Rapidan Dam Investigations
Blue Earth River Feasibility Study
Ecosystem Restoration

US Army Corps
of Engineers ®

Prepared by:
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September 2009

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DISCLAIMER
Preliminary Documents
Blue Earth River Ecosystem Restoration Feasibility Study
September 2009

This document is a preliminary engineering product. St. Paul District, US Army Corps of Engineers prepared this document as part of the Blue Earth River Ecosystem Restoration Feasibility study. The information has undergone a District Quality Control review, but no independent technical review has been conducted. Users of this information must be aware that the information is preliminary in nature and not intended as a final product of the feasibility study.

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Executive Summary

Study Background

The Blue Earth River Basin feasibility study was undertaken to assess the potential for federal ecosystem restoration activities in the Blue Earth River basin. The study was initiated on October 5, 2007 with the execution of a cost-share agreement between the Department of the Army and Blue Earth County. The study is authorized by a May 10, 1962 resolution of the Committee on Public Works, U.S. House of Representatives.

Initial study efforts focused on the Rapidan Dam, because the dam blocks fish passage from the Minnesota River to the Blue Earth River and Watonwan River watersheds upstream and affects the fishery in the study area. In order to assess the expected "future without project" conditions, it was necessary to assess the current and likely future condition of the dam. To that end, the Corps conducted geotechnical investigations and preliminary hydraulic and hydrologic analyses. Those efforts identified a lack of energy dissipation and the need for a stilling basin below the dam to reduce the risk of failure during large flood events. A conceptual stilling basin design was developed for cost estimating purposes. A construction estimate and operation and maintenance estimate were prepared, and finally a life-cycle cost analysis was conducted. This report presents the preliminary information generated during the study from its inception through September 2009.

It should be noted that the analyses presented in this report have undergone a quality control review within the St. Paul District office, but no independent technical review has been conducted. This report is being prepared only to document the preliminary work of the study team. The results are **preliminary** in nature, and not intended to serve as final recommendations of the U.S. Army Corps of Engineers. The Rapidan Dam is not required to meet and does not currently meet Corps of Engineers dam safety criteria. The improvements described in this report were not intended to bring the dam into compliance with any accepted dam safety standards. However, the conceptual features described herein would significantly reduce the risk of scour below the dam that could impact its foundation.

Summary of Major Findings

- Geotechnical analyses found little potential for high gradients that could cause piping in the foundation, but the foundation material is highly vulnerable to erosion. Measures to protect the abutments and sandstone downstream are essential for continued safe operation of the dam.
- The available hydrologic information for Rapidan dam is out of date. The probable maximum flood, base safety standard and threshold flood should be determined based on current methodology.
- Rapidan Dam does not meet current Corps of Engineers dam safety criteria for either spillway capacity or energy dissipation downstream.
- It does not appear that constructing a spillway to support continued hydroelectric generation of the dam would be cost-effective from the County's perspective.

- Continued operation of the dam without additional energy dissipation measures was not analyzed. The current situation poses significant risk of severe erosion that could lead to environmental damage from an uncontrolled release of sediment. The uncertainty regarding future maintenance needs and financial impacts from a catastrophic event makes it difficult to reliably estimate future costs.

History and Description of Rapidan Dam

The Rapidan Dam is located on the Blue Earth River approximately 12 miles upstream of Mankato in Blue Earth County, Minnesota. The dam, built in 1910, supports hydroelectric power generation, but it also blocks fish passage between the Minnesota River and the 1,200 miles of perennial tributary streams above the dam. The Federal Energy Regulatory Commission (FERC) has classified the dam as having a significant downstream hazard potential based on the environmental damage that would be caused by an uncontrolled release of the agriculturally impacted sediments behind the dam.

The dam is an Ambursen Dam consisting of concrete structures founded on friable sandstone bedrock in a steep U-shaped valley. The overall length of the dam is approximately 475 feet and the maximum height is approximately 90 feet. The reservoir upstream of the dam provides storage for power generation, recreation and conservation value. The reservoir also serves as a sedimentation basin and is essentially full of sediment. Therefore, sediment in current runoff passes downstream.

The dam served as an electric power generating facility for Northern States Power Company until it was damaged by flooding in 1965. Blue Earth County obtained ownership of the structure in 1970. Under an agreement with the county, Rapidan Redevelopment, Ltd. redeveloped the dam for producing hydroelectric power in 1984. As part of the redevelopment, the powerhouse, draft tubes, and penstocks were modified; new turbines, new tainter gates, and a low-flow valve were installed; the upstream tainter gate piers and upstream forebay wall were repaired, and the corbels were post-tensioned. In 2002, extensive undermining of the dam's foundation was discovered, and emergency repairs were required to prevent a dam failure. Additional apron, foundation and abutment repairs have been conducted since 2002. North American Hydro currently operates the hydroelectric generation equipment at the dam under a lease agreement with Blue Earth County.

Geotechnical Investigations

The Corps conducted geotechnical investigations at the Rapidan Dam in February 2008. Particular attention was focused on the potential for piping. The dam foundation, made up of the Jordan Sandstone formation, was characterized with a drilling program in the abutments and laboratory testing. The sandstone was generally composed of poorly or uncemented zones interbedded with some hard, well-cemented zones. Significant lengths of core were not recovered at elevations corresponding to the foundation, probably indicating that the material at those elevations is poorly cemented or uncemented. Piezometer instrumentation at the site indicates that the regional water table roughly coincides with the tailwater elevation, while a small perched water table exists around pool elevation. The presence of two separate water tables suggests that pool waters are insulated from the sandstone by various fine-grained deposits, thereby minimizing seepage through the foundation and the abutments. A finite element model was constructed in order to further understand seepage conditions at the site. The results supported findings from the field. It was found that the vast majority of available head was dissipated by fine-grained sediment that has accumulated behind that dam over time, providing little potential for the development of high vertical gradients required to initiate piping. However, vulnerability of the foundation material to erosion (due to piping or scour) remains a serious concern. Measures providing adequate protection to the abutments and the sandstone downstream of the dam are essential for continued safe operation.

Hydrologic Investigations

Hydrologic investigations were conducted to assess the probable maximum flood (PMF), appropriate dam safety criteria, and criteria to be used for design of a stilling basin at the dam. Study activities included researching prior reports and updating the discharge-frequency relationship to reflect the available data through 2007.

The PMF assumed in previous studies is outdated and should be redeveloped for the specific location of the Rapidan Dam based on current National Oceanic and Atmospheric Administration guidance. The assumed PMF was based on one originally developed by the Corps of Engineers in a 1970 report for a proposed Blue Earth Dam located downstream of the Le Sueur River confluence with the Blue Earth River. The PMF at Rapidan was later estimated (by others) to be 164,000 cubic feet per second (cfs) based on drainage area transfer using the information in the 1970 report.

A Base Safety Standard (BSS) should also be determined. The BSS is the inflow design flood where there is no significant increase in adverse consequences from dam failure compared to non-failure adverse consequences.

The discharge-frequency relationship at the USGS gage near Rapidan was developed. The 500-yr discharge value is 45,000 cfs. The 100-yr discharge value is 30,000 cfs. These estimates are based on a graphical plot due to the non-stationarity and non-homogeneity of the recorded data.

The threshold flood should be determined and considered as an option for design of a stilling basin. The threshold flood is the flood that fully uses the existing dam or just exceeds the design maximum water surface elevation at the dam. This would include minimum of 3 ft of freeboard and could be higher considering other factors outlined in Engineer Regulation 1110-8-2(FR) such as wind setup and wave runup. Preliminary estimates are that at elevation 878.1 ft., which provides three feet of freeboard, the spillway discharge is approximately 48,300 cfs. This is approximately the threshold flood for the dam and has an exceedence frequency of 0.15 percent (~ 670-yr recurrence interval). Previous studies indicated that the spillway capacity is 51,700 cfs.

The two most likely candidates for design flow of a stilling basin would then be the threshold flow or ½ PMF (approximately 82,000 cfs). Corps Standard 3 criteria would require minimum ½ PMF. Federal Enegy Regulatory Commission (FERC) standards may also require ½ PMF. Significant modifications of the dam structure would be required to safely pass such a flow. This standard is based on a hazard classification of SIGNIFICANT which was in turn based on the environmental impact of sediment release downstream.

Hydraulic Investigations

Hydraulic investigations were conducted to assess the current spillway capacity and existing apron configuration at the dam. Preliminary hydraulic design was also prepared for a stilling basin adequate to pass the flood of record. Since the Rapidan Dam is not a Corps dam, it is not required to meet Corps of Engineers standards. FERC standards would apply to issues regarding licensing for power generation. According to Corps of Engineers standards, it is likely that ½ PMF or a threshold flow would be adopted as the design flow for stilling basin design. Both of those potential flows are larger than the flood of record at Rapidan Dam.

For the purposes of cost estimating, conceptual designs for stilling basins were produced based on the flood of record. This criterion was chosen only because it is the highest flow for which the tailwater conditions are known. Any further development of these conceptual designs must first determine the appropriate design flow and include a survey to estimate the tailwater conditions for all flows. The designs should then be adjusted appropriately. It should be noted that the spillway design described in this report is based on smaller flows than would likely be required to meet appropriate dam safety criteria, but it was beyond the scope of this effort to develop the information needed to design a larger stilling basin.

The spillway discharge capacity is significantly higher than previously published values if the depressed water surface elevation near the spillway crest due to high velocities is taken into account. Without collecting cross sectional surveys to establish a tailwater rating curve for flows higher than 43,100 cfs; a conceptual stilling basin design cannot be determined for higher flows.

The stilling basin, under existing conditions, has sufficient tail water depth to insure the hydraulic jump will occur at or near to the toe of the spillway. The length of the hydraulic jump, however, is expected to extend downstream of the areas that have had recent reinforcement during high flows. The jump will extend past the concrete section of the apron for flows greater than 5,500 cfs and will extend past the grouted rock section of the apron for flows greater than 28,500 cfs. The apron materials are likely not designed to be stable under the conditions of a developing hydraulic jump.

This report does not provide a recommended or approved design for improvements to Rapidan Dam. In order to continue with design, the following aspects of this design must be completed:

1. Thorough study to determine the design criteria based on appropriate standard. FERC is the regulating agency for Rapidan Dam.

2. Determination of complete tailwater rating curve at the dam site. Cross-sectional surveys should be done on the river channel and floodplain downstream of the dam such that a hydraulic model could be created to determine the tailwater rating curve at the toe of the dam for the full range of flows.

3. Down-watering to determine the velocity and depth of the water at the toe of the dam should be done in a manner that accounts for the turbulent boundary layer development losses on the spillway crest.

Stilling Basin Conceptual Structural Design

The purpose for this structural design was to create a feasibility level design that could be used for cost estimating purposes. The conceptual stilling basing consists of two T-walls on either side of the stilling basin slab. The slab was reduced in width by the toe length of the T-walls. The walls have a base width of 60’, a total height of 53’, and a length of 72’. It was calculated that the base slab would need to be 12.18’ thick. This was rounded up to a thickness of 12.5’. The width of the slab is 238’ and the length is 67’. Additional features include baffle piers, chute blocks, and an end sill.

Life-Cycle Cost/Benefit Analysis

The Corps worked with the Blue Earth County Engineer to identify necessary repairs to the dam and likely operation and maintenance activities. Cost estimates were developed for these activities. In addition, an estimate was prepared for a conceptual stilling basin design that would be adequate to prevent scour downstream of the dam for all flows up to the flood of record (the 1965 event). Both the conceptual design and the cost estimate should be considered preliminary approximations of what would be needed to address the scour issue at the dam.

Blue Earth County’s current practice is to monitor the dam and perform maintenance on an as-needed basis. For this study, the Corps of Engineers did not analyze continued operation without the addition of a stilling basin. Without additional measures to dissipate energy downstream of the dam, a large flood event could cause catastrophic erosion leading to loss of the dam. Such an event could cause an uncontrolled release of large volumes of sediment causing significant environmental damage downstream. Smaller flood events could cause significant erosion that would need to be repaired. The uncertainty regarding future maintenance needs and potential financial impacts from a catastrophic event makes it difficult to reliably estimate future costs. It was beyond the scope of this effort to quantify the risk and costs associated with continued operation of the dam without additional energy dissipation measures.

Throughout this analysis, price levels are stated as of June 2009, with a Federal discount rate of 4 5/8 percent for water resource projects being used to amortize costs and to discount benefits to a common period of time, and a 50-year period of analysis.

The repair option assumes that Rapidan Dam will continue to generate hydroelectric power. This will result in revenues based on the county's lease agreement with the operator of the hydroelectric generating facilities and Minnesota's Renewable Energy Production Incentive Program. Based on the lease agreement, the county receives approximately 5 percent of the total revenues generated at the dam. The county's annual revenues from the facility are estimated at $37,000. The county receives an additional $189,000 (approximately) in annual revenue from Minnesota's Renewable Energy Production Incentive Program. This program is currently set to expire in 2013. For this analysis, future revenue streams were calculated both with and without an extension of the incentive program.

The estimated initial cost for the proposed stilling basin and other immediate repairs/improvements to the Rapidan Dam is $10,367,000. This equates to an annual cost of $535,000. Average annual operation and maintenance costs are estimated at $85,000. A sensitivity analysis was also conducted assuming that the dam would be removed in year 50 at a cost of approximately $29 million.

This study concludes that the average annual revenue to the County is negative for all of the alternatives studied. The study found that it would cost Blue Earth County between $394,000 and $696,000 per year for 50 years to construct a stilling basin and maintain the dam to support continued hydropower generation.

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Part A
Geotechnical Investigations

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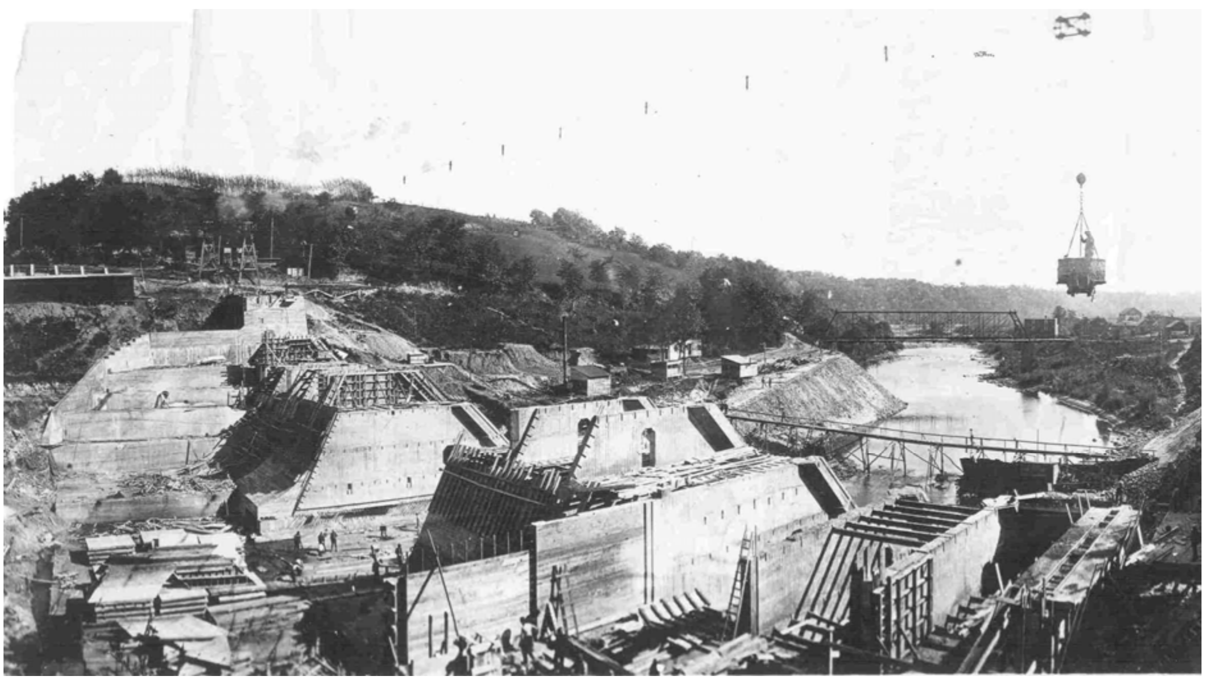
BLUE EARTH RIVER FEASIBILITY STUDY

GEOTECHNICAL INVESTIGATION OF THE RAPIDAN DAM



July 2008

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Executive Summary

The Rapidan Dam is an Ambursen Dam located on the Blue Earth River several miles southwest of Mankato, MN. A geotechnical investigation of the dam was conducted as part of a dam removal feasibility study, with particular attention focused on the potential for piping. The dam foundation, made up of the Jordan Sandstone formation, was characterized with a drilling program in the abutments and laboratory testing. The sandstone was generally composed of poorly or uncemented zones interbedded with some hard, well-cemented zones. Significant lengths of core were not recovered at elevations corresponding to the foundation, probably indicating that the material at those elevations is poorly cemented or uncemented. Piezometer instrumentation at the site indicates that the regional water table roughly coincides with the tailwater elevation, while a small perched water table exists around pool elevation. The presence of two separate water tables suggests that pool waters are insulated from the sandstone by various fine-grained deposits, thereby minimizing seepage through the foundation and the abutments. A finite element model was constructed in order to further understand seepage conditions at the site. The results supported findings from the field. It was found that the vast majority of available head was dissipated by fine-grained sediment that has accumulated behind that dam over time, providing little potential for the development of high vertical gradients required to initiate piping. However, vulnerability of the foundation material to erosion (due to piping or scour) remains a serious concern. Measures providing adequate protection to the abutments and the sandstone downstream of the dam are essential for continued safe operation.

1 Introduction

1.1 Background. The Rapidan Dam is located on the Blue Earth River, about five miles southwest of Mankato, MN. The Ambursen-type gravity dam was built between 1909 and 1911, and has since served mainly as a hydroelectric power generation facility. Originally owned by Northern States Power Company (NSP), the structure was severely damaged in the 1965 flood, leading NSP to abandon its power-generating operation. Several years later, in 1970, Blue Earth County purchased the dam from NSP. In 1984, following two decades of disuse, the county formed an agreement with Rapidan Redevelopment, LTD. (currently Rapidan Hydroelectric, LLC) to redevelop the dam for hydroelectric power production. A number of studies have since been commissioned to determine the future of the dam as it approaches 100 years since its construction.

1.2 The dam was constructed within the Jordan Sandstone formation, which has been found to be highly friable and permeable in places. Interestingly, seepage control measures appear not to have been seriously considered in the design. The dam's foundation and abutments lack an effective cutoff. Four sluice gates built into the upstream slab of the structure for the purpose of discharging river sediment malfunctioned shortly after construction, leading to the accumulation of fine-grained sediment behind the dam. Ironically, the presence of the sediment has likely been one of the reasons the dam has survived for so long, as it appears to be the main factor mitigating seepage problems.

1.3 Throughout the history of Rapidan Dam, springs have been reported at various locations in the apron and base slab. The most significant springs were discovered several years after the construction of the dam, and were located at the left and right edges of the downstream portion of the original concrete apron. Documentation shows that erosion of the apron resulted from the springs. French drains were constructed to reduce pressures beneath the apron, and additional concrete was placed in order to extend erosion protection. A recent diving inspection (2006) detected a weak spring, noting a drop in temperature but no flow, though it is believed to be old and stable. A single spring is visible in the upstream portion of the base slab within the structure, though records show that it has been present since the 1920s.

1.4 Soundings performed by Blue Earth County in December 1999 noted significant changes in the downstream apron. Subsequent and more thorough inspections through February 2002 showed an alarming degree of undermining of the structure such that large pieces of the base slab failed and presumably fell into the voids. Soundings performed in April of 2002 indicated even more extensive voiding than was first thought, prompting an emergency repair effort involving the district office. In order to resolve the undermining and stabilize the foundation, a rock berm was installed immediately below and downstream of the end of the spillway and concrete was pumped into the voids further upstream beneath the base slab. Additional rock was placed in the location of the failed apron and covered with reinforced concrete.

1.5 Since the repairs, the state of the downstream apron has been subject to more scrutiny. Several soundings have been performed since 2002, fortunately finding little to no change in the apron. Some other repairs have been made at the site, however. An inspection performed in the summer of 2007 for a potential failure mode analysis (PFMA) discovered some locations where the buttresses were not in contact with the 2002 concrete, requiring the placement of some additional concrete. Also, the rock face on the downstream right abutment was historically protected by a concrete façade, which has since failed. Progressive spalling of the exposed face was addressed in late 2007 with the installation of rock riprap.

1.6 Present Study. This report is part of a larger feasibility study aimed at evaluating several alternatives for the dam, including its possible removal, with a federal interest in restoring the aquatic ecosystem of the Blue Earth River. Future phases of the study will include a cost-benefit analysis that must incorporate maintenance costs associated with prolonging the life of the structure. Understanding the condition of the dam foundation is critical to defining the nature of future repairs required to achieve an acceptable level of safety. An initial evaluation of the cause of voiding of the foundation prior to the 2002 emergency repairs (Barr 2002) suggested that potential for piping of the foundation was low, and that scour was the primary cause of the undermining. However, the extent of the voids upstream of the spillway led to doubts about whether a scour mechanism could produce such an undermining. The costs associated with seepage remediation of the foundation material would be significant, so a better understanding of the dam foundation was required in order to perform an accurate cost/benefit analysis. A geotechnical investigation was therefore undertaken with the principal goal of evaluating the potential for piping in the dam foundation.

1.7 The investigation effort consisted of three main components: (i) performing borings in the pool and abutments and collecting samples for testing in order to better characterize the soils and bedrock in the vicinity of the dam, (ii) installing and collecting readings from a system of piezometers in the abutments, with the aim of describing the local groundwater regime, and (iii) performing a seepage analysis using data collected from (i) and (ii) to evaluate the potential for piping beneath the dam and downstream apron.

2 Geology

2.1 General Geology. The Blue Earth River, a major tributary on the Minnesota River, lies within the Western Lake Region of the Central Lowlands Physiographic Province, and is more precisely located in the sub-region known as the Blue Earth Till Plain Province of the State of Minnesota.

2.2 In the project vicinity the Minnesota River Basin is characterized as an elevated plain with drainage northward by the Blue Earth River to it's confluence with the Le Sueur River approximately two miles south of Mankato. The upland plain is essentially flat, but is deeply incised with series of complicated dendritic gulches or valleys which join the main Minnesota River Valley. The elevation of the upland near the Minnesota River valley ranges between 975 and 1000 feet MSL.

2.3 The valleys and gulches may be incised as deep as 200 feet into the overlying plain of glacial deposits and their widths vary from 300 to 500 feet. The upland plain is composed of a relatively thin mantle of clayey glacial lake silts, glacial drift and/or till, and stream alluvium. In the project vicinity the overlying deposits rest generally on eroded Cretaceous Period shale and Paleozoic Era dolomites, siltstones and sandstones. The Cretaceous shale deposits are absent at the project site however.

2.4 Site Geology. At the project site the overlying soils vary from the right abutment to the left abutment (facing downstream). The right abutment, which is closer to the valley wall, has a thicker section of glacial lake silts overlying the bedrock that forms the dam abutments. Approximately 20 feet of silts and another 25 feet of gravelly, sandy, clay till rest directly on the Jordan Sandstone or on a remnant portion of weathered clayey shale known as the Blue Earth Siltstone Member of the Ordovician Period Prairie Du Chien Formation. Where encountered the Blue Earth Siltstone is 2-3 feet thick. Below this siltstone the Cambrian Period Jordan Sandstone (El. 848) is deposited. This material forms most of the visible valley walls at the dam. Seepage from joints in the sandstone is readily visible at the site. The Jordan Sandstone is characterized as a massive to cross-bedded, wet, jointed, fine grained quartz sandstone. It is commonly iron stained and may be well cemented, but more commonly is poorly to un-cemented. Interbedded within this less indurated rock are very hard, well cemented beds of purple quartzite-type rock. These hard beds can range from 1-2 up to 10 feet in thickness, but generally comprise about 20% of the formation at this site. Based on one boring, total thickness of the Jordan Sandstone is approximately 86 feet at the project site.

2.5 The left abutment (facing downstream) is characterized by a thinner layer of overlying silty soils and no till. This is likely due to its location near the center of the valley which exposed the valley walls to more erosion over time. The overburden is composed primarily of 0-10 feet of sandy gravelly colluvium and/or fill, which in turn overlies about 10-feet of clayey, gravelly, colluvium. A 14-foot thick portion of the Blue Earth Siltstone Member (El. 863) lies below the overburden soils. Generally it is weathered, laminated, uncemented to poorly cemented, and wet. Again, beneath the Blue Earth Siltstone the Jordan Sandstone (El. 848) is deposited. Its character is similar to that as described in the right abutment. The hard purple quartzite-like beds in the left abutment can range from 1-2 up to 10-feet in thickness, but generally comprise about 20% of the formation at this site. The Jordan Sandstone rests on a dolomitic sandy clayey siltstone unit known as the St. Lawrence Formation (El. 764). This formation was not penetrated in its entirety at the project site so its total thickness is not known.

2.6 The bedrock at the centerline of Rapidan Dam and its river valley were not investigated for this report. It is known that the dam itself rests on the Jordan Sandstone Formation. The conservation pool upstream of the dam has an approximately 30 ft-thick layer of sediment deposited beneath the lake waters since the dam began operations. Samples obtained in the pool indicate that the bottom sediments are composed of organic-rich sands, silts and clays likely derived from the farm fields located on the flat upland plain adjacent to the river valley.

3 Investigation of local groundwater regime

3.1 In order for piping to occur, high upward hydraulic gradients are required downstream of the dam. High gradients can occur in the abutments or foundation if the foundation material provides insufficient dissipation of the differential head created by the dam. Therefore, measuring the head at various locations in the foundation and abutments is an effective means of evaluating the potential for piping. For this study, the groundwater regime was studied by installing a number of piezometers in the abutments. Data was also obtained from two pre-existing standpipe piezometers inside the dam to determine head levels below the structure.

3.2 Determining the mechanical properties of the sandstone foundation was another important aspect of evaluating the potential for piping failure. Rock descriptions obtained during the borings, as well as laboratory testing performed on core samples, formed the basis for the characterization of the sandstone.

3.3 Instrumentation plan. A total of six grout-in-place piezometers were installed in the abutments of Rapidan Dam. Locations of the boreholes in which the piezometers were installed are shown in Fig. 3.1. Table 3.1 lists the sensors with their corresponding installation elevation. Boreholes 08-2M and 08-4M each contain two piezometers installed at different depths in order to allow for the calculation of vertical hydraulic gradients. Holes 08-3M and 08-5M each contained a single piezometer at elevations corresponding with the "deep" sensors in the other holes.

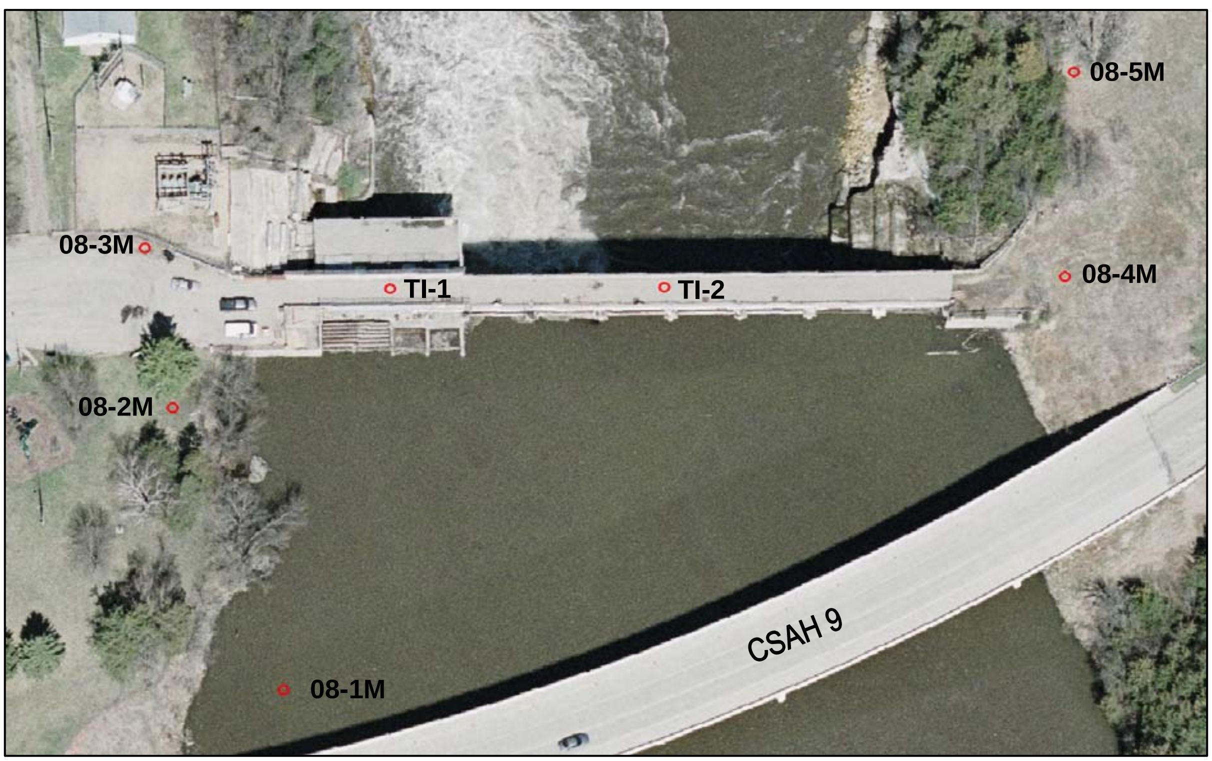

Figure 3.1 A site map showing locations of the borings in the abutments and pre-existing piezometers inside the dam (NAD – 83'/UTM).

Table 3.1

Piezometer installation elevations	
08-2M (shallow)	837.8
08-2M (deep)	798.0
08-3M	805.2
08-4M (shallow)	850.6
08-4M (deep)	800.6
08-5M	800.2

3.4 Additionally, two open-tube standpipe piezometers were installed below the lower walkway inside the dam at some point in the late 1970s or early 1980s in order to comply with Federal Energy Regulatory Commission (FERC) inspection requirements. Both were installed to several feet below the base slab beneath the lower walkway. A single reading was taken from each tube on February 21, 2008. Prior to taking the readings the piezometers were flushed and bailed to ensure that they were still functional. Installation elevations and head measurements are presented in Table 3.2. The measured head levels were no more than about one foot above tailwater elevation.

Table 3.2

piezometer	install elev.	head elev. (21 Feb 08)
TI-1	805.5	811.5
TI-2	802.35	810.7
(Pool)	-	871.3
(Tailwater)	-	810.4

3.5 Instrumentation results. All piezometers have provided stable readings since installation. Head measurements are plotted over time in Fig. 3.2. Two sensors were installed in “shallow” locations, either in the upper reaches of or just above the Jordan formation. Sensor 08-4M (shallow), which was positioned just above the sandstone on the right abutment, shows an initial drop in pressure following installation, followed by a fairly steady rise to the most recent reading (approx. El. 860). Data from 08-2M (shallow), which was installed near the top of the sandstone on the left abutment, are unusual in that they suggest “negative” pressure. This may be a result of some type of soil suction, which would indicate that the soil at that location is unsaturated. Alternatively, the sensor could be malfunctioning, though readings are stable and show some correspondence with data other piezometers.

3.6 The remaining sensors were installed at deeper locations, between El. 798 and El. 805. All show very similar readings, with head levels several feet above tailwater (approx. El. 815). Figure 3.3 shows the readings from the deep piezometers in detail.

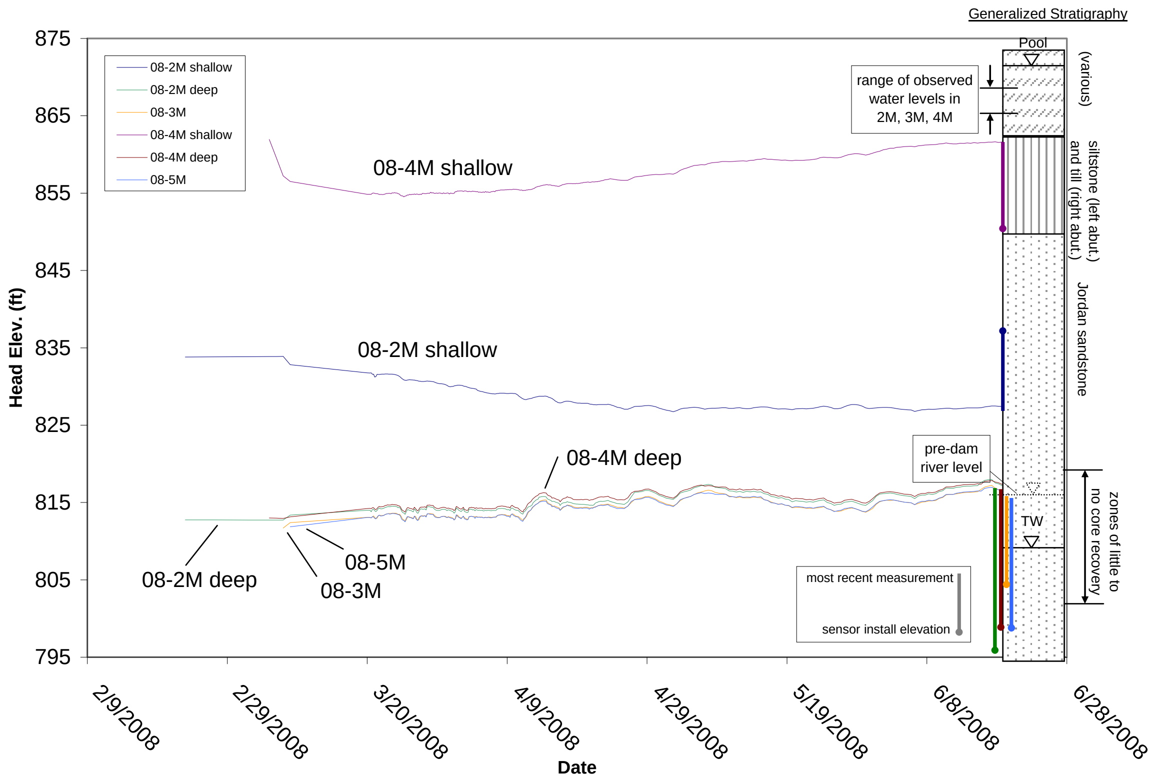

Figure 3.2 Piezometer readings at Rapidan Dam between February and June 2008.

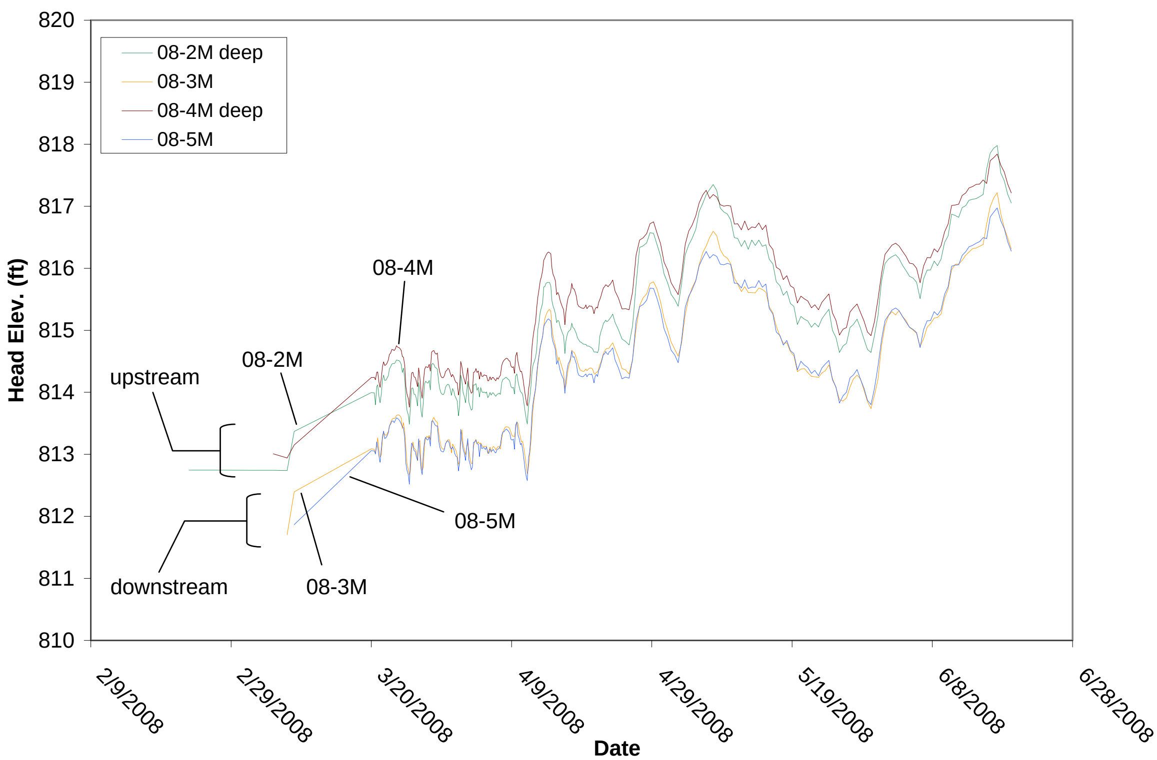

Figure 3.3 Detail of readings from piezometers installed at "deep" locations.

