

**APPENDIX 33-E
KSM WATER STORAGE DAM VALUE
ENGINEERING STUDY REPORT**

December 17, 2012

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Dear Mr. Smolik:

KSM Water Storage Dam - Value Engineering Study Report

Klohn Crippen Berger Ltd. is pleased to submit this report presenting a value engineering assessment of alternative dams - an asphalt core rockfill dam and a roller compacted concrete dam – as the Water Storage Dam for the KSM development.

Yours truly,

KLOHN CRIPPEN BERGER LTD.



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GWS:

EXECUTIVE SUMMARY

Klohn Crippen Berger Ltd. (KCB) has carried out a value engineering comparison of two types of dam – asphalt core rockfill dam (ACRD) and roller compacted concrete (RCC) dam – for the proposed Water Storage Dam on Mitchell Creek. The dam will collect mine area contact water for treatment. Water will be delivered to a water treatment plant located downstream of the dam.

The dam will be approximately 165 m high. The design life is expected to be 1,000 years. Contact water will be acidic, with an expected pH in the range of 3.5 to 5.5. The dam is designed to store mine contact water up to the 1:200 year “wet year”. A spillway will be provided to safely discharge higher flows, up to the Probable Maximum Flood.

The scope of work included conceptual level design of the ACRD and RCC dam; stability checks for static and seismic loading conditions; preparation of order of magnitude cost estimate; and comparison of the two alternatives on the basis of design issues, constructability, performance, maintenance requirements and cost.

Components common to both dams such as the grout curtain, drainage facilities, and water delivery system to the water treatment plant, are not addressed in this report. The cost estimates address only the major quantity items.

The ACRD will comprise a central core of asphalt; two transition zones of gravel on each side of the core; central zones of non-acid reactive rock (non-reactive) in the upstream and downstream shells; and rockfill zones that could comprise reactive rock in the outer parts of the shells. Aggregate for the asphalt, as well as the transition fills, would be manufactured from non-reactive rock. A drain layer at the base of the downstream shell would also comprise non-reactive rock. A central zone of the foundation would be excavated to bedrock; beneath most of the shells, weak foundation soil would be removed but dense soil would be left in place. The steep rock slopes immediately above the creek would be excavated back to 45° in the central zone of the dam. The dam meets all stability requirements. Estimated deformation due to the Maximum Design Earthquake is less than 100 mm. Settlement of the dam over a 1,000 year life (extrapolated from case history data that only covers about 50 years) is 1350 mm.

The estimated cost of the ACRD, using unit rates developed by Moose Mountain Technical Services that reflect the use of large mining equipment, is \$127 million. Use of the largest equipment planned for use at the mine would require that the permanent access road be in place. Using unit rates that might be expected for a large earthworks project, we estimate the cost to be \$235 million.

The RCC dam will be constructed with earth moving equipment. Conventional concrete would be placed on the upstream and downstream faces to provide freeze-thaw protection. An impervious PVC membrane, 2.5 mm thick, would be installed on the upstream face. Aggregate for RCC and conventional concrete would be manufactured from non-reactive rock. The dam is stable for all static and seismic load cases. The downstream slope, assumed to be 0.7H:1V for this conceptual layout, could likely be steepened somewhat and the RCC volume reduced accordingly.

Foundation preparation for the RCC dam includes excavation of all overburden beneath the dam footprint. The steep lower canyon slopes would be excavated to 45° for the full extent of the dam.

The RCC dam will be resistant to overtopping by water, either due to avalanche-induced waves or during large floods. The dam could be lowered, compared to the ACRD. In addition, a stepped spillway can be constructed on the downstream face of the dam, at little incremental cost compared to the dam construction, eliminating the need for a separate spillway on the abutment.

Large quantities of cement and fly ash would be required on an ongoing basis during RCC dam construction. The permanent access road would be required for delivery of these materials.

The estimated cost of the RCC dam is \$443 million. Constructing a spillway on the dam would result in savings in the order of \$50 million compared to the ACRD with its separate spillway on the abutment. Additional savings would result from shorter diversion tunnel compared to that for the ACRD, and if the dam crest is made narrower than 10 m. However, it is not expected that the total savings will be sufficient to result in a cost similar to that for the ACRD.

The ACRD could be constructed in about two years. The RCC dam would take three years. Dam construction can take place on the abutments while the diversion tunnel is being excavated; however, it is unlikely that a full construction season could be eliminated. For either alternative, careful planning will be required to sustain the high placement rates required.

Relative benefits in terms of design issues, construction, cost and risks are compared. Asphalt is resistant to acidic environments. The ACRD is cost effective, utilizing local fill and mine equipment. In the case of the RCC dam, the long term performance of the PVC in the acidic environment is unknown; allowance would be required for periodic replacement. The concrete is not resistant to acidic water. The RCC dam would take longer than the ACRD to construct.

On the basis of the comparison of the two options, we recommend that the asphalt core rockfill dam be adopted for the Water Storage Dam.

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1 INTRODUCTION

A Water Storage Dam (WSD) is required in Mitchell Creek to collect mine area contact water in order to attenuate wet year excesses of water or extreme event flows and to regulate the flow of water to the water treatment plant, prior to release. Non-contact water will be diverted around the WSD. The design life of the dam is assumed to be 1,000 years.

This report presents evaluation of two alternative options listed below for the WSD:

- An asphalt core rockfill dam (ACRD); and
- A roller compacted concrete (RCC) gravity dam.

An ACRD was recommended in 2011 for the site, as asphalt (bitumen) has a very long life and is resistant to acidic environments.

An RCC dam was selected for consideration as an alternative because this type of dam often has the following benefits: relatively rapid construction rate; small footprint for site preparation; spillway can be constructed on the dam rather than as a separate structure; relatively small quantity compared to rockfill and earthfill dams results in competitive cost despite the high unit rate for concrete.

For the purpose of evaluation, conceptual level designs of the main dams were developed and order of magnitude cost estimates were prepared and presented in this report. The evaluation included consideration of the following factors:

- design;
- constructability;
- schedule for construction;
- construction issues and risk;
- maintenance issues;
- reliability and performance over design life;
- risks associated with extreme events such as flood and earthquake;
- risks associated with performance due to the low pH water;
- tangible mitigation measures to manage risk; and
- cost (as-constructed).

Only the main dams are compared. Common associated components such as the grout curtain, drainage facilities and means of water delivery from the reservoir to the water treatment plant are generally the same or similar for the two alternatives and are not addressed. There are differences in diversion and spillway; these are described but not addressed in detail in this report.

The geology of the site and details of site investigations are presented in KCB (2012). The geotechnical conditions are adequate for founding either type of dam being considered for this study.

1.1 Objective and Scope

The objective was to evaluate the two alternative options for the WSD: Asphalt Core Rockfill Dam (ACRD) and Roller Compacted Concrete (RCC) Dam on the basis of their cost, constructability, reliability and performance.

The key requirements for the WSD which must be addressed in the design are:

- The design life of the dam is expected to be 1,000 years;
- The dam should be able to retain low pH water, in the range of 3.5 to 5.5, without causing any significant deterioration that would have adverse impact on the performance of the dam during its design life;
- The WSD performance/operational requirements, together with the location selected for the dam result in the WSD have a maximum height of approximately 165 m and crest length of approximately 650 m on the preferred alignment; and
- The dam will be designed to minimize seepage through the dam and dam foundation.

The scope of work summarized in this report includes:

- Conceptual level design of the ACRD and RCC dam;
- Stability checks under the static and seismic loadings for the two options;
- Preparation of order of magnitude cost estimate for the two options;
- Relative comparison of the two alternative options on the basis of their cost, schedule, constructability, reliability (or expected performance over its design life of 1000 years) and maintenance issues.

The scope of work did not include the design or comparison of the following items:

- Foundation grout curtain and consolidation grouting;
- Spillway;
- Drainage galleries and adits;
- Water handling facilities related to delivery of water to the water treatment plant; and
- Any common facilities.

Unit rates used for comparative purposes do not include design, construction management, or contingency.

1.2 Asphalt Core Rockfill Dam (ACRD)

A zoned rockfill dam with a central asphalt core is considered as one option. The upstream and downstream slopes of the dam are taken as 2.25H:1V and 1.75H:1V, respectively. A tapered central asphalt core with thickness increasing from 1 m at the top to 2 m at the base of the maximum section is provided as the impervious element, to minimize seepage through the dam.

Asphalt liners are used in industrial acidic water handling applications due to asphalt's acid resistance and long term durability (KCB 2012, Appendix VII). Asphalt core placement is less disrupted by poor weather conditions than placement of clay and till cores. The plastic, self healing nature of asphalt cores makes this type of dam resistant to leakage from settling of fill or earthquake deformation. The first asphalt core dam was built in Germany in 1961-1962. Since then, more than 150 ACRD dams have been built in many regions around the world but mostly in northern Europe, including several in Norway. Currently, two dams of similar height are under construction in China and one is in the detailed design stage in Turkey.

1.3 Roller Compacted Concrete (RCC) Dam

A gravity dam constructed of RCC is considered as the alternative to ACRD. RCC was developed in the 1980s, and there are now about 450 RCC dams world-wide. The cementitious content (cement, fly ash, sometimes other pozzolanic materials) is relatively low compared to conventional concrete. RCC is spread and compacted using earthfill equipment. High placement rates can be achieved. The RCC is compacted in typically 300 mm thick layers. The upstream and downstream faces are formed. The upstream face is most often vertical; conventional concrete formwork is usually used. The downstream face is usually stepped; a variety of formwork has been used including conventional concrete formwork and concrete blocks. Several RCC dams of similar height have been constructed around the world (International Water Power & Dam Construction, 2012).

Most RCC dams rely on an upstream face of conventional concrete, grout-enriched RCC (grout is added to RCC so the resulting mix can be vibrated as is conventional concrete), or the RCC without modification to provide an impervious barrier to water. However, many dams incorporate a geomembrane, most commonly polyvinyl chloride (PVC) on the upstream face to prevent seepage through the dam. PVC facings have been in place on dams since the early 1970s in the Italian Alps. The material formulation includes stabilizers to mitigate UV deterioration.

1.4 Foundation Grout Curtain and Blanket Grouting

A grout curtain will be constructed to minimize seepage through the dam foundation. The grout curtain for both dams will be of comparable size and constructed along the same alignment. For the ACRD, the grout curtain will be located directly beneath center of the dam below the asphalt core. For the RCC dam, it will be installed from a plinth just in front of the upstream face of the RCC dam. Note that the dam axis for the ACRD and the upstream face of the RCC dam are coincident in the designs developed for this report.

