2013Google, Imagery Mar 29, 2009

January 21, 2014

Middle and Lower Chase River Dams Progress Update

Golder Associates

Agenda

- Data Review Findings
  - Review of Historical Information
  - Seismic Hazard Assessment Review
  - Performance of Concrete Dam Core
  - Hydrology Study Review
- Data Gaps and Design Unknowns
- Dam System
- Risk Assessment Update
- Conceptual Design Options
- Next Steps
  - Additional Investigations
  - Geophysics Update
  - Additional Studies

Data Review Findings – Middle Dam

Early years

- 1888: Built by logs to supply water for the city?
- ~1910: concrete dam constructed by Western Fuel Company/ Wellington Colliery/ Harewood Colliery for coal wash water ?
- Former presence of railway line/ spur lines from Harewood Mine to Coal wharves suggests dams built with rock fill <0.6 m dia. from mine. Photos of the rock fill on the upstream and downstream side of the Middle Chase Dam during the 1980 rehabilitation works, indicates the rock fill particle size is generally <0.6 m dia.
- ~1950: Additional fill material added to DS face by end dumping with little/no compaction -Debris (car parts) limited presence.
- Nov 1955: Flooding (heavy rain) problems on Chase River likely occasion that prompted hole made in dam to increase discharge capacity of reservoir.

1976 and 1977 (Dam Inspections)

- Seepage on right abutment contact 3-5 cfs and seepage at DS toe of 2-3 cfs
- Serious piping may be present in the center of the embankment or LLO.
- Underwater inspection – heavy silt conditions. 2” pipe found-old valve stem?

Data Review Findings – Middle Dam

<u>September 1, 1976 Inspection</u>

Typical Reinforcing used in dam – car springs, railway rails, drill steel

Seepage along right abutment contact – scap material in embankment

Data Review Findings – Middle Dam

1978 (Dam Investigation – Golder)

- Test pit (TP) and borehole (BH) investigation (Golder – basis of 1980 work)
- Top soil and bedrock in 2 TPs. Loose gravel, trace sand and silt and loose to compact sand and gravel some clayey silt, cobbles and boulders in 2 TPs.
- Loose to compact sand and gravel, cobbles and boulders (fill) followed by rock fill encountered in BH investigation.
- Till-like material encountered at 12.5 m depth in 1 BH.

1980 (Dam Investigation and Remediation)

- 300mm high concrete addition placed on upstream face concrete wall
- Hole in concrete core (from 1955) patched
- Saturated material was removed and the 3’ thick drainage blanket was placed, shot rock buttress placed ontop
- Digging continued until a intact log crib was exposed. LLO not located.
- Replacement of DS fill in 12-18” lifts. Compacted and verification testing.
- Heavy seepage at DS toe at temporary weir. Drainage trench to intercept seepage.
- New concrete spillway walls and new walkway over spillway.

Beginning of fill replacement – note vibratory compaction in left foreground of photograph

August 22, 1980 Inspection

Original log crib exposed – logs were still intact

Concrete plug from upstream face after stripping

Data Review Findings – Middle Dam

<u>1981 and 1982 (Dam Inspections)</u>

- Seepage noted at DS toe both years

<u>1983 to 1986 (EBA Assessment of Increase in Seepage 1993)</u>

- City records indicate depth of water over weir 25 to 32 mm. -> 3.7 to 5.1 l/s

<u>1985, 1986 and 1987 (Dam Inspections and Improvements)</u>

- Concrete spillway wall extension to direct flows away from toe and concrete weir replaced temporary weir
- Minor seepage from contact between old and new walls of spillway. Spalling.
- Erosion and undercutting of hillside on left side of spillway. Erosion near toe and along right abutment contact

<u>1992 (Dam Inspection)</u>

- Seepage on right abutment and through right channel wall. Flow through seepage weir (Dec 1) was ~0.3cfs

<u>1993 (Seepage Monitoring and Assessment of Increase in Seepage - EBA)</u>

- Seepage at 2 areas on DS shell near contact with right abutment mid-way up dam. 2 pipes installed to collect water from main seepage and sump installed downhill of pipe discharge to collect material from seepage flow.

Data Review Findings – Middle Dam

1993 (Seepage Monitoring and Assessment of Increase in Seepage - EBA)

- First pipe installed early Sept:
  - Sep 18-Oct 3: flows 0 to 24 l/min, Sept 28- Oct 2: weight of dried sediment 23.1 to 189.5 gms (fine to coarse sand with trace fine gravel)
- Second pipe installed early Oct:
  - Oct 5: flow 1 l/min (first pipe), flow 30 l/min (second pipe)
- Combined flow both pipes decreased end of Oct from total flow of 50 l/min to 20 l/min. Carried material decreased from 20 gms/day to <10 gms/day
- City records (1992-1993) indicate depth of water over weir 32 to 64 mm. -> 5.1 to 14.4 l/s.
- EBA notes general increase in flow over past 10 years
  - EBA suggests deterioration along bedding planes in bedrock
  - Loss of fines from embankment
- Seepage at 3 locations: near DS end of concrete training wall along right side of spillway - bedding plane 0.2 m below this interface, bedding plane approx. mid-height of the embankment at the right abutment contact, approx. rock fill 1.5 m left of the source at the right abutment contact

Data Review Findings – Middle Dam

<u>1995 (Dam Inspection)</u>

- Extension of concrete spillway wall and installation of wall seepage monitoring weir.

<u>1994, 1995, 1996 (City of Nanaimo)</u>

- Sediment and flow test spillway records

<u>1998 (Dam Inspection)</u>

- Visual seepage observation of 2cfs through notch left side of spillway. No seepage in upper drain pipes. Large seepage flow near toe. Gauge at weir – 4”.

<u>2003 (Dam Inspection)</u>

- Profiling sonar bathymetry – possible LLO and valve detected?

<u>2004 (Dam Inspection)</u>

- Installed concrete chamber over seepage weir and installed telemetry equipment to continuously monitor seepage.

Data Review Findings – Middle Dam

2009 (EBA Seismic Hazard Assessment)

- Diver searched for LLO. Inlet ~20 m from dam face, 9 m from right abutment.
- LLO appears to pass below the patched area in concrete (may have been valve stem)
- LLO appears to be located on left abutment of dam, appears to pass beneath the original fill that was left in place during the 1980 excavation.
- Seepage probably from abandoned LLO
- Diver inspection indicated conditions of exposed wood of LLO at the upstream end were very poor – rotted and partially collapsed. Approx. 1 m from inlet, the LLO was encased in an unknown thickness of concrete. Then buried in sediments.

2013 Anecdotal Information (email from Solomon Hunter, January 5, 2014)

- The LLO pipe at the base was thought to be black cast iron probably 30" or 36". The pipe was not visible from the upstream side but was visible on the downstream face (there was no berm behind the dam).
- The valves to operate the pipe were probably inoperable sometime in the 1950's and eventually removed.

Data Review Findings – Lower Dam

Early Years

- 1887: 5.5 m high built by log cribbing to supply water for the city?
- ~1910: concrete dam constructed by Wester Fuel Company/ Harewood Colliery/ Wakesiah Colliery?
- Former presence of railway line/ spur lines from Harewood Mine to Coal wharves suggests dams built with rock fill <0.6 m dia. from mine. Photos of the rock fill on the upstream and downstream side of the Middle Chase Dam during the 1980 rehabilitation works, indicates the rock fill particle size is generally <0.6 m dia.
- 1918 Railway constructed over the Dam, fill added to permit crossing at orientation not parallel to concrete wall
- ~1918 Slag, cinders, sand and gravel very loose to loose fill end dumped on downstream side

1978 Investigation

- Test pit (TP) and borehole (BH) investigation (Golder – basis of 1980 work)
- Loose cinders, slag, sand and gravel fill 4 TPs and 4 BHs. Rockfill 1 TP and 4 BHs. Till-Like Material in 2 TPS and 1 BH.

Data Review Findings – Lower Dam

1978 Investigation (continued)

- Till-like material encountered at 14.9 m depth in BH9.
- No Bedrock encountered below dam.
- Concrete wall 0.3 m thick to 0.6 m below crest and then increase to 1.2 m thick.
- 3 control valves encountered, two near concrete wall third few meters upstream.
- Coarse rockfill, encountered voids (up to 0.3 m in size)
- Assumed abutments founded on till-like material due to 1 borehole in center and two test pits at edge of right abutment
- No Bedrock encountered below dam, dam on steep sided ravine

1980 (Dam Inspection and Improvement)

- 2 LLO plugged with concrete and all valves chambers filled and capped with concrete
- Poor concrete was removed and replaced on spillway, spillway founded on bedrock
- Spillway walls raised by 300 mm

Data Review Findings – Lower Dam

1980 (Dam Inspection and Improvement continued – Photos below)

- Stabilizing sand and gravel toe berm constructed
- Seepage collection trench installed at toe and backfilled with drain rock
- Crack noted in cinders, slag sand and gravel deposit at crest of filter berm and monitoring commenced

1981 (Dam Inspection)

- Erosion observed on left abutment (no inspection or repair report)

Close up view of crack

Stabilizing berm viewed from left abutment

Drain rock toe drain used to pick up embankment seepage

Data Review Findings – Lower Dam

<u>1983 (Dam Inspection)</u>

- Seepage noted on left side of spillway.
- Noted that sandbagging had been required in recent years at intake to prevent overtopping of spillway walls

<u>2003 (Dam Inspection and Improvement)</u>

- Staff gauge installed to measure reservoir level along with precipitation gauge
- Seepage estimated from flow in seepage collection trench

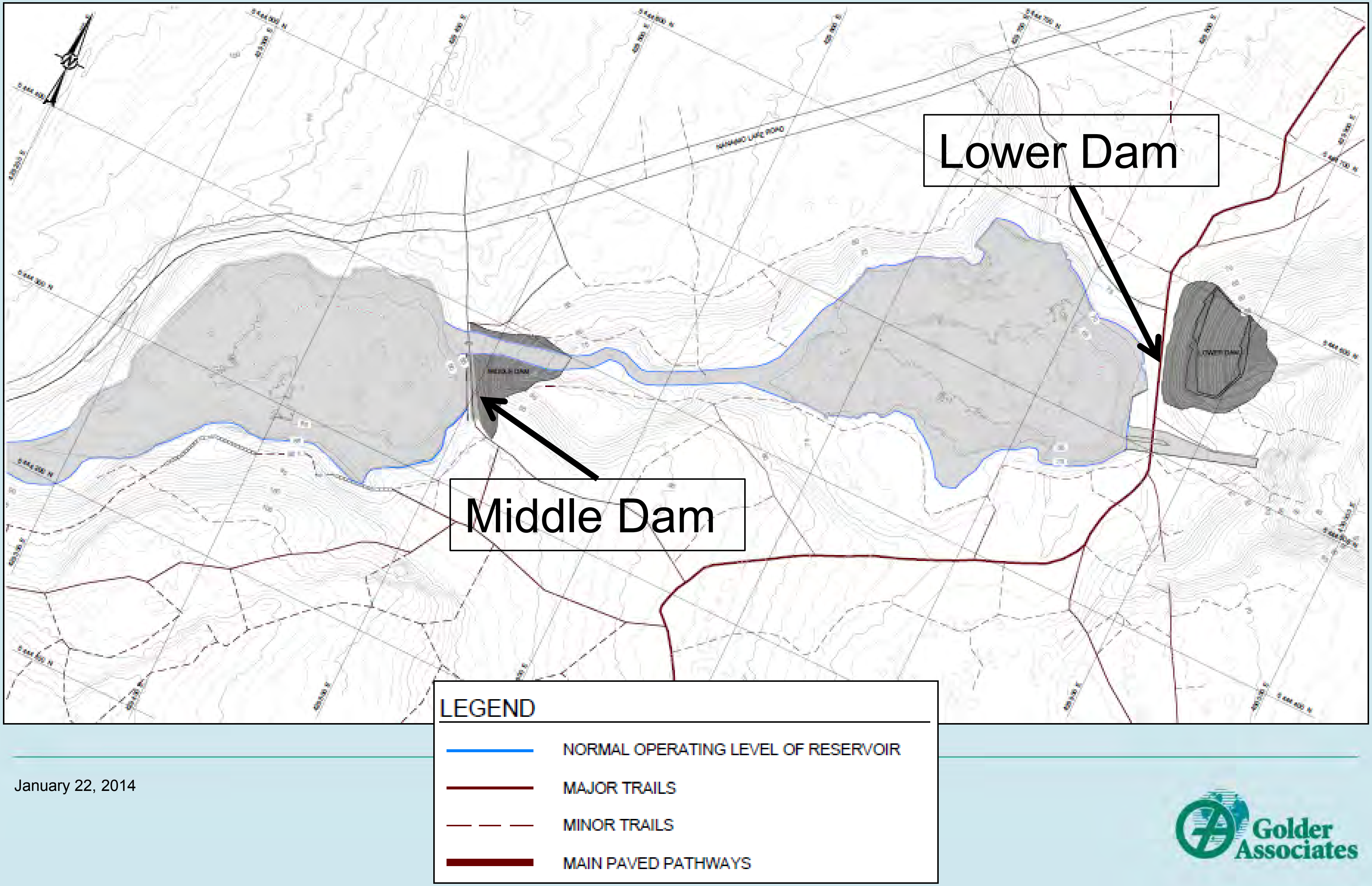
<u>2009 (reported by EBA)</u>

- Diver visual observation of rockfill at surface of upstream shell

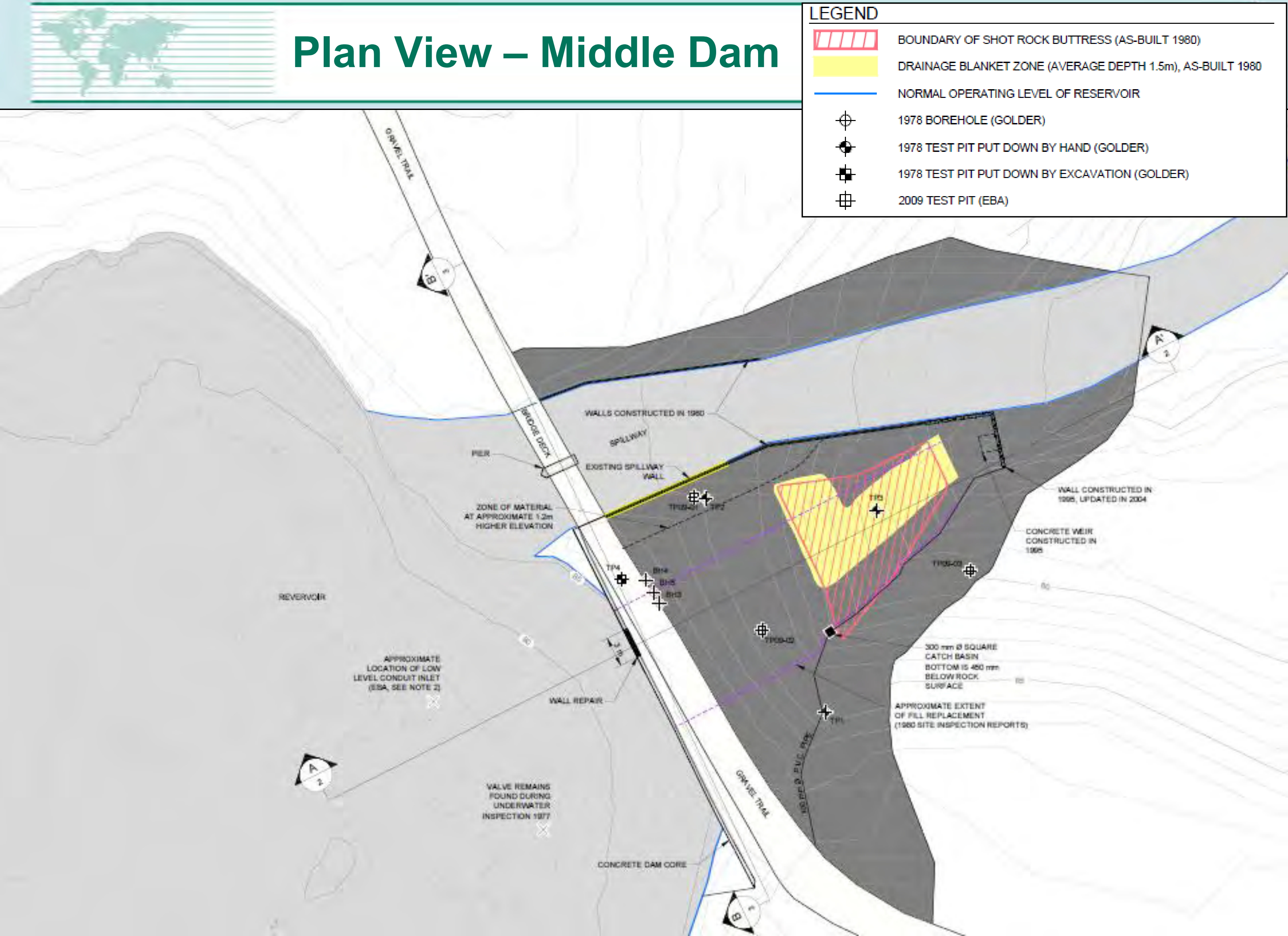
<u>2013 (reported by Klohn)</u>

- GPR – evidence of vertical rebar near the centre of the 1.2 m thick wall at ~760 mm spacing for short distance above water level only
- Coring – evidence of ~16 mm horizontal square twist bars on ~760 mm intervals at center of core
- Minimal reinforcing evidence – general reinforcing or dowels used at cold joints between concrete pours

