Quinte Conservation Authority

Bellrock Dam Stability Analysis and Anchor Design

H368596-0000-230-230-0001
Rev. 1
December 12, 2023

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Report

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DATE	REV.	STATUS	PREPARED BY	CHECKED BY	APPROVED BY
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1. Introduction

In 2004, Hatch Ltd. (Hatch) completed a Dam Safety Assessment (DSA) for the Bellrock Dam that recommended the use of anchors to stabilize the structure. Quinte Conservation Authority has engaged Hatch to review the stability of the structure and complete the anchor design. This report summarizes the stability review of the Bellrock Dam and the design of the anchors.

2. Stability Assessment

Stability calculations of the three concrete sections were made in accordance with MNRF's 2011 Structural Design and Factors of Safety Technical Bulletin with stability parameters adapted from the 2004 DSA (1). Figure 2-1 shows a plan and elevation view of the Bellrock Dam.

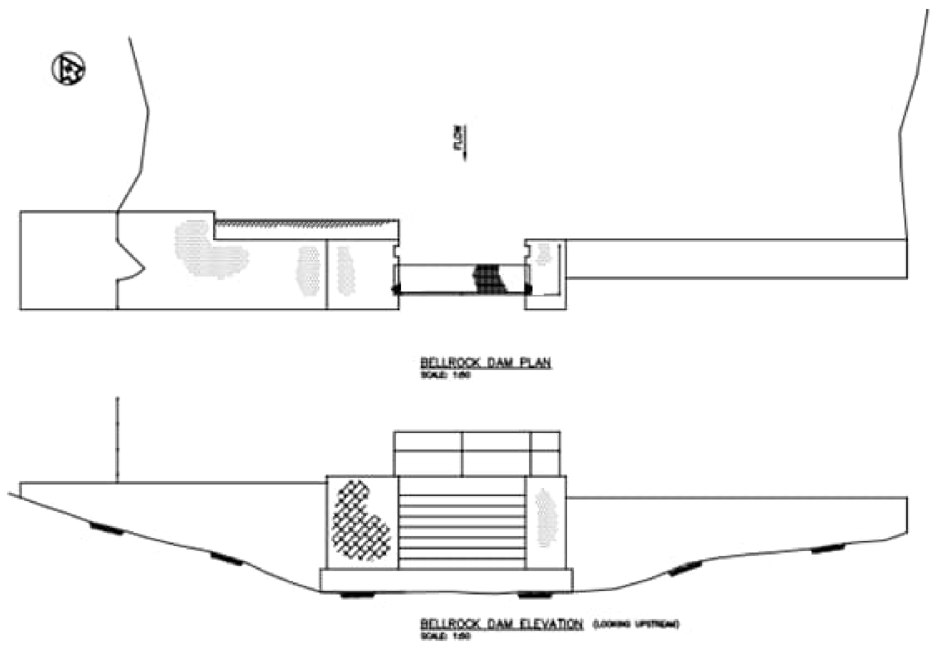


Figure 2-1: Plan and Elevation View of Bellrock Dam

2.1 Design Parameters

The design parameters and assumptions used in the standards-based stability assessment analyses of Bellrock Dam are provided in Table 2-1.

Table 2-1: Summary of Design Parameters and Assumptions Used in the Analysis of the Bellrock Dam Concrete Structures

Parameter	Value
Cohesion (Bonded Interface)	0
Friction Angle (Bonded Interface)	45 deg
Tensile Strength	0
Uplift Condition	Full Uplift
PGA[1]	5.0%*g*
Normal HWL	140.98 m
Normal TWL	139.00 m
IDF HWL	142.10 m
IDF TWL	139.97 m

[1] No earthquake analysis performed in 2004. Current value selected based on industry practicable minimum requirements.

2.1.1 Load Combinations

The following eight load combinations were used for the analysis. Load combinations are depicted in Figure 2-2.

1. Usual – Summer
2. Usual – Winter
3. Unusual – Winter
4. Unusual – IDF
5. Post-Seismic – Summer
6. Post-Seismic – Winter
7. Extreme – Summer Earthquake
8. Extreme – Winter Earthquake

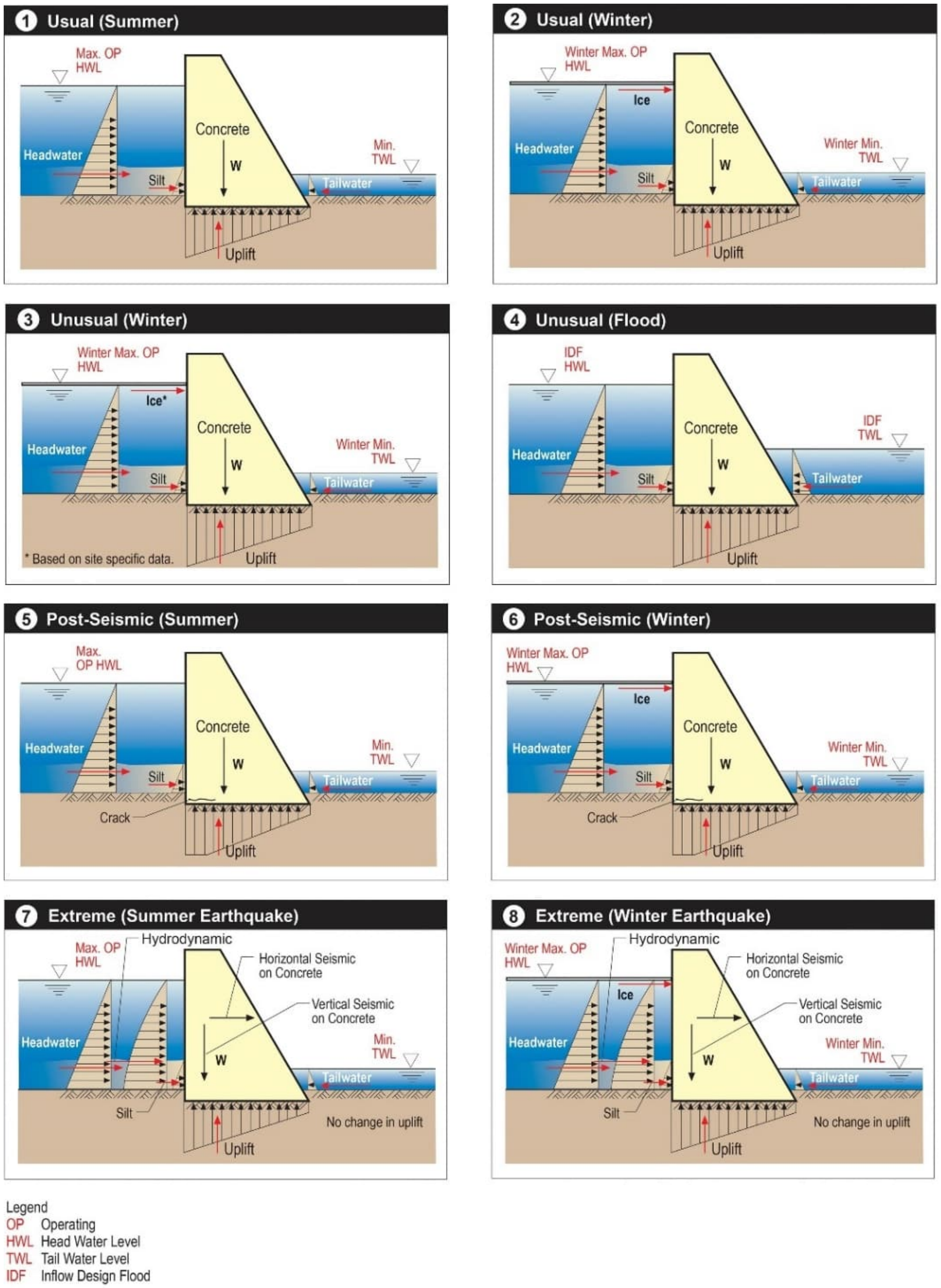


Figure 2-2: Summary of Load Combinations

2.2 Description of Loading Conditions

2.2.1 Dead Loads

Dead loads are based on the following mass densities:

1. Reinforced Concrete 2400 kg/m^3 (150 pcf)
2. Water 1000 kg/m^3 (62.4 pcf)

If soil data is unavailable, a granular backfill will be assumed for the design review with the following parameters:

1. Moist Unit Weight (bulk) 2163 kg/m^3 (135 pcf)
2. Submerged Unit Weight 1249 kg/m^3 (78 pcf)
3. Angle of Internal Friction 33°

2.2.2 Ice Loads

A report issued by the Canadian Electrical Association (CEA) Technologies, Incorporated in 2003 (2) outlines the methods used to determine both the usual (thermal only) and unusual (thermal and jacking) ice forces, this method is often referred to as the CEATI ice load model. To determine the unusual ice loads (if applicable), a review of the head pond operating regime is performed. One of the more influential aspects to the magnitude of the ice load is related to the fluctuations of the headpond during the winter season. In general, it was determined that the amplitude and frequency of the headpond fluctuations have a direct impact on the ice load felt by the dam.

The following two ice loading conditions must be considered:

1. Winter Usual Load Case: Ice load generated solely from temperature effects. This case is based on thermal ice load only and is assumed to be a linear load of 75 kN/m, as outlined in the MNRF technical bulletins. This load is typically applied 0.3 m below the water surface.
2. Winter Unusual Load Case: This case uses an ice load based on a combination of temperature effects and headpond water level changes as determined by the CEATI ice load model.

The two conclusions reached on the basis of this assessment are:

1. The Usual Ice Load (thermal) was selected to be 75 kN/m
2. The Unusual Ice Load is not considered at the structure as no headwater elevation study has been completed as part of this study.

2.2.3 Hydrostatic Loads

Headwater and tailwater pressures are assumed as a triangular distribution. Water levels used in the assessment of the various load cases are derived based on standard operation procedures, dam classification and the IDF. The cyclical nature of wave forces has been ignored in this analysis.

2.2.4 Live Loads

Live loads on bridges or pedestrian walkways are ignored in the stability analysis.

2.2.5 Snow Loads

Snow loads are not taken into account as a resisting load for the stability calculations.

2.2.6 Wind Loads

The effects of wind are ignored on all concrete structures for the analysis.

2.2.7 Uplift

Hydrostatic uplift may be considered differently depending on several factors. Specifically, uplift load can be applied in the stability analysis as follows:

1. **Condition 1**

   For dams with no foundation drains or pressure relief systems, full uplift varying linearly from 100% headwater pressure at the upstream face to 100% tailwater pressure at the downstream face is assumed to act on the entire base area of the dam.

2. **Condition 2**

   For dams equipped with an effective drainage and/or pressure relief system where there are field investigations and/or monitoring data are available, reduced uplift can be used. The reduced uplift is considered to vary from a minimum of 67% of upstream headwater pressure at the line of drainage to 100% tailwater pressure at the downstream face, provided that the actual recorded uplift is equal to or less than this assumption.

   Uplift corresponds to current water levels and does not consider ‘locked in’ pressures. If base tensions exceed allowable limits, it is assumed that cracking of the base occurs which changes the uplift pressures.

No foundation drains are known to exist at the Bellrock Dam. Therefore, **Condition 1** will be applied to the structure.

2.2.8 Earthquake Loads

A pseudostatic stability analysis is conducted for the structure. The following equations are used to determine the vertical and horizontal earthquake loads being applied to the structure.

$$PHGA = PGA * \frac{2}{3}$$

where,

PGA = peak ground acceleration
PHGA = peak horizontal ground acceleration.

$$PVGA = PHGA * \frac{2}{3}$$

where,

PVGA = peak vertical ground acceleration
PHGA = peak horizontal ground acceleration.

2.2.9 Additional Stabilizing Forces

Passive anchorage such as rock dowels increase structural stability both with respect to sliding and overturning. The dowels provide shear resistance across the concrete-rock interface as well as crack control in the event they are subject to direct tension. Post-tensioned anchors add additional normal stress thereby enhancing the frictional resistance of the sliding interface.

Remedial post-tensioned anchor forces were included in the stability assessment for the proposed designs.

2.3 Design Equations

Stability analysis for the structure has been carried out using the Gravity Method of analysis. The structure is analyzed under the desired loading conditions and a cracked base analysis is performed to estimate the length of the cracks (if any) caused by tensile stresses and an increased uplift pressure. Full uplift is applied along the length of the crack if a crack forms due to the tensile forces. The factor of safety (FOS) against sliding is calculated using the following equation.

$$FS_{sliding} = \frac{C \cdot A_c + \sum V \cdot tan\phi}{\sum H}$$

where,

FSsliding = factor of safety versus sliding (dimensionless)
C = cohesion (kPa)
Ac = base area under compression (m^2)
V = vertical forces acting on the section (kN)
ϕ = friction angle along plane being analyzed (degrees)
H = horizontal forces acting on the section (kN).