1.5 Seepage Collection Dam

A seepage collection dam about 20 m high will be built downstream of the main ACRD or RCC dam to collect seepage that bypasses the main dam. The seepage collected from this dam will be sent to the Water Treatment Plant (WTP). The foundation of the seepage collection dam for both options will be provided with a grout curtain of comparable size to reduce foundation seepage.

For the ACRD option, the seepage collection dam will also be an asphalt core dam with rockfill shells and side slopes of 2.5H:1V.

For the RCC dam, the seepage collection dam will also be an RCC dam with downstream slope of 0.7H:1V.

1.6 Drainage Measures

Both dams will be provided with a drainage curtain immediately downstream of the grout curtain to intercept and collect seepage within the foundation. The drainage curtain will comprise adits in the foundation at two levels (approximately at El. 550 m and El. 620 m) within the foundation, and drain holes drilled up and down from each adit to intercept seepage that invariably bypasses the grout curtain. Intercepted seepage will be directed downstream to the seepage collection dam reservoir. The approximate size of the adits will be 3.5 m by 3.5m.

The RCC dam also will be provided with drainage galleries located approximately 8 m downstream of the upstream face at the same levels as the adjacent adits. A series of 75 mm diameter drains at approximately 3 m centres will be drilled from the upper drainage gallery and adits to the lower ones, to the dam crest and into the foundation near the abutment to intercept any seepage which may leak through the dam and foundation grout curtain.

Thus for either dam, seepage through the dam as well as seepage at shallow depths beneath the dam will be led to the reservoir formed by the seepage collection dam.

1.7 Water Delivery System

The acidic water in the reservoir will be delivered by pumping over the dam via pipeline to a water treatment plant located downstream of the dam. It is assumed that a similar conveyance system would be implemented for either option.

1.8 Diversion during Construction

Water will be diverted out of Mitchell Creek and through a tunnel in the right (looking downstream) abutment during dam construction. When the tunnel and portal structures are complete, cofferdams will be built upstream and downstream of the dam site to lead water into the tunnel and around the site. The combination of tunnel size and cofferdam height is commonly selected to provide protection from a specified flood event, often about the 1:10 year and 1:50 year flood for RCC and rockfill dams, respectively. A more frequent design flood event can be chosen for RCC dams as they can withstand overtopping with little or no damage. The design flood for rockfill dams can be decreased by

incorporating measures in the downstream slope to prevent erosion in the event of a flood that causes overtopping. However, some damage would occur to the rockfill in the event of overtopping, in particular the finer materials close to the asphalt core.

1.9 Spillway

The dam height and reservoir storage volume are selected to prevent spill of acidic water past the dam for events with total volume smaller than the 1:200 year “wet year”. However, the dam will be designed to safely pass the Probable Maximum Flood. Therefore, a spillway is required to safely discharge high flows.

In the case of the ACRD, the spillway will be a rock cut on the left abutment. It will be concrete-lined in the vicinity of the dam. The spillway channel will lead flows to discharge into the creek valley downstream of the seepage collection dam. It is currently envisaged that downstream of the dam vicinity, the spillway channel would comprise an excavation into rock. A concrete flip bucket will be required near the valley bottom to facilitate energy dissipation.

For the RCC dam, the spillway can be incorporated on the downstream slope. A stepped spillway would follow the downstream slope, and would utilize the steps that are commonly built into the dam to dissipate energy. The dam crest would be at the maximum reservoir level at the spillway section and walls on the downstream slope would contain the flow. Some portion of the maximum design flow could be permitted to overtop the entire dam for a short time, rather than designing a spillway that can handle 100% of the design flood.

2 DESIGN BASIS

The design basis relevant to the design of the ACRD is summarized below based on the information provided in KCB (2012). The design basis for the RCC dam is the same in most respects. However, since the RCC dam can withstand some overtopping without adverse effects, freeboard could be less than is required for the rockfill dam.

2.1 General Design Basis

- Design life : Minimum 1,000 years.
- Water Quality: pH will be in the range of 3.5 to 5.5.
- Storage Requirement: Store 1:200-year wet year runoff without discharge to limit potential release of contact water to the environment.
- Required Storage Volume:
 - Minimum Volume: 1 Mm³ for sediment control.
 - Maximum volume: 60 Mm³ (peak of 1:200 year wet year).
- Reservoir Elevations:
 - Normal Operating Reservoir Elevations: Low reservoir elevation of El. 610 m in spring and high reservoir of El. 665 m in fall.
 - Maximum Reservoir Elevation: El. 706 m during a 1:200 year wet year.
- Freeboard and Minimum Dam Crest Elevation:
 - Freeboard: 8 m to accommodate wave run-up due to avalanche and landslide generated waves. This freeboard exceeds requirements for passing the Inflow Design Flood (IDF). IDF and avalanche are assumed to not occur at the same time. The freeboard is considered on top of 1:200 year wet year water level of El. 706 m. Thus, the resulting dam crest elevation including the freeboard is El. 716 m.
 - Minimum Dam Crest Elevation: El. 716 m.

According to the 2007 Canadian Dam Association Guidelines (CDA, 2007), the consequence category of the water storage dam is taken as “Very High”. The following criteria related to the static and seismic stability of the dam are adopted based on the CDA (2007) guidelines:

2.2 Static Stability

- Static factors of safety:
 - Long term steady state FOS > 1.5;

- End of Construction FOS > 1.3; and
- Rapid Drawdown FOS > 1.2.

2.3 Seismic Stability

Maximum Design Earthquake (MDE) ground motions are taken as the 1:10,000 year earthquake ground motions with Peak ground Acceleration (PGA) of 0.14 g based on the site specific seismic hazard assessment conducted for the KSM site by KCB (2010). The corresponding earthquake magnitude is taken as Mw7. Note that the 1:10,000 year ground motions specified for “extreme” consequence category structures are adopted to add robustness to the dam design; the 1:5,000 year earthquake could be adopted for “very high” consequence dams (CDA, 2007).

- Seismic stability factors of safety:
 - Pseudo-static FOS > 1.0; and
 - Post-earthquake FOS > 1.2;

2.4 Static and Seismic Deformations

- The static and seismic horizontal and vertical shear displacements at the core shall not be greater than 0.5 m and 1.0 m, respectively. The intent of this requirement is to keep potential displacements to less than one-half the core width so its function is not jeopardised.

2.5 Spillway

- The selected inflow design flood (IDF) is the Probable Maximum Flood (PMF) event, with snowmelt. Note that a “very high” consequence dam such as the WSD could be designed for the lesser criteria of 2/3 between the 1/1,000 year earthquake and the PMF.
- Spillway invert elevation for the ultimate dam is taken at El. 706 m.

2.6 Seepage Collection Dam

Potential seepage of acidic water past the WSD will be captured by a seepage collection dam located a short distance from the downstream toe of the main dam. Safety and flood criteria are based on a “significant” consequence category under the 2007 CDA guidelines.

- Flood: 1:200-year, 24-hour flood with snowmelt will be stored without discharge.
- Operating Surge Storage: Greater than 14 days of WSD seepage and catchment runoff assuming failure of WTP system.
- Spillway: Designed to pass the 1:500 year 24 hour flood event.
- Seismic: 10,000 year return period ground motions with PGA of 0.14g.

- Stability:
 - Static FOS > 1.5;
 - Pseudo-Static FOS > 1.0.

3 ASPHALT CORE ROCKFILL DAM

3.1 Geometry and Zonation

Figure 1 shows a plan view of the Asphalt Core Rockfill Dam (ACRD) and Figure 2 shows the geometry and material zonation of the dam. The upstream and downstream slopes of the dam are taken as 2.25H:1V and 1.75H:1V. The maximum height of the dam is 165 and the crest length is approximately 650m. The dam comprises the following key zones:

- A low permeable central asphalt core with 1 m width at the top and 2 m at the bottom at the maximum section. The maximum width of asphalt core constructed to date, in a dam of similar height, is 1.6 m. The 2 m width provides a level of redundancy in the unlikely case that some deterioration could occur in the acidic environment. Aggregate for the asphalt will be manufactured from non-reactive rock;
- A transition zone of non-reactive gravel adjacent to and on each side of the asphalt core. The top width (minimum) of this zone is 2 m and it increases to 4 m at the bottom at the maximum section. The material will be broadly graded between a maximum grain size of 60 mm and less than 10% finer than 0.075 mm;
- A second transition zone of non-reactive, crushed rock adjacent to the first transition on both sides of the core. The top width (minimum) is 3.5 m and it increases to 10 m at the bottom at the maximum section. The transition material is graded from a maximum size of 150 mm to less than 3% finer than 0.075 mm;
- A well compacted shell zone of quarried, non-reactive rock adjacent to the second transition zone on both sides of the core. The width of this zone on the upstream side is significantly greater than width of the downstream zone as this upstream zone will be in contact with low pH water. Maximum particle size is 700 mm;
- Well compacted shell zones with quarried rock adjacent to the non-reactive rock. Maximum particle size is 1000 mm;
- Well compacted shell zone of natural gravel or quarried rock. Maximum particle size is 1000 mm; and
- Blanket drain with crushed rock, placed on the foundation under the downstream shell, manufactured from non-reactive rock.

The two zones with maximum particle size 1000 mm could, in fact, be from the same source.

The asphalt core and adjacent transition zones will be founded on a concrete plinth. The concrete provides a uniform surface on which to compact the initial layers of core and transition, as well as the base for the grout curtain. The concrete will receive a bituminous coating to protect it from the acidic environment.

The use of non-reactive rock in the inner part of the dam fill is intended to ensure long term support for the asphalt core, despite the low pH of the reservoir water.

The central part of the dam consisting of the asphalt core and the adjacent transition zones will be founded on bedrock. Overburden in this area will be excavated and the upper part of the bedrock will be treated with blanket grouting. A grout curtain will be constructed to minimize seepage through the dam foundation. The dam outside the central part will be founded on dense overburden soils after stripping and removal of any loose, surficial soil.

The lower foundation area in the creek gorge is very steep. It will require trimming to a slope of 45° in the central section of dam near the core, in order to mitigate arching that can occur in narrow valleys. In this phenomenon, the fills adjacent to the steep bedrock slope are prevented from settling due to friction, while the fills in the middle of the valley can settle. The differential settlement results in relatively low stress in the middle of the valley. In the extreme, unless mitigated by proper design of the foundation trimming and adequate compaction effort, the stresses can be lower than the pressure of the reservoir, resulting in “hydrofracturing” of the fills and leakage from the reservoir.

