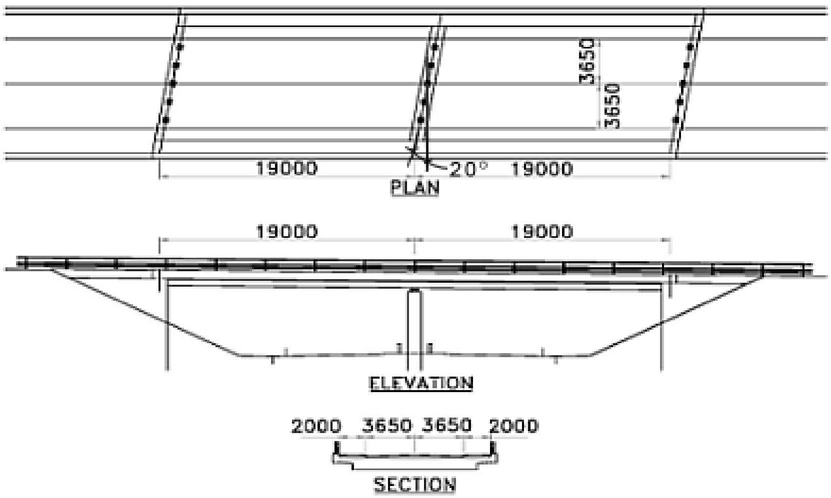
Disclaimer: This example has been prepared to illustrate the design principles for a reinforced concrete bridge deck on reinforced concrete abutments and pier to suit a hypothetical set of conditions. Users of this example should verify for themselves the appropriateness of any parts of the example which 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.

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.

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

BS 5400 Pt 2 **Loading**

Cl. 5.1.1 Dead: Reinforced Concrete density = 25 kN/m^3 — γ_{conc} = 25 kN/m^3

Cl.5.2.1 Mastic Asphalt Road Surfacing Density = 22 kN/m^3 — γ_{surf} = 22 kN/m^3

Footway infill say = 22 kN/m^3 — γ_{infill} = 22 kN/m^3

Cl. 4.3.1.2 Differential Settlement:

The soil investigation report estimates that there will be 20 to 25mm settlement at the abutments and 5 to 10mm settlement at the pier. Therefore allow for 15mm differential settlement. — Diff.Set. = 15mm

Cl.6.1 Live:

Cl. 3.2.9.1 Carriageway width = 7.3m

Cl. 3.2.9.3.1 Number of notional lanes = 2

Notional Lane width w_l = 3.65m

Cl. 6.1.1 Consider: (i) HA loading to Cl. 6.4.1

(ii) HA + HB loading to Cl. 6.4.2

Cl. 6.2 HA Loading

Cl.6.2.1 UDL: $W = 336(1/L)^{0.67}$

For one span loaded L = 19m Hence W = 46.7 kN/m

For two spans loaded L = 38m Hence W = 29.4 kN/m

Cl.6.2.2 KEL: KEL = 120 kN per notional lane

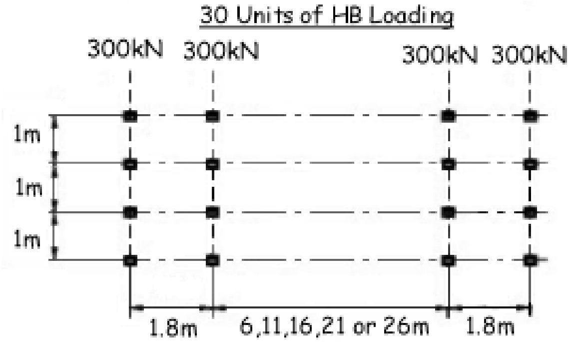
Cl.6.4.1.1 HA Lane factors from Table 14:

$\beta_1 = \beta_2 = \alpha_1 = 0.274 \times 3.65 = 1.0$

Cl. 6.3.1 HB Loading

The road carried by the bridge is not a Principal road (see DMRB BD 37/01) therefore design for 30units of type HB loading

30 Units of HB Loading



Cl. 6.5 Footway Loading

Cl. 6.5.1.1 a) For one span loaded L = 19m Hence w = 5.0 kN/m^2

b) For two spans loaded L = 38m

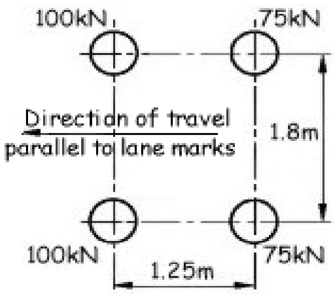
Hence w = 5.0 x [(29.4 x 10) / (38 + 270)] = 4.8 kN/m^2

Cl. 6.5.1.2 Reduction for elements supporting carriageway and 2m footway:

a) For one span loaded w = 0.8 x 5.0 = 4.0 kN/m^2

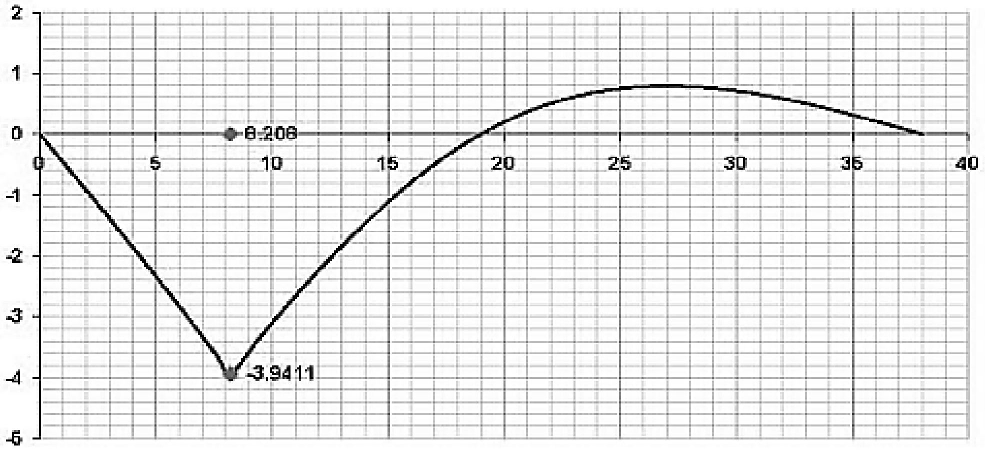
b) For two spans loaded w = 0.8 x 4.8 = 3.8 kN/m^2

Cl. 6.6 Accidental Loading : Vehicle on Footways



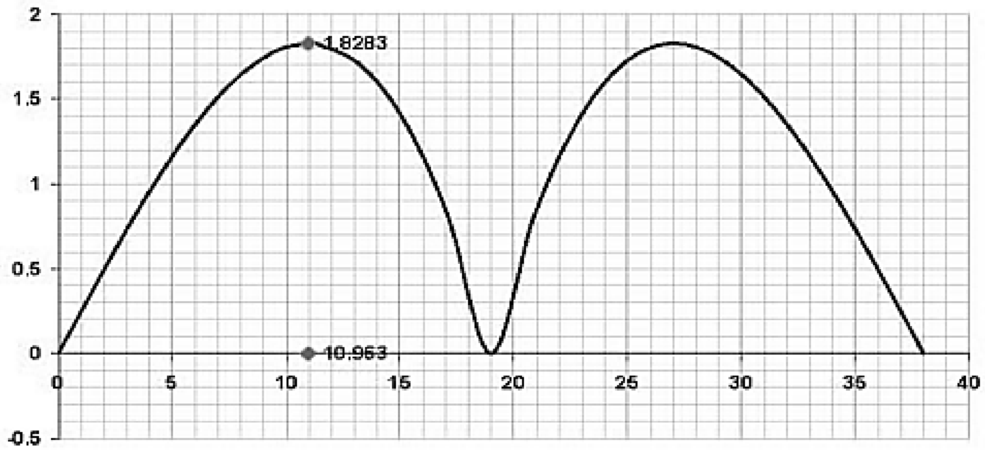
Influence Lines (Use spreadsheet "Infline.xls")

Influence Line for Max Sag Moment in Span 1



For maximum sagging moment in deck apply HA UDL in one span with KEL at 8.2m from end of deck.

Influence Line for Hog Moment at Support 2



For maximum hogging moment in deck apply HA UDL in both spans with KEL at 10.9m from end of deck,
The KEL needs to positioned adjacent to the supports to obtain the critical loading cases for shear effects.

A simple line beam analysis will provide guidance for the HB loading.
For critical loading positions of HB vehicle use spreadsheet MovinLd.xls
For maximum sagging moment position leading axle of HB vehicle at 15.067m from abutment; the maximum moment occurs under the third axle at 7.267m from the abutment. The HB vehicle with the 6m inner axle spacing will give the critical sagging moment.
For maximum hogging moment over the centre support position the leading axle in span 2 at 9.804m from the centre support. The HB vehicle with the 16m inner axle spacing will give the critical hogging moment.

