Structural Retaining Walls Memorandum

New Campus Development for The Ottawa Hospital
Phase 4: Main Hospital Project
Ottawa, Ontario

January 12th, 2026
(Issued for SPC Resubmission)

Prepared by WSP Canada Inc.

CA0027758.0-51

VERSION NUMBER AND DATE

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1.0 BACKGROUND INFORMATION

PCL/ED has retained WSP to provide Civil Engineering Services in support of development of the New Civic Campus for the Ottawa Hospital, located in Ottawa, Ontario, Canada. The New Civic Campus will be located on Carling Avenue, southwest of the intersection of Preston Street and Carling Avenue. The new Civic Campus is part of a multi-phased development that includes a parking garage, a new research facility, a future University of Ottawa Heart Institute, a new Main Hospital Building and a Central Utility Plant.

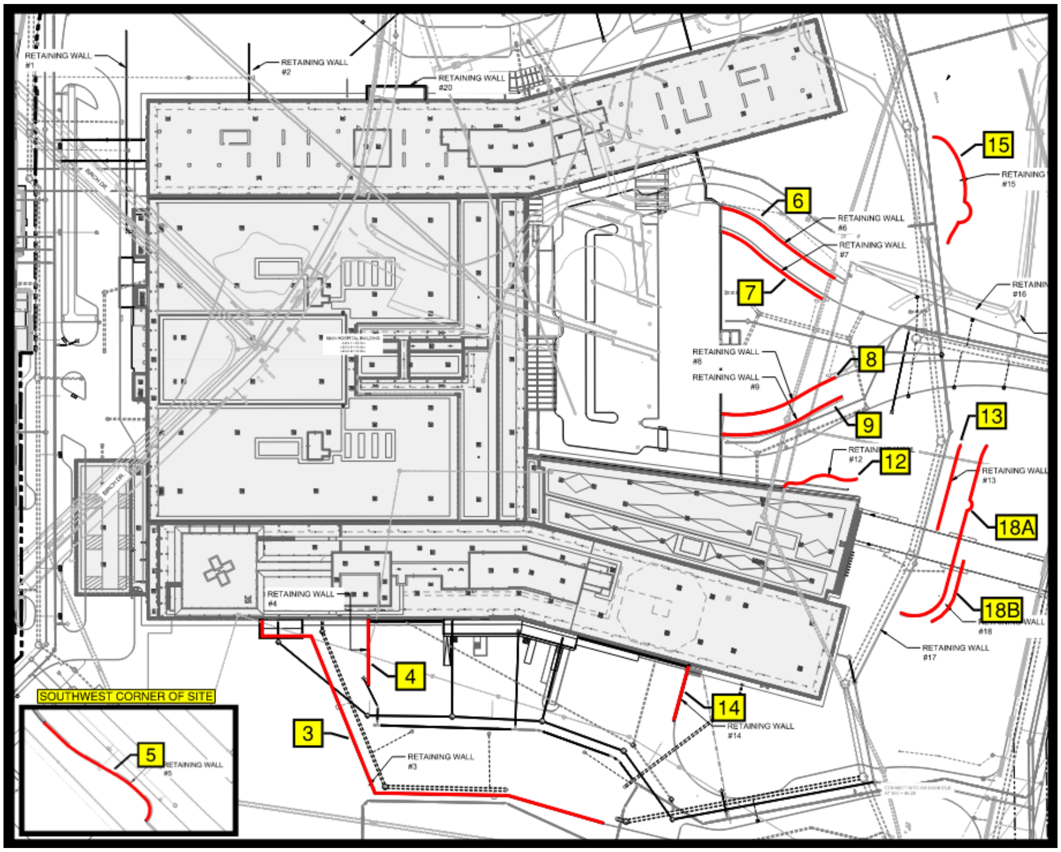
As part of WSP's engineering services WSP is responsible for the design of retaining walls throughout the project site. This memorandum provides a summary of the design concept, parameters and rationale that were employed in the design of the retaining walls. The actual details of the retaining walls are illustrated in the Site Plan Control Resubmission (SPC) drawings for reference only.

2.0 RETAINING WALL LAYOUT

The positions and lengths of the retaining walls in the Site Plan Control Resubmission Drawings generally reflect those of the Indicative Schematic Design (ISD); however, the ISD retaining walls have been impacted by the continued design development of the site and Advanced Work (AW). These impacts include the addition of new walls, coordination with other engineering disciplines, accommodation of utilities, some wall alignment changes, removal of some of the ISD walls, and changes in the wall types. All the changes from ISD as related to the design of the retaining walls are reflected on the 95% DD drawings and addressed in this Memorandum as applicable.

Each retaining wall has been numbered for reference; please note that the addition and deletion of retaining walls throughout the design development process, has created "gaps" in the identification numbers. Figure 2.1 below shows the walls found in the 95% DD drawings.

Figure 2.0-1: 95% DD Retaining Walls



3.0 WALL TYPES

There are generally two types of retaining walls in the 95% DD: pre-cast concrete block walls (proprietary modular block products) and cast-in-place (CIP) reinforced concrete walls. In general, pre-cast concrete block walls were selected for use in areas where walls are less visible to the public (ex. loading dock walls), while CIP walls were selected in highly visible public areas and in locations where a pre-cast block wall would not meet structural requirements (ex. where precast concrete block walls were technically unsuitable to accommodate conflicts with other structures and/or utilities). Table 3.1 below provides a schedule of the retaining wall locations and types.

Table 3.0-1: Retaining Wall Schedule

ID	LOCATION	TYPE
3	Loading dock	Pre-cast block
4	Loading dock	CIP
5	South Road E (wall extension)	Pre-cast block
6	Main entrance	CIP
7	Main entrance	CIP
8	Main entrance	CIP
9	Main entrance	CIP
12	Pavilion	CIP
13	Road A	CIP
14	Loading dock	CIP
15	Woodland path	Pre-cast block
18	Road A	Pre-cast block

4.0 WALL DESIGN

The site retaining walls have been designed as per the requirements of Schedule 15 of the Project Agreement (PA); the following information is provided to demonstrate how the design of the retaining walls meets the requirements of Schedule 15.

Structural

- All retaining walls are designed in accordance with the CSA S6-19 Canadian Highway Bridge Design Code (CHBDC).
- All retaining walls are designed for a minimum service life of 75 years.
- Cast-in-place retaining walls are designed to limit long-term (post construction and completion of backfill operation) lateral displacements of the top of the wall to lesser of the following:
  - 1 horizontal to 500 vertical, relative to bottom of the wall, or
  - 10mm
  - ***Note:*** Pre-cast block walls are inherently flexible structures by-design, due to tolerances in manufacturing and installation, reduced frost protection requirements and design-movement during seismic events. Therefore, the above requirement for long-term lateral displacement is not applicable to these walls. Small pre-cast block wall movements are generally imperceptible due to the batter in the wall face as well as the rough surface texture of the blocks.
- Design calculations for CIP walls are provided in Appendix A of this memorandum.
- Pre-cast block retaining walls are shown schematically in the 95% DD drawings; detailed design of the pre-cast block walls, and final signed and sealed shop drawings, will be provided by the manufacturer of the pre-cast block wall units.
- All new concrete for CIP walls will be Class C-1 exposure as per CSA requirements with target strength of 35 MPa at 56 days. Higher strength concrete is normally utilized for precast concrete modular block walls.
- The drainage of the walls will be primarily buried in field perforated pipe drains discharging into storm sewer installation. The back of the CIP retaining walls will be waterproofed with waterproofing dimple board channeling any water in backfill to the subdrain pipes. No drainage holes are utilized in the CIP walls.
- The walls are designed with appropriate articulation with use of expansion and control/construction joints to accommodate seasonal thermal changes.
- The frost protection for cast in place retaining walls is provided either by sufficient soil frost cover or where, or where full frost cover is not possible, high-grade insulation is specified.

Conflict With Adjacent Properties

- The retaining walls are located near the main hospital structure and are not in proximity to adjacent properties lines. Wall #5, which is a short extension of an Advanced Works (AW) wall, is the closest wall to an adjacent property, however, this is a relatively short wall with no tiebacks or deep excavations which would encroach onto adjacent property.

Traffic Barriers and Railings

- Where required due to proximity to live traffic, rigid traffic barriers are provided on top of the retaining walls to provide appropriate performance level protection as per the CHBDC. Walls with traffic barriers are designed to resist collision loads as per CHDBC.
- Hand railings or fences will be installed to the top of all walls which do not have a traffic barrier; the minimum fence height is 1800mm and the minimum railing height is 1070mm above the top of the retaining wall. The retaining walls are designed to resist all loads from fences and railings. Gaps in the fencing and safety railings will be 100 mm or less.

Wall Finishes

- Architectural finishes of the pre-cast block retaining walls and CIP retaining walls will be confirmed during the design development and will require approval from The Ottawa Hospital (TOH).
- TOH will approve all aesthetic wall features; aesthetic considerations shall not negatively affect the maintenance or reparability of the walls.
- An anti-graffiti coating will be applied to all site retaining walls.

Construction

The Schedule 15 requirements concerning construction of the site retaining walls (by others) is quoted below for reference. The design of the walls was carried out so that the Schedule 15 requirements could be fully met for constructability.

- Structural stability of walls and excavations will be maintained throughout construction
- Dewatering provisions will be provided to control groundwater during all phases of construction
- Temporary Shoring Movement Tolerances: To reduce construction costs, PCL-ED has proposed to relax the lateral shoring movement tolerances for any shoring that is not directly adjacent to a foundation wall element. This proposal was accepted by the CA and is documented in RFI-00305.

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APPENDIX A: CIP WALL DESIGN CALCULATIONS

1. Design Parameters:

soil =	0.7	
ULS soil bearing capacity (kPa) =	250	
ULS soil bearing capacity (kPa)	200	
Unit weight of soil (kN/m3) =	22	
Unit weight of exisitng soil (kN/m3) =	20	
Unit weight of concrete (kN/m3) =	24	
Granular A friction angle (°) =	38	
Lateral earth pressure at rest coefficient =	0.38	
Height of retaining wall at grade, H1 (m) =	1.65	

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m³. The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m³, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.
Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented below.

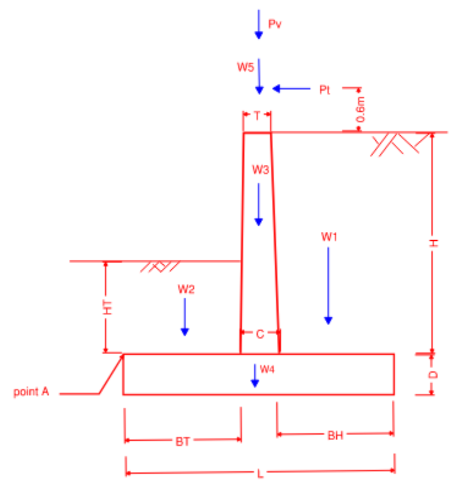
Table 6.9-2 – Geotechnical parameters for backfill material

Material Description	Unit Weight (kN/m³)		Friction Angle (°)	Friction Factor, tan	Lateral Earth Pressure Coefficients		
	Drained γ_{dr}	Effective $\dot{\gamma}$	ϕ	δ	Active K_a	At Rest K_o	Passive K_P
Granular A (Crushed Stone)	22	13.5	38	0.6	0.24	0.38	4.20
Granular B Type II (Crushed Stone)	22	13.5	40	0.6	0.22	0.36	4.60

Notes:
The properties of backfill materials are for a condition of 98% of the materials SPMDD.
Earth pressure coefficients provided are for the horizontal backfill profile.
For soil above the water table, the "drained" unit weight must be used and below the water table, the "effective" unit weight must be used.

2. Dimension Estimates:

Retaining wall height, H (m) =	1.65	
Base width, L (m) =	3.2	L=0.5H to 2/3H
Thickness of base, D (m) =	0.6	D=0.1H
Stem thickness at the bottom, C (m) =	0.6	C=0.1H
Stem thickness at the top, T (m) =	0.46	T=0.25m min.
Width of the toe, B_T (m) =	0.5	B=0.25L to 0.33L
Parapet height (m) =	0.8	
Parapet width (m) =	0.3	

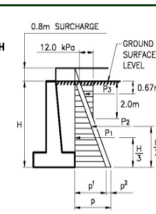


3. Stability Check:

Soil height at toe, H_T (m) =	0.45	
Width of the toe, B_T (m) =	0.5	
Soil height at heel, H_H (m) =	1.65	
Width of the heel, B_H (m) =	2.1	
Weight due to soil, W1 (kN) =	78.77	
Weight due to soil, W2 (kN) =	0.00	4.95
Weight due to stem, W3 (kN) =	20.99	
Weight due to base, W4 (kN) =	46.08	
Weight due to barrier, W5 (kN) =	6.16	
Total (kN) =	152.00	

ACTIVE EARTH PRESSURE WITH 800 mm SURCHARGE

q = EQUIVALENT FLUID PRESSURE (kPa)
p = UNIT PRESSURE AT DEPTH H METRES
p^1 = UNIT PRESS. DUE TO BACKFILL AT DEPTH H
p^2 = UNIT PRESSURE DUE TO 0.8m SURCHARGE
P = TOTAL FORCE PER LINEAR METRE OF WALL
M = OVERTURNING MOMENT (kN–m)



AT SERVICEABILITY LIMIT STATES

$\alpha_E = 1.0$
$q = 7.0 \times (H)$

AT ULTIMATE LIMIT STATES

$\alpha_E = 1.25$
$q = 1.25 \times (7.0) \times (H)$

$\phi = 30°$
$P = P_1 + P_2 + P_3$
$p = p^1 + p^2$

– RESULTS INCLUDE THE EFFECT OF COMPACTION SURCHARGE

SLS				ULS			
H (m)	p (kPa)	P (kN)	M (kN–m)	H (m)	p (kPa)	P (kN)	M (kN–m)
1.0	13	18	9	1.0	16	23	11
1.5	16	28	20	1.5	20	35	25
2.0	20	37	37	2.0	25	46	46
2.5	23	48	58	2.5	29	60	73
3.0	27	60	85	3.0	34	75	106
3.5	30	75	118	3.5	38	94	148
4.0	34	90	159	4.0	43	113	199
4.5	37	108	209	4.5	46	135	261
5.0	41	128	268	5.0	51	160	335
5.5	44	149	337	5.5	55	186	421
6.0	48	172	417	6.0	60	215	521

THE METHOD OF EQUIVALENT FLUID PRESSURES IS LIMITED TO A MAXIMUM HEIGHT OF 6.0m. FOR RETAINING WALLS WITH HEIGHTS > 6.0m, THE EARTH PRESSURE DISTRIBUTION SHALL BE ESTABLISHED BY A GEOTECHNICAL ENGINEER.