Inflows to the reservoir in excess of the 200-year wet year flows will be released via a spillway excavated in the left abutment. The spillway invert will be El. 706 m. A concrete structure will protect the dam and prevent rock erosion in the dam vicinity. Much of the spillway channel to river level will be in rock excavation; the excavated rock will be used as dam fill. A stepped spill channel will lead lower flows down the steepest portion of the gorge to creek level; a concrete flip bucket will be used to discharge higher flows.

The diversion tunnel for the ACRD will be about 1200 m long. For safety, the upstream portal must be located upstream of an avalanche chute which is very close to the upstream toe of the dam. The downstream portal will be between the main dam and seepage collection dam, so that following tunnel closure, seepage from it can be captured for delivery to the water treatment plant.

3.2 Stability Assessment

Static and seismic stability assessments of the asphalt core rockfill dam were conducted. Material parameters used in the assessment are listed in Table 1. The dam is to be founded on dense foundation soils except near the center where it will be directly founded on bedrock. Foundation soils and dam fills are not susceptible to liquefaction or significant strength loss under the seismic loading.

Table 1. Material Properties for WSD Stability Analysis

Material	Unit Weight	Strength	
	γ (kN/m ³)	ϕ' (°)	c' or s_u (kPa)
Quarried Rock	21	40	0
Crushed Rock	21	40	0
Gravel	20	36	0
Asphalt Concrete Core	23	25*	0
Drain	21	35	0

Material	Unit Weight	Strength	
	γ (kN/m ³)	ϕ' (°)	c' or s_u (kPa)
Overburden	20	32*	0

Note: *conservative estimates.

Two-dimensional limit equilibrium stability analyses were carried out using the computer program SLOPE/W (Geo-Slope, 2011) and the Morgenstern-Price method of slices to determine the factor of safety (FOS). The dam cross-section at its maximum height was analysed.

Analyses of the upstream and downstream slope were conducted for various conditions listed in Table 2. The water levels considered in the analyses are also listed in Table 2.

Table 2. Summary of WSD Stability Analyses

Case #	Slope	Static or Seismic	Water Elevation (m)		Horizontal Earthquake Acceleration (g)	Factor of Safety	
						Target	Calculated
1	Downstream Stability	Static	Maximum Normal	665	0.00	1.5	1.49
2		Static	200-year Wet Year Maximum	706	0.00	1.5	1.49
3		Seismic ¹	Maximum Normal	665	0.195	-	1.00
4	Upstream Stability	Static	Operating	665	0.00	1.5	1.83
5		Static	200-year Wet Year Maximum	706	0.00	1.5	1.91
6		Static	Rapid Draw Down ²	607	0.00	1.3	1.75
7		Seismic ¹	Maximum Normal	665	0.180	-	1.00

Note: ¹Yield acceleration is determined which is the acceleration required to bring the FOS to unity. ²As the slip-surface did not pass through the core, the case is effectively low-water condition.

The horizontal seismic displacements under the Maximum Design Earthquake (MDE) with Peak Ground Acceleration (PGA) of 0.14 g and earthquake magnitude of M7 were estimated using the methods proposed by Hynes and Franklin (1984) and Bray and Travararou (2007). The estimated 84th percentile displacements are less than 0.1 m by both methods. The seismic settlements were estimated using the method proposed by Swaisgood (2003) and Bureau (1997) using case history-based relationships. The estimated seismic settlement was less than 0.1 m by both methods.

The construction settlement at the mid-height of the core was estimated as 0.5% of the dam height, or about 0.850 m. Based on the data presented in Fell et al. (2005), the post-construction settlement of the dam over its design life of 1000 years is estimated as 1.35 m, which is approximately 0.8% of the dam height. Note that the empirical data on post-construction settlement of dams are limited to less than 50 years. Thus, the settlement over a period of 1000 years was estimated by extrapolation.

Settlement of 0.3% of the dam height in the first 10 years and 0.25% for each of the two subsequent log cycles (i.e. to 100 years and 1000 years) is assumed. Note that the static displacement of the asphalt core depends on many factors including the stresses imposed on the asphalt core due to the adjacent transition zones, reservoir load and self-weight.

3.3 Quantities and Cost Estimate

The major quantities for excavation and fill are summarized in Table 3. The total dam fill is approximately 10.7 million cubic metres, of which 5.3 million cubic metres will be non-reactive rockfill, transitions, asphalt aggregate and drain rock. Table 3 also summarizes excavation quantities for overburden removal and rock shaping. As noted earlier, only the major quantities for items that provide a comparison of the two alternatives are presented. Components such as grouting, which will be the same for both options, are not included.

Table 3 also presents the cost estimate for construction of the listed components. Unit rates for placed fill in Table 3 were developed by KCB and Stan Burton, using unit rates for blasting, loading and hauling of mine rock by Moose Mountain Technical Services, with input from Seabridge Gold. Those rates yield an estimated cost of \$127 million. However, some of the unit rates are very low (e.g. \$4/m³) for rockfill that is quarried, hauled and dumped, spread, and compacted using smaller equipment. The rates used in Table 3 reflect the use of large, owner-supplied equipment for the mine development (e.g. 400 tonne trucks and large shovels), which would require that the permanent mine access road and the full haul road to Sulphurets pit be in place to get that equipment to the site and allow hauling back to the WSD. While much of the rockfill, in particular the non-reactive rockfill, will be sourced from required excavation at the mine and so could be discounted (excavation is required in any event), that rock has to be hauled several kilometres to the dam site. About 50% of the rockfill will be from quarries near the dam site and cannot be discounted as mine rock stripped for preproduction or for other development excavations; however, large mine equipment may be available for quarrying in these areas. We have increased unit rates to a range of \$16 to \$25/m³ in Table 8, i.e. the range of unit rates that might be expected on a large contractor-based earthworks project, yielding an estimated cost of \$235 million. The current plan for construction of the WSD involves the use of 30 tonne and 100 tonne trucks (brought in over the winter access road in sections) in order to meet the construction schedule (WSD required by end of Year -2).

Table 3. Quantities and Construction Cost Estimate for ACRD

Item	Description	Quantity	Unit	Unit Cost	Total Cost
1	Foundation Preparation				
1.1	Logging, Grubbing, Clearing and Stripping		LS	500,000	500,000
1.2	Foundation Overburden sub-excavation for Grout Curtain and Grouting	407,000	m ³	8.5	3,459,500
1.3	Bedrock Shaping and Trimming	10,000	m ³	25	250,000
2	ACRD Dam				
2.1	Asphalt Concrete (Bitumen and aggregate)	66,000	m ³	300	19,800,000
2.2	Gravel Transition Zone A	263,000	m ³	20	5,260,000
2.3	Crushed Rock Transition Zone B	802,000	m ³	20	16,040,000
2.4	Well Compacted Quarried Non-Reactive Rock (0-700mm)	3,787,000	m ³	10	37,870,000
2.5	Well compacted Quarried Rock (0-1000mm)	3,821,000	m ³	8	30,568,000
2.6	Well Compacted Natural Gravel or Quarried Rock (0-1000 mm)	1,554,000	m ³	4	6,216,000
2.7	Drain Blanket	358,000	m ³	21	7,339,000
	TOTAL				127,300,000

Table 4. Quantities and Construction Cost Estimate for ACRD with Revised Unit Rates

Item	Description	Quantity	Unit	Unit Cost	Total Cost
1	Foundation Preparation				
1.1	Logging, Grubbing, Clearing and Stripping		LS	\$ 500,000	\$ 500,000
1.2	Foundation Overburden sub-excavation for Grout Curtain and Grouting	407,000	m ³	\$ 9	\$ 3,459,500
1.3	Bedrock Shaping and Trimming	10,000	m ³	\$ 25	\$ 250,000
2	ACRD Dam				
2.1	Asphalt Concrete (bitumen and aggregate)	66,000	m ³	\$ 300	\$ 19,800,000
2.2	Gravel Transition Zone A	263,000	m ³	\$ 25	\$ 6,575,000
2.3	Crushed Rock Transition Zone B	802,000	m ³	\$ 25	\$ 20,050,000
2.4	Well Compacted Quarried Non-Reactive Rock (0-700mm)	3,787,000	m ³	\$ 20	\$ 75,740,000
2.5	Well compacted Quarried Rock (0-1000mm)	3,821,000	m ³	\$ 20	\$ 76,420,000
2.6	Well Compacted Natural Gravel or Quarried Rock (0-1000 mm)	1,554,000	m ³	\$ 16	\$ 24,864,000
2.7	Drain Blanket	358,000	m ³	\$ 21	\$ 7,518,000
	TOTAL				\$ 235,200,000

3.4 Schedule

We understand that the dam is to be constructed in approximately two years (in addition to the time for diversion tunnel construction). This will require the placement of about 5 million cubic metres of fill per year. This is an aggressive schedule that will require fill placement through most of the year. A benefit of rockfill is that it can be placed in freezing conditions. The road from the non-reactive monzonite to the dam will have to be well-maintained nearly year-round to achieve the schedule.

The asphalt core can be placed to a few degrees below zero Celsius, using insulated machinery and covering the asphalt immediately following placement. The core and adjacent transition zones (which

are placed concurrently) will be halted once sustained temperatures fall several degrees below zero. Mean daily temperatures are above freezing from May to October (KCB, 2012). The rockfill can be placed in freezing temperatures. The outer transition zones and rockfill zones will be stepped to the outer rockfill where placement can continue as long as snowfall can be cleared from the placement area.

The total volume of the asphalt core zone is 66,000 m³. The bitumen content will be about 6% by mass of the asphalt/aggregate mix, (Hoeg, 1993), or about 8,700 tonnes of bitumen over the two-year construction period. Assuming the core is placed in six months of each year, about 24 tonnes of bitumen would be required for each day of placement. It would be necessary to store bitumen on site in advance of core placement, to ensure the continuity of supply. Bitumen could be hauled, in small unit loads that could be handled on the temporary access road, while the diversion tunnel is being constructed.

4 ROLLER COMPACTED CONCRETE (RCC) DAM

4.1 Dam Geometry

Figures 3 and 4 show the plan and sections of the RCC Dam. The RCC dam is assumed to have a 10 m crest width (same as for the ACRD), vertical upstream face and downstream slope of 0.7H:1V. The entire dam will be founded on bedrock; all overburden will be removed prior to construction. The steep slopes near the deep canyon along the dam axis will be flattened to 45 degrees. The bedrock foundation close to the surface will be improved with consolidation grouting to shallow depth. A grout curtain will be installed just in front of the upstream face of the dam from a plinth to minimize foundation seepage. The RCC dam will include upstream and downstream facings with conventional concrete or grout-enriched RCC to improve its constructability and durability and to minimize seepage through the dam.