The locations of the resultant and normal stresses along the base of the structure are also critical criteria to determine the stability of the dam.

The location of the resultant acting on the plane under consideration is determined as follows:

$$a = \frac{\sum M_{st} - \sum M_{ov}}{\sum V}$$

where,

a	=	location of the resultant from the toe
$\sum M_{st}$	=	sum of stabilizing moments about toe
$\sum M_{ov}$	=	sum of overturning moments about toe.

The stresses may be computed by the following equations:

$$f_1 = -\frac{\sum V}{A}(1 - \frac{6e}{B})$$

$$f_2 = -\frac{\sum V}{A}(1 + \frac{6e}{B})$$

where,

f_1 & f_2	=	stresses acting on the plane considered at the heel and toe of the structure, respectively
A	=	area of the plane = W x B
E	=	eccentricity = $\frac{B}{2} - a$
B	=	length of base.

2.4 Standards-Based Acceptance Criteria

In accordance with the Canadian Dam Association (CDA) and the MNRF technical bulletins, adequate sliding resistance for concrete structures is normally indicated by sliding factors, equal to or exceeding the minimum values listed in Table 2-2.

Table 2-2: Minimum Sliding Factors of Safety for Concrete Structures

Load Combination	Sliding Factor (Friction Only Resistance)	Sliding Factor (Friction and Cohesion[1])	
		With Tests[2]	No Tests
Usual	1.5	2.0	3.0
Unusual	1.3	1.5	2.0
Extreme	1.1	1.3	1.5
Post-Earthquake	1.1	N/A	N/A

Notes:

1. Cohesion refers to the shear strength or adhesion of material(s) when normal stress across the prospective failure plane is zero. The failure plane under consideration can be either at the bedrock-concrete interface or at a concrete joint within the structure. Cohesion is generally determined by direct tension and/or triaxial compression tests and is measured in force per square area. Cohesion represents a shear strength or adhesion of the materials across the failure plane under consideration. Analysis based on zero cohesion shall be documented in all cases.
2. Test data refers to the laboratory tested parameters of structural or foundation materials. Adequate test data refers to testing which has taken place at the site being assessed. The higher factors of safety are reserved for sites where cohesion values are obtained based on extrapolations from testing performed at nearby sites which are considered to be representative.

Table 2-3 summarizes additional criteria that must be satisfied for the analysis of concrete structures, according to CDA and MNRF standards.

Table 2-3: Additional Acceptance Criteria

Load Case	Position of Resultant Force	Normal Compression Stress[1, 2]
Normal	Middle third of the base (100% compression)[3]	<0.3 x f'_c
Unusual	Middle half of the base	<0.5 x f'_c
Extreme	Within the base	<0.9 x f'_c
Post-Earthquake	Within the base	<0.5 x f'_c

Notes:

1. Where f'_c is the compression strength of concrete.
2. The minimum between the provided value and the bearing strength of the foundation should be used. Foundation bearing strength shall be calculated by dividing the ultimate compressive strength of the foundation by factors outlined in Table 2-2.
3. Small portion of the base is allowed to be under zero compression for existing structures as long as all other acceptance criteria is met.

2.5 Shear Strength Parameters

For the purposes of this assessment, the interface between the concrete and the bedrock was assumed to represent the critical sliding surface. Rough, unbonded shear strength parameters were assumed to exist along this interface. Estimation of these parameters was performed using Barton-Bandis theory (3) by means of a simplified approach outlined by Donnelly, 2005 (4). The results of this assessment indicated that a shear strength of 45° was available.

3. Results of Structural Assessments

3.1 Summary of 2004 Results

outlines the stability results performed in 2004. The results indicate that the gravity section satisfied the summer normal loading conditions. The sluiceway and overflow sections developed cracks at the concrete-rock interface as well as having substandard factors of safety for sliding. This means these two sections do not meet the current guidelines under summer normal conditions.

All sections developed unstable cracks on the analysis plane considered under winter ice load conditions and therefore fail to meet current guidelines.

The gravity section satisfied criteria under the 1:50-yr floor condition. The overflow and sluiceway both developed unstable cracks along the base and, therefore, do not meet requirements.

Table 3-1: Summary of 2004 Stability Results Bellrock Dam (1)

Section	Residual	Peak		Load Case	FOS Against Sliding				Location of Resultant	Minimum Base Friction Angle to Satisfy Sliding Criteria (deg)	Minimum % Bonded Area to Satisfy Peak Sliding Criteria	Notes
					Residual Case		Peak					
	Phi (deg)	c (MPa)	Phi (deg)		Req'd	Actual	Req'd	Actual				
Overflow Section	43	0.38	53	Normal	1.5	1.45	3.0	25.20	Outside middle third	44.0	6.11	2, 5
				Normal with ice	1.5	0.25	3.0	0.36	Outside	79.8	38.21	uc, 3
				Flood	1.3	0.47	2.0	14.32	Outside	68.7	7.99	uc, 3
Gravity Section	43	0.38	53	Normal	1.5	6.18	3.0	61.97	Within	12.8	0.00	1
				Normal with ice	1.5	0.78	3.0	1.11	Outside	60.9	13.44	uc, 3
				Flood	1.3	1.97	2.0	23.05	Outside middle third	31.6	1.65	1
Sluiceway	43	0.38	53	Normal	1.5	1.13	3.0	36.82	Outside middle third	51.2	1.08	2, 5
				Normal with ice	1.5	0.23	3.0	0.32	Outside	80.7	4.41	uc, 3
				Flood	1.3	0.16	2.0	20.32	Outside	82.4	1.48	uc, 3

Notes:
uc = unstable crack
Note 1 = dam section satisfies dam safety criteria.
Note 2 = dam section satisfies dam safety criteria under peak strength assumptions.
Note 3 = dam section deemed to satisfy dam safety criteria for low hazard dams [Figure 7.1, Note (f) of the draft ODSG].
Note 4 = bearing stress at toe of dam exceeds criteria.
Note 5 = position of resultant does not satisfy criteria.
Note 6 = does not satisfy dam safety criteria for sliding stability.
Note 7 = rock anchor taken into account.

3.2 2022 Stability Results with Anchors

One of the most efficient means of addressing stability deficiencies is through the installation of post-tensioned anchors. Typically, these high-strength steel bars are drilled vertically into the bedrock foundation and tensioned to provide a beneficial stabilizing force. Specific details are incorporated into the design to ensure corrosion protection, thereby ensuring a design life of many decades.

3.2.1 Post-Tensioning Force Requirements

In order to meet the stability requirements, the following anchor loads have been applied to the structures:

1. 204 kN/m applied to the overflow section.
2. 151 kN/m applied to the gravity section.
3. 550 kN applied to each pier of the spillway section.

The following sections outline the calculated results for concrete sliding assuming the implementation of the post-tensioned anchors.

3.2.2 Overflow Section

The results of the 2022 standards-based assessment for the overflow section are summarized in Table 3-2.

Table 3-2: Standards-Based Stability Results – Overflow Section

Loading Case			Normal		Unusual	Post Seismic		Extreme	
			Case 1	Case 2	Case 4	Case 5	Case 6	Case 7	Case 8
Factor of Safety For Sliding		Required	1.50	1.50	1.30	1.30	1.30	1.10	1.10
		Computed	10.54	2.43	5.92	10.54	2.43	9.37	2.40
Stress (kPa) -ve : Compression +ve : Tension	At Heel	Computed	-275.55	0.00	-204.63	-275.55	0.00	-266.27	0.00
	At Toe	Computed	-75.28	-653.42	-136.99	-75.28	-653.41	-82.72	-667.24
	Allowable		-2000	-2000	-2308	-2308	-2308	-2727	-2727
Position of Resultant from Toe (m)			0.833	0.244	0.746	0.833	0.244	0.823	0.238
Location of Resultant Within Dam Base			1/3	Base	1/3	1/3	Base	1/3	Base
Required Location of Resultant			1/3	1/3	1/2	Base	Base	Base	Base

The stability of the section along the assumed sliding surface met or exceeded the recommendations of the MNRF guidelines for all load combinations. The calculated FOS for sliding are above the required values. In addition, all other performance indicators are met for all loading combinations analyzed. The location of the resultant is located outside of the middle third of the dam, this is deemed acceptable under the MNRF LRIA technical bulletins if the FOS is sufficient and all other performance indicators are met. Detailed calculations can be found in Appendix A.

3.2.3 Gravity Section

The results of the 2022 standards-based assessment for the gravity section are summarized in Table 3-3.

Table 3-3: Standards-Based Stability Results – Gravity Section

Loading Case			Normal		Unusual	Post Seismic		Extreme	
			Case 1	Case 2	Case 4	Case 5	Case 6	Case 7	Case 8
Factor of Safety For Sliding		Required	1.50	1.50	1.30	1.30	1.30	1.10	1.10
		Computed	9.94	2.46	5.38	9.94	2.45	8.16	2.40
Stress (kPa) -ve : Compression +ve : Tension	At Heel	Computed	-151.34	0.00	-105.50	-151.34	0.00	-141.61	0.00
	At Toe	Computed	-86.47	-302.10	-128.36	-86.47	-302.09	-93.53	-307.81
	Allowable		-2000	-2000	-2308	-2308	-2308	-2727	-2727
Position of Resultant from Toe (m)			1.151	0.543	1.021	1.151	0.543	1.127	0.531
Location of Resultant Within Dam Base			1/3	1/2	1/3	1/3	1/2	1/3	1/2
Required Location of Resultant			1/3	1/3	1/2	Base	Base	Base	Base

The stability of the section along the assumed sliding surface met or exceeded the recommendations of the MNRF guidelines for all load combinations. The calculated FOS for sliding are above the required values. In addition, all other performance indicators are met for all loading combinations analyzed. The location of the resultant is located outside of the middle third of the dam, this is deemed acceptable under the MNRF LRIA technical bulletins if the FOS is sufficient and all other performance indicators are met. Detailed calculations can be found in Appendix A

3.2.4 Spillway

The results of the 2022 standards-based assessment for the spillway section are summarized in Table 3-4.

Table 3-4: Standards-Based Stability Results – Spillway Section

Loading Case			Normal		Unusual	Post Seismic		Extreme	
			Case 1	Case 2	Case 4	Case 5	Case 6	Case 7	Case 8
Factor of Safety For Sliding		Required	1.50	1.50	1.30	1.30	1.30	1.10	1.10
		Computed	10.68	2.64	5.77	10.68	2.63	9.50	2.60
Stress (kPa) -ve : Compression +ve : Tension	At Heel	Computed	-166.42	0.00	-129.52	-166.42	0.00	-162.41	0.00
	At Toe	Computed	-67.79	-242.31	-96.93	-67.79	-242.30	-70.59	-243.56
	Allowable		-2000	-2000	-2308	-2308	-2308	-2727	-2727
Position of Resultant from Toe (m)			1.283	0.723	1.179	1.283	0.722	1.273	0.718
Location of Resultant Within Dam Base			1/3	1/2	1/3	1/3	1/2	1/3	1/2
Required Location of Resultant			1/3	1/3	1/2	Base	Base	Base	Base

The stability of the section along the assumed sliding surface met or exceeded the recommendations of the MNRF guidelines for all load combinations. The calculated FOS for sliding are above the required values. In addition, all other performance indicators are met for all loading combinations analyzed. The resultant is located outside of the middle third of the dam; this is deemed acceptable under the MNRF LRIA technical bulletins if the FOS is sufficient and all other performance indicators are met. Detailed calculations can be found in Appendix A.

4. Post–Tensioning Anchor Design Summary

As a result of the stability analysis, post-tensioned anchors are required in order to achieve the required stability criteria. The estimated post-tensioning forces from stability analyses have been used to calculate the type, size, length, and spacing for each section.

All anchors have a design load of 550 kN. The threaded bar anchors have a nominal diameter of 36 mm and are to be Dywidag hot-rolled threadbar (or similar) of grade ASTMA722. The anchors must have a minimum bond length of 3 m in bedrock and a minimum free stressing length of 3 m. Accordingly, the spacing of the anchors is as follows:

- 2700 mm for the overflow section
- 3600 mm for the gravity section
- 1 anchor per pier in the spillway section.

Detailed anchor calculations are included in Appendix B. The anchor layout and general anchor details are presented in Appendix C (DWG H368596-0000-220-270-0001).

5. References

1. **Acres International.** *Final Report - Bellrock Dam - Napanee Watershed.* 2004.

