Springbank Dam North Embankment Repairs

prepared for the

Upper Thames River Conservation Authority

Riggs Engineering Ltd.

1240 Commissioners Road West

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London, Ontario

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May, 2011

RIGGS ENGINEERING

May 3, 2011

Mr. Rick Goldt, CET
Supervisor, Water Control Structures
Upper Thames River Conservation Authority
1424 Clark Road
London, Ontario
N5V 5B9

Dear Mr. Goldt,

Re: London, Ontario
Springbank Dam North Embankment Repairs

Please find attached our final report titled Springbank Dam North Embankment Repairs. We appreciate the opportunity to provide our services on this project.

Yours truly,

Riggs Engineering Ltd.

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Summary

Riggs Engineering was hired by the Upper Thames River Conservation Authority to investigate drainage behind the Springbank Dam north embankment crib wall. Surface runoff and erosion of the valley slope above the crib have washed out the the top meter of fill material behind the crib. Movement of fine particles from the slope to the top of the crib has created drainage problems in the fill behind the crib. The material at the top of the crib is wet and bog-like year round, with a number of small puddles and pools observed even during periods free of rain.

This report focuses on determining possible remedial options for restoring the drainage characteristics of the fill material behind the crib. An inspection of the crib and valley slope above the crib revealed that crib structure is in generally good condition, while the slope is very steep (36 to 40 degrees from horizontal) and currently eroding. The slope is interspersed with vegetation, including some dead trees. The fill behind the crib is no longer free draining, and is responsible for current drainage problems. The top 1.0 m of material near the crib face has been completely lost to erosion, as well as the majority of the fill of downstream return portions of the crib. Restoring the lost material and providing adequate draining of the crib is the focus of the report.

Access to the Springbank Dam north embankment crib is by crossing the deck of the dam. Remedial options may be limited to the weight of equipment and material that will need to be transported to the north embankment. Calculations for the load carrying capacity of the dam deck were carried out. It was determined that the dam deck has capacity for a 6 kPa load, equivalent to a single small pick up truck moving back and forth carrying equipment and material to the top of the crib. Slope stability analysis was also carried out, as extra weight of equipment and material at the top of the crib can theoretically jeopardize the crib and/or the slope above. Our computations confirmed that localized sloughing from the slope are likely. Deep seated failure of the slope that extend below the base of the crib have been computed with safety factors at or near unity for a variety of water levels. The greatest sensitivity of the slope stability have been identified as the location of the phreatic surface. The assumed ground water elevation at the top of the valley slope produces the largest effect on the computed safety factors for global stability. Lesser sensitivities in the global safety factor are noted by changes in the Thames River water level (high and low levels).

Deep seated failure mechanisms extending below the crib wall would likely be preceded by slope failures immediately upstream and downstream of the crib. Valley slope upstream and downstream of the crib are similar in geometry to the slope above the crib.

A number of restoration options were considered as remedial measures for improving the drainage characteristics of the soil behind the crib, and are listed below with estimated costs:

- Option 1 (restore original grade, excavate a trench near crib face and fill with granular material), $68,000,
- Option 2 (install French drain near toe of slope, regrade soil towards drain), $61,250

- Option 3 (installation of a hard engineered surface such as interlocking paving stones, asphalt or concrete), cost not estimated,
- Option 4 (do nothing), $0

Option 1 includes filling the downstream return portion of the crib with crushed rock, and restoring the original grade (2% slope towards the river). The eroded material at the top of the crib is to be removed and small drainage tile installed. The excavation required for the installation of the new drainage tile is expected to be burdensome (as crib's cross ties are spaced every 2.4 m), which is responsible for higher overall costs compared to other options.

Option 2 includes grading the fill material with a 2% slope toward the toe of slope, and installing a swale and a French drain at the toe of valley slope behind the crib. A French drain is a small drainage ditch filled with granular material that would transport seepage and runoff water from the slope and direct it towards the river below. One of the advantages with Option 2 is that it mimics the natural channel that currently exists at the toe of the slope. Other advantages include the ability of the French drain to move quickly the runoff and seeping water from the top of the crib, and the fact that estimated project cost for implementing Option 2 is lower than Option 1.

Option 3 includes a hardened surface at top of the crib, such as a interlocking brick tiles, or asphalt or concrete paved surfaces. Due to relatively high costs of implementing a hardened surface, Option 3 is removed from further consideration.

Consideration was also given to a do nothing strategy (Option 4). The disadvantage of the do nothing strategy is that seepage and erosion would continue. The surface erosion would continue to erode and trap fine particles in the soil behind the crib, which result in the continuation of saturated conditions. Since the saturated soil conditions at the top of the crib exacerbate the global safety factors, the do nothing option is not recommended.

Option 2 (French drain) is recommended as the potential solution for the Springbank Dam north embankment crib restoration. It is important to note that Option 2, even though it will improve drainage of the soil behind the crib, will not improve the overall stability of the slope (currently estimated as nearly unity). The global stability safety factors before and after the proposed remedial measures are not expected to change as the low global stability safety factors are dependent on slope geometry (the valley slope is very steep), phreatic surface (i.e., the ground water level), and the soil strengths. The location of the phreatic surface has been determined to be most sensitive to the computed safety factors, and in particular the ground water elevation at the top of the valley slope. The water level of the river has been shown to influence the safety factors, but not to a significant amount. Furthermore, monitoring of the valley slope should be undertaken at regular intervals and be compared to a base case (such as this report).

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1. Introduction and background

The purpose of this document is to present results on the preliminary engineering investigation of the drainage behind the Springbank Dam north embankment crib wall. An inspection of the valley slope and soil retained by the crib wall indicates that some remedial actions are required. The remedial measures are related to surface erosion of the soil retained by the crib, as well as slope drainage and seepage management. Additional fill material and re-grading of the soil at the top of the crib will be necessary. Throughout this report terms slope and valley slope are used interchangeably, and refer to the valley slope above the Springbank Dam north embankment crib wall.

Springbank Dam (see Figures 1 and 2) was designed and built in 1929 to provide the reservoir for water supply and recreation for the City of London. The dam was extensively rehabilitated in 1967, where deteriorated concrete was repaired, a new pier and sheet pile wall installed on the south bay. A number of mechanical components of the dam were upgraded. The concrete crib on the north abutment was also constructed in 1967 and additional work was done on the revetment downstream of the dam. A number of erosion protection measures were implemented throughout the years.

The Springbank Dam (see Figure 3 for section of the dam) has been operated in the past for the purposes of retaining water for recreation purposes. Prior to recent rehabilitation, Springbank Dam was operated with automatic water level controls using three gates during the summer months (May to November). The operating target water level was at EL. 229.4 m. During winter periods (November to April) the gates and timber stop logs were removed, ensuring that the dam freely passes all flows without raising the water level upstream. The Springbank Dam was designed to pass flows with probability of exceedence of 0.004 any given year (1/250 yr flood). During most recent rehabilitation, new gates were installed on the Springbank Dam. However, during the testing of the gates in June of 2008, a malfunction was detected that permanently lowered the gates. Currently, the gates remain in the down position.

The concrete crib on the north embankment is the focus of this project. The original 1928 drawings show that north abutment consists of a bulkhead wall that extends 12 m into the riverbank, followed by a check for sheet piling. A check is a slot in the concrete bulkhead wall that accommodated installation of sheet piling further into the bank. The original drawings from 1928 show 15 m of sheet piling extended into the riverbank past the concrete bulkhead. Currently, a concrete wall extends some 10 m upstream and 10 m downstream from the dam (exact extents were not provided to us for review), located at the base of the crib structure. The concrete crib (built in 1967) retains the fill on the north embankment, upstream and downstream of the dam. The crib (see Figure 4) consists of precast interlocking beams (305 mm by 305 mm by 2240 mm) resting on a concrete footing. Asphalt shingles are placed as elastomeric bearing

