Construction of a Plastic Concrete Seepage Cutoff Wall for the New Coquitlam Dam

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ABSTRACT

Coquitlam Dam, constructed in the early 1910s, is a 30 m high hydraulic fill embankment. The dam is situated in a region of high seismic hazard in British Columbia, Canada. The existing dam core and shells, and part of the dam foundation are deemed to be liquefiable under the design earthquake. A new 30 m high compacted earth core rockfill embankment dam is currently being constructed at the downstream toe of the existing dam. As part of the construction of the new dam, a seepage cutoff wall has been completed underneath the central core of the new embankment to control foundation seepage gradients and to minimize piping potential of the foundation soils. The wall was constructed of plastic concrete using slurry panel construction method. Plastic concrete was selected to provide a seepage cutoff wall that has sufficient strength to withstand both static and seismic stresses beneath the new embankment, and yet is flexible enough to undergo seismic deformations, without cracking, with the surrounding soils. This paper describes the construction of the plastic concrete cutoff wall for the new Coquitlam dam, including the field and laboratory testing performed to confirm design wall stiffness, strength, and hydraulic conductivity requirements. The trial laboratory and field testing programs to determine plastic concrete mix design, and the QA/QC testing conducted during construction, including measurement of in-situ hydraulic conductivity of the constructed plastic concrete panels, are presented.

PROJECT BACKGROUND

Coquitlam Dam, as shown on Figure 1, is located on the Coquitlam River 15 km upstream of its confluence with the Fraser River, near the city of Port Coquitlam. The dam, owned and operated by BC Hydro, impounds Coquitlam Reservoir which provides water storage for power generation and potable water to the Greater Vancouver Water District (GVWD). Failure of the dam would have significant consequences due to downstream population and development.

Coquitlam Dam is a hydraulic fill embankment structure constructed between 1911 and 1913 as part of the Coquitlam-Buntzen Hydro-Electric Development Project. This type of earthfill structure is known to be vulnerable to earthquake damage, as evident from the near failure event of the Lower San Fernando hydraulic fill dam during the 1971 M6.6 San Fernando Earthquake (Seed et al. 1975). Coquitlam Dam was seismically improved in 1980 and 1984/85 by placing rockfill on its upstream and downstream slopes (see Figure 2) to limit seismically-induced deformation under the then Maximum Design Earthquake (MDE), M7.0 with PGA=0.35g. However, since the last seismic improvement, the seismic design parameters have increased significantly together with the profession's understanding of soil behavior under seismic loading. Deficiency investigations carried out between 1998 and 2003 concluded that the existing dam does not meet the present day dam safety standards, and a new embankment dam is currently being constructed at the downstream toe of the existing dam to mitigate seismic risks at Coquitlam Dam (Yan et al. 2007).

A plan and section of the new downstream embankment dam are shown on Figures 3 and 4, respectively. The new embankment dam comprises an earth core of till flanked by zones of filter, transition and rockfill shells, founded either on competent soils or bedrock. Because of the absence of bedrock at a high enough elevation, a mass concrete transition is founded on the left abutment bedrock. The reverse curvature of the new embankment axis is required so that the impervious core abuts the end of the concrete transition. The new dam is of the same height as the existing dam with the following main characteristics: (1) crest width of 8.0 m; (2) crest El. of 161.4 m; (3) upstream slope of 1.8H:1V; and (4) downstream slope of 1.7H:1V. Details of the new embankment are discussed in Yan et al. (2007).

As part of the construction of the new dam, a seepage cutoff wall was completed underneath the central core of the new embankment to control foundation seepage gradients and to minimize piping potential of the foundation soils. The wall was constructed of plastic concrete using slurry panel construction method. Plastic concrete was selected to provide a seepage cutoff wall that has sufficient strength to withstand both static and seismic stresses beneath the new embankment, and yet is flexible enough to undergo seismic deformations with the surrounding soils.

SITE GEOLOGY AND FOUNDATION SOILS

Based on regional geology and site investigations carried out during original dam construction, and more detailed geotechnical information obtained for recent deficiency investigations and design of the new embankment dam, a site geology model consisting of ten foundation soil units (with higher number designating geologically younger units) was developed as shown in Table 1. An interpreted surficial geology of the existing dam site is shown on Figures 5 and 6. It is shown that the existing dam site is generally underlain by a glaciated stiff silt layer (Unit 2A) which is in turn underlain by various glaciated dense overburden materials (Units 1B and 1A), except at the original river channel where the stiff silt (Unit 2A) is overlain by alluvial deposit (Unit 6A), and at the left abutment where bedrock outcrops. At the original river channel beneath the downstream slope of the existing dam, there exists an opening in stiff Silt (Unit 2A) where the stiff Silt (Unit 2A) was eroded by the river, and the Sand and Gravel alluvium (Unit 6A) overlying Unit 2A is in direct contact with the underlying dense Sand and Gravel (Unit 1B) which in turn is underlain by the dense Sand deposit (Unit 1A). The axis of the new embankment dam is 80 m downstream of the existing dam axis so that the entire new embankment dam is founded on the stiff Silt Unit 2A (Figure 4). The opening in Unit 2A at the original river channel is located near the upstream toe of the new dam. To control the foundation seepage, a plastic concrete seepage cutoff wall is designed and constructed as part of the new downstream embankment dam.

DESIGN ISSUES

The plastic concrete cutoff wall is 0.8 m wide by 150 m long, and typically 20 m deep (Figures 3 and 4), providing a partial foundation seepage cutoff. Its purpose is to reduce the exit gradients at the downstream toe of the new dam and to minimize potential piping of foundation Unit 2A silt into the underlying Unit 1B sand and gravel. As shown on Figure 4, the cutoff wall, located beneath the core of the new embankment, is typically extended through Unit 2A (silt) and Unit 1B (sand and gravel) into Unit 1A (sand). The Unit 1A sand is filter compatible with Units 2A and 1B. Any Unit 2A silt, if eroded into the underlying Unit 1B (sand and gravel), would be retained in the Unit 1B by the underlying Unit 1A sand and the downstream plastic concrete cutoff wall. The cutoff wall is embedded into the base of the core to provide a longer seepage path at its contact with the impervious core of the dam. The maximum amount of embedment is about 0.16 of the maximum dam height. The top of the cutoff wall is capped with a plastic core to accommodate potential arching around the cutoff wall.

A longitudinal section through the cutoff wall axis is shown on Figure 7. At the left abutment, the cutoff wall is keyed 0.3 m into the bedrock where the bedrock surface is less than 20 m deep. The left lateral extent of the cutoff wall is terminated when Unit 2A is in direct contact with the bedrock. At the right abutment where bedrock is very deep, the end of cutoff wall is extended about 50 m into the right abutment so that the average hydraulic gradient in Unit 1B around the end of cutoff wall is limited to less than 0.3, which would control the flow velocities around the end of cutoff in Unit 1B below Unit 2A (silt) such that fines movement originating from Unit 2A silt, if it ever occurred, is kept to a minimum. Prior to actual construction of the cutoff wall, additional drill holes were put down at both left and right abutment areas to better define the top of bedrock and Unit 1A sand, respectively.