3.1 Check for Overturning Moment:

earth pressure table =	1.19	
live load factor =	1.7	
dead load factor-earth fill =	0.8	CHBDC Table 3.3
dead load factor-concrete =	0.9	CHBDC Table 3.3
TL-4 load transverse load, Pt (kN) =	100	
stem (m) =	5.75	
due to TL-4 (kNm)	69.48	
due to soil pressure, surcharge, compact =	37.38	obtained from Design Aid 2-11
ULS total, Mo (kNm) =	106.86	
Mr (kNm) =	219.68	0.8 x (W1 x arm + W2 x arm) + 0.9 x ((W3+W5) x arm + W4 x arm)
pressure on the toe side, Mp (kNm) =	0.00	
Totoal ULS overturning resistance, M_{tr}	219.68	Mr + Mp
M_{tr}/M_o =	2.1	resistance

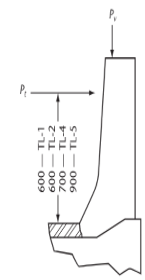
Table 3.7
Loads on traffic barriers
(See Clauses 3.8.8.1 and 12.4.3.2.5.)

Performance level	Transverse load, kN	Longitudinal load, kN	Vertical load, kN
TL-1	25	10	10
TL-2	50	20	10
TL-4	100	30	30
TL-5	210	70	90

Notes:
(1) *Traffic barrier types are illustrative only and other types may be used.*
(2) *Transverse load P_t shall be applied over a barrier length of 1200 mm for TL-1 and TL-2 barriers, 1050 mm for TL-4 barriers, and 2400 mm for TL-5 barriers.*
(3) *Longitudinal load P_ℓ shall be applied at the same locations and over the same barrier lengths as P_t. For post and railing barriers, the longitudinal load shall not be distributed to more than three posts.*
(4) *Vertical load P_v shall be applied over a barrier length of 5500 mm for TL-1, TL-2, and TL-4 barriers and 12 000 mm for TL-5 barriers.*
(5) *These loads shall be used for the design of traffic barrier anchorages and decks only.*

Earth pressure and hydrostatic pressure	Maximum α_E	Minimum α_E
Passive earth pressure, considered as a load*	1.25	0.50
At-rest earth pressure	1.25	0.80
Active earth pressure	1.25	0.80
Backfill pressure	1.25	0.80
Hydrostatic pressure	1.10	0.90

Retaining systems	Bearing, ϕ_{gu}	Analysis	0.45	0.50	0.60
	Overturning, ϕ_{gu}	Analysis	0.45	0.50	0.55
	Base sliding, ϕ_{gu}	Analysis	0.70	0.80	0.90
	Facing interface sliding, ϕ_{gu}	Test	0.75	0.85	0.95
	Connections, ϕ_{gu}	Test	0.65	0.70	0.75
	Settlement, ϕ_{gs}	Analysis	0.7	0.8	0.9
	Deflection/tilt, ϕ_{gs}	Analysis	0.7	0.8	0.9



3.2 Check for Sliding:

The passive soil pressure is neglected in the sliding check.

ULS sliding force (kN) =	45.74	obtained from Design Aid 2-11
ULS TL-4 transverse force, (kN) =	29.57	
Total, Fs (kN) =	75.31	
ULS sliding resistance, Fr (kN) =	109.15	total weight x friction coef.
Fr/Fs =	1.45	must be greater than 1.25, CHBDC Table 6.2

4. Check for Bearing Pressure under Footing:

Base area, A (m2) =	3.20	
Area property, S (m3) =	1.71	neglecting eccentric axial
ULS Tl-4 load, Pv (kN) =	9.27	
ULS total vertical load, P (kN) =	201.80	
moment at bottom of the stem, M (kNm) =	37.38	Mo
Reaction at toe (kPa) =	84.96	P/A+M/S
Reaction at heel (kPa) =	41.16	P/A-M/S
ULS soil bearing capacity (kPa) =	250.00	
Net max. reaction (kPa) =	39.96	
Ratio of Demand/Capacity =	0.16	soil beairng capacity is Okay

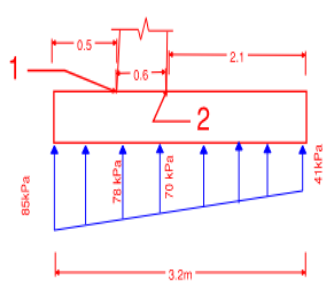
5. Stem Design:

ULS design loads bottom of the stem

M (kNm) =	106.86	D/R =	0.71
V (kN) =	75.31	D/R =	0.38

15M@300 on all faces

6. Base Design:



shear at section 1, V1 (kN) =	40.75	D/R =	0.21
shear at section 2, V2 (kN) =	116.55	D/R =	0.59

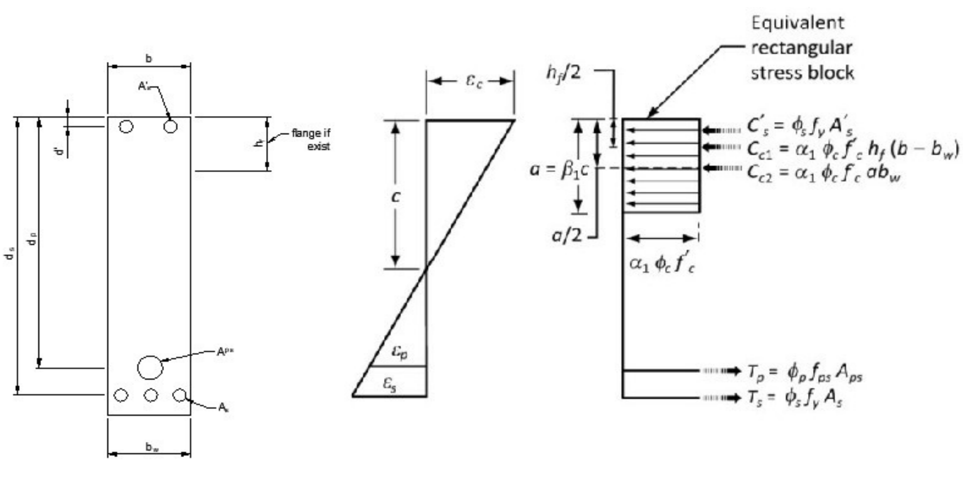
15M@300 at all faces

Wall flexual capacity calculation - at bottom of wall stem

Symbols	Data	Unit	Notes
b	1000.00	mm	Total width of flange (including web)
b_w	1000.00	mm	Web width, when neutral axil is located in flange, bw should be taken as b (C8.8.4.1)
h_f	0.00	mm	thickness of flange
f_c'	30.00	Mpa	Concrete strength
α_1	0.81		α1=0.85-0.0015*fc' ≥ 0.67 (CHBDC 8.8.3)
β_1	0.90		β1=0.97-0.0025*fc' ≥ 0.67 (CHBDC 8.8.3)
c	19.25	mm	c = (Ts-Cc1)/α1*β1*ϕc*fc'*bw , Distance from extrem compression fibre to neutral axis, α1*ϕc*fc' is concrete stress uniformly distributed over an equivalent compression zone
a	17.23	mm	a = β1*c , Equivalent rectangular compression zone height
ϕ_c	0.75		Resistance factor for concrete
C_{c1}	0.00	N	Cc1 = α1*ϕc*fc'*hf*(b-bw) Amount of compression in flange (C8.8.3)
C_{c2}	312000.00	N	Cc2 = α1*ϕc*fc'*a*bw Amount of compression in web (C8.8.3)
A_s	867	mm^2	Area of rebars on the flexural tension side
f_y	400.00	Mpa	Yield strength of tensile rebar
ϕ_s	0.90		Resistance factor of reinforcing steel
T_s	312000.00	N	Ts = ϕs*fy*As Amount of tension in reinforcing steel
h	600.00	mm	Overall height of beam
d_s	492.00	mm	Distance from centroid of rebars to extrem compression fibre of concrete beam
M_r	**150.82**	kN-m	Moment resistance, Mr = Ts*(ds-a/2) - Cc1*(hf/2-a/2) (C8.8.4.1)

Limit for Min. & Max. Reinforcement Ratios

M_r(min.)	157744097	N-mm	Min. Reinforcement: Factored Mr is at least 1.2*Mcr. Mcr = fcr*I/y (CHBDC 8.8.4.3)
I	18000000000	mm^4	Moment of inertia
y	300.00	mm	Distance to netural axis
f_{cr}	2.19	Mpa	Cracking strengtch for normal-density concrete 0.4*√fc'
M_{cr}	131453413.80	N-mm	Cracking moment
c/d (max.)	0.04		Max. Reinforcement: c/d not exceeding 0.5, (CHBDC 8.8.4.5)



Strain **Stress block**

PROJECT	The Ottawa Hospital New Campus Development - Phase 4 Retaining Wall Calculation	SHEET	
PROJECT No.	CA0027758.0-51	DATE	08-Jan-26
SUBJECT	Retaining Wall #4 - shear capacity calculation	DESIGNER	Hui Liu
		CHECKER	Felix Wasiewicz

Wall stem shear capacity calculation - section at bottom of wall

	Rmark	Symbols	Data	Unit	Notes
Section		h	600	mm	Overall thickness
		d	492	mm	Effective depth (distance from extrem compression fibre to centroid of tensile force)
		d_v	442.8	mm	0.72h - taken as the greater; 0.9d - taken as the greater
		b_v	600	mm	Effective web width, taken as minimum web witdth whin the depth dv
Vc		ϕ_c	0.75		Resistance factor for concrete
		f_c'	30	Mpa	Concrete strength
		f_{cr}	2.19089	Mpa	Not be greater than 3.2MPa; 0.4√fc' - nomal-density concrete
		β	0.18		1) β = 0.18 -with tansverse reinforcement; 2)β = 0.18 -without tansverse reinforcement, <3dv; 3) β = 230/(1000+dv) without transverse reinforcement, but having max. size of coarse aggregate not less than 20mm
		V_c	**196450.6**	N	Vc = 2.5*β*ϕc*fcr*bv*dv, (CHBDC 8.9.3.4)
Vs		ϴ	42	°	ϴ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.733038	rad	Convert Dgrees to Radians
		ϕ_s	0.9		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotϴ	1.110613		cosϴ/sinϴ
		s	100	mm	Spacing of stirups
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*cotϴ/s (CHBDC 8.9.3.5)
Vs - with inclination		ϴ	42	°	ϴ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.733038	rad	Convert Dgrees to Radians
		ϕ_s	0.95		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotϴ	1.110613		cosϴ/sinϴ
		s	200	mm	Spacing of stirups
		α	45	°	Transverse reinforcement inclined at an angle to the longitudinal axis
		Radians	0.785398	rad	Convert Degrees to Radians
		cotα	1		cosα/sinα
		sinα	0.707107		
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*(cotϴ + cotα)*sinα/s Trans verse reinforcement inclined at an angle to the longitudinal axis, and in the direction that will intersec diagonal cracks caused by the shear (CHBDC 8.9.3.5)
Vr	Automati	Limit of $(V_c + V_s)$	1494450	N	Vc + Vs shall not exceed 0.25*ϕc*fc'*bv*dv
	Φ_p = 0.95	V_p	0	N	Component in the direction of the applied shear of all of the effective prestressing forces crossing the critical section factored by ϕp (taken as positive if resisting the applied shear)
	Total	**V_r**	**196.45**	kN	Vr = Vc +Vs+Vp

Min. Av, Vf limit requiring transverse reinforcement

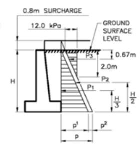
		V (limit)	87311	N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25 Tcr (CHBDC 8.9.1.2)
		$0.25T_{cr}$		N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25*Tcr , Tcr = 0.8 * φc*fcr* (Acp2/Pc)*[1+fce/(0.8* φc *fcr)]0.5 (CHBDC 8.9.1.1)
		A_v(min.)	49	mm^2	Min. amount of transverse reinforcement: Av is not less than 0.15*fcr(bv*s/fy)

1. Design Parameters:

Friction coefficient between conc. and soil =	0.7	
ULS Soil bearing capacity (kPa) =	250	
ULS soil bearing capacity (kPa) =	200	
Unit weight of soil (kN/m3) =	22	
Unit weight of existing soil (kN/m3) =	20	
Unit weight of concrete (kN/m3) =	24	
Height of retaining wall, H (m) =	3.7	
At rest lateral soil pressure coefficient, Ko =	0.38	
Lateral soil pressure modification factor =	1.19	

DESIGN AID 2-11
ACTIVE EARTH PRESSURE WITH 800 mm SURCHARGE

q = EQUIVALENT FLUID PRESSURE (kPa)
p = UNIT PRESSURE AT DEPTH H METRES
p^1 = UNIT PRESS. DUE TO BACKFILL AT DEPTH H
p^2 = UNIT PRESSURE DUE TO 0.8m SURCHARGE
P = TOTAL FORCE PER LINEAR METRE OF WALL
M = OVERTURNING MOMENT (kN–m)



AT SERVICEABILITY LIMIT STATES

$\alpha_E = 1.0$
$q = 7.0 \times (H)$

AT ULTIMATE LIMIT STATES

$\alpha_E = 1.25$
$q = 1.25 \times (7.0) \times (H)$

$\phi = 30°$
$P = P_1 + P_2 + P_3$
$p = p^1 + p^2$

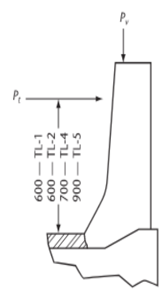
– RESULTS INCLUDE THE EFFECT OF COMPACTION SURCHARGE

SLS				ULS			
H (m)	P (kPa)	P (kN)	M (kN–m)	H (m)	P (kPa)	P (kN)	M (kN–m)
1.0	13	18	9	1.0	16	23	11
1.5	16	28	20	1.5	20	35	25
2.0	20	37	37	2.0	25	46	46
2.5	23	48	58	2.5	29	60	73
3.0	27	60	85	3.0	34	75	106
3.5	30	75	118	3.5	38	94	148
4.0	34	90	159	4.0	43	113	199
4.5	37	108	209	4.5	46	135	261
5.0	41	128	268	5.0	51	160	335
5.5	44	149	337	5.5	55	186	421
6.0	48	172	417	6.0	60	215	521

THE METHOD OF EQUIVALENT FLUID PRESSURES IS LIMITED TO A MAXIMUM HEIGHT OF 6.0m. FOR RETAINING WALLS WITH HEIGHTS > 6.0m, THE EARTH PRESSURE DISTRIBUTION SHALL BE ESTABLISHED BY A GEOTECHNICAL ENGINEER.