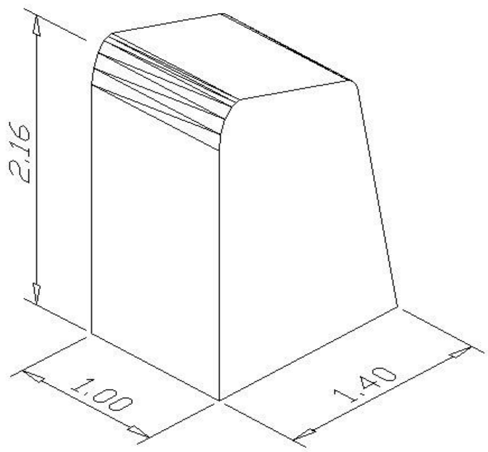
2. **CEA Technologies.** *Static Ice Loads on Hydro-electric Structures Report T002700-0206.* August 2003.

3. **Bandis, S. C.** *Scale Effects in the Shear Strength and Deformability of Rock and Rock Joints.* Rotterdam : s.n., 1990.

4. *General Report Q97 - Spillways .* **Donnelly, C. Richard.** s.l. : ICOLD, 2015.

5. **Canadian Dam Association (CDA).** *2007 Dam Safety Guidelines (2013 update).* 2013.

Appendix A
Stability Calculations



Geometrical Definitions

Base Elevation	138.790	m
Log Top Elevation (Summer)	140.950	m
H.W.L. (Summer)	140.980	m
T.W.L. (Summer)	139.000	m
Log Top Elevation (Winter)	140.950	m
H.W.L. (Winter)	140.980	m
T.W.L. (Winter)	139.000	m
Log Top Elevation (Flood I)	140.950	m
H.W.L. (Flood I)	142.100	m
T.W.L. (Flood I)	139.970	m
Log Top Elevation (Flood II)	140.950	m
H.W.L. (Flood II)	142.100	m
T.W.L. (Flood II)	139.970	m
Deck Top Elevation	140.950	m
Thickness of Deck	0.000	m
Ice Elevation	140.680	m
Volume of Section	2.46	m^3
Centre of Gravity X	0.794	m
Centre of Gravity Y	0.962	m
Length of Pier Section	1.400	m
Width of Pier Section	1.000	m
Length of Sluiceway #1 Section	0.000	m
Width of Sluiceway #1 Section	0.000	m
Distance to Edge of Sluiceway #1 Section	0.000	m
Length of Sluiceway #2 Section	0.000	m
Width of Sluiceway #2 Section	0.000	m
Distance to Edge of Sluiceway #2 Section	0.000	m

Material Properties

f_c'	20.00	MPa	Concrete Compressive Strength
f_{b1}	3.00	MPa	Rock Bearing Strength
f_{b2}	0.00	MPa	Till Bearing Strength
ϕ_1	35.0	°	Angle of Friction #1
ϕ_2	40.0	°	Angle of Friction #2
ϕ_3	45.0	°	Specified Angle of Sliding Friction
ϕ_4	50.0	°	Angle of Friction #4
ϕ_5	55.0	°	Angle of Friction #5
τ_n	0.00	MPa	Cohesion
τ_1	0.38	MPa	$(0.17\sqrt{f_c'})/2$
τ_2	0.76	MPa	$(0.17\sqrt{f_c'})$
τ_3	1.00	MPa	$(0.05f_c')$
γ_{conc}	23.50	kN/m^3	Unit Weight of Concrete
γ_{water}	9.81	kN/m^3	Unit Weight of Water
ϕ_β	35.0	°	Basic Friction Angle

Loadings

2.22	%g	Vertical Ground Acceleration (Summer)
3.33	%g	Horizontal Ground Acceleration (Summer)
0.89	%g	Vertical Ground Acceleration (Winter, DEice)
1.33	%g	Horizontal Ground Acceleration (Winter, DEice)
75	kN/m	Ice Force on Concrete
75	kN/m	Ice Force on Logs/Gates

Input Summary

	Load Case									
	#1	#2	#3 (Sum)	#3 (Win)	#4	#5 (Sum)	#5 (Win)	#6		
M_1	57.9	57.9	57.9	57.9	57.9	57.9	57.9	57.9	kN	Weight of Section
V_{water}	0.02	0.02	0.02	0.02	0.83	0.02	0.02	0.83	m^3	Volume of Water Over Section
M_2	0.20	0.20	0.20	0.20	8.10	0.20	0.20	8.10	kN	Weight of Water Over Section
x	1.12	1.12	1.12	1.12	0.90	1.12	1.12	0.90	m	Location of Water Force Along X-Axis
ICE	-	75.00	-	75.00	-	-	75.00	-	kN	Total Ice Force
y	-	1.89	-	1.89	-	-	1.89	-	m	Location of Ice Force Along Y-Axis
W	-	-	-	-	-	0.84	0.33	-	kN	Westergaards Force
y	-	-	-	-	-	0.90	0.90	-	m	Location of Westergaards along Y-Axis
S_H	-	-	-	-	-	3.33	1.33	-	%g	Horizontal Seismic Coefficient
S_V	-	-	-	-	-	2.22	0.89	-	%g	Vertical Seismic Coefficient
w_1	23.52	23.52	23.52	23.52	47.25	23.52	23.52	47.25	kN	Hydrostatic Pressure From Headwater
y	0.73	0.73	0.73	0.73	0.91	0.73	0.73	0.91	m	Location of Headwater Force Along Y-Axis
w_2	0.22	0.22	0.22	0.22	6.83	0.22	0.22	6.83	kN	Hydrostatic Pressure From Tailwater
y	0.07	0.07	0.07	0.07	0.39	0.07	0.07	0.39	m	Location of Tailwater Force Along Y-Axis
H_1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	kN	Other Horizontal Force
y	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	m	Location of Other Horizontal Force Along Y-Axis
V_1	204.00	204.00	204.00	204.00	204.00	204.00	204.00	204.00	kN	Other Vertical Force
x	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	m	Location of Other Vertical Force Along X-Axis

Results

		Load Case #1 - Usual (Summer)				Load Case #2 - Usual (Winter)			
Cohesion	MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	16.48	16.48	16.48	16.48	22.97	21.37	16.48	16.48
Effective Base	%	100.0	100.0	100.0	100.0	52.3	64.0	100.0	100.0
Length of Base in Compression	m	1.40	1.40	1.40	1.40	0.73	0.90	1.40	1.40
Resultant	m	0.833	0.833	0.833	0.833	0.244	0.246	0.256	0.256
Stress at Heel	kPa	-275.55	-275.55	-275.55	-275.55	0.00	0.00	158.17	158.17
Cracked		NO	NO	NO	NO	YES	YES	NO	NO
Stress at Toe	kPa	-75.28	-75.28	-75.28	-75.28	-653.42	-632.05	-509.00	-509.00
Allowable Stress at Toe	kPa	-2000	-1500	-1500	-1500	-2000	-1500	-1500	-1500
F.S. Overturning		7.42	7.42	7.42	7.42	1.33	1.33	1.36	1.36
F.S. Sliding ϕ=	35	7.38	30.22	53.05	67.45	1.70	4.73	10.01	12.61
F.S. Sliding ϕ=	40	8.84	31.68	54.52	68.92	2.04	5.07	10.36	12.96
F.S. Sliding ϕ=	45	10.54	33.37	56.21	70.61	2.43	5.46	10.76	13.36
F.S. Sliding ϕ=	50	12.56	35.40	58.23	72.63	2.90	5.93	11.24	13.84
F.S. Sliding ϕ=	55	15.05	37.89	60.72	75.12	3.47	6.51	11.83	14.43
Accepted F.S. Sliding		1.50	2.00	2.00	2.00	1.50	2.00	2.00	2.00

		Load Case #4 - Flood I				Load Case #6 - Flood II			
Cohesion	MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	30.83	30.83	30.83	30.83	-13.04	-14.09	30.83	30.83
Effective Base	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Length of Base in Compression	m	1.40	1.40	1.40	1.40	5.60	5.70	1.40	1.40
Resultant	m	0.746	0.746	0.746	0.746	0.992	1.002	0.746	0.746
Stress at Heel	kPa	-204.63	-204.63	-204.63	-204.63	47.35	47.08	-204.63	-204.63
Cracked		NO	NO	NO	NO	NO	NO	NO	NO
Stress at Toe	kPa	-136.99	-136.99	-136.99	-136.99	-148.44	-146.75	-136.99	-136.99
Allowable Stress at Toe	kPa	-2308	-2000	-2000	-2000	-2308	-2000	-2000	-2000
F.S. Overturning		3.63	3.63	3.63	3.63	-7.12	-6.39	3.63	3.63
F.S. Sliding ϕ=	35	4.14	17.31	30.47	38.78	4.90	45.50	30.47	38.78
F.S. Sliding ϕ=	40	4.96	18.13	31.29	39.60	5.87	46.48	31.29	39.60
F.S. Sliding ϕ=	45	5.92	19.08	32.25	40.55	7.00	47.61	32.25	40.55
F.S. Sliding ϕ=	50	7.05	20.22	33.38	41.68	8.34	48.95	33.38	41.68
F.S. Sliding ϕ=	55	8.45	21.61	34.78	43.08	10.00	50.62	34.78	43.08
Accepted F.S. Sliding		1.30	1.50	1.50	1.50	1.30	1.50	1.50	1.50

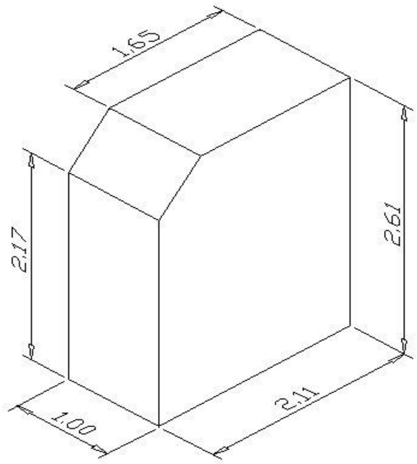
Input Summary

	Load Case									
	#1	#2	#3 (Sum)	#3 (Win)	#4	#5 (Sum)	#5 (Win)	#6		
M_1	57.9	57.9	57.9	57.9	57.9	57.9	57.9	57.9	kN	Weight of Section
V_{water}	0.02	0.02	0.02	0.02	0.83	0.02	0.02	0.83	m^3	Volume of Water Over Section
M_2	0.20	0.20	0.20	0.20	8.10	0.20	0.20	8.10	kN	Weight of Water Over Section
x	1.12	1.12	1.12	1.12	0.90	1.12	1.12	0.90	m	Location of Water Force Along X-Axis
ICE	-	75.00	-	75.00	-	-	75.00	-	kN	Total Ice Force
y	-	1.89	-	1.89	-	-	1.89	-	m	Location of Ice Force Along Y-Axis
W	-	-	-	-	-	0.84	0.33	-	kN	Westergaards Force
y	-	-	-	-	-	0.90	0.90	-	m	Location of Westergaards along Y-Axis
S_H	-	-	-	-	-	3.33	1.33	-	%g	Horizontal Seismic Coefficient
S_V	-	-	-	-	-	2.22	0.89	-	%g	Vertical Seismic Coefficient
w_1	23.52	23.52	23.52	23.52	47.25	23.52	23.52	47.25	kN	Hydrostatic Pressure From Headwater
y	0.73	0.73	0.73	0.73	0.91	0.73	0.73	0.91	m	Location of Headwater Force Along Y-Axis
w_2	0.22	0.22	0.22	0.22	6.83	0.22	0.22	6.83	kN	Hydrostatic Pressure From Tailwater
y	0.07	0.07	0.07	0.07	0.39	0.07	0.07	0.39	m	Location of Tailwater Force Along Y-Axis
H_1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	kN	Other Horizontal Force
y	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	m	Location of Other Horizontal Force Along Y-Axis
V_1	204.00	204.00	204.00	204.00	204.00	204.00	204.00	204.00	kN	Other Vertical Force
x	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	m	Location of Other Vertical Force Along X-Axis