The strength and stiffness characteristics of the plastic concrete cutoff wall were determined based on 2-dimensional static and dynamic finite element stress analyses using FEADAM (Duncan et al. 1984) and FLUSH (Lysmer et al. 1975) computer programs, respectively. The maximum bending stresses in the cutoff wall occur at the depth corresponding to the interface of Unit 2A (silt) and Unit 1B (dense sand and gravel). The combined static and dynamic stress envelops (i.e. maximum and minimum fiber stresses) induced in the cutoff wall due to seismic shaking with a full reservoir were computed as a function of the stiffness of the cutoff wall, as shown on Figure 8. As expected, a stiffer cutoff wall attracts higher stresses, and thus requires higher strength for the plastic concrete mix. Note that the stress and stiffness of the cutoff wall shown on Figure 8 were evaluated under the confinement of the soils. Therefore, the strength and stiffness design parameters are specified with an effective confining soil stress of about 300 kPa, corresponding to the depth at the interface of Units 2A and 1B. As shown on Figure 8, for a modulus less than 300 MPa, the confined strength required for a factor of safety of 1.5 is larger than 2.5 MPa. As the confined compressive strengths are typically 1.3 to 1.5 times higher than the corresponding unconfined compressive strength (UCS), thus a UCS of 2.0 to 3.0 MPa at 28 days is specified to control the strength.

Other criteria include a ductility requirement that the axial strain before failure at 28 days shall be larger than 5%, and a permeability coefficient at 28 days of less than 2.0×10^{-7} cm/s. The confined stiffness and strength, ductility and permeability criteria are determined from triaxial consolidated undrained (CU) tests on cylindrical samples (100 mm diameter by 200 mm long) of plastic concrete mix cast and cured for different ages. A slump of 200 ± 20 mm was specified for the plastic concrete to ensure its workability and minimize the risks of defect during tremie placement.

Prior to tender, a series of laboratory trial mix tests were carried out during preliminary design to assess the design criteria. Six plastic concrete trial mixes were prepared as shown in Table 2. The materials used for the plastic concrete trial mix consisted of Type 10 Portland cement, Premium Gel bentonite manufactured by Cetco, Wyoming, and fine/coarse aggregates meeting CSA A23.1-M with the coarse aggregates comprising 14x5 mm rounded particles. Potable water was added as required to achieve a target slump of approximately 200 mm. The trial mix test results, which included UCS test (CSA A23.3-9C), triaxial permeability test (ASTM D5084) with hydraulic gradients of 20 and 40, and triaxial consolidated undrained (CU) test (ASTM D4767) under a confining stress of 300 kPa, are shown in Table 3. The trial program shows Mix 2, with bentonite to cement ratio of 0.2, gives results that are closest to meeting the design requirements. Typical triaxial test results are plotted on Figure 9, showing a ductile plastic concrete response. These results were provided to bidders as reference information during tendering. The contractor was responsible for the final design to produce a plastic concrete meeting the design criteria.

CONSTRUCTION ASPECTS

Specification Requirements

The contract requires that the contractor conduct a laboratory trial program with selected material sources to design a plastic concrete mixture that meets the design requirements, following the testing standards and requirements specified in Table 4. After the plastic concrete design mixture has been determined in the laboratory, the contractor is then required, prior to actual production mixing of plastic concrete, to perform field trial mix of a minimum 10 m^3 of plastic concrete at site in accordance with the contractor's design mix using the materials, mixer and procedures to be used for the production. Laboratory testing as per Table 4 is required on each 5 m^3 batch of plastic concrete mixed. The field trial mix and tests are to be repeated until test results show the plastic concrete properties meet the design requirements. Once the contractor has demonstrated in their field trial that the plastic concrete mix can be consistently produced to meet the design requirements, the production of plastic concrete during construction will then only be controlled and checked by relatively simple tests, such as slump and UCS measurements.

Pre-Construction Plastic Concrete Design Mix

Five laboratory mixes were prepared and tested in the contractor's laboratory trial program, from which a plastic concrete design mix, Mix "1B", as shown in Table 5 was selected for the field trial. Bara-kade bentonite and coarse angular aggregates with max 14 mm particle size were used in the mix. The selected mix met all the design criteria except for modulus and slump, i.e. the initial modulus was higher, and the slump was lower, than the specifications. The higher initial modulus from laboratory Mix "1B" was considered acceptable due to its higher confined compressive strength, as shown on Figure 8. The concern was the low slump mix that may affect the tremie placement.

During the field trials, attempts were made by the contractor to modify the laboratory Mix "1B" to produce a field mix that would result in a lower initial modulus but a higher slump, and in the meantime keep other parameters within the specifications. Eleven trials were made; however, all failed to produce a field mix that would meet all design criteria. Particularly, the contractor failed to produce a higher slump mix without compromising the strength criteria. The typical slump from field trials ranged from 150 mm to 170 mm, all lower than the specified values. In the end, the contractor performed a field trial of tremie placement of plastic concrete in a 2 m deep x 5 m long x 1.5 m wide test pit to demonstrate the workability of the low slump concrete. The field trial indicated that a plastic concrete with a 150 mm slump is workable, and able to flow laterally to a distance of 2.5 m under about 2 m head. As small panel length was used in construction, the laboratory Mix "1B" was then accepted for production. To account for the lower slump of the concrete, the top of granular working platform was also raised about 0.3 to 0.5 m higher than the top of concrete guidewalls to provide extra head during concrete placement. The final accepted mix design was as follows (for 1 m^3 plastic concrete):

- 145 kg cement;
- 29 kg bentonite;
- 954 kg coarse aggregates;
- 780 kg fine aggregates; and
- 338 kg water (producing water/cement ratio of 2.33)

The field trials also showed that the plastic concrete mix properties were very sensitive to water-cement ratio. Careful control and monitoring of water content during construction, not only in bentonite slurry but also in both fine and coarse aggregates, are important in quality control to ensure a consistent plastic concrete product. Therefore, accurate measurement method and procedures were established to account for all water contents in the mix. As the majority of water was in the bentonite slurry to produce a pumpable slurry, only a very small amount of additional water could be allowed in the concrete aggregates in order to meet a water/cement ratio of 2.33. Thus, allowable water contents in the aggregates were tightly controlled during the production, e.g. the aggregates had to be protected from the rain.