Preliminary Design

Use span to depth ratio about 20:1.
D = 19000 / 20 = 950 mm
Check maximum moment over centre pier using a line beam analysis:

Permanent Loads

For 3.65m (notional lane width) width of deck:
Weight of concrete = 3.65 x 0.95 x 25 = 86.7 kN/m
Weight of surfacing = 3.65 x 0.125 x 22 = 10.0 kN/m
Apply load factors for ultimate limit state from Table 1 then

UDL = (86.7 x 1.15) + (10.0 x 1.75) = 117.2 kN/m

Moment = $wL^2/8$ = 117.2 x 19^2 / 8 = 5290 kNm

Differential settlement:

BS 5400 Pt 4
Cl. 4.3.2.1 From Table 3 - Modulus of Elasticity = 34 kN/mm^2

Long Term E = 34 / 2 = 17 kN/mm^2

I_{xx} = 3.65 x 0.95^3 / 12 = 0.261m^4

EI_{xx} = 17 x 10^6 x 0.261 = 4.437 x 10^6 m^4

Using spreadsheet LneBem.xls with -15mm settlement at centre support:

Moment over Pier = 553 kNm

At Ultimate Limit State Moment = 1.2 x 553 = 660 kNm

HA UDL for 2 spans loaded = 29.4 kN/m KEL = 120 kN
At Ultimate Limit State UDL = 29.4 x 1.5 = 44.1 kN/m
At Ultimate Limit State KEL: = 120 x 1.5 = 180 kN
Using spreadsheet LneBem.xls with KEL at 8.2m from left hand end:

Moment over Pier = 2290 kNm

Using spreadsheet MovinLd.xls to analyse 30 units of HB loading

Moment over Pier = 2153 kNm

At Ultimate Limit State Moment = 1.3 x 2153 = 2800 kNm

HB moment > HA moment hence:
Ultimate Design Moment = γ_{f3} x (M_{dead} + M_{HB})
Ultimate Design Moment M_D = 1.1 x (5290 + 660 + 2800) = **9625 kNm**

BS 8500 Pt 1
Table A5 Deck concrete is grade C40/50 with Class designation XD1 requires a cover to reinforcement of 35 + Δc = 35 + 15 = 50mm

BS 5400 Pt 4
Cl. 5.3.2.3 Assume 40mm dia. reinforcement then d = 950 - 50 - 20 = 880mm
Bars at 125 to 150mm c/c will generally meet crack control requirements so try 40mm bars at 150 c/c then

As = (π x 40^2 / 4) x (3.65 x 1000 / 150) = 30580mm^2

f_{cu} = 50 N/mm^2

fy = 500N/mm^2

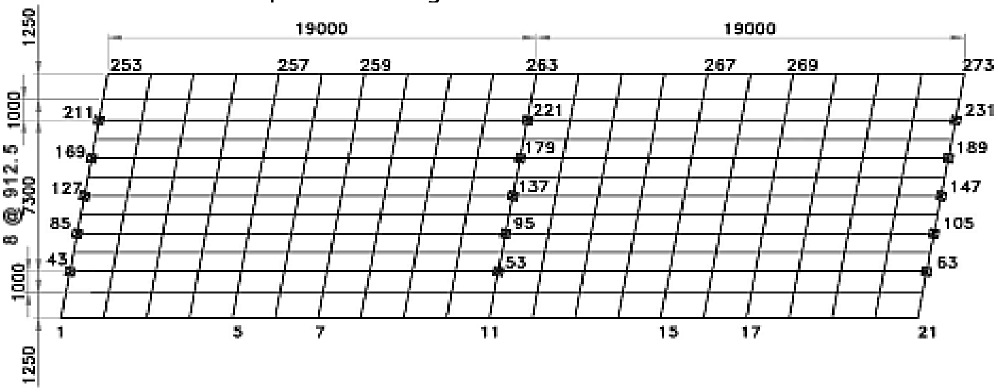
z = [1 - (1.1 x 500 x 30580) / (50 x 3650 x 880)]d = 0.895d

Mu = 0.87 x 500 x 30580 x 0.895 x 880 x 10^{-6} = **10480 kNm**

10480 > 9625 kNm therefore continue with 950mm deck depth — D = 950mm
Check for the additional load effects caused by the skew by using a grillage analysis.

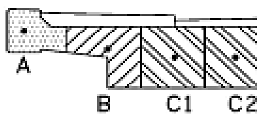
Grillage Analysis

As the skew angle is relatively small (20° < 35°) then a skew mesh will give sufficient accuracy. The maximum length to breadth ratio of the mesh should be limited to about 2:1. Dividing the deck into suitable members to coincide with critical positions we get:



Member Properties: (Use proforma "Secprop.xls")

i) Longitudinal Members:



	Area	Ixx	Jxx
A	5.07E+05	1.76E+10	*1.79E+10*
B	8.04E+05	5.50E+10	*2.20E+10*
C	8.67E+05	6.52E+10	*5.27E+10*
	(mm^2)	(mm^4)	(mm^4)

ii) Transverse Members:

	Area	Ixx	Jxx
Cantilever	8.30E+05	1.32E+10	*2.26E+10*
Deck	1.81E+06	1.36E+11	*1.86E+11*
	(mm^2)	(mm^4)	(mm^4)

Note: The torsional Inertia Jxx estimates the torsional stiffness of the deck slab. This results in a torsional moment being induced in the grillage members. An analysis will be required to convert this twisting moment into the reinforcement directions, usually carried out using the Wood and Armer equations.

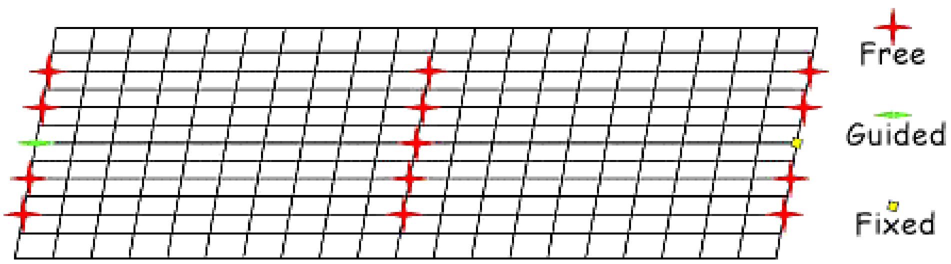
Providing the skew angle is not large (<35°) then it is usually more economical to position the transverse reinforcement in the skew direction, i.e. in the same direction as the grillage members. In this case an approximation of the Wood and Armer effects on the main reinforcement can be obtained by using Jxx = 0 (set to $10^{-6}mm^4$) in the grillage analysis.

Supports:

Provide bearings to allow rotational movement hence all 'free' supports to resist vertical loads only.

Provide 'fixed' bearing at abutment with lowest carriageway level hence central support at right hand end to resist vertical and horizontal loads.

Provide 'sliding-guided' bearing at centre of left hand end abutment to resist vertical and transverse forces but allow expansion and contraction movement of deck.



Load Cases:

Dead Load - Apply as UDL to longitudinal members

Concrete:

UDL on A = 5.07E+05 x 25 x 10E-06 = 12.68 kN/m
UDL on B = 8.04E+05 x 25 x 10E-06 = 20.10 kN/m
UDL on C = 8.67E+05 x 25 x 10E-06 = 21.68 kN/m

Surfacing:

UDL on C1 = 0.5 x 0.913 x 0.125 x 22 = 1.26 kN/m
UDL on C2 = 0.913 x 0.125 x 22 = 2.51 kN/m

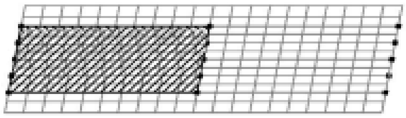
Footway:

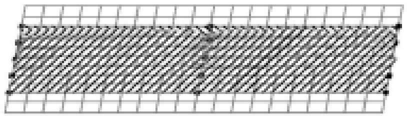
UDL on A = 0.375 x 0.25 x 22 = 2.06 kN/m
UDL on B = 1.125 x 0.25 x 22 = 6.19 kN/m
UDL on C1 = 0.50 x 0.25 x 22 = 2.75 kN/m

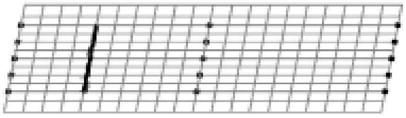
Parapet:

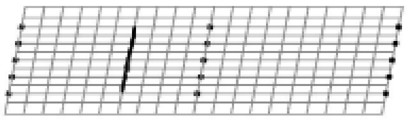
UDL on A = say 1kN/m (steel parapet with infill mesh)

Live Load

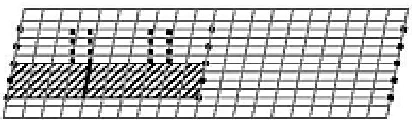

UDL SPAN 1

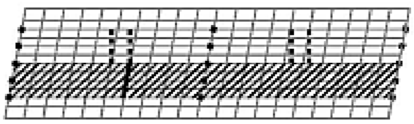

UDL SPANS 1 & 2


KEL SPAN 1
(max sag)

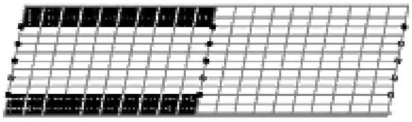

KEL SPAN 1
(max hog)

Some grillage models allow patch loads to be applied for uniform distributed loads otherwise the UDL may be divided as for the Dead Load cases above. The Knife Edge Loads may be applied as uniformly distributed loads on the transverse members. The loads should be applied along the grillage transverse member closest to the critical position determined from the influence line diagram.
The KEL needs to positioned adjacent to the supports to obtain the critical loading cases for shear effects.
The KEL should also be positioned on transverse members adjacent to the estimated critical positions to ensure the skew effects are included.

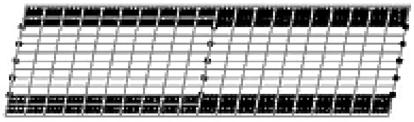

HB & HA SPAN 1
(max sag)


HB & HA SPANS 1 & 2
(max hog)

The loads in a slab deck will tend to take the shortest path to the supports, the shortest path between the pier and abutment being at right angles to the centre-line of the bearings. The obtuse corner of a skewed deck will therefore attract more load so the HB vehicle needs to be positioned in the lane as shown above to check for the worst sagging moment.
A few positions adjacent to the critical line beam result should again be checked to cover the effects of skew.



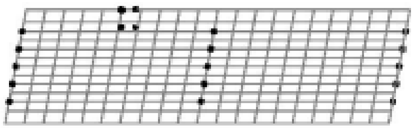
FOOTWAY UDL
(max sag)



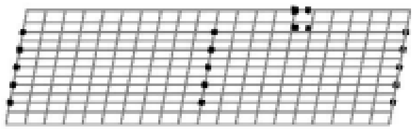
FOOTWAY UDL
(mag hog)

The footway UDL is included with the HA and HB load cases, although if the accidental wheel load can load the footway then this load case will generally govern the design of the cantilever members.