A 2.5 mm thick impervious polyvinyl chloride (PVC) geomembrane, bonded to a geotextile backing (such as that manufactured by Carpi Tech SA of Switzerland and Italy) will be installed on the upstream face of the dam to prevent seepage through the dam and to minimize interaction of the low pH water with the concrete. This membrane has been in place on dams in the Italian Alps for 40 years and has shown negligible deterioration due to UV exposure. However, we are not aware of any long term testing in aggressive chemical environments. Carpi's suppliers have provided test data for up to two years in a variety of aggressive environments including hydrochloric acid with pH much lower than 3.5. The suppliers have advised that for pH 3.5 to 5.5 they are confident in the PVC performance and will provide their standard warranty, which we understand is for 10 years. Carpi has developed a methodology for underwater placement of the PVC; this would be of benefit should the liner have to be replaced (or a new membrane added over top of the existing) in the future. Drainage galleries will be provided in the dam, extending into adits in the abutments.

The geotextile backing on the PVC provides a means to drain any leakage that could develop through the membrane. The dam face can be zoned, that is, sections of the geotextile can be drained into the galleries at discrete locations; if leakage is detected, the general area of the leak will be known.

The crest elevation of the RCC dam is assumed the same as for the ACRD (i.e. EL 716 m) for this options comparison. The alignment of the upstream face of the RCC dam matches the axis of the ACRD. This will increase the storage capacity of the reservoir slightly compared to the ACRD. A stepped spillway corresponding to spillway invert elevation of EL. 706 m can be incorporated into the RCC dam as an integral spillway; a separate spillway on the left abutment is not required.

Aggregate for the RCC will be from a non-reactive source, possibly the monzonite at the mine site. Alternatively, some aggregate testing has been performed on an alluvial deposit downstream of the dam site, at the confluence of Ted Morris and Sulphurets Creeks, and aggregate could be sourced from this area.

The diversion tunnel for the RCC dam would be about 400 m long. With the relatively small dam footprint, the upstream portal can be located downstream of the nearby avalanche chute. The

downstream portal would be between the main dam and seepage collection dam, similar to the arrangement for the ACRD.

4.2 Mix Design

Based on recent projects, in particular projects in northern climates, we assume that an RCC mix with 90-day strength of 25 MPa and maximum aggregate size of 50 mm will be as presented in Table 5. Although the RCC is lacking some common requirements for freeze-thaw resistance such as entrained air, it has historically performed well in northern climates and at high altitude.

Table 5. Mix Design for RCC

Cement kg/m ³	Fly Ash kg/m ³	c + f kg/m ³	Water kg/m ³	w/(c + f)	Coarse Aggregate kg/m ³	Fine Aggregate kg/m ³	Total kg/m ³
115	55	170	122	0.72	1280	950	2522

The conventional vibrated concrete (CVC) at the dam faces will meet the following criteria in order to achieve adequate freeze-thaw resistance (CSA 2009):

- Maximum water to cement ratio (w/c) of 0.5;
- Minimum 28-day strength of 30 MPa;
- Air content of 4% - 7%; and
- Total of 7 days curing time with temperature above 10°.

The recommended CVC facing mix proportions for the concept layout are summarized in Table 6. They are based on those used for the Lake Roberson project (Robitaille and Beaulieu, 1995) in Quebec, which experiences relatively cold winters.

Table 6. Mix Design for Conventional Concrete Facing Mix

Cement kg/m ³	Fly Ash kg/m ³	c + f kg/m ³	Water kg/m ³	w/(c + f)	Coarse Aggregate kg/m ³	Fine Aggregate kg/m ³	Total Mass kg/m ³	Slump mm	Air %
224	96	320	143	0.45	1200	718	2381	20 - 60	4 - 7

The cement would be sulphate resistant type, to provide some protection in the acidic environment.

4.3 Stability Assessment

The static and seismic stability of the RCC dam was assessed using the US Army Corps of Engineers (USACE, 1995) guidelines. The maximum height section of the dam was analyzed.

An average bedrock slope of 5% dipping in the downstream direction was conservatively assumed in the stability assessment. The interface friction angle between the dam and the foundation bedrock was taken as 45 degrees based on the review of limited available information on the foundation bedrock.

The static stability of the dam was assessed under the maximum normal operating water level at El. 665 m and under 200-year wet year maximum water level at El. 706 m. The seismic stability was assessed for the maximum normal operating water level at El. 665 m. These water levels are same as those used in the stability assessment of the ACRD. No tailwater or silt load was assumed in the stability checks. The ice load on the dam will have negligible impact on the factor of safety and thus is not included. The uplift force acting on the dam was calculated assuming that the drainage gallery will be 50% efficient as recommended by USACE (1995). It was assumed that the drainage gallery will be located 5 m above the base of the dam, at a horizontal distance of 8 m from the upstream face.

The Maximum Design Earthquake (MDE) peak ground acceleration (PGA) was taken as 0.14 g corresponding to 10,000 year return period earthquake. In the seismic stability assessment, the horizontal seismic coefficient acting on the dam was taken equal to the PGA.

Table 7 summarizes the factors of safety from the stability analysis against the sliding and overturning failure modes.

Table 7. Summary of RCC Dam Stability Analyses

Failure Mode	Static or Seismic	Factor of Safety	
		Maximum Normal Water Level El. 665 m	200-Year Wet Year Maximum Water Level El. 706 m
Sliding	Static	2.9	1.6
	Seismic	1.8	-
Overturning	Static	>2.0	2.0
	Seismic	2.4	

Factors of safety exceeding 1.5 against both sliding and overturning under both static and seismic loading conditions are considered adequate for the RCC dam. Given the relatively high factors of safety, the dam volume could be decreased by steepening the downstream slope slightly. However, that refinement has not been made for this conceptual level arrangement.

4.4 Quantities and Cost Estimate

Table 8 lists the estimated quantities and conceptual level construction unit rates and cost estimate for the RCC Dam. The unit rate for RCC is based on experience with recent projects.

Table 8. Quantities and Construction Cost Estimate for RCC Dam

Item	Description	Quantity	Unit	Unit Cost	Total Cost
1	Foundation Preparation				
	Logging, Grubbing, Clearing and Stripping		LS	\$ 150,000	\$ 150,000
	Foundation Overburden Sub-excavation for Dam and Grout curtain	781,000	m ³	\$ 9	\$ 6,638,500
	Bedrock Shaping and Trimming	20,800	m ³	\$ 25	\$ 520,000
2	Dam Construction				
	2.1 RCC Concrete	2,613,000	m ³	\$ 155	\$ 405,015,000
	2.2 Conventional Concrete Facing (Upstream)	17,900	m ³	\$ 310	\$ 5,549,000
	2.3 Conventional Concrete Facing (Downstream)	31,700	m ³	\$ 310	\$ 9,827,000
	2.4 Reinforced Concrete on Crest	1,950	m ³	\$ 600	\$ 1,170,000
	2.5 Upstream CARPI Membrane	51,000	m ²	\$ 275	\$ 14,025,000
	Total				\$ 442,900,000

The RCC dam will be founded entirely on bedrock. The volume of overburden requiring excavation was estimated using the existing drill hole and seismic survey data. The overburden excavation includes the entire width of the RCC dam, plus allowances for the upstream plinth and a working area upstream and downstream.

The cost estimate presented Table 8 is for the construction cost of the dam only. It does not include the following items:

- Cost of construction of integral spillway and its associated components, located on the downstream slope of the dam.
- Cost for instrumentation (this is expected be of similar magnitude as for the ACRD); and
- Cost for diversion structures required for construction.

As the RCC dam can be allowed to overtop and due to increase in the storage volume (with the upstream RCC dam face at the same location as the ACRD centreline, the height of the RCC dam can be reduced. If the RCC dam crest elevation is reduced to El. 709 (with freeboard of 3 m for the 200-year wet year maximum water level of El. 706 m), the construction cost can be reduced by approximately \$40 million to \$403 million.

The RCC dam crest width is assumed to be 10 m, the same as the ACRD (this width is required for the ACRD, in part to accommodate the construction equipment). However, the RCC dam crest could be constructed narrower if there is not a reason for the width. Each metre reduction in crest width would save about \$8 million.

Since the RCC dam will not require a separate spillway on the left abutment, there is a potential additional saving of about \$8 million for the RCC dam option; some of this would be offset by incremental costs for spillway walls on the dam. As noted earlier, the diversion tunnel could be about 800 m shorter, resulting in a saving in the order of \$4 million. Also as noted earlier, the downstream slope could be steepened somewhat and still satisfy stability criteria.

The total cost for RCC, about \$443 million, appears to be significantly higher than that for the ACRD. All economies discussed above would reduce the RCC cost to about \$360 million, still well above the estimated \$235 million for the ACRD alternative. The cost difference results because the RCC dam would require a specialist contractor, requires substantial quantities of cementitious material from off site, and cannot take advantage of available mine equipment for construction except, perhaps, for trucks to haul non-reactive rock to the dam vicinity for aggregate manufacture.

4.5 Schedule

Efficient RCC placement will require careful planning and preparation. Sustained placement rates in excess of 10,000 m³/day have been achieved on several large dams (Dunstan, 2012). This usually requires advance preparation and storage of aggregate, as well as storage of significant quantities of cement and fly ash to ensure reliability of supply. For preliminary planning purposes, an average rate of 6,000 m³/day would result in 435 days or about 17 working months. Placement will not be possible during sustained freezing conditions. Assuming that placement can occur for six months per year, a three year (three full seasons) construction schedule would be required. We assume that the foundation preparation would take place while the diversion tunnel is being constructed. The grout curtain would be installed from a plinth at the upstream heel of the dam, so as to not interfere with RCC placement.

RCC dams are constructed with joints at regular intervals, commonly 20 m to 40 m along the dam axis, so that the shrinkage due to concrete cooling takes place at defined locations, rather than generating random cracks in the dam body. These construction joints contain water stops and drains near the upstream face, to prevent seepage through the dam. Rather than raise the entire dam from bottom to top, the dam can be constructed in blocks (“monoliths”); the ends of a monolith would be formed, and they would be locations for construction joints when the adjacent RCC is constructed. For the WSD, the RCC monoliths on the abutments could be constructed while the diversion tunnel is under construction, leaving the highest dam section in the creek to be constructed after diversion. However, it is unlikely that a full season could be saved in the schedule as foundation preparation would have to occur prior to RCC placement.