Guidewall Construction

Prior to commencement of construction of the PCCW, up to about 6 m thick of the new embankment fill was placed and compacted to allow for construction of the concrete guidewalls, and embedment of the PCCW into the core of the new embankment dam. The top 1.2 m of the initial embankment also served as a working platform for PCCW construction that was removed later with the concrete guidewalls after completion of the PCCW. The guidewalls were constructed of 25 MPa concrete with 9 kg/m^3 of steel reinforcement cage, which was cast-in-place in a trench excavated in the impervious core of the initial embankment. The guidewalls were 1.2 m in depth and approximately 0.65m in width. The opening space between the two opposing guidewalls was a minimum of 0.85 m and 0.90 m in the straight and curved sections of the cutoff wall, respectively. The top of the cutoff wall was flush with the bottom of the guidewall. After completion of the cutoff wall, the concrete guidewalls were removed together with the working platform materials prior to resumption of fill placement for the embankment.

Panel Excavation

The cutoff wall was constructed by the slurry panel method of excavation. The slurry mixture supports the walls of the trench and maintains stability during the excavation and concreting process. Panels were excavated by a hydraulic clamshell bucket mounted on a 90 ton Lieberr HS-853HD crawler crane. The clamshell bucket was 800mm wide with a bite length of 2.7 m. Chisels, 8 and 12 tonnes, were used to break up obstruction (e.g. boulders) that was not able to be excavated with the hydraulic clam shell. Chisels were also used to excavate the minimum 0.3 m deep key into the bedrock at the left abutment where the most eastern 14 m of the cutoff wall was embedded into the bedrock. Cleanup of the chiseled rock was carried out with the hydraulic clam. Cuttings from the trench excavation were directly placed into a Cat D400D articulated dump truck and hauled to and disposed of at spoil disposal site.

A total of 22 panels (see Figure 10) were excavated as ‘opening’, ‘running’ or ‘closing’ panels with each panel excavated in three bites; a left bite, right bite and a final center bite. Each bite was excavated to target depth prior to commencing another bite. During excavation, the bottom of each panel was confirmed by viewing the cuttings retrieved in the clamshell bucket and sounding the depth of the trench. Following excavation and prior to concrete backfill, each panel was checked for depth, width, length, cleaned joints and verticality, and was cleaned of any slough that may have accumulated at the bottom of the trench. Panel lengths ranged from 6.0 to 6.9 m with the exception of two longer panels (Panel Nos. 23/20 and 20/16) at the left abutment that were 10.4 m and 7.7 m in length. The longer panels facilitated the keying-in and clean up process of the steep bedrock profile. This is because in order for proper cleanup of the sloping bottom of the bedrock at the panel joint, an additional bite had to be taken into the previously poured panel to allow for the 2.7 m long hydraulic clam to reach the sloping bedrock at the bottom of the panel joint.

Trench Slurry and Stability

The stability assessment by the contractor concluded that for a 6.5 m long panel and a bentonite slurry density of 1040 kg/m^3, the slurry level in the trench should be at least 1.5 m above the surrounding groundwater table. This was easily met with the site dewatering required for the embankment construction. The groundwater table was monitored in an adjacent piezometer over the duration of cutoff wall excavation. During construction, the slurry level in the trench was maintained as high as possible, typically 0.3 m below the top of the guidewall.

Trench slurry was mixed onsite and stored in five Baker tanks, each with a capacity of 79,500L (21,000 US gal). Three of the five tanks were designated for storage of fresh slurry and the other two tanks were designated for storage of used slurry that had passed through a desanding unit. A piping network consisting of 75 mm (3-inch) HDPE pipes was constructed to deliver slurry to the trench during excavation and to return slurry to the tank farm during desanding and backfill placement. The bentonite used in the slurry mix was the same as the one used for plastic concrete mix, i.e. Bara-Kade SP Grade (200 mesh) bentonite, meeting the Wyoming grade API 13A.

Prior to tremie placement of plastic concrete backfill, the slurry was desanded to remove sand that had accumulated on the bottom of the trench, or suspended in the slurry. Desanding was accomplished with a TOYO submersible pump specifically designed to pump solids that have settled out of fluid suspension in an excavated panel. The pump agitated the sediments on the bottom and the suction force of the pump removed slurry with all materials less than 13 mm, which was then circulated through the slurry return line to a desanding unit at the tank farm. Fresh or reclaimed slurry was delivered back to the top of the trench to make up for the slurry pumped out of the trench. The desanding process was carried out on the right, left and middle bites of the excavated panel and at various depths progressing from about mid-depth to the bottom of the panel. Progress of desanding was monitored by visual inspection of the sand ejected from the desander and from field tests performed on slurry samples taken from the trench until the test results showed the sand content of the slurry was less than 5% by volume specified in the contract.

Formation of Panel Joints

Panel joints were formed using the CWS (Coffrage avec WaterStop) joint system, as described in Vanel (1992). For an “opening” panel, the CWS end forms were installed vertically at both ends, after panel excavation, using a utility crane and held in place by a temporary steel guide template bolted to the top of the concrete guidewalls (Figure 11a). The CWS end forms remained in the trench during pouring of the “opening” panel, and during excavation of the “running” or “closing” panels, where they served as a guide for excavation at the joint with the previous panels. Guidance of the hydraulic clamshell by interlocking the clam with the in-place CWS end forms during excavation ensured a good alignment of the adjoining panels. The CWS end forms installed in the “opening” panel were only removed after completing the excavation, but prior to pouring, of the adjoining “running” or closing panels. For a secondary panel, the CWS end form was installed, after panel excavation, at either left or right joint, depending on whether it is a left or right running panel. No end joints were installed after excavation of the closing panel. After extracting the CWS end forms, the panel joints were cleaned, prior to backfill placement, by brushing the joint slot full depth with a custom made wire rope brushes cut to match the shape of the panel joint slot (Figure 11b).

At one panel joint location, difficulties were encountered in removal of CWS end forms as it was jammed by the edge of a boulder. Removal of a boulder by chiseling adjacent to the CWS end forms was not successful. Finally a cross bite excavation had to be carried out at the panel joint to the depth of the boulder obstruction to remove the boulder, and then the CWS end forms (see Figure 12).

Concrete Placement

Plastic concrete was mixed in an onsite batch plant, and transported to the dam site by transit mixers. Calculated amount of cement, aggregates and water were first batched in the concrete batch plant, which were then added to the transit mixer to mix with a measured amount of bentonite slurry. The bentonite slurry was hydrated at least 24 hours prior to the plastic concrete production. The bentonite slurry for use in the plastic concrete mix was mixed and stored in two 21,000 US gallon Baker tanks. During batching, the moisture content of the aggregates at the batch plant was measured to determine if any additional water was needed in the mix with consideration of the water content already in the aggregates and the hydrated bentonite slurry.