2. Dimension Estimates:

Retaining wall height, H (m) =	3.7	
Base width, L (m) =	3	L=0.5H to 2/3H
Thickness of base, D (m) =	0.7	D=0.1H
Stem thickness at the bottom, C (m) =	0.7	C=0.1H
Stem thickness at the top, T (m) =	0.45	T=0.25m min.
Width of the toe, B_T (m) =	0.8	B=0.25L to 0.33L
Parapet height (m) =	0.8	
Parapet width (m) =	0.3	



Soil height at toe, H_T (m) =	0
Soil height at heel, H_H (m) =	3.7

Table 3.7
Loads on traffic barriers
(See Clauses 3.8.8.1 and 12.4.3.2.5.)

Performance level	Transverse load, kN	Longitudinal load, kN	Vertical load, kN
TL-1	25	10	10
TL-2	50	20	10
TL-4	100	30	30
TL-5	210	70	90

Notes:
(1) *Traffic barrier types are illustrative only and other types may be used.*
(2) *Transverse load P_t shall be applied over a barrier length of 1200 mm for TL-1 and TL-2 barriers, 1050 mm for TL-4 barriers, and 2400 mm for TL-5 barriers.*
(3) *Longitudinal load P_l shall be applied at the same locations and over the same barrier lengths as P_t. For post and railing barriers, the longitudinal load shall not be distributed to more than three posts.*
(4) *Vertical load P_v shall be applied over a barrier length of 5500 mm for TL-1, TL-2, and TL-4 barriers and 12 000 mm for TL-5 barriers.*
(5) *These loads shall be used for the design of traffic barrier anchorages and decks only.*

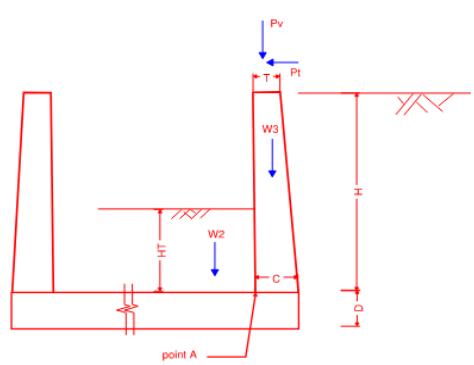
Table 6.9-2 – Geotechnical parameters for backfill material

Material Description	Unit Weight (kN/m³) Drained γ_{dr}	Unit Weight (kN/m³) Effective γ'	Friction Angle (°) ϕ	Friction Factor, tan δ	Lateral Earth Pressure Coefficients Active K_a	At Rest K_o	Passive K_p
Granular A (Crushed Stone)	22	13.5	38	0.6	0.24	0.38	4.20
Granular B Type II (Crushed Stone)	22	13.5	40	0.6	0.22	0.36	4.60

Notes:
The properties of backfill materials are for a condition of 98% of the materials SPMDD.
Earth pressure coefficients provided are for the horizontal backfill profile.
For soil above the water table, the "drained" unit weight must be used and below the water table, the "effective" unit weight must be used.

3.1 Moment at Bottom of Stem :

live load factor =	1.7	
dead load factor-earth fill =	0.8	CHBDC Table 3.3
min. dead load factor-concrete =	0.9	CHBDC Table 3.3
max. dead load factor-concrete =	1.2	
TL-4 load transverse load, Pt (kN) =	100	
stem (m) =	9.85	
due to TL-4 (kNm) =	75.94	
due to soil pressure, surcharge, compact =	200.64	obtained from Design Aid 2-11
ULS total, Mo (kNm) =	276.58	



3.2 Shear at Bottom of Stem:

The passive soil pressure is neglected in the sliding check.

ULS sliding force (kN) =	121.82	obtained from Design Aid 2-11		
ULS TL-4 transverse force, (kN) =	17.26			
Total, Fs (kN) =	139.08			

5. Stem Design:

ULS design loads bottom of the stem

M (kNm) =	276.58	D/R =	0.83
V (kN) =	139.08	D/R =	0.56

15M @ 150

70mm cover

6. Base Design:

Parapet weight (kN/m) =	5.76		
Stem weight (kN/m) =	51.06		
Vertical collision force, Pv (kN/m) =	1.82		
ULS total (kN/m) =	56.82		
Shear at base (kN/m) =	56.82	D/R =	0.23
Positive moment at edge (kNm) =	276.58	D/R =	0.87
Negative mement at mid-span (kNm) =	104.4	D/R =	0.57

15M @ 300 at positive moment area

15M @ 150 at negative moment area

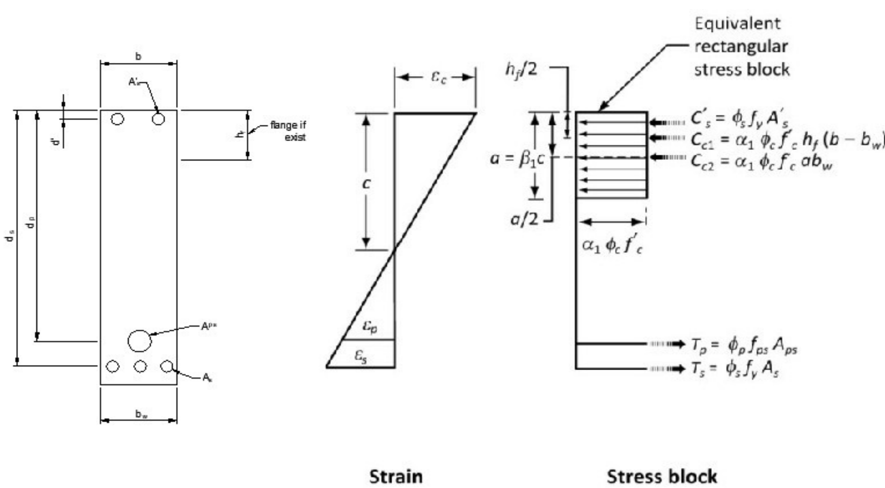
100mm cover

Wall flexual capacity calculation - at bottom of wall stem

Symbols	Data	Unit	Notes
b	1000.00	mm	Total width of flange (including web)
b_w	1000.00	mm	Web width, when neutral axil is located in flange, bw should be taken as b (C8.8.4.1)
h_f	0.00	mm	thickness of flange
f_c'	30.00	Mpa	Concrete strength
α_1	0.81		α1=0.85-0.0015*fc' ≥ 0.67 (CHBDC 8.8.3)
β_1	0.90		β1=0.97-0.0025*fc' ≥ 0.67 (CHBDC 8.8.3)
c	34.05	mm	c = (Ts-Cc1)/α1*β1*ϕc*fc'*bw , Distance from extrem compression fibre to neutral axis, α1*ϕc*fc' is concrete stress uniformly distributed over an equivalent compression zone
a	30.48	mm	a = β1*c , Equivalent rectangular compression zone height
ϕ_c	0.75		Resistance factor for concrete
C_{c1}	0.00	N	Cc1 = α1*ϕc*fc'*hf*(b-bw) Amount of compression in flange (C8.8.3)
C_{c2}	552000.00	N	Cc2 = α1*ϕc*fc'*a*bw Amount of compression in web (C8.8.3)
A_s	1533	mm^2	Area of rebars on the flexural tension side
f_y	400.00	Mpa	Yield strength of tensile rebar
ϕ_s	0.90		Resistance factor of reinforcing steel
T_s	552000.00	N	Ts = ϕs*fy*As Amount of tension in reinforcing steel
h	700.00	mm	Overall height of beam
d_s	620.00	mm	Distance from centroid of rebars to extrem compression fibre of concrete beam
M_r	**333.83**	kN-m	Moment resistance, Mr = Ts*(ds-a/2) - Cc1*(hf/2-a/2) (C8.8.4.1)

Limit for Min. & Max. Reinforcement Ratios

M_r(min.)	113	kN-m	Min. Reinforcement: Factored Mr is at least 1.2*Mcr. Mcr = fcr*I/y (CHBDC 8.8.4.3)
I	28583333333	mm^4	Moment of inertia
y	665.95	mm	Distance to netural axis
f_{cr}	2.19	Mpa	Cracking strengtch for normal-density concrete 0.4*√fc'
M_{cr}	94035734.05	N-mm	Cracking moment
c/d (max.)	0.05		Max. Reinforcement: c/d not exceeding 0.5, (CHBDC 8.8.4.5)



Wall stem shear capacity calculation - section at bottom of wall

	Rmark	Symbols	Data	Unit	Notes
Section		h	700	mm	Overall thickness
		d	622	mm	Effective depth (distance from extrem compression fibre to centroid of tensile force)
		d_v	559.8	mm	0.72h - taken as the greater; 0.9d - taken as the greater
		b_v	600	mm	Effective web width, taken as minimum web witdth whin the depth dv
Vc		ϕ_c	0.75		Resistance factor for concrete
		f_c'	30	Mpa	Concrete strength
		f_{cr}	2.19089	Mpa	Not be greater than 3.2MPa; 0.4√fc' - nomal-density concrete
		β	0.18		1) β = 0.18 -with tansverse reinforcement; 2)β = 0.18 -without tansverse reinforcement, <3dv; 3) β = 230/(1000+dv) without transverse reinforcement, but having max. size of coarse aggregate not less than 20mm
		V_c	**248358.2**	N	Vc = 2.5*β*ϕc*fcr*bv*dv, (CHBDC 8.9.3.4)
Vs		Θ	42	°	Θ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.733038	rad	Convert Dgrees to Radians
		ϕ_s	0.9		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotΘ	1.110613		cosΘ/sinΘ
		s	100	mm	Spacing of stirups
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*cotΘ/s (CHBDC 8.9.3.5)
Vs - with inclination		Θ	42	°	Θ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.733038	rad	Convert Dgrees to Radians
		ϕ_s	0.95		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotΘ	1.110613		cosΘ/sinΘ
		s	200	mm	Spacing of stirups
		α	45	°	Transverse reinforcement inclined at an angle to the longitudinal axis
		Radians	0.785398	rad	Convert Degrees to Radians
		cotα	1		cosα/sinα
		sinα	0.707107		
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*(cotΘ + cotα)*sinα/s Trans verse reinforcement inclined at an angle to the longitudinal axis, and in the direction that will intersec diagonal cracks caused by the shear (CHBDC 8.9.3.5)
Vr	Automati	Limit of (V_c + V_s)	1889325	N	Vc + Vs shall not exceed 0.25*ϕc*fc'*bv*dv
	Φ_p = 0.95	V_p	0	N	Component in the direction of the applied shear of all of the effective prestressing forces crossing the critical section factored by ϕp (taken as positive if resisting the applied shear)
	Total	**V_r**	**248.36**	kN	Vr = Vc +Vs+Vp

Min. Av, Vf limit requiring transverse reinforcement

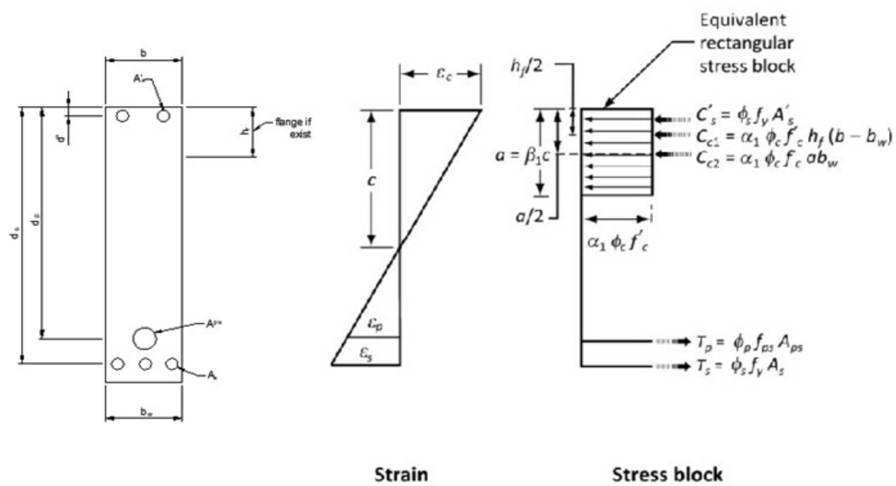
		V (limit)	110381	N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25 Tcr (CHBDC 8.9.1.2)
		$0.25T_{cr}$		N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25*Tcr , Tcr = 0.8 * φc*fcr* (Acp2/Pc)*[1+fce/(0.8* φc *fcr)]0.5 (CHBDC 8.9.1.1)
		A_v(min.)	49	mm^2	Min. amount of transverse reinforcement: Av is not less than 0.15*fcr(bv*s/fy)