Large quantities of cement and fly ash would be required for RCC dam construction. The permanent road to the site would have to be in place to permit efficient delivery of the materials. For the RCC mix with cementitious (cement plus fly ash) content of 170 kg per cubic metre of RCC, over 440,000 tonnes of cementitious material would be required. An additional 16,000 tonnes would be required for the conventional concrete facing. The total cementitious is equivalent to about 950 tonnes per day during the placement season. The permanent access road will be required to achieve this delivery rate. The materials would be delivered in advance and stockpiled, but the quantity is such that daily delivery rates would still be large and the permanent access roads would have to be in place.

5 COMPARISON OF ALTERNATIVE OPTIONS

Each option offers benefits compared to the other, with respect to design, construction, maintenance and cost. These are described in Table 9 following this report. They may be summarized as:

5.1 Design Issues

5.1.1 ACRD Alternative

- Asphalt is known to have a long life and to be resistant to acidic environments. The ACRD can be constructed using local rockfill; only the asphalt aggregate and transition zones will require manufacture; other zones will be taken directly from the quarry (or from mine waste) to the dam. The spillway would be a separate structure; however, the excavated rock could be placed in the dam fill.

5.2 RCC Alternative

- The long term acid resistance of the PVC facing membrane of the RCC dam is unknown. However, the PVC would be exposed where it can be monitored and, if necessary, replaced. All aggregate for construction would have to be manufactured. Concrete, even when manufactured with sulphate-resistant cement, cannot withstand long term exposure to acidic conditions with pH as low as 3.5. The spillway could be located on the dam.

5.3 Construction Issues

5.3.1 ACRD Alternative

- The ACRD would require a longer diversion tunnel.
- The dam would be constructed largely of unprocessed rockfill. Rockfill can be placed in freezing conditions. The asphalt core can be placed to a few degrees below freezing, with appropriate modifications to the equipment.
- The 2.0 m wide asphalt core width at the base of the dam will require a purpose-built paving machine. Currently, the maximum constructed asphalt core width to date is 1.6 m.
- North American experience with asphalt core dams is very limited. The 15 m high Nemiscau I dam was constructed in 2008 by Hydro Quebec as a demonstration project in a region where glacial till, the typical core material for embankment dams in Quebec, is not available. Currently five ACRDs between 26 m and 109 m height are under construction for Hydro Quebec's Romaine II project. Only one contractor in North America has ACRD experience. However, we are confident that the ACRD could be constructed at KSM by including European-based expertise into the contracting group or Owner's construction team, depending on the delivery method.

5.3.2 RCC Alternative

- The diversion tunnel for the RCC dam would be shorter.
- The RCC dam would require a three year construction period. Portions of the RCC dam on the abutments could be constructed while the diversion tunnel is being excavated; however, this will not reduce the construction time to two years. The highest dam section in the creek would be constructed after diversion.
- The permanent mine access road would have to be in place in order to haul the large quantities of cement and fly ash required. RCC cannot be placed in sustained freezing temperatures.

5.4 Cost

5.4.1 ACRD Alternative

- The order of magnitude cost for the construction of ACRD based on using large mine equipment is \$128 million (for the components that were costed). Some of the unit rates do not appear to be appropriate for reasons outlined in Section 3.3. The construction cost using more realistic unit costs will be in the order of \$235 million. The ACRD is cost effective, taking advantage of mine equipment and local materials, including rockfill that would otherwise be waste from the mine.

5.4.2 RCC Alternative

- The order of magnitude cost for the construction of RCC dam is \$443 million. This cost assumes that the crest elevation of the RCC dam will be the same as that for the ACRD dam (i.e. El. 716 m). However, since the RCC dam can be allowed to overtop, the dam crest can be lower. If the crest elevation of the RCC dam is lowered by 7m to El. 709 m, the construction cost will reduce to about \$400 million. A further reduction is likely possible by reducing the crest width and steepening the downstream slope of the dam, and there would be some savings in the shorter diversion tunnel and elimination of a separate spillway, but the cost of the dam is still unlikely to approach that of the ACRD.
- The upstream PVC will have to be replaced after some period of time as a regular maintenance item/cost. The frequency of replacement cannot be accurately estimated but may be in the order of 25 to 50 years, depending on the aggressiveness of the contact water.

5.5 Risks

5.5.1 Schedule Risk

- There is a risk of not meeting the construction schedule and resulting consequential delay of mine development. We understand that the dam is to be constructed in approximately two years. Careful planning will be required for either alternative dam. Early dam construction will

require the early construction of the road, to a standard acceptable for hauling large quantities, from the non-reactive monzonite source to the dam site. In the case of the RCC dam, the main access road is required to ensure reliability of supply of the large quantities of cementitious material.

5.5.2 Performance Risk

- Leakage through and beneath the dam should be contemplated in all dams and appropriate measures put in place to mitigate the risk. Leakage most commonly develops due to construction defects. For both the ACRD and RCC dam, leakage through the dam would be delivered via drains/galleries or the blanket drain to the seepage collection dam reservoir.
- The risk of seepage through the dam foundation is similar for the two alternatives. Both dams would include drainage adits in the abutments to intercept and monitor seepage. The adits and galleries also provide a means to grout the foundation, should it be found necessary in the future. All leakage intercepted by the drainage system would be delivered to the seepage collection dam reservoir.

6 CONCLUSIONS AND RECOMMENDATION

The ACRD will incorporate an asphalt core as the water-retaining component of the dam. Asphalt is known to survive for hundreds of years. It is resistant to acidic environments. In contrast, the long term performance of PVC in acidic environments is unknown, while concrete is known to suffer damage in the long term in such environments.

The ACRD will be constructed almost entirely of onsite materials (rockfill, processed rockfill, aggregate). Bitumen for the asphalt core would be delivered in advance, in order to stockpile a sufficient quantity to not impede construction progress. Asphalt can be placed in freezing conditions, with appropriate equipment. Rockfill can be placed in freezing conditions. The ACRD could be completed in two construction seasons. On the other hand, construction of the RCC dam would require that the permanent site access road be completed, in order to deliver the large quantities of cement and fly ash that would be required. RCC cannot be placed in freezing conditions. RCC dam construction would take three seasons. Although sections (monoliths) of the RCC dam can be constructed on the abutments prior to river diversion, it is unlikely that a full season could be saved. The RCC dam will take longer to construct than will the ACRD.

The comparison of asphalt core rockfill dam and roller compacted dam indicates that the construction cost of the former is significantly lower. The use of mine equipment for dam construction is a significant benefit in regard to cost. In addition, since the rockfill dam is constructed of local materials it is not necessary to have the permanent road in place, provided that the construction equipment can be mobilized over the temporary (winter) road.

Leakage through or under either dam could be monitored via drain holes into the seepage adits and, for the RCC alternative, galleries. These openings would provide access for additional grouting, should the seepage rate increase over time. The galleries and adits would direct all intercepted seepage through the dam and through the shallow foundation beneath the dam to the reservoir formed by the seepage collection dam.

Although the RCC dam offers several benefits compared to the ACRD dam – spillway on the dam; possibility of lowering the dam; shorter diversion tunnel – these are not adequate to offset the unknown long term performance of the RCC dam, longer construction schedule, and cost. It is KCB's recommendation that the ACRD alternative should be selected as the preferred option for the Water Storage Dam.

7 LIMITATIONS OF REPORT

This report is an instrument of service of Klohn Crippen Berger Ltd. and has been prepared for the exclusive use of Seabridge Gold Inc. for specific application to the Kerr-Sulphurets-Mitchell Project. The contents of this report may not be relied upon by any other party without the express written permission of Klohn Crippen Berger. The material in this report reflects best judgment of Klohn Crippen Berger in light of the information available at the time of preparation. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Klohn Crippen Berger Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

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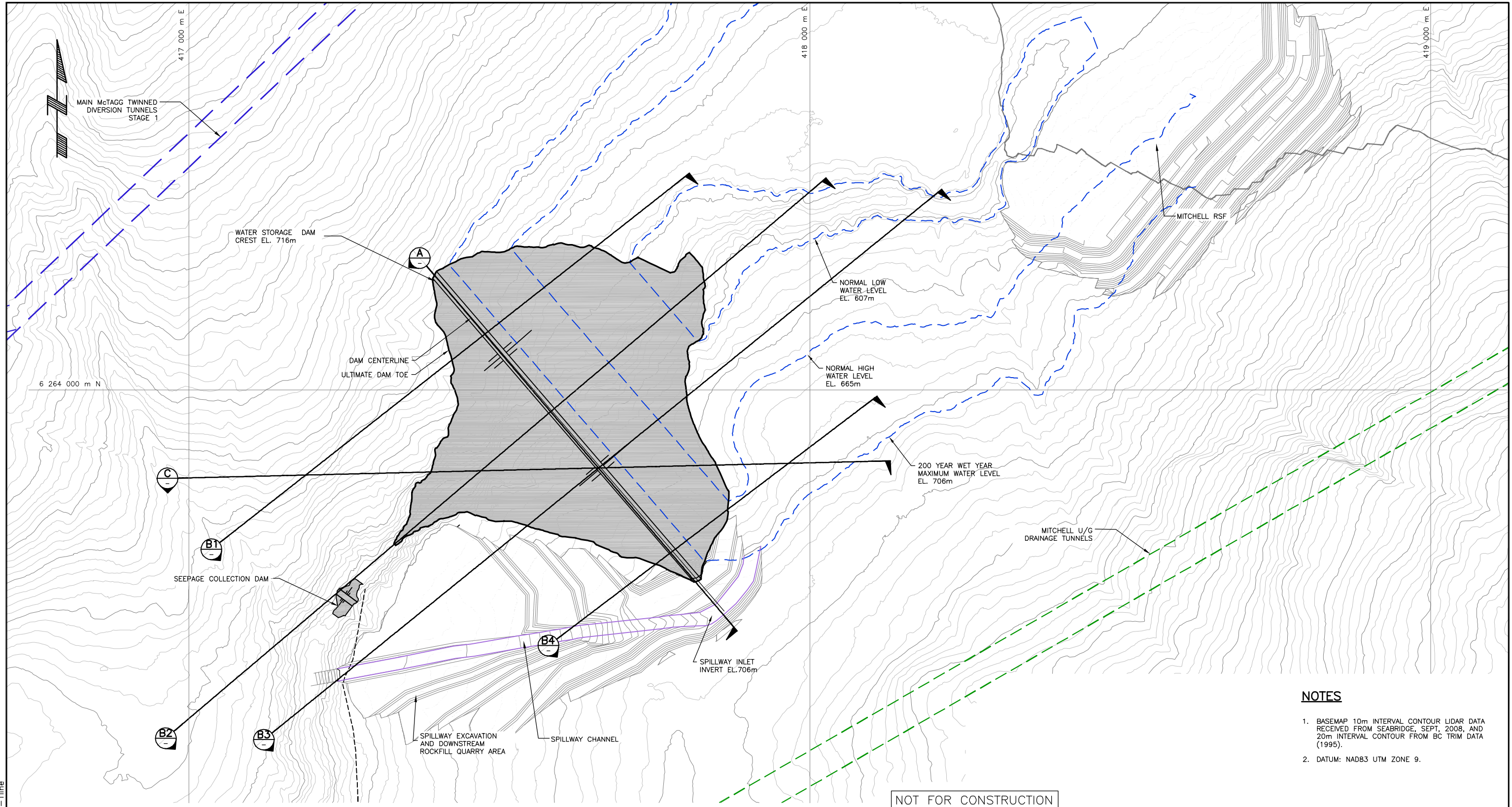
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FIGURES

