Parkway Infrastructure Engineers

Hatch Mott MacDonald

Calculation Cover Sheet

No. of Sheets 51

Project Title Windsor Essex Parkway | **Project No.** 285380 | **File No.** N/A

Section TB-4 - Substructure | **Subject** Calculation Rev. 1 | **Calc No.** 285380-03-103-0162

Project Manager Biljana Rajlic

Designer Boris Malac

Design Phase
- **A** Concept or preliminary
- **B** Analysis and Detailed design x
- **C** Design verification
- **D** Other (specify)

Computer Applications Used

Title	Version/Date
MS Excel	2010
STAAD.Pro V8.i	20.07.09.11

Scopes of Checking for Manual and Computer Generated Calculations

Conformance with CAN/CSA S6-06 and PA

Sheets Checked*	Calculations by Name	Signature	Date	Checked By Name	Signature	Date
1001-1020	B.M.		06-24-2014	F.A.		07-21-2014
2001-2020	B.M.		06-24-2014	F.A.		~~07-21-2014~~
3001-3003	A.V.(AMEC)	Ashley Verge	07-17-2014	D.D.(AMEC)		07-17-2014
4001-4003	J.L.		07-16-2014	B.M.		07-21-2014

*If an Excel spreadsheet or other computer file has been checked and printout has not been attached, enter the name, date and full file path or iPas location of the file that was checked.
(If held in iPas, giving the iPas nickname or pasting the shortlink from Properties – General could also be useful.)

a) Basic Design Information or Source and Reference:

Project Agreement and Geotechnical Report

b) Identify documents/technical records where output will be used:

Basis for structural drawing package

Approved by Project Manager Biljana Rajlic **Signature** Biljana Rajlic **Date** JULY 28, 2014

Distribution
Original to project file

CALCULATION INDEX SHEET
(Reference Procedure: DQSP 1.2 - Design and Checking)

Segment: Phase 1
Design Element: Trail Bridge TB-4 - Structural
Submittal: IFC

Checker: Francesco Addario
Date of Check: 7/21/2014
Checkers Document No.: 285380-03-103-0169

Originator: Boris Malac
Date of Issue: 22-Jul-14

Item	Page No.	Description	Prepared by:	Checked by:	Comment
1	1000 -	East Abutment	BMa	FA	
2	2000 -	West Abutment	BMa	FA	
3	3000 -	Geotechnical Review	AMEC	AMEC	
4	4000 -	Miscellaneous	JL	BMa	

TB-4: East Abutment Foundation Design

Hatch Mott MacDonald

Project Name	**Windsor - Essex - Parkway**		
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Subject	**TB-4 East Abutment**	Sheet #	
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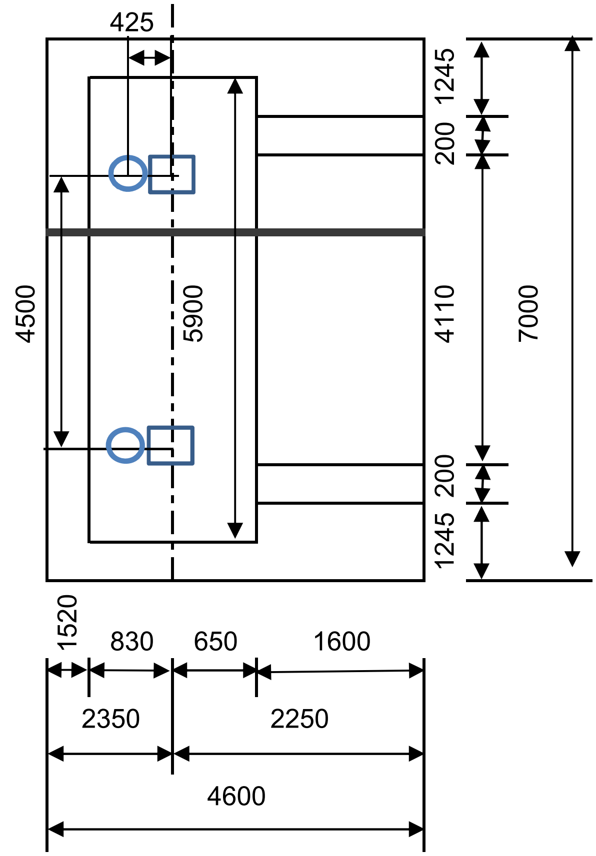
1. Abutment Geometry

All Sketch Dimensions in: mm
Unless mentioned

1.1 Dimensions

Total Length of Abutment =	5.900	m
c/l bearings to abut. face =	0.830	m
Avg. Ballast Wall Thickness =	0.380	m
Ballast Wall Height =	0.900	m
Abut. Stem Thickness =	1.480	m
Abutment Stem Height =	5.250	m
Total Abut. Ht. =	7.150	m
Abut. Ht. Above footing =	6.150	m
Jacking location offset =	0.425	m
Footing Thickness =	1.000	m
Footing Length =	7.000	m
Footing Width =	4.600	m
"Toe Length" =	1.520	m
"Heel Length" =	1.600	m
height of soil on on toe =	0.000	m
Wing Wall Thickness =	0.200	m
Inside to Inside of Wing Wall =	4.110	m
u/s brgs. to u/s ftg. =	6.250	m

C.L.



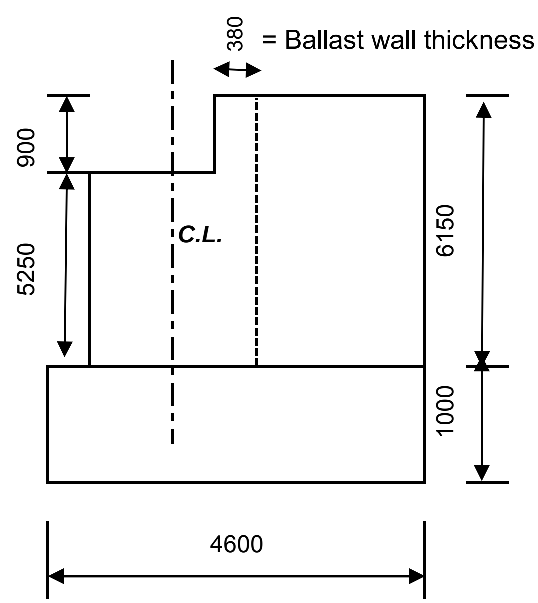
1.2 Geometry

*Area or section taken along red line above.

Area* =	1.3E+07	mm^2
Centroid 'y' =	2576.9	mm
Centroid 'x' =	2310.73	mm

Total

Volume=	83.5214	m^3

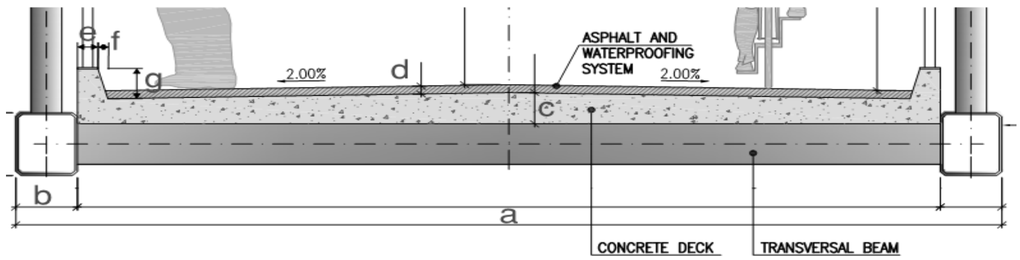


2. Loads on Bearings

2.1 Structural Self weight

Span s= 40 m

a	4814
b	254
c	222
d	50
e	102
f	51
g	150



slope 2 %

Concrete area = 914182 mm^2 Concrete unit weight = 24 kN/m^3
Asphalt area = 200000 mm^2 Asphalt unit weight = 24 kN/m^3

Concrete weight = 877.6 kN
Asphalt weight = 188 kN
Steel Weight = 27115 kg = 266 kN

2.2 Live Load

Uniform. Distributed = 5.0 - s / 30 = 3.7 kN/m^2
Total UDL Load = 586.7 kN

Maintenance vehicle = 80 kN
Vehicle with DLA = 96 kN

2.3 Temperature Load

*Assume temperature loading is 9 % of Vertical Load
Horizontal Load/Abut = 63 kN

2.4 Wind

Cl. 3.10

return period for determining of wind pressure is considered as 50 years
as the maximum span length is less than 125 m — Cl. 3.10.1.2(b)

hourly mean reference wind pressure: q = 470 Pa — Table A3.1.1
location: Windsor

gust effect coefficient: C_g = 2.5 — Cl. 3.10.1.3

height above ground: H = 9.7 m — Cl. 3.10.1.4

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wind exposure coefficient: C_e = $(0.1H)^{0.2} \leq 1.0$ => C_e = 1.0 — Cl. 3.10.1.4

superstructure:

horizontal wind drag coefficient: C_h = 2.0 — Cl. 3.10.2.2

horizontal drag load: $F_h = q\, C_e\, C_g\, C_h$ = 2350 Pa = 2.35 kN/m^2 — Cl. 3.10.2.2

Exposed area of superstructure to wind:

Drawings scale 1: 141

Truss	Length		Width	Amount	Area
	Plan	Real			
	[mm]	[mm]	[mm]	[-]	[m^2]
Top chord	251	35391	254	1	9.0
Bottom chord	290	40890	254	1	10.4
Verticals	24	3384	152	15	7.7
Diagonals - internal	29	4089	152	14	8.7
Diagonals - end	29	4089	152	2	1.2
Diag./vert. - reduction due slab		-350	152	31	-1.6
Rail top	290	40890	51	1	2.1
Rail - bars	290	40890	51	1	2.1
Rail middle		1100	25	320	8.8
Sum					**48.4**

Concrete slab	290	40890	332	1	**13.6**

X/h = 1.0 m — Cl. C3.10.2.2

Exposed frontal area A_s = 48.4 m^2

Gross area in elevation A = 160.0 m^3

A_s/A = 0.3

Shielding factor K_x = 0.48 — Table C3.6

lateral load on windward side: F_{h1} = 114 kN

lateral load on leeward side: F_{h2} = 55 kN

lateral load on bridge deck F_{h3} = 32 kN

lateral load on superstructure: F_h = 200 kN

vertical wind coefficient: C_v = 1.0 — Cl. 3.10.2.3

vertical load: $F_v = q\, C_e\, C_g\, C_v$ = 1175 Pa = 1.18 kN/m^2 — Cl. 3.10.2.3

exposed width of the superstructure: = 5.10 m

vertical line load per unit length: F_v = 5.99 kN/m

2.5 Factored Bearing Load

S6-06 Load Factors

	D1	D2	D3	E	L	K	W
ULS 1	1.1	1.2	1.5	1.25	1.7	0	0
ULS 2	1.1	1.2	1.5	1.25	1.6	1.15	0
ULS 3	1.1	1.2	1.5	1.25	1.4	1	0.5
ULS 4	1.1	1.2	1.5	1.25	0	1.25	1.65
ULS 9	1.35	1.35	1.35	1.25	0	0	0
SLS 1	1	1	1	1	0.9	0.8	0

D1--steel
D2--Cast in place
D3--Asphalt

	Vertical forces							Longit.	Lateral
	Concrete	Asphalt	Steel	LL	Wind	Total	Tot.per Abut.	per Abut.	per Abut.
ULS 1	1053	282	293	1161	0	2788	1394	0	0
ULS 2	1053	282	293	1092	0	2720	1360	72	0
ULS 3	1053	282	293	956	120	2703	1352	63	50
ULS 4	1053	282	293	0	396	2023	1012	78	165
ULS 9	1185	254	359	0	0	1798	899	0	0
SLS 1	878	188	266	614	0	1946	973	50	0