3.7 Interpretation. The piezometer readings, along with water level observations made during drilling, indicate that two separate water tables exist in the abutments. A perched water table is likely fed by pool waters, while the lower water table within the Jordan Sandstone near the tailwater probably represents the regional water table.

3.8 First, there is an upper, perched water table slightly lower than the pool elevation. This interpretation is based on piezometer 08-4M (shallow) and water level observations made during drilling in boreholes 08-2M, 08-3M, and 08-4M. In each of these borings, water was observed several feet below the pool elevation (Table 3.3). In boring 08-5M, no water was observed in the borehole, perhaps an indication of the perched nature of the water table. Finally, seepage has been consistently observed at about El. 850 on the downstream left abutment, slightly above the first appearance of the Jordan Sandstone. (Fig. 3.4).

Table 3.3

Water level observations from drilling	
Pool	871.3
Tailwater	810.4
08-2M	868.5
08-3M	865
08-4M	864.8

3.9 Data collected at the site also indicates there is a lower, regional groundwater table slightly above the tailwater elevation (approx. El. 810). Readings from all four piezometers installed at deep elevations (08-2M (deep), 08-3M, 08-4M (deep), and 08-5M), indicate head several feet above the tailwater elevation. This is consistent with visual observation of seepage on the downstream left abutment (Fig. 3.4). Similar seepage has been observed on the right abutment behind the stepped concrete retaining wall during previous inspections. However, the area is now blocked by riprap, limiting access.

3.10 By all indications, there is little to no flow from the perched upper water table to the lower regional water table. As stated earlier, sensor 08-2M (shallow), which is situated in the upper reaches of the Jordan sandstone, does not show positive pressures. Also, while seepage on the left abutment is observed at the two differing elevations, there is little to no seepage between the two sources. Finally, the color and quantity of water distinguish the seeps. Seepage higher up on the wall is sparse and clear in color, while seepage below near the tailwater is heavy and rust-colored.

3.11 The site geology as discussed in section 2 presents a number of potential aquitards to explain the separate water tables. Upstream of the dam, pool sediments provide a barrier between water in the pool and the sandstone. Fine-grained deposits are also found above the sandstone in the abutments. On the left abutment, the Blue Earth Siltstone overlies the Jordan Sandstone. Borings on the right abutment show massive glacial till deposits above the sandstone.

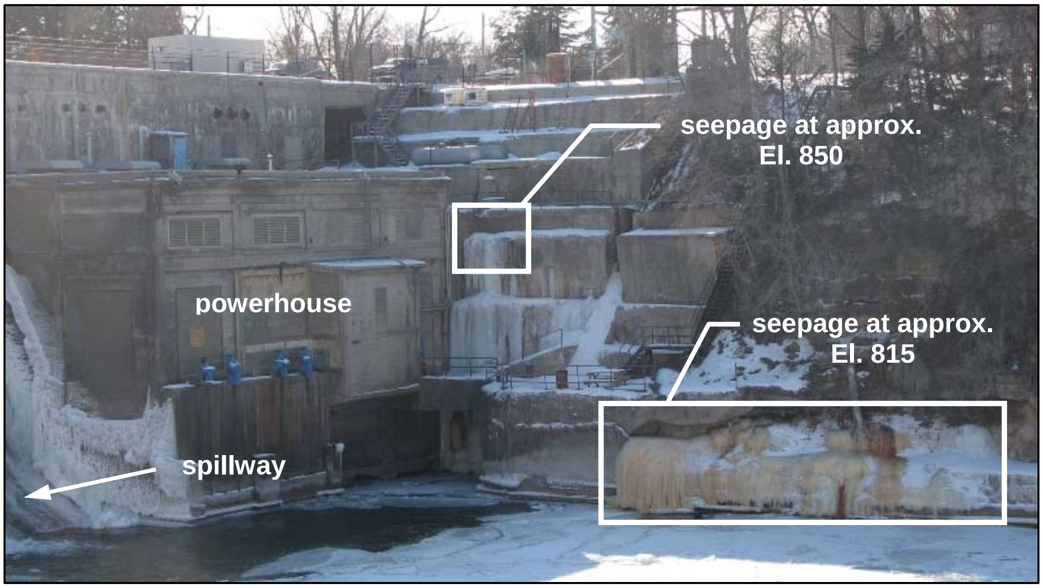

Figure 3.4 Photograph of downstream left abutment showing frozen seepage at two locations (February 2008).

4 Laboratory Testing

4.1 Two material properties in particular were of interest for this study: soil and rock permeabilities and resistance of the sandstone to erosion. Most testing was focused on determining these properties. Some additional testing was performed in order to better classify or characterize particular materials.

4.2 Permeability. Knowledge of permeability of the various rocks and soils that comprise the local geology provides a context in which to explain head measurements and understand the local hydrogeology. Reliable permeability estimates are also important for constructing a useful seepage model. Several constant head and falling head permeability tests were requested in order to directly measure permeability. Additionally, mechanical analyses of a number of other samples were requested, provided estimated ranges of permeability using empirical correlations.

4.3 Results of permeability testing are summarized in Table 4.1. Falling head permeability tests of the pool sediment yielded uniform results, showing that the organic clay is effectively impervious.

4.4 Permeability of the sandstone was much less uniform. Test results ranged from 5.0×10^{-3} ft/min (very permeable) to 1.6×10^{-9} ft/min (effectively impervious). Higher values resulted in poorly cemented highly quartzose samples (08-2M [2], 08-2M [7]) while lower values were obtained for hard, well-cemented sandstones (08-4M #1, 08-2M #6) and silty sandstones (08-3M #2). Medium values were found for some silty sandstones (08-2M #8, 08-2M #10).

Table 4.1

Summary of permeability test results					
hole #	**sample/ core #**	**elev.**	**soil/rock type**	**test type**	**permeability k (ft/min)**
08-1MU	1	857.1	organic clay (OH)	flex-wall falling head	2.1E-07
08-1MU	2	850.2	organic clay (OH)	flex-wall falling head	2.5E-07
08-2M	2	844.5	sandstone	flex-wall constant head	2.8E-03
08-2M	6	824.5	sandstone	flex-wall falling head	9.4E-08
08-2M	7	808.5	sandstone	flex-wall constant head	5.0E-03
08-2M	8	798.5	sandstone	flex-wall constant head	7.5E-04
08-2M	10	780.0	sandstone	flex-wall constant head	1.1E-03
08-3M	2	803.4	sandstone	flex-wall constant head	6.1E-05
08-4M	1	840.0	sandstone	flex-wall falling head	1.6E-09

4.5 Unconfined compressive strength. Both proposed mechanisms by which the undermining could have occurred (piping and scour) involve erosion of the sandstone by water. Therefore, it was important to characterize the ability of the foundation material to resist erosion. While at least one specialized test is available to measure erosion resistance directly (jet index test), it is most effective when performed in-situ and is not widely available. Instead, unconfined compressive strength testing is a relatively cheap and practical alternative. In sandstone, cementation of the grains is what provides both cohesive shear strength and resistance to erosion. Therefore, a number of unconfined compressive strength tests were requested for rock core samples at various depths.

4.6 One caveat of using unconfined compressive strength to judge resistance to erosion is that the test requires solid cores with an aspect ratio of at least 1W:2L in order to produce accurate test results. The test therefore favors hard, well-cemented zones where few mechanical breaks occurred. For this project, a significant proportion of the recovered cores were not “solid”, thus precluding them from testing. Furthermore, large sections of core samples in every boring were not recovered, especially between El. 800 and 820. This likely indicates that rock at those elevations was uncemented and soft, perhaps due to leaching of cementation by the regional groundwater table. It is worth noting that in many cases the elevations of no core recovery correspond well with the elevation of the dam foundation.

4.7 For the testing, four core samples of varying texture and composition were chosen. Three core samples were taken from hole 08-2M and one from hole 08-5M. Results are shown in Table 4.2. The results show a high degree of variability of the sandstone. Three of the samples represent the medium-hard moderately to well-cemented sandstone, while the other represents silty poorly-cemented soft sandstone. Unconfined shear strength varied by two orders of magnitude between the groups. It is thought that most of the material between El. 800 and 820 best resembles core #10 from 08-2M. Figures 4.1 and 4.2 present photographs of recovered sandstone cores to provide a visual sense of the variability in texture.

Table 4.2

Summary of unconfined compression test results				
hole #	**core #**	**elev.**	**rock description**	**UCS (tsf)**
08-2M	4	835.4	breaks readily, m. hard, mod. to well-cemented, fe-stained, white to yellow to red	35.21
08-2M	9	795.2	quartzose, breaks readily, m. hard in zones, soft, uncemented in zones, lt. fe-stained in zones, brn. to white	13.55
08-2M	10	779.0	quartzose, silty, soft, poorly cemented, grey	0.21
08-5M	1	822.5	quartzose, mod. to well-cemented, v. to unweathered, fe-stained in zones, brittle fracture, orange to white	14.68

4.8 Other tests. Several other tests were performed for secondary reasons. Shear strength of the pool sediment was of interest for general characterization of the material, and may be pertinent to the dam removal alternative (e.g. if excavations or slopes are required). To that end, two unconsolidated undrained (UU) triaxial shear tests at different depths were requested for undisturbed samples taken from 08-1MU. The clay (CH) samples were found to exhibit plastic behavior and fairly low shear strength, as shown in Table 4.3.

Table 4.3

Summary of U-U triaxial test results							
hole #	**sample #**	**elev.**	**LL**	**PL**	**MC (mean)**	**ϕ_u (deg)**	**c_u (tsf)**
08-1MU	1	857.1	86.0	34.4	69.9	2.3	0.16
08-1MU	2	850.2	78.2	31.4	61.2	1.9	0.18

4.9 Additionally, Atterberg limits were requested for several soils overlying the Jordan Sandstone in order to ensure proper classification. Moisture contents of fine-grained soils were also requested in order to characterize the material behavior. These results were intended mostly for classification purposes and are not used or discussed in this report.

Figure 4.1 A box of rock cores from hole 08-2M showing typical variation of the Jordan Sandstone at the Rapidan Dam. Core samples ranged from hard, well-cemented, purple (left) to medium-hard with medium to poor cementation (middle) to crushed and uncemented (right).

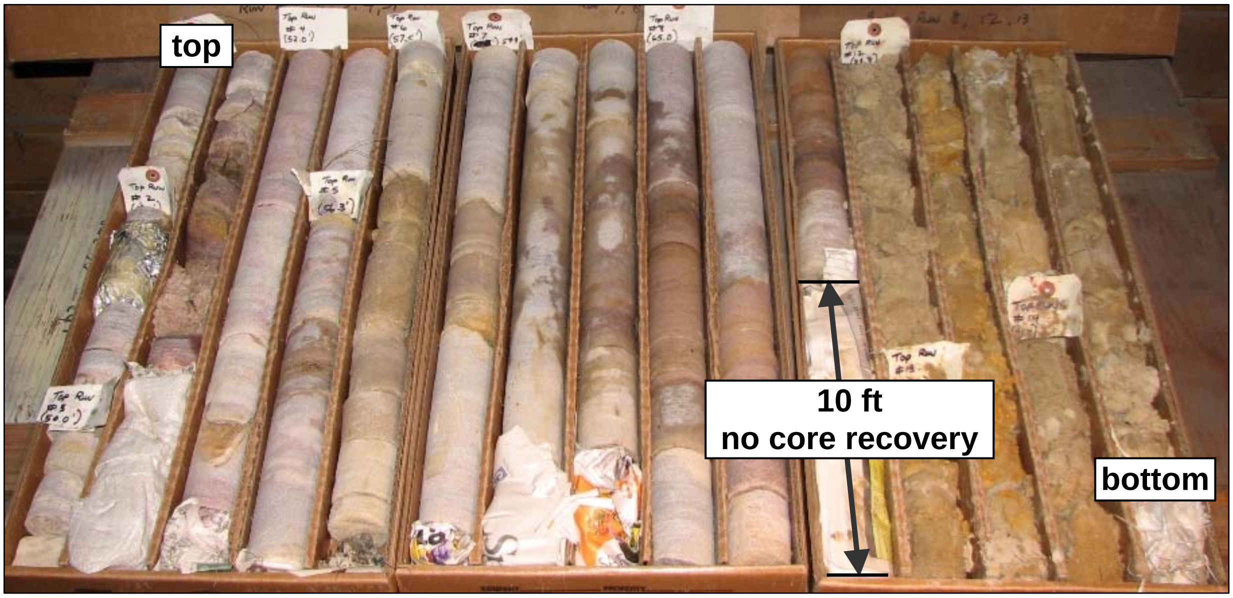

Figure 4.2 Core samples from 08-4M show a stark change in texture. Hard, well-cemented purple sandstone (left, middle) dominated at the top of the formation and transitioned to crushed, uncemented sandstone (right) below about El. 820. The transition is marked by roughly 10 ft of unrecovered core.

5 Seepage Analysis

5.1 The basic goal of the seepage analysis was to consider a wide variety of scenarios that might yield hydraulic gradients capable of initiating piping. Factors that were considered included gaps or openings of some sort along the foundation and apron, as well as varying levels of sediment in the pool. Using laboratory testing data, a critical vertical gradient was calculated to which gradients computed in the models were compared in order to evaluate the likelihood of piping for each scenario.

5.2 Seep/W model. A cross-section model of the dam (Fig. 5.1) was created using the geometry specified in the original plans. Important features of the dam geometry included the lengths of the base slab and the downstream apron, the keys on the upstream and downstream edges of the dam, and the end sill of the concrete apron. Although there have been extensions and modifications of the apron, they are not well documented. It was judged that modeling the original apron would yield conservative results.

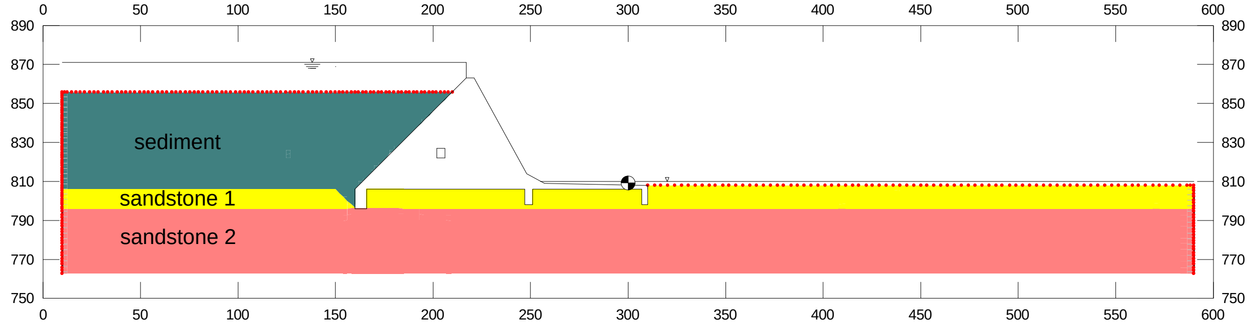

Figure 5.1 Seep/W model of the Rapidan Dam.

5.3 Three main materials were used in the model. Sediment behind the dam was modeled as a single material about 50 ft thick (Barr 2000). The Jordan Sandstone was divided into two layers (sandstone 1 and sandstone 2) based on testing data used to estimate permeabilities. Finally, a wedge of material representing clay backfill was included immediately behind the upstream key.

Table 5.1 Permeabilities used in the Seep/W model.

Seepage Model Stratigraphy				
layer	base elev.	top elev.	permeability k	
			(ft/min)	(cm/s)
Sediment	806	856	2.3E-07	1.2E-07
Clay backfill	-	-	2.3E-07	1.2E-07
Sandstone 1	796	806	1.5E-02	7.6E-03
Sandstone 2	763	796	4.0E-03	2.0E-03

5.4 Permeabilities chosen for the materials are shown in Table 5.1. The sediment was assigned a permeability based on the average of the two tests from 08-1MU described in paragraph 4.8. For the sandstone, results of permeability testing were used in conjunction with correlations based on grain size in order to estimate permeabilities (see Fig. 5.2). Values determined from direct testing of the rock cores are likely lower than average since core samples selected for testing were generally better cemented, and therefore less permeable, than the surrounding material. On the other hand, mechanical analyses of jar samples favored uncemented material, likely yielded permeability values greater than the average. Therefore, intermediate values were ultimately chosen for the model.

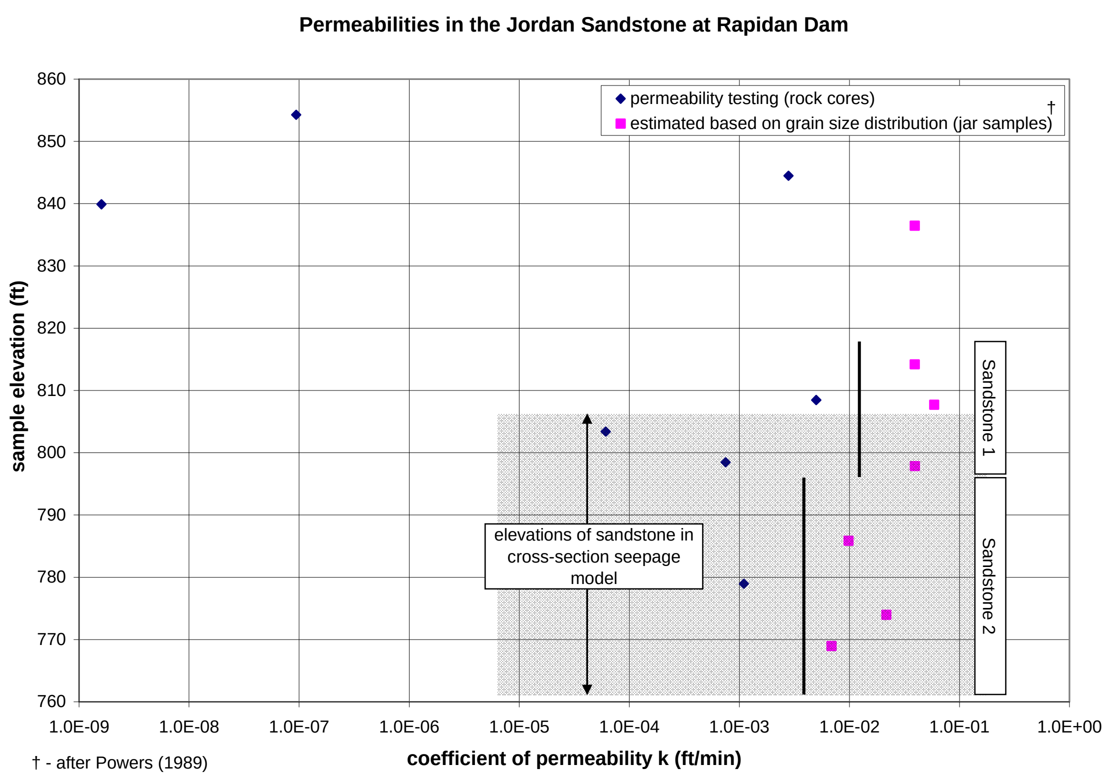

Figure 5.2 Permeability data for the Jordan Sandstone at Rapidan Dam.

5.5 Head boundary conditions were chosen based on measurements taken during February 2008. The upstream boundary condition was determined by the pool (El. 871), while the downstream boundary condition was determined by the tailwater (El. 810.4). Small variations in either level were found to be insignificant to the output. Infinite elements were employed on the upstream and downstream edges of the model in order to account for the undefined nature of those boundaries.

5.6 Scenarios modeled. A number of seepage scenarios were investigated in order to determine the potential for piping. Beginning with a basic configuration, several modifications and/or additions were made to the downstream apron or the base slab in order to represent possible physical phenomena that might allow high gradients. The critical gradient for piping initiation was computed to be $i_{cr} = 1.05$ based on dry unit weight and moisture content data using the below equation. It was assumed for this calculation that the cores were saturated, which is a conservative assumption.

$$i_{cr} = \frac{\gamma_{sat} - \gamma_w}{\gamma_w}$$

5.7 The first scenario that was investigated was a model of the dam in a basic configuration. Only the basic head boundary conditions upstream and downstream of the dam were applied. The vertical gradient was recorded at the boundary node immediately downstream of the apron.

5.8 Two subsequent scenarios postulated an area of erosion immediately downstream of the apron, forming a wedge-shaped void. To model this lack of material, the geometry of the material downstream of the apron was altered, and the tailwater boundary condition was applied to the newly exposed nodes (Fig. 5.2).

5.9 The next case considered some defect in the apron further upstream near the spillway that would expose the sandstone to the tailwater head level (Fig. 5.3). The defect could be a crack or gap in the concrete due to scour.

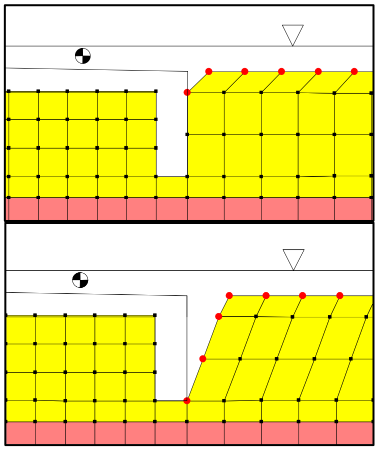

Figure 5.2 Scenarios considering a gap due to erosion downstream of the apron sill.

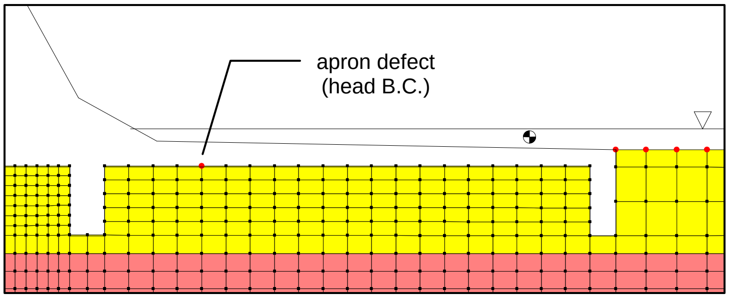

Figure 5.3 Scenario considering a defect such as a gap in the downstream apron.

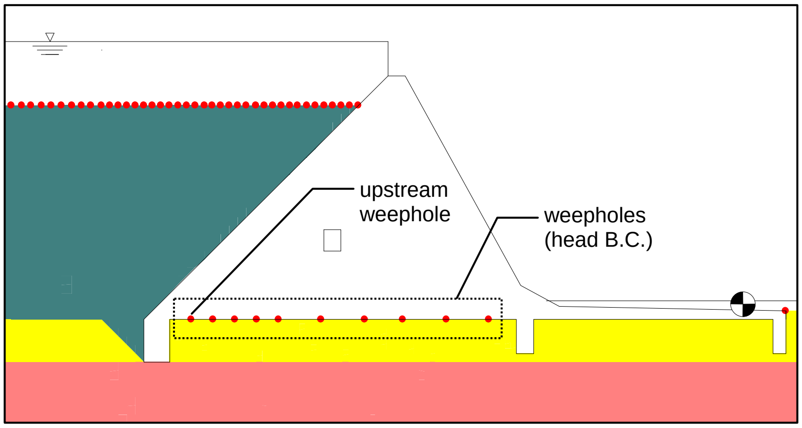

Figure 5.4 Weepholes were modeled by applying a head boundary condition (tailwater) at the weephole location.

5.10 The final scenarios considered weepholes in the base slab presumably incorporated into the design in order to relieve water pressure in the foundation. The holes were not investigated but are assumed to exist as shown in the 1910 drawings. In order to model the holes, a head boundary condition was applied at the weephole location (Fig. 5.4), as was done for the apron defect. This case was of interest due to at least one spring that has been identified inside the dam, and is thought to coincide with a weephole location. At the time the weepholes were designed and constructed, filter criteria for preventing soil particle transport had not yet been developed, and thus the weepholes could allow material loss under high upward pressures. One scenario considers just the weephole furthest upstream, where the highest pressures are expected. In the other, all weepholes are modeled.

5.11 Results of the above scenarios are summarized in Table 5.2. The highest gradient was obtained from the scenario considering a single weephole at the location furthest upstream. Still, the result (i = 0.148) is about an order of magnitude smaller than the computed critical gradient. The model leaves little doubt that piping is unlikely in the dam's current configuration. As additional confirmation, the head levels at the locations where the open-tube piezometers were installed were computed to be between 810 and 813 in all of these scenarios, coinciding well with actual head measurements reported in Table 3.2.

Table 5.2

scenario	gradient location	i_{max}
basic	downstream of apron	0.0176
gap downstream of apron (small)	bottom of gap	0.0244
gap downstream of apron (large)	bottom of gap	0.0246
apron defect (no weepholes)	at defect	0.0733
single weephole (upstream)	upstream weephole	0.148
all weepholes	upstream weephole	0.0509

5.12 As an additional check, a number of the above scenarios were considered for varying levels of pool sediment. Having accumulated over time, there was likely little fine-grained sediment in the pool immediately after construction. While the model shows that piping is unlikely at current levels of sedimentation, the presence of less sediment would result in higher gradients throughout the foundation and apron. Thus, the purpose of this check was to determine if critical gradients could be achieved at past sediment levels.

5.13 Results are shown in Table 5.3 and plotted in Figure 5.5 on a semi-log scale. Plots of total head and vertical gradient contours for select cases can be found in Appendix A. Only with zero pool sediment did critical gradients (i > 1.05) develop, highlighting the substantial ability of the fine-grained sediment to dissipate head. Of the critical scenarios for zero sediment, (iii) appears to offer the greatest potential for piping.

Table 5.3

sediment thickness (ft)	scenario			
	(i)	**(ii)**	**(iii)**	**(iv)**
0	0.258	1.61	5.30	2.01
1	0.0491	0.236	0.468	0.155
5	0.0252	0.112	0.221	0.07438
10	0.0211	0.0909	0.182	0.06167
20	0.0189	0.0801	0.161	0.05501
50	0.0176	0.0733	0.148	0.05091

scenario	gradient location
(i) basic	downstream of apron
(ii) apron defect (no weepholes)	at defect
(iii) single weephole (upstream)	upstream weephole
(iv) all weepholes	upstream weephole

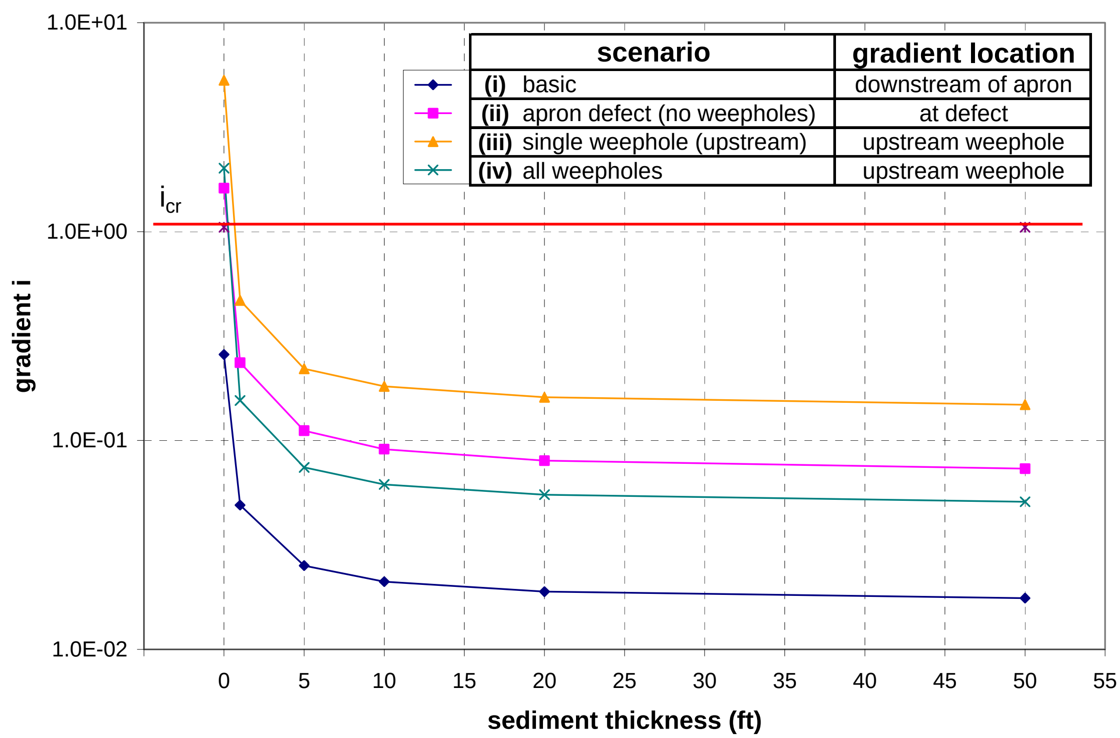

Figure 5.5 Computed vertical hydraulic gradients for several scenarios with varying sediment thickness.

5.14 The only scenarios for which piping seems possible are those with no pool sediment, indicating that piping of material could have occurred immediately following the construction of the dam. The historical potential for piping gives rise to the possibility that piping was initiated early on, but was increasingly inhibited by the collection of sediment until ceasing altogether. This would have resulted in some voiding beneath the dam that could have been exposed with the progression of downstream scour. However, this seems unlikely given that the highest gradients occur at the upstream weepholes, whereas the voids discovered in 2002 were concentrated in the downstream portions of the foundation.

5.15 Despite the possibility of historical potential for piping, there currently do not appear to be any seepage problems at the Rapidan Dam. The only evidence of upward flow is several reported springs either inside the dam or in the downstream apron. The springs could be paths of relatively high hydraulic conductivity that developed as a result of material loss early in the dam's history. Another explanation would be the presence of high-conductivity joints within the sandstone. In any case, the springs are reported to be weak and stable, and thus are of little concern.

6 Conclusions

6.1 The geotechnical investigation of the Rapidan Dam yielded a number of important conclusions regarding the potential for piping in the foundation. Data from borings, piezometer readings, laboratory testing and seepage modeling were used to assess the erodibility of the foundation material and characterize potential seepage below and around the dam.

6.2 An evaluation of the foundation materials revealed that the Jordan Sandstone is pervious and very friable in places. The zones of little to no core recovery between El. 800 and El. 820 are particularly disconcerting, as they roughly coincide with the elevation of the foundation. The small amount of material that was recovered in those zones generally exhibited little to no cementation. The loose, soft rock would be sensitive to upward flow beneath the dam or downstream apron. Any erosive process, whether scour or piping, would be expected to progress fairly rapidly within this material.

6.3 With respect to piping potential, groundwater measurements from beneath the dam indicate that the differential head is dissipated almost entirely by the fine-grained sediment that has accumulated behind the structure. Potential flow through the abutments appears to be impeded by fine-grained rocks and soils located above the sandstone formation as well as the pool sediment.

6.4 Seepage analyses performed with Seep/W support the interpretation of the field evidence, indicating low head levels below the dam and downstream apron. In a 2D cross-section model, it was found that even a thin blanket of fine-grained sediment in the pool dissipates much of the available differential head, restricting the development of vertical gradients beneath the dam and throughout the downstream apron.

6.5 The findings of this study suggest that it was very unlikely that recent piping was involved in the undermining discovered in 2002. Given the lateral variation in the permeability of the Jordan Sandstone, as was found in the borings, it is possible that areas of localized flow exist that are not reflected in the head measurements. The likely presence of joints in the rock also provides opportunities for the development of localized groundwater flow. Nevertheless, evidence of localized flow is confined to several weak, stable springs throughout the foundation and downstream apron that are of little concern.

6.6 Although the data gathered for this report suggest that current seepage is not a problem, this study was not exhaustive and there remains the possibility that seepage issues exist. The more critical finding of the report is the significant vulnerability of the foundation to erosion. Adequate measures must be taken to properly protect the abutments and foundation from erosion, and frequent inspections should be conducted in order to verify that the protection is effective.

References

Barr Engineering Company. (2000). *Rapidan Dam Feasibility Study: Dam Removal Option.* Minneapolis, MN.

Barr Engineering Company. (2002). *Rapidan Dam Feasibility Study: Dam Repair Option*. Minneapolis, MN.

Powers, J. P. (1981). *Construction Dewatering: A Guide to Theory and Practice.* John Wiley & Sons, Inc. New York.

U.S. Army Corps of Engineers. (2002). Rapidan Dam Emergency Repairs After Action Report. St. Paul, MN.

