,mm$

Area enclosed with centerline of transverse rebar

Torsion block 1 $\quad A_{oh1} := (b1 - 40mm\cdot 2 - 19mm)\cdot(h1 - 40mm\cdot 2 - 19mm)$

Torsion block 2 $\quad A_{oh2} := (b2 - 40mm\cdot 2 - 16mm)\cdot(h2 - 40mm\cdot 2 - 16mm)$

Area enclosed by shear flow path

Torsion block 1 $\quad A_{o1} := 0.85A_{oh1}$

Torsion block 2 $\quad A_{o2} := 0.85A_{oh2}$

Calculate torsion moments carried by each block:

Torsion block 1 $\quad T_{c1} := \frac{A_{o1}}{A_{o1} + A_{o2}}\cdot T_c \quad\quad T_{c1} = 1443\,kN\cdot m$

Torsion block 2 $\quad T_{c2} := \frac{A_{o2}}{A_{o1} + A_{o2}}\cdot T_c \quad\quad T_{c2} = 1098\,kN\cdot m$

Transverse Reinforcement for Shear and Torsion - Torsion Block 1

Depth of section:

$$h_s := v1$$

Effective depth of coping:

$$d_e := (h_s - 150\text{mm}) \qquad d_e = 2582\,\text{mm}$$

Width of coping:

$$b := b_f - h5\cdot 2 \qquad b = 1400\,\text{mm}$$

Critical section for shear shall be taken as d_v from the internal face of the support (AASHTO LRFD Article 5.8.3.2)

$$d_v := \begin{cases} 0.9\cdot d_e & \text{if } 0.9\cdot d_e > 0.72\cdot h_s \\ 0.72\cdot h_s & \text{otherwise} \end{cases}$$

$$d_v = 2323.8\,\text{mm}$$

Calculate nominal shear resistance of concrete section assuming beam section:

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

$$V_c = 2958\,\text{kN}$$

Strength reduction factor for shear:

$$\Phi_s = 0.7$$

Required nominal shear resistance of transverse reinforcement:

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

$$V_s = 4659\,\text{kN}$$

Provide 19mmϕ shear links wth 4 legs across the section

$$\phi_{link} := 19\text{mm}$$

$$A_v := \pi\frac{\phi_{link}^2}{4}\cdot 4 \qquad A_v = 1134\,\text{mm}^2$$

Determine required spacing of transverse reinforcement

$$s_{t1} := \begin{vmatrix} s_{t1} \leftarrow \dfrac{A_v\cdot f_y}{0.0083\cdot\sqrt{\dfrac{f_c}{\text{MPa}}}\cdot\text{MPa}\cdot b} \\ s_{t2} \leftarrow \dfrac{A_v\cdot f_y\cdot d_v}{V_s} \ \text{ if } V_s > 0\text{kN} \\ s_{t2} \ \text{ if } (V_s > 0\text{kN})\cdot(s_{t2} \le s_{t1}) \\ s_{t1} \ \text{ otherwise} \end{vmatrix} \qquad s_{t1} = 221\,\text{mm}$$

For reinforced concrete the angle of inclination of diagonal stresses, θ, can be assumed to be 45 degrees:

$\theta := 45deg$

Determine required transverse reinforcement for torsion:

Area of one leg of transverse torsion reinforcement $A_t := \pi \frac{\phi_{link}^2}{4}$ $A_t = 284\,mm^2$ $\phi_{link} = 19\,mm$

Required spacing of torsional reinforcement $s_{t2} := \frac{2 \cdot A_{o1} \cdot A_t \cdot f_y \cdot \cot(\theta)}{T_{c1}} \cdot \Phi_s$ $s_{t2} = 312\,mm$

Calculate combined spacing of shear and torsion transverse reinforcement:

$$\left(\frac{1}{s_{t1}} + \frac{1}{s_{t2}}\right)^{-1} = 129.303\,mm$$

Provide 19mm dia transverse reinforcement at 100mm c/c in main body of coping

Transverse Reinforcement for Shear and Torsion - Torsion Block 2

Provide 16mmϕ shear links wth 2 legs across the section

$\phi_{link} := 16mm$

$A_v := \pi \frac{\phi_{link}^2}{4} \cdot 2$ $A_v = 402\,mm^2$

Determine required transverse reinforcement for torsion:

Area of one leg of transverse torsion reinforcement $A_t := \pi \frac{\phi_{link}^2}{4}$ $A_t = 201\,mm^2$ $\phi_{link} = 16\,mm$

Required spacing of torsional reinforcement $s_{t2} := \frac{2 \cdot A_{o2} \cdot A_t \cdot f_y \cdot \cot(\theta)}{T_{c2}} \cdot \Phi_s$ $s_{t2} = 222\,mm$

Provide 16mm dia transverse reinforcement at 200mm c/c in beam ledges

Design for Loads from Deck Jacking

AASHTO LRFD requires that the beam ledge is designed to resist deck jacking forces.