FIGURE 1
SITE PLAN

SPRINGBANK DAM NORTH EMBANKMENT REPAIRS

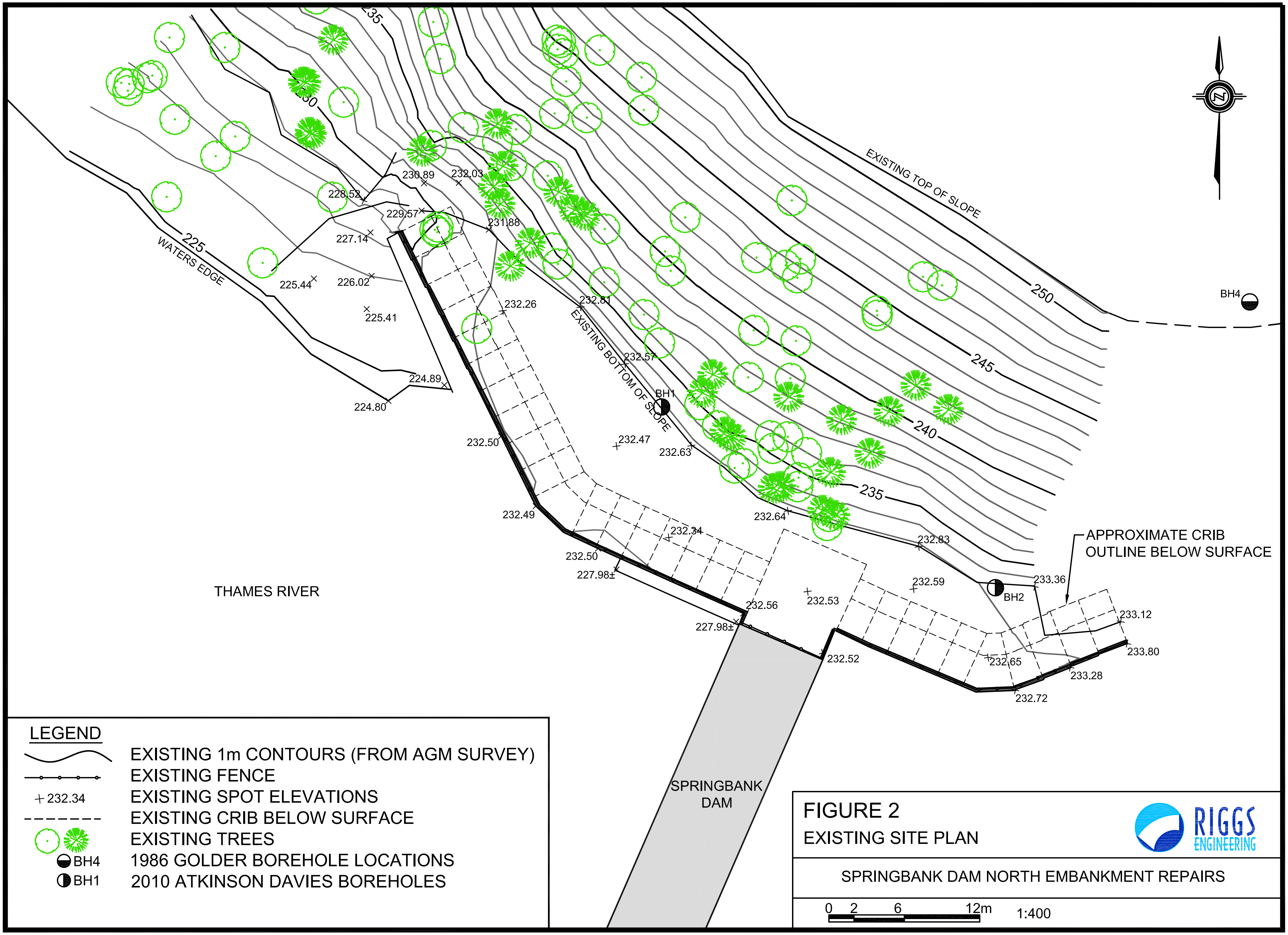

FIGURE 2
EXISTING SITE PLAN

SPRINGBANK DAM NORTH EMBANKMENT REPAIRS

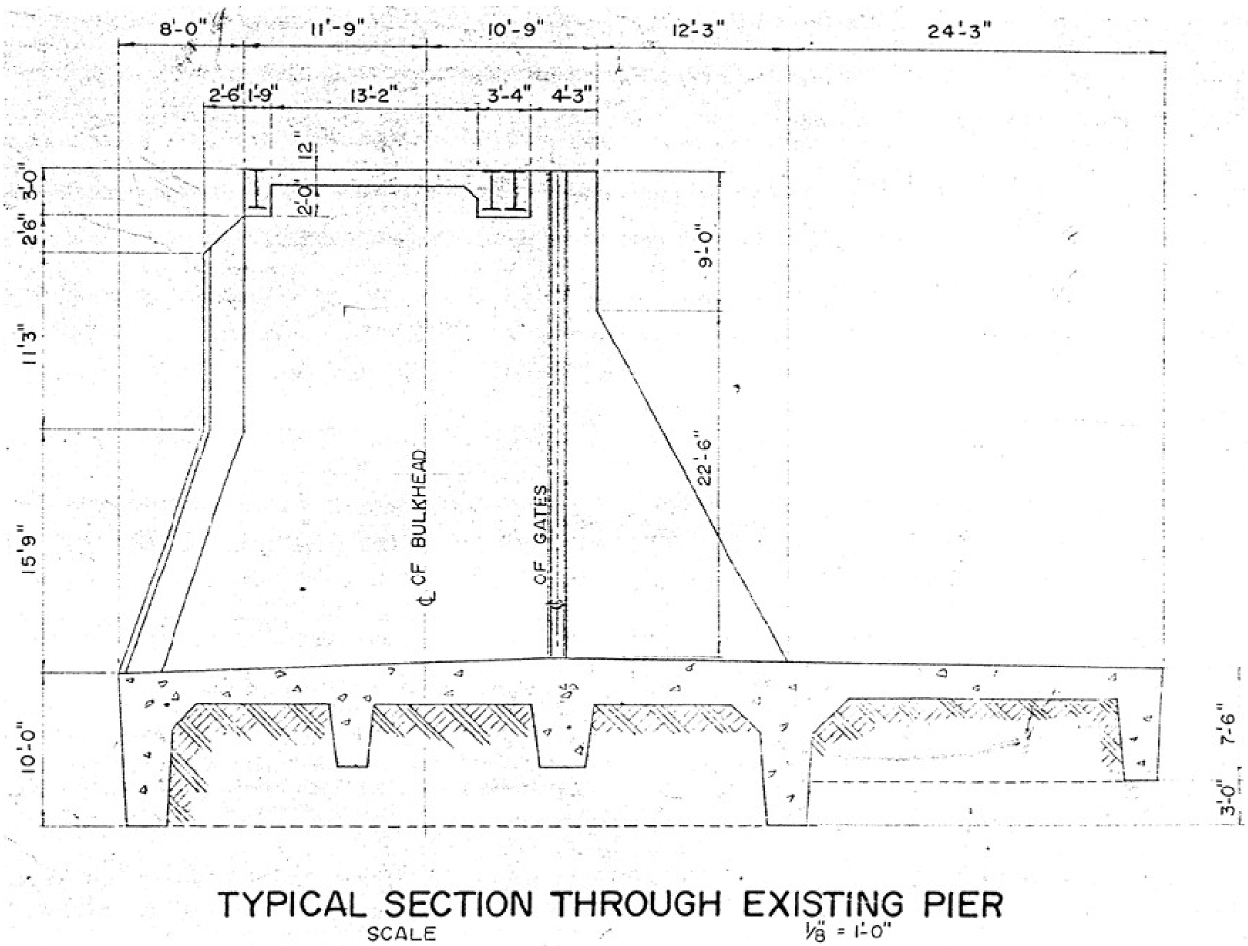

Figure 3: Springbank Dam typical section (extracted from 1967 rehabilitation drawings)

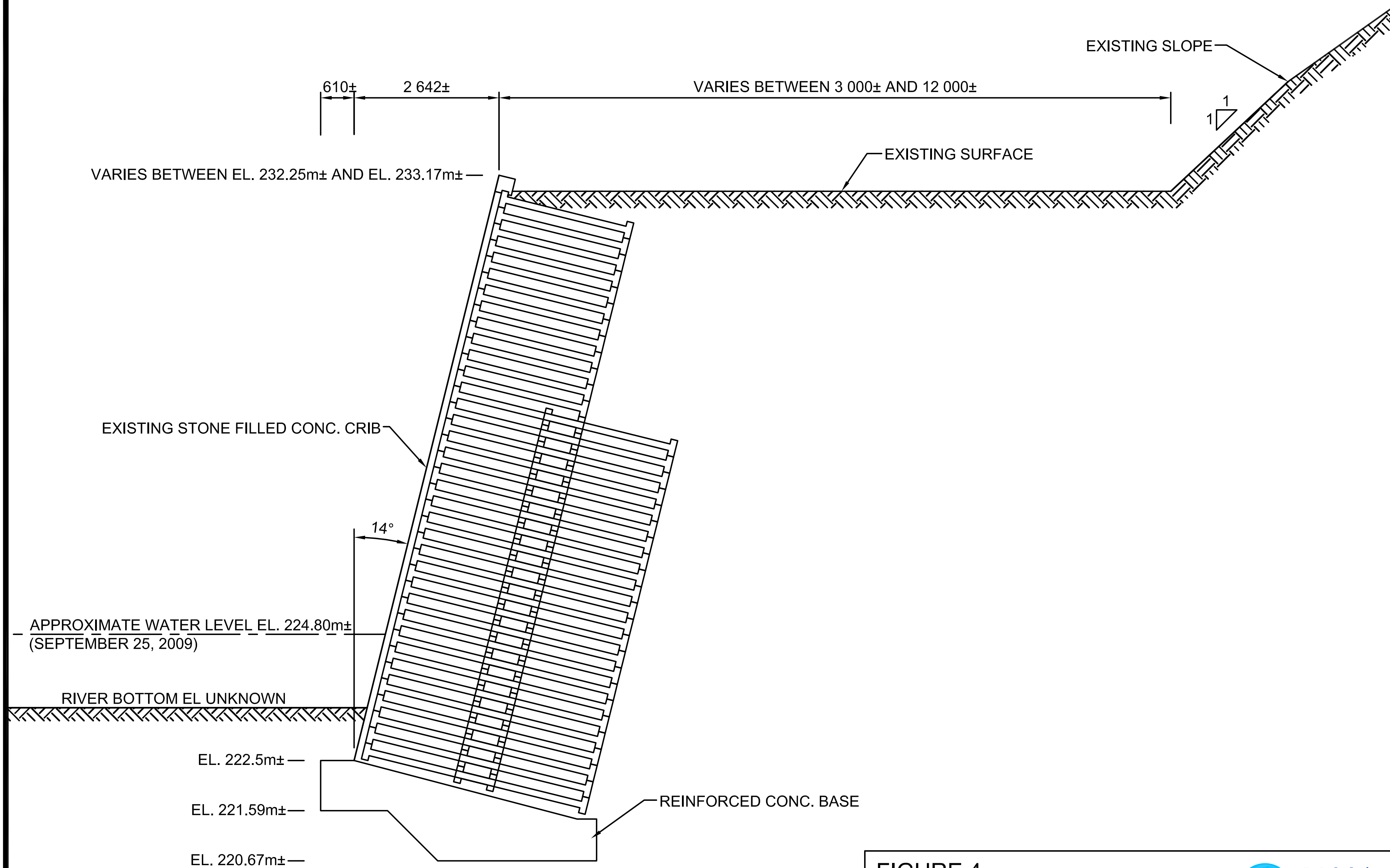

FIGURE 4

TYPICAL SECTION OF EXISTING CRIB

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SPRINGBANK DAM NORTH EMBANKMENT REPAIRS

0 0.5 1.5 3m 1:100

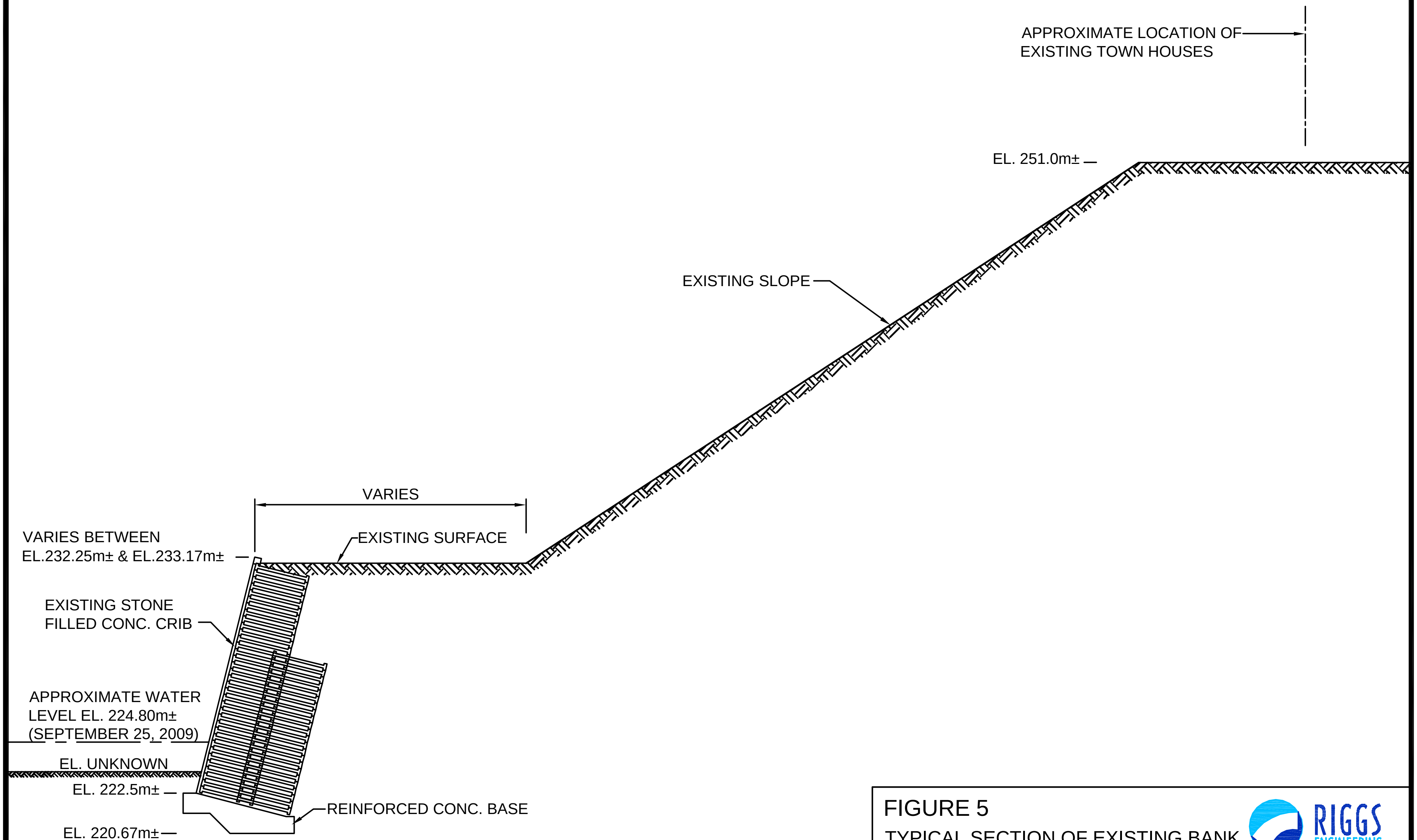

FIGURE 5
TYPICAL SECTION OF EXISTING BANK

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SPRINGBANK DAM NORTH EMBANKMENT REPAIRS

0 1.25 3.75 7.5m 1:250

pads for the precast concrete pieces making up the crib. Although filter fabric was not specified in the original drawing, it is visible over some sections of the crib. Figure 5 shows the crib as well as top of slope.

