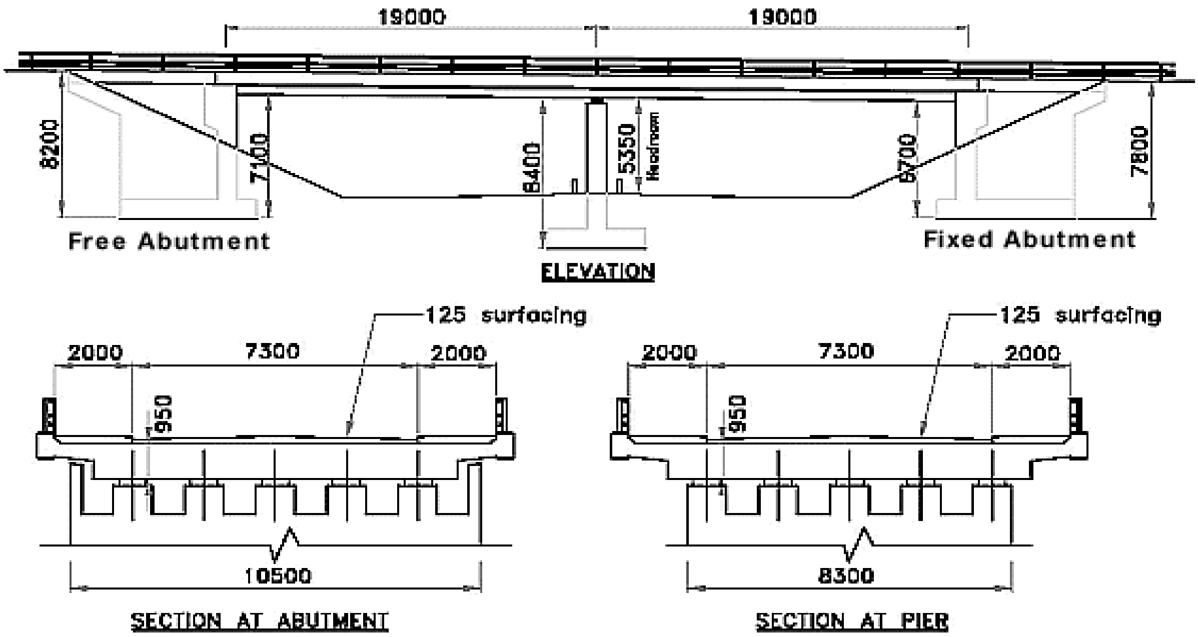
Disclaimer: This example has been prepared to illustrate the design principles for a reinforced concrete bridge deck on reinforced concrete abutments and pier to suit a hypothetical set of conditions. Users of this example should verify for themselves the appropriateness of any parts of the example which they use for their own bridge designs and their own particular conditions, and should check for themselves the correctness of any calculations that they copy. David Childs has prepared this document to assist designers, but he takes no responsibility for how the example is used.



Geometry

2 No 19m spans centre to centre of bearings.
20° Skew.
7.3m wide carriageway with 2 No 2m wide footways.
125mm thick surfacing, which includes waterproofing.
Minimum headroom = 5.3m + S (See DMRB TD 27/05)
National speed limit 60mph (100kph) across deck.
The bridge will carry an Urban All-Purpose Single Carriageway road and is located in York, North Yorkshire, England.

Note: It is important to be involved with the highway design at an early stage to be able to influence the road alignments for the benefit of the bridge geometry and aesthetics. An adequate longitudinal gradient is required to ensure surface water is shed from the deck. Also changes in gradient or horizontal alignment on the deck can result in the deck appearing to be twisted and should be avoided. If the deck needs to accommodate a curved horizontal road alignment then try to ensure that the radius and superelevation are constant throughout the full length of the deck.
If the bridge crosses another road then the road below will need to have suitable visibility through the bridge opening to achieve the correct standard of overtaking and stopping sight distances. Positions of the piers or abutments may need to be adjusted to achieve the correct sight lines.

Soil Investigation Report

It is essential that a soil investigation is carried at the bridge site to obtain details of the ground conditions and soil properties below the abutments and pier foundations. Assume for the example that the following information was obtained:

Abutments to be founded on dense sand and gravel with an allowable bearing pressure of 500 kN/m^2. Predicted settlement from a preliminary estimate of foundation loads is 20 to 25mm.

Centre pier to be founded on lightly weathered mudstone with an allowable bearing pressure of 800 kN/m^2. Predicted settlement from a preliminary estimate of foundation loads is 5 to 10mm.

Test results show the founding strata to have an angle of shearing resistance (φ) = 30^o

Materials

Concrete to BS 8500:2006 (See Tables A.1 and A.5)

Deck concrete : C40/50 for exposure condition XD1 — f_{cu}=50N/mm^2

Note: condition XC3 is allowed under waterproofing but maintain XD1 for the deck.

Parapet cantilever conc. : C40/50 for exposure condition XD3 — f_{cu}=50N/mm^2

Pier concrete : C40/50 for exposure condition XD3 — f_{cu}=50N/mm^2

Abutment and wing wall conc. : C32/40 for exp. condition XD2 — f_{cu}=40N/mm^2

Steel reinforcement : Grade B500B to BS 4449:2005 — f_y=500N/mm^2

Backfill material will be Class 6N in accordance with the Highways Agency Manual of Contract Documents for Highway Works Series 600 clause 610 and Table 6/1 with an effective angle of internal friction (ϕ') = 35^o and density (γ) = 19kN/m^3

Loading From the Deck

Grillage analysis RCGrillageZeroJ gave the following reactions for the various load cases:

Abutment

	Max Reaction (kN)		Total Reaction (kN)	
	SLS	ULS	SLS	ULS
Dead Load	710	834	2200	2592
Dead + HA Load		1270		4302
Dead + HB Load		1385		4469

Pier

	Max Reaction (kN)		Total Reaction (kN)	
	SLS	ULS	SLS	ULS
Permanent Load	2359	2772	7631	8995
Dead + HA + Fwy Load		3589		11985
Dead + HB + Fwy Load		3636		11904

Note: BD 30/87 (DMRB Ref.) Clause 5.2.4.2 says the overturning design criteria given in BS 5400 Pt 2 is not applicable and that factors of safety given in CP2 and BS 2004 shall be used on the nominal loads.

BS 5400 Pt2

Cl. 4.7 Use BS 8004 to design foundations using nominal loads

Loads from the bearings will distribute through the abutment walls and the pier. Bearing loads under dead load are reasonably uniform and contribute to about 50% of the total reaction under dead and live load. Consequently it may be assumed that the distribution of the loads across the width of the bridge at foundation level will be reasonably uniform and can therefore be analysed using a unit strip method.

Estimate nominal loads on a 1 metre width of abutment and pier:

Abutment (10.5m wide)

Dead load = 2200 / 10.5 = **210** kN

Dead + HA Load = 210 + (4302 - 2592) / (1.5 x 10.5) = 210 + **109** = 319 kN

Dead + HB Load = 210 + (4469 - 2592) / (1.3 x 10.5) = 210 + **138** = 348 kN

Pier (8.3m wide)

Permanent Load = 7631 / 8.3 = **919** kN

Dead + HA + Fwy Load = 919 + (11985 - 8995) / (1.5 x 8.3) = 919 + **240** = 1159 kN

Dead + HB + Fwy Load = 919 + (11904 - 8995) / (1.3 x 8.3) = 919 + **270** = 1189 kN

Temperature Effects

Cl. 5.4.2 The bridge is sited in York, England so from Figures 7 and 8 the minimum and maximum shade air temperatures are -17°C and 34°C respectively.

Fig. 9 The deck type is classed as Group 4 (concrete slab).

Table 10 Minimum effective bridge temperature = -10°C

Table 11 Maximum effective bridge temperature = 34°C

Hence temperature range = 34 + 10 = 44°C

Cl. 5.4.6 Range of movement at pier = 44 x 12 x 10^{-6} x 19000 = 10mm

Range of movement at free abutment = 44 x 12 x 10^{-6} x 38000 = 20mm

The maximum reaction on one bearing at the pier is 3636kN; this will limit the type of bearing that may be used. A disc bearing is likely to be suitable rather than an elastomeric bearing.

Freyssinet manufacture Tetron Disc Bearings; their bearing catalogue can be obtained from: **http://www.freyssinet.co.uk/brochures**

Freyssinet D3E 400 is suitable for SLS permanent loads of 2600 kN > 2359

The maximum ULS reaction at one bearing at the abutment = 1385 kN

However the SLS permanent load reaction = 710 kN which governs the bearing: use Freyssinet D3E 125 (800 kN > 710).

Cl. 5.4.7.3 Nominal load from frictional bearing restraint under dead load, superimposed dead load and snow load.

Average SLS Dead + Super'dead reaction at pier = 7631 / 5 = 1526 kN

Average SLS Dead + Super'dead reaction at free abutment = 2200 / 5 = 440 kN

Basic snow loading from BS 6399: Part 3: 1988 = 0.6 kN/m^2
Plan area of one span of deck = 19 x 12.3 = 234 m^2
Total nominal load on one span = 234 x 0.6 = 140 kN.
Reaction on Abutment Bearing = 0.375 x 140 / 5 = 11 kN
Reaction on Pier Bearing = 1.25 x 140 / 5 = 35 kN

Hence total nominal reaction for frictional bearing restraint:
At Pier = 1526 + 35 = 1561 kN
At Abutment = 440 + 11 = 451 kN

Sliding surfaces are made from P.T.F.E. to achieve a low frictional restraint.
D3E 400 bearing (pier) has a disc diameter of 350mm (Area = 96200mm^2)
Bearing Stress = 1561000 / 96200 = 16 N/mm^2
D3E 125 bearing (abutment) has a disc diameter of 195mm (Area = 29900mm^2)
Bearing Stress = 451000 / 29900 = 15 N/mm^2

Coefficient of friction from BS 5400 Pt 9 Section 9.1 Table 3
Coefficient of friction for bearing stress of 10N/mm^2 = 0.06
Coefficient of friction for bearing stress of 20N/mm^2 = 0.04
By interpolation for 15 & 16 N/mm^2 use coefficient of friction = 0.048

Combination 5 loading

Sliding bearings will be provided at the pier and free abutment to accommodate temperature movements in the deck. Consequently the stability and bearing pressures for the pier and free abutment foundations will need to consider frictional restraint loads acting at bearing level.
Similarly the fixed abutment will need to resist the frictional restraint at both the free abutment and the pier.
Restraint at top of pier = 1561 x 0.048 x 5 / 8.3 = **45**kN/m
Restraint at free abutment = 451 x 0.048 x 5 / 10.5 = **10**kN/m
Reaction at fixed abutment = (1561 + 451) x 0.048 x 5 / 10.5 = **46**kN/m

BS 5400 Pt 2
Cl. 6.10 Traction and Braking Load

Nominal Load for HA = 8kN/m x 38m + 250kN = 554kN
Nominal Load for HB = 25% of 30units x 10kN x 4axles = 300kN
554 > 300kN hence HA braking is critical.
Braking load on 1m width of abutment = 554 / 10.5 = **53kN/m**.
This load is applied on the deck and will act on the fixed abutment only.

Allow for traction and braking on the back of the abutment by considering the HB vehicle load = 300kN.
Braking load on 1m width of abutment = 300 / 10.5 = **29kN/m**.
This load is applied at the top of the abutment and can act on both abutments.

Cl. 6.11 Skidding Load

Nominal Load = 300kN
300 < 554kN hence braking load is critical in the longitudinal direction.
When this load is applied on the deck it will act on the fixed abutment only.