- Figure 1 Water Storage Dam, Asphalt Core Rockfill Dam Option - Plan
- Figure 2 Water Storage Dam, Asphalt Core Rockfill Dam Option – Sections
- Figure 3 Water Storage Dam, RCC Dam Option, Plan and Upstream Elevation
- Figure 4 Water Storage Dam, RCC Dam Option, Section and Details

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 Drawing File: Z:\P\VCR\PO9819A01_SBR - KSM Water Storage Dam\400 Drawings\Fig 1 Plan Asphalt Core Earthfill Dam.dwg (jinaco)
 Xref File(s): contours; Design
 Image File(s): Seabridge_Gold_RGB-1line




NOTES

1. BASEMAP 10m INTERVAL CONTOUR LIDAR DATA RECEIVED FROM SEBRIDGE, SEPT, 2008, AND 20m INTERVAL CONTOUR FROM BC TRIM DATA (1995).
2. DATUM: NAD83 UTM ZONE 9.

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		TITLE WATER STORAGE DAM ASPHALT CORE ROCKFILL DAM OPTION - PLAN	
		PROJECT No. P09819A01	FIG. No. FIG. 1

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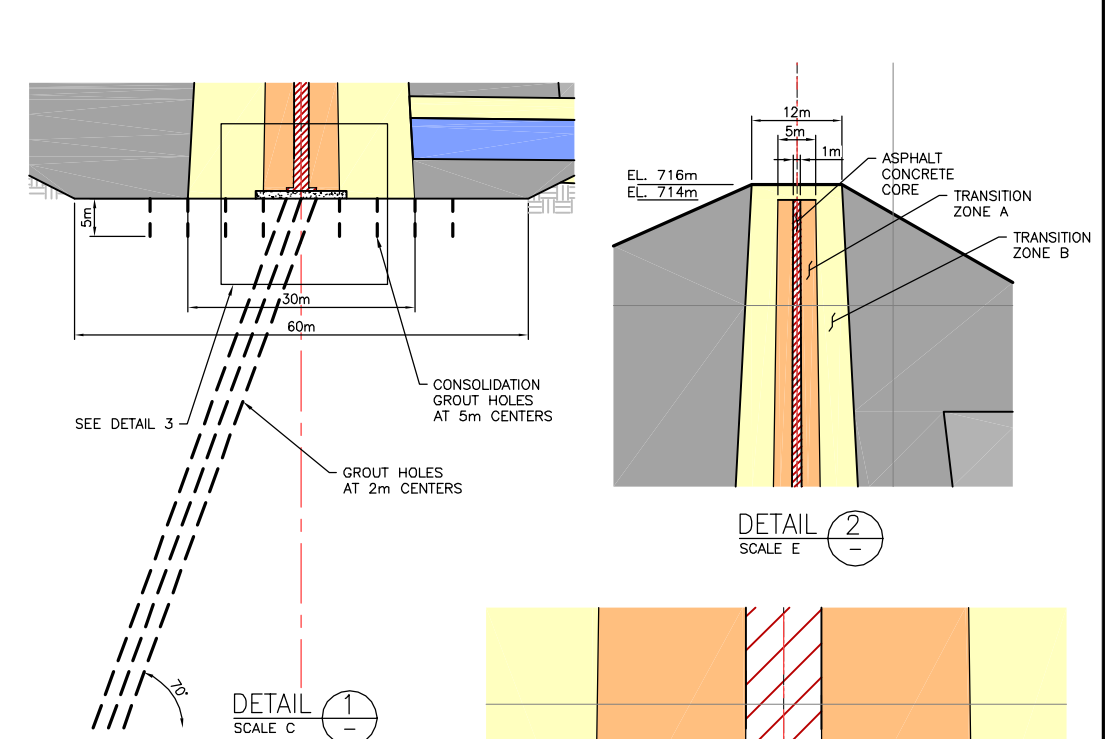
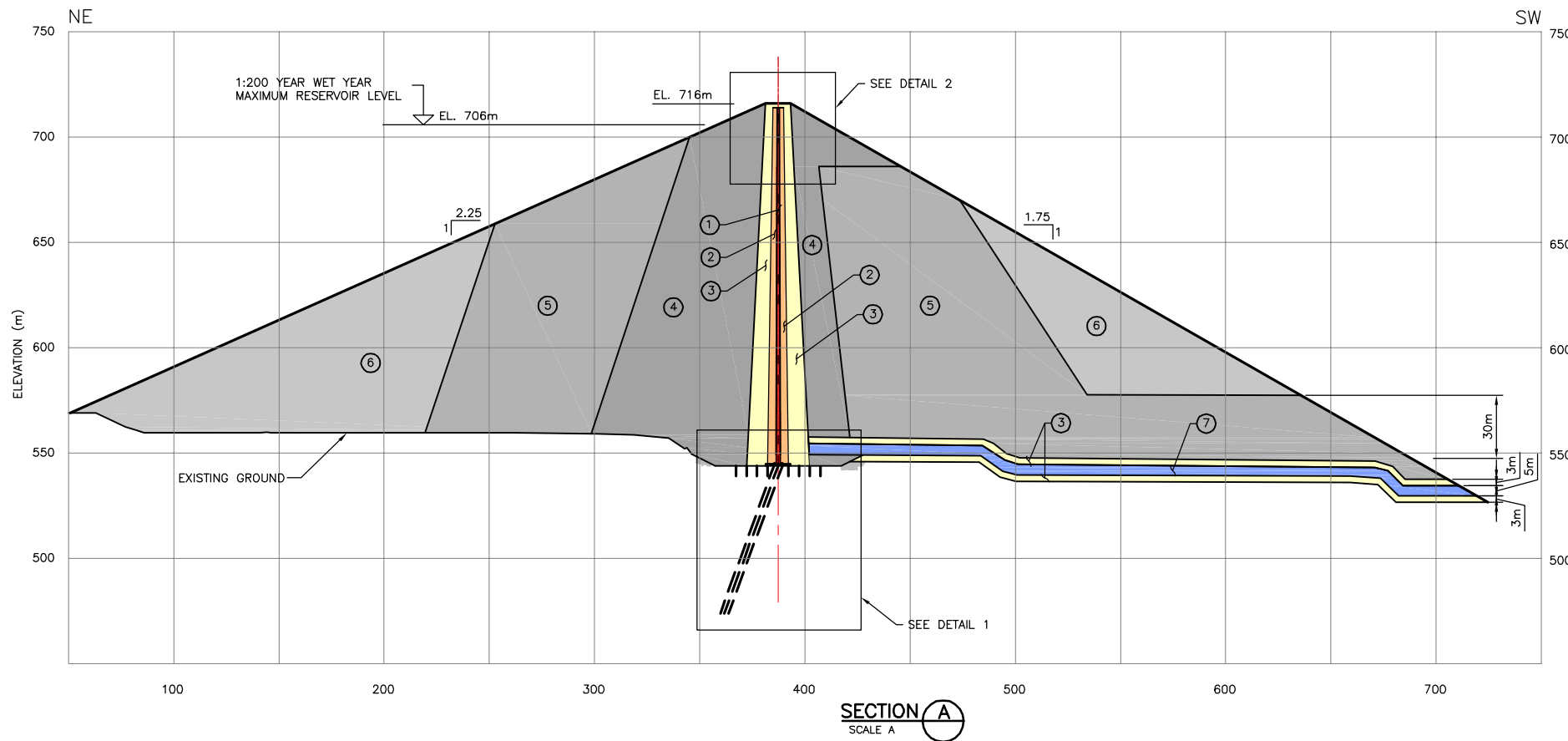
Date: 12/17/2012

Scale: 1:2,584g(PS)

Drawing File: Z:\P\VC\P09819A01_SBR - KSM Water Storage Dam\400 Drawings\Fig 2 Sections Asphalt Core.dwg (jinaco)