Footing slab flexual capacity calculation

	Symbols	Data	Unit	Notes
	b	1000.00	mm	Total width of flange (including web)
	b_w	1000.00	mm	Web width, when neutral axil is located in flange, bw should be taken as b (C8.8.4.1)
	h_f	0.00	mm	thickness of flange
	f_c'	30.00	Mpa	Concrete strength
	α_1	0.81		α1=0.85-0.0015*fc' ≥ 0.67 (CHBDC 8.8.3)
	β_1	0.90		β1=0.97-0.0025*fc' ≥ 0.67 (CHBDC 8.8.3)
	c	34.05	mm	c = (Ts-Cc1)/α1*β1*ϕc*fc'*bw , Distance from extrem compression fibre to neutral axis, α1*ϕc*fc' is concrete stress uniformly distributed over an equivalent compression zone
	a	30.48	mm	a = β1*c , Equivalent rectangular compression zone height
	ϕ_c	0.75		Resistance factor for concrete
	C_{c1}	0.00	N	Cc1 = α1*ϕc*fc'*hf*(b-bw) Amount of compression in flange (C8.8.3)
	C_{c2}	552000.00	N	Cc2 = α1*ϕc*fc'*a*bw Amount of compression in web (C8.8.3)
	A_s	1533	mm^2	Area of rebars on the flexural tension side
	f_y	400.00	Mpa	Yield strength of tensile rebar
	ϕ_s	0.90		Resistance factor of reinforcing steel
	T_s	552000.00	N	Ts = ϕs*fy*As Amount of tension in reinforcing steel
	h	700.00	mm	Overall height of beam
	d_s	592.00	mm	Distance from centroid of rebars to extrem compression fibre of concrete beam
	M_r	**318.37**	kN-m	Moment resistance, Mr = Ts*(ds-a/2) - Cc1*(hf/2-a/2) (C8.8.4.1)

Limit for Min. & Max. Reinforcement Ratios

	M_r(min.)	113	kN-m	Min. Reinforcement: Factored Mr is at least 1.2*Mcr. Mcr = fcr*I/y (CHBDC 8.8.4.3)
	I	28583333333	mm^4	Moment of inertia
	y	665.95	mm	Distance to netural axis
	f_{cr}	2.19	Mpa	Cracking stregntch for normal-density concrete 0.4*√fc'
	M_{cr}	94035734.05	N-mm	Cracking moment
	c/d (max.)	0.06		Max. Reinforcement: c/d not exceeding 0.5, (CHBDC 8.8.4.5)

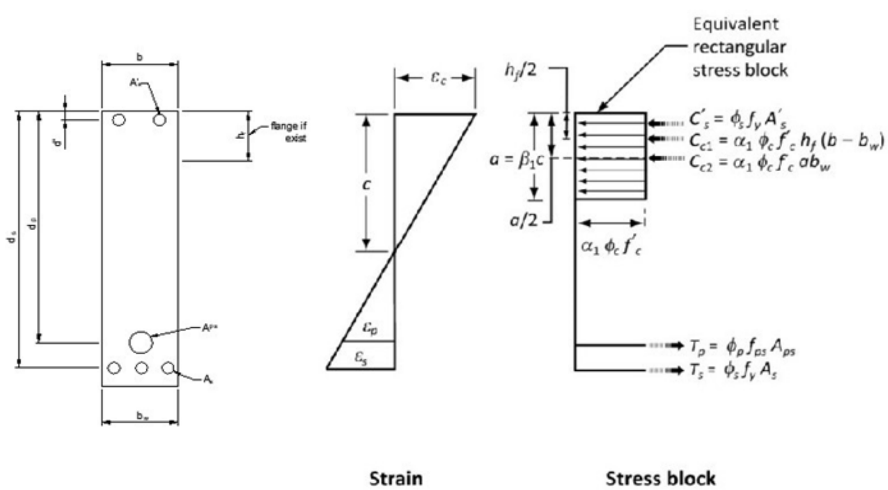


Footing slab flexual capacity calculation

	Symbols	Data	Unit	Notes
	b	1000.00	mm	Total width of flange (including web)
	b_w	1000.00	mm	Web width, when neutral axil is located in flange, bw should be taken as b (C8.8.4.1)
	h_f	0.00	mm	thickness of flange
	f_c'	30.00	Mpa	Concrete strength
	α_1	0.81		α1=0.85-0.0015*fc' ≥ 0.67 (CHBDC 8.8.3)
	β_1	0.90		β1=0.97-0.0025*fc' ≥ 0.67 (CHBDC 8.8.3)
	c	19.25	mm	c = (Ts-Cc1)/α1*β1*ϕc*fc'*bw , Distance from extrem compression fibre to neutral axis, α1*ϕc*fc' is concrete stress uniformly distributed over an equivalent compression zone
	a	17.23	mm	a = β1*c , Equivalent rectangular compression zone height
	ϕ_c	0.75		Resistance factor for concrete
	C_{c1}	0.00	N	Cc1 = α1*ϕc*fc'*hf*(b-bw) Amount of compression in flange (C8.8.3)
	C_{c2}	312000.00	N	Cc2 = α1*ϕc*fc'*a*bw Amount of compression in web (C8.8.3)
	A_s	867	mm^2	Area of rebars on the flexural tension side
	f_y	400.00	Mpa	Yield strength of tensile rebar
	ϕ_s	0.90		Resistance factor of reinforcing steel
	T_s	312000.00	N	Ts = ϕs*fy*As Amount of tension in reinforcing steel
	h	700.00	mm	Overall height of beam
	d_s	592.00	mm	Distance from centroid of rebars to extrem compression fibre of concrete beam
	M_r	**182.02**	kN-m	Moment resistance, Mr = Ts*(ds-a/2) - Cc1*(hf/2-a/2) (C8.8.4.1)

Limit for Min. & Max. Reinforcement Ratios

	M_r(min.)	110	kN-m	Min. Reinforcement: Factored Mr is at least 1.2*Mcr. Mcr = fcr*I/y (CHBDC 8.8.4.3)
	I	28583333333	mm^4	Moment of inertia
	y	680.75	mm	Distance to netural axis
	f_{cr}	2.19	Mpa	Cracking strengtch for normal-density concrete 0.4*√fc'
	M_{cr}	91990641.89	N-mm	Cracking moment
	c/d (max.)	0.03		Max. Reinforcement: c/d not exceeding 0.5, (CHBDC 8.8.4.5)

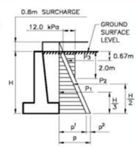


1. Design Parameters:

Friction coefficient between conc. and soil	=	0.7
ULS Soil bearing capacity (kPa)	=	250
ULS soil bearing capacity (kPa)	=	200
Unit weight of soil (kN/m3)	=	22
Unit weight of existing soil (kN/m3)	=	20
Unit weight of concrete (kN/m3)	=	24
Height of retaining wall, H (m)	=	5.6
At rest lateral soil pressure coefficient, Ko	=	0.38
Lateral soil pressure modification factor	=	1.19

DESIGN AID 2-11
ACTIVE EARTH PRESSURE WITH 800 mm SURCHARGE

q = EQUIVALENT FLUID PRESSURE (kPa)
p = UNIT PRESSURE AT DEPTH H METRES
p^1 = UNIT PRESS. DUE TO BACKFILL AT DEPTH H
p^2 = UNIT PRESSURE DUE TO 0.8m SURCHARGE
P = TOTAL FORCE PER LINEAR METRE OF WALL
M = OVERTURNING MOMENT (kN-m)



AT SERVICEABILITY LIMIT STATES

$\alpha_E = 1.0$
$q = 7.0 \times (H)$

AT ULTIMATE LIMIT STATES

$\alpha_E = 1.25$
$q = 1.25 \times (7.0) \times (H)$

$\phi = 30°$
$P = P_1 + P_2 + P_3$
$p = p^1 + p^2$

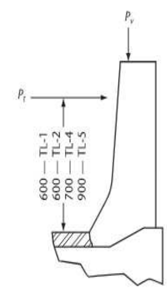
– RESULTS INCLUDE THE EFFECT OF COMPACTION SURCHARGE

SLS				ULS			
H (m)	P (kPa)	P (kN)	M (kN-m)	H (m)	P (kPa)	P (kN)	M (kN-m)
1.0	13	18	9	1.0	16	23	11
1.5	16	28	20	1.5	20	35	25
2.0	20	37	37	2.0	25	46	46
2.5	23	48	58	2.5	29	60	73
3.0	27	60	85	3.0	34	75	106
3.5	30	75	118	3.5	38	94	148
4.0	34	90	159	4.0	43	113	199
4.5	37	108	209	4.5	46	135	261
5.0	41	128	268	5.0	51	160	335
5.5	44	149	337	5.5	55	186	421
6.0	48	172	417	6.0	60	215	521

THE METHOD OF EQUIVALENT FLUID PRESSURES IS LIMITED TO A MAXIMUM HEIGHT OF 6.0m. FOR RETAINING WALLS WITH HEIGHTS > 6.0m, THE EARTH PRESSURE DISTRIBUTION SHALL BE ESTABLISHED BY A GEOTECHNICAL ENGINEER.

2. Dimension Estimates:

Retaining wall height, H (m)	=	5.6	
Base width, L (m)	=	3	L=0.5H to 2/3H
Thickness of base, D (m)	=	1	D=0.1H
Stem thickness at the bottom, C (m)	=	1	C=0.1H
Stem thickness at the top, T (m)	=	0.45	T=0.25m min.
Width of the toe, B_T (m)	=	0.8	B=0.25L to 0.33L
Parapet height (m)	=	0.8	
Parapet width (m)	=	0.3	



Soil height at toe, H_T (m)	=	0
Soil height at heel, H_H (m)	=	5.6

Table 3.7
Loads on traffic barriers
(See Clauses 3.8.8.1 and 12.4.3.2.5.)

Performance level	Transverse load, kN	Longitudinal load, kN	Vertical load, kN
TL-1	25	10	10
TL-2	50	20	10
TL-4	100	30	30
TL-5	210	70	90

Notes:
(1) *Traffic barrier types are illustrative only and other types may be used.*
(2) *Transverse load P_t shall be applied over a barrier length of 1200 mm for TL-1 and TL-2 barriers, 1050 mm for TL-4 barriers, and 2400 mm for TL-5 barriers.*
(3) *Longitudinal load P_l shall be applied at the same locations and over the same barrier lengths as P_t. For post and railing barriers, the longitudinal load shall not be distributed to more than three posts.*
(4) *Vertical load P_v shall be applied over a barrier length of 5500 mm for TL-1, TL-2, and TL-4 barriers and 12 000 mm for TL-5 barriers.*
(5) *These loads shall be used for the design of traffic barrier anchorages and decks only.*

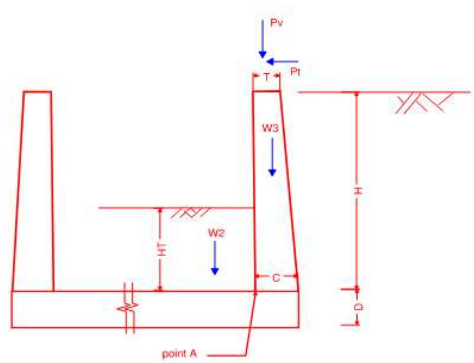
Table 6.9-2 – Geotechnical parameters for backfill material

Material Description	Unit Weight (kN/m³) Drained γ_{dr}	Effective γ'	Friction Angle (°) ϕ	Friction Factor, tan δ	Lateral Earth Pressure Coefficients Active K_a	At Rest K_o	Passive K_P
Granular A (Crushed Stone)	22	13.5	38	0.6	0.24	0.38	4.20
Granular B Type II (Crushed Stone)	22	13.5	40	0.6	0.22	0.36	4.60

Notes:
The properties of backfill materials are for a condition of 98% of the materials SPMDD.
Earth pressure coefficients provided are for the horizontal backfill profile.
For soil above the water table, the "drained" unit weight must be used and below the water table, the "effective" unit weight must be used.

3.1 Moment at Bottom of Stem :

live load factor	=	1.7	
dead load factor-earth fill	=	0.8	CHBDC Table 3.3
min. dead load factor-concrete	=	0.9	CHBDC Table 3.3
max. dead load factor-concrete	=	1.2	
TL-4 load transverse load, Pt (kN)	=	100	
stem (m)	=	13.65	
due to TL-4 (kNm)	=	78.46	
due to soil pressure, surcharge, compact	=	526.68	obtained from Design Aid 2-11
ULS total, Mo (kNm)	=	605.14	



3.2 Shear at Bottom of Stem:

The passive soil pressure is neglected in the sliding check.