CL. 5.3 Wind Loading

Bridge site in York, England:

Cl. 5.3.2.2 $V_s = V_b\ S_p\ S_a\ S_d$

Cl. 5.3.2.2.1 V_b = 24 m/s (from Fig.2)

Cl. 5.3.2.2.2 $S_p = 1.05$

Cl. 5.3.2.2.3 $S_a = 1 + 0.001\Delta$ At 20m above mean sea level $S_a = 1.02$

Cl. 5.3.2.2.4 $S_d = 1.0$

$V_s = 24 \times 1.05 \times 1.02 \times 1.0 = 25.7$ m/s

Cl. 5.3.2.3 $S_g = S_b\ T_g\ S_h'$

Considering wind loading on the side of the deck:

$S_b = S_b'\ K_F$

Cl. 5.3.2.3.1 $S_b' = 1.51$ $K_F = 0.87$ $S_b = 1.51 \times 0.87 = 1.31$

Cl. 5.3.2.3.2 $T_g = 1.0$

Cl. 5.3.2.3.3 $S_h' = 1.0$

$S_g = 1.31 \times 1.0 \times 1.0 = 1.31$

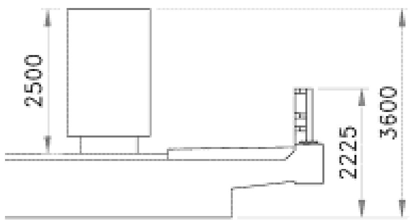
Cl. 5.3.2.1 $V_d = S_g\ V_s$

$V_d = 1.31 \times 25.7 = 33.7$ m/s

Cl. 5.3.3 Transverse wind load = $P_t = q\ A_1\ C_d$

$q = 0.613\ V_d^2 = 0.613 \times 33.7^2 = 696$ N/m^2

Cl. 5.3.3.1.2 $A_1 = 38 \times 3.6 = 136.8$ m^2



Cl. 5.3.3.2.1 $b/d = 12.3 / 3.6 = 3.4$

Fig. 5 $C_d = 1.4$

$P_t = 696 \times 136.8 \times 1.4 \times 10^{-3} = 133.3$ kN

Wind Load
$P_t = 133.3$ kN

BD 30/87 **Abutment Design**

Cl. 5.3 Stability calculations are carried out using active earth pressures.
The strength of the members are designed to resist 'at rest' earth pressures at serviceability and ultimate limit states.

Loading at Rear of Abutment:

Backfill

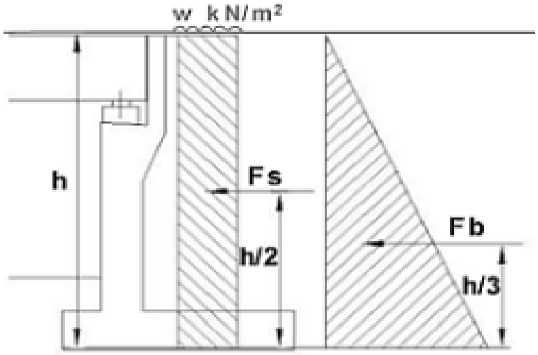
For Stability calculations use active earth pressures = Ka γ h

K_a for Class 6N material:

K_a = (1-Sin35) / (1+Sin35) = 0.27

Density of Class 6N material:

γ = 19kN/m3



Active Pressure at depth h = 0.27 x 19 x h = 5.13h kN/m^2

Hence Fb = 5.13h^2 / 2 = 2.57h^2 kN/m

Cl. 5.8.2 Surcharge

For HA loading surcharge = **10 kN/m^2**

For 30 units of HB loading surcharge = **12 kN/m^2**

Assume a surcharge loading for the compaction plant to be equivalent to 30 units of HB

Hence Compaction Plant surcharge = **12 kN/m^2**.

For surcharge of w kN/m^2 :

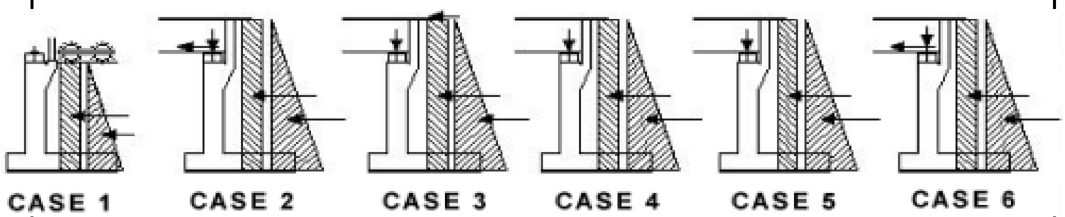
Fs = K_a w h = 0.27wh kN/m

BD 30/87

Cl. 6.3 The abutments are designed in accordance with the Highways Agency document BD 30/87 which requires that drainage is provided at the rear of the abutment wall to collect and dispose of any water percolating through the fill. Pore water pressures will therefore not be allowed to develop on the back of the wall and will not be required in the analysis of the structure.

1) Stability Check

Use spreadsheets "FixedAbut.xls" and "FreeAbut.xls" contained in "104.zip" to analyse load cases. Adjust base dimensions to obtain suitable bearing pressures and factors of safety against sliding and overturning.
The following load combinations are analysed:



Case 1 : Backfill + construction surcharge

Case 2 ; Backfill + HA surcharge + Deck dead load + Deck contraction

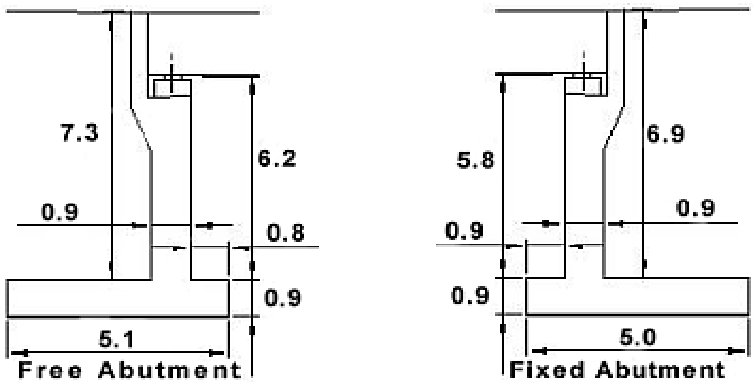
Case 3: Backfill + HA surcharge + Braking behind abutment + Deck dead load

Case 4: Backfill + HB surcharge + Deck dead load

Case 5: Backfill + HA surcharge + Deck dead load + HB on deck

Case 6: Backfill + HA surcharge + Deck dead load + HA on deck + Braking on deck (Fixed Abutment Only)

Analysing the fixed abutment with Load Cases 1 to 6 and the free abutment with Load Cases 1 to 5 using the spreadsheets the following results are obtained:



Free Abutment:

(Units: kN & m)

	Factors of Safety		Bearing Pressures	
	Overturning	Sliding	At Toe	At Heel
Case 1	5.06	2.63	189.00	83.00
Case 2a	3.85	2.73	349.00	31.00
Case 3	3.05	2.50	388.00	0.00
Case 4	4.24	2.82	338.00	45.00
Case 5	4.64	3.27	401.00	33.00

Results obtained using active earth pressures and nominal loads

Wall Moments and Shears

Section	Depth	Moments SLS Dead+Live	Moments SLS Total	ULS	Shear
H/8	0.913	1+28	29	42	52
H/4	1.825	8+60	68	101	75
3H/8	2.738	28+95	123	186	109
H/2	3.65	66+134	200	307	155
5H/8	4.563	128+177	305	474	211
3H/4	5.475	222+222	444	698	279
7H/8	6.388	352+272	624	989	358
H	7.3	525+326	851	1357	447

Base Moments and Shears

	Moments SLS	Moments ULS	Shear
At d from front of wall	0	0	0
At front of wall	145	214	510
At back of wall	727	1139	466

Results obtained using 'at rest' earth pressures and factored loads.

Fixed Abutment:

(Units: kN & m)

	Factors of Safety		Bearing Pressures	
	Overturning	Sliding	At Toe	At Heel
Case 1	5.40	2.67	158.00	96.00
Case 2a	2.85	2.33	376.00	0.00
Case 3	3.18	2.53	356.00	6.00
Case 4	4.54	2.89	307.00	57.00
Case 5	5.03	3.39	367.00	50.00
Case 6	2.83	2.52	451.00	0.00

Results obtained using active earth pressures and nominal loads

Wall Moments and Shears

Section	Depth	Moments SLS Dead+Live	Moments SLS Total	Moments ULS	Shear
H/8	0.863	1+26	27	40	87
H/4	1.725	7+56	63	93	108
3H/8	2.588	23+94	117	175	139
H/2	3.45	55+150	205	311	180
5H/8	4.313	108+210	318	488	230
3H/4	5.175	187+273	460	712	291
7H/8	6.038	297+340	637	993	362
H	6.9	444+409	853	1339	443

Base Moments and Shears

	Moments SLS	Moments ULS	Shear
At d from front of wall	1	2	53
At front of wall	225	309	648
At back of wall	682	1011	461

Results obtained using 'at rest' earth pressures and factored loads.

Reinforcement Design

Substructure concrete : C32/40 for exposure condition XD2

Steel reinforcement : Grade B500B to BS 4449:2005

BS 8500 Pt 1 — f_{cu} = 40 N/mm^2 f_y = 500 N/mm^2

Table A5 — Nominal cover to reinforcement = 45 + Δc = 45 + 15 = 60mm

BS 5400 Pt 4

Cl. 5.6.1.1 — Total maximum ultimate axial load on abutment = 4469 kN

$0.1 f_{cu} A_c = 0.1 \times 40 \times 10^3 \times 10.5 \times 0.9 = 37800$ kN > 4469

Hence design as a cantilever slab in accordance with clause 5.4

Fixed Abutment Wall

Cl. 5.4.2 — Design moments in accordance with clause 5.3.2.

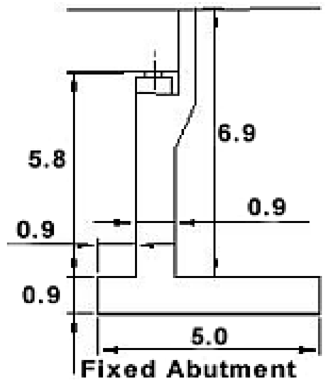
At bottom of wall:

M_{ult} =1339 kNm

M_{sls} = 853 kNm

M_q/M_g = 409/444 = 0.921

V_{ult} = 443 kN



Fixed Abutment

Using spreadsheet 'reinfPt4.xls' contained in 303.zip' then try B32 @ 150 c/c

d = 900 - 60 - 16 = 824mm

$A_s = (\pi \times 32^2 / 4) \times (1000 / 150) = 5362$ mm^2

Cl. 5.3.2.3 — $M_u = (0.87 f_y) A_s z$

z = [1-(1.1 x 500 x 5362) / (40 x 1000 x 824)]d = 0.911d

$M_u = 0.87 \times 500 \times 5362 \times 0.911 \times 824 \times 10^{-6}$ = 1750 kNm > 1339 Hence OK

Check capacity based on limiting concrete strength:

$M_u = 0.15 f_{cu} b d^2 = 0.15 \times 40 \times 1000 \times 824^2 \times 10^{-6}$ = 4074 kNm

> 1750 Hence steel strength governs.

Cl. 5.8.8.1 — Use 25mm aggregate then minimum distance between bars = 25 + 5 = 30mm
In practice this should be an absolute minimum as vibrating pokers will often become jammed; spacings in excess of 50mm should be aimed at.
32mm bars at 150 c/c gives a spacing of 118mm which is satisfactory.

Cl. 5.8.8.2 — Maximum distance between bars for crack control:
Results obtained using spreadsheet 'CrackControl.xls' contained in '302.zip'.