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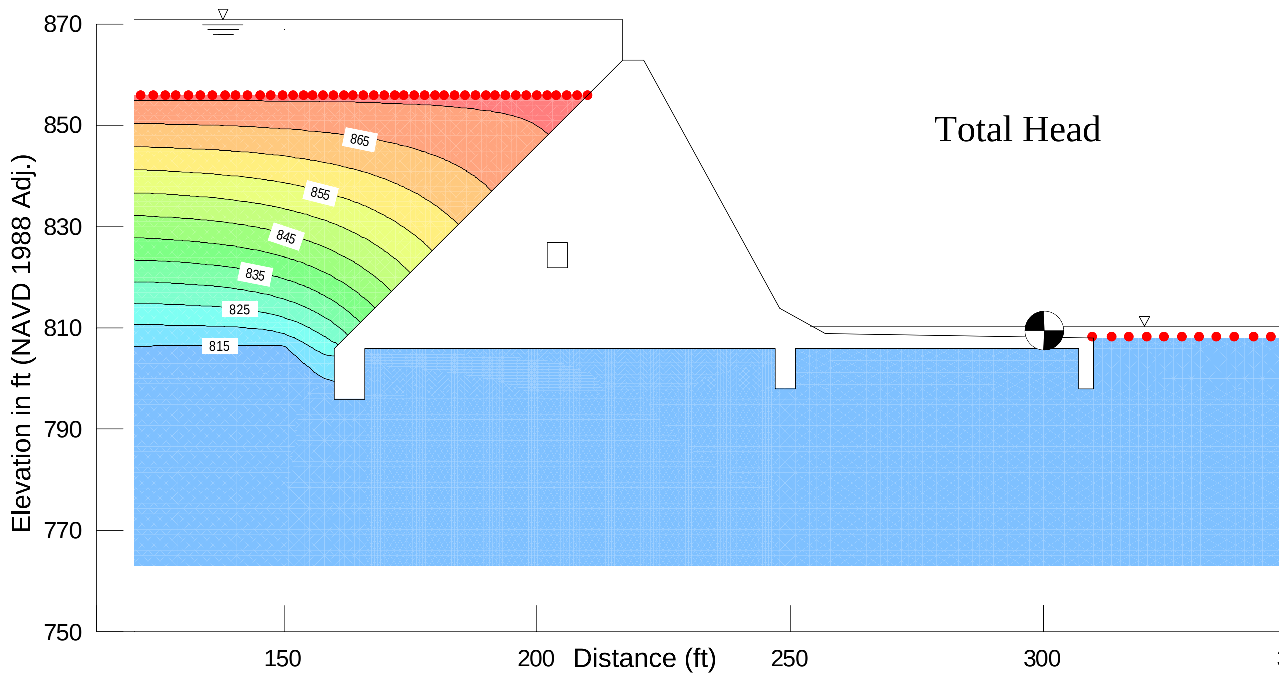
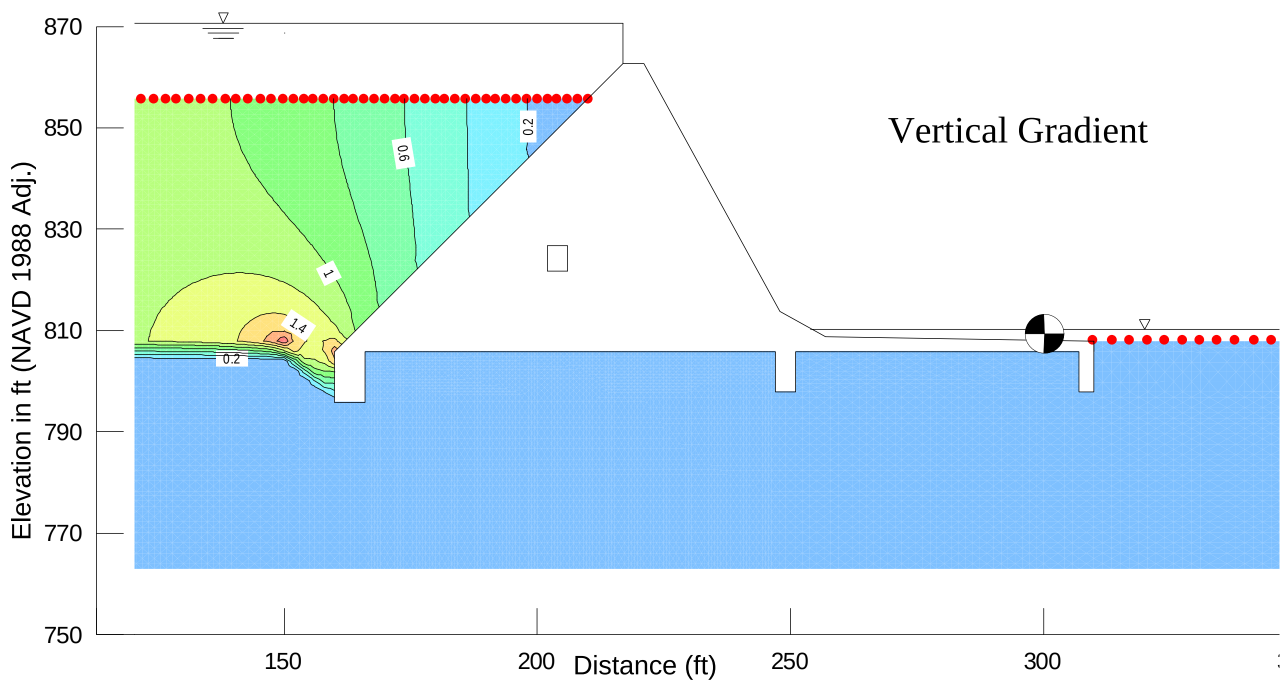
Appendix A: Seepage Modeling Graphical Results

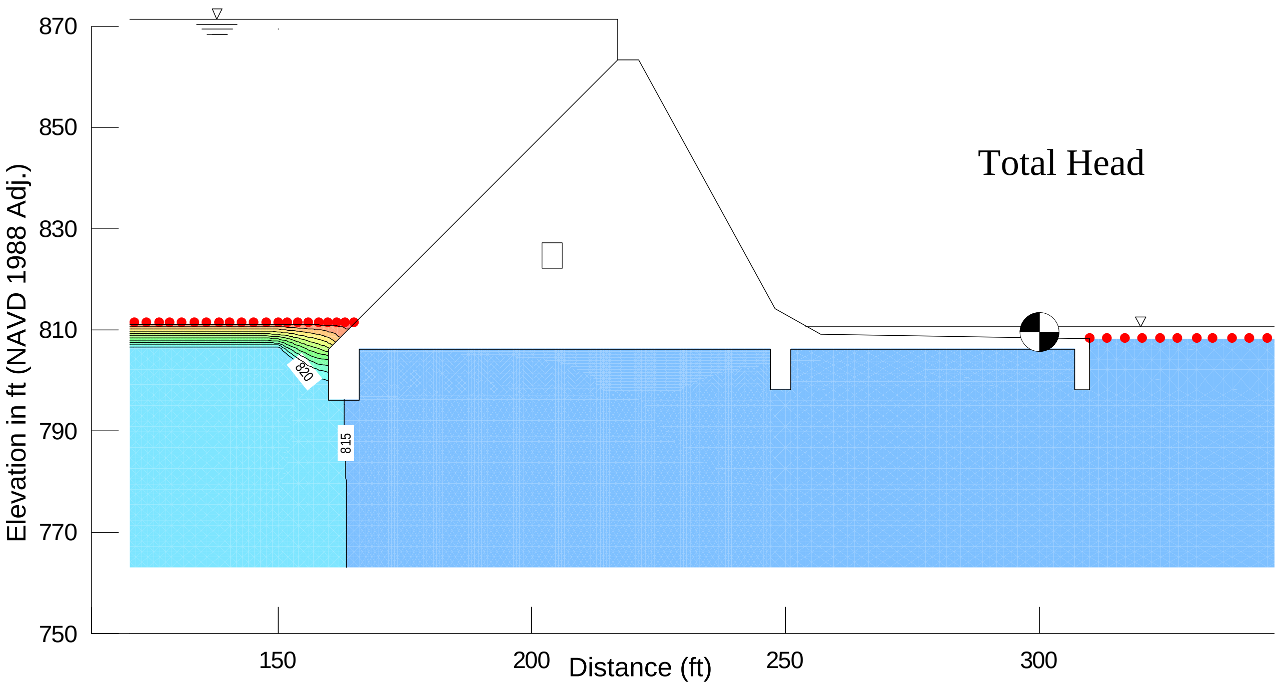
The following plots of total head and vertical hydraulic gradient have been produced for selected cases. For sediment levels where only scenario (i) is presented (50 ft and 5 ft), there was no discernable difference in the plots for the other scenarios (ii), (iii), and (iv).

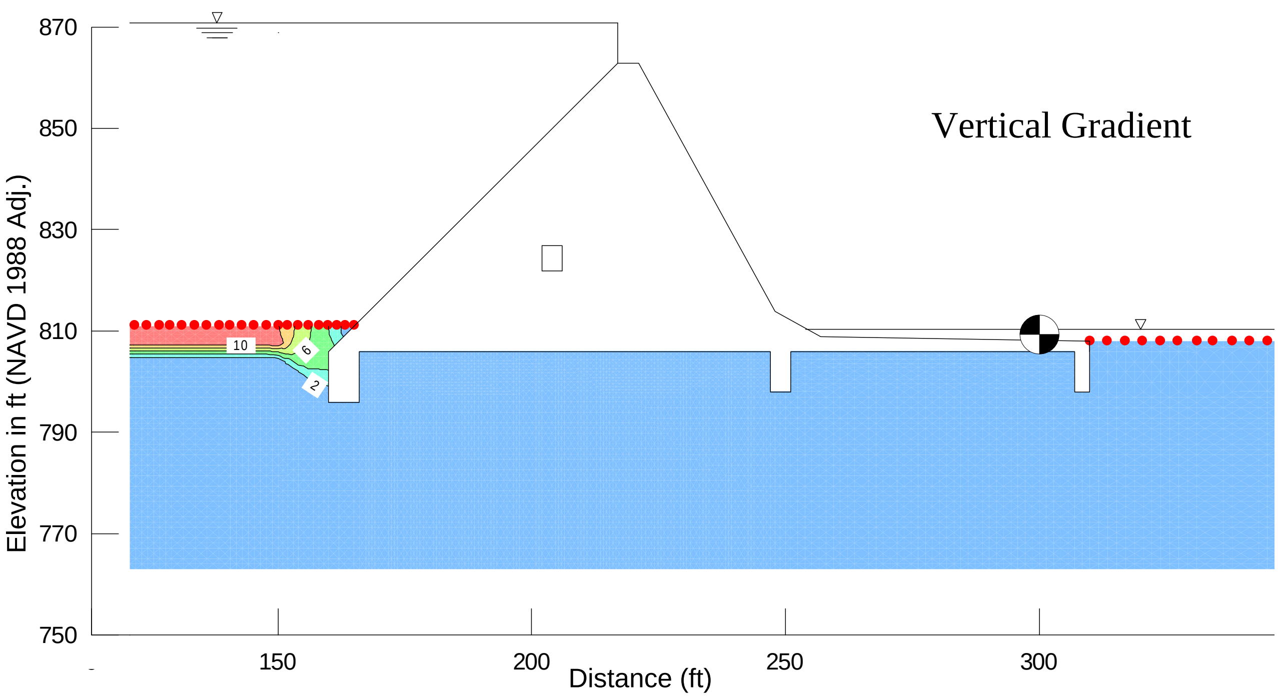
50 ft sediment - basic scenario (i)



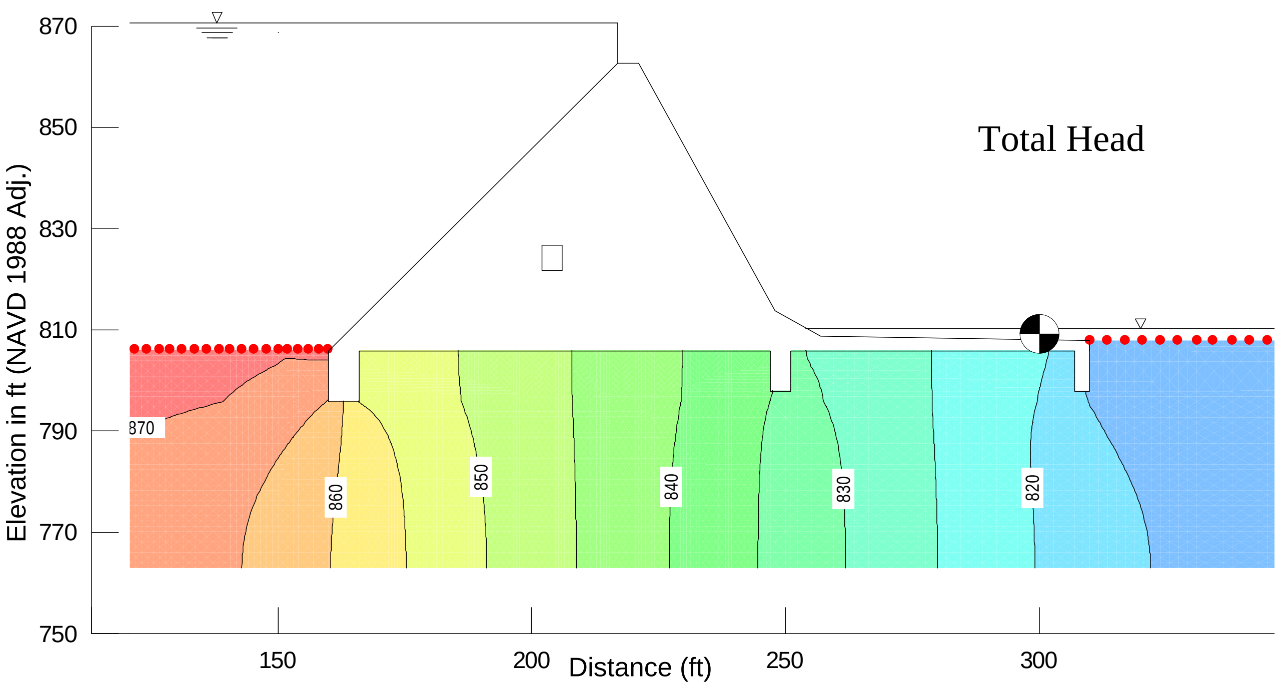


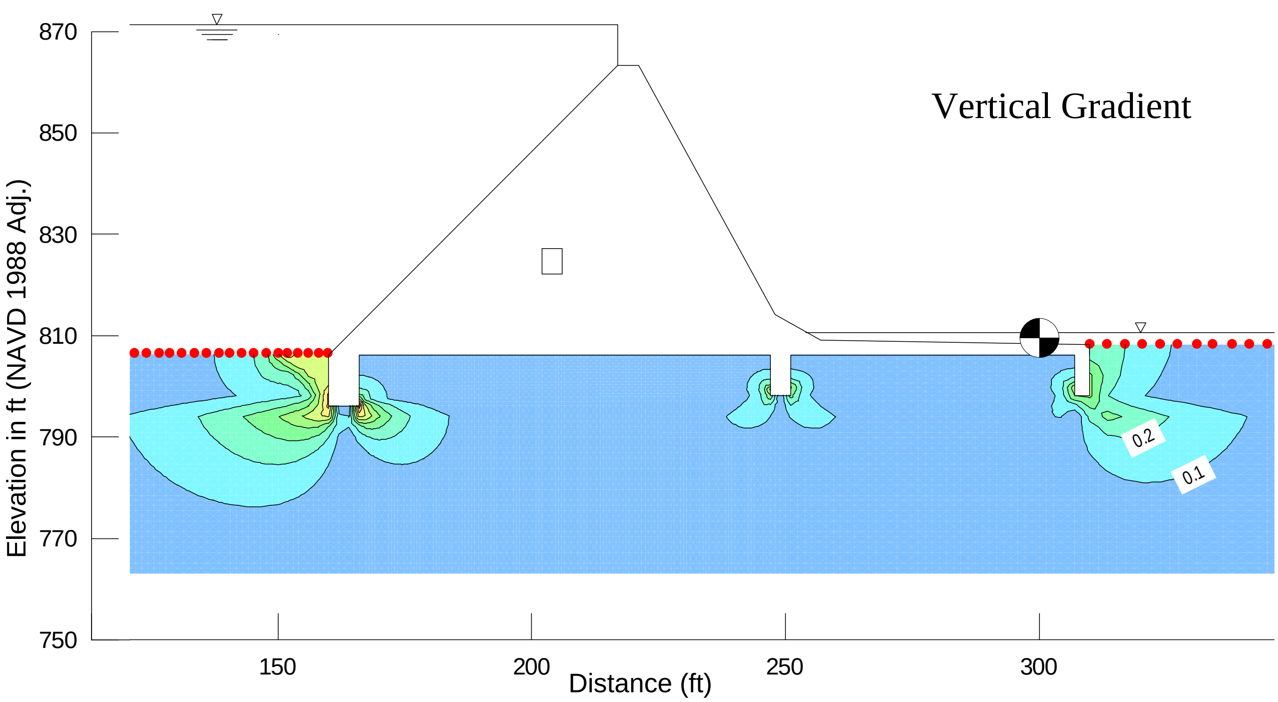
5 ft sediment - basic scenario (i)



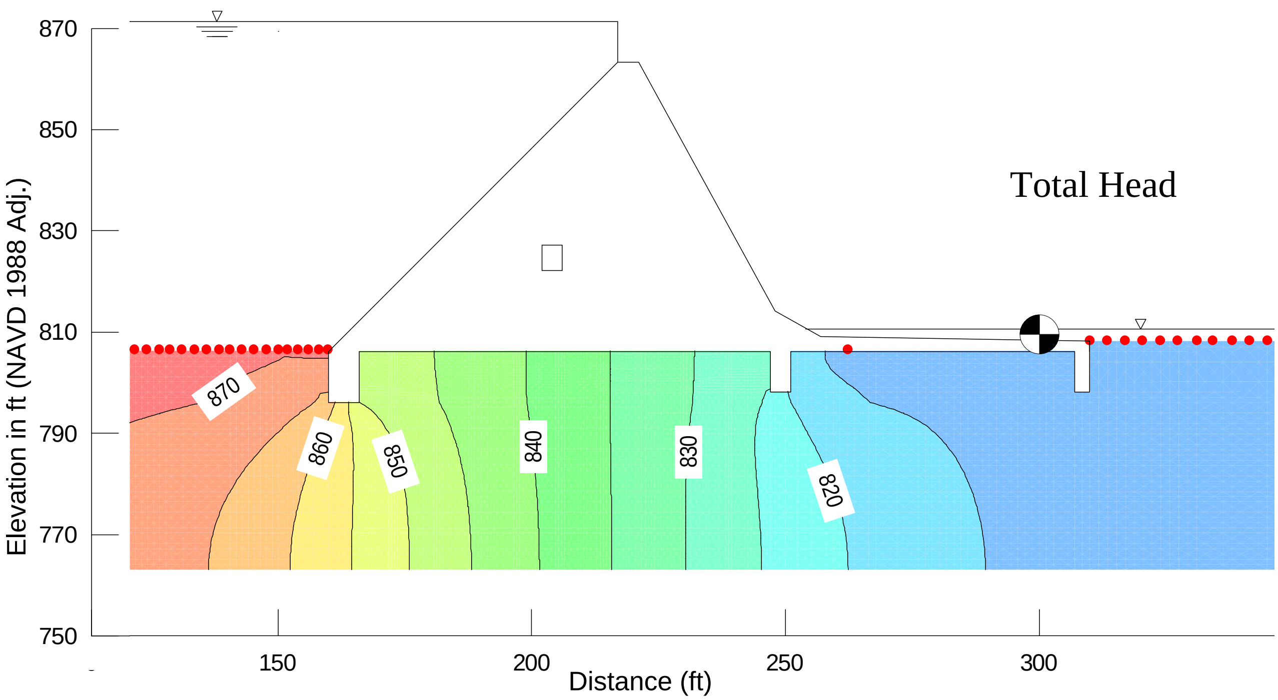


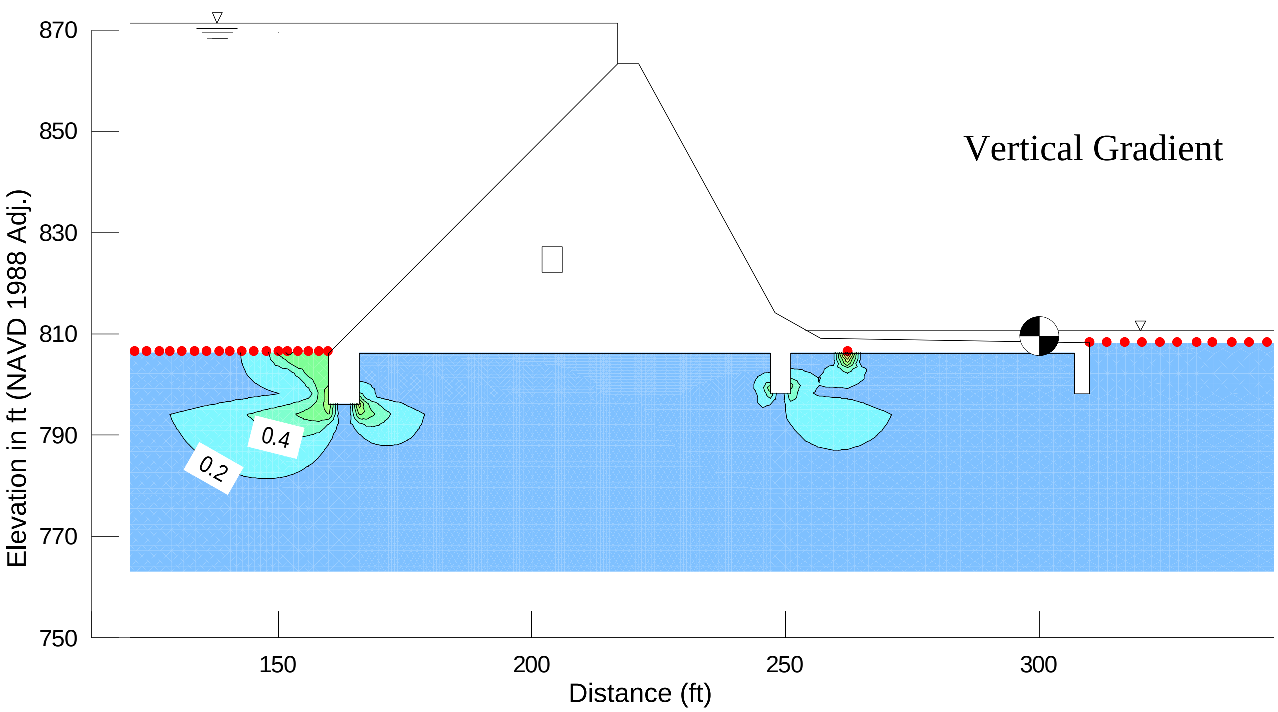
no sediment - basic scenario (i)



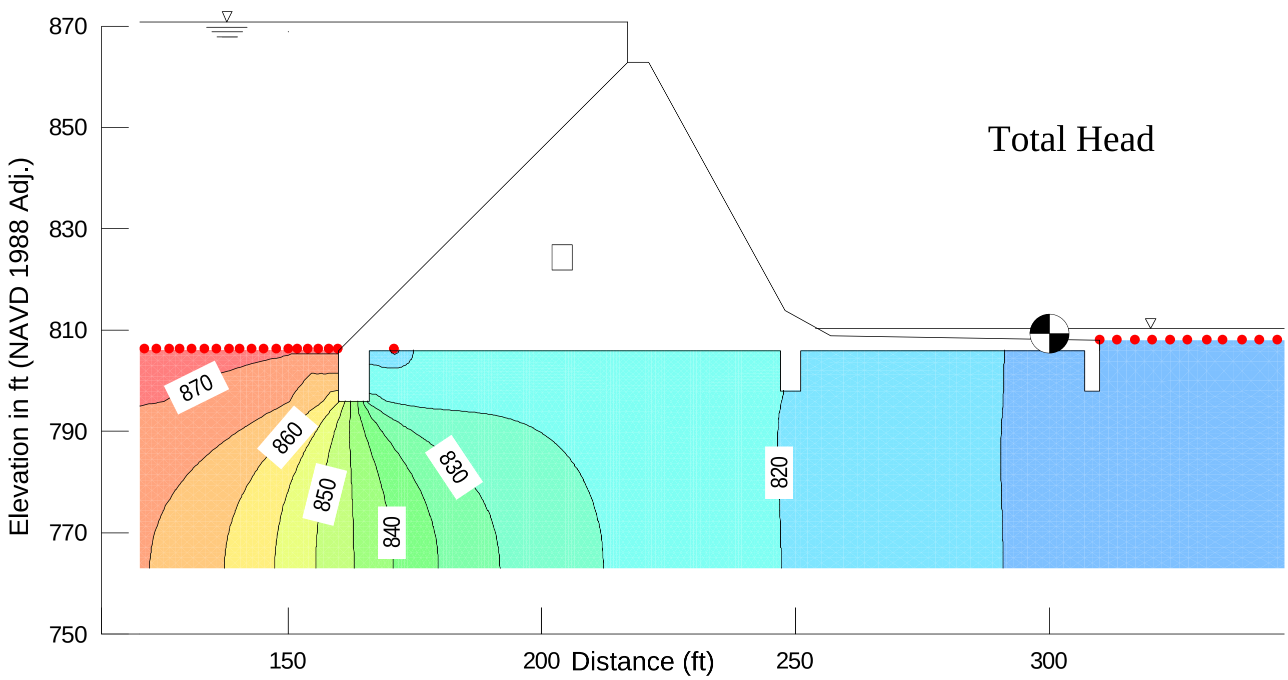


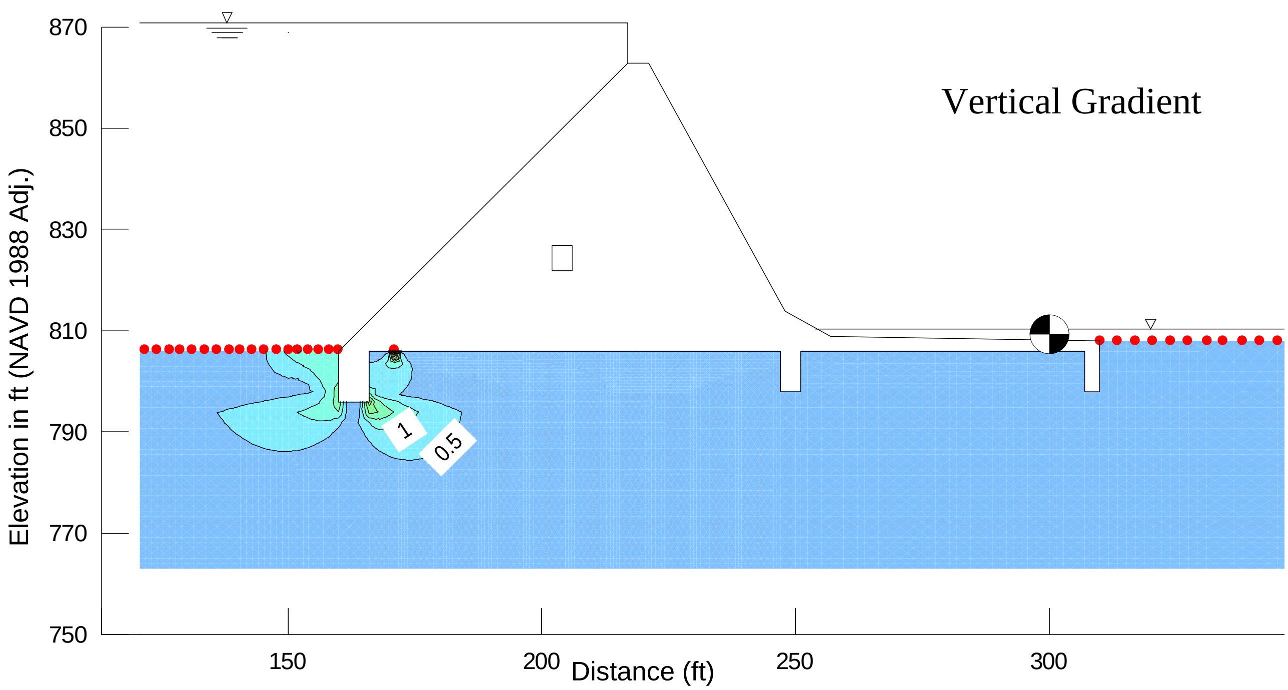
no sediment – apron defect (ii)



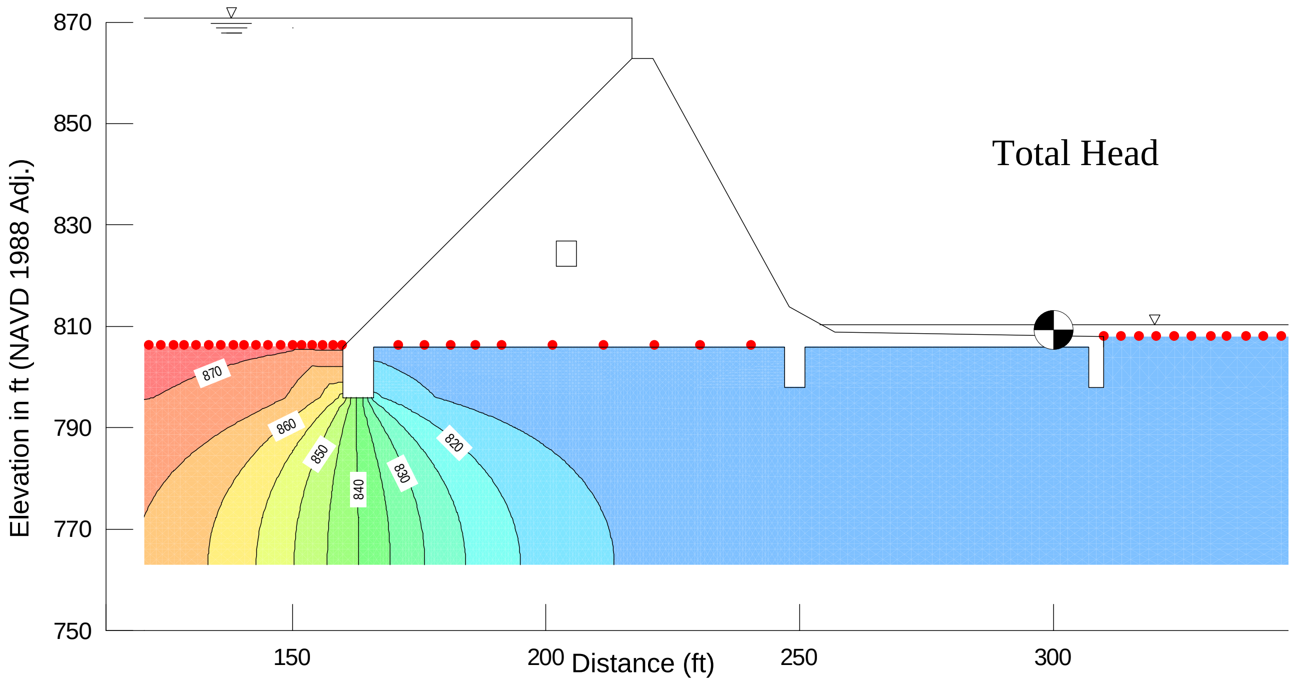


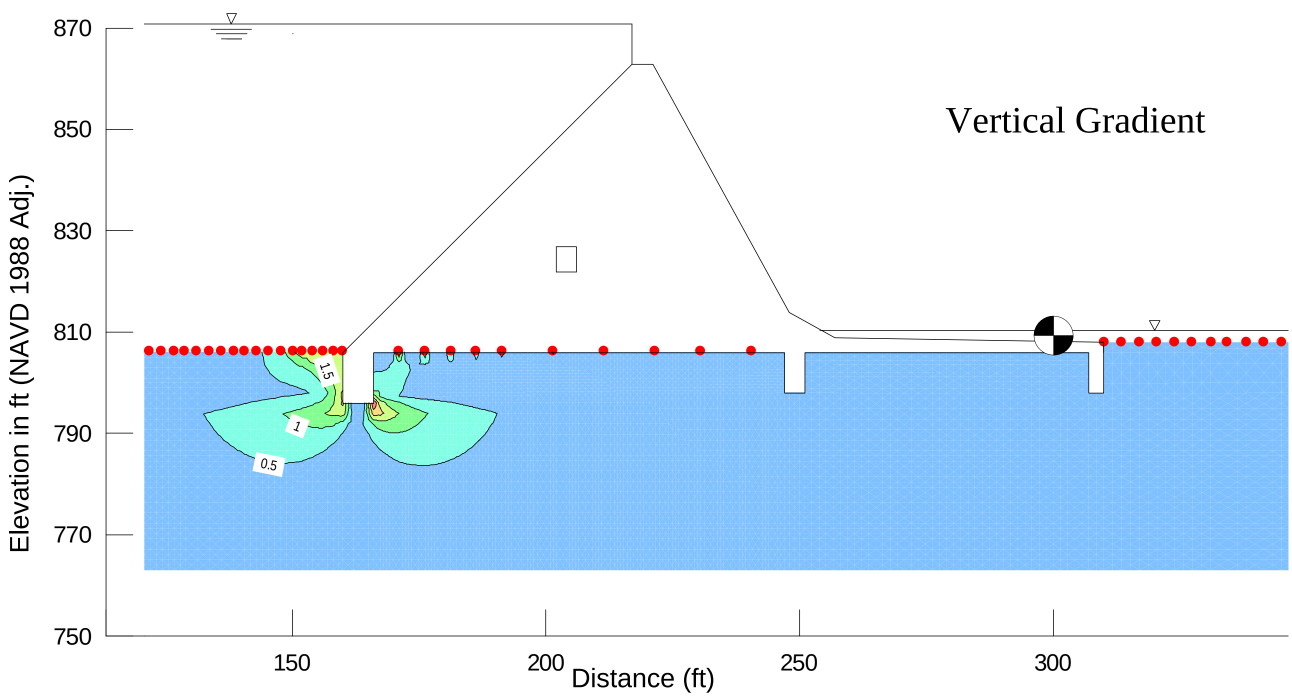
no sediment – single upstream weephole (iii)

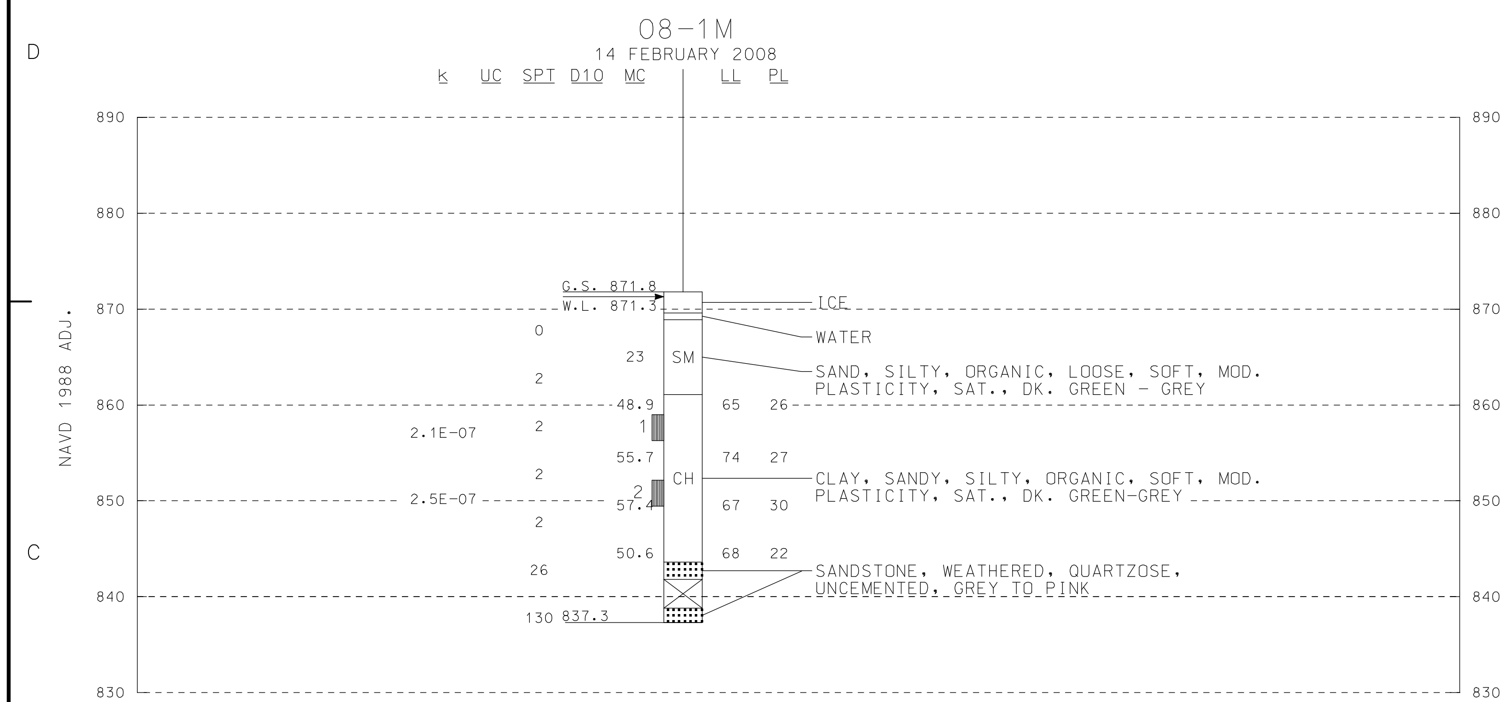




no sediment – all weepholes (iv)







NOTES:

1. WATER LEVEL IS SURFACE EL. OF BLUE EARTH RIVER ON DAY OF BORING.
2. HOLE STABILIZED WITH HOLLOW STEM AUGER TO EL. 838.8'.
3. 5-INCH UNDISTURBED SAMPLES OBTAINED IN OFFSET BORING.
4. HOLE BACKFILLED WITH TREMIED BENTONITE GROUT.

GENERAL BORING LEGEND

84-1M	YEAR OF BORING-BORING NUMBER, BORING TYPE (EG: M=MACHINE, A=AUGER, TP=TEST PIT, P=PIEZOMETER).
1 MAY 1984	DATE OF BORING
W.S. 1026.7	WATER SURFACE ELEVATION ON DAY OF BORING
G.S. 1020.2	GROUND SURFACE ELEVATION AT BORING
GW	WELL GRADED GRAVELS, GRAVEL - SAND MIXTURE, LITTLE OR NO FINES
GP	POORLY GRADED GRAVELS, LITTLE OR NO FINES
GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
SM	SILTY SANDS, SAND - SILT MIXTURES
SC	CLAYEY SANDS, SAND - CLAY MIXTURES
ML	INORGANIC SILTS, LIQUID LIMIT LESS THAN 50
MH	INORGANIC SILTS, LIQUID LIMIT GREATER THAN 50
CL	INORGANIC CLAYS, LOW TO MEDIUM PLASTICITY, LIQUID LIMIT LESS THAN 50
CH	INORGANIC CLAYS, HIGH PLASTICITY, LIQUID LIMIT GREATER THAN 50
OL	ORGANIC SILTS OR CLAYS, LOW PLASTICITY, LIQUID LIMIT LESS THAN 50
OH	ORGANIC SILTS OR CLAYS, MEDIUM TO HIGH PLASTICITY, LIQUID LIMIT GREATER THAN 50
PT	PEAT
SP-SM	BORDERLINE MATERIAL
SP&SM	STRATIFIED MATERIAL
	SANDSTONE
	SILTSTONE
	SHALE
1	LOCATION AND SAMPLE NUMBER FOR UNDISTURBED SAMPLE
	DRILL OUT
W.L. 726.7	WATER LEVEL ON DATE OF BORING
700.1	ELEVATION AT BOTTOM OF BORING
(238.56)	ELEVATION IN METERS

GENERAL BORING NOTES

1. GENERAL: THE UNIFIED SOIL CLASSIFICATION SYSTEM IS USED TO IDENTIFY BASIC SOIL TYPE. THE LEGEND REPRESENTS ONLY THE BASIC SOILS. TO COMPLETE THE CLASSIFICATION, PERTINENT INFORMATION IS ADDED TO THE RIGHT OF THE BORING STAFF. NOTES PERTAINING TO A SPECIFIC BORING ARE SHOWN BELOW THE BORING STAFF.