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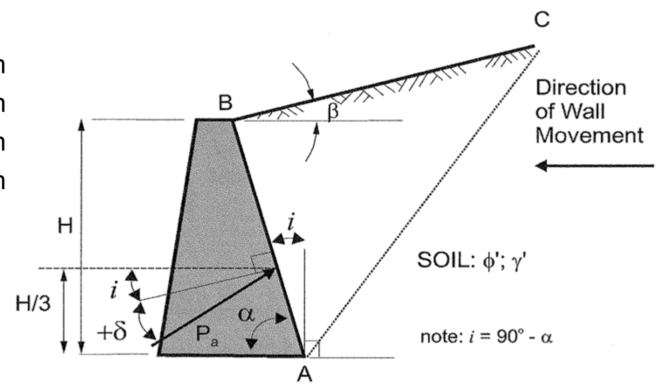
3. Loads on Abutment

3.1 Geometry behind stem

Asphalt thickness	=	80	mm
Approach slab thickness	=	250	mm
Soil fill thickness	=	0	mm
Depth of soil @ toe	=	1200	mm



Angles						
α	=	90 °	=	1.571	rad
β	=	-5 °	=	-0.09	rad
i	=	0 °	=	0	rad

3.2 Material Properties

Concrete unit weight		=	24.0 kN/m^3			
Asphalt unit weight		=	23.5 kN/m^3			
Soil fill Unit weight		=	21 kN/m^3			
ϕ of soil		=	33 °	=	0.576	rad
angle of wall friction	δ	=	16.5 °	=	0.288	rad

$$K_a = \cos(\delta+i)\frac{\sin^2(\alpha+\phi')}{\sin^2\alpha\ \sin^2(\alpha-\delta)\left(1+\sqrt{\dfrac{\sin(\delta+\phi')\ \sin(\phi'-\beta)}{\sin(\alpha-\delta)\ \sin(\alpha+\beta)}}\right)^2} = 0.243$$

3.3 Surcharge loads

LL surcharge taken as 0 m fill — cl.6.9.5

q_L =	0.0	kN/m^2	QL	0.0	kN/m

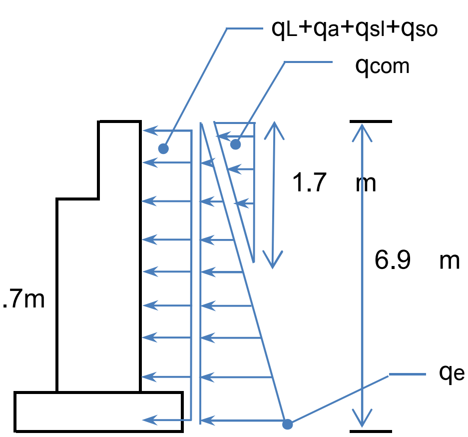
DL surcharge

asphalt (q_a)	0.458	kN/m^2	Qa	3.157	kN/m
slab (q_{sl})	1.46	kN/m^2	Qsl	10.07	kN/m
soil (q_{so})	0	kN/m^2	Qso	0	kN/m

3.4 Lateral Earth Pressure

Lateral Earth Pressure q_e = 35.26 kN/m^2
Q_e = 121.7 kN/m

Compaction (cl. 6.9.3)
q_{com} = 12 kN/m^2 from 0 -1.7m
Q_{com} = 10 kN/m



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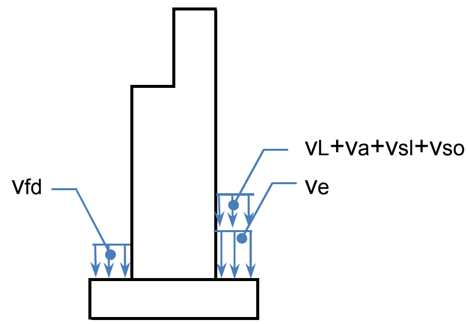
3.5 Earth Pressure Vertical Loads

Surcharge loads					
vL	0.00	kN/m^2	VL	0	kN/m
va	1.88	kN/m^2	Va	3.008	kN/m
vsl	6	kN/m^2	Vsl	9.6	kN/m
vso	0	kN/m^2	Vso	0	kN/m
ve	129.2	kN/m^2	Ve	206.64	kN/m
Surcharge on toe					
vfd	25.2	kN/m^2	Vfd	38.304	kN/m



3.6 S6-06 Load Factors

D1--steel
D2--Cast in place
D3--Asphalt

Max Load Factors

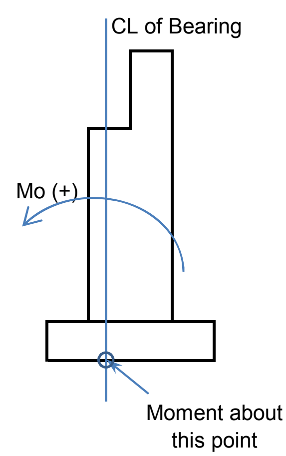
	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
D1	1.1	1.1	1.1	1.1	1.35	1
D2	1.2	1.2	1.2	1.2	1.35	1
D3	1.5	1.5	1.5	1.5	1.35	1
E	1.25	1.25	1.25	1.25	1.25	1
L	1.7	1.6	1.4	0	0	0.9
K	0	1.15	1	1.25	0	0.8

Min Load Factors

	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
D1	0.95	0.95	0.95	0.95	1.35	1
D2	0.9	0.9	0.9	0.9	1.35	1
D3	0.65	0.65	0.65	0.65	1.35	1
E	0.8	0.8	0.8	0.8	0.8	1
L	1.7	1.6	1.4	0	0	0.9
K	0	1.15	1	1.25	0	0.8

*Section below assumes no loads applied to wingwalls. Both Min and Max factors above are used to produce maxium negative and maximum positive bending moments.

*Bending moments are taken about the bearing center at the underside of footing, Mo



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3.7 Construction Loads

3.7.1 Construction Unfactored Loads

*This section is used to determine the natural sense of the each load components. i.e. if the unfactored component contibutes to overturning or not.

	H	Lever Arm	M (h)	V	Lever Arm	M (v)
LL Surcharge (QL) =	0	3.45	0	0	1.45	0
Asphalt Surcharge (Qa) =	0	3.45	0	0	1.45	0
Slab Surcharge (Qsl) =	0	3.45	0	0	1.45	0
Soil Surcharge (Qso)=	0	3.45	0	0	1.45	0
Lateral Earth Pre. (Qe)=	500	2.30	1150	849	1.45	-1231
Compaction (Qcom)=	42	6.33	266	0	0.00	0
Toe Soil Pressure (Vfd)=	0	0.00	0	268	1.59	426
Toe Pressure Sides (Vfd2)=	0	0.00	0	193	0.71	-137
Foundation Self Weight=	0	0.00	0	2005	-0.06	122

3.7.2 Construction Maximum Factored Horizontal Loads

Horizontal Loads in kN	Horizontal	Vertical	*Using Factors for Max (+) Moment	*Using Factors for Max (-) Moment
	SLS 1	SLS 1	SLS 1	SLS 1
LL Surcharge (QL) =	0	0	0	0
Asphalt Surcharge (Qa) =	0	0	0	0
Slab Surcharge (Qsl) =	0	0	0	0
Soil Surcharge (Qso)=	0	0	0	0
Lateral Earth Pre. (Qe)=	500	849	-81	-81
Compaction (Qcom)=	42	0	266	0
Toe Soil Pressure (Vfd)=	0	268	426	426
Toe Pressure Sides (Vfd2)=	0	193	-137	-137
Bearing Horizontal Load =	0	0	0	0
Foundation Self Weight=	0	2005	122	122
Total=	542	3315	595	329

3.8 Service Loads

3.8.1 Unfactored Loads

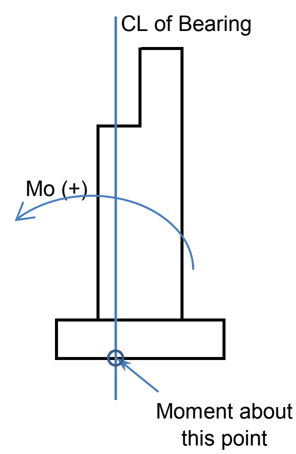
*This section is used to determine the natural sense of the each load components. i.e. if the unfactored component contibutes to overturning or not.

	H	Lever Arm	M (h)	V	Lever Arm	M (v)
LL Surcharge (QL) =	0	3.45	0	0	1.45	0
Asphalt Surcharge (Qa) =	13	3.45	45	12	1.45	-17.9
Slab Surcharge (Qsl) =	41	3.45	143	39	1.45	-57.2
Soil Surcharge (Qso)=	0	3.45	0	0	1.45	0
Earth Pressure (Qe)=	500	2.30	1150	849	1.45	-1231
Compaction (Qcom)=	0	0.00	0	0	0.00	0
Toe Soil Pressure (Vfd)=	0	0.00	0	268	1.59	426.3
Toe Pressure Sides (Vfd2)=	0	0.00	0	193	0.71	-137
Foundation Self Weight=	0	0.00	0	2005	-0.06	121.7

3.8.2 Maximum Factored Horizontal Loads

Horizontal Loads in kN

	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
LL Surcharge (QL) =	0	0	0	0	0	0
Asphalt Surcharge (Qa) =	19	19	19	19	18	13
Slab Surcharge (Qsl) =	50	50	50	50	56	41
Soil Surcharge (Qso)=	0	0	0	0	0	0
Earth Pressure (Qe)=	625	625	625	625	625	500
Compaction (Qcom)=	0	0	0	0	0	0
Toe Soil Pressure (Vfd)=	0	0	0	0	0	0
Toe Pressure Sides (Vfd2)=	0	0	0	0	0	0
Bearing Horizontal Load =	0	72	63	78	0	50
Foundation Self Weight=	0	0	0	0	0	0
Total=	694	766	757	772	698	604



3.8.3 Maximum Factored Vertical Loads

Vertical Loads in kN

	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
LL Surcharge (QL) =	0	0	0	0	0	0
Asphalt Surcharge (Qa) =	19	19	19	19	17	12
Slab Surcharge (Qsl) =	47	47	47	47	53	39
Soil Surcharge (Qso)=	0	0	0	0	0	0
Earth Pressure (Qe)=	1062	1062	1062	1062	1062	849
Compaction (Qcom)=	0	0	0	0	0	0
Toe Soil Pressure (Vfd)=	335	335	335	335	335	268
Toe Pressure Sides (Vfd2)=	242	242	242	242	242	193
Bearing Vertical Load =	1394	1360	1352	1012	899	973
Foundation Self Weight=	2405	2405	2405	2405	2706	2005
Total=	5504	5470	5461	5121	5313	4340

3.8.4 Maximum Factored Moment

Moment in kN.m

	*Using Factors for Max (+) Moment						
	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1	
LL Surcharge (QL) =	0	0	0	0	0	0	
Asphalt Surcharge (Qa) =	40	40	40	40	36	27	
Slab Surcharge (Qsl) =	103	103	103	103	116	86	
Soil Surcharge (Qso)=	0	0	0	0	0	0	
Earth Pressure. (Qe)=	-65	-65	-65	-65	-65	-81	
Compaction (Qcom)=	0	0	0	0	0	0	
Toe Soil Pressure (Vfd)=	533	533	533	533	533	426	
Toe Pressure Sides (Vfd2)=	-110	-110	-110	-110	-110	-137	
Bearing Horizontal Load =	0	450	391	489	0	313	
Bearing Vertical Load =					382		Jacking
Foundation Self Weight=	146	146	146	146	164	122	
Total	647	1097	1038	1136	1056	755	

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	*Using Factors for Max (-) Moment					
	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
LL Surcharge (QL) =	0	0	0	0	0	0
Asphalt Surcharge (Qa) =	17	17	17	17	36	27
Slab Surcharge (Qsl) =	77	77	77	77	116	86
Soil Surcharge (Qso)=	0	0	0	0	0	0
Earth Pressure. (Qe)=	-102	-102	-102	-102	-102	-81
Compaction (Qcom)=	0	0	0	0	1	0
Toe Soil Pressure (Vfd)=	341	341	341	341	341	426
Toe Pressure Sides (Vfd2)=	-172	-172	-172	-172	-172	-137
Bearing Horizontal Load =	0	-450	-391	-489	0	-313
Bearing Vertical Load =					382	
Foundation Self Weight=	110	110	110	110	164	122
Total	272	-178	-119	-217	766.9	129