	Concrete Strength f_{cu} =	40	N/mm^2	Concrete Section	
	Steel Strength f_y =	500	N/mm^2	Breadth	Depth
Cl. 4.3.2.2	Young's Modulus for Steel E_s =	200000	N/mm^2	1000	900
Table 13	Environment Conditions for nominal cover:		Severe		
	Reinforcement controlling crack width :			Steel Reinforcement	
	Cover from notional surface =	35	mm	Area	Depth
	Bar diameter (ϕ)=	32	Comp.	0	0
	Spacing (s) =	150	Ten.	5362	824

Factored Dead Load Moment(Mg) = 444 kNm

Factored Live Load Moment(Mq) = 409 kNm

Table 3 — Young's Modulus for Concrete E_c = 31 kN/mm^2

Cl. 4.3.2.1b) — Modified E_c = Ec(1-0.5Mg/(Mg+Mq)) = 22.93 kN/mm^2

x/d_t = 0.285
x = 234.757 mm

Second Mom of Area of Cracked Section = $Ixx = bx^3/3+\alpha_e A_{sc}(x-d_c)^2+\alpha_e A_{st}(d_t-x)^2$
Ixx = 2.06E+10 mm^4
Steel stress = $\sigma_s = \alpha_e(Mg+Mq)(d_t-x)/Ixx$ = 213.319 N/mm^2
Steel strain = $\varepsilon_s = \sigma_s/E_s$ = 0.00107

Distance from Comp Face to notional surface = d_{ns} = 875 mm
Strain at notional surface = $\varepsilon_1 = \varepsilon_s(d_{ns}-x)/(d_t-x)$ = 0.00116
equation 25 — Stiffening Effect of Conc in Tension = 4.54E-05
Modified Strain at notional surface = ε_m = 0.00111
Distance from crack to bar = 74.697 mm

Cl 5.8.8.2 — Design Crack Width = 0.2229 mm

equation 24 — Maximum Crack Width from Table 1 : 0.25 mm — Hence OK

Stress Limitations

Cl 4.1.1.3 — Compressive stress in concrete = 9.74 N/mm^2
Compressive stress due to Temperature Difference = 0 N/mm^2
Total Compressive stress in concrete = 9.74 N/mm^2
Table 2 — Allowable compressive stress = 0.5fcu = 20 N/mm^2 — Hence OK

Tensile stress in steel = 213.319 N/mm^2
Tensile stress due to Temperature Difference = 0 N/mm^2
Total Tensile stress in steel = 231.319 N/mm^2
Table 2 — Allowable tensile stress = 0.75fy = 375 N/mm^2 — Hence OK

At Bottom of Wall
B32@150c/c

Using spreadsheet 'reinfPt4.xls' contained in '303.zip' to determine curtailment of main bars.

At 3H/4 (1.725m above base):

M_{ult} = 712 kNm	**Use B25 @ 150c/c**
M_{sls} = 460 kNm	M_{ult} = 1114 > 712 kNm
M_q/M_g = 273/187 = 1.46	M_{sls} = 569 > 460 kNm
V_{ult} = 291 kN	V_{ult} = 396 > 291 kN

At 3H/8 (4.312m above base):

M_{ult} = 175 kNm	**Use B16 @ 150c/c**
M_{sls} = 117 kNm	M_{ult} = 474 > 175 kNm
M_q/M_g = 94/23 = 4.087	M_{sls} = 237 > 117 kNm
V_{ult} = 139 kN	V_{ult} = 294 > 139 kN

Cl 5.8.9 Shrinkage and temperature reinforcement.

This clause provides a method of determining the reinforcement required which is considered unreliable and the method detailed in BD 28/87 should be used.

BD 28/87 *Results obtained using spreadsheet 'EarlyThermal.xls' contained in '302.zip'*

The wall section is usually poured after the base concrete has cured. This can result in thermal cracking in the wall concrete unless sufficient distribution steel is provided.

Restrained Section Length L = 10500 mm

Restrained Section Thickness T = 900 mm

Reinforcement Strength f_y = 500 N/mm^2

Concrete Strength f_{cu} = 40 N/mm^2

BS 8500-1 Wall concrete : C32/40 for exposure condition XD2

Table A.5 Cement content = 340 kg/m^3

BD 28/87 Assume 18mm ply formwork is used and deck is poured in Summer

Cl.5.8 Short term fall in temperature T_1 = 35 +10 = 45°

A_c for outer 250mm of section for 1m length of section = 500000 mm^2

Tensile strength of immature concrete $f_{ct} = 0.12*f_{cu}^{0.7}$ = 1.5872 N/mm^2

Using the prediction method (Section 5.1)

Minimum area of reinforcement = $f_{ct}*A_c/f_y$ = 1587.17 mm^2/m........(2)

For crack control:

BS 5400-Pt4 f_{ct}/f_b = 0.67 for type 2 deformed bars

Table 1 Crack width = 0.25 mm

Ultimate tensile strain of concrete ε_{ult}= 200 microstrains

Shrinkage strain of concrete $\varepsilon_{sh}= 0.5*\varepsilon_{ult}$ = 100 microstrains

Clause 5.7 Thermal Strain:

Coefficient of thermal expansion = α = 12 x 10^{-6}

Long-term fall in temperature T_2 = 0 (restraint exposed to same climatic exposure)

Thermal strain of concrete $\varepsilon_{th}= 0.8*\alpha*(T_1+T_2)$ = 0.00043

Restraint Factor = 0.6 (Wall cast onto base)

Try 20 mm ϕ bars then:

$A_s = (f_{ct}/f_b)*A_c*\phi*[R*(\varepsilon_{sh}+\varepsilon_{th})-0.5*\varepsilon_{ult}]/(2*w)$ = 2937.28 mm^2/m ....(3)

Dimension of member perpendicular to restraint = 5800 mm

Length to height ratio = 1.81034 :1

Hence minimum area from equation (3) governs = 2937.28 mm^2/m

Minimum area A_s in each face = 1468.6 mm^2/m

Length/Height for reinforcement = 2900 mm

B20 @ 200 c/c = 1571 > 1468.6 mm^2/m Hence OK

Transverse distribution
B20 @ 200c/c up to 2.9m from base

Cl. 5.8.4.2 Minimum area of secondary reinforcement in top half of wall (above 2.9m from base) = 0.12% x b_{td} x d = 0.12% x 1000 x 824 = 989 mm^2/m

Use B16@200 c/c (A_s = 1005 mm^2/m > 989 Hence OK)

B16 @ 200c/c above 2.9m

Cl. 5.4.4 Shear Resistance

Cl. 5.4.4.1 Shear stress = $v = V/bd = 443 \times 10^3 / (1000 \times 824) = 0.54$ N/mm^2

Table 9 $\xi_s = (500/d)^{1/4} = (500 / 824)^{1/4} = 0.883$

$v_c = (0.27/\gamma_m)(100A_s/b_wd)^{1/3}(f_{cu})^{1/3}$

$v_c = (0.27/1.25)(100x5362/1000/824)^{1/3}(40)^{1/3}$

0.64 N/mm^2

Cl. 5.3.3.1 Max. allowable shear stress = $0.75(f_{cu})^{1/2}$ or 4.75 N/mm^2

= $0.75(40)^{1/2}$ = 4.74 or 4.75 i.e. = 4.74 N/mm2

0.54 < 4.75 N/mm^2 Hence OK

Cl. 5.4.4.1 Max. allowable shear stress without shear reinforcement = 0.883x0.64 = 0.565

0.565 < 0.54 Hence shear links not required.

Additional longitudinal reinforcement to enable truss action to develop:

$A_{se} > V/2(0.87f_y)$

$V/2(0.87f_y) = 443 \times 10^3 / (2 \times 0.87 \times 500)$

= 509 mm^2

A_{st} provided = 5362 mm^2

A_{st} req'd for Design Moment = 5362 x (1339 / 1750) = 4103 mm^2

A_{st} available for truss action = 5362 - 4103 = 1259 mm^2 > 509 Hence OK

Fixed Abutment Base

At front face of wall: M_{uls} = 309 kNm

M_{sls} = 225 kNm

Critical Case 6 includes live load HA surcharge and HA braking on deck.
By entering values of zero for these loads then the spreadsheet will provide an estimation of the dead load effects.
This gives SLS Dead Load Moment Mg = 146 kNm

Thus Mq = 225 - 146 = 79 kNm. Mq / Mg = 79 / 146 = 0.54 — **Base Reinf:**

Using spreadsheet 'reinfPt4.xls' contained in '303.zip' then use B16 @ 150 c/c

d = 900 - 60 - 8 = 832mm — Bottom face of toe:

M_u = 474 kNm > 309 Hence OK

M_{crack} = 366 kNm > 225 Hence OK — **B16@150 c/c**

Cl. 5.7.3.2. (a) Check shear at d from the front face of the abutment

At d V_{ult} = 53 kN < 294 (from 'reinfPt4.xls') Hence OK

At rear face of wall: M_{uls} = 1011 kNm

M_{sls} = 682 kNm

Using the same estimation as above SLS Dead Load Moment Mg = 362 kNm

Thus Mq = 682 - 362 = 320 kNm. Mq / Mg = 320 / 362 = 0.88

Using spreadsheet 'reinfPt4.xls' contained in '303.zip' then use B32 @ 150 c/c

d = 900 - 60 - 16 = 824mm — Top face of heel

M_u = 1750 kNm > 1011 Hence OK

M_{crack} = 960 kNm > 682 Hence OK — **B32@150 c/c**

Cl. 5.4.4.1 Check shear at rear face of the abutment

V_{ult} = 461 kN < 466 (from 'reinfPt4.xls') Hence OK

Distribution steel:

Check for early thermal cracking effects of pouring base concrete onto blinding:

Using spreadsheet 'EarlyThermal.xls' contained in '302.zip' — Distribution steel

Minimum steel area required = 1587mm^2/m (793.6 mm^2/m in each face)

Use B16@200 c/c (A_s = 1005 > 793.6 Hence OK) — **B16@200 c/c**

Local Effects on Curtain Wall

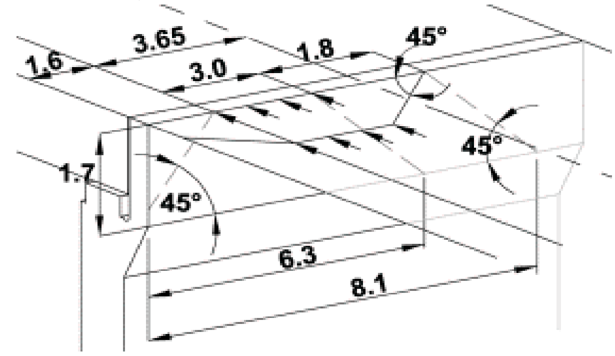
The wall is designed to be cast onto the top of the abutment after the deck has been built. This enables the gap at the end of the deck to be set to the correct width to accommodate the temperature movements.

Loading will be applied to the wall from the backfill, surcharge and braking loads on top of the wall.

BS 5400 Pt 2 Cl. 6.10 Consider HB braking load acting on top of curtain wall.

Braking load on each axle = 25% x 30 units x 10 kN x 4 / 2 = 150 kN

Assume a 45° horizontal dispersal to, and down, the curtain wall and a maximum dispersal of the width of the abutment (10.5m) then:

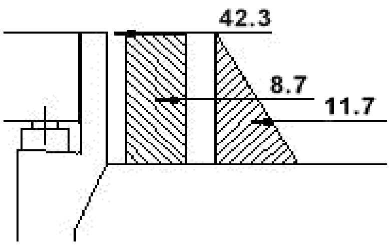


Worst position for HB vehicle will be with wheel adjacent to kerb-line as shown.

First axle load on top of curtain wall = 150 / 3.0 = 50 kN/m
Second axle load on top of curtain wall = 150 / 6.4 = 23.4 kN/m
First axle load on base of curtain wall = 150 / 6.3 = 23.8 kN/m
Second axle load on base of curtain wall = 150 / 8.1 = 18.5 kN/m
Max load at road acting on base of curtain wall = 23.8 + 18.5 = 42.3 kN/m

Bending and Shear at Base of Curtain Wall

Allow 0.6m high for bearing plinth then curtain wall height = 1.1 + 0.6 = 1.7m
Using 'at rest' pressures:
Horizontal load due to HB surcharge = 0.426 x 12 x 1.7 = 8.7 kN/m
Horizontal load due to backfill = 0.426 x 19 x 1.7^2 / 2 = 11.7 kN/m



SLS Moment = (42.3 x 1.7) + (8.7 x 0.85) + (11.7 x 0.567) = 85.9 kNm/m (79.3 live + 6.6 dead)
ULS Moment = 1.1 x {(1.1 x 42.3 x 1.7) + (1.5 x 8.7 x 0.85) + (1.5 x 11.7 x 0.567)} = 110.2 kNm/m
ULS Shear = 1.1 x {(1.1 x 42.3) + (1.5 x 8.7) + (1.5 x 11.7)} = 85 kN/m

Using spreadsheet 'reinfPt4.xls' contained in '303.zip' then:
Use B20 @ 150 c/c in a 350mm thick wall
M_{ult} = 229 > 110.2 kNm/m Hence OK
M_{crack} = 123 > 85.9 kNm/m Hence OK (for 0.25mm crack width)
V_{ult} = 217 > 85 kN/m Hence OK

Note: A computer analysis gives a peak SLS moment under the wheel loads of 115.2kNm which reduces to 85.2kNm at the edge of the wheel loads so M_{crack} is adequate for even the peak value.

