Project Memo

January 31, 2023

To: Mike Smith From: G. Ainslie

cc: C. Phillibert
J. Platt

Quinte Conservation Authority

Bellrock Dam Concrete Condition Assessment

1. Introduction

Figure 1-1: View of the Bellrock Dam

Hatch has recently undertaken a global stability assessment of the Bellrock Dam. During the assessment it was determined that stabilizing remedial actions would be required for the dam to meet current NDMNRF dam safety guidelines. Hatch performed the design for the installation of post-tensioned anchors for the structure. Quinte Conservation (QC) would like the condition of the concrete assessed prior to drilling and anchoring to verify the integrity of the structure. To do this, Hatch had suggested concrete coring be completed in the three distinct sections of the dam; the overflow weir, the spillway pier and the gravity section. The objective of this investigation was to determine if the anchoring program should continue as designed, if the anchoring design must be modified to accommodate poor concrete conditions, or if an alternate approach should be taken.

1.1 Structure History

The Bellrock dam was originally constructed in 1958, in 1982, an effort to rehabilitate the deteriorated concrete took place. It is assumed that the work outlined on Cumming – Cockburn & Associates Limited, Drawing Nos. 4014-1 & 4014-2 has been completed, however, it should be noted the drawings are not labeled as “As-Built”. The scope of the rehabilitation was to resurface the overflow weir and the right pier and rebuild the left pier. Passive rock anchors are also included in the drawings between bedrock and the newly placed concrete. In 2004, Hatch performed a Dam Safety Assessment and determined that none of the components of the dam completely satisfy the global stability requirements for concrete dams. In 2010, a new gantry was installed over the spillway bay. It is understood that the history of the dam may not be complete as limited documentation was provided to Hatch for this assessment.

1.1.1 Concrete Rehabilitation - 1982

The following work was specified on Drawings 4014-1 & 4014-2:

1. Weir – Remove minimum 150 mm of concrete on the crest of the weir and all deteriorated concrete along upstream and downstream surfaces. Install four passive rock anchors (25M) on the upstream side of the weir. Install 15M rebar matt on 300 mm centers, and place concrete over entire weir.
2. Left pier – The existing left pier was removed and replaced with newly placed concrete, with a passive rock anchor (25M) installed into bedrock. The steel reinforcement used was a 300 mm x 300 mm matt cage.
3. Right Pier – A minimum of 300 mm of existing concrete to sound concrete was removed and replaced with new concrete. A passive rock anchor (25M) was installed near the upstream edge of the pier. A 600 mm x 600 mm dowel grid was used to at the interface of existing and new concrete. The steel reinforcement used was a 300 x 300 mm matt cage.

2. Concrete Coring

Concrete coring took place on October 20th, 2022 and a total of two cores were drilled and taken for further testing. The next day, one core was taken from the overflow section and later sent to Hatch for examination and laboratory testing.

2.1 Site Representation

Hatch personnel was present on site and directed the drilling for October 20th and advised the location for the final core prior to demobilizing from site. PVC piping (or alternative) for transporting the cores was not provided by the contractor.

2.2 Drilling Layout

Three concrete cores were taken during the drilling exercise; a core in each representative section of the structure that will be anchored to improve the global stability of the dam. Core 1 was taken from the right gravity section, Core 2 was taken from the right pier of the spillway and Core 3 was taken from the overflow spillway. Since the left pier was rebuilt in 1982, it is assumed to be in equal or greater condition than the right pier and was therefore not selected for coring.

The diameter of core samples for compressive strength testing is generally dictated by the size of the aggregates in the concrete. Given the age of the structure, and no construction documentation, it is assumed large aggregates were used during the original construction. Therefore, a 6-in. diameter core bit was used. The desired length to diameter ratio for compressive testing is around 2 : 1, therefore a target core sample length of twelve inches was selected.

2.3 Core Samples

Photographs of the samples were taken shortly after they were removed from the dam. The three samples are shown in Figure 2-1.

Figure 2-1: Three Concrete Cores Taken from Bellrock Dam

All three of the samples indicate concrete resurfacing had been performed on the structure in the past. The intention of the drilling program was to retrieve a sample of the concrete below the rehabilitated surface concrete. However, with varying concrete depths, drill bit/extension limitations, and concrete cracking, this was not achieved for all of the cores.

3. Bellrock Dam Condition Assessment

An assessment on the quality of the concrete and applicability of the proposed anchorage program was performed by visual and laboratory results of the concrete.

3.1 Visual Assessment

During the coring of the dam, a visual assessment of the concrete was performed, the following section outlines observed concrete deficiencies.

3.1.1 Slurry Deposit

During the drilling of concrete Core 1, diluted concrete slurry was seeping through a joint in the right wingwall to the leveling slab. Figure 3-1 shows the slurry deposit downstream of Hole 1 as well a view of the inside of Hole 1.

Figure 3-1: Slurry Deposit Downstream of Core Hole 1, Internal View of Core Hole 1

The slurry deposit and empty core hole are clear indications that there are seepage paths from the core to the downstream face of the dam. This will impact on the anchoring program as the drilled holes must pass a watertightness test prior to the installation of anchors. It should be assumed that all of the anchor holes will need to be grouted and re-drilled to satisfy the pressure testing requirements. Procedures and requirements for this are specified on the anchoring drawings.

3.1.2 Overflow Weir Surface Erosion

The overflow weir exhibits moderate surface erosion as shown in Figure 3-2.

Figure 3-2: Exposed Aggregate on the Overflow Weir

The overflow section has eroded and exposed aggregate along the entire surface of the structure. A core was taken from the crest of the overflow to help estimate the condition and strength of the concrete and to reveal the depth of the rehabilitated concrete. The depth from the sample indicates approximately 14 cm of sound concrete remaining on the surface of the overflow section.

3.1.3 Leveling Slab downstream of the Right Wingwall

Undermining of the leveling slab downstream of the right wingwall has occurred, shown in Figure 3-3.

Figure 3-3: Undermining of the Right Wingwall Leveling Slab

The base of the leveling slab has eroded and undermining has begun. The undermining does not appear to have occurred beneath the dam proper. Large rocks appear to have been used in the concrete of the leveling slab, likely a different mix than has been included in the dam

3.1.4 Concrete to Bedrock Interface

Two notable locations along the downstream toe of the dam have active leakage. The two locations are shown in Figure 3-1 and Figure 3-4.

Figure 3-4: Seepage Points Downstream of Overflow Weir

Seepage beneath concrete structures does not always indicate a concern for dam safety. However, seepage should be generally controlled to avoid excessive erosion to the concrete and foundation of the dam.

3.2 Concrete Laboratory Results

Hatch engaged WSP to perform concrete compressive strength testing of the three samples obtained from the Bellrock Dam. The Concrete Core Test Report is available in Appendix A. The three core samples were delivered to WSP as shown in Figure 3-5. Testing was performed according to CSA A23.2-14c.

Figure 3-5: Cores Prior to Delivery to WSP

Core 1 contains a notably large aggregate and an area of honeycombed concrete or deteriorated concrete that was not removed and replaced during the deck resurfacing. Core 2 has longitudinal cold joint that runs nearly the entire length of the core. The majority of Core 3 appears to be the remediated concrete surface. The trimmed cores are shown in Figure 3-6 after they have been tested.

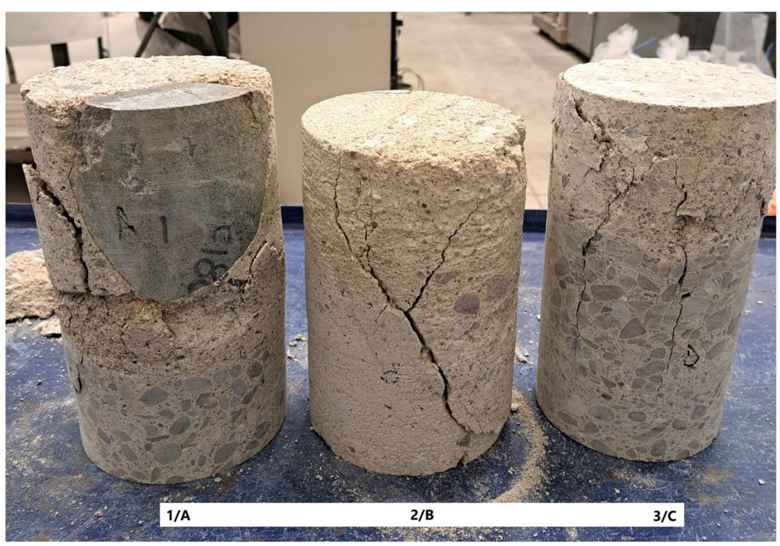

Figure 3-6: Cores after Compressive Testing

A summary of the laboratory results is presented in Table 3-1.

Table 3-1: Compressive Test Results

Core Sample	1/A	2/B	3/C
Corrected Strength (MPa)	10.4	6.8	25.5
Failure Type	3	2	3

The results of the compressive strength testing show significant variability in the compressive strength of the concrete cores. This is to be expected in cores taken from a structure dating back to the 1950s that has undergone concrete remediations. Cores 1 and 2 contain defects that when exposed to unconfined compression tests would be expected to present lower strength results than the concrete in-situ, which inherently enjoys some degree of confinement.

The strength of the concrete will have an impact on the mechanism transferring the load of the anchors to concrete of the dam. Typically, a lower strength concrete will require a larger bearing area for the anchor head (upper end) to accommodate the same compressive load. Engagement of foundation bedrock is not expected to present any capacity problems. To increase the bearing area at the anchor head, a larger steel plate can be specified, or a concrete pedestal can be formed to distribute the load to a larger area.

4. Conclusions and Recommendations

The following is a summary of the observations and conclusions made from the Bellrock Dam concrete assessment:

1. The overflow weir has experienced moderate surface erosion with exposed aggregates. At this time, this surface deterioration is not a dam safety concern.
2. As exhibited during drilling of Core 1, it is anticipated that many of the anchor locations will have seepage paths or voids within the concrete, which will necessitate hole sealing (as specified) during anchor installation.
3. Undermining of the leveling slab downstream of the right wingwall has occurred.
4. There is evidence of seepage along the interface of the bedrock and concrete along the downstream side of the overflow section and between the leveling slab concrete and the right wingwall. Grout injection at these interfaces may have impacts on the assumed shear strength parameters of the assumed failure plane. Crack injection may have a negative impact on the hydrostatic uplift profile if the crack injection does not target the preferred upstream portion of the dam body and it is possible the leakage will only migrate to a different location along the downstream side of the dam.
5. The results of the compression test indicate that two of the concrete samples tested exhibited lower than assumed compressive strength.

4.1 Recommendations

Hatch recommends the following:

1. The condition of the overflow weir should be monitored over time during annual inspections. The current surface erosion of the weir does not pose a dam safety concern, however, should be monitored for possible accelerated deterioration.
2. It should be assumed that all of the anchor holes will need to be grouted and re-drilled to satisfy the pressure testing requirements. This assumption should be noted in the tender documents and reflected in the pricing schedule.
3. The condition of the concrete leveling slab downstream of the right wingwall should be monitored for further undermining. A repair plan can be considered at this time and should be implemented when undermining of the dam proper begins.
4. An investigation into a grouting program should be performed. The open concrete to bedrock and concrete to leveling slab crack injection should target the upstream surface of the dam.
5. The anchor head design will be modified to accommodate for the lower than assumed strength of the concrete. Revisions to the Bellrock Dam Post Tensioned Anchors Drawing H368596-0000-220-270-0001 are shown in Appendix B.

Appendix A: Concrete Core Compressive Strength Report

CONCRETE CORE TEST REPORT

Obtaining and Testing Drilled Cores for Compressive Strength Testing CSA A23.2 - 14C

PROJECT #: OMATB2283.2000
PROJECT: Hatch - Materials Lab Testing
DATE RECEIVED: 29-Nov-22
LOCATION: H369335 - Bellrock Dam Concrete Condition Assess.

CLIENT: Hatch
DATE REPORTED: 14-Dec-22
SAMPLED BY: Client
CONTRACTOR: n/a
CLASS: n/a
CONDITIONED: Dry

TESTED BY: J. Pasqua

TEST RESULTS

Lab No.:	C1806-22a	C1806-22a	C1806-22a			
Core No.:	A	B	C			
Location of Core:	H369335 - Bellrock Dam Concrete Condition Assess.					

Date Of Pour:	n/a	n/a	n/a			
Date Sampled:	n/a	n/a	n/a			
Date Tested:	13-Dec-22	13-Dec-22	15-Dec-22			
Age At Test (Days):	n/a	n/a	n/a			

length (mm)	As received	301.3	315.0	317.5			
	As tested	272.6	244.4	269.7			
Diameter *(mm)*		157.1	156.9	144.1			
Volume *(cm^3)*		5284.1	4725.4	4398.4			
Mass *(g)*		11482	9162	10098			
Density *(kg/m^3)*		2173	1939	2296			

Load *(N)*	205156	135584	415533			
Strength *(MPa)*	10.6	7.0	25.5			
Failure Type	3	2	3			
Corrected Strength *(MPa)**	10.4	6.8	25.2			

Note: *If length to diameter ratio (L/D) is less than 1.8

All portions of the testing performed by Wood E&IS is in accordance with current CSA procedures. Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request.

Prepared by: J Pasqua

Reviewed by: O.Lazic

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Burlington, ON, L7N 3W5

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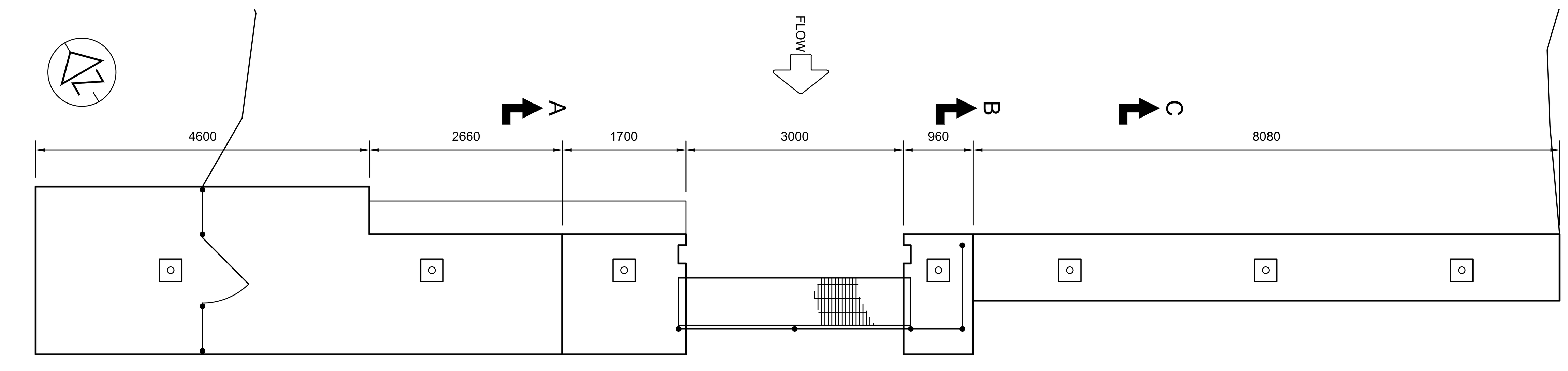
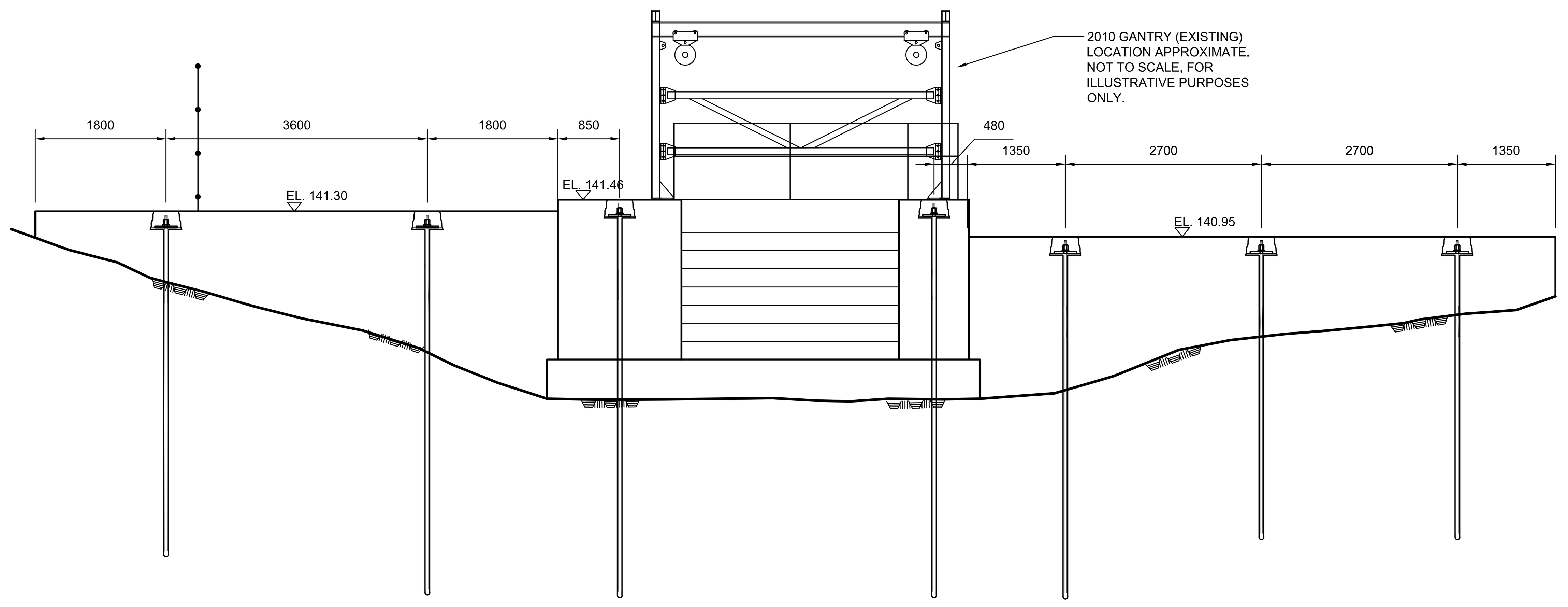
Appendix B:
Bellrock Dam Post Tensioned Anchors Drawing



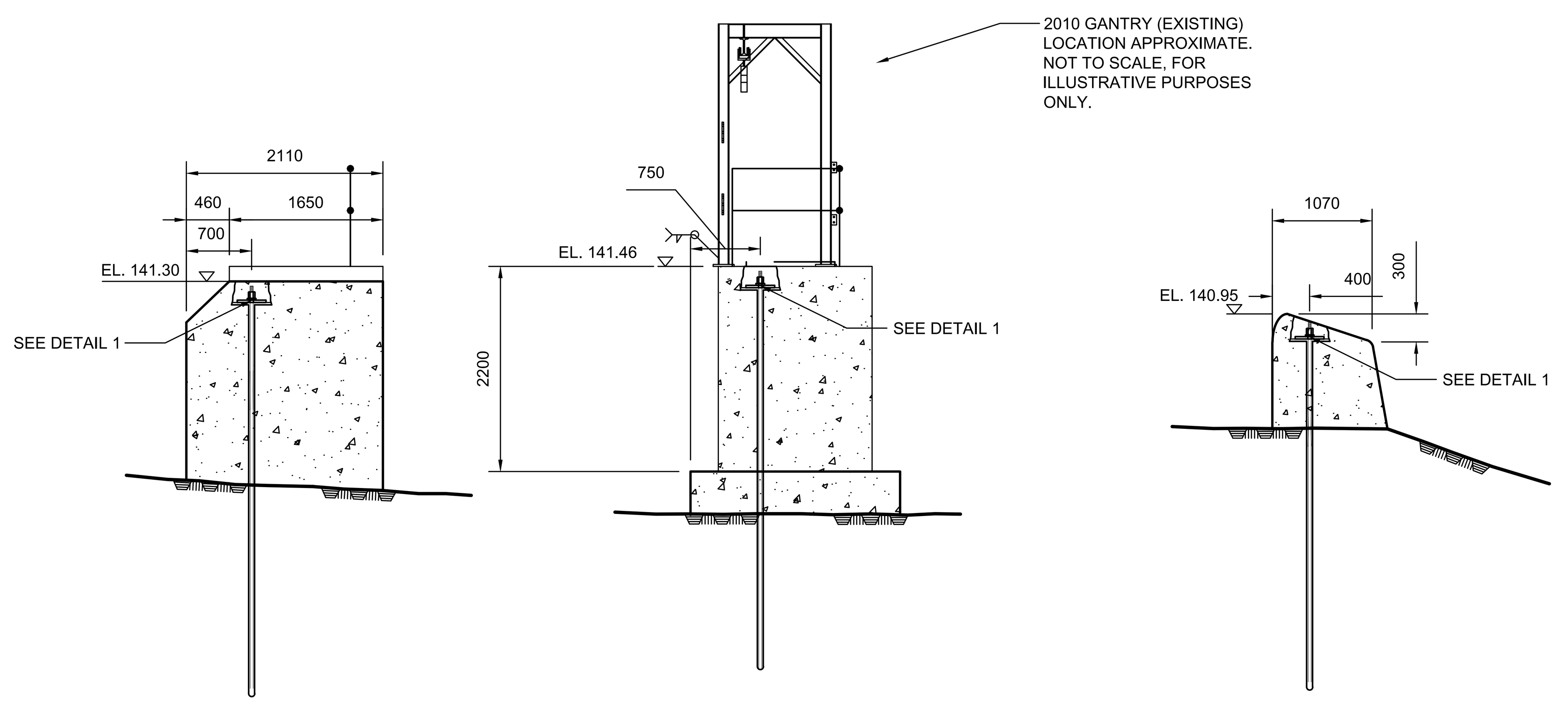
MAIN DAM PLAN VIEW

SCALE 1:50



ELEVATION

SCALE 1:50



SECTION A-A

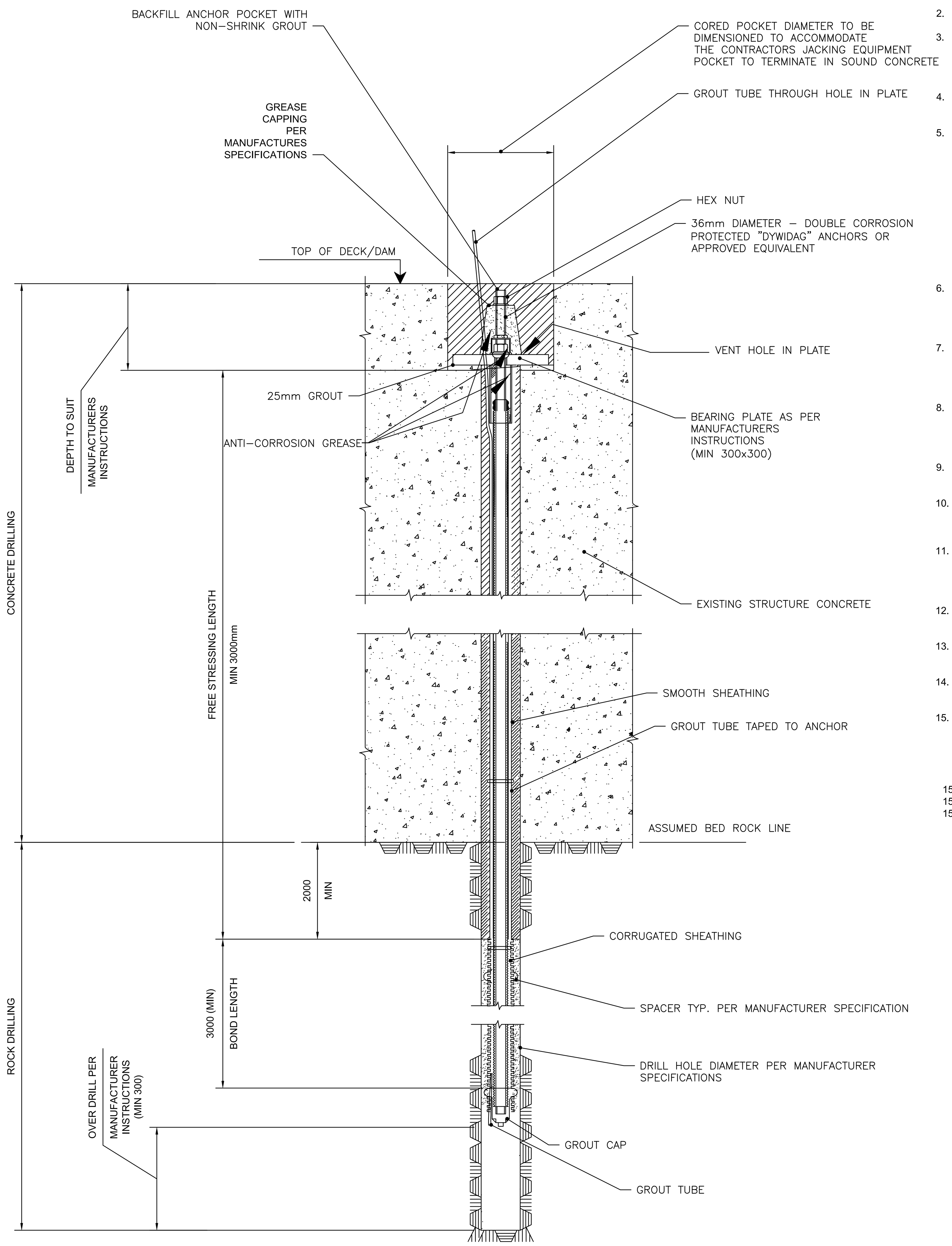
SCALE 1:50

SECTION B-B

SCALE 1:50

SECTION C-C

SCALE 1:50



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 WATERTIGHNESS 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	

NAME

SIGNATURE

ENG REG NUMBER

REVISION DATE

ORIGINAL DATE

REG. PROFESSIONAL

Reserved space for Issue Reason stamp applied by Document Control

No.	DESCRIPTION	BY	CHK'D	DATE
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	G. Ainslie	NR	
CHECKER	B. MacTavish		6/21/2022
DESIGN COORD.			
RESP. ENG.	B. MacTavish		6/21/2022
LEAD DISC. ENG.			
ENG. MANAGER			
PROJ. MANAGER	G. Ainslie		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 B

SHEET SIZE: B1

$USERNAME$ $DATE$ $FILE$ $TIME$

DWG. No. H368596-0000-220-270-0001