2. PERMEABILITY (K): PERMEABILITY IN FT/MIN AS DETERMINED FROM CONSTANT AND FALLING HEAD TESTS IS SHOWN TO THE LEFT OF THE BORING STAFF.

3. UNCONFINED COMPRESSIVE STRENGTH (UC): UNCONFINED COMPRESSIVE STRENGTH IN TSF IS SHOWN TO THE LEFT OF THE BORING STAFF.

4. MOISTURE CONTENT (MC): NATURAL MOISTURE CONTENT IN PERCENT OF DRY WEIGHT ARE SHOWN TO THE LEFT OF THE BORING STAFF.

5. BLOW COUNT (SPT): BLOW COUNTS ARE SHOWN TO THE LEFT OF THE BORING STAFF AND, EXCEPT AS NOTED, ARE THE NUMBER OF BLOWS NECESSARY TO DRIVE THE SAMPLER USED A DISTANCE OF 12-INCHES. STANDARD BLOW COUNTS ARE FOR A STANDARD PENETRATION TEST (SPT) USING A 1" X 2-INCH SAMPLER, 140 LB. HAMMER, AND A 30-INCH DROP. FOR NON-STANDARD BLOW COUNTS SAMPLER SIZE, HAMMER WEIGHT, AND HEIGHT OF DROP ARE AS SHOWN.

6. ATTERBERG LIMITS: LIQUID LIMIT (LL) AND PLASTIC LIMIT (PL) ARE SHOWN TO THE RIGHT OF THE BORING STAFF.

7. D10 SIZE: THE GRAIN SIZE IN MILLIMETERS OF WHICH 10% OF THE SAMPLE IS FINER IS SHOWN TO THE LEFT OF THE BORING STAFF.

8. % RECOVERY: PERCENT CORE RECOVERY IS SHOWN TO THE LEFT OF THE BORING STAFF. PERCENT RECOVERY IS LENGTH OF CORE RECOVERED/LENGTH OF CORE CUT X 100. UNLESS SPECIFIED OTHERWISE, ALL CORE IS 4-INCH DIAMETER.

8. ELEVATIONS REFERENCED TO N.A.V.D. 1988 ADJ. UNLESS SPECIFIED OTHERWISE.

9. THE BORINGS SHOW SUMMARIES OF INFORMATION RECORDED ON THE ORIGINAL FIELD LOGS. THESE LOGS ARE AVAILABLE FOR INSPECTION AT THE ST. PAUL DISTRICT OFFICE. ARRANGEMENTS TO INSPECT LOGS CAN BE MADE BY CALLING (651) 290-5599.

US Army Corps of Engineers
St. Paul District

Symbol	Description	Date	Appr.

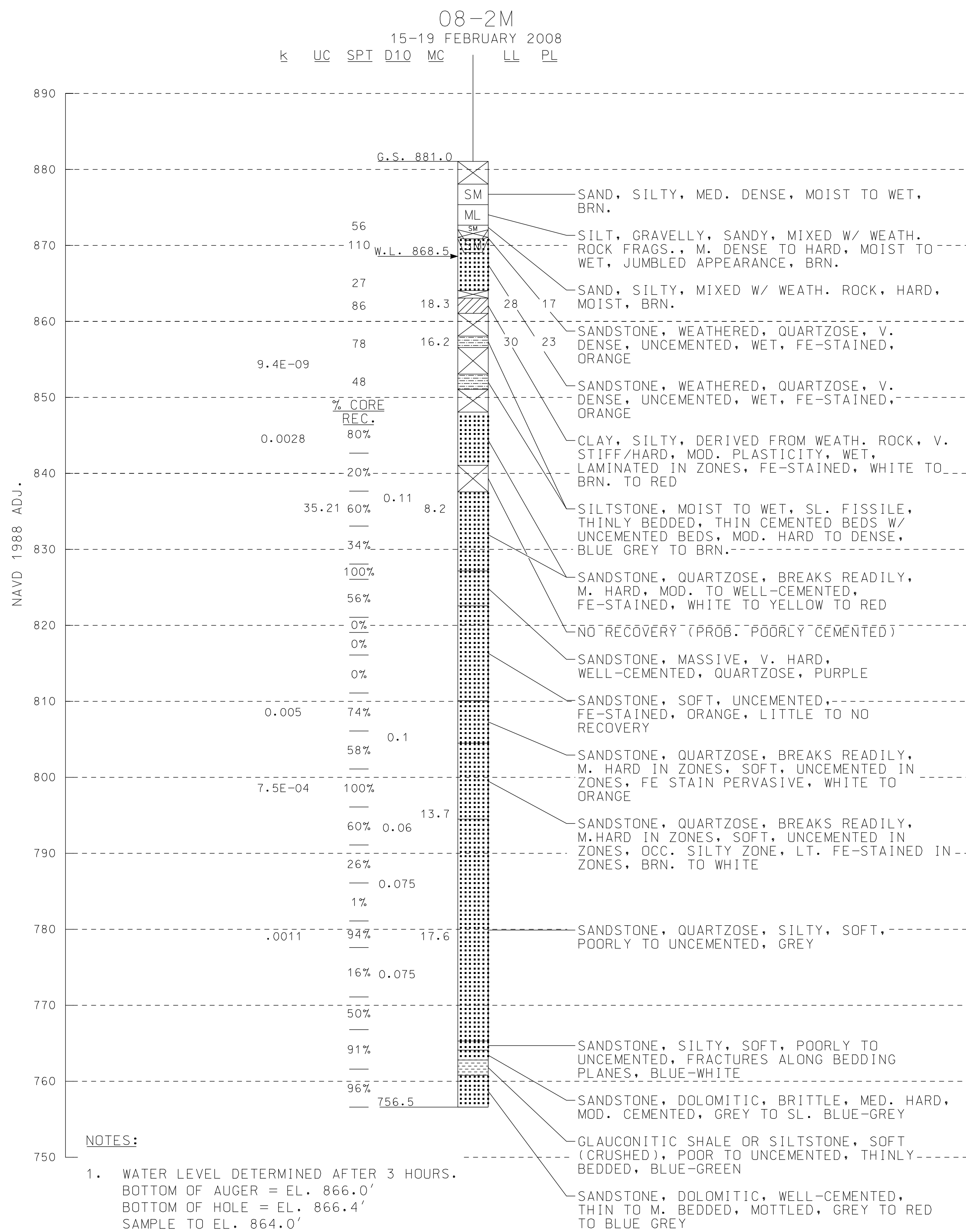
Designed by: GSW | Dwn by: GSW | Ckd by: GAR | Date: 25 AUG 2008 | Rev.
Solicitation No.:
Drawing Number:
Authority:
File Name:
Plot Scale:
Plot Date:

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT
CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

Horizontal Coordinate System:
Vertical Coordinate System: NAVD 1988 Adj.

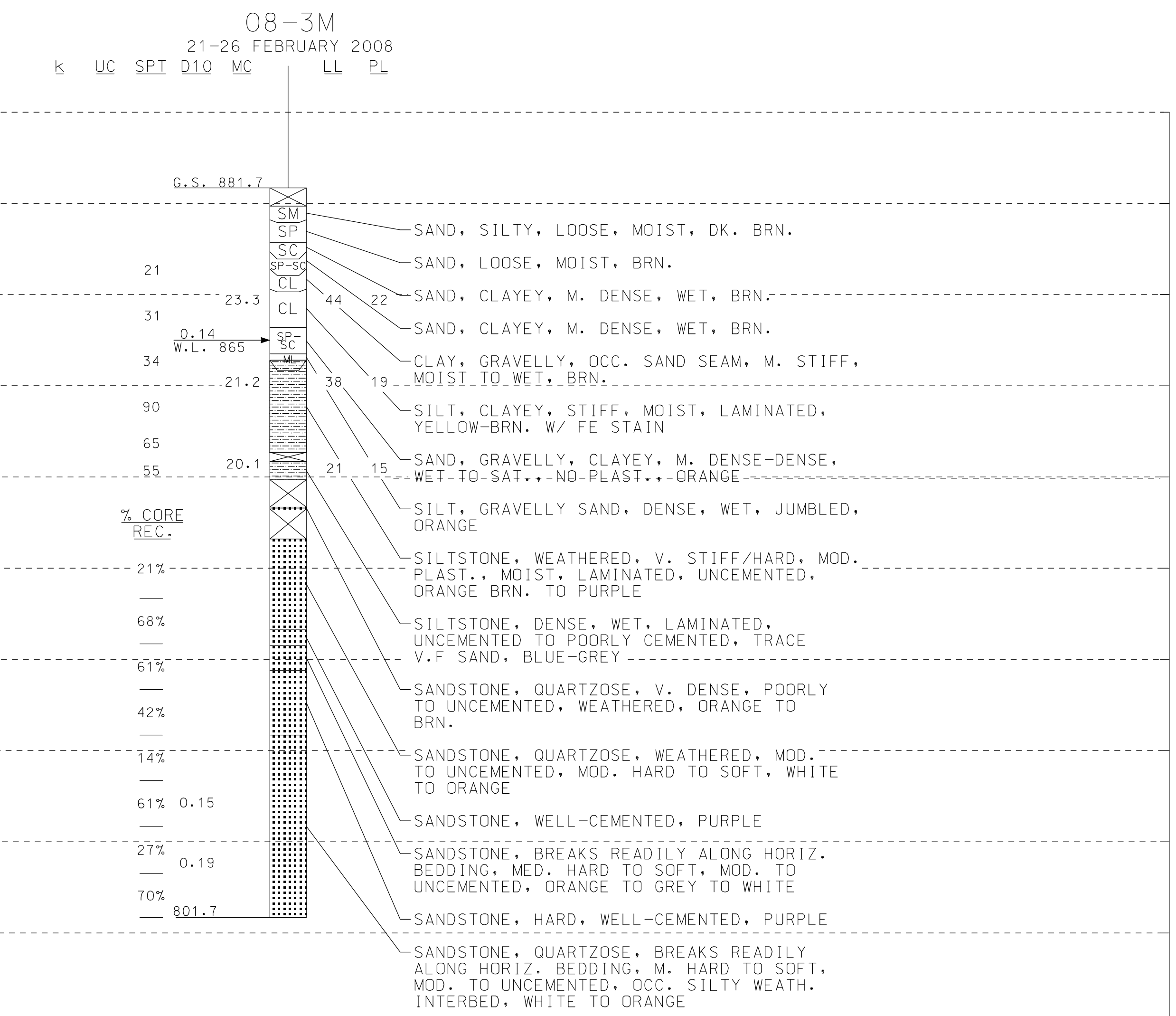
BLUE EARTH RIVER
FEASIBILITY STUDY
RAPIDAN DAM
GEOTECHNICAL INVESTIGATION

Sheet reference number:



NOTES:

1. WATER LEVEL DETERMINED AFTER 3 HOURS.
   BOTTOM OF AUGER = EL. 866.0'
   BOTTOM OF HOLE = EL. 866.4'
   SAMPLE TO EL. 864.0'
   BLUE EARTH RIVER WATER SURFACE = EL. 871.3'.
2. HOLE STABILIZED WITH HOLLOW STEM AUGER TO EL. 866.0'. AUGER PULLED AND 4-INCH FLUSH JOINT SET TO EL. 865.9'. HOLE STABILIZED WITH BENTONITE DRILLING FLUID TO EL. 848.0'. SET NX FLUSH JOINT CASING TO EL. 848.0'. HOLE STABILIZED WITH DRILLING MUD TO EL. 817.5'. RESET NX CASING TO EL. 817.5'. HOLE STABILIZED WITH DRILLING MUD TO BELOW EL. 817.5'.
3. 2-INCH CORE BARREL WITH DIAMOND BIT USED BELOW EL. 847.6'.
4. MUD LOSS DURING CORE RUN BETWEEN EL. 811'-806'.
5. VIBRATING WIRE PIEZOMETERS INSTALLED IN COMPLETED BOREHOLE.



NOTES:

1. WATER LEVEL DETERMINED AFTER 2 HOURS.
   BOTTOM OF AUGER = EL. 863.2'
   BOTTOM OF HOLE = EL. 863.3'
   SAMPLE TO EL. 861.7'
2. HOLE STABILIZED WITH HOLLOW STEM AUGER TO EL. 863.2'. PULL AUGER AND RESET 4-INCH FLUSH JOINT CASING TO EL. 863.2'. HOLE STABILIZED WITH DRILLING MUD TO EL. 843.2'. SET NX FLUSH JOINT CASING TO EL. 843.2'. HOLE STABILIZED WITH DRILLING MUD BELOW EL. 843.2'.
3. 2-INCH CORE BARREL USED BELOW EL. 843.2'.
4. CONSTANT DRILLING MUD LOSS BETWEEN EL. 841.7'-826.7'. HOLE CAVED AFTER CORE RUN TO EL. 826.7' AND HOLE COLLAPSED; CORE BARREL LOST. ABANDON HOLE AND REDRILL APPROX. 4 FEET WEST. SET 4-INCH FLUSH JOINT CASING TO EL. 861.7'. HOLE STABILIZED WITH DRILLING MUD BELOW EL 861.7'. SET NX CASING TO EL. 826.5'. CONSTANT MUD LOSS BELOW EL. 826.5'.
5. VIBRATING WIRE PIEZOMETER INSTALLED IN COMPLETED BOREHOLE.

US Army Corps of Engineers
St. Paul District

Symbol	Description	Date	Appr.

Designed by: GSW | Dwn by: GSW | Ckd by: GAR | Date: 25 AUG 2008 | Rev. | Solicitation No.: | Drawing Number: | Authority: | File Name: | Plot Scale: | Plot Date:

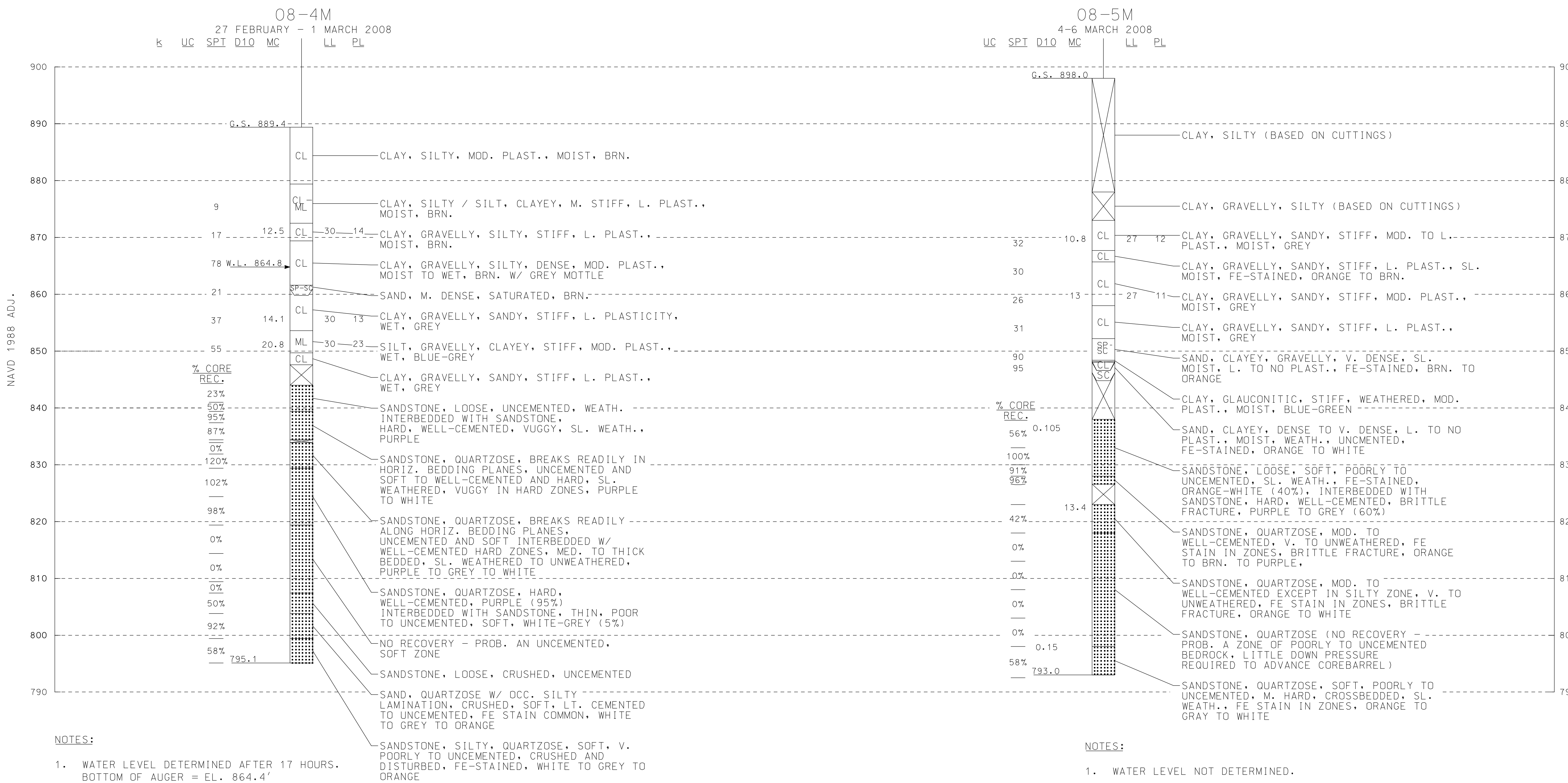
DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT
CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

Horizontal Coordinate System:
Vertical Coordinate System: NAVD 1988 Adj.

BLUE EARTH RIVER FEASIBILITY STUDY
RAPIDAN DAM
GEOTECHNICAL INVESTIGATION

Sheet reference number:

Sheet 2 of 3



NOTES:

1. WATER LEVEL DETERMINED AFTER 17 HOURS. BOTTOM OF AUGER = EL. 864.4′ BOTTOM OF HOLE = EL. 864.0′ SAMPLE TO EL. 859.4′
2. HOLE STABILIZED WITH HOLLOW STEM AUGER TO EL. 864.4′. HOLE STABILIZED WITH BENTONITE DRILLING MUD BETWEEN EL. 864.4′-844.0′. SET FLUSH JOINT NX CASING TO EL. 844.0′. HOLDE STABILIZED WITH DRILLING MUD BETWEEN EL. 844.0′-834.4′. RESET AUGER TO EL. 859.4′. RESET NX CASING TO EL. 834.8′. HOLDE SABILIZED WITH DRILLING MUD BELOW EL. 834.8′.
3. 2-INCH CORE BARREL USED BELOW EL. 844.0′.
4. AFTER STANDING OVERNIGHT BOREHOLE COLLAPSED BETWEEN EL. 848.1′-834.4′.
5. VIBRATING WIRE PIEZOMETERS INSTALLED IN COMPLETED BOREHOLE.

NOTES:

1. WATER LEVEL NOT DETERMINED.
2. HOLE STABILIZED WITH HOLLOW STEM AUGER TO EL. 846.5′. HOLE STABILIZED WITH BENTONITE DRILLING MUD BETWEEN EL. 846.5′-838.0′. SET NX FLUSH JOING CASING TO EL. 840.7′. HOLE STABILIZED WITH DRILLING MUD BELOW EL. 840.7′.
3. 2-INCH CORE BARREL USED BELOW EL. 838.0′.
4. SLOW DRILLING MUD LOSS BETWEEN EL. 818.0′-793.0′.
5. VIBRATING WIRE PIEZOMETER INSTALLED IN COMPLETED BOREHOLE.

US Army Corps of Engineers
St. Paul District

Symbol	Description	Date	Appr.

Designed by: GSW | Date: 25 AUG 2008 | Rev.
Dwn by: GSW | Ckd by: GAR | Solicitation No.:
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DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT
CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

Horizontal Coordinate System:
Vertical Coordinate System: NAVD 1988 Adj.

BLUE EARTH RIVER FEASIBILITY STUDY
RAPIDAN DAM GEOTECHNICAL INVESTIGATION

Sheet reference number:

Sheet 3 of 3

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MIDWEST TESTING LABORATORY

1555 North 42nd Street – Unit B / Grand Forks, ND 58203-0809
Phone (701) 772-2832 / Fax (701) 772-2633

REPORT OF: SOIL INDEX PROPERTIES

PROJECT: Rapidan Dam
Mankato, MN

DATE: March 14, 2008

REPORTED TO: Department of Geotechnical & Geology
ATTN: Grant Riddick (ED-D)
U.S. Army Corps of Engineers
St. Paul District
190 Fifth Street East
St. Paul, MN 55101-1638

COPIES:

PROJECT NO.: G3729

BORING NUMBER: 08-1M

DATE: 2-14-08

Sample No.	Depth (ft.)	Classification	In-Situ Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index
1	6.5 – 7.0	SM	23.0		Non-Plastic	
2	11.5 – 12.0	CH	48.9	65	26	39
3	17.0 – 17.5	CH	55.7	74	27	47
4	22.0 – 22.5	CH	57.4	67	30	37
5	27.0 – 27.5	CH	50.6	68	22	46

SIGNED ____________

MIDWEST TESTING LABORATORY

1555 North 42nd Street - Unit B. / Grand Forks, ND 58203
Phone (701) 772-2832 / Fax (701) 772-2633

REPORT OF: SOIL GRAIN SIZE DISTRIBUTION

PROJECT: Rapidan Dam
Mankato, MN

DATE: March 14, 2008

REPORTED TO: Department of Geotechnical & Geology
ATTN: Grant Riddick (ED-D)
U.S. Army Corps of Engineers
St. Paul District
190 Fifth Street East
St. Paul , MN 55101-1638

COPIES:

PROJECT NO.: G3729

BORING NO.:	08-1M
DATE SAMPLED:	2/14/2008
SAMPLE NO.:	1
DEPTH (ft.):	6.5 - 7.0

MECHANICAL ANALYSIS (ASTM D 422)(see attached curve):

% Passing	
1 1/2" (38mm)	100
1" (25.4)	100
3/4" (19.05)	100
3/8" (9.525)	100
#4 (4.75)	99
10 (2.0)	99
20 (0.85)	99
40 (0.425)	98
60 (0.25)	81
100 (0.15)	55
200 (0.075)	30.9

PARTICLE SIZE DISTRIBUTION

Gravel (%)	
Coarse (plus 3")	0
Fine (3" - #4)	1
Sand (%)	
Coarse (#4 - #10)	0
Medium (#10 - #40)	1
Fine (#40 - #200)	67
Fines (%)	30.9

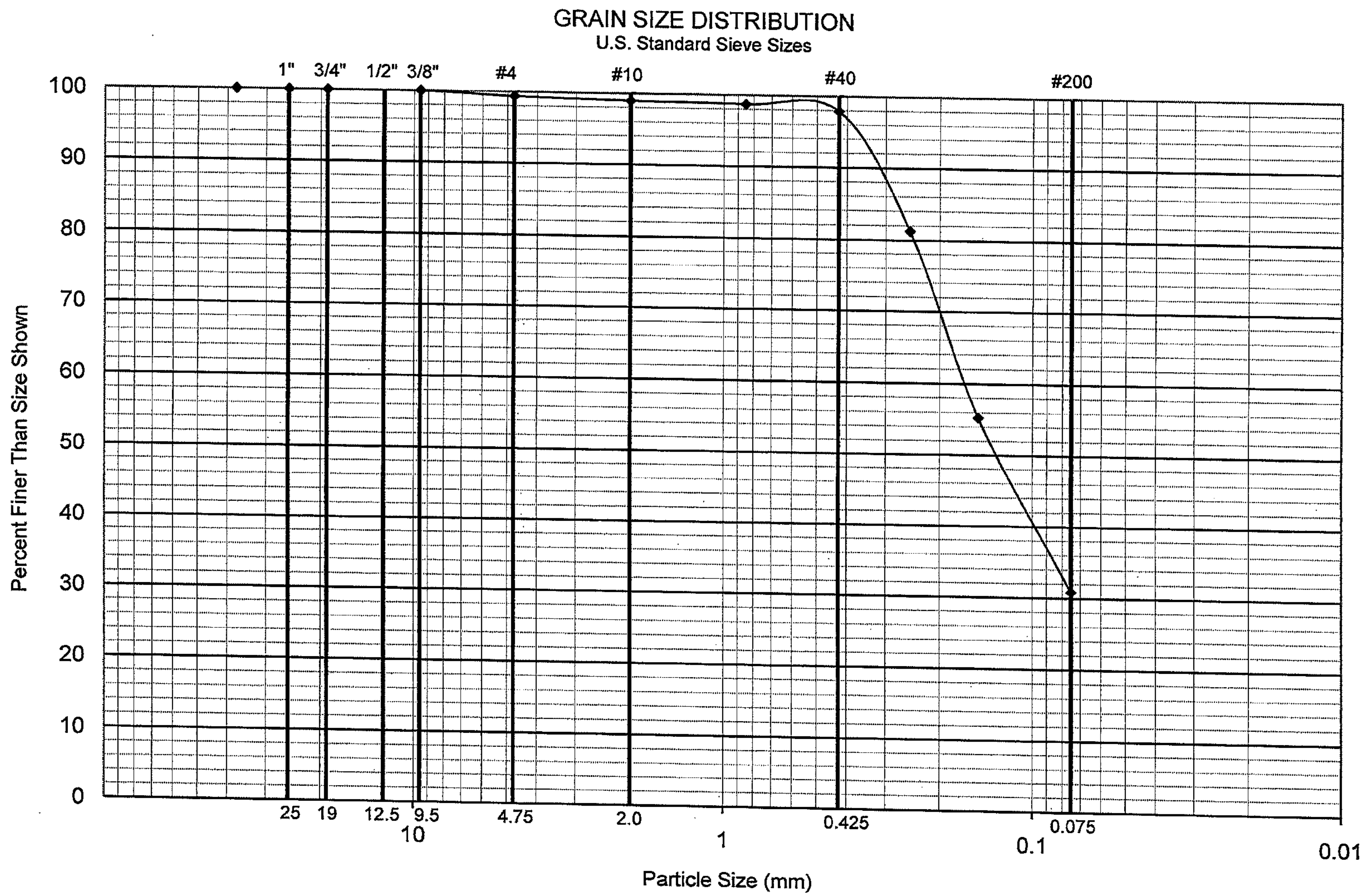
REMARKS:

Signed [signature]

MIDWEST TESTING LABORATORY

Project: Rapidan Dam
Boring No.: 08-1M
Sample No.: 1
Depth (ft.): 6.5 - 7.0

Project No.: G3729
Reported To: U.S. Army Corps of Engineers



√

MIDWEST TESTING LABORATORY

1555 North 42nd Street – Unit B / Grand Forks, ND 58203-0809
Phone (701) 772-2832 / Fax (701) 772-2633

REPORT OF: SOIL INDEX PROPERTIES

PROJECT: Rapidan Dam
Mankato, MN

DATE: March 14, 2008

REPORTED TO: Department of Geotechnical & Geology
ATTN: Grant Riddick (ED-D)
U.S. Army Corps of Engineers
St. Paul District
190 Fifth Street East
St. Paul, MN 55101-1638

COPIES:

PROJECT NO.: G3729

BORING NUMBER: 08-2M

DATE: 2-15-08

Sample No.	Depth (ft.)	Classification	In-Situ Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index
5	18.5 – 19.0	CL	18.3	28	17	11
6	23.5 – 24.0	ML	16.2	30	23	7

SIGNED_______________

MIDWEST TESTING LABORATORY

1555 North 42nd Street – Unit B / Grand Forks, ND 58203-0809
Phone (701) 772-2832 / Fax (701) 772-2633

REPORT OF: SOIL INDEX PROPERTIES

PROJECT: Rapidan Dam
Mankato, MN

DATE: March 14, 2008

REPORTED TO: Department of Geotechnical & Geology
ATTN: Grant Riddick (ED-D)
U.S. Army Corps of Engineers
St. Paul District
190 Fifth Street East
St. Paul, MN 55101-1638

COPIES:

PROJECT NO.: G3729

BORING NUMBER: 08-3M

DATE: 2-21-08 & 2-22-08

Sample No.	Depth (ft.)	Classification	In-Situ Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index
3	12.0 – 12.5	CL	23.3	44	22	22
7	21.0 – 21.5	CL	21.2	38	19	19
9	30.5 – 31.0	CL – ML	20.1	21	15	6

SIGNED ________________

MIDWEST TESTING LABORATORY

1555 North 42nd Street - Unit B. / Grand Forks, ND 58203
Phone (701) 772-2832 / Fax (701) 772-2633

REPORT OF: SOIL GRAIN SIZE DISTRIBUTION

PROJECT:	Rapidan Dam Mankato, MN	DATE:	March 14, 2008
REPORTED TO:	Department of Geotechnical & Geology ATTN: Grant Riddick (ED-D) U.S. Army Corps of Engineers St. Paul District 190 Fifth Street East St. Paul , MN 55101-1638	COPIES:	
PROJECT NO.:	G3729		

BORING NO.:	08-3M
DATE SAMPLED:	2/21/2008
SAMPLE NO.:	4
DEPTH (ft.):	16.0 - 16.5

MECHANICAL ANALYSIS (ASTM D 422)(see attached curve):

% Passing

1 1/2" (38mm)	100
1" (25.4)	100
3/4" (19.05)	100
3/8" (9.525)	91
#4 (4.75)	85
10 (2.0)	65
20 (0.85)	29
40 (0.425)	18
60 (0.25)	14
100 (0.15)	10
200 (0.075)	7.3

PARTICLE SIZE DISTRIBUTION

Gravel (%)	
Coarse (plus 3")	0
Fine (3" - #4)	15
Sand (%)	
Coarse (#4 - #10)	20
Medium (#10 - #40)	47
Fine (#40 - #200)	11
Fines (%)	7.3

REMARKS:

Signed

MIDWEST TESTING LABORATORY

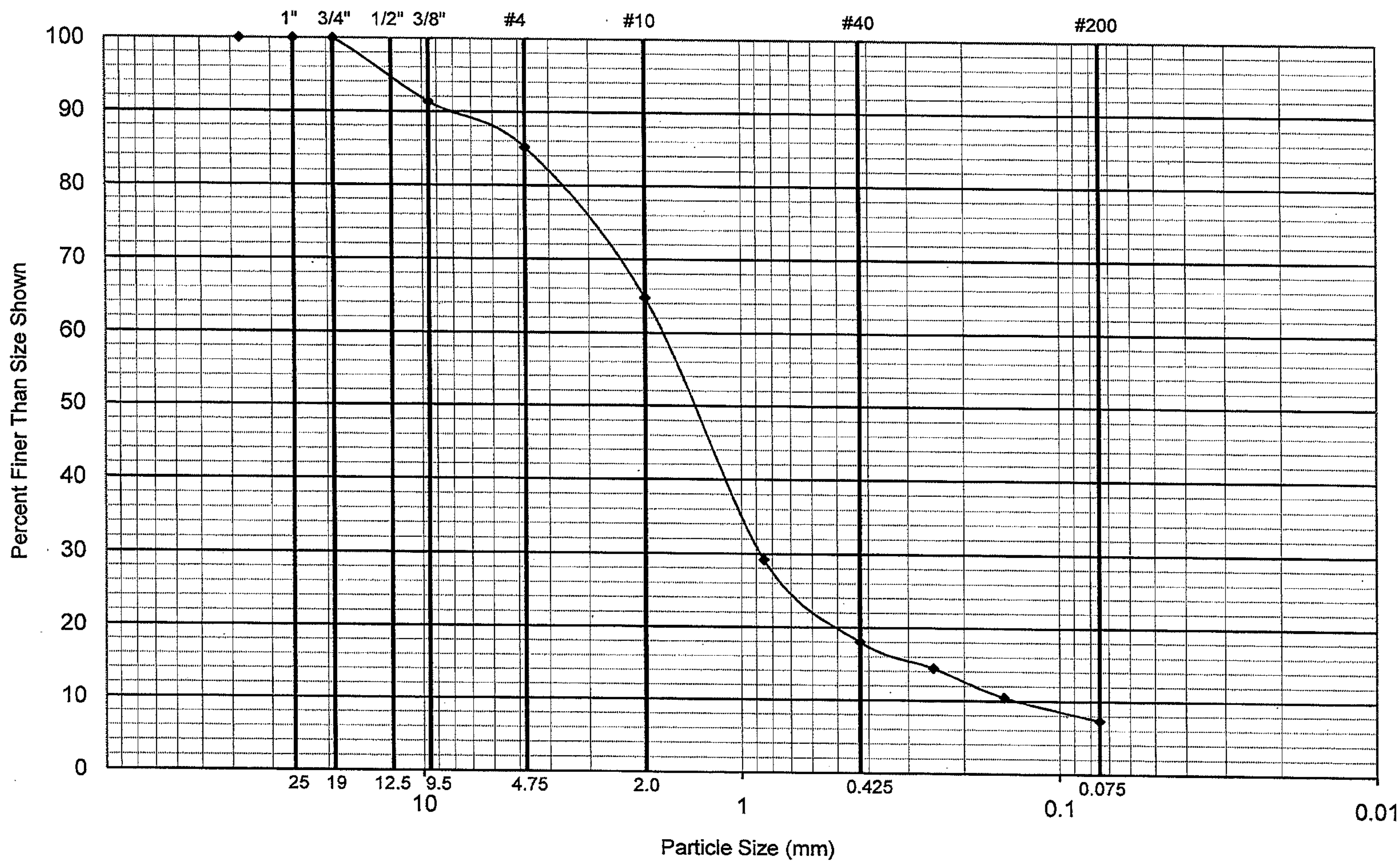
Project: Rapidan Dam
Boring No.: 08-3M
Sample No.: 4

Depth (ft.): 16.0 - 16.5

Project No.: G3729

Reported To: U.S. Army Corps of Engineers

GRAIN SIZE DISTRIBUTION
U.S. Standard Sieve Sizes



MIDWEST TESTING LABORATORY

1555 North 42nd Street – Unit B / Grand Forks, ND 58203-0809
Phone (701) 772-2832 / Fax (701) 772-2633

REPORT OF: SOIL INDEX PROPERTIES

PROJECT: Rapidan Dam
Mankato, MN

DATE: March 14, 2008

REPORTED TO: Department of Geotechnical & Geology
ATTN: Grant Riddick (ED-D)
U.S. Army Corps of Engineers
St. Paul District
190 Fifth Street East
St. Paul, MN 55101-1638

COPIES:

PROJECT NO.: G3729

BORING NUMBER: 08-4M

DATE: 2-27-08

Sample No.	Depth (ft.)	Classification	In-Situ Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index
2	18.0 – 18.5	CL	12.5	30	14	16
5	33.5 – 34.0	CL	14.1	30	13	17
6	38.0 – 38.5	ML	20.8	30	23	7

SIGNED

MIDWEST TESTING LABORATORY
1555 North 42nd Street – Unit B / Grand Forks, ND 58203-0809
Phone (701) 772-2832 / Fax (701) 772-2633

REPORT OF: SOIL INDEX PROPERTIES

PROJECT: Rapidan Dam
Mankato, MN

DATE: March 14, 2008

REPORTED TO: Department of Geotechnical & Geology
ATTN: Grant Riddick (ED-D)
U.S. Army Corps of Engineers
St. Paul District
190 Fifth Street East
St. Paul, MN 55101-1638

COPIES:

PROJECT NO.: G3729

BORING NUMBER: 08-5M

DATE: 3-4-08

Sample No.	Depth (ft.)	Classification	In-Situ Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index
1	28.0 – 28.5	CL	10.8	27	12	15
3	38.0 – 38.5	CL	13.0	27	11	16