Upstream and downstream portions of the crib are generally in good condition. The Dam Safety Assessment report from 2002 notes that the fill at the top of the crib was removed by erosion of the slope above the crib. Currently, material at the top of the crib does not drain freely (surface puddles were evident from our inspections in October of 2009, and again in January of 2010). The soil at the top of the crib was also observed to be wet during the inspection performed for the 2002 Dam Safety Assessment report. Even though the top of the fill is wet, virtually no seepage was observed to occur through the crib face. This was confirmed by our inspection in late 2009 and again in early 2010. Also noted in the January 2010 inspection (undertaken at the time of the geotechnical investigation by Atkinson Davies Inc.) was a small surface drainage channel that formed at the toe of slope (on top of the crib), eventually taking the water to the ends of the crib and into Thames River.

Extensive vegetation is observed on the faces of the crib, and is supported by sand and silt that has accumulated during periods of high runoff and erosion. One of the recommendation from the 2002 Dam Safety Assessment report is to perform maintenance and improve the surface drainage and seepage of water from the slope to a downstream outlet. Currently, the soil at the top of the crib is not free draining, is typically wet with small pool and puddles readily visible (even during periods free from rain).

Prior to undertaking the recommended repairs, an investigation is needed to assess if placement of construction equipment (for possible remedial measures of the crib) on top of the dam deck will jeopardize the stability of the dam. Structural analysis is therefore necessary to identify constraints and limitations the deck structure may impose on the future remedial measures of the crib.

Placement of construction equipment and fill material on top of the crib may also exacerbate stability of the slope above the crib. Slope stability analysis of the slope above the crib is therefore necessary to identify if placement of construction equipment and fill material will reduce stability of the slope.

Assessment of the current conditions of the crib, slope above the crib, and the dam deck is presented in Section 2. Summary of calculations of the carrying capacity of the Springbank Dam deck is outlined in Section 3, while Section 4 presents results from our slope stability analysis. Lastly, Section 5. outlines our proposed options for the restoration of the crib, while Sections 6 and 7 presents conclusions and recommendations.

1.1 Sources of information

The following documents were used in preparation of this report:

- Springbank Dam Drawings (1928). H.G. Acres & Co. Limited, Consulting Engineers, Niagara Falls, Ontario, April 1928.
- Springbank Dam Rehabilitation Drawings (1967). Peter T. Mitches & Associated Limited, Consulting Engineers, London, Ontario, August, 1967.
- Borehole No. 4, from Development of Condominiums at top of slope at Springbank Dam (1986), Golder Associates Limited, London, Ontario. (Note that Golder Associates Ltd. provided us with the borehole taken at top of the slope which is closest to the crib. Complete Golder's report was not available for our review.)
- Upper Thames River Conservation Authority, Dam Inspection Reports (1995). B.M. Ross and Associates Limited, Consulting Engineers, Goderich, Ontario, October, 1995.
- Dam Safety Assessment Report for Springbank Dam (2002). Acres Intonational, Oakville, Ontario, May, 2002.
- Topographical Sketch of part of Thames River, West of the Springbank Dam (2008). Archibald, Gray & McKay Limited, London, Ontario, September 2008.

2. Existing conditions

2.1 Crib and slope

As part of our work, an inspection of the crib wall, the fill retained by the crib, as well as the slope above the crib was performed by Riggs Engineering. In general, the conditions described in the 2002 Dam Safety Assessment report were verified. The following observations were noted:

- At the downstream edge of the crib, surface runoff forms a flow pathway that runs past the downstream edge of the crib. Water is constantly seen seeping past the downstream extend of the crib (see Photos 1 and 2).
- The downstream return portion of the crib wall has been washed out from surface runoff and erosion (see Photos 3 and 4).
- The upstream return portions of the crib are protected by large size riprap that exends up the slope (see Photo 5). Some soil material is lost, although significantly less than the downstream return portions.
- The slope on the upstream side of the crib does show runoff from seepage and/or the top of crib (see Photo 6)
- The slope above the crib is covered by brush type vegetation, and is bare in some sections. Dead trees are prominent along the slope.
- The slope was measured (with a hand held slope inclinometer) to be in the range between 36 and 40 degrees from the horizontal.
- Small portions of the valley slope were observed to be nearly vertical and currently eroding. Some portions of the slope are held together by tree roots, some of which are currently dead (see Photos 7 and 8).
- The fill at the top of the crib is overgrown with vegetation, and appears wet even during periods free of rain. Seepage and surface runoff from the slope ends up on the fill on top of the crib and remain there in puddles and small pools (see Photo 9).
- The top 1.0 m of the fill material at the top of the crib has been washed away by surface runoff and erosion from the valley slope (see Photo 10). The surface above the crib is irregular, is abundant with brush vegetation, and appears to be marsh and bog-like.
- During our January 2010 inspection seepage from the valley slope (from top of the crib) was observed to form small drainage channels that eventually take the water to the return portions of the crib. Photos 11 and 12 show the seepage channel on the top of the crib on the downstream and upstream sides of the crib, respectively.

2.2 Erosion protection works

The downstream portion of the crib's slope to the river is partly protected by large size riprap that has over time embedded into the side slopes (see Photo 1). The original dam construction from the 1920's shows that below the spillway a 7 m cast-in-place concrete apron was installed. In the 1968 construction of the crib, 0.6 m thick grouted riprap was placed for an additional 30 m length downstream. Based on visual inspections performed in October of 2009 and again in January of 2010, it appears that the grouted riprap has not undermined the base of the crib on the downstream side.

The riprap erosion protection on the upstream side of the crib was also inspected during low water levels in January of 2010. The riprap on the upstream side of the crib protects the north bank of Thames River for a distance of approximately 150 m upstream of the dam (see Photo 13). Our inspection focused on the riprap that may have the potential to impact the stability of the crib, and was limited to a distance of about 30 m upstream of the dam. The riprap in this zone extends from the river bank onto an approximately 2H:1V slope to a vertical distance that aligns with the return portion of the crib (see Photos 5 and 6). Photo 14 show the riprap on the banks of the slope upstream of the crib. The upstream riprap varies in size from 0.1 m to 1.0 m. Even though some cracking of the stones are evident, the riprap is in generally good condition. Visual inspection also confirmed that no scour holes were evident in the river bottom in front of the crib on the upstream side (see Photo 15).

2.3 Dam deck

A review of the 1928 drawings and an inspection of the Springbank Dam deck were undertaken by staff of Riggs Engineering. Our inspection pertained only to the examination of the deck for the purpose of estimating its load carrying capacity for currently proposed remedial operations. Detailed inspection of the underside of the deck was not undertaken. Complete condition survey of concrete and steel structural components were not included in the scope of work for this project, and are therefore not provided.

Springbank Dam superstructure consists of four reinforced concrete piers spaced every 15 m. The deck spans the piers and is supported by three 15 m long girders (see Figure 6). Between the steel girders is a one-way reinforced concrete slab, that spans 4.0 m (13'-2”). The deck contains holes for lifting and locking old gate guides, spaced approximately 3 m on the upstream side of the deck. The holes are 0.9 m by 0.3 m in plan, and spaced every 3.0 m (in the north-south direction).

During our inspection of the deck, cracks were observed in the deck slab near locations of every hole for the gate guide. The cracks in every case span from the hole (upstream side) to the downstream edge of the slab (the location of the two girders). A photograph of the deck is shown in Photo 16, while the cracks spanning the length of the one-way slabs are shown in Photo 17. The deck cracks identified above were not documented in previous inspection reports (from 1995 and 2000), nor in the Dam Safety report of 2002. Since previous inspections did not focus on evaluating the structural capacity of the deck, it is likely that the above cracks would not have been reported. The observed cracks likely appeared few years after the construction of the dam's deck, and were likely not reported by others because of their insignificant nature.

The deck on the dam is in generally good condition. Previous inspection reports (from 1995 and 2000) indicated that north end of the deck has a transverse crack that should continually be monitored. A photograph of this crack is provided in Photos 18 and 19. Other portions of the deck were previously observed to have minor cracking, as well as some surface water ponding on the deck.

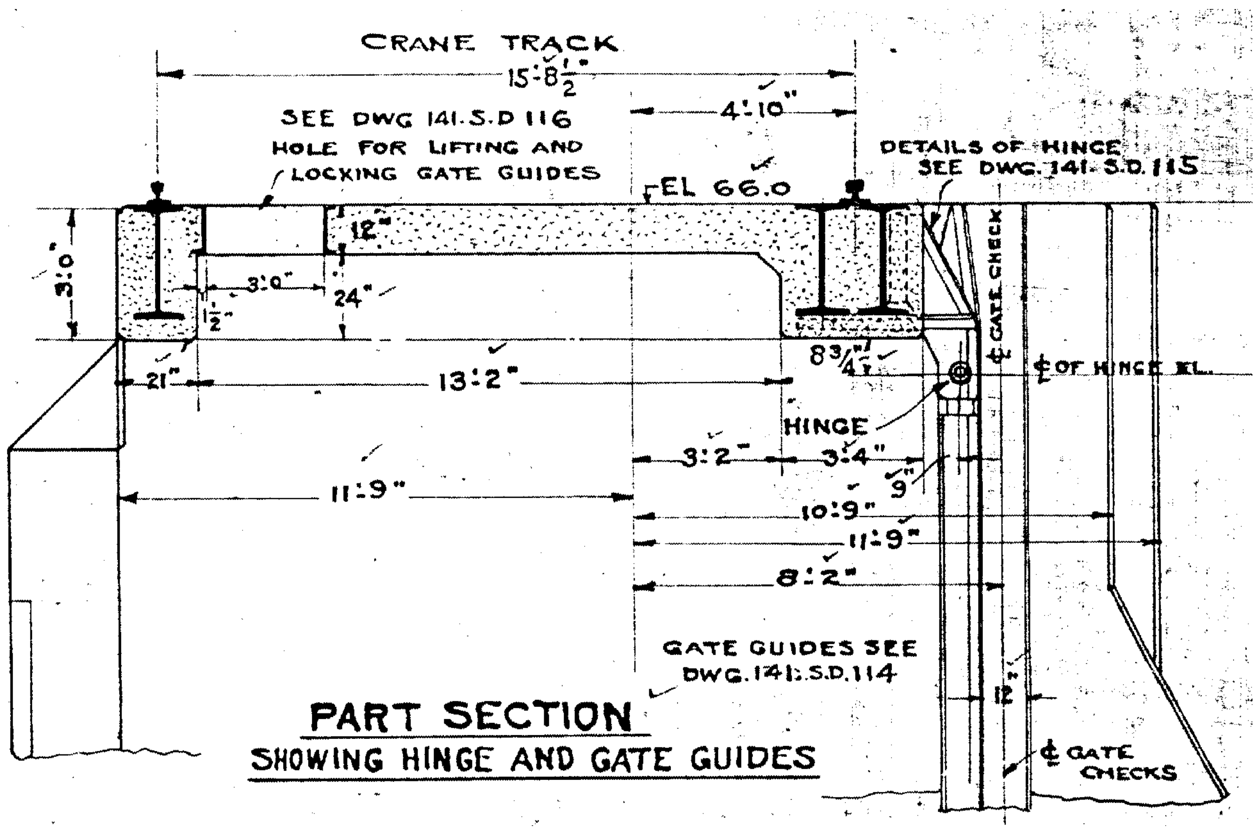


Figure 6: Typical section of the deck, extracted from 1928 drawings

3. Deck load carrying capacity

We have undertaken the necessary inspections of the Springbank Dam deck for the purpose of determining the load carrying capacity of the deck for the proposed repairs. Future remedial operations of the crib wall and/or valley slope above the crib may require service vehicle(s) to move across the dam. Furthermore, supply and re-grading of the fill material for the top of the crib, as well as drainage management will require transportation of fill and other materials across the dam. The amount of material that can be transported will be governed by the load carrying capacity of the deck.