Jacking

3.9 Reactions in Footing Bottom

		ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1	
Longitudinal Forces		694	766	757	772	698	604	kN
Vertical Forces		5504	5470	5461	5121	5313	4340	kN
Longitudinal Bending Moments	max	647	1097	1038	1136	1056	755	kN.m
	min	272	-178	-119	-217	767	129	kN.m
Lateral Forces		0	0	50	165	0	0	kN
Lateral Bending Moments		0	0	313	1032	0	0	kN.m

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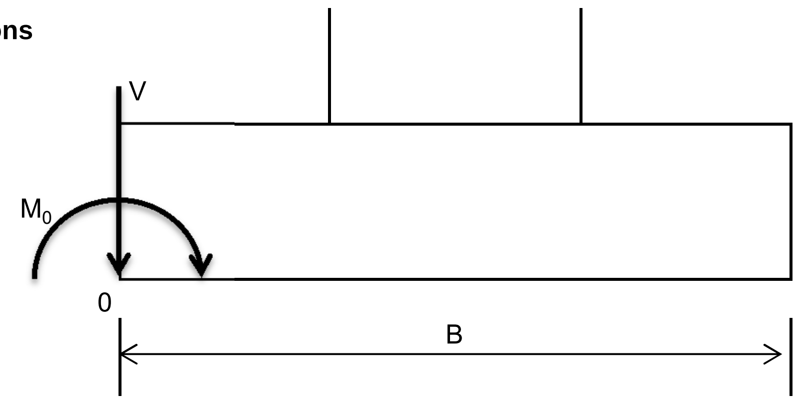
4. Spread Footing

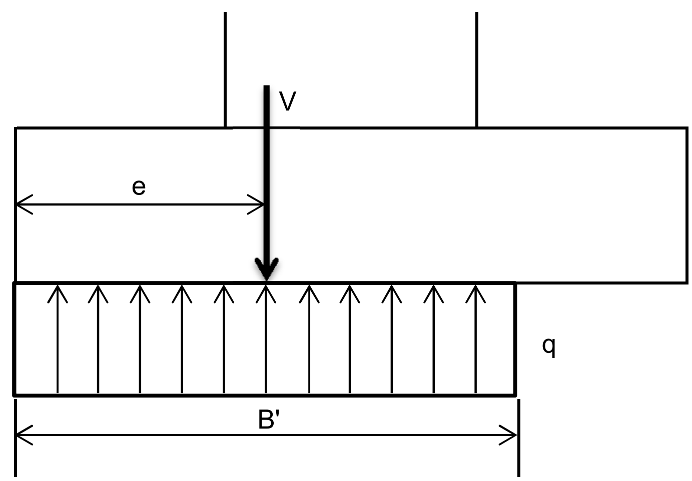
Spread footing geometry and reactions

Length L = 7.00 m

Width B = 4.60 m

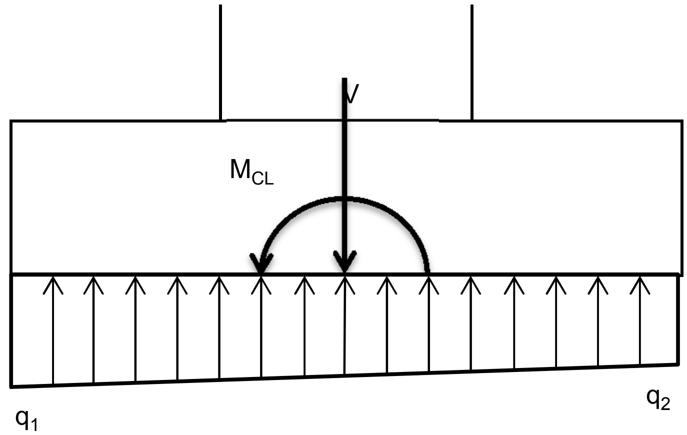


Uniform load distribution



		ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1	
Vertical force	V =	5504	5470	5461	5121	5313	4340	kN
Bending moment	M_0 =	12287	11757	11796	10899	11430	9444	kNm
Bending moment-trans.	M_T =	0	0	50	165	0	0	kNm
Excentricity	e_L =	0.00	0.00	0.01	0.03	0.00	0.00	m
Excentricity	e_B =	2.23	2.15	2.16	2.13	2.15	2.18	m
Effective length	$L' = L - 2\,e_L$ =	7.00	7.00	6.98	6.94	7.00	7.00	m
Effective width	B' =	4.46	4.30	4.32	4.26	4.30	4.35	m
Stress in footing bottom	$q = V / (B' L')$ =	**176**	**182**	**181**	**173**	**176**	**142**	kN/m^2

Linear load distribution @ ULS2



Bending moment at CL

M_{CL} = V (B/2 - e) = 823.2 kNm

Stress in footing bottom

$q_1 = (V\,B + 6M_{CL})/(B^2\,L) =$ 203 kPa

$q_2 = (V\,B - 6M_{CL})/(B^2\,L) =$ 137 kPa

Note: This is just approximate calculation. The foundation design is carried by AMEC based on foundation loads supplied by HMM.

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5. Internal forces

Max Load Factors

	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
E	1.25	1.25	1.25	1.25	1.25	1

5.1 Footing

Length of cantilever L = 1.52 m

		ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1	
Uniform load distribution in footing bottom	$q =$	176	182	181	173	176	142	kPa
Bending moment (factored)	$M = 1/2\ q\ L^2 =$	203	210	209	200	204	165	kNm/m
Shear force (factored)	$V = q\ L =$	268	276	275	264	268	217	kN/m

5.2 Stem

Lateral earth pressure $q_e =$ 36.1 kPa

Surcharge $q_s =$ 1.9 kPa

Height of stem L = 6.15 m

Bending moment $M = 1/6\ q_e\ L^2 + 1/2\ q_s\ L^2 =$ 264 kNm/m

Shear force $V = 1/2\ q_e\ L + q_s\ L =$ 123 kN/m

Height under bearings L = 5.25 m

Factored internal forces

	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1	
Bending Moment	330	394	386	400	330	309	kN.m/m
Shear Force	154	166	164	167	154	131	kN/m

5.3 Ballast wall

Lateral earth pressure $q_e =$ 5.3 kPa

Surcharge $q_s =$ 1.9 kPa

Height of stem L = 0.90 m

Bending moment $M = 1/6\ q_e\ L^2 + 1/2\ q_s\ L^2 =$ 1 kNm/m

Shear force $V = 1/2\ q_e\ L + q_s\ L =$ 4 kN/m

Factored internal forces

	ULS 1	ULS 2	ULS 3	ULS 4	SLS 1	
Bending Moment	2	2	2	2	1	kN.m/m
Shear Force	5	5	5	5	4	kN/m

6.1 Cross section check - Bending — Footing

Mat'l Strength - Concrete =	35	MPa	β_1 = 0.97 - 0.0025 $f_C' \geq$ 0.67	Cl. 8.8.3 (f)
- Steel =	400	MPa	.:. β_1 = 0.88	
Material Factors - Concrete=	0.75		α_1 = 0.85-0.0015 $f_C' \geq$ 0.67	Cl. 8.8.3 (f)
- Steel =	0.9		.:. α_1 = 0.80	
$E_c=(3000(f_C')^{0.5}+6900)\ (g_C / 2300)^{1.5}$=	27932	MPa		Cl. 8.4.1.7
E_s =	200000	MPa		

Width of the member: 1000 mm
Depth of the member: 1000 mm
Cover to rebar: 120 mm

	SLS	ULS 1	ULS 2	ULS 3	ULS 4
6. Factored design moment [kNm]:	0	203	210	209	200

DESIGN REINFORCING:

	ULS 1	ULS 2	ULS 3	ULS 4
FOR ULS, THE REQ'D REINF. RATIO =	0.00075	0.00078	0.00077	0.00074
.:. REQ'D AREA OF REINF. [mm²] =	**654**	**675**	**672**	**644**

d = **870** mm

INPUT STEEL AREA PROVIDED: 20M x 5 pcs = **1571** mm^2

O.K. for strength requirements

CHECK IF SECTION IS CRACKED UNDER SLS

fcr = $0.4(fc')^{0.5}$ = 2.37 MPa

Mcr = fcr [b h^2 / 6] = 394 kNm

M SLS = 0 kNm < Mcr

.:. Section is **Uncracked**

Note: If section is cracked, check crack control in the following (next page).
If section is uncracked, crack control is not needed.

CHECK MIN. STEEL: where, Mr $\geq$ 1.2 Mcr ….. OR ….. Mr > 1.333 Mf — Cl. 8.8.4.3

a = $\phi_S A_S f_y/[\alpha_1 \phi_C f_C' b]$ = 27 mm

.:. Mr = $\phi_S A_S f_y$ [d-a/2] = 484 kNm

Cl. 8.4.1.8

1.20 Mcr = 473 kNm < Mr **O.K. FOR MIN. STEEL**

CHECK MAX. STEEL: c / d $\leq$ 0.5 (ensures reinforcement yields) — Cl. 8.8.4.5

c = a/b_1 = 30.61 mm

then, c/d = 0.04 < 0.5 **O.K. FOR MAX. STEEL**

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6.2 Cross section check - Shear   Footing

Factored shear force at ULS   Vf = 276 kN

bv =	1000	mm	Cl. 8.9.1.6
h =	1000	mm	
d =	870	mm	
dv =max(0.72h,0.9d)=	783	mm	Cl. 8.9.1.5

Simplified method for shear

β =	0.180	Cl. 8.9.3.6
θ =	42	

$V_C = 2.5*\beta\ (\phi_C)\ f_{CR}\ b_V\ d_V$ = 625.4 > V_f = 276.3 kN   Cl. 8.9.3.4

BY 126 %

O.K., NO SHEAR BARS ARE NEEDED

6.3 Cross section check - Bending — Stem

Mat'l Strength - Concrete = 35 MPa — $\beta_1 = 0.97 - 0.0025\, f_C' \geq 0.67$ — Cl. 8.8.3 (f)
- Steel = 400 MPa — $\therefore \beta_1 = 0.88$

Material Factors - Concrete= 0.75 — $\alpha_1 = 0.85\text{-}0.0015\, f_C' \geq 0.67$ — Cl. 8.8.3 (f)
- Steel = 0.9 — $\therefore \alpha_1 = 0.80$

$E_c=(3000(f_C')^{0.5}+6900)\,(g_C / 2300)^{1.5}=$ 27932 MPa — Cl. 8.4.1.7
$E_s =$ 200000 MPa

Width of the member: 1000 mm
Depth of the member: 1200 mm
Cover to rebar: 70 mm

	SLS	ULS 1	ULS 2	ULS 3	ULS 4
6. Factored design moment [kNm]:	309	330	394	386	400
<u>DESIGN REINFORCING:</u>					
FOR ULS, THE REQ'D REINF. RATIO =		0.00074	0.00088	0.00086	0.0009
.:. REQ'D AREA OF REINF. [mm^2] =		**826**	**987**	**966**	**1001**

d = **1117.5** mm

INPUT STEEL AREA PROVIDED: 25M x 5 pcs = **2454** mm^2

O.K. for strength requirements

<u>CHECK IF SECTION IS CRACKED UNDER SLS</u>

fcr = $0.4(fc')^{0.5}$ = 2.37 MPa
Mcr = fcr [b h^2 / 6] = 568 kNm
M SLS = 309 kNm < Mcr
.:. Section is **Uncracked**

<u>Note</u>: If section is cracked, check crack control in the following (next page).
If section is uncracked, crack control is not needed.