Results

		Load Case #3 - Post-Earthquake (Summer)				Load Case #3 - Post-Earthquake (Winter)			
Cohesion	MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	16.48	16.48	16.48	16.48	23.13	21.37	16.48	16.48
Effective Base	%	100.0	100.0	100.0	100.0	52.2	64.0	100.0	100.0
Length of Base in Compression	m	1.40	1.40	1.40	1.40	0.73	0.90	1.40	1.40
Resultant	m	0.833	0.833	0.833	0.833	0.244	0.246	0.256	0.256
Stress at Heel	kPa	-275.55	-275.55	-275.55	-275.55	0.00	0.00	158.17	158.17
Cracked		NO	NO	NO	NO	YES	YES	NO	NO
Crack Propagated		NO	NO	NO	NO	NO	NO	NO	NO
Stress at Toe	kPa	-75.28	-75.28	-75.28	-75.28	-653.41	-632.05	-509.00	-509.00
Allowable Stress at Toe	kPa	-2308	-2000	-2000	-2000	-2308	-2000	-2000	-2000
F.S. Overturning		7.42	7.42	7.42	7.42	1.33	1.33	1.36	1.36
F.S. Sliding ϕ= 35		7.38	30.22	53.05	67.45	1.70	4.73	10.01	12.61
F.S. Sliding ϕ= 40		8.84	31.68	54.52	68.92	2.04	5.07	10.36	12.96
F.S. Sliding ϕ= 45		10.54	33.37	56.21	70.61	2.43	5.46	10.76	13.36
F.S. Sliding ϕ= 50		12.56	35.40	58.23	72.63	2.90	5.93	11.24	13.84
F.S. Sliding ϕ= 55		15.05	37.89	60.72	75.12	3.47	6.51	11.83	14.43
Accepted F.S. Sliding		1.10	1.50	1.50	1.50	1.10	1.50	1.50	1.50

		Load Case #5 - Earthquake (Summer)				Load Case #5 - Earthquake (Winter)			
Cohesion	MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	16.48	16.48	16.48	16.48	22.97	21.37	16.48	16.48
Effective Base	%	100.0	100.0	100.0	100.0	51.1	64.0	100.0	100.0
Length of Base in Compression	m	1.40	1.40	1.40	1.40	0.72	0.90	1.40	1.40
Resultant	m	0.823	0.823	0.823	0.823	0.238	0.240	0.251	0.251
Stress at Heel	kPa	-266.27	-266.27	-266.27	-266.27	0.00	0.00	161.89	161.89
Cracked		NO	NO	NO	NO	YES	YES	NO	NO
Crack Propagated		NO	NO	NO	NO	YES	NO	NO	NO
Stress at Toe	kPa	-82.72	-82.72	-82.72	-82.72	-667.24	-640.60	-511.98	-511.98
Allowable Stress at Toe	kPa	-2727	-2308	-2308	-2308	-2727	-2308	-2308	-2308
F.S. Overturning		6.68	6.68	6.68	6.68	1.32	1.32	1.35	1.35
F.S. Sliding ϕ= 35		6.56	26.98	47.39	60.26	1.68	4.64	9.86	12.43
F.S. Sliding ϕ= 40		7.86	28.28	48.69	61.56	2.01	4.97	10.20	12.77
F.S. Sliding ϕ= 45		9.37	29.78	50.20	63.07	2.40	5.36	10.60	13.16
F.S. Sliding ϕ= 50		11.17	31.58	52.00	64.87	2.86	5.83	11.07	13.64
F.S. Sliding ϕ= 55		13.38	33.80	54.21	67.08	3.43	6.40	11.66	14.22
Accepted F.S. Sliding		1.10	1.30	1.30	1.30	1.10	1.30	1.30	1.30



Geometrical Definitions

Base Elevation	138.690	m
Log Top Elevation (Summer)	141.300	m
H.W.L. (Summer)	140.980	m
T.W.L. (Summer)	139.000	m
Log Top Elevation (Winter)	141.300	m
H.W.L. (Winter)	140.980	m
T.W.L. (Winter)	139.000	m
Log Top Elevation (Flood I)	141.300	m
H.W.L. (Flood I)	142.100	m
T.W.L. (Flood I)	139.970	m
Log Top Elevation (Flood II)	141.300	m
H.W.L. (Flood II)	142.100	m
T.W.L. (Flood II)	139.970	m
Deck Top Elevation	141.300	m
Thickness of Deck	0.000	m
Ice Elevation	140.680	m
Volume of Section	5.40	m^3
Centre of Gravity X	1.038	m
Centre of Gravity Y	1.281	m
Length of Pier Section	2.110	m
Width of Pier Section	1.000	m
Length of Sluiceway #1 Section	0.000	m
Width of Sluiceway #1 Section	0.000	m
Distance to Edge of Sluiceway #1 Section	0.000	m
Length of Sluiceway #2 Section	0.000	m
Width of Sluiceway #2 Section	0.000	m
Distance to Edge of Sluiceway #2 Section	0.000	m

Material Properties

f_c'	20.00	MPa	Concrete Compressive Strength
f_{b1}	3.00	MPa	Rock Bearing Strength
f_{b2}	0.00	MPa	Till Bearing Strength
ϕ_1	35.0	°	Angle of Friction #1
ϕ_2	40.0	°	Angle of Friction #2
ϕ_3	45.0	°	Specified Angle of Sliding Friction
ϕ_4	50.0	°	Angle of Friction #4
ϕ_5	55.0	°	Angle of Friction #5
τ_n	0.00	MPa	Cohesion
τ_1	0.38	MPa	$(0.17\sqrt{f_c'})/2$
τ_2	0.76	MPa	$(0.17\sqrt{f_c'})$
τ_3	1.00	MPa	$(0.05f_c')$
γ_{conc}	23.50	kN/m^3	Unit Weight of Concrete
γ_{water}	9.81	kN/m^3	Unit Weight of Water
ϕ_β	35.0	°	Basic Friction Angle

Loadings

2.22	%g	Vertical Ground Acceleration (Summer)
3.33	%g	Horizontal Ground Acceleration (Summer)
0.89	%g	Vertical Ground Acceleration (Winter, DEice)
1.33	%g	Horizontal Ground Acceleration (Winter, DEice)
75	kN/m	Ice Force on Concrete
75	kN/m	Ice Force on Logs/Gates

Input Summary

	Load Case									
	#1	#2	#3 (Sum)	#3 (Win)	#4	#5 (Sum)	#5 (Win)	#6		
M_1	126.8	126.8	126.8	126.8	126.8	126.8	126.8	126.8	kN	Weight of Section
V_{water}	0.00	0.00	0.00	0.00	1.78	0.00	0.00	1.78	m^3	Volume of Water Over Section
M_2	0.00	0.00	0.00	0.00	17.46	0.00	0.00	17.46	kN	Weight of Water Over Section
x	0.00	0.00	0.00	0.00	1.10	0.00	0.00	1.10	m	Location of Water Force Along X-Axis
ICE	-	75.00	-	75.00	-	-	75.00	-	kN	Total Ice Force
y	-	1.99	-	1.99	-	-	1.99	-	m	Location of Ice Force Along Y-Axis
W	-	-	-	-	-	0.92	0.37	-	kN	Westergaards Force
y	-	-	-	-	-	0.94	0.94	-	m	Location of Westergaards along Y-Axis
S_H	-	-	-	-	-	3.33	1.33	-	%g	Horizontal Seismic Coefficient
S_V	-	-	-	-	-	2.22	0.89	-	%g	Vertical Seismic Coefficient
w_1	25.72	25.72	25.72	25.72	53.90	25.72	25.72	53.90	kN	Hydrostatic Pressure From Headwater
y	0.76	0.76	0.76	0.76	1.04	0.76	0.76	1.04	m	Location of Headwater Force Along Y-Axis
w_2	0.47	0.47	0.47	0.47	8.04	0.47	0.47	8.04	kN	Hydrostatic Pressure From Tailwater
y	0.10	0.10	0.10	0.10	0.43	0.10	0.10	0.43	m	Location of Tailwater Force Along Y-Axis
H_1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	kN	Other Horizontal Force
y	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	m	Location of Other Horizontal Force Along Y-Axis
V_1	151.00	151.00	151.00	151.00	151.00	151.00	151.00	151.00	kN	Other Vertical Force
x	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	m	Location of Other Vertical Force Along X-Axis

Results

		Load Case #1 - Usual (Summer)				Load Case #2 - Usual (Winter)			
Cohesion	MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	26.91	26.91	26.91	26.91	31.57	26.91	26.91	26.91
Effective Base	%	100.0	100.0	100.0	100.0	77.3	100.0	100.0	100.0
Length of Base in Compression	m	2.11	2.11	2.11	2.11	1.63	2.11	2.11	2.11
Resultant	m	1.151	1.151	1.151	1.151	0.543	0.556	0.556	0.556
Stress at Heel	kPa	-151.34	-151.34	-151.34	-151.34	0.00	49.67	49.67	49.67
Cracked		NO	NO	NO	NO	YES	NO	NO	NO
Stress at Toe	kPa	-86.47	-86.47	-86.47	-86.47	-302.10	-287.48	-287.48	-287.48
Allowable Stress at Toe	kPa	-2000	-1500	-1500	-1500	-2000	-1500	-1500	-1500
F.S. Overturning		6.23	6.23	6.23	6.23	1.64	1.68	1.68	1.68
F.S. Sliding ϕ= 35		6.96	38.72	70.49	90.52	1.72	8.57	15.40	19.70
F.S. Sliding ϕ= 40		8.34	40.10	71.87	91.90	2.06	8.92	15.74	20.05
F.S. Sliding ϕ= 45		9.94	41.70	73.46	93.50	2.46	9.32	16.15	20.45
F.S. Sliding ϕ= 50		11.84	43.61	75.37	95.40	2.93	9.80	16.63	20.93
F.S. Sliding ϕ= 55		14.19	45.95	77.72	97.75	3.51	10.40	17.22	21.52
Accepted F.S. Sliding		1.50	2.00	2.00	2.00	1.50	2.00	2.00	2.00

		Load Case #4 - Flood I				Load Case #6 - Flood II			
Cohesion	MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	48.54	48.54	48.54	48.54	48.54	11.04	48.54	48.54
Effective Base	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Length of Base in Compression	m	2.11	2.11	2.11	2.11	2.11	5.70	2.11	2.11
Resultant	m	1.021	1.021	1.021	1.021	1.021	1.229	1.021	1.021
Stress at Heel	kPa	-105.50	-105.50	-105.50	-105.50	-105.50	35.19	-105.50	-105.50
Cracked		NO	NO	NO	NO	NO	NO	NO	NO
Stress at Toe	kPa	-128.36	-128.36	-128.36	-128.36	-128.36	-134.93	-128.36	-128.36
Allowable Stress at Toe	kPa	-2308	-2000	-2000	-2000	-2308	-2000	-2000	-2000
F.S. Overturning		3.19	3.19	3.19	3.19	3.19	21.40	3.19	3.19
F.S. Sliding ϕ= 35		3.77	21.26	38.75	49.78	3.77	41.81	38.75	49.78
F.S. Sliding ϕ= 40		4.51	22.00	39.49	50.52	4.51	42.67	39.49	50.52
F.S. Sliding ϕ= 45		5.38	22.87	40.36	51.39	5.38	43.67	40.36	51.39
F.S. Sliding ϕ= 50		6.41	23.90	41.39	52.42	6.41	44.86	41.39	52.42
F.S. Sliding ϕ= 55		7.68	25.17	42.66	53.69	7.68	46.32	42.66	53.69
Accepted F.S. Sliding		1.30	1.50	1.50	1.50	1.30	1.50	1.50	1.50

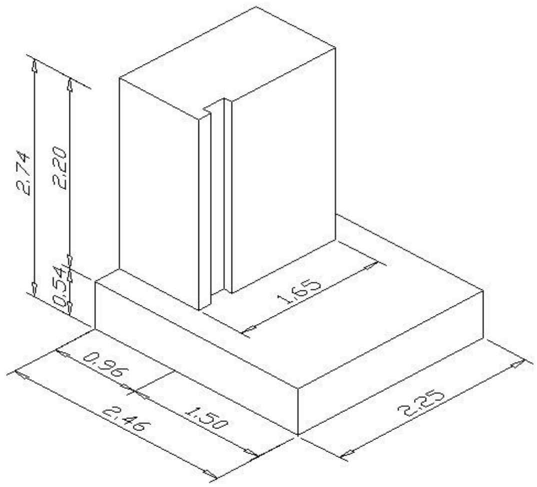
Input Summary

	Load Case									
	#1	#2	#3 (Sum)	#3 (Win)	#4	#5 (Sum)	#5 (Win)	#6		
M_1	126.8	126.8	126.8	126.8	126.8	126.8	126.8	126.8	kN	Weight of Section
V_{water}	0.00	0.00	0.00	0.00	1.78	0.00	0.00	1.78	m^3	Volume of Water Over Section
M_2	0.00	0.00	0.00	0.00	17.46	0.00	0.00	17.46	kN	Weight of Water Over Section
x	0.00	0.00	0.00	0.00	1.10	0.00	0.00	1.10	m	Location of Water Force Along X-Axis
ICE	-	75.00	-	75.00	-	-	75.00	-	kN	Total Ice Force
y	-	1.99	-	1.99	-	-	1.99	-	m	Location of Ice Force Along Y-Axis
W	-	-	-	-	-	0.92	0.37	-	kN	Westergaards Force
y	-	-	-	-	-	0.94	0.94	-	m	Location of Westergaards along Y-Axis
S_H	-	-	-	-	-	3.33	1.33	-	%g	Horizontal Seismic Coefficient
S_V	-	-	-	-	-	2.22	0.89	-	%g	Vertical Seismic Coefficient
w_1	25.72	25.72	25.72	25.72	53.90	25.72	25.72	53.90	kN	Hydrostatic Pressure From Headwater
y	0.76	0.76	0.76	0.76	1.04	0.76	0.76	1.04	m	Location of Headwater Force Along Y-Axis
w_2	0.47	0.47	0.47	0.47	8.04	0.47	0.47	8.04	kN	Hydrostatic Pressure From Tailwater
y	0.10	0.10	0.10	0.10	0.43	0.10	0.10	0.43	m	Location of Tailwater Force Along Y-Axis
H_1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	kN	Other Horizontal Force
y	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	m	Location of Other Horizontal Force Along Y-Axis
V_1	151.00	151.00	151.00	151.00	151.00	151.00	151.00	151.00	kN	Other Vertical Force
x	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	m	Location of Other Vertical Force Along X-Axis