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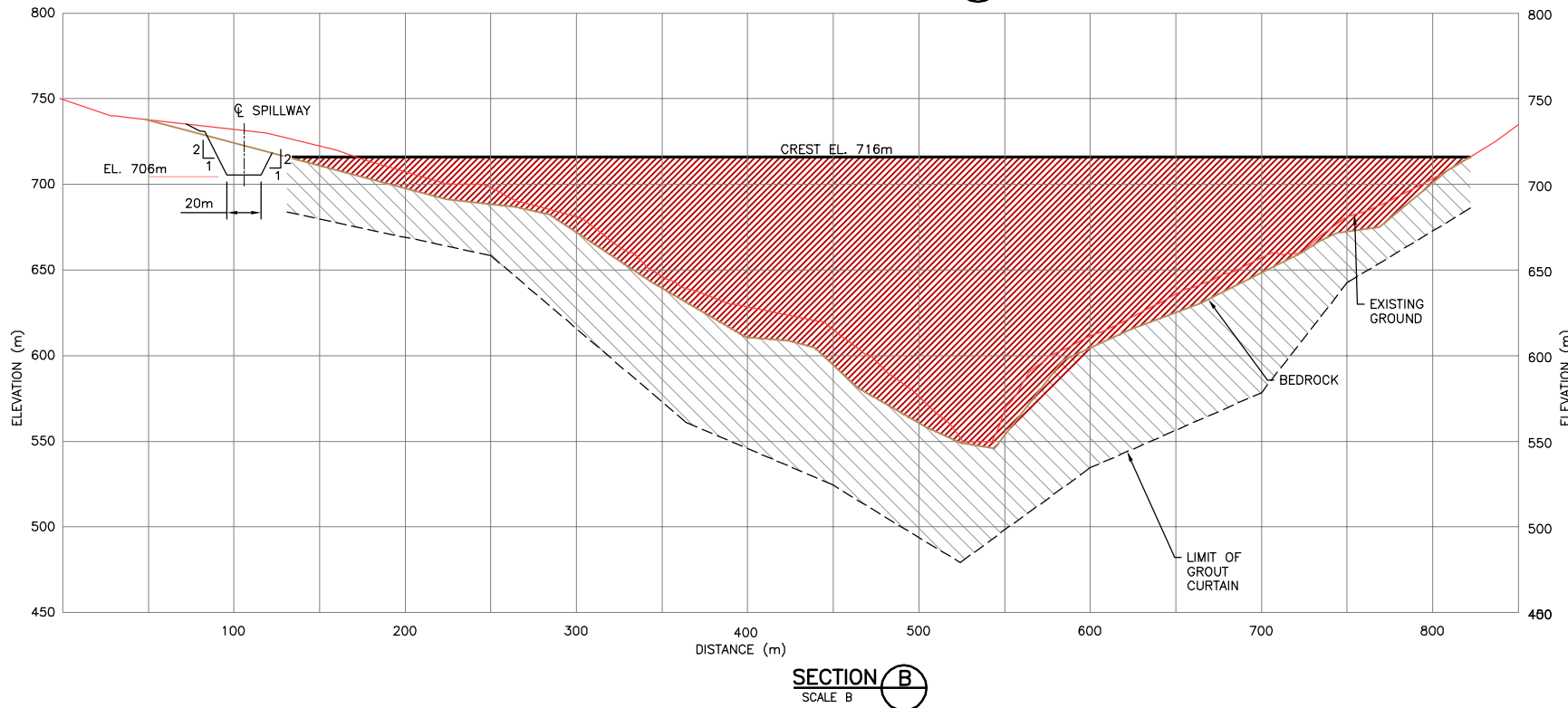
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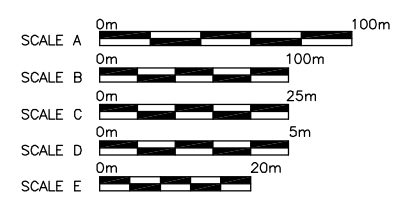
NOTES

- SEE FIGURE 1 FOR LOCATION OF SECTIONS A AND B.
- GROUT CURTAIN AND SPILLWAY DETAILS ARE SHOWN FOR ILLUSTRATION ONLY AND MAY NOT BE COMPLETE.
- DRAINAGE ADITS AND TUNNELS IN THE FOUNDATION ARE NOT SHOWN.

ZONE	MATERIAL
①	ASPHALT CONCRETE CORE
②	TRANSITION ZONE A OF GRAVEL (0-60MM)
③	TRANSITION ZONE B OF CRUSHED ROCK (0-150MM)
④	WELL COMPACTED QUARRIED NON-REACTIVE ROCK (0-700MM)
⑤	WELL COMPACTED QUARRIED ROCK (0-700MM)
⑥	WELL COMPACTED QUARRIED ROCK (0-1000MM)
⑦	DRAIN



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<p>AS A MUTUAL PROTECTION TO OUR CLIENT, THE PUBLIC AND OURSELVES, ALL REPORTS AND DRAWINGS ARE SUBMITTED FOR THE CONFIDENTIAL INFORMATION OF OUR CLIENT FOR A SPECIFIC PROJECT AND AUTHORIZATION FOR USE AND/OR PUBLICATION OF DATA STATEMENTS, CONCLUSIONS OR ABSTRACTS FROM OR REGARDING OUR REPORTS AND DRAWINGS IS RESERVED PENDING OUR WRITTEN APPROVAL.</p>	<p>CLIENT</p> <p>SEABRIDGE GOLD</p>	<p>PROJECT</p> <p>KSM PROJECT</p>
	<p>Klohn Crippen Berger</p>	<p>TITLE</p> <p>WATER STORAGE DAM ASPHALT CORE ROCKFILL DAM OPTION - SECTIONS</p>
<p>PROJECT No.</p> <p>P09819A01</p>	<p>FIG. No.</p> <p>FIG. 2</p>	

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Time: 14:34:13

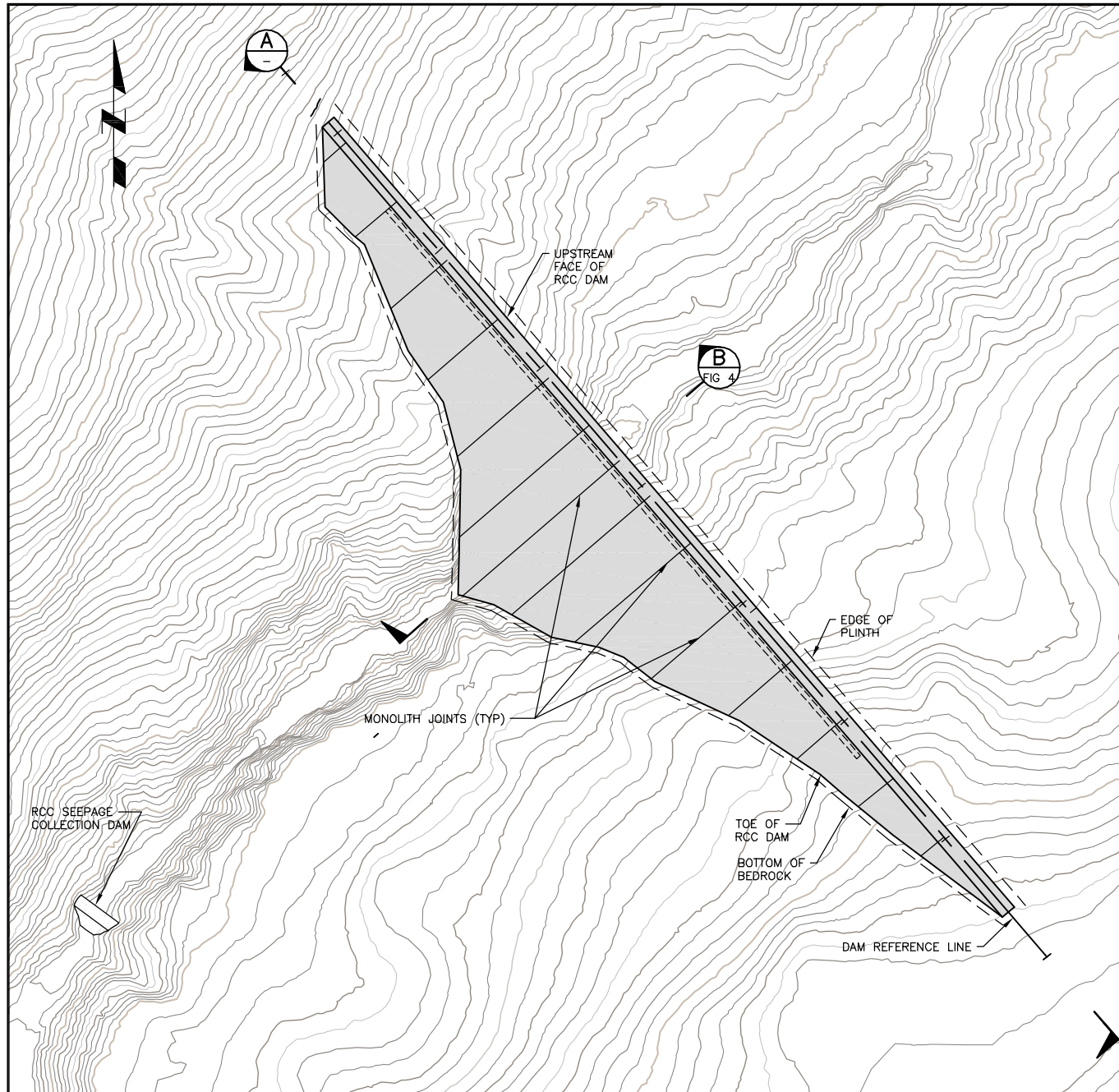
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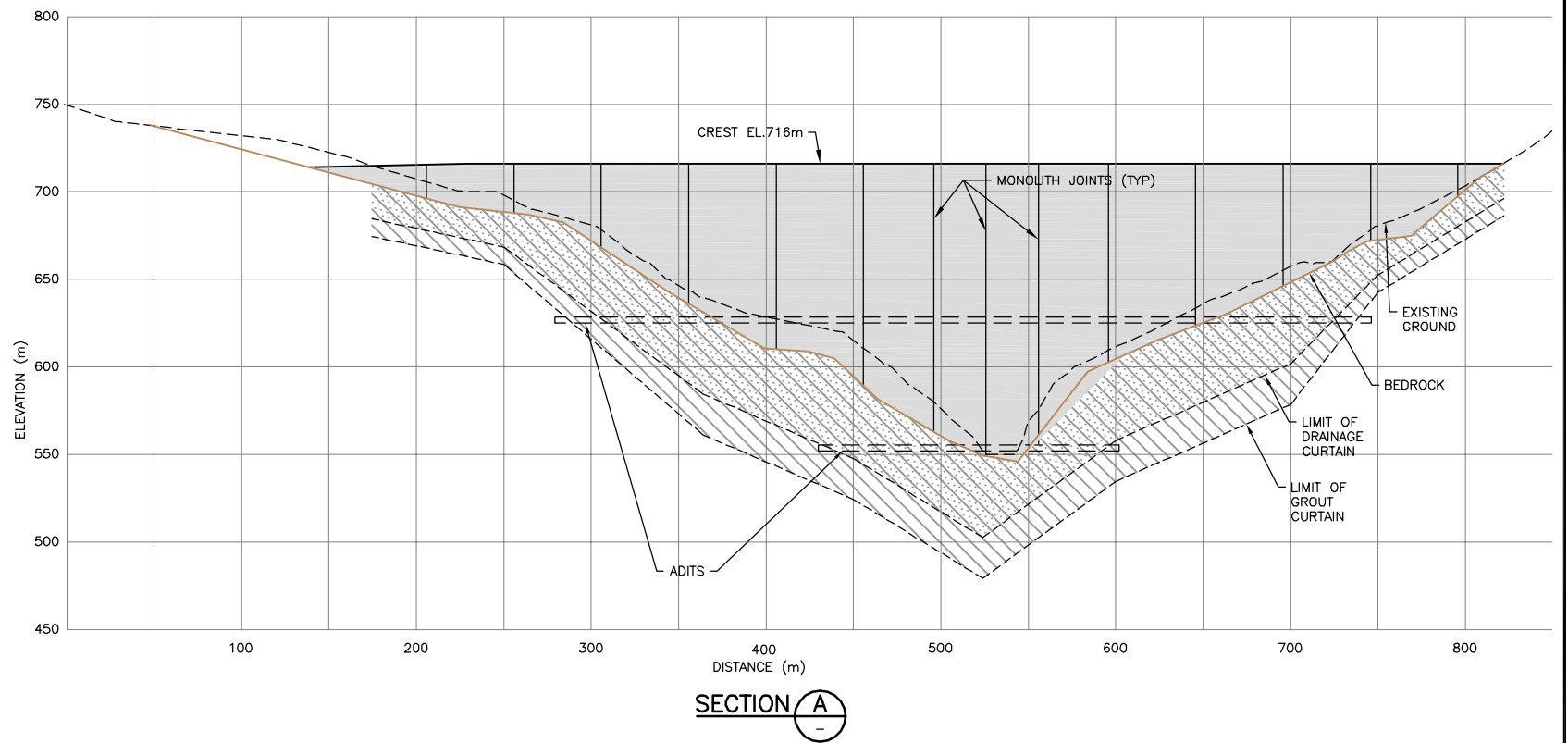
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Xref File(s): WSF dam section; Design