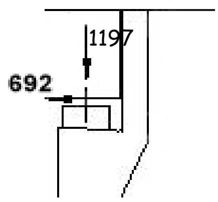
Local Effect on Bearing Plinth

The object of providing a plinth for the bearing is:

1) Provide access to inspect the bearing.
2) Provide a recess adjacent to the bearing to accommodate a jack for lifting the deck to replace the bearings.
3) Provide access to the rear of the bearings to enable the drainage channel to be kept clear.

Worst combined load on fixed bearing plinth = Dead + HA + HA Braking.
Max Dead load reaction at ULS = 834 kN
Max Dead + HA live load reaction at ULS = 1270 kN
γ_{fL} for HA live load = 1.5
γ_{fL} for braking load = 1.25
Approx. ULS vertical reaction for braking = [(1270 - 834) x 1.25 / 1.5] + 834 = 1197 kN
Horizontal ULS load on fixed bearing plinth = 1.25 x 554 = 692 kN

BS 5400 Pt4
Cl. 5.5.1.2 Effective height of plinth l_e = 1.4 l_o (from Table 11)
le = 1.4 x 600 = 840mm



The Freyssinet D3T 500 fixed bearing will accommodate the vertical and lateral loads. The size of the base plate for this bearing is 680 x 515 mm.
Make the bearing plinth 800 x 800mm in plan to accommodate possibly larger bearings to avoid limiting supply by one manufacturer.

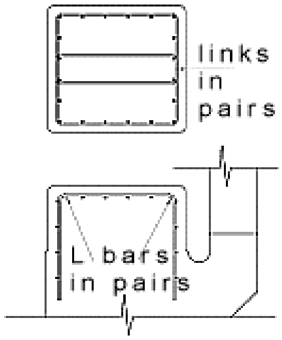
Cl. 5.5.1.1 l_e/h = 840 / 800 = 1.05 < 12 Hence plinth is defined as a short column.
Cl. 5.5.3.1 Increase moment on the plinth to allow for positioning tolerance for bearing.
Eccentricity = 0.05 x 800 = 40 mm but not more than 20mm
Adjusted M = (692 x 0.6) + (1197 x 0.02) = 439 kNm

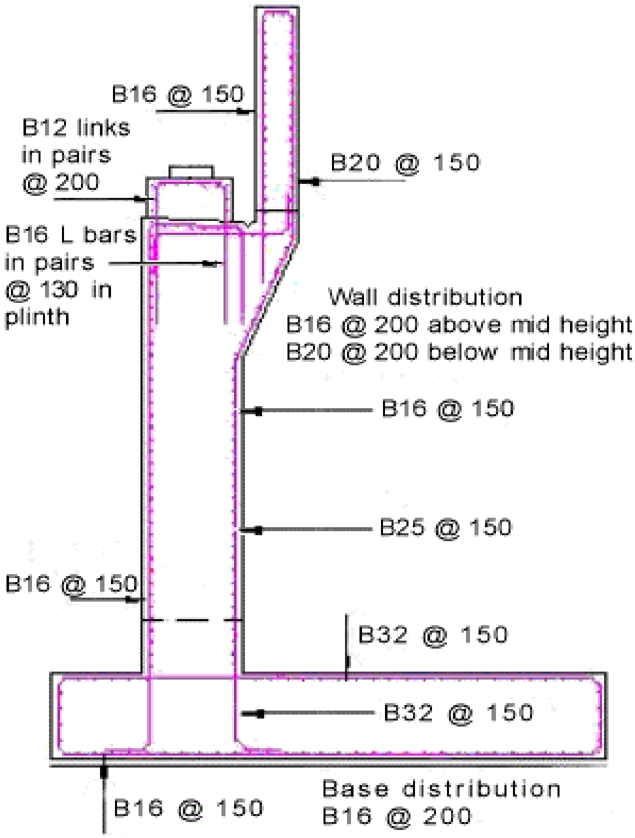
Cl. 5.5.3.5 Simplified design:
Eccentricity e = M / N = 439 / 1197 = 0.367m
Assume B16 reinforcement with 60mm cover and 12mm links then:
d' = d_2 = 60 + 12 + 16/2 = 80mm
(h/2 - d') = (0.8/2 - 0.08) = 0.32m < 0.367m Hence use solution (b)
$M_a = M + N(h/2 - d_2)$
$M_a = \gamma_{f3}[439 + 1197(0.8/2 - 0.08)]$
M_a = 1.1[439 + 383] = 904 kNm
Cl. 5.3.2.3 Approximate z = 0.95d = 0.95 x (800 - 80) = 684mm
$M_u = 0.87 f_y A_s z$
904 = 0.87 x 500 x A_s x 684 x 10^{-6}
A_s = 3038mm^2
Cl. 5.5.3.5 Reduce area of tensile steel by $N/0.87f_y$ = 1197 x 10^3 / (0.87 x 500) = 2752mm^2
A_s required = 3038 - 2752 = 286mm^2
Cl. 5.8.4.1 Minimum Area of tensile reinforcement = 0.15% of $b_a d$
A_s = 0.15 x 800 x 720 / 100 = 864mm^2 (Provide 6 B16 A_s = 1206mm^2)
Provide B16@130 as inverted U bars (formed by L bars in pairs) so that the compression face of the plinth also has B16@130.
With B16's in all faces then total cross-sectional area = 20 x π x 16^2 / 4 = 4021mm^2
1% of cross section = 0.01 x 800 x 800 = 6400mm^2
$0.15N/f_y$ = 0.15 x 1.1 x 1197 x 10^3 / 500 = 395 mm^2 4021 > (min. of 6400 or 395) Hence OK

Cl 5.8.4.3 Minimum Area of links:

Minimum diameter = compression bar dia. / 4 = 5mm
Preferred smallest dia. for all bars = 12mm.
Maximum spacing = compression bar dia. x 12 = 240mm
Also spacing not to exceed 0.75d = 0.75 x 918 = 689mm.

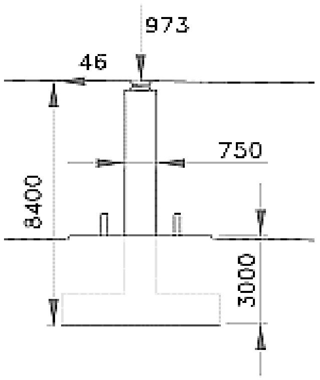
Hence provide 12mm links in pairs at 200 c/c.





Centre Pier

Thickness of wall to accommodate bearing plinth:
Freyssinet bearing D3E 400 has base plate size 555 x 420 + movement in increments of 100mm.
Movement at pier = 10mm so base plate = 555 x 520.
Make pier 750mm thick minimum for bearing.



Nominal reaction for bearing restraint calculation (dead + snow) = 1561 kN
Load per metre of pier = 1561 x 5 / 8.3 = 940 kN/m
Combination 5 loading:
Restraint at top of pier = 45 kN/m

Make an initial estimation of the base size:
Average density of fill and concrete below ground level = (25 + 19) / 2 = 22 kN/m^3
Try base width = 2.5m
Weight = 3.0 x 2.5 x 22 = 165 kN/m
Weight of pier above ground level = 0.75 x 5.4 x 25 = 101 kN/m

Total vertical load P = 940 + 101 + 165 = 1206 kN/m
Moment due to bearing restraint at base = 45 x 8.4 = 378 kNm/m
Eccentricity e = 378 / 1206 = 0.313

Size of Base to suit allowable bearing pressure:
Let B = width of base then for unit length of pier
$P / (B \times 1) + 6 \times P \times e / (B^2 \times 1) = 800$ kN/m^2
$1206 / B + 6 \times 1206 \times 0.313 / B^2 = 800$
$800B^2 - 1206B - 2265 = 0$
$B = [1206 + (1206^2 + 4 \times 800 \times 2265)^{0.5}] / (2 \times 800) = 2.6$m
Make base say 2.8m wide

Check bearing pressure under maximum deck loading (Dead + HB + Ftwy)
Approximate total nominal load = 7631 + (11904 - 8995) / 1.3 = 9869 kN
Load per metre of pier = 9869 / 8.3 = 1189 kN/m
Eccentricity of load due to temperature movement in deck and construction tolerances = e = 10 + 20 = 30mm
Maximum pressure under base = $(1189 / 2.8) + (1189 \times 0.03 \times 6 / 2.8^2)$
= 427 + 27 = 454 kN/m^2 < 800 Hence OK

Check stability about edge of base under frictional bearing restraint condition:
Frictional restraint = 45 kN/m
Load from deck = 940 kN/m
Weight of pier = 101 kN/m
Weight of Base and fill = 2.8 x 3 x 22 = 185 kN/m

Overturning moment = 45 x 8.4 = 378 kNm
Restoring moment = (940 + 101 + 185) x 1.4 = 1716 kNm
Factor of Safety against overturning = 1716 / 378 = 4.54 > 2.0 Hence OK

Factor of safety against sliding = (940 + 101 + 185) x tan 30° / 45 = 15.7 > 2.0 Hence OK

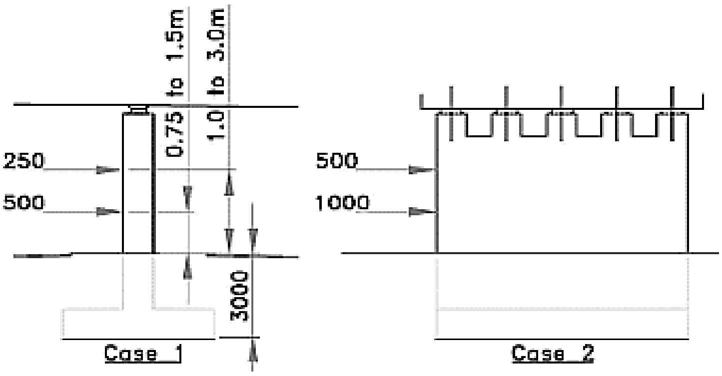
BS 5400 Pt 2 Cl. 6.8

Collision Loading

It is usual for the Highway Authority to adopt the Highways Agency Standard BD 60/04 (DMRB ref. 1.3.5).

BD 60/04 Table 3

Collision loads are considered to act normal to or parallel with the carriageway:



Cl. 2.9(b)

Use 50% of collision loads to check bearing pressures and sliding, and use 100% of collision loads to check for overturning.

Total nominal dead load from the deck = 7631 kN

Weight of pier = 101 x 8.3 = 838 kN

Weight of base and fill = 185 x 8.3 = 1536 kN

Total weight P = 7631 + 838 + 1536 = 10005 kN

Restoring moment about edge of foundation = 10005 x 1.4 = 14007 kNm

Case 1:

With loads at maximum height in range then:

Overturning moment about base = (250 x 6.0) + (500 x 4.5) = 3750 kNm

Factor of Safety against overturning = 14007 / 3750 = 3.7 > 2.0 Hence OK

Moment for bearing pressure = 50% x 3750 = 1875 kNm

Eccentricity = e = 1875 / 10005 = 0.187m

Bearing pressure = [10005 / (2.8 x 8.3)] +/- [(6 x 10005 x 0.187) / (8.3 x 2.8^2)]

Bearing pressure = 431 +/- 173 = 604kN/m^2 & 258kN/m^2 < 800 Hence OK

Case 2:

Total horizontal load = 1000 + 500 = 1500kN

Load required for sliding = 50% x 1500 = 750kN

Load resisting sliding = 10005 x tan 30° = 5776kN (ignoring any passive resistance of soil)

Factor of safety against sliding = 5776 / 750 = 7.7 > 2.0 Hence OK

Hence Pier is stable under collision loading

BS 5400 Pt 4 Pier Reinforcement

Cl. 5.6.1.1 Wall Section: $0.1f_{cu}A_c = 0.1 \times 50 \times 8300 \times 750 \times 10^{-3} = 31125$kN

ULS Weight of wall = 1.15 x 8.4 x 8.3 x 0.75 x 25 = 1503kN

Total ULS load from deck = 11904 kN

Total Ultimate Axial Load = γ_{f3} x (11904 + 1503)

= 1.1 x 13407 = 14748kN

14748 < 31125 Hence design as a cantilever slab in accordance with Cl. 5.4

Assume base thickness = 750mm then:

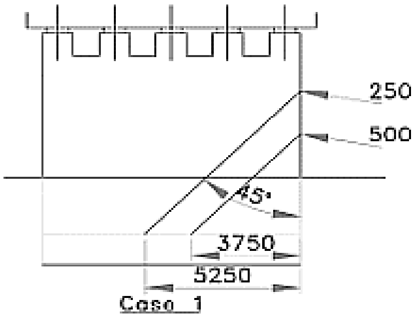
Load case A

Moment at base of wall due to frictional bearing restraint = 45 x 7.65 = 344kNm/m

Combination 5: SLS moment = 344kNm/m

ULS moment = 1.3 x 344 = 447kNm/m

Load case B



Collision load case 1:
Assumed restricted distribution from load down to base of wall as shown.
250 kN load will distribute 5.25m
500 kN load will distribute 3.75m

Moment at base of wall = (250 x 5.25 / 5.25) + (500 x 3.75 / 3.75) = 750 kNm/m (max)

Combination 4: SLS moment = 750 kNm/m

BD 60/04 ULS moment = 1.5 x 750 = 1125 kNm/m

Cl. 2.6 Stage 1 design check is carried out at ULS only.