No other live loads are considered on the deck with the accidental wheel load unless the parapet is required to be high containment (see Cl. 6.7.2.2)



Accidental Wheel
(max sag)



Accidental Wheel
(max hog)

Grillage Analysis Results

The program 'SuperSTRESS was used to obtain the following results from a grillage analysis of the permanent and live load cases described above.

Results from RCGrillage950.pdf (With torsional member stiffness).

COMBINATION 1 LOADING (Bending Effects)	SLS (kNm)	ULS (kNm)	
Longitudinal Mid Span Effects			
Dead Load + HA + Footway	1188	1447	Member 105 (C5&C6)
Dead Load + HB + Footway	**1226**	**1461**	Member 145 (C7&C8)
Longitudinal Effects at Pier			
Dead Load + HA + Ftwy + Settl.	2032	2446	Member 50 (C9&C10)
Dead Load + HB + Ftwy + Settl.	**2089**	**2491**	Member 211 (C11&C12)

We can compare these results with the analysis using no torsional stiffness in the grillage members.

Results from RCGrillageZeroJ950.pdf (With no torsional member stiffness).

COMBINATION 1 LOADING (Bending Effects)	SLS (kNm)	ULS (kNm)		
Longitudinal Mid Span Effects				
Dead Load + HA + Footway	1198	1460	Member 124 (C5&C6)	Sagging Moment
Dead Load + HB + Footway	**1256**	**1497**	Member 205,185 (C7&C8)	SLS = 1256 kNm
				ULS = 1497 kNm
Longitudinal Effects at Pier				
Dead Load + HA + Ftwy + Settl.	2034	2448	Member 210 (C9&C10)	Hogging Moment
Dead Load + HB + Ftwy + Settl.	**2117**	**2524**	Member 210 (C11&C12)	SLS = 2117 kNm
				ULS = 2524 kNm

It can be seen that there is only about a 1% to 3% increase in the design moments using no torsional member stiffness so we shall continue the design using these slightly higher results to avoid designing for torsional effects.
Flexural shear effects are considered at sections away from the supports.
Concrete slabs on isolated bearings will usually fail by punching shear rather than flexural shear. Punching shear is considered at 1.5d away from the support consequently flexural shear will not usually dominate within this boundary. Grillage results for shear effects are therefore considered on grid lines at d away from the support.

COMBINATION 1 LOADING (Shear Effects)	ULS (kN)	
Adjacent to Abutments		
Dead Load + HA + Footway	**250**	Member 102 (C13)
Dead Load + HB + Footway	247	Member 82 (C14)
Adjacent to Pier		
Dead Load + Settlement + HA + Footway	445	Member 89 (C19)
Dead Load + Settlement + HB + Footway	**449**	Member 92 (C20)

Combination 2 loading considers the effects of wind on the structure. The large mass and shape of the concrete deck will generally ensure that this load combination will not be critical for the design of the deck. The wind load will however be considered later in the lateral forces acting on the bearings.

Combination 3 loading:
Restraint to expansion and contraction will induce axial loads in the deck. The restraint will be provided by friction or shear stiffness of the bearings and/or the flexure of the substructure, both of which are insignificant compared with the axial stiffness of the deck. This effect may be generally ignored in the design of the deck but will be considered later in the bearing and substructure design.

BS 5400 Pt 2 Cl. 5.4.1(b)

Temperature differences between the top and bottom of the deck will cause the deck to deflect and these effects need to be considered.

Differential Temperature (Use spreadsheet "TempDiff5400.xls" contained in "105.zip")

Vertical Temperature Differences Using Non-Linear Effects

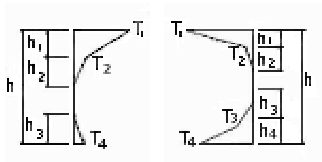
BS 5400-2:2006 Clause 5.4.5

Construction Depth h = 950 mm

Depth of Surfacing ds = 125 mm

Idealised Section

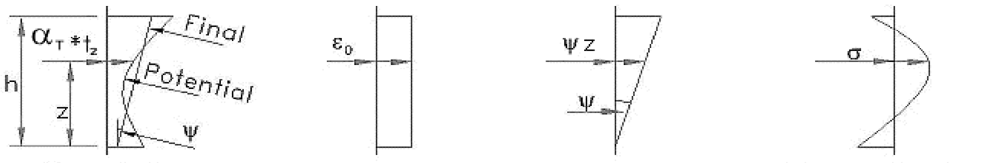
Positive	Type 4	Reverse
h_1 = 150		h_1 = 190
h_2 = 250		h_2 = 200
h_3 = 225		h_3 = 200
		h_4 = 190
T_1 = 11.75		T_1 = -7.1
T_2 = 2.75	Temperature	T_2 = -1.35
T_3 = 2.25	Differences	T_3 = -1.6
		T_4 = -6.213



Section Details:

Positive	Top Width	Bottom Width	Depth	Top Temp.	Bottom Temp.	Modulus of E.	Coeff. $\alpha_T \times 10^{-6}$
Section 1	912.5	912.5	150	11.75	2.75	34000	12
	912.5	912.5	250	2.75	0	34000	12
	912.5	912.5	325	0	0	34000	12
	912.5	912.5	225	0	2.25	34000	12
Reverse							
Section 1	912.5	912.5	190	-7	-1.35	34000	12
	912.5	912.5	200	-1.35	0	34000	12
	912.5	912.5	170	0.00	0	34000	12
	912.5	912.5	200	0.00	-1.6	34000	12
	912.5	912.5	190	-1.60	-6.2125	34000	12

Using the strain method described in 'Concrete Bridge Design to BS 5400'



For force equilibrium $\int_0^h \sigma b_z dz = 0$

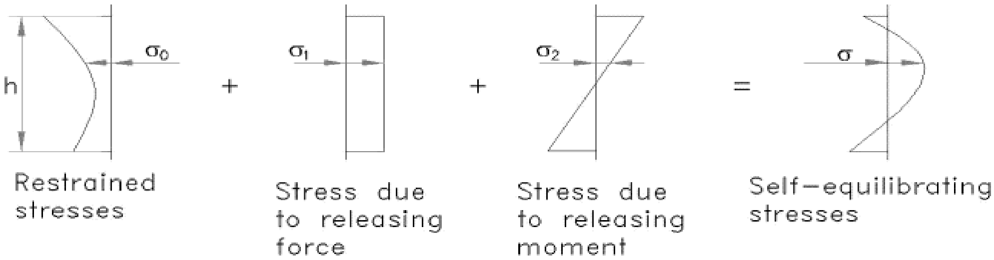
For moment equilibrium $\int_0^h \sigma b_z z dz = 0$

The stress σ at level $\sigma = E_z(\varepsilon_0 + \psi z - \alpha_z t_z)$

The axial strain ε_0 and curvature ψ can be obtained by substituting the equation for stress σ into the equations for force and moment equilibrium.

	Positive			Reverse	
Axial strain ε_0 =	-1.3E-05		Axial strain ε_0 =	-2E-05	
Curvature y =	7.3E-08		Curvature y =	-3E-09	
Centroid of Section =	475	mm	Centroid of Section =	475	mm
EI about centroid =	2216672	kNm^2	EI about centroid =	2216672	kNm^2
EA for Section =	2.9E+07	kN	EA for Section =	2.9E+07	kN
Releasing Moment M =	-161.466	kNm	Releasing Moment M =	6.35388	kNm
Releasing Force F =	627.093	kN	Releasing Force F =	-681.47	kN

The stresses due to a releasing force and releasing moment can now be calculated using the values for ε_0 and ψ.



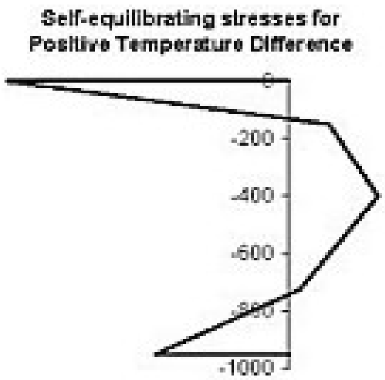
Stresses due to Positive Temperature Differences

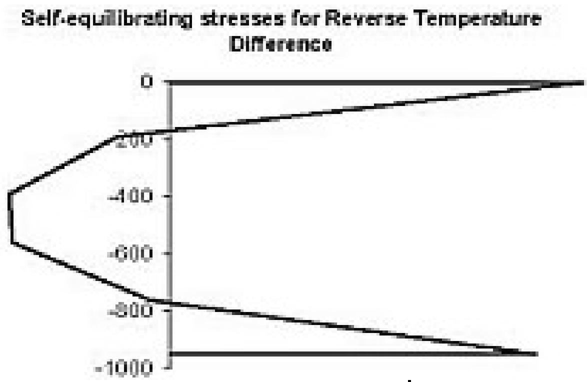
Depth from top	Restrained Stress	Releasing Force	Releasing Moment	Self-equilibrating stresses (N/mm^2)
0	-4.794	0.72339	1.17639	-2.8942
150	-1.122	0.72339	0.8049	0.40629
400	0	0.72339	0.18575	0.90914
725	0	0.72339	-0.6192	0.10424
950	-0.918	0.72339	-1.1764	-1.371

Stresses due to Reverse Temperature Differences

Depth from top	Restrained Stress	Releasing Force	Releasing Moment	Self-equilibrating stresses (N/mm^2)
0	2.856	-0.7861	-0.0463	2.02358
190	0.5508	-0.7861	-0.0278	-0.2631
390	0	-0.7861	-0.0083	-0.7944
560	0	-0.7861	0.00828	-0.7778
760	0.6528	-0.7861	0.02778	-0.1055
950	2.5347	-0.7861	0.04629	1.79487

Notation: Tensile stresses are positive.