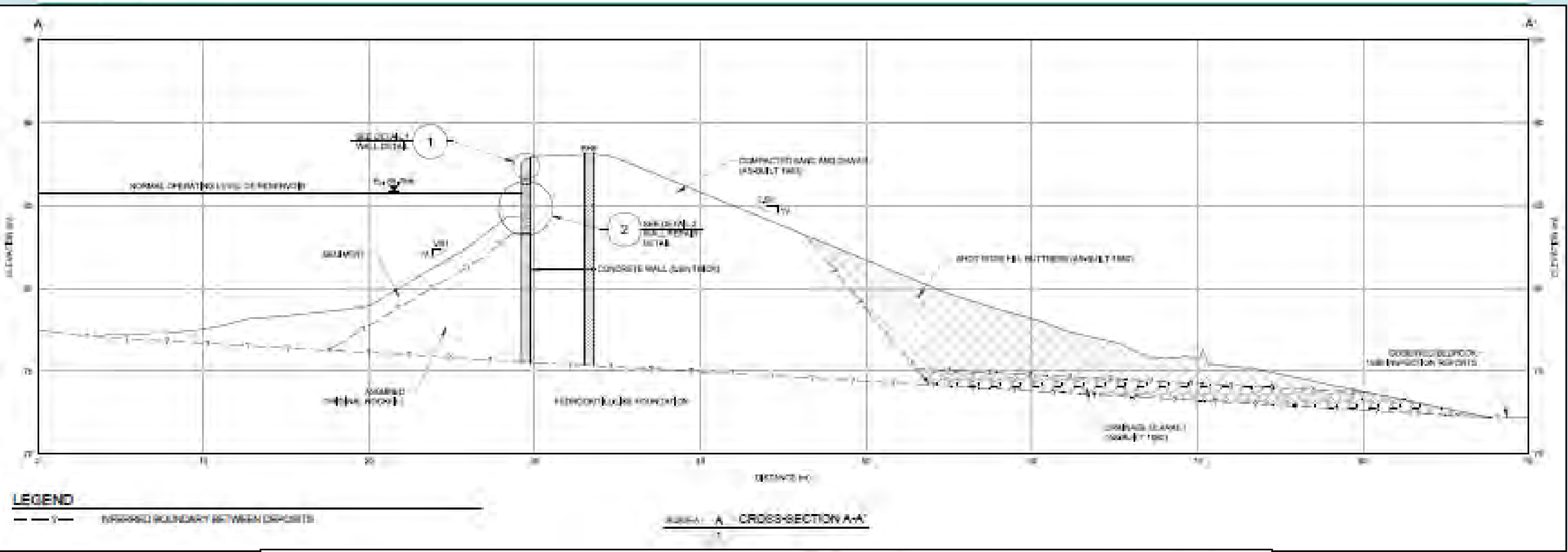
Plan View – Middle and Lower Dam

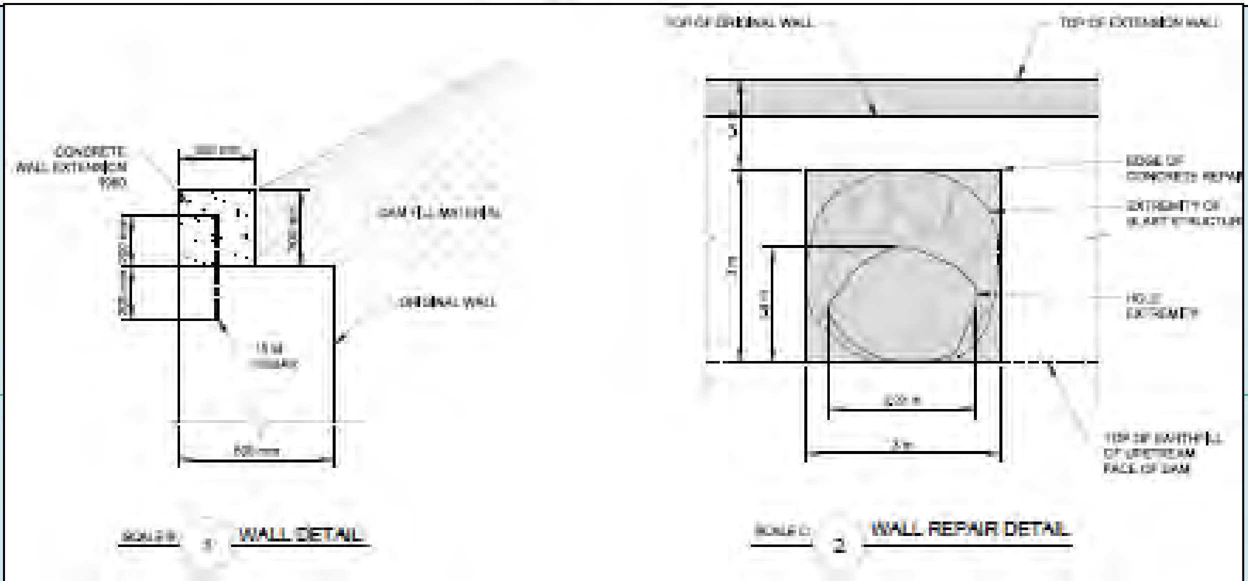


Plan View – Middle Dam



Section A-A' – Middle Dam





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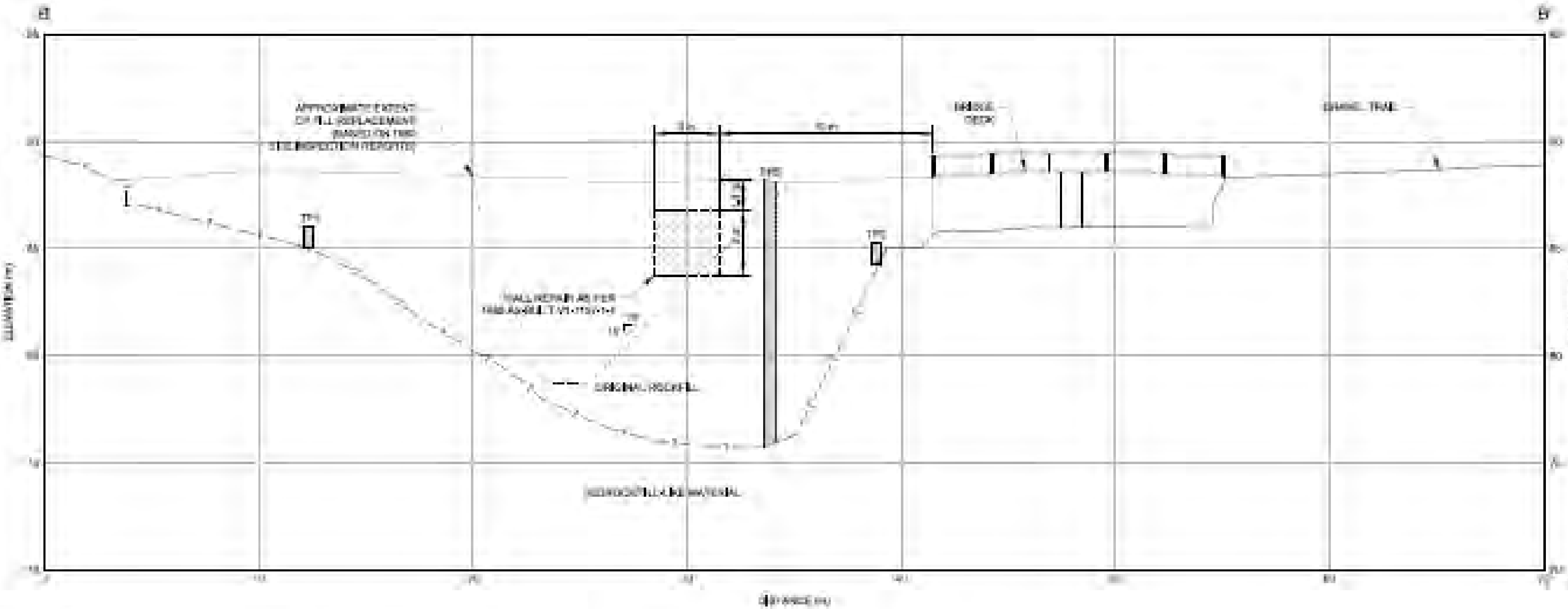
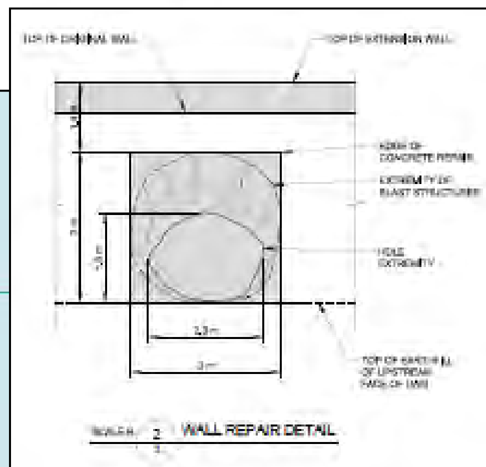
Section B-B' – Middle Dam