Our inspection of the deck and analysis of the 1928 drawings were used to assess its capacity for the feasibility of the proposed remedial options. The limiting cases were determined to be the capacity of the girders spanning the piers of the dam (15 m in length), as well as reinforced one-way slabs spanning between the girders (see Figure 6). The capacity calculations included checks using steel (Fy = 250 MPa) and concrete strengths (fc' = 20 MPa) typical of the 1930's era. These strength values used are typical in evaluations of older structures.

In order to check the capacity of deck, we have used the loading conditions specified in the National Building Code of Canada (NBCC, 2005) for design of garage structures for light trucks and unloaded buses. The applied live load according to NBCC is 6 kPa, and is approximately equivalent to a single one tonne truck running back and forth carrying fill material to the north embankment crib.

Drawing from 1928 were used to locate size and spacing of the reinforcing steel of the one-way deck slab that spans a total of 4.0 m (in the upstream-downstream direction). Size of the reinforcing steel shown on the drawing are 19 mm diameter bars, spaced every 150 mm. Bending moment capacity of the one-way slab sections with and without holes for the old guide gates was shown to be adequate for the 6 kPa live load criterion. Furthermore, steel girders spanning the pier (span of 15 m) are estimated to have enough reserve capacity to carry the 6 kPa live load distributed over the deck on the entire deck structure.

It has come to our attention that improvements to the control building in 2007 included some extra weight on top of the deck slab. Our analysis shows that existing structures (steel girders and reinforced concrete slabs) have enough reserve capacity to adequately above specified loads.

It is noteworthy to mention that there may be capacity in the structural members to have some material stockpiled on the deck. However, due to age of the structure and the possibility of the fill material escaping and falling into the river, stockpiling of material on the deck is not recommended.

4. Slope stability analysis

The Springbank Dam north bank valley slope is quite steep (measurements of slope range from 36 to 40 degrees from the horizontal), and is partially held by the crib. Slope stability analysis was performed for the valley slope above the crib in order to investigate if the additional fill material to be placed as part of potential remedial measures would put the slope in jeopardy.

In order to assess the slope, we have obtained borehole data from Golder Associates, and Atkinson Davies, Inc. The original borehole was part of Golder's geotechnical investigation for the development of condominiums located on the top of the riverbank slope (see Figure 1). Note that the Golder's borehole from 1986 was taken at the top of the slope, upstream of the Springbank Dam. Only the borehole closest to the crib was provided to Riggs Engineering, together with its approximate location. Golder's original report to the company who carried out the condominium development was not available for our review.

Geometry of the crib was extracted from the 1967 rehabilitation drawings, while the slope geometry was inferred from an AGM survey from 2008 (of the islands immediately downstream of Springbank Dam and the valley slope above the crib). Golder's borehole was used to characterize the soil profile of the slope, as well as groundwater conditions. A conservative assumption was made that 0.6 m of riprap material with tremmie concrete at the base of the crib (resting on the river bed) does not provide passive resistance in stability calculations.

The soil stratigraphy was inferred from the Golder's borehole, and consists of layers of dense sand and gravel (with some silt) from the top of slope (EL. 250.8 m) to approximately EL. 227.5 m. Past the dense sand layer rests hard grey clayey silt till that likely behaves like a brittle solid (Atterberg Limit's test indicate that natural water content of this layer is below the plastic limit). Water level during drilling by Golder in 1984 was observed at EL. 235.0 m. The design peak flows and water levels for the Springbank Dam are shown in Table 1 (taken from the 2002 Dam Safety Assessment report). Even though above conditions are no longer valid (i.e., the current dam does not have stop logs), Table 1 presents a range of design flood water levels at the base of the crib. The target operating water elevation upstream of the Springbank Dam has historically been 229.4 m, some 3.0 m below the top of the crib. The range of water levels identified above has been used in determining the phreatic surface, a requirement when performing slope stability analyses.

Table 1: Design peak flows and water levels at Springbank Dam (taken from the 2002 Dam Safety Assessment report)

Spring rain on snowmelt			Summer storm		
Return Period (yr)	Peak flow (m3/s)	Peak Level (m)	Return Period (yr)	Peak flow (m3/s)	Peak Level (m)
2	580	228.58	2	189	230.30
5	821	229.65	5	344	231.20
10	979	230.29	10	460	231.71
20	1130	230.92	20	584	232.20
50	1330	231.72	50	765	232.70
100	1470	232.04	100	916	233.00
250	1667	232.51	250	1080	233.60

Notes:
Dam deck is at EL. 233.30 m.
Spring rain on snowmelt assumes all stop logs and support beams to be removed for winter.
Summer storm assumed all stop logs left in place, and vertical lift gate open.

The layer of dense sand deposit encountered in Golder's borehole 4 taken at the top of the slope (extends down to EL. 227.5 m, or some 6 m below the top of the crib) is inferred to have a relatively high permeability that facilitates movement of water quickly. The layer of hard clayey silt till, found below the layer of sand is very dense and highly compact. The permeability of the clayey silt till is likely to be orders of magnitude lower than those of the dense sand above it. Therefore, the clayey silt layer will in all likelihood act as an impermeable boundary to the seeping water from the slope moving towards the river.

As part of this work, a small scale geotechnical investigation was undertaken by our consultant, Atkinson Davies Inc. The purpose of the extra geotechnical work was to verify the elevation of the clay till layer at the top of the crib. It was feared that if the clay till layer is present at a different elevation than provided in the Golder borehole, the seepage and groundwater drainage characteristics might be different than can be reasonable inferred from the Golder's borehole. Different clay till elevation might also affect the type of solution that can be proposed to address the drainage issues. Due to the limited accessibility of the site, a hand held drill was used to place two shallow boreholes on the top of the crib. The depths drilled were 3.1 and 3.7 m below the top of the crib. The clay till material was not encountered in the shallow boreholes (only fill material was found). It is noteworthy to mention that the water level after drilling was found to be approximately 0.3 m below the surface at the toe of the slope for both of the shallow boreholes. The Atkinson Davies borehole report and the letter summarizing inspection notes and soil parameters are attached to the appendix of this report.

The slope stability calculations were performed using plane strain finite analysis code Plaxis 2D. The advantage of the finite element analysis over classical limit equilibrium methods is that failure surfaces in the soil continua do not have to be initially assumed. With 2D plain strain finite element modeling, the failure surface with the lowest factor of safety is automatically determined. Even though the finite element analysis does not require assumptions regarding failure surface(s), it does require an assumption of the soil constitutive model (where its strength and stiffness characteristics are specified). Note that in classical limit equilibrium analysis, strength characteristics are needed, as are assumed failure surfaces. For this and other work, we have successfully implemented the Mohr-Coulomb constitutive model with Plaxis 2D for representing the soil continua. Estimating stiffness values of dense sand and the clayey silt till layers was completed by using typical values recommended from the literature and our engineering experience. The assumed stiffness values for finite element calculations of safety factors are relatively unimportant, as deflection of the soil continua are not of interest. The strength parameters are what govern the failure of the slope. The soil parameters were provided by our geotechnical expert, and are listed in Table 2. Estimates of permeability shown in Table 2 are based on typical values available in the literature. A conservative load of 30 kPa was placed on top of the slope (10 m away from the edge of the slope) to take into account weight of the condominiums.

Table 2: Soil parameters used in the slope stability analysis

Top EL (m)	Layer Name (-)	Unit Weight (kN/m3)	Friction Angle (deg)	Cohesion (kPa)	Permeability (m/s)
250.4	Dense Sand	22	40	0	1.0E-04
227.3	Clay Till	22	28	12.5	1.0E-08

4.1 Slope stability results

Our slope stability analysis indicates that computed safety factors are highly sensitive to the assumed level of the groundwater table. For our calculations the groundwater level was assumed to be present at EL. 243.0 m at the right boundary of our model (35 m north from the top of the slope). The water level at the base of the crib (river level) was assumed to be at 224.5 m for the downstream sections of the crib, and 229.4 m for the upstream section (dam's high water level). Permeability constants for dense sand, and the clayey silt till from Table 2 were used to perform a steady state seepage analysis in order to estimate the phreatic surface within the soil mass. The computed phreatic surface produced results similar to what was observed during the field work undertaken in January 2010 by Atkinson Davies Inc.

The finite element model set-up is shown in Figure 7 (where the phreatic surface, uniform load and material boundaries are shown), while a result from the base case is shown in Figure 8. The resulting failure surface is shown as deep seated slope failure, extending below the footing on which the crib rests. Note that in our analysis, we have prevented shallow slides from occurring along the slope face by using a material property through which failure surfaces can not pass. This assumption is consistent with recommendation from Terraprobe's manual “Geotechnical

Principles of Stable Slopes" (MNR, 1998) where computed slides must be at least 2 m deep (MNR, 1998, p. 165). The shallower slides, even if they have lower safety factors, can be controlled by surface stabilization measures and vegetation.

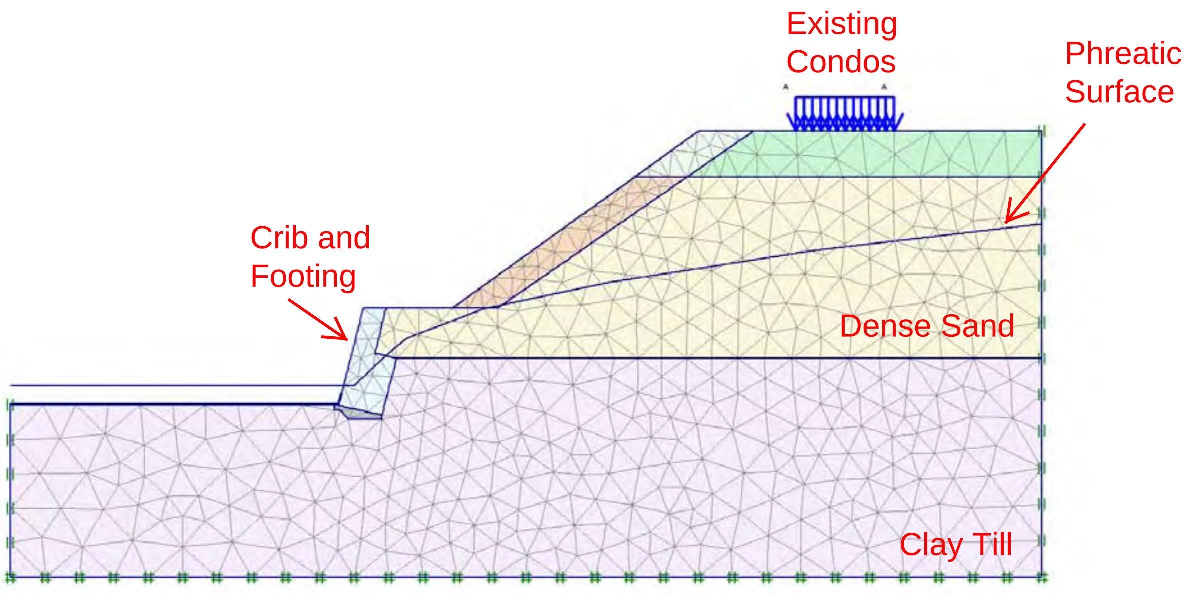


Figure 7: Geometry of the Plaxis 2D finite element model. Approximately 5 m zone of soil was modelled as elastic material to take prevent shallow slope failures (i.e., sloughing)