The releasing moments need to be distributed on the continuous deck slab. This is most conveniently carried out using the spreadsheet "LbeamDTRelMom.xls"
Positive temperature release moment = -161.5 kNm
-161.5 is entered in both spans on the spread sheet. The distribution results in a sagging moment of 242 kNm over the pier with an uplift reaction of 25.5 kN at the pier. Sagging moment at mid span = 121 kNm

Mid Span Moment
121 kNm

The width of the longitudinal grillage member is 0.913m which has been used for the width of the deck section in the temperature analysis. The uplift reaction at the bearing will therefore be 2x25.5 = 51 kN. The minimum dead load reaction is about 400 kN > 51 kN therefore uplift will not occur.
Shear at Abutment = 12.75 kN

Shear at Abut.
13 kN

Reverse temperature release moment = 6.35kNm

When this is distributed then the reaction at the pier is 1 kN and the hogging moment = 9.5 kNm.

Moment at Pier
10 kNm

The reverse temperature effect will increase the dead and live load moments and reactions at the pier and will therefore need to be considered in this combination.

Shear at Pier = 0.5 kN

Shear at Pier
1 kN

COMBINATION 3 LOADING (Bending Effects)	SLS (kNm)	ULS (kNm)		
Longitudinal Mid Span Effects				
Dead Load + HA + Footway	1198	1460	Member 125	
Dead Load	696	819		
Hence HA + Footway	502	641		
Adjust Live Ld γ_{fL} from combination 1 to 3	502	534	(1.25/1.5)	
Factored Moment due to Diff. Temp.	97	121	Positive Temp.	
Dead Load + HA + Footway + Diff. Temp.	1295	1474		
Dead Load + HB + Footway	1256	1497	Member 205	
Dead Load	696	819		
Hence HB + Footway	560	678		
Adjust Live Ld γ_{fL} from combination 1 to 3	560	574	(≏ 1.1/1.3)	
Factored Moment due to Diff. Temp.	97	121	Positive Temp.	
Dead Load + HB + Footway + Diff. Temp.	**1353**	**1514**	HB critical	
Longitudinal Effects at Pier				
Dead Load + Settlement + HA + Footway	2034	2448	Member 210	
Dead Load + Settlement	1622	1915		
Hence HA + Footway	412	533		
Adjust Live Ld γ_{fL} from combination 1 to 3	412	444	(1.25/1.5)	
Factored Moment due to Diff. Temp.	8	10	Reverse Temp.	
Dead Load + HA + Footway + Diff. Temp.	2042	2369		
Dead Load + Settlement + HB + Footway	2117	**2524**	Member 210	Combination 1 critical
Hence HA + Footway	495	609		
Adjust Live Ld γ_{fL} from combination 1 to 3	495	515	(≏ 1.1/1.3)	
Factored Moment due to Diff. Temp.	8	10	Reverse Temp.	
Dead Load + HA + Footway + Diff. Temp.	**2125**	2440		

COMBINATION 3 LOADING (Shear Effects)	ULS (kN)		
Adjacent to Abutments			
Dead Load + HA + Footway	250	Member 102	
Dead Load	171		
Hence HA + Footway	79		
Adjust Live Ld γ_{fL} from combination 1 to 3	66	(1.25/1.5)	
Factored shear due to Diff. Temp.	13		Combination 1
Dead Load + HA + Footway + Diff. Temp.	**250**		and 3 critical
Dead Load + HB + Footway	247	Member 82	
Dead Load	171		
Hence HB + Footway	76		
Adjust Live Ld γ_{fL} from combination 1 to 3	64	(≏ 1.1/1.3)	
Factored shear due to Diff. Temp.	13		
Dead Load + HA + Footway + Diff. Temp.	248		
Adjacent to Pier			
Dead Load + Settlement + HA + Footway	445	Member 89	
Dead Load + Settlement	351		
Hence HA + Footway	94		
Adjust Live Ld γ_{fL} from combination 1 to 3	78	(1.25/1.5)	
Factored shear due to Diff. Temp.	1		
Dead Load + HA + Footway + Diff. Temp.	430		
Dead Load + Settlement + HB + Footway	**449**	Member 92	Combination 1
Dead Load + Settlement	351		critical
Hence HB + Footway	98		
Adjust Live Ld γ_{fL} from combination 1 to 3	83	(≏ 1.1/1.3)	
Factored shear due to Diff. Temp.	1		
Dead Load + HB + Footway + Diff. Temp.	435		

BS 5400 Pt 2

Cl. 4.1.3 Load Effects to be used for design = γ_{f3} x effects of design loads

BS 5400 Pt 4

Cl. 4.2.2 Serviceability Limit State : γ_{f3} = 1.0

Cl. 4.2.3 Ultimate Limit State : γ_{f3} = 1.1

Load Effects Summary

i) Bending at mid span

SLS = **1353** kNm (sag) (dead = 696kNm, live = 657kNm)

Additional stress in concrete due to Positive Temperature Difference:

SLS Compressive stress due to Diff. Temp = 0.8 x 2.894 at top = **2.315**N/mm^2

ULS = 1.1 x 1514 = **1665** kNm (sag)

ii) Bending at pier

SLS = **2125** kNm (hog) (dead = 1622kNm, live = 503kNm)

Additional stress in reinforcement due to Reverse Temperature Difference:

Tensile self-equilibrating stress at reinforcement level (at top):

Using 40mm ϕ bars with 50mm cover then

σ = 2.024 - [(2.024 - {-0.263}) x 70 / 190] = 1.181 N/mm^2

E_s = 200000 N/mm^2 Ec = 34000 N/mm^2

Tensile stress in reinforcement = 1.181 x 200000 / 34000 = 6.95 N/mm^2

SLS Tensile stress due to Diff. Temp = 0.8 x 6.95 = **5.56**N/mm^2

ULS = 1.1 x 2524 = **2776** kNm (hog)

iii) Shear at Abutment

ULS = 1.1 x 250 = **275** kN

iv) Shear at Pier

ULS = 1.1 x 449 = **494** kN

BS 5400 Pt 4 Reinforced Concrete Slab Design

Cl. 5.1.2.1 **Design** the slab for Ultimate Limit State and **Check** for Serviceability Limit State.

Section over Pier

Cl. 5.4.2 Cl. 5.3.2.3 $Mu = (0.87f_y)A_s z$

Deck concrete : C40/50 for exposure condition XD1

Steel reinforcement : Grade B500B to BS 4449:2005

$f_{cu} = 50\ N/mm^2$ $f_y = 500\ N/mm^2$

Width of grillage member = b = 0.913m

BS 8500 Pt 1

Table A5 Deck concrete is grade C40/50 with Class designation XD1 requires a cover to reinforcement of $35 + \Delta c = 35 + 15 = 50mm$

try 40mm bars at 150 c/c then

$d = 950 - 50 - 20 = 880mm$

$As = (\pi \times 40^2 / 4) \times (0.913 \times 1000 / 150) = 7649mm^2$

$z = [1 - (1.1 \times 500 \times 7649) / (50 \times 913 \times 880)]d = 0.895d$

$Mu = 0.87 \times 500 \times 7649 \times 0.895 \times 880 \times 10^{-6} = 2620\ kNm < 2776\ kNm$ Fail

Use **40mm bars at 125 c/c** then

$As = (\pi \times 40^2 / 4) \times (0.913 \times 1000 / 125) = 9178mm^2$

$z = [1 - (1.1 \times 500 \times 9178) / (50 \times 913 \times 880)]d = 0.874d$

$Mu = 0.87 \times 500 \times 9178 \times 0.874 \times 880 \times 10^{-6} = \mathbf{3070}\ kNm > 2776\ kNm$ OK

Check capacity based on limiting concrete strength:

$Mu = 0.15\ f_{cu}\ b\ d^2$

$Mu = 0.15 \times 50 \times 913 \times 880^2 \times 10^{-6} = 5303\ kNm > 3070\ kNm$

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.
40mm bars at 125 c/c gives a spacing of 85mm 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} =	50	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	913	950
Table 13	Environment Conditions for nominal cover:		Moderate		
	Reinforcement controlling crack width :			Steel Reinforcement	
	Cover from notional surface =	25	mm	Area	Depth
	Bar diameter (ϕ)=	40	Comp.	0	0
	Spacing (s) =	125	Ten.	9178	880

Factored Dead Load Moment(Mg) = 1622 kNm
Factored Live Load Moment(Mq) = 503 kNm

Table 3 — Young's Modulus for Concrete E_c = 34 kN/mm^2

Cl. 4.3.2.1b) — Modified $E_c = Ec(1-0.5Mg/(Mg+Mq))$ = 21.02 kN/mm^2

Modular ratio $\alpha_e = E_s/E_c$ = 9.574

$x/d_t = \alpha_e(A_{st}/(bd_t)+A_{sc}/(bd_t)+[(\alpha_e{}^2\{A_{st}/(bd_t)+A_{sc}/(bd_t)\}^2+2\alpha_e\{A_{st}/(bd_t)+A_{sc}d_c/(bd_t{}^2)\}]^{0.5}$

x/d_t = 0.370

x = 0.370 x 880 = 325.622 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 = 3.734E+10 mm^4

Steel stress = $\sigma_s = \alpha_e(Mg+Mq)(d_t-x)/Ixx$ = 300.122 N/mm^2

Steel strain = $\varepsilon_s = \sigma_s/E_s$ = 0.00150

Distance from Comp Face to notional surface = d_{ns} = 925 mm

Strain at notional surface = $\varepsilon_1 = \varepsilon_s(d_{ns}-x)/(d_t-x)$ = 0.0016

equation 25 — Stiffening Effect of Conc in Tension = 0.00016

Modified Strain at notional surface = ε_m = 0.00146

Distance from crack to bar = $[(25+40/2)^2+(125/2)^2]^{0.5}-40/2$

Distance from crack to bar = 57.015 mm

Cl 5.8.8.2 — Design Crack Width = 0.2271 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 = 18.53 N/mm^2