SIGNED

MIDWEST TESTING LABORATORY

1555 North 42nd Street - Unit B. / Grand Forks, ND 58203
Phone (701) 772-2832 / Fax (701) 772-2633

REPORT OF: SOIL GRAIN SIZE DISTRIBUTION

PROJECT: Rapidan Dam
Mankato, MN

DATE: March 14, 2008

REPORTED TO: Department of Geotechnical & Geology
ATTN: Grant Riddick (ED-D)
U.S. Army Corps of Engineers
St. Paul District
190 Fifth Street East
St. Paul , MN 55101-1638

COPIES:

PROJECT NO.: G3729

BORING NO.:	08-5M
DATE SAMPLED:	3/4/2008
SAMPLE NO.:	6
DEPTH (ft.):	50.5 - 51.0

MECHANICAL ANALYSIS (ASTM D 422)(see attached curve):

% Passing	
1 1/2" (38mm)	100
1" (25.4)	100
3/4" (19.05)	100
3/8" (9.525)	99
#4 (4.75)	99
10 (2.0)	99
20 (0.85)	98
40 (0.425)	93
60 (0.25)	84
100 (0.15)	66
200 (0.075)	38.1

PARTICLE SIZE DISTRIBUTION

Gravel (%)	
Coarse (plus 3")	0
Fine (3" - #4)	1
Sand (%)	
Coarse (#4 - #10)	0
Medium (#10 - #40)	5
Fine (#40 - #200)	55
Fines (%)	38.1

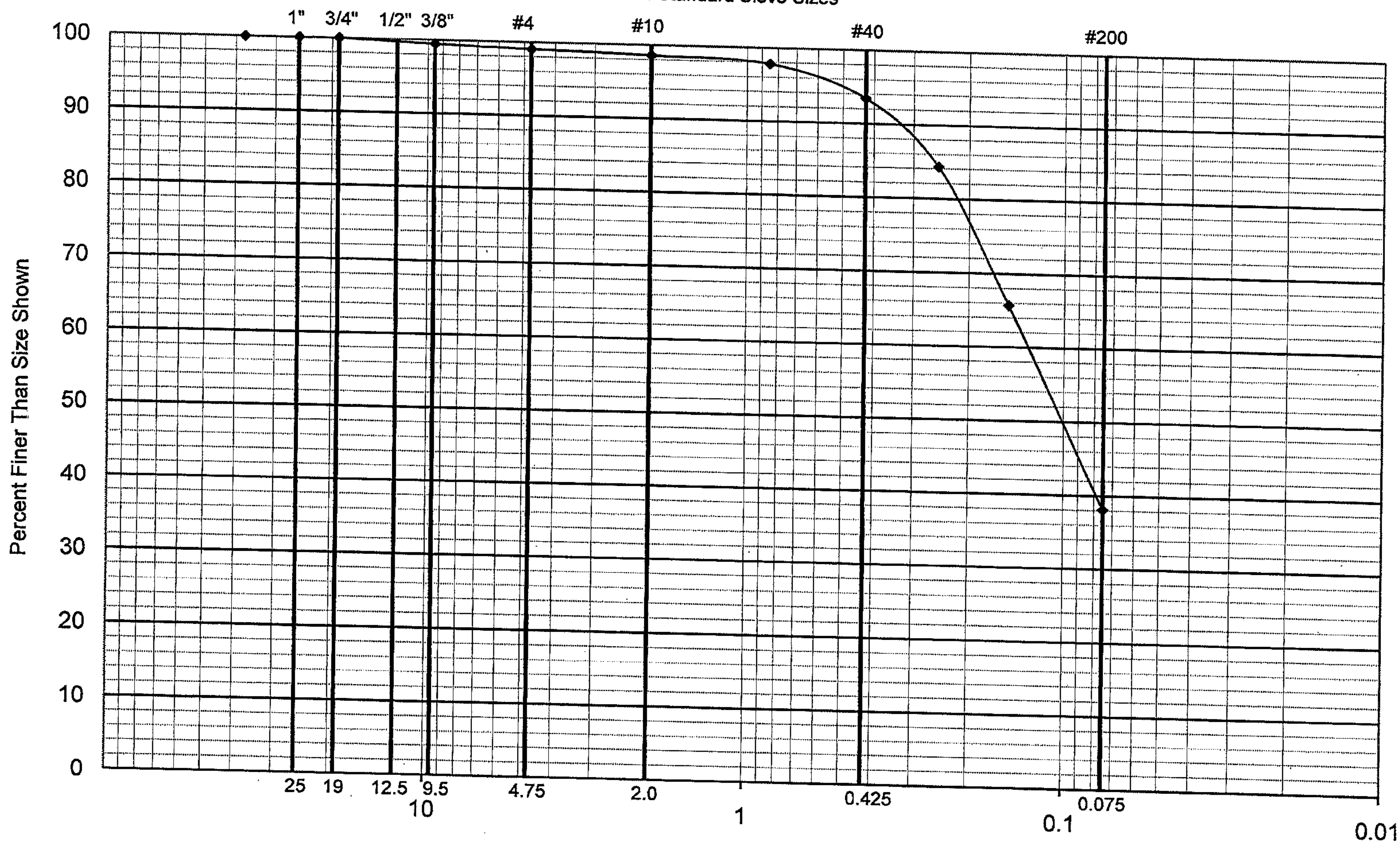
REMARKS:

Signed

MIDWEST TESTING L.BORATORY

Project: Rapidan Dam
Boring No.: 08-5M
Sample No.: 6
Depth (ft.): 50.5 - 51.0
Project No.: G3729
Reported To: U.S. Army Corps of Engineers

GRAIN SIZE DISTRIBUTION
U.S. Standard Sieve Sizes

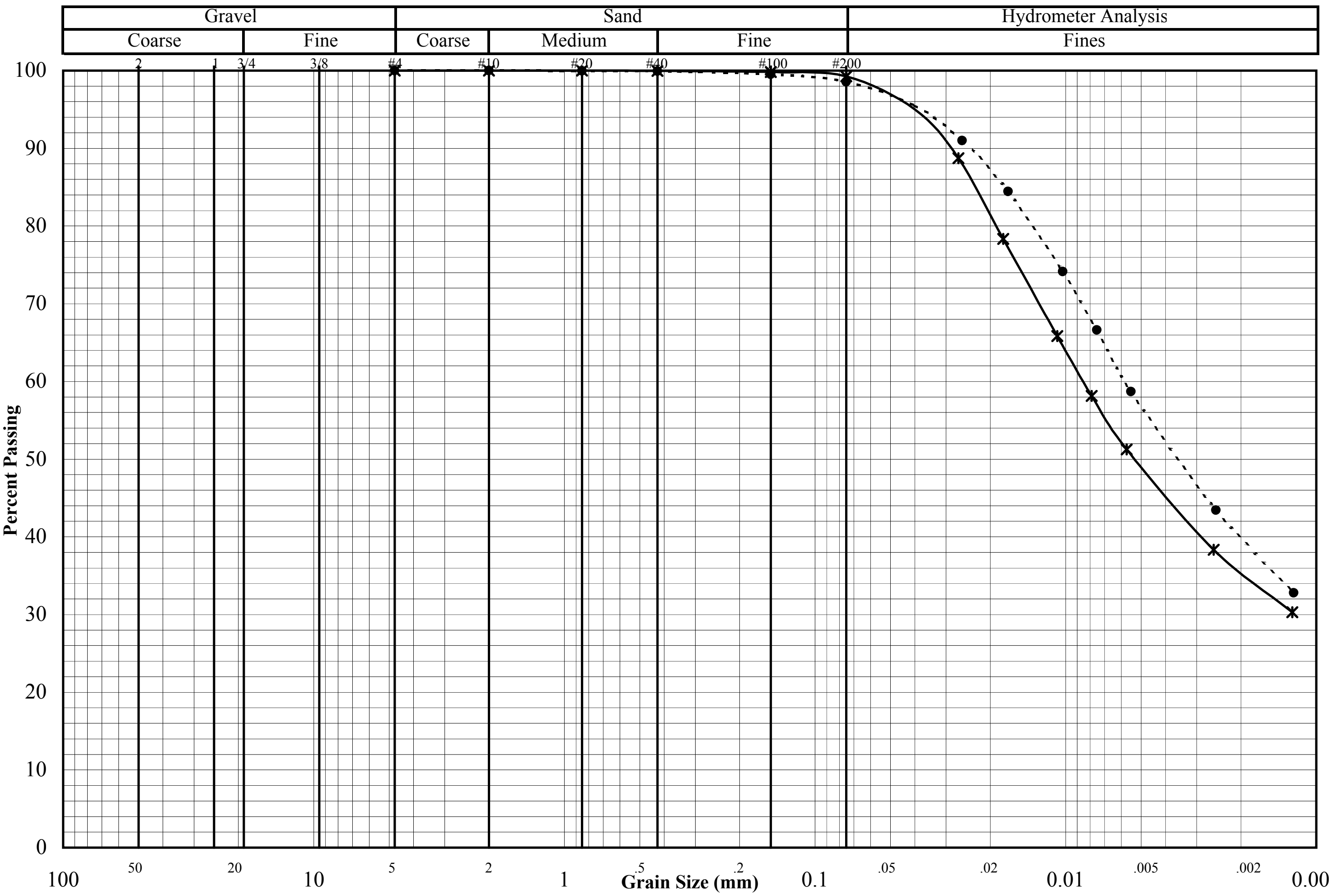


Grain Size Distribution ASTM D422

Job No. : **6471**

Project:	Rapidan Dam, Blue Earth River Feasibility Study - #'W912ES-08-T-0017	Test Date:	4/22/08
Reported To:	USACE -Geotech. & Geology Section	Report Date:	4/24/08

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	08-1 mu	2	21-22.3	TWT	Lean Clay (CL)
●	08-1 mu	1	14-15.4	TWT	Lean Clay (CL)
◇					



Other Tests

	*	●	◇
Liquid Limit			
Plastic Limit			
Plasticity Index			
Water Content			
Dry Density (pcf)			
Specific Gravity	2.7*	2.7*	
Porosity			
Organic Content			
pH			
Shrinkage Limit			
Penetrometer			
Qu (psf)			

(* = assumed)

Percent Passing

	*	●	◇
Mass (g)	165.3	138.5	
2"			
1.5"			
1"			
3/4"			
3/8"			
#4	100.0	100.0	
#10	100.0	100.0	
#20	100.0	100.0	
#40	100.0	99.9	
#100	99.8	99.5	
#200	99.2	98.5	

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

9301 Bryant Ave. South, Suite 107

SOIL ENGINEERING TESTING, INC.

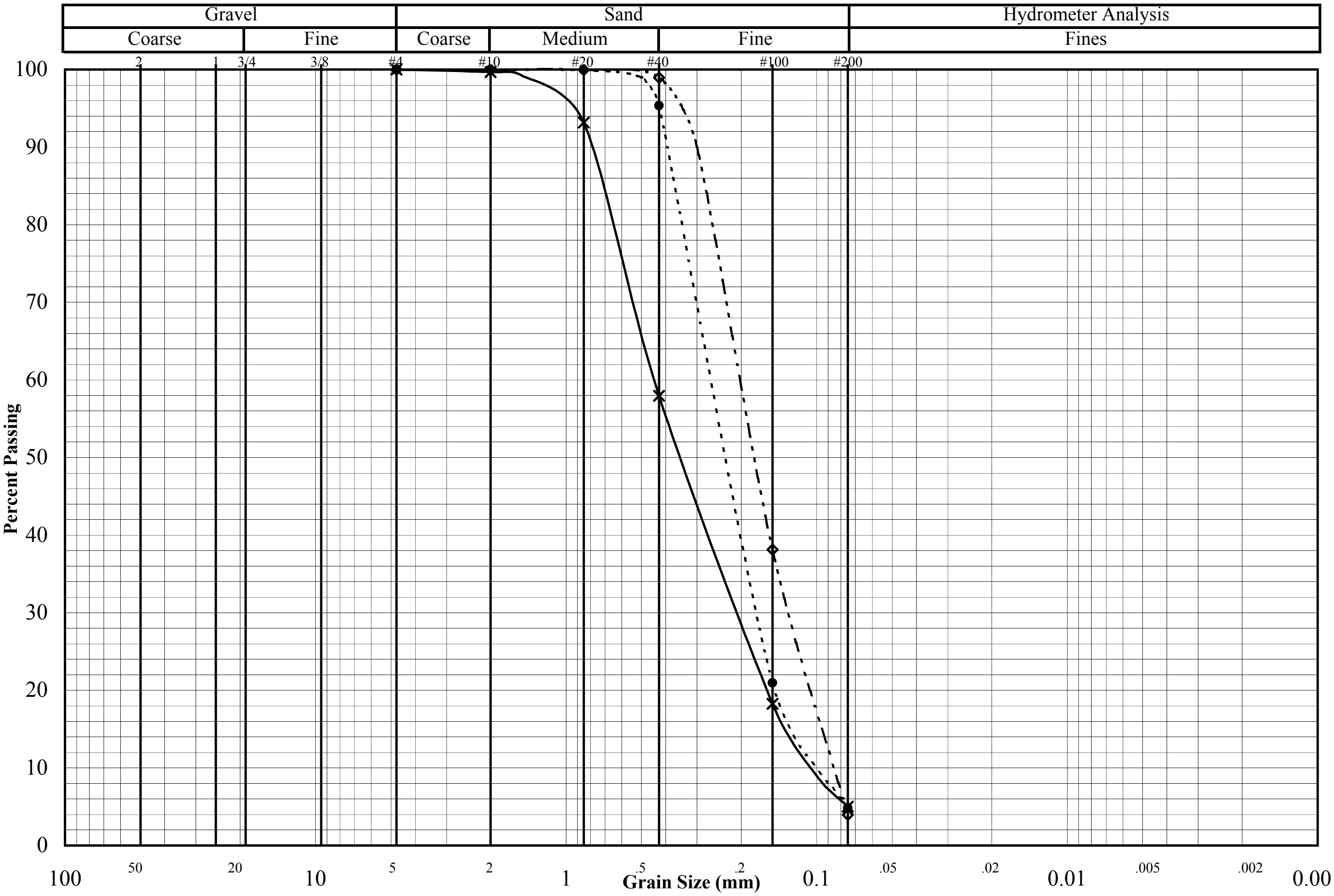
Bloomington, Minnesota 55420-3436

Grain Size Distribution ASTM D422

Job No. : **6471**

Project:	Rapidan Dam, Blue Earth River Feasibility Study - #'W912ES-08-T-0017	Test Date:	4/14/08
Reported To:	USACE -Geotech. & Geology Section	Report Date:	4/28/08

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	08-2m	8	44.2-44.4	Jar	Uncemented Sandstone, Sand w/silt, fine to medium grained (SP-SM)
●	08-2m	9	75.7-75.9	Jar	Uncemented Sandstone, Sand, fine grained (SP)
◇	08-2m	10	87.6-87.8	Jar	Uncemented Sandstone, Sand, fine grained (SP)



Other Tests

	*	●	◇
Liquid Limit			
Plastic Limit			
Plasticity Index			
Water Content	7.5	12.9	1.8
Dry Density (pcf)			
Specific Gravity			
Porosity			
Organic Content			
pH			
Shrinkage Limit			
Penetrometer			
Qu (psf)			

(* = assumed)

Percent Passing

	*	●	◇
Mass (g)	239.8	259.1	202.7
2"			
1.5"			
1"			
3/4"			
3/8"			
#4	100.0	100.0	100.0
#10	99.7	100.0	100.0
#20	93.2	99.9	100.0
#40	58.0	95.3	99.0
#100	18.3	21.0	38.1
#200	5.0	4.8	3.9

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

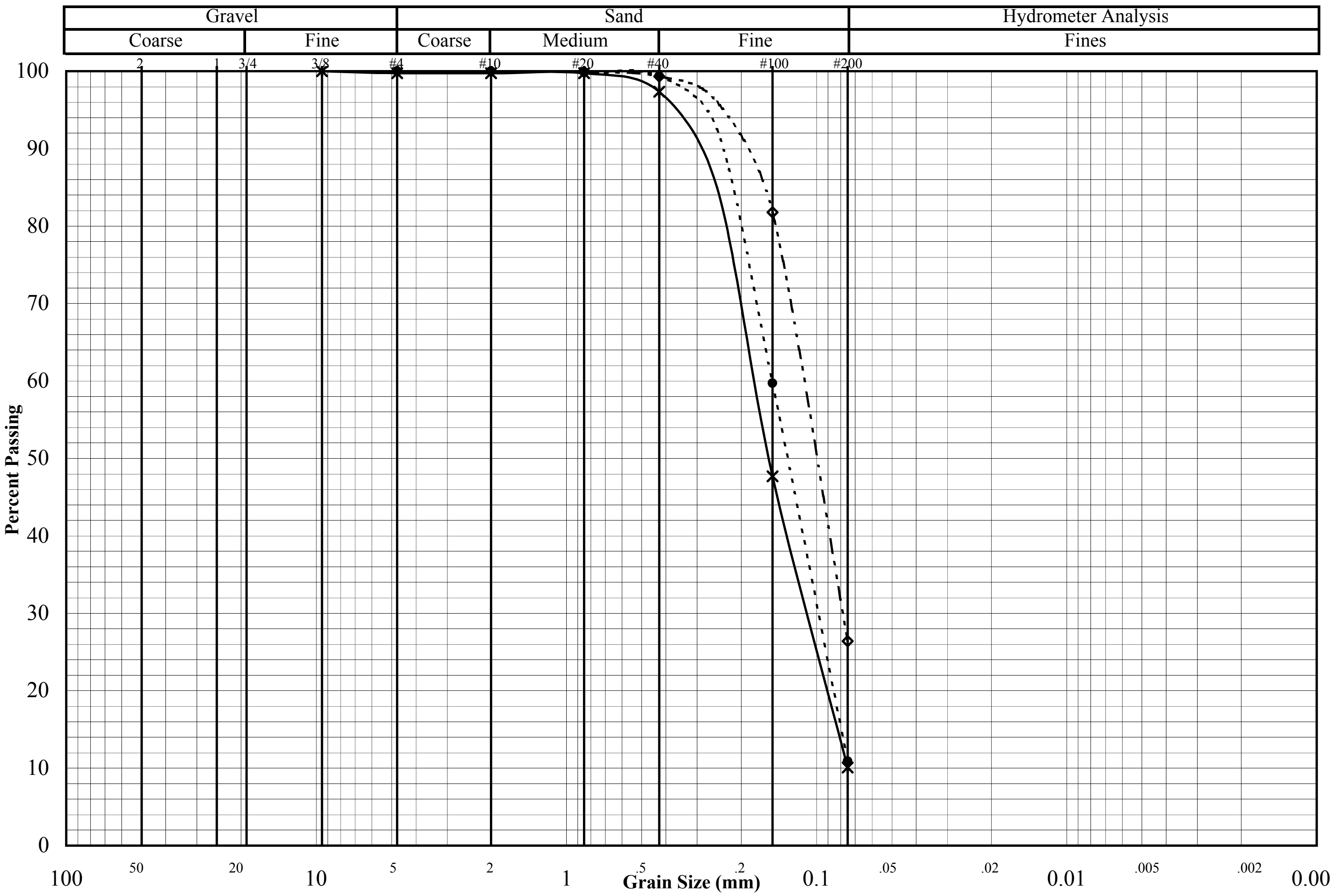
Sample 1 contained pieces of cemented sandstone that were retained on the #10 sieve.

9301 Bryant Ave. South, Suite 107 SOIL ENGINEERING TESTING, INC. Bloomington, Minnesota 55420-3436

Grain Size Distribution ASTM D422

Job No. :	6471		
Project:	Rapidan Dam, Blue Earth River Feasibility Study - #'W912ES-08-T-0017	Test Date:	4/14/08
Reported To:	USACE -Geotech. & Geology Section	Report Date:	4/29/08

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	08-2m	11	95.0-95.2	Jar	Uncemented Sandstone, Sand w/silt, fine grained (SP-SM)
●	08-2m	12	106.8-107.1	Jar	Uncemented Sandstone, Sand w/silt, fine grained (SP-SM)
◇	08-2m	13	111.9-112.1	Jar	Uncemented Sandstone, Silty Sand, fine grained (SM)



Other Tests

	*	●	◇
Liquid Limit			
Plastic Limit			
Plasticity Index			
Water Content	21.1	19.9	18.8
Dry Density (pcf)			
Specific Gravity			
Porosity			
Organic Content			
pH			
Shrinkage Limit			
Penetrometer			
Qu (psf)			

(* = assumed)

Percent Passing

	*	●	◇
Mass (g)	200.3	240.9	182.9
2"			
1.5"			
1"			
3/4"			
3/8"	100.0		
#4	99.8	100.0	100.0
#10	99.8	100.0	100.0
#20	99.8	99.9	99.9
#40	97.4	99.4	99.3
#100	47.7	59.7	81.8
#200	10.1	10.9	26.4

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

Sample 1 contained a piece of cemented sandstone that was retained on the #4 sieve.

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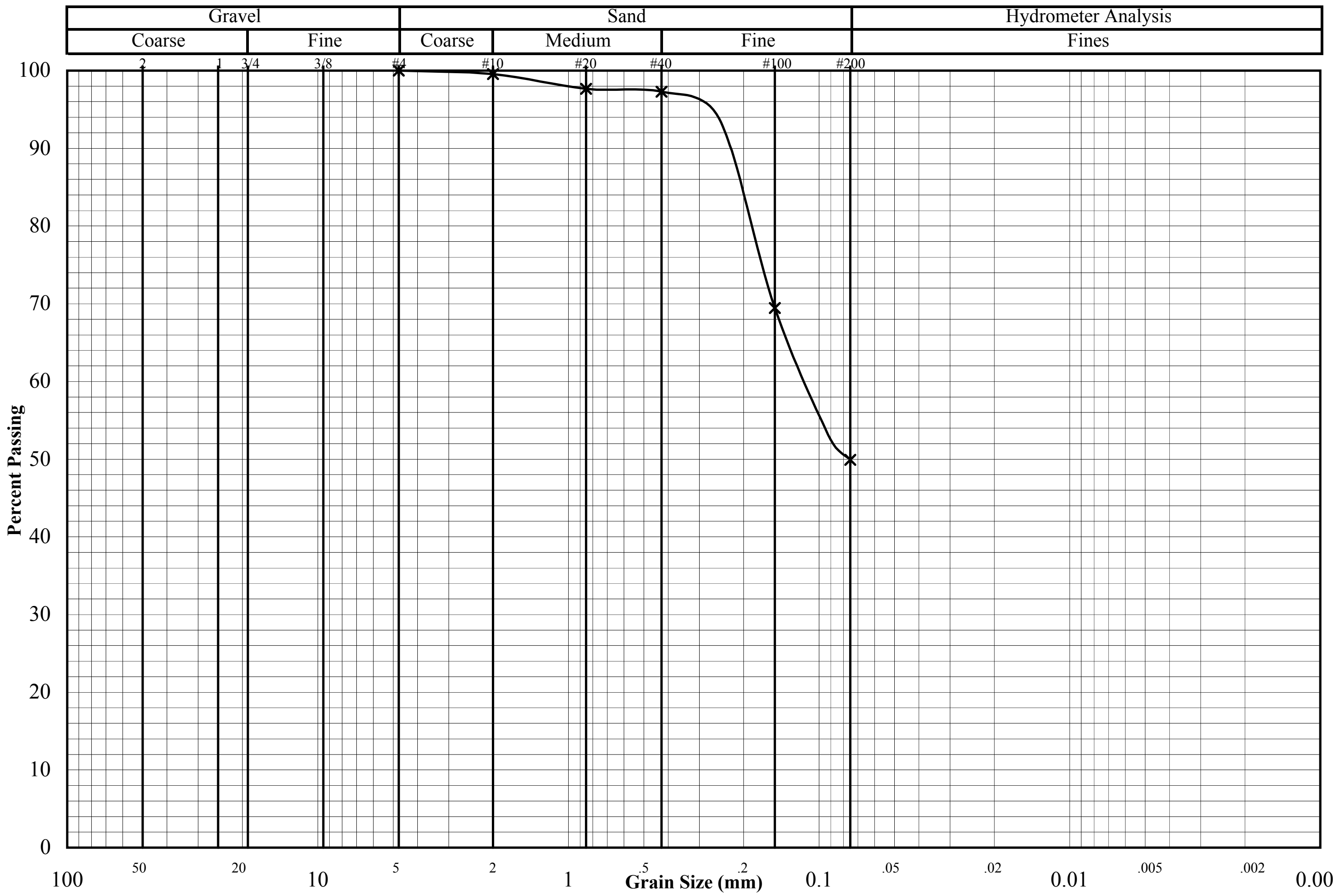
Grain Size Distribution ASTM D422

Project:	Rapidan Dam, Blue Earth River Feasibility Study - #'W912ES-08-T-0017	Job No. :	6471
		Test Date:	4/14/08
Reported To:	USACE -Geotech. & Geology Section	Report Date:	4/28/08

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	08-2m	14	118.8-119.0	Jar	Siltstone, Silty Sand (SM/ML)
●					
◇					



Other Tests	*	●	◇
Liquid Limit			
Plastic Limit			
Plasticity Index			
Water Content	20.6		
Dry Density (pcf)			
Specific Gravity			
Porosity			
Organic Content			
pH			
Shrinkage Limit			
Penetrometer			
Qu (psf)			

(* = assumed)

Percent Passing	*	●	◇
Mass (g)	114.6		
2"			
1.5"			
1"			
3/4"			
3/8"			
#4	100.0		
#10	99.6		
#20	97.6		
#40	97.3		
#100	69.5		
#200	49.9		

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

Pieces of cemented sandstone retained on the #10 sieve.

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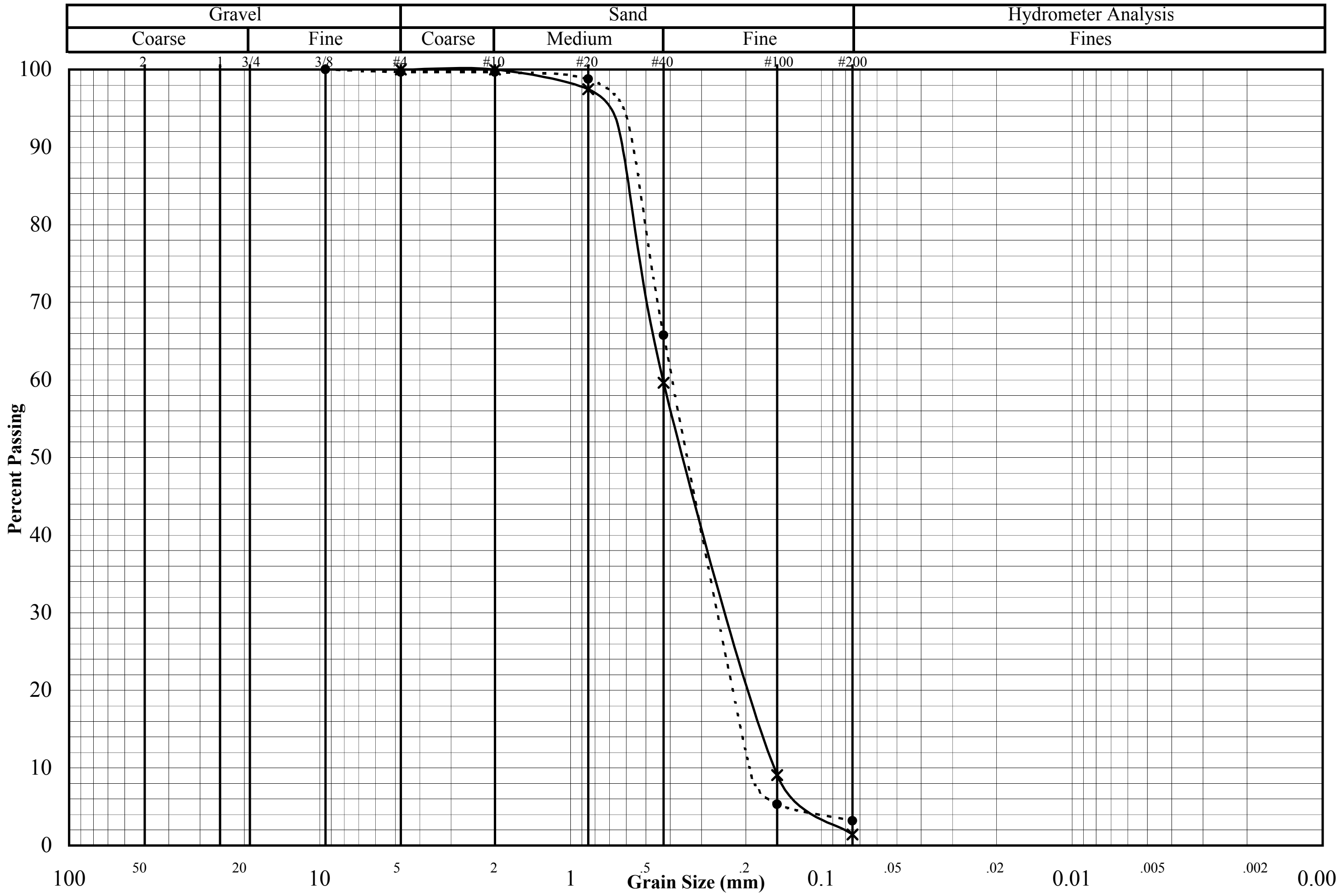
Grain Size Distribution ASTM D422

Job No. : 6471

Project:	Rapidan Dam, Blue Earth River Feasibility Study - #'W912ES-08-T-0017	Test Date:	4/14/08
Reported To:	USACE -Geotech. & Geology Section	Report Date:	4/28/08

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	08-3m	10	67.3-67.6	Jar	Uncemented Sandstone, Sand, fine to medium grained (SP)
●	08-3m	11	73.8-74.2	Jar	Uncemented Sandstone, Sand, fine to medium grained (SP)
◇					



Other Tests

	*	●	◇
Liquid Limit			
Plastic Limit			
Plasticity Index			
Water Content	20.6	17.6	
Dry Density (pcf)			
Specific Gravity			
Porosity			
Organic Content			
pH			
Shrinkage Limit			
Penetrometer			
Qu (psf)			

(* = assumed)

Percent Passing

	*	●	◇
Mass (g)	299.9	284.7	
2"			
1.5"			
1"			
3/4"			
3/8"		100.0	
#4	100.0	99.6	
#10	100.0	99.6	
#20	97.5	98.8	
#40	59.6	65.8	
#100	9.1	5.3	
#200	1.4	3.2	

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

Sample 2 contained a piece of cemented sandstone that was retained on the #4 sieve.

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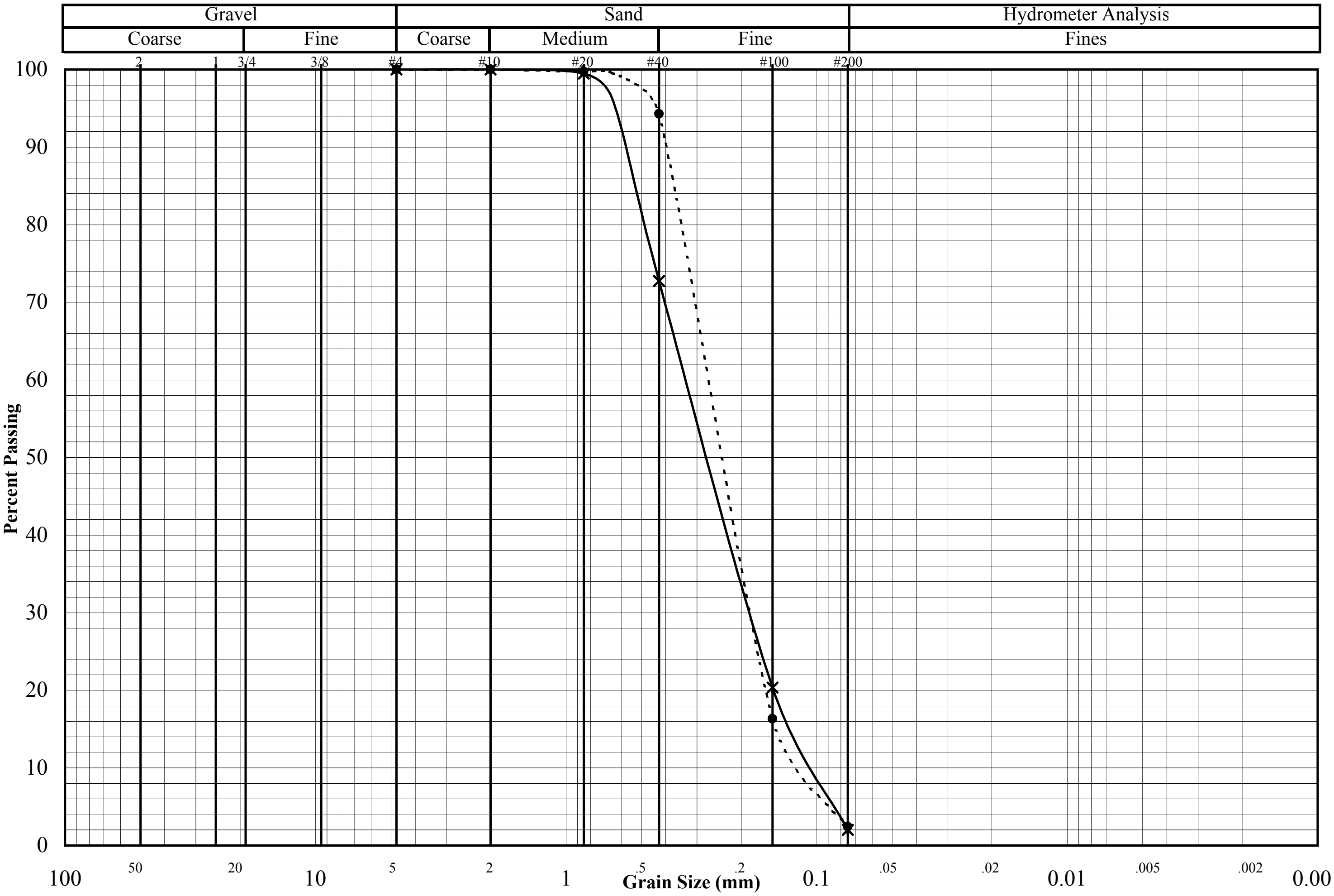
Bloomington, Minnesota 55420-3436

Grain Size Distribution ASTM D422

Project:	Rapidan Dam, Blue Earth River Feasibility Study - #'W912ES-08-T-0017	Job No. :	**6471**
		Test Date:	4/14/08
Reported To:	USACE -Geotech. & Geology Section	Report Date:	4/28/08

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	08-5m	7	61.4-61.7	Jar	Uncemented Sandstone, Sand, fine to medium grained (SP)
●	08-5m	8	100.0-100.3	Jar	Uncemented Sandstone, Sand, fine grained (SP)
◇					



Other Tests

	*	●	◇
Liquid Limit			
Plastic Limit			
Plasticity Index			
Water Content	8.0	11.3	
Dry Density (pcf)			
Specific Gravity			
Porosity			
Organic Content			
pH			
Shrinkage Limit			
Penetrometer			
Qu (psf)			

(* = assumed)