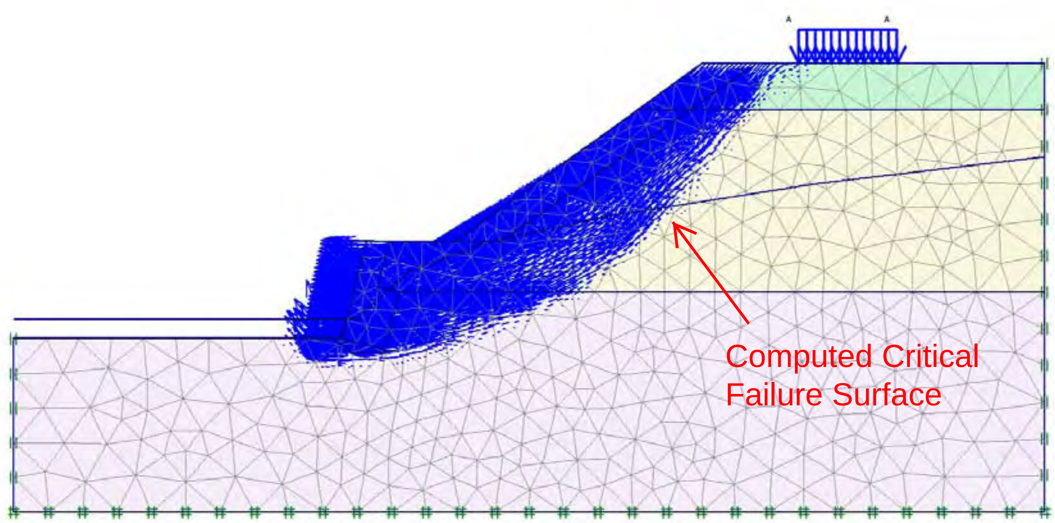


Figure 8: Non-circular failure surface extending through the base of the footing as computed by Plaxis 2D (SF = near unity)

The general pattern resulting from our analysis indicates that the computed safety factors for deep seated failure condition are approximately unity. These results agree with comments provided by Atkinson Davies Inc. In performing the sensitivity analysis, we have found that the groundwater level in the slope above the crib plays a major role in the computed safety factor. In the one scenario if the groundwater level at the top of the slope is at the same level as the river level, the safety factor against deep seated slope stability is computed as approximately being 1.2. In another scenario where the entire slope above the crib is saturated, the computed safety factor is approximately unity. Observations from January 2010 are consistent with the latter scenario.

The above results apply for portions of the crib that rest on a footing only. The 1967 rehabilitation drawings indicate that at the dam, the crib footing sits on a concrete wall that extends a distance into the clayey silt till. This concrete wall would likely prevent deep seated failure conditions typical at crib sections upstream and downstream of the dam. The case where the crib footing rests on the concrete wall was not tested in our analysis, as the wall (and the dam structure itself) would reduce the likelihood of deep seated failure condition described previously.

Deep seated failure mechanisms extending below the crib wall would likely be preceded by slope failures immediately upstream and downstream of the crib. Slopes upstream and downstream of the crib are similar in geometry to the valley slope above the crib. Failure conditions postulated could be encountered if the groundwater levels rise significantly (after periods of prolonged heavy rains) in the sand deposit. Seasonal variation of the groundwater level in sandy material are not expected to be significant due to sand's high levels of permeability.

The consequence of slope failure for the sections where the crib rests only on the footing are not expected to compromise the overall safety of Springbank dam. Likely consequence of failure would be slope (and crib) material ending up in the river, which would likely impair dam operations.

The important fact to note is that the slope has not failed yet, despite the computed low safety factors. Considering that the slope had not yet failed, it is likely that banks of the Thames in the vicinity of Springbank Dam have, over time, come into equilibrium and stabilized with a safety factor near unity. These comments were provided by our geotechnical consultant, Atkinson Davies Inc.

We should note that the above safety factors are less than the design minimum factors of safety set out in Terraprobe's manual (MNR, 1998). The land use categorized by Active – “habitable or occupied structures near slope; residential, commercial, and industrial buildings, retaining walls, storage warehousing of non-hazardous substances” have a design minimum safety factor between 1.3 to 1.5 (MNR, 1998, p.165).

Obtaining additional borehole data and/or laboratory strength and permeability test results would provide more confidence on the parameters used in our calculations. However, extra investigations are unlikely to provide drastically different global stability results (the valley slope above the crib is simply too steep). Monitoring the ground water level at the top of the valley slope would be beneficial, as it would able to provide a refinement to the computed safety factor and thus give a direct indication of the slope stability.

5. Crib drainage maintenance

Maintenance work is required to restore eroded and washed out fill material at the top of the crib. Currently, the surface runoff and groundwater seepage from the slope ends up on top of the crib. The water that ends up on top of the crib eventually runs of from the top of the crib and ends up in the Thames River. Over time, the process of surface runoff and seepage from the slope have eroded portion of the fill material on top of the crib (see Photo 10). Addressing the surface runoff and seepage at the top of the crib is the focus of this section. A number of alternative maintenance options are presented next.

5.1 Option 1 – Restore original grade

The as-built drawings of the crib from 1967 show that the flat bench at the top of the crib is to be graded with a 2% slope draining towards the river. One alternative to the remedial measure is to restore the eroded fill on top of the crib to specifications found on the 1967 drawings (2% slope on top of the crib).

The downstream return portions of the crib (i.e., downstream crib wing) currently has large amount of material missing (see Photos 3 and 4) and should be lined with filter cloth and filled with crushed gabion rock fill (100-200 mm in diameter) prior to grading and placement of the fill material. The crushed rock and filter cloth system provide a drainage path for water to exit the crib. The return portions of the downstream most crib wing is currently exposed, and approximately 1.0 cubic meter gabion basket can be placed to firmly secure the exposed crib into the slope.

In order to restore the fill material at the top of the crib, the eroded section of material in a zone 3.0 m from the crib face, and 1.5 m deep is to be excavated everywhere along the top of the crib. This excavation is expected to be cumbersome as crib's cross ties are spaced every 2.4 m along the top. Prior to backfilling the excavated trench, a 100 mm diameter drainage pipe with sock (i.e., Big 'O' drainage tile) is to be installed. The new drain pipe should be equipped with a rodent guard, to prevent small burrowing animals from nesting inside the pipe. After placement of the drainage pipe, the trench is to be filled with existing material, and supplemented with a known quantity of imported gradual B material to restore the original 2% grade towards the river. Table 3 outlines our estimates of the cost associated with Option 1, while Figures 9 and 10 show drawings of the proposed maintenance repairs.

Periodic inspections should be performed of the drainage system (probably at the same interval as the dam safety assessments for the Springbank Dam). The inspection should take place during (or shortly after the freshet), when the seepage of the slope is likely to be greatest.

Table 3: Option 1 - Restore original grade cost estimate

Item No.	Title	Unit	Quantity	Unit Price	Total
1	Mobilization	lump sum	1	20,000	20,000
2	Gabion basket	m3	1	2,000	2,000
3	Crushed rock fill	tons	20	50	1,000
4	Geotextile	lump sum	1	500	500
5	Excavation	m	75	150	11,250
6	Granular B fill	m3	150	30	4,500
7	Drainage tubing	m	75	50	3,750
8	Grading	lump sum	1	2,000	2,000
9	Plan, spec, tender	lump sum	1	8,000	8,000
10	Contract admin	lump sum	1	5,000	5,000
11	Contingency	lump sum	1	10,000	10,000
				TOTAL	68,000