The deck jacking loads shall not be less than 1.3 times the permanent load reaction at the bearing adjacent to the point of jacking.

PC Deck Dead Load Reaction - max at the bearing

$$Vdl_{pc} := \left|\frac{V2_{dl_2}}{2}\right| + \frac{\left|T_{dl_2}\right|}{h1_c \cdot 2} \qquad Vdl_{pc} = 752.9\,kN$$

Steel Deck Dead Load Reaction - max at the bearing

$$Vdl_{st} := \left|\frac{V2_{dl_3}}{2}\right| + \frac{\left|T_{dl_3}\right|}{h1_s \cdot 2} \qquad Vdl_{st} = 665.9\,kN$$

PC Deck Superimposed Dead Load Reaction - max at the bearing

$$Vsdl_{pc} := \left|\frac{V2_{sdl_2}}{2}\right| + \frac{\left|T_{sdl_2}\right|}{h1_c \cdot 2} \qquad Vsdl_{pc} = 144.9\,kN$$

Steel Deck Superimposed Dead Load Reaction - max at the bearing

$$Vsdl_{st} := \left|\frac{V2_{sdl_3}}{2}\right| + \frac{\left|T_{sdl_3}\right|}{h1_s \cdot 2} \qquad Vsdl_{st} = 121.2\,kN$$

Maximum design load reaction due to deck jacking:

$$V_{Jack} := 1.3\max\left(Vdl_{pc} + Vsdl_{pc}, Vdl_{st} + Vsdl_{st}\right)$$

$$V_{Jack} = 1167\,kN$$

Assuming the worst case for positioning of the jack and ignoring width of loaded area from jack gives:

Horizontal tensile force	$N_{uc} := 0.2 \cdot V_{Jack}$	$N_{uc} = 233.4\,kN$
Shear span	$a_v := h5$	$a_v = 1200\,mm$
Design Moment	$M_u := V_{Jack} \cdot a_v + N_{uc} \cdot \left(h - d_e\right)$	$M_u = 1078.0\,kN \cdot m$
Loaded width during jacking	$w := 1000mm$	$w = 1000\,mm$
Depth of beam ledge	$h := v3$	$h = 1200\,mm$
Effective depth	$d_e := h - 65mm$	$d_e = 1135\,mm$
Width of the interface	$b_v := w$	$b_v = 1000\,mm$
Area of concrete resisting shear transfer	$A_{cv} := b_v \cdot d_e$	$A_{cv} = 1.135\ m^2$

Calculate required nominal shear strength

$$V_n := \frac{V_{Jack}}{\Phi_s} \qquad V_n = 1667.4\,\text{kN}$$

Calculate shear friction reinforcement, A_{vf}

Cohesion and friction values for monolithically cast concrete

$$c := 1.0\text{MPa}$$

$$\lambda := 1.00$$

$$\mu := 1.4 \cdot \lambda$$

Shear reinforcement is then given by:

$$A_{vf} := \left| \begin{array}{l} A_{vf1} \leftarrow \dfrac{V_n - c \cdot A_{cv}}{f_y \cdot \mu} \\ A_{vf2} \leftarrow 0.35 \cdot \dfrac{b_v}{mm} \cdot \dfrac{MPa}{f_y} \cdot mm^2 \\ A_{vf} \leftarrow A_{vf1} \ \text{if} \ A_{vf1} > A_{vf2} \\ A_{vf} \leftarrow A_{vf2} \ \text{if} \ A_{vf2} > A_{vf1} \\ 0 \ \text{if} \ \dfrac{V_n}{A_{cv}} < 0.7\text{MPa} \end{array} \right. \qquad \frac{V_n}{A_{cv}} = 1.469\,\text{MPa}$$

$$A_{vf} = 975\,\text{mm}^2$$

Design the primary tensile force reinforcement A_s:

Strength reduction factor for bending $\Phi = 0.8$

Width of section $b := 1000\text{mm}$

Determine area of primary reinforcement, A_s, to resist flexure
(ref AASHTO LFRD Article 5.7.3.2.3 - see notes on flexural design for rearrangement of terms):

$$R := \frac{M_u}{\Phi \cdot b \cdot d_e^{\,2} \cdot f_y} \qquad R = 0.0027 \qquad M := \frac{0.85 \cdot f_c}{f_y} \qquad M = 0.0654$$

$$\rho := M \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R}{M}}\right) \qquad \rho = 0.002739$$

$$A_s := \rho \cdot b \cdot d_e \qquad A_s = 3109.2\,\text{mm}^2$$

Design the tensile force reinforcement A_n:

$$A_n := \frac{N_{uc}}{\Phi \cdot f_y} \qquad A_n = 748\,\text{mm}^2$$

Check that the Area of Primary Reinforcement, A_s, satisfies code requirements:

$$A_s := \begin{cases} A_s & \text{if } A_s > \frac{2}{3} \cdot A_{vf} + A_n \\ \frac{2}{3} \cdot A_{vf} + A_n & \text{otherwise} \end{cases}$$

$$A_s := \begin{cases} 0.04 \cdot \frac{f_c}{f_y} \cdot b \cdot d_e & \text{if } A_s < 0.04 \cdot \frac{f_c}{f_y} \cdot b \cdot d_e \\ A_s & \text{otherwise} \end{cases}$$

$$A_s = 3492\,\text{mm}^2$$

$$A_s := \frac{A_s}{b} \qquad A_s = 3492\,\frac{\text{mm}^2}{\text{m}}$$

check area of steel required

$$\rho_{REQUIRED} := \frac{A_s \cdot m}{b \cdot d_e} \cdot 100 \qquad \rho_{REQUIRED} = 0.31$$ PERCENT

Determine area of closed stirrups or ties, Ah:

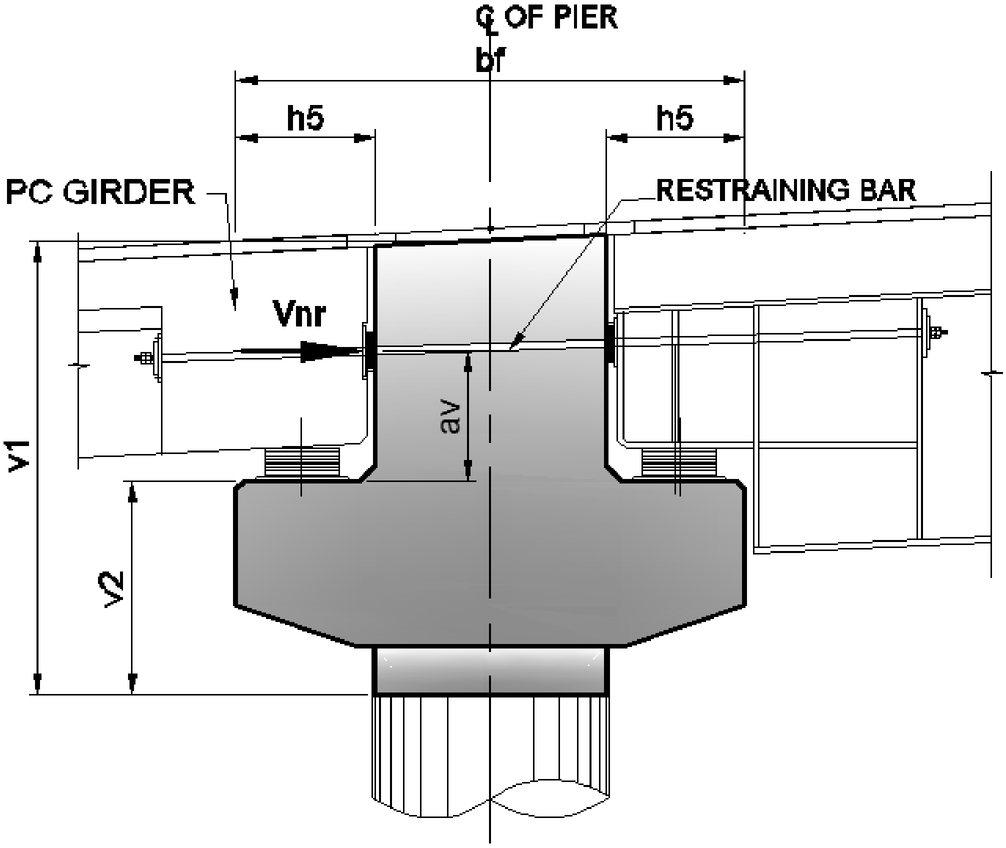
$$A_h := 0.5 \cdot \left(A_s \cdot b - A_n\right) \qquad A_h = 1372\,\text{mm}^2$$

This reinforcement shall be uniformly distributed within two-thirds of the effective depth of the beam ledge adjacent to As.