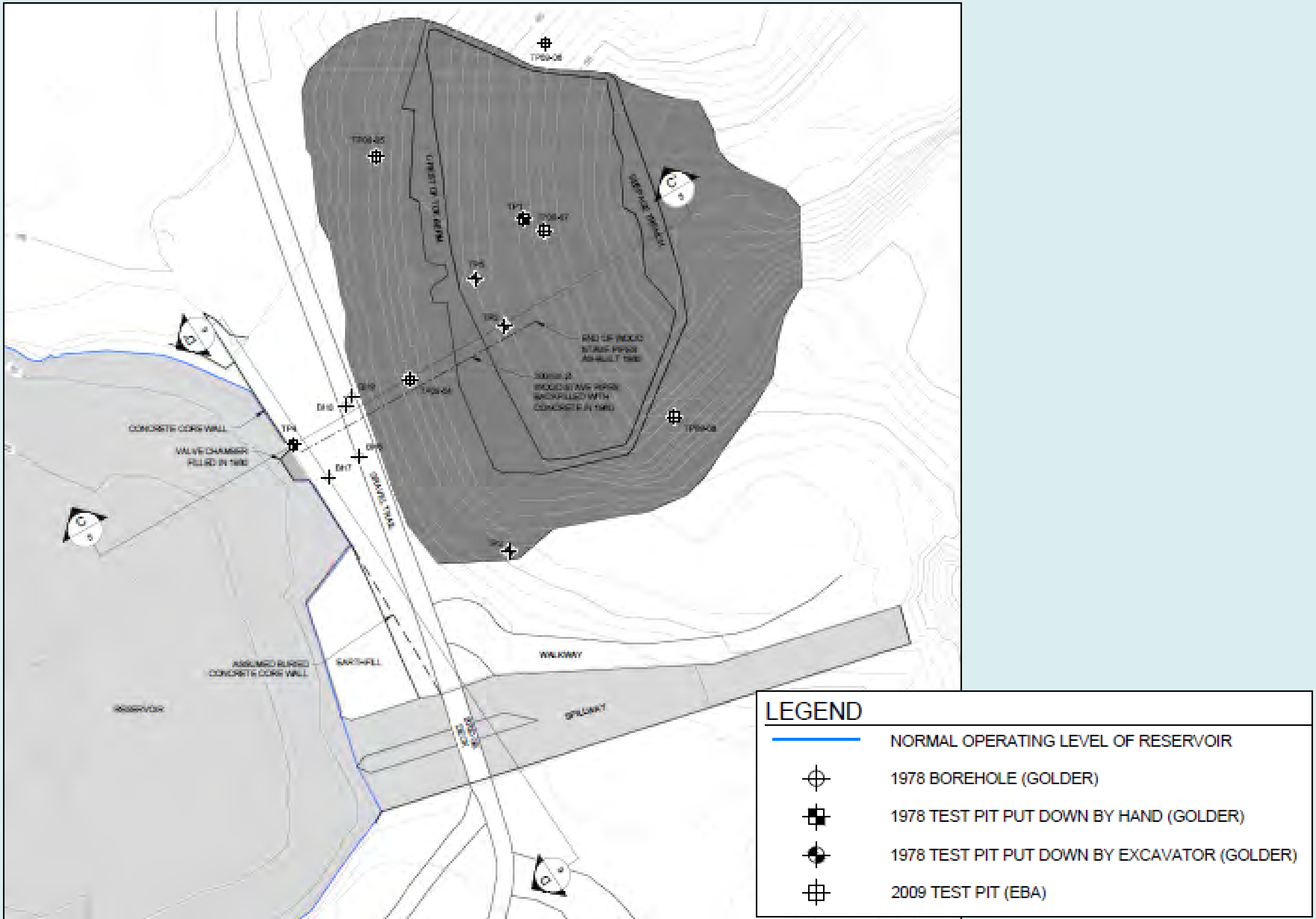


LEGEND

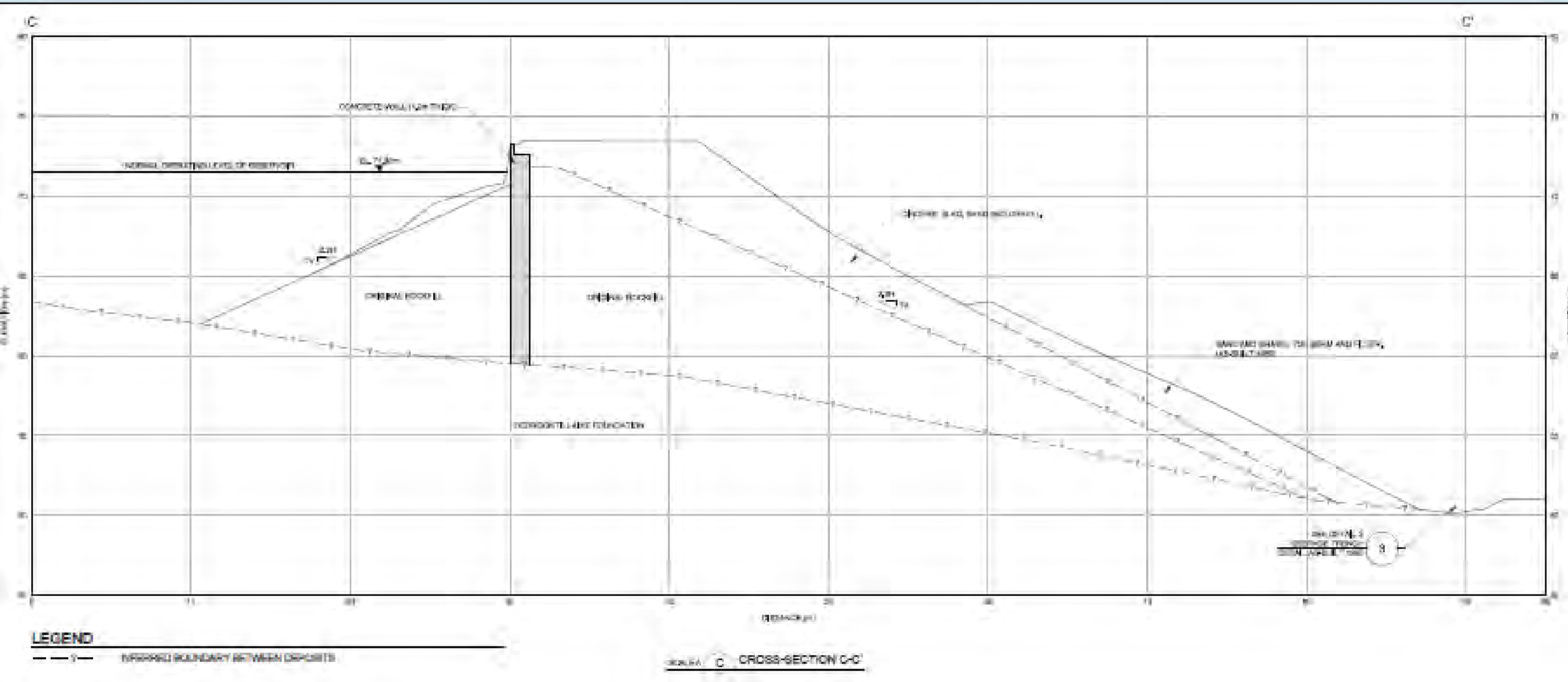
INFERRED BOUNDARY BETWEEN DEPOSITS

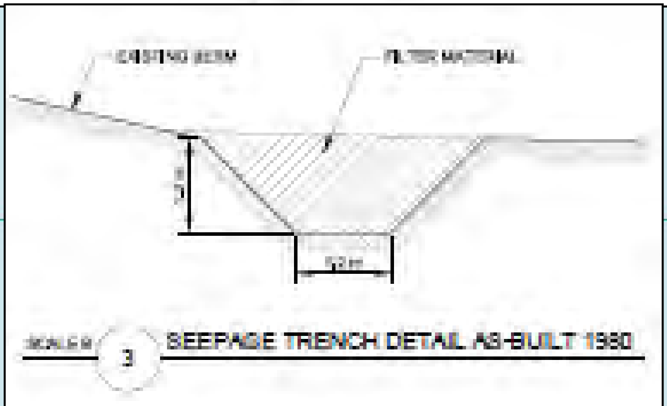


Plan View – Lower Dam



Section C-C' – Lower Dam





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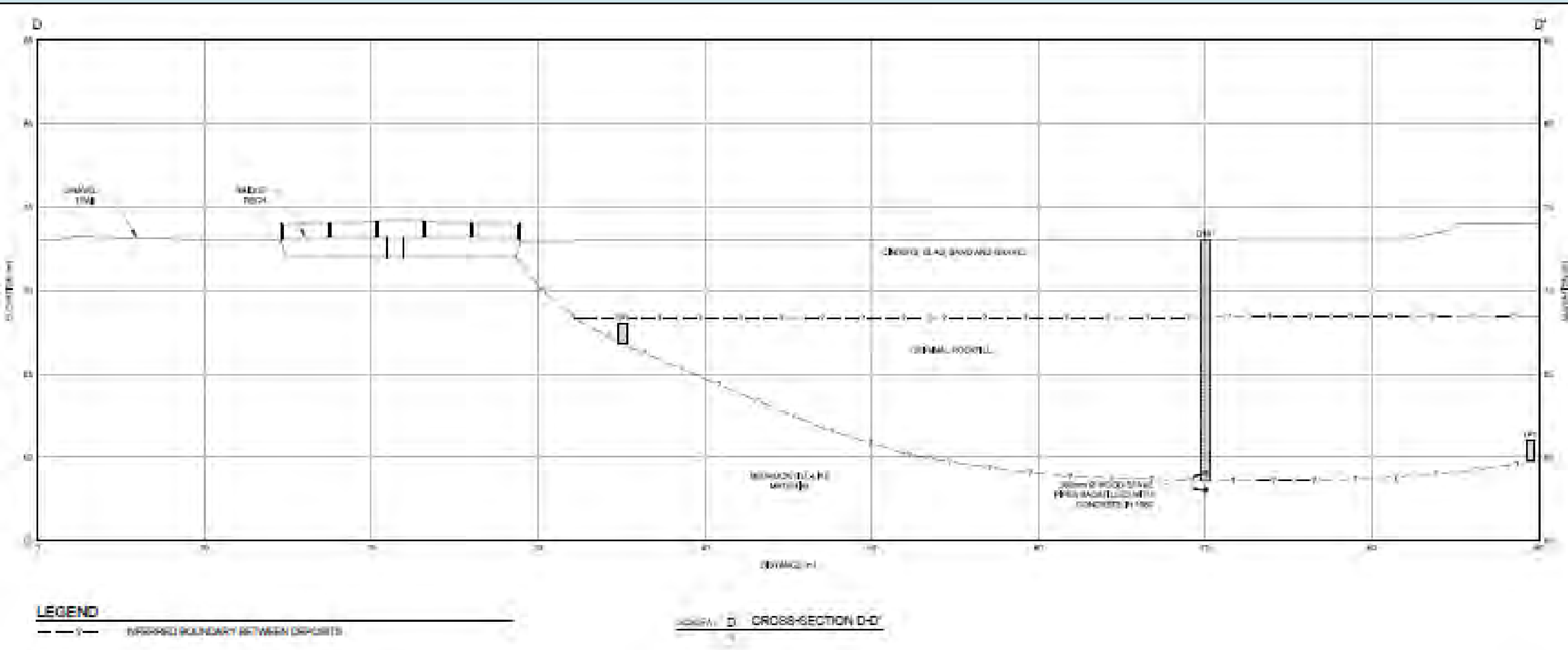
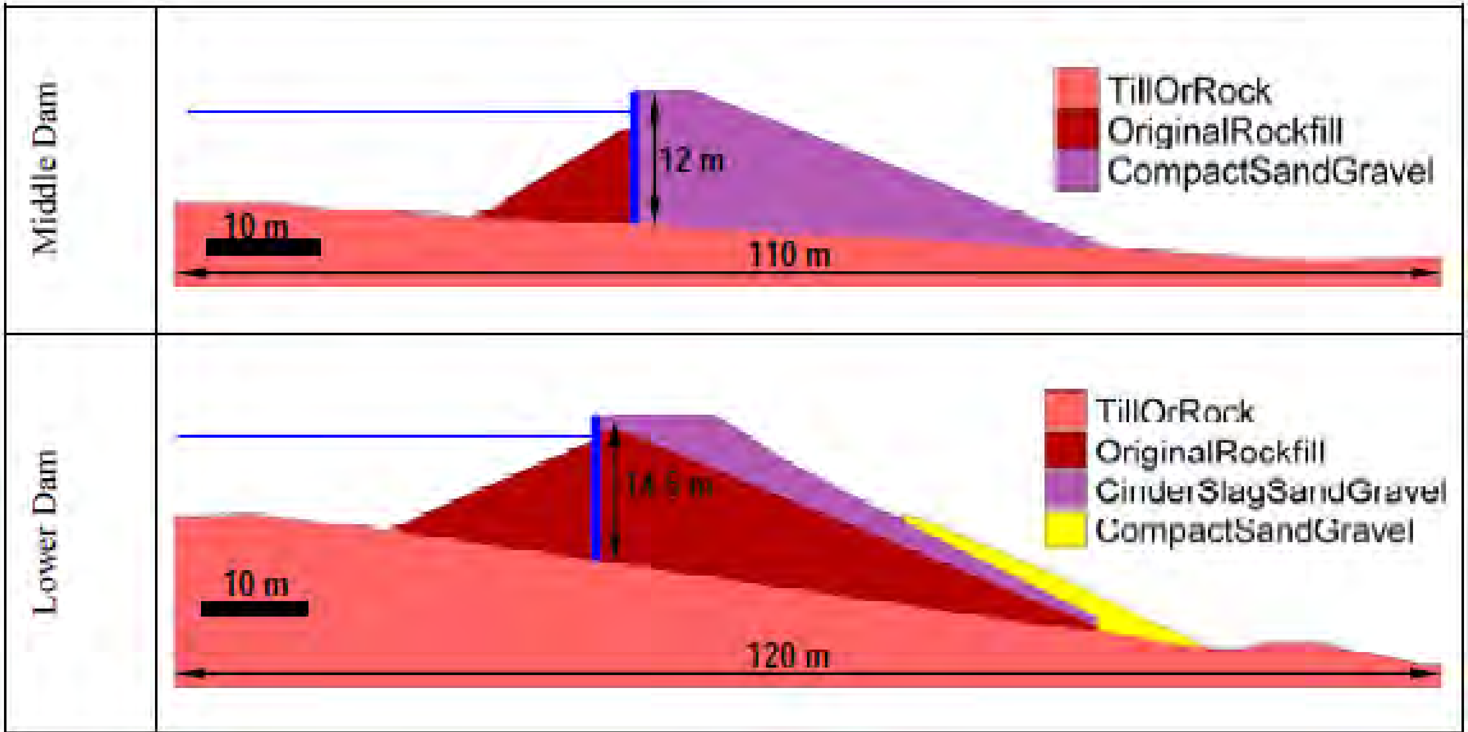
Section D-D' – Lower Dam



Seismic Hazard Assessment Review

- Parametric 2D analysis carried out by EBA (April 2010), based on 3 assumed scenarios
  - Best case scenario material properties reasonably assumed
  - Reasonably worst case scenario material properties assumed
  - Most likely case scenario material properties assumed based on available data and engineering judgment
- Friction angle and shear modulus numbers based on SPT "N" values available for Rock Fill, Sand/Gravel and Cinder/Slag.

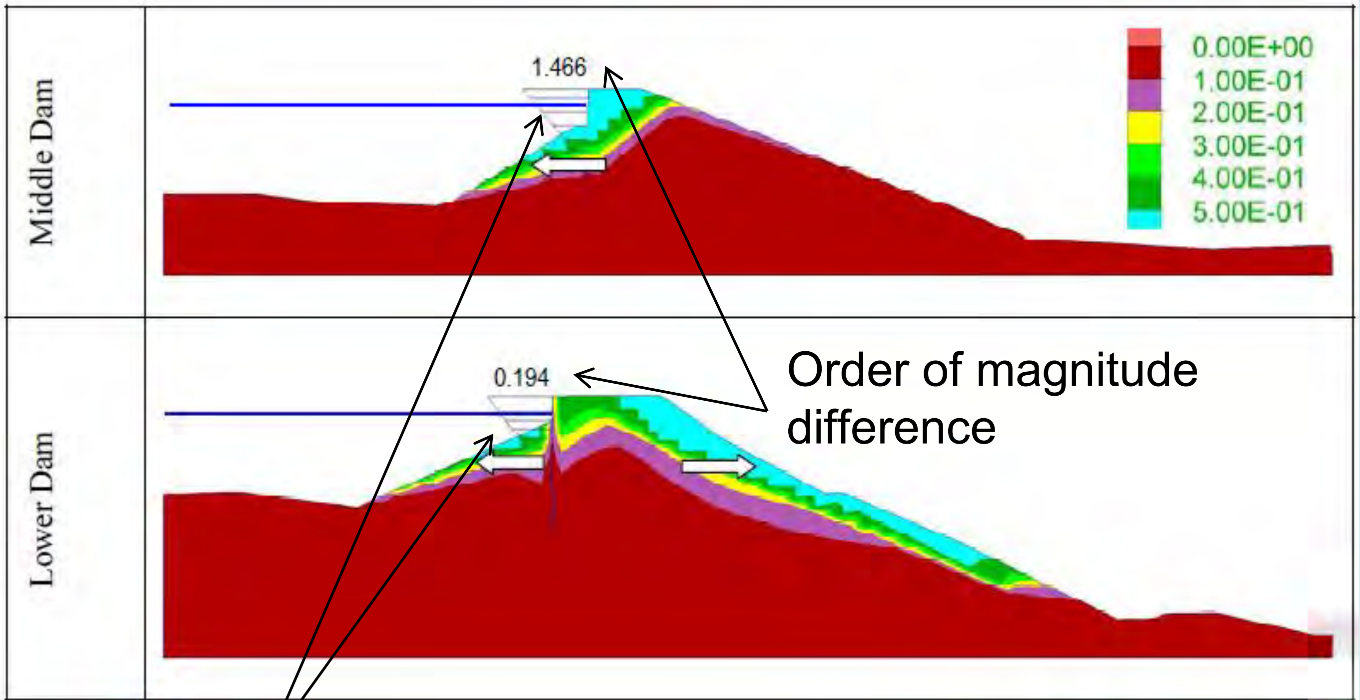
The dam till-like foundation parameters used for analysis not clear



Seismic Hazard Assessment Review

- Safety of the dams are controlled by the concrete walls
- Concrete wall was modelled assuming "elastic and inelastic (moment capacity) beam" behavior for analyses. The effective moment of inertia of the wall was selected as 20% of the value for the cross section and damping ratio was set to 20%.
  - No concrete/ soil interface taken into account
- Cracking of the concrete core requires further study
  - Does cracking of the concrete core = failure?
- Dynamic analyses performed in time domain using the acceleration response spectrum (earthquake motions) for a design event of 1:3000 per annum for dam sites (similar to design event - Vancouver 1:2475)
  - 1:10,000 event would require another seismology study
- Groundwater level in DS shells assumed to be within 0.5 m above base of dams
- Analyses with concrete wall of plastic moment capacity showed large residual displacements of the wall towards upstream
- The dam may be damaged in an earthquake with lower excitation levels (return-period) that was not investigated by EBA fully

Seismic Hazard Assessment Review



Deformed shape of wall (exaggerated) towards reservoir (Most likely case scenario)

- Illustrates effect of <u>inelastic</u> behavior of concrete walls on deflections. Moment capacities of 150 and 600 KN were used for 0.6 and 1.2 m wide dams, respectively. This is the <u>most critical case</u>.

- Contours of Total displacement, deformed shape of concrete wall and its top deflection (m) for Most Likely case

Performance of Concrete Dam Core

How does concrete core behave during seismic event

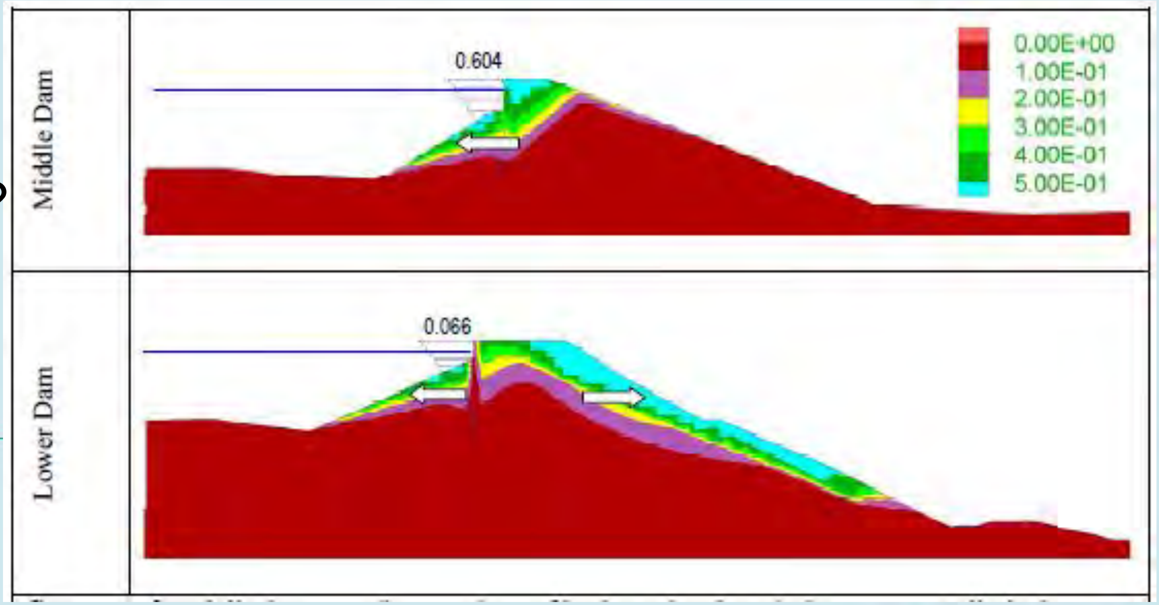
- Toppling, or shearing?
- Cracking, and what is the extent of cracking – effectiveness as a water barrier

Lower Dam.

- Much smaller deformations, thicker concrete section – significant cracking, but maintain containment?

Middle Dam

- Toppling or shearing more likely
  - Loss of containment?



Data Gaps and Design Unknowns – Middle Dam

- Condition of all fills. Some “old” fill left in near spillway (?)
- LLO location still unknown
- Design Criteria for materials in the dam
  - Fill - “new fill” well compacted so should have competent fill properties (can reasonably be estimated)
  - “Old fill”- unknown and questionable, possibly loose (?)
  - Concrete wall – thought to have reinforcement steel (?)
- Performance under earthquakes not established
  - Seismic resistance calculations carried out – need to be verified
- Wall toppling /deformation need to be confirmed. Does Middle core wall topple (?)
  - Seepage from wall if damaged needs to be confirmed
- Performance under floods not established
  - Review of estimated floods needs to be verified
- Capacity of spillway needs to verified
- Fault associated with Chase River Valley

Data Gaps and Design Unknowns – Lower Dam

- Design Criteria for materials in the dam not established
  - Presence and condition of rockfill unknown
  - Presence and condition of ash fill unknown
  - Condition of compacted berm known due to construction records
- Performance under floods not established
  - Review of estimated floods needs to be verified
- Capacity of spillway needs to verified

Dam Core - Issues

- Current condition of concrete core
  - How effective is the current containment
    - Current seepage rates,
    - Changes in seepage rates over time?
- Thickness of walls
- Concrete Reinforcement
  - Spacing
  - Condition of reinforcement
- Concrete condition
  - Uniformity – weak spots, weak filler materials, etc
  - Cold joints, etc
- Dam core will crack under seismic shaking (size of crack (?))
- What is seepage through cracks (?)
    - With cracks will short term piping cause dam failure (?)

Hydrology Study Review

CITY OF NANAIMO

MIDDLE AND LOWER CHASE RIVER DAMS
SPILLWAY HYDROLOGY STUDY

WATER
MANAGEMENT
CONSULTANTS

WATER
MANAGEMENT
CONSULTANTS

April 30, 2002

City of Nanaimo
455 Wallace Street
Nanaimo BC
V9R 5J6

Attention: Scott Pamminger, A.Sc.T.
Engineering Services Technician

Dear Sir:

Re: Middle and Lower Chase River Dams Spillway Hydrology Study

Water Management Consultants is pleased to present our report on the Middle and Lower Chase River Dams Spillway Hydrology study.