For most of the panels, plastic concrete was tremie placed through two 254 mm ID steel tremie pipes. However, for the two easternmost panels at the left abutment, three tremie pipes were used due to larger panel lengths (i.e. panel lengths of 10.4 m and 7.7 m). The tremie pipes were set approximately equidistantly spaced through the panel length, and about 0.3 m above bottom of the trench at the start of backfill placement. At the start of each pour, a plug was made in the tremie using vermiculite to separate the initial plastic concrete from the bentonite in the tremie pipe. During the pour, the tremie pipes were raised to maintain an immersion depth of 2 m and 4 m in the plastic concrete backfill, and displaced slurry was pumped to the storage tanks. During placement, soundings with a weighted open reel tape measure were made at approximately 2 m spacing within the trench. The soundings were taken after each truck load, and the depth to concrete versus volume placed was plotted and compared to a theoretical neat volume curve based on the design dimension. This plot, as shown on Figure 13, was used to determine areas of over-pour (below the theoretical line) or any potential areas of sidewall collapse during the placement (above the theoretical line).

Grout Curtain in Bedrock below Cutoff Wall

A series of grout pipes comprising 114mm diameter HDPE pipes were embedded in a section of the plastic concrete cutoff wall where the cutoff wall is in direct contact with the bedrock. These grout pipes were used after completion of the cutoff wall to perform a single line curtain grouting in bedrock and at the contact between the bedrock and the cutoff wall.

QA/QC PROGRAM

Trench Bentonite Slurry

The Contractor's quality control program consisted of testing the slurry for density, pH, viscosity, sand content, water loss in the filter press and filter cake. Testing was made twice daily during excavation with samples taken at approximately 5 m depth spacing. Fresh slurry had a density of about 1020 to 1060 kg/m^3. The density of the slurry in the trench during excavation ranged from 1020 to 1230 kg/m^3 and the density of the slurry in the trench after desanding just prior to backfill placement varied from 1020 to 1070 kg/m^3. The average density of the slurry prior to backfill placement was 1043 kg/m^3. The sand content of the slurry in the trench during excavation ranged from 0.5 to 8 percent and the sand content prior to backfill placement ranged from 0.5 to 1.5 percent with an average value of 0.7 percent. Just prior to backfill placement, the slurry was tested to ensure it was sufficiently thin for thorough displacement of the slurry during tremie placement of plastic concrete.

Plastic Concrete

Recognizing the deficiencies in their laboratory and field mix trial program, an augmented QC program was implemented by the Contractor with additional testing on UCS and confined initial tangent modulus of each poured panel at different ages to demonstrate that the engineering characteristics of Lab Mix 1B had been replicated in the field and the required values had been met consistently. For each panel, plastic concrete cylinders were cast and tested to determine UCS and confined initial tangent modulus at 7, 14 and 28 days. If UCS and initial tangent modulus did not meet the specifications, the confined strength and modulus were then used as per Figure 8 to determine if the plastic concrete had acceptable strength and stiffness characteristics. Deficiencies with low UCS strength were found in the first panel (Panel 1). The average 7 and 28 day UCS strength were 0.84 and 1.2 MPa, respectively. Cylinders from Panel 1 were also tested at 56 days and had an average UCS of 1.39 MPa. Based on the low UCS values, the panel was rejected and excavated to full depth, and replaced by a new panel.

In-panel Hydraulic Conductivity Measurements

Hydraulic conductivity tests were performed on plastic concrete cylinders, and showed that the permeability coefficient specification was met (i.e. less than 2×10^{-7} cm/s). In order to estimate the in-situ hydraulic conductivity of the plastic concrete cutoff wall, test holes were formed in selected panels by presetting a length of NW casing (with removable close-end plug at the bottom) in the panel prior to the pour and then removing the casing after the plastic concrete had cured for 48 to 72 hours, and the bottom 4 m of the hole was grouted with cement-bentonite. Figure 14 shows the preset NW casing with an oversized 250 mm diameter casing used to centralize the smaller casing in the panel. The casings were periodically rotated 2 to 3 times daily during the curing period. NW series drill casing is made from steel tubing and is flush inside and outside with no internal coupling, and has a specified outside diameter of 89 mm (3.5 inches). Tests consisted of a series of falling head tests in open holes and selected response zones isolated by pneumatic packers in the test holes. After tests, the test holes were grouted with cement-bentonite. Test results based on simplified analytical solutions suggest that the plastic concrete cutoff wall has an in-situ hydraulic conductivity in the order of 10^{-5} cm/s, i.e. much higher than those determined from cylinder tests. Similar trend of field vs. laboratory plastic concrete permeability coefficients had been reported by Singh et al. (2005). Seepage analyses were performed, and confirmed that a field hydraulic conductivity of 10^{-5} cm/s would meet the design intent of the seepage cutoff wall.

CONCLUSIONS

As part of the seismic upgrade of the Coquitlam Dam, a plastic concrete seepage cutoff wall was constructed beneath the impervious core of the new downstream embankment dam. The design and construction aspects of the plastic concrete cutoff wall are discussed. It is found that the plastic concrete is a difficult material to produce, and often design has conflicting demands on its strength and stiffness characteristics. A strength vs. stiffness relationship chart, as used in this project, provides a practical guide to resolve some of these apparent conflicting requirements. Therefore, it is important to have well-defined design criteria on the plastic concrete, and a trial mix design prior to construction. During construction, a well defined and executed QA/QC program is essential to ensure that the plastic concrete seepage cutoff wall is constructed to the design requirements. Field permeability tests in plastic concrete panels indicate in-situ permeability coefficients of about two orders of magnitude greater than those from laboratory cylinder tests. Design should consider in-situ permeability rather than laboratory permeability values of plastic concrete.

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Table 1 – Summary of Geologic Units and Characteristics of Relevant Foundation Soils

Unit	Soil Description	Geologic Period	Deposit	Soil Characteristics
7	Rubble	Postglacial	Colluvial (rockfall)	---
6B	Sand		Alluvial	---
6A	Sandy Gravel, Gravel and Sand, Cobbles/Boulders		Alluvial	$(N_1)_{60}$=10 – 30 Fines < 10%
5	Silt, Clay	Post-Vashon and Pre-Postglacial	Glacio-lacustrine	Soft to firm
4	Sand, Silt and Sand, Gravelly Sand		Glaciofluvial	$(N_1)_{60}$= 5 – 14 Fines = 5 -70%
3	Sand and Gravelly Sand	Vashon Stade	Subglacial	$(N_1)_{60}$=18 - 35 Fines = 5 -25%
2B	Sand and Gravel, Sand	Pre- Vashon Stade	Glaciofluvial	--
2A	Silt, Clay		Glacio-lacustrine	Stiff to hard
1B	Sandy Gravel, Gravel and Sand, Cobbles/Boulders		Glaciofluvial	Very dense Fines < 10%
1A	Sand		Glaciofluvial	$(N_1)_{60}$=28 – 30 Fines < 5%

Table 2 – Pre-Tender Mix Design Parameters for Plastic Concrete

Material Proportions	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5	Mix 6
Cement (kg/m3)	145	145	120	120	165	165
Bentonite (kg/m3)	40	29	30	40	30	50
Bentonite/Cement Ratio	0.28	0.20	0.25	0.33	0.18	0.30
Water/Cement Ratio	2.72	2.12	2.50	3.08	1.92	2.52
Fine/Coarse Aggregate Ratio	1.0	1.0	1.0	1.0	1.0	1.0
Slump (mm)	205	185	180	210	185	235