Results

			Load Case #3 - Post-Earthquake (Summer)				Load Case #3 - Post-Earthquake (Winter)			
Cohesion		MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face		%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift		kN	26.91	26.91	26.91	26.91	31.93	26.91	26.91	26.91
Effective Base		%	100.0	100.0	100.0	100.0	77.1	100.0	100.0	100.0
Length of Base in Compression		m	2.11	2.11	2.11	2.11	1.63	2.11	2.11	2.11
Resultant		m	1.151	1.151	1.151	1.151	0.543	0.556	0.556	0.556
Stress at Heel		kPa	-151.34	-151.34	-151.34	-151.34	0.00	49.67	49.67	49.67
Cracked			NO	NO	NO	NO	YES	NO	NO	NO
Crack Propagated			NO	NO	NO	NO	NO	NO	NO	NO
Stress at Toe		kPa	-86.47	-86.47	-86.47	-86.47	-302.09	-287.48	-287.48	-287.48
Allowable Stress at Toe		kPa	-2308	-2000	-2000	-2000	-2308	-2000	-2000	-2000
F.S. Overturning			6.23	6.23	6.23	6.23	1.63	1.68	1.68	1.68
F.S. Sliding ϕ=	35		6.96	38.72	70.49	90.52	1.72	8.57	15.40	19.70
F.S. Sliding ϕ=	40		8.34	40.10	71.87	91.90	2.06	8.92	15.74	20.05
F.S. Sliding ϕ=	45		9.94	41.70	73.46	93.50	2.45	9.32	16.15	20.45
F.S. Sliding ϕ=	50		11.84	43.61	75.37	95.40	2.92	9.80	16.63	20.93
F.S. Sliding ϕ=	55		14.19	45.95	77.72	97.75	3.50	10.40	17.22	21.52
Accepted F.S. Sliding			1.10	1.50	1.50	1.50	1.10	1.50	1.50	1.50

			Load Case #5 - Earthquake (Summer)				Load Case #5 - Earthquake (Winter)			
Cohesion		MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face		%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift		kN	26.91	26.91	26.91	26.91	31.57	26.91	26.91	26.91
Effective Base		%	100.0	100.0	100.0	100.0	75.5	100.0	100.0	100.0
Length of Base in Compression		m	2.11	2.11	2.11	2.11	1.59	2.11	2.11	2.11
Resultant		m	1.127	1.127	1.127	1.127	0.531	0.544	0.544	0.544
Stress at Heel		kPa	-141.61	-141.61	-141.61	-141.61	0.00	53.56	53.56	53.56
Cracked			NO	NO	NO	NO	YES	NO	NO	NO
Crack Propagated			NO	NO	NO	NO	YES	NO	NO	NO
Stress at Toe		kPa	-93.53	-93.53	-93.53	-93.53	-307.81	-290.30	-290.30	-290.30
Allowable Stress at Toe		kPa	-2727	-2308	-2308	-2308	-2727	-2308	-2308	-2308
F.S. Overturning			5.37	5.37	5.37	5.37	1.61	1.65	1.65	1.65
F.S. Sliding ϕ=	35		5.72	32.11	58.50	75.14	1.68	8.33	14.95	19.12
F.S. Sliding ϕ=	40		6.85	33.24	59.63	76.27	2.01	8.67	15.29	19.46
F.S. Sliding ϕ=	45		8.16	34.55	60.94	77.59	2.40	9.06	15.68	19.85
F.S. Sliding ϕ=	50		9.73	36.12	62.51	79.15	2.86	9.53	16.15	20.32
F.S. Sliding ϕ=	55		11.66	38.05	64.44	81.08	3.42	10.11	16.72	20.90
Accepted F.S. Sliding			1.10	1.30	1.30	1.30	1.10	1.30	1.30	1.30



Geometrical Definitions

Base Elevation	138.720	m
Log Top Elevation (Summer)	141.010	m
H.W.L. (Summer)	140.980	m
T.W.L. (Summer)	139.000	m
Log Top Elevation (Winter)	141.010	m
H.W.L. (Winter)	140.980	m
T.W.L. (Winter)	139.000	m
Log Top Elevation (Flood I)	141.010	m
H.W.L. (Flood I)	142.100	m
T.W.L. (Flood I)	139.970	m
Log Top Elevation (Flood II)	141.010	m
H.W.L. (Flood II)	142.100	m
T.W.L. (Flood II)	139.970	m
Deck Top Elevation	141.460	m
Thickness of Deck	0.025	m
Ice Elevation	140.680	m
Volume of Section	6.42	m^3
Centre of Gravity X	1.120	m
Centre of Gravity Y	1.002	m
Length of Pier Section	2.250	m
Width of Pier Section	0.960	m
Length of Sluiceway #1 Section	2.250	m
Width of Sluiceway #1 Section	1.500	m
Distance to Edge of Sluiceway #1 Section	0.000	m
Length of Sluiceway #2 Section	0.000	m
Width of Sluiceway #2 Section	0.000	m
Distance to Edge of Sluiceway #2 Section	0.000	m

Material Properties

f_c'	20.00	MPa	Concrete Compressive Strength
f_{b1}	3.00	MPa	Rock Bearing Strength
f_{b2}	0.00	MPa	Till Bearing Strength
ϕ_1	35.0	°	Angle of Friction #1
ϕ_2	40.0	°	Angle of Friction #2
ϕ_3	45.0	°	Specified Angle of Sliding Friction
ϕ_4	50.0	°	Angle of Friction #4
ϕ_5	55.0	°	Angle of Friction #5
τ_n	0.00	MPa	Cohesion
τ_1	0.38	MPa	$(0.17\sqrt{f_c'})/2$
τ_2	0.76	MPa	$(0.17\sqrt{f_c'})$
τ_3	1.00	MPa	$(0.05f_c')$
γ_{conc}	23.50	kN/m^3	Unit Weight of Concrete
γ_{water}	9.81	kN/m^3	Unit Weight of Water
ϕ_β	35.0	°	Basic Friction Angle

Loadings

2.22	%g	Vertical Ground Acceleration (Summer)
3.33	%g	Horizontal Ground Acceleration (Summer)
0.89	%g	Vertical Ground Acceleration (Winter, DEice)
1.33	%g	Horizontal Ground Acceleration (Winter, DEice)
75	kN/m	Ice Force on Concrete
75	kN/m	Ice Force on Logs/Gates

Input Summary

	Load Case									
	#1	#2	#3 (Sum)	#3 (Win)	#4	#5 (Sum)	#5 (Win)	#6		
M_1	150.8	150.8	150.8	150.8	150.8	150.8	150.8	150.8	kN	Weight of Section
V_{water}	1.66	1.66	1.66	1.66	5.26	1.66	1.66	5.26	m^3	Volume of Water Over Section
M_2	16.28	16.28	16.28	16.28	51.56	16.28	16.28	51.56	kN	Weight of Water Over Section
x	2.05	2.05	2.05	2.05	1.53	2.05	2.05	1.53	m	Location of Water Force Along X-Axis
ICE	-	184.50	-	184.50	-	-	184.50	-	kN	Total Ice Force
y	-	1.96	-	1.96	-	-	1.96	-	m	Location of Ice Force Along Y-Axis
W	-	-	-	-	-	2.19	0.88	-	kN	Westergaards Force
y	-	-	-	-	-	0.93	0.93	-	m	Location of Westergaards along Y-Axis
S_H	-	-	-	-	-	3.33	1.33	-	%g	Horizontal Seismic Coefficient
S_V	-	-	-	-	-	2.22	0.89	-	%g	Vertical Seismic Coefficient
w_1	61.63	61.63	61.63	61.63	127.42	61.63	61.63	127.42	kN	Hydrostatic Pressure From Headwater
y	0.75	0.75	0.75	0.75	1.00	0.75	0.75	1.00	m	Location of Headwater Force Along Y-Axis
w_2	0.95	0.95	0.95	0.95	18.85	0.95	0.95	18.85	kN	Hydrostatic Pressure From Tailwater
y	0.09	0.09	0.09	0.09	0.42	0.09	0.09	0.42	m	Location of Tailwater Force Along Y-Axis
H_1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	kN	Other Horizontal Force
y	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	m	Location of Other Horizontal Force Along Y-Axis
V_1	550.00	550.00	550.00	550.00	550.00	550.00	550.00	550.00	kN	Other Vertical Force
x	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	m	Location of Other Vertical Force Along X-Axis

Results

			Load Case #1 - Usual (Summer)				Load Case #2 - Usual (Winter)			
Cohesion		MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face		%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift		kN	68.96	68.96	68.96	68.96	70.91	68.96	68.96	68.96
Effective Base		%	100.0	100.0	100.0	100.0	96.4	100.0	100.0	100.0
Length of Base in Compression		m	2.25	2.25	2.25	2.25	2.17	2.25	2.25	2.25
Resultant		m	1.283	1.283	1.283	1.283	0.723	0.725	0.725	0.725
Stress at Heel		kPa	-166.42	-166.42	-166.42	-166.42	0.00	7.81	7.81	7.81
Cracked			NO	NO	NO	NO	YES	NO	NO	NO
Stress at Toe		kPa	-67.79	-67.79	-67.79	-67.79	-242.31	-242.01	-242.01	-242.01
Allowable Stress at Toe		kPa	-2000	-1500	-1500	-1500	-2000	-1500	-1500	-1500
F.S. Overturning			6.77	6.77	6.77	6.77	1.92	1.93	1.93	1.93
F.S. Sliding ϕ=	35		7.48	42.15	76.82	98.69	1.85	10.16	18.48	23.72
F.S. Sliding ϕ=	40		8.96	43.63	78.31	100.17	2.21	10.53	18.84	24.09
F.S. Sliding ϕ=	45		10.68	45.35	80.02	101.89	2.64	10.96	19.27	24.51
F.S. Sliding ϕ=	50		12.73	47.40	82.07	103.94	3.14	11.46	19.78	25.02
F.S. Sliding ϕ=	55		15.25	49.93	84.60	106.46	3.76	12.09	20.40	25.65
Accepted F.S. Sliding			1.50	2.00	2.00	2.00	1.50	2.00	2.00	2.00

			Load Case #4 - Flood I				Load Case #6 - Flood II			
Cohesion		MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face		%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift		kN	125.70	125.70	125.70	125.70	92.11	91.10	125.70	125.70
Effective Base		%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Length of Base in Compression		m	2.25	2.25	2.25	2.25	5.60	5.70	2.25	2.25
Resultant		m	1.179	1.179	1.179	1.179	1.252	1.256	1.179	1.179
Stress at Heel		kPa	-129.52	-129.52	-129.52	-129.52	20.93	21.97	-129.52	-129.52
Cracked			NO	NO	NO	NO	NO	NO	NO	NO
Stress at Toe		kPa	-96.93	-96.93	-96.93	-96.93	-135.71	-135.18	-96.93	-96.93
Allowable Stress at Toe		kPa	-2308	-2000	-2000	-2000	-2308	-2000	-2000	-2000
F.S. Overturning			3.55	3.55	3.55	3.55	5.09	5.18	3.55	3.55
F.S. Sliding ϕ=	35		4.04	23.42	42.80	55.02	4.26	32.56	42.80	55.02
F.S. Sliding ϕ=	40		4.84	24.22	43.60	55.83	5.10	33.41	43.60	55.83
F.S. Sliding ϕ=	45		5.77	25.15	44.53	56.76	6.08	34.39	44.53	56.76
F.S. Sliding ϕ=	50		6.88	26.26	45.64	57.86	7.25	35.56	45.64	57.86
F.S. Sliding ϕ=	55		8.24	27.62	47.00	59.23	8.69	37.00	47.00	59.23
Accepted F.S. Sliding			1.30	1.50	1.50	1.50	1.30	1.50	1.50	1.50