Percent Passing

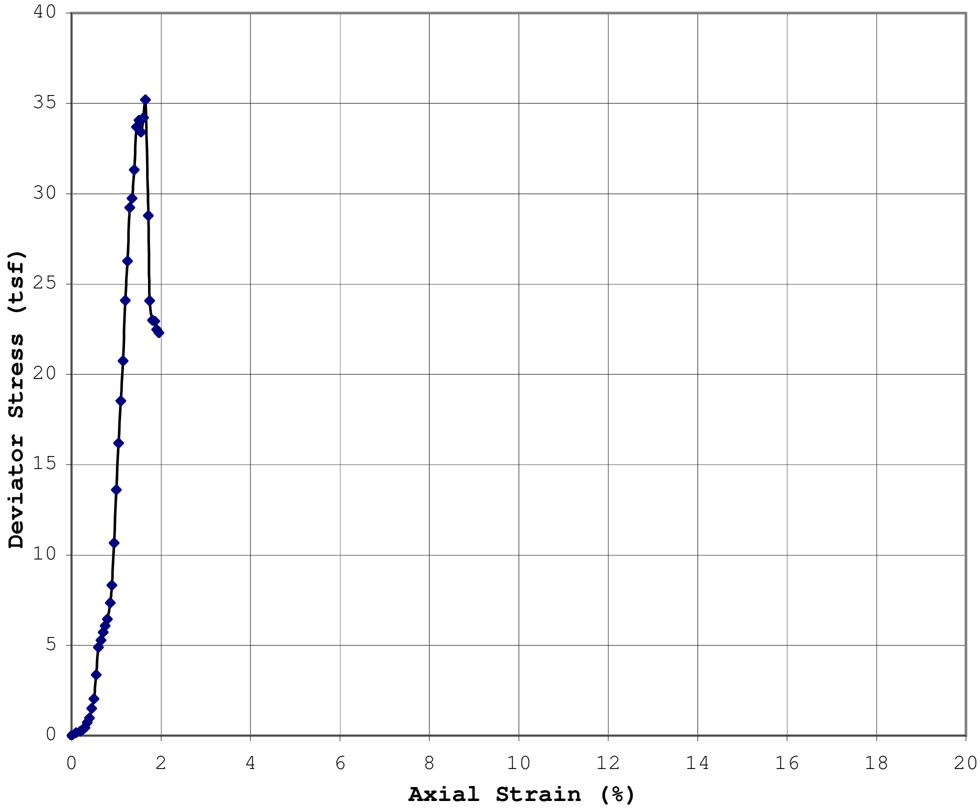
	*	●	◇
Mass (g)	114.4	259.1	
2"			
1.5"			
1"			
3/4"			
3/8"			
#4	100.0	100.0	
#10	100.0	100.0	
#20	99.5	99.7	
#40	72.7	94.3	
#100	20.4	16.3	
#200	2.0	2.4	

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

Unconfined Stress/Strain Curves **ASTM: D2166**

Project:	Rapidan Dam, Blue Earth River Feasibility Study - #'W912ES-08-T-0017	Job:	6471
Client:	USACE -Geotech. & Geology Section	Date:	4/23/08
Remarks:			



Boring: 08-2M Depth: 45.3 - 46.0
Sample #: 4

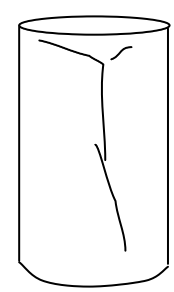
Soil Type: Jordan Sandstone

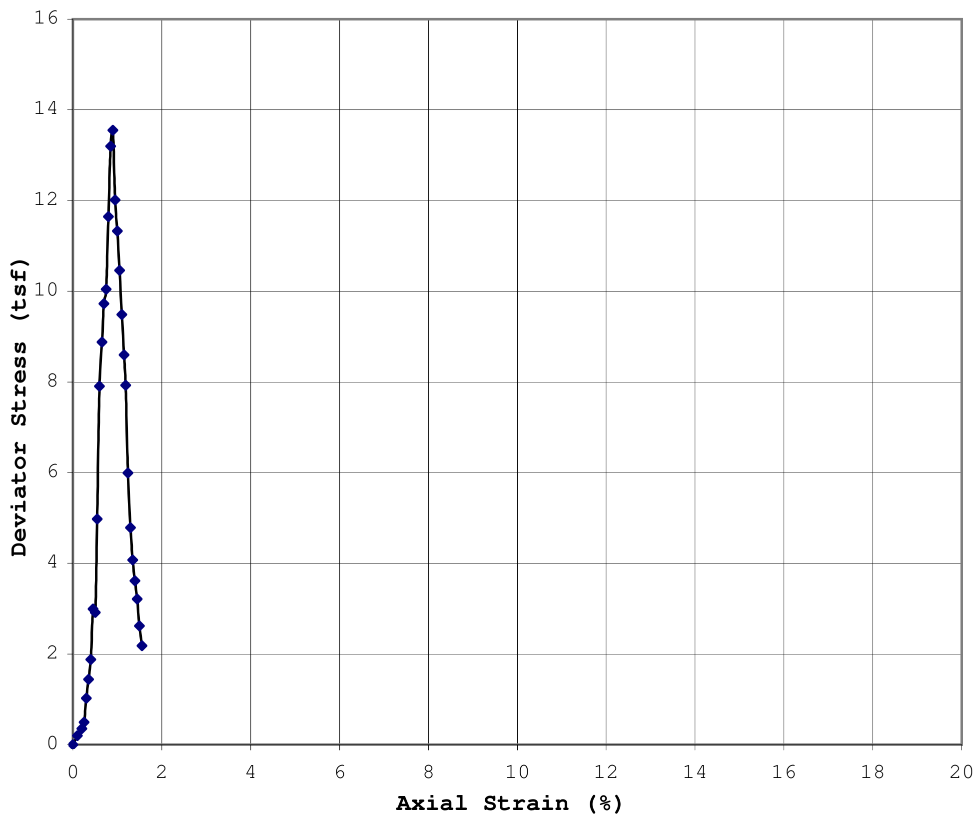
Strain Rate (in/min): 0.009
Sample Type: Core
Dia. (in) 2.00 Ht. (in) 3.99
Height to Diameter Ratio: 1.99

Unconfined Comp. Strength: 35.21 tsf
Strain at Failure (%): 1.66

W.C. (%): 8.2
Yd (pcf): 129.8

Sketch of Specimen After Failure





Boring: 08-2M Depth: 85.6 - 86.1
Sample #: 9

Soil Type: Jordon Sandstone

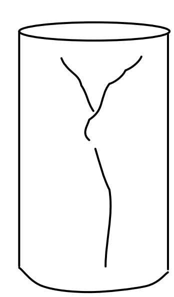
Strain Rate (in/min): 0.009
Sample Type: Core

Dia. (in): 1.91 Ht. (in): 4.00
Height to Diameter Ratio: 2.09

Unconfined Comp. Strength: 13.55 tsf
Strain at Failure (%): 0.90

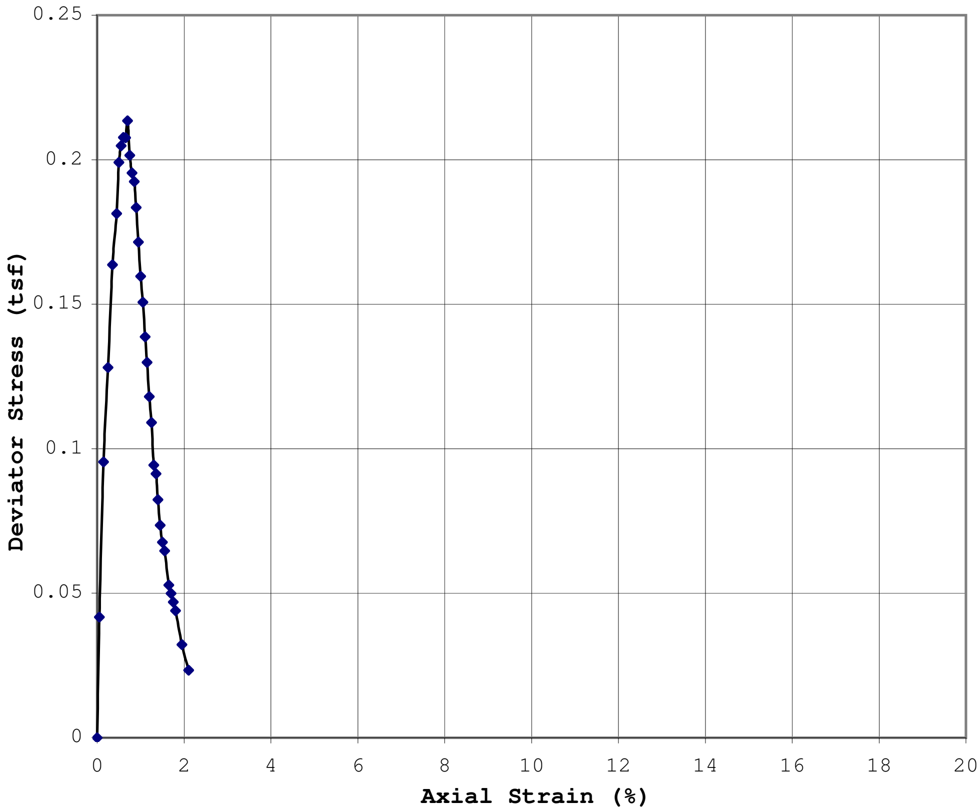
W.C. (%): 13.7
Yd (pcf): 116.7

Sketch of Specimen After Failure



Unconfined Stress/Strain Curves
ASTM: D2166

Project: Rapidan Dam, Blue Earth River Feasibility Study - #'W912ES-08-T-0017
Job: 6471
Client: USACE -Geotech. & Geology Section
Date: 4/23/08
Remarks:



Boring: 08-2M Depth: 101.7 - 102.2
Sample #: 10

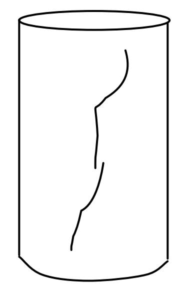
Soil Type: Jordan Limestone

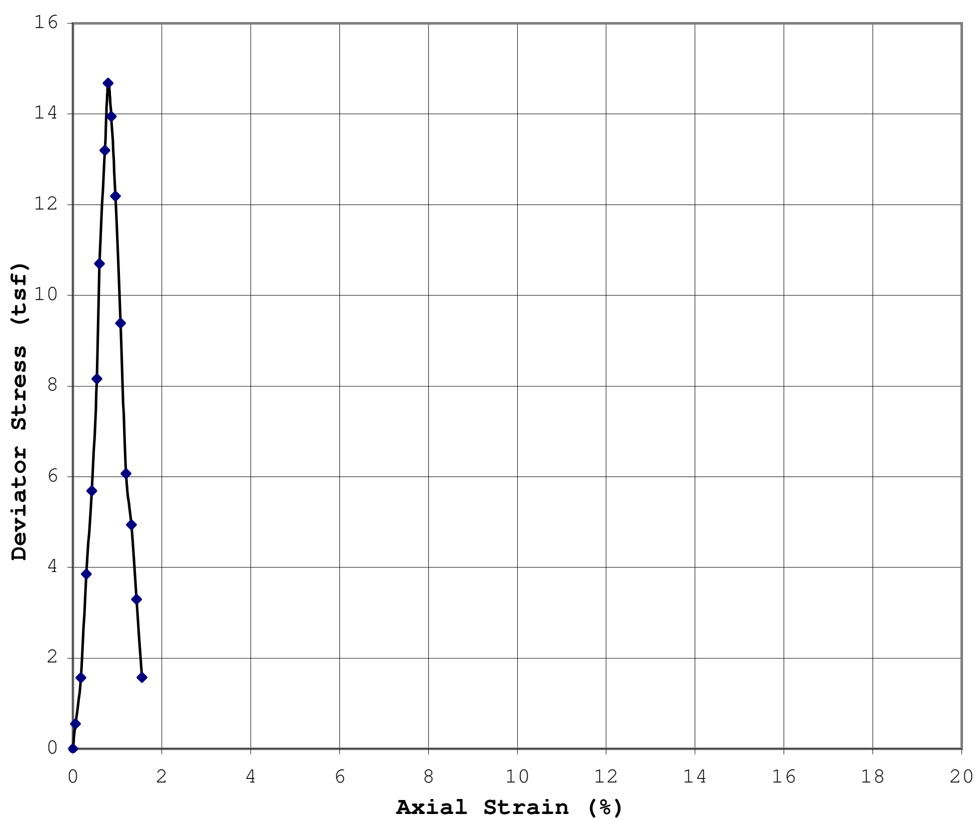
Strain Rate (in/min): 0.009
Sample Type: Core
Dia. (in) 2.03 Ht. (in) 3.99
Height to Diameter Ratio: 1.97

Unconfined Comp. Strength: 0.21 tsf
Strain at Failure (%): 0.70

W.C. (%): 17.6
Yd (pcf): 111.2

Sketch of Specimen After Failure





Boring: 08-5M Depth: 75.3 - 75.7
Sample #: 1

Soil Type: Jordan Sandstone

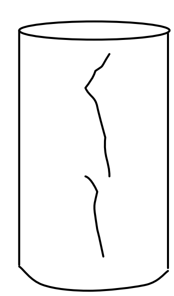
Strain Rate (in/min): 0.009
Sample Type: Core

Dia. (in): 1.92 Ht. (in): 3.35
Height to Diameter Ratio: 1.75

Unconfined Comp. Strength: 14.68 tsf
Strain at Failure (%): 0.79

W.C. (%): 13.4
Yd (pcf): 113.6

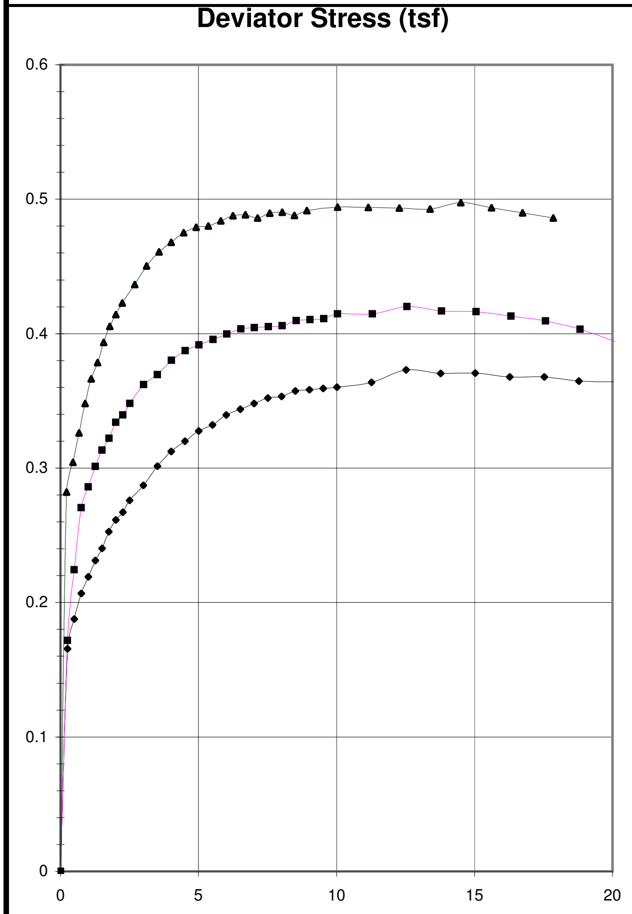
Sketch of Specimen After Failure

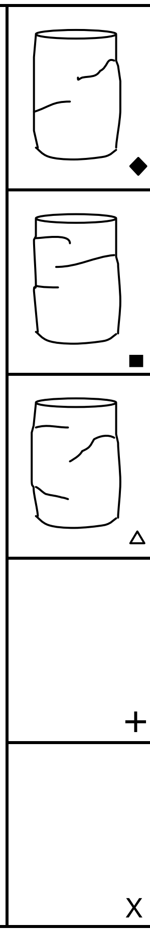


TRIAXIAL TEST ASTM: D 2850

Job No. **6471**
Date: **4/23/08**

Project: **Rapidam Dam - Blue Earth River Feasibility Study - #W912ES-08-T-001**
Boring #: **08-1MU** Sample #: **1** Type: **5T** Depth (ft): **14.0 - 15.4 (Bot.)**
Soil Type: **Organic Clay (OH)**





Failure Criterion: **Max. Deviator Stress**

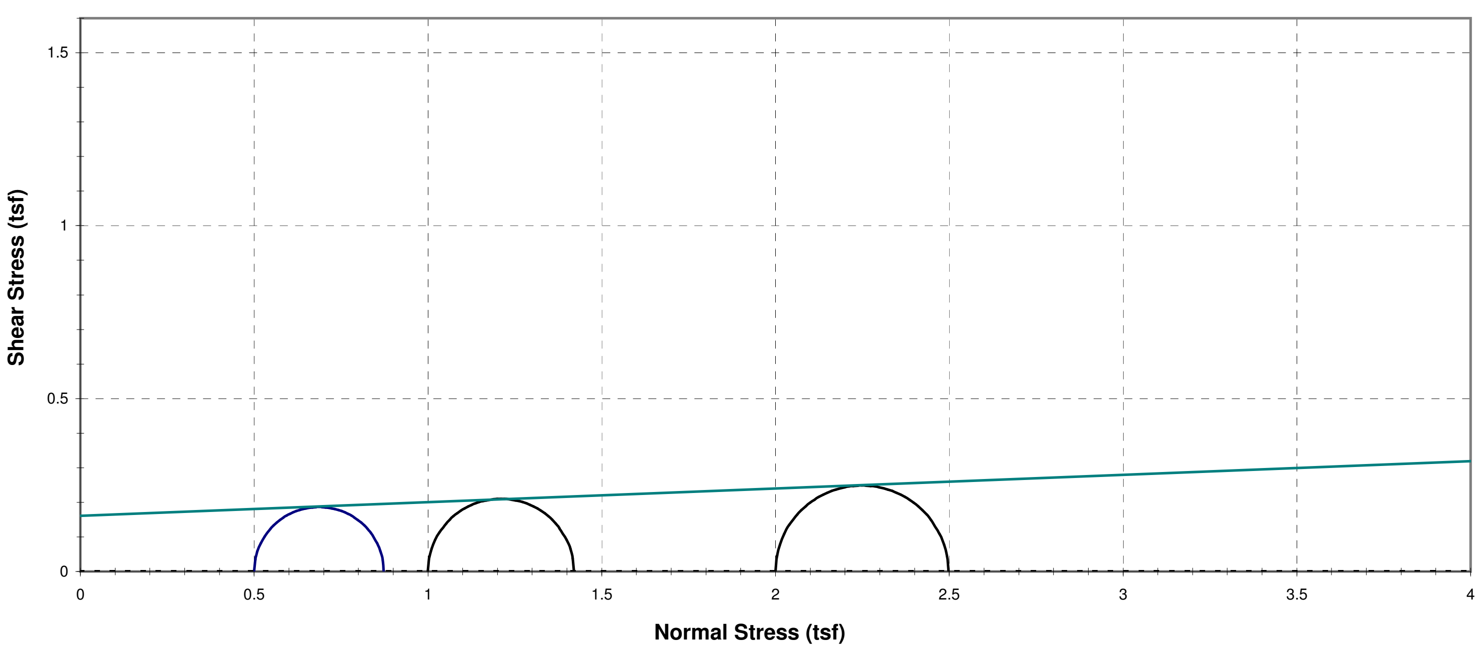
Angle of internal friction, ϕ_u = **2.3 °**
Apparent Cohesion, c_u = **0.16 (tsf)**

Test Date:	4/18/08	Liquid Limit:	86.0
Test Type:	U-U	Plastic Limit:	34.4
Strain Rate (in/min):	0.05	Plasticity Index:	51.6
Strain Rate (%/min):	1.252	Spec. Gravity (Assumed):	2.57

Before Consolidation	A	B	C	D	E
Diameter (in)	1.94	1.94	1.94		
Height (in)	3.99	3.98	4.48		
Water Content (%)	73.3	71.9	64.6		
Dry Density (pcf)	54.3	54.8	55.4		
Void Ratio	1.95	1.93	1.89		
After Consolidation					
Diameter (in)					
Height (in)					
Water Content (%)					
Dry Density (pcf)					
Void Ratio					
Back Pressure (tsf)					
Minor Principal Stress (tsf)	0.50	1.00	2.00		
Max. Deviator Stress (tsf)	0.37	0.42	0.50		
Ultimate Deviator Stress (tsf)	0.36	0.39	0.49		
Deviator Stress at Failure (tsf)	0.37	0.42	0.50		
Max. Pore Pressure Buildup (tsf)	--------	--------	--------		
Pore Pressure Parameter "B"	--------	--------	--------		
Pct. Axial Strain at Failure	12.5	12.6	14.5		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Specimens trimmed to given sizes; Allowed to adjust under applied confining pressures for about 10 minutes.



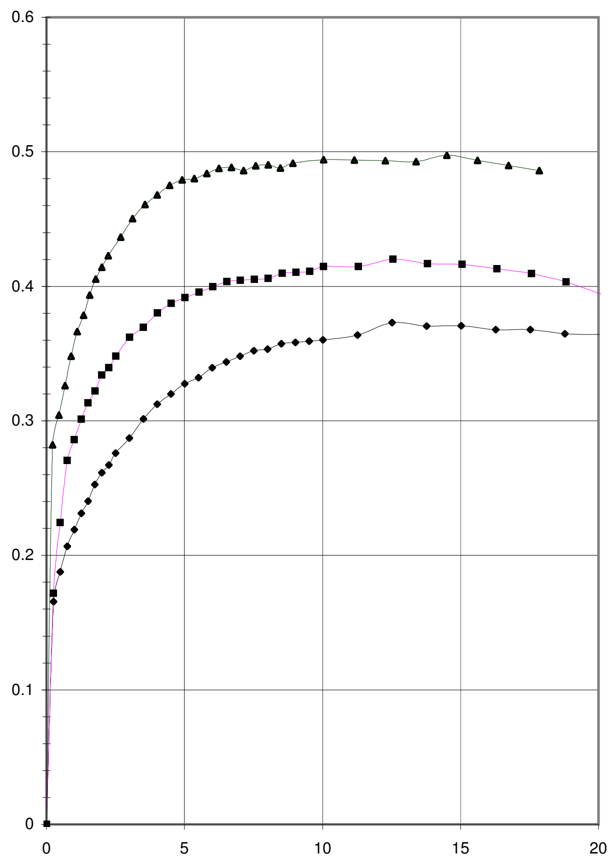
Total ϕ_u: 2.3 ° c_u= 0.16 (tsf)

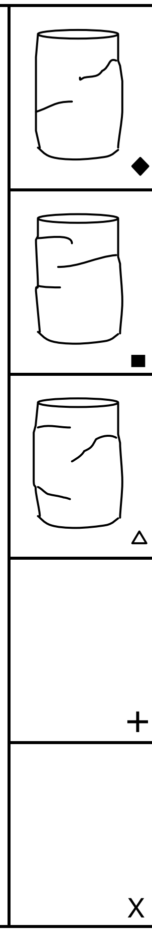
Job No. 6471
Date: 4/23/08

TRIAXIAL TEST ASTM: D 2850

Project: **Rapidam Dam - Blue Earth River Feasibility Study - #W912ES-08-T-001**
Boring #: **08-1MU** Sample #: **1** Type: **5T** Depth (ft): **14.0 - 15.4 (Bot.)**
Soil Type: **Organic Clay (OH)**

Deviator Stress (tsf)





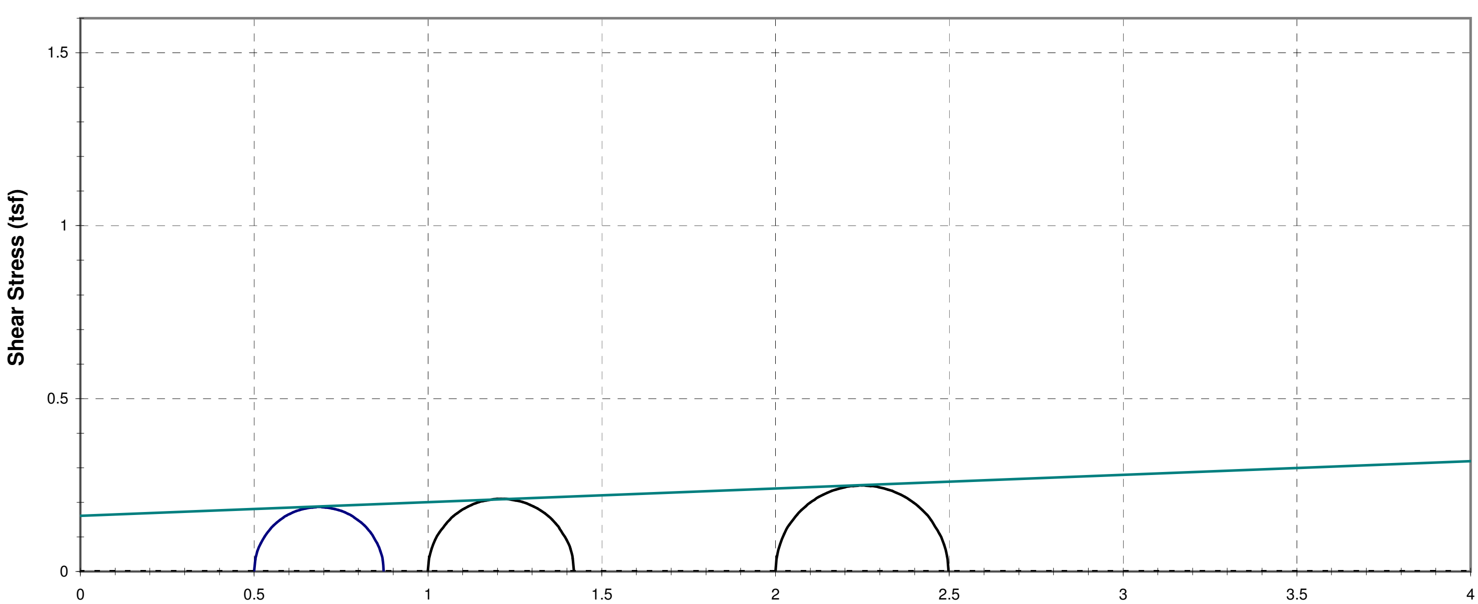
Failure Criterion: **Max. Deviator Stress**
Angle of internal friction, ϕ_u = **2.3 °**
Apparent Cohesion, c_u = **0.16 (tsf)**

Test Date:	4/18/08	Liquid Limit:	86.0
Test Type:	U-U	Plastic Limit:	34.4
Strain Rate (in/min):	0.05	Plasticity Index:	51.6
Strain Rate (%/min):	1.252	Spec. Gravity (Assumed):	2.57

Before Consolidation	A	B	C	D	E
Diameter (in)	1.94	1.94	1.94		
Height (in)	3.99	3.98	4.48		
Water Content (%)	73.3	71.9	64.6		
Dry Density (pcf)	54.3	54.8	55.4		
Void Ratio	1.95	1.93	1.89		
After Consolidation					
Diameter (in)					
Height (in)					
Water Content (%)					
Dry Density (pcf)					
Void Ratio					
Back Pressure (tsf)					
Minor Principal Stress (tsf)	0.50	1.00	2.00		
Max. Deviator Stress (tsf)	0.37	0.42	0.50		
Ultimate Deviator Stress (tsf)	0.36	0.39	0.49		
Deviator Stress at Failure (tsf)	0.37	0.42	0.50		
Max. Pore Pressure Buildup (tsf)	--------	--------	--------		
Pore Pressure Parameter "B"	--------	--------	--------		
Pct. Axial Strain at Failure	12.5	12.6	14.5		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Specimens trimmed to given sizes; Allowed to adjust under applied confining pressures for about 10 minutes.



Total ϕ_u: 2.3 ° c_u= 0.16 (tsf)

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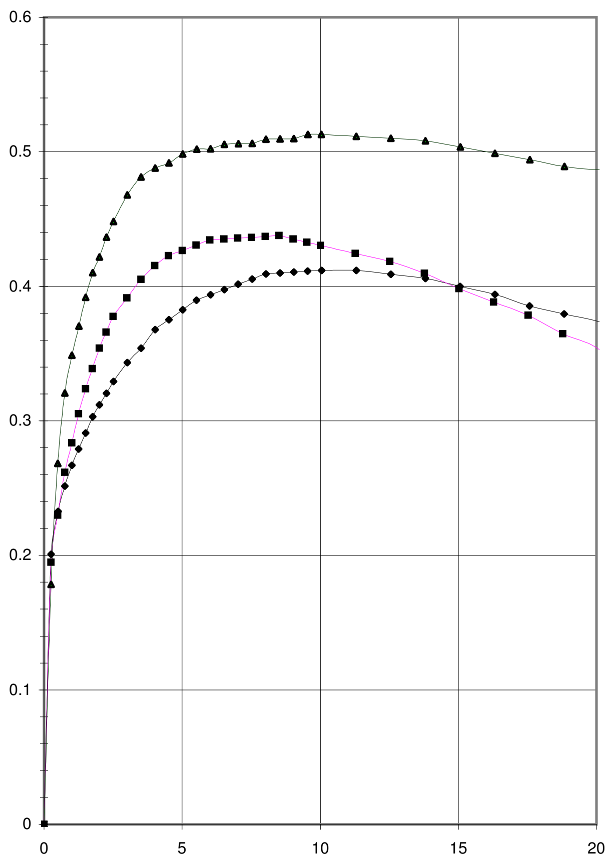
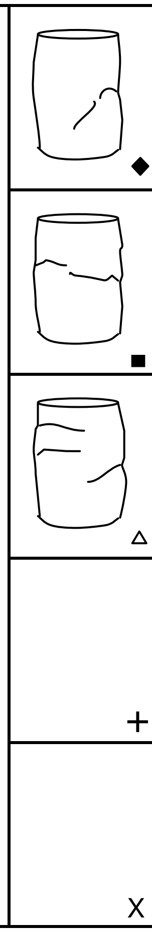
TRIAXIAL TEST ASTM: D 2850

Job No. **6471**
Date: **4/23/08**

Project: **Rapidam Dam - Blue Earth River Feasibility Study - #W912ES-08-T-001**
Boring #: **08-1MU** Sample #: **2** Type: **5T** Depth (ft): **21.0 - 22.3**
Soil Type: **Organic Clay (OH)**

Deviator Stress (tsf)





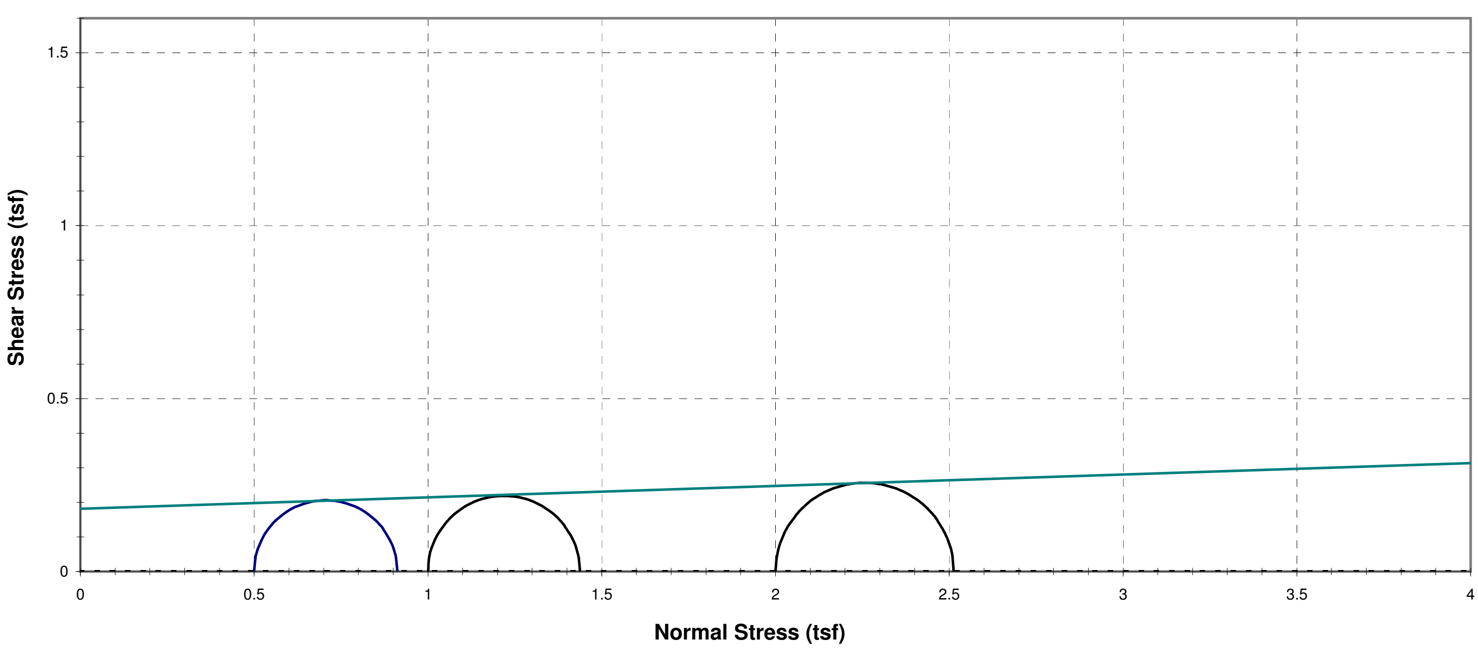
Failure Criterion: **Max. Deviator Stress**
Angle of internal friction, ϕ_u = **1.9 °**
Apparent Cohesion, c_u = **0.18 (tsf)**

Test Date:	4/18/08	Liquid Limit:	78.2
Test Type:	U-U	Plastic Limit:	31.4
Strain Rate (in/min):	0.05	Plasticity Index:	46.8
Strain Rate (%/min):	1.255	Spec. Gravity (Assumed):	2.57

Before Consolidation	A	B	C	D	E
Diameter (in)	1.94	1.94	1.94		
Height (in)	3.98	3.99	3.98		
Water Content (%)	61.3	62.6	59.7		
Dry Density (pcf)	60.9	59.9	64.7		
Void Ratio	1.63	1.68	1.48		
After Consolidation					
Diameter (in)					
Height (in)					
Water Content (%)					
Dry Density (pcf)					
Void Ratio					
Back Pressure (tsf)					
Minor Principal Stress (tsf)	0.50	1.00	2.00		
Max. Deviator Stress (tsf)	0.41	0.44	0.51		
Ultimate Deviator Stress (tsf)	0.36	0.30	0.45		
Deviator Stress at Failure (tsf)	0.41	0.44	0.51		
Max. Pore Pressure Buildup (tsf)	--------	--------	--------		
Pore Pressure Parameter "B"	--------	--------	--------		
Pct. Axial Strain at Failure	11.3	8.5	10.0		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Specimens trimmed to given sizes; Allowed to adjust under applied confining pressures for about 10 minutes.