ULS sliding force (kN) =	229.30	obtained from Design Aid 2-11		
ULS TL-4 transverse force, (kN) =	12.45			
Total, Fs (kN) =	241.76			

5. Stem Design:

ULS design loads bottom of the stem

M (kNm) =	605.14	D/R =	0.81	
V (kN) =	241.76	D/R =	0.44	same shear capacites are taken for footing slab and wall stem

20M @ 150

70mm cover

6. Base Design:

Parapet weight (kN/m) =	5.76		
Stem weight (kN/m) =	97.44		
Vertical collision force, Pv (kN/m) =	1.82		
ULS total (kN/m) =	103.2		
Shear at base (kN/m) =	103.2	D/R =	0.19
Positive moment at edge (kNm) =	605.14	D/R =	0.84
Negative mement at mid-span (kNm) =	159.6	D/R =	0.58

15M @ 300 at positive moment area

20M @ 150 at negative moment area

100mm cover

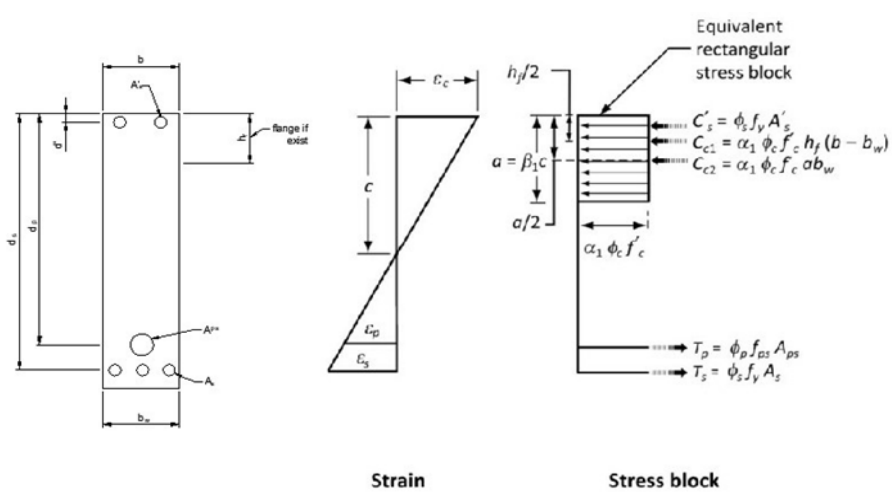
PROJECT	The Ottawa Hospital New Campus Development - Phase 4 Retaining Wall Calculation	SHEET	
PROJECT No.	CA0027758.0-51	DATE	08-Jan-26
SUBJECT	Retaining Wall #6 #7 #8 & #9 tall sections - flexual capacity calculation	DESIGNER	Hui Liu
		CHECKER	Felix Wasiewicz

Footing slab flexual capacity calculation

	Symbols	Data	Unit	Notes
	b	1000.00	mm	Total width of flange (including web)
	b_w	1000.00	mm	Web width, when neutral axil is located in flange, bw should be taken as b (C8.8.4.1)
	h_f	0.00	mm	thickness of flange
	f_c'	30.00	Mpa	Concrete strength
	α_1	0.81		α1=0.85-0.0015*fc' ≥ 0.67 (CHBDC 8.8.3)
	β_1	0.90		β1=0.97-0.0025*fc' ≥ 0.67 (CHBDC 8.8.3)
	c	51.08	mm	c = (Ts-Cc1)/α1*β1*ϕc*fc'*bw , Distance from extrem compression fibre to neutral axis, α1*ϕc*fc' is concrete stress uniformly distributed over an equivalent compression zone
	a	45.71	mm	a = β1*c , Equivalent rectangular compression zone height
	ϕ_c	0.75		Resistance factor for concrete
	C_{c1}	0.00	N	Cc1 = α1*ϕc*fc'*hf*(b-bw) Amount of compression in flange (C8.8.3)
	C_{c2}	828000.00	N	Cc2 = α1*ϕc*fc'*a*bw Amount of compression in web (C8.8.3)
	A_s	2300	mm^2	Area of rebars on the flexural tension side
	f_y	400.00	Mpa	Yield strength of tensile rebar
	ϕ_s	0.90		Resistance factor of reinforcing steel
	T_s	828000.00	N	Ts = ϕs*fy*As Amount of tension in reinforcing steel
	h	1000.00	mm	Overall height of beam
	d_s	890.00	mm	Distance from centroid of rebars to extrem compression fibre of concrete beam
	M_r	**717.99**	kN-m	Moment resistance, Mr = Ts*(ds-a/2) - Cc1*(hf/2-a/2) (C8.8.4.1)

Limit for Min. & Max. Reinforcement Ratios

	M_r(min.)	231	kN-m	Min. Reinforcement: Factored Mr is at least 1.2*Mcr. Mcr = fcr*I/y (CHBDC 8.8.4.3)
	I	83333333333	mm^4	Moment of inertia
	y	948.92	mm	Distance to netural axis
	f_{cr}	2.19	Mpa	Cracking strengtch for normal-density concrete 0.4*√fc'
	M_{cr}	192401560.01	N-mm	Cracking moment
	c/d (max.)	0.06		Max. Reinforcement: c/d not exceeding 0.5, (CHBDC 8.8.4.5)



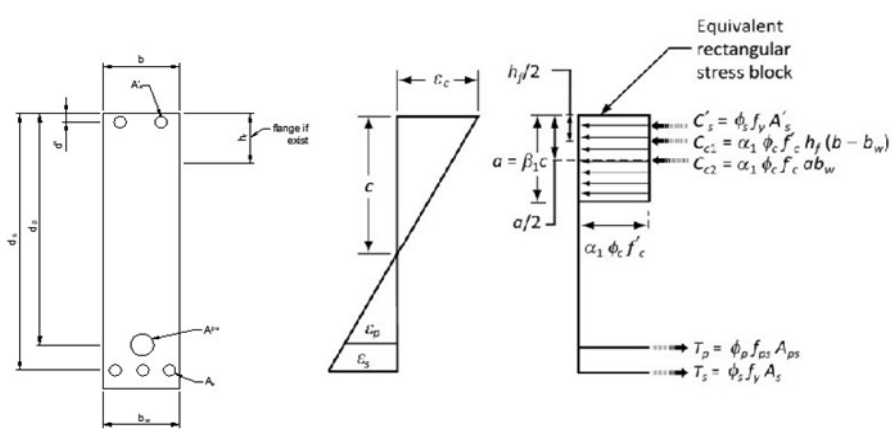
PROJECT	The Ottawa Hospital New Campus Development - Phase 4 Retaining Wall Calculation	SHEET	
PROJECT No.	CA0027758.0-51	DATE	08-Jan-26
SUBJECT	Retaining Wall #6 #7 #8 & #9 tall sections - flexual capacity calculation	DESIGNER	Hui Liu
		CHECKER	Felix Wasiewicz

Footing slab flexual capacity calculation

	Symbols	Data	Unit	Notes
	b	1000.00	mm	Total width of flange (including web)
	b_w	1000.00	mm	Web width, when neutral axil is located in flange, bw should be taken as b (C8.8.4.1)
	h_f	0.00	mm	thickness of flange
	f_c'	30.00	Mpa	Concrete strength
	α_1	0.81		α1=0.85-0.0015*fc' ≥ 0.67 (CHBDC 8.8.3)
	β_1	0.90		β1=0.97-0.0025*fc' ≥ 0.67 (CHBDC 8.8.3)
	c	19.25	mm	c = (Ts-Cc1)/α1*β1*ϕc*fc'*bw , Distance from extrem compression fibre to neutral axis, α1*ϕc*fc' is concrete stress uniformly distributed over an equivalent compression zone
	a	17.23	mm	a = β1*c , Equivalent rectangular compression zone height
	ϕ_c	0.75		Resistance factor for concrete
	C_{c1}	0.00	N	Cc1 = α1*ϕc*fc'*hf*(b-bw) Amount of compression in flange (C8.8.3)
	C_{c2}	312000.00	N	Cc2 = α1*ϕc*fc'*a*bw Amount of compression in web (C8.8.3)
	A_s	867	mm^2	Area of rebars on the flexural tension side
	f_y	400.00	Mpa	Yield strength of tensile rebar
	ϕ_s	0.90		Resistance factor of reinforcing steel
	T_s	312000.00	N	Ts = ϕs*fy*As Amount of tension in reinforcing steel
	h	1000.00	mm	Overall height of beam
	d_s	890.00	mm	Distance from centroid of rebars to extrem compression fibre of concrete beam
	M_r	**274.99**	kN-m	Moment resistance, Mr = Ts*(ds-a/2) - Cc1*(hf/2-a/2) (C8.8.4.1)

Limit for Min. & Max. Reinforcement Ratios

	M_r(min.)	223	kN-m	Min. Reinforcement: Factored Mr is at least 1.2*Mcr. Mcr = fcr*I/y (CHBDC 8.8.4.3)
	I	83333333333	mm^4	Moment of inertia
	y	980.75	mm	Distance to netural axis
	f_{cr}	2.19	Mpa	Cracking strengtch for normal-density concrete 0.4*√fc'
	M_{cr}	186157069.39	N-mm	Cracking moment
	c/d (max.)	0.02		Max. Reinforcement: c/d not exceeding 0.5, (CHBDC 8.8.4.5)



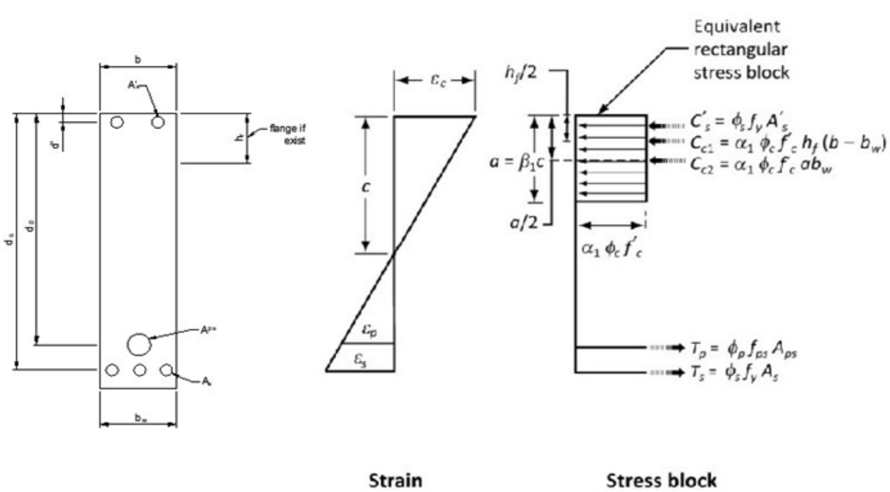
Strain Stress block

Wall flexual capacity calculation - at bottom of wall stem

	Symbols	Data	Unit	Notes
	b	1000.00	mm	Total width of flange (including web)
	b_w	1000.00	mm	Web width, when neutral axil is located in flange, bw should be taken as b (C8.8.4.1)
	h_f	0.00	mm	thickness of flange
	f_c'	30.00	Mpa	Concrete strength
	α_1	0.81		α1=0.85-0.0015*fc' ≥ 0.67 (CHBDC 8.8.3)
	β_1	0.90		β1=0.97-0.0025*fc' ≥ 0.67 (CHBDC 8.8.3)
	c	51.08	mm	c = (Ts-Cc1)/α1*β1*ϕc*fc'*bw , Distance from extrem compression fibre to neutral axis, α1*ϕc*fc' is concrete stress uniformly distributed over an equivalent compression zone
	a	45.71	mm	a = β1*c , Equivalent rectangular compression zone height
	ϕ_c	0.75		Resistance factor for concrete
	C_{c1}	0.00	N	Cc1 = α1*ϕc*fc'*hf*(b-bw) Amount of compression in flange (C8.8.3)
	C_{c2}	828000.00	N	Cc2 = α1*ϕc*fc'*a*bw Amount of compression in web (C8.8.3)
	A_s	2300	mm^2	Area of rebars on the flexural tension side
	f_y	400.00	Mpa	Yield strength of tensile rebar
	ϕ_s	0.90		Resistance factor of reinforcing steel
	T_s	828000.00	N	Ts = ϕs*fy*As Amount of tension in reinforcing steel
	h	1000.00	mm	Overall height of beam
	d_s	920.00	mm	Distance from centroid of rebars to extrem compression fibre of concrete beam
	M_r	**742.83**	kN-m	Moment resistance, Mr = Ts*(ds-a/2) - Cc1*(hf/2-a/2) (C8.8.4.1)

Limit for Min. & Max. Reinforcement Ratios

	M_r(min.)	231	kN-m	Min. Reinforcement: Factored Mr is at least 1.2*Mcr. Mcr = fcr*I/y (CHBDC 8.8.4.3)
	I	83333333333	mm^4	Moment of inertia
	y	948.92	mm	Distance to netural axis
	f_{cr}	2.19	Mpa	Cracking strengtch for normal-density concrete 0.4*√fc'
	M_{cr}	192401560.01	N-mm	Cracking moment
	c/d (max.)	0.06		Max. Reinforcement: c/d not exceeding 0.5, (CHBDC 8.8.4.5)



Wall stem shear capacity calculation - section at bottom of wall

	Rmark	Symbols	Data	Unit	Notes
Section		h	1000	mm	Overall thickness
		d	920	mm	Effective depth (distance from extrem compression fibre to centroid of tensile force)
		d_v	828	mm	0.72h - taken as the greater; 0.9d - taken as the greater
		b_v	900	mm	Effective web width, taken as minimum web witdth whin the depth dv
Vc		ϕ_c	0.75		Resistance factor for concrete
		f_c'	30	Mpa	Concrete strength
		f_{cr}	2.19089	Mpa	Not be greater than 3.2MPa; 0.4√fc' - nomal-density concrete
		β	0.18		1) β = 0.18 -with tansverse reinforcement; 2)β = 0.18 -without tansverse reinforcement, <3dv; 3) β = 230/(1000+dv) without transverse reinforcement, but having max. size of coarse aggregate not less than 20mm
		V_c	**551019.8**	N	Vc = 2.5*β*ϕc*fcr*bv*dv, (CHBDC 8.9.3.4)
Vs		Θ	42	°	Θ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.733038	rad	Convert Dgrees to Radians
		ϕ_s	0.9		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotΘ	1.110613		cosΘ/sinΘ
		s	100	mm	Spacing of stirups
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*cotΘ/s (CHBDC 8.9.3.5)
Vs - with inclination		Θ	42	°	Θ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.733038	rad	Convert Dgrees to Radians
		ϕ_s	0.95		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotΘ	1.110613		cosΘ/sinΘ
		s	200	mm	Spacing of stirups
		α	45	°	Transverse reinforcement inclined at an angle to the longitudinal axis
		Radians	0.785398	rad	Convert Degrees to Radians
		cotα	1		cosα/sinα
		sinα	0.707107		
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*(cotΘ + cotα)*sinα/s Trans verse reinforcement inclined at an angle to the longitudinal axis, and in the direction that will intersec diagonal cracks caused by the shear (CHBDC 8.9.3.5)
Vr	Automati	Limit of $(V_c + V_s)$	4191750	N	Vc + Vs shall not exceed 0.25*ϕc*fc'*bv*dv
	Φ_p = 0.95	V_p	0	N	Component in the direction of the applied shear of all of the effective prestressing forces crossing the critical section factored by ϕp (taken as positive if resisting the applied shear)
	Total	V_r	**551.02**	kN	Vr = Vc +Vs+Vp