Image File(s): Seabridge_Gold_RGB-1line



PLAN AT TOP OF RCC
1:2000



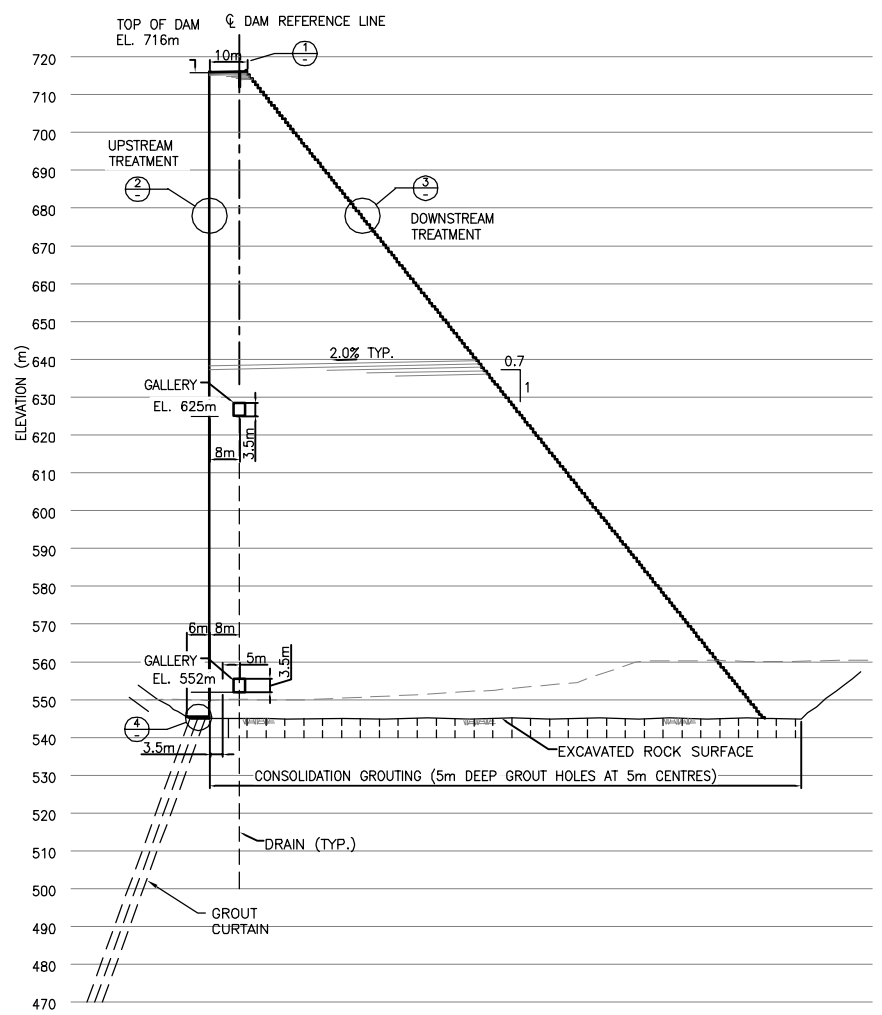
NOTES

1. INTEGRAL SPILLWAY ON THE DAM NOT SHOWN.
2. SEEPAGE COLLECTION DAM, GROUT CURTAIN, DRAINAGE CURTAIN AND DRAINAGE ADITS ARE SHOWN FOR ILLUSTRATION ONLY AND MAY NOT BE COMPLETE.
3. DETAILS OF PVC GEOMEMBRANE ON THE UPSTREAM FACE NOT SHOWN.
4. DRAINS ON THE DAM ARE NOT SHOWN.

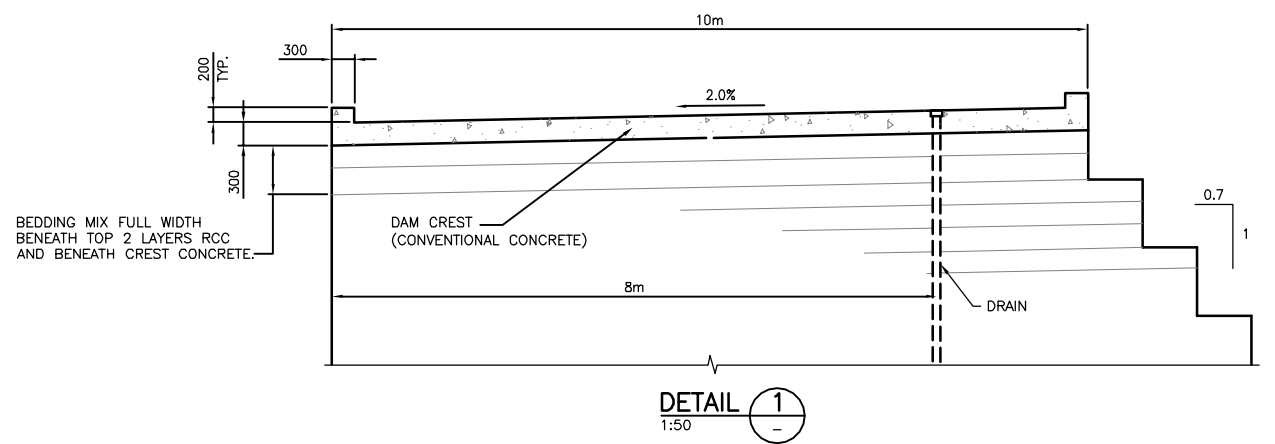
NOT FOR CONSTRUCTION

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	<p>Klohn Crippen Berger</p>	<p>TITLE</p> <p>WATER STORAGE DAM RCC DAM OPTION PLAN AND UPSTREAM ELEVATION</p>
<p>PROJECT No.</p> <p>P09819A01</p>	<p>FIG. No.</p> <p>FIG. 3</p>	

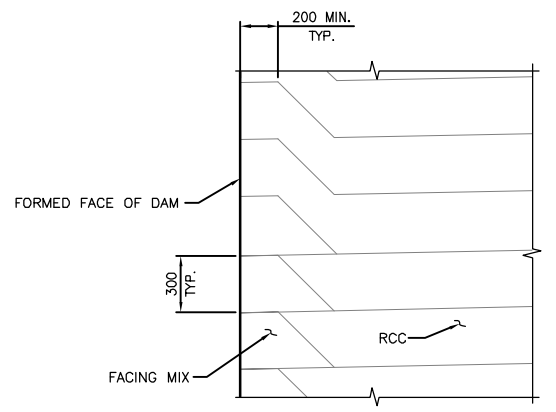
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SECTION B
SCALE 1:1000 FIG. 3

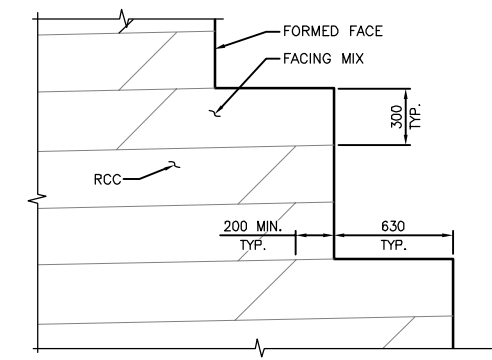


DETAIL 1
1:50



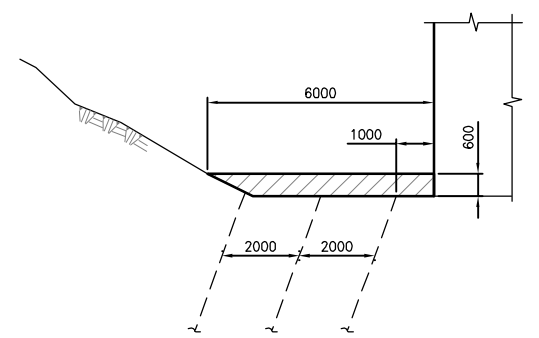
UPSTREAM FACE TREATMENT

DETAIL 2
1:20



DOWNSTREAM FACE TREATMENT

DETAIL 3
1:20



DETAIL 4
1:100

NOT FOR CONSTRUCTION

NOTES

1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS NOTED OTHERWISE.
2. SEE FIG. 3 FOR LOCATION OF SECTION.
3. DETAILS OF GEOMEMBRANE ON THE UPSTREAM FACE NOT SHOWN.
4. DRAINAGE ADITS AND DRAINS ON THE DAM ARE NOT SHOWN.

Time: 17:56:29
Date: 11/9/2012
Scale: 1:2,584g(PS)
Drawing File: Z:\VCR\PO9819A01 SBR - KSM Water Storage Dam\400 Drawings\0003.dwg (jtinaco)
Xref File(s):
Image File(s): Seabridge_Gold_RGB-1line

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	<p>Klohn Crippen Berger</p>	<p>TITLE</p> <p>WATER STORAGE DAM RCC DAM OPTION SECTION AND DETAILS</p>
<p>PROJECT No.</p> <p>P09819A01</p>		<p>FIG. No.</p> <p>FIG. 4</p>

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