Input Summary

	Load Case #1	#2	#3 (Sum)	#3 (Win)	#4	#5 (Sum)	#5 (Win)	#6		
M_1	150.8	150.8	150.8	150.8	150.8	150.8	150.8	150.8	kN	Weight of Section
V_{water}	1.66	1.66	1.66	1.66	5.26	1.66	1.66	5.26	m^3	Volume of Water Over Section
M_2	16.28	16.28	16.28	16.28	51.56	16.28	16.28	51.56	kN	Weight of Water Over Section
x	2.05	2.05	2.05	2.05	1.53	2.05	2.05	1.53	m	Location of Water Force Along X-Axis
ICE	-	184.50	-	184.50	-	-	184.50	-	kN	Total Ice Force
y	-	1.96	-	1.96	-	-	1.96	-	m	Location of Ice Force Along Y-Axis
W	-	-	-	-	-	2.19	0.88	-	kN	Westergaards Force
y	-	-	-	-	-	0.93	0.93	-	m	Location of Westergaards along Y-Axis
S_H	-	-	-	-	-	3.33	1.33	-	%g	Horizontal Seismic Coefficient
S_V	-	-	-	-	-	2.22	0.89	-	%g	Vertical Seismic Coefficient
w_1	61.63	61.63	61.63	61.63	127.42	61.63	61.63	127.42	kN	Hydrostatic Pressure From Headwater
y	0.75	0.75	0.75	0.75	1.00	0.75	0.75	1.00	m	Location of Headwater Force Along Y-Axis
w_2	0.95	0.95	0.95	0.95	18.85	0.95	0.95	18.85	kN	Hydrostatic Pressure From Tailwater
y	0.09	0.09	0.09	0.09	0.42	0.09	0.09	0.42	m	Location of Tailwater Force Along Y-Axis
H_1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	kN	Other Horizontal Force
y	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	m	Location of Other Horizontal Force Along Y-Axis
V_1	550.00	550.00	550.00	550.00	550.00	550.00	550.00	550.00	kN	Other Vertical Force
x	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	m	Location of Other Vertical Force Along X-Axis

Results

		Load Case #3 - Post-Earthquake (Summer)				Load Case #3 - Post-Earthquake (Winter)			
Cohesion	MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	68.96	68.96	68.96	68.96	71.29	68.96	68.96	68.96
Effective Base	%	100.0	100.0	100.0	100.0	96.3	100.0	100.0	100.0
Length of Base in Compression	m	2.25	2.25	2.25	2.25	2.17	2.25	2.25	2.25
Resultant	m	1.283	1.283	1.283	1.283	0.722	0.725	0.725	0.725
Stress at Heel	kPa	-166.42	-166.42	-166.42	-166.42	0.00	7.81	7.81	7.81
Cracked		NO	NO	NO	NO	YES	NO	NO	NO
Crack Propagated		NO	NO	NO	NO	NO	NO	NO	NO
Stress at Toe	kPa	-67.79	-67.79	-67.79	-67.79	-242.30	-242.01	-242.01	-242.01
Allowable Stress at Toe	kPa	-2308	-2000	-2000	-2000	-2308	-2000	-2000	-2000
F.S. Overturning		6.77	6.77	6.77	6.77	1.92	1.93	1.93	1.93
F.S. Sliding ϕ= 35		7.48	42.15	76.82	98.69	1.84	10.16	18.48	23.72
F.S. Sliding ϕ= 40		8.96	43.63	78.31	100.17	2.21	10.53	18.84	24.09
F.S. Sliding ϕ= 45		10.68	45.35	80.02	101.89	2.63	10.96	19.27	24.51
F.S. Sliding ϕ= 50		12.73	47.40	82.07	103.94	3.14	11.46	19.78	25.02
F.S. Sliding ϕ= 55		15.25	49.93	84.60	106.46	3.76	12.09	20.40	25.65
Accepted F.S. Sliding		1.10	1.50	1.50	1.50	1.10	1.50	1.50	1.50

		Load Case #5 - Earthquake (Summer)				Load Case #5 - Earthquake (Winter)			
Cohesion	MPa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	68.96	68.96	68.96	68.96	70.91	68.96	68.96	68.96
Effective Base	%	100.0	100.0	100.0	100.0	95.7	100.0	100.0	100.0
Length of Base in Compression	m	2.25	2.25	2.25	2.25	2.15	2.25	2.25	2.25
Resultant	m	1.273	1.273	1.273	1.273	0.718	0.720	0.720	0.720
Stress at Heel	kPa	-162.41	-162.41	-162.41	-162.41	0.00	9.41	9.41	9.41
Cracked		NO	NO	NO	NO	YES	NO	NO	NO
Crack Propagated		NO	NO	NO	NO	YES	NO	NO	NO
Stress at Toe	kPa	-70.59	-70.59	-70.59	-70.59	-243.56	-243.13	-243.13	-243.13
Allowable Stress at Toe	kPa	-2727	-2308	-2308	-2308	-2727	-2308	-2308	-2308
F.S. Overturning		6.29	6.29	6.29	6.29	1.90	1.91	1.91	1.91
F.S. Sliding ϕ= 35		6.65	37.63	68.62	88.16	1.82	9.99	18.16	23.31
F.S. Sliding ϕ= 40		7.97	38.95	69.94	89.48	2.18	10.35	18.52	23.67
F.S. Sliding ϕ= 45		9.50	40.48	71.47	91.01	2.60	10.77	18.94	24.09
F.S. Sliding ϕ= 50		11.32	42.30	73.29	92.83	3.10	11.27	19.44	24.59
F.S. Sliding ϕ= 55		13.56	44.55	75.53	95.07	3.71	11.89	20.05	25.20
Accepted F.S. Sliding		1.10	1.30	1.30	1.30	1.10	1.30	1.30	1.30

Appendix B
Anchor Design

HATCH

Calculation Cover Sheet

Client:	Quinte Conservation	**Project No:**	H368596
Project Title:	Bellrock Stability Assessment and Anchor Design	**Discipline:**	Civil-Structural
Calculation No:	1	**EWP No:**	
Number of sheets: (incl. cover sheet)	6		
Calculation Title:	Rock Anchor Design		

Category of calculation checking required ✓ *(tick box)* ☐ Full ☐ Spot ☐ Self

Prepared By: Gavin Ainslie *(Print Name)* Date: 2022-06-06 (YYYY-MM-DD)

Preliminary Review By: Bruce MacTavish *(Print Name)* Date: ______ (YYYY-MM-DD)

Can the calculation now be released for work? ○ Yes ○ No **To the Client?** ○ Yes ○ No

General Notes:

			Gavin Ainslie		
2022-02-15			Sheheryar Qureshi	Bruce MacTavish	
DATE	**REV**	**STATUS**	**PREPARED BY**	**CHECKED BY**	**APPROVED BY**

Superseded by Calculation No: **Date:** (YYYY-MM-DD)

Reason Voided:

HATCH

Anchor Design - West Dam Pier 1 to 6

References:

- Post Tensioning Institute - Recommendations for Prestressed Rock and Soil Anchors (PTI DC35.1 - 2014)

Data

$P_a := 550kN$	Required anchorage load
$d_a := 1\frac{3}{8}\ in = 35 \cdot mm$	Diameter of anchor threaded bar
$cc_g := 25mm$	Grout cover over the anchor
$ASTM_a := 722$	Grade of prestressing steel
$f_{pu} := 1035MPa$	Ultimate tensile strength of the prestressing steel
$type := 1$	0 = Strand Tendon, 1 = Bar Tendon

HATCH

$A_a := 1019mm^2$ Area of anchor threaded bar

830/1035 MPa ASTM A722	26	1"	548	457	567	4.48	30.5	18.3	R
Hot-Rolled Threadbar	32	1 1/4"	806	673	834	6.53	36.6	18.3	R
PT Ground Anchors Post-Tensioning	36	1 3/8"	1,019	851	1,054	8.27	41.4	18.3	R
*Cold-Rolled Threadbar	*46	1 3/4"	1,664	1,459	1,779	13.72	51.0	13.7	R
Steel Grade for 46Ø: 876/1069 MPa	*66	2 1/2"	3,331	2,754	3,442	27.10	70.9	13.7	R
	*75	3"	4,419	3,656	4,568	35.90	79.9	13.7	R

$\gamma_{rock} := 26\frac{kN}{m^3}$ Saturated unit weight of rock

$\gamma_w := 9.8\frac{kN}{m^3}$ Unit weight of water

$\gamma_{subrock} := \gamma_{rock} - \gamma_w$ Submerged unit weight of rock

$\phi_c := 0.65$ Concrete resistance factor (CSA A23.-14 cl 8.4.2).

$f_c := 20MPa$ Concrete compressive strength.

$H_{str} := 1m = 1\,m$ $\gamma_{subrock} = 16.2\cdot\frac{kN}{m^3}$ Submerged unit weight of rock

Calculations

1. Working Load of Anchor

$P_r := 0.6\cdot f_{pu}\cdot A_a$ Working load of provided anchor, PTI, Section 6.6 $P_r = 632.8\cdot kN$

$Check_1 := \begin{cases} \text{"OK"} & \text{if } P_r \ge P_a \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$ $Check_1 = \text{"OK"}$

2. Anchor Bond Length

$FS := 2.0$ Factor of safety on average ultimate bond strength, PTI Section 6.6

$\tau_u := 1.7MPa$ Average ultimate bond strength along interface between grout and ground , PTI Table C6.1, as shown below

$d_{hole} := Ceil(d_a + 2cc_g, 25mm)$ Drill hole diameter $d_{hole} = 100\cdot mm$

Table C6.1 — Typical average ultimate bond strengths—rock/grout

Rock	Average ultimate bond strength—rock/grout, MPa (psi)
Granite and basalt	1.7 to 3.1 (250 to 450)
Dolomite limestone	1.4 to 2.1 (200 to 300)
Soft limestone	1.0 to 1.4 (150 to 200)
Slates and hard shales	0.8 to 1.4 (120 to 200)
Soft shales	0.2 to 0.8 (30 to 120)
Sandstones	0.8 to 1.7 (120 to 250)
Weathered sandstones	0.7 to 0.8 (100 to 120)
Chalk	0.2 to 1.1 (30 to 155)
Weathered marl	0.15 to 0.25 (25 to 35)
Concrete	1.4 to 2.8 (200 to 400)

$L_b := \frac{P_r \cdot FS}{\pi \cdot d_{hole} \cdot \tau_u}$ Required Bond Length, PTI, Section 6.7 $L_b = 2.37\,m$

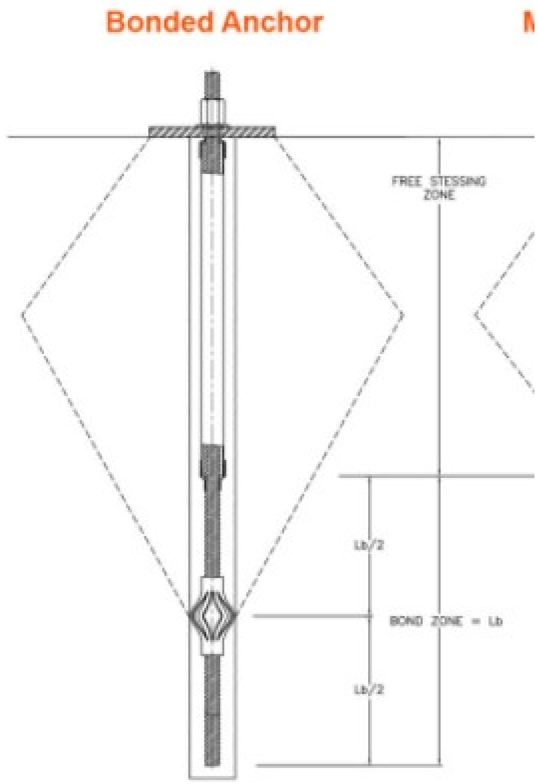
$L_{bmin} := \begin{cases} 4.5m & \text{if } ASTM_a = 416 \\ 3.0m & \text{if } ASTM_a = 722 \wedge d_a \le 44mm \\ 4.5m & \text{otherwise} \end{cases}$ Minimum tendon bond length, PTI, Section 6.7 $L_{bmin} = 3\,m$

$L_{b.prov} := \max(L_b, L_{bmin})$ Provided Bond Length, $L_{b.prov} = 3\,m$

3. Rock Cone Displacement and Minimum Embedment

C6.7.1 — Rock anchors

For conventional rock anchors installed in competent rock, the bond stresses are typically concentrated at the top of the bond length. The maximum strain in the tendon bond length occurs at the top of the tendon bond length and may cause local load redistribution within the rock or the displacement of a small cone of rock. When this occurs, the peak stress position moves down the tendon bond length.