Hence design moment = 1.1 x 1125 = 1238 kNm

Design shear = 1.1 x 1.5 x (250 / 5.25 + 500 / 3.75) = 299kN/m

Using spreadsheet 'reinfPt4.xls' contained in '303.zip' then:

Use **B32 @ 150 c/c** in a 750mm thick wall

M_{ult} = 1434 > 1238 kNm/m Hence OK

M_{crack} = 432 > 344 kNm/m (Load Case A) Hence OK (for 0.15mm crack width)

V_{ult} = 428 > 299 kN/m Hence OK

Pier wall reinforcement B32@150c/c at base

BD 28/87 Use spreadsheet 'EarlyThermal.xls' contained in '302.zip' to check distribution reinforcement requirements for casting the wall onto the base slab. For a crack width requirement of 0.15mm and restraint factor of 0.6 then **B25 @ 150 c/c** are required to prevent cracking. These bars are required up to mid height of the wall.

B25@150c/c distribution up to mid height

Check Section above collision load (5.25m above base)

Moment in wall due to frictional bearing restraint = 45 x 2.4 = 108kNm/m

Combination 5: SLS moment = 108kNm/m

ULS moment = 1.3 x 108 = 140kNm/m

ULS shear = 1.1 x 1.3 x 45 = 64 kN/m

Using spreadsheet 'reinfPt4.xls' contained in '303.zip' then:

Use **B20 @ 150 c/c**

B20@150c/c at 5.25m above top of base

M_{ult} = 599 > 140 kNm/m Hence OK

M_{crack} = 173 > 108 kNm/m (Load Case A) Hence OK (for 0.15mm crack width)

V_{ult} = 314 > 64 kN/m Hence OK

BS 5400 Pt4 Cl. 5.8.4.2

Minimum area of distribution reinforcement = 0.12% x $b_t d$

= 0.12% x 1000 x (750 - 60 - 10)

= 816 mm^2/m

Use **B16 @ 150 c/c** (A_s = 1340 mm^2/m)

B16@150c/c distribution above mid height

Base Reinforcement

Load case A : Frictional bearing restraint

Load from deck = 940 kN/m

Wall = 7.65 x 0.75 x 25 = 143 kN/m

Fill = 2.05 x 2.25 x 19 = 88 kN/m

Base = 2.8 x 0.75 x 25 = 53 kN/m

Total Vertical load P = 940 + 143 + 88 + 53 = 1224 kN/m

Moment at base = 45 x 8.4 = 378 kNm/m

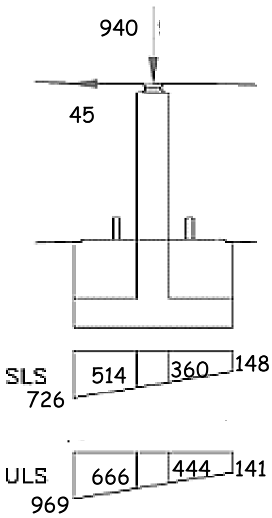
Eccentricity = e = 378 / 1224 = 0.309m

Bearing Pressure at SLS:

σ = 1224 / 2.8 +/- 1224 x 0.309 x 6 / 2.8^2

Max σ = 437 + 289 = 726 kN/m^2

Min σ = 437 - 289 = 148 kN/m^2



At ULS: P = 1.1 x [(1.15 x (940 + 143 + 53) + (1.2 x 88)] = 1553 kN/m

M = 1.1 x 1.3 x 378 = 541 kNm/m

e = 541 / 1553 = 0.348m

Bearing Pressure at ULS:

σ = 1553 / 2.8 +/- 1553 x 0.348 x 6 / 2.8^2

Max σ = 555 + 414 = 969 kN/m^2

Min σ = 555 - 414 = 141 kN/m^2

Maximum Moment to face of pier wall:

M_{sls} = 514 x 1.025^2 / 2 + (726 - 514) x 1.025^2 / 3 - 1.025^2 x (19 x 2.25 + 25 x 0.75) / 2

M_{sls} = 312 kNm/m

M_{uls} = 666x1.025^2/2+(969 - 666)x1.025^2/3 - 1.025^{2}x1.1x(19x2.25x1.2+25x0.75x1.15)/2

M_{uls} = 414 kNm/m

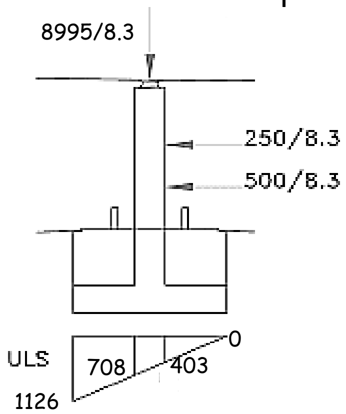
BD 60/04 *Load Case B* : Collision Load Case 1

Cl. 2.9(a) Check base slab for ULS condition only.

At ULS: P = 1.1 x [8995 / 8.3 + (1.15 x (143 + 53) + (1.2 x 88)] = 1556 kN/m

M = 1.1 x 1.5 x (250 x 6 + 500 x 4.5) / 8.3 = 745 kNm/m

e = 745 / 1556 = 0.479m



Bearing Pressure at ULS:

σ = 1556 / 2.8 +/- 1556 x 0.479 x 6 / 2.8^2

Max σ = 556 + 570 = 1126 kN/m^2

Min σ = 556 - 570 = -14 kN/m

Correct for uplift

L = 3 x (2.8/2 - 0.479) = 2.763

Max pressure = 2 x 1556 / 2.763 = 1126 (uplift negligible)

M_{uls} = 708x1.025^2/2+(1126 - 708)x1.025^2/3 - 1.025^{2}x1.1x(19x2.25x1.2+25x0.75x1.15)/2

M_{uls} = 476 kNm/m

Critical design moments:

M_{sls} = 312 kNm/m

M_{uls} = 476 kNm/m

Using spreadsheet 'reinfPt4.xls' contained in '303.zip' then:

Use **B25 @ 150 c/c** with a 750mm deep slab — B25 @ 150 in bottom

M_{ult} = 900 > 476 kNm/m Hence OK

M_{crack} = 467 > 312 kNm/m Hence OK (for 0.25mm crack width)

V_{ult} = 364 kN/m

BS 5400 Pt 4

Cl. 5.7.3.2. (a) Check shear at d from the front face of the abutment

d = 750 - 60 - 13 = 677mm

Bearing pressure at d = 1126 x (2.8 - 1.025 + 0.677) / 2.8 = 986 kN/m^2

Shear at d = (1126+986)x(1.025-0.677)/2-1.1x(1.025-0.677)x(19x2.25x1.2+25x0.75x1.15)

Shear at d = 367 - 28 = 339 kN/m < 364 Hence OK

Distribution steel:

Check for early thermal cracking effects of pouring base concrete onto blinding:

Using spreadsheet 'EarlyThermal.xls' contained in '302.zip'

Minimum steel area required = 1587mm^2/m (793.6 mm^2/m in each face)

Use **B16 @ 200** c/c (A_s = 1005 > 793.6 Hence OK) — B16 @ 200 distribution

Cl. 5.8.4.2 Minimum area of reinforcement = 0.12% x 1000 x 750 = 900mm^2/m

Use **B16 @ 150** c/c in top of slab (A_s = 1340 > 900 Hence OK) — B16 @ 150 in top

Local Effect on Bearing Plinth

1) Worst vertical load on sliding bearing plinth = Dead + HB + Footway

Max vertical reaction at ULS = 3636 kN

2) Worst horizontal load on sliding bearing plinth = Combination 5 loads

Nominal vertical load = N = 1561 kN $\qquad N_{ult}$ = 1561 x 1.3 = 2029 kN

Nominal horizontal load = 1561 x 0.048 = 75kN $\qquad H_{ult}$ = 1.3 x 75 = 98 kN

BS 5400 Pt4

Cl. 5.5.1.2 Effective height of plinth $l_e = 1.4\, l_o$ (from Table 11)

le = 1.4 x 600 = 840mm

Freyssinet bearing D3E 400 has base plate size 555 x 420 + movement in increments of 100mm.
Movement at pier = 10mm so base plate = 555 x 520.
Make plinth 700x700mm

Cl. 5.5.1.1 l_e/h = 840 / 700 = 1.2 < 12 Hence plinth is defined as a short column.

Cl. 5.5.3.1 Increase moment on the plinth to allow for positioning tolerance for bearing.
Eccentricity = 0.05 x 700 = 35 mm but not more than 20mm
1) Moment from max reaction = 3636 x 0.02 = 73 kNm
2) Moment from Comb. 5 load = (98 x 0.6) + (2029 x 0.02) = 99 kNm

Cl. 5.5.3.5 Simplified design:

Eccentricity e = M / N

1) e_1 = 73 / 3636 = 0.02

2) e_2 = 99 / 2029 = 0.05

Assume B12 reinforcement with 60mm cover and 12mm links then:

d' = d_2 = 60 + 12 + 12/2 = 78mm

(h/2 - d') = (0.7/2 - 0.078) = 0.272m > 0.05m

Also $\quad 0.45f_{cu}b(h-2e) = 0.45 \times 50 \times 10^3 \times 0.7 \times (0.7 - 2 \times 0.05)$ = 9450kN > 1.1x3636 =4000

Hence use solution (a) - Provide nominal reinforcement

Cl. 5.8.4.1 Minimum Area of tensile reinforcement = 0.15% of $b_a d$

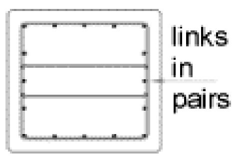
$A_s = 0.15 \times 700 \times 622 / 100 = 653\text{mm}^2$ (Provide 6B12 $A_s = 679\text{mm}^2$)

Provide B12@110 as inverted U bars (formed by L bars in pairs)

With B12's in all faces then total cross-sectional area = $20 \times \pi \times 12^2 / 4 = 2262\text{mm}^2$

1% of cross section = $0.01 \times 700 \times 700 = 4900\text{mm}^2$

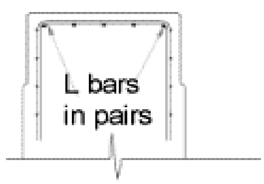
$0.15N/f_y = 0.15 \times 1.1 \times 3636 \times 10^3 / 500 = 1200\text{ mm}^2$



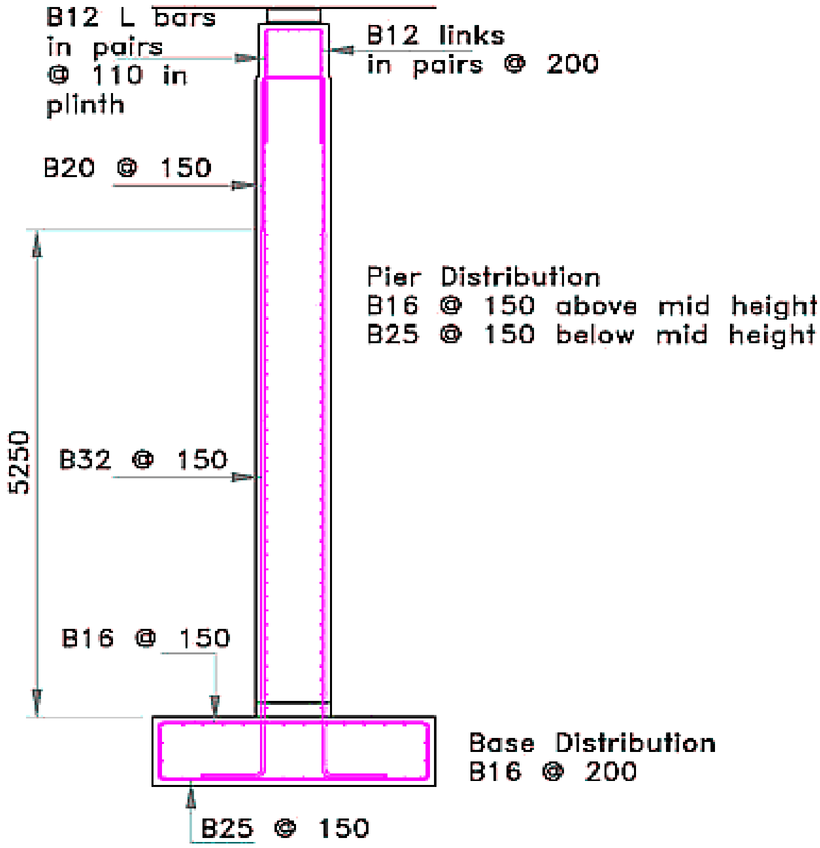
2262 > (minimum of 4900 or 1200) Hence OK

Cl 5.8.4.3 Minimum Area of links:

Minimum diameter = compression bar dia. / 4 = 5mm
Preferred smallest dia. for all bars = 12mm.
Maximum spacing = compression bar dia. x 12 = 240mm
Also spacing not to exceed 0.75d = 0.75 x 918 = 689mm.