We have enjoyed working with you and City of Nanaimo staff on this interesting and challenging study.

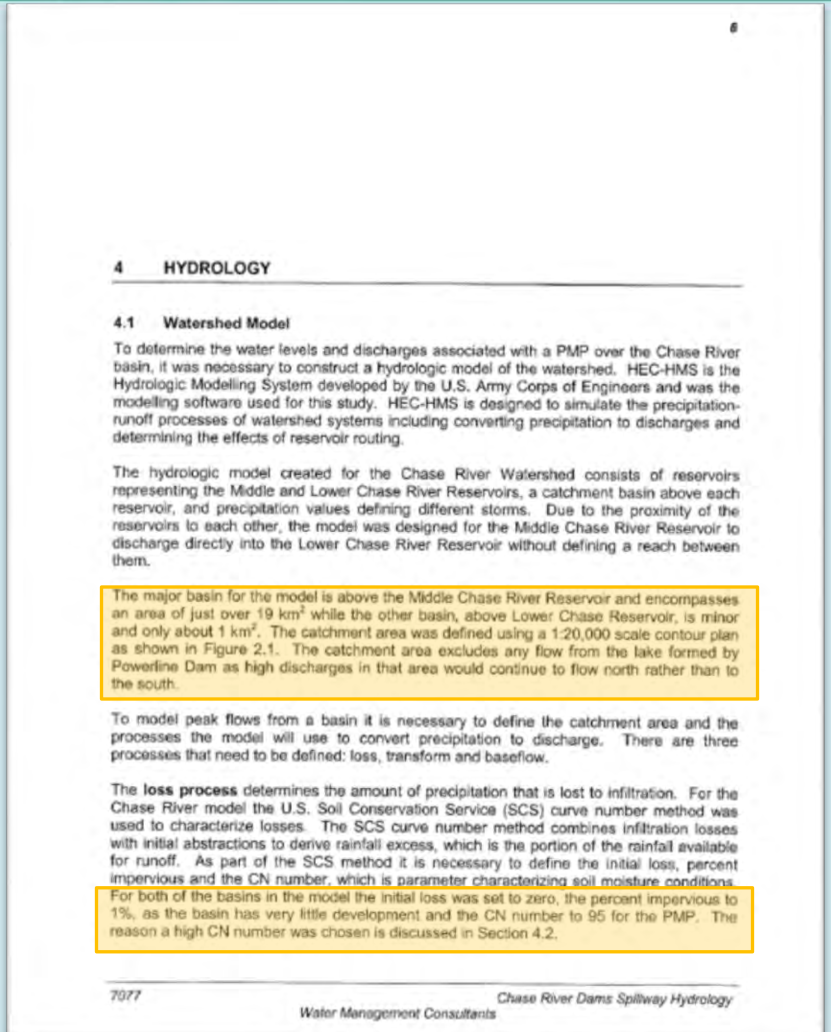
Thank you for this opportunity to provide consulting services to the City of Nanaimo.

Yours truly,

CDN WATER MANAGEMENT CONSULTANTS INC.

C. David Sellars, P.Eng.
Project Manager

Hydrology Study Review



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4 HYDROLOGY

4.1 Watershed Model

To determine the water levels and discharges associated with a PMP over the Chase River basin, it was necessary to construct a hydrologic model of the watershed. HEC-HMS is the Hydrologic Modelling System developed by the U.S. Army Corps of Engineers and was the modelling software used for this study. HEC-HMS is designed to simulate the precipitation-runoff processes of watershed systems including converting precipitation to discharges and determining the effects of reservoir routing.

The hydrologic model created for the Chase River Watershed consists of reservoirs representing the Middle and Lower Chase River Reservoirs, a catchment basin above each reservoir, and precipitation values defining different storms. Due to the proximity of the reservoirs to each other, the model was designed for the Middle Chase River Reservoir to discharge directly into the Lower Chase River Reservoir without defining a reach between them.

The major basin for the model is above the Middle Chase River Reservoir and encompasses an area of just over 19 km^2 while the other basin, above Lower Chase Reservoir, is minor and only about 1 km^2. The catchment area was defined using a 1:20,000 scale contour plan as shown in Figure 2.1. The catchment area excludes any flow from the lake formed by Powerline Dam as high discharges in that area would continue to flow north rather than to the south.

To model peak flows from a basin it is necessary to define the catchment area and the processes the model will use to convert precipitation to discharge. There are three processes that need to be defined: loss, transform and baseflow.

The **loss process** determines the amount of precipitation that is lost to infiltration. For the Chase River model the U.S. Soil Conservation Service (SCS) curve number method was used to characterize losses. The SCS curve number method combines infiltration losses with initial abstractions to derive rainfall excess, which is the portion of the rainfall available for runoff. As part of the SCS method it is necessary to define the initial loss, percent impervious and the CN number, which is parameter characterizing soil moisture conditions. For both of the basins in the model the initial loss was set to zero, the percent impervious to 1%, as the basin has very little development and the CN number to 95 for the PMP. The reason a high CN number was chosen is discussed in Section 4.2.

7077 *Chase River Dams Spillway Hydrology*

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Item	Notes
Other hydraulic structures	• It appears the influence of other hydraulic structures upstream of the Middle Dam have been ignored. These include two other dams and Hwy 19 (Nanaimo Parkway).
CN = 95	• A very high curve number for such an undeveloped basin. • Basin predominately forested. • CN_{AMCIII} for forest in good condition ranges from 43 to 89 depending on hydrologic soil group. • No information about hydrologic soil group(s) provided.
Section 4.2	• Refers to this section for discussion on the high CN.

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Hydrology Study Review

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To estimate short duration rainfall the SCS Type 1A storm distribution was used, which is recommended for this area. (Haan *et al.*, 1994). The one-hour PMP storm based on the SCS Type 1A storm distribution was found to be 40 mm. This rainfall is likely to occur in the cold season (November through March) and would have very wet antecedent conditions. The distribution of the SCS Type 1A storm is shown on Figure 4.2.

Using a 24-hour PMP rainfall, a CN value of 95 and the Type 1A distribution, the PMF inflow peak was calculated to be 192 m^3/s for the Middle Chase Reservoir. The CN value of 95 was chosen to simulate extreme wet antecedent conditions, which would be typical for a winter storm and consistent with standard PMF procedures which require adverse, conservative conditions to be modelled. The rainfall hyetograph and the runoff hydrograph are shown in Figure 4.3. The storm was modelled for 24 hours but the first 12 hours are shown in this graph to provide greater definition, which is possible because the peak occurs within the first 12 hours. The peak PMF at the Lower Chase River Dam was calculated to be 198 m^3/s. There is very little peak flow attenuation provided by these small reservoirs.

As a sensitivity test it was found that increases in the lag time of 20% decreased the peak PMF by 8%. A decrease in the lag time of 20% increased the peak flow by 10%.

Consideration was also given to the occurrence of a local storm. The procedures used to develop a local storm applied generalized relationships from the HMR-57 study as follows:

- Use generalized local storm maps from HMR-57 to identify 1 hour, 1 square mile storm
- Adjust for duration using depth-duration curves provided in HMR-57
- Adjust for basin area using depth-area relationships

The analysis indicated that a local storm could produce a one-hour PMP of 60 mm. However, this storm would occur in the summer when the antecedent soil moisture conditions are dry. The highest intensities with this type of storm occur within the first hour so there would be considerable losses to satisfy soil moisture deficits and depression storage. Thus there would be less rainfall excess available for runoff and thus a reduced peak flow. HMR-57 notes that there is very little information in the region available to characterize this type of storm particularly in Canada where there are fewer recording rain gauges. Given the uncertainty inherent in characterizing a local storm and the large losses that would certainly occur, it was decided to base the PMP/PMF calculations using the conventional approach, which is based on a regional winter storm.

To verify this conclusion, a local storm with a one-hour precipitation value of 60 mm was input to the model. A CN value of 93 with the local storm produced the same outflow from the catchment as the winter storm and a CN value of 95. The CN value for characterizing soil conditions in the summer months would be expected to be much lower than 93. Therefore it was concluded that the conventional approach for calculating the PMP/PMF was appropriate.

The winter storm produces a higher peak flow than the summer storm even though the one-hour rainfall is less. This is because the 24-hour rainfall depth in the summer is much less than the winter storm, which contributes to the wet antecedent conditions and causes an increase of flow in the watershed prior to the occurrence of the peak rainfall intensity.

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Item	Notes
CN = 95	• This is the explanation referred to in Section 4.1. Does not provide technical justification for the use of the high Curve Number.

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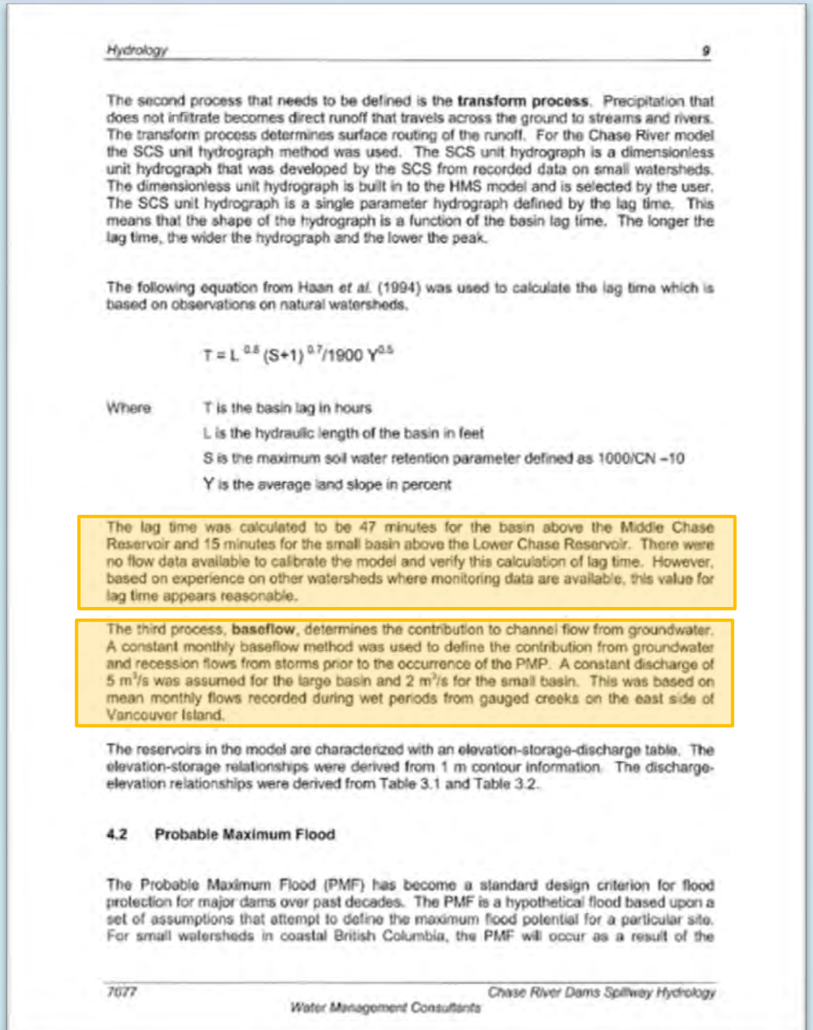
Hydrology Study Review



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The second process that needs to be defined is the **transform process**. Precipitation that does not infiltrate becomes direct runoff that travels across the ground to streams and rivers. The transform process determines surface routing of the runoff. For the Chase River model the SCS unit hydrograph method was used. The SCS unit hydrograph is a dimensionless unit hydrograph that was developed by the SCS from recorded data on small watersheds. The dimensionless unit hydrograph is built in to the HMS model and is selected by the user. The SCS unit hydrograph is a single parameter hydrograph defined by the lag time. This means that the shape of the hydrograph is a function of the basin lag time. The longer the lag time, the wider the hydrograph and the lower the peak.

The following equation from Haan *et al.* (1994) was used to calculate the lag time which is based on observations on natural watersheds.

$$T = L^{0.8} (S+1)^{0.7} / 1900\, Y^{0.5}$$

Where T is the basin lag in hours

L is the hydraulic length of the basin in feet

S is the maximum soil water retention parameter defined as 1000/CN –10

Y is the average land slope in percent

The lag time was calculated to be 47 minutes for the basin above the Middle Chase Reservoir and 15 minutes for the small basin above the Lower Chase Reservoir. There were no flow data available to calibrate the model and verify this calculation of lag time. However, based on experience on other watersheds where monitoring data are available, this value for lag time appears reasonable.

The third process, **baseflow**, determines the contribution to channel flow from groundwater. A constant monthly baseflow method was used to define the contribution from groundwater and recession flows from storms prior to the occurrence of the PMP. A constant discharge of 5 m^3/s was assumed for the large basin and 2 m^3/s for the small basin. This was based on mean monthly flows recorded during wet periods from gauged creeks on the east side of Vancouver Island.

The reservoirs in the model are characterized with an elevation-storage-discharge table. The elevation-storage relationships were derived from 1 m contour information. The discharge-elevation relationships were derived from Table 3.1 and Table 3.2.

4.2 Probable Maximum Flood

The Probable Maximum Flood (PMF) has become a standard design criterion for flood protection for major dams over past decades. The PMF is a hypothetical flood based upon a set of assumptions that attempt to define the maximum flood potential for a particular site. For small watersheds in coastal British Columbia, the PMF will occur as a result of the

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Item	Notes
Lag = 47 Minutes	• This is a quick lag for a 19 sq-km basin. However, may be valid due to steep topography.
Baseflow	• 5 cms from 19 sq-km basin. • 2 cms from 1 sq-km basin. • Assume that total baseflow to Lower Dam adds to 7 cms. • Lower dam reported total spillway capacity = 25 cms (baseflow takes up 28% of total capacity?). • Utilizing weir equation and parameters given in this report for the Middle Dam spillway, 5 cms would flow at a depth of 0.4 meters. Available photos do not suggest this depth of base flow.

Hydrology Study Review

Table 3.2: Lower Chase Dam Spillway Capacity

Reservoir elevation m	Head m	Discharge m^3/s
71.6	0.0	0.0
71.9	0.3	2.0
72.2	0.6	6.0
72.4	0.8	10.0
72.6	1.0	15.0
72.8	1.2	20.0
73.0	1.4	25.0
73.2	1.6	30.0
73.4 (Dam crest)	1.8	35.0

Note: Maximum capacity is 25 m^3/s without overtopping the spillway chute wing walls

Previous studies noted in EBA, 1992b indicated that the maximum capacity was 55 m^3/s. The detail of the calculation procedures were not given but it is likely that it was assumed that critical depth would occur at the spillway entrance. In other words the spillway would act as a broad crested weir. In fact, the detailed spillway hydraulic analysis carried out for this reports shows that there are head losses in the approach channel and critical depth does not occur until a point downstream of the bridge. As a result the maximum capacity was determined to be lower than in previous studies.

It was concluded from this detailed study that the original design of the spillway was not very efficient. At the time it was constructed (about 1910) it was not possible to carry out the detailed hydraulic modelling that was used in the current study. The spillway chute does not have a steep enough gradient to overcome the contraction. The designers should have used less of a contraction to maintain super-critical flow for the full range of discharges.

Item	Notes
Baseflow (Continued)	• How does the recent January 10-11, 2014 storm compare? • Reported 50mm of rainfall in 9 hours. • Lower Dam’s spillway peaked at 41 cm of depth. • The calculated flows from this event are inconsistent with the calculated base flows (2002 report).

Hydrology Study Review

3 SPILLWAY CAPACITIES

3.1 Middle Chase River Dam Spillway

The elevation-discharge relationship for the Middle Chase spillway was calculated using the weir equation $Q=CLH^{1.5}$ where:

Q is the discharge in m^3/s
C is a coefficient
L is the crest length in m
H is the elevation difference in m between the crest and the reservoir level

The width of the opening under the bridge is approximately 12.2 m as determined from detailed surveys carried out by the City of Nanaimo. For high reservoir levels, the C coefficient was estimated to be 1.67 based on tables in King and Brater (1953). The entrance to the spillway is roughly flush with the leading approach of the reservoir bottom justifying a lower value for the coefficient.

A steady state backwater analysis was also carried out using the U.S. Army Corps of Engineers HEC-RAS modelling software to determine the Q-H relationship for the spillway. The model predicted discharges for the spillway similar to that calculated by the weir equation with a coefficient of 1.67.

The bridge over the spillway limits the height of the water surface profile over the spillway which could potentially limit the discharge capacity. However, it was determined from the backwater analysis that critical depth occurs under the bridge and the flow depth at the bridge is about 0.1 m below the soffit when the reservoir level is at the crest of the dam. Therefore the reservoir level defines the maximum capacity of the spillway. With the reservoir level near the crest of the dam (elevation 88.3 m), the head above the spillway sill is about 2.1 m resulting in a peak discharge in the spillway of 62 m^3/s. The maximum capacity had been estimated in previous studies to be 42 m^3/s (EBA, 1992a). In the previous study, it had been assumed that the elevation of the water surface at the soffit of the bridge was equivalent to the reservoir level. However, the elevation of the water surface profile is lower at the bridge than in the reservoir and hence the spillway capacity is greater than previously computed.