Compressive stress due to Temperature Difference = 0 N/mm^2

Total Compressive stress in concrete = 18.53 N/mm^2

Table 2 — Allowable compressive stress = 0.5fcu = 25 N/mm^2 Hence OK

Tensile stress in steel = 300.122 N/mm^2

Tensile stress due to Temperature Difference = 5.56 N/mm^2

Total Tensile stress in steel = 305.68 N/mm^2

Table 2 — Allowable tensile stress = 0.75fy = 375 N/mm^2 Hence OK

Section over pier B40@125c/c

Transverse Reinforcement

Maximum ULS moment in deck at pier = 1236 kNm (Combination 12)

Cl. 5.4.2 Cl. 5.3.2.3 — $Mu = (0.87f_y)A_s z$

Deck concrete : C40/50 for exposure condition XD1

Steel reinforcement : Grade B500B to BS 4449:2005

f_{cu} = 50 N/mm^2 f_y = 500 N/mm^2

Width of grillage member = b = 1.9m

try 25mm bars at 200 c/c then

d = 950 - 50 - 40 - 13 = 847mm

As = (π x 25^2 / 4) x (1.9 x 1000 / 200) = 4663mm^2

z = [1 - (1.1 x 500 x 4663) / (50 x 1900 x 847)]d = 0.97d > 0.95d hence use 0.95d

Mu = 0.87 x 500 x 4663 x 0.95 x 847 x 10^{-6} = 1632 kNm > 1236 kNm Hence OK

Use B25 @ 200 over the Pier

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'*

Cl.6.3 Although the deck is only 950mm thick (less than 1m) there will still be a large mass of concrete which will generate heat when the deck is poured and it is worth considering the internal restraint.

Restrained Section Length L = 38000 mm

Restrained Section Thickness T = 950 mm

Reinforcement Strength f_y = 500 N/mm^2

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

BS 8500-1 Deck concrete : C40/50 for exposure condition XD1

Table A.5 Cement content = 360 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 $f_{ct} = 0.12*f_{cu}^{0.7}$ = 1.8555 N/mm^2

Using the prediction method (Section 5.1)

Minimum area of reinforcement = $f_{ct}*A_c/f_y$ = 1855.5 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 = 0.5 (Internal)

Try 16 mm ϕ bars then:

$A_s = (f_{ct}/f_b)*A_c*\phi*[R*(\varepsilon_{sh}+\varepsilon_{th})-0.5*\varepsilon_{ult}]/(2*w)$ = 1779.52 mm^2/m ....(3)

Hence minimum area from equation (2) governs = 1855.5 mm^2/m

Minimum area A_s in each face = 927.75 mm^2/m

B16 @ 200 c/c = 1005 > 928 mm2/m Hence OK

Transverse distribution
B16 @ 200c/c

Cl. 5.4.4 **Shear Resistance**

Cl. 5.4.4.1 Shear stress = $v = V/bd = 494 \times 10^3 / (913 \times 880) = 0.61$ N/mm^2

Table 9 $\xi_s = (500/d)^{1/4} = (500 / 880)^{1/4} = 0.89$

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

$v_c = (0.27/1.25)(100 \times 9178/913/880)^{1/3}(40)^{1/3}$

0.77 N/mm^2

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

= $0.75(50)^{1/2} = 5.3$ or 4.75 i.e. = 4.75 N/mm2

0.61 < 4.75 N/mm^2 Hence OK

Cl. 5.4.4.1 Max. allowable shear stress without shear reinforcement = 0.89x0.77 = 0.69

0.61 < 0.69 Hence shear links not required.

Additional longitudinal reinforcement to enable truss action to develop:

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

$V/2(0.87f_y) = 494 \times 10^3 / (2 \times 0.87 \times 500)$

= 568 mm^2

A_{st} provided = 9178 mm^2

A_{st} req'd for Design Moment = 9178 x (2911 / 3070) = 8703 mm^2

A_{st} available for truss action = 9178 - 8073 = 475 mm^2 < 568 Hence Fail

Option 1: increase main bar size or reduce spacing.
Option 2: provide additional steel in second layer.

Option 1 is undesirable as 50mm ϕ bars are a non-preferred size and reducing the spacing to 100mm will only leave a 60mm gap; this could be further reduced by 15mm to 45mm by the fixing tolerance Δc.

Cl. 5.8.8.1(a1) Bars in second row shall be in-line with the main steel and distance between rows shall be greater than $h_{agg.}$ With 25mm aggregate then use 32mm ϕ bar spacers.

Try B20 @ 250c/c $A_{st} = \pi \times 20^2 \times 913 / (4 \times 250) = 1147$mm^2

Effective depth = d = 880 - 20 - 32 - 10 = 818mm

Equivalent A_{st} at main steel level = 1147 x 818 / 880 = 1066 mm^2 > 568mm^2 Hence OK

Cl. 5.4.4.2 Punching Shear

Maximum ULS reaction at pier bearing:

Combination 12, ULS reaction at node 221 = 1.1 x 3636 = 4000kN

The SLS reaction for this load combination = 3049kN (Combination 11)

Bearing dimensions are usually designed to achieve a bearing stress of about 20N/mm^2 under nominal loading.

Say nominal reaction is about 3000kN then square bearing dimension is :

$$b = (3000 \times 10^3 / 20)^{0.5} = 400\text{mm}$$

B40 @ 125 in longitudinal direction B25 @ 200 in transverse direction

Critical perimeter at 1.5d from loaded area:

Length in longitudinal direction = 2 x 1.5 x 880 + 400 = 3040 mm

Length in transverse direction = 2 x 1.5 x 847 + 400 = 2941 mm

As the spacing between the bearings = 1825 mm is less than 2941 mm then the shear perimeter will not be formed around an isolated reaction and shear will need to be considered across two failure lines either side of the pier.

Shear perimeter = 2 x 9.3 / cos20^o = 19.8m

Maximum total reaction at pier (C 19) = 3550 + 1257 + 2371 + 1256 + 3551
= 11985 kN

Ultimate load = 1.1 x 11985 = 13184 kN

Dead load of deck between shear perimeters = Deck concrete + surfacing + fill

Cross sectional area of deck = (2x0.507+2x0.804+9x0.867) = 10.425 m^2

Cross sectional area of surfacing = 7.3 x 0.125 = 0.91 m^2

Cross sectional area of footway fill = 2 x 0.25 x 2.0 = 1.0 m^2

Distance between shear perimeters = 3.040 m

Ultimate Dead load = 3.04x(1.15x10.425x25 + 1.75x0.91x22 + 1.2x1.0x22)
= 1098 kN

Live load on deck between shear perimeters = HA udl = 2 x 29.4 x 3.04 = 179 kN

Ultimate live load = 1.5x179 = 269 kN

Load on shear perimeters = 13184 - 1.1(1098+269) = 11680 kN

V_c is based on the reinforcement in the tension zone perpendicular to the shear plane, i.e.

B40 @ 125 c/c (d = 880) A_s = 10053 mm^2/m and

B20 @ 250 (d = 818) A_s = 1257 mm^2/m

Total steel area = 10053 + 1257 = 11310 mm^2/m

Effective depth d = (10053 x 880 + 1257 x 818) / 11310 = 873 mm

Transform A_s perp. to shear plane = 11310 x cos^{2}20^o = 9987 mm^2/m

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

$$v_c = (0.27/1.25)(100 \times 9987/1000/873)^{1/3}(40)^{1/3}$$

$$v_c = 0.773 \text{ N/mm}^2$$

$$\xi_s = (500/d)^{1/4} = (500 / 873)^{1/4} = 0.87$$

Max. allowable shear stress $\xi_s v_c$ = 0.87x0.773 = 0.673 N/mm^2

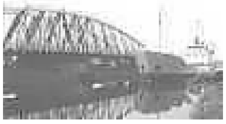
$V_c = \Sigma\xi_s v_c bd$ = 0.673 x 19800 x 873 x 10^{-3}
= 11633 kN < 11680Hence Fail

Increase second layer of steel to B25 @ 125 (A_s = 3927mm^2/m) then:

Total steel area = 10053 + 3927 = 13980 mm^2/m

Effective depth d = (10053 x 880 + 3927 x 815) / 13980

d = 862 mm



Transform A_s perp. to shear plane = 13980 x $\cos^2 20°$ = 12345 mm^2/m

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

$v_c = (0.27/1.25)(100 \times 12345/1000/862)^{1/3}(40)^{1/3}$

$v_c = 0.833\ N/mm^2$

$\xi_s = (500/d)^{1/4} = (500 / 862)^{1/4} = 0.873$

Max. allowable shear stress $\xi_s v_c$ = 0.873x0.833 = 0.727 N/mm^2

$V_c = \Sigma\xi_s v_c bd$ = 0.727 x 19800 x 862 x 10^{-3}

= 12408 kN > 11680 Hence OK

Increase 2nd layer over pier to B25@125c/c

Check shear on perimeters at 0.75d from critical perimeter:

0.75d = 0.75 x 862 = 647mm

@ (1.5 + 0.75)d either side of bearings:

distance between shear perimeters = 2 x 2.25 x 862 + 400 = 4279mm

Load on shear perimeters = 13184 - 1.1(1098+269)(4.279/3.040)