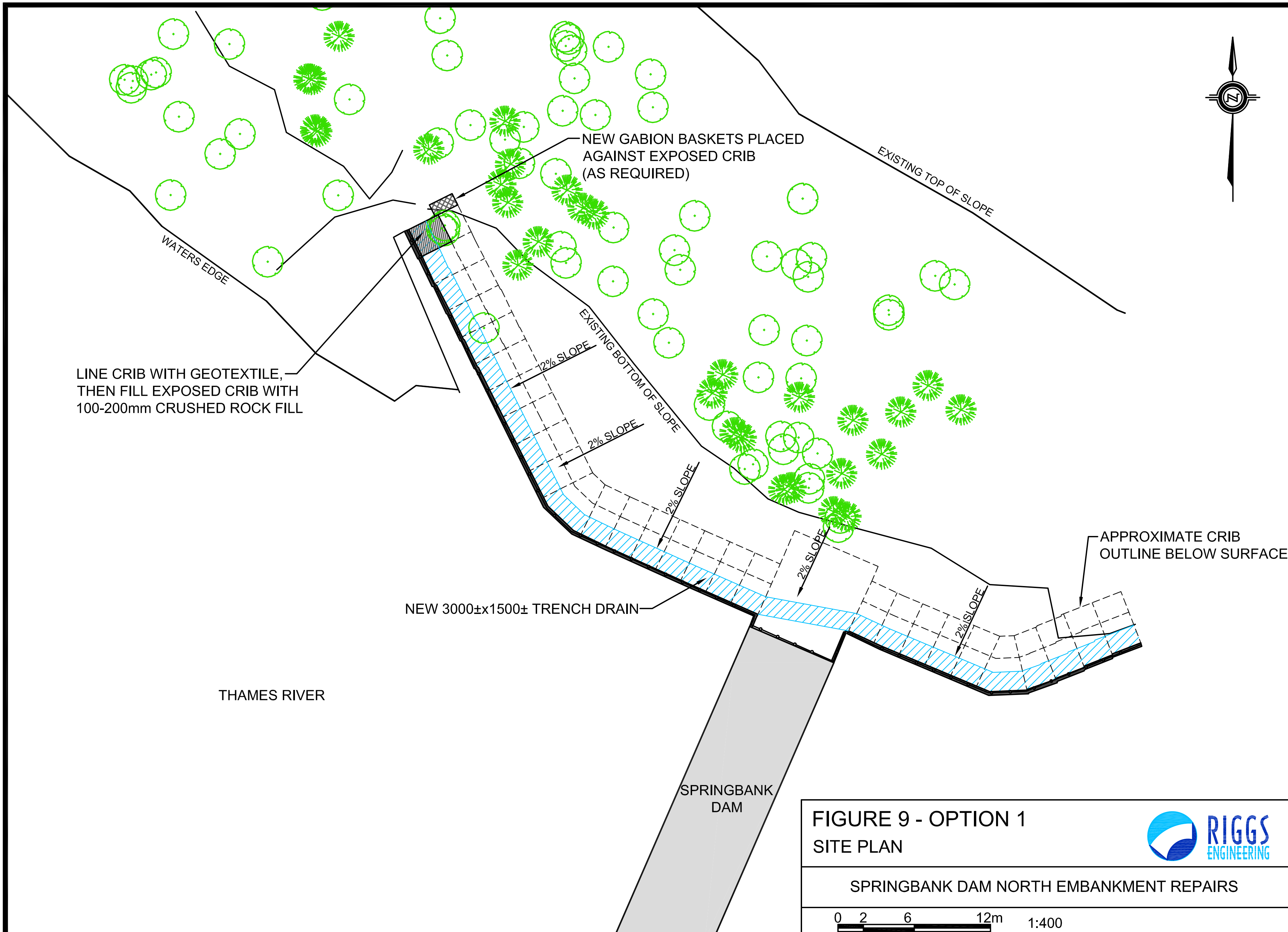


FIGURE 9 - OPTION 1

SITE PLAN

RIGGS ENGINEERING

SPRINGBANK DAM NORTH EMBANKMENT REPAIRS

0 2 6 12m 1:400

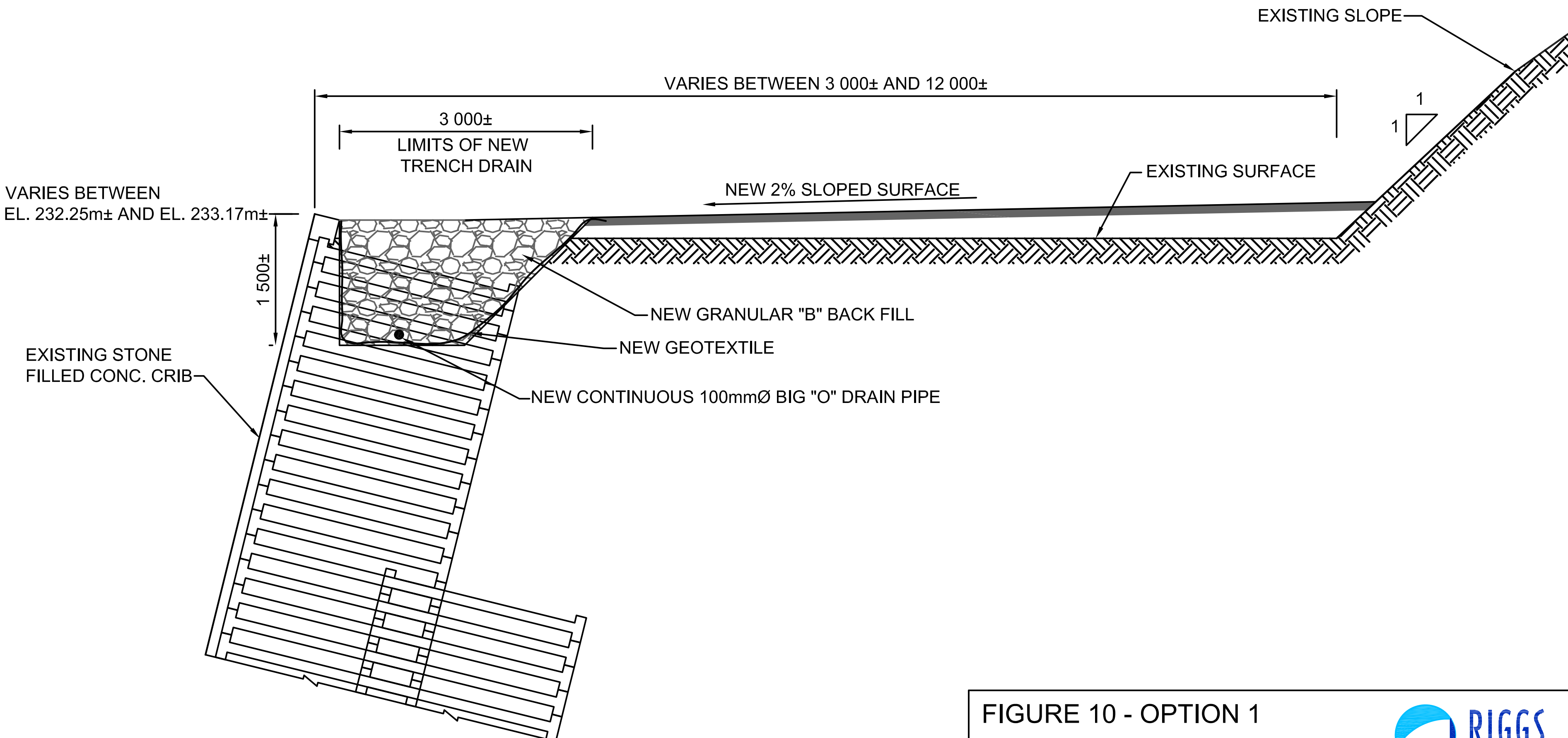


FIGURE 10 - OPTION 1
TYPICAL DRAINAGE DETAIL

RIGGS ENGINEERING

SPRINGBANK DAM NORTH EMBANKMENT REPAIRS

0 1 2 3m 1:75

5.2 Option 2 – French drain

This option includes placement of filter cloth and backfilling with crushed stone (same as in Option 1) on the downstream return portion of the crib. The exposed downstream return portion of the crib is likewise to be secured with a 1 cubic meter gabion basket (as in Option 1). To restore the eroded fill at the top of the crib, existing material is to be brought in (if needed) and top of the crib should be re-graded to the original 2% slope which slopes towards the new drain. In order to mimic the drainage patterns observed during the January 2010 inspection, a French drain is recommended to be installed 2 m from the toe of valley slope in the trench along the entire plan extent of the crib. Table 4 shows our estimate of the cost, while Figures 11 and 12 outline the general arrangements and details of Option 2. Note that our geotechnical consultant Atkinson Davies Inc. agrees with the above configuration of the above maintenance option.

A French drain is a ditch covered with gravel or rock material that collects surface and groundwater away from an area. Typical drains of this type also have perforated pipes along the bottom of the ditch that would move the draining water rather quickly. Both the swale, and the ditch of the French drain should be lined with filter cloth to reduce movement of fine particles and limit the potential clogging of the system.

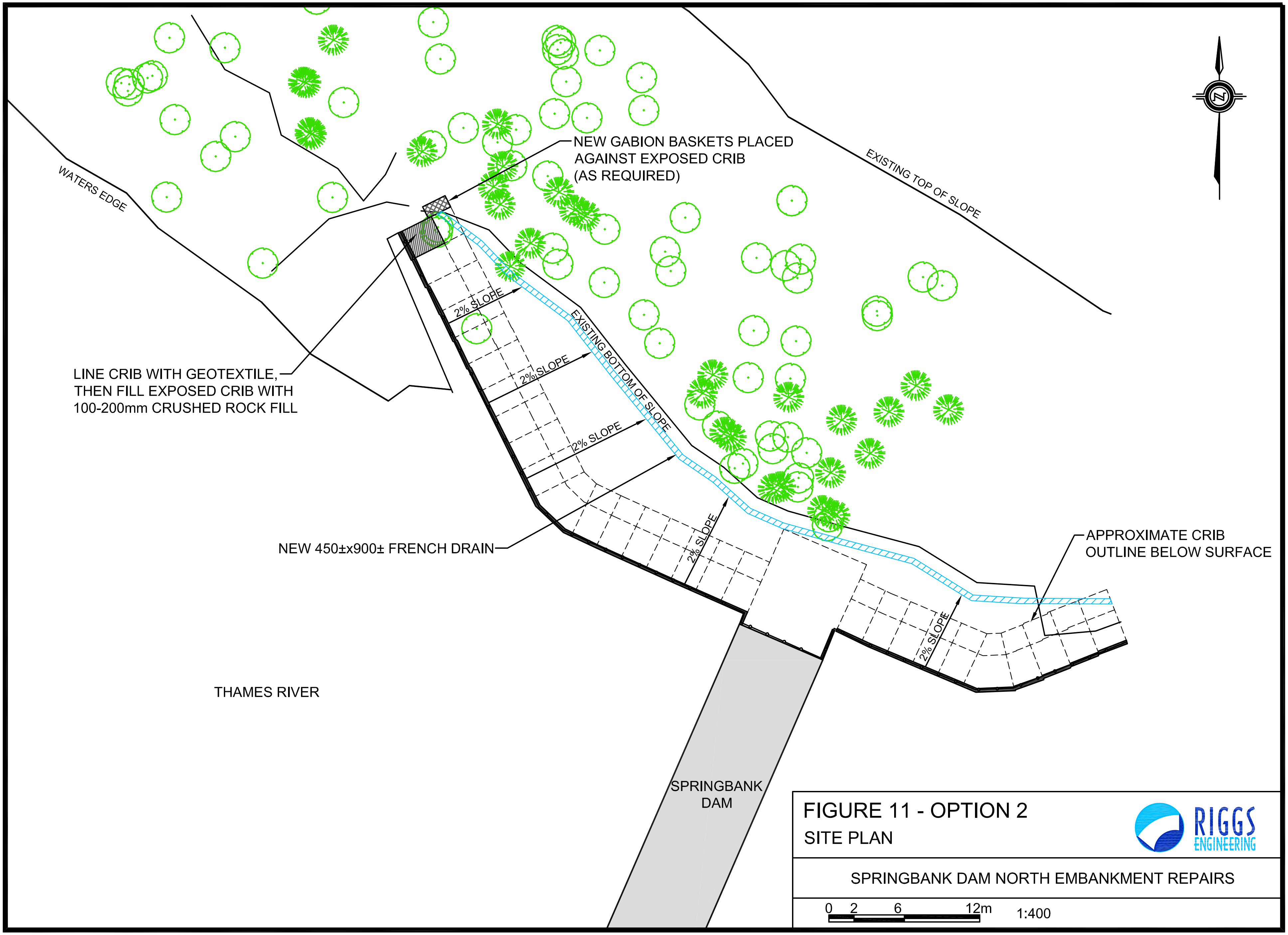
A number of alternatives to filling the downstream return portion of the crib with crushed rock were considered. These alternatives would include filling and grading the crib with granular material, and connecting the French drain to the downstream end slope that would move the water away from the crib. The possible options would be to line the downstream slope past the crib with:

i) corrugated steel pipe,
ii) rock chute (slope lined with rock),
iii) corrugated steel half-pipe,

These alternatives remain viable if the cost of crushed rock fill is deemed excessive.

Table 4: Option 2 - French drain cost estimate

Item No.	Title	Unit	Quantity	Unit Price	Total
1	Mobilization	lump sum	1	20,000	20,000
2	Gabion basket	m3	1	2,000	2,000
3	Crushed rock fill	tons	20	50	1,000
4	Geotextile	lump sum	1	500	500
5	Excavation	m	75	50	3,750
6	Granular B fill	m3	50	30	1,500
7	French drain	m	75	100	7,500
8	Grading/swale	lump sum	1	2000	2,000
9	Plan, spec, tender	lump sum	1	8,000	8,000
10	Contract admin	lump sum	1	5,000	5,000
11	Contingency	lump sum	1	10,000	10,000
				TOTAL	61,250


NEW GABION BASKETS PLACED AGAINST EXPOSED CRIB (AS REQUIRED)
EXISTING TOP OF SLOPE
WATERS EDGE
2% SLOPE
2% SLOPE
2% SLOPE
2% SLOPE
2% SLOPE
EXISTING BOTTOM OF SLOPE
LINE CRIB WITH GEOTEXTILE, THEN FILL EXPOSED CRIB WITH 100-200mm CRUSHED ROCK FILL
NEW 450±x900± FRENCH DRAIN
APPROXIMATE CRIB OUTLINE BELOW SURFACE
THAMES RIVER
SPRINGBANK DAM
FIGURE 11 - OPTION 2
SITE PLAN
RIGGS ENGINEERING
SPRINGBANK DAM NORTH EMBANKMENT REPAIRS
0 2 6 12m
1:400