Table 3.1 shows the spillway discharges for different reservoir elevations.

Item	Notes
Weir Equation	• Weir equation used to develop a rating curve for the Middle Dam spillway. • Acceptable, but given the contraction, bridge, pier, and irregular channel, HEC-RAS would be more appropriate to model.
HEC-RAS	• Consultant states that a HEC-RAS model was created and that it predicted similar results as the weir equation.

Hydrology Study Review

Spillway Capacities 6

Table 3.1: Middle Chase River Dam Spillway Capacity

Reservoir elevation m	Head m	Discharge m^3/s
86.2	0.0	0.0
86.4	0.2	2.0
86.8	0.6	10.0
87.5	1.3	30.0
88.0	1.8	50.0
88.3 (Dam crest)	2.1	62.0

3.2 Lower Chase River Dam Spillway

The elevation-discharge relationship for the Lower Chase spillway was calculated using the HEC-RAS backwater analysis program. This program was developed by the U.S. Army Corps of Engineers to calculate steady state water surface profiles in channels. It is ideally suited to the conditions at the Lower Chase River Dam spillway as it can model both sub-critical and super-critical flow and channels of varying width and slope.

Fourteen cross-sections were used in the model to characterize the spillway. Four sections define upstream of the bridge, two sections are at the bridge, five sections define the 25 m reach downstream of the bridge where the grade is still near 1% and the side walls are constricting the width, and three sections define the reach downstream of the break point to the free over-fall. The break point in the spillway occurs approximately 25 m downstream of the bridge where the slope changes from 1 to 20%.

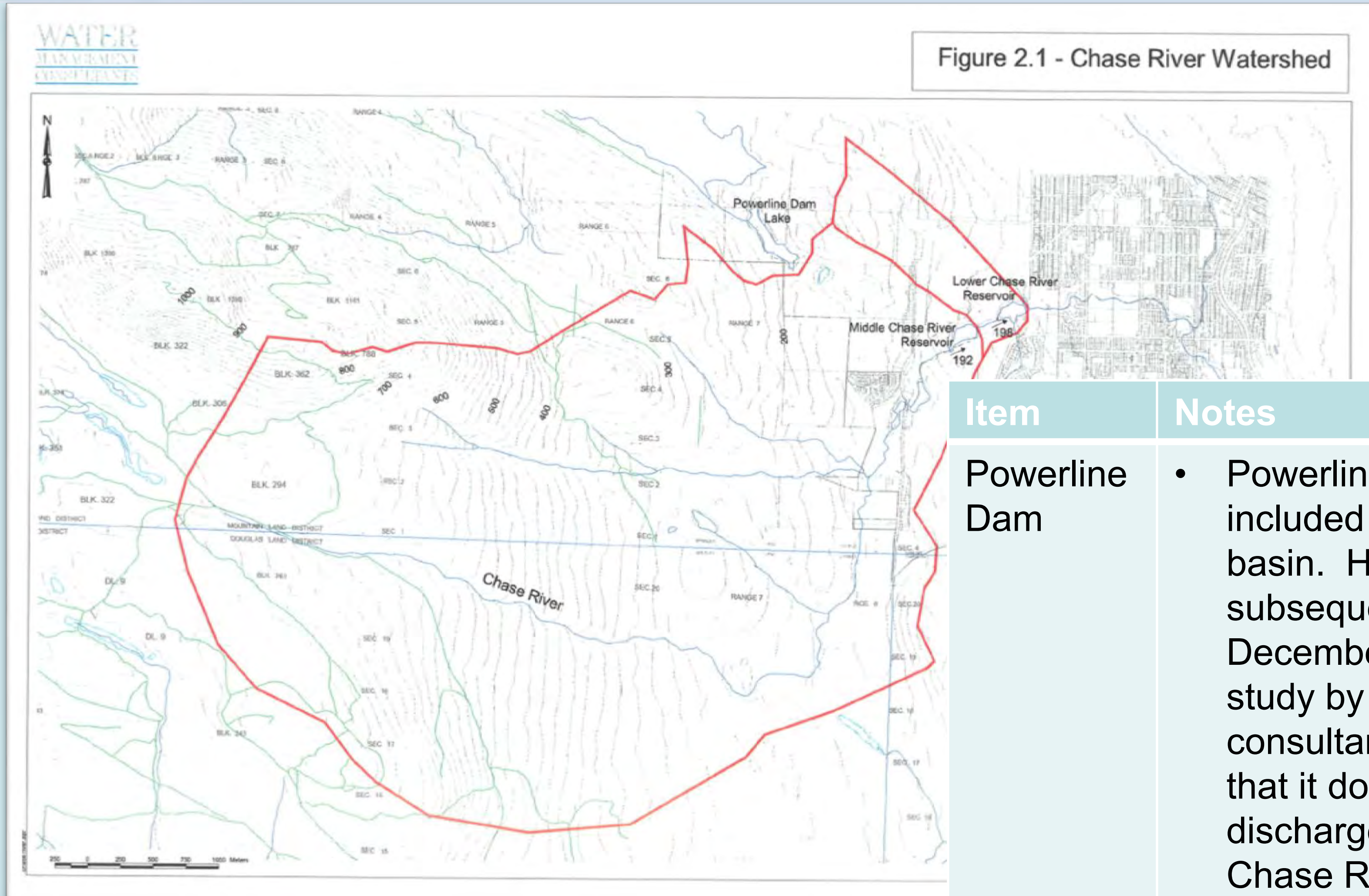
It was found that at low discharges of less than 5 m^3/s, critical depth occurs at the bridge and remains as super-critical flow for the remaining length of the spillway as shown in Figure 3.1a. At higher discharges, however, the constriction in the spillway prior to the break point causes the flow to decelerate, resulting in sub-critical flow and an increase in water levels. As the discharge increases, the point at which flow changes to sub-critical moves upstream until the entire reach before the break point is sub-critical. This occurs when the discharge reaches 20 m^3/s. Flow downstream of the break point is always super-critical.

Table 3.2 shows the spillway discharges for different reservoir elevations. The lowest point on the dam crest is 73.4 m so the maximum capacity of the spillway, with zero freeboard on the dam, is 35 m^3/s. However, the HEC-RAS backwater analysis demonstrated that the increase in water levels due to the constriction upstream of the break point results in overtopping the sidewalls at a discharge of 25 m^3/s. The water surface profile for a discharge of 35 m^3/s and the right wing wall elevations are shown on Figure 3.1b. Overtopping of the sidewalls is unacceptable due to the dangers associated with uncontrolled flow and thus the sidewall elevations limit the capacity of the spillway to 25 m^3/s.

7077 *Chase River Dams Spillway Hydrology*
Water Management Consultants

Item	Notes
HEC-RAS	• HEC-RAS used to develop rating curve for the Lower Dam Spillway.
Spillway hydraulic problems	• HEC-RAS model predicts a hydraulic jump for all flows greater than 5 cms on the flat 1% spillway section. • What are the potential consequences associated with this flow depth exceeding the wall height? • What are some possible remedies to address these consequences? • Has this hydraulic jump been observed in the field (assume flows exceeding 5 cms are common).

Hydrology Study Review



Item	Notes
Powerline Dam	• Powerline dam not included in drainage basin. However, subsequent December 17, 2002 study by the same consultant indicates that it does discharge into the Chase River basin.

Hydrology Study Review

Final Recommendations

Many other studies including preliminary designs, dam breach analyses, etc appear to be based in part on the findings and analysis of this study. If the hydrology and hydraulics (H&H) change, those changes will have ripple effects to those other studies.

Recommend that hydrology be further studied to:

- Account for upstream hydraulic structures/storage.
- Refine Curve Numbers and Lag Times.
- Refine Baseflow values.
- Verify basin delineations (Powerline Dam).

Recommend that spillway hydraulics be further studied to:

- Verify Middle Dam's rating curve.
- Verify Lower Dam's rating curve.
- Verify Lower Dam's hydraulic jump caused by convergence.
- Evaluate consequences of the jump depth exceeding the Lower Dam's spillway walls and possible ways to remedy.

Recommend a formal incremental damage assessment be performed using updated H&H info and revised breach parameters.

Dam System

- The two dams act as a system.
  - Middle Dam fails = Lower Dam <u>may</u> fail
  - The mechanism of failure:
    - Seismic event: core toppling and overtopping and failure
    - Flood event: overtopping and failure
- Considerations
  - Possible Event
  - Possible Failure Mode
  - Possible Consequence
  - Effect Downstream

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Design Issues - Summary

- The two dams act as a system.
  - Middle Dam fails = Lower Dam may fail -> dams in EXTREME Category
- The mechanism of failure:
  - Seismic event: core toppling and overtopping and failure
  - Flood event: overtopping and failure
- The control of the consequence designation is the Middle Dam.
- Under present Consequence category, it is necessary to fix the Middle dam for seismic event, check performance of the Lower dam under the new conditions, and provide flood capacity for handling the design floods. With a RISK assessment a different approach is possible.
- If the Middle Dam is fixed for seismic event (no core toppling) then the Lower dam may survive with some damage but still retain water?
- If the Middle Dam fails due to a seismic event and the lower dam is reinforced to withstand a seismic event, will the overtopping flood downstream be acceptable?
- Both dam spillways are undersized for design IDF flood even if the Consequence Category can be downgraded by fixing the Middle Dam for seismic event and ensuring that the cracking of the Lower Dam does not result in uncontrolled dam failure and flooding downstream.

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Risk Assessment

Initial Conceptual Ideas – Increase Flood Routing Capacity

- Allow overtopping of the dam (reinforce downstream face of dam)
    - Roller Compacted Concrete and Soil Cement
    - Conventional/Mass Concrete Slabs
    - Precast Concrete Blocks
    - Gabions
    - Vegetative cover Reinforced and artificial turf
    - Rock fill reinforced rockfill
    - RipRap
    - Geomembranes and Geocells and fabric formed concrete
    - Open stone asphalt (possible)
    - Allu
- Increase spillway capacity
    - Reconfigure (straighten), deepen, (possible Obermeyer weir)

Roller Compacted Concrete and Soil Cement

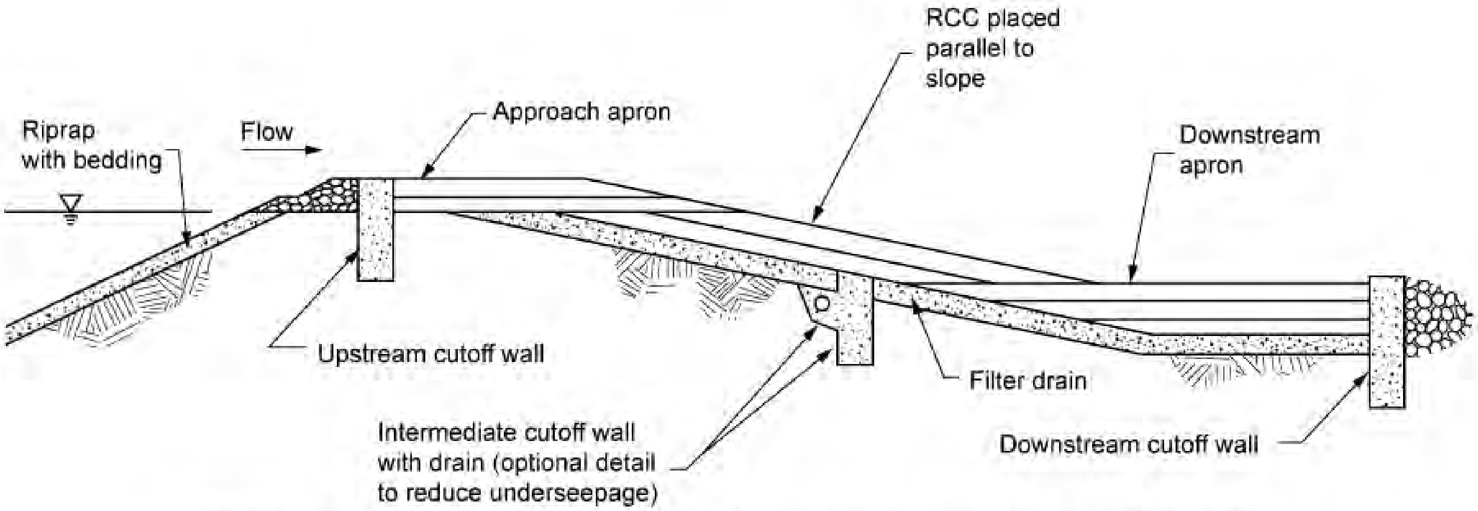

Figure 2.—Overtopping protection with RCC placed parallel to slope (PCA, 2002).

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Roller Compacted Concrete and Soil Cement

- Roller Compacted Concrete: Concrete compacted by roller compaction, concrete that in its unhardened state will support a roller while being compacted. RCC differs from soil cement (SC) in that it may have coarse aggregate and develops properties similar to conventional placed concrete.
-
- Generally RCC used instead of (SC) because of less strength and thicker section usually needed to provide the same security as RCC
- Example where SC has been used is the Alvin Wirtz dam in Austin Texas – 105 ft. high dam
- Nakusp water front protection -- ~ 30m high slope

Mass Concrete Slabs

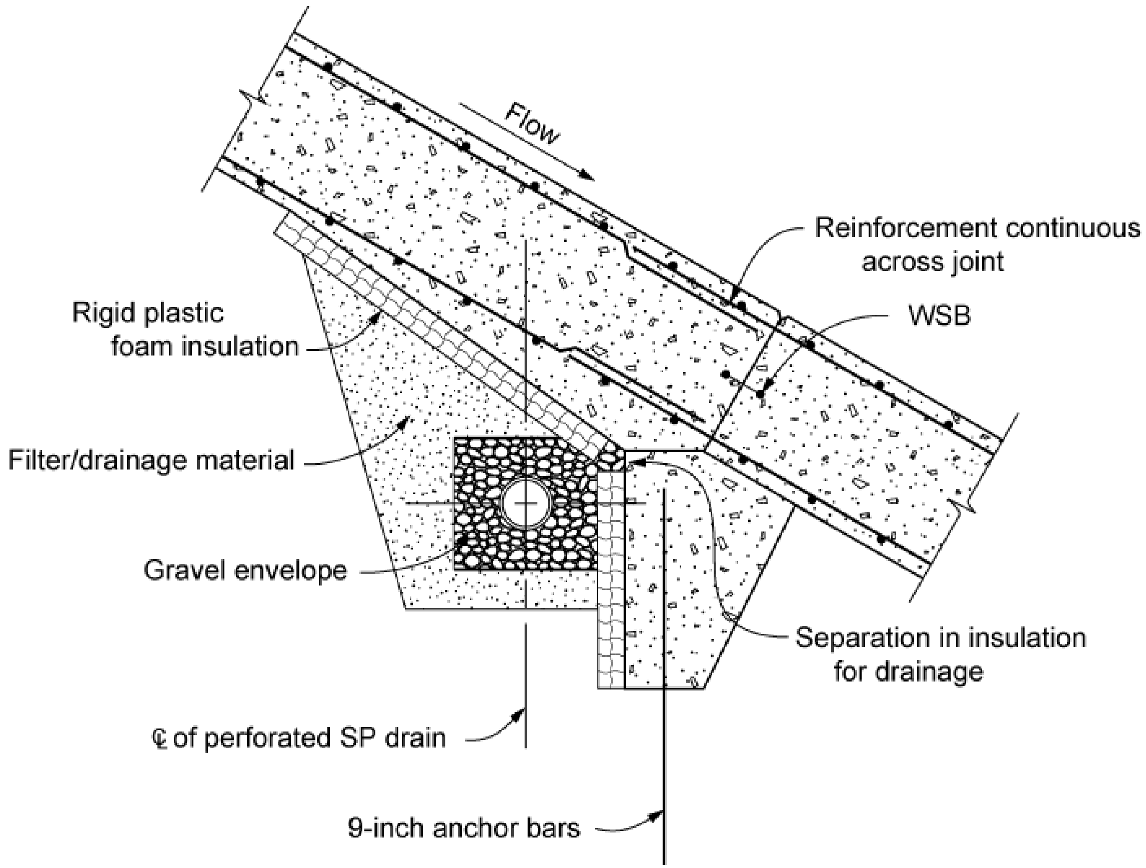


Figure 3.—Defensive design measures for concrete chutes to prevent uplift failure.