= 11067 kN < 11633 kN

Hence second layer can be reduced to B20@250c/c

Section at mid span

Maximum ULS Moment = 1665 kNm

Cl. 5.4.2 Cl. 5.3.2.3 $Mu = (0.87f_y)A_s z$

Deck concrete : C40/50 for exposure condition XD1

Steel reinforcement : Grade B500B to BS 4449:2005

$f_{cu} = 50$ N/mm^2 $f_y = 500$ N/mm^2

Width of grillage member = b = 0.913m

BS 8500 Pt 1

Table A5 Deck concrete is grade C40/50 with Class designation XD1 requires a cover to reinforcement of 35 + Δc = 35 + 15 = 50mm

try 32mm bars at 125 c/c then

d = 950 - 50 - 16 = 884mm

$A_s = (\pi \times 32^2 / 4) \times (0.913 \times 1000 / 125) = 5874$mm^2

z = [1 - (1.1 x 500 x 5874) / (50 x 913 x 884)]d = 0.92d

Mu = 0.87 x 500 x 5874 x 0.92 x 884 x 10^{-6} = 2078 kNm > 1665 kNm Hence OK

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} =	50	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	913	950
Table 13	Environment Conditions for nominal cover:		Moderate		
	Reinforcement controlling crack width :			Steel Reinforcement	
	Cover from notional surface =	30	mm	Area	Depth
	Bar diameter (ϕ)=	32	Comp.	0	0
	Spacing (s) =	125	Ten.	5874	884

Factored Dead Load Moment(Mg) = 696 kNm

Factored Live Load Moment(Mq) = 657 kNm

Table 3 Young's Modulus for Concrete E_c = 34 kN/mm^2

Cl. 4.3.2.1b) Modified E_c = Ec(1-0.5Mg/(Mg+Mq)) = 25.25 kN/mm^2

Modular ratio $\alpha_e = E_s/E_c$ = 7.921

$x/d_t = \alpha_e(A_{st}/(bd_t)+A_{sc}/(bd_t)+[(\alpha_e^2\{A_{st}/(bd_t)+A_{sc}/(bd_t)\}^2+2\alpha_e\{A_{st}/(bd_t)+A_{sc}d_c/(bd_t^2)\}]^{0.5}$

x/d_t = 0.287

x = 0.287 x 884 = 253.477 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 = 2.345E+10 mm^4

Steel stress = $\sigma_s = \alpha_e(Mg+Mq)(d_t-x)/Ixx$ = 288.099 N/mm^2

Steel strain = $\varepsilon_s = \sigma_s/E_s$ = 0.00144

Distance from Comp Face to notional surface = d_{ns} = 930 mm

Strain at notional surface = $\varepsilon_1 = \varepsilon_s(d_{ns}-x)/(d_t-x)$ = 0.00155

equation 25 Stiffening Effect of Conc in Tension = 2.12E-05

Modified Strain at notional surface = ε_m = 0.00152

Distance from crack to bar = $[(30+32/2)^2+(125/2)^2]^{0.5}-32/2$

Distance from crack to bar = 61.603 mm

Cl 5.8.8.2	Design Crack Width =	0.2583	mm		
equation 24	Maximum Crack Width from Table 1 :	0.25	mm	**Hence Fail**	
		Use B32@250c/c alternating with B40@250c/c			
		Increased steel area reduces crack width to 0.20mm			
	Stress Limitations (from analysis of B32@125c/c)				
Cl 4.1.1.3	Compressive stress in concrete =	14.63	N/mm^2		
	Compressive stress due to Temperature Difference =	2.32	N/mm^2		
	Total Compressive stress in concrete =	16.95	N/mm^2		
Table 2	Allowable compressive stress = 0.5fcu =	25	N/mm^2	Hence OK	
	Tensile stress in steel =	288.099	N/mm^2		
	Tensile stress due to Temperature Difference =	0	N/mm^2		**Mid Span**
	Total Tensile stress in steel =	288.099	N/mm^2		**section**
Table 2	Allowable tensile stress = 0.75fy =	375	N/mm^2	Hence OK	**B40@250c/c +**
					B32@250c/c

Cl. 5.4.4 **Shear adjacent to abutment**

As there is only minimal bending in the longitudinal direction adjacent to the abutment then the minimum steel area will be required.

Minimum steel required to prevent early thermal cracking = B16@200c/c

Steel in longitudinal direction is at 125c/c hence check shear capacity of

B16 @ 125 c/c $A_s = \pi \times 16^2/4 \times 913/125 = 1469\ mm^2$ $d = 950 - 50 - 8 = 892mm$

Max. ULS flexural shear = 275 kN

Cl. 5.4.4.1 Shear stress = $v = V/bd = 275 \times 10^3 / (913 \times 892) = 0.34\ N/mm^2$

Table 9 $\xi_s = (500/d)^{1/4} = (500 / 892)^{1/4} = 0.865$

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

$v_c = (0.27/1.25)(100x1469/913/892)^{1/3}(40)^{1/3}$

$0.42\ N/mm^2$

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

$= 0.75(50)^{1/2} = 5.3$ or 4.75 i.e. = 4.75 N/mm2

$0.42 < 4.75\ N/mm^2$ Hence OK

Cl. 5.4.4.1 Max. allowable shear stress without shear reinforcement = 0.865x0.42 = 0.36

0.36 > 0.34 Hence shear links not required.

At Abutment
B16@125c/c

Cl. 5.4.4.2 Punching Shear

Punching shear for wheel loads need only be checked for thin slabs, as in a beam and slab deck. However the calculation will be carried out for completeness.

BS 5400 Pt. 2

Cl.6.2.5 Nominal wheel load = 100 kN acting on a contact area 300x300mm.

Cl. 6.2.6 Dispersal through surfacing is 1:2 so area on top face of concrete = 425x425

Note: BS 5400 Pt 4 Clause 5.4.4.2 overwrites dispersal down to neutral axis.

BS 5400 Pt. 4 Using the minimum steel area provided near end of deck:

Longitudinal steel B16@125c/c ($A_s = 1608mm^2/m$, d = 950 - 50 - 8 = 892mm)

Skew transverse steel B16@200c/c ($A_s = 1005mm^2/m$, d = 892- 16 =876mm)

Transform skew steel into longitudinal and square transverse directions:

Longitudinal = $1005 \times \sin^2 20^\circ = 118mm^2/m$

Transverse = $1005 \times \cos^2 20^\circ = 887mm^2/m$ (d = 876mm)

Σ Longitudinal steel = $1608 + 118 = 1726\ mm^2/m$

$d = [(1608x892)+(118x876)]/1726 = 891mm$

Cl. 5.4.4.2 Critical perimeter at 1.5d from loaded area:

Length in longitudinal direction = 2 x 1.5 x 892 + 425 = 3101 mm

Length in transverse direction = 2 x 1.5 x 847 + 425 = 2966 mm

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

Longitudinal steel: $v_c = (0.27/1.25)(100x1726/1000/891)^{1/3}(40)^{1/3}$

$v_c = 0.427\ N/mm^2$

$\xi_s = (500/d)^{1/4} = (500 / 891)^{1/4} = 0.866$

Allowable shear stress $\xi_s v_c = 0.866x0.427 = 0.37\ N/mm^2$

Transverse steel: $v_c = (0.27/1.25)(100x887/1000/876)^{1/3}(40)^{1/3}$

$v_c = 0.344\ N/mm^2$

$\xi_s = (500/d)^{1/4} = (500 / 876)^{1/4} = 0.869$

Allowable shear stress $\xi_s v_c = 0.869x0.344 = 0.299\ N/mm^2$

$V_c = \Sigma\xi_s v_c bd = 2[(0.37x2966x891)+(0.299x3101x876)]10^{-3}$

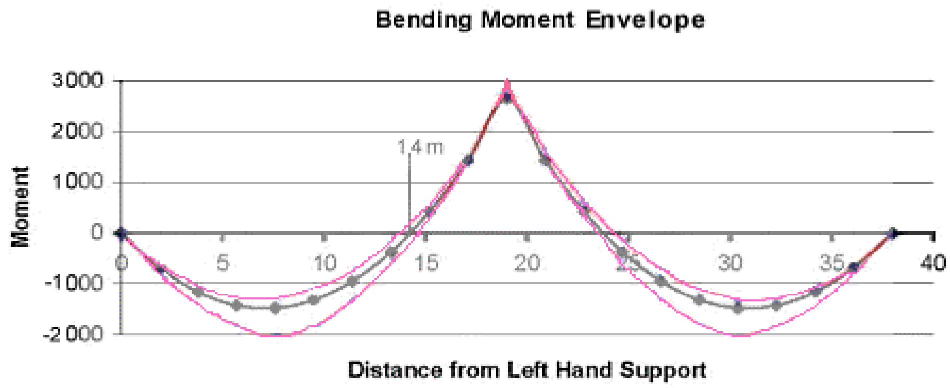
$= 3580$ kN

Ultimate shear force from wheel load = $\gamma_{f3} \times \gamma_{fL} \times 100$ kN

= 1.1 x 1.5 x 100 = 165 kN < 3580 kN Hence OK

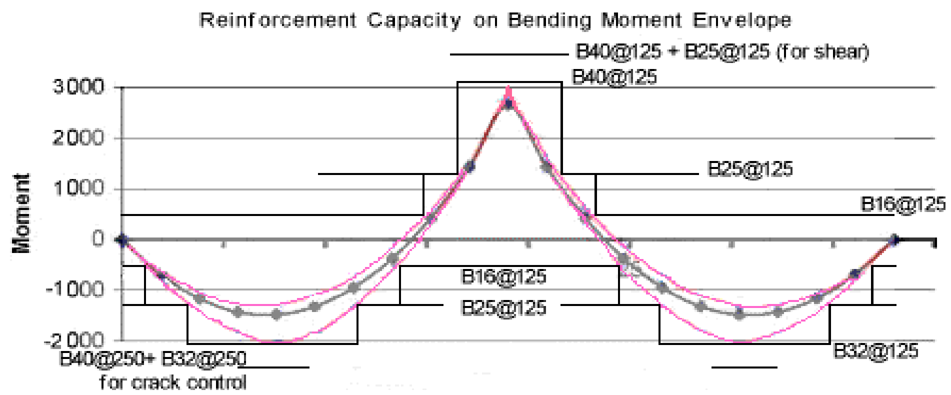
Curtailment

A line beam analysis of the dead load bending moment shows the point of contraflexure to be at about 14m from the abutment bearing (5m from the centre-line of the pier bearings.