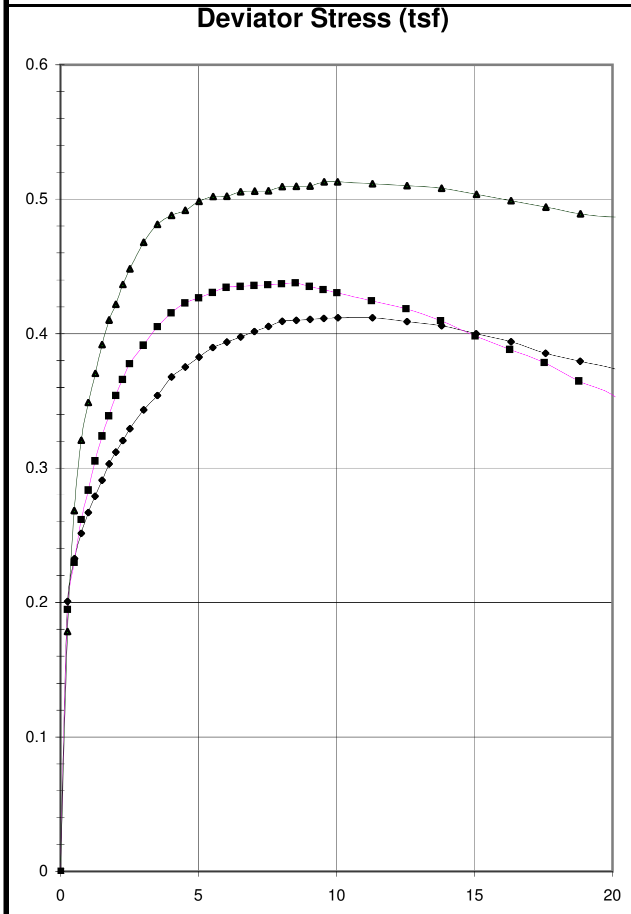


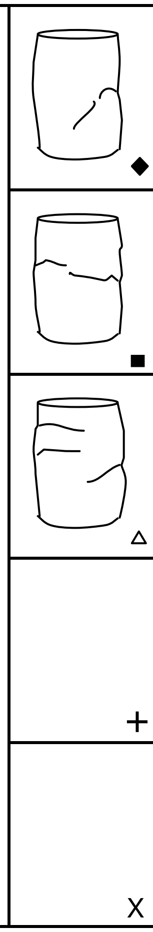
Total ϕ_u:	1.9 °	c_u=	0.18	(tsf)

TRIAXIAL TEST ASTM: D 2850

Job No. **6471**
Date: **4/23/08**

Project:	**Rapidam Dam - Blue Earth River Feasibility Study - #W912ES-08-T-001**				
Boring #:	**08-1MU**	Sample #: **2**	Type: **5T**	Depth (ft):	**21.0 - 22.3**
Soil Type:	**Organic Clay (OH)**				





Failure Criterion: **Max. Deviator Stress**

Angle of internal friction, ϕ_u = **1.9 °**

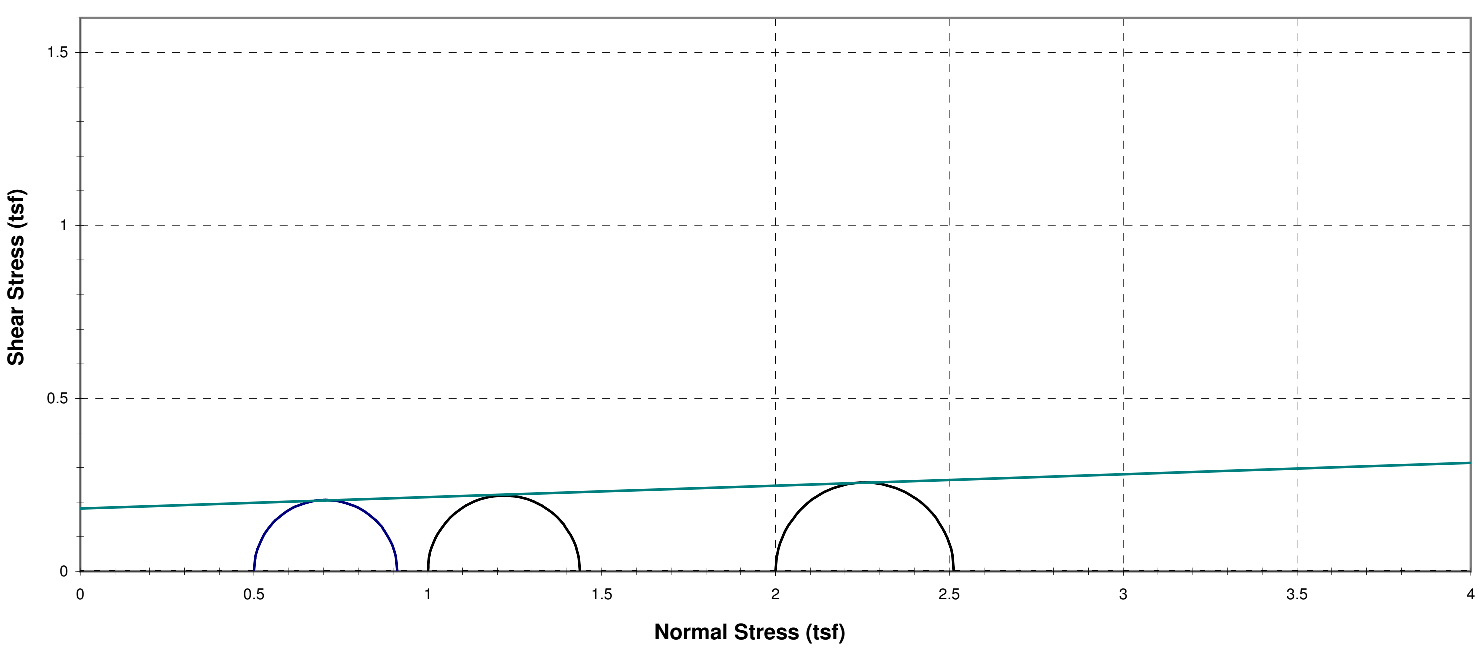
Apparent Cohesion, c_u = **0.18 (tsf)**

Test Date:	4/18/08	Liquid Limit:	78.2
Test Type:	U-U	Plastic Limit:	31.4
Strain Rate (in/min):	0.05	Plasticity Index:	46.8
Strain Rate (%/min):	1.255	Spec. Gravity (Assumed):	2.57

Before Consolidation	A	B	C	D	E
Diameter (in)	1.94	1.94	1.94		
Height (in)	3.98	3.99	3.98		
Water Content (%)	61.3	62.6	59.7		
Dry Density (pcf)	60.9	59.9	64.7		
Void Ratio	1.63	1.68	1.48		
After Consolidation					
Diameter (in)					
Height (in)					
Water Content (%)					
Dry Density (pcf)					
Void Ratio					
Back Pressure (tsf)					
Minor Principal Stress (tsf)	0.50	1.00	2.00		
Max. Deviator Stress (tsf)	0.41	0.44	0.51		
Ultimate Deviator Stress (tsf)	0.36	0.30	0.45		
Deviator Stress at Failure (tsf)	0.41	0.44	0.51		
Max. Pore Pressure Buildup (tsf)	--------	--------	--------		
Pore Pressure Parameter "B"	--------	--------	--------		
Pct. Axial Strain at Failure	11.3	8.5	10.0		

"These test results are for informational purposes only and must be reviewed by a qualified professional engineer to verify that the test parameters shown are appropriate for any particular design"

Remarks: Specimens trimmed to given sizes; Allowed to adjust under applied confining pressures for about 10 minutes.



Total ϕ_u:	1.9 °	c_u=	0.18	(tsf)

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Hydraulic Conductivity Test Data

Project: Rapidan Dam, Blue Earth River Feasibility Study - #'W912ES-08-T-0017 Date: 4/28/2008

Reported To: USACE -Geotech. & Geology Section Job No.: 6471

Boring No.:	08-2M	08-2M	08-2M	08-2M	08-3M		
Sample No.:	2	7	8	10	2		
Depth (ft)	36.3-36.7	72.4-72.7	82.3-82.7	101.7-102.2	78.1-78.5		
Location:	LT. ABUT.	LT. ABUT.	LT. ABUT.	LT. ABUT.	LT. ABUT.		
Sample Type:	Core	Core	Core	Core	Core		
Soil Type:	Jordan Sandstone	Jordan Sandstone	Jordan Sandstone	Jordan Sandstone	Jordan Sandstone		
Atterberg Limits							
LL							
PL							
PI							
Permeability Test	Flex-Wall	Flex-Wall	Flex-Wall	Flex-Wall	Flex-Wall		
Before Test Conditions: Saturation %:							
Porosity:							
Ht. (in):	2.13	1.89	1.89	1.38	2.07		
Dia. (in):	1.94	1.83	1.95	1.89	2.02		
Dry Density (pcf):	117.8	117.4	114.6	121.7	115.7		
Water Content:	8.3%	9.2%	13.8%	19.7%	13.6%		
Test Type:	Constant	Constant	Constant	Constant	Constant		
Max Head (ft):	5.0	5.0	5.0	5.0	5.0		
Confining press. (Effective-psi):	30.4	65.1	69.2	77.0	61.1		
Trial No.:	6-10	6-10	6-10	6-10	6-10		
Water Temp ℃:	22.0	22.0	22.0	22.0	22.0		
% Compaction							
% Saturation (After Test)							
Coefficient of Permeability							
K @ 20 ℃ (cm/sec)	**1.4×10^{-3}**	**2.6×10^{-3}**	**3.8×10^{-4}**	**5.6×10^{-4}**	**3.1×10^{-5}**		
K @ 20 ℃ (ft/min)	**2.8×10^{-3}**	**5.0×10^{-3}**	**7.5×10^{-4}**	**1.1×10^{-3}**	**6.1×10^{-5}**		
Notes:	About 100+cc thru specimen before starting test.						

Hydraulic Conductivity Test Data

Project: Rapidan Dam, Blue Earth River Feasibility Study - #'W912ES-08-T-0017 Date: 4/15/2008

Reported To: USACE -Geotech. & Geology Section Job No.: 6471

Boring No.:	08-01MU	08-01MU	08-02MU	08-04MU			
Sample No.:	1	2	6	1			
Depth (ft)	14.0-15.4 (Top)	21.0-22.3 (Bot.)	56.5-56.8	49.3-49.6			
Location:							
Sample Type:	5" TWT	5" TWT	Core	Core			
Soil Type:	Organic Clay (OH)	Organic Clay (OH)	Jordan Sandstone	Jordan Sandstone			
Atterberg Limits							
LL							
PL							
PI							
Permeability Test	Flex-Wall	Flex-Wall	Flex-Wall	Flex-Wall			
Before Test Conditions: Saturation %:							
Porosity:							
Ht. (in):	2.67	2.93	2.00	1.90			
Dia. (in):	2.84	2.84	2.10	2.00			
Dry Density (pcf):	71.2	56.9	146.8	156.8			
Water Content:	38.8%	79.9%	1.6%	1.1%			
Test Type:	Falling	Falling	Falling	Falling			
Max Head (ft):	5.0	5.0	10.0	20.0			
Confining press. (Effective-psi):	4.5	6.5	47.2	52.4			
Trial No.:	19-23	19-23	24-29	17-2			
Water Temp ℃:	21.0	21.0	22.0	22.0			
% Compaction							
% Saturation (After Test)	98.1%	99.9%	97.3%	96.0%			
Coefficient of Permeability							
K @ 20 ℃ (cm/sec)	**1.1×10^{-7}**	**1.3×10^{-7}**	**4.8×10^{-8}**	**7.9×10^{-10}**			
K @ 20 ℃ (ft/min)	**2.1×10^{-7}**	**2.5×10^{-7}**	**9.4×10^{-8}**	**1.6×10^{-9}**			
Notes:							

Project:	Rapidan Dam, Blue Earth River Feasibility Study - #W912ES-08-T-0017	Date:	5/20/08
Client:	USACE - Geotech. & Geology Section	Job:	6471

Sample Information & Classification

Boring	08-02M	
Sample #	5	
Depth (ft)	52.1-52.5	
Type or BPF	Core	
Soil Classification	Jordan Sandstone	

Test Results

Water Content (%)	13.3	
Dry Density (pcf)	114.3	

9301 Bryant Ave. South Suite 107 SOIL ENGINEERING TESTING, INC. Bloomington, Minnesota 55420-3436

Rapidan Dam Investigations
Blue Earth River Feasibility Study
Ecosystem Restoration

Part B
Hydrologic Investigations

September 2009

Prepared by:

U.S. Army Corps of Engineers
St. Paul District
190 Fifth Street East, Suite 401
St. Paul, Minnesota 55101-1638

Rapidan Dam Plan & Profile

The St. Paul District searched through files at the Minnesota Department of Natural Resources (MNDNR), Blue Earth County Public Works Department (BEC), and the St. Paul District for prior reports and existing information on Rapidan Dam. **Figures 1** and **2** show a plan and profile view of the dam for reference regarding the following discussion. **Figure 3** shows modifications that were made on the downstream apron.

Probable Maximum Flood (PMF)

Previous reports for the Radian Dam location have estimated a PMF discharge value of 164,000 cfs. The St. Paul District researched previous studies to determine the basis for this value. The PMF value originated from a flood control study for the Blue Earth River and computations are documented in an Interim Survey for Flood Control, dated February 1970 **(reference 1)**. The PMF was generated for a proposed reservoir located at mile 3, just upstream of the confluence with the Minnesota River. The drainage area for the proposed dam site was 3,545 square miles.

In the Interim Study, a PMF was computed for each of two seasons; summer season and snowmelt season (month of March). The most critical season was the summer season with an instantaneous peak discharge of 206,000 cfs. The criterion used for the development of this event was Hydrometeorlogical Report No. 44 (**reference 2)**. Current criteria for development of this event in this region is, "NOAA Hydro meteorological Report No. 51 **(reference 3)** and "Hydro meteorological Report No. 52.

The PMF for the Rapidan location was first documented in a hydropower reconnaissance study for Rapidan dam dated August 1979 **(reference 4)**. In this study, a factor was applied to the ordinates of the PMF hydrograph for the Blue Earth dam site. This factor was computed as the ratio of the drainage areas to the 0.6 power[1] using a drainage area for Rapidan as 2,430 square miles[2]. The resulting PMF peak discharge was rounded up to 165,000 cfs. The 164,000 cfs value has been used in subsequent reports. This gauge is located approximately 0.2 miles downstream of the dam.

Dam Safety Standards

In April 2008, a Potential Failure Mode Analysis Study **(reference 5)** was done for this project. As a result of this analysis, the Federal Energy Regulatory Commission (FERC) classified Rapidan Dam as having a SIGNIFICANT downstream hazard potential. The hazard rating is based on the environmental damage that would be caused by an uncontrolled release of the agriculturally impacted sediments that fill the reservoir. The inundation areas downstream of the dam have no permanent inhabited structures, but

[1] Area ratio adopted from peak transfer technique as per USGS report, "Techniques for Estimating Magnitude and Frequency of Floods in Minnesota, WRI 77-31", May 1977.
[2] The USGS has listed the drainage area at the USGS gauge for the Blue Earth River near Rapidan, no. 05320000, which is only 0.2 mi. downstream of the dam, as 2,410 square miles.

there is a park located just downstream of the dam which attracts hikers and fishermen. The park has primitive camp sites and a canoe launch.

U.S. Army Corps criteria for design flood standards are outlined in ER 1110-8-2(FR) **(reference 6)**. This regulation outlines Safety Dam Standards. Based on review of various reports and the FERC hazard classification of Significant, it appears that Rapidan would be assigned "Standard 3". Standard 3 is described as follows:

> Standard 3 applies to dams where an analysis clearly demonstrates that failure could be tolerated at some flood magnitude. The recommended plan should be for a dam which meets or exceeds a base safety standard. The base safety standard will be met when a dam failure related to hydraulic capacity will result in no measurable increase in population at risk and a negligible increase in property damages over that which would have occurred if the dam had not failed. Determination of the IDF that identifies the base safety standard will require definition of the relationship between flood flows and adverse impacts (population at risk and property damages) with and without dam failure for a range of floods up to the probable maximum flood (PMF). Appropriate freeboard will be included for all evaluations. Selection of a base condition predicated on the risk to life from dam failure will require supporting information to demonstrate the increment of population that would actually be threatened. The evaluation should distinguish between population downstream of a dam and the population that would likely be in a life threatening situation given the extent of prefailure flooding, evacuation opportunities, and other factors that might affect the occupancy of the incrementally inundated area at the time the failure occurs. The occurrence of overtopping floods must be relatively infrequent to make standard 3 acceptable. One-half of the PMF is the minimum acceptable IDF for standard 3 dams.

One-half PMF, the minimum Inflow Design Flood for a Standard 3 dam, is 82,000 cfs for Rapidan, although some studies have listed 84,000 cfs as ½ PMF.

Dam Break Studies

Corps of Engineers 1979 Study

Two dam break studies have been identified. The St. Paul District conducted a study done in 1978 **(reference 7**). Although attempts were made to obtain this report, it was not located. However, the 1979 hydropower reconnaissance study included in Appendix I portions of this study pertaining to overall technical assessment and recommendations. Two studies by FERC also referenced portions of the study (**reference 8 & 9).** One of these was an Emergency Action Plan dated July 1984. The dam break report addressed the discharge capacity, flood of record, PMF and the likely consequences of overtopping of the dam.

The study indicated the existing spillway has a discharge capacity of 51,700 cfs with the reservoir elevation at 881.5 ft. just prior to overtopping the abutments. The maximum flood of record had a peak discharge of 43,100 cfs. The likely effect of dam failure would be significant economic damage, but would not increase the risk to human life.

The discharge effect of a dam break at Mankato for a failure during a river discharge of 51,700 cfs was an increase in discharge to 103,600 cfs. Elevation would increase by 10.6 ft from 779.7 ft to 790.3 ft.[3] in about 3.7 hours. These elevations are minimum elevations since Minnesota and Le Sueur River contributions included no backwater effect. The relative difference in elevations and discharges would be the same if this condition were included. The Corps report concluded that an increase in discharge capacity is desirable to decrease the possibility of downstream economic loss but is not needed to insure public safety. No recommendation regarding discharge capacity was made.

Rapidan Redevelopment LTD 1991 Study

A second dam break study was done by Rapidan Redevelopment LTD in 1991. This study addressed three hydrologic failure modes; sunny day, 500-yr, and ½ PMF. The 500-yr inflow flood had a discharge value of 49,900 cfs. The ½ PMF inflow hydrograph used a fuse plug scenario in the right abutment. The ½ PMF breach hydrograph did not significantly differ from the ½ PMF hydrograph without failure because the breach occurred before the peak of the inflow hydrograph.[4]

Downstream flows on the Minnesota River and Le Sueur River, according to the 1989 submittal, were assumed to be the 100-yr floods. These values are 81,100 cfs and 23,000 cfs respectively according to the 1986 Blue Earth Flood Insurance Study.

The effect of the sunny day failure was an increase in elevation at Le Hillier of 0.9 ft for elevations from 769.7 ft to 770.6 ft. and for a 210 ft. breach width. For the 500-yr inflow flood and same breach width, the increase in elevation was 0.6 ft. from elevation 791.6 ft to 792.2 ft. The ½ PMF hydrograph reached a peak elevation of 801.0 which is 6 ft. above the top of levee elevation. The report concluded that the worst case scenario is the 500-yr flood with a 105 foot wide, 0.10 hour breach.

Discharge Frequency

An annual, instantaneous, peak, discharge-frequency relationship was developed at the USGS gage (05320000) for the Blue Earth River near Rapidan, MN. At this location the contributing drainage area is 2,410 square miles. This gage has 95 years of broken systematic record (1910, 1912-46,1948,1950-2007) and 100 years of historic record with the 1965 event discharge of 43,100 cfs tagged as the largest event to occur since 1908. **Table 1** lists the instantaneous, peak discharge values and corresponding dates and **Table 2**. lists these values along with their rank and plotting position. Flows are based on Water Year.

[3] A later dam break study dated 1991 (**reference 10**) by Rapidan Redevelopment LTD, indicated the top of levee elevation at Le Hillier was approximately 795 ft.
[4] Silt has consumed most of the available storage in the reservoir; therefore, top of sediment pool was assumed to be 862 ft.

The flows at the gage are considered to be regulated based on operation of the dam. The regulation and storage effect varied throughout the projects history because of the sediment loading into the reservoir to the point where the storage space has been mostly consumed. Even though operation of the dam is now run-of-river, the peak flow set at the downstream USGS gauge is considered to be non-homogenous and non-stationary. Therefore, instead of applying a Log Pearson Type III distribution to these flows as per US Water Resource Council, Bulletin 17B **(reference 11),** a graphical fit was made to the flows values with their corresponding Weibull plotting position. **Figure 4** shows the discharge-frequency plot.

Table 3 lists the 10-, 50-, 100-, and 500-yr flows obtained from the graphical plot. The 100-yr discharge value for is 30,000 cfs. The 500-yr discharge value is 45,000 cfs which is lower than the previous 49,900 cfs value used for the 500-yr flood in the 1991 EAP. It is also higher than the 1965 flood of record of 43,100 cfs.

Summary

The PMF assumed in previous studies is outdated and should be redeveloped based on HMR 51 & 52. In addition the PMF was originally developed for the proposed Blue Earth Dam located at mile 3 from the confluence with the Minnesota River and downstream of the Le Sueur River confluence with the Blue Earth River. The PMF at Rapidan was then estimated based on drainage area transfer.

For the Probable Failure Mode Analysis conducted in 2008, the hazard classification was updated to SIGNIFICANT and therefore, may require a minimum IDF of ½ PMF. A Base Safety Standard BSS should also be determined. The BSS is the inflow design flood where there is no significant increase in adverse consequences from dam failure compared to non-failure adverse consequences. According to the 1991 EAP, it appears that the BSS is less than ½ PMF because simulation of the ½ PMF in this study resulted in dam failure at elevation 881.5 ft (top of dam) before the peak of the inflow hydrograph occurred. Therefore, there was little difference between with and without failure discharges.

Previous studies indicated that the spillway capacity is 51,700 cfs. The threshold flood should be determined and considered as an option for design of a stilling basin. The threshold flood is the flood that fully uses the existing dam or just exceeds the design maximum water surface elevation at the dam. This would include minimum of 3 ft of freeboard and could be higher considering other factors outlined in ER 1110-8-2(FR) such as wind setup and wave runup.

Updating the discharge-frequency relationship decreased the 500-yr discharge value from 49,900 cfs to 45,000 cfs. The 100-yr discharge value was 30,000 cfs. These estimates are based on a graphical plot due to the non-stationarity and non-homogeneity of the recorded data.

The two most likely candidates for design flow of a stilling basin would then be the threshold flow or ½ PMF. Corps Standard 3 criteria would require minimum ½ PMF.

FERC standards may also require ½ PMF. Significant modifications of the dam structure would be required. This standard is based on a hazard classification of SIGNIFICANT which was in turn based on the environmental impact of sediment release downstream. Preliminary estimates are that at elevation 878.1 ft., which provides three feet of freeboard, the spillway discharge is approximately 48,300 cfs. This is approximately the threshold flood for the dam and has an exceedence frequency of 0.15 percent (~ 670-yr recurrence interval).

Table 1 USGS Recorded Annual Peak Discharges

Blue Earth County, Minnesota
Hydrologic Unit Code 07020009
Latitude 44°05'44", Longitude 94°06'33" NAD27
Drainage area 2,410 square miles
Gage datum 808.80 feet above sea level NGVD29

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Water Year	Date	Gage Height (feet)	Stream-flow (cfs)	Water Year	Date	Gage Height (feet)	Stream-flow (cfs)
1910	Mar. 12, 1910		15,600	1960	May 24, 1960	11.52	16,600[6]
1912	May 04, 1912		1,810[1,6]	1961	Mar. 28, 1961	9.35	11,200[6]
1913	Apr. 14, 1913		3,190[1,6]	1962	Apr. 02, 1962	11.53	16,600[6]
1914	Jul. 03, 1914		3,250[1,6]	1963	Jul. 24, 1963	9.36	11,200[6]
1915	Mar. 28, 1915		11,510[1,6]	1964	May 15, 1964	6.46	5,240[6]
1916	Mar. 25, 1916		10,300[1,6]	1965	Apr. 09, 1965	21.36	43,100[6,7]
1917	Mar. 27, 1917		13,280[1,6]	1966	Apr. 02, 1966	6.16	4,760[6]
1918	Aug. 22, 1918		6,690[1,6]	1967	Jun. 17, 1967	7.04	6,300[6]
1919	Apr. 18, 1919		9,610[1,6]	1968	Sep. 25, 1968	5.04	3,160[6]
1920	Mar. 16, 1920		7,250[1,6]	1969	Apr. 10, 1969	13.54	21,100[6]
1921	Jun. 13, 1921		2,310[1,6]	1970	May 16, 1970	5.26	3,460[6]
1922	Mar. 14, 1922		3,090[1,6]	1971	Mar. 21, 1971	8.15	8,580[6]
1923	Mar. 26, 1923		374[1,6]	1972	Jun. 09, 1972	8.43	9,200[6]
1924	Aug. 24, 1924		1,260[1,6]	1973	Mar. 13, 1973	9.69	8,380[6]
1925	Jun. 17, 1925		4,250[1,6]	1974	Jun. 11, 1974	7.01	6,220[6]
1926	Sep. 28, 1926		1,160[1,6]	1975	May 01, 1975	7.13	6,460[6]
1927	1927		4,500[6,B]	1976	Mar. 20, 1976	4.03	1,240[6]
1928	Mar. 16, 1928		1,960[1,6]	1977	Jun. 17, 1977	3.73	1,500[6]
1929	1929		11,600[6,B]	1978	Jun. 21, 1978	6.31	4,920[6]
1930	1930		2,750[6,B]	1979	Mar. 31, 1979	9.31	11,100[6]
1931	Mar. 24, 1931		210[1,6]	1980	Jun. 03, 1980	8.42	9,170[6]
1932	1932		6,000[6,B]	1981	Jun. 26, 1981	7.45	7,100[6]
1933	1933		10,800[6,B]	1982	Mar. 24, 1982	6.67	5,760[6]
1934	1934		1,600[6,B]	1983	Mar. 03, 1983	10.11	12,800[6]
1935	1935		2,200[6,B]	1984	Jun. 23, 1984	8.79	9,810[6]
1936	1936		13,600[6,B]	1985	Mar. 17, 1985	9.38	11,100[6]
1937	1937		2,500[6,B]	1986	Mar. 23, 1986	11.17	15,200[6]
1938	1938		13,900[6,B]	1987	Oct. 15, 1986	5.85	4,300[6]
1939	1939		4,700[6,B]	1988	Mar. 24, 1988	5.36	3,570[6]
1940	Jun. 07, 1940	3.97	1,710[6]	1989	Mar. 27, 1989	7.36	6,890[6]
1941	Mar. 31, 1941	6.13	4,390[6]	1990	Jul. 28, 1990	7.00	6,170[6]
1942	Mar. 29, 1942	5.02	2,790[6]	1991	Jun. 08, 1991	10.09	12,800[6]
1943	Jun. 16, 1943	7.85	6,940[6]	1992	Mar. 03, 1992	9.12	10,500[6]
1944	May 22, 1944	9.41	11,000[6]	1993	Jun. 20, 1993	13.32	20,300[6]
1945	Jun. 15, 1945	9.01	9,500[6]	1994	Mar. 16, 1994	8.15	8,450[6]
1946	1946		7,500[6,B]	1995	Apr. 24, 1995	7.67	7,310[6]
1948	1948		11,600[6,B]	1996	Jun. 20, 1996	8.08	8,700[6]
1950	Mar. 30, 1950	6.07	4,390[6]	1997	Mar. 23, 1997	8.62	9,440[6]

1951	Apr. 08, 1951	14.97	26,100[6]
1952	Apr. 01, 1952	11.17	14,700[6]
1953	Jun. 09, 1953	12.91	19,700[6]
1954	Jun. 25, 1954	7.12	6,230[6]
1955	Mar. 12, 1955	5.44	3,550[6]
1956	Jun. 18, 1956	6.80	5,670[6]
1957	May 28, 1957	5.39	3,440[6]
1958	Apr. 08, 1958	3.59	1,270[6]
1959	Jun. 06, 1959	5.53	3,670[6]
1998	Mar. 31, 1998	8.31	8,780[6]
1999	Jun. 14, 1999	8.92	10,100[6]
2000	May 22, 2000	6.21	4,870[6]
2001	Apr. 14, 2001	12.04	17,200[6]
2002	Aug. 23, 2002	5.66	4,010[6]
2003	May 14, 2003	6.68	5,660[6]
2004	Sep. 19, 2004	9.84	12,200[6]
2005	Sep. 29, 2005	9.02	10,600[6]
2006	Apr. 10, 2006	10.01	12,600[6]
2007	Mar. 21, 2007	9.98	12,500[5]

Table 2 Recorded Peak Discharges, Rank & Plotting Position

Events Analyzed				Ordered Events			
Day	Mon	Year[5]	FLOW cfs	Rank	Water Year	FLOW cfs	Weibull Plot Pos
12	Mar	1910	15,600	1	1965	43,100*	1.04
04	May	1912	1,810	2	1951	26,100	2.08
14	Apr	1913	3,190	3	1969	21,100	3.12
03	Jul	1914	3,250	4	1993	20,300	4.17
28	Mar	1915	11,510	5	1953	19,700	5.21
25	Mar	1916	10,300	6	2001	17,200	6.25
27	Mar	1917	13,280	7	1962	16,600	7.29
22	Aug	1918	6,690	8	1960	16,600	8.33
18	Apr	1919	9,610	9	1910	15,600	9.38
16	Mar	1920	7,250	10	1986	15,200	10.42
13	Jun	1921	2,310	11	1952	14,700	11.46
14	Mar	1922	3,090	12	1938	13,900	12.50
26	Mar	1923	374	13	1936	13,600	13.54
24	Aug	1924	1,260	14	1917	13,280	14.58
17	Jun	1925	4,250	15	1991	12,800	15.62
28	Sep	1926	1,160	16	1983	12,800	16.67
01	Jan	1927	4,500	17	2006	12,600	17.71
16	Mar	1928	1,960	18	2007	12,500	18.75
01	Jan	1929	11,600	19	2004	12,200	19.79
01	Jan	1930	2,750	20	1948	11,600	20.83
24	Mar	1931	210	21	1929	11,600	21.88
01	Jan	1932	6,000	22	1915	11,510	22.92
01	Jan	1933	10,800	23	1963	11,200	23.96
01	Jan	1934	1,600	24	1961	11,200	25.00
01	Jan	1935	2,200	25	1985	11,100	26.04
01	Jan	1936	13,600	26	1979	11,100	27.08
01	Jan	1937	2,500	27	1944	11,000	28.12
01	Jan	1938	13,900	28	1933	10,800	29.17
01	Jan	1939	4,700	29	2005	10,600	30.21
07	Jun	1940	1,710	30	1992	10,500	31.25
31	Mar	1941	4,390	31	1916	10,300	32.29
29	Mar	1942	2,790	32	1999	10,100	33.33
16	Jun	1943	6,940	33	1984	9,810	34.38
22	May	1944	11,000	34	1919	9,610	35.42
15	Jun	1945	9,500	35	1945	9,500	36.46
01	Jan	1946	7,500	36	1997	9,440	37.50
01	Jan	1948	11,600	37	1972	9,200	38.54
30	Mar	1950	4,390	38	1980	9,170	39.58
08	Apr	1951	26,100	39	1998	8,780	40.62
01	Apr	1952	14,700	40	1996	8,700	41.67
09	Jun	1953	19,700	41	1971	8,580	42.71
25	Jun	1954	6,230	42	1994	8,450	43.75
12	Mar	1955	3,550	43	1973	8,380	44.79
18	Jun	1956	5,670	44	1946	7,500	45.83
28	May	1957	3,440	45	1995	7,310	46.88
08	Apr	1958	1,270	46	1920	7,250	47.92
06	Jun	1959	3,670	47	1981	7,100	48.96
24	May	1960	16,600	48	1943	6,940	50.00
28	Mar	1961	11,200	49	1989	6,890	51.04
02	Apr	1962	16,600	50	1918	6,690	52.08
24	Jul	1963	11,200	51	1975	6,460	53.12
15	May	1964	5,240	52	1967	6,300	54.17
09	Apr	1965	43,100	53	1954	6,230	55.21
02	Apr	1966	4,760	54	1974	6,220	56.25
17	Jun	1967	6,300	55	1990	6,170	57.29
25	Sep	1968	3,160	56	1932	6,000	58.33
10	Apr	1969	21,100	57	1982	5,760	59.38
16	May	1970	3,460	58	1956	5,670	60.42
21	Mar	1971	8,580	59	2003	5,660	61.46
09	Jun	1972	9,200	60	1964	5,240	62.50
13	Mar	1973	8,380	61	1978	4,920	63.54
11	Jun	1974	6,220	62	2000	4,870	64.58
01	May	1975	6,460	63	1966	4,760	65.62
20	Mar	1976	1,240	64	1939	4,700	66.67
17	Jun	1977	1,500	65	1927	4,500	67.71
21	Jun	1978	4,920	66	1950	4,390	68.75
31	Mar	1979	11,100	67	1941	4,390	69.79
03	Jun	1980	9,170	68	1987	4,300	70.83
26	Jun	1981	7,100	69	1925	4,250	71.88

[5] Missing dates were assigned 01 Jan.

Table 2 Recorded Peak Discharges, Rank & Plotting Position (continued)

24 Mar 1982	5,760	70	2002	4,010	72.92
03 Mar 1983	12,800	71	1959	3,670	73.96
23 Jun 1984	9,810	72	1988	3,570	75.00
17 Mar 1985	11,100	73	1955	3,550	76.04
23 Mar 1986	15,200	74	1970	3,460	77.08
15 Oct 1986	4,300	75	1957	3,440	78.12
24 Mar 1988	3,570	76	1914	3,250	79.17
27 Mar 1989	6,890	77	1913	3,190	80.21
28 Jul 1990	6,170	78	1968	3,160	81.25
08 Jun 1991	12,800	79	1922	3,090	82.29
03 Mar 1992	10,500	80	1942	2,790	83.33
20 Jun 1993	20,300	81	1930	2,750	84.38
16 Mar 1994	8,450	82	1937	2,500	85.42
24 Apr 1995	7,310	83	1921	2,310	86.46
20 Jun 1996	8,700	84	1935	2,200	87.50
23 Mar 1997	9,440	85	1928	1,960	88.54
31 Mar 1998	8,780	86	1912	1,810	89.58
14 Jun 1999	10,100	87	1940	1,710	90.62
22 May 2000	4,870	88	1934	1,600	91.67
14 Apr 2001	17,200	89	1977	1,500	92.71
23 Aug 2002	4,010	90	1958	1,270	93.75
14 May 2003	5,660	91	1924	1,260	94.79
19 Sep 2004	12,200	92	1976	1,240	95.83
29 Sep 2005	10,600	93	1926	1,160	96.88
10 Apr 2006	12,600	94	1923	374*	97.92
21 Mar 2007	12,500	95	1931	210*	98.96

* Outlier

Table 3 Discharge Frequency Values

Recurrence Interval	Discharge, cfs
10-yr	15,000
50-yr	25,000
100-yr	30,000
500-yr	45,000