Precast Concrete Blocks

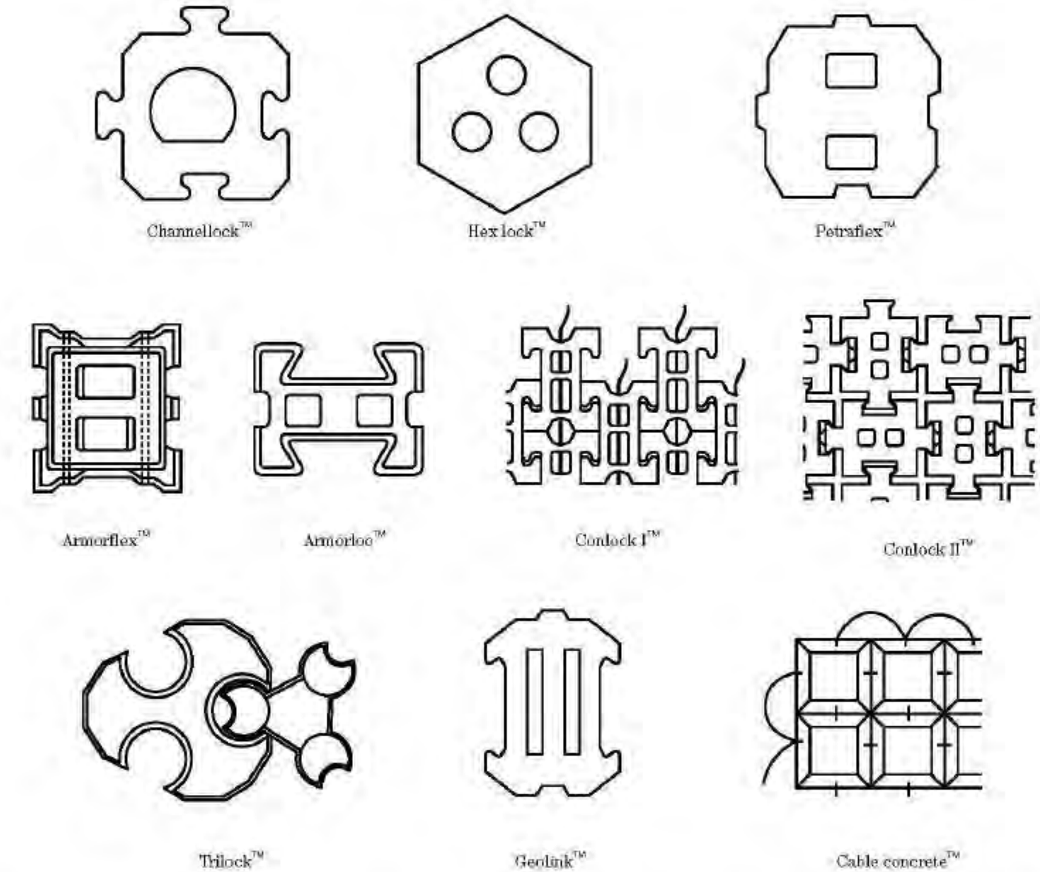


Figure 2.—Some common examples of precast revetment systems. (Figure courtesy of HCFCD and NRCS manual.)

Precast Concrete Blocks - Continued

- Cable tied
- Interlocking
- Overlapping
- Butt-jointed

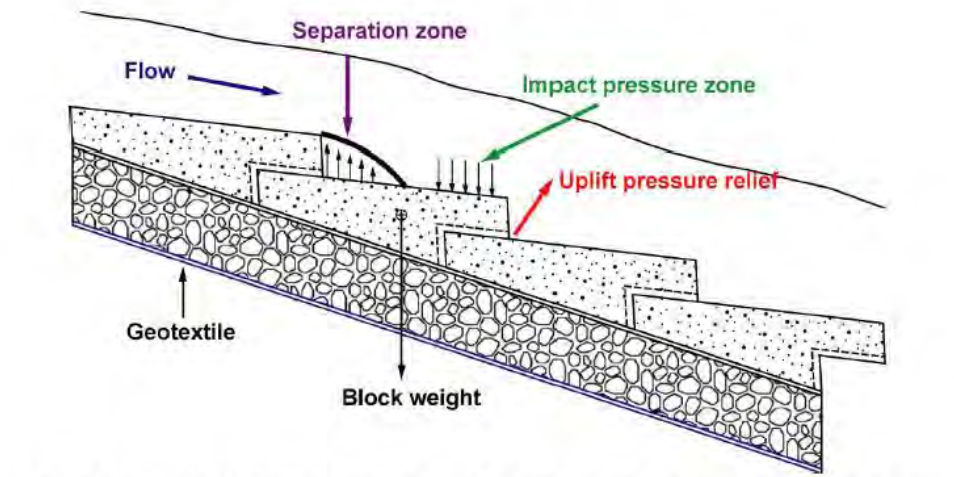


Figure 5.—Typical forces on a wedge-block ACB system (Courtesy of Armortec, Inc.).

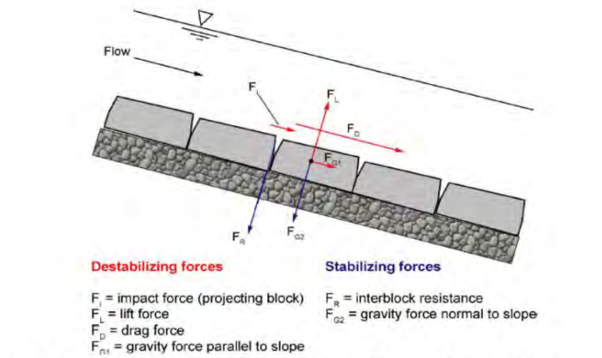
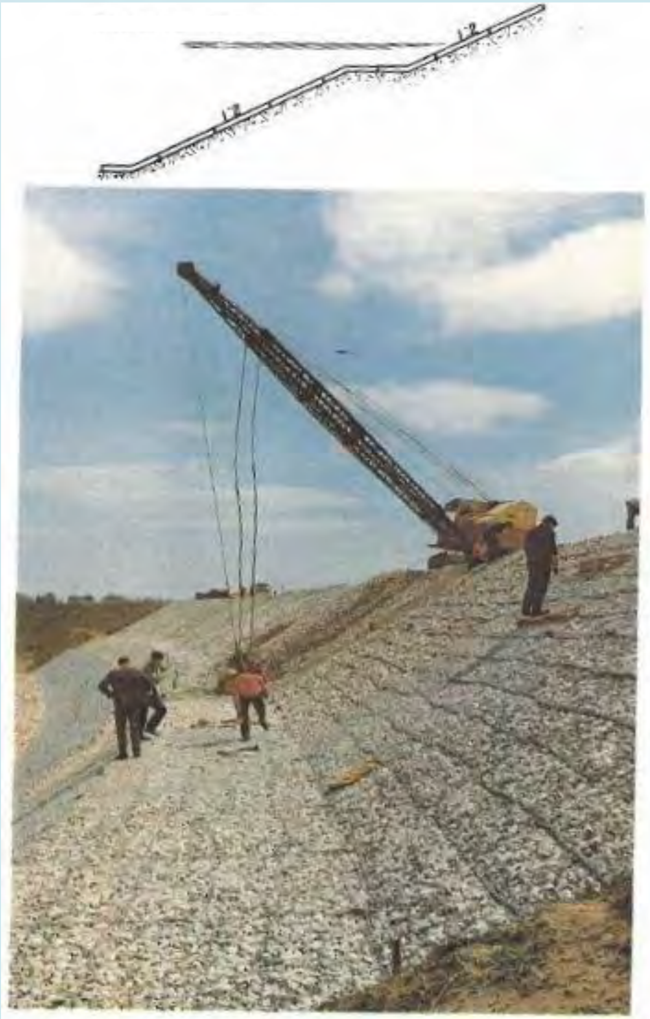


Figure 4.—Hydraulic forces on the typical cable-tied ACB system (Courtesy of Armortec, Inc.)

Gabions



Vegetative Cover

- Vegetative protection prevents erosion
- Can be natural or artificial
- Resistant fabrics can be installed in the turf to assist in resistance
- Usually for fairly low velocities and flows.

Rock fill - Reinforced rock fill

- Rock fill sizes can resist erosion
- Reinforcement done by installing rebar
- Also can be reinforced by synthetic geogrid.

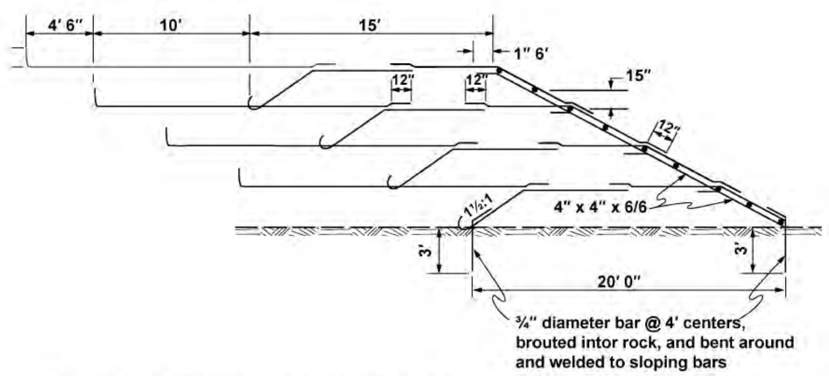


Figure 4.—Construction sequence detail—Des Arc Bayou site No. 3 (Henry, 1977).

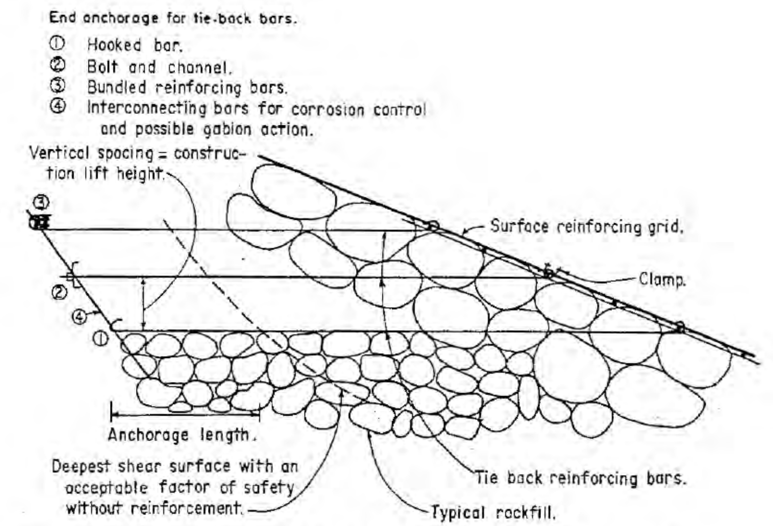


Figure 5.—Schematic detail of tie-back steel reinforcing bars with end-anchorage alternatives (Brown et al., 1983).

Riprap

- Large size rock fragments are sized to resist erosion.

Geomembranes, Geocells and fabric formed concrete

- Synthetic fabrics and cells are frequently used to resist erosion.

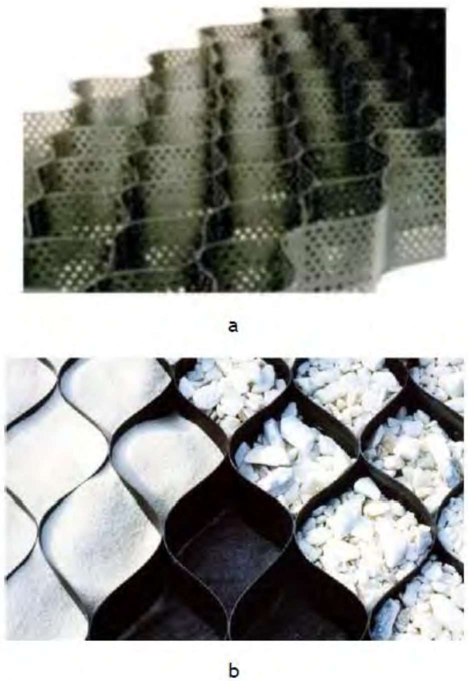


Figure 1.—Examples of a perforated and solid geoweb system with various fill materials (a) Geoweb™ (Source: Presto Products Company); (b) TerraCell® (Source: WEBTEC, Inc).

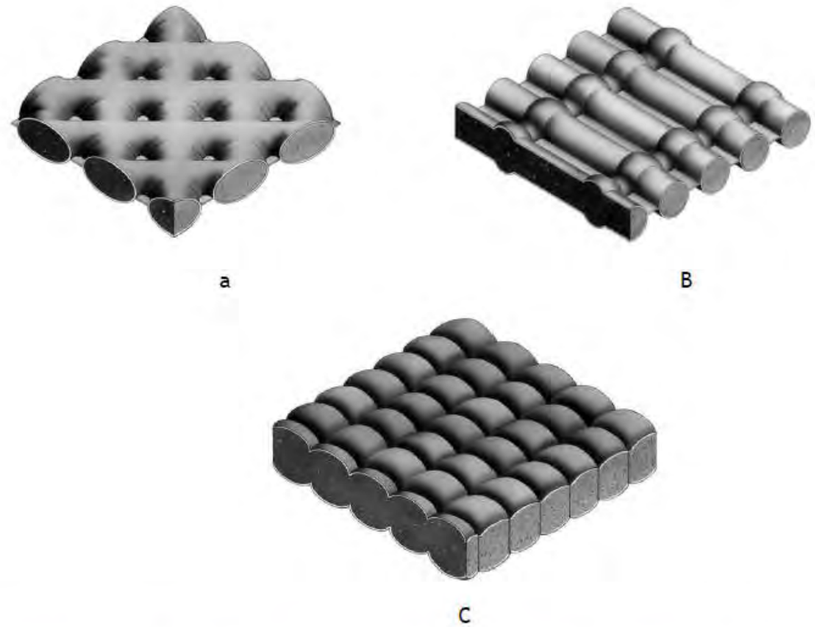


Figure 2.—(a) Filter Point™ fabric form pumped with concrete (Source: Texicon and Hydrotex); (b) Filter Band™ fabric form pumped with concrete (Source: Hydrotex); (c) Uniform Section™ fabric form (Source: Hydrotex).

Geomembranes, Geocells and fabric formed concrete

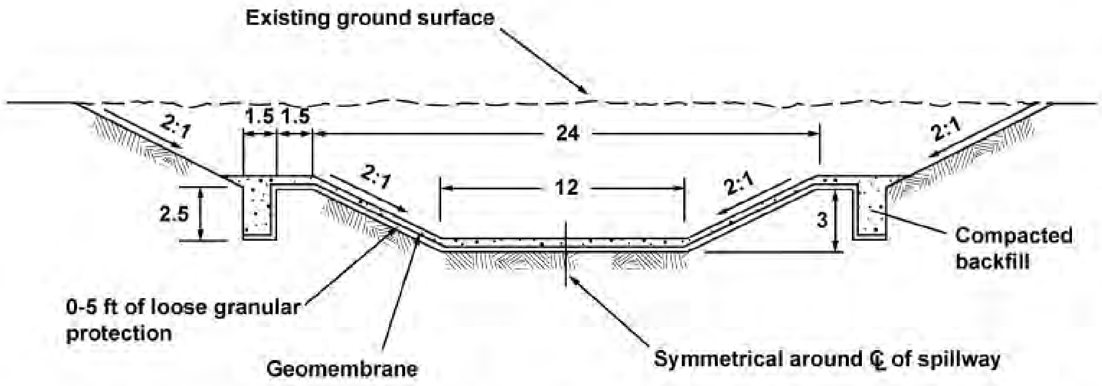


Figure 4.—Typical cross-section, showing the geomembrane installation procedure along the channel.

Figure 7.—Fabric forms being filled with fine aggregate concrete. (Courtesy of Donnelly Fabricators.)

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Open stone asphalt (possible)

Mass Stabilization

MASS STABILIZATION IN GENERAL

Mass Stabilization is a **ground improvement method** for soft soil foundations where the **binder** (cement, lime etc.) is **mixed** to the treated soil. As a result of the treatment, the **strength** and **deformation** properties are **significantly improved** comparing with the original soil.

The mass stabilization job carried out by:

Feeding the binder by **ALLU PF** or **PFM** into the soil whilst mixing it with a **ALLU PMX** mounted on an excavator.

The process is controlled by **ALLU DAC** monitoring & control system.