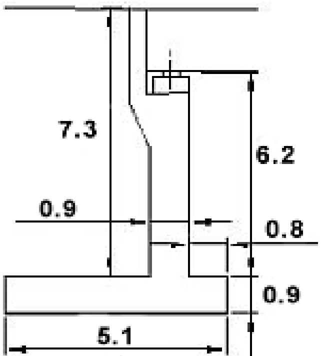


Hence provide 12mm links in pairs at 200 c/c.



Pier Reinforcement Layout

Free Abutment



Free Abutment

Comparing the design load effects in the wall and base with the capacities of the reinforcement used in the fixed abutment then it can be seen that the same reinforcement layout will also be suitable for the free abutment.

Local Effect on Bearing Plinth

The free abutment supports a sliding-guided bearing. This bearing acts in conjunction with the fixed bearing on the fixed abutment to resist all lateral loads acting on the deck.

1) Worst vertical load on sliding-guided bearing plinth = Dead + HB

Max vertical reaction at ULS = 1385 kN

BS 5400 Pt 2 2) Worst horizontal load on sliding-guided bearing plinth:

Cl. 6.11 a) Skidding at free abutment = 300 kN

Cl. 5.3 b) Wind load shared between abutments = P_t / 2 = 133.3 / 2 = 67 kN

300 > 67 kN Hence Skidding worst case

Associated vertical load from HA loading at ULS = 1270 kN

Max horizontal reaction at SLS = 300 kN

Max horizontal reaction at ULS = 1.25 x 300 = 375 kN

Freyssinet bearing D3F 200 would be suitable; this bearing has base plate size 475 x 335 + movement in increments of 100mm.

Movement at free abutment = 20mm so base plate = 475 x 430.

Make sliding-guided bearing plinth 700mm square as for sliding bearings..

BS 5400 Pt4

Cl. 5.5.1.2 Effective height of plinth l_e = 1.4 l_o (from Table 11)

le = 1.4 x 600 = 840mm

Cl. 5.5.1.1 l_e/h = 840 / 700 = 1.2 < 12 Hence plinth is defined as a short column.

Cl. 5.5.3.1 Increase moment on the plinth to allow for positioning tolerance for bearing.

Eccentricity = 0.05 x 700 = 35 mm but not more than 20mm

1) Moment from max reaction = 1385 x 0.02 = 28 kNm

2) Moment from skidding load = (375 x 0.6) + (1270 x 0.02) = 250 kNm

Cl. 5.5.3.5 Simplified design:

Eccentricity e = M / N

1) e_1 = 28 / 1385 = 0.02

2) e_2 = 250 / 1270 = 0.2

Assume B12 reinforcement with 60mm cover and 12mm links then:

d' = d_2 = 60 + 12 + 12/2 = 78mm

(h/2 - d') = (0.7/2 - 0.078) = 0.272m > 0.2m

Also $0.45f_{cu}b(h-2e) = 0.45 \times 50 \times 10^3 \times 0.7 \times (0.7 - 2 \times 0.2)$ = 4725kN > 1.1x1385 = 1524

Hence use solution (a) - Provide nominal reinforcement

Cl. 5.8.4.1 Minimum Area of tensile reinforcement = 0.15% of $b_a d$

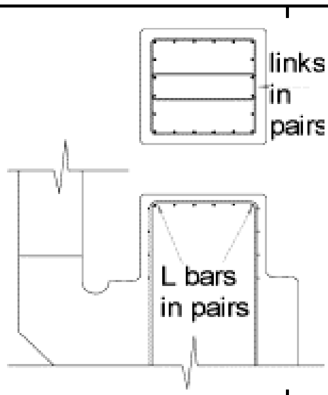
$A_s = 0.15 \times 700 \times 620 / 100 = 651mm^2$ (Provide 6B12 $A_s = 679mm^2$)

Provide B12@110 as inverted U bars (formed by L bars in pairs)

With B12's in all faces then total cross-sectional area = $20 \times \pi \times 12^2 / 4 = 2262mm^2$

1% of cross section = 0.01 x 700 x 700 = 4900mm^2

$0.15N/f_y$ = 0.15 x 1.1 x 1385 x 10^3 / 500 = 457 mm^2

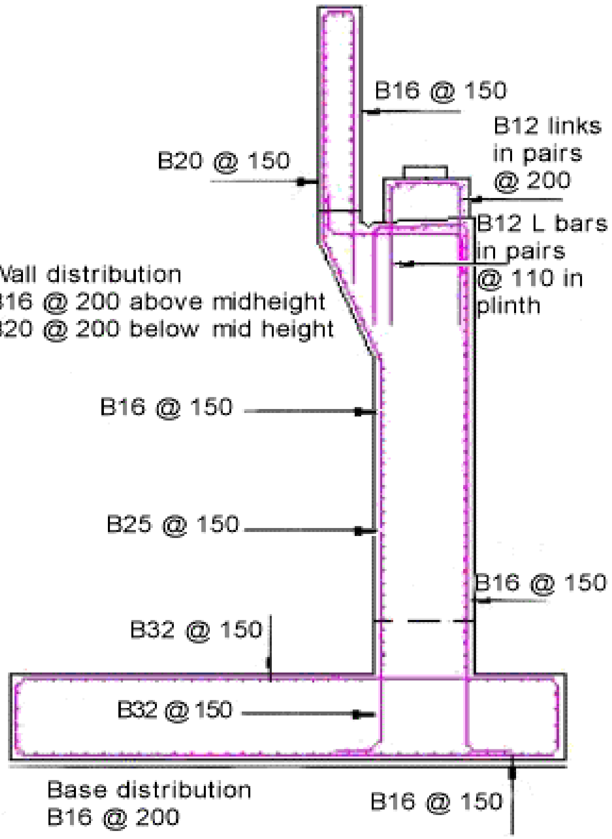


2262 > (minimum of 4900 or 457) Hence OK

Cl 5.8.4.3 Minimum Area of links:

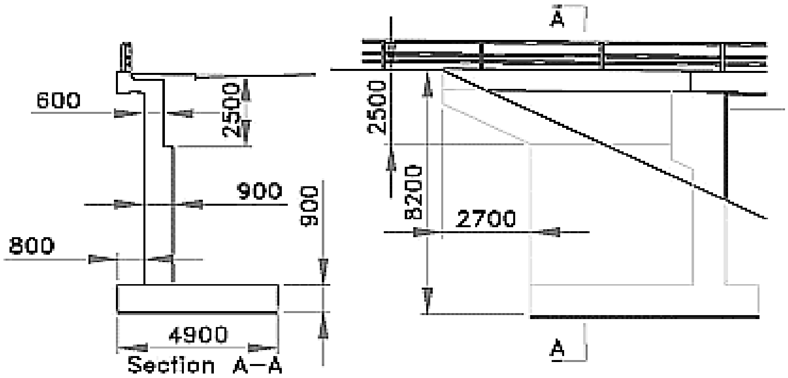
Minimum diameter = compression bar dia. / 4 = 5mm
Preferred smallest dia. for all bars = 12mm.
Maximum spacing = compression bar dia. x 12 = 240mm
Also spacing not to exceed 0.75d = 0.75 x 918 = 689mm.

Hence provide 12mm links in pairs at 200 c/c.



Wing Walls

The wing walls may be designed as free standing cantilever retaining walls. The Abutment spreadsheets can be used to obtain suitable section sizes by considering loading from the backfill and surcharge only.



The wing wall is often made integral with the abutment wall and base. A complicated analysis would be required to obtain the distribution of loading through the structure. This is not usually justified bearing in mind the over-simplification of the soil behaviour on the back of the wall, that is assuming a triangular distribution with the maximum pressure at the base of the wall.

Analysing the wall as a free standing cantilever wall with HA surcharge of 10 kN/m^2 and HB and Construction surcharge of 12 kN/m^2 the load effects at the base of the wall are:

M_{sls} = 662 kNm/m	Using spreadsheet 'reinfPt4.xls' then: Use **B32 @ 150** c/c
Mq/Mg = 136 / 525 = 0.26	M_u = 1750kNm/m > 1092
M_{uls} = 1092 kNm/m	M_{crack} = 1140kNm/m > 662
V_{uls} = 418 kN/m	V_u = 466kN/m > 418

Load effects on the base are:	V_{uls}	M_{uls}	M_{sls}	
At d from front of wall	0	0	0	
At front of wall	363	152	80+13=93	Mq/Mg=0.16
At back of wall	450	986	482+109=591	Mq/Mg=0.23

Using spreadsheet 'reinfPt4.xls' then:

Top of base:	Bottom of base:
Use **B32 @ 150** c/c	Use **B16 @ 150** c/c
M_u = 1750kNm/m > 986	M_u = 474kNm/m > 152
M_{crack} = 1147kNm/m > 591	M_{crack} = 394kNm/m > 93
V_u = 466kN/m > 450	V_u = 394kN/m > 0

Use distribution reinforcement as for the abutment wall:

B16 @ 200 c/c in base slab.

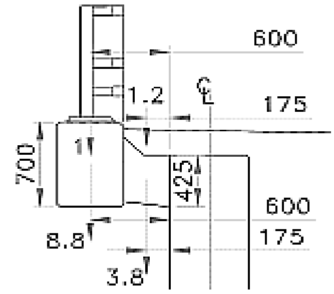
B20 @ 200 c/c in bottom half of wall.

Reduce the thickness of the top section of wall for reduced load effects:
At 3H/8 (2.738m) below ground level on a 600mm thick wall:

	Using spreadsheet 'reinfPt4.xls' then:
M_{sls} = 47 kNm/m	B12 @ 150 c/c
Mq/Mg = 19 / 28 = 0.68	M_u = 172kNm/m > 77
M_{uls} = 77 kNm/m	M_{crack} = 126kNm/m > 47
V_{uls} = 73 kN/m	V_u = 202kN/m > 73

We also need to consider the effects of accidental wheel load on the cantilever and collision loads on the parapet which may increase the reinforcement requirement.