<u>CHECK MIN. STEEL:</u> where, Mr $\geq$ 1.2 Mcr ….. OR ….. Mr > 1.333 Mf — Cl. 8.8.4.3

$a = \phi_S A_S f_y/[\alpha_1 \phi_C f_C' b] =$ 42 mm
$\therefore Mr = \phi_S A_S f_y [d-a/2] =$ 969 kNm

Cl. 8.4.1.8

1.20 Mcr = 682 kNm < Mr **O.K. FOR MIN. STEEL**

<u>CHECK MAX. STEEL:</u> c / d $\leq$ 0.5 (ensures reinforcement yields) — Cl. 8.8.4.5

$c = a/b_1 =$ 47.83 mm

then, c/d = 0.04 < 0.5 **O.K. FOR MAX. STEEL**

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6.4 Cross section check - Shear Stem

Factored shear force at ULS Vf = 167 kN

bv =	1000	mm	Cl. 8.9.1.6
h =	1200	mm	
d =	1118	mm	
dv =max(0.72h,0.9d)=	1006	mm	Cl. 8.9.1.5

Simplified method for shear

β =	0.180	Cl. 8.9.3.6
θ =	42	

$V_C = 2.5 * \beta\ (\phi_C)\ f_{CR}\ b_V\ d_V$ = 803.3 > V_f = 166.9 kN — Cl. 8.9.3.4

BY 381 %

O.K., NO SHEAR BARS ARE NEEDED

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6.5 Cross section check - Bending — Ballast wall

Mat'l Strength - Concrete = 35 MPa — $\beta_1 = 0.97 - 0.0025\ f_C' \geq 0.67$ — Cl. 8.8.3 (f)

- Steel = 400 MPa — $\therefore\ \beta_1 = 0.88$

Material Factors - Concrete= 0.75 — $\alpha_1 = 0.85\text{-}0.0015\ f_C' \geq 0.67$ — Cl. 8.8.3 (f)

- Steel = 0.9 — $\therefore\ \alpha_1 = 0.80$

$E_c=(3000(f_C')^{0.5}+6900)\ (g_C / 2300)^{1.5}=$ 27932 MPa — Cl. 8.4.1.7

$E_s =$ 200000 MPa

Width of the member: 1000 mm
Depth of the member: 380 mm
Cover to rebar: 70 mm

	SLS	ULS 1	ULS 2	ULS 3	ULS 4
6. Factored design moment [kNm]:	1	2	2	2	2
<u>DESIGN REINFORCING:</u>					
FOR ULS, THE REQ'D REINF. RATIO =		5.7E-05	5.7E-05	5.7E-05	5.7E-05
.:. REQ'D AREA OF REINF. [mm²] =		**17**	**17**	**17**	**17**

d = **302.5** mm

INPUT STEEL AREA PROVIDED: 15M x 5 pcs = **884** mm^2

O.K. for strength requirements

<u>CHECK IF SECTION IS CRACKED UNDER SLS</u>

fcr = $0.4(fc')^{0.5}$ = 2.37 MPa

Mcr = fcr [b h^2 / 6] = 57 kNm

M SLS = 1 kNm < Mcr

.:. Section is **Uncracked**

<u>Note</u>: If section is cracked, check crack control in the following (next page).
If section is uncracked, crack control is not needed.

<u>CHECK MIN. STEEL:</u> where, Mr $\geq$ 1.2 Mcr ….. OR ….. Mr > 1.333 Mf — Cl. 8.8.4.3

$a = \phi_S A_S f_y/[\alpha_1 \phi_C f_C' b] =$ 15 mm

$\therefore\ Mr = \phi_S A_S f_y [d\text{-}a/2] =$ 94 kNm

Cl. 8.4.1.8

1.20 Mcr = 68 kNm < Mr **O.K. FOR MIN. STEEL**

<u>CHECK MAX. STEEL:</u> c / d $\leq$ 0.5 (ensures reinforcement yields) — Cl. 8.8.4.5

c = a/b_1 = 17.22 mm

then, c/d = 0.06 < 0.5 **O.K. FOR MAX. STEEL**

6.6 Cross section check - Shear Ballast wall

Factored shear force at ULS Vf = 5 kN

bv =	1000	mm	Cl. 8.9.1.6
h =	380	mm	
d =	303	mm	
dv =max(0.72h,0.9d)=	274	mm	Cl. 8.9.1.5

Simplified method for shear

β =	0.180	Cl. 8.9.3.6
θ =	42	

$V_C = 2.5*\beta\ (\phi_C)\ f_{CR}\ b_V\ d_V$ = 218.5 > V_f = 5.1 kN Cl. 8.9.3.4

BY 4157 %

O.K., NO SHEAR BARS ARE NEEDED

7. Layout and reinforcement

Suggested layout and reinforcement

	Thickness of section [mm]	Rebar Diameter [mm]	Rebar Spacing [mm]
Ballast wall	380	15M	200
Stem	1200	25M	200
Footing	1000	20M	200

No shear reinforcement needed.

TB-4: West Abutment Foundation Design

Hatch Mott MacDonald

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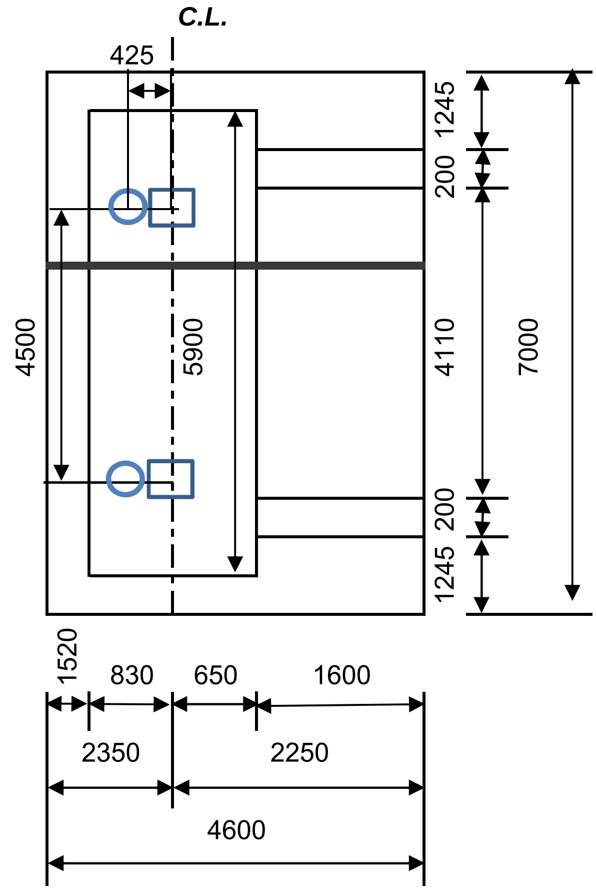
1. Abutment Geometry

All Sketch Dimensions in: mm
Unless mentioned

1.1 Dimensions

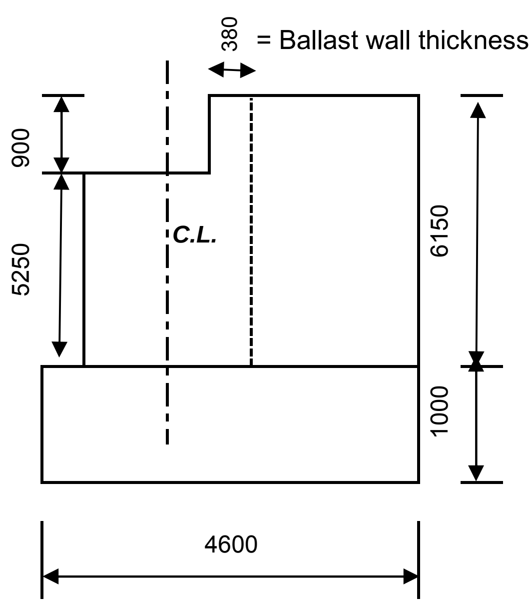
Total Length of Abutment =	5.900	m
c/l bearings to abut. face =	0.830	m
Avg. Ballast Wall Thickness =	0.380	m
Ballast Wall Height =	0.900	m
Abut. Stem Thickness =	1.480	m
Abutment Stem Height =	5.250	m
Total Abut. Ht. =	7.150	m
Abut. Ht. Above footing =	6.150	m
Jacking location offset =	0.425	m
Footing Thickness =	1.000	m
Footing Length =	7.000	m
Footing Width =	4.600	m
"Toe Length" =	1.520	m
"Heel Length" =	1.600	m
height of soil on on toe =	0.000	m
Wing Wall Thickness =	0.200	m
Inside to Inside of Wing Wall =	4.110	m
u/s brgs. to u/s ftg. =	6.250	m



1.2 Geometry

*Area or section taken along red line above.

Area* =	1.3E+07	mm^2
Centroid 'y' =	2576.9	mm
Centroid 'x' =	2310.73	mm
Total		
Volume=	83.5214	m^3

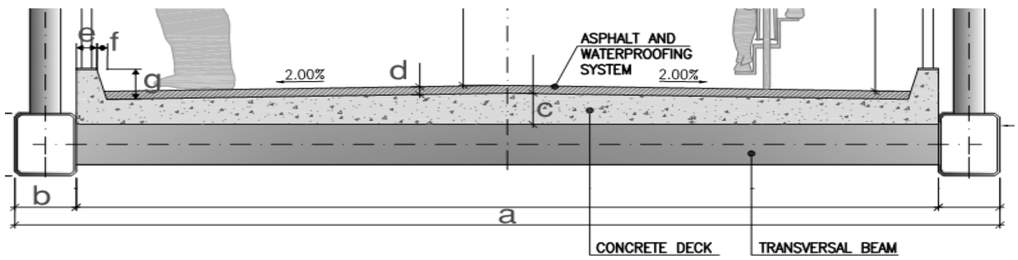


2. Loads on Bearings

2.1 Structural Self weight

Span s= 40 m

a	4814
b	254
c	222
d	50
e	102
f	51
g	150



slope 2 %

Concrete area = 914182 mm^2 Concrete unit weight = 24 kN/m^3
Asphalt area = 200000 mm^2 Asphalt unit weight = 24 kN/m^3

Concrete weight = 877.6 kN
Asphalt weight = 188 kN
Steel Weight = 27115 kg = 266 kN

2.2 Live Load

Uniform. Distributed = 5.0 - s / 30 = 3.7 kN/m^2
Total UDL Load = 586.7 kN

Maintenance vehicle = 80 kN
Vehicle with DLA = 96 kN

2.3 Temperature Load

*Assume temperature loading is 9 % of Vertical Load
Horizontal Load/Abut = 63 kN

2.4 Wind

Cl. 3.10

return period for determining of wind pressure is considered as 50 years
as the maximum span length is less than 125 m Cl. 3.10.1.2(b)

hourly mean reference wind pressure: q = 470 Pa Table A3.1.1
location: Windsor

gust effect coefficient: C_g = 2.5 Cl. 3.10.1.3

height above ground: H = 9.7 m Cl. 3.10.1.4

wind exposure coefficient: C_e = $(0.1H)^{0.2} \leq 1.0$ => C_e = 1.0 Cl. 3.10.1.4

superstructure:

horizontal wind drag coefficient: C_h = 2.0 Cl. 3.10.2.2

horizontal drag load: $F_h = q\, C_e\, C_g\, C_h$ = 2350 Pa = 2.35 kN/m^2 Cl. 3.10.2.2

Exposed area of superstructure to wind:

Drawings scale 1: 141

Truss	Length		Width	Amount	Area
	Plan	Real			
	[mm]	[mm]	[mm]	[-]	[m^2]
Top chord	251	35391	254	1	9.0
Bottom chord	290	40890	254	1	10.4
Verticals	24	3384	152	15	7.7
Diagonals - internal	29	4089	152	14	8.7
Diagonals - end	29	4089	152	2	1.2
Diag./vert. - reduction due slab		-350	152	31	-1.6
Rail top	290	40890	51	1	2.1
Rail - bars	290	40890	51	1	2.1
Rail middle		1100	25	320	8.8
Sum					**48.4**