Table 3 – Summary of Plastic Concrete Pre-Tender Trial Mix Test Results

Test Property	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5	Mix 6	Criteria
Unconfined Comp. Strength (MPa) 7 days 18 days 28 days	 0.90 1.30 1.28	 1.68 2.32 2.37	 1.19 1.72 1.75	 0.69 0.99 1.03	 2.32 3.11 3.28	 1.15 1.68 1.84	 > 1.0 2.0 – 3.0
Permeability (x 10^{-7} cm/s) 7 days 28 days	 3.8	 2.2 1.2	 3.2 1.1	 5.6	 1.4	 2.6	 ≤ 2 x 10^{-7}
Confined Comp. Strength (MPa) 7 days 28 days	 1.28	 2.36 2.82	 1.96 2.20	 1.03	 2.57	 1.25	
Initial Confined Modulus (MPa) 7 days 28 days	 243	 242 333	 281 344	 145	 429	 181	 ≤ 300
Axial Strain at Failure (%) 7 days 28 days	 11	 7 5	 9 6	 9	 4	 9	 ≥ 5

Table 4 – Laboratory Testing of Plastic Concrete Mix Specimen

Tests	Standards
Slump	ASTM C143
Unit Weight	CSA-A23.2-M
Hydraulic Conductivity with hydraulic gradient of 20 at 7 and 28 Days	ASTM D5084
Unconfined Compressive Test with initial tangent modulus measurements at 7, 14, and 28 Days	CSA-A23.2-M or ASTM C39/C39M
Consolidated Undrained Triaxial Test (initial tangent modulus, ductility and strength) at 7 and 28 Days	ASTM D4767
Gradation and Specific Gravity on selected aggregates	ASTM C136, ASTM C127, ASTM C128

Note: Compressive strengths were tested on 100 mm diameter by 200 mm long cylindrical specimens.

Table 5 – Pre-Construction Plastic Concrete Design Mix from Laboratory Trial

Property	**Unit**	**Mix "1B"**	**Specification**
Cement content	kg/m^3	145	
Bentonite content	kg/m^3	29	
Water content	kg/m^3	338	
Coarse Aggregate	kg/m^3	954	
Fine Aggregate	kg/m^3	780	
Fine/coarse aggregate		45/55	
Water/cement		2.33	
Water/bentonite		11.64	
Slump	mm	170	200 ± 20
Unconfined	**Age Days**		
Compressive Strength, MPa	7	1.57	> 1.0
	28	2.17	2.0 to 3.0
Confined at 300 kPa	**Age Days**		
Compressive Strength, MPa	28	3.35	
Initial tangent modulus, MPa	28	345	< 300
Axial Strain, %	28	15	> 5.0
Hydraulic conductivity, cm/s (x10^{-7})	28	0.7	< 2.0

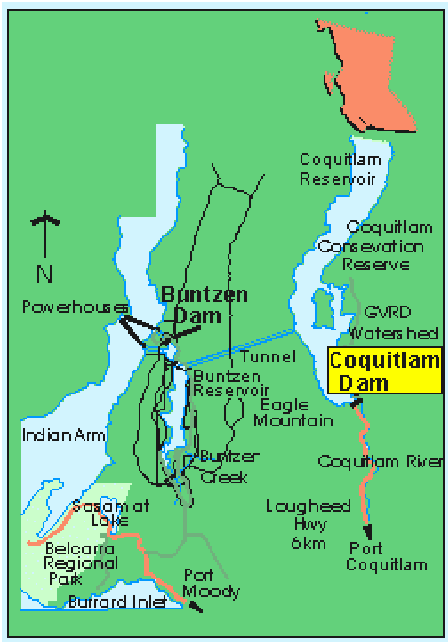

Figure 1 – Location of Coquitlam Dam

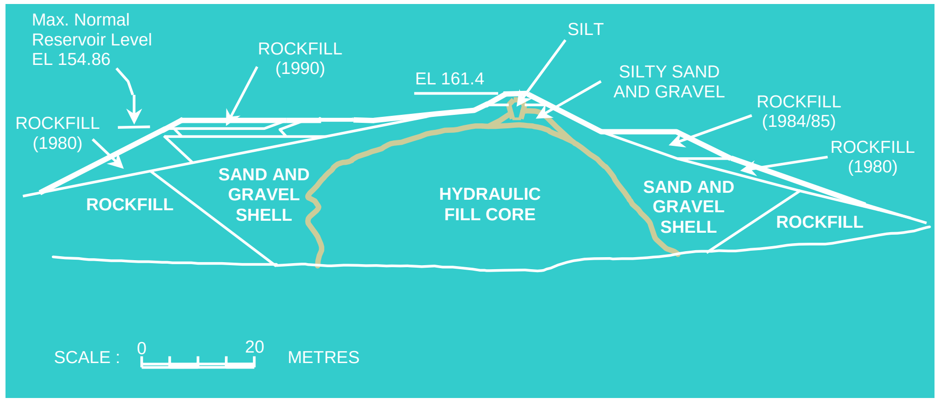

Figure 2 – Cross Section of Existing Coquitlam Dam

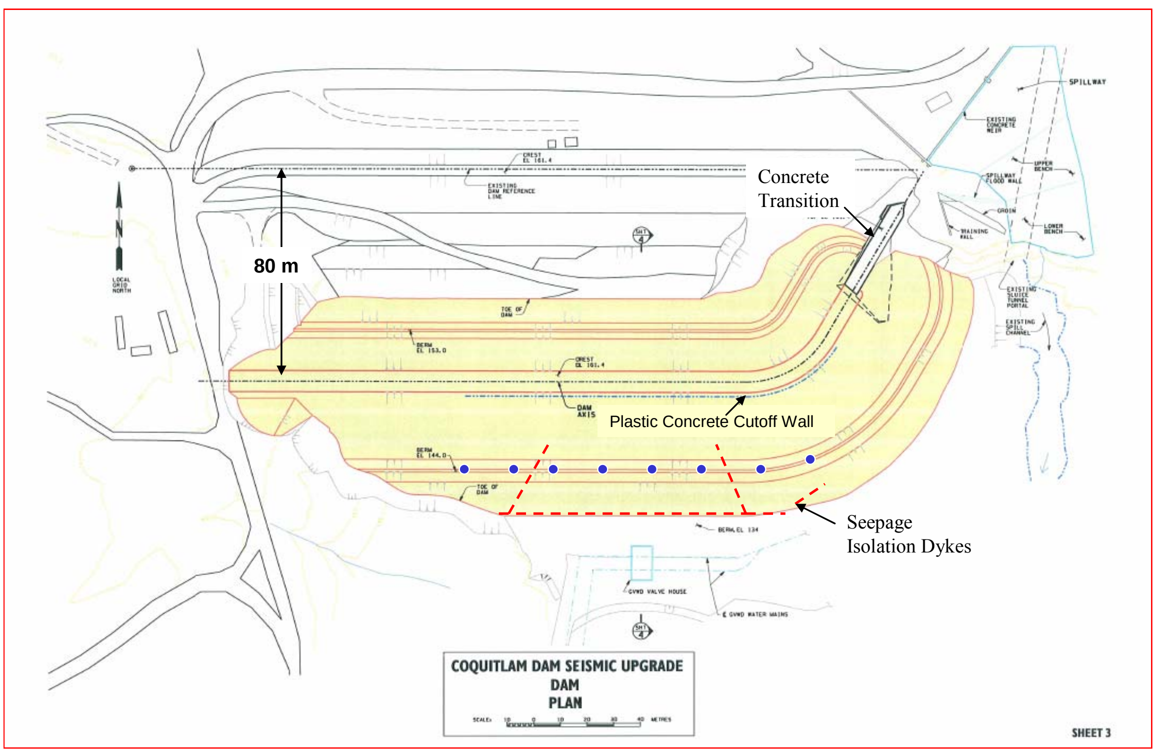

Figure 3 – New Downstream Earth Core Rockfill Dam – Plan

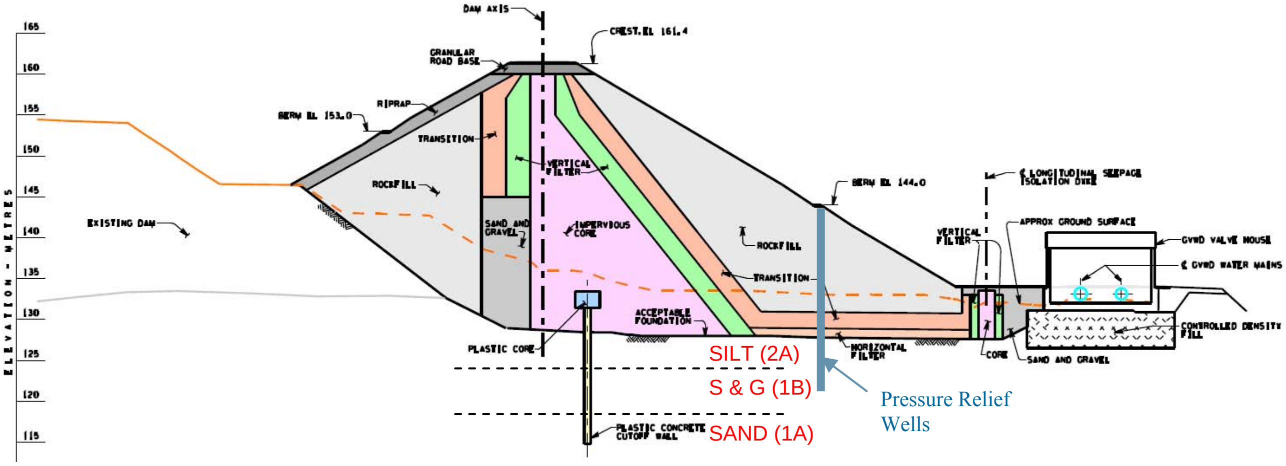

Figure 4 – Typical Section of New Downstream Rockfill Dam

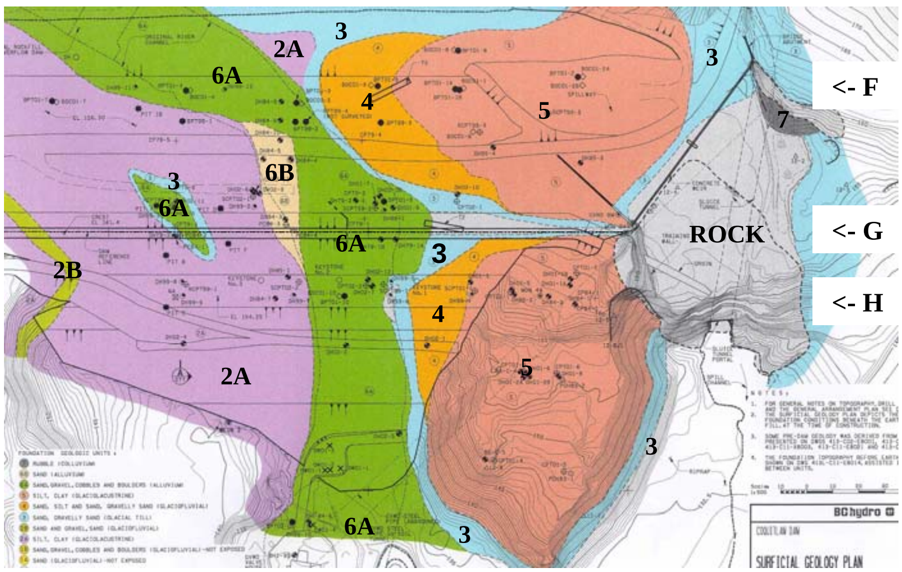

Figure 5 – Surficial Geological Units for Coquitlam Dam Site

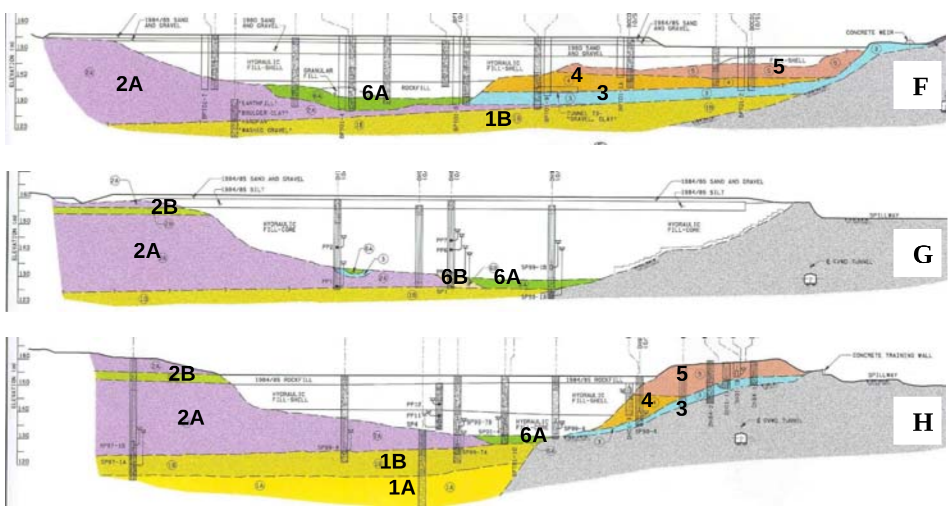

Figure 6 – Longitudinal Geological Sections F, G, and H (Looking Upstream)

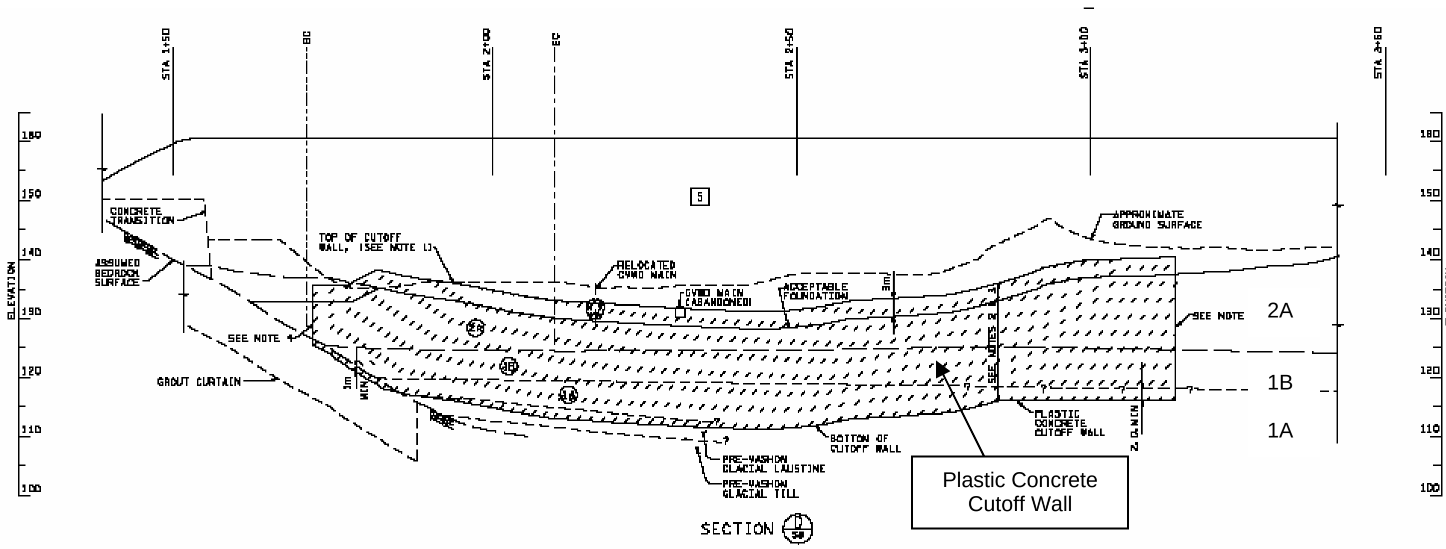

Figure 7 – Longitudinal Section of the Cutoff Wall (Looking Downstream)

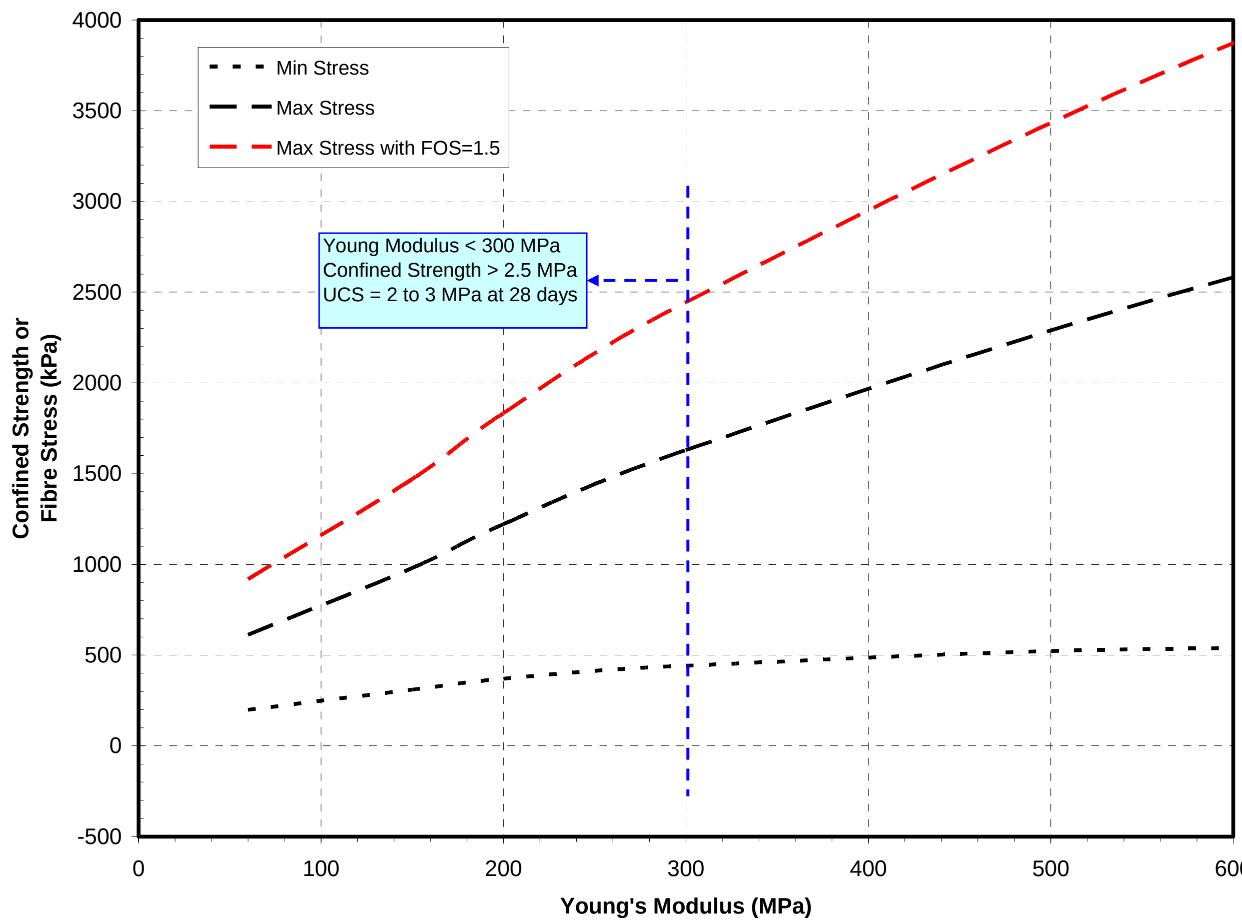

Figure 8 – Plastic Concrete Mix Strength and Modulus Design Chart

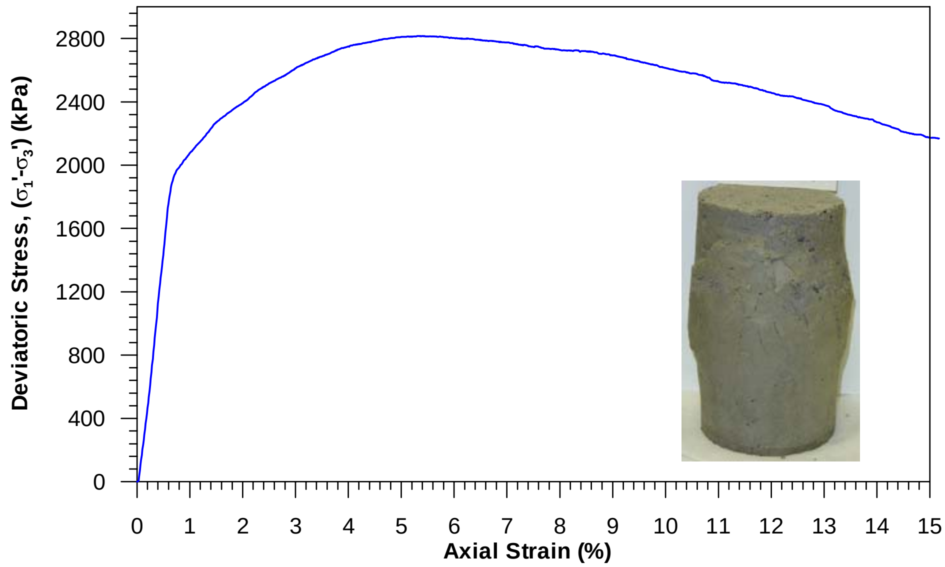

Figure 9 – Typical CU Triaxial Stress-Strain Test Results of Plastic Concrete Trial Mixes

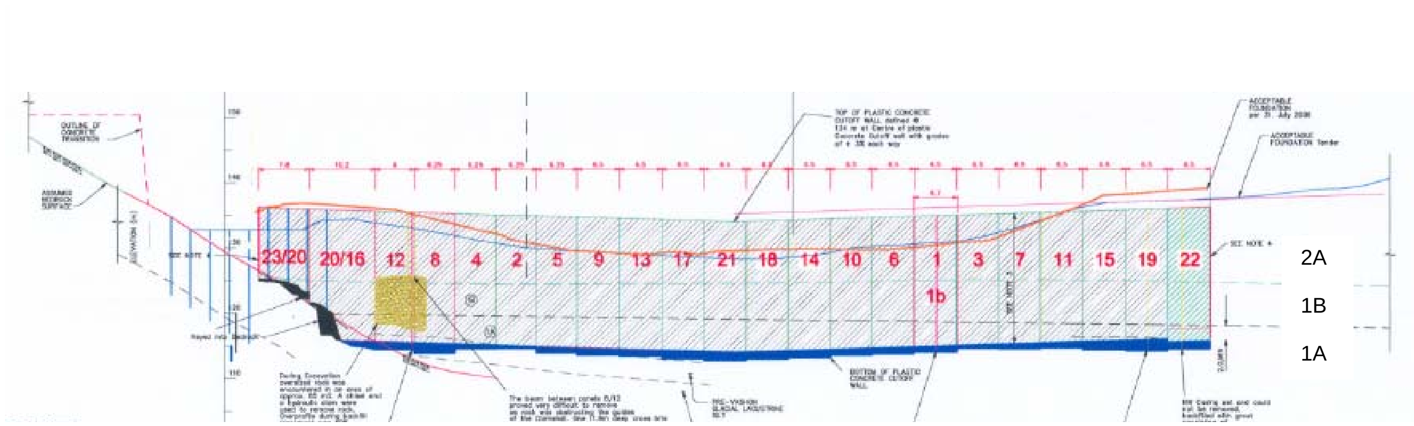

Figure 10 – Panels of Plastic Concrete Cutoff Wall (Looking Downstream)

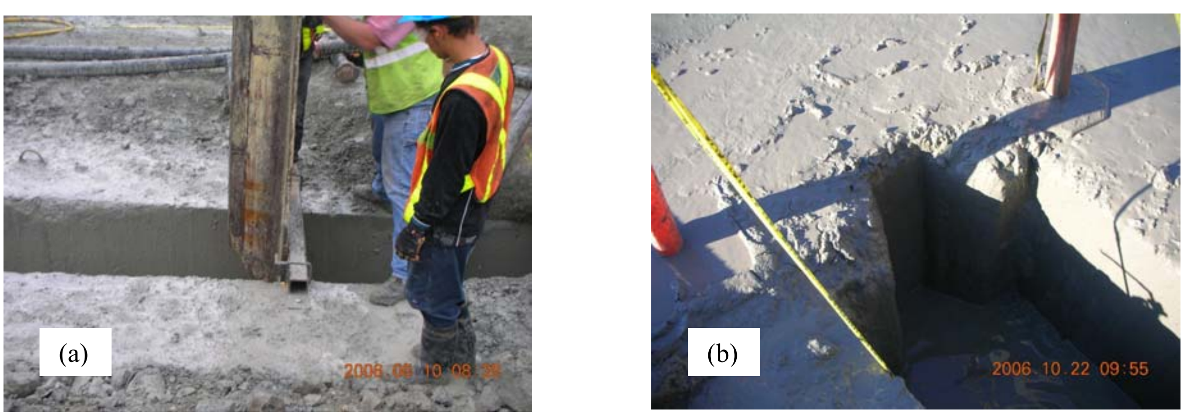

Figure 11 – (a) Installation of CWS End Joint Forms; (b) Key Slot formed by CWS End Joint System

Figure 12 – Cross Bite Excavation at Panel Joint Location to remove boulder and CWS End Joint Forms
(Note: a section of concrete guidewall was removed to perform the cross bite at the panel joint)

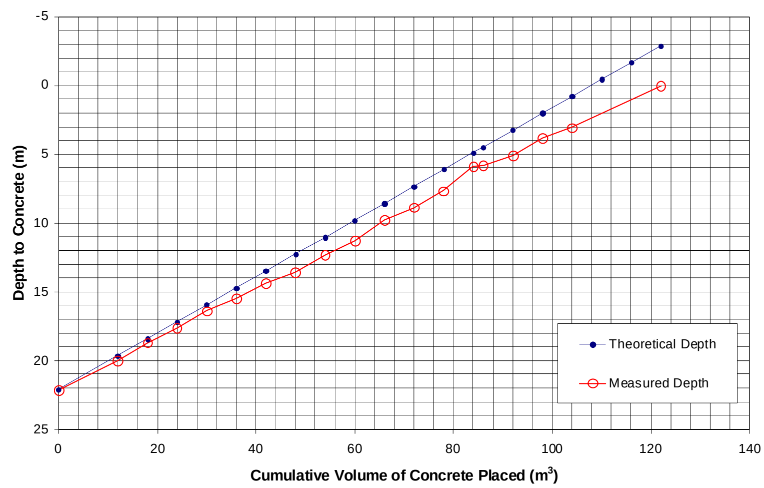

Figure 13 – Plastic Concrete Placement Volume Control Chart

Figure 14 - NW Casing for Hydraulic Conductivity Test Holes