An envelope of the bending moment results for all dead and live load cases will show the extent that reinforcement is required. We can superimpose the bending capacities for various bar sizes to determine where bars can be reduced.

A useful spreadsheet for obtaining the moment and shear capacities for various bar arrangements is '*reinfPt4.xls*' contained in '*303.zip*'.



Standard bar sizes in the UK are 12, 16, 25, 32 and 40mm diameter. It is good practice when reducing bar sizes to not miss more than one bar size. So the B25 bar is included in the layout above to avoid lapping a B40 bar to a B16.

Also bar diameters less than 12mm are avoided as the reinforcement needs to be robust to avoid being damaged on site.

Cl. 5.8.6.7 <u>Lap Lengths</u>

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

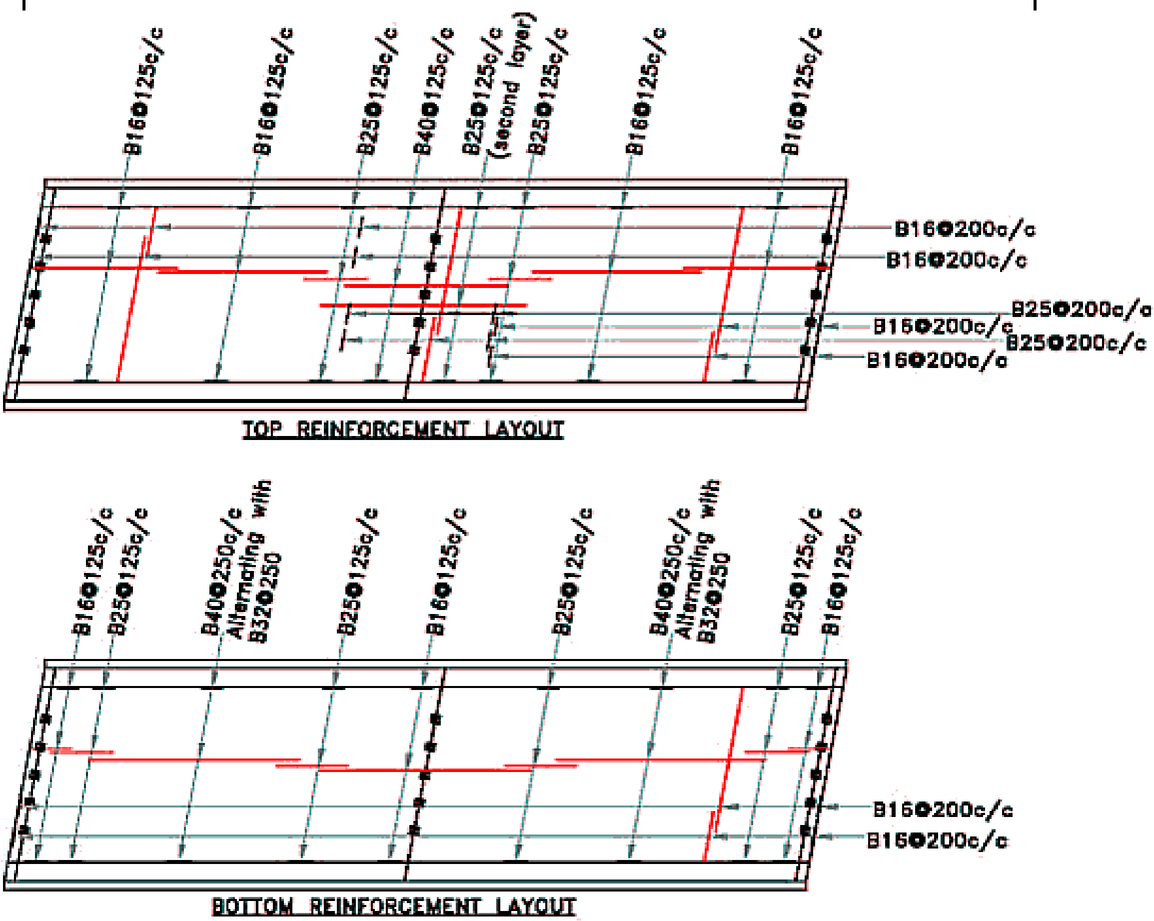
Care is needed in detailing to avoid the 1.4 or 2.0 enhancement factor required for the conditions highlighted in (a), (b) and (c) of Cl. 5.8.6.7.

Condition (b) is avoided by staggering bar laps in adjacent bars.

Although condition (a) applies to the 40mm bars in the top of the slab over the pier, the stress in the bar where a 25mm bar is lapped is only :

$$435 \times (25^2 / 40^2) = 0.4 \times 435 \text{ N/mm}^2$$

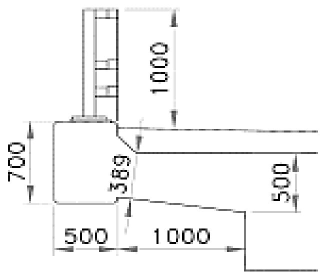
So $1.4 \times 0.4 = 0.56 < 1.0$ is still less than the lap required for the 25 mm bar



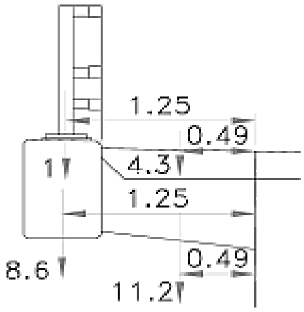
Note: Layout has been simplified. Laps in bars should be staggered.

<u>DECK REINFORCEMENT LAYOUT</u>

Parapet Cantilever



The maximum hogging moment in the cantilever, from the grillage analysis, occurs at the pier section. This load effect can be reduced by constructing the cantilever and parapet edge beam after the main deck concrete has been poured and dead load deflections have been allowed by releasing the soffit falsework. This also helps to achieve a uniform alignment to the edge beam.



Determine load effects about root of cantilever:
Dead Load:
Parapet = say 1kN/m
Parapet edge beam = 8.6kN/m
Cantilever = 11.2kN/m
Footway fill & surfacing = 4.3kN/m

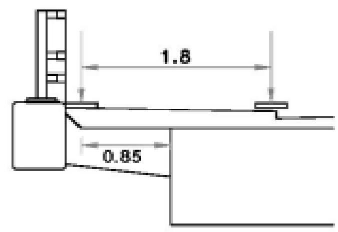
SLS Moment = (1x1.25)+(8.6x1.25)+(11.2x1.49)+(4.3x0.49)
= 1.25 + 10.75 + 16.69 + 2.1 = 30.8 kNm/m

ULS Moment = (1.2x1.25)+(1.15x10.75)+(1.15x16.69)+(1.2x2.1) = 35.6 kNm/m

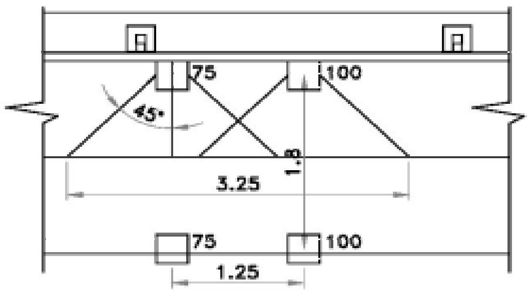
ULS Shear = (1.2x1.25)+(1.15x8.6)+(1.15x11.2)+(1.2x4.3) = 29.4 kN/m

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 root of the cantilever then the length of cantilever supporting these wheels is 3.25m (assuming a 300 x 300mm contact area).

SLS Moment = [1.2x0.85x(100+75)] / 3.25 = 54.9 kNm/m

ULS Moment = [1.5x0.85x(100+75)] / 3.25 = 68.7 kNm/m

ULS Shear = 1.5(100+75) / 3.25 = 80.8 kN/m

BS 8500 Pt 1 Depth of section = 500mm

Table A5 Deck concrete is grade C40/50 with Class designation XD1 requires a cover to reinforcement of 35 + Δc = 35 + 15 = 50mm

Transverse bars in main deck are at 200mm c/c and will align with the bars in the cantilever. The maximum cover to these bars will therefore be:

cover = 50 + 40(main longt'l bar dia.) = 90mm

Total SLS Moment = 30.8 + 54.9 = 85.7 kNm/m — SLS M=85.7kNm

Design ULS Moment = 1.1x(35.6 + 68.7) = 114.7 kNm/m — ULS M=114.7kNm

BS 5400 Pt 4 Design ULS Shear = 1.1x(29.4 + 80.8) = 121.7 kN/m — ULS V=121.7kN

Cl.5.8.8.2 M_q / M_g = 54.9 / 30.8 = 1.78

Using spreadsheet *'reinfPt4.xls'* contained in *'303.zip'* **B20@200 c/c** looks satisfactory:

BS 8500 Pt 1 — d = 500 - 50 - 40 - 10 = 400 (to align with main deck reinforcement)

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

We will need to check later that this cover can be provided when the size of the longitudinal and transverse steel has been determined.

$A_s = (\pi \times 20^2 / 4) \times (1000 / 200) = 1571$mm2

Cl. 5.3.2.3 — z = [1 - (1.1 x 500 x 1571) / (50 x 1000 x 400)]d = 0.957d hence use 0.95d

$M_u = (0.87f_y)A_s z = 0.87 \times 500 \times 1571 \times 0.95 \times 400 \times 10^{-6} = 260$ kNm > 114.7 kNm Hence OK

Note : With the wheel loads at the end of the deck the load distribution will be truncated and the maximum length of cantilever supporting the wheel loads

ULS design moment = 114.7 x 3.25 / 2.25 = 166 < 260 Hence B20@200 c/c still OK

Max ULS M=166kNm

will be reduced from 3.25m to 2.25m.