Concrete slab	290	40890	332	1	**13.6**

X/h = 1.0 m Cl. C3.10.2.2

Exposed frontal area A_s = 48.4 m^2

Gross area in elevation A = 160.0 m^3

A_s/A = 0.3

Shielding factor K_x = 0.48 Table C3.6

lateral load on windward side: F_{h1} = 114 kN

lateral load on leeward side: F_{h2} = 55 kN

lateral load on bridge deck F_{h3} = 32 kN

lateral load on superstructure: F_h = 200 kN

vertical wind coefficient: C_v = 1.0 Cl. 3.10.2.3

vertical load: $F_v = q\, C_e\, C_g\, C_v$ = 1175 Pa = 1.18 kN/m^2 Cl. 3.10.2.3

exposed width of the superstructure: = 5.10 m

vertical line load per unit length: F_v = 5.99 kN/m

2.5 Factored Bearing Load

S6-06 Load Factors

	D1	D2	D3	E	L	K	W
ULS 1	1.1	1.2	1.5	1.25	1.7	0	0
ULS 2	1.1	1.2	1.5	1.25	1.6	1.15	0
ULS 3	1.1	1.2	1.5	1.25	1.4	1	0.5
ULS 4	1.1	1.2	1.5	1.25	0	1.25	1.65
ULS 9	1.35	1.35	1.35	1.25	0	0	0
SLS 1	1	1	1	1	0.9	0.8	0

D1--steel
D2--Cast in place
D3--Asphalt

	Vertical forces							Longit.	Lateral
	Concrete	Asphalt	Steel	LL	Wind	Total	Tot.per Abut.	per Abut.	per Abut.
ULS 1	1053	282	293	1161	0	2788	1394	0	0
ULS 2	1053	282	293	1092	0	2720	1360	72	0
ULS 3	1053	282	293	956	120	2703	1352	63	50
ULS 4	1053	282	293	0	396	2023	1012	78	165
ULS 9	1185	254	359	0	0	1798	899	0	0
SLS 1	878	188	266	614	0	1946	973	50	0

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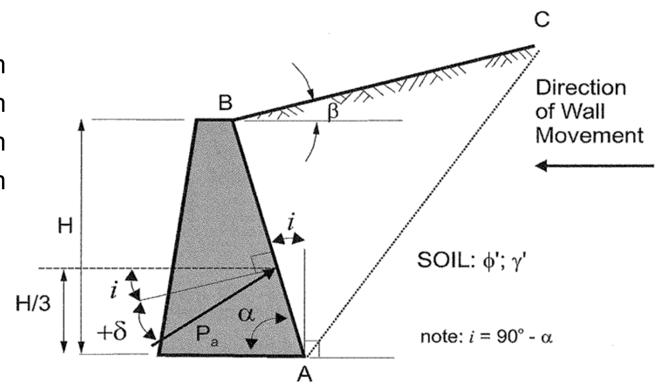
3. Loads on Abutment

3.1 Geometry behind stem

Asphalt thickness	=	80	mm
Approach slab thickness	=	250	mm
Soil fill thickness	=	0	mm
Depth of soil @ toe	=	1200	mm



Angles	α	=	90 °	=	1.571	rad
	β	=	-3 °	=	-0.05	rad
	i	=	0 °	=	0	rad

3.2 Material Properties

Concrete unit weight		=	24.0 kN/m^3			
Asphalt unit weight		=	23.5 kN/m^3			
Soil fill Unit weight		=	21 kN/m^3			
φ of soil		=	33 °	=	0.576	rad
angle of wall friction	δ	=	16.5 °	=	0.288	rad

$$K_a = \frac{\cos(\delta+i)\ \sin^2(\alpha+\phi')}{\sin^2\alpha\ \sin^2(\alpha-\delta)\left(1+\sqrt{\dfrac{\sin(\delta+\phi')\ \sin(\phi'-\beta)}{\sin(\alpha-\delta)\ \sin(\alpha+\beta)}}\right)^2} = 0.248$$

3.3 Surcharge loads

LL surcharge taken as 0 m fill cl.6.9.5

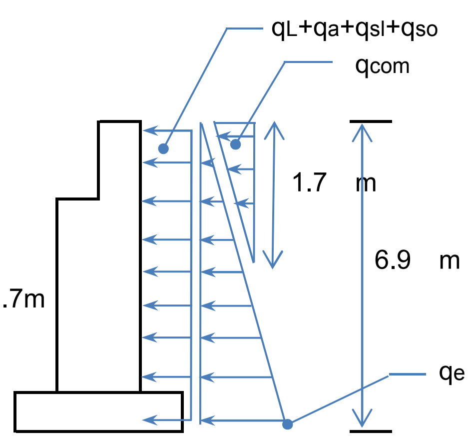
q_L =	0.0	kN/m^2	QL	0.0	kN/m

DL surcharge

asphalt (qa)	0.467	kN/m^2	Qa	3.22	kN/m
slab (qsl)	1.49	kN/m^2	Qsl	10.28	kN/m
soil (qso)	0	kN/m^2	Qso	0	kN/m

3.4 Lateral Earth Pressure

Lateral Earth Pressure qe = 35.97 kN/m^2
Qe = 124.1 kN/m



Compaction q_{com} = 12 kN/m^2 from 0 -1.7m
(cl. 6.9.3) Q_{com} = 10 kN/m

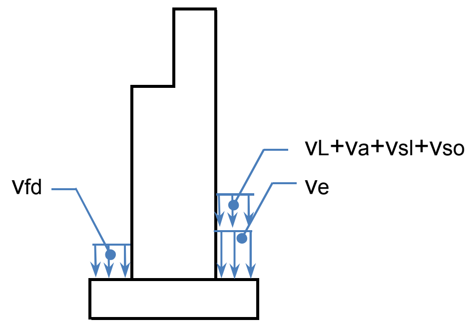
3.5 Earth Pressure Vertical Loads

Surcharge loads

vL	0.00	kN/m^2	VL	0	kN/m
va	1.88	kN/m^2	Va	3.008	kN/m
vsl	6	kN/m^2	Vsl	9.6	kN/m
vso	0	kN/m^2	Vso	0	kN/m
ve	129.2	kN/m^2	Ve	206.64	kN/m

Surcharge on toe

vfd	25.2	kN/m^2	Vfd	38.304	kN/m



3.6 S6-06 Load Factors

D1--steel
D2--Cast in place
D3--Asphalt

Max Load Factors

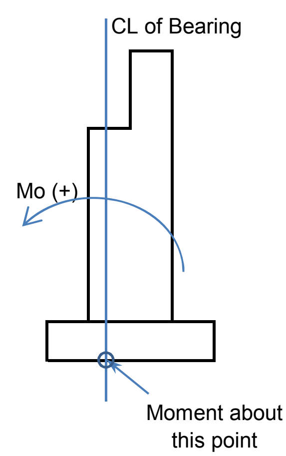
	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
D1	1.1	1.1	1.1	1.1	1.35	1
D2	1.2	1.2	1.2	1.2	1.35	1
D3	1.5	1.5	1.5	1.5	1.35	1
E	1.25	1.25	1.25	1.25	1.25	1
L	1.7	1.6	1.4	0	0	0.9
K	0	1.15	1	1.25	0	0.8

Min Load Factors

	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
D1	0.95	0.95	0.95	0.95	1.35	1
D2	0.9	0.9	0.9	0.9	1.35	1
D3	0.65	0.65	0.65	0.65	1.35	1
E	0.8	0.8	0.8	0.8	0.8	1
L	1.7	1.6	1.4	0	0	0.9
K	0	1.15	1	1.25	0	0.8

*Section below assumes no loads applied to wingwalls. Both Min and Max factors above are used to produce maxium negative and maximum positive bending moments.

*Bending moments are taken about the bearing center at the underside of footing, Mo



3.7 Construction Loads

3.7.1 Construction Unfactored Loads

*This section is used to determine the natural sense of the each load components. i.e. if the unfactored component contibutes to overturning or not.

	H	Lever Arm	M (h)	V	Lever Arm	M (v)
LL Surcharge (QL) =	0	3.45	0	0	1.45	0
Asphalt Surcharge (Qa) =	0	3.45	0	0	1.45	0
Slab Surcharge (Qsl) =	0	3.45	0	0	1.45	0
Soil Surcharge (Qso)=	0	3.45	0	0	1.45	0
Lateral Earth Pre. (Qe)=	510	2.30	1173	849	1.45	-1231
Compaction (Qcom)=	42	6.33	266	0	0.00	0
Toe Soil Pressure (Vfd)=	0	0.00	0	268	1.59	426
Toe Pressure Sides (Vfd2)=	0	0.00	0	193	0.71	-137
Foundation Self Weight=	0	0.00	0	2005	-0.06	122

3.7.2 Construction Maximum Factored Horizontal Loads

Horizontal Loads in kN	Horizontal	Vertical	*Using Factors for Max (+) Moment	*Using Factors for Max (-) Moment
	SLS 1	SLS 1	SLS 1	SLS 1
LL Surcharge (QL) =	0	0	0	0
Asphalt Surcharge (Qa) =	0	0	0	0
Slab Surcharge (Qsl) =	0	0	0	0
Soil Surcharge (Qso)=	0	0	0	0
Lateral Earth Pre. (Qe)=	510	849	-58	-58
Compaction (Qcom)=	42	0	266	0
Toe Soil Pressure (Vfd)=	0	268	426	426
Toe Pressure Sides (Vfd2)=	0	193	-137	-137
Bearing Horizontal Load =	0	0	0	0
Foundation Self Weight=	0	2005	122	122
Total=	552	3315	618	353

3.8 Service Loads

3.8.1 Unfactored Loads

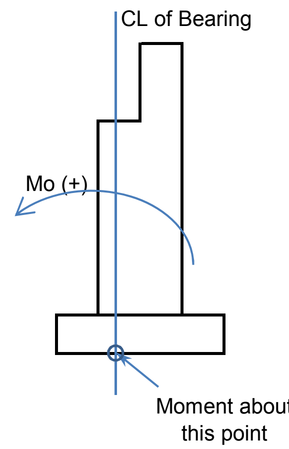
*This section is used to determine the natural sense of the each load components. i.e. if the unfactored component contibutes to overturning or not.