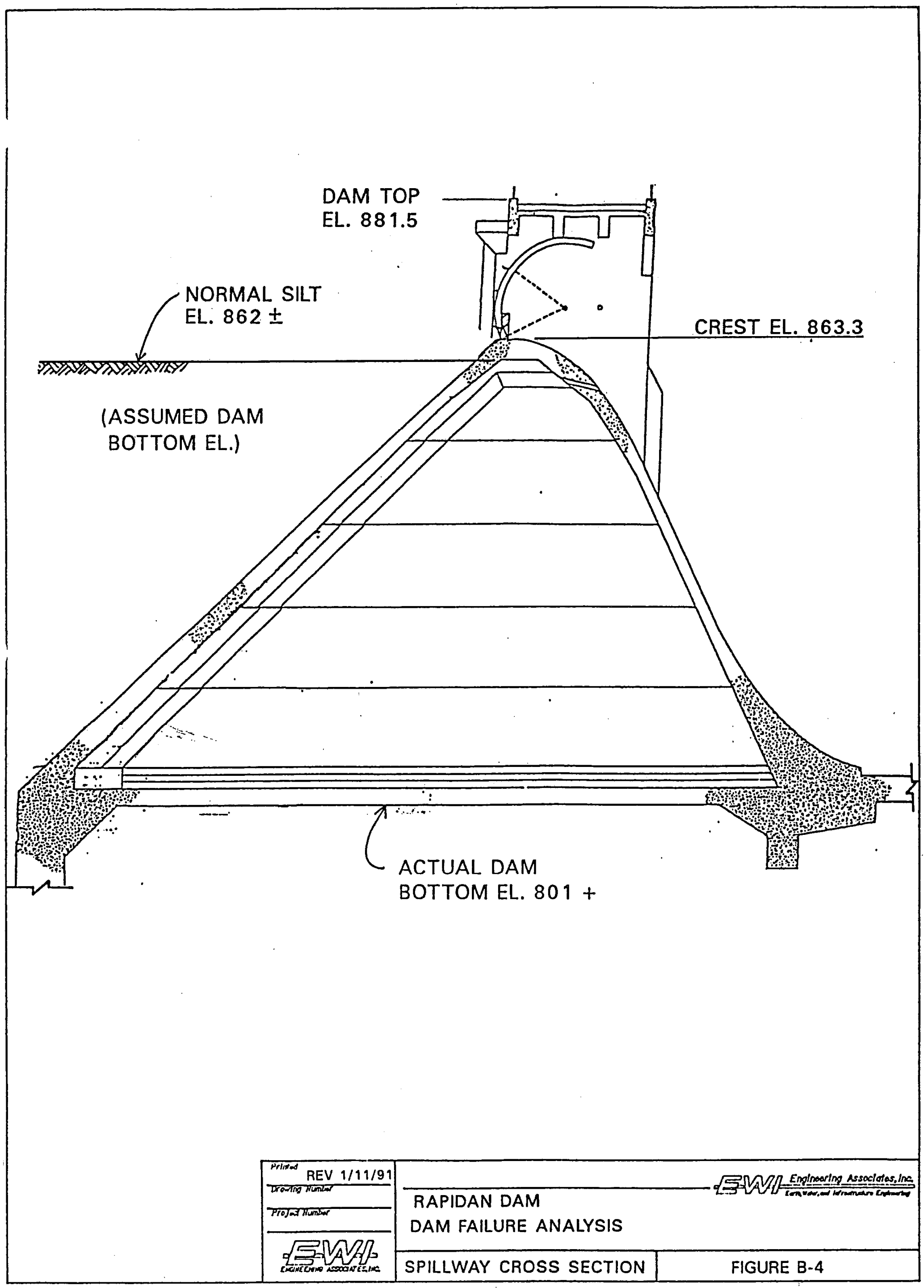
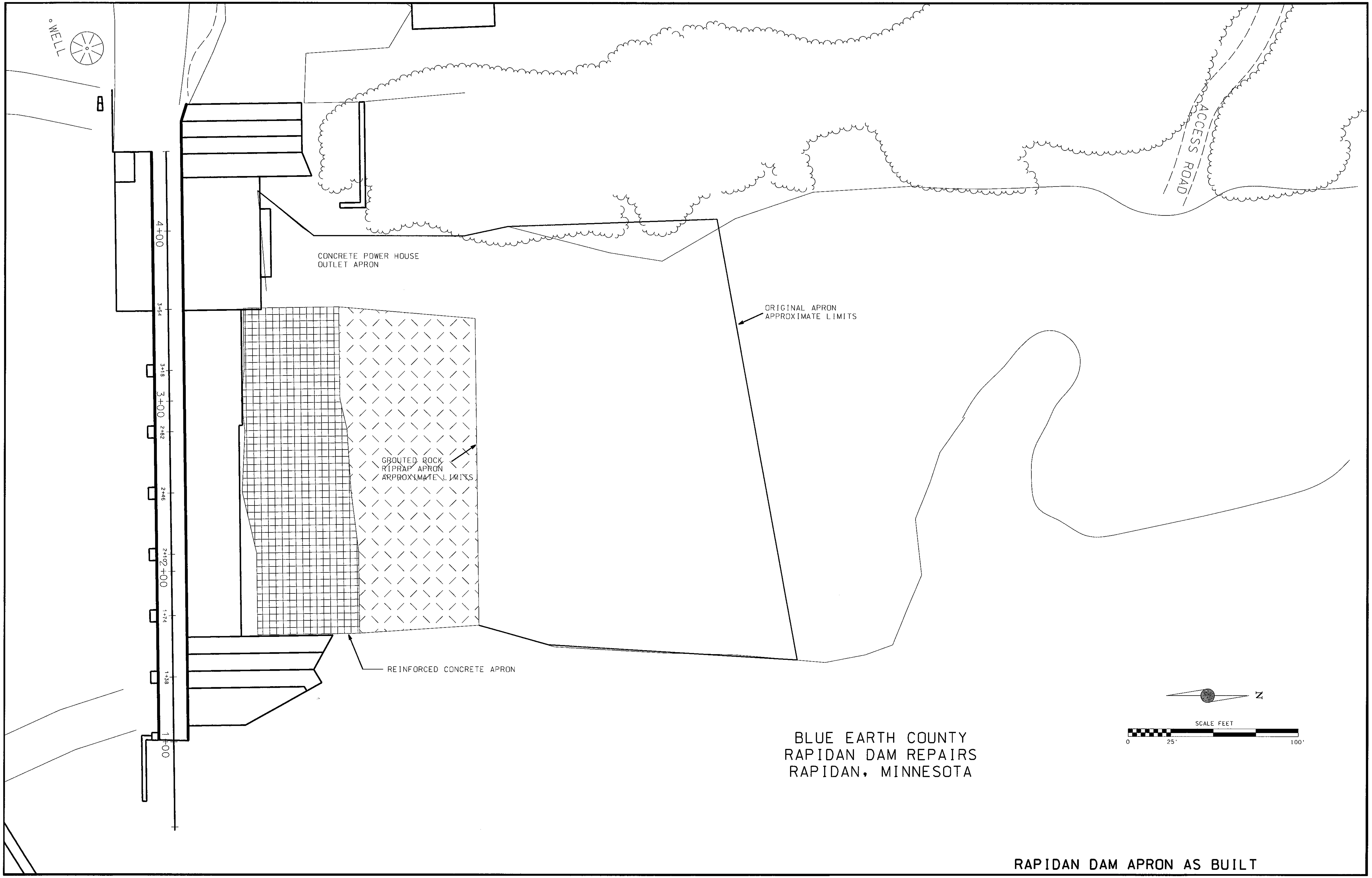


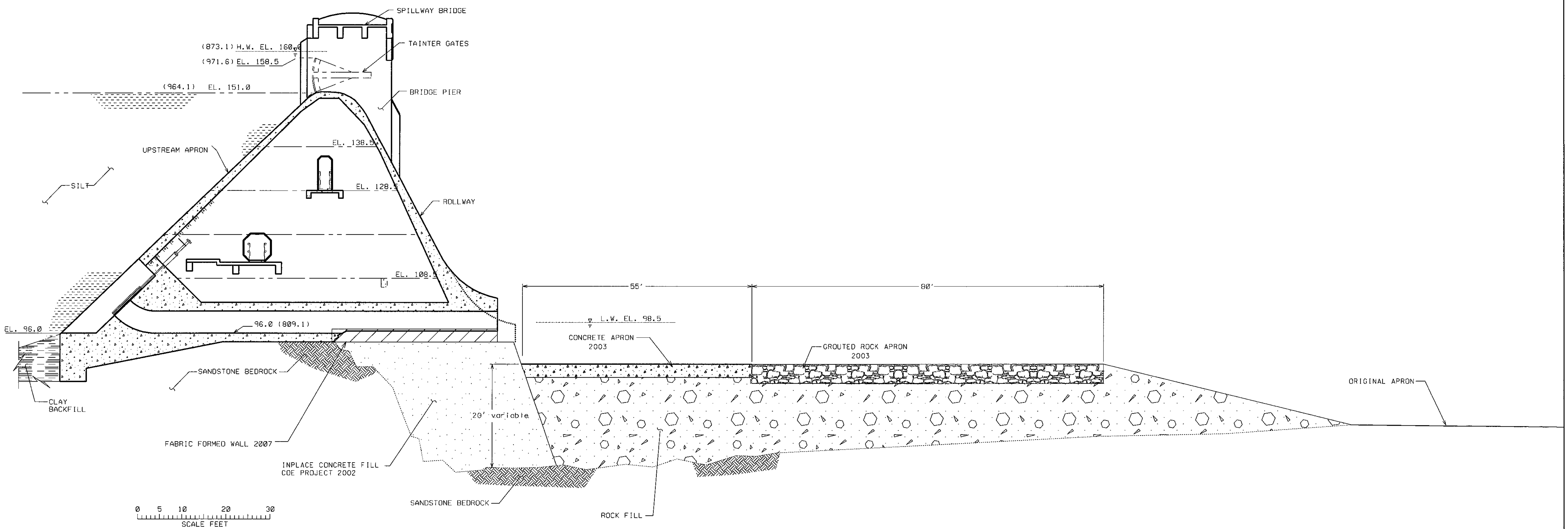
Figure 1 Dam Cross Section



.../Dam/Apron_as-built_plan.dgn 7/29/2008 8:24:24 AM

FIGURE 2

BLUE EARTH COUNTY
RAPIDAN DAM REPAIRS
RAPIDAN, MINNESOTA



RAPIDAN DAM AS BUILT

FIGURE 3

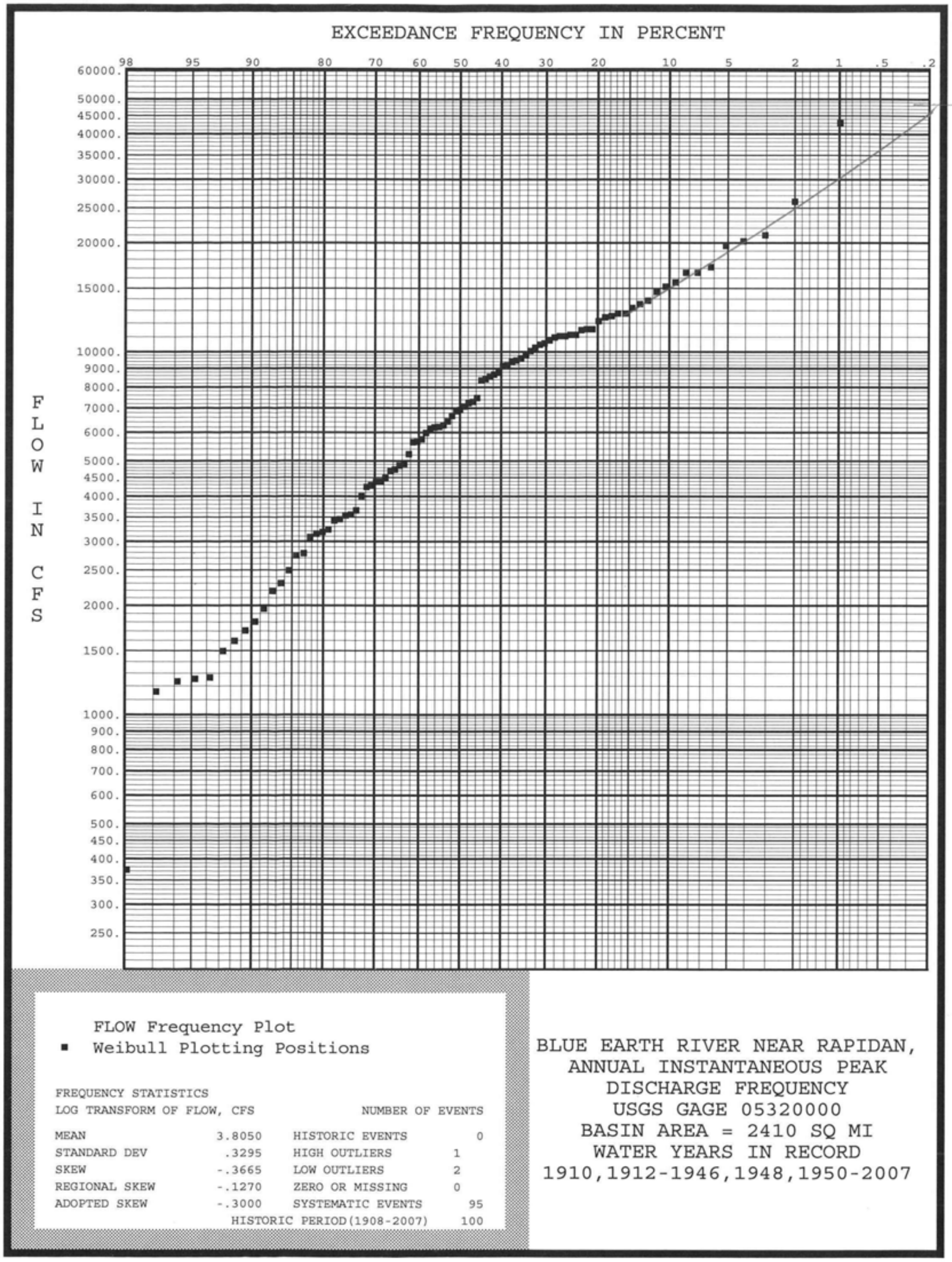


Figure 4 Discharge-Frequency Curve, Blue Earth River near Rapidan, MN

References

1. U.S. Department of the Army, Corps of Engineers (COE), "Interim Survey of Blue Earth River, Minnesota for Flood Control, Appendix A, Economic Growth and Development", February 1970.

2. U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA), "Hydrometeorological Report No. 51, Probable Maximum Precipitation Estimates – United States East of the 105th Meridian", June 1978.

3. U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA), "Hydrometeorological Report No. 52, Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian", August 1982.

4. U.S. Department of Energy, Federal Energy Regulatory Commission (FERC), "Potential Failure Mode Analysis Study Report, Rapidan Dam, FECRC Project No. 3071", 17 April 2008.

5. U.S. Department of Energy, Federal Energy Regulatory Commission (FERC), "Potential Failure Mode Analysis Study Report, Rapidan Dam, FECRC Project No. 3071", 17 April 2008.

6. U.S. Department of the Army, Corps of Engineers (COE), "ER 1110-8-2(FR), Inflow Design Floods for Dams and Reservoirs", 01 March 1991.

7. U.S. Department of the Army, Corps of Engineers (COE), Blue Earth River, Rapidan Dam, Blue Earth County, Minnesota, Inventory No. 512, 1978.

8. U.S. Department of Energy, Federal Energy Regulatory Commission (FERC), "Feasibility Study, Rapidan Dam", April 1981.

9. Rapidan Redevelopment LTD, "Emergency Action Plan, Rapidan Hydroelectric Project, FERC No. 3071", July 1984.

10. Rapidan Redevelopment LTD, "Emergency Action Plan, Rapidan Hydroelectric Project, FERC No. 3071", submitted July 1989, revision January 1991.

11. U.S. Department of the Interior, Geological Survey, "Guidelines for Determining Flood Flow Frequency, Bulletin # 17B", Reston, VA, March 1982.

Rapidan Dam Investigations
Blue Earth River Feasibility Study
Ecosystem Restoration

Part C
Hydraulic Investigations

September 2009

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Rapidan Dam:

Hydraulic Assessment of Spillway Capacity & Energy Dissipation Characteristics

US Army Corps of Engineers, St. Paul District

August 2008

I. Hydraulic Structure Data:

--All elevations reported in NGVD 1929

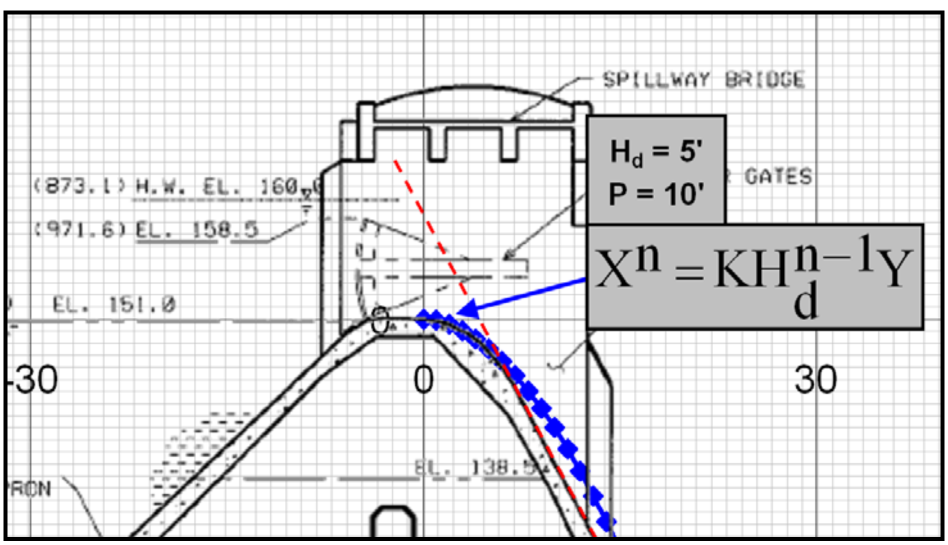
Low Chord of Service Bridge: 877.29 feet
Top Deck of Service Bridge: 881.56 feet
Downstream Horizontal Concrete Apron: 802.06 feet
As-built Length of Horizontal Apron (concrete only) = 55 feet
As-built Length of Horizontal Apron (concrete plus grouted rock) = 135 feet
Spillway Slope: 1 H : 1.9 V

<u>Tainter Bays</u>
Number of Bays: 5
Width of Each Bay: 34 feet
Spillway Crest: 864.06 ft
Estimated Design Head: 5 ft (back calculated based on crest shape, see Sect II)

<u>Needle Bays:</u>
Number of Bays: 2
Width of Each Bay: 34 feet
Spillway Crest: 866.56 ft

II. Estimated Open Gate Headwater Rating Curve

The headwater rating curve was calculated for the Rapidan Dam by first estimating the design head based on the elliptical shape of the spillway crest. The design head was found to be approximately 5', which results in a downstream elliptical shape very close to the actual shape shown in the as-built drawings[1]. The approach height was estimated as 10', which is much smaller than the original design, to account for siltation that has taken place in the reservoir



[1] Design of downstream elliptical crest shape from COE "Hydraulic Design of Spillways" EM 1110-2-1603, section 3-3

Figure 1. Back calculation of design head based on spillway shape

Knowing the original design head for the spillway crest, discharge based on the upstream energy head was calculated accounting for the variable coefficient of discharge[2]. Although the water surface profile through the spillway crest is difficult to estimate; an envelope of possible water surface elevations that account for velocity head was also estimated according to the equations below. The 1965 high water mark seems to indicate that the actual water surface is within this envelope.

Minimum WSEL / Maximum Velocity Head:

$$H_v = \left(\frac{Q}{LD_c}\right)^2 \frac{1}{2g}$$

$$Max.\,WSEL = Crest\ EL + H_e - H_v$$

Maximum WSEL / Minimum Velocity Head:

$$H_v = \left(\frac{Q}{LH_e}\right)^2 \frac{1}{2g}$$

$$Min.\,WSEL = Crest\ EL + H_e - H_v$$

[2] Coefficient of discharge varies according to Hydraulic Design Criteria 111-21/1, reproduced as plate 3-4 in COE "Hydraulic design of Spillways" EM 1110-2-1603

Upstream Energy Head Elevation	5 Tainter Bays H_e	C	Q	2 Needle Bays H_e	C	Q	Total Flow (cfs)	WSEL Envelope at Crest of Tainter Bays Max: $H_v = q/H_e$	Min: $H_v = q/D_c$
864.06	0	3.10	0				0		
864.5	0.44	3.19	158				158	864.43	864.35
865	0.94	3.29	509				509	864.84	864.67
866	1.94	3.49	1602				1602	865.63	865.30
867	2.94	3.71	3179	0.44	3.188	63	3242	866.37	865.89
868	3.94	3.84	5109	1.44	3.388	398	5507	867.10	866.48
869	4.94	3.97	7418	2.44	3.61	937	8354	867.79	867.05
870	5.94	4.07	10027	3.44	3.78	1641	11667	868.47	867.62
871	6.94	4.16	12915	4.44	3.92	2494	15409	869.14	868.18
872	7.94	4.22	16042	5.44	4.02	3472	19513	869.81	868.74
873	8.94	4.26	19347	6.44	4.12	4573	23921	870.48	869.31
874	9.94	4.30	22896	7.44	4.20	5789	28685	871.15	869.87
875	10.94	4.30	26451	8.44	4.24	7065	33517	871.86	870.45
876	11.94	4.30	30159	9.44	4.28	8437	38596	872.57	871.04
877	12.94	4.30	34027	10.44	4.30	9863	43890	873.28	871.62
878	13.94	4.30	38046	11.44	4.30	11314	49360	874.00	872.21
879	14.94	4.30	42213	12.44	4.30	12829	55042	874.71	872.79
880	15.94	4.30	46521	13.44	4.30	14407	60928	875.42	873.38
881	16.94	4.30	50967	14.44	4.30	16045	67011	876.14	873.96
882	17.94	4.30	55546	15.44	4.30	17740	73286	876.85	874.54
883	18.94	4.30	60254	16.44	4.30	19491	79745	877.56	875.13
884	19.94	4.30	65089	17.44	4.30	21296	86385	878.27	875.71
885	20.94	4.30	70046	18.44	4.30	23154	93199	878.99	876.30
886	21.94	4.30	75123	19.44	4.30	25062	100185	879.70	876.88
887	22.94	4.30	80317	20.44	4.30	27021	107338	880.41	877.47

Figure 2. Computed rating table for free flow conditions

The water surface comes into contact with the low chord of the service bridge above the spillway at elevation 877.29 feet. At this point the free-flow weir equation is no longer valid and flow must be estimated using a sluice gate type pressure flow equation. The discharge over the dam under pressure flow was calculated using the method outlined in FHWA 1986 and the results are shown below[3].

Upstream Energy Head Elevation	Q (cfs)
878.0	35918
879.0	37671
880.0	39363
881.0	40998
882.0	42582
883.0	44119
884.0	45612
885.0	47065
886.0	48480
887.0	49860
888.0	51208
889.0	52526
890.0	53815

Figure 3. Computed rating table for pressure flow conditions

[3] Method for computation of pressure flow discharge taken from FHWA 1986 "Bridge Waterways Analysis Manual: Research Report", Chapter III—Orifice Flow

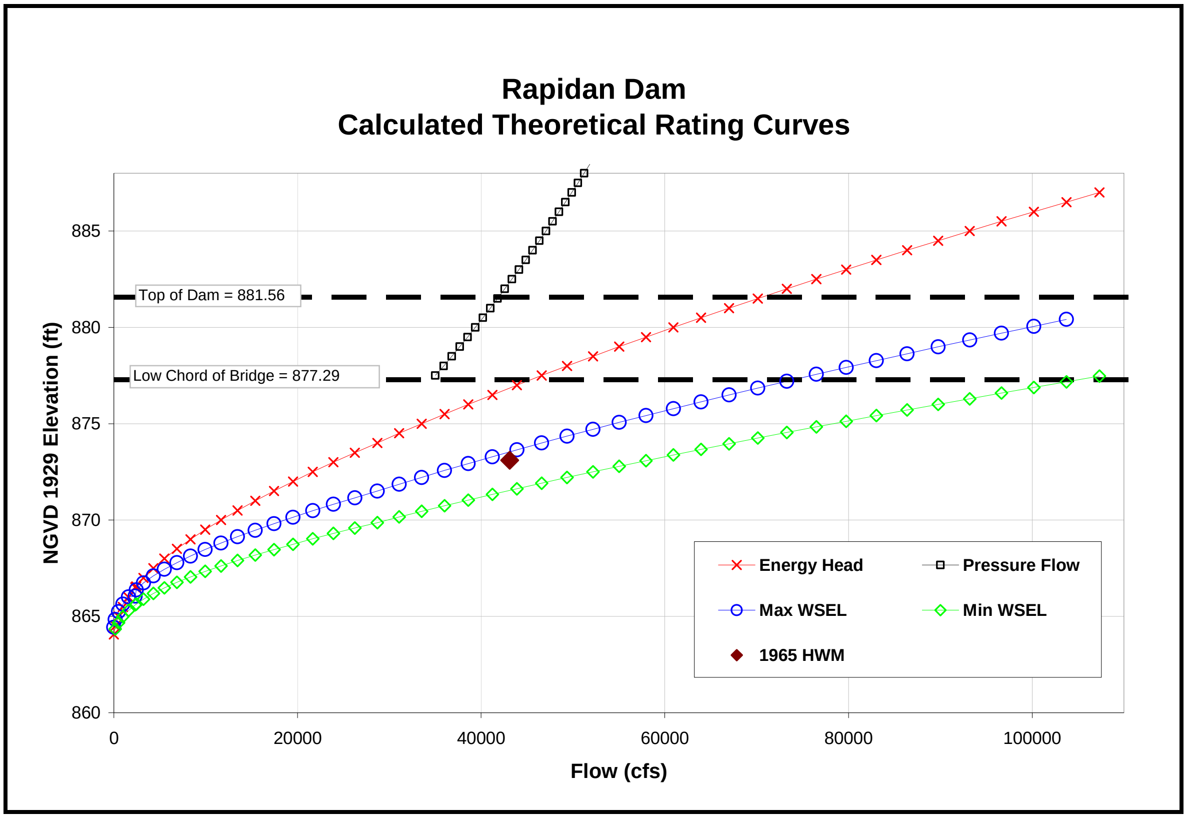


Figure 4: Theoretical rating curves for Rapidan Dam spillway.

The water surface is significantly lower than the energy grade line near the crest due to high velocities. Due to the difficulty of accurately modeling water surface profiles over the spillway crest, the point where the spillway would theoretically switch to a pressure flow regime is not well defined. Other factors that influence the switch to pressure flow include pier effects and potential obstruction caused by tainter gate trunions. For this reason, there is a great variance in the theoretical maximum spillway discharge.

Method	Maximum Spillway Capacity
Pressure Flow, Energy Head = 881.56, Top of Dam (no freeboard)	41,891 cfs
Energy Head = Low Chord Elevation	45,461 cfs
WSEL = Low Chord (min. vel. envelope)	77,262 cfs
WSEL = Low Chord (max. vel. envelope)	105,167 cfs

Figure 5. Maximum spillway discharge.

III. Existing Hydraulic Conditions for Energy Dissipation

The existing energy dissipation structure at Rapidan Dam follows no standard energy dissipation design. This configuration does not meet safety criteria for a Corps of Engineers project. For the purposes of initial planning, an investigation of the hydraulic characteristics of the existing stilling basin was done.

The tailwater rating curve was taken from the published rating table for USGS gage #05320000 which is located approximately 1,000 feet downstream from the spillway and is limited to a maximum discharge of 43,100 cfs. A Flood Insurance Study revised in 1999 exists for Blue Earth County and includes various cross section surveys on the Blue Earth River, however the nearest cross section is located approximately 37,500' below Rapidan Dam. No other information for the tailwater rating curve above 43,100 cfs was found. For more detailed design, this tailwater rating curve should be extended for higher flows and translated upstream to the dam site. However, for the purposes of this feasibility level investigation the direct use of this rating curve is considered adequate for flows up to 43,100 cfs and slightly conservative due to its location downstream of the dam.

In order to evaluate the existing hydraulic conditions at the toe of the spillway apron, a numerical hydraulic model was constructed using HEC-RAS. HEC-RAS is a one-dimensional hydraulic model that has the ability to calculate energy losses over the spillway and through the spillway basin. It was necessary to make the following simplifying assumptions:

1) The spillway extends at a 1H : 1.9V slope until it reaches the horizontal concrete apron at elevation 802.06'. In other words, the effects from the flared downstream end of the spillway were ignored. Some minor energy losses would be expected from this flared end, thus to ignore this geometry is conservative.

2) The flow condition is dominated by the five downstream left bays that have tainter gates. The two downstream right bays that have needle gates flow through a complicated two-dimensional geometry of a stepped energy dissipation structure before entering the primary spillway basin. These needle bays also have a higher spillway crest which makes their contribution to the overall flow proportionately smaller than that from the tainter bays.

3) Energy losses from the turbulent boundary layer development at the spillway crest do not significantly affect the evaluation of hydraulic conditions. The HEC-RAS model does not have the capability to account for these losses. Although these losses could be significant, their relative magnitude compared to the 62 feet of elevation difference between the horizontal spillway apron and the spillway crest is small.

The model used a manning's n-value of 0.013 to calculate energy losses over the spillway apron. Estimates of the supercritical depth and velocity for various flow rates at the toe of the spillway (horizontal apron elevation = 802.06') were taken from the HEC-RAS model output and conjugate depths were calculated and compared to the tail-water rating curve. The maximum flow analyzed was 105,167 cfs.

The resulting conjugate depth curve is lower than the tailwater rating curve for all flows up to 43,100 cfs (upper limit of USGS rating curve), which means a hydraulic jump is expected to occur near or above the toe of the spillway for the range of flows from 0 to 43,100 cfs. The conjugate depths for higher flow rates were calculated, but the tailwater conditions are unknown.

Using the incoming velocity and depth at the horizontal apron from the HEC-RAS model, the expected length of the hydraulic jump for the range of flow rates was also determined using results from experimental data on horizontal aprons[4]. This analysis shows that the full development of the hydraulic jump will occur past the recently constructed concrete apron (55 feet past the toe of spillway) for flows greater than 5,500 cfs and will occur past the grouted rip-rap portion of the spillway apron (135 feet past the toe of the spillway) for flows greater than 28,500 cfs. It is likely that the in-place horizontal apron is not currently designed to remain stable under these flow conditions (i.e. when the hydraulic jump is not fully developed).

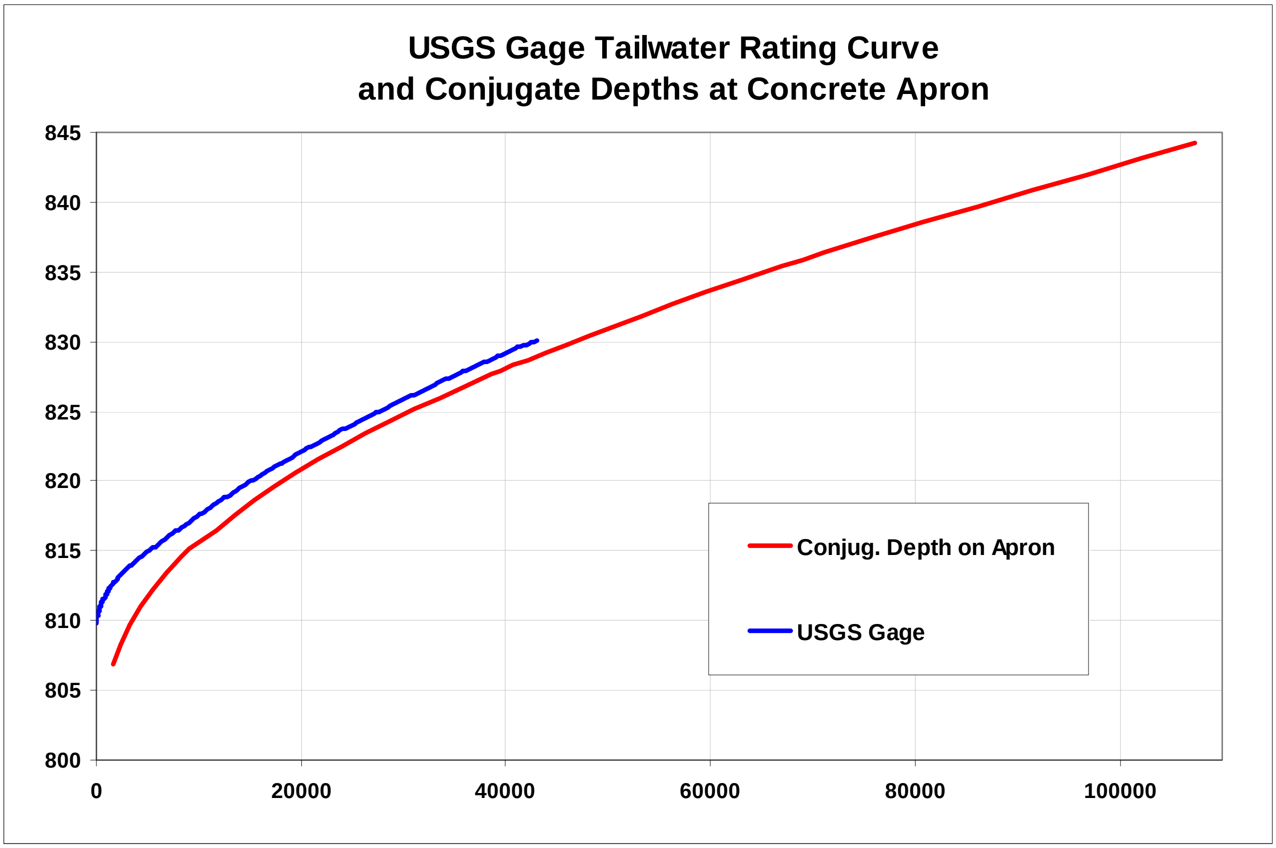


Figure 6. Conjugate depths for hydraulic jump and tailwater rating curve.

[4] Data taken from "Hydraulic Design of Stilling Basins and Energy Dissipators", US Bureau of Reclamation (1978) Section 1, Page 13, Figure 6.

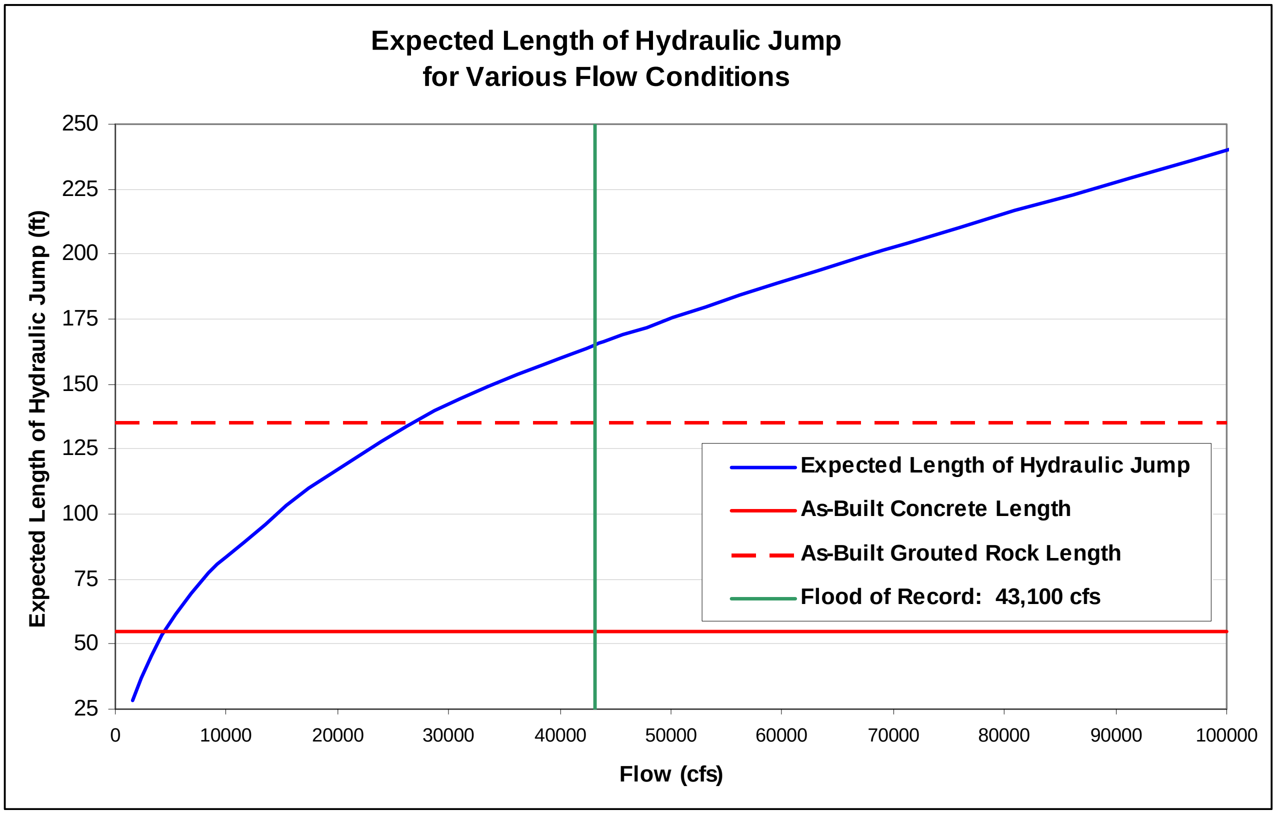


Figure 7. Expected length of hydraulic jump for a range of flows

IV. Determination of Design Flow

According to ER 1110-8-2 (FR), the design flow for the stilling basin should be selected based on the safety dam standards, which likely will result in the selection of either ½ PMF or 3’ of freeboard[5]. It is likely that the top of the service bridge would be adopted as the top of dam (elevation 881.56’). Therefore the threshold flow would likely be chosen as the flow when the energy head has 3’ of freeboard measured from the top of dam. Another possible criterion is one in which the water surface is expected to come into contact with the low chord of the service bridge, which results in a range from 77,258 cfs – 105,162 cfs depending on the method used to estimate the water surface profile through the spillway.

The required design criteria will ultimately be determined by the Federal Energy Regulatory Commission, the regulating agency for Rapidan Dam.

[5] Discussion on the selection of design flow is contained in St Paul District “Rapidan Hydrology” (July 2008).

Criteria	**Flow**	**Tailwater Condition**
½ PMF	82,000 cfs	Unknown
Threshold: 3’ energy head freeboard from Top of Dam, weir free-flow (He=878.56’)	52,517 cfs	Unknown
Threshold: Water Surface at Low Chord (WS = 877.29’)	77,258 – 105,162 cfs	Unknown
Flood of Record (1965)	43,100 cfs	830.1’

Figure 8. *Possible design flow for stilling basin design based on various criteria.*

V. Summary

The design criteria for stilling basin design will ultimately be decided by the FERC, the regulating agency for Rapidan Dam. According to Corps of Engineers standards, it is likely that ½ PMF or a threshold flow would be adopted as the design flow for stilling basin design. The spillway discharge capacity is significantly higher than previously published values if the depressed water surface elevation near the spillway crest due to high velocities is taken into account. Without collecting cross sectional surveys to establish a tailwater rating curve for flows higher than 43,100 cfs; a conceptual stilling basin design cannot be determined for higher flows.

The stilling basin, under existing conditions, has sufficient tail water depth to insure the hydraulic jump will occur at or near to the toe of the spillway. The length of the hydraulic jump, however, is expected to extend downstream of the areas that have had recent reinforcement during high flows. The jump will extend past the concrete section of the apron for flows greater that 5,500 cfs and will extend past the grouted rock section of the apron for flows greater than 28,500 cfs. The apron materials are likely not designed to be stable under the conditions of a developing hydