GEOTECHNICAL INVESTIGATION AND DESIGN REPORT KASHWAKAMAK LAKE DAM REPLACEMENT, TOWNSHIP OF NORTH FRONTENAC, ONTARIO

Project No.: CCO-23-3603

Prepared for:

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Revision 2 – June 2024

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1.0 INTRODUCTION

McIntosh Perry Consulting Engineers (McIntosh Perry) was retained by Mississippi Valley Conservation Authority (Client, MVCA) to complete a geotechnical investigation and design recommendation for the proposed replacement/rehabilitation of Kashwakamak Lake dam (Project) in the Township of North Frontenac. The dam is located on Kashwakamak Lake and forms part of the Mississippi River Watershed.

This geotechnical investigation and design recommendations are provided as part of the Class Environmental Assessment for the Kashwakamak Lake Dam at the request of the MVCA as outlined in the project RFP dated January 18, 2023. A proposal was submitted to the Client on March 03, 2023 and was accepted by the Client by means of signed Agreement dated march 20, 2023. A scope change "Scope Change #1" was requested on July 7, 2023 and was approved by the Client by means of signed back proposal on July 10, 2023.

The fieldwork was carried out between September 18 and 25 and comprised of four (4) boreholes advanced into the bedrock to a maximum depth of 9 meter below existing ground surface (mbgs) (El. 253.1 m) in BH23-4 which was drilled at the north (left) dam abutment. The other three boreholes were drilled downstream to a maximum drilling depth of 6.3 mbgs (El. 252.9 m) in BH23-5.

The purpose of the investigation was to explore the subsurface conditions at this site and to provide borehole location plans, record of borehole logs, and laboratory test results. This report provides anticipated geotechnical conditions influencing the design and construction of the proposed replacement and rehabilitation of the dam structure, as well as recommendations for foundation design.

This report is prepared for the sole use of the Client. The use of this report, or any reliance on it by any third party, is the responsibility of such a third party. This report is subject to the limitations shown in Appendix A. It is understood that the Project will be performed in accordance with all applicable codes and standards present within its jurisdiction.

2.0 PROJECT UNDERSTANDING

Kashwakamak Lake Dam is located approximately 8 km east of Femleigh on Lot 21, Concession IX, Clarendon Ward, North Frontenac Township. The dam is one of six major dams that acts as a flood and drought control structures along the Mississippi River, protecting people, property, infrastructure, and natural ecosystems both upstream and downstream of the dam. The dam was built in 1910 by the Mississippi River Improvement Company and its ownership and operation were transferred to MVCA in 1991. The dam also includes a small concrete saddle dam structure that is built to the north of the main dam structure. The saddle dam prevent and control the water from flowing through the natural channel behind the saddle dam into the marsh.

The dam underwent extensive maintenance in 1988 that was completed to the concrete surfaces of the weir. In 1995, MVCA undertook a repair program to reduce or eliminate the seepage around the earth embankment at the entrance to the dam. Terraprobe 1997 performed a limited geotechnical investigation and drilled five boreholes at the north (left) abutment to investigate water seepage through the rock. In 2000, MVCA undertook a grouting program and repairs to cracked and spalled concrete on the weir and the abutments. In 2002, the deck of the dam was replaced. In 2020, a proposed repair option of the Kashwakamak Lake Dam rehabilitation was prepared by Cleland Jardine Engineering Ltd. These repairs were not implemented. In 2020, MVCA conducted a Risk Assessment and in 2022 a Dam Safety Review (DSR) that concluded that structural issues at the dam needed to be addressed within 5 years based on MVCA website. The dam was inspected in 2022 by MVCA and seepage was observed through the embankment and was observed to come from through the rock.

The Dam Safety Review for the main dam and the saddle dam in 2022 completed by Hatch Ltd. included discussion on anticipated dam replacement/rehabilitation options. The report states that the existing dam concrete structures are deteriorating and in poor to fair condition. Major concrete repairs are required, specially at the overflow structure, showing signs of extensive spalled concrete surfaces at the upstream face and a severely deteriorated horizontal joint at the toe. The dam must undergo substantial rehabilitation or replacement within the next five years. The report concluded that both structures are founded on good to excellent quality bedrock foundation with adequate permeability, bearing capacity, strength, and rock quality. The report stated that no rock anchors or dowels are known to have been installed in the dam sections.

Based on the current condition of the Kashwakamak Lake Dam, it is understood to be in poor to fair condition and will require substantial rehabilitation or replacement within the next five years. A decision needs to be made on whether to rehabilitate (Option 1), or to decommission the existing and construct a new dam. It is also understood that replacement options may include replacing the dam with a similar structure at the same location (Option 2) or a new structure to the east of the existing dam (Option 3).

3.0 SITE CONDITIONS

3.1 Local Geology

Based on the published physiography maps of the area (Ontario Geological Survey), the site is located within the boundary zone of Georgian Bay Fringe from the south and Algonquin Highlands from the north. Surficial geology maps of Southern Ontario indicate that the surficial geology within the site is Precambrian bedrock and bounded by bedrock-drift complex in Precambrian terrain from the south and west. The site is also bounded by geological surficial formation of ice-contact stratified deposits composed of sand and gravel, minor silt, clay and till from the north.

Bedrock geology maps of Southern Ontario indicate the bedrock formation within the site is carbonate metasedimentary bedrock composed of marble, calc-silicate rocks, skarn, tectonic breccias from the Grenville super group and the Finton group.

3.2 Site Description

Kashwakamak Lake Dam is located approximately 8 km east of Femleigh on Lot 21, Concession IX, Clarendon Ward, North Frontenac Township. It was built in 1910.

The dam consists of a concrete overflow weir spillway at the south side and a sluiceway containing two stop log bays, each are 10 timber stop logs of 0.30 m high by 0.30 m wide by 3.43 m long, at the north. A small concrete saddle dam structure that is considered as a part of the Kashwakamak Dam built to the north of the main dam structure. The dam is provided with a floating safety/debris broom located upstream and a steel handrail around the control structure. Drawings of the Kashwakamak Lake Dam structures as received from the Client are included in Appendix G.

The surrounding area of the site comprised of a Kashwakamak Lake on the west side of the existing dam, forest area on the south side of the dam, a downstream flow on the east side of the site property and only north side is accessible for the dam site leading to Gutheinz Road. Recreational development along the shoreline of Kashwakamak Lake includes over 500 residences/cottages and at least five marinas/resorts. There are also several wetlands around the perimeter of the lake and manòmin (wild rice) crops downstream of the dam. The dam site location is shown in Figure 1, Appendix B.

4.0 FIELD INVESTIGATION

The staff of McIntosh Perry Consulting Engineers (McIntosh Perry) conducted an on-site visit prior to the planned drilling date and marked the proposed borehole locations; additionally, requisitions were submitted to Ontario One Call (ON1Call) to obtain public utility clearance locates, obtained private utility clearance locates and approval permits, and coordinated with the client regarding the intended geotechnical exploration drill-date.

The fieldwork was conducted between September 18 and 25 and comprised of four (4) boreholes advanced into the bedrock. BH23-4 was drilled at the north (left) dam abutment and advanced to a maximum depth of 9 mbgs (El. 253.1 m). The other three boreholes were drilled downstream. BH23-1 was drilled to a maximum depth of 6.5 mbgs (El. 252.8 m), BH23-2 was drilled to a maximum depth of 5.6 mbgs (El. 253.0 m), and BH23-5 was drilled to a maximum depth of 6.3 mbgs (El. 252.9 m). The other three boreholes were drilled downstream to a maximum drilling depth of 6.3 mbgs (El. 252.9 m) in BH23-5.

BH23-4 was drilled using a CME 75 truck-mounted drilling rig, outfitted with casing, while the rest of the boreholes were drilled using portable Hilti Drill. The equipment used for drilling was owned and operated by Ohlmann Geotechncial Services (OGS) of Almonte, Ontario. The bedrock was cored and sampled in all boreholes from the top of the encountered bedrock surface to the bottom of the boreholes. The bedrock was cored and sampled in BH23-1 from the ground surface (El. 259.3 m) to 6.5 mbgs (El. 252.8 m), in BH23-2 from the ground surface (El. 258.6 m) to 5.6 mbgs (El. 253.0 m), in BH23-4 from 0.4 mbgs (El. 261.7 m) to 9.0 mbgs (El. 253.1 m), and in BH23-5 from ground surface (El. 259.2 m) to 6.3 mbgs (El. 252.9 m). NQ size rock cores were obtained using diamond drilling and wireline tooling. Rock cores were retrieved in double-walled NQ coring methods.

Packer testing was performed in all boreholes. The test was performed from the bottom of boreholes towards the top of boreholes. The first test in each borehole was performed within the bottom 1.5 m of the hole. Then the bladder of the Packer test system was pulled up adding another 1.5 m to the tested section, except in BH23-2. In BH23-2, the first tested section was from 1.1 mbgs to the bottom of the borehole, and the second tested section was from 2.6 mbgs to the bottom of the borehole. The procedure was repeated up to the last 1.5 m of the hole near the ground surface. The results of the Packer tests are summarized in Section 6.3 and in Tables D.1 to D.12, in Appendix D.

A 51 mm diameter standpipe monitoring well was installed in BH23-4 with screen installed in the bedrock. The well was protected in flush-mount caps. Details and location information of the well are provided in Section 6.2 and summarized in Tables 6-2.

The bedrock core hole was sealed with bentonite holeplug and the boreholes were backfilled with auger cuttings and holeplug and restored to the original ground surface with cold patch asphalt. The boreholes were surveyed with a GPS unit to record their locations and elevations. Borehole locations are shown in Figure 2, included in Appendix B.

Table 4-1: Borehole Designations, Locations, and Depth

BH No.	Drilling Date	Coordinates (UTM Zone 18T)			Borehole Termination		Remarks
		Northing	Easting	Surface El. (m)	Depth (mbgs)	El. (m)	
23-1	Sept. 22-25, 2023	4972860	345362	259.3	6.5	252.8	- Bedrock was cored from the ground surface. - Rock core ~ 6.5 m
23-2	Sept. 20, 2023	4972859	345352	258.6	5.6	253.0	- Bedrock was cored from the ground surface. - Rock core ~ 5.6 m
23-4	Sept. 18, 2023	4972865	345350	262.1	9.0	253.1	- 0.4 m topsoil - Well installed in bedrock

							- Rock core ~ 8.6 m
23-5	Sept. 19, 2023	4972839	345348	259.2	6.3	252.9	- Bedrock was cored from the ground surface. - Rock core ~ 6.3 m

The field investigation, including drilling and sampling, was supervised on a full-time basis by McIntosh Perry. All boreholes were logged during the drilling progress. All samples were labelled by waterproof paper one by one as they retrieved. All soil samples were preserved in double plastic bags to mitigate the risk of moisture loss during transportation to the geotechnical laboratory. Rock cores were laid and labelled in specialty boxes made for rock core transportation. The Rock Quality Designation was measured for the first time in the field immediately after drilling to reduce the measurement errors caused by transportation induced damages to the rock cores.

5.0 LABORATORY INVESTIGATIONS

Geotechnical Laboratory testing on representative rock cores was performed at McIntosh Perry Geotechnical Laboratory and included rock compressive strength on 10 rock cores. The laboratory tests were performed in accordance with American Society for Testing Materials (ASTM) test procedures.

Paracel Laboratories Ltd., in Ottawa, Ontario carried out chemical testing on a representative surface water sample to determine the potential susceptibility to corrosion to ductile iron elements and concrete attack parameters. The chemical parameters consisted of pH, chloride, sulphate, and resistivity. Laboratory test results are included in Appendix E.

As per the request of the MVCA, the rest of the soil samples and rock cores will be stored in McIntosh Perry storage facility until McIntosh Perry receives a further notice from MVCA to dispose them.

6.0 SUBSURFACE CONDITIONS

6.1 Subsoil Conditions

The site stratigraphy at the drilled borehole locations consisted of a thin layer of topsoil encountered in BH23-4 only, underlain by bedrock. In all other boreholes, the bedrock was observed at the ground surface and cored and sampled to the bottom of the boreholes.

The topsoil and bedrock that were encountered during the course of the investigation, together with the field and laboratory test results are shown on the borehole records included in Appendix C. Laboratory test results are included in Appendix E. Description of the strata encountered are given below.

6.1.1 Topsoil

A thin topsoil layer of approximately 0.4 m was observed in BH23-4 only on the dam north (left) abutment. No soil testing was performed on the topsoil sample.

6.1.2 Bedrock

Bedrock was encountered and cored in all boreholes as described in Table 6-1. The bedrock was observed at the ground surface in BH23-1, 23-2, and 23-5 and was observed below the topsoil in BH23-4 on the north (left) abutment). The bedrock was cored and sampled to the bottom of all boreholes.

During the core drilling, measurements including Total Core Recovery (TCR) and Rock Quality Designation (RQD) were carried out as part of the rock quality classification. TCR is defined as the sum of all recovered rock core pieces from a core run expressed as a percent of the total length of the core run. The RQD is defined as a percentage of the sum of the intact core pieces over 100 mm divided by the total length of core run. The TCR and RQD for the rock cores are presented in the borehole log records in Appendix C.

Based on the retrieved rock cores from borehole, the bedrock was identified as Carbonate Metasedimentary bedrock diagonally parting Marble. It was observed to be slightly weathered and slightly fractured with moderately close, horizontal to diagonal joints. A few vertical cracks were observed in BH23-4 between El. 259.5 and 259.3 m, between El. 256.9 and 256.7 m, and between El. 256.2 and 255.9 m. Also, vertical cracks were observed in BH23-5 between El. 255.6 and 255.4 m, and between El. 253.8 and 253.6 m.

The Carbonate Metasedimentary bedrock was observed to be strong, grey to dark grey with white bands of Marble, medium to thinly bedded. In BH23-1 and 23-2, the bedrock was observed to have good to excellent quality based on RQD value of 75 to 100%. In BH23-4 and 23-5, the bedrock quality was fair to excellent based on RQD value of 56 to 98%. The rock cores are shown in Figures 3, Appendix C.

Table 6-1: Bedrock Core Summary

BH No.	Ground Surface El. (m)	Bedrock Surface El. (m)	Sound Bedrock El. (m)	Rock Core #	El. (m)	Recovery (%)	RQD (%)	UCS (MPa)
23-1	259.3	259.3	259.3 – 252.8	RC-1	259.3 – 258.2	100	93	
				RC-2	258.2 – 257.1	100	89	164
				RC-3	257.1 – 256.2	100	94	
				RC-4	256.2 – 255.1	98	84	167
				RC-5	255.1 – 254.1	100	100	
				RC-6	254.1 – 253.4	98	83	177
				RC-7	253.4 – 252.8	100	92	
23-2	258.6	258.6	258.6 – 253.1	RC-1	258.6 – 257.4	99	91	201

BH No.	Ground Surface El. (m)	Bedrock Surface El. (m)	Sound Bedrock El. (m)	Rock Core #	El. (m)	Recovery (%)	RQD (%)	UCS (MPa)
				RC-2	257.4 – 256.2	92	92	
				RC-3	256.2 – 255.5	100	100	
				RC-4	255.5 – 254.6	97	76	208
				RC-5	254.6 – 253.8	100	97	
				RC-6	253.8 – 253.6	94	75	
				RC-7	253.6 – 253.1	100	100	194.8
23-4	262.1	261.8	261.8 – 253.1	RC-2	261.8 – 261.5	100	63	
				RC-3	261.5 – 260.9	98	95	
				RC-4	260.9 – 260.6	100	67	
				RC-5	260.6 – 259.2	86	56	
				RC-6	259.2 – 257.6	104	82	
				RC-7	257.6 – 256.1	100	97	211
				RC-8	256.1 – 254.6	100	73	
				RC-9	254.6 – 253.1	98	91	
23-5	259.2	259.2	259.2 – 252.9	RC-1	259.2 – 258.3	100	87	
				RC-2	258.3 – 257.2	100	92	211
				RC-3	257.2 – 256.4	95	95	
				RC-4	256.4 – 255.5	102	81	173
				RC-5	255.5 – 254.4	100	93	
				RC-6	254.4 – 253.8	96	64	126
				RC-7	253.8 – 252.9	103	98	

6.2 Groundwater

Groundwater was not observed during the site of investigation in open BH23-1, 23-4 and 23-5. However, minor artesian pressure observed in BH23-1 which dissipated shortly after completing drilling. One standpipe well was installed in BH23-4. These boreholes were denoted with “MW”. The groundwater was measured in the well on September 26, 2023. The measured groundwater depth in the well with standpipe well information is presented in Table 6-2.

Groundwater levels are expected to fluctuate due to extreme weather events and seasonal changes.

Table 6-2: Monitoring Wells Summary

BH/MW ID	Screen Interval El. (m)	Groundwater Level Observation				Remarks
		Installation Date	Measurement Date	Depth (mbgs)	GW Elev. (m)	
BH23-4 MW	258.3 – 253.1	Sept. 18, 2023	Sept. 26, 2023	1.5	260.6	Screen in the bedrock

6.3 Packer Testing

Twelve (12) Single-Packer tests were performed in total concurrently with the geotechnical drilling program in the drilled boreholes. The tests were performed using a constant head (Lugeon) packer injection test method. The boreholes were first drilled, and the bedrock was cored to the planned depths. Then cumulative single packer tests were performed from the bottom of the boreholes towards the top of boreholes.

The test procedure involved lowering a single packer assembly inside the open boreholes to the top of the test interval. The test section then was isolated by inflating the packer bladder using pressurized water. Once a successful seal was established, water was pumped into the isolated test interval through the injection pipe until a constant differential head and inflow rate were established.

The test was performed by applying a total of three ascending water pressure steps (i.e., 10, 15, and 20 psi) followed by two descending water pressure steps (i.e., 15, and 10 psi) within each test interval. A regulated constant head achieved by controlling the injection flow rate using a bypass valve. For each step, the pressure and injected quantity of water was recorded at one-minute intervals for a total of five (5) minutes until it had stabilized. During the Packer testing, difficulties associated with maintaining a steady pressure were encountered in BH23-4 in the first test, which were fixed, and the test proceeded.

The first test in each borehole was performed within the bottom 1.5 m of the hole. Then the packer bladder was pulled up adding another 1.5 m to the tested section, except in BH23-2. In BH23-2, the first test section was from 1.1 mbgs to the bottom of the borehole, and the second test section was from 2.6 mbgs to the bottom of the borehole. The procedure was repeated up to the last 1.5 m of the hole near the ground surface. The results of the Packer tests are summarized in Tables D.1 to D.12 in Appendix D.

6.4 Chemical Analysis

Chemical analyses were conducted by Paracel Laboratories in Ottawa, ON, to determine the resistivity, pH, sulphate and chloride content of a water sample collected from the lake. A summary of chemical analysis results is shown in Table 6-3 and the laboratory results are shown in Appendix E.

Table 6-3: Chemical Analysis Summary

SAMPLE	DEPTH (m)	Chemical Analysis			
		pH (pH units)	Resistivity (Ohm.cm)	Chloride (%)	sulphate (%)
Surface water	--	7.9	9170	0.0005	0.0003

7.0 DISCUSSIONS AND RECOMMENDATIONS

Based on the results of the geotechnical field and laboratory investigation performed, the following discussion is provided to assist the Client and the Designer with the proposed replacement/rehabilitation of Kashwakamak Lake Dam Project. The recommendations provided within this report are based on our understanding of the proposed Project which is summarized above in "Section 2" and through the interpretation of factual information obtained from the boreholes advanced during this subsurface investigation. If any of these understandings change, McIntosh Perry should be contacted to assess the implications of those changes on the recommendations provided herein.

Based on the subsurface conditions observed in the boreholes, and assuming they are representative of soil and bedrock conditions across the Site, the most important geotechnical considerations for the design and construction of the proposed dam are expected to be the following:

- **Proposed Replacement and Rehabilitation Options:** It is understood that MVCA is considering three replacement and rehabilitation options. The first option is to rehabilitate the existing dam structure which may necessitate executing grouting at the abutment and the dam foundation. The second option involves replacing the existing main structure with a similar one constructed at the same place. The third option is to construct a new dam to the east of the existing at the downstream side while taking advantage of the existing dam to control the surface water during construction.

- **Bedrock Subgrade Preparation:** Information about the foundation level of the existing dam is approximate. It was assumed that the proposed replacement will be also constructed on sound bedrock foundation at elevations varies from approximately El. 257 to 258 m at the Sluiceway to 259 to 260 m at the abutments. The existing saddle dam is assumed to be constructed on sound rock at El. 260 to 261 m. It is also assumed that the proposed saddle dam replacement will be founded on sound bedrock at El. 260 to 261 m. These elevations are estimated based on the provided survey in "Drawing No. 1434-19-01" by Ecos Garatech Consulting Engineers dated Oct. 1998. The bedrock subgrade should be cleaned of any loose or unstable rock pieces from the dam influence zone. Lean mixed concrete should be used for levelling the sound bedrock. The lean mix concrete shall have a minimum compressive strength of 30 MPa. If lean mixed concrete is used below dam at the bedrock surface (i.e., not confined within bedrock), it must extend a minimum of 0.3 m beyond the edge of the dam and then downward at a 1H:1V. The bedrock subgrade has to be approved by the geotechnical engineer.

- **Seismic Site Classification:** The proposed dam will be designed in accordance with Part Four of CSA S6 2020 . Based upon the results of the site investigation, the subject site for the proposed building can be designed to "Site Class C to B" in accordance with Table 4.4.3.2 of the CSA 2020, and subject to the limitations of the code. It should be confirmed by structural engineer based on shear wave velocity.

- **Temporary Construction Dewatering:** Effective water control and management prior to and during construction will be required for the dam replacement options. Water quantities will depend on seasonal conditions, depths of excavations, and the duration that excavations are left open. The water level in the lake may fluctuate in response to extreme weather events and seasonal changes. Temporary water cut-off system such as cofferdams or secant pile wall should be constructed around the excavation and sump pumps may be used to drain the water from the confined zone during the construction. However, it is the Contractors' responsibility to design the dewatering method based on the expected water levels in the lake and based on the low permeability of bedrock. Recommendations for appropriate dewatering measures to effectively control the water levels shall be provided by a specialized dewatering contractor. It is recommended to plan the excavation during the dry season to reduce the dewatering pumping requirements. The groundwater disposal should be performed in accordance with applicable regulations. Assessment of the dewatering requirements and the need for registration on the Environmental Activity and Sector Registry (EASR) or a Permit to take Water (PTTW) should be carried out by specialists experienced in this field.

- **Rehabilitation of the Existing Dam:** It is understood that rehabilitation of the existing dam structure may be considered for this project. There is limited information available with respect to the existing dam foundation. For both replacement options "Option 2 and Option 3", bedrock area grouting will be necessary for the foundation bedrock. Grouting is recommended at the upstream face for seepage cut-off through the foundation. Grouting at the north (left) abutment is recommended as water seepage through bedrock was reported within the north (left) abutment and also minor artesian pressure was observed in open BH23-1 which was dissipated shortly after finishing the drilling. It is also recommended to perform grouting at the south (right) abutment of the dam.

- **Existing Dam Removal:** It is understood that the new replacement options include removing the existing dam and either constructing a new dam in place the existing dam or at a new location to the east of the existing. It is understood that rapid drawdown as a result of the dam removal is not allowed or expected. For replacement "Option 2" with a new structure at the same location, a temporary cofferdam or a secant pile wall can be utilized at the upstream to allow for the removal of the old dam and the construction of the new structure. For replacement "Option 3" with a new structure to the east of the existing dam, the existing dam can act as coffer dam to control the water flowing and allowing for the construction of the new dam. It is also understood that the existing saddle dam and the existing natural channel may be used as a contingency bypass to control the water level in the lake during construction. The flow control during removal of the existing concrete dam shall be outlined by the geomorphologist.

It is understood that Hazard Potential Classification (HPC) and design flood for the replacement and rehabilitation of the dam according to the CDA and MNRF guidelines have indicated in Hydraulic Analysis Memorandum.

The comments made regarding the construction of the proposed dam replacement/rehabilitation are intended to highlight those aspects which could impact or affect the detail design of the proposed structure, for which special provisions may be required in the Contract Documents. Comments related to construction aspects are not intended to dictate construction equipment or methods. Relevant parties should make their own interpretation of the factual data presented in the report. Interpretation of the data presented may affect equipment selection, proposed construction methods, and scheduling of construction activities.

7.1 Site Preparation and Grading

For replacement option "Option 2", the existing dam shall be demolished to allow for the construction of the new proposed dam. The demolition the existing structure and the construction of the new dam shall be conducted within the confines of a temporary cofferdam, or a secant pile wall designed and installed in accordance with OHSA. The flow control during removal of the existing concrete dam shall be outlined by the geomorphologist.

For replacement option "Option 3", it is understood that the existing dam is planned to be used to control the water level during the construction phase. Therefore, it will not be demolished until the construction of the new dam to the east of the existing is complete.

The site should be graded in the early stages of construction to provide positive control of surface water and directing it away from excavations and subgrades. The Contractor should take appropriate measurements for collection and disposal of surface and groundwater and runoff including an adequate pumping system.

7.1.1 Buried Services

Public and private utility owners should be notified prior to the commencement of any construction activities. Existing underground utilities in the vicinity of the proposed excavation should be reviewed before commencing any excavation works to identify potential damage hazards due to the proposed excavation. Existing utilities that are excavated or exposed as part of the construction will need to be supported and rerouted during the construction. The contractor shall inform owners of all existing utilities before proceeding with excavation. The utility owners may provide the permissible deformation that a particular utility may tolerate. Shoring shop drawings should be stamped by a professional engineer.

7.2 Excavation

7.2.1 Existing Topsoil

Topsoil shall be removed from within the footprint of the proposed structure, to expose the bedrock subgrade. Any over excavation shall be leveled by lean concrete or a concrete mix of the same strength as the foundation system.

The excavated materials and any corresponding excess soils should be disposed of in accordance with all applicable environmental legislation. Excess soils management and evaluation of the environmental quality of subsoils is not within the scope of this geotechnical investigation.

7.2.2 Bedrock Excavation

For excavations into bedrock, the bedrock was observed to be in excellent quality based on RQD values of the retrieved rock cores. In general, sound bedrock was observed in all drilled boreholes at the bedrock surface. The bedrock quality and site-specific requirements need to be assessed during construction by the geotechnical engineer.

The excavations for the proposed dam and abutments will extend to sound bedrock. All excavations must be undertaken in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA), Regulations for Construction O.Reg. 213/91, with specific reference to acceptable size slopes and stabilization requirements. For planning purposes, a weathered bedrock is recommended to be treated as a Type 2 Soil. Sound rock would generally be self-supporting, however, as a precautionary measure, it should be back-sloped at 10V:1H. All rock excavations should be scaled, to remove loose rock fragments to ensure safe working conditions. All rock faces should be reviewed by a geotechnical engineer to look for loose pieces and wedge failures. Rock bolting for worker safety may be necessary depending on the layout and field condition at that time.

The stability of the excavation side slopes is highly dependent on the Contractor’s methodology and layout. Bedrock excavation will require pneumatic or hydraulic breakers such as hoe-rams or heavy rock excavation equipment capable of breaking and ripping sound Carbonate Metasedimentary bedrock. Line drilling for this site can be considered and can be done by drilling 75 to 100 mm holes at 200 to 300 mm spacing but this should be independently assessed by the Contractor. Bedrock excavation should be carried out as per OPSS.MUNI 403.

7.2.3 Subgrade Preparation

The excavations for the proposed dam replacement are generally expected to extend down to sound bedrock. Based on the recent boreholes the sound bedrock is expected to be encountered at shallow depth near the ground surface. The sound bedrock was observed in the cored boreholes at elevations range between which is corresponding to approximate El. 258.6 to 259.3 m in BH23-1, 23-2, and 23-5, while in BH23-4 which was drilled on the north (left) abutment, the sound bedrock was observed at approximately 262.1 m. Moderate bedrock excavation is expected to expose sound bedrock which is expected to generate a manageable amount of excavated rock materials.

Subgrade preparation for footings founded on rock will involve the removal of all soils and weathered bedrock to expose a sound bedrock. Any pieces of rock that can be manipulated by conventional excavation equipment should be removed, and as directed by the geotechnical engineer. Final subgrade surfaces should be brushed and cleaned. The exposed bedrock surface should be examined and approved by the geotechnical engineer to confirm the competency to support the design bearing pressures. Lean mixed concrete should be used for levelling the sound bedrock. The lean mix concrete shall have a minimum compressive strength of 30 MPa. If lean mixed concrete is used below dam at the bedrock surface (i.e., not confined within bedrock), it must extend a minimum of 0.3 m beyond the edge of the dam and then downward at a 1H:1V.

Confirmation of bedrock quality during construction will require the contractor to perform probing of the bedrock using 50 mm diameter drill holes drilled to a depth of 1.5 m within the footprint of the dam. These holes will need to be reviewed by the geotechnical engineer to confirm that no significant mud seams or voids exist at the proposed dam replacement location. If mud seams are found, localized areas may need to be lowered below the mud seam. The locations of these probe holes should be selected under the direction of the geotechnical engineer during construction. Contractors should plan for one probe per pad footing and a minimum or 1 probe every 6 m in strip footings/dam.

7.2.4 Temporary Construction Dewatering

It is understood that the existing saddle dam and the existing natural channel behind it may be used as a contingency bypass to control the water level in the lake during construction.

Groundwater was observed in the monitoring well installed in BH23-4 on the north (left) abutment, and groundwater was at El. 260.6 m which is the approximately same as the water level in the lake upstream. Water quantities will depend on seasonal conditions, depths of excavations, and the duration that excavations are left open. The water level in the lake may fluctuate in response to extreme weather events and seasonal changes. Temporary water cut-off system such as cofferdams or secant pile wall should be constructed around the excavation and sump pumps may be used to drain the water from the confined zone during the construction. However, it is the Contractors' responsibility to design the dewatering method based on the expected water levels in the lake and based on the low permeability of bedrock. Recommendations for appropriate dewatering measures to effectively control the water levels shall be provided by a specialized dewatering contractor. It is recommended to plan the excavation during the dry season to reduce the dewatering pumping requirements. All construction activities and grouting shall be carried in dry conditions.

The groundwater disposal should be performed in accordance with applicable regulations. A PTTW from the Ontario Ministry of the Environment, Conservation and Parks (MECP) will be required if the quantity of water to be pumped from the Site exceeds 400,000 L/day. For expected groundwater extraction between 50,000 and 400,000 L/day, an EASR permit is adequate. Assessment of the dewatering requirements and the need for registration on the EASR or a PTTW should be carried out by specialists experienced in this field.

7.2.5 Temporary Water Cut-off System Installation and Design

The proposed replacement options of the existing dam and the saddle dam will require dry condition during demolition, excavation and construction. For Option 3, the existing dam can act as a cofferdam during the construction phase. The removal of the existing dam can be done once the new dam is completed with the aid of temporary cofferdam.

For Option 2 and saddle dam, demolition of the existing dam and constructing the new replacement shall be performed within the confines of a temporary cofferdam, or a secant pile wall designed and installed in accordance with OHSA. Based on the encountered bedrock during the site investigation and based on our understanding of the site geology, sheet pile cofferdam is not feasible for this site. Temporary water cut-of alternatives could include portable cofferdam, inflatable bladder, sandbags and plastic sheeting or similar systems. Limitations associated with using such systems are that they can only provide protection up to a limited height. Also, as with other cofferdam methods, dry condition cannot be fully achieved within the work area inside.

A secant pile wall may also be considered. A secant pile wall consists of overlapping (secant) piles to form structural or cut-off walls and achieve the required water tightness. This option involves coring the bedrock and installing reinforced and nonreinforced piles. The secant pile wall is permanent and more expansive but can provide flexibility with respect to the water height behind it.

The contractor should hire an experienced professional geostructural engineer to provide a detailed design for the cut-off system considering the space restrictions, estimated costs, and availability of materials. The designer must take into consideration the loads from water pressure, and seismic loading. Also, it should consider the freeze-thaw action, expansion and contraction of cut-off elements, and construction vibrations.

The General Contractor should count for this in their design and choose suitable system and construction method for this site. The General Contractor shall choose the most suitable option based on their experience, available equipment, and their understanding of the factual information provided in this report. Shop drawings should be submitted to the designers and reviewed by the geotechnical engineer well in advance of mobilization.

7.2.6 Permeability of Bedrock and Packer Testing

Hydraulic conductivity of bedrock was calculated from the analysis of Packer testing performed in the cored boreholes. The calculated hydraulic conductivity values of the bedrock spanned over three orders of magnitude from 7.84×10^{-6} m/sec in BH23-4 to 8.44×10^{-9} m/sec in BH23-5, with a geometric mean value for all the Packer tests of 3.67×10^{-7} m/sec.

A summary of the calculated hydraulic conductivity values for the bedrock at Kashwakamak Dam is presented Table 7-1.

Table 7-1: Summary of Calculated Hydraulic Conductivity for Bedrock

BH No.	Test Number	Interval El. (m)	Hydraulic Conductivity (m/sec)			Flow Behavior	Representative Lugeon Value (l/min/m)
			Minimum	Maximum	Average		
23-1	Test 1/3	254.4 - 252.8	1.28×10^{-6}	2.49×10^{-6}	1.86×10^{-6}	Void Filling	12.15
	Test 2/3	255.9 - 252.8	1.55×10^{-6}	4.09×10^{-6}	2.35×10^{-6}	Turbulent	13.62
	Test 3/3	257.5 - 252.8	1.29×10^{-6}	2.35×10^{-6}	1.59×10^{-6}	Void Filling	9.73
23-2	Test 1/2	258.1 - 253.7	7.51×10^{-8}	4.48×10^{-7}	1.92×10^{-7}	Wash-out	3.41
	Test 2/2	256.6 - 253.7	3.75×10^{-7}	7.18×10^{-7}	5.99×10^{-7}		5.97
23-4	Test 1/4	254.6 - 253.1	1.31×10^{-6}	7.84×10^{-6}	3.78×10^{-6}	Void Filling	14.69
	Test 2/4	256.1 - 253.1	6.50×10^{-7}	1.49×10^{-6}	9.95×10^{-7}	Wash-out	12.30
	Test 3/4	257.7 - 253.1	7.21×10^{-8}	5.06×10^{-8}	6.21×10^{-8}	Void Filling	0.38
	Test 4/4	259.2 - 253.1	2.52×10^{-7}	1.45×10^{-7}	1.80×10^{-7}	Void Filling	1.04
23-5	Test 1/3	257.3 - 255.8	2.65×10^{-8}	1.47×10^{-7}	7.3×10^{-8}	Void Filling	0.70
	Test 2/3	258.8 - 255.8	2.72×10^{-8}	1.01×10^{-7}	5.95×10^{-8}	Dilation	0.34
	Test 3/3	260.4 - 255.8	8.44×10^{-9}	1.19×10^{-7}	7.66×10^{-8}	Void Filling	0.06

Five flow behaviors through bedrock are typically expected. Darcy's law is predicated on laminar flow (termed Darcian flow) where, for a given geometry (for example a borehole test section), the injection flow rate and the access head pressure have a linearly proportional relationship. It is generally accepted that Packer tests in rock where flow is predominantly via fine fracture networks are dominated by Darcian "Laminar" flow.

However, where more open fractures are present, allowing higher flow rates, non-Darcian (Turbulent) flow will occur, and the flow rate will increase under-proportionally with excess head, as energy is lost to turbulence.

Dilation flow is an indication of maximum pressure that can be applied without risk of dilating or displacing existing fractures/joints (known as hydrojacking) in the rock around or above the test section.

Wash-out flow behavior may be explained as an increase in hydraulic conductivity of the rock caused by the test, due to movement/erosion of infill in fractures in such a way that they do not block flow paths, or permanent rock movements caused by the testing. It could also interpret as leakage past the packers that disturbs or erodes the rock, so that leakage paths do not close with reduced excess head.

Void Filling flow behavior may be explained as a decrease in hydraulic conductivity of the rock caused by the test, with possible mechanisms including:

- Water filling and pressurising of voids or discontinuities not linked to a wider network,
- Movement or swelling of infill in fractures in such a way that they become trapped and block flow paths, and
- Clogging of rock fractures due to use of dirty water for injection.

Based on the Packer testing results, the following is noted:

- The calculated hydraulic permeability in BH23-1, which was drilled downstream on the north channel bank, was relatively high comparing to other boreholes and was generally in the order 1.28 x 10-6 m/sec to 4.09 x 10-6 m/sec with “Void Filling” flow behavior. As noted earlier, minor artesian pressure was observed in open BH23-1 which was dissipated shortly after finishing the drilling.
- The calculated hydraulic permeability in BH23-2, which was drilled downstream behind the north (left) abutment of the dam, was generally in the order 7.51 x 10-8 cm/sec to 3.75 x 10-7 m/sec with “Wash-out” flow behavior.
- The highest calculated hydraulic conductivity value, 7.84 x 10-6 m/sec was in BH23-4, which was drilled the north (left) abutment of the dam, between El. 254.6 - 253.1 m in test “Test 1/4”. The flow behavior was observed to be “Void Filling” behavior which was corresponding to a Lugeon value of 14.69. A “Wash-out” flow behavior observed in the second test “Test 2/4” between El. 256.1 - 253.1 m. In tests “Test 3/4, and Test 4/4”, the hydraulic conductivity observed to become lower with “Void Filling” behavior.
- The calculated hydraulic permeability in BH23-5, which was drilled downstream behind the spillway dam, was generally in the order 8.44 x 10^{-9} m/sec to 1.47 x 10^{-7} m/sec with “Void Filling” flow behavior for “Test 1/3 and Test 3/3”. A “Dilation” flow was observed during “Test 2/3”.

The Packer testing results summarized in this section are preliminary in nature and shall be referred to for general understanding only.

7.2.7 *Bedrock Grouting*

As discussed earlier, the rock quality was observed to be generally in fair to excellent condition based on RQD values of the retrieved rock cores. Vertical and diagonal fractures were observed in rock cores retrieved from

BH23-4 which was drilled on the north abutment. Vertical cracks were also observed in rock cores retrieved from BH23-5 which was drilled downstream. Discoloring was observed around the edges of these cracks which may indicate chemical erosion. Discontinuities in the rock mass may be joined to create a continuous seepage path through bedrock. Previous observations of water seepage through the north abutment were reported.

Bedrock grouting is recommended at the upstream face for seepage cut-off through the foundation. Also, grouting at the north (left) abutment is recommended as water seepage through bedrock was reported within the north (left) abutment and also minor artesian pressure was observed in open BH23-1 which was dissipated shortly after finishing the drilling. It is also recommended to perform grouting at the south (right) abutment of the dam.

The design of the grouting program is the responsibility of the General Contractor. It is important to emphasize that the Packer testing results summarized in this report in Section “7.2.6 Permeability of Bedrock and Packer Testing” are preliminary and the Contractor may refer to the Packer testing results for general understanding only. The Contractor shall perform field testing that are suitable for the Contractor’s construction and grouting methods including, but not limited to, performing Packer testing to determine grout pressure, grout holes depths and spacing based on their test results for the design of the grouting program. High mobility grout is recommended to seal these cracks. Grouting pressures shall not exceed the overburden pressure. Grouting shall be carried out in dry conditions.

7.3 Foundations

7.3.1 Geotechnical Bearing Resistance for the Proposed Building

Provided there are no continuous soil-filled seams or mud seams present at shallow depth in the sound bedrock below the founding level, conventional pad and strip footings founded on the sound bedrock, a factored bearing resistance of 1,000 kPa under Ultimate Limit States (ULS) conditions is recommended for the proposed dam. This includes for a geotechnical resistance factor of $\Phi = 0.5$. The factored ULS bearing resistance was estimated using the Rock Mass Rating (RMR) method by Bieniawski (1989).

The size of the selected footings shall be determined by the structural engineer. The selected size of the footing shall have adequate compressive strength to provide resistance to the structural loads from the proposed replacement. Designers should keep footing dimensions to a minimum of 1.5 m for pad footings, and 1.0 m for strip footings regardless of the bearing pressure being used.

Provided the bedrock surface is properly cleaned of soil and weathered material at the time of construction, settlement under the ULS condition is expected to be negligible. Therefore, there is no corresponding design bearing pressure recommended under Serviceability Limit State (SLS) conditions for bedrock.

Subgrade preparation shall be in accordance with Section “7.2.3 Subgrade Preparation”.

7.3.2 *Lateral Resistance of the Proposed Dam*

The factored ultimate resistance of the footings to lateral loading ‘shear resistance for sliding’ across the interface between the footing, and the bedrock may be calculated using Mohr-Coulomb criterion below with load and resistance factors given in Table 7-2.

$$\tau = f_c c' + (\sigma - f_U U) f_\phi tan\phi'$$

where c' is cohesion, ϕ' is shearing angle, U is water pressure, and σ is the normal stress on the sliding surface.

Table 7-2: Minimum Lateral Load and Resistance Factors after Meyerhof (1984) (Wyllie 2009)

Category	Item	Load Factor	Resistance Factor
Loads	Dead Loads, (f_{DL})	1.25 **(0.8)***	--
	Live Loads, Wind, earthquake, (f_{LL})	1.5	--
	Water Pressure, (f_U)	1.25 **(0.8)***	--
Shear strength	Cohesion “*c*” - stability, earth pressure, (f_c)	--	0.65
	Cohesion “*c*” – Foundation, (f_c)	--	0.5
	Friction angle “ϕ”, (f_ϕ)	--	0.8

*** The values given in the parenthesis apply to beneficial loading conditions such as dead loads resist overturning or up lift.**

It is prudent to ignore the cohesion component when estimating the shear resistance against sliding. This is because the cohesive bond may be lost when separation takes place between concrete and rock foundation upon relative movement. The shearing angle ϕ' may be taken as 35 deg.

To increase the lateral resistance against sliding, the footings shall be supplied with a shear key and/or anchored to the bedrock by means of rock anchors (i.e., dowels or rebars). The design of both, the shear key (i.e., width and impediment), and the rock anchor system (i.e., the number and interval of the anchors, and the embedment length of anchors in concrete and rock) shall be provided by a structural engineer.

7.3.3 *Uplift and Overturning Resistance*

Uplift is an active force due to hydrostatic pressure which must be included in the analysis of stability. The uplift pressures act between the dam and its foundation, and within the foundation below the contact plane and it should also be considered within any cracks within the dam.

Uplift at the foundation-concrete interface for structures having no foundation drains or an unverified drainage system should be assumed to vary as a straight line from 100% of the headwater pressure at the upstream face (heel) to 100% of the tailwater pressure at the downstream face (toe) applied over 100% of the base area.

The dead load of the dam can provide resistance to uplift and overturning forces that the proposed dam foundation may experience. Additional resistance can be provided by increasing the dead weight of the structure using additional concrete elements or by using rock anchors.

Grouted rock anchors may be designed based on frictional stress between the grout and intact bedrock. The bond zone must be entirely within sound bedrock. The design of rock anchors can be performed extending the Limit State Design (LSD) method. The Ultimate Limit States (ULS) and Serviceability Limit States (SLS) bond stress values must be based on both performance and structural criteria. However, based upon typical published values, the unfactored ULS bond stress values for limestone bedded with shale may be approximately 800 kPa to more than 1,400 kPa as per Ground Anchors and Anchored System (FHWA-IF-99-015).

CFEM (2006) recommends a geotechnical resistance factor of 0.3 be applied to the empirical unfactored ULS values. Performance testing is recommended to be carried out at the outset of the Project to verify the anchor capacities. Performance tests shall be performed on the first three production anchors installed and thereafter on a minimum of 2% of the remaining production anchors. Designers may take the approach that working stress value is approximately equivalent to the SLS value. We recommend that a conservative allowable working stress value of 240 kPa be used to calculate the length of the required bond zone. The estimated value includes a geotechnical resistance factor of 0.3. The resistance factor can be increased to Φ=0.4 based on the performance testing results and the allowable working stress can be optimized, if required. The bond zone must be entirely within sound bedrock.

In order to mobilize the shear stress in the rock, the load at the top of the anchor must be properly transferred through the upper bedrock to the bond zone to prevent progressive grout fail and ensure proper performance. Therefore, a “free length” is required through the foundation element, and down to the bond zone.

The mass of rock mobilized by a rock anchor may be assumed to be based upon a 60°cone drawn upward from a point located at the lower one-third point of the bond zone and spaced such that the theoretical cones do not overlap. Designers should review the spacing of anchors and take into account of any overlapping cones (i.e., avoid doubling-up on rock mass calculations for overlapping cones). The bulk unit weight of bedrock may be assumed to be approximately 26 kN/m^3. The corresponding buoyant unit weight would be approximately 16 kN/m^3. It is recommended that the designer uses submerged unit weights for the rock mass calculations since it is below water level.

7.3.4 Geotechnical Parameters

The geotechnical parameters used for the slope stability analyses are summarized in Table below.

Table 7-3: Geotechnical Parameters Used for Dam Design

Zones	Material	Parameters		
		Saturated Unit Weight, γ (kN/m^3)	Cohesion, c′ (kPa)	Internal Friction Angle, ϕ′ (degree)
Foundation Materials	In-situ Contact Bedrock	Impenetrable		
	In-situ Fractured Bedrock	Impenetrable		
	Concrete	24	High Strength	
	Riprap	22	0	40
	Granular B Type II	21	0	32
	Rockfill	21	0	35
	Grout	Impenetrable		

7.4 Frost Protection

Bedrock subgrade is not frost heave susceptible.

Frost penetration depth in overburden is 1.8 m below the surface for the subject site. Frost penetration depth is estimated based on the OPSD 3090.101. For protection against frost effects, earth cover of 1.8 m must be provided for all footings in unheated or isolated structures. In the absence of adequate soil cover, equivalent synthetic insulation material can be used.

Backfill soils should not be placed in a frozen condition or placed on frozen subgrades.

7.5 Site Classification for Seismic Site Response

The National Building Code of Canada is not applicable for the design of dams since the seismic zoning maps generated for the National Building Code of Canada are specifically provided for the seismic design of common buildings only. Recommendations for safety analysis of existing dams and design of new dams for seismic loads should be performed in accordance with Canadian Dam Association (CDA) Guidelines.

Dam Class “Very High” in accordance with Table 2-1 of the Canadian Dam Safety Guidelines 2007 (2013 Edition) is recommended. Table 6-1B of the Guidelines should be consulted to estimate the flood and earthquake hazards, traditional Standard-Based Approach. The minimum Annual Exceedance Probability (AEP) for Dam Class “Very High” can be:

- For Floods: two-third between 1/1000 and probable maximum flood (PMF); and
- For Earthquakes: ½ between 1/2475 and 1/10,000 or maximum credible earthquake (MCE).

For earthquakes, the annual exceedance probability (1/2475) was selected for consistency seismic design levels given in the National Building Code of Canada.

Selected spectral responses in the general vicinity of the site for a 2% chance of exceedance in 50 years (2475 years return period) are as indicated in Table 7-3, based on the National Building Code Seismic Hazard published by Natural Resources Canada 2015.

Table 7-3: Selected Seismic Spectral Responses (2% in 50 Yrs)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)	PGV (m/s)
0.175	0.117	0.069	0.036	0.106	0.100

Given the shallow bedrock across the site and the proposed replacement will be founded on sound bedrock, the site can be classified as Seismic Site Class (C).

7.6 Lateral Earth Pressure

Active earth pressure is the minimum value of the lateral earth pressure, which a soil mass can apply against an unrestrained structure. On the other hand, passive earth resistance is the maximum value of lateral pressure, which can be mobilized in the soil by the structure moving toward the soil mass.

This report provides coefficients of lateral earth pressure. Static lateral pressure can be calculated by using the following equation:

$$P_h = K \times (\gamma h + q)$$

In this equation, the provided unit weight of the soil, γ, is for a moist soil above the groundwater table. Pseudo-dynamic effects of seismic activities are considered based on Mononobe-Okabe method.

The backfill material shall be ‘free draining’ and to follow OPSS.MUNI 1010 recommendation for grain size distribution. However, if there is a chance of hydrostatic pressure build-up behind the wall, the designer shall consider the fluid pressure in the analysis of retaining wall pressure.

Calculation of all live load and dead load surcharges are the responsibility of the designer.

The PGA for this Site is 0.106 based on Site Class C and probability of exceedance per annum of 0.000404.

Table 7-4: Lateral Earth Pressure Design Parameters for Backfill and Native Soil

Design Parameters	Material		
	Granular A	Granular B	Rock Fill
Unit Weight, γ (kN/m^3)	21	21	21
Internal Friction Angle, ϕ (°)	32	32	35
Static at-rest pressure, K_o	0.47	0.47	0.43
Static active pressure, K_a	0.31	0.31	0.27
Static passive pressure, K_p	3.25	3.25	3.69
Dynamic active pressure, K_{AE}	0.34	0.34	0.30
Dynamic passive pressure, K_{PE}	0.85	0.85	0.81

The above noted lateral pressure coefficients are calculated assuming the wall back angel is vertical and the backslope of the retained soil is horizontal. The wall-soil interaction angle is assumed to equal to 0.5ϕ as per CFEM. If Engineered Shoring is used, then designers should refer to CFEM for design assistance and a geotechnical engineer should be retained to perform the shoring design review.

7.7 Backfill

The backfill placed against exterior retaining walls shall be free draining granular material meeting the grading requirements of an OPSS.MUNI 1010 Granular A or Granular B Type II. However, other suitable granular materials may be proposed and considered depending on the site-specific conditions.

The exterior backfill should be placed and compacted as outlined below:

- Backfill should not be placed in frozen condition, or placed on a frozen subgrade;
- Backfill should be placed and compacted in maximum loose lift thickness compatible with the selected construction equipment, but not thicker than 0.3 m. Each lift should be uniformly compacted to achieve 98% of its SPMDD.
- In landscaped areas the upper 0.3 m of backfill below landscape details should be a low permeable soil to reduce surface water infiltration;
- Lateral earth pressure shall be estimated in accordance with Section “7.6 Lateral Earth Pressure”. At rest condition shall be assume for fully restrained retaining wall, and active lateral earth pressure condition should be assumed if relative outward movement is expected;
- For backfill that would underlie paved areas, sidewalks or exterior slabs-on-grade, each lift should be uniformly compacted to achieve 98% of its SPMDD;

- For backfill that would underlie landscaped areas, each lift should be uniformly compacted to at least 95% of its SPMDD;

7.8 Underground Utilities

At the subject site, the burial depth of water-bearing utility lines is typically 2.4 m below the ground surface or as dictated by local applicable codes. If this depth is not achievable, equivalent thermal insulation should be provided. The contractor should retain a professional engineer to provide detailed drawings for excavation and temporary support of the excavation walls during construction.

The Occupational Health and Safety Act (OHSA) of Ontario indicated that side slopes in fill above the water could be classified as Type 3 soil and sloped no steeper than 1H:1V or be shored. Below the groundwater level, the fill is considered to be Type 4 Soil and the excavation side slopes must be sloped from their bottom cut back at 3H:1V. Otherwise, lateral support for all excavations such as trench boxes should be used.

For excavation in rock, please refer to Section “7.2.2 Bedrock Excavation”.

The engineer designing utilities shall ensure the proposed utility pipes can tolerate compaction loads.

The recommendations within this section are intended to be a supplement to, and not a replacement of the most recent local municipal requirements.

7.8.1 Bedding and Cover

The following are recommendations for service trench bedding and cover materials:

- Bedding for buried utilities should consist of an OPSS.MUNI 1010 "Granular A" material and be placed in accordance with municipal requirements, assuming the subgrade soils are not allowed to become disturbed. All utility pipes and high amps electrical conduits shall receive a minimum of 150 mm bedding.
- The use of clear stone is not recommended for use as pipe bedding.
- The cover material should be a service sand material or an OPSS.MUNI 1010 "Granular A". The dimensions should comply with the pertinent specification section.
- The bedding, spring line, and cover should be compacted to at least 98% of its SPMDD.
- All covers are to be compacted to 100% SPMDD if they are intersecting structural elements.
- Compaction equipment should be used in such a way that the utility pipes are not damaged during construction.

7.8.2 Trench Backfill

Backfill above the cover for buried utilities should be in accordance with the following recommendations:

- For service trenches underlying pavement areas, the backfill should be placed and compacted in uniform lift thickness compatible with the selected compaction equipment and not thicker than 300 mm. Each lift should be compacted to a minimum of 98% of its SPMDD. The upper 0.3 m immediately below the pavement elevation should be compacted to a minimum of 100% of its SPMDD.
- During backfilling, care should be taken to ensure the backfill proceeds in equal stages simultaneously on both sides of the pipe; and
- No frozen material should be used as backfill; neither should the trench base be allowed to freeze.

The quality and workmanship in the construction are as important as the compaction standards themselves. It is imperative that the guidelines for the compaction be followed for the full depth of the trench to achieve satisfactory performance.

8.0 CEMENT TYPE AND CORROSION POTENTIAL

A water sample was submitted to Parcel laboratories for testing of chemical properties relevant to exposure of concrete elements to sulphate attacks as well as potential corrosivity effects on buried metallic structural elements. Test results are presented in Table 6-3 and the laboratory results for the chemical analysis are shown in appendix E.

Based on electrical resistivity results and pH-value, the corrosion potential for steel elements in contact to surface water is within the non-aggressive range.

The analytical results of the water sample were compared with applicable Canadian Standards Association (CSA) A23.1-04 and are given in Table 8-1 below.

Table 8-1: Additional Requirement for Concrete Subjected to Sulphate Attack

Class of Exposure	Degree of Exposure	Water Soluble Sulphate in Soil Sample (%)	Cementing Material to be Used
S-1	Very Severe	> 2.0	HS or HSb
S-2	Severe	0.2 – 2.0	HS or HSb
S-3	Moderate	0.1 – 0.2	MS, MSb, LH, HS, or HSb

The chemical sulphate content analyses for selected soil samples tested indicate a sulphate concentration of maximum of a 0.0003 % in water, as shown in Table 6-3, indicating a “moderate to low” risk for sulphate attack on concrete material.

The potential for sulphate attack on concrete structures is moderate to low. Therefore, Type GU Portland cement may be adequate to protect buried concrete elements in the subsurface conditions encountered.

9.0 CONSTRUCTION CONSIDERATIONS

The recommendations presented in this report are based on the assumption that an adequate level of construction monitoring by qualified geotechnical personnel during construction will be provided. The bedrock quality during construction should be confirmed by extending a 1.5 m probe holes into the bedrock within the footing footprints. These holes will need to be reviewed by the geotechnical engineer to confirm that no significant mud seams or voids exist. The holes must be filled with grout after inspection is completed. All bearing surfaces should be inspected and approved by experienced geotechnical personnel prior to placing the footings or lean mix concrete slabs.

In addition, an adequate level of construction monitoring should include laboratory and field test during construction. This includes Full time compaction testing of backfill behind retaining walls and part time compaction testing of general backfill with laboratory testing for the proposed fill soils for this site. Also, periodic testing of concrete is required.

All backfilling shall comply with the OPSS.MUNI 501 for compaction requirements, unless the design recommendations included in this report exceed provisions of OPSS.MUNI 501.

10.0 CLOSURE

We trust this geotechnical investigation and design recommendation report meets the requirements of your project. The “Limitations of Report” presented in Appendix A are an integral part of this report. Please contact the undersigned should you have any questions or concerns.

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PROPOSED KASHWAKAMAK LAKE DAM REPLACEMENT – NORTH FRONTENAC TWN. ON

APPENDIX A
LIMITATIONS OF REPORT

LIMITATIONS OF REPORT

McIntosh Perry Consulting Engineers Ltd. (McIntosh Perry) carried out the field work and prepared the report. This document is an integral part of the Foundation Investigation and Design report presented.

The conclusions and recommendations provided in this report are based on the information obtained at the borehole locations where the tests were conducted. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the specific locations where tests were conducted and conditions may become apparent during construction, which were not detected and could not be anticipated at the time of the site investigation. The benchmark level used and borehole elevations presented in this report are primarily to establish relative differenced in elevations between the borehole locations and should not be used for other purposes such as to establish elevations for grading, depth of excavations or for planning construction.

The recommendations presented in this report for design are applicable only to the intended structure and the project described in the scope of the work, and if constructed in accordance with the details outlined in the report. Unless otherwise noted, the information contained in this report does not reflect on any environmental aspects of either the site or the subsurface conditions.

The comments or recommendation provided in this report on potential construction problems and possible construction methods are intended only to guide the designer. The number of boreholes advanced at this site may not be sufficient or adequate to reveal all the subsurface information or factors that may affect the method and cost of construction. The contractors who are undertaking the construction shall make their own interpretation of the factual data presented in this report and make their conclusions, as to how the subsurface conditions of the site may affect their construction work.

The boundaries between soil strata presented in the report are based on information obtained at the borehole locations. The boundaries of the soil strata between borehole locations are assumed from geological evidences. If differing site conditions are encountered, or if the Client becomes aware of any additional information that differs from or is relevant to the McIntosh Perry findings, the Client agrees to immediately advise McIntosh Perry so that the conclusions presented in this report may be re-evaluated.

Under no circumstances shall the liability of McIntosh Perry for any claim in contract or in tort, related to the services provided and/or the content and recommendations in this report, exceed the extent that such liability is covered by such professional liability insurance from time to time in effect including the deductible therein, and which is available to indemnify McIntosh Perry. Such errors and omissions policies are available for inspection by the Client at all times upon request, and if the Client desires to obtain further insurance to protect it against any risks beyond the coverage provided by such policies, McIntosh Perry will co-operate with the Client to obtain such insurance.

McIntosh Perry prepared this report for the exclusive use of the Client. Any use which a third party makes of this report, or any reliance on or decision to be made based on it, are the responsibility of such third parties. McIntosh Perry accepts no responsibility and will not be liable for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this report.

PROPOSED KASHWAKAMAK LAKE DAM REPLACEMENT – NORTH FRONTENAC TWN. ON

APPENDIX B

SITE AND BOREHOLE LOCATION PLANS

McINTOSH PERRY

500 250 0 500

Scale 1:20,000

Metres

LEGEND

○ Site Location

Local Road

Major Road

Watercourse

🟦 Waterbody

🟩 Wooded Area

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2023.

CLIENT:
MISSISSIPPI VALLEY
CONSERVATION AUTHORITY

PROJECT:
GEOTECHNICAL INVESTIGATION AND DESIGN REPORT
KASHWAKAMAK LAKE DAM REPLACEMENT- ONTARIO

TITLE:
SITE LOCATION

McINTOSH PERRY
115 Walgreen Road, RR3, Carp, ON K0A1L0
Tel: 613-836-2184 Fax: 613-836-3742
www.mcintoshperry.com

PROJECT NO:CCO-24-3603	
Date	Nov., 18, 2023
GIS	MG
Checked By	JP

FIGURE:
1

C:\Users\M.Grunstra\McIntosh Perry\GIS - Documents\Projects\2023\CCO\CCO-23-3603_MVCA_Kashwakamak Dam EA\aprx\Geotech\CCO-23-3603 Borehole Locations.aprx

20 10 0 20

Scale 1:600

Metres

LEGEND

- ⊕ Borehole Location
- ⊕ Basemark Location (B.M. = 262.00 (Abutment planted in 1967))

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2023.

CLIENT: MISSISSIPPI VALLEY CONSERVATION AUTHORITY

PROJECT: GEOTECHNICAL INVESTIGATION AND DESIGN REPORT KASHWAKAMAK LAKE DAM REPLACEMENT- ONTARIO

TITLE: BOREHOLE LOCATIONS

McINTOSH PERRY

115 Walgreen Road, RR3, Carp, ON K0A1L0
Tel: 613-836-2184 Fax: 613-836-3742
www.mcintoshperry.com

PROJECT NO:CCO-24-3603

Date	Nov., 25, 2023	FIGURE: 2
GIS	MG	
Checked By	JP	

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PROPOSED KASHWAKAMAK LAKE DAM REPLACEMENT – NORTH FRONTENAC TWN. ON

APPENDIX C
BOREHOLE LOG RECORDS

MCINTOSH PERRY

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60˚ CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
Φ'	$-^{o}$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	$-^{o}$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
Υ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{,max} - e}{e_{max} - e_{min}}$
P_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
Υ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
Υ	kN/m^3	UNIT WEIGHT OF SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
Υ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L – W_L)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W – W_P)/ I_P$	i	1	HYDAULIC GRADIENT
Υ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(W_L – W) / 1_P$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERED SOIL	$e_{,max}$	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
Υ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

McINTOSH PERRY

PROJECT NO.: **CCO-23-3603**

PROJECT: Geotechnical Investigation - Kashwakamak Lake Dam

CLIENT: Mississippi Valley Conservation Authority

PROJECT LOCATION: Township of North Frontenac, ON

Drilling Date: Sep-22-2023 - Sep-25-2023

BH Location: N 4972860; E 345362

Drilling Equipment: Portable Hilti Drill

Drilling Method: Portable Hilti Drill

Remarks: Coordinate System - UTM Zone 18 T

BH No: 23-1

Datum: Geodetic

Elevation: 259.3 m

Compiled by: JF

Checked by: MAK

ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m RQD (%)	RECOVERY (%)	GROUNDWATER CONDITIONS	DEPTH (m)	ELEVATION (m)	Remarks and Grain Size Distribution (%)
259.3	Bedrock									
0.0	**Carbonite Metasedimentary Bedrock** marble parting diagonally, strong, grey to darck grey with white bands of marble, fresh to slightly wethered, medium to thinly bedded, good to excellent quality based on RQD.	▨	1	RC	93	100%		0.0	259	
			2	RC	89	100%		1.0	258	UCS = 164 Mpa
			3	RC	94	100%		2.0	257	
			4	RC	84	98%		3.0	256	UCS = 167 Mpa
			5	RC	100	100%		4.0	255	
			6	RC	83	98%		5.0	254	UCS = 177 Mpa
252.8			7	RC	92	100%		6.0	253	
6.5	**End of borehole**									

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80

SHEAR STRENGTH (kPa)
Field. Shear Vane (x) & Sensitivity (s)
Pocket Penetrometer ✖
● Quick Triaxial ○ Unconfined
20 40 60 80

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%)
10 20 30 40 50 60 70 80 90

Unit Weight (kN/m^3)
Pocket Penetro. (kPa)
GR SA SI CL

GRAPH NOTES

$\overset{30}{\underset{3}{\times}}$ Upper value = Field Vane Shear Strength
Lower value = Vane Sensitivity

○ ε=3% Strain at Failure

McINTOSH PERRY

PROJECT NO.: **CCO-23-3603**

PROJECT: Geotechnical Investigation - Kashwakamak Lake Dam

CLIENT: Mississippi Valley Conservation Authority

PROJECT LOCATION: Township of North Frontenac, ON

Drilling Date: Sep-20-2023 - Sep-20-2023

BH Location: N 4972859; E 345352

Drilling Equipment: Portable Hilti Drill

Drilling Method: Portable Hilti Drill

Remarks: Coordinate System - UTM Zone 18 T

BH No: 23-2

Datum: Geodetic

Elevation: 258.6 m

Compiled by: JF

Checked by: MAK

SOIL PROFILE			SAMPLES				GROUNDWATER CONDITIONS	DEPTH (m)	ELEVATION (m)	Remarks and Grain Size Distribution (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m RQD (%)	RECOVERY (%)				GR SA SI CL
258.6	Bedrock							0.0		
0.0	**Carbonite Metasedimentary Bedrock** marble parting diagonally, strong, grey to darck grey with white bands of marble, fresh to slightly wethered, medium to thinly bedded, excellent quality based on RQD.		1	RC	91	99%		1.0	258	UCS = 201 Mpa
			2	RC	92	92%		2.0	257	
			3	RC	100	100%		3.0	256	
			4	RC	76	97%		4.0	255	UCS = 208 Mpa
			5	RC	97	100%			254	
			6	RC	75	94%		5.0		
253.0			7	RC	100	100%				UCS = 195 Mpa
5.6	**End of borehole**									

DYNAMIC CONE PENETRATION RESISTANCE PLOT

20 40 60 80

SHEAR STRENGTH (kPa)

Field. Shear Vane (x) & Sensitivity (s)

Pocket Penetrometer ✖

● Quick Triaxial ○ Unconfined

20 40 60 80

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%)

10 20 30 40 50 60 70 80 90

Unit Weight (kN/m^3)

Pocket Penetro. (kPa)

GRAPH NOTES

$\overset{30}{\underset{3}{\times}}$ Upper value = Field Vane Shear Strength

Lower value = Vane Sensitivity

○ ε=3% Strain at Failure

McINTOSH PERRY

PROJECT NO.: **CCO-23-3603**

PROJECT: Geotechnical Investigation - Kashwakamak Lake Dam

CLIENT: Mississippi Valley Conservation Authority

PROJECT LOCATION: Township of North Frontenac, ON

Drilling Date: Sep-18-2023 - Sep-18-2023

BH Location: N 4972865; E 345350

Drilling Equipment: CME 75 Trackmount

Drilling Method: CME 75 Trackmount

Remarks: Coordinate System - UTM Zone 18 T

BH No: 23-4 MW

Datum: Geodetic

Elevation: 262.1 m

Compiled by: JF

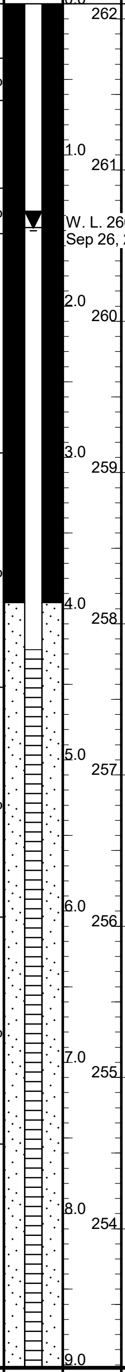
Checked by: MAK

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
262.1	Access Road
0.0	**Topsoil**
261.7 0.4	**Carbonite Metasedimentary Bedrock** marble parting diagonally, strong, grey to darck grey with white bands of marble, fresh to slightly wethered, medium to thinly bedded, good to excellent quality based on RQD.
253.1 9.0	**End of borehole** Monitoring Well Installed

SAMPLES

NUMBER	TYPE	"N" BLOWS/0.3 m RQD (%)	RECOVERY (%)	Remarks and Grain Size Distribution (%)
1	SS	50/ 50mm	86%	
2	RC	63	100%	
3	RC	95	98%	
4	RC	67	100%	
5	RC	56	86%	
6	RC	82	104%	
7	RC	97	100%	UCS = 158 Mpa
8	RC	73	100%	Verical crack encountered from 6.7 to 6.9 m.
9	RC	91	98%	



DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80

SHEAR STRENGTH (kPa)
Field. Shear Vane (x) & Sensitivity (s)
Pocket Penetrometer ✖
● Quick Triaxial ○ Unconfined
20 40 60 80

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30 40 50 60 70 80 90

Remarks and Grain Size Distribution (%)
Unit Weight (kN/m^3)
Pocket Penetro. (kPa)
GR SA SI CL

GRAPH NOTES
$\overset{30}{\underset{3}{\times}}$ Upper value = Field Vane Shear Strength
Lower value = Vane Sensitivity
○ ε=3% Strain at Failure

McINTOSH PERRY

PROJECT NO.: **CCO-23-3603**

PROJECT: Geotechnical Investigation - Kashwakamak Lake Dam

CLIENT: Mississippi Valley Conservation Authority

PROJECT LOCATION: Township of North Frontenac, ON

Drilling Date: Sep-19-2023 - Sep-19-2023

BH Location: N 4972839; E 345348

Drilling Equipment: Portable Hilti Drill

Drilling Method: Portable Hilti Drill

Remarks: Coordinate System - UTM Zone 18 T

BH No: 23-5

Datum: Geodetic

Elevation: 259.2 m

Compiled by: JF

Checked by: MAK

SOIL PROFILE		SAMPLES				GROUNDWATER CONDITIONS	Remarks and Grain Size Distribution (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m RQD (%)	RECOVERY (%)		GR SA SI CL
259.2	Bedrock						
0.0	**Carbonite Metasedimentary Bedrock** marble parting diagonally, strong, grey to darck grey with white bands of marble, fresh to slightly wethered, medium to thinly bedded, good to excellent quality based on RQD.	1	RC	87	100%		
		2	RC	92	100%		UCS = 211 Mpa
		3	RC	95	95%		
		4	RC	81	102%		UCS = 174 Mpa
		5	RC	93	100%		
		6	RC	64	96%		UCS = 126 Mpa
		7	RC	98	103%		
252.9 6.3	**End of borehole**						

DEPTH (m): 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0

ELEVATION (m): 259, 258, 257, 256, 255, 254, 253

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80

SHEAR STRENGTH (kPa)
Field. Shear Vane (x) & Sensitivity (s)
Pocket Penetrometer ✖
● Quick Triaxial ○ Unconfined
20 40 60 80

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30 40 50 60 70 80 90

Unit Weight (kN/m^3)
Pocket Penetro. (kPa)

GRAPH NOTES

$\overset{30}{\underset{3}{\times}}$ Upper value = Field Vane Shear Strength
Lower value = Vane Sensitivity

○ ε=3%
Strain at Failure

Retrieved Rock Cores

(BH23-1)

RC 1 – 259.3 to 258.18 m
(RQD = 93%)

RC 2 – 258.18 to 257.04 m
(RQD = 89%)

(BH23-1)

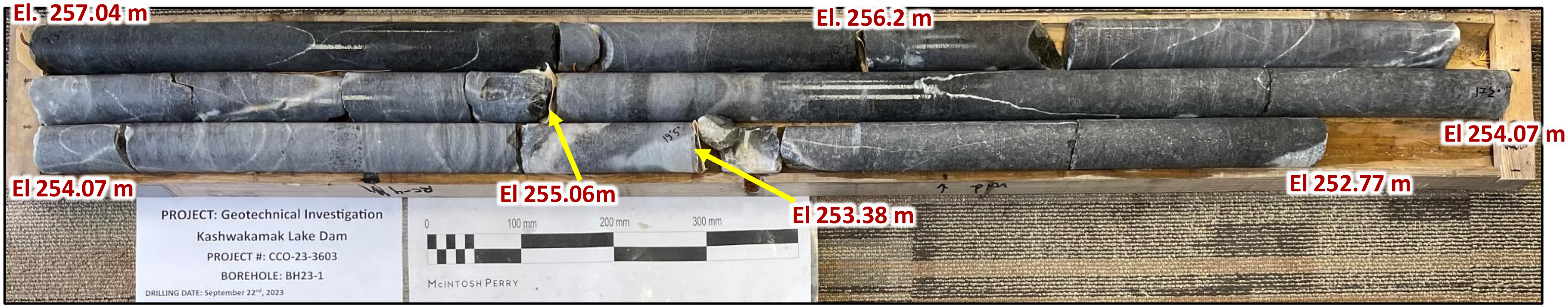
RC3 – 257.04 to 256.2m
(RQD = 94%)

RC4 – 256.2 to 255.06 m
(RQD = 84%)

RC5 – 255.06 to 254.07 m
(RQD = 100%)

RC6 – 254.07 to 253.38
(RQD = 83%)

RC7 – 253.38 to 252.77
(RQD =92%)



McINTOSH PERRY

Kashwakamak Lake Dam

Township of North Frontenac, ON

Project No: CCO-23-3603
Client: MVCA

Retrieved Rock Cores (BH23–2)

RC1 – 258.6 to 257.38 m **(RQD=91%)**	**RC2 – 257.38 to 256.11 m** **(RQD=92%)**	**RC3 – 256.11 to 255.45 m** **(RQD=100%)**	**RC4 – 255.45 to 254.59 m** **(RQD=76%)**

RC5 – 254.59 to 253.72m
(RQD = 97%)

RC6 – 253.72 to 253.52 m
(RQD = 75%)

RC7 – 253.52 to 253.01
(RQD = 100%)

McINTOSH PERRY

Kashwakamak Lake Dam

Township of North Frontenac, ON

Project No: CCO-23-3603
Client: MVCA

Retrieved Rock Cores

(BH23-4)

RC2 – 262.1 to 261.82 m
(RQD = 63%)

RC3 – 261.82 to 261.24 m
(RQD = 95%)

RC4 – 261.24 to 260.94 m
(RQD = 67%)

RC5 – 260.94 to 259.49 m
(RQD = 56%)

RC6 – 259.49 to 257.94 m
(RQD = 82%)

(BH23-4)

RC7 – 257.94 to 256.42m
(RQD = 97%)

RC8 – 256.42 to 254.92 m
(RQD = 73%)

RC9 – 254.92 to 253.44 m
(RQD=91%)

McINTOSH PERRY

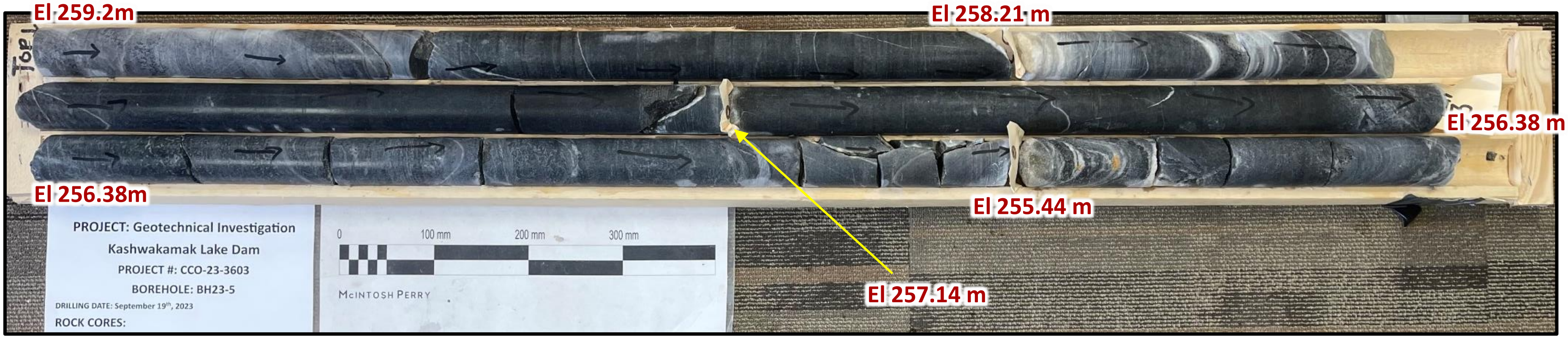
Kashwakamak Lake Dam

Township of North Frontenac, ON

Project No: CCO-23-3603
Client: MVCA

Retrieved Rock Cores (BH23-5)

RC1 – 259.2 to 258.21 m **(RQD=87%)**	**RC2 – 258.21 to 257.14 m** **(RQD=92%)**	**RC3 – 257.14 to 256.38 m** **(RQD=95%)**	**RC4 – 256.38 to 255.44 m** **(RQD=81%)**



PROJECT: Geotechnical Investigation
Kashwakamak Lake Dam
PROJECT #: CCO-23-3603
BOREHOLE: BH23-5
DRILLING DATE: September 19th, 2023
ROCK CORES:

RC5 – 255.44 to 254.32m
(RQD = 93%)

RC6 – 254.32 to 253.71 m
(RQD = 64%)

RC7 – 253.71 to 252.9
(RQD = 98%)

PROJECT: Geotechnical Investigation
Kashwakamak Lake Dam
PROJECT #: CCO-23-3603
BOREHOLE: BH23-5
DRILLING DATE: September 19th, 2023

McIntosh Perry	**Kashwakamak Lake Dam** **Township of North Frontenac, ON**	**Project No: CCO-23-3603** **Client: MVCA**

PROPOSED KASHWAKAMAK LAKE DAM REPLACEMENT – NORTH FRONTENAC TWN. ON

APPENDIX D
PACKER TESTING RESULTS

McINTOSH PERRY

Packer Testing Analysis
Kashwakamak Lake Dam Replacement– Ontario

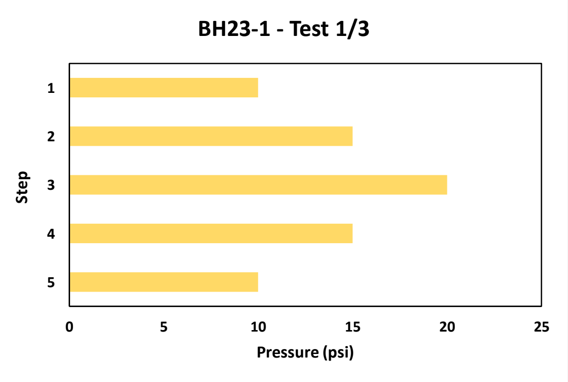
CCO-23-3603

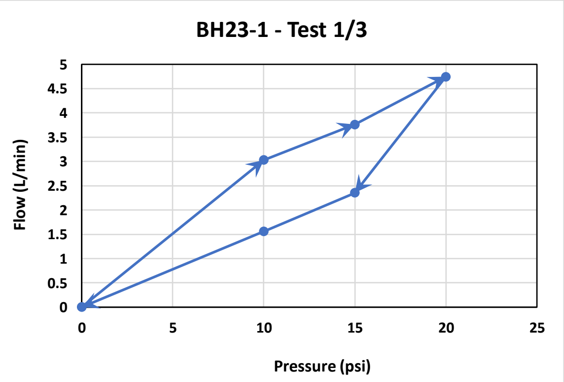
Table D.1

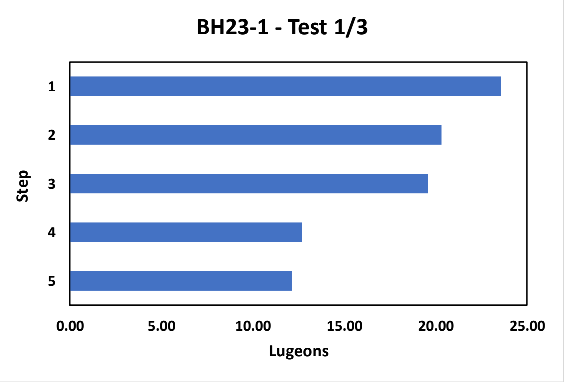
BH23-1 - Test 1/3

Top of section(ft) = 16
Bottom of Section (ft) = 21.42
Total length (ft)= 5.42
Total length (m)= 1.65
Surface Elevation (m) = 259.3
Section Int. El. (m) 254.4 252.8
Depth to center of tested section (m) = 5.7
Depth to Groundwater (m) = 0
H_{gauge} above ground (m) = 0.9

	Flow (Gallon)														
Presure (Psi)	0 min	1 min	2 min	3 min	4 min	5 min	Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= $P_{gauge}/\rho.g$	ΔH (m)	Net Inj Pressure *p* (bar)	**K (m/s)**	**Lugeons, V**
0							0	0	0	0					
10	3737.15	3737.98	3738.81	3739.55	3740.38	3741.15	0.8	3.028328	0.003028328	5.04721E-05	7.03	7.93	0.778	**2.49E-06**	**23.59**
15	3742.1	3742.22	3744.2	3745.43	3746.09	3747.07	0.994	3.76269754	0.003762698	6.27116E-05	10.54	11.44	1.122	**2.145E-06**	**20.32**
20	3747.98	3749.33	3750.61	3751.88	3753.13	3754.25	1.254	4.74690414	0.004746904	7.91151E-05	14.06	14.96	1.468	**2.069E-06**	**19.60**
15	3755.26	3755.93	3756.64	3757.35	3758.04	3758.37	0.622	2.35452502	0.002354525	3.92421E-05	10.54	11.44	1.122	**1.342E-06**	**12.72**
10	3759.33	3759.79	3760.23	3760.64	3760.99	3761.39	0.412	1.55958892	0.001559589	2.59931E-05	7.03	7.93	0.778	**1.282E-06**	**12.15**
0							0	0	0	0					
													Average Values =	**1.866E-06**	**17.1**







Flow Behavior: **Void Filling**
Lugeon: **12.15**
Hydraulic Conductivity: **1.28239E-06**

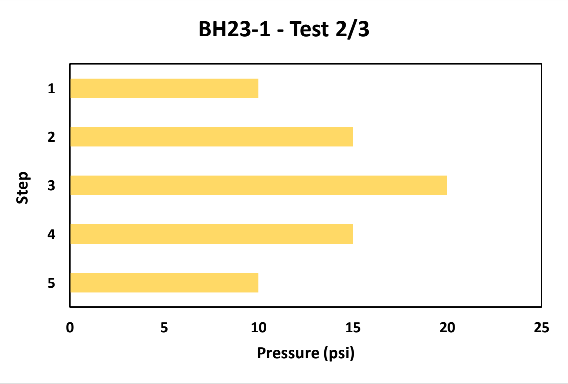
Packer Testing Analysis
Kashwakamak Lake Dam Replacement– Ontario

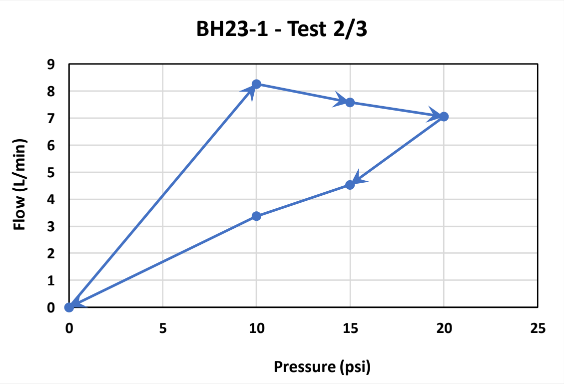
CCO-23-3603

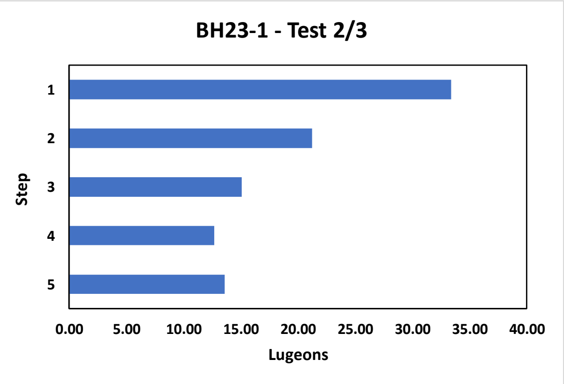
Table D.2

BH23-1 - Test 2/3

Top of section(ft) =		11	Bottom of Section (ft) =			21.42	Total length (ft)=			10.42	Total length (m)=				3.18
Surface Elevation (m) =		259.3	Section Int. El. (m)		255.9	252.8	Depth to center of tested section (m) =					4.94			
Depth to Groundwater (m) =		0	H_{gauge} above ground (m) =			0.9									
	Flow (Gallon)						Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= $P_{gauge}/\rho.g$	ΔH (m)	Net Inj Pressure *p* (bar)	**K (m/s)**	**Lugeons, V**
Presure (Psi)	0 min	1 min	2 min	3 min	4 min	5 min									
0							0	0	0	0					
10	3761.35	3763.92	3766.44	3768.41	3770.37	3772.26	2.182	8.25976462	0.008259765	0.000137663	7.03	7.93	0.778	**4.094E-06**	**33.39**
15	3773.98	3776.18	3778.26	3780.29	3782.17	3783.99	2.002	7.57839082	0.007578391	0.000126307	10.54	11.44	1.122	**2.604E-06**	**21.24**
20	3786.7	3789	3791.1	3793.22	3795.29	3796.01	1.862	7.04843342	0.007048433	0.000117474	14.06	14.96	1.468	**1.852E-06**	**15.10**
15	3799.2	3801.8	3802.27	3803.72	3803.72	3805.18	1.196	4.52735036	0.00452735	7.54558E-05	10.54	11.44	1.122	**1.556E-06**	**12.69**
10	3807.95	3808.82	3809.76	3810.67	3811.55	3812.4	0.89	3.3690149	0.003369015	5.61502E-05	7.03	7.93	0.778	**1.67E-06**	**13.62**
0							0	0	0	0	0				
													Average Values =	**2.355E-06**	**19.2**







Flow Behavior: Turbulent
Lugeon: 15.10
Hydraulic Conductivity: 1.8519E-06

Packer Testing Analysis
Kashwakamak Lake Dam Replacement– Ontario

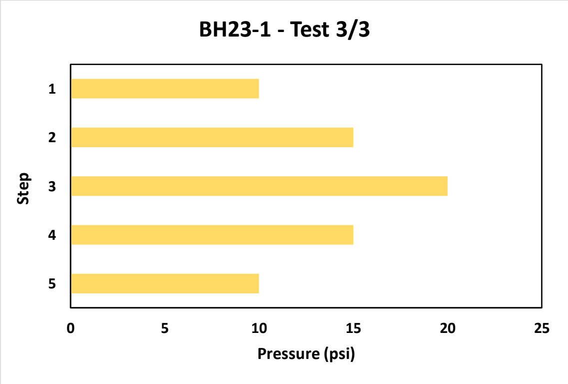
CCO-23-3603

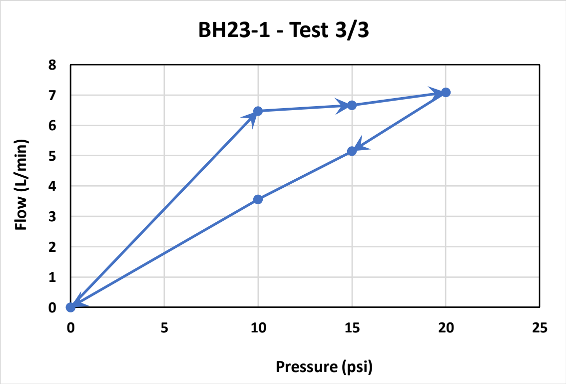
Table D.3

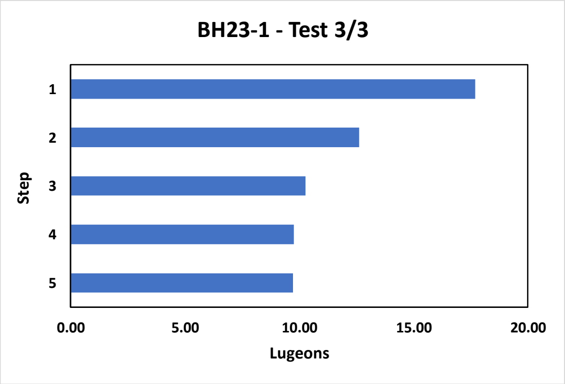
BH23-1 - Test 3/3

Top of section(ft) = 6
Bottom of Section (ft) = 21.42
Total length (ft)= 15.42
Total length (m)= 4.7
Surface Elevation (m) = 259.3
Section Int. El. (m) 257.5 252.8
Depth to center of tested section (m) = 4.18
Depth to Groundwater (m) = 0
H_{gauge} above ground (m) = 0.9

	Flow (Gallon)														
Presure (Psi)	0 min	1 min	2 min	3 min	4 min	5 min	Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= $P_{gauge}/\rho.g$	ΔH (m)	Net Inj Pressure p (bar)	**K (m/s)**	**Lugeons, V**
0							0	0	0	0					
10	3812.4	3814.36	3816.27	3817.85	3819.45	3820.95	1.71	6.4730511	0.006473051	0.000107884	7.03	7.93	0.778	**2.351E-06**	**17.70**
15	3822.2	3823.1	3825.87	3827.6	3829.31	3831	1.76	6.6623216	0.006662322	0.000111039	10.54	11.44	1.122	**1.677E-06**	**12.63**
20	3832.94	3835	3836.86	3838.7	3840.56	3842.3	1.872	7.08628752	0.007086288	0.000118105	14.06	14.96	1.468	**1.364E-06**	**10.27**
15	3843.6	3845.06	3846.42	3847.78	3849.19	3850.4	1.36	5.1481576	0.005148158	8.58026E-05	10.54	11.44	1.122	**1.296E-06**	**9.76**
10	3851.4	3852.4	3853.34	3854.29	3855.25	3856.1	0.94	3.5582854	0.003558285	5.93048E-05	7.03	7.93	0.778	**1.292E-06**	**9.73**
0							0	0	0	0					
													Average Values =	**1.596E-06**	**12.0**





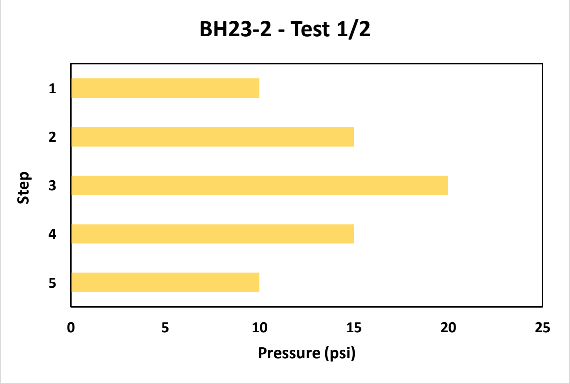


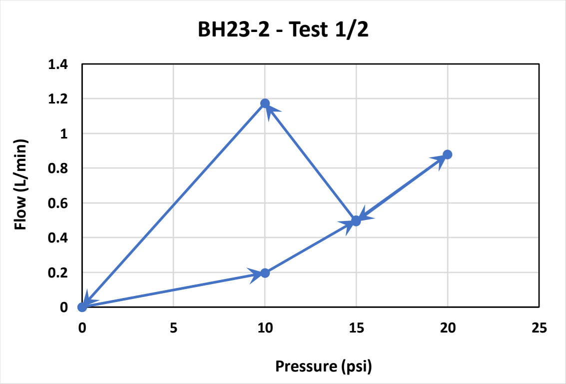
Flow Behavior: **Void Filling**
Lugeon: **9.73**
Hydraulic Conductivity: **1.29225E-06**

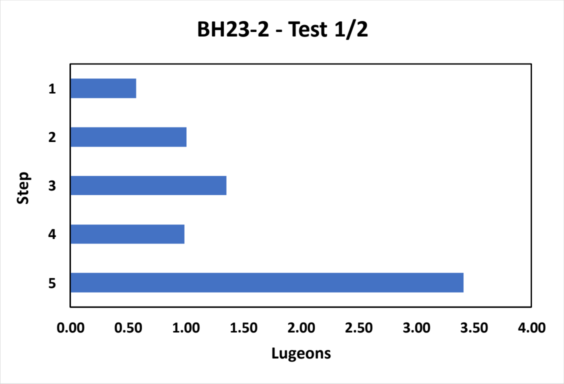
Table D.4

BH23-2 - Test 1/2

Top of section(ft) =		3.83	Bottom of Section (ft) =			18.33	Total length (ft)=			14.5	Total length (m)=				4.42
Surface Elevation (m) =		259.3	Section Int. El. (m)		258.1	253.7	Depth to center of tested section (m) =			3.38					
Depth to Groundwater (m) =		0	H_{gauge} above ground (m) =			0.9									
	Flow (Gallon)														
Presure (Psi)	0 min	1 min	2 min	3 min	4 min	5 min	Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= P_{gauge}/ρ.g	ΔH (m)	Net Inj Pressure *p* (bar)	**K (m/s)**	**Lugeons, V**
0							0	0	0	0					
10	3717.56	3717.65	3717.7	3717.73	3717.77	3717.82	0.052	0.19684132	0.000196841	3.28069E-06	7.03	7.93	0.778	**7.51E-08**	**0.57**
15	3718	3718.16	3718.2	3718.42	3718.54	3718.66	0.132	0.49967412	0.000499674	8.3279E-06	10.54	11.44	1.122	**1.321E-07**	**1.01**
20	3718.83	3719.08	3719.35	3719.53	3719.76	3719.99	0.232	0.87821512	0.000878215	1.46369E-05	14.06	14.96	1.468	**1.776E-07**	**1.35**
15	3720.13	3720.27	3720.4	3720.53	3720.66	3720.78	0.13	0.4921033	0.000492103	8.20172E-06	10.54	11.44	1.122	**1.301E-07**	**0.99**
10	3721.17	3721.55	3721.86	3722.18	3722.44	3722.72	0.31	1.1734771	0.001173477	1.9558E-05	7.03	7.93	0.778	**4.477E-07**	**3.41**
0							0	0	0	0					
													Average Values =	**1.925E-07**	**1.5**







Flow Behavior: Wash-out
Lugeon: 3.41
Hydraulic Conductivity: 4.47709E-07

Packer Testing Analysis
Kashwakamak Lake Dam Replacement– Ontario

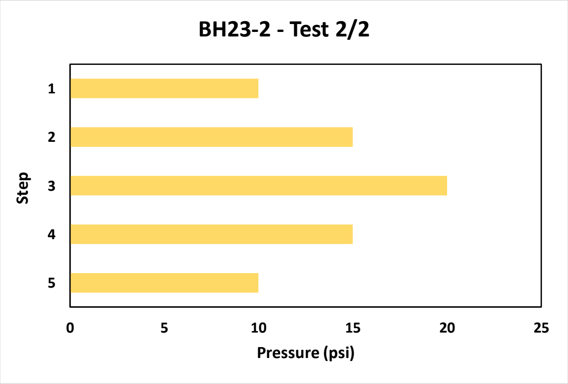
CCO-23-3603

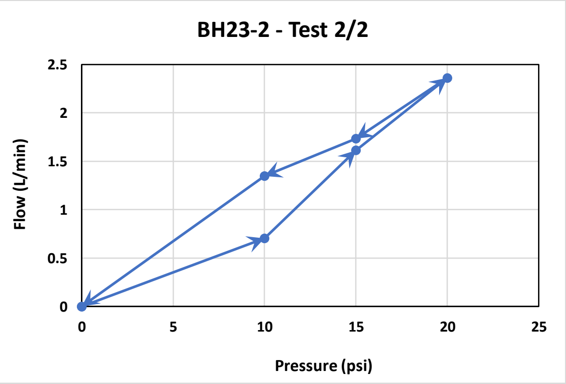
Table D.5

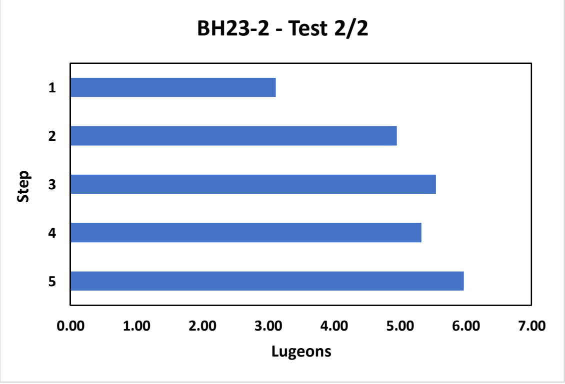
BH23-2 - Test 2/2

Top of section(ft) = 8.83
Bottom of Section (ft) = 18.33
Total length (ft)= 9.5
Total length (m)= 2.9
Surface Elevation (m) = 259.3
Section Int. El. (m) 256.6 253.7
Depth to center of tested section (m) = 4.14
Depth to Groundwater (m) = 0
H_{gauge} above ground (m) = 0.9

	Flow (Gallon)														
Presure (Psi)	0 min	1 min	2 min	3 min	4 min	5 min	Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= P_{gauge}/p.g	ΔH (m)	Net Inj Pressure *p* (bar)	**K (m/s)**	**Lugeons, V**
0							0	0	0	0					
10	3723.13	3723.38	3723.54	3723.72	3723.89	3724.06	0.186	0.70408626	0.000704086	1.17348E-05	7.03	7.93	0.778	**3.752E-07**	**3.12**
15	3724.49	3724.93	3725.34	3725.73	3726.1	3726.62	0.426	1.61258466	0.001612585	2.68764E-05	10.54	11.44	1.122	**5.957E-07**	**4.95**
20	3728.22	3728.95	3729.44	3730.15	3730.74	3731.34	0.624	2.36209584	0.002362096	3.93683E-05	14.06	14.96	1.468	**6.672E-07**	**5.55**
15	3731.82	3732.33	3732.75	3733.29	3733.65	3734.11	0.458	1.73371778	0.001733718	2.88953E-05	10.54	11.44	1.122	**6.404E-07**	**5.33**
10	3734.46	3734.82	3735.21	3735.54	3735.9	3736.24	0.356	1.34760596	0.001347606	2.24601E-05	7.03	7.93	0.778	**7.181E-07**	**5.97**
0							0	0	0	0					
													Average Values =	**5.993E-07**	**5.0**







Flow Behavior: Wash-out
Lugeon: 5.97
Hydraulic Conductivity: 7.18119E-07

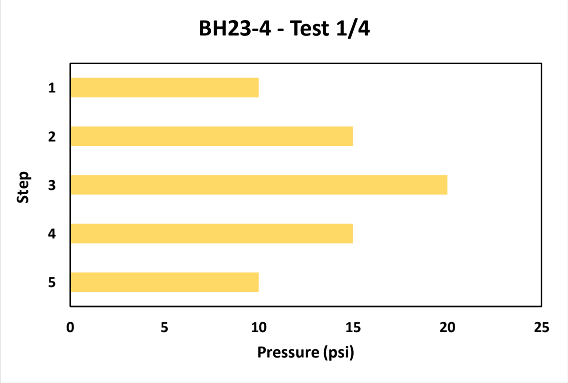
Packer Testing Analysis
Kashwakamak Lake Dam Replacement– Ontario

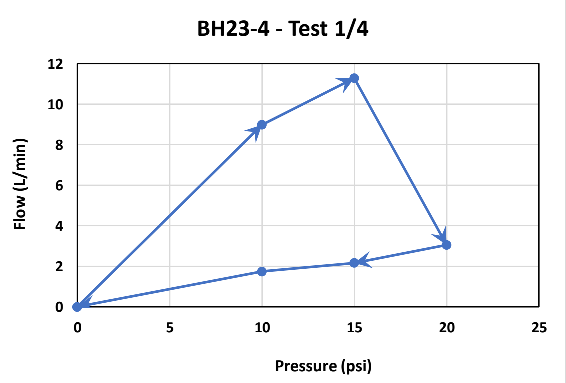
CCO-23-3603

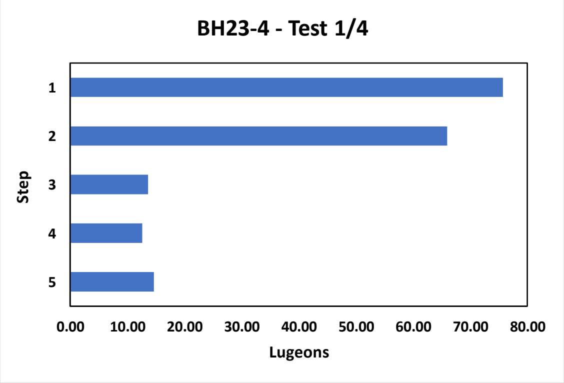
Table D.6

BH23-4 - Test 1/4

Top of section(ft) =		24.58	Bottom of Section (ft) =			29.58	Total length (ft)=			5	Total length (m)=				1.524
Surface Elevation (m) =		262.1	Section Int. El. (m)		254.6	253.1					Depth to center of tested section (m) =	8.253984			
Depth to Groundwater (m) =		0	H_{gauge} above ground (m) =			0.9									
	Flow (Gallon)						Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= $P_{gauge}/\rho.g$	Net Inj Head ΔH	Net Inj Pressure *p* (bar)	K (m/s)	Lugeons, V
Presure (Psi)	0 min	1 min	2 min	3 min	4 min	5 min									
0							0	0	0	0					
10	75.3	78.23	78.78	81.93	84.05	87.16	2.372	8.97899252	0.008978993	0.00014965	7.03	7.93	0.778	7.837E-06	75.74
15	88.7	92.55	95.32	99.62	102.77	103.6	2.98	11.2805218	0.011280522	0.000188009	10.54	11.44	1.122	6.825E-06	65.96
20	108.06	108.97	109.66	110.47	111.29	112.09	0.806	3.05104046	0.00305104	5.08507E-05	14.06	14.96	1.468	1.412E-06	13.64
15	113.73	114.4	115.13	115.7	116.34	116.58	0.57	2.1576837	0.002157684	3.59614E-05	10.54	11.44	1.122	1.305E-06	12.62
10	118.3	119.05	119.41	120.95	120.26	120.6	0.46	1.7412886	0.001741289	2.90215E-05	7.03	7.93	0.778	1.52E-06	14.69
0							0	0	0	0					
													Average Values =	3.78E-06	36.5







Flow Behavior: Void Filling
Lugeon: 14.69
Hydraulic Conductivity: 1.51981E-06

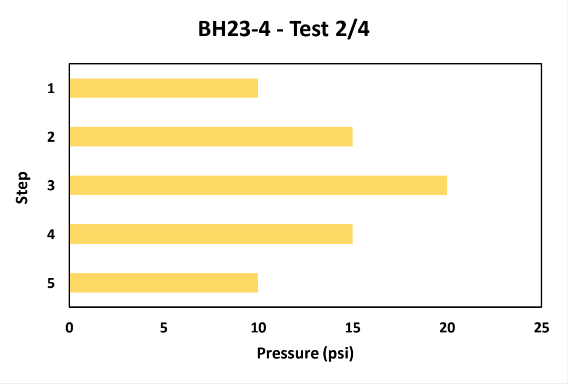
Packer Testing Analysis
Kashwakamak Lake Dam Replacement– Ontario

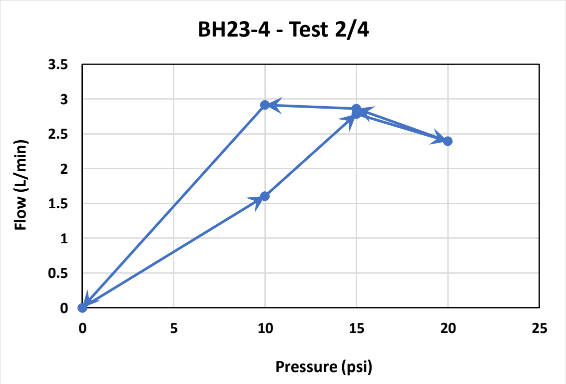
Table D.7

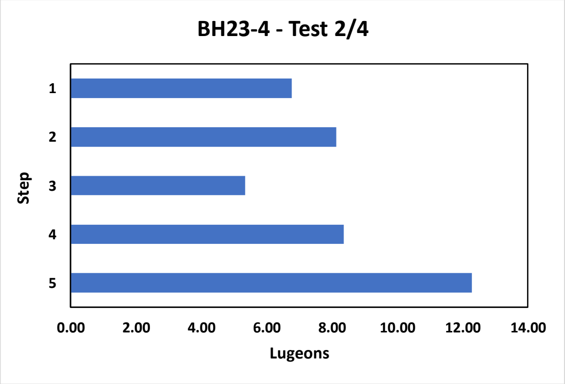
BH23-4 - Test 2/4

Top of section(ft) = 19.58
Bottom of Section (ft) = 29.58
Total length (ft)= 10
Total length (m)= 3.048
Surface Elevation (m) = 262.1
Section Int. El. (m) 256.1 253.1
Depth to center of tested section (m) = 7.491984
Depth to Groundwater (m) = 0
H_{gauge} above ground (m) = 0.9

	Flow (Gallon)						Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= $P_{gauge}/\rho.g$	ΔH (m)	Net Inj Pressure p (bar)	**K (m/s)**	**Lugeons, V**
Presure (Psi)	0 min	1 min	2 min	3 min	4 min	5 min									
0							0	0	0	0					
10	21.2	21.66	22.1	22.51	22.9	23.32	0.424	1.60501384	0.001605014	2.67502E-05	7.02829766	7.92829766	0.778	**8.227E-07**	**6.77**
15	24.11	25.77	25.25	26.75	26.22	27.79	0.736	2.78606176	0.002786062	4.64344E-05	10.5424465	11.4424465	1.123	**9.895E-07**	**8.14**
20	27.27	27.91	28.44	29.18	29.86	30.43	0.632	2.39237912	0.002392379	3.9873E-05	14.0565953	14.9565953	1.467	**6.5E-07**	**5.35**
15	31	31.76	32.59	33.23	33.95	34.78	0.756	2.86176996	0.00286177	4.76962E-05	10.5424465	11.4424465	1.123	**1.016E-06**	**8.36**
10	35.55	36.12	36.7	37.27	38.83	39.4	0.77	2.9147657	0.002914766	4.85794E-05	7.02829766	7.92829766	0.778	**1.494E-06**	**12.30**
0							0	0	0	0					
													Average Values =	**9.945E-07**	**8.2**







Flow Behavior: Wash-out
Lugeon: 12.30
Hydraulic Conductivity: 1.49406E-06

Packer Testing Analysis
Kashwakamak Lake Dam Replacement– Ontario

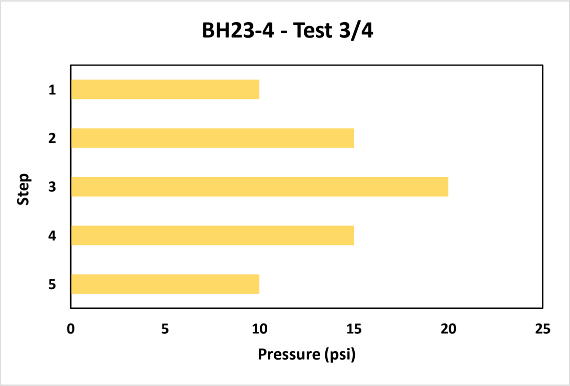
CCO-23-3603

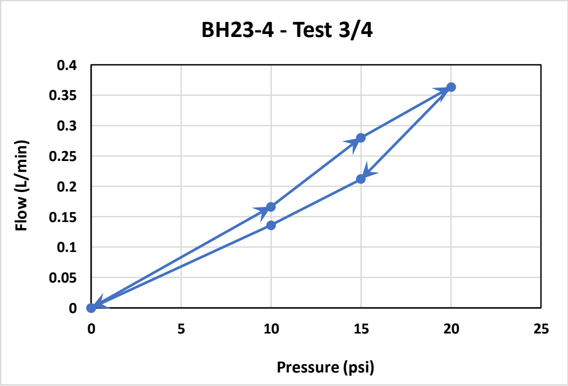
Table D.8

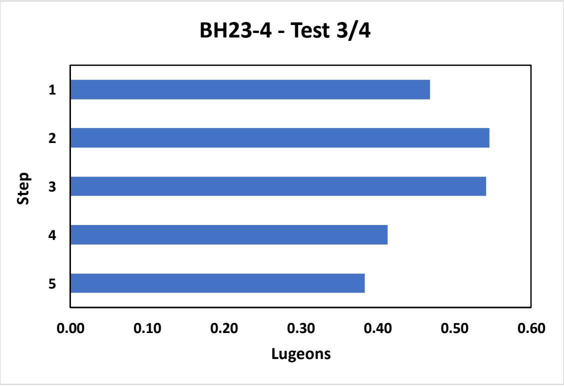
BH23-4 - Test 3/4

Top of section(ft) = 14.58
Bottom of Section (ft) = 29.58
Total length (ft)= 15
Total length (m)= 4.572
Surface Elevation (m) = 262.1
Section Int. El. (m) 257.7 253.1
Depth to center of tested section (m) = 6.729984
Depth to Groundwater (m) = 0
H_{gauge} above ground (m) = 0.9

	Flow (Gallon)														
Presure (Psi)	0 min	1 min	2 min	3 min	4 min	5 min	Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= P_{gauge}/ρ.g	ΔH (m)	Net Inj Pressure *p* (bar)	**K (m/s)**	**Lugeons, V**
0							0	0	0	0					
10	39.89	39.96	39.93	40.09	40.05	40.11	0.044	0.16655804	0.000166558	2.77597E-06	7.02829766	7.92829766	0.778	**6.186E-08**	**0.47**
15	40.21	40.37	40.31	40.47	40.42	40.58	0.074	0.28012034	0.00028012	4.66867E-06	10.5424465	11.4424465	1.123	**7.208E-08**	**0.55**
20	40.52	40.62	40.72	40.81	40.91	41	0.096	0.36339936	0.000363399	6.05666E-06	14.0565953	14.9565953	1.467	**7.154E-08**	**0.54**
15	41.06	41.11	41.17	41.22	4138	41.34	0.056	0.21198296	0.000211983	3.53305E-06	10.5424465	11.4424465	1.123	**5.455E-08**	**0.41**
10	41.59	41.53	41.67	41.6	41.64	41.77	0.036	0.13627476	0.000136275	2.27125E-06	7.02829766	7.92829766	0.778	**5.061E-08**	**0.38**
0							0	0	0	0					
													Average Values =	**6.213E-08**	**0.5**







Flow Behavior: Void Filling
Lugeon: 0.38
Hydraulic Conductivity: 5.06115E-08

McINTOSH PERRY

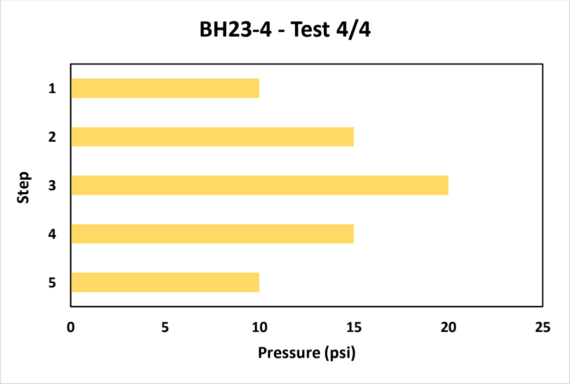
Packer Testing Analysis
Kashwakamak Lake Dam Replacement– Ontario

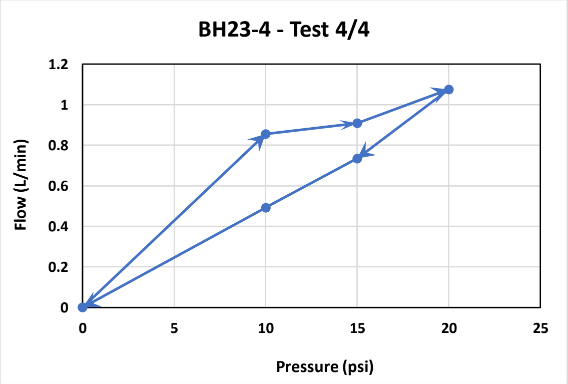
CCO-23-3603

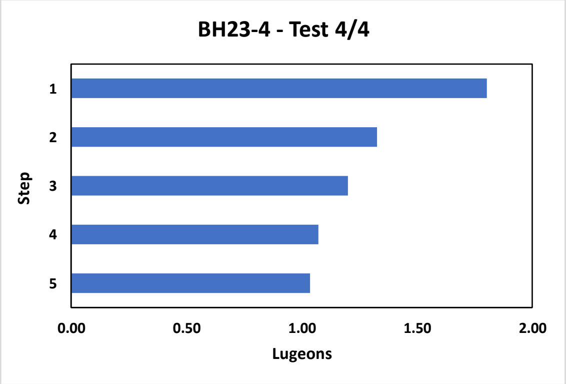
Table D.9

BH23-4 - Test 4/4

Top of section(ft) =		9.58	Bottom of Section (ft) =			29.58	Total length (ft)=				20	Total length (m)=			6.1
Surface Elevation (m) =		262.1	Section Int. El. (m)		259.2	253.1	Depth to center of tested section (m) =					5.97			
Depth to Groundwater (m) =		0	H_{gauge} above ground (m) =			0.9									
	Flow (Gallon)														
Presure (Psi)	0 min	1 min	2 min	3 min	4 min	5 min	Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= $P_{gauge}/\rho.g$	ΔH (m)	Net Inj Pressure p (bar)	**K (m/s)**	**Lugeons, V**
0							0	0	0	0					
10	41.06	41.29	41.43	41.76	41.98	42.19	0.226	0.85550266	0.000855503	1.42584E-05	7.02829766	7.92829766	0.778	**2.517E-07**	**1.80**
15	42.33	42.67	42.82	43.05	43.39	43.53	0.24	0.9084984	0.000908498	1.51416E-05	10.5424465	11.4424465	1.123	**1.852E-07**	**1.33**
20	43.74	44.04	44.3	44.69	44.98	45.16	0.284	1.07505644	0.001075056	1.79176E-05	14.0565953	14.9565953	1.467	**1.676E-07**	**1.20**
15	45.36	45.58	45.79	45.91	46.12	46.33	0.194	0.73436954	0.00073437	1.22395E-05	10.5424465	11.4424465	1.123	**1.497E-07**	**1.07**
10	46.49	46.64	64.79	46.84	47.09	47.14	0.13	0.4921033	0.000492103	8.20172E-06	7.02829766	7.92829766	0.778	**1.448E-07**	**1.04**
0							0	0	0	0					
													Average Values =	**1.798E-07**	**1.3**







Flow Behavior: Void Filling
Lugeon: 1.04
Hydraulic Conductivity: 1.44765E-07

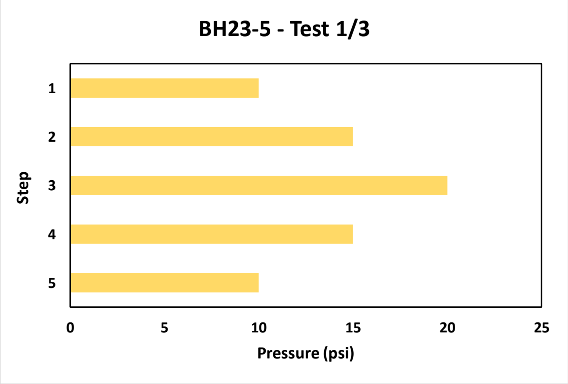
Packer Testing Analysis
Kashwakamak Lake Dam Replacement– Ontario

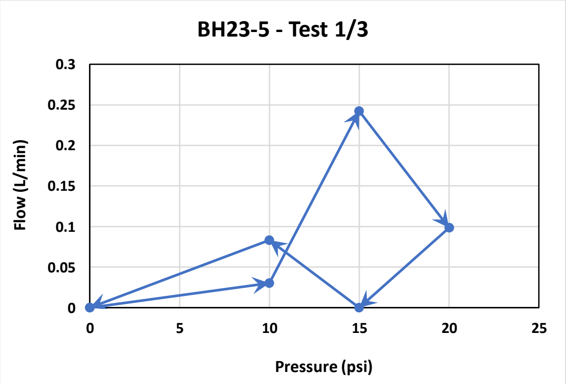
CCO-23-3603

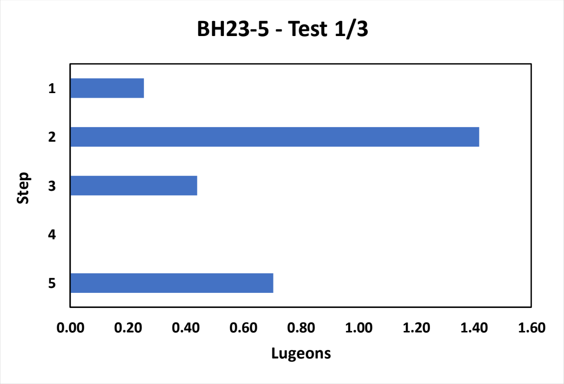
Table D.10

BH23-5 - Test 1/3

Top of section(ft) =		15.67	Bottom of Section (ft) =			20.67	Total length (ft)=			5	Total length (m)=				1.52
Surface Elevation (m) =		262.1	Section Int. El. (m)		257.3	255.8	Depth to center of tested section (m) =			5.54					
Depth to Groundwater (m) =		0	H_{gauge} above ground (m) =			0.9									
	Flow (Gallon)														
Presure (Psi)	0 min	1 min	2 min	3 min	4 min	5 min	Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= $P_{gauge}/\rho.g$	ΔH (m)	Net Inj Pressure p (bar)	K (m/s)	Lugeons, V
0							0	0	0	0					
10	58.46	58.58	58.59	58.5	58.5	58.5	0.008	0.03028328	3.02833E-05	5.04721E-07	7.02829766	7.92829766	0.778	2.649E-08	0.26
15	58.63	58.71	58.88	58.82	58.96	58.95	0.064	0.24226624	0.000242266	4.03777E-06	10.5424465	11.4424465	1.123	1.468E-07	1.42
20	58.46	58.48	58.58	58.59	58.59	58.59	0.026	0.09842066	9.84207E-05	1.64034E-06	14.0565953	14.9565953	1.467	4.564E-08	0.44
15	58.5	58.5	58.5	58.5	58.5	58.5	0	0	0	0	10.5424465	11.4424465	1.123	0	0.00
10	58.55	58.55	58.55	58.66	58.66	58.66	0.022	0.08327902	8.3279E-05	1.38798E-06	7.02829766	7.92829766	0.778	7.285E-08	0.70
0							0	0	0	0					
													Average Values =	7.295E-08	0.6





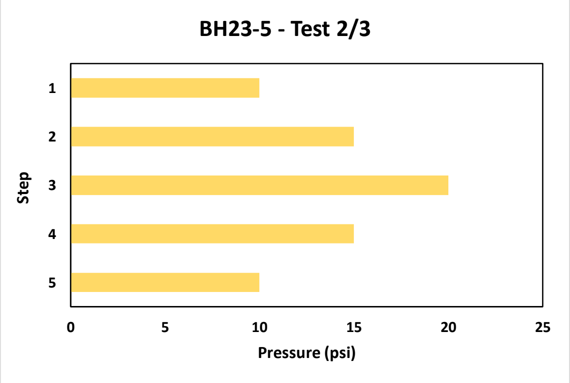


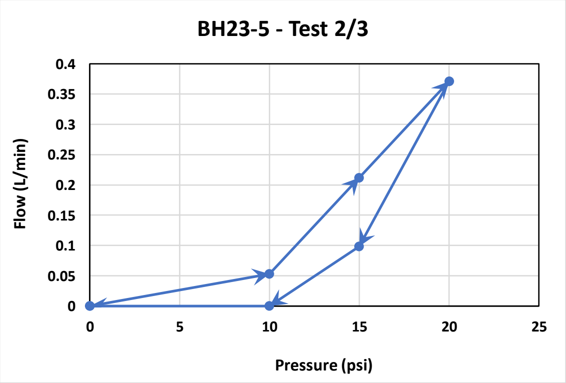
Flow Behavior: Void Filling
Lugeon: 0.70
Hydraulic Conductivity: 7.28453E-08

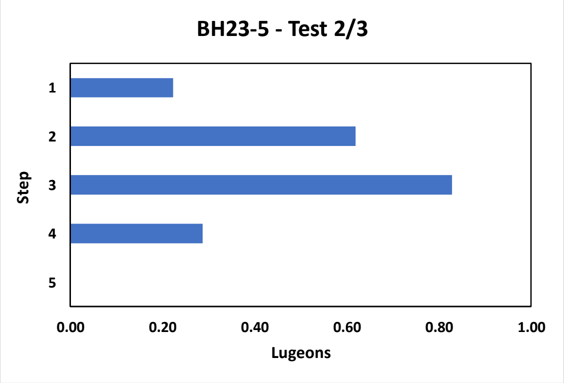
Table D.11

BH23-5 - Test 2/3

Top of section(ft) =		10.67	Bottom of Section (ft) =			20.67	Total length (ft)=			10	Total length (m)=				3.05
Surface Elevation (m) =		262.1	Section Int. El. (m)		258.8	255.8	Depth to center of tested section (m) =					4.78			
Depth to Groundwater (m) =		0	H_{gauge} above ground (m) =			0.9									
	Flow (Gallon)														
Presure (Psi)	0 min	1 min	2 min	3 min	4 min	5 min	Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= $P_{gauge}/\rho.g$	ΔH (m)	Net Inj Pressure *p* (bar)	**K (m/s)**	**Lugeons, V**
0							0	0	0	0					
10	59.63	59.78	59.79	59.79	59.7	59.7	0.014	0.05299574	5.29957E-05	8.83262E-07	7.02829766	7.92829766	0.778	**2.715E-08**	**0.22**
15	59.71	59.75	59.89	59.82	59.86	59.99	0.056	0.21198296	0.000211983	3.53305E-06	10.5424465	11.4424465	1.123	**7.525E-08**	**0.62**
20	60.09	60.12	60.21	60.31	60.49	60.58	0.098	0.37097018	0.00037097	6.18284E-06	14.0565953	14.9565953	1.467	**1.007E-07**	**0.83**
15	60.52	60.55	60.67	60.69	60.62	60.65	0.026	0.09842066	9.84207E-05	1.64034E-06	10.5424465	11.4424465	1.123	**3.494E-08**	**0.29**
10	60.65	60.65	60.65	60.65	60.65	60.65	0	0	0	0	7.02829766	7.92829766	0.778	**0**	**0.00**
0							0	0	0	0					
													Average Values =	**5.952E-08**	**0.4**







Flow Behavior: Dilation
Lugeon: 0.22
Hydraulic Conductivity: 4.14817E-08

Packer Testing Analysis
Kashwakamak Lake Dam Replacement– Ontario

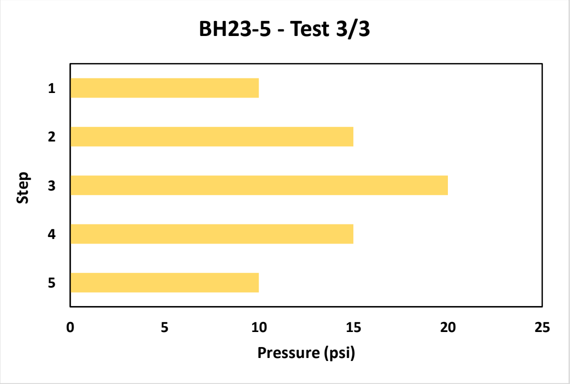
CCO-23-3603

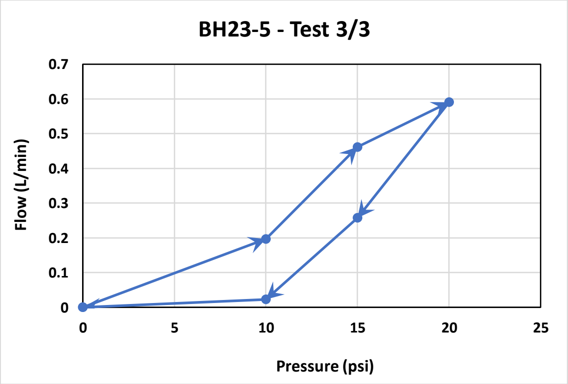
Table D.12

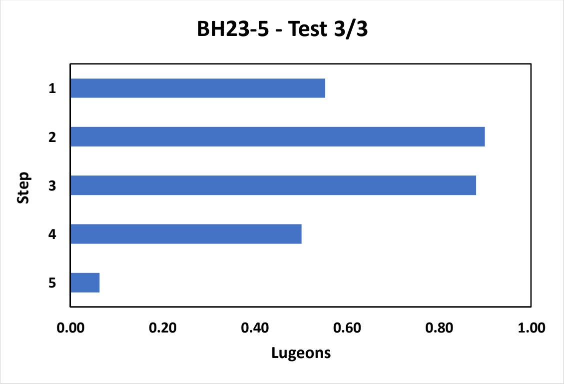
BH23-5 - Test 3/3

Top of section(ft) =	5.67	Bottom of Section (ft) =	20.67	Total length (ft)=	15	Total length (m)=	4.57
Surface Elevation (m) =	262.1	Section Int. El. (m)	260.4	255.8	Depth to center of tested section (m) =	4.01	
Depth to Groundwater (m) =	0	H_{gauge} above ground (m) =	0.9				

Presure (Psi)	Flow (Gallon)						Average Flow Rate (Gallon/min)	Average Flow Rate (L/min)	Average Flow Rate (m3/min)	Average Flow Rate (m3/Sec)	H_{gauge}= $P_{gauge}/\rho.g$	ΔH (m)	Net Inj Pressure p (bar)	K (m/s)	Lugeons, V
	0 min	1 min	2 min	3 min	4 min	5 min									
0							0	0	0	0					
10	50.98	50.96	51.04	51.11	51.27	51.24	0.052	0.19684132	0.000196841	3.28069E-06	7.02829766	7.92829766	0.778	7.313E-08	0.55
15	51.33	51.57	51.69	51.71	51.83	51.94	0.122	0.46182002	0.00046182	7.697E-06	10.5424465	11.4424465	1.123	1.189E-07	0.90
20	52.01	52.15	55.39	52.42	52.66	52.79	0.156	0.59052396	0.000590524	9.84207E-06	14.0565953	14.9565953	1.467	1.163E-07	0.88
15	52.75	52.89	52.84	53	52.95	53.09	0.068	0.25740788	0.000257408	4.29013E-06	10.5424465	11.4424465	1.123	6.626E-08	0.50
10	53.17	53.19	53.12	53.16	53.27	53.2	0.006	0.02271246	2.27125E-05	3.78541E-07	7.02829766	7.92829766	0.778	8.438E-09	0.06
0							0	0	0	0					
													Average Values =	7.66E-08	0.6







Flow Behavior: Void Filling
Lugeon: 0.06
Hydraulic Conductivity: 8.43821E-09

PROPOSED KASHWAKAMAK LAKE DAM REPLACEMENT – NORTH FRONTENAC TWN. ON

APPENDIX E
LAB RESULTS

McINTOSH PERRY

Unconfined Compressive Strength of Intact Rock Cores

ASTM D7012 Method C

Project No.:	CCO-23-3603-03	**Date Issued:**	Nov 14,2023
Lab No.:	OL-23063	**Report No.:**	1 of 4
Project Name:	Kashwakamak Dam		

Core No.:	1	**Moisture Condition:**	Dry as received		
Borehole Location:	BH23-1	**RC:**	2	**Depth (ft):**	5'3.5"-5'8.5"
Date Sampled:	Sept 25,2023	**Received:**	Nov 2,2023	**Tested:**	Nov 7,2023
Core No.:	2	**Moisture Condition:**	Dry as received		
Borehole Location:	BH23-1	**RC:**	4	**Depth (ft):**	11'2"-11'6"
Date Sampled:	Sept 25,2023	**Received:**	Nov 2,2023	**Tested:**	Nov 7,2023
Core No.:	3	**Moisture Condition:**	Dry as received		
Borehole Location:	BH23-1	**RC:**	6	**Depth (ft):**	18'2"-18'7"
Date Sampled:	Sept 25,2023	**Received:**	Nov 2,2023	**Tested:**	Nov 7,2023

Core No. :	1	2	3
Diameter (mm)	49.1	49.1	49.1
Thickness/Height (mm)	117.2	97.6	118.8
Density (Kg/m^3)	2747	2718	2732
Compressive Strength (Mpa)	164	167.2	177.3
Mass of Core (kg)	0.6084	0.4991	0.6157
Description of Failure	3	3	1

Remarks: Core# 1&2 Columnar vertical cracking through both ends. No well formed Cones on ether end.

Core # 3 Reasonably well formed cones on both ends.

Reviewed By: **Date:** Nov 14,2023

Jason Hopwood-Jones
Laboratory Manager

Unconfined Compressive Strength of Intact Rock Cores

ASTM D7012 Method C

Project No.:	CCO-23-3603-03	**Date Issued:**	Nov 14,2023
Lab No.:	OL-23063	**Report No.:**	2 of 4
Project Name:	Kashwakamak Dam		

Core No.:	4	**Moisture Condition:**	Dry as received		
Borehole Location:	BH23-2	**RC:**	1	**Depth (ft):**	2'2"-2'7"
Date Sampled:	Sept 25,2023	**Received:**	Nov 2,2023	**Tested:**	Nov 7,2023
Core No.:	5	**Moisture Condition:**	Dry as received		
Borehole Location:	BH23-2	**RC:**	4	**Depth (ft):**	11'8"-12'1"
Date Sampled:	Sept 25,2023	**Received:**	Nov 2,2023	**Tested:**	Nov 7,2023
Core No.:	6	**Moisture Condition:**	Dry as received		
Borehole Location:	BH23-2	**RC:**	7	**Depth (ft):**	17'5"-17'10"
Date Sampled:	Sept 25,2023	**Received:**	Nov 2,2023	**Tested:**	Nov 7,2023

Core No. :	4	5	6
Diameter (mm)	49.1	49.1	49.1
Thickness/Height (mm)	120.1	114.8	118
Density (Kg/m^3)	2785	2793	2762
Compressive Strength (Mpa)	201.1	208.2	194.8
Mass of Core (kg)	0.6332	0.6070	0.6171
Description of Failure	1	2	2

Remarks: Core # 4 Reasonably well formed cones on both ends.

Core # 5 & 6 Well formed cone on one end and vertical cracking through bottom.

Reviewed By: **Date:** Nov 14,2023

Jason Hopwood-Jones
Laboratory Manager

McINTOSH PERRY

Unconfined Compressive Strength of Intact Rock Cores

ASTM D7012 Method C

Project No.:	CCO-23-3603-03	**Date Issued:**	Nov 14,2023
Lab No.:	OL-23063	**Report No.:**	3 of 4
Project Name:	Kashwakamak Dam		

Core No.:	7	**Moisture Condition:**		Dry as received	
Borehole Location:	BH23-4	**RC:**	7	**Depth (ft):**	15'1"-15'6"
Date Sampled:	Sept 25,2023	**Received:**	Nov 2,2023	**Tested:**	Nov 7,2023
Core No.:	8	**Moisture Condition:**		Dry as received	
Borehole Location:	BH23-5	**RC:**	2	**Depth (ft):**	4'11"-5'4"
Date Sampled:	Sept 25,2023	**Received:**	Nov 2,2023	**Tested:**	Nov 7,2023
Core No.:	9	**Moisture Condition:**		Dry as received	
Borehole Location:	BH23-5	**RC:**	4	**Depth (ft):**	10'10"-11'3"
Date Sampled:	Sept 25,2023	**Received:**	Nov 2,2023	**Tested:**	Nov 7,2023

Core No. :	7	8	9
Diameter (mm)	47.4	49.4	49.4
Thickness/Height (mm)	116.2	118.8	117.1
Density (Kg/m^3)	2736	2793	2844
Compressive Strength (Mpa)	157.8	211.3	173.9
Mass of Core (kg)	0.5610	0.6359	0.6383
Description of Failure	4	4	1

Remarks: Core # 7 & 8 Diagonal fracture with some cracking through ends.

Core # 9 Reasonably well formed cones on both ends.

Reviewed By:

Date: Nov 14,2023

Jason Hopwood-Jones
Laboratory Manager

Unconfined Compressive Strength of Intact Rock Cores

ASTM D7012 Method C

Project No.:	CCO-23-3603-03	**Date Issued:**	Nov 14,2023
Lab No.:	OL-23063	**Report No.:**	4 of 4
Project Name:	Kashwakamak Dam		

Core No.:	10	**Moisture Condition:**	Dry as received		
Borehole Location:	BH23-5	**RC:**	6	**Depth (ft):**	16'9"-17'2"
Date Sampled:	Sept 25,2023	**Received:**	Nov 2,2023	**Tested:**	Nov 7,2023
Core No.:		**Moisture Condition:**			
Borehole Location:		**RC:**		**Depth (ft):**	
Date Sampled:		**Received:**		**Tested:**	
Core No.:		**Moisture Condition:**			
Borehole Location:		**RC:**		**Depth (ft):**	
Date Sampled:		**Received:**		**Tested:**	

Core No. :	10		
Diameter (mm)	50.7		
Thickness/Height (mm)	116.9		
Density (Kg/m^3)	2757		
Compressive Strength (Mpa)	126.2		
Mass of Core (kg)	0.6506		
Description of Failure	1		

Remarks: Core # 10 Reasonably well formed cones on both ends.

Reviewed By: **Date:** Nov 14,2023

Jason Hopwood-Jones
Laboratory Manager

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Certificate of Analysis

McIntosh Perry Consulting Eng. (Nepean)
215 Menten Place, Unit 104
Nepean, ON K2H 9C1
Attn: Jeff Forrester

Report Date: 3-Nov-2023
Order Date: 31-Oct-2023

Client PO: CCO-23-3603
Project: CCO-23-3603 (Kashwakamak Dam)
Custody: 66483

Order #: 2344177

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
2344177-01	CCO-23-3603

Approved By:

Dale Robertson, BSc
Laboratory Director

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC	1-Nov-23	1-Nov-23
pH	EPA 150.1 - pH probe @25 °C	1-Nov-23	1-Nov-23
Resistivity	EPA 120.1 - probe	1-Nov-23	1-Nov-23

Certificate of Analysis

Report Date: 03-Nov-2023

Client: **McIntosh Perry Consulting Eng. (Nepean)**

Order Date: 31-Oct-2023

Client PO: CCO-23-3603

Project Description: CCO-23-3603 (Kashwakamak Dam)

Client ID:		CCO-23-3603	-	-	-		
Sample Date:		31-Oct-23 09:00	-	-	-	-	-
Sample ID:		2344177-01	-	-	-		
Matrix:		Surface Water	-	-	-		
	MDL/Units						
General Inorganics							
pH	0.1 pH Units	7.9	-	-	-	-	-
Resistivity	0.01 Ohm.m	91.7	-	-	-	-	-
Anions							
Chloride	1 mg/L	5	-	-	-	-	-
Sulphate	1 mg/L	3	-	-	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions								
Chloride	ND	1	mg/L					
Sulphate	ND	1	mg/L					
General Inorganics								
Resistivity	ND	0.01	Ohm.m					

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	490	5	mg/L	498			1.6	20	
Sulphate	213	1	mg/L	210			1.3	10	
General Inorganics									
pH	7.9	0.1	pH Units	8.0			0.4	3.3	
Resistivity	12.9	0.01	Ohm.m	12.8			1.3	20	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	9.78	1	mg/L	ND	97.8	78-114			
Sulphate	220	1	mg/L	210	100	74-126			

Qualifier Notes:

Sample Data Revisions:

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

NC: Not Calculated

PARACEL
LABORATORIES LTD.

Paracel ID: 2344177

Paracel Order Number
(Lab Use Only)
2344177

Chain Of Custody
(Lab Use Only)
№ 66483

Client Name: McIntosh Perry
Contact Name: Jeff Forrester
Address: 215, Menten Place, Unit 104,
Ottawa, ON, K2H-9C1
Telephone:

Project Ref: CCO-23-3603 (Kashwakamak Dam)
Quote #:
PO #: CCO-23-3603
E-mail: j.forrester@mcintoshperry.com

Page 1 of 1

Turnaround Time
☐ 1 day ☐ 3 day
☐ 2 day ☒ Regular
Date Required:

☐ REG 153/04 ☐ REG 406/19
☐ Table 1 ☐ Res/Park ☐ Med/Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/Other
☐ Table
For RSC: ☐ Yes ☐ No

Other Regulation
☐ REG 558 ☐ PWQO
☐ CCME ☐ MISA
☐ SU - Sani ☐ SU - Storm
Mun:
☐ Other:

Matrix Type: S (Soil/Sed.) GW (Ground Water)
SW (Surface Water) SS (Storm/Sanitary Sewer)
P (Paint) A (Air) O (Other)

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken		Required Analysis											
					Date	Time	Corrosivity Package	(Sulphate, pH,	Chloride, Resistivity).									
1	CCO-23-3603	SW	0	1	Oct, 31, 2023	09:00 AM												
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

Comments:
Test for corrosivity Package (Sulphate, PH, chloride, Resistivity).

Method of Delivery: Walk-in

Relinquished By (Sign):
Relinquished By (Print): Jay Patel
Date/Time: October 31, 2023

Received By Driver/Depot: Scouse
Date/Time: 31/10/23 1232
Temperature: 19.7 °C

Received at Lab:
Date/Time: Oct.31 2023 1327
Temperature: 11.1 °C

Verified By: Hisal
Date/Time: Oct 31, 23 | 15:1
pH Verified: ☐ By:

Chain of Custody (Blank).xlsx

Revision 4.0

PROPOSED KASHWAKAMAK LAKE DAM REPLACEMENT – NORTH FRONTENAC TWN. ON

APPENDIX F

SEISMIC HAZARD CALCULATIONS

McINTOSH PERRY

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 44.892N 76.959W **User File Reference:** Kashwakamak Lake Dam 2023-11-16 19:28 UT

Requested by: McIntosh Perry

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.139	0.087	0.058	0.021
Sa (0.1)	0.184	0.120	0.082	0.032
Sa (0.2)	0.175	0.116	0.080	0.033
Sa (0.3)	0.146	0.098	0.069	0.028
Sa (0.5)	0.117	0.079	0.054	0.022
Sa (1.0)	0.069	0.046	0.031	0.011
Sa (2.0)	0.036	0.023	0.015	0.005
Sa (5.0)	0.010	0.006	0.004	0.001
Sa (10.0)	0.004	0.002	0.002	0.001
PGA (g)	0.106	0.069	0.046	0.018
PGV (m/s)	0.100	0.063	0.041	0.014

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

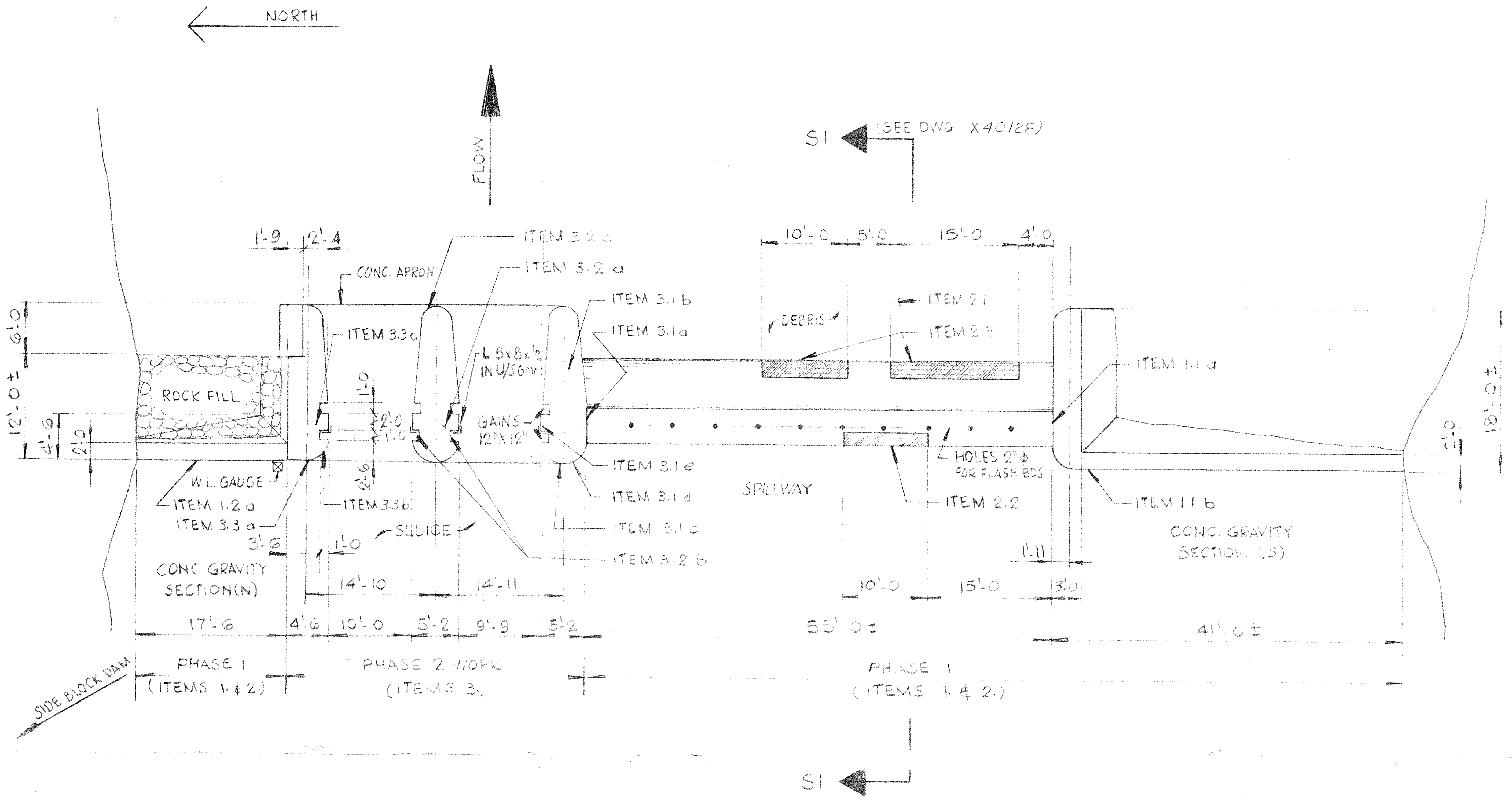
Natural Resources Canada Ressources naturelles Canada

Canada

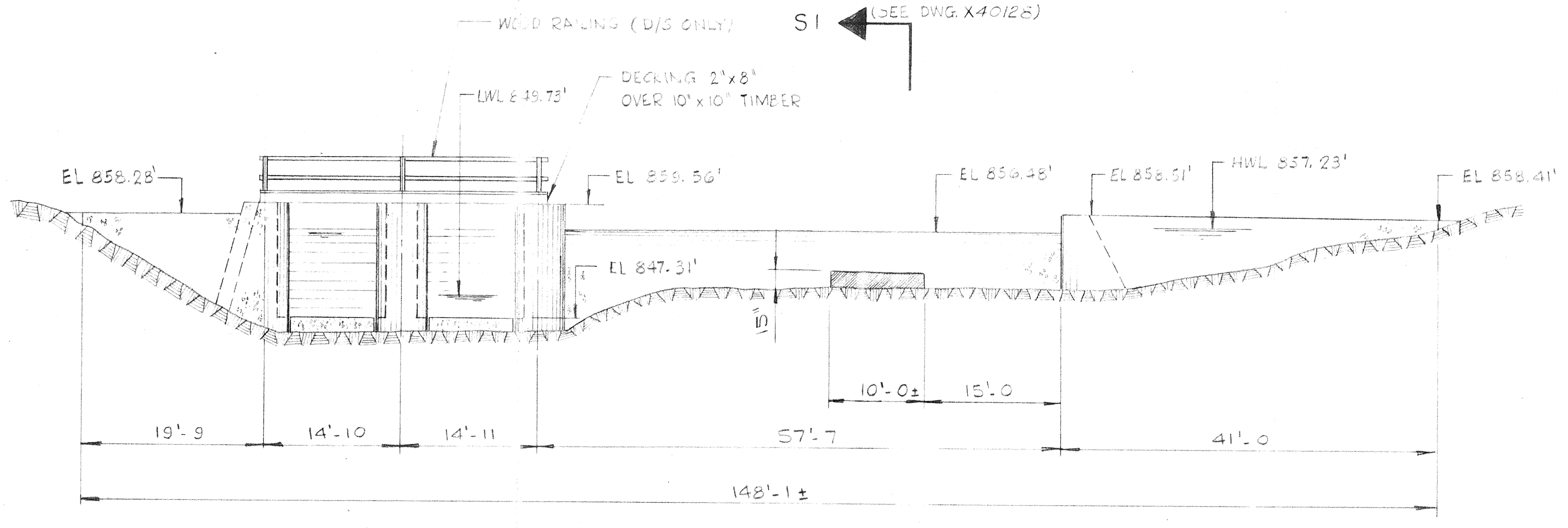
PROPOSED KASHWAKAMAK LAKE DAM REPLACEMENT – NORTH FRONTENAC TWN. ON

APPENDIX G
AVAILABEL DRAWINGS
AND DOCUMENTS

McINTOSH PERRY



PLAN
1" = 10'-0



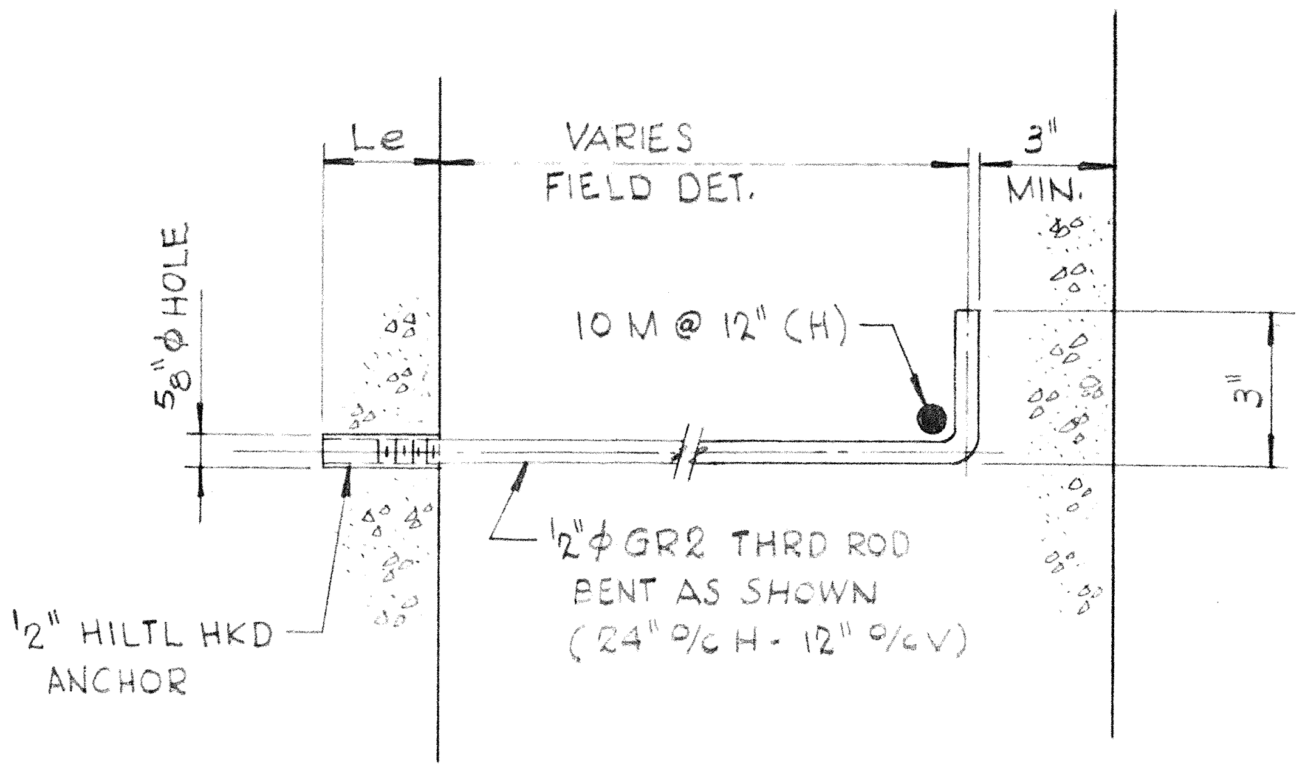
ELEVATION
VIEW LOOKING D/S
1"=1'-0

rev no	date	particulars	dwn / chkd	appd

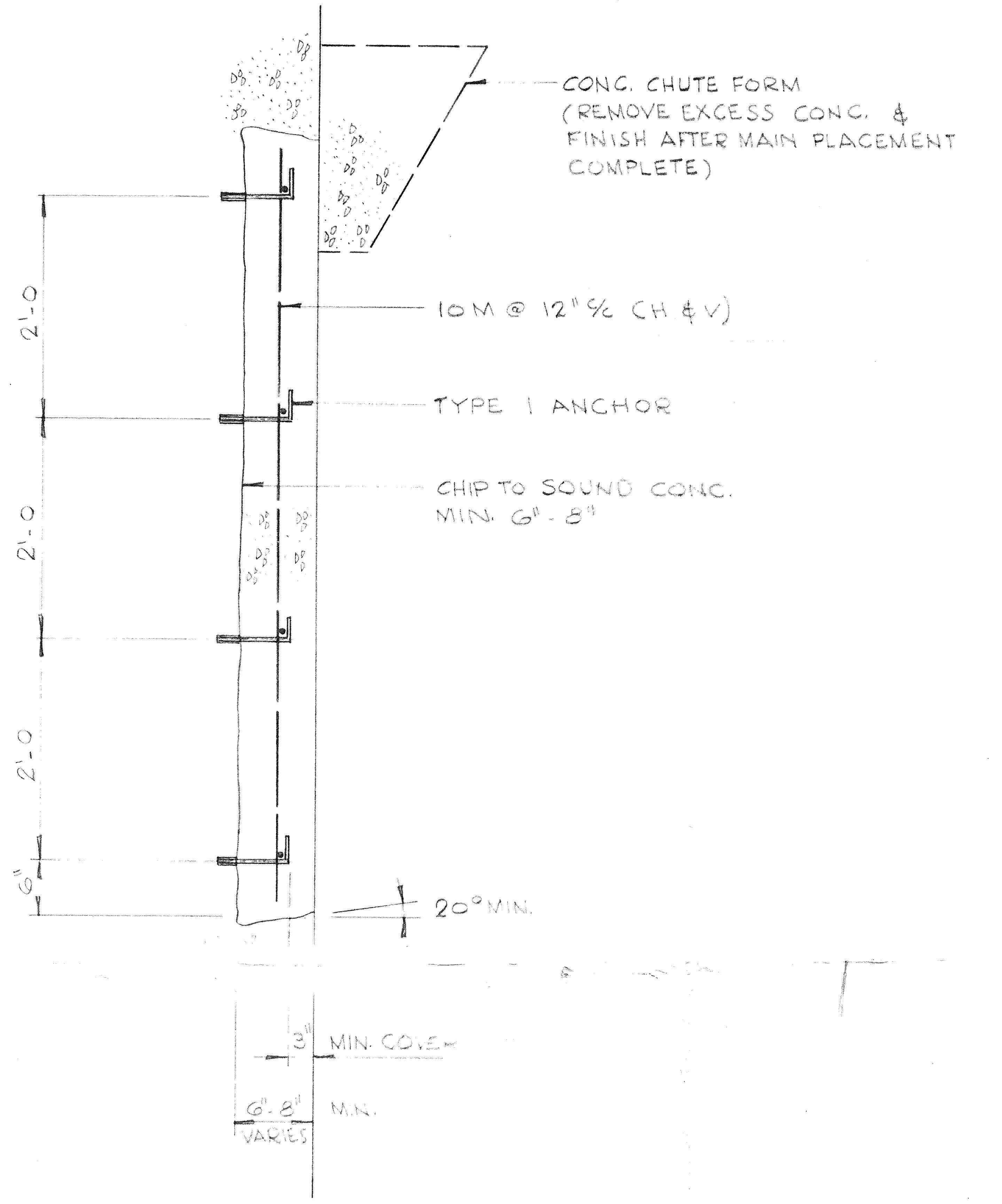
Ontario Hydro
thermal & hydraulic generation division
civil works department

KASHWAKAMAK LAKE DAM
SPILLWAY AND GRAVITY SECTIONS
CONCRETE MAINTENANCE
(PHASE 1 - 1986)
PLAN - ELEVATION - SCOPE OF WORK

charge/work order	designed by	design checked by
	drawn by RZ	drawing checked by
scale 1" = 10'-0	date 86-05	approved LW Coley
drawing no. X40127		rev. no. 0



DETAIL 1
TYPE 1 ANCHOR
3" = 1'-0



DETAIL 2
CONC. REPAIR
1" = 1'-0

USE FOR SCOPE ITEMS 1.1 AND 1.2

NOTES

MATERIALS

1- TYPE 1 ANCHORS - SAE J429d GR2 ROD COMPLETE WITH HILTI HKD ANCHORS.

2. REINFORCING - CSA G30-12M77 (Fy = 350MPa)

3- CONCRETE -
  (a) CEMENT TYPE - TYPE 30
  (b) MAX. AGG. SIZE - 10mm (3/8")
  (c) AIR CONTENT - 6-9 PER CENT
  (d) STRENGTH 28 DAYS - 25MPa
  (e) HYDRO REF. SEC. APPLYING
    - W-25.83 - CONC. WORKMANSHIP
    - M 182m - 78 CONC. AGGREGATES
    - M 241 - 84 READY MIXED CONC.
  (f) CONC. PATCH (WALER BOLTS ETC.)
    - THORITE BY THOROSYSTEM PRODUCTS OF CANADA LTD.
  (g) MOIST CORE 7 DAYS MIN.

METHODS

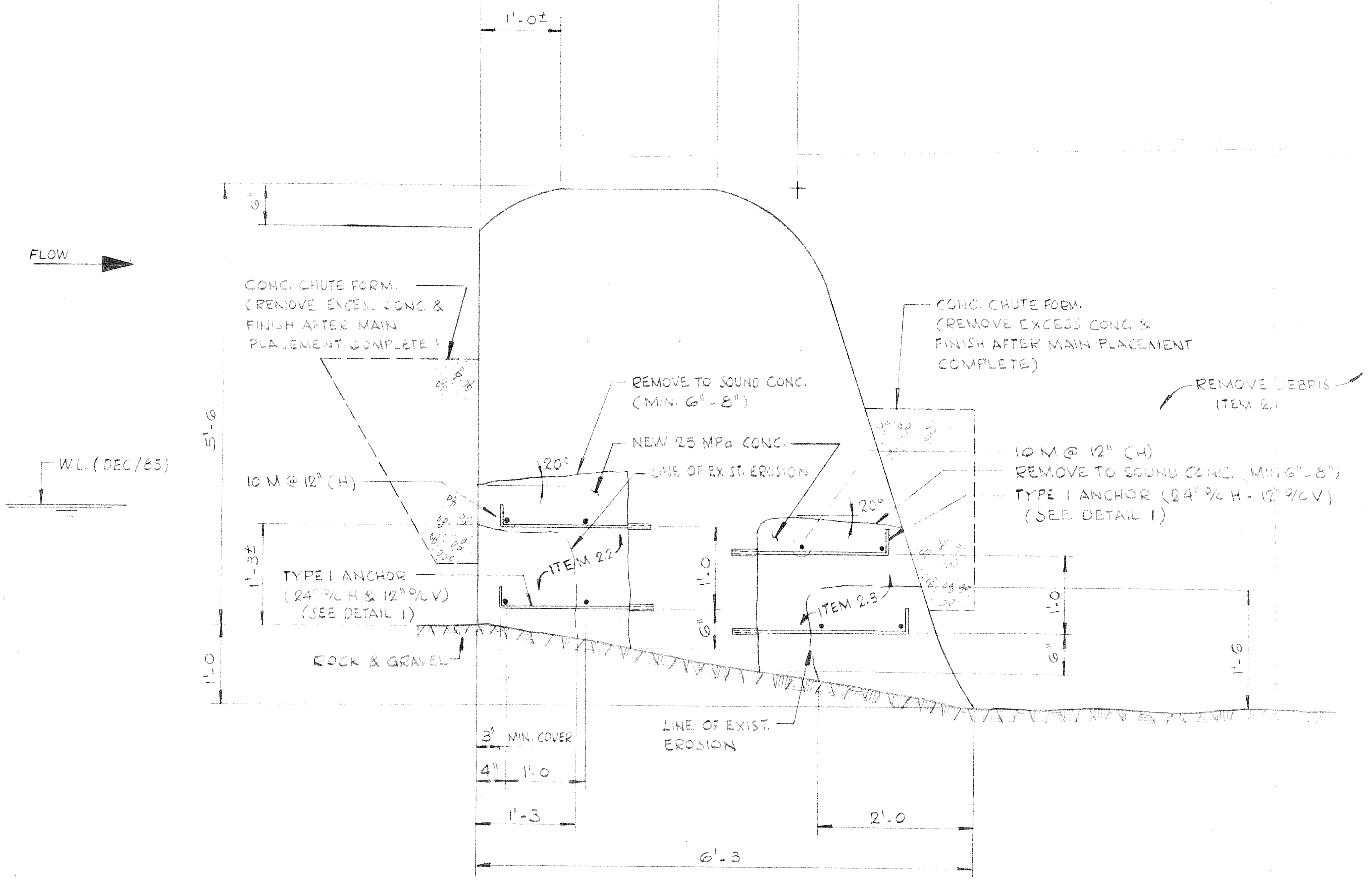
1- CHIP TO SOUND CONCRETE FROM EXISTING LINE OF EROSION 6"-8" MIN.

2. WATER JET EXISTING SURFACE PRIOR TO PLACING THE NEW CONCRETE.

3- MOISTEN EXISTING CONCRETE THOROGHLY PRIOR TO PLACING NEW CONCRETE.

4- CONCRETE WORKMANSHIP TO O.H.S. W-25

5. HEAT FOR CURING PERIOD WILL BE REQUIRED WHEN AIR TEMP. IS < 10°C



SECTION S1 (SEE DWG. X40127)
1" = 1'-0

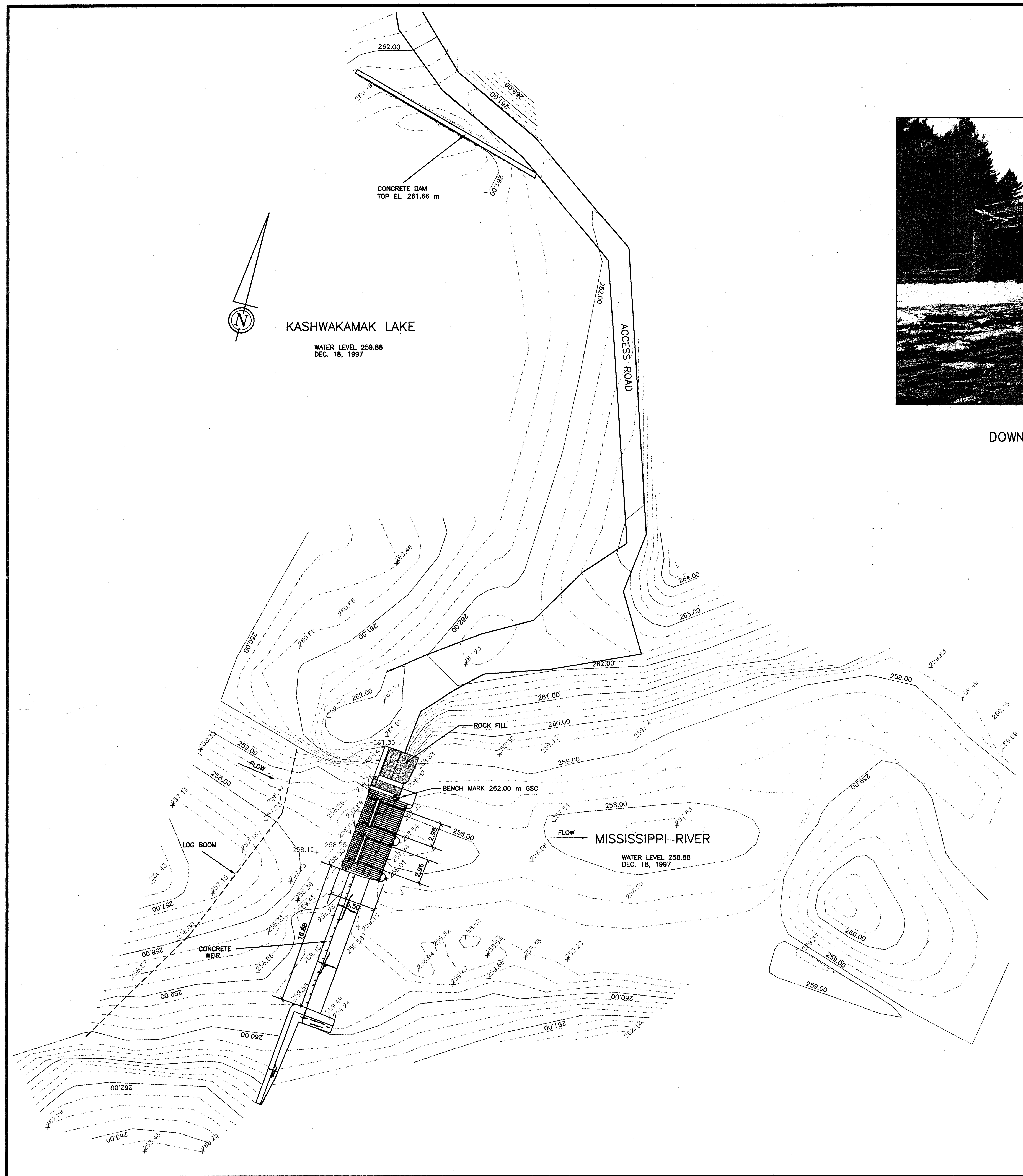
USE FOR SCOPE ITEMS 2.2 & 2.3

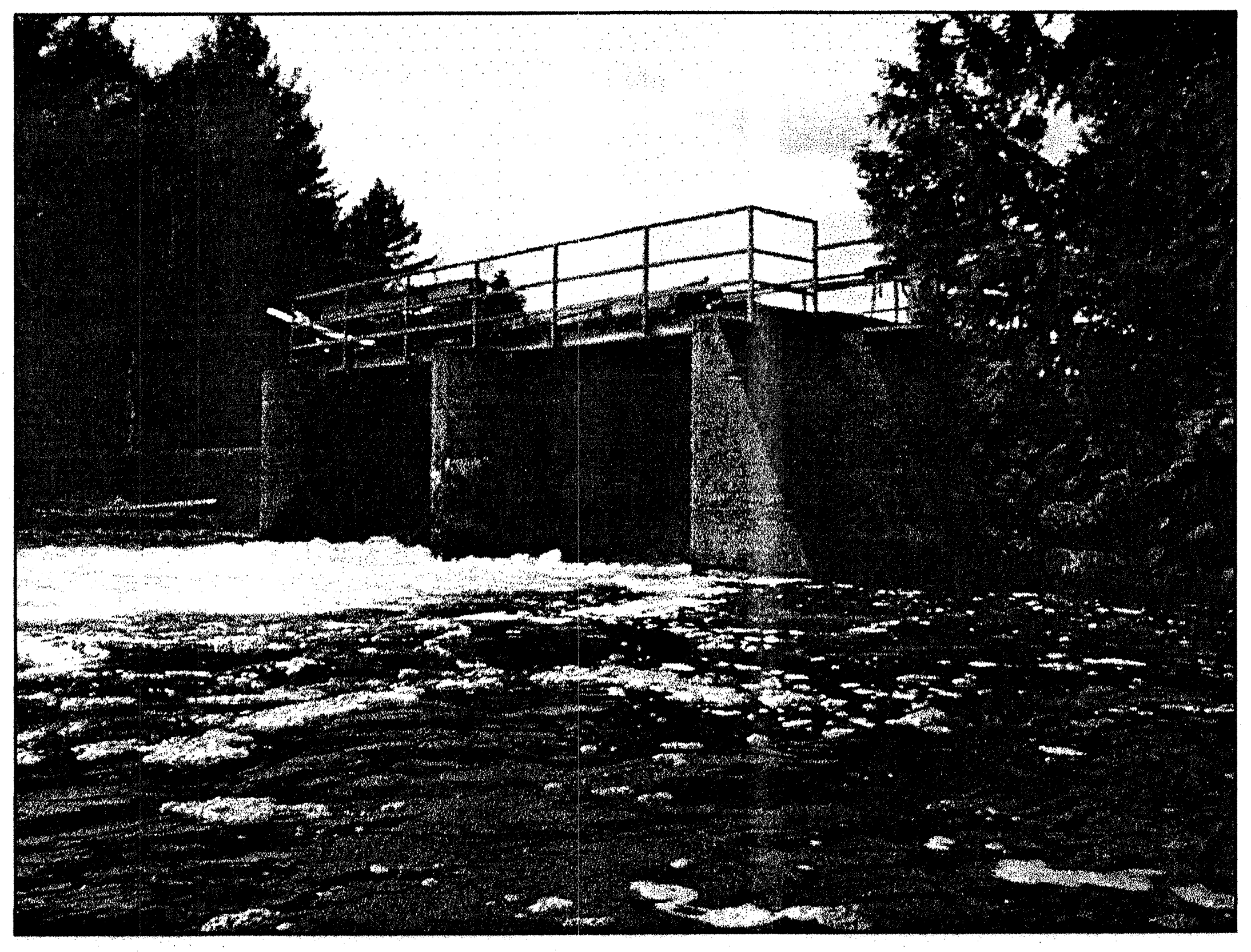
Ontario Hydro
thermal & hydraulic generation division
civil works department

KASHWAKAMAK LAKE DAM
SPILLWAY AND GRAVITY SECTIONS
CONCRETE MAINTENANCE
(PHASE 1 - 1986)
SECTIONS AND DETAILS

charge/work order	designed by	design checked by
	drawn by R. Z.	drawing checked by
scale AS NOTED	date 86-05	approved L N Coley
drawing no. X40128		rev. no. 0

rev no	date	particulars	dwn/chkd	appd





DOWNSTREAM VIEW OF DAM

NO.	REVISIONS	DATE	APP.
B	ISSUED FOR CONSTRUCTION	JULY 26 1999	
A	RELEASED FOR INFORMATION	OCT. 22 1998	

GENERAL NOTES

- WORK IN THE DRY WHERE POSSIBLE.
- PROVIDE SEDIMENT AND EROSION CONTROL MEASURES.
- CLEAN EQUIPMENT BEFORE ENTERING WATER.
- CHECK EQUIPMENT FOR OIL LEAKS BEFORE ENTERING WATER.

SITE BENCHMARKS

BRASS CAP, TOP OF MOST NORTHERLY CONCRETE PIER
BETWEEN WOOD DECK AND STEEL CATWALK,
ELV. 262.00 m GSC.

EGA **ECOS GARATECH**
CONSULTING ENGINEERS
BRAMPTON (905) 458–4110 FAX: (905) 458–1479

OWNER/CLIENT:
MISSISSIPPI VALLEY CONSERVATION

PROJECT:
KASHWAKAMAK LAKE DAM

TITLE:
SITE PLAN TB

DESIGNED: N.A.	DRAWN: P.L.	REV. NO.: B
CHECKED: M.D.G.	DATE: OCT., 1998	DWG. NO. 1434–19–01
SCALE: 1:250	JOB: 1434–19	

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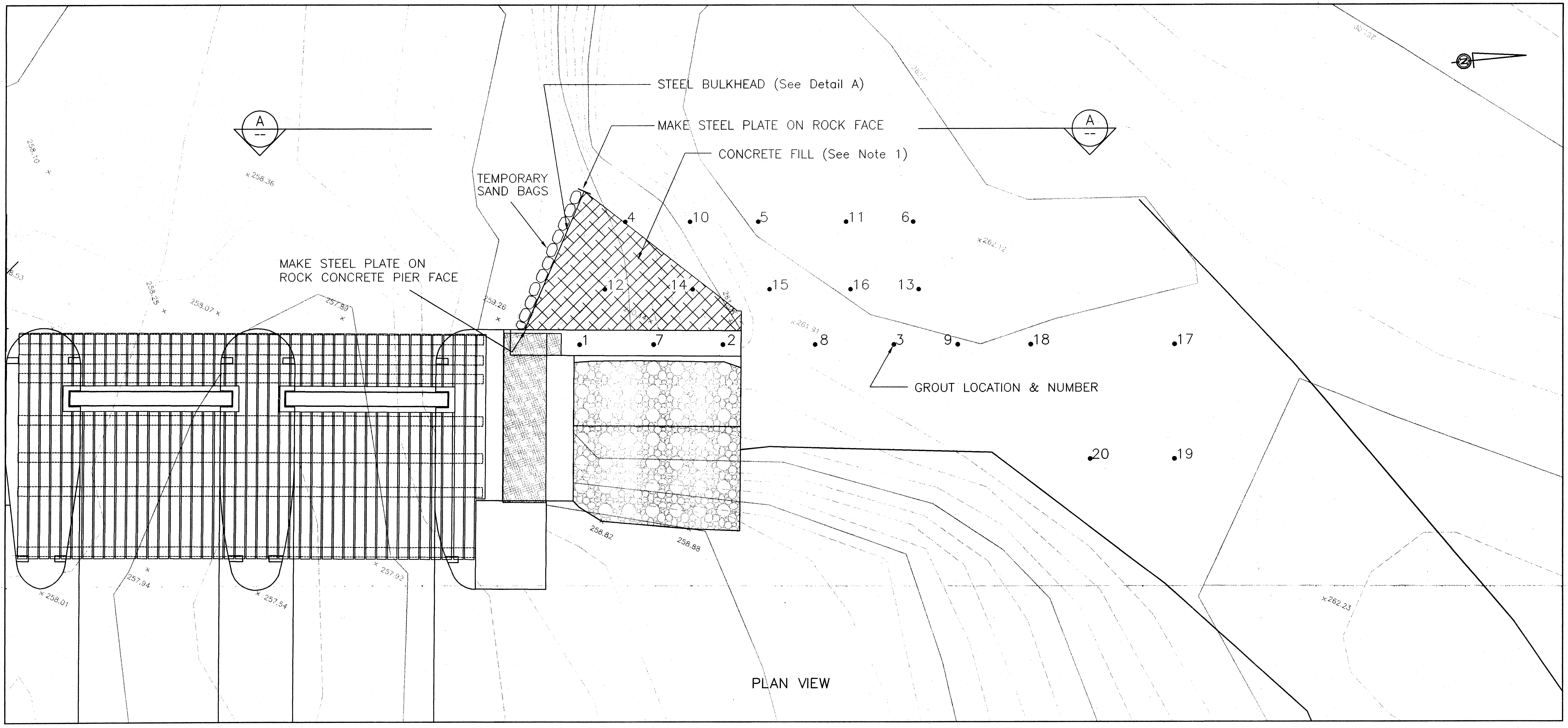
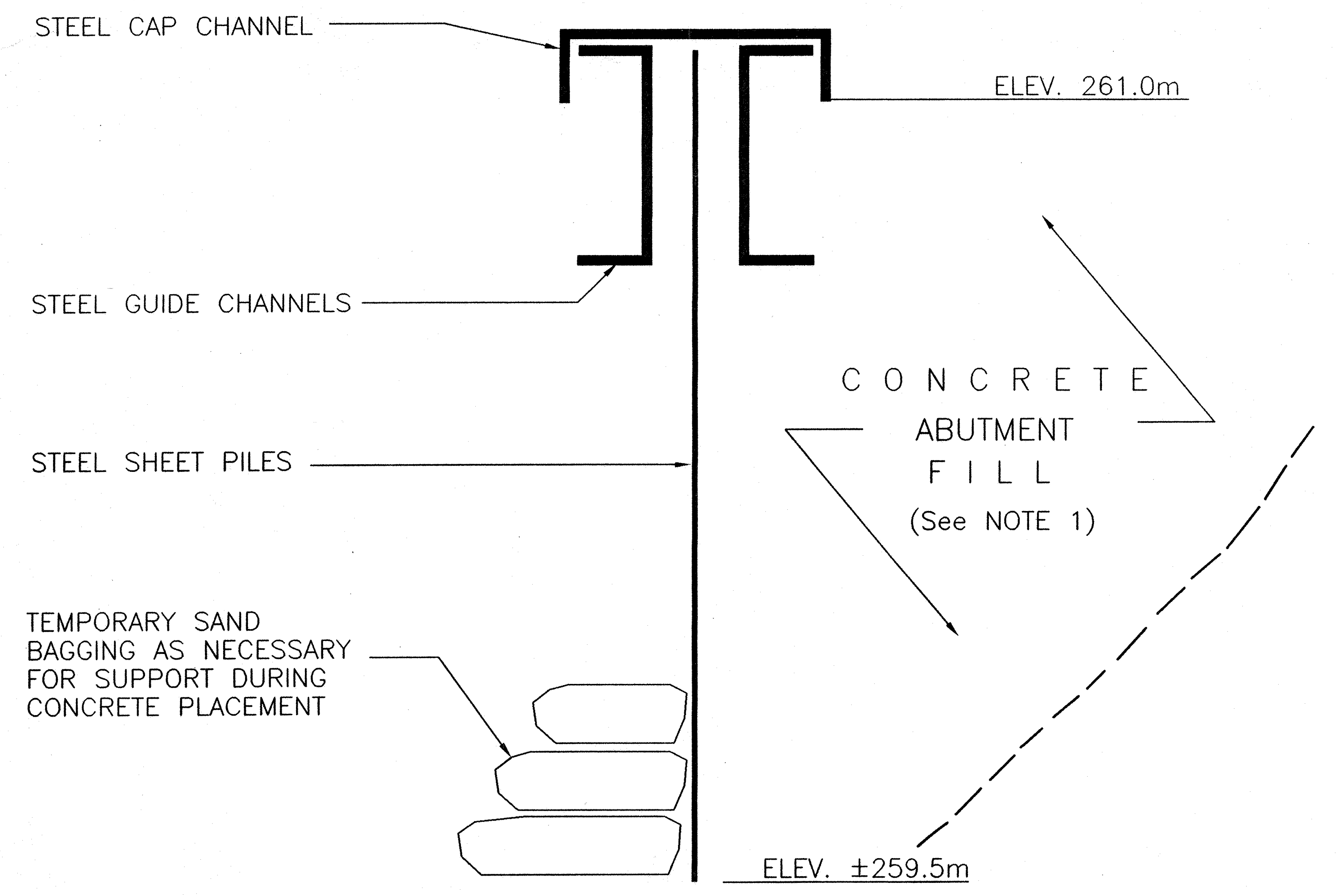
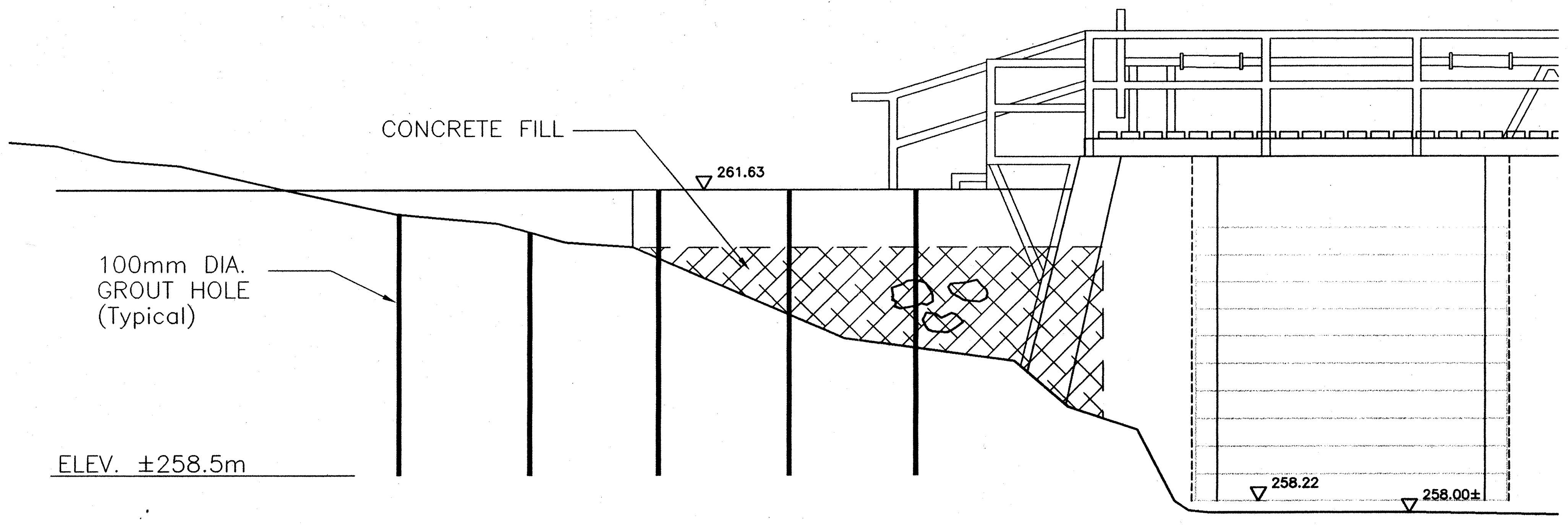


PLAN VIEW



DETAIL A
NOT TO SCALE

SUPPORT TO BE
BY PROFESSIONAL ENGINEER
SUBMIT STAMPED SHOP DRAWINGS



CONTROL STRUCTURE AND NORTH WINGWALL
UPSTREAM ELEVATION
SECTION A–A

NO.	REVISIONS	DATE	APP.
B	ISSUED FOR CONSTRUCTION	JULY 26 1999	
A	RELEASED FOR INFORMATION	OCT. 22 1998	

NOTE 1:
CONCRETE FILL BEFORE GROUTING

- REMOVE ALL DELETERIOUS MATERIAL, WOOD, LOOSE ROCK etc.
- INSTALL ROCK DOWELS & REBAR MAT.
- PLACE CONCRETE FILL TO ELEVATION 261.0m BY TREMIE METHOD OR APPROVED ALTERNATIVE. CONCRETE CSA A23.1–F2 EXPOSURE. FLOAT CONCRETE SURFACE AND BROOM FINISH.
- REMOVE TEMPORARY SAND BAGGING/FORMWORK
- FILL ALL WORKSPACE AREAS WITH CONCRETE.
- TRIM/BACKFILL AGAINST OUTER FACE OF APRON.

NOTE 2:
GROUTING

- DRILL HOLE TO MIN ELEV. 255m OR 5m DEPTH FROM EXISTING GRADE.
- WATER TEST ALL HOLES.

- GROUT SEQUENCE –
  1,2,3
  4,5,6,7,8,9
  10,11
  12,13
  14,15,16
  17,18,19,20

- GROUT: CEMENTIOUS GROUT
  1:1 CEMENT, WATER
  BY VOLUME

SITE BENCHMARKS

BRASS CAP, TOP OF MOST NORTHERLY CONCRETE PIER
BETWEEN WOOD DECK AND STEEL CATWALK,
ELV. 262.00 m GSC.

EGA **ECOS GARATECH**
CONSULTING ENGINEERS
BRAMPTON (905) 458–4110 FAX: (905) 458–1479

Terraprobe

OWNER/CLIENT:
MISSISSIPPI VALLEY CONSERVATION

PROJECT:
KASHWAKAMAK LAKE DAM

TITLE:
CONTROL STRUCTURE
AND NORTH BANK

DESIGNED: N.A.	DRAWN: P.L.	REV. NO.: B
CHECKED: M.D.G.	DATE: OCT., 1998	DWG. NO.
SCALE: 1:50	JOB: 1434–19	1434–19–02

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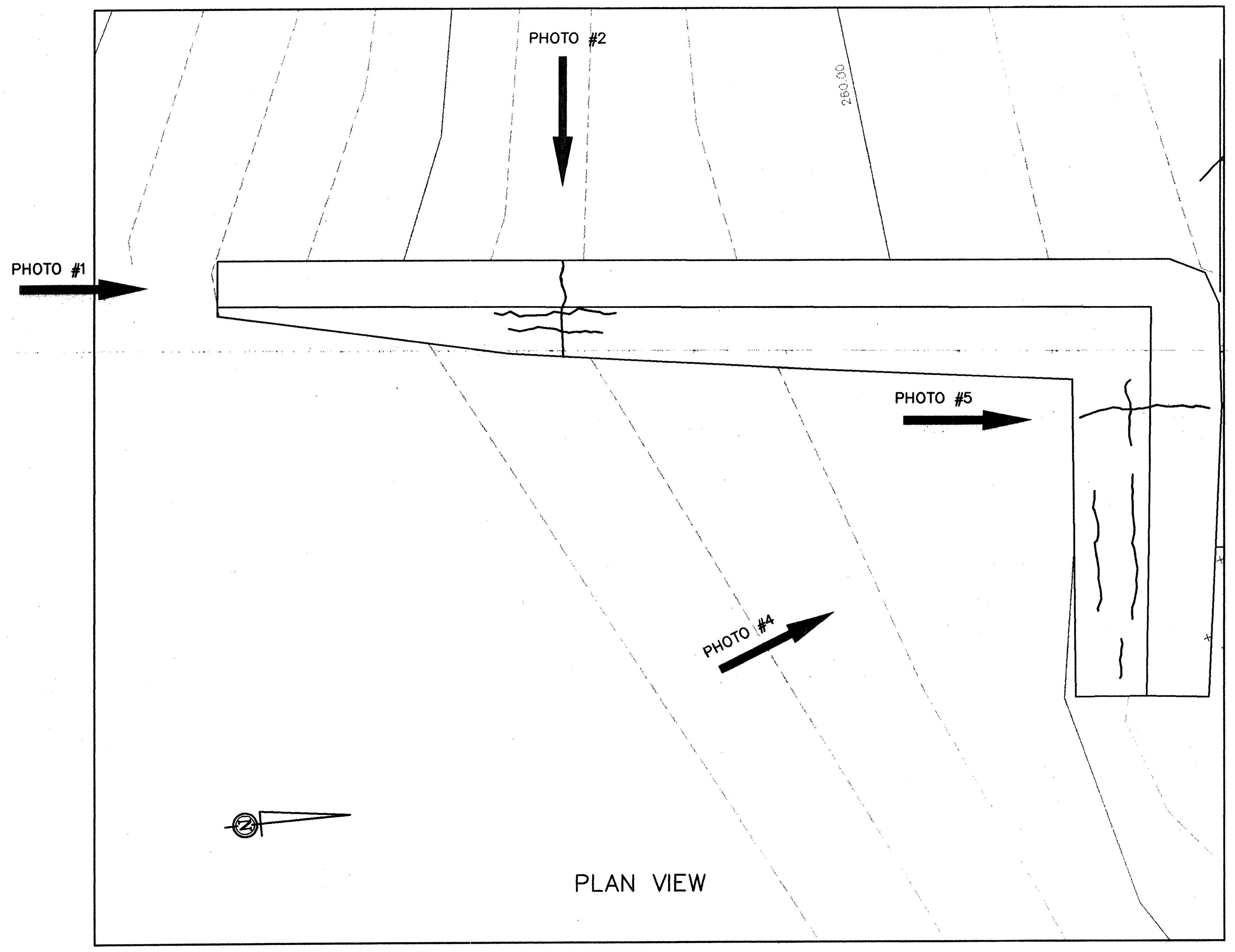
PHOTO #1

SEVERE
STRUCTURAL
CRACK

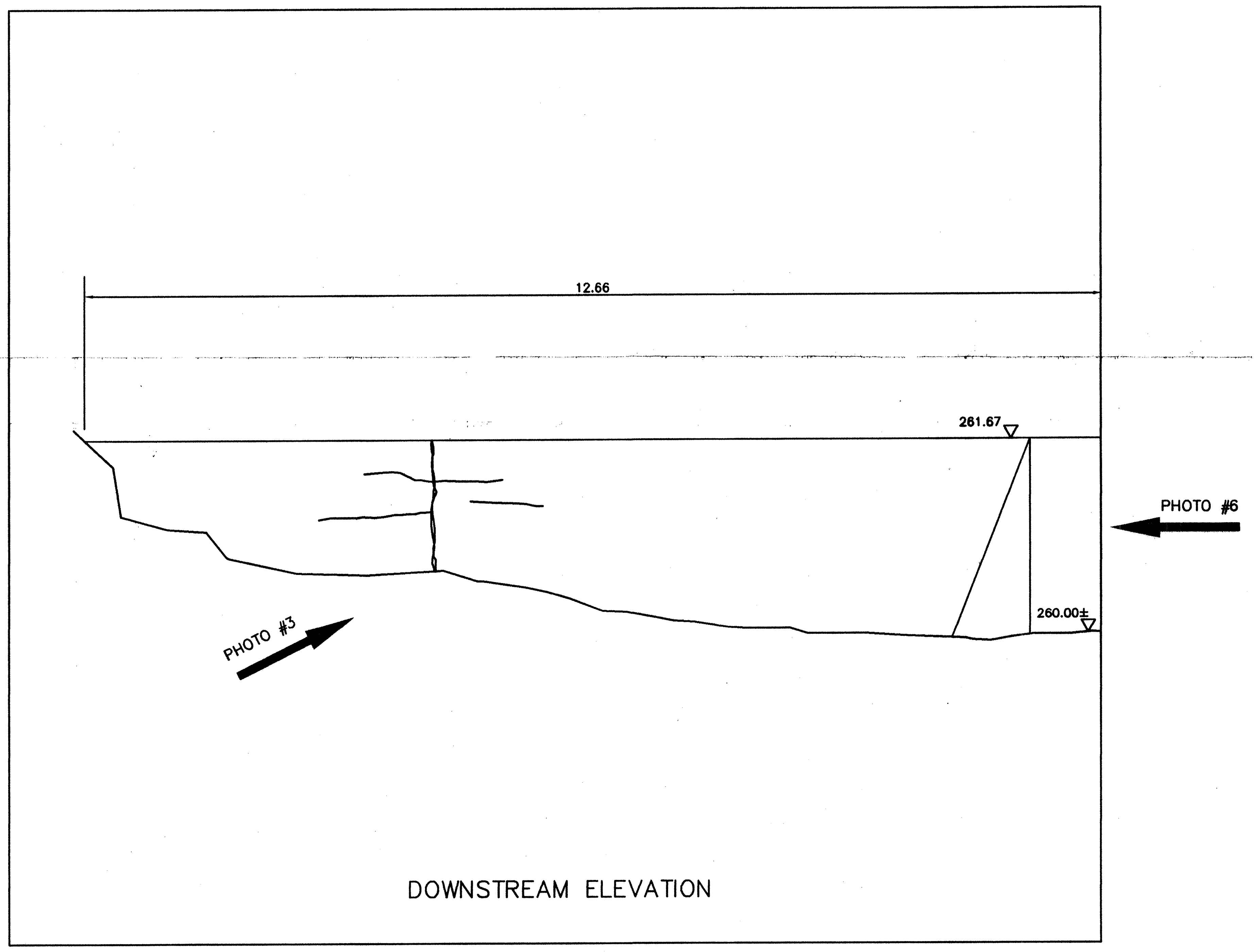
PHOTO #2

SEVERE
STRUCTURAL
CRACK

PHOTO #3

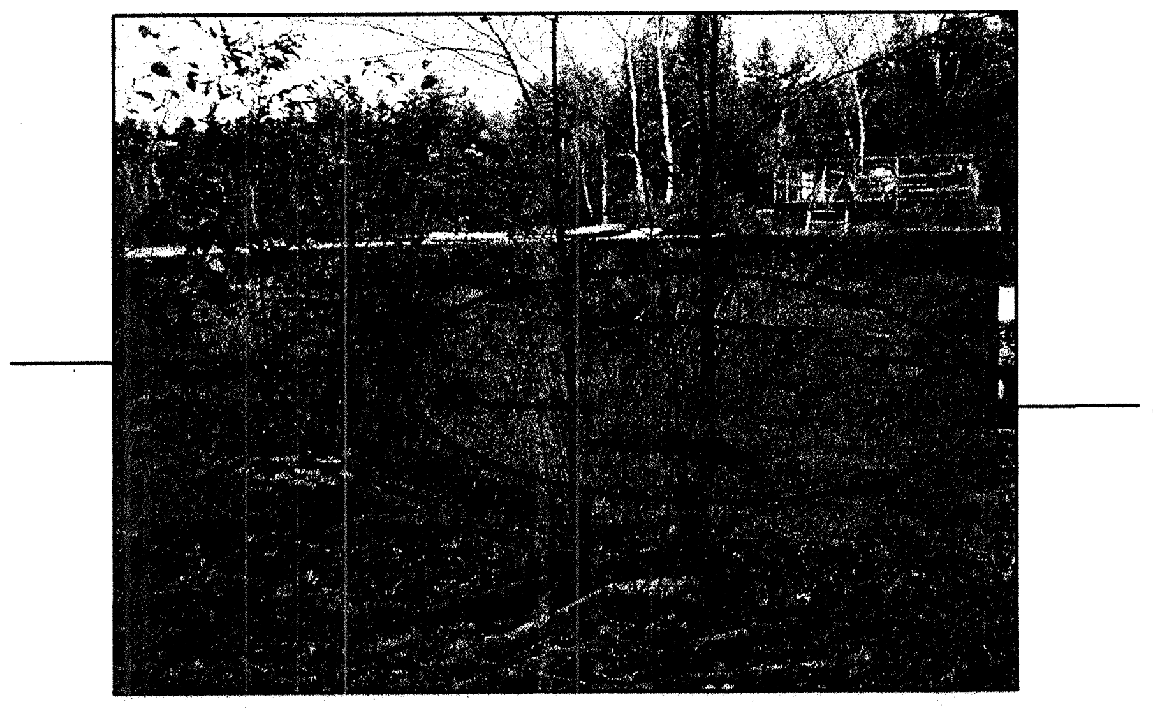


PLAN VIEW



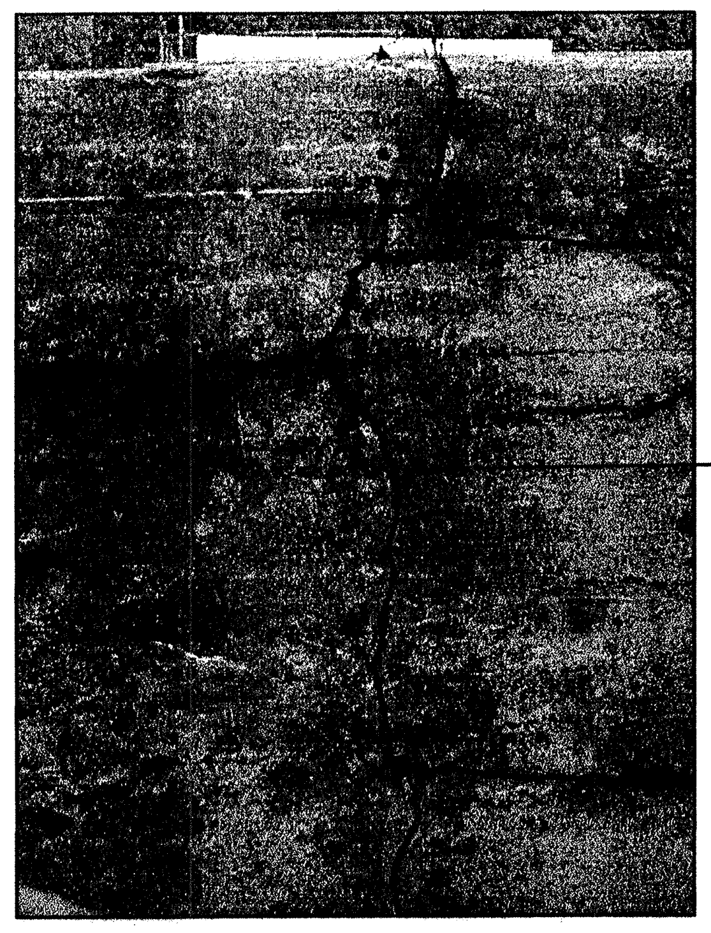
DOWNSTREAM ELEVATION

SEVERE
STRUCTURAL
CRACK



COLD
JOINT
DETERIORATION

PHOTO #4



SEVERE
STRUCTURAL
CRACK

PHOTO #5

PHOTO #6

NO.	REVISIONS	DATE	APP.
B	ISSUED FOR CONSTRUCTION	JULY 23 1999	
A	RELEASED FOR INFORMATION	OCT. 22 1998	

STRUCTURAL CRACKS AND COLD JOINTS

- LAYOUT & CONFIRM ALL CRACKS AND JOINTS TO BE REPAIRED.
- PRESSURE WASH.
- REMOVE LOOSE DELETERIOUS MATERIALS.
- SUFFICIENTLY GROUT CRACKS TO BE ABLE TO PRESSURE INJECT HILTI RM700 EP REPAIR MORTAR.
- CLEAN CONCRETE FACE OF SURPLUS REPAIR MATERIALS.

SITE BENCHMARKS

BRASS CAP, TOP OF MOST NORTHERLY CONCRETE PIER
BETWEEN WOOD DECK AND STEEL CATWALK,
ELV. 262.00 m GSC.

EGA **ECOS GARATECH**
CONSULTING ENGINEERS
BRAMPTON (905) 458-4110 FAX: (905) 458-1479

OWNER/CLIENT:
MISSISSIPPI VALLEY CONSERVATION

PROJECT:
KASHWAKAMAK LAKE DAM

TITLE:
SOUTH WINGWALL

DESIGNED: N.A.	DRAWN: P.L.	REV. NO.: B
CHECKED: M.D.G.	DATE: OCT., 1998	DWG. NO. 1434-19-03
SCALE: 1:50	JOB: 1434-19	

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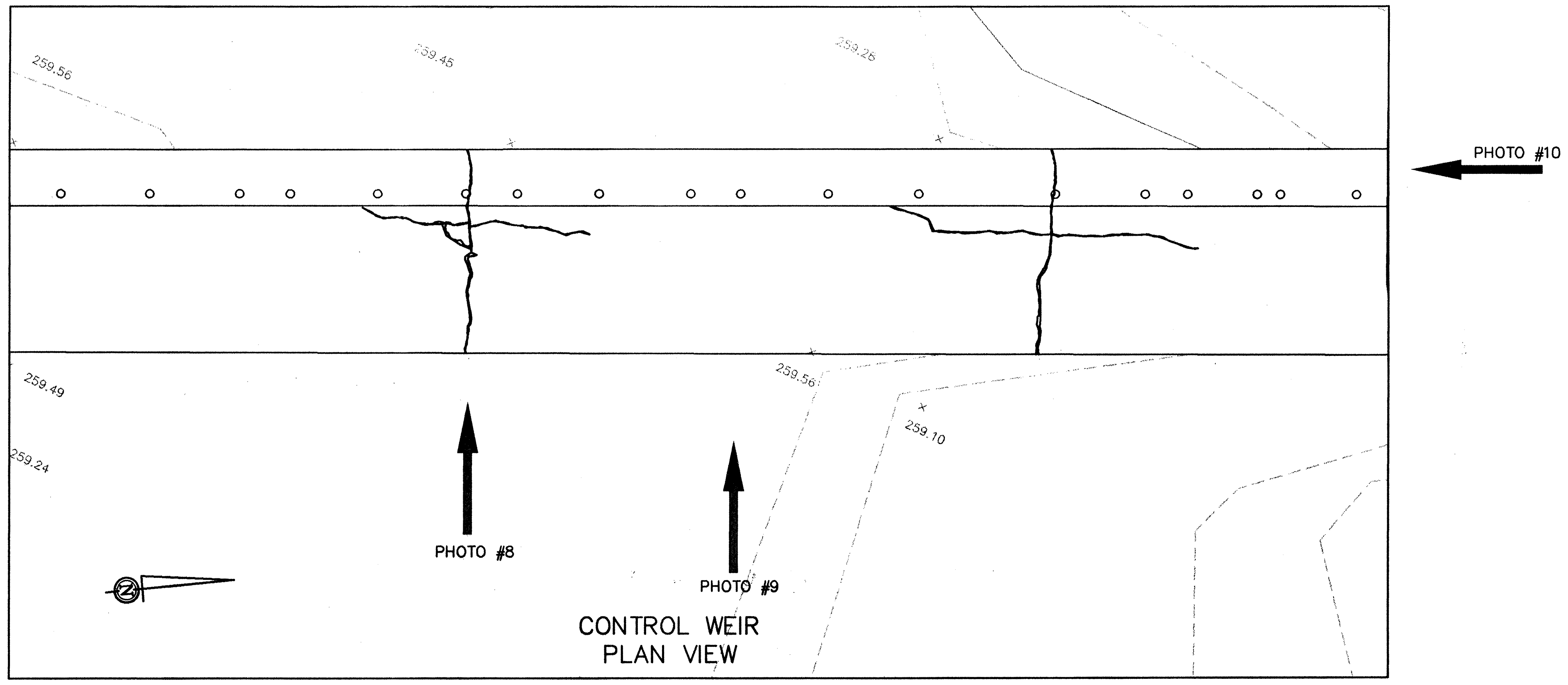
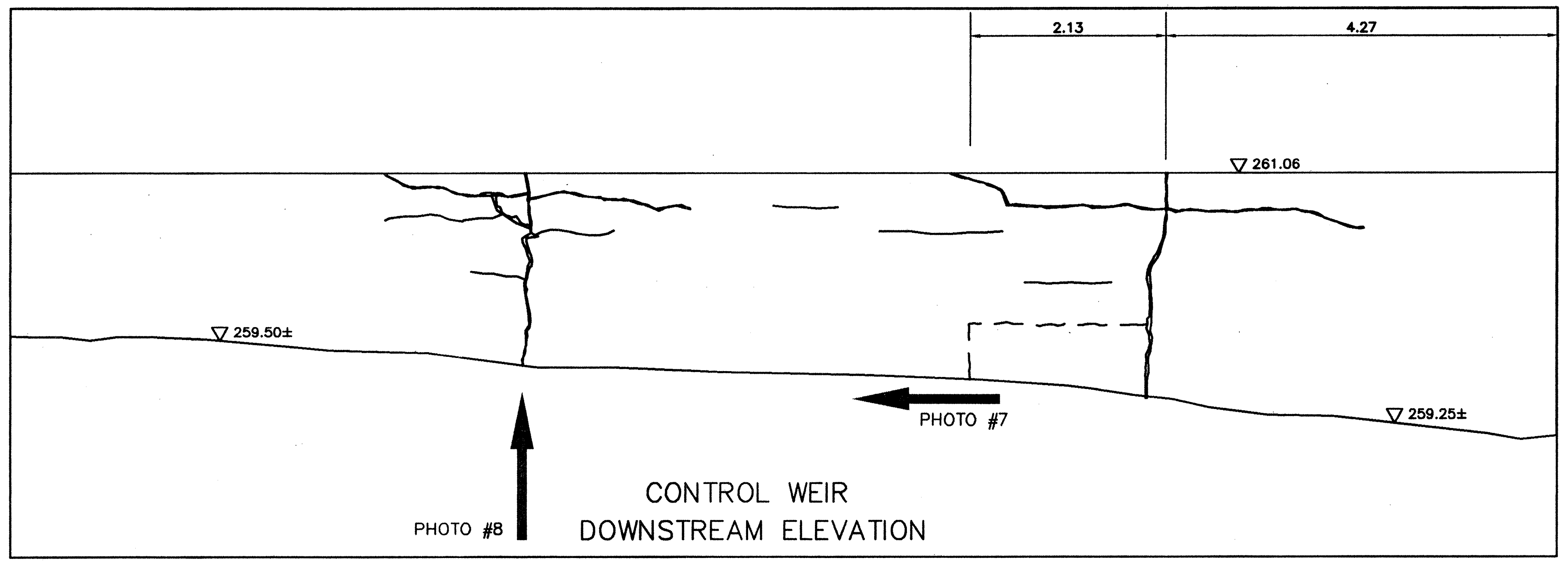
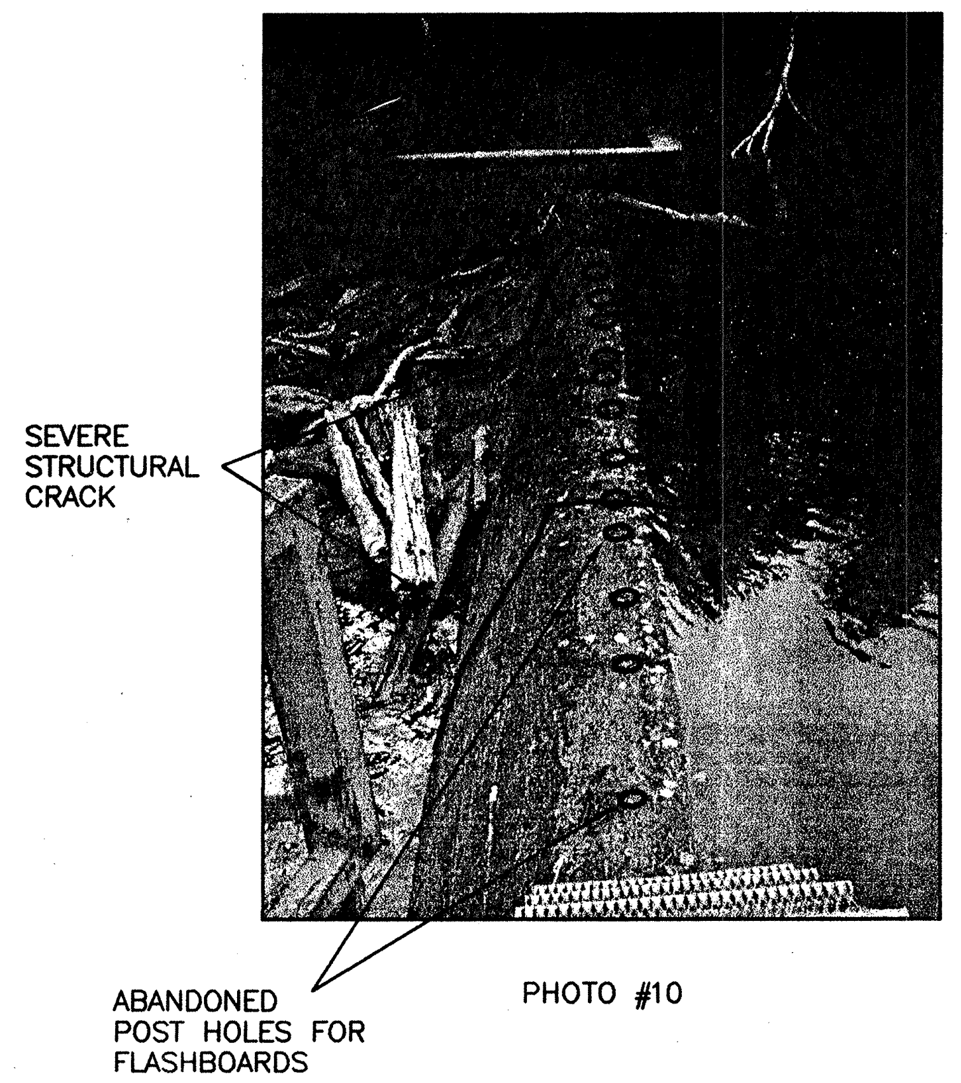


CONTROL WEIR
PLAN VIEW

PHOTO #7



CONTROL WEIR
DOWNSTREAM ELEVATION



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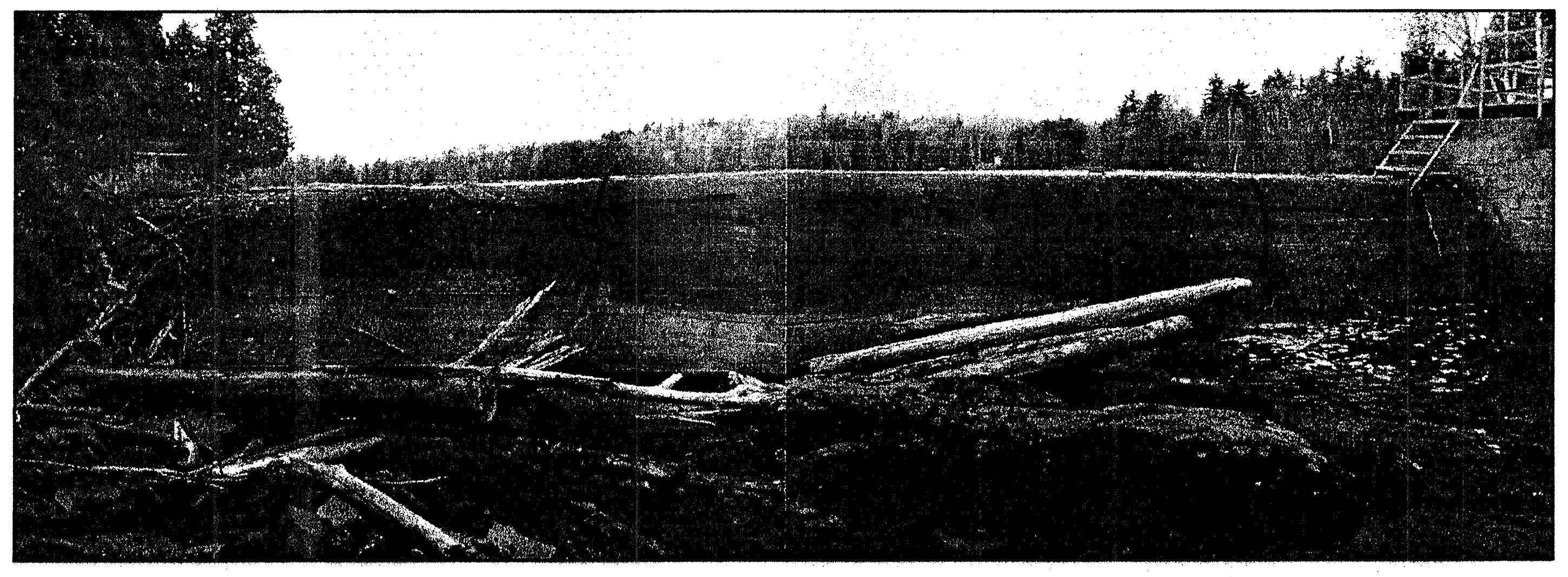

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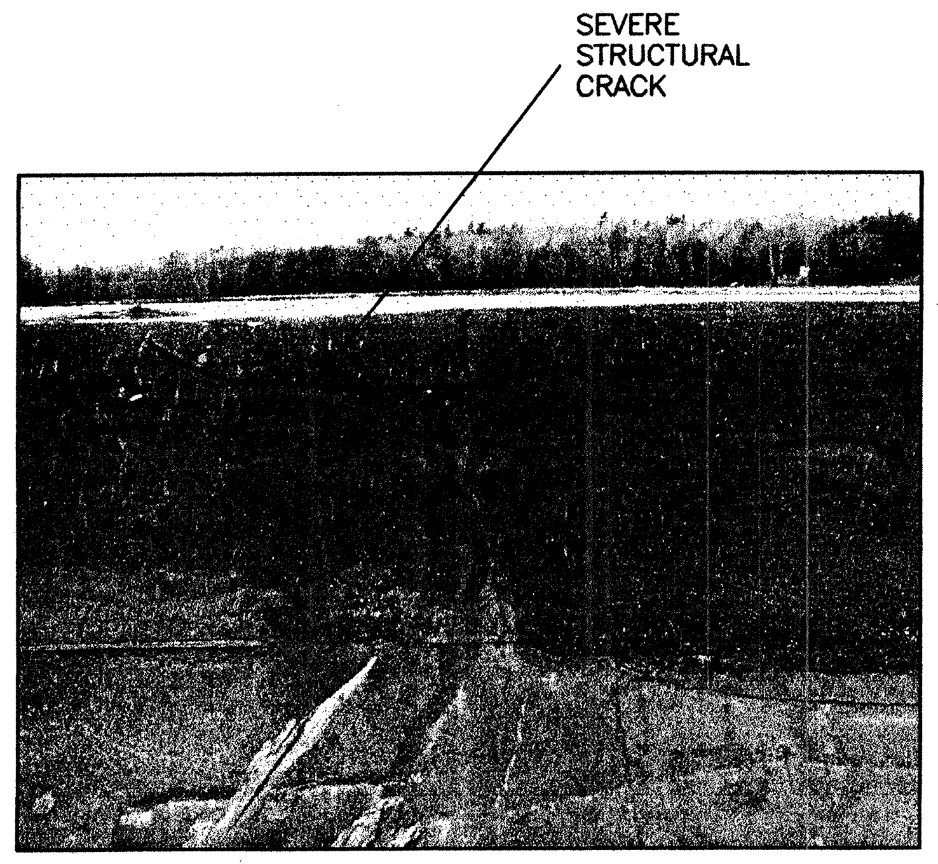

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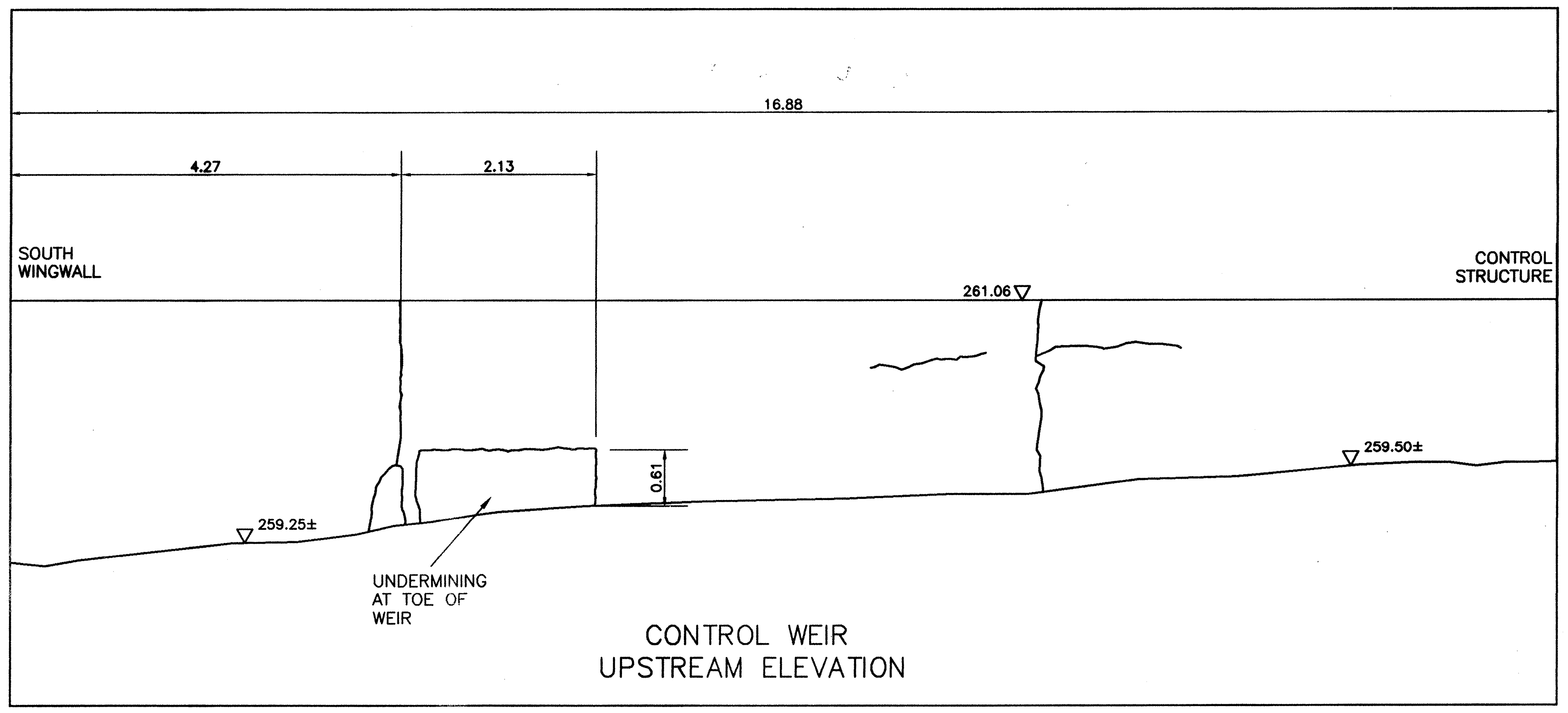

CONTROL WEIR
UPSTREAM ELEVATION

NO.	REVISIONS	DATE	APP.
B	ISSUED FOR CONSTRUCTION	JULY 26 1999	
A	RELEASED FOR INFORMATION	OCT. 22 1998	

STRUCTURAL CRACKS AND POST HOLES

- PRESSURE WASH.
- REMOVE LOOSE DELETERIOUS MATERIALS.
- SUFFICIENTLY GROUT CRACKS TO BE ABLE TO PRESSURE INJECT HILTI RM700 EP REPAIR MORTAR.
- CLEAN CONCRETE FACE OF SURPLUS REPAIR MATERIALS.

TOE OF WEIR & CONCRETE SPALLING

- REMOVE LOOSE DELETERIOUS MATERIALS & PREPARE SURFACE.
- PROTECT & FORM WORKING AREA.
- PLACE TREMIE CONCRETE (30 mPa) TO SUFFICIENTLY TREAT UNDERMINING UPSTREAM TOE OF WEIR & AREAS OF SPALLING OF THE CONCRETE FACE.
- REMOVE FORMS & SCRAPE SURFACE SMOOTH OF ROUGH PROTRUSIONS.

SITE BENCHMARKS

BRASS CAP, TOP OF MOST NORTHERLY CONCRETE PIER BETWEEN WOOD DECK AND STEEL CATWALK, ELV. 262.00 m GSC.

EGA
ECOS GARATECH
CONSULTING ENGINEERS
BRAMPTON (905) 458-4110 FAX: (905) 458-1479

OWNER/CLIENT:
MISSISSIPPI VALLEY CONSERVATION

PROJECT:
KASHWAKAMAK LAKE DAM

TITLE:
CONTROL WEIR

DESIGNED: N.A.	DRAWN: P.L.	REV. NO.: B
CHECKED: M.D.G.	DATE: OCT., 1998	DWG. NO.
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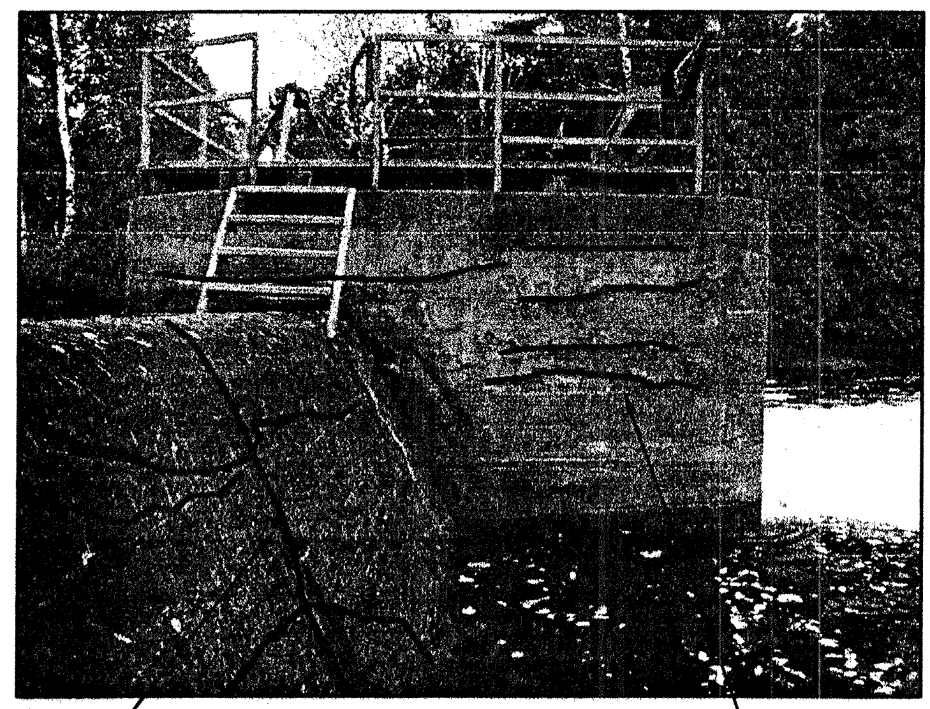


PHOTO #11

SEVERE
STRUCTURAL
CRACKS

COLD JOINT
DETERIORATION

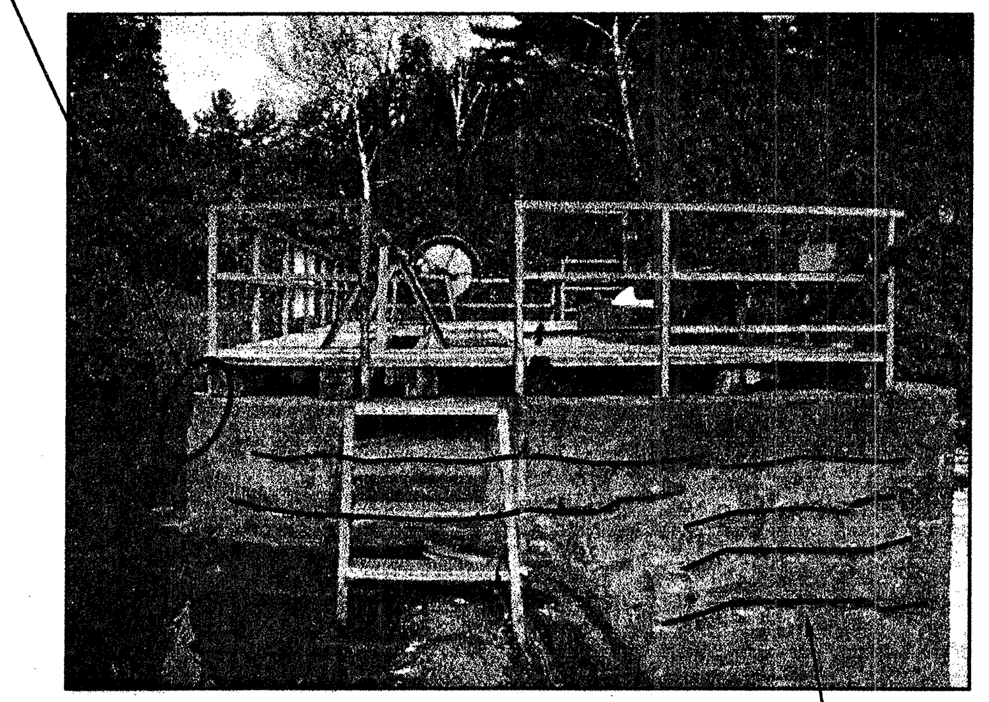

SEVERE
CONCRETE
SPALLING

PHOTO #12

COLD JOINT
DETERIORATION

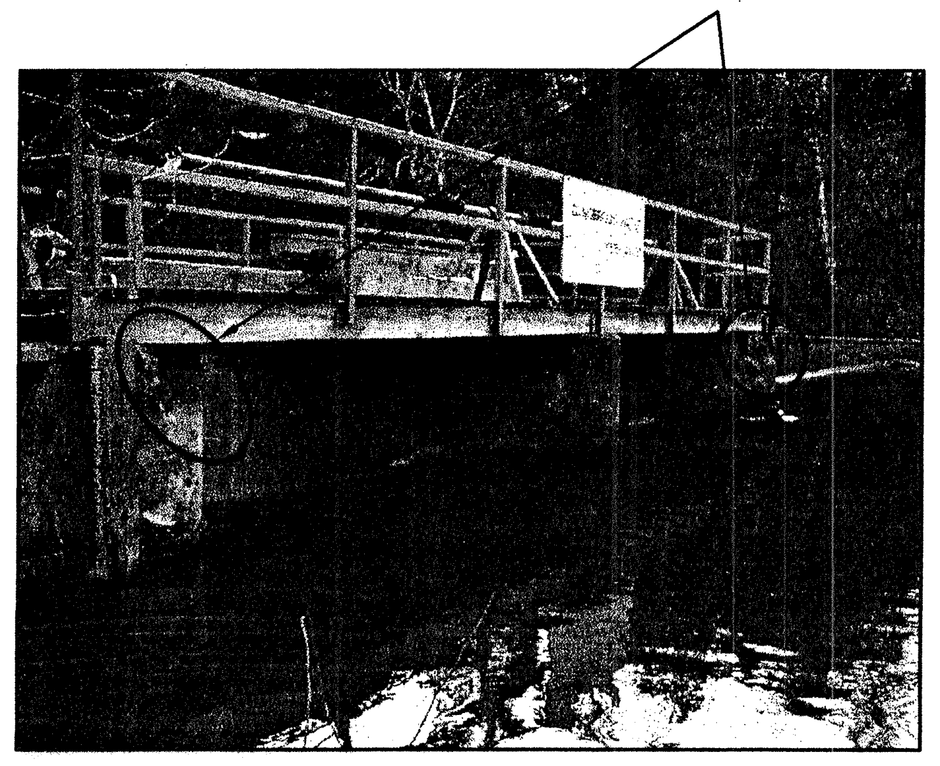
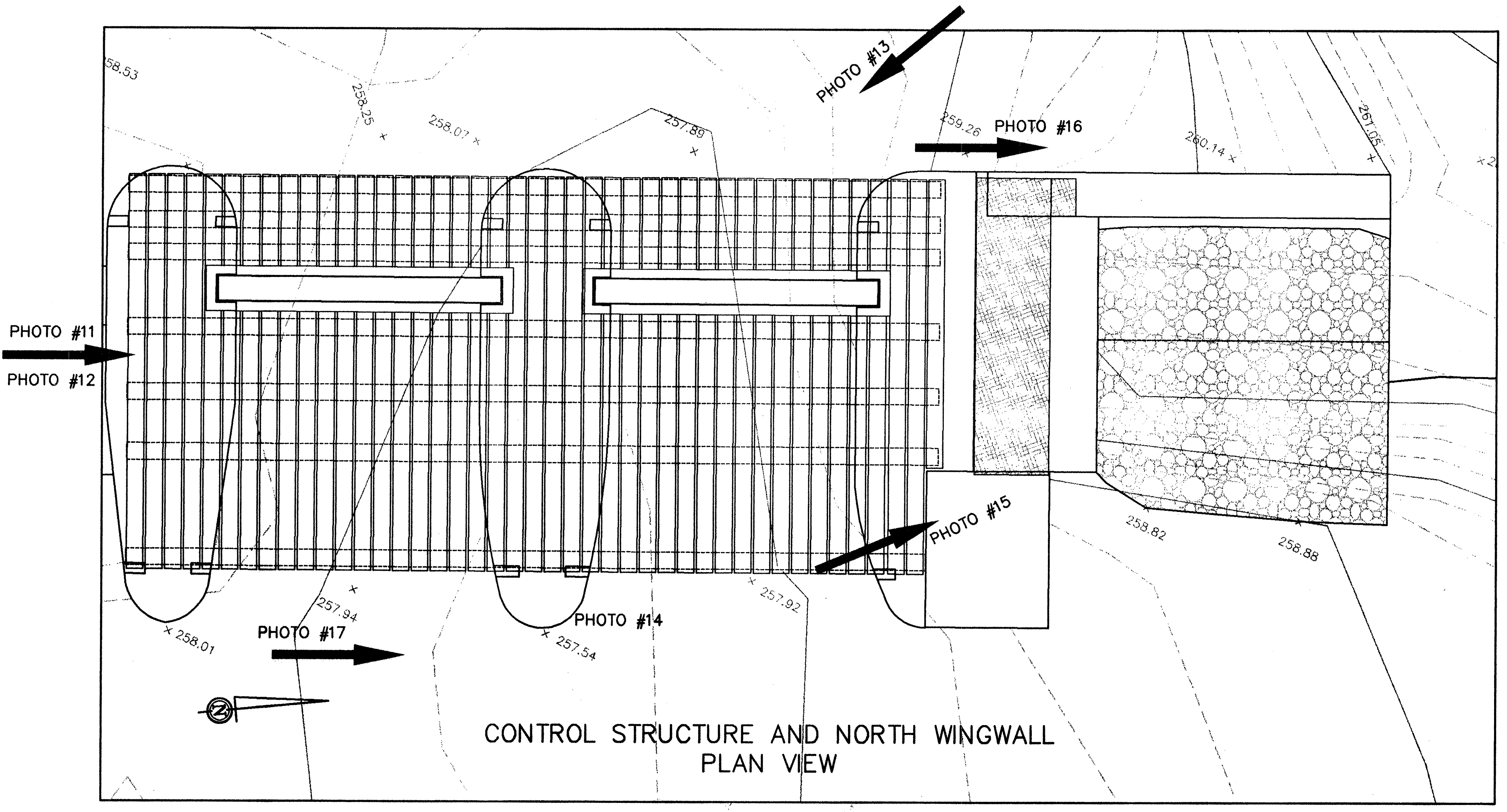
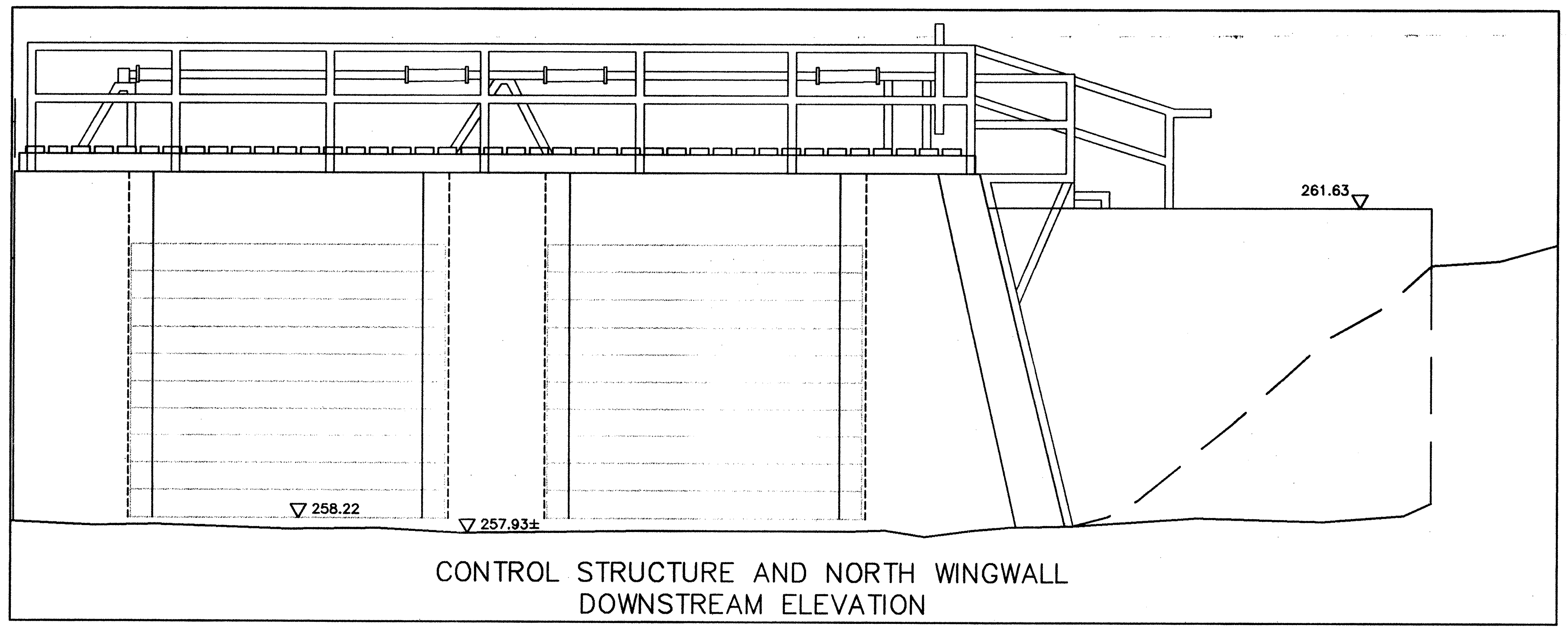

SEVERE
CONCRETE
SPALLING

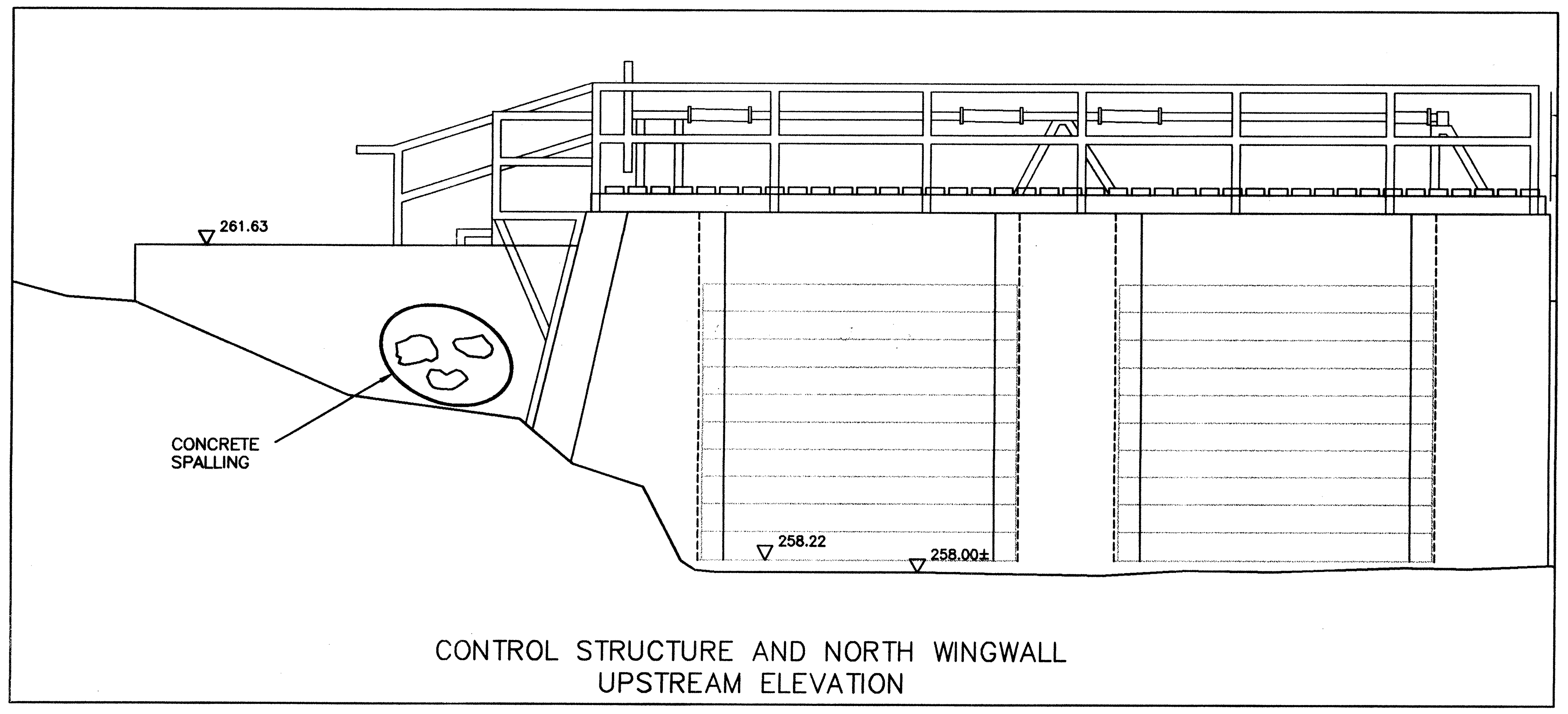
PHOTO #13



CONTROL STRUCTURE AND NORTH WINGWALL
PLAN VIEW



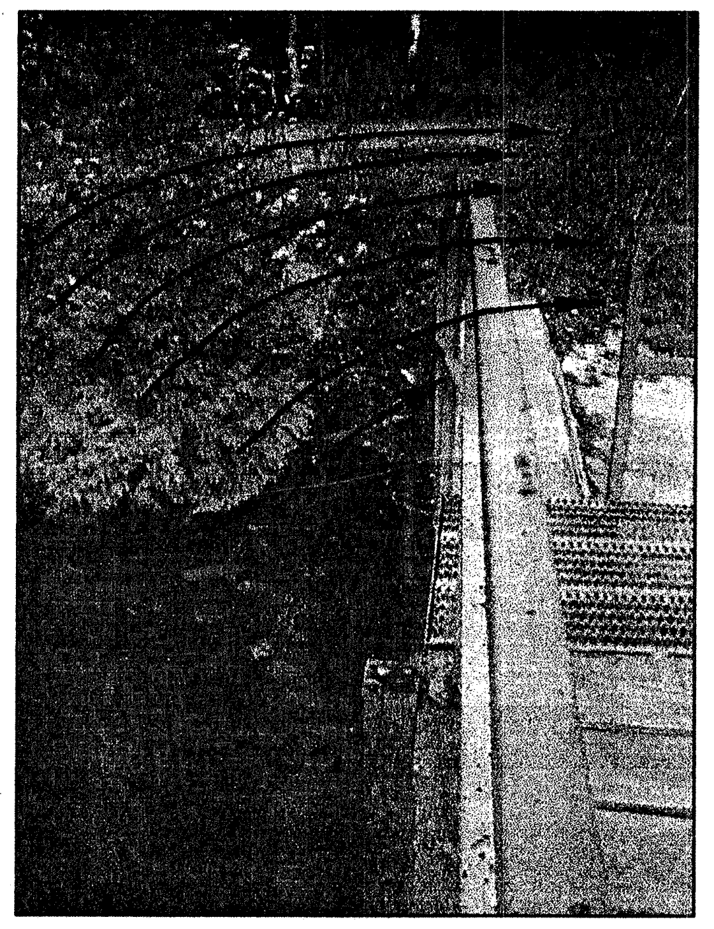
CONTROL STRUCTURE AND NORTH WINGWALL
DOWNSTREAM ELEVATION



CONTROL STRUCTURE AND NORTH WINGWALL
UPSTREAM ELEVATION

PHOTO #14


PHOTO #15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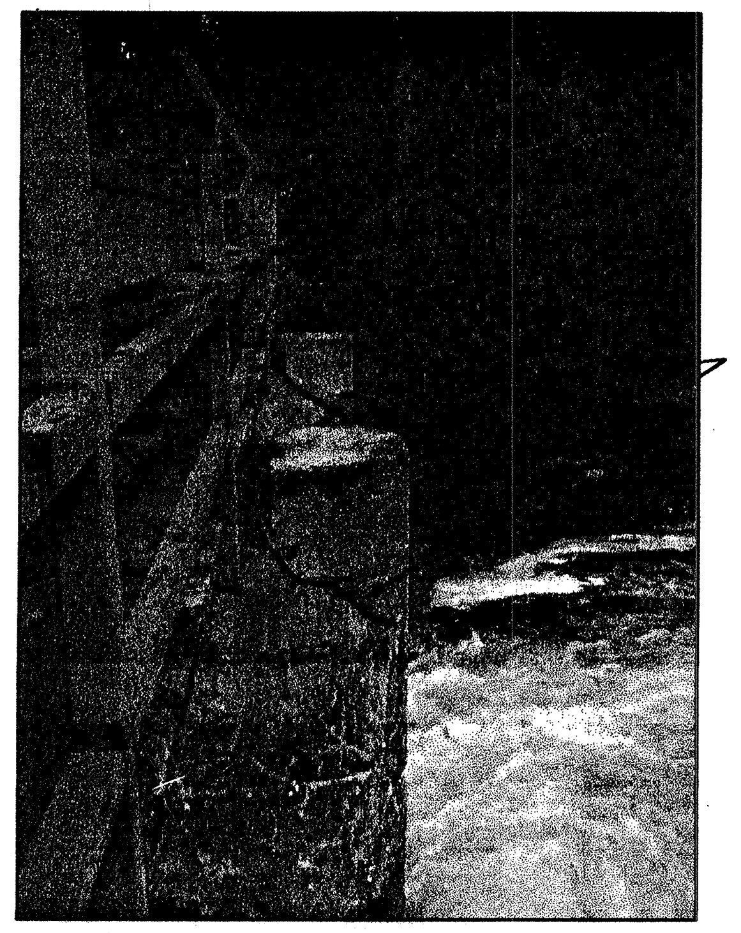
SEVERE
STRUCTURAL
CRACKS

PHOTO #17

NO.	REVISIONS	DATE	APP.
B	ISSUED FOR CONSTRUCTION	JULY 26 1999	
A	RELEASED FOR INFORMATION	OCT. 22 1998	

STRUCTURAL CRACKS, SPALLING AND COLD JOINTS

- PRESSURE WASH.
- REMOVE LOOSE DELETERIOUS MATERIALS & PREPARE SURFACE.
- SUFFICIENTLY GROUT CRACKS TO BE ABLE TO PRESSURE INJECT HILTI RM700 EP REPAIR MORTAR.
- CLEAN CONCRETE FACE OF SURPLUS REPAIR MATERIALS.
- PROTECT & FORM WORKING AREA FOR REPAIR OF CONCRETE FACE IN SPALLING AREAS.
- REMOVE FORMS; SCRAPE SURFACE SMOOTH OF ROUGH PROTRUSIONS.

SITE BENCHMARKS

BRASS CAP, TOP OF MOST NORTHERLY CONCRETE PIER
BETWEEN WOOD DECK AND STEEL CATWALK,
ELV. 262.00 m GSC.

EGA **ECOS GARATECH**
CONSULTING ENGINEERS
BRAMPTON (905) 458–4110 FAX: (905) 458–1479

OWNER/CLIENT:
MISSISSIPPI VALLEY CONSERVATION

PROJECT:
KASHWAKAMAK LAKE DAM

TITLE:
CONTROL STRUCTURE

DESIGNED: N.A.	DRAWN: P.L.	REV. NO.: B
CHECKED: M.D.G.	DATE: OCT., 1998	DWG. NO. 1434–19–05
SCALE: 1:50	JOB: 1434–19	

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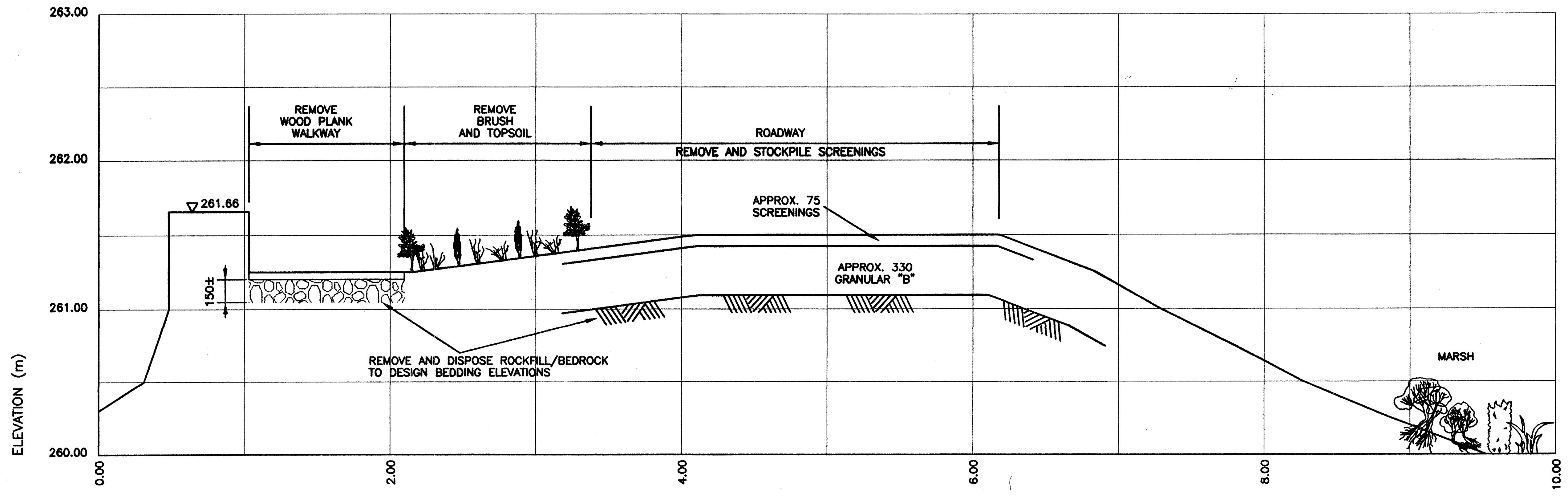
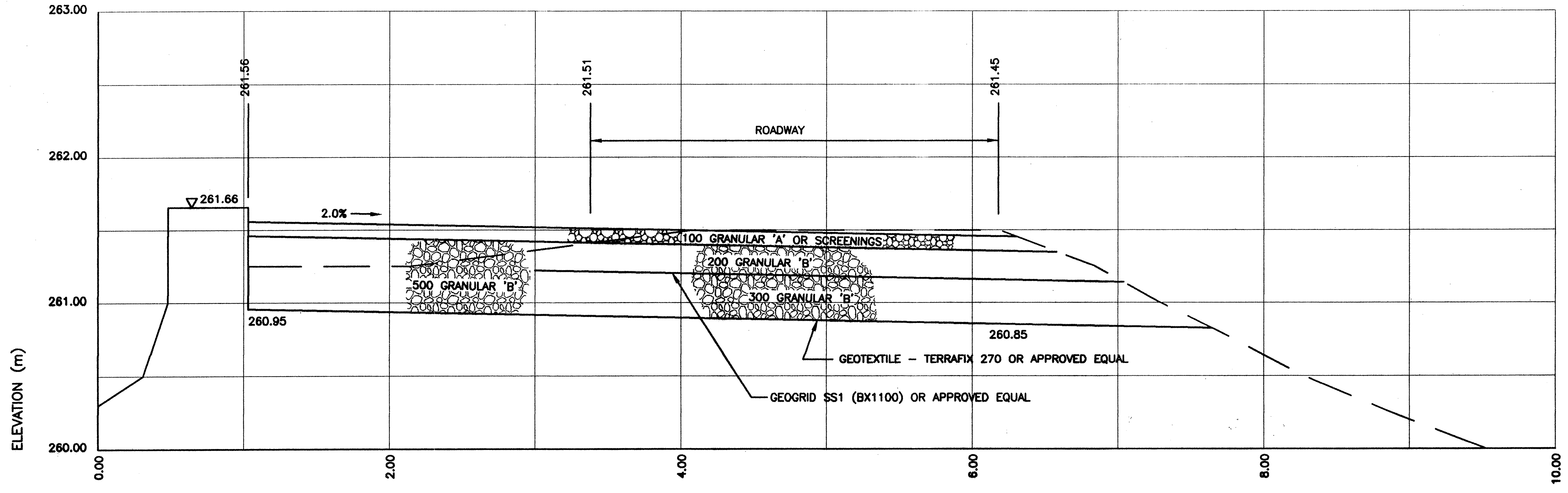


SECTION A–A – EXISTING

SCALE – 1:25

* ROAD WORKS NOT PART OF THIS CONTRACT



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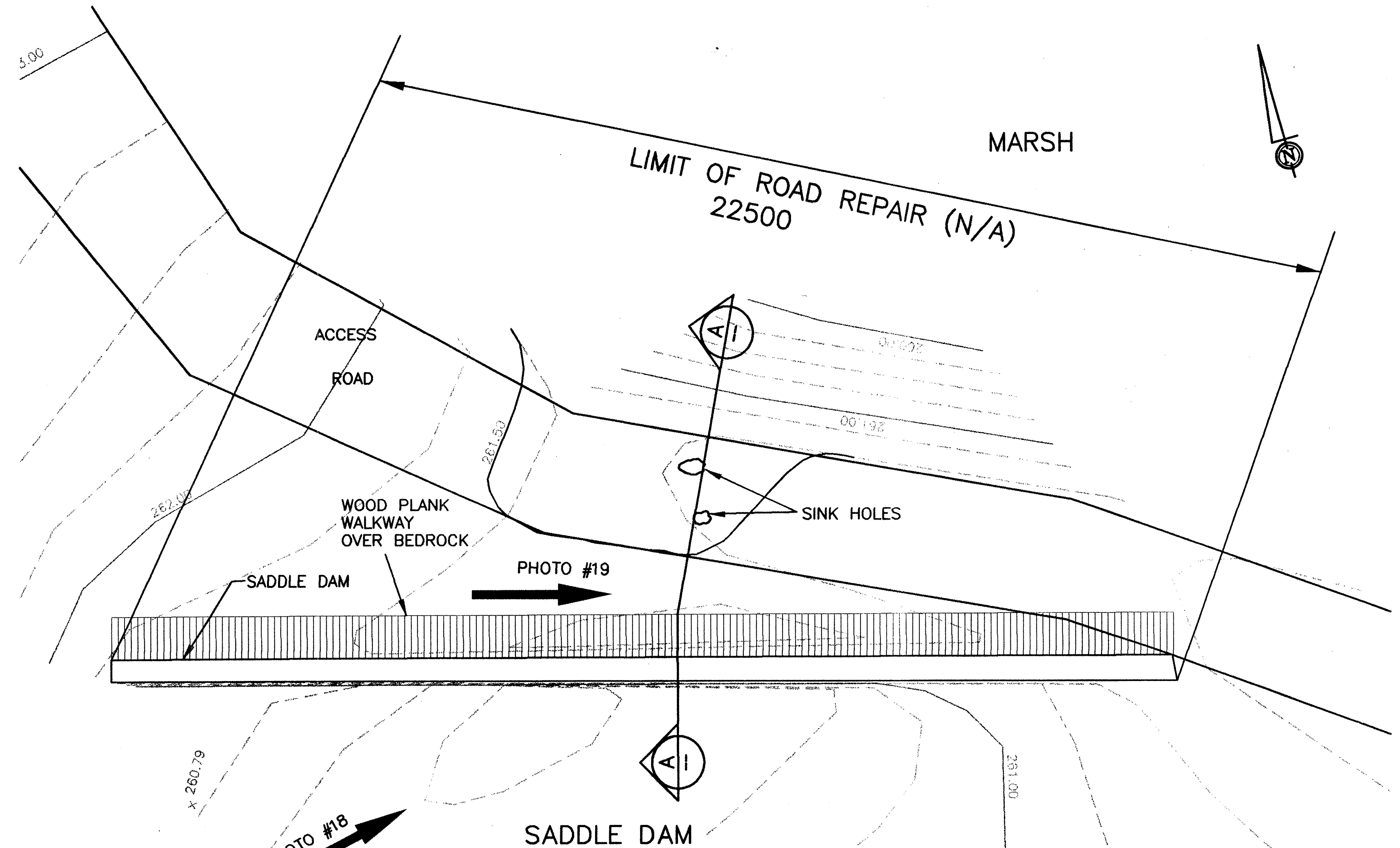
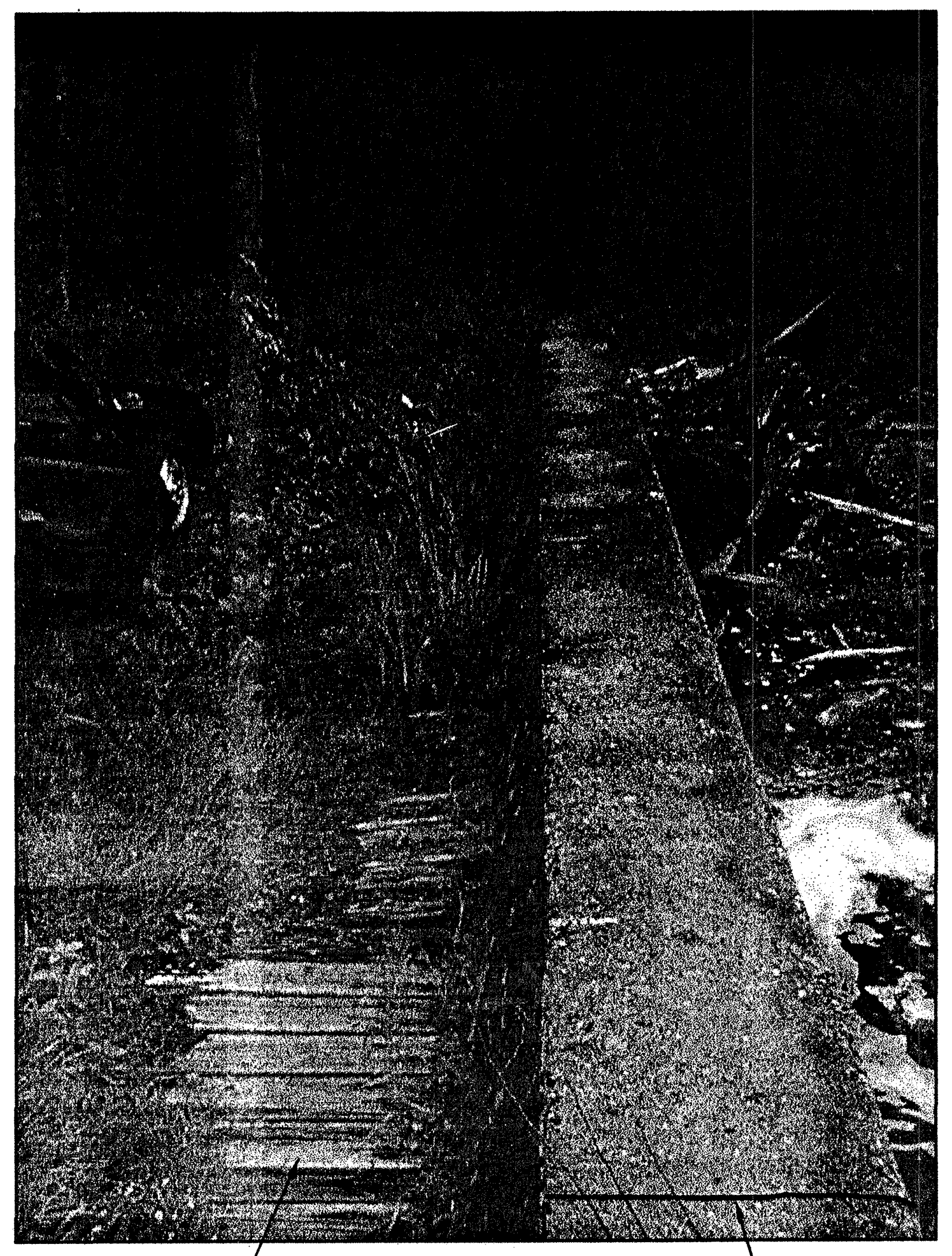


SADDLE DAM PLAN VIEW

SEVERE STRUCTURAL CRACK

PHOTO #18



WOOD PLANK WALKWAY OVER ROCK FILL/BEDROCK

PHOTO #19

SEVERE STRUCTURAL CRACK

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STRUCTURAL CRACKS

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- REMOVE LOOSE DELETERIOUS MATERIALS.
- SUFFICIENTLY GROUT CRACKS TO BE ABLE TO PRESSURE INJECT HILTI RM700 EP REPAIR MORTAR.
- CLEAN CONCRETE FACE OF SURPLUS REPAIR MATERIALS.

SITE BENCHMARKS

BRASS CAP, TOP OF MOST NORTHERLY CONCRETE PIER BETWEEN WOOD DECK AND STEEL CATWALK,
ELV. 262.00 m GSC.

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SCALE: 1:100 U/N	JOB: 1434–19	

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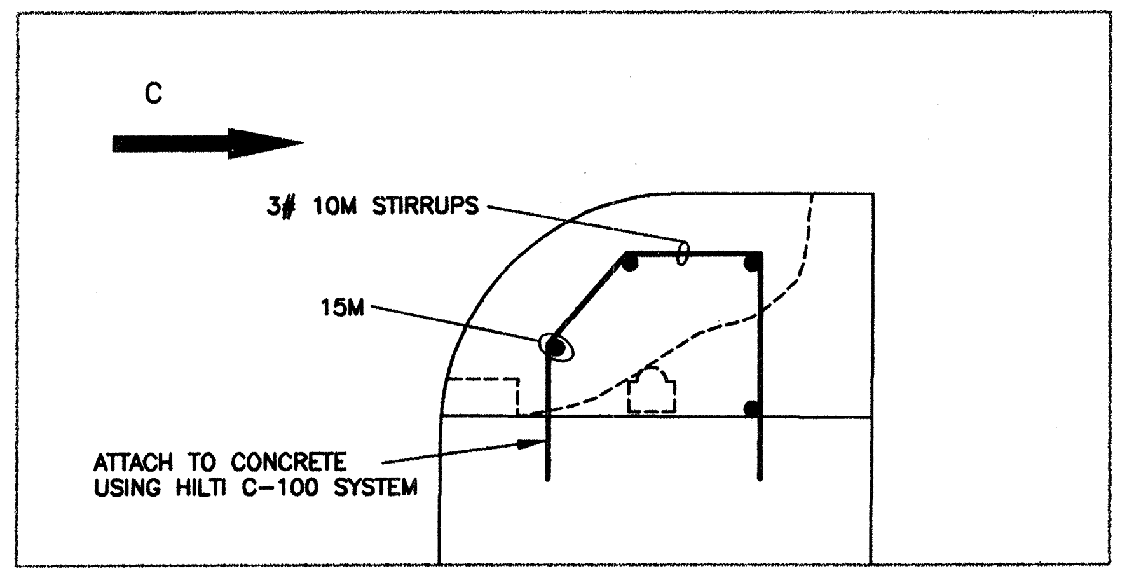
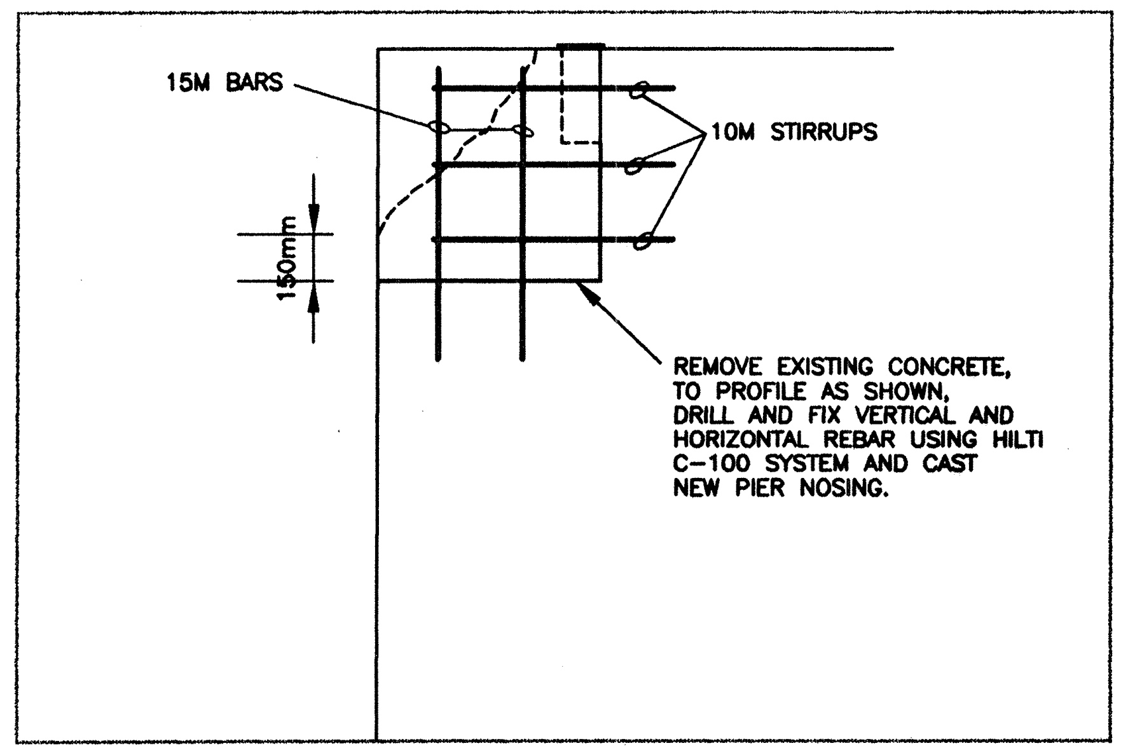
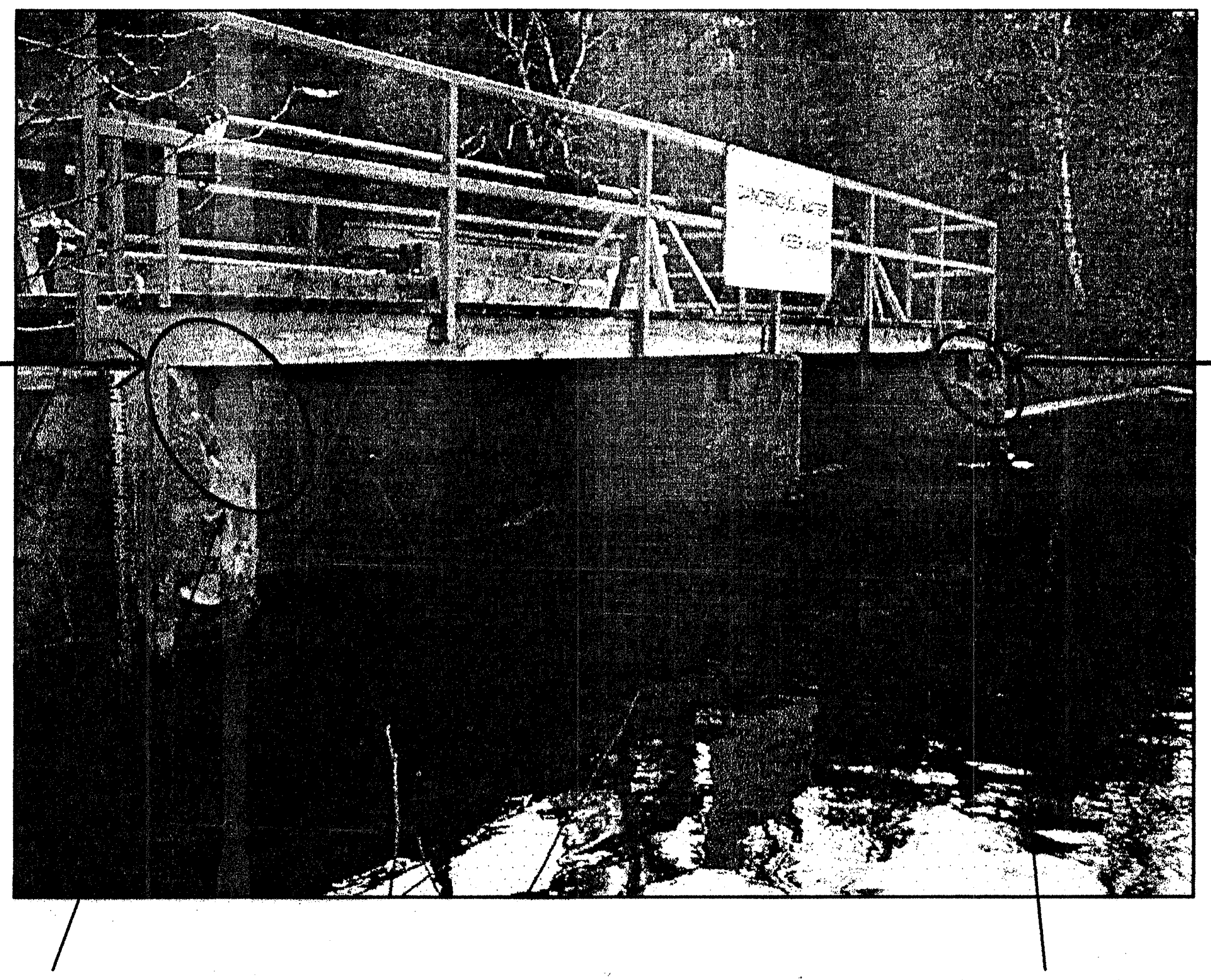


PLAN VIEW



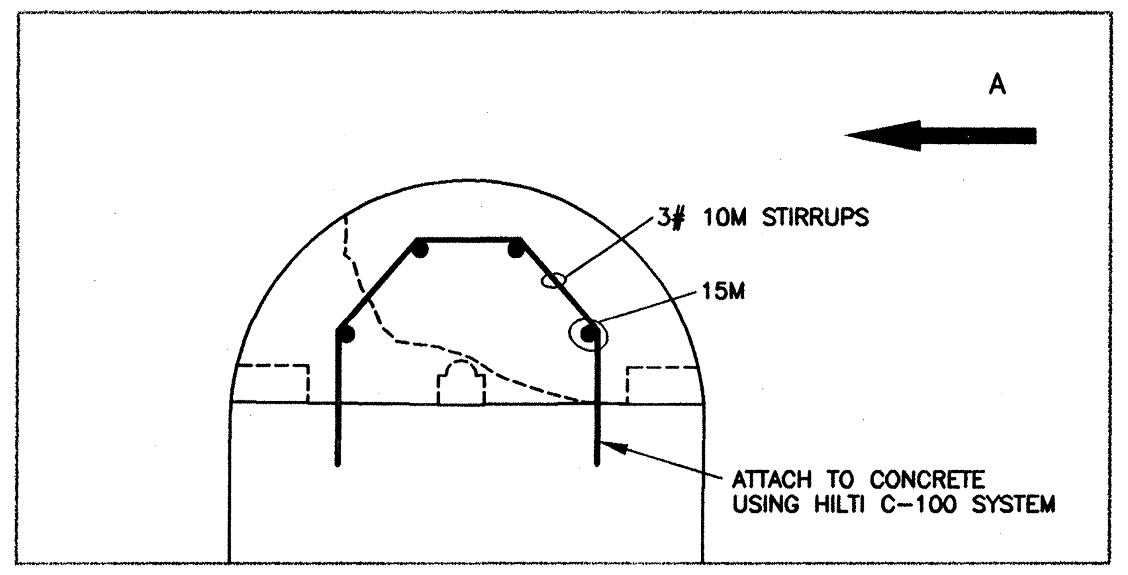
ELEVATION C

NORTH PIER – UPSTREAM SIDE

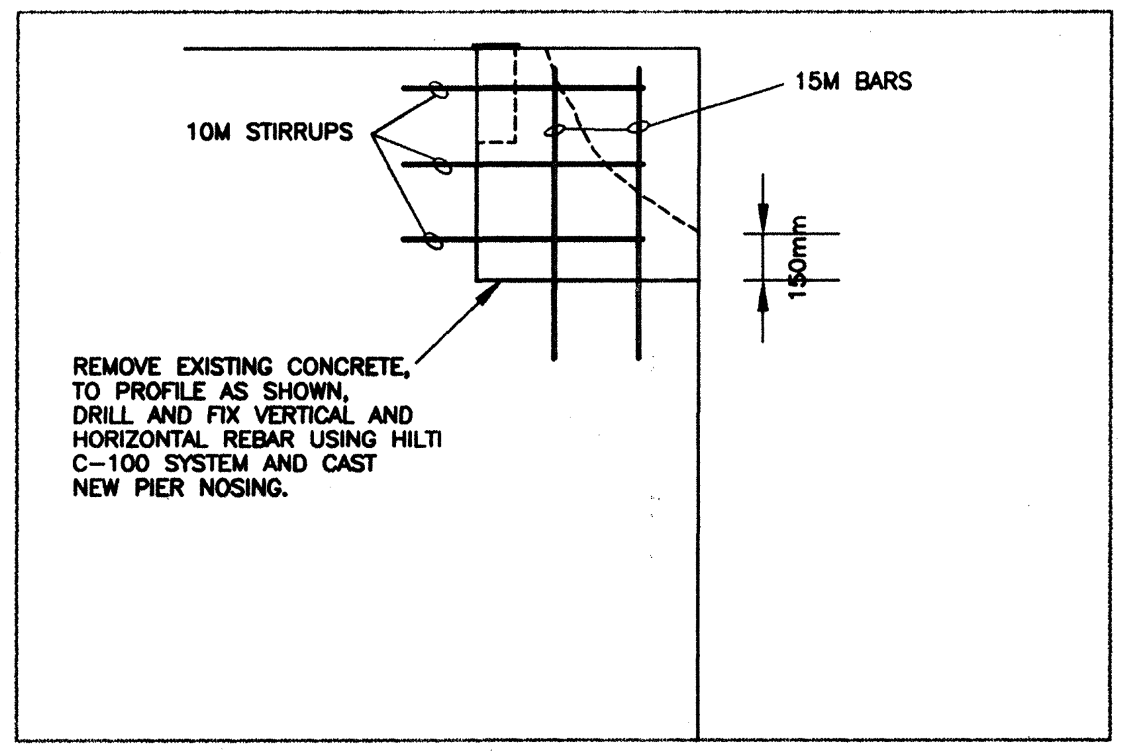


UPSTREAM NORTH PIER
SEVERE CONCRETE SPALLING

UPSTREAM SOUTH PIER
SEVERE CONCRETE SPALLING

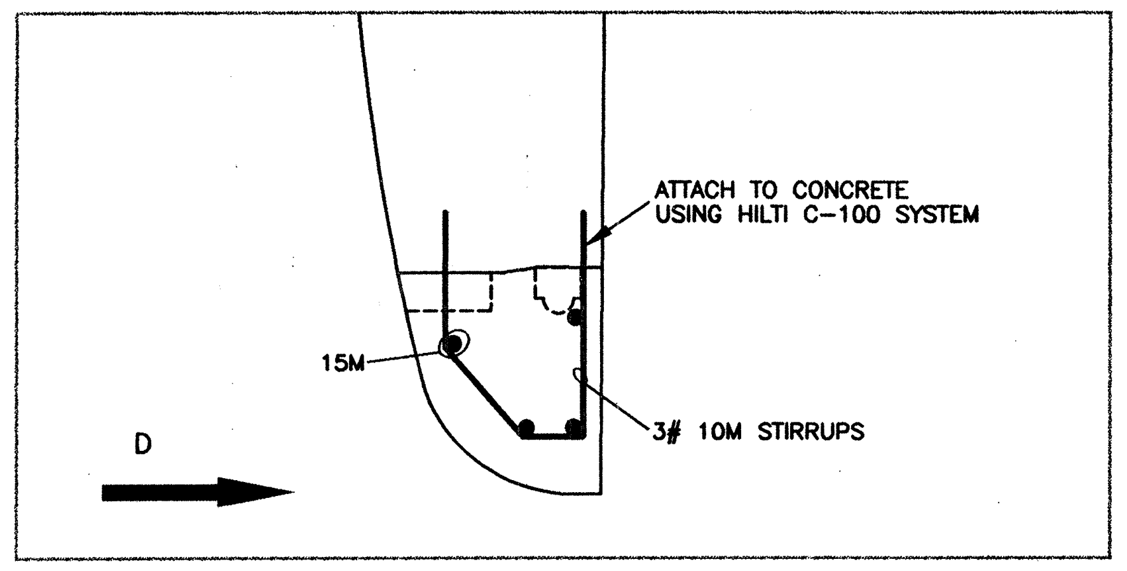


PLAN VIEW

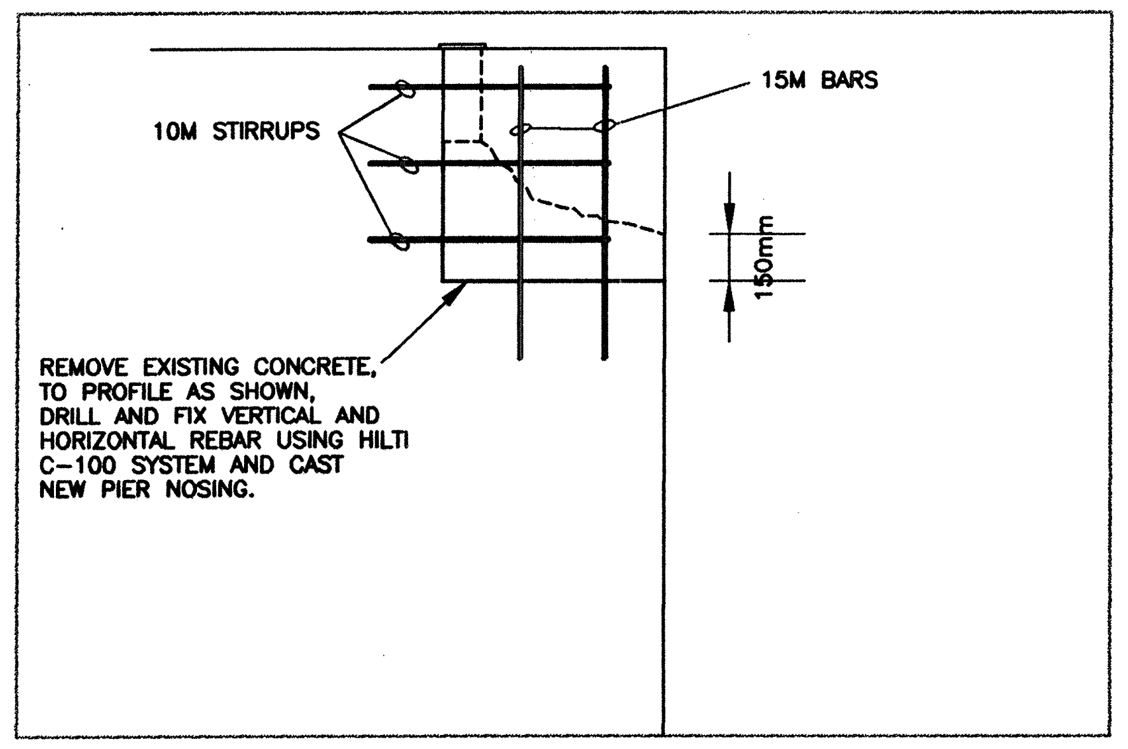


ELEVATION A

SOUTH PIER – UPSTREAM SIDE

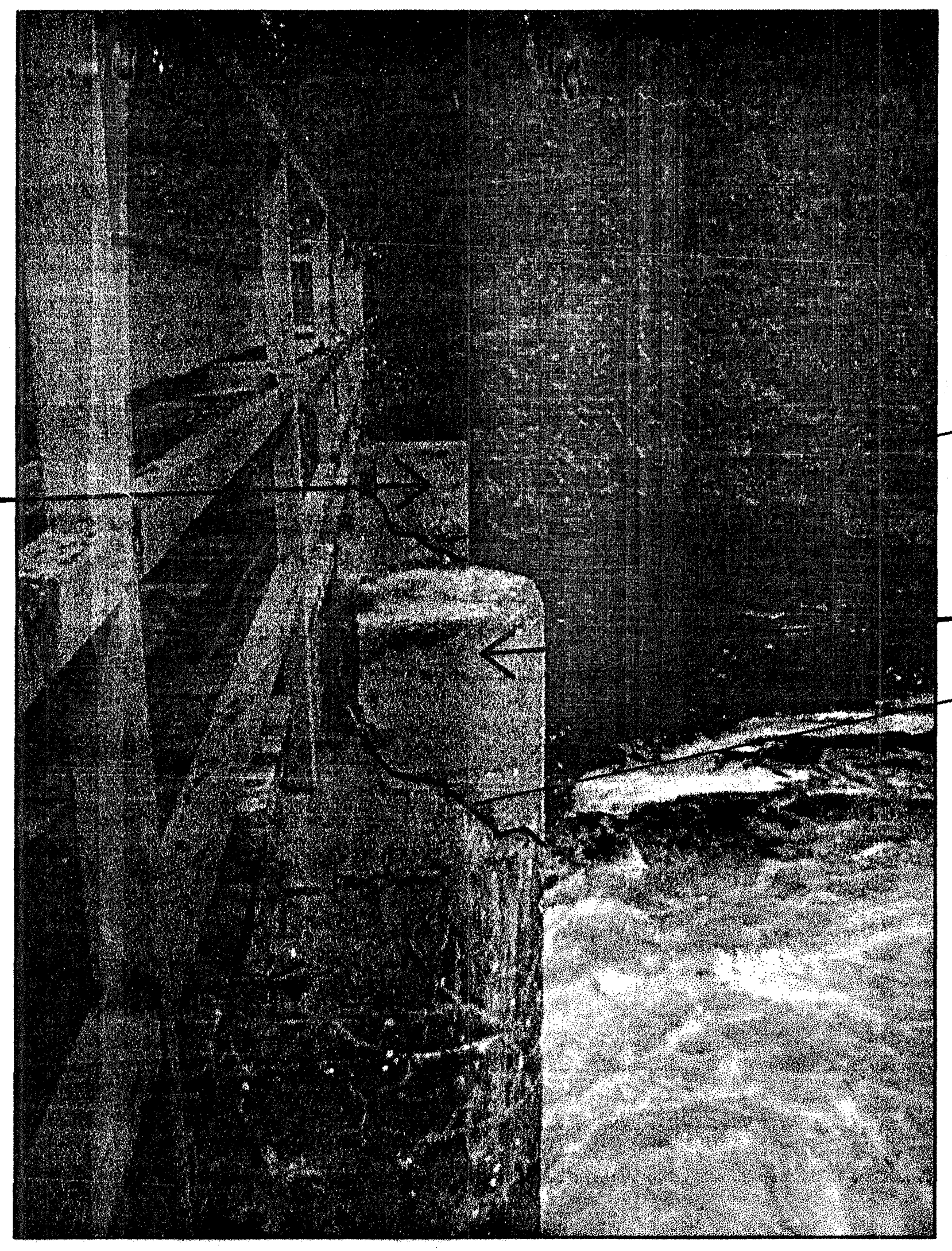


PLAN VIEW



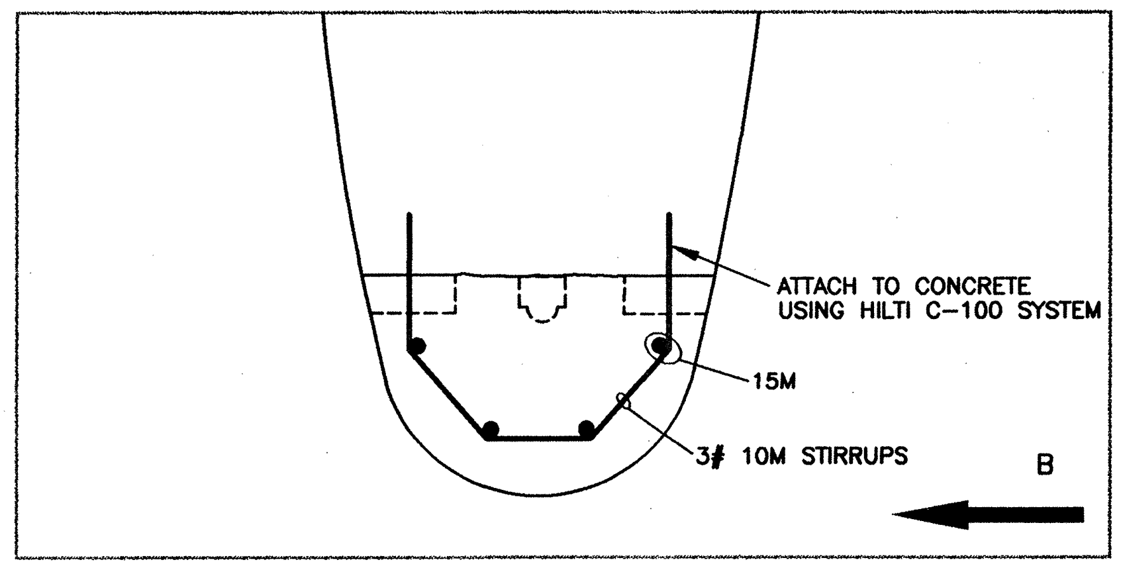
ELEVATION D

NORTH PIER – DOWNSTREAM SIDE

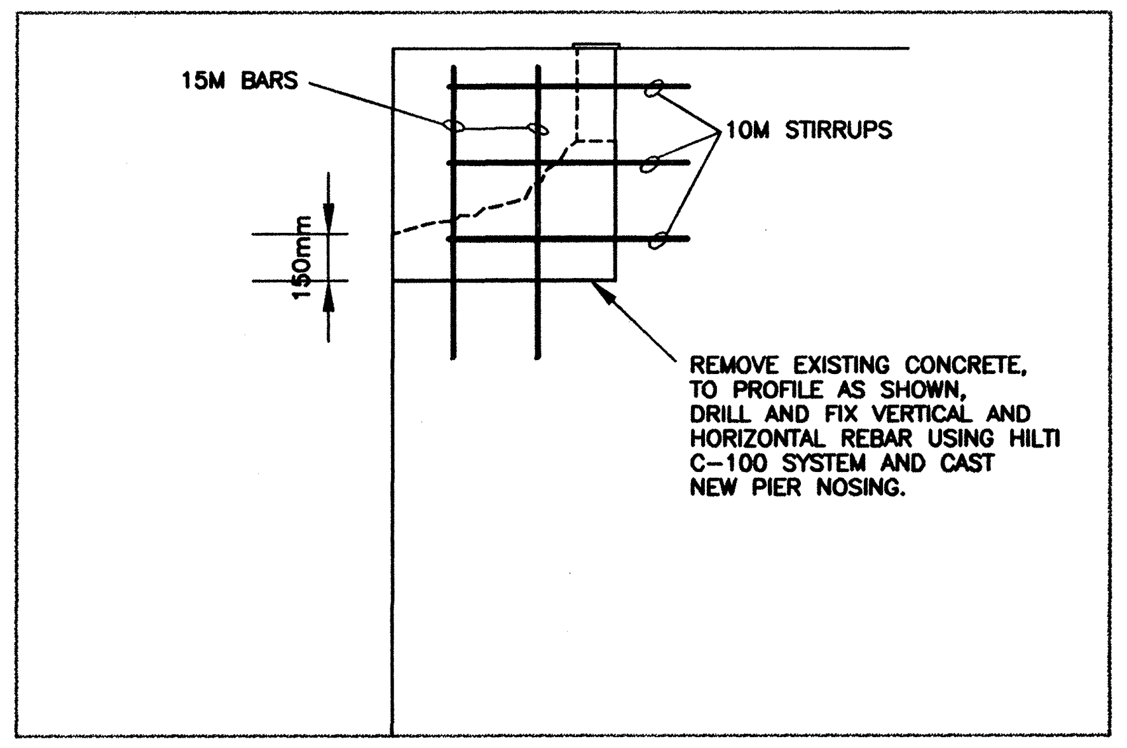


DOWNSTREAM NORTH PIER
SEVERE STRUCTURAL CRACK

DOWNSTREAM CENTRE PIER
SEVERE STRUCTURAL CRACK



PLAN VIEW



ELEVATION B

CENTRE PIER – DOWNSTREAM SIDE

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GENERAL STRUCTURAL NOTES:

- CONCRETE TO BE CSA STANDARD CAN3–A23.1 WITH MIN. STRENGTH OF 30 MPa IN 28 DAYS. EXPOSURE CLASS B.
- ALL PIER REINFORCING TO BE VERIFIED BY DESIGN ENGINEER OR HIS REPRESENTATIVE PRIOR TO POURING CONCRETE.
- ALL FASTENERS TO BE CADMIUM–PLATED CARBON STEEL.
- CONCRETE COVER TO REINFORCING STEEL:
  - 75 mm UNLESS NOTED
- MAINTAIN/PROVIDE MISCELLANEOUS METAL FOR DECK ANCHOR/SUPPORT AT COLUMNS AS NECESSARY.

SITE BENCHMARKS

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SCALE: 1:25	JOB: 1434–19	1434–19–07

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PROPOSED KASHWAKAMAK LAKE DAM REPLACEMENT – NORTH FRONTENAC TWN. ON

APPENDIX H
SITE PHOTOS

McINTOSH PERRY

Figure 3: Study area landscape overview, approach access to dam site.

Figure 4: Study area landscape overview, Mississippi River.

Figure 5: Kashwakamak Lake Dam, looking northwest.

Figure 6: Mississippi River, looking west from Kashwakamak Lake Dam site.

Figure 7: Kashwakamak Lake Dam, looking west.

Figure 8: Kashwakamak Lake Dam, sluiceway and overflow looking west.

Figure 9: Kashwakamak Lake Dam, overflow spillway.

Figure 10: Kashwakamak Lake Dam, sluiceway and deck, looking north.

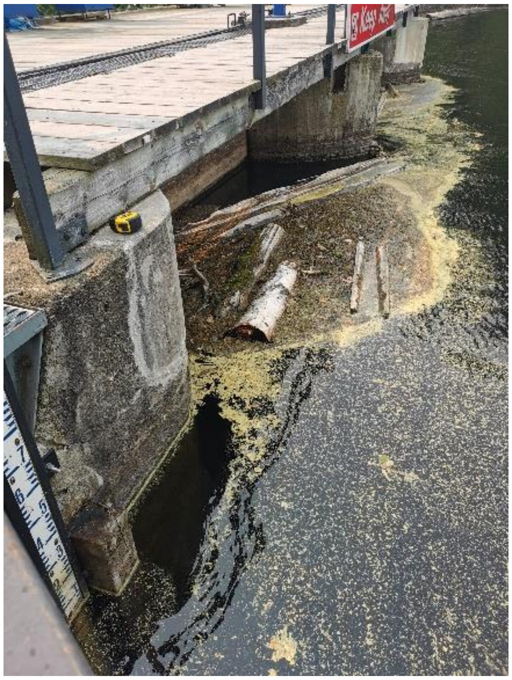

Figure 11: Kashwakamak Lake Dam, sluiceway and deck, detail.

Figure 12: Kashwakamak Lake Dam, spillway detail.

Figure 13: Kashwakamak Lake Dam, left concrete abutment and earthen enbankment.

Figure 14: Kashwakamak Lake Dam, concrete detail.