Min. Av, Vf limit requiring transverse reinforcement

		V (limit)	244898	N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25 Tcr (CHBDC 8.9.1.2)
		$0.25T_{cr}$		N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25*Tcr , Tcr = 0.8 * φc*fcr* (Acp2/Pc)*[1+fce/(0.8* φc *fcr)]0.5 (CHBDC 8.9.1.1)
		A_v(min.)	74	mm^2	Min. amount of transverse reinforcement: Av is not less than 0.15*fcr(bv*s/fy)

1. Design Parameters:

Friction coefficient between conc. and soil =	0.7	
ULS soil bearing capacity (kPa) =	250	
ULS soil bearing capacity (kPa)	200	
Unit weight of soil (kN/m3) =	22	
Unit weight of existing soil (kN/m3)	20	
Unit weight of concrete (kN/m3) =	24	
Granular A friction angle (°) =	38	
:eral earth pressure at rest coefficient, Ko =	0.38	
Height of retaining wall at grade, H1 (m) =	5.3	
Railing height (m) =	1.05	

2. Dimension Estimates:

Retaining wall height, H (m) =	5.3	
Base width, L (m) =	4.2	L=0.5H to 2/3H
Thickness of base, D (m) =	0.7	D=0.1H
Stem thickness at the bottom, C (m) =	0.6	C=0.1H
Stem thickness at the top, T (m) =	0.4	T=0.25m min.
Width of the toe, B_T (m) =	1	B=0.25L to 0.33L
Parapet height (m) =	0.3	
Parapet width (m) =	0.4	
Railing height (m) =	1.05	

3. Stability Check:

Soil height at toe, H_T (m) =	1.8	
Width of the toe, B_T (m) =	1	
Soil height at heel, H_H (m) =	5.3	
Width of the heel, B_H (m) =	2.6	
Weight due to soil, W1 (kN) =	303.16	
Weight due to soil, W2 (kN) =	0.00	43.56 actual weight
Weight due to stem, W3 (kN) =	63.60	
Weight due to base, W4 (kN) =	70.56	
Weight due to parapet & railing, W5 (kN) =	3.68	
Total (kN) =	441.00	

3.1 Check for Overturning Moment:

soil pressure table =	1.19	Yko is 7.0 on the table, this
live load factor =	1.7	
dead load factor-earth fill =	0.8	CHBDC Table 3.3
dead load factor-concrete =	0.9	CHBDC Table 3.3
pedestrian load-transverse, Wp (kN) =	1.2	
stem (m) =	1	
due to Wp (kNm)	13.57	
due to soil pressure, surcharge, compact =	356.47	11, increased 4% for the
ULS total, Mo (kNm) =	370.04	
Mr (kNm) =	909.35	0.9 x ((W3+W5) x arm + W4 x
pressure on the toe side, Mp (kNm) =	0.00	Design Aid 2-11, converted to

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m³. The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m³, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

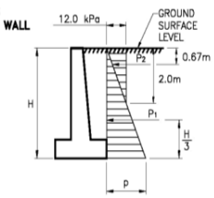
Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented below.

3.8.8.2 Pedestrian and bicycle barriers

The load on pedestrian and bicycle barrier railings shall be a uniform load of 1.20 kN/m applied laterally and vertically simultaneously.

DESIGN AID 2-10
ACTIVE EARTH PRESSURE WITHOUT SURCHARGE

q = EQUIVALENT FLUID PRESSURE (kPa)
p = UNIT PRESSURE AT DEPTH H METRES
P = TOTAL FORCE PER LINEAR METRE OF WALL
M = OVERTURNING MOMENT (kN–m)



AT SERVICEABILITY LIMIT STATES

α_E = 1.0
q = 7.0 x (H)

AT ULTIMATE LIMIT STATES

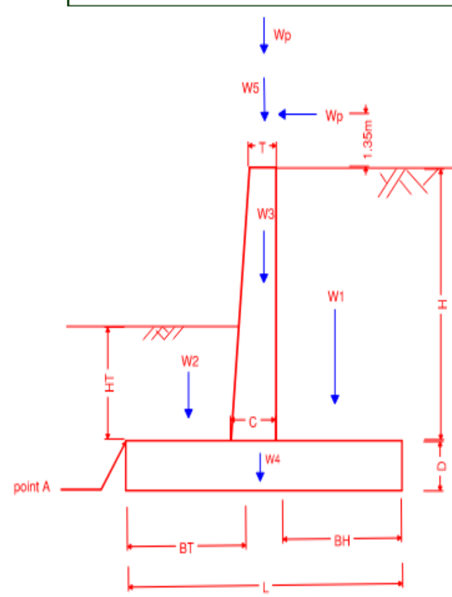
α_E = 1.25
q = 1.25 x (7.0) x (H)

ϕ = 30°
$P = P_1 + P_2$

– RESULTS INCLUDE THE EFFECT OF COMPACTION SURCHARGE

SLS				ULS			
H (m)	P (kPa)	P (kN)	M (kN–m)	H (m)	P (kPa)	P (kN)	M (kN–m)
1.0	7	13	6	1.0	9	16	8
1.5	11	19	14	1.5	13	24	18
2.0	14	26	25	2.0	18	33	31
2.5	18	33	40	2.5	22	43	50
3.0	21	44	59	3.0	26	55	74
3.5	25	55	84	3.5	31	69	105
4.0	28	68	114	4.0	35	85	143
4.5	32	83	152	4.5	40	104	190
5.0	35	100	197	5.0	44	125	246
5.5	39	118	251	5.5	48	148	314
6.0	42	138	315	6.0	53	173	394

THE METHOD OF EQUIVALENT FLUID PRESSURES IS LIMITED TO A MAXIMUM HEIGHT OF 6.0m. FOR RETAINING WALLS WITH HEIGHTS > 6.0m, THE EARTH PRESSURE DISTRIBUTION SHALL BE ESTABLISHED BY A GEOTECHNICAL ENGINEER.



Totoal ULS overturning resistance, M_{tr}	909.35	Mr + Mp
Mr/Mo =	2.46	passive soil pressure

3.2 Check for Sliding:

The passive soil pressure is neglected in the sliding check.

ULS sliding force (kN) =	172.65	11, increased 4% for sloped
ULS TL-1 transverse force, (kN) =	2.04	25kN/1.2m
Total, Fs (kN) =	174.69	
ULS sliding resistance, Fr (kN) =	329.37	total weight x friction coef.
Fr/Fs =	1.89	CHBDC Table 6.2

Retaining systems	Bearing, ϕ_{gu}	Analysis	0.45	0.50	0.60
	Overturning, ϕ_{gu}	Analysis	0.45	0.50	0.55
	Base sliding, ϕ_{gu}	Analysis	0.70	0.80	0.90
	Facing interface sliding, ϕ_{gu}	Test	0.75	0.85	0.95
	Connections, ϕ_{gu}	Test	0.65	0.70	0.75
	Settlement, ϕ_{gs}	Analysis	0.7	0.8	0.9
	Deflection/tilt, ϕ_{gs}	Analysis	0.7	0.8	0.9

4. Check for Bearing Pressure under Footing:

Base area, A (m2) =	4.20	
Area property, S (m3) =	2.94	neglecting eccentric
ULS Tl-1 load, Pv (kN) =	3.09	
ULS total vertical load, P (kN) =	601.90	
moment at bottom of the stem, M (kNm) =	356.47	Mo
Reaction at toe (kPa) =	264.56	P/A+M/S
Reaction at heel (kPa) =	22.06	P/A-M/S
Soil excavatrion weight (kPa) =	20.00	
ULS soil bearing capacity (kPa) =	250.00	
Net max. reaction (kPa) =	144.56	
Ratio of Demand/Capacity =	0.58	soil beairng capacity is Okay

Earth pressure and hydrostatic pressure	Maximum α_E	Minimum α_E
Passive earth pressure, considered as a load*	1.25	0.50
At-rest earth pressure	1.25	0.80
Active earth pressure	1.25	0.80
Backfill pressure	1.25	0.80
Hydrostatic pressure	1.10	0.90

Material Description	Unit Weight (kN/m^3) Drained γ_{dr}	Unit Weight (kN/m^3) Effective γ'	Friction Angle (°) ϕ	Friction Factor, tan δ	Lateral Earth Pressure Coefficients Active K_a	Lateral Earth Pressure Coefficients At Rest K_o	Lateral Earth Pressure Coefficients Passive K_P
Granular A (Crushed Stone)	22	13.5	38	0.6	0.24	0.38	4.20
Granular B Type II (Crushed Stone)	22	13.5	40	0.6	0.22	0.36	4.60

Notes:
The properties of backfill materials are for a condition of 98% of the materials SPMDD.
Earth pressure coefficients provided are for the horizontal backfill profile.
For soil above the water table, the "drained" unit weight must be used and below the water table, the "effective" unit weight must be used.

Bearing resistance values for the design of bridge abutment footings at serviceability limit states (SLS) and ultimate limit states (ULS) at **200 kPa** and **250 kPa**, respectively, are considered acceptable for conventional spread footings supported on an engineered fill and/or native soil bearing surface improved by the ground improvement program described in the following sections and reviewed as necessary to match the structural requirements.

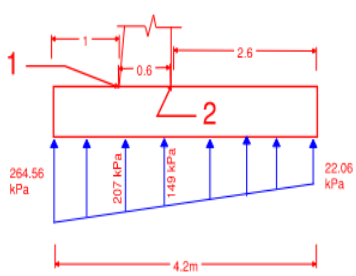
5. Stem Design:

ULS design loads bottom of the stem

M (kNm) =	370.04	D/R	=	0.90
V (kN) =	174.69	D/R	=	0.89

20M @ 150 vertical
15M @ 300 vertical & horizontal
70mm concrete cover

6. Base Design:



Cast against and permanently exposed to earth	(1) Footing	Reinforcing steel	100 ± 25	—	
	(2) Caisson	Reinforcing steel	100 ± 25	—	
		Post-tensioning ducts	120 ± 15	—	
Various	Components other than those covered elsewhere in this Table	Reinforcing steel	70 ± 20§	55 ± 10§	
		Pretensioning strands	—	70 ± 5§	
		Post-tensioning ducts	90* ± 15§	80* ± 10§	

shear at section 1, V1 (kN) =	235.78	D/R	=	0.85497
shear at section 2, V2 (kN) =	222.38	D/R	=	0.80637

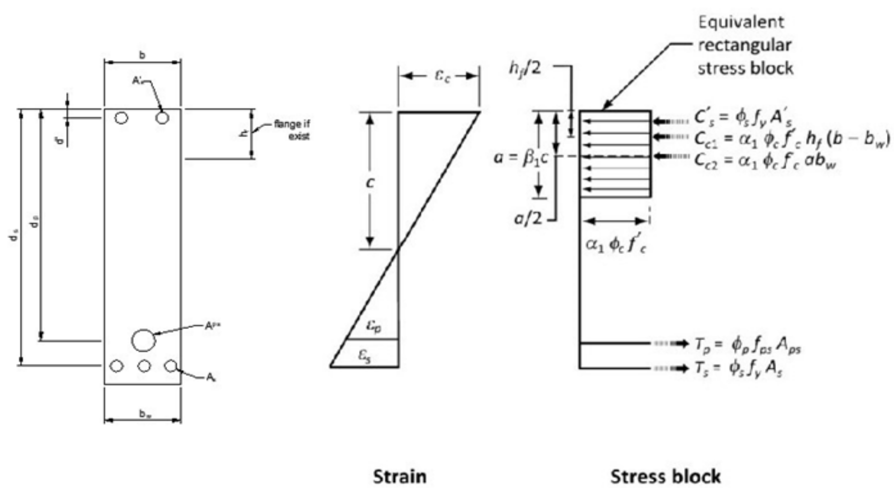
20M @ 150 transverse
15M @ 300 longitudinal
100mm concrete cover at bottom, 70mm other locations

Wall stem flexual capacity calculation - at bottom of wall stem

	Symbols	Data	Unit	Notes
	b	1000.00	mm	Total width of flange (including web)
	b_w	1000.00	mm	Web width, when neutral axil is located in flange, bw should be taken as b (C8.8.4.1)
	h_f	0.00	mm	thickness of flange
	f_c'	30.00	Mpa	Concrete strength
	α_1	0.81		α1=0.85-0.0015*fc' ≥ 0.67 (CHBDC 8.8.3)
	β_1	0.90		β1=0.97-0.0025*fc' ≥ 0.67 (CHBDC 8.8.3)
	c	51.08	mm	c = (Ts-Cc1)/α1*β1*ϕc*fc'*bw , Distance from extrem compression fibre to neutral axis, α1*ϕc*fc' is concrete stress uniformly distributed over an equivalent compression zone
	a	45.71	mm	a = β1*c , Equivalent rectangular compression zone height
	ϕ_c	0.75		Resistance factor for concrete
	C_{c1}	0.00	N	Cc1 = α1*ϕc*fc'*hf*(b-bw) Amount of compression in flange (C8.8.3)
	C_{c2}	828000.00	N	Cc2 = α1*ϕc*fc'*a*bw Amount of compression in web (C8.8.3)
	A_s	2300	mm^2	Area of rebars on the flexural tension side
	f_y	400.00	Mpa	Yield strength of tensile rebar
	ϕ_s	0.90		Resistance factor of reinforcing steel
	T_s	828000.00	N	Ts = ϕs*fy*As Amount of tension in reinforcing steel
	h	600.00	mm	Overall height of beam
	d_s	520.00	mm	Distance from centroid of rebars to extrem compression fibre of concrete beam
	M_r	**411.63**	kN-m	Moment resistance, Mr = Ts*(ds-a/2) - Cc1*(hf/2-a/2) (C8.8.4.1)