Load Effects on Cantilever Section



Determine load effects about face of wall:
Dead Load:
Parapet = say 1kN/m
Parapet edge beam = 8.8kN/m
Cantilever = 3.8kN/m
Footway fill & surfacing = 1.2kN/m

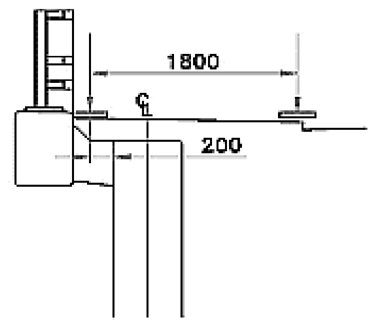
SLS Moment = (1x0.6)+(8.8x0.6)+(3.8x0.175)+(1.2x0.175)
= 0.6 + 5.3 + 0.67 + 0.21 = 6.8 kNm/m

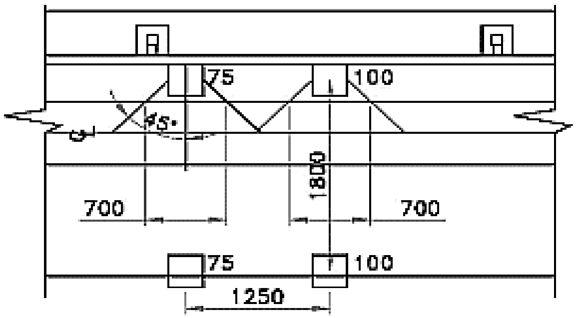
ULS Moment = (1.2x0.6)+(1.15x5.3)+(1.15x0.67)+(1.2x0.21) = 8 kNm/m

ULS Shear = (1.2x1.0)+(1.15x8.8)+(1.15x3.8)+(1.2x1.2) = 17 kN/m

Live Load:
By inspection the accidental wheel load will be more onerous than the pedestrian live load. No other live loads are to be considered.

BS 5400 Pt 2
Cl. 6.6





Section Plan

Assuming the load effects from the outer wheels are dispersed at 45° back to the face of the wall then the length of cantilever supporting these wheels is 0.7m (assuming a 300 x 300mm contact area).

SLS Moment = [1.2x0.2x(100+75)] / 0.7 = 60 kNm/m

ULS Moment = [1.5x0.2x(100+75)] / 0.7 = 75 kNm/m

ULS Shear = 1.5(100+75) / 0.7 = 375 kN/m

BS 8500 Pt 1 Depth of cantilever section = 425mm

Table A5 Deck concrete is grade C40/50 with Class designation XD3 requires a cover to reinforcement of 45 + Δc = 45 + 15 = 60mm

Total SLS Moment = 6.8 + 60 = 66.8 kNm/m

Design ULS Moment = 1.1x(8 + 75) = 91.3 kNm/m

BS 5400 Pt 4 Design ULS Shear = 1.1x(17 + 375) = 431 kN/m

Cl.5.8.8.2 Mq / Mg = 60 / 6.8 = 8.82

Using spreadsheet *'reinfPt4.xls'* contained in *'303.zip' then* Use **B20@150 c/c**

d = 425 - 60 - 10 = 355

M_u = 302kNm/m > 91.3

M_{crack} = 95kNm/m > 66.8

V_u = 479kN/m > 431

Cl. 5.3.3.3 Note: A shear enhancement factor of 2 has been applied to Vu as only a limited length of cantilever has been considered and the section is assumed to act like a beam rather than a slab.

Use spreadsheet *'EarlyThermal.xls'* contained in *'302.zip'* to assess the shrinkage effects of casting the cantilever and edge beam after the wall concrete has cured.

Restrained Section Length L = 7500 mm

Restrained Section Thickness T = 425 mm

Reinforcement Strength f_y = 500 N/mm^2

Concrete Strength f_{cu} = 50 N/mm^2

BS 8500 -1 Cantilever concrete : C40/50 for exposure condition XD3

Table A.5 Cement content = 350 kg/m^3

BD 28/87 Assume 18mm ply formwork is used and deck is poured in Summer

Cl. 5.8 Short term fall in temperature T_1 = 35°

Ac for outer 250mm of section for 1m length of section = 425000 mm^2

Tensile strength of immature concrete $f_{ct} = 0.12 * f_{cu}^{0.7}$ = 1.8555 N/mm^2

Using the prediction method (Section 5.1)

Minimum area of reinforcement = $f_{ct} * A_c / f_y$ = 1577.17 mm^2/m........(2)

For crack control:

BS 5400-Pt4 f_{ct}/f_b = 0.67 for type 2 deformed bars

Table 1 Crack width = 0.25 mm

Ultimate tensile strain of concrete ε_{ult}= 200 microstrains

Shrinkage strain of concrete $\varepsilon_{sh} = 0.5 * \varepsilon_{ult}$ = 100 microstrains

Cl. 5.7 Thermal strain:

Coefficient of thermal expansion = α = 12 x 10^{-6}

Long-term fall in temperature T_2 = 0 (restraint exposed to same climatic exposure)

Thermal strain of concrete eth= $0.8 * \alpha * (T_1 + T_2)$ = 0.00034

Restraint Factor = 0.8 (Edge element cast onto slab)

Try 16 mm ϕ bars then:

As = $(f_{ct}/f_b) * A_c * \phi * [R * (\varepsilon_{sh} + \varepsilon_{th}) - 0.5 * \varepsilon_{ult}] / (2 * w)$ = 2667.07 mm^2/m..................(3)

Hence minimum area from equation (3) governs = 2667.07 mm^2/m

Min As in each face = 2667.07 / 2 = 1333.5 mm^2/m

B16 @ 150 c/c As = 1340 > 1333.5 Hence OK

Transverse Cantilever bars **B16 @ 150** c/c

<u>Secondary Reinforcement</u>

Cl. 5.8.4.2 Minimum area of secondary reinforcement = 0.12% of $b_t d$

0.12% x 1000 x 355 = 426 mm^2/m Use **B12 @ 200** c/c (A_s=565)

TD 19/06 **Parapet Edge Beam**

Cl.4.46 The member supporting the parapet must be designed for a parapet impact loading in accordance with Departmental Standard BD 37.

Cl. 4.48 The supporting structure shall be designed to resist, without damage, all loads that the parapet is capable of transmitting.

The standard of parapet to be used is obtained by carrying out the Road Restraint Risk Assessment Process (RRRAP). An Excel spreadsheet is available from the Highways Agency to enable the assessment to be made.

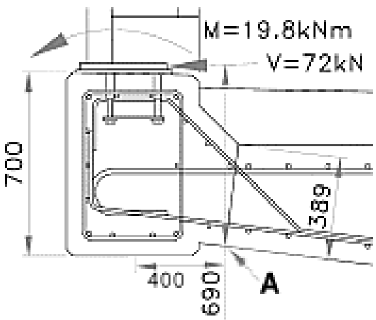
We shall assume for the purpose of the design that the RRRAP indicates that
BS 5400 Pt2 a parapet is required to be provided at the Normal Containment Level N2.
CL. 6.7.1 Parapet is not high containment therefore only local effects need be considered. For metal parapet the nominal collision load is:

a) The ultimate design moment of resistance (M_{ult}) of the post
and
b) the lesser of:
i) M_{ult} of post divided by the height of the lowest rail
or ii) Ultimate Shear Resistance (V_{ult}) of parapet post

We now have to refer to parapet manufacturer's data sheets to obtain details of the post strength and rail layout.
Corus manufacture a Protect 365™ N2 parapet and give the following data:

Post size = 100 x 100mm
Post Ultimate Moment Capacity = 19.8 kNm
Post Ultimate Shear Capacity = 164.9 kN
Height to centre-line of lowest rail = 275mm

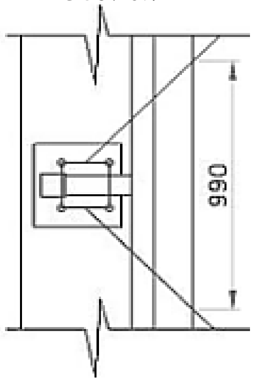
So Nominal loads are:
a) M_{ult} = 19.8 kNm
b) (i) M_{ult} / h = 19.8 / 0.275 = 72 kN
b) (ii) V_{ult} = 164.9 kN
Hence b) V = 72 kN



Section

Check that the B20@150c/c main bars in the top of the cantilever can resist the collision loading on the reduced depth of cantilever adjacent to the parapet edge beam (389mm)

Assume neutral axis of section is near the bottom face of the cantilever at Point A. This is an overestimation, but safe.
Mass of edge beam = 8.8 kN/m
Mass of parapet = 1kN/m approx.



Plan

Assume collision loads on post disperse at 45° from the centre of the anchorage bolts back to the section under consideration, this gives a length of 990mm of cantilever to support the loads locally.

Taking moments at A on 990mm length of cantilever:
SLS Dead Load Mom. = (8.8 + 1) x 0.4 x 0.99 = 3.9 kNm
SLS Live Load Mom. = 1.2 x (19.8 + 0.69 x 72) = 83.4 kNm
Total SLS Moment = 3.9 + 83.4 = 87.3 kNm

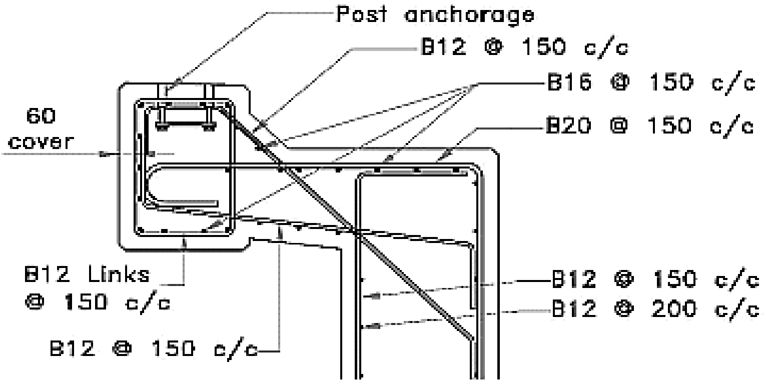
ULS Dead Load Mom. = [(1.15 x 8.8) + (1.2 x 1)] x 0.4 x 0.99 = 4.5 kNm
ULS Live Load Mom. = 1.5 x (19.8 + 0.69 x 72) = 104.2 kNm
BS 8500 Pt 1 — Total ULS Design Moment = 1.1 (4.5 + 104.2) = 119.6 kNm
Table A5 — Cantilever concrete is grade C40/50 with Class designation XD3 requires a cover to reinforcement of 45 + Δc = 45 + 15 = 60mm

d = 389 - 60 - 10 = 319 mm
Mq / Mg = 83.4 / 3.9 = 21.4

Using spreadsheet *'reinfPt4.xls'* contained in *'303.zip'*:
B20@150 c/c gives M_{ult} = 267 kNm & M_{sls} = 84 kNm

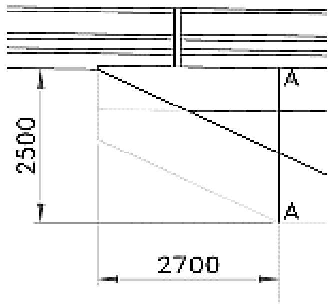
Although the M_{sls} moment is slightly less than required (84 < 87.3) the analysis has been simplified. Sloping bars have been provided to the splay, these are used to prevent cracking in the splay and also to stabilise the reinforcing bars during concreting. These bars will also assist with the bending and shear capacity of the section. — Splay bars B12 @ 150 c/c

ULS Design Shear (Horizontal) = 1.1 x 1.5 x 72 = 119 kN
Assume load on end post, i.e. shear to be resisted on one side:
BS5400 Pt 4 — Assume 12mm ϕ link bars.
Cl. 5.3.3.1 — d = 500 - 60 - 12 - 8 = 420mm (to 16mm ϕ bars on outside face)
$v = 119 \times 10^3 / (700 \times 420) = 0.41$ N/mm^2
Cl. 5.3.3.2 — A_s = Area of longitudinal bars in outside face of edge beam
A_s = 5 No B16 bars from early thermal expansion calculations — Edge Beam
$A_s = 5 \times \pi \times 16^2 / 4 = 1005$ mm^2 — B16 @ 150 c/c
Table 8 — $v_c = (0.27/\gamma_m)(100A_s/b_wd)^{1/3}(f_{cu})^{1/3}$
$v_c = (0.27/1.25)(100\text{x}1005/700/420)^{1/3}(40)^{1/3}$
$= 0.52$ N/mm^2
$\xi_s = (500/d)^{1/4} = (500 / 420)^{1/4} = 1.04$
Table 7 — $\xi_s v_c = 0.52 \times 1.04 = 0.54$ N/mm^2 > 0.41
Hence $A_{sv} > 0.4bs_v/0.87f_{yv}$
Using 12mm ϕ links then $A_s = 2 \times \pi \times 12^2 / 4 = 226$mm^2
Hence $s_v < 226 \times 0.87 \times 500 / (0.4 \times 700) = 351$mm — Edge Beam
Use s_v = 150mm to align with bars in cantilever. — B12 links @ 150
$A_{sa} > V / (2\text{x}0.87\text{x}f_y)$
$V / (2\text{x}0.87\text{x}f_y) = 119 \times 10^3 / (2 \times 0.87 \times 500) = 137$mm^2
A_{sa} = 1005 > 137 Hence OK



End of Cantilever Wall Section

The far end of the wing wall will cantilever from the main wing wall.