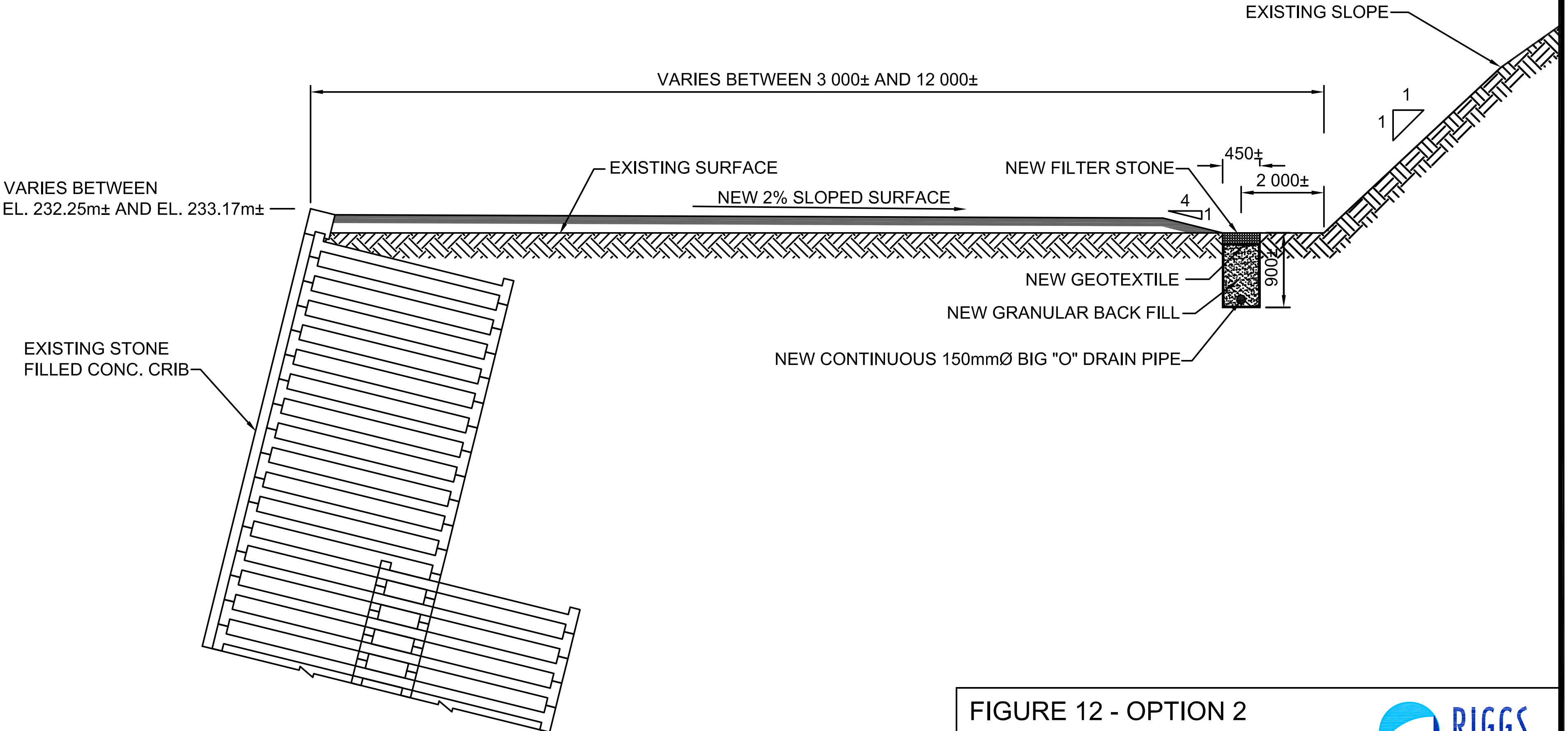


FIGURE 12 - OPTION 2

TYPICAL DRAINAGE DETAIL

RIGGS ENGINEERING

SPRINGBANK DAM NORTH EMBANKMENT REPAIRS

0 1 2 3m 1:75

5.3 Option 3 – Hardened surface

An option where the fill material is restored at top of the crib, and the bench at top of the slope is covered with a hardened surface (hand placed concrete, grout type mattress, etc.). Such a solution would certainly keep the fill material in place, even during severe storm events when surface runoff and erosion are expected to be highest. However, a solution with a hardened surface will not be needed if an alternative solution (such as Options 1 and 2) are deemed adequate.

5.4 Option 4 – Do nothing

The do nothing option is always an alternative. Erosion of the material from the slope and top of the crib would continue (at greater rates during years with heavy storms). The fill material at top of the crib would also continue to trap fine particles that erode from the slope, which would make the soil at top of the crib saturated nearly at all times. Our slope stability analysis shows that saturated soil behind the crib reduce the factors of safety in global stability analysis of the slope and crib.

5.5 Discussion of options

Comparison of the above alternatives leads to the following conclusions: Option 1 is more expensive, and also more limited in providing drainage at the top of the crib. Option 2 is less costly, and has higher capacity to drain surface and groundwater (i.e., a French drain has more capacity than a Option 1). Option 2 also has the advantage of mimicking the drainage characteristics observed during our inspection, while at the same time removing most of the water that seeps from the above slope away from the soil at the top of the crib.

The difference in cost between option 1 and 2 is not, in our opinion, significant. Benefit obtained from lower cost and draining the water more quickly recommend Option 2 could as a viable option for future consideration.

It is important to note that neither of the options presented above are expected to improve the safety factor again deep seated slope stability analysis. On the contrary, placement of additional fill material within the active wedge can theoretically lower the safety factor by a small amount, but not enough to be of concern. Our analysis has shown that the largest influence in the safety factor against global slope stability appears to be the groundwater level in the slope above the crib. The recommendation to install the French drain at the toe of the slope would act to control the groundwater level at the top of the crib, but would not measurably lower the phreatic surface in the above slope and in areas below the crib. Thus, the proposed drainage options at the top of the crib would not improve the safety factors for the overall global slope stability.

6. Conclusions

Preliminary engineering design has been completed for the restoration of the Springbank Dam North Embankment Crib. Over time, surface runoff and groundwater seepage from the slope above the crib have eroded the top 1 m of soil material on top of the crib. As a result of the erosion, the fill material on the top of the crib is no longer free draining. Puddles, pool and saturated soil are constantly present at the top of the crib, even during periods free from rain. During the inspection of January 2010, the groundwater level was observed to be approximately 0.3 m below the surface at the top of the crib, near the toe of the slope. Artificial channels were seen at the toe of the slope (both upstream and downstream of the dam) that carried water from the top of the crib down to the river below.

As the access to the top of the crib is limited to going over the Springbank Dam, calculations were performed to check if the deck of the dam would be able to support a typical one tonne truck carrying equipment and materials. Our calculations show that the existing structure would support a one tonne truck. Stockpiling of the materials on top of the deck of the dam is not recommended in order to limit the material escaping into the river below.

On order to improve the drainage characteristics of the fill material at the top of the crib, a number of options were considered. There is a concern that placing additional material on top of the crib could aggravate the stability of the above slope. Slope stability analysis was performed using soil parameters provided by our geotechnical consultant, and it was determined that the slope above the crib has a global stability factor of safety near unity. Our analysis also demonstrates that location of the groundwater level in the slope above the crib heavily influences the computed safety factor. For the water levels observed in the field during our January 2010 inspection, the global stability safety factor was determined as near unity. Placement of extra material on top can theoretically reduce the global safety factor by a small amount, but not by an amount that will affect the above conclusion (i.e., safety factor before, during and after the maintenance repairs is expected to remain near unity).

7. Recommendations

A number of options were considered for improving the drainage characteristics of the soil behind the crib. The Option 2 (French drain at the toe of slope) is recommended as the preferred alternative. The recommended French drain option would collect the surface runoff and seepage water from the above slope, and carry it down to the river. The French drain option would mimic what currently exists in the soil retained by the crib during the high slope runoff and seepage events. Based on our assessment, it is our recommendation that the Option 2 alternative be implemented within 1-5 years. It is important to note that implementation of the recommended alternative would not increase the global stability of the valley slope. Slope stability is influenced most by the geometry of the slope (the slope is approximately 1.2H:1V in some areas), the location of the phreatic surface, and the soil strengths. The French drain option would however improve the drainage characteristics of the soil behind the crib.

In light of the results presented in this report, a monitoring program should be devised that periodically assesses changes to the valley slope from a base point. The inspections should include monitoring of slope movement, surface erosion, seepage and drainage characteristics.

Since the global stability safety factors have been shown to be sensitive to the location of the phreatic surface, and since the safety factors (before and after the drainage improvement works) are estimated as being marginally above unity, it is recommended that a ground water monitoring program be considered for the near future. A minimum of two piezometers (one at the top of the valley slope, and one through the soil behind the crib) are recommended.

Photographic Log

Photo 1: North embankment crib, downstream portion (standing on bottom and looking downstream). Most of the surface runoff flows bypasses the crib.

Photo 2: Downstream end of the crib, looking towards the south. Water on the bottom shows the surface runoff bypassing the downstream portions of the crib.


Photo 3: Downstream end of the crib. Note the material lost.


Photo 4: Downstream end, material lost
due to surface runoff and erosion.


Photo 5: Upstream return portion of the crib, showing the upstream extent of riprap.


Photo 6: Slope at the upstream end of the crib, showing runoff from the top of the crib

Photo 7: Slope above the crib. Portion of the slope are steep and are eroding.

Photo 8: Slope above the crib. Slope is held together by roots, sometimes by trees that are dead.

Photo 9: Puddles and pools on the fill material on top of the crib

Photo 10: Top of crib on the upstream side of the crib, looking upstream. Shown in the figure is material eroded from top of the crib.

Photo 11: Toe of slope downstream of the dam, standing on top of crib. Seepage from the slope is seen to drain in a small channel that eventually empties at the downstream side of the crib

Photo 12: Toe of slope upstream of the dam, standing on top of crib. Again seepage from the slope drains in a less defined channel (due to trees) and eventually empties at the upstream side of the crib


Photo 13: Riprap on the upstream side of the crib, standing on dam and looking upstream. Seeping water is evident on the outer side adjacent to the crib. Evidence of seepage has also been observed at the treeline (top of riprap).


Photo 14: Riprap protection at the upstream side of the crib (stone diameter varies from 0.1 to 1.0 m). Some cracking in the stones is evident, but the riprap is in generally good condition

Photo 15: Riverbed stone upstream on the upstream side of the dam adjacent to the crib shows no localized undermining

Photo 16: Springbank Dam bridge deck, looking north. Note the holes in the concrete for the old gate guides (welded shut by thin plates).

Photo 17: Typical view of the deck slab at the old gate guides (looking downstream). Note the crack running along the length of the slab from the pipes to the old gate guides (see pen for scale).

Photo 18: North end of the deck, vertical cracks are observed adjacent to the stairs and adjacent to the fence post.

Photo 19: Vertical crack adjacent to the fence post on the north end of the deck.

RIGGS ENGINEERING LTD.
1240 Commissioners Road West
Suite 205
London, Ontario
N6K 1C7

Report On

GEOTECHNICAL INVESTIGATION

for

SPRINGBANK DAM STUDY
LONDON, ONTARIO

Ref.: 1-4478

February 17, 2010

ATKINSON, DAVIES INC. Since 1980
Consulting Geotechnical, Environmental and Construction Materials Engineers
60 Meg Drive, Unit 12, London, Ontario, N6E 3T6 **Telephone:** (519) 685-6400 **Fax:** (519) 685-0943
Website: www.atkinsondavies.com **Email:** atkinsondavies@atkinsondavies.com

February 17, 2010

Ref.: 1-4478

Riggs Engineering Ltd.
1240 Commissioners Road West
Suite 205
London, Ontario
N6K 1C7

Attention: Mr. Peter Crook, P. Eng.

Dear Mr. Crook:

Re: Geotechnical Investigation for Spingbank Dam Study, London, Ontario

We have completed this project in accordance with your instructions and authorization, and this report contains a record of our findings.

FIELD WORK

The field work was carried out on January 22, 2010, and consisted of two boreholes located as shown on Enclosure 1. Due to inaccessibility with the drill rig, the boreholes were advanced to depths of 3.1 to 3.7 metres with Pionjar hand equipment.

The elevation of the ground surface at each borehole location was related to a local benchmark, which was taken as the top of the concrete deck of the dam at the location shown on Enclosure 1.

SUBSURFACE CONDITIONS

Descriptions of the strata encountered in each borehole are given on the borehole logs comprising Enclosures 2 and 3. The following notes are intended only to amplify this data.

Beneath the surface layer of topsoil, measuring 100mm thick, layers of sand and gravel fill materials were encountered, which were underlain by grey clayey silt and sand to silty clay fill materials at depths of 1.2 to 2.5 metres. The fill samples yielded moisture contents ranging from 11% to 48%.