Limit for Min. & Max. Reinforcement Ratios

	M_r(min.)	86	kN-m	Min. Reinforcement: Factored Mr is at least 1.2*Mcr. Mcr = fcr*I/y (CHBDC 8.8.4.3)
	I	18000000000	mm^4	Moment of inertia
	y	548.92	mm	Distance to netural axis
	f_{cr}	2.19	Mpa	Cracking strengtch for normal-density concrete 0.4*√fc'
	M_{cr}	71842597.05	N-mm	Cracking moment
	c/d (max.)	0.10		Max. Reinforcement: c/d not exceeding 0.5, (CHBDC 8.8.4.5)



Wall stem shear capacity calculation - section at bottom of wall stem

	Rmark	Symbols	Data	Unit	Notes
Section		h	600	mm	Overall thickness
		d	492	mm	Effective depth (distance from extrem compression fibre to centroid of tensile force)
		d_v	442.8	mm	0.72h - taken as the greater; 0.9d - taken as the greater
		b_v	600	mm	Effective web width, taken as minimum web witdth whin the depth dv
Vc		ϕ_c	0.75		Resistance factor for concrete
		f_c'	30	Mpa	Concrete strength
		f_{cr}	2.19089	Mpa	Not be greater than 3.2MPa; 0.4√fc' - nomal-density concrete
		β	0.18		1) β = 0.18 -with tansverse reinforcement; 2)β = 0.18 -without tansverse reinforcement, <3dv; 3) β = 230/(1000+dv) without transverse reinforcement, but having max. size of coarse aggregate not less than 20mm
		V_c	**196450.6**	N	Vc = 2.5*β*ϕc*fcr*bv*dv, (CHBDC 8.9.3.4)
Vs		ϴ	42	°	ϴ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.733038	rad	Convert Dgrees to Radians
		ϕ_s	0.9		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotϴ	1.110613		cosϴ/sinϴ
		s	100	mm	Spacing of stirups
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*cotϴ/s (CHBDC 8.9.3.5)
Vs - with inclination		ϴ	42	°	ϴ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.733038	rad	Convert Dgrees to Radians
		ϕ_s	0.95		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotϴ	1.110613		cosϴ/sinϴ
		s	200	mm	Spacing of stirups
		α	45	°	Transverse reinforcement inclined at an angle to the longitudinal axis
		Radians	0.785398	rad	Convert Degrees to Radians
		cotα	1		cosα/sinα
		sinα	0.707107		
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*(cotϴ + cotα)*sinα/s Trans verse reinforcement inclined at an angle to the longitudinal axis, and in the direction that will intersec diagonal cracks caused by the shear (CHBDC 8.9.3.5)
Vr	Automati	Limit of $(V_c + V_s)$	1494450	N	Vc + Vs shall not exceed 0.25*ϕc*fc'*bv*dv
	Φ_p = 0.95	V_p	0	N	Component in the direction of the applied shear of all of the effective prestressing forces crossing the critical section factored by ϕp (taken as positive if resisting the applied shear)
	Total	**V_r**	**196.45**	kN	Vr = Vc +Vs+Vp

Min. Av, Vf limit requiring transverse reinforcement

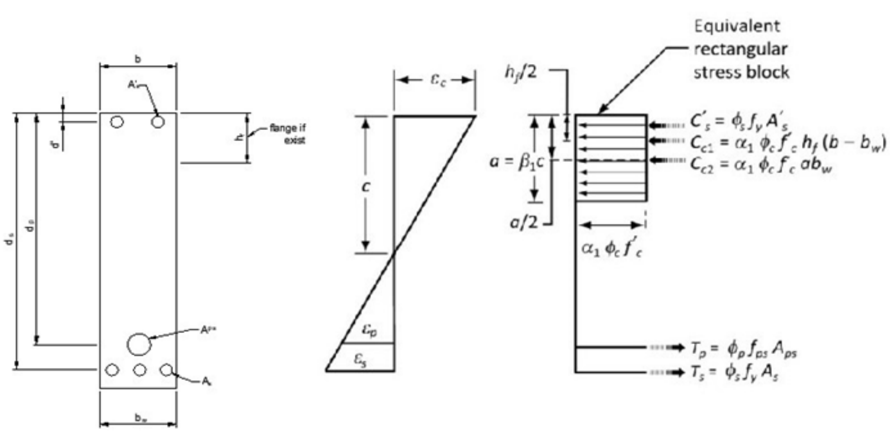
		V (limit)	87311	N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25 Tcr (CHBDC 8.9.1.2)
		$0.25T_{cr}$		N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25*Tcr , Tcr = 0.8 * φc*fcr* (Acp2/Pc)*[1+fce/(0.8* φc *fcr)]0.5 (CHBDC 8.9.1.1)
		A_v(min.)	49	mm^2	Min. amount of transverse reinforcement: Av is not less than 0.15*fcr(bv*s/fy)

Footing slab flexual capacity calculation

	Symbols	Data	Unit	Notes
	b	1000.00	mm	Total width of flange (including web)
	b_w	1000.00	mm	Web width, when neutral axil is located in flange, bw should be taken as b (C8.8.4.1)
	h_f	0.00	mm	thickness of flange
	f_c'	30.00	Mpa	Concrete strength
	α_1	0.81		α1=0.85-0.0015*fc' ≥ 0.67 (CHBDC 8.8.3)
	β_1	0.90		β1=0.97-0.0025*fc' ≥ 0.67 (CHBDC 8.8.3)
	c	51.08	mm	c = (Ts-Cc1)/α1*β1*ϕc*fc'*bw , Distance from extrem compression fibre to neutral axis, α1*ϕc*fc' is concrete stress uniformly distributed over an equivalent compression zone
	a	45.71	mm	a = β1*c , Equivalent rectangular compression zone height
	ϕ_c	0.75		Resistance factor for concrete
	C_{c1}	0.00	N	Cc1 = α1*ϕc*fc'*hf*(b-bw) Amount of compression in flange (C8.8.3)
	C_{c2}	828000.00	N	Cc2 = α1*ϕc*fc'*a*bw Amount of compression in web (C8.8.3)
	A_s	2300	mm^2	Area of rebars on the flexural tension side
	f_y	400.00	Mpa	Yield strength of tensile rebar
	ϕ_s	0.90		Resistance factor of reinforcing steel
	T_s	828000.00	N	Ts = ϕs*fy*As Amount of tension in reinforcing steel
	h	700.00	mm	Overall height of beam
	d_s	590.00	mm	Distance from centroid of rebars to extrem compression fibre of concrete beam
	M_r	**469.59**	kN-m	Moment resistance, Mr = Ts*(ds-a/2) - Cc1*(hf/2-a/2) (C8.8.4.1)

Limit for Min. & Max. Reinforcement Ratios

	M_r(min.)	116	kN-m	Min. Reinforcement: Factored Mr is at least 1.2*Mcr. Mcr = fcr*I/y (CHBDC 8.8.4.3)
	I	28583333333	mm^4	Moment of inertia
	y	648.92	mm	Distance to netural axis
	f_{cr}	2.19	Mpa	Cracking strengtch for normal-density concrete 0.4*√fc'
	M_{cr}	96502952.91	N-mm	Cracking moment
	c/d (max.)	0.09		Max. Reinforcement: c/d not exceeding 0.5, (CHBDC 8.8.4.5)



Strain **Stress block**

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PROJECT No.	CA0027758.0-51	DATE	08-Jan-26
SUBJECT	Retaining Wall #12 - footing slab shear capacity calculation	DESIGNER	Hui Liu
		CHECKER	Felix Wasiewicz

Footing slab shear capacity calculation

	Rmark	Symbols	Data	Unit	Notes
Section		h	700	mm	Overall thickness
		d	592	mm	Effective depth (distance from extrem compression fibre to centroid of tensile force)
		d_v	532.8	mm	0.72h - taken as the greater; 0.9d - taken as the greater
		b_v	700	mm	Effective web width, taken as minimum web witdth whin the depth dv
Vc		ϕ_c	0.75		Resistance factor for concrete
		f_c'	30	Mpa	Concrete strength
		f_{cr}	2.19089	Mpa	Not be greater than 3.2MPa; 0.4√fc' - nomal-density concrete
		β	0.18		1) β = 0.18 -with tansverse reinforcement; 2)β = 0.18 -without tansverse reinforcement, <3dv; 3) β = 230/(1000+dv) without transverse reinforcement, but having max. size of coarse aggregate not less than 20mm
		V_c	**275776.1**	N	Vc = 2.5*β*ϕc*fcr*bv*dv, (CHBDC 8.9.3.4)
Vs		ϴ	42	°	ϴ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.733038	rad	Convert Dgrees to Radians
		ϕ_s	0.9		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotϴ	1.110613		cosϴ/sinϴ
		s	100	mm	Spacing of stirups
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*cotϴ/s (CHBDC 8.9.3.5)
Vs - with inclination		ϴ	42	°	ϴ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.733038	rad	Convert Dgrees to Radians
		ϕ_s	0.95		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotϴ	1.110613		cosϴ/sinϴ
		s	200	mm	Spacing of stirups
		α	45	°	Transverse reinforcement inclined at an angle to the longitudinal axis
		Radians	0.785398	rad	Convert Degrees to Radians
		cotα	1		cosα/sinα
		sinα	0.707107		
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*(cotϴ + cotα)*sinα/s Trans verse reinforcement inclined at an angle to the longitudinal axis, and in the direction that will intersec diagonal cracks caused by the shear (CHBDC 8.9.3.5)
Vr	Automati	Limit of $(V_c + V_s)$	2097900	N	Vc + Vs shall not exceed 0.25*ϕc*fc'*bv*dv
	Φ_p = 0.95	V_p	0	N	Component in the direction of the applied shear of all of the effective prestressing forces crossing the critical section factored by ϕp (taken as positive if resisting the applied shear)
	Total	$\mathbf{V_r}$	**275.78**	kN	Vr = Vc +Vs+Vp

Min. Av, Vf limit requiring transverse reinforcement

		V (limit)	122567	N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25 Tcr (CHBDC 8.9.1.2)
		$0.25T_{cr}$		N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25*Tcr , Tcr = 0.8 * φc*fcr* (Acp2/Pc)*[1+fce/(0.8* φc *fcr)]0.5 (CHBDC 8.9.1.1)
		A_v(min.)	58	mm^2	Min. amount of transverse reinforcement: Av is not less than 0.15*fcr(bv*s/fy)

1. Design Parameters:

Friction coefficient between conc. and =	0.7	
ULS soil bearing capacity (kPa) =	250	
ULS soil bearing capacity (kPa)	200	
Unit weight of soil (kN/m3) =	22	
Unit weight of existing soil (kN/m3)	20	
Unit weight of concrete (kN/m3) =	24	
Granular A friction angle (°) =	38	
:eral earth pressure at rest coefficient, Ko =	0.38	
Height of retaining wall at grade, H1 (m) =	2.13	
Railing height (m) =	1.05	

2. Dimension Estimates:

Retaining wall height, H (m) =	2.13	
Base width, L (m) =	2.1	L=0.5H to 2/3H
Thickness of base, D (m) =	0.6	D=0.1H
Stem thickness at the bottom, C (m) =	0.6	C=0.1H
Stem thickness at the top, T (m) =	0.4	T=0.25m min.
Width of the toe, B_T (m) =	0.5	B=0.25L to 0.33L
Parapet height (m) =	0.3	
Parapet width (m) =	0.4	
Railing height (m) =	1.05	

3. Stability Check:

Soil height at toe, H_T (m) =	0.45		
Width of the toe, B_T (m) =	0.5		
Soil height at heel, H_H (m) =	2.13		
Width of the heel, B_H (m) =	1		
Weight due to soil, W1 (kN) =	65.60		
Weight due to soil, W2 (kN) =	0.00	4.95	actual weight
Weight due to stem, W3 (kN) =	25.56		
Weight due to base, W4 (kN) =	30.24		
Weight due to parapet & railing, W5 (kN) =	3.68		
Total (kN) =	125.08		

3.1 Check for Overturning Moment:

Modification factor for the MTO lateral =	1.19	Yko is 7.0 on the table, this
live load factor =	1.7	
dead load factor-earth fill =	0.8	CHBDC Table 3.3
dead load factor-concrete =	0.9	CHBDC Table 3.3
pedestrian load-transverse, Wp (kN) =	1.2	
Wp distribution length at bottom of the =	1	
ULS overturning moment about point A	7.10	
ULS overturning moment about point A =	50.64	obtained from Design Aid 2-
ULS total, Mo (kNm) =	57.74	
ULS overturning resistance due to dead, =	130.97	0.8 x (W1 x arm + W2 x arm) +
ULS overturning resistance due to soil =	0.00	
Totoal ULS overturning resistance, M_{tr}	130.97	Mr + Mp
Mr/Mo =	2.27	must be greater than 2.0

3.2 Check for Sliding:

The passive soil pressure is neglected in the sliding check.

ULS sliding force (kN) =	46.58	obtained from Design Aid 2-
Railing force, (kN) =	2.04	
Total, Fs (kN) =	48.62	
ULS sliding resistance, Fr (kN) =	89.96	total weight x friction coef.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m³. The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m³, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

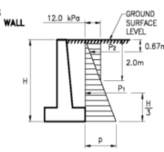
Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented below.

3.8.8.2 Pedestrian and bicycle barriers

The load on pedestrian and bicycle barrier railings shall be a uniform load of 1.20 kN/m applied laterally and vertically simultaneously.