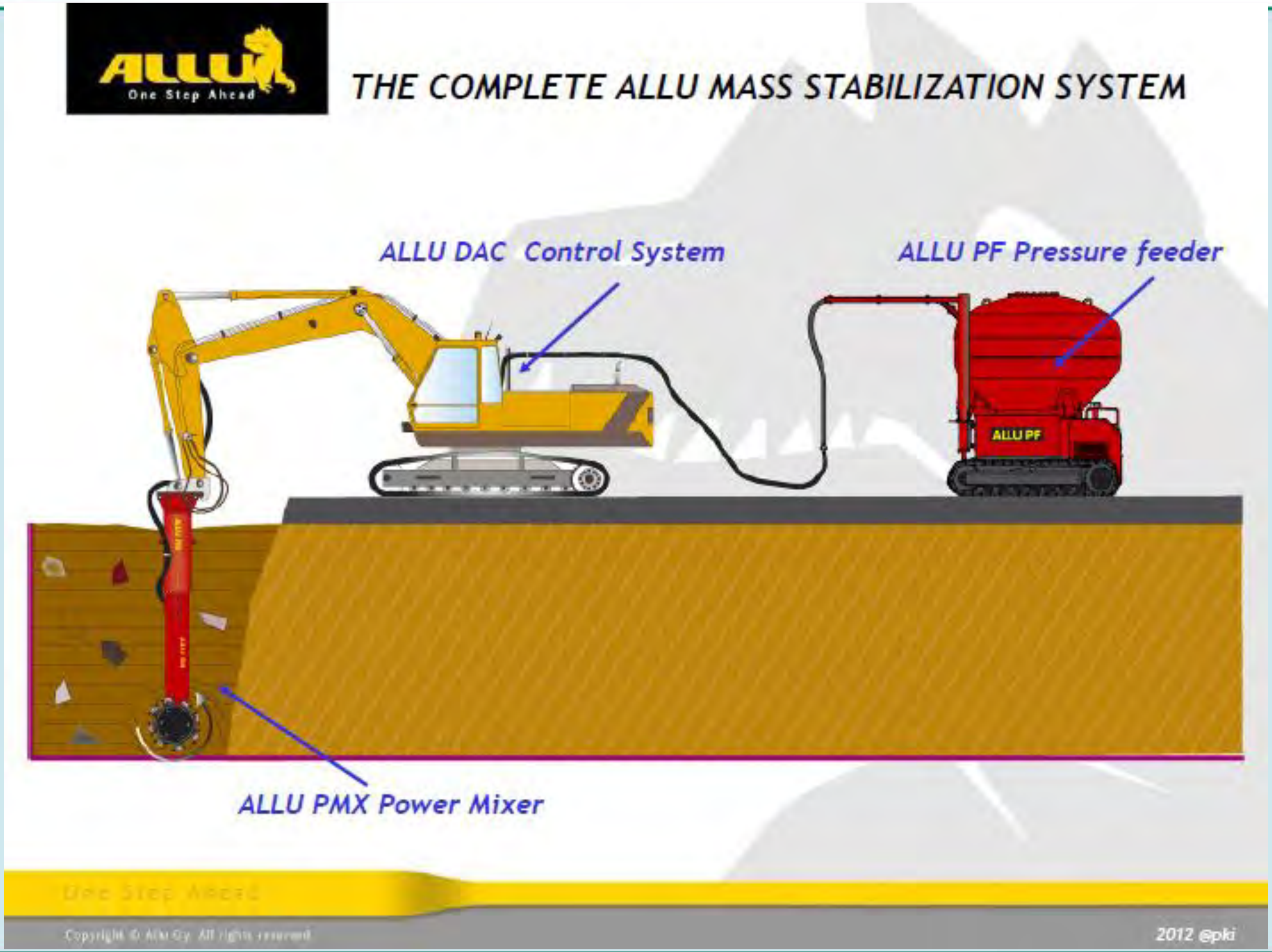
Mass Stabilization-Allu

- Mass Stabilization -
  a versatile technology

  for

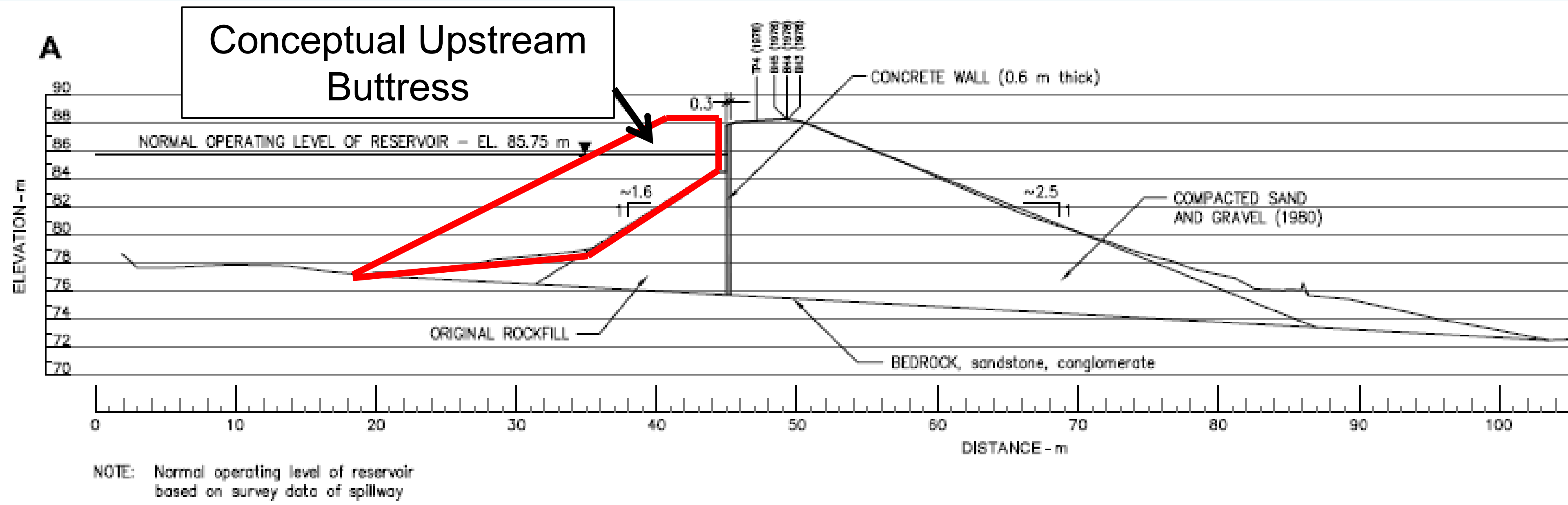
  - in-situ improving of soft soils
  - remediation of contaminated soils
  - improving and utilizing clean and contaminated soft sediments

Mass Stabilization



Conceptual Design Options - Middle Dam Stabilization

- Buttress upstream of Middle Dam to limit deformations
- Find and fill low level outlet



Middle Dam Cross-Section (EBA, 2010).

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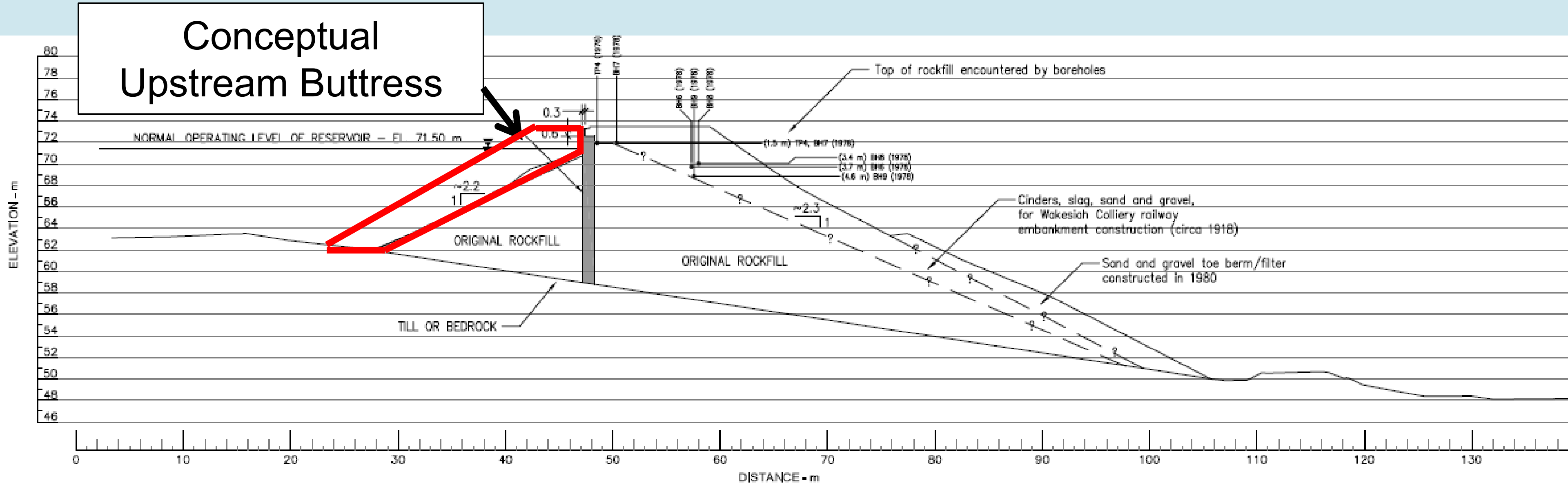
Conceptual Design Options – Lower Dam Stabilization

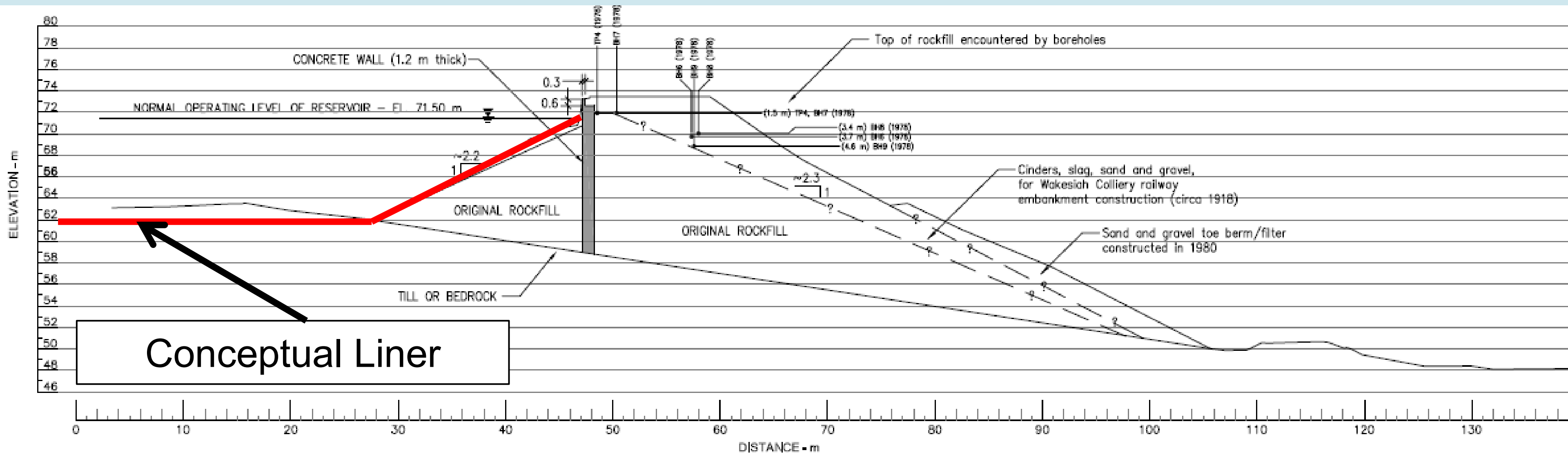
- Buttress upstream on Lower Dam to limit deformations if needed



Lower Dam Cross-Section (EBA, 2010).

Conceptual Design Options - Dam Stabilization

- Install additional barrier (Middle or Lower Dams) – if needed
  - Liner
    - PVC
    - Coletanche
    - GCL
    - Bitumen



Lower Dam Cross-Section (EBA, 2010).

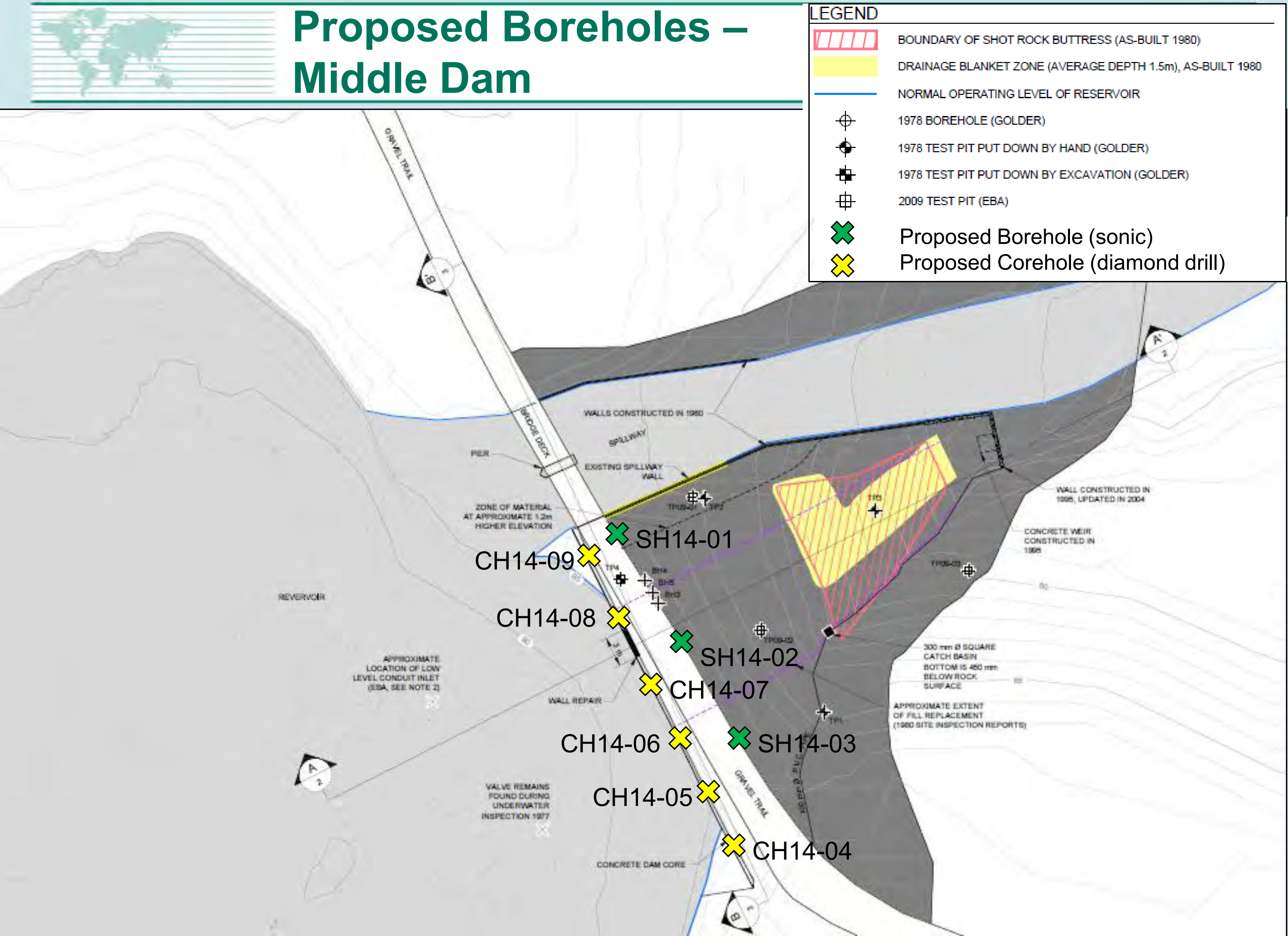
Next Steps – Additional Information

- Collect and re-evaluate existing information.
- Assess need for any additional geotechnical information
  - Geophysical survey on surface of downstream and cores of both dams
  - Borehole or test pit investigations **(mid-February)**
    - Collect information on properties of dam fills
    - Collect information on dam foundations
    - Install water level monitoring instruments
    - Determine dam zonation
- This information needed for design
  - Basis for analysis
    - Input to numerical modelling
    - Piping assessment, etc
- This information needed for construction and tendering
- Additional information on concrete core (re-inforcement)?