	H	Lever Arm	M (h)	V	Lever Arm	M (v)
LL Surcharge (QL) =	0	3.45	0	0	1.45	0
Asphalt Surcharge (Qa) =	13	3.45	46	12	1.45	-17.9
Slab Surcharge (Qsl) =	42	3.45	146	39	1.45	-57.2
Soil Surcharge (Qso)=	0	3.45	0	0	1.45	0
Earth Pressure (Qe)=	510	2.30	1173	849	1.45	-1231
Compaction (Qcom)=	0	0.00	0	0	0.00	0
Toe Soil Pressure (Vfd)=	0	0.00	0	268	1.59	426.3
Toe Pressure Sides (Vfd2)=	0	0.00	0	193	0.71	-137
Foundation Self Weight=	0	0.00	0	2005	-0.06	121.7

3.8.2 Maximum Factored Horizontal Loads

Horizontal Loads in kN

	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
LL Surcharge (QL) =	0	0	0	0	0	0
Asphalt Surcharge (Qa) =	20	20	20	20	18	13
Slab Surcharge (Qsl) =	51	51	51	51	57	42
Soil Surcharge (Qso)=	0	0	0	0	0	0
Earth Pressure (Qe)=	638	638	638	638	638	510
Compaction (Qcom)=	0	0	0	0	0	0
Toe Soil Pressure (Vfd)=	0	0	0	0	0	0
Toe Pressure Sides (Vfd2)=	0	0	0	0	0	0
Bearing Horizontal Load =	0	72	63	78	0	50
Foundation Self Weight=	0	0	0	0	0	0
Total=	708	780	771	786	712	616



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3.8.3 Maximum Factored Vertical Loads

Vertical Loads in kN

	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
LL Surcharge (QL) =	0	0	0	0	0	0
Asphalt Surcharge (Qa) =	19	19	19	19	17	12
Slab Surcharge (Qsl) =	47	47	47	47	53	39
Soil Surcharge (Qso)=	0	0	0	0	0	0
Earth Pressure (Qe)=	1062	1062	1062	1062	1062	849
Compaction (Qcom)=	0	0	0	0	0	0
Toe Soil Pressure (Vfd)=	335	335	335	335	335	268
Toe Pressure Sides (Vfd2)=	242	242	242	242	242	193
Bearing Vertical Load =	1394	1360	1352	1012	899	973
Foundation Self Weight=	2405	2405	2405	2405	2706	2005
Total=	5504	5470	5461	5121	5313	4340

3.8.4 Maximum Factored Moment

Moment in kN.m

	*Using Factors for Max (+) Moment						
	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1	
LL Surcharge (QL) =	0	0	0	0	0	0	
Asphalt Surcharge (Qa) =	42	42	42	42	37	28	
Slab Surcharge (Qsl) =	106	106	106	106	120	89	
Soil Surcharge (Qso)=	0	0	0	0	0	0	
Earth Pressure. (Qe)=	-47	-47	-47	-47	-47	-58	
Compaction (Qcom)=	0	0	0	0	0	0	
Toe Soil Pressure (Vfd)=	533	533	533	533	533	426	
Toe Pressure Sides (Vfd2)=	-110	-110	-110	-110	-110	-137	
Bearing Horizontal Load =	0	450	391	489	0	313	
Bearing Vertical Load =					382		Jacking
Foundation Self Weight=	146	146	146	146	164	122	
Total	670	1120	1061	1159	1080	782	

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	*Using Factors for Max (-) Moment					
	ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
LL Surcharge (QL) =	0	0	0	0	0	0
Asphalt Surcharge (Qa) =	18	18	18	18	37	28
Slab Surcharge (Qsl) =	80	80	80	80	120	89
Soil Surcharge (Qso)=	0	0	0	0	0	0
Earth Pressure. (Qe)=	-73	-73	-73	-73	-73	-58
Compaction (Qcom)=	0	0	0	0	1	0
Toe Soil Pressure (Vfd)=	341	341	341	341	341	426
Toe Pressure Sides (Vfd2)=	-172	-172	-172	-172	-172	-137
Bearing Horizontal Load =	0	-450	-391	-489	0	-313
Bearing Vertical Load =					382	
Foundation Self Weight=	110	110	110	110	164	122
Total	304	-146	-87	-185	800.9	156

Jacking

3.9 Reactions in Footing Bottom

		ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1	
Longitudinal Forces		708	780	771	786	712	616	kN
Vertical Forces		5504	5470	5461	5121	5313	4340	kN
Longitudinal Bending Moments	max	670	1120	1061	1159	1080	782	kN.m
	min	304	-146	-87	-185	801	156	kN.m
Lateral Forces		0	0	50	165	0	0	kN
Lateral Bending Moments		0	0	313	1032	0	0	kN.m

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4. Spread Footing

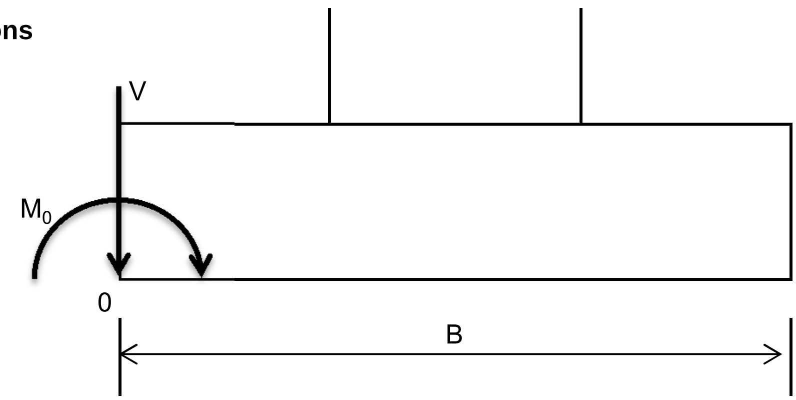
Spread footing geometry and reactions

Length $L =$ 7.00 m

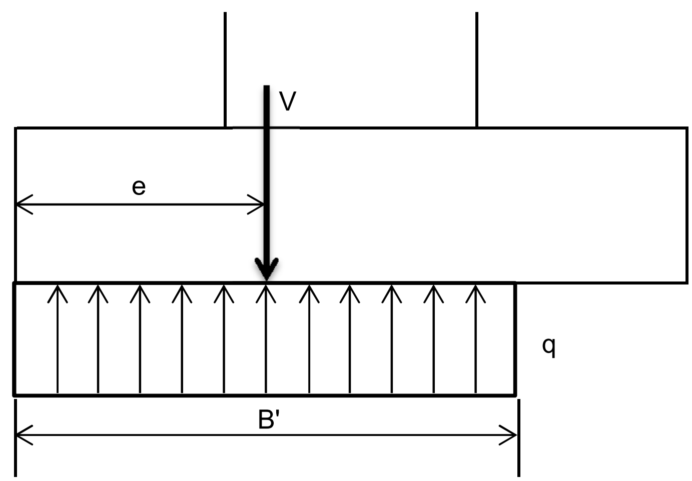
Width $B =$ 4.60 m

Vertical force $V =$ kN

Bending moment $M_0 =$ kNm



Uniform load distribution



		ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1	
Vertical force	$V =$	5504	5470	5461	5121	5313	4340	kN
Bending moment	$M_0 =$	12264	11734	11773	10876	11406	9417	kNm
Bending moment-trans.	$M_T =$	0	0	50	165	0	0	kNm
Excentricity	$e_L =$	0.00	0.00	0.01	0.03	0.00	0.00	m
Excentricity	$e_B =$	2.23	2.15	2.16	2.12	2.15	2.17	m
Effective length	$L' = L - 2\,e_L =$	7.00	7.00	6.98	6.94	7.00	7.00	m
Effective width	$B' =$	4.46	4.29	4.31	4.25	4.29	4.34	m
Stress in footing bottom	$q = V / (B'\,L') =$	**176**	**182**	**181**	**174**	**177**	**143**	kN/m^2

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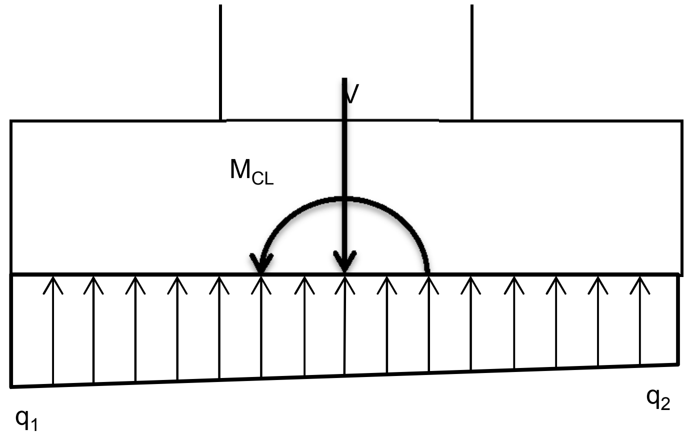
Linear load distribution @ ULS2



Bending moment at CL

M_{CL} = V (B/2 - e) = 846.6 kNm

Stress in footing bottom

$q_1 = (V B + 6M_{CL})/(B^2 L)$ = 204 kPa

$q_2 = (V B - 6M_{CL})/(B^2 L)$ = 136 kPa

Note: This is just approximate calculation. The foundation design is carried by AMEC based on foundation loads supplied by HMM.

5. Internal forces

Max Load Factors

	ULS 1	ULS 2	ULS 3	ULS 4	SLS 1
E	1.25	1.25	1.25	1.25	1

5.1 Footing

Uniform load distribution in footing bottom	$q =$	182	kPa
Length of cantilever	$L =$	1.52	m
Bending moment (factored)	$M = 1/2\ q\ L^2 =$	210	kNm/m
Shear force (factored)	$V = q\ L =$	277	kN/m

5.2 Stem

Lateral earth pressure	$q_e =$	36.9	kPa
Surcharge	$q_s =$	2.0	kPa
Height of stem	$L =$	6.15	m
Bending moment	$M = 1/6\ q_e\ L^2 + 1/2\ q_s\ L^2 =$	269	kNm/m
Shear force	$V = 1/2\ q_e\ L + q_s\ L =$	125	kN/m

Height under bearings	$L =$	5.25	m

Factored internal forces

	ULS 1	ULS 2	ULS 3	ULS 4	SLS 1	
Bending Moment	337	401	392	406	314	kN.m/m
Shear Force	157	169	167	170	134	kN/m

5.3 Ballast wall

Lateral earth pressure	$q_e =$	5.4	kPa
Surcharge	$q_s =$	2.0	kPa
Height of stem	$L =$	0.90	m
Bending moment	$M = 1/6\ q_e\ L^2 + 1/2\ q_s\ L^2 =$	2	kNm/m
Shear force	$V = 1/2\ q_e\ L + q_s\ L =$	4	kN/m

Factored internal forces

	ULS 1	ULS 2	ULS 3	ULS 4	SLS 1	
Bending Moment	2	2	2	2	2	kN.m/m
Shear Force	5	5	5	5	4	kN/m

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6.1 Cross section check - Bending — Footing

Mat'l Strength - Concrete = 35 MPa — $\beta_1 = 0.97 - 0.0025\, f_C' \geq 0.67$ — Cl. 8.8.3 (f)
- Steel = 400 MPa — $\therefore \beta_1 = 0.88$

Material Factors - Concrete= 0.75 — $\alpha_1 = 0.85\text{-}0.0015\, f_C' \geq 0.67$ — Cl. 8.8.3 (f)
- Steel = 0.9 — $\therefore \alpha_1 = 0.80$

$E_c=(3000(f_C')^{0.5}+6900)\,(g_C / 2300)^{1.5}=$ 27932 MPa — Cl. 8.4.1.7
$E_s =$ 200000 MPa

Width of the member: 1000 mm
Depth of the member: 1000 mm
Cover to rebar: 120 mm

	SLS	ULS 1	ULS 2	ULS 3	ULS 4
6. Factored design moment [kNm]:	175	210	210	210	210
DESIGN REINFORCING:					
FOR ULS, THE REQ'D REINF. RATIO =		0.00078	0.00078	0.00078	0.00078
.:. REQ'D AREA OF REINF. [mm^2] =		**676**	**676**	**676**	**676**

d = **870** mm

INPUT STEEL AREA PROVIDED: 20M x 5 pcs = **1571** mm^2

O.K. for strength requirements

CHECK IF SECTION IS CRACKED UNDER SLS

fcr = $0.4(fc')^{0.5}$ = 2.37 MPa
Mcr = fcr [b h^2 / 6] = 394 kNm
M SLS = 175 kNm < Mcr
.:. Section is **Uncracked**

Note: If section is cracked, check crack control in the following (next page).
If section is uncracked, crack control is not needed.