Cl. 5.8.8.2 — Maximum distance between bars for crack control:

Results obtained using spreadsheet 'CrackControl.xls' contained in '302.zip'

			Concrete Section	
Concrete Strength f_{cu} =	50	N/mm^2	Breadth	Depth
Steel Strength f_y =	500	N/mm^2	1000	500
Young's Modulus for Steel Es = (Cl. 4.3.2.2)	200000	N/mm^2		
Environment Conditions for nominal cover: (Table 13)	Moderate			
Reinforcement controlling crack width :			Steel Reinforcement	
Cover from notional surface =	25	mm	Area	Depth
Bar diameter (ϕ)=	20	Comp.	0	0
Spacing (s) =	200	Ten.	1571	400

Factored Dead Load Moment(Mg) = 30.8 kNm

Factored Live Load Moment(Mq) = 54.9 kNm

Table 3 — Young's Modulus for Concrete E_c = 34 kN/mm^2

Cl. 4.3.2.1b) — Modified Ec = Ec(1-0.5Mg/(Mg+Mq)) = 27.89 kN/mm^2

x/d = 0.211

x = 84.334 mm

Second Mom of Area of Cracked Section = 1.32E+09 mm^4

Steel Stress = 146.687 N/mm^2

Steel Strain = 0.00073

Distance from Comp Face to notional surface = 435 mm

Strain at notional surface = 0.00081

Stiffening Effect of Conc in Tension = 0 (from Equation 25)

Modified Strain at notional surface = 0.00081

Distance from crack to bar = 95.948 mm

Cl. 5.8.8.2 — Design Crack Width = 0.1748 mm (eqn. 24)

Minimum Crack Width from Table 1: 0.25 mm Hence OK

Stress Limitations

Cl. 4.1.1.3 Compressive stress in concrete = 5.47 N/mm^2
Compressive stress due to Temperature Difference = 0.00 N/mm^2
Total Compressive stress in concrete = 5.47 N/mm^2
Table 2 Allowable compressive stress = $0.5f_{cu}$ = 25 N/mm^2 Hence OK

Tensile stress in steel = 146.687 N/mm^2
Tensile stress due to Temperature Difference = 0.00 N/mm^2
Total Tensile stress in steel = 146.687 N/mm^2 — Main Cantilever bar B20@200c/c
Table 2 Allowable tensile stress = $0.75f_y$ = 375 N/mm^2 Hence OK

Use spreadsheet *'EarlyThermal.xls'* contained in *'302.zip'* to assess the shrinkage effects of casting the cantilever after the main deck has cured.

Restrained Section Length L = 38000 mm
Restrained Section Thickness T = 500 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 = 500000 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 = $f_{ct}*A_c/f_y$ = 1855.5 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 = 12×10^{-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 = $(f_{ct}/f_b)*A_c*\phi*[R*(\varepsilon_{sh}+\varepsilon_{th})-0.5*\varepsilon_{ult}]/(2*w)$ = 2667.14 mm^2/m..................(3)
Hence minimum area from equation (3) governs = 2667.14 mm^2/m
Min As in each face = 2667.14 / 2 = 1333.6 mm^2/m

B16 @ 150 c/c As = 1340 > 1333.6 Hence OK — Transverse Cantilever bars B16 @ 150 c/c

Check that 60mm cover can be achieved for exposure condition XD3:
Cover to main bars = 90mm hence cover to anti-cracking steel = 90 - 16 = 74 > 60mm Hence OK

Cl. 5.4.4 Shear Resistance

Cl. 5.4.4.1 Shear stress = v = V/bd = $121.7 \times 10^3 / (1000 \times 400) = 0.30$ N/mm^2

Table 9 $\xi_s = (500/d)^{1/4} = (500 / 400)^{1/4} = 1.057$

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

$v_c = (0.27/1.25)(100 \times 1571/1000/400)^{1/3}(40)^{1/3}$

$= 0.54$ N/mm^2

Max allowable shear stress without shear reinforcement = 1.057x0.54 = 0.57

0.30 < 0.57 Hence shear links not required.

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

0.30 < 4.75 Hence shear links not required.

Cl. 5.3.3.2 Additional Longitudinal reinforcement to enable truss action to develop:

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

$V/2(0.87f_y) = 121.7 \times 10^3 / (2 \times 0.87 \times 500)$

$= 140$ mm^2

A_{st} provided = 1571 mm^2

A_{st} req'd for Design Moment = 1571 x (160 / 260) = 967 mm^2

A_{st} available for truss action = 1571 - 967 = 604 mm^2 > 140 Hence OK

Secondary Reinforcement

Cl. 5.8.4.2 Minimum area of secondary reinforcement = 0.12% of b_td

0.12% x 1000 x 400 = 480 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 a parapet is required to be provided at the Normal Containment Level N2.

BS 5400 Pt2

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.

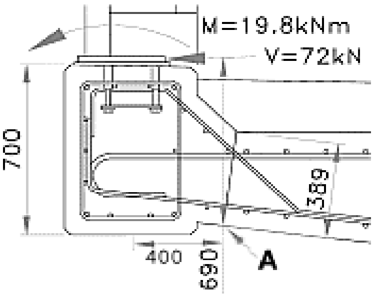
Corus manufacture a Protect 365^{TM} N2 parapet and give the following data:

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

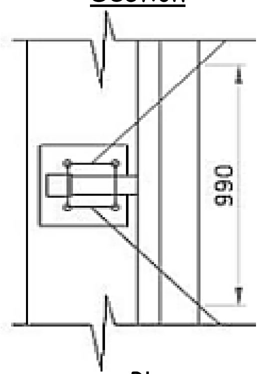
So Nominal loads are:

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



Plan

Check that the B20@200c/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 = 8.6 kN/m
Mass of parapet = 1kN/m approx.

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. = (8.6 + 1) x 0.4 x 0.99 = 3.8 kNm
SLS Live Load Mom. = 1.2 x (19.8 + 0.69 x 72) = 83.4 kNm
Total SLS Moment = 3.8 + 83.4 = 87.2 kNm

ULS Dead Load Mom. = [(1.15 x 8.6) + (1.2 x 1)] x 0.4 x 0.99 = 4.4 kNm
ULS Live Load Mom. = 1.5 x (19.8 + 0.69 x 72) = 104.2 kNm
Total ULS Design Moment = 1.1 (4.4 + 104.2) = 119.5 kNm

BS 8500 Pt 1
Table A5 Cantilever concrete is grade C40/50 with Class designation XD3 requires a cover to reinforcement of 45 + Δc = 45 + 15 = 60mm

Cover provided to main bars = 90mm (see before).
d = 389 - 90 - 10 = 289 mm
Mq / Mg = 83.4 / 3.8 = 21.9

Using spreadsheet *'reinfPt4.xls'* contained in *'303.zip'*:
B20@200 c/c gives M_{ult} = 184 kNm & M_{sls} = 85 kNm

Although the M_{sls} moment is slightly less than required (85 < 87.2) 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 @ 200 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)

$v = 119 \times 10^3 / (700 \times 420) = 0.41\ \text{N/mm}^2$

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

$A_s = 5 \times \pi \times 16^2 / 4 = 1005\ \text{mm}^2$

Edge Beam
B16 @ 150 c/c

Table 8 — $v_c = (0.27/\gamma_m)(100A_s/b_wd)^{1/3}(f_{cu})^{1/3}$

$v_c = (0.27/1.25)(100\text{x}1005/700/420)^{1/3}(40)^{1/3}$

$= 0.52\ \text{N/mm}^2$

$\xi_s = (500/d)^{1/4} = (500 / 420)^{1/4} = 1.04$

Table 7 — $\xi_s v_c = 0.52 \times 1.04 = 0.54\ \text{N/mm}^2 > 0.41$

Hence $A_{sv} > 0.4bs_v/0.87f_{yv}$

Using 12mm ϕ links then $A_s = 2 \times \pi \times 12^2 / 4 = 226\text{mm}^2$

Hence $s_v < 226 \times 0.87 \times 500 / (0.4 \times 700) = 351\text{mm}$

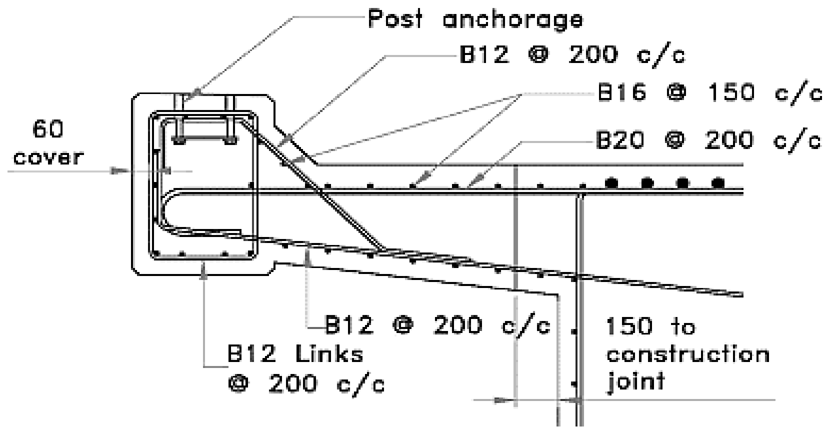
Use s_v = 200mm to align with bars in cantilever.

Edge Beam
B12 links @ 200

$A_{sa} > V / (2\text{x}0.87\text{x}f_y)$

$V / (2\text{x}0.87\text{x}f_y) = 119 \times 10^3 / (2 \times 0.87 \times 500) = 137\text{mm}^2$

A_{sa} = 1005 > 137 Hence OK



Reinforcement Layout