$h_{cone} := \left(\dfrac{P_r}{\frac{1}{3}\pi \cdot \gamma_{subrock}} \right)^{\frac{1}{3}}$ — Height of rock cone — $h_{cone} = 3.34\,m$

$Emb_{min} := Ceil\left(h_{cone} + \dfrac{L_b}{2}, 0.5m \right)$ — Minimum embedment into competent rock — $Emb_{min} = 5\,m$

4. Free Stressing Length of Anchor

$L_{unbmin} := \begin{cases} 4.5m & \text{if } type = 0 \\ 3m & \text{otherwise} \end{cases}$ — Minimum free stressing length (unbonded), PTI, Section 6.8 — $L_{unbmin} = 3\,m$

$Allow := 1m$ — Allowance for free stressing length to account unforeseen circumstances

$L_{unbprov} := H_{str} + Allow$ — Provided free stressing length (unbonded) — $L_{unbprov} = 2\,m$

$Check_2 := \begin{cases} \text{"OK"} & \text{if } L_{unbprov} \ge L_{unbmin} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$ — $Check_2 = \text{"NOT OK}$

5. Concrete Bearing Resistance

$A_{br} := \dfrac{P_r}{(0.85 \cdot \phi_c \cdot f_c)}$ Required area of bearing, CSA A23.3, Clause 10.8.1 $A_{br} = 5.73 \times 10^4 \cdot mm^2$

$Side_{br} := Ceil\left(A_{br}^{0.5}, 25mm\right)$ Required area of bearing $Side_{br} = 250 \cdot mm$

Appendix C
Revised Anchor Head Calculation Based on Concrete Condition Assessment

HATCH

Calculation Cover Sheet

Client:	Quinte Conservation Authority				
Project Title:	Bellrock Dam PT Anchor Plate Design				
Discipline:	Structural				
Calculation No.:	H368596-0000-230-230-0001	**File No:**	H368596-0000-230-230-0001	**Number of Sheets:**	4

Description: This
This calculation checks the adequacy of the bearing plate for the post tension anchors as designed and stamped by Bruce M.T. on 2023/04/28 with the details in drawing H368596-0000-220-270-0001. The assumption for the concrete compressive strength was taken from the core sample 1/A presented in the Table 3-1 of the report H369335-0000-230-230-0001, Rev A Page 8.

Category of calculation verification required *tick box* ☐ 1 ☐ 2 ☐ 3 ☐ 4

Prepared by:	Farrokh Rasouli	**Date:**	July 1, 2023
Print Name >	(Responsible Engineer)		
Preliminary Review by:		**Date:**	
Print Name >			

Can the calculation now be released for work? ☐ Yes ☐ No **To the Client?** ☐ Yes ☐ No

Checked by: by:	Robert MacCrimmon	**Date:**	July 6, 2023
Print Name >			
Reviewed by:		**Date:**	
Print Name >			
Approved by:	Sophie Alrhieh	**Date:**	August 11, 2023
Print Name >			

General Notes:

Revisions

Rev.	Date	Prepared by	Checked by	Approved by	Description
0	June 21, 2021	F. Rasouli	R. MacCrimmon	S. Alrhieh	Internal Review
		F. Rasouli	R. M	Alrhieh, Safaa	Digitally signed by Alrhieh, Safaa DN: cn=Alrhieh, Safaa, email=sophie.alrhieh@hatch.com Date: 2023.08.14 11:19:55 -04'00'

Superseded by Calculation No. **Date:**

Reason voided:

BELLROCK DAM PT ANCHOR PLATE DESIGN

Calculation description

1. All anchors have a design load of 550 kN.
2. The threaded bar anchors have a nominal diameter of 36 mm and are to be Dywidag hot-rolled threadbar (or similar) of grade ASTM A722.
3. The anchors must have a minimum bond length of 3 m in bedrock and a minimum free stressing length of 3 m.
4. Accordingly, the spacing is as follows:

- 2700 mm for the overflow section
- 360 mm for the gravity section
- 1 anchor per pier in the spillway section.

DYWIDAG THREADBARS - TECHNICAL DATA [METRIC UNITS] August 15, 2014, DSI Surrey, BC

STEEL GRADE f_y / f_u MPa	Nominal Bar Diameter mm		Steel Area A_s mm²	Yield Load $P_y = f_y A_s$ kN	Ultimate Load $P_u = f_u A_s$ kN	Lineal Weight kg/m	Max. Bar Ø Across Ribs mm	Mill Length m	Direction of Thread L / R
900/1100 MPa Hot-Rolled Threadbar Form-Ties, Post-Tensioning, Ground Anchors	15	5/8"	177	159	**195**	1.44	17.6	5.9	R
	20	7/8"	316	284	**348**	2.60	23.0	11.9	R
830/1035 MPa ASTM A722 Hot-Rolled Threadbar PT Ground Anchors Post-Tensioning	26	1"	548	457	**567**	4.48	30.5	18.3	R
	32	1 1/4"	806	673	**834**	6.53	36.6	18.3	R
	36	1 3/8"	1,019	851	**1,054**	8.27	41.4	18.3	R
*Cold-Rolled Threadbar Steel Grade for 46Ø: 876/1069 MPa	*46	1 3/4"	1,664	1,459	**1,779**	13.72	51.0	13.7	R
	*66	2 1/2"	3,331	2,754	**3,442**	27.10	70.9	13.7	R
	*75	3"	4,419	3,656	**4,568**	35.90	79.9	13.7	R

References

1. Dywidag Threadbars - Technical Data Metric 2018.
2. Recommendations for Prestressed Rock and Soil Anchors, Post Tensioning Institute (PTI DC35.1-14).
3. Concrete Design Handbook, Fourth Edition, Cement Association of Canada.
4. Design of Concrete Structures, CSA A23.3-14.
5. Design of Steel Structures, CSA S16-01.

Input data

$d_a := 36mm$ Anchor diameter.

$d_{hole} := 100mm$ Hole diameter.

$A_a := 1019mm^2$ Anchor area.

$P_f := 550kN$ Anchor design factored load.

$f_{u1} := 1054kN$ Anchor ultimate factored load.

$f_{y1} := 851\,kN$	Anchor yield load.
$Pr := 0.6 \cdot f_{u1} = 632.4\,kN$	Anchor design working load.
$f_c := 10.4MPa$	Concrete compressive strength.
$\gamma_{rock} := 26.20 \frac{kN}{m^3}$	Rock unit weight.
$\gamma_w := 9.807 \frac{kN}{m^3}$	Water unit weight
$\gamma_{subrock} := 16.39 \frac{kN}{m^3}$	Rock submerged unit weight.

HATCH

Detailed Calculations

Anchor Base Plate Design

General notes

Considering the maximum testing load at 80% ultimate tensile strength, the thickness of the base plate is checked for allowable bending stress. For the equivalent square base plate, based on considering the plate as a double cantilevered section over bearing sections, the minimum thickness required is calculated.

$d_{basepl} := 330mm$ — Equivalent square base plate.

$t_{bpl} := 40mm$ — Thickness of the base plate.

$\nu := 0.3$ — Poisson's ratio for steel.

$f_{y2} := 350MPa$ — Yield stress of 350W steel.

$d_0 := 40mm$ — Effective inner opening diameter.

$\Phi_c := 0.65$

$\Phi_s := 0.85$

$\Phi_0 := 0.90$

$$t_{min1} := \left(\frac{d_{basepl} - d_0}{2} \cdot \sqrt{\frac{2 \cdot 0.8 \cdot 1 \cdot f_{u1}}{d_{basepl} \cdot d_{basepl} \cdot f_{y2}}}\right) = 30.5\,mm$$

Minimum base plate thickness.

Minimum base plate thickness < base plate thickness, therefore ok.

$$V_1 := \frac{0.8 \cdot 1 \cdot f_{u1}}{d_{basepl}}\left(d_{basepl} - d_0\right) = 740.99\,kN$$

Base plate shear.

$$\frac{V_1}{0.66 \cdot \Phi_0 \cdot t_{min1} \cdot d_{basepl} \cdot f_{y2}} = 0.35$$

0.35 < 1.0, therefore ok.

At the base plate location of the PT anchors, the concrete bearing stress experienced by the concrete needs to be checked to eliminate crushing of concrete. The factored bearing resistance of concrete is calculated based on Post-Tensioning Manual Cl 3.1.7.

$0.7 \cdot f_c = 7.28\,MPa$ — Concrete compressive strength at poststressing.

$A_b := d_{basepl}^{\;2} = 108900\,mm^2$ — Base plate area.

$A_2 := \left(d_{basepl} + 2 \cdot 150mm\right)^2 = 396900\,mm^2$ — Area of lower base of the largest frustum of pyramid.

At Service Load

$$f_{cp1} := \min\left[\left[0.6 \cdot 0.7 \cdot f_c \cdot \sqrt{\left(\frac{A_2}{A_b}\right)}\right], \left(1.25 \cdot f_c\right)\right] = 8.34\,MPa$$

$$P_{all1} := f_{cp1} \cdot A_b = 908.11\,kN$$ Base permissible load.

$$\frac{0.6 \cdot f_{u1}}{P_{all1}} = 0.7$$

0.7 < 1.0, therefore ok.

At Transfer Load

$$f_{cp2} := \min\left[\left[0.8 \cdot 0.7 \cdot f_c \cdot \sqrt{\left(\frac{A_2}{A_b}\right) - 0.2}\right], \left(1.25 \cdot 0.7 \cdot f_c\right)\right] = 9.1\,MPa$$

$$P_{all2} := f_{cp2} \cdot A_b = 990.99\,kN$$ Base permissible load.

$$\frac{0.8 \cdot f_{u1}}{P_{all2}} = 0.85$$

0.85 < 1.0, therefore ok.

Bending Stress Check

$$q := \frac{0.8 \cdot f_{u1}}{A_b} = 7.74\,MPa$$ Base pressure.

$$L_0 := \frac{d_{basepl} - d_0}{2} = 145\,mm$$ Base plate cantilever length.

$$M := q \cdot d_{basepl} \cdot L_0 \cdot \frac{L_0}{2} = 26.86\,kN \cdot m$$ Base plate moment.

$$I := d_{basepl} \cdot \frac{t_{bpl}^{\;3}}{12} = 1760000\,mm^4$$ Base plate moment of inertia.