CONCLUSION - PROVIDE SAME REINFORCEMENT FOR MAIN BEAM LEDGE DESIGN ACROSS ENTIRE BEAM LEDGE WIDTH - EXCEPT OVER WIDTH OF COLUMN.

JACKING POINTS SHALL NOT BE CLOSER THAN 500mm FROM FACE OF COLUMN.

Design for Loads from Longitudinal Restrainers, Article 3.10.9.5



This calculation note is intended for the design of the reinforced concrete elements of the pier coping supporting the restrainers.

For the design of the restrainers themselves, inlcuding any local bursting reinforcement required, refer to a separatee calculation.

Restrainers shall be designed for a force calculated as the acceleration coefficient times the permanent load of the lighter of the two adjoining spans.
Acceleration coefficient

$$A := 0.40$$

Permanent load of the lighter of the two spans assuming, conservatively that only 40% of the total load appears as a reaction at the pier coping:

$$PERM_{LOAD} := \left(V2_{pc} + V2_{sdl_2}\right)\frac{1}{40\%} \qquad PERM_{LOAD} = 2606\,kN$$

Design load in restrainer

$$REST_{LOAD} := PERM_{LOAD} \cdot A \qquad REST_{LOAD} = 1042\,kN$$

Required nominal shear strength of pier coping supporting the restrainers:

$$V_{nr} := \frac{REST_{LOAD}}{\Phi_s} \qquad V_{nr} = 1489\,kN$$

Assume conservatively that this load is carried at a single point with a 150mm x 150mm bearing plate located at mid height of the PC deck. Given that the applied load will be supported by a coping width that is at least equal to the shear span, a_v, of the load - design shear friction reinforcement in accordance with Article 5.13.2.5.2 and Article 5.8.4.

Shear span of the load $a_v := v1 - v2 - \frac{1200mm}{2}$ $a_v = 932\,mm$

The width of the concrete face assumed to participate in resistance to shear is as defined below:

$$b_v := \left| \begin{array}{l} S \leftarrow \min(2 \cdot h1_c, 2 \cdot h1_s) \\ d_w \leftarrow 150mm + 4 \cdot a_v \\ d_w \ \text{ if } \ d_w < S \\ S \ \text{ otherwise} \end{array} \right. \qquad b_v = 3878\,mm$$

Depth of support $h := b_f - h5 \cdot 2$ $h = 1400\,mm$

Effective depth $d_e := h - 65mm$ $d_e = 1335\,mm$

Area of concrete resisting shear transfer $A_{cv} := b_v \cdot d_e$ $A_{cv} = 5.177\ m^2$

Calculate limiting nominal shear strength

$$V_{nlimit} := \left| \begin{array}{l} 0.2 \cdot f_c \cdot A_{cv} \ \text{ if } \ 0.2 \cdot f_c \cdot A_{cv} < 5.5 \cdot \frac{A_{cv}}{mm^2} \cdot N \\ 5.5 \cdot \frac{A_{cv}}{mm^2} \cdot N \ \text{ otherwise} \end{array} \right. \qquad V_{nlimit} = 28474.2\,kN$$

Check that the required nominal shear strength is less than that provided

$$PierCoping_{Capacity} := \left| \begin{array}{l} \text{"OK"} \ \text{ if } \ V_{nr} \le V_{nlimit} \\ \text{"INADEQUATE"} \ \text{ otherwise} \end{array} \right. \qquad PierCoping_{Capacity} = \text{"OK"}$$

Calculate shear friction reinforcement, A_{vf}

Cohesion and friction values for monolithically cast concrete

$c := 1.0MPa$ $\lambda := 1.00$ $\mu := 1.4 \cdot \lambda$

Shear reinforcement is then given by:

$$A_{vf} := \left| \begin{array}{l} A_{vf1} \leftarrow \frac{V_{nr} - c \cdot A_{cv}}{f_y \cdot \mu} \\ A_{vf2} \leftarrow 0.35 \cdot \frac{b_v}{mm} \cdot \frac{MPa}{f_y} \cdot mm^2 \\ A_{vf} \leftarrow A_{vf1} \ \text{ if } \ A_{vf1} > A_{vf2} \\ A_{vf} \leftarrow A_{vf2} \ \text{ if } \ A_{vf2} > A_{vf1} \\ 0 \ \text{ if } \ \frac{V_{nr}}{A_{cv}} < 0.7MPa \end{array} \right. \qquad \frac{V_{nr}}{A_{cv}} = 0.288\,MPa$$

$A_{vf} = 0\,mm^2$

Conclude that the concrete section alone has adequate capacity to carry the loads from the longitudinal restrainers without the need for addiional shear reinforcement.