To simplify the calculations we shall assume that the wall is a rectangle 2.5 x 2.7m with the soil pressure at 2.5m deep acting on the back of the wall.

Backfill pressure = $K_o\gamma h$ = 0.426 x 19 x 2.5 = 20 kN/m^2

HB surcharge pressure = 0.426 x 12 = 5 kN/m^2

Force on back of wall due to backfill = 20 x 2.5 x 2.7 / 2 = 68 kN
Force on back of wall due to surcharge = 5 x 2.5 x 2.7 = 34 kN
Allow for the shear load from the collision force on an end post = 72 kN
Taking moments about the vertical axis A-A:

M_{sls} = [(68 + 34) x 2.7 / 2] + (72 x 2.7) = 332 kNm

Mq/Mg = 240 / 92 = 2.6

M_{uls} = 1.1 x 1.5 x 332 = 548 kNm

V_{uls} = 1.1 x 1.5 x (68 + 34 + 72) = 287 kNm

Using spreadsheet 'reinfPt4.xls' for a 2.2m length of 600mm thick wall then:

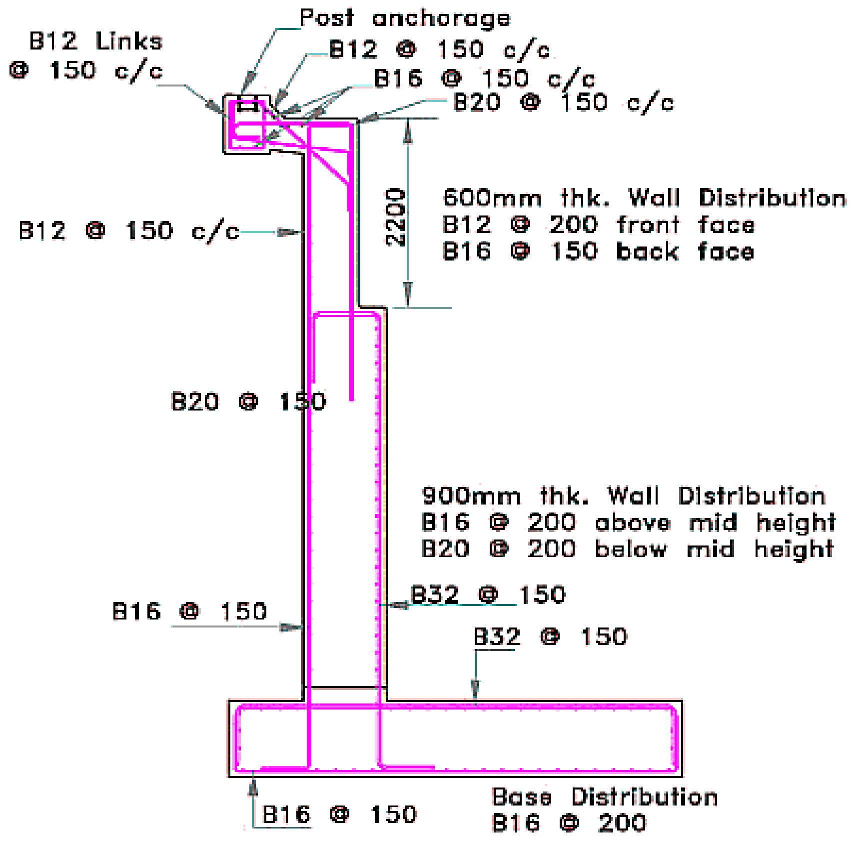
B16 @ 150 c/c

M_u = 659kNm > 548

M_{crack} = 335kNm > 332

V_u = 538kN > 287

This is very conservative as it ignores the capacity of the parapet edge beam, however further refinement is not justified.



Cl. 5.8.6.7 **Reinforcement Lap Lengths**

Lap lengths are usually stated on drawings as being so many bar diameters which simplifies the fixing on site.

The minimum lap length = anchorage length to Cl. 5.8.6.3

Maximum design stress in the bar = $0.87 \times f_y = 0.87 \times 500 = 435$ N/mm^2

Maximum design force in the bar = $435 (\pi \times \phi^2 / 4)$ ……………………………(1)

Cl. 5.8.6.3 Allowable bond stress for (f_{cu} of 50 N/mm^2)= 3.3 N/mm^2 (Table 15)

Maximum force to achieve bond stress = 3.3 (Anchorage length $\times \pi \times \phi$) ……(2)

Equating (1) and (2) then Anchorage length = $[435 / (4 \times 3.3)] \phi = 33 \phi$

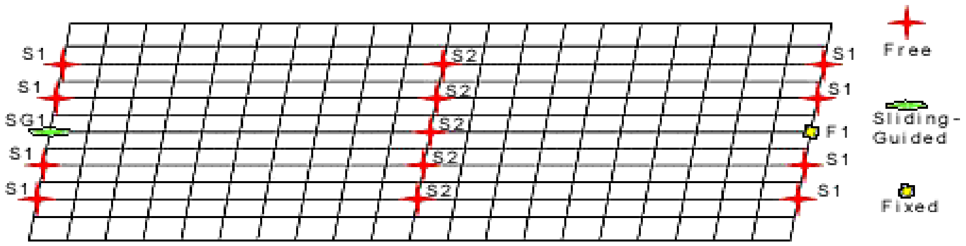
The minimum lap length of 25 times the smaller bar ϕ + 150mm for a 12 mm bar gives a lap length of $(25 + 150 / 12) \phi = 38 \phi$

So generally a minimum lap length of 40 ϕ will be suitable for most instances.

Minimum lap 40 ϕ

Care is needed in detailing to avoid the 1.4 or 2.0 enhancement factor required for the conditions highlighted in (a), (b) and (c) of Cl. 5.8.6.7.
Condition (b) is avoided by staggering bar laps in adjacent bars.

Bearings



The deck design justifies the use of four different bearings:
1) Free sliding bearings on the abutments (8No S1)
2) Free sliding bearings on the piers (5No S2)
3) Fixed bearing on the fixed abutment (1No F1)
4) Sliding guided bearing on the free abutment (1No SG1)

This bearing arrangement will allow the deck to expand in any direction away from the fixed bearing but resist vertical and horizontal loading applied to the deck.

The deck will also deflect due to the loading and produce a rotation at each bearing. This rotation needs to be quantified to ensure that a suitable bearing can be provided to accommodate this movement. Most computer analysis programs will determine the rotations at nodes and can be output in a similar way as the load effects.

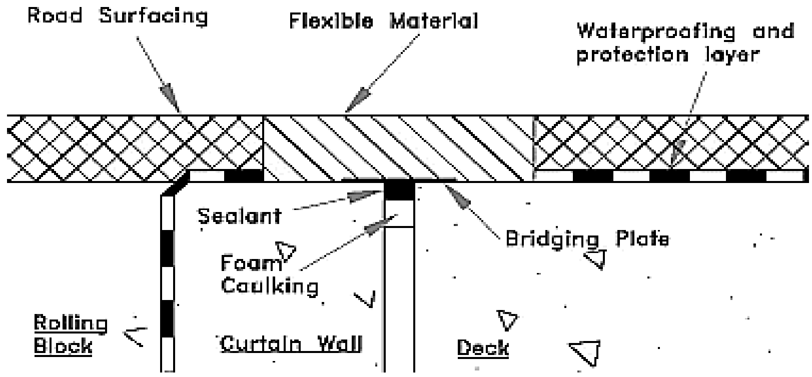
It is useful to prepare a bearing schedule on the basis as described in the Highways Agency Manual of Contact Documents for Highway Works Specification Appendix 21/1:

Bearing identification mark				S1	S2	F1	SG1
Number off				8	5	1	1
Seating material upper surface				concrete	concrete	concrete	concrete
Seating material lower surface				concrete	concrete	concrete	concrete
Allowable average contact pressure (N\mm^2)		Upper face SLS		-	-	-	-
		Upper face ULS		-	-	-	-
		Lower face SLS		15	15	15	15
		Lower face ULS		18	18	18	18
Design load effects (kN)	SLS	Vertical	max.	1176	3049	851	851
			permanent	710	2359	500	500
			min.	665	2272	455	455
		Transverse		0	0	300	300
		Longitudinal		0	0	554	0
	ULS	Vertical		1385	3619	1004	1004
		Transverse		0	0	375	375
		Longitudinal		0	0	692	0
Translation (mm)	SLS	Irreversible	Transverse	1	1	0	0
			Longitudinal	11	6	0	11
		Reversible	Transverse	2	2	0	0
			Longitudinal	20	10	0	20
	ULS	Irreversible	Transverse	1	1	0	0
			Longitudinal	15	7	0	15
		Reversible	Transverse	3	3	0	0
			Longitudinal	26	13	0	26
Rotation (radians)	SLS	Irreversible	Transverse	0	0	0	0
			Longitudinal	0	0	0	0
		Reversible	Transverse	0.0006	0.0001	0.0006	0.0006
			Longitudinal	0.0031	0.0001	0.0031	0.0029
	Maximum rate (radians/100kN)		Transverse	0.00004	3.35×10^{-6}	0.00004	0.00004
			Longitudinal	0.00026	4.18×10^{-6}	0.00026	0.00025
Maximum bearing dimensions (mm)	Upper surface		Transverse	600	600	700	600
			Longitudinal	600	600	700	600
	Lower surface		Transverse	600	600	700	600
			Longitudinal	600	600	700	600
	Overall height			300	300	300	300
Type of fixing required			Upper face	Bolts	Bolts	Bolts	Bolts
			Lower face	Bolts	Bolts	Bolts	Bolts

The method of fixing has been scheduled as requiring cast-in sockets for the lower plate and a purpose made steel plate to suit the top plate. The purpose made plate will have drilled and tapped holes and dowel bar anchors which are cast into the deck.

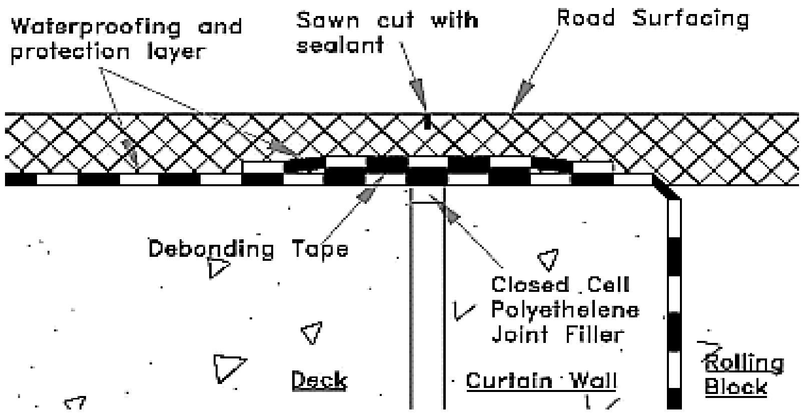
BD 33/94 **Expansion Joint**

Table 1 The movement at the free end of the deck due to temperature variations has been calculated as 20mm. If the shade air temperature is measured when the joint is installed then the joint can be set to accommodate a +/- 10mm movement. Table 1 of BD 33/94 lists the Asphaltic Plug Joint as being suitable for a total longitudinal movement o 40mm.



Asphaltic Plug Joint

Adequate sub-surface drainage needs to be provided and well maintained. Most joint manufacturers will supply a drainage system however strict attention to detail is required during installation to ensure that the system operates correctly. Overviews of different joints can be obtained from the Bridge Joint Association website at http://www.bridgejoints.org.uk.



Fixed Abutment Joint

The joint at the fixed end of the deck needs to accommodate the rotational movement of the end of the deck. This is usually achieved with two layers of sprayed waterproofing material separated by a layer of debonding tape as shown above.