CHECK MIN. STEEL: where, Mr $\geq$ 1.2 Mcr ….. OR ….. Mr > 1.333 Mf — Cl. 8.8.4.3

$a = \phi_S A_S f_y/[\alpha_1 \phi_C f_C' b] =$ 27 mm
$\therefore Mr = \phi_S A_S f_y [d-a/2] =$ 484 kNm

Cl. 8.4.1.8

1.20 Mcr = 473 kNm < Mr **O.K. FOR MIN. STEEL**

CHECK MAX. STEEL: c / d $\leq$ 0.5 (ensures reinforcement yields) — Cl. 8.8.4.5

c = a/b$_1$ = 30.61 mm

then, c/d = 0.04 < 0.5 **O.K. FOR MAX. STEEL**

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6.2 Cross section check - Shear   Footing

Factored shear force at ULS Vf = 277 kN

bv =	1000	mm	Cl. 8.9.1.6
h =	1000	mm	
d =	870	mm	
dv =max(0.72h,0.9d)=	783	mm	Cl. 8.9.1.5

Simplified method for shear

β =	0.180	Cl. 8.9.3.6
θ =	42	

$V_C = 2.5 * \beta\ (\phi_C)\ f_{CR}\ b_V\ d_V$ = 625.4 > V_f = 276.8 kN Cl. 8.9.3.4

BY 126 %

O.K., NO SHEAR BARS ARE NEEDED

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6.3 Cross section check - Bending   Stem

Mat'l Strength - Concrete = 35 MPa
- Steel = 400 MPa

$\beta_1 = 0.97 - 0.0025\ f_C' \geq 0.67$ Cl. 8.8.3 (f)

$\therefore \beta_1 = 0.88$

Material Factors - Concrete= 0.75
- Steel = 0.9

$\alpha_1 = 0.85 - 0.0015\ f_C' \geq 0.67$ Cl. 8.8.3 (f)

$\therefore \alpha_1 = 0.80$

$E_c = (3000(f_C')^{0.5}+6900)\ (g_C / 2300)^{1.5} =$ 27932 MPa   Cl. 8.4.1.7

$E_s =$ 200000 MPa

Width of the member: 1000 mm
Depth of the member: 1200 mm
Cover to rebar: 70 mm

	SLS	ULS 1	ULS 2	ULS 3	ULS 4
6. Factored design moment [kNm]:	314	337	401	392	406

DESIGN REINFORCING:

	ULS 1	ULS 2	ULS 3	ULS 4
FOR ULS, THE REQ'D REINF. RATIO =	0.00075	0.0009	0.00088	0.00091
.:. REQ'D AREA OF REINF. [mm^2] =	**843**	**1004**	**982**	**1018**

d = **1117.5** mm

INPUT STEEL AREA PROVIDED: 25M x 5 pcs = **2454** mm^2

O.K. for strength requirements

CHECK IF SECTION IS CRACKED UNDER SLS

fcr = $0.4(fc')^{0.5}$ = 2.37 MPa

Mcr = fcr [b h^2 / 6] = 568 kNm

M SLS = 314 kNm < Mcr

.:. Section is **Uncracked**

Note: If section is cracked, check crack control in the following (next page).
If section is uncracked, crack control is not needed.

CHECK MIN. STEEL: where, $Mr \geq 1.2\ Mcr$ ….. OR ….. $Mr > 1.333\ Mf$ Cl. 8.8.4.3

$a = \phi_S\ A_S\ f_y / [\alpha_1\ \phi_C\ f_C'\ b] =$ 42 mm

$\therefore Mr = \phi_S\ A_S\ f_y\ [d - a/2] =$ 969 kNm

Cl. 8.4.1.8

1.20 Mcr = 682 kNm < Mr **O.K. FOR MIN. STEEL**

CHECK MAX. STEEL: $c / d \leq 0.5$ (ensures reinforcement yields)   Cl. 8.8.4.5

$c = a/b_1 =$ 47.83 mm

then, c/d = 0.04 < 0.5 **O.K. FOR MAX. STEEL**

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6.4 Cross section check - Shear Stem

Factored shear force at ULS Vf = 277 kN

bv =	1000	mm	Cl. 8.9.1.6
h =	1200	mm	
d =	1118	mm	
dv =max(0.72h,0.9d)=	1006	mm	Cl. 8.9.1.5

Simplified method for shear

β =	0.180	Cl. 8.9.3.6
θ =	42	

$V_C = 2.5 * \beta\ (\phi_C)\ f_{CR}\ b_V\ d_V$ = 803.3 > V_f = 276.8 kN Cl. 8.9.3.4

BY 190 %

O.K., NO SHEAR BARS ARE NEEDED

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6.5 Cross section check - Bending — Ballast wall

Mat'l Strength - Concrete =	35	MPa	$\beta_1 = 0.97 - 0.0025\, f_C' \geq 0.67$	Cl. 8.8.3 (f)
- Steel =	400	MPa	.:. $\beta_1 = 0.88$	
Material Factors - Concrete=	0.75		$\alpha_1 = 0.85\text{-}0.0015\, f_C' \geq 0.67$	Cl. 8.8.3 (f)
- Steel =	0.9		.:. $\alpha_1 = 0.80$	
$E_c=(3000(f_C')^{0.5}+6900)\,(g_C / 2300)^{1.5}=$	27932	MPa		Cl. 8.4.1.7
$E_s =$	200000	MPa		
Width of the member:	1000	mm		
Depth of the member:	380	mm		
Cover to rebar:	70	mm		

	SLS	ULS 1	ULS 2	ULS 3	ULS 4
6. Factored design moment [kNm]:	2	2	2	2	2

DESIGN REINFORCING:

	ULS 1	ULS 2	ULS 3	ULS 4
FOR ULS, THE REQ'D REINF. RATIO =	5.8E-05	5.8E-05	5.8E-05	5.8E-05
.:. REQ'D AREA OF REINF. [mm²] =	**17**	**17**	**17**	**17**

d = **302.5** mm

INPUT STEEL AREA PROVIDED: 15M x 5 pcs = **884** mm^2

O.K. for strength requirements

CHECK IF SECTION IS CRACKED UNDER SLS

$fcr = 0.4(fc')^{0.5} =$ 2.37 MPa

$Mcr = fcr\,[\,b\,h^2 / 6\,] =$ 57 kNm

M SLS = 2 kNm < Mcr

.:. Section is **Uncracked**

Note: If section is cracked, check crack control in the following (next page).
If section is uncracked, crack control is not needed.

CHECK MIN. STEEL: where, $Mr \geq 1.2\, Mcr$ ….. OR ….. $Mr > 1.333\, Mf$ — Cl. 8.8.4.3

$a = \phi_S\, A_S\, f_y/[\alpha_1\, \phi_C\, f_C'\, b] =$ 15 mm

.:. $Mr = \phi_S\, A_S\, f_y\,[d\text{-}a/2] =$ 94 kNm

Cl. 8.4.1.8

1.20 Mcr = 68 kNm < Mr **O.K. FOR MIN. STEEL**

CHECK MAX. STEEL: $c / d \leq 0.5$ (ensures reinforcement yields) — Cl. 8.8.4.5

$c = a/b_1 =$ 17.22 mm

then, c/d = 0.06 < 0.5 **O.K. FOR MAX. STEEL**

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6.6 Cross section check - Shear — Ballast wall

Factored shear force at ULS Vf = 5 kN

bv =	1000	mm	Cl. 8.9.1.6
h =	380	mm	
d =	303	mm	
dv =max(0.72h,0.9d)=	274	mm	Cl. 8.9.1.5

Simplified method for shear

β =	0.180	Cl. 8.9.3.6
θ =	42	

$V_C = 2.5 * \beta\ (\phi_C)\ f_{CR}\ b_V\ d_V$ = 218.5 > V_f = 5.2 kN — Cl. 8.9.3.4

BY 4073 %

O.K., NO SHEAR BARS ARE NEEDED

7. Layout and reinforcement

Suggested layout and reinforcement

	Thickness of section [mm]	Rebar Diameter [mm]	Rebar Spacing [mm]
Ballast wall	380	15M	200
Stem	1200	25M	200
Footing	1000	20M	200

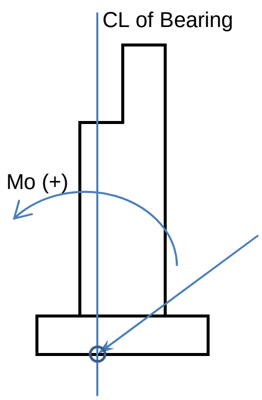
No shear reinforcement needed.

TB-4: Foundation Geotechnical Checks

(AMEC)

Footing Stability Calculation
Notations and Formulae

Forces Diagram:

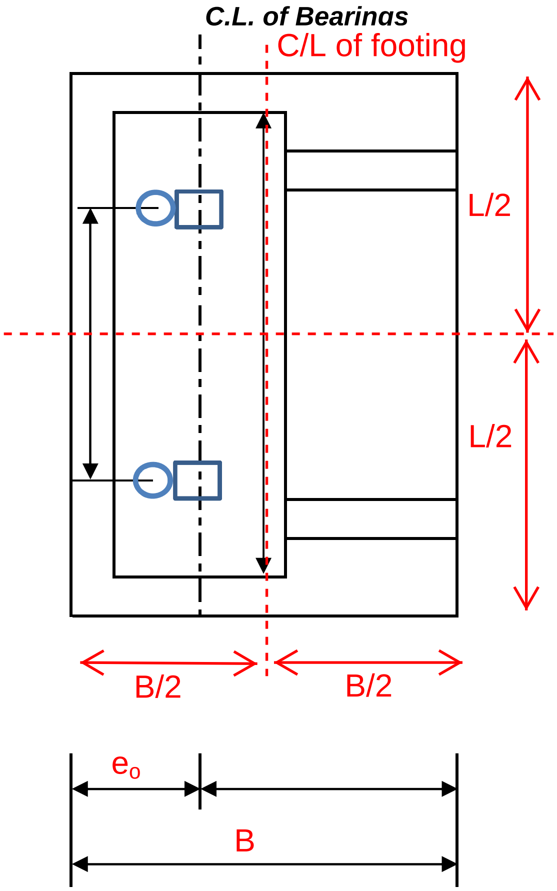


Forces

H
V
M
Lateral H
Lateral M

		ULS 1	ULS 2	ULS 3	ULS 4	ULS 9	SLS 1
Longitudinal Forces		383	455	446	462	385	357
Vertical Forces		4956	4922	4913	4573	4733	3894
Longitudinal Bending Moments	max	1327	1737	1684	1773	1755	1295
	min	758	348	401	312	1278	724
Lateral Forces		0	0	50	165	0	0
Lateral Bending Moments		0	0	285	941	0	0

Geometry



$x_2 = B/2 - e_o$

$M_t = M + V*x_2$

$e = M_t/V$

e_L = Lateral M/V

$B' = B - 2*|e|$

$L' = L - 2*|e_L|$

$q_{min} = V*(1-6*|e|/B-6*|e_L|/L)/(B*L)$

$q_{max} = V*(1+6*|e|/B+6*|e_L|/L)/(B*L)$

$q_{ULS} = V/(B'*L')$

S_u = Design undrained shear strength

Shape factor = 1+0.2*B'/L'

Depth factor = 1+0.2*(depth/B')

Inclination factor = $(1-\tan^{-1}((H^2+\text{Lateral } H^2)^{0.5}/V)/90)^2$

Net bearing resistance = $0.5*5.14*S_u$*Shape factor*Depth factor*Inclination factor