DESIGN AID 2-10
ACTIVE EARTH PRESSURE WITHOUT SURCHARGE

q = EQUIVALENT FLUID PRESSURE (kPa)
p = UNIT PRESSURE AT DEPTH H METRES
P = TOTAL FORCE PER LINEAR METRE OF WALL
M = OVERTURNING MOMENT (kN–m)



AT SERVICEABILITY LIMIT STATES

$\alpha_E = 1.0$
$q = 7.0 \times (H)$

AT ULTIMATE LIMIT STATES

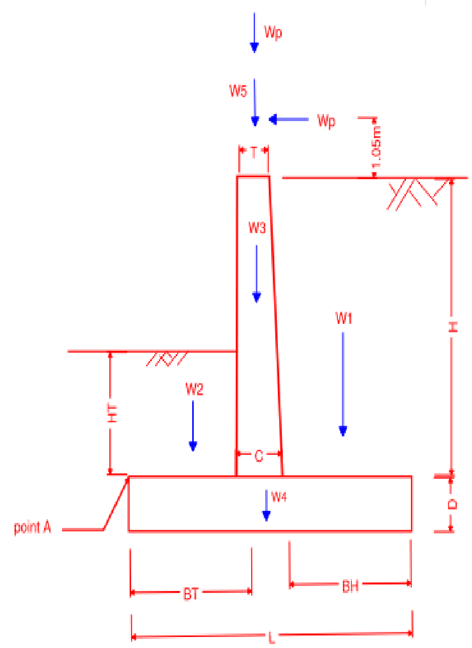
$\alpha_E = 1.25$
$q = 1.25 \times (7.0) \times (H)$

$\phi = 30°$
$P = P_1 + P_2$

– RESULTS INCLUDE THE EFFECT OF COMPACTION SURCHARGE

SLS				ULS			
H (m)	P (kPa)	P (kN)	M (kN–m)	H (m)	P (kPa)	P (kN)	M (kN–m)
1.0	7	13	6	1.0	9	16	8
1.5	11	19	14	1.5	13	24	18
2.0	14	26	25	2.0	18	33	31
2.5	18	33	40	2.5	22	43	50
3.0	21	44	59	3.0	26	55	74
3.5	25	55	84	3.5	31	69	105
4.0	28	68	114	4.0	35	85	143
4.5	32	83	152	4.5	40	104	190
5.0	35	100	197	5.0	44	125	246
5.5	39	118	251	5.5	48	148	314
6.0	42	138	315	6.0	53	173	394

THE METHOD OF EQUIVALENT FLUID PRESSURES IS LIMITED TO A MAXIMUM HEIGHT OF 6.0m. FOR RETAINING WALLS WITH HEIGHTS > 6.0m, THE EARTH PRESSURE DISTRIBUTION SHALL BE ESTABLISHED BY A GEOTECHNICAL ENGINEER.



Earth pressure and hydrostatic pressure	Maximum α_E	Minimum α_E
Passive earth pressure, considered as a load*	1.25	0.50
At-rest earth pressure	1.25	0.80
Active earth pressure	1.25	0.80
Backfill pressure	1.25	0.80
Hydrostatic pressure	1.10	0.90

Retaining systems	Bearing, ϕ_{gu}	Analysis	0.45	0.50	0.60
	Overturning, ϕ_{gu}	Analysis	0.45	0.50	0.55

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Base sliding, ϕ_{gu}	Analysis	0.70	0.80	0.90
Facing interface sliding, ϕ_{gu}	Test	0.75	0.85	0.95
Connections, ϕ_{gu}	Test	0.65	0.70	0.75
Settlement, ϕ_{gs}	Analysis	0.7	0.8	0.9
Deflection/tilt, ϕ_{gs}	Analysis	0.7	0.8	0.9

Fr/Fs = 1.85 must be greater than 1.25,

4. Check for Bearing Pressure under Footing:

Table 6.9-2 – Geotechnical parameters for backfill material

Material Description	Unit Weight (kN/m³)		Friction Angle (°)	Friction Factor, tan	Lateral Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'	ϕ	δ	Active K_a	At Rest K_o	Passive K_P
Granular A (Crushed Stone)	22	13.5	38	0.6	0.24	0.38	4.20
Granular B Type II (Crushed Stone)	22	13.5	40	0.6	0.22	0.36	4.60

Notes:
The properties of backfill materials are for a condition of 98% of the materials SPMDD.
Earth pressure coefficients provided are for the horizontal backfill profile.
For soil above the water table, the "drained" unit weight must be used and below the water table, the "effective" unit weight must be used.

Base area, A (m2)	=	2.10	
Area property, S (m3)	=	0.74	neglecting eccentric
ULS Tl-1 load, Pv (kN)	=	3.09	
ULS total vertical load, P (kN)	=	162.66	
moment at bottom of the stem, M (kNm)	=	50.64	Mo
Reaction at toe (kPa)	=	146.35	P/A+M/S
Reaction at heel (kPa)	=	8.56	P/A-M/S
Soil excavatrion weight (kPa)	=	20.00	
ULS soil bearing capacity (kPa)	=	250.00	
Net max. reaction (kPa)	=	91.75	
Ratio of Demand/Capacity	=	0.37	soil beairng capacity is Okay

Bearing resistance values for the design of bridge abutment footings at serviceability limit states (SLS) and ultimate limit states (ULS) at **200 kPa** and **250 kPa**, respectively, are considered acceptable for conventional spread footings supported on an engineered fill and/or native soil bearing surface improved by the ground improvement program described in the following sections and reviewed as necessary to match the structural requirements.

5. Stem Design:

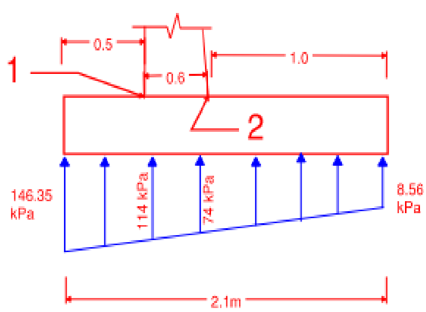
ULS design loads bottom of the stem

M (kNm)	=	57.74	D/R	=	0.38
V (kN)	=	48.62	D/R	=	0.25

15M @ 300 vertical & horizontal
70mm concrete cover

Cast against and permanently exposed to earth	(1) Footing	Reinforcing steel	100 ± 25	—
	(2) Caisson	Reinforcing steel	100 ± 25	—
		Post-tensioning ducts	120 ± 15	—
Various	Components other than those covered elsewhere in this Table	Reinforcing steel	70 ± 20§	55 ± 10§
		Pretensioning strands	—	70 ± 5§
		Post-tensioning ducts	90* ± 15§	80* ± 10§

6. Base Design:



shear at section 1, V1 (kN)	=	65.09	D/R	=	0.33
shear at section 2, V2 (kN)	=	41.28	D/R	=	0.21

15M @ 300 all faces
100mm concrete cover at bottom, 70mm other locations

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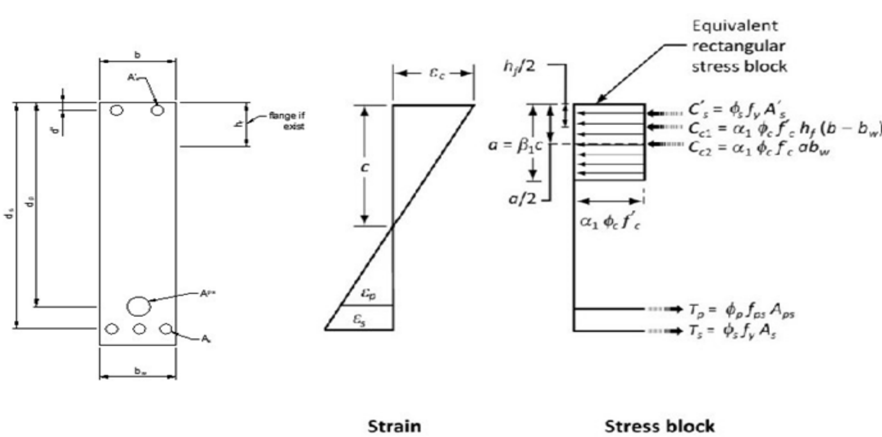
		CHECKER	Felix Wasiewicz

Wall stem flexual capacity calculation - at bottom of wall stem

	Symbols	Data	Unit	Notes
	b	1000.00	mm	Total width of flange (including web)
	b_w	1000.00	mm	Web width, when neutral axil is located in flange, bw should be taken as b (C8.8.4.1)
	h_f	0.00	mm	`
	f_c'	30.00	Mpa	Concrete strength
	α_1	0.81		α1=0.85-0.0015*fc' ≥ 0.67 (CHBDC 8.8.3)
	β_1	0.90		β1=0.97-0.0025*fc' ≥ 0.67 (CHBDC 8.8.3)
	c	19.25	mm	c = (Ts-Cc1)/α1*β1*ϕc*fc'*bw , Distance from extrem compression fibre to neutral axis, α1*ϕc*fc' is concrete stress uniformly distributed over an equivalent compression zone
	a	17.23	mm	a = β1*c , Equivalent rectangular compression zone height
	ϕ_c	0.75		Resistance factor for concrete
	C_{c1}	0.00	N	Cc1 = α1*ϕc*fc'*hf*(b-bw) Amount of compression in flange (C8.8.3)
	C_{c2}	312000.00	N	Cc2 = α1*ϕc*fc'*a*bw Amount of compression in web (C8.8.3)
	A_s	867	mm^2	Area of rebars on the flexural tension side
	f_y	400.00	Mpa	Yield strength of tensile rebar
	ϕ_s	0.90		Resistance factor of reinforcing steel
	T_s	312000.00	N	Ts = ϕs*fy*As Amount of tension in reinforcing steel
	h	600.00	mm	Overall height of beam
	d_s	492.00	mm	Distance from centroid of rebars to extrem compression fibre of concrete beam
	M_r	**150.82**	kN-m	Moment resistance, Mr = Ts*(ds-a/2) - Cc1*(hf/2-a/2) (C8.8.4.1)

Limit for Min. & Max. Reinforcement Ratios

	M_r(min.)	81	kN-m	Min. Reinforcement: Factored Mr is at least 1.2*Mcr. Mcr = fcr*I/y (CHBDC 8.8.4.3)
	I	18000000000	mm^4	Moment of inertia
	y	580.75	mm	Distance to netural axis
	f_{cr}	2.19	Mpa	Cracking strengtch for normal-density concrete 0.4*√fc'
	M_{cr}	67904934.44	N-mm	Cracking moment
	c/d (max.)	0.04		Max. Reinforcement: c/d not exceeding 0.5, (CHBDC 8.8.4.5)



Wall stem shear capacity calculation - section at bottom of wall stem

	Rmark	Symbols	Data	Unit	Notes
Section		h	600	mm	Overall thickness
		d	492	mm	Effective depth (distance from extrem compression fibre to centroid of tensile force)
		d_v	442.8	mm	0.72h - taken as the greater; 0.9d - taken as the greater
		b_v	600	mm	Effective web width, taken as minimum web witdth whin the depth dv
Vc		ϕ_c	0.75		Resistance factor for concrete
		f_c'	30	Mpa	Concrete strength
		f_{cr}	2.1908902	Mpa	Not be greater than 3.2MPa; 0.4√fc' - nomal-density concrete
		β	0.18		230/(1000+dv) without transverse reinforcement, but having max. size of coarse aggregate not less than 20mm (CHBDC 8.9.3.6)
		V_c	**196450.55**	N	Vc = 2.5*β*ϕc*fcr*bv*dv, (CHBDC 8.9.3.4)
Vs		ϴ	42	°	ϴ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.7330383	rad	Convert Dgrees to Radians
		ϕ_s	0.9		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotϴ	1.1106125		cosϴ/sinϴ
		s	100	mm	Spacing of stirups
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*cotϴ/s (CHBDC 8.9.3.5)
Vs - with inclination		ϴ	42	°	ϴ = 42° , non-prestressed, not subjected to axial tension, fy≤400MPa, fc'≤ 60MPa; Angle of inclination of the principal diagonal compressive stresses to the longitudinal axis of beam
		Radians	0.7330383	rad	Convert Dgrees to Radians
		ϕ_s	0.95		Resistance factor for rebars
		f_y	400	Mpa	Specified yield strength of rebars
		A_v	0	mm^2	Area of transverse shear reinforcement perpendicular to the axis of beam within a distance of s
		cotϴ	1.1106125		cosϴ/sinϴ
		s	200	mm	Spacing of stirups
		α	45	°	Transverse reinforcement inclined at an angle to the longitudinal axis
		Radians	0.7853982	rad	Convert Degrees to Radians
		cotα	1		cosα/sinα
		sinα	0.7071068		
		V_s	**0**	N	Vs = ϕs*fy*Av*dv*(cotϴ + cotα)*sinα/s Trans verse reinforcement inclined at an angle to the longitudinal axis, and in the direction that will intersec diagonal cracks caused by the shear (CHBDC 8.9.3.5)
Vr	Automati	Limit of (V_c + V_s)	1494450	N	Vc + Vs shall not exceed 0.25*ϕc*fc'*bv*dv
	Φ_p = 0.95	V_p	0	N	Component in the direction of the applied shear of all of the effective prestressing forces crossing the critical section factored by ϕp (taken as positive if resisting the applied shear)
	Total	V_r	**196.45**	kN	Vr = Vc +Vs+Vp

Min. Av, Vf limit requiring transverse reinforcement

		V (limit)	87311	N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25 Tcr (CHBDC 8.9.1.2)
		$0.25T_{cr}$		N	Rigions requiring transverse reinforcement. Vf is greater than (0.2*φc*fcr*bv*dv+0.5φp*Vp) and Tf is greater than 0.25*Tcr , Tcr = 0.8 * φc*fcr* (Acp2/Pc)*[1+fce/(0.8* φc *fcr)]0.5 (CHBDC 8.9.1.1)
		A_v(min.)	49	mm^2	Min. amount of transverse reinforcement: Av is not less than 0.15*fcr(bv*s/fy)