At the completion of drilling, water levels were measured in the boreholes at depths of 0.2 to 0.4 metres (El. 99.0±).

We trust this report is sufficient for your present requirements, however if further discussion is required, please contact our office. The Statement of Limitation, Appendix ‘A’, should be read in connection with the report.

LICENSED PROFESSIONAL ENGINEER
C.J.W. ATKINSON
1588011
PROVINCE OF ONTARIO

Yours very truly,

ATKINSON, DAVIES INC.

C.J.W. Atkinson, M.Sc., P.Eng.

CJWA/wrs
Enclosures

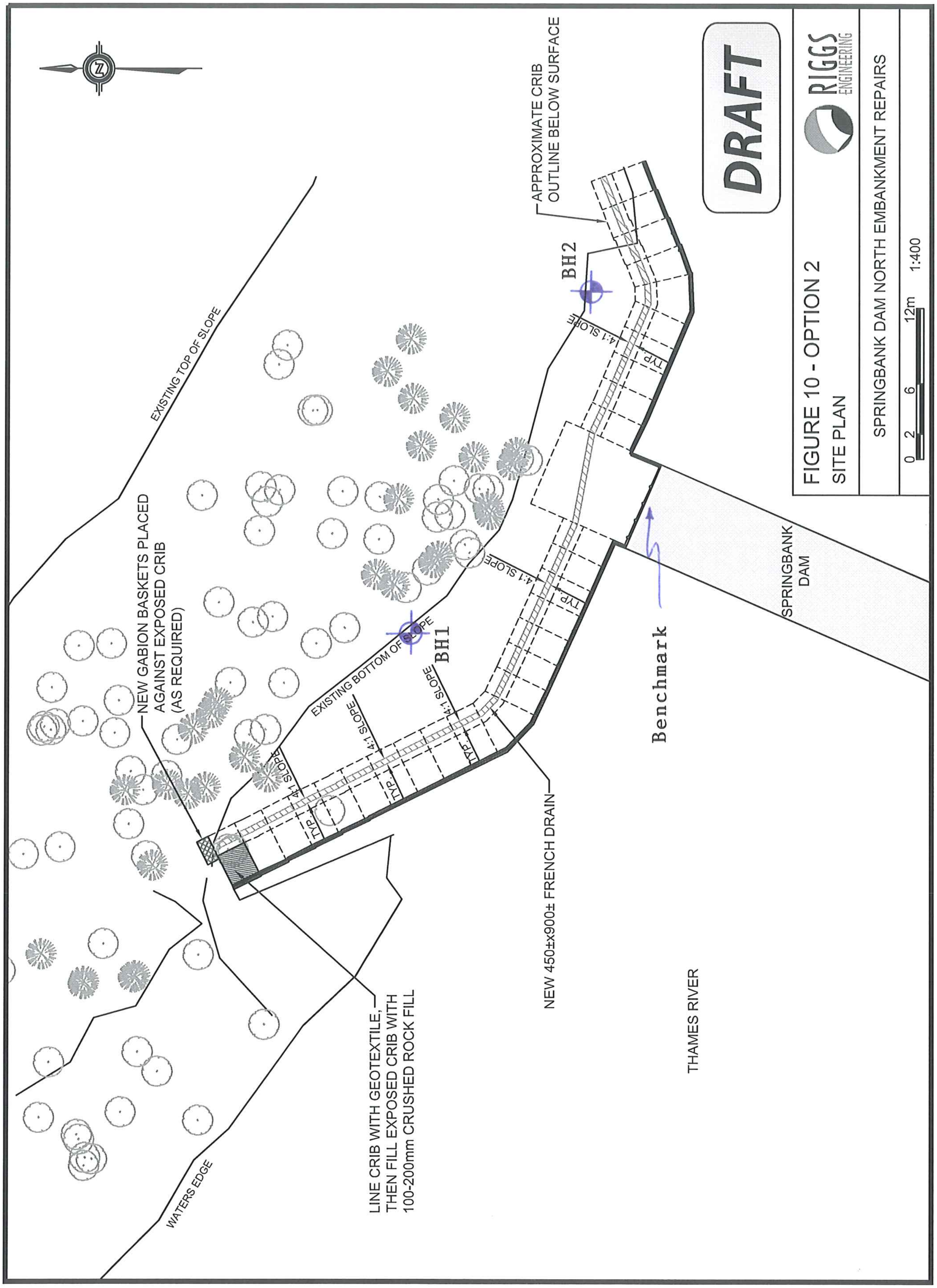
STATEMENT OF LIMITATION

The conclusions and recommendations in this report are based on information determined at the borehole and test pit locations and on geological data of a general nature which may be available for the area investigated. Soil and groundwater conditions between and beyond the boreholes and test pits may differ from those encountered at the borehole and test pit locations and conditions may become apparent during construction which could not be detected or anticipated at the time of the soil investigation. The passage of time also must be considered, and it must be recognized that, due to natural occurrences or direct or indirect human intervention at the site or distant from it, actual conditions discovered may quickly change. The information contained within this report in no way reflects the environmental aspect of the site or soil, unless specifically reported upon.

The comments given in this report on potential construction problems and possible methods of construction are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all of the factors that may affect construction methods and costs (e.g. the thickness of surficial topsoil and fill layers can vary markedly and unpredictably). The contractors bidding on this project or undertaking the construction should therefore make their own interpretations of the presented factual information and draw their own conclusions as to how the subsurface conditions may affect their work.

We recommend that we be retained to ensure that all necessary stripping, subgrade preparation and compaction requirements are met, and to confirm that the soil conditions do not deviate materially from those encountered in the boreholes and test pits. **In cases where this recommendation is not followed, the company's responsibility is limited to interpreting accurately the information encountered in the boreholes and test pits.**

This report is applicable only to the project described in the introduction, constructed substantially in accordance with details of alignment and elevation quoted in the text.


DRAFT
RIGGS ENGINEERING
FIGURE 10 - OPTION 2
SITE PLAN
SPRINGBANK DAM NORTH EMBANKMENT REPAIRS
0 2 6 12m
1:400
APPROXIMATE CRIB OUTLINE BELOW SURFACE
BH2
BH1
EXISTING TOP OF SLOPE
EXISTING BOTTOM OF SLOPE
4:1 SLOPE
TYP.
NEW GABION BASKETS PLACED AGAINST EXPOSED CRIB (AS REQUIRED)
SPRINGBANK DAM
Benchmark
NEW 450±x900± FRENCH DRAIN
LINE CRIB WITH GEOTEXTILE, THEN FILL EXPOSED CRIB WITH 100-200mm CRUSHED ROCK FILL
THAMES RIVER
WATERS EDGE

Atkinson Davies Inc.

CONSULTING SOILS AND MATERIALS ENGINEERS

12 - 60 Meg Drive, London, ON, N6E 3T6

Phone: 519-685-6400 Fax: 519-685-0943

REF. NO.: 1-4478
CLIENT: Riggs Engineering Ltd
PROJECT: Springbank Dam
LOCATION: London, Ontario
DATUM ELEVATION: Top of north end of dam, 100.0m

LOG OF BOREHOLE NO. 1

Encl. No. 2 (Sheet 1 of 1)
DRILLING DATA: Hand Equipment
METHOD: Pionjar
DIAMETER: 50mm
DATE: Jan 22, 2010

SUBSURFACE PROFILE

● Penetration Resistance Blows/ft: 20, 40, 60, 80

Undrained Shear Strength kPa — ▲ Field Vane Test ★ Compression Test: 20, 40, 60, 80

Elev. metres	Depth metres	DESCRIPTION	NUMBER	TYPE	"N" Blows/ft	PLASTIC LIMIT %	NATURAL WATER %	LIQUID LIMIT %
99.22	0	100mm TOPSOIL.						
		Sand & gravel FILL with clayey seams.	1	ss			48	
			2	ss			15	
98		Grey, clayey silt & sand FILL, some gravel.	3	ss			16	
			4	ss			16	
			5	ss			19	
	3	End of Borehole. Water level at 0.2m depth at completion.						

Atkinson Davies Inc. CONSULTING SOILS AND MATERIALS ENGINEERS
12 - 60 Meg Drive, London, ON, N6E 3T6 Phone: 519-685-6400 Fax: 519-685-0943

REF. NO.: 1-4478
CLIENT: Riggs Engineering Ltd
PROJECT: Springbank Dam
LOCATION: London, Ontario
DATUM ELEVATION: Top of north end of dam, 100.0m

LOG OF BOREHOLE NO. 2

Encl. No. 3 (Sheet 1 of 1)
DRILLING DATA: Hand Equipment
METHOD: Pionjar
DIAMETER: 50mm
DATE: Jan 22, 2010

SUBSURFACE PROFILE

Elev. metres	Depth metres	DESCRIPTION	GROUND WATER	NUMBER	TYPE	"N" Blows/ft	PLASTIC LIMIT %	NATURAL WATER %	LIQUID LIMIT %
99.40	0	100mm TOPSOIL.							
		Sand & Gravel FILL, some silt & clay.	▼	1	ss				
				2	ss				
				3	ss			11	
				4	ss			13	
		Grey, silty clay FILL, sand & gravel seams.		5	ss			15	
				6	ss			14	
		End of Borehole. Water level at 0.4m depth at completion.							

Penetration Resistance Blows/ft: 20 40 60 80
Undrained Shear Strength kPa: ▲ Field Vane Test ★ Compression Test: 20 40 60 80

ATKINSON, DAVIES INC. Since 1980
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Website: www.atkinsondavies.com
Email: atkinsondavies@atkinsondavies.com

17 March 2010

Ref.: 1-4478

Riggs Engineering Ltd.
1240 Commissioners Road West
Suite 205
London, Ontario
N6K 1C7

Attention: Mr. Peter Crook, P.Eng.

Dear Mr. Crook:

Re: Review of Draft Report for Springbank Dam North Abutment Repairs

We confirm the following observations and discussion pertaining to our site inspection on September 10, 2009.

1. The relatively flat area behind the north abutment was observed to be wet, indicating poor drainage of the fill material placed behind the concrete crib.

2. Measurements of the slope along the north side of the Thames River, adjacent to the dam, revealed slope angles ranging from 38 to 40 degrees, which is typical for dense granular material. The soil profiles from the Golder borehole (BH4) confirmed that the very dense granular strata extend to a depth of 23.3 metres. It may therefore be assumed that the angle of internal friction for the granular subsoil is close to the slope angle of 40 degrees. The soil profile below the very dense granular subsoil consists of hard clayey silt till and very dense silt, and typical parameters for this type of material are given in a published paper (Soderman, Kenney and Loh, "Geotechnical Properties of Glacial Clays in Lake St. Clair Region of Ontario). Typical values from a similar clay in Rodney revealed c^1 = 260 p.s.f. (12.5 kPa) and $\phi^1 = 28°$. The

inspection confirmed that the existing slope is stable with respect to deep circular failure, however the existing gradient is compatible with a factor of safety slightly above unity. Bare areas on the existing slope are the result of surface sliding due to oversteepening and surface erosion.

3. Groundwater was encountered in the borehole at EL. 235, and because of the permeable nature of the upper granular deposits, seasonal fluctuation of the groundwater table is anticipated to be 0.3m.

4. We have reviewed the recommendations for remedial construction and it is our opinion that Option 2 (The French drain) would be the most suitable for the site condition. We would also suggest that the line of the drain be moved close to the bottom of the slope to intersect surface run-off from the slope, which is probably the main cause of the wet surficial condition.

We trust that these comments contain sufficient information for design purposes, however if we can be of further assistance, please do not hesitate to contact us.

Yours very truly,

ATKINSON, DAVIES INC.

C.J.W. Atkinson, P.Eng.

CJWA/wrs