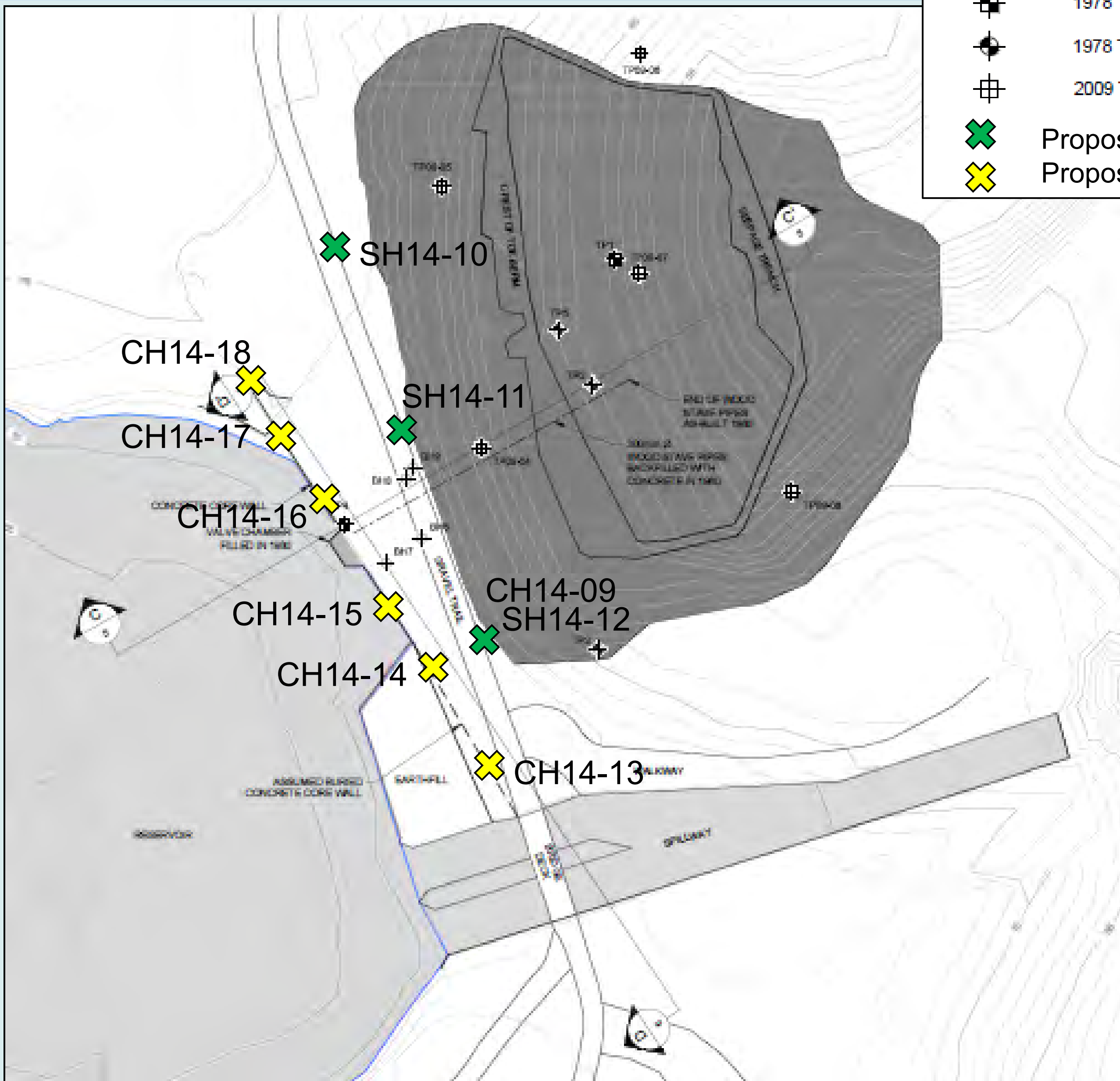
Next Steps – Additional Information

- Additional information on concrete core
  - Concrete quality
  - Concrete uniformity
  - Reinforcement
    - Spacing, condition
- Proposed program
  - Multiple, fully cored holes through core into foundation
  - Downhole geophysics
    - Information on concrete strength, rebar and concrete thickness

Proposed Boreholes – Middle Dam



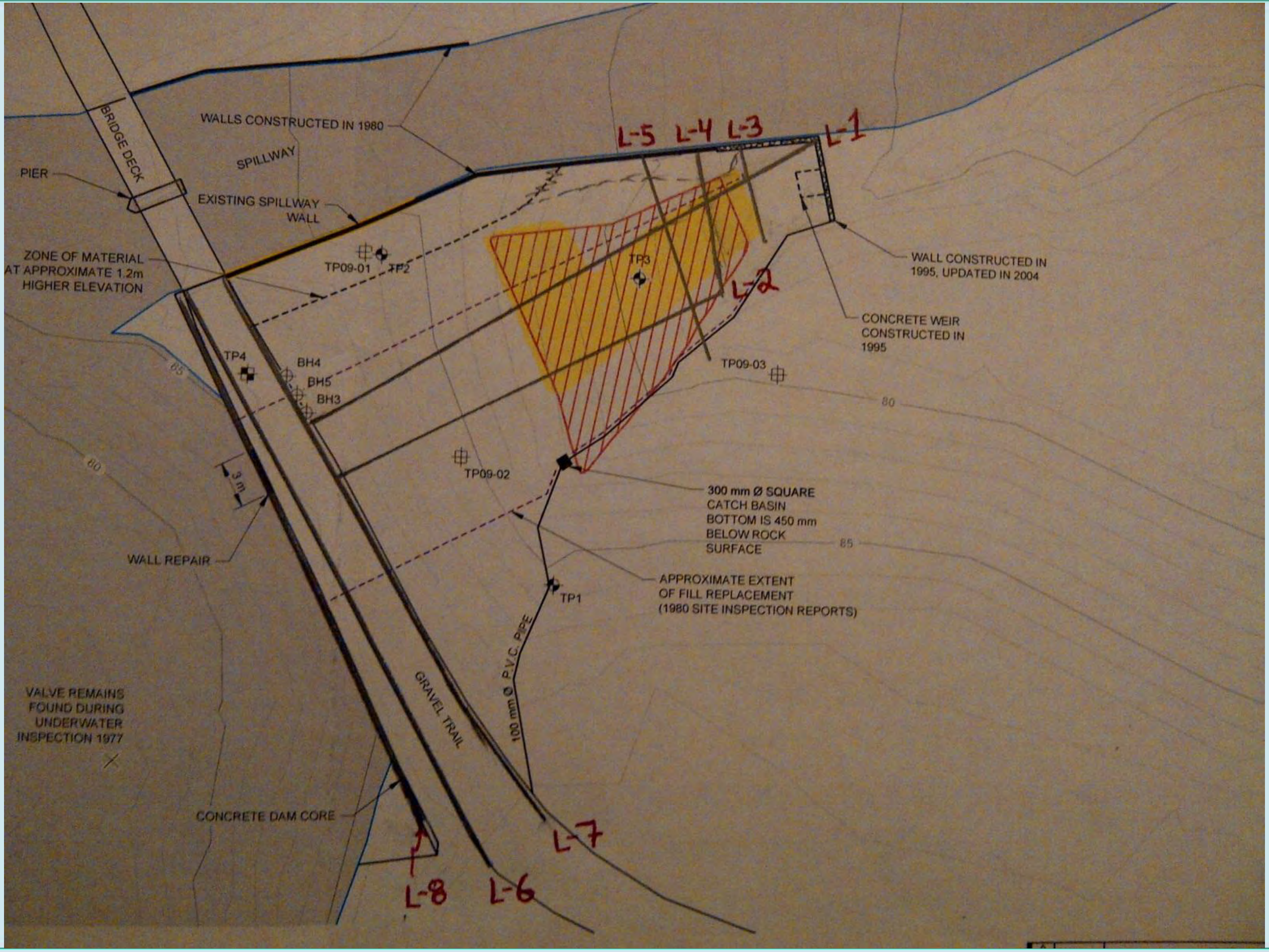
Proposed Boreholes – Lower Dam



LEGEND

- NORMAL OPERATING LEVEL OF RESERVOIR
- 1978 BOREHOLE (GOLDER)
- 1978 TEST PIT PUT DOWN BY HAND (GOLDER)
- 1978 TEST PIT PUT DOWN BY EXCAVATOR (GOLDER)
- 2009 TEST PIT (EBA)
- Proposed Borehole (sonic)
- Proposed Corehole (diamond drill)

Geophysics Survey Lines-Middle Dam

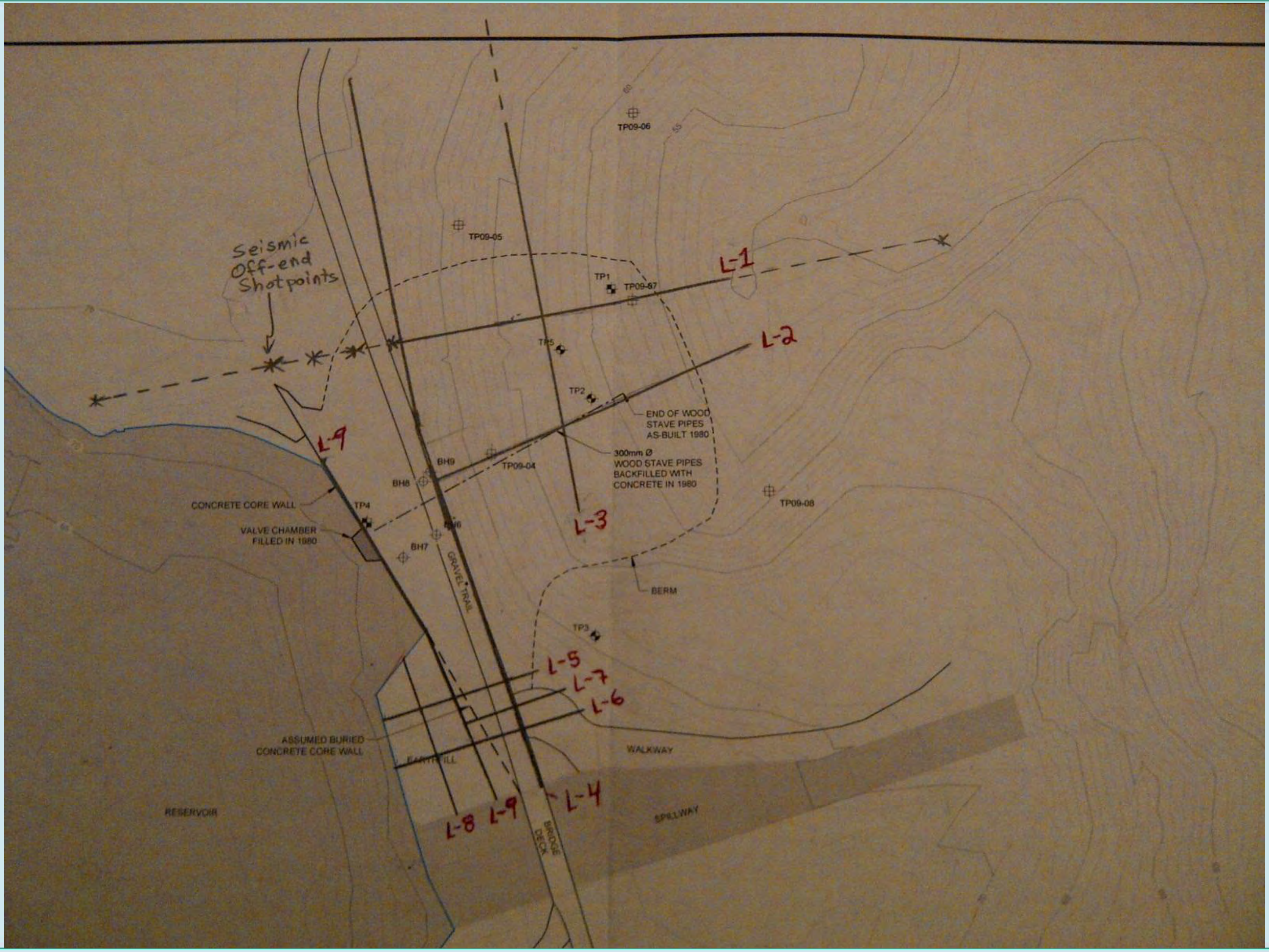


Geophysics Survey Lines-Middle Dam

- Geophysical Survey Coverage consists of **8 GPR profiles**:
  - **Downstream Face (5 profiles) –**
    - 2 profiles from toe to crest
      - Profile nearest the spillway also acquired with the higher-powered/lower frequency GPR system;
    - 3 cross-profiles within lower 20 m of face to focus on LLO;
  - **Crest of Dam (3 profiles) –**
    - one each near the upstream and downstream edges of the crest;
    - one on top of concrete wall.

GPR—High Power 50 MHz-Middle Dam (L-1)

Geophysics Survey Lines-Lower Dam



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Geophysics Survey Lines-Lower Dam

- Geophysical Survey Coverage consists of **9 GPR profiles & 2 Seismic Profiles:**
  - **Downstream Face (3 GPR profiles, 2 coincident seismic profiles)** –
    - 2 profiles from toe to crest near the middle (L-1 & L-2)
      - North Profile (L-1)-Lower Half- also acquired with the higher-powered/lower frequency GPR system
      - L-1 also covered by seismic (MASW and refraction);
    - 1 cross-profile (L-3) along "bench" across middle of downstream face – GPR and seismic (MASW and refraction);
  - **Crest of Dam (7 GPR profiles)** –
    - 1 along downstream edge (~1.5 m from fence, 95 m long);
    - 3 traversing the peninsula (2 west-east, 1 north-south);
    - 3 traversing suspected buried wall (2 are extensions of peninsula profiles);
    - 1 crossing width of crest—extending L-1;
    - 1 along top of concrete wall

Geophysics—GPR 200MHz-Lower Dam (L-1)

GPR—High Power 50 MHz-Lower Dam (L-1)

GPR 400 MHz-Lower Dam, Top of Wall (L-9)

Geophysics Survey- Objectives: To-Date Status

- **Profile internal layering of middle and lower dams:**
  - GPR profiles show some internal layering within both dams (GPR data still to be processed and interpreted);
  - Radar depth of penetration – To Be Determined;
- **Identify "water table," and other possible variations in water saturation, within middle and lower dams (if conditions allow):**
  – To Be Determined;
- **Profile underlying foundation (bedrock or till interface):**
  - Middle Dam – Appears that GPR profile may track bedrock to ~20-25 m up the face from the toe (outcrop observed at the toe). Depth to be determined (GPR data still to be processed and interpreted). Note—the distanced along face from toe to crest is ~40 m;
  - Lower Dam – Does not appear that GPR reached the base; however, one or both of the seismic refraction lines appear to profile bedrock or till. All data still to be processed and interpreted;

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Geophysics Survey- Objectives: To-Date Status

- **Characterize amount of reinforcement (such as rebar) within the concrete wall as discernable from the top of the wall along the dam crest:**
  - **Both Dams** – Depths of radar penetration to be determined (possibly to ~2 m). **GPR data yet to be processed and interpreted**;
  - **Middle Dam** – Possibly see 2 or 3 horizons of rebar (shallowest rebar spaced approximately 0.8 m).
    - Possible layering is apparent (possibly 4 interfaces w/in depth range);
    - Anomalously reflective zones – suggest possible variations in moisture content and/or other properties;
  - **Lower Dam** – Rebar not immediately obvious; however a variety of internal structures are apparent—possibly irregularly spaced rebar and nonexistent in some sections.

Geophysics Survey- Objectives: To-Date Status

- **Middle Dam—Attempt a limited search of the Low-Level-Outlet (LLO) within the lower down-stream face:**
  - GPR data yet to be processed and interpreted;
  - Expect that identifying LLO is unlikely, due to variable ground fills and topography that complicate the images.
- **Lower Dam—Confirm location/existence of buried concrete core-wall through peninsula and obtain information regarding peninsula fill material;**
  - GPR data yet to be processed and interpreted (acquired 3 profiles across expected buried wall);
  - Field plots suggest that the wall is visible, however, it is not obvious due to existence of what appear to be adjacent “blocks of material” (poss. including boulders/cobbles) within the fill that produce similar radar signatures.
  - Within peninsula, material appears to be mostly coarse with 1 or 2 distinct layers

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Geophysics Survey- Objectives: To-Date Status

- **Lower Dam--Obtain general seismic shear-wave velocity (Vs) versus depth (1-D) profile (from MASW—Multichannel Analysis of Surface Waves):**
  - Depth extent of these values to be determined – data yet to be processed and interpreted.

- **Lower Dam– Additionally, obtain general seismic refraction (compression-wave) profiles (2-D), coincident with the two MASW profiles, to further explore basal profile of dam:**
  - Field records suggest sharp bedrock signal w/in northern half of dam, possibly stepping down deeper w/in southern half of dam – data yet to be processed and interpreted.

Next Steps

- Hydrology and hydraulics
  - Recommend that hydrology be further studied to:
    - Account for upstream hydraulic structures/storage.
    - Refine Curve Numbers and Lag Times.
    - Refine Baseflow values.
    - Verify basin delineations (Powerline Dam).
  - Recommend that spillway hydraulics be further studied to:
    - Verify Middle Dam’s rating curve.
    - Verify Lower Dam’s rating curve.
    - Verify Lower Dam’s hydraulic jump caused by convergence.
    - Evaluate consequences of the jump depth exceeding the Lower Dam’s spillway walls and possible ways to remedy.
- Evaluate dam breach characteristics - Time to failure
  - Breach by overtopping – several scenarios
  - Breach due to seismic shaking – several scenarios

Next Steps

- Seismic analyses
  - Conduct site investigations (concrete core, embankments)
  - Evaluate integrity of core – decision point
  - Evaluate seismic resistance of core
    - FLAC analyses – deformation of dam – several scenarios
    - Evaluate response of concrete core – cracking (Herold Engineering)
    - Resistance (post cracking) – potential for dam breach
  - Lower Dam and Middle Dam
    - Middle Dam, with upstream buttress
- Evaluate other Failure modes
  - Piping, static failure
- Updated Inundation Modelling - AE
  - Updated Hydrology, dam breach scenarios
- Updated Damage Assessment – Golder
  - Based on updated inundation models

Next Steps

- Risk Assessment
  - Develop risk model
  - Develop risk inputs
    - Initial risk workshop – subjective inputs (based on current studies)
    - Subsequent risk inputs – based on analyses outlined above.
  - Risk modelling – use to inform selection of remediation options
    - Remediate both dams, or Lower Dam only
    - Inflow Design Flood (IDF) requirements
    - Requirements for design for seismic resistance

Next Steps

- Develop design (dam remediation) options
  - Spillways
    - Spillway improvement options
    - New spillway options
  - Designs for dam overtopping
  - Buttressing of dams
  - Preliminary cost estimates