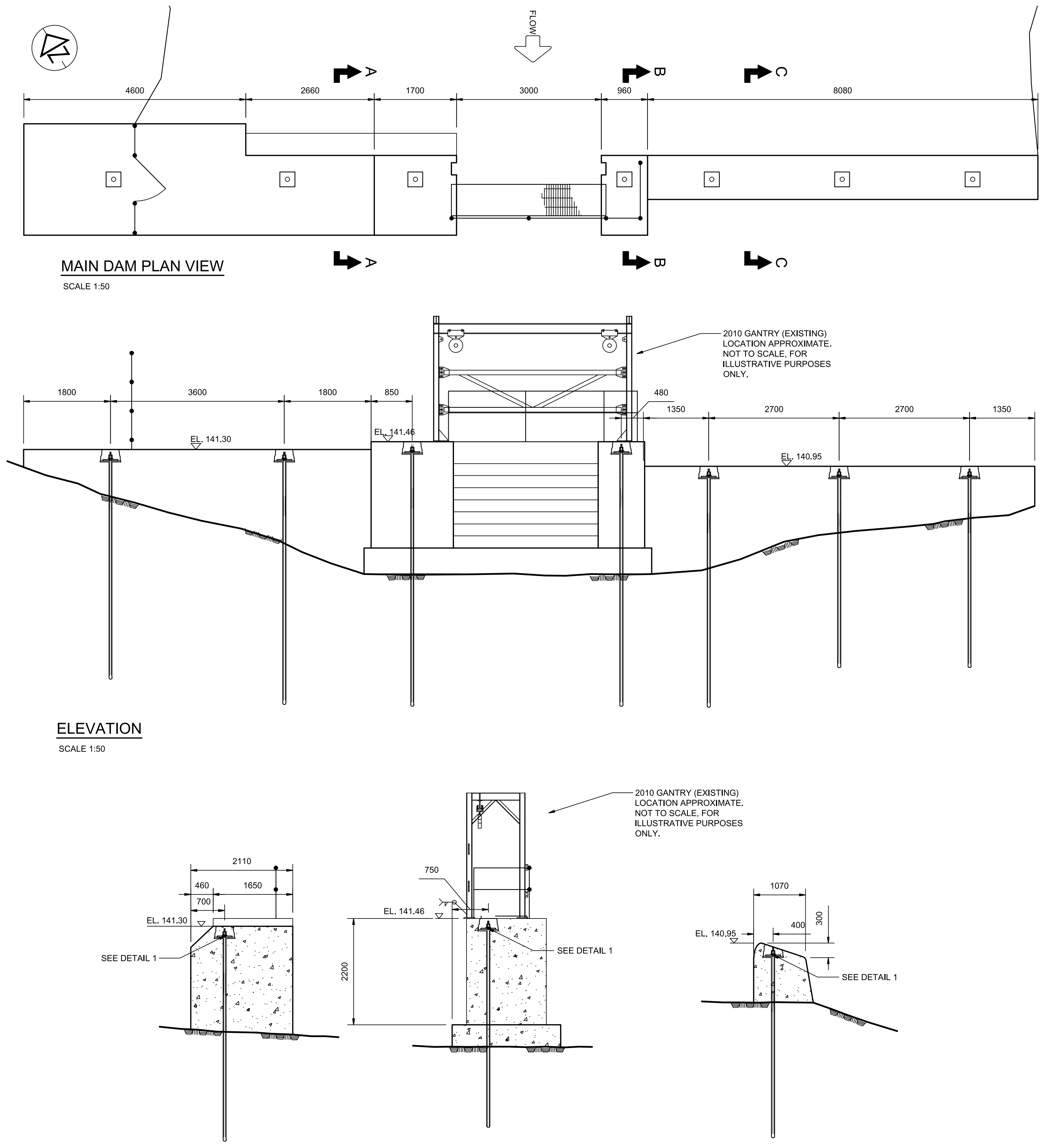
$$\frac{M \cdot \frac{t_{bpl}}{2}}{I} = 305.24\,MPa$$ Base plate bending stress.

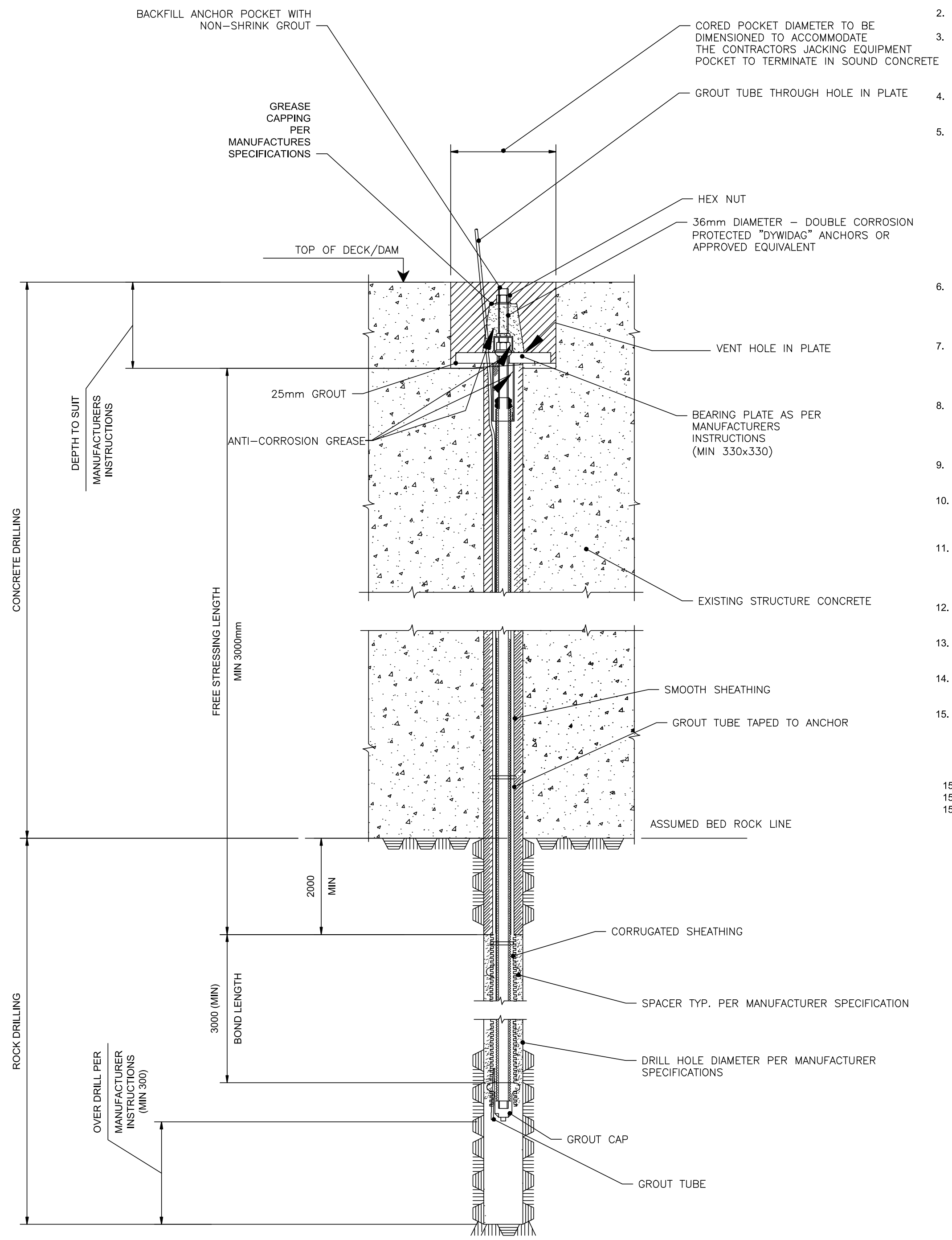
Base plate bending stress < 350 MPa, therefore ok.

Summary of results

Use PL 330x330x40 mm of Grade 350W steel base plate.

Appendix D
Bellrock Dam Post-Tensioned Anchors Section and Layout





DETAIL 1

TYP. POST TENSIONED ANCHOR (36mm BAR DIAMETERS)

SCALE 1:75

NOTES:

1. CONTRACTOR TO FIELD VERIFY ALL MEASUREMENTS.
2. VENDOR DETAILS TO OVERRIDE HARDWARE DETAILS HEREIN.
3. CLEAN ALL CUTTING AND DEBRIS OUT OF THE HOLE. USE A METHOD THAT REACHES TO THE BOTTOM OF THE BOND LENGTH OF THE ANCHOR. SURFACE WATER SHALL NOT BE ALLOWED TO ENTER THE HOLE.
4. CONTRACTOR TO ENSURE NO DRILLING CHIPS OR DEBRIS ENTERS THE WATERWAY.
5. TEST EACH HOLE FOR WATER TIGHTNESS NOT MORE THAN 7 DAYS PRIOR TO ANCHOR INSTALLATION. CONTRACTOR TO FILL ENTIRE HOLE IN ROCK WITH WATER AND SUBJECT THE THIS WATER TO A PRESSURE OF 35KPA IN EXCESS OF THE HYDROSTATIC HEAD MEASURE AT THE TOP OF THE HOLE. THE TEST INTERVAL SHALL BE 20 MINUTES. THE CONTRACTOR SHALL SUPPLY A FLOW METER CAPABLE OF ACCURATELY MEASURING FLOW TO 0.1 LITERS AND PRESSURE TO 35 KPA. IF THE WATER LEAKAGE FROM THE HOLE EXCEEDS 10L OF WATER OVER A 20 MINUTE PERIOD, THE HOLE SHALL BE REGROUTED, RE-DRILLED AND RE-TESTED UNTIL THIS WATERTIGHTNESS CRITERION IS SATISFIED.
6. THE ACTUAL DIAMETER OF DRILLED HOLES IN EXISTING CONCRETE & ROCK, & THE ACTUAL DIAMETER OF PRESTRESSING DUCK, IN THE CONCRETE, TO BE DETERMINED BY ANCHOR MANUFACTURER TO SUIT ANCHOR ARRANGEMENT.
7. STRESSING SHALL NOT COMMENCE UNTIL THE ANCHOR GROUT HAS REACHED A MINIMUM COMPRESSIVE STRENGTH OF 35MPa & NOT UNTIL THE NEW BEARING PLATE GROUT HAS REACHED A MINIMUM COMPRESSIVE STRENGTH OF 40MPa.
8. SIZES OF DRILL HOLE, CORRUGATED SHEATHING, SMOOTH SHEATHING & DIMENSIONS OF SHEATHING AROUND COUPLERS ACCORDING TO THE ANCHOR MANUFACTURE'S CORROSION PROTECTION SYSTEM.
9. ALL GROUTING AND STRESSING OF ANCHORS TO BE DONE IN ACCORDANCE WITH MANUFACTURE'S WRITTEN SPECIFICATIONS.
10. ALL MATERIALS (GROUT, PVC/PE SHEATHING, CORROSION INHIBITOR COMPOUND, ECT) PER MANUFACTURER'S WRITTEN SPECIFICATIONS.
11. ROCK ANCHOR BAR TO BE IN ACCORDANCE WITH ASTM A772 (GRADE 830, ULT. 1035 MPa), GUARANTEED ULTIMATE STRENGTH (GUTS) TO BE 1054 kN.
12. ALL ANCHORS TO BE TESTED TO 65% GUTS. - TEST LOAD = 687 kN.
13. ALL ANCHORS TO BE LOCKED-OFF TO 60% GUTS. - LOCKED-OFF LOAD = 632 kN.
14. RESIDUAL MINIUM DESIGN LOAD (WORKING LOAD) IN ANCHORS IS ASSUMED TO BE 55% GUTS. DESIGN LOAD = 579 kN.
15. PROOF TESTS MUST BE PERFORMED FOR ALL ANCHORS. THE PROOF TESTS CONSISTS OF INCREMENTALLY LOADING THE ANCHORS TO 65% OF THE ULTIMATE LOAD AND TAKING ELONGATION READINGS TO 0.03MM ACCURACY WITH THE SCHEDULE BELOW:
   - 15.1. PULT. = ULTIMATE LOAD 1054 kN (GUTS)
   - 15.2. P= DESIGN LOAD = 0.53 PULT = 559 kN
   - 15.3. AL= ALIGNMENT LOAD (5% PULT)

	APPLIED INCREMENT LOAD AS PERCENTAGE OF PULT
	5
	13
	26
	39
	52
	62
TEST LOAD	65
LOCK -OFF LOAD	60

HOLD TEST LOAD INCREMENT (65%) AND TAKE ELONGATION READINGS AT: 1 MIN., 2 MIN., 3 MIN., 4 MIN., 5 MIN., 6 MIN., 10 MIN. **

** = IF THE MOVEMENT BETWEEN 1 MINUTE AND 10 MINUTES EXCEEDS 1mm, HOLD TEST LOAD FOR ADDITIONAL 50 MINUTES AND RECORD MOVEMENT AT 15, 20, 25, 30, 45 AND 60 MINUITES. THE "HOLD-LOAD TIME" STARTS WHEN THE PUMP BEGINS TO LOAD THE ANCHOR. CONTRACTOR TO PROVIDE LOAD-EXTENSION DATA & PLOTS

DRAWING No.	DRAWING TITLE
REFERENCE DRAWINGS	

LICENSED PROFESSIONAL ENGINEER — B. A. MacTAVISH — 2023-04-28 — PROVINCE OF ONTARIO

NAME / SIGNATURE / ENG REG NUMBER / REVISION DATE / ORIGINAL DATE / REG. PROFESSIONAL

No.	DESCRIPTION	BY	CHK'D	DATE
1	ANCHOR HEAD REVISED		BM	04/28/2023
B	ISSUED FOR REVISION	JH	GA	1/31/2023
0	ISSUED FOR CONSTRUCTION	JH	GA	6/21/2022

REVISIONS

HATCH

ROLE	NAME	SIGNATURE	DATE
DRAFTSPERSON	J. HOWCROFT	NR	5/24/2022
DESIGNER		NR	
CHECKER			6/21/2022
DESIGN COORD.			
RESP. ENG.			6/21/2022
LEAD DISC. ENG.			
ENG. MANAGER			
PROJ. MANAGER			6/21/2022

DRAWING APPROVAL STATUS: Preliminary

QUINTE CONSERVATION AUTHORITY

BELLROCK DAM ANCHORAGE SECTION AND LAYOUT

SCALE NTS OR AS NOTED | DWG. No. H368596-0000-220-270-0001 | REV 1

SHEET SIZE: B1

$USERNAMES $DATES $FILES $TIMES

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