Gross bearing resistance=Net bearing resistance +depth*unit weight of soil

$R_h = 0.8*B'*L'*S_u$

FS bearing = Gross bearing resistance/q_{ULS}

FS sliding = $R_h/(H^2+\text{Lateral } H^2)^{0.5}$

TB-4 East

Abutment	B	L
East	4.60	7.00

Foundation Depth	0.0
U.Weight soil	21.0

Trail Bridge 4 - East   ULS   As-Proposed - 12 June 2014

	B	L	e_o	V	H	M	Lateral H	Lateral M	x_2 Distance from bearing C/L to footing C/L	M_t Moment about footing C/L	e Footing eccentricity from C/L	e_L Eccentricity along lateral	B'	L'	q_{min}	q_{max}	q_{ULS}	Su	Shape factor	Depth factor	Inclination factor	"NET" Bearing Resistance (0.5*(5.14'*Su))	Gross Bearing Resistance	Rh (0.8*B'*L*Su)	FS Bearing	FS Sliding
ULS1-max	4.60	7	2.35	5504	694	647	0	0	-0.05	371.8	0.07	0.00	4.46	7.00	156	186	176	80	1.127569	1	0.846676	196	196	2000	1.11	2.88
ULS 2-max	4.60	7	2.35	5470	766	1097	0	0	-0.05	823.5	0.15	0.00	4.30	7.00	137	203	182	80	1.122826	1	0.830697	192	192	1926	1.05	2.51
ULS3-max	4.60	7	2.35	5461	757	1038	50	313	-0.05	764.95	0.14	0.06	4.32	6.89	130	209	184	80	1.125479	1	0.831967	193	193	1904	1.05	2.51
ULS4-max	4.60	7	2.35	5121	772	1136	165	1032	-0.05	879.95	0.17	0.20	4.26	6.60	96	222	182	80	1.129039	1	0.814736	189	189	1797	1.04	2.28
ULS9-max	4.60	7	2.35	5313	698	1056	0	0	-0.05	790.35	0.15	0.00	4.30	7.00	133	197	176	80	1.122928	1	0.840595	194	194	1928	1.10	2.76
ULS1-min	4.60	7	2.35	5504	694	272	0	0	-0.05	-3.2	0.00	0.00	4.60	7.00	171	171	171	80	1.131395	1	0.846676	197	197	2060	1.15	2.97
ULS 2-min	4.60	7	2.35	5470	766	-178	0	0	-0.05	-451.5	-0.08	0.00	4.43	7.00	152	188	176	80	1.126712	1	0.830697	192	192	1987	1.09	2.59
ULS3-min	4.60	7	2.35	5461	757	-119	50	313	-0.05	-392.05	-0.07	0.06	4.46	6.89	145	194	178	80	1.129446	1	0.831967	193	193	1964	1.09	2.59
ULS4-min	4.60	7	2.35	5121	772	-217	165	1032	-0.05	-473.05	-0.09	0.20	4.42	6.60	112	206	176	80	1.133857	1	0.814736	190	190	1864	1.08	2.36
ULS9-min	4.60	7	2.35	5313	698	767	0	0	-0.05	501.35	0.09	0.00	4.41	7.00	145	185	172	80	1.126036	1	0.840595	195	195	1976	1.13	2.83

TB-4 West

Abutment	B	L
West	4.60	7.00

Foundation Depth	0.0
U.Weight soil	21.0

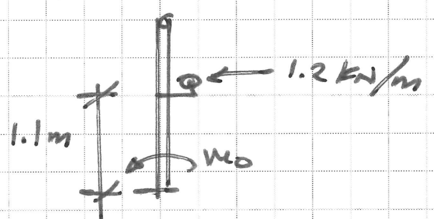
Trail Bridge 4 - West   ULS   As-Proposed- 12 June 2014

	B	L	e_o	V	H	M	Lateral H	Lateral M	x_2 Distance from bearing C/L to footing C/L	M_t Moment about footing C/L	e Footing eccentricity from C/L	e_L Eccentricity along lateral	B'	L'	q_{min}	q_{max}	q_{ULS}	Su	Shape factor	Depth factor	Inclination factor	0.5*(5.14'*Su) "NET" Bearing Resistance	Gross Bearing Resistance	0.8*B'*L*Su Rh	FS Bearing	FS Sliding
ULS1-max	4.60	7.00	2.35	5504	708	670	0	0	-0.05	394.8	0.07	0.00	4.46	7.00	155	187	176	80	1.12733	1	0.843746	196	196	1997	1.11	2.82
ULS 2-max	4.60	7.00	2.35	5470	780	1120	0	0	-0.05	846.5	0.15	0.00	4.29	7.00	136	204	182	80	1.122586	1	0.827788	191	191	1922	1.05	2.46
ULS3-max	4.60	7.00	2.35	5461	771	1061	50	313	-0.05	787.95	0.14	0.06	4.31	6.89	129	210	184	80	1.125234	1	0.829056	192	192	1900	1.04	2.46
ULS4-max	4.60	7.00	2.35	5121	786	1159	165	1032	-0.05	902.95	0.18	0.20	4.25	6.60	95	223	183	80	1.128767	1	0.811738	188	188	1793	1.03	2.23
ULS9-max	4.60	7.00	2.35	5313	712	1080	0	0	-0.05	814.35	0.15	0.00	4.29	7.00	132	198	177	80	1.12267	1	0.837575	193	193	1923	1.09	2.70
ULS1-min	4.60	7.00	2.35	5504	708	304	0	0	-0.05	28.8	0.01	0.00	4.59	7.00	170	172	171	80	1.13113	1	0.843746	196	196	2056	1.15	2.90
ULS 2-min	4.60	7.00	2.35	5470	780	-146	0	0	-0.05	-419.5	-0.08	0.00	4.45	7.00	153	187	176	80	1.127046	1	0.827788	192	192	1992	1.09	2.55
ULS3-min	4.60	7.00	2.35	5461	771	-87	50	313	-0.05	-360.05	-0.07	0.06	4.47	6.89	147	193	178	80	1.129786	1	0.829056	193	193	1969	1.08	2.55
ULS4-min	4.60	7.00	2.35	5121	786	-185	165	1032	-0.05	-441.05	-0.09	0.20	4.43	6.60	114	204	175	80	1.134236	1	0.811738	189	189	1869	1.08	2.33
ULS9-min	4.60	7.00	2.35	5313	712	801	0	0	-0.05	535.35	0.10	0.00	4.40	7.00	143	187	173	80	1.125671	1	0.837575	194	194	1971	1.12	2.77

TB-4: Miscellaneous (Railings/Luminaires)

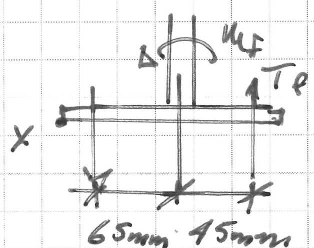
Hatch Mott MacDonald	CALCULATION SHEET			SHEET NO. 1 OF 1
DESCRIPTION: TRAIL BRIDGE RAILING		PROJECT NO: 285380	MADE BY: JL DATE: 7/16/14	CHECKED BY: DATE:

RAILING ANCHORAGE DESIGN



$M_o = 1.2\text{kN/m} \times 1\text{m} \times 1.1\text{m}$

$M_f = 1.32\text{kN.m} \times 1.5 = 1.98\text{ kN.m}$



About ℄

$T_f = 1.95\text{kN.m} / 0.045\text{m} = 44\text{ kN}$ (CONSERVATIVE)

About PT X

$T_f = 1.95\text{kNm} / 0.135\text{m} = 14.4\text{kN}$

TRY 12.7 DIA. HAS. SS. ANCHORS

YIELD STRENGTH = 66.3kN > 44 ok

TENSILE ULS = 79kN > TF ok.

CHECK HILTI HIT-RE 500

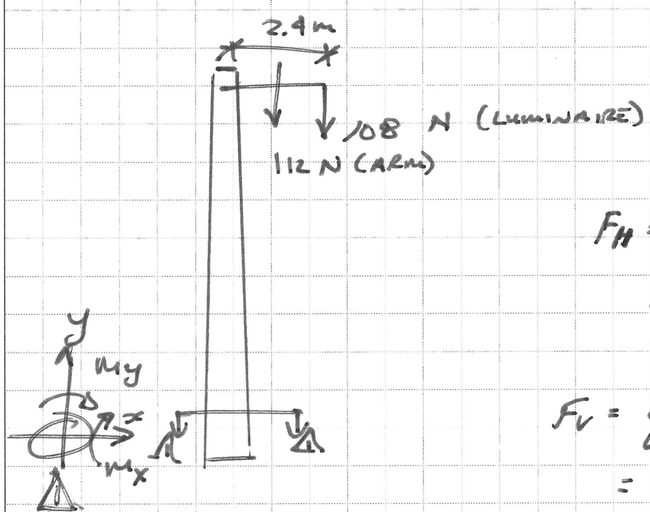
ULTIMATE TENSILE STRENGTH @ 152 EMBEDMENT = 95.7kN > TF ok.

Hatch Mott MacDonald

CALCULATION SHEET

SHEET NO. 1 OF 2

DESCRIPTION	PROJECT NO	MADE BY	CHECKED BY
LUMINAIRE POST FOOTING ON EPS	285380	JL DATE 7/16/14	DATE



TRAIL POLES, H = 6m

$F_H = q\,C_e C_g C_h$

$= 535\,pa \times 2.5 \times 1.1 \times 1.2$

$= 1765.5\,pa$

$F_V = q\,C_e C_g C_v$

$= 535\,pa \times 2.5 \times 1.1 \times 1.0 = 1471\,pa$

pole weight ≈ 6 kN

My (EPS DESIGN IS FOR 1% COMPACTION ∴ SLS)

Component weight

$M_{SW} = 108(2.4) + 112(1.2) = 0.394\ kN\cdot m$

WIND - HORIZONTAL

$M_w = 1.7655\ \frac{kN}{m^2} \times 1.35\ m^2 \times 2.3\ m = 5.50\ kN\cdot m$

$M_T = 5.5\ kN\cdot m + 0.394\ kN\cdot m$

$= 5.9\ kN\cdot m$

Mx

WIND ON COMPONENTS, ARM AREA = $0.15 m^2$, LUMINAIRE AREA = $0.22\ m^2$

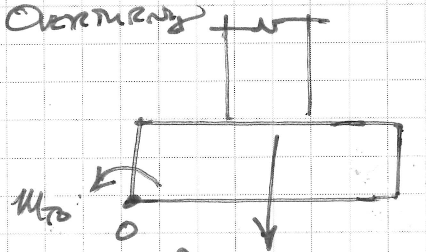
$M_{PW_C} = 1.7655\ kN/m^2 \times 0.37\ m^2 \times 6\ m = 4.0\ kN\cdot m$

$M_{T_0} = 5.5 + 4.0 = 9.5\ kN\cdot m$

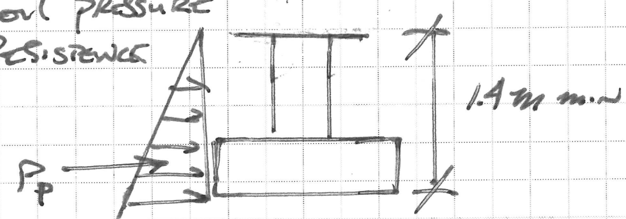
Hatch Mott MacDonald	CALCULATION SHEET	SHEET NO. 2 OF 2

DESCRIPTION	PROJECT NO	MADE BY	CHECKED BY
Luminaire Footing Check	285380	JL DATE 7/16/14	DATE

Overturning



Soil pressure resistence



$P_T = P_{conc} + P_{soil} + P_{pole}$

$K_p = \dfrac{1+\sin\phi}{1-\sin\phi} = 3$ use 50% of K_p

$P_{conc} = (1.5\times1.5\times0.4 + \pi\, 0.38^2\times1)\,24 = 32.5\,kN$

$P_{soil} = (1.5\times1.5\times1 - \pi\, 0.38^2\times1)\,21 = 37.7\,kN$

$P_{pole} = 6\,kN$

$P_T = 76.2\,kN$

$P_p = \gamma K_p h\, h/2 = 21\,\frac{kN}{m^3}\times1.5\,m\times1.4\times1.4\times0.5$

$= 30.9\,kN/m\times0.76\,m$

$= 23.8\,kN$

Overturning

$M_o = -76.2\,(0.75\,m) + 9.5\,kN\cdot m - 23.8\,(\tfrac{1}{3}(1.4)) = -58.7\,kN\cdot m$

ok.

Sliding

$$FS_{(sliding)} = \frac{(\Sigma V)\tan(k_1\phi_2') + \cancel{Bk_2 c_2'}^{\text{assume}=0} + P_p}{P_a\cos\alpha}$$

$= [76.2\,kN\ \tan(0.5(30)) + 23.8] / (1.35\,m^2 + 0.37\,m^2)\,1.7655\,\frac{kN}{m^2}$

$= 14.5 > 1.5$ ok.

Check EPS Pressure

$M_{TO} = 9.5\,kN\cdot m < M_{pp} = 11.1\,kN\cdot m$ ∴ pressure is from vertical load only

$\sigma = P/A = 76.2/(1.5\,m\times1.5\,m) = 34\,kN/m^2 < 50\,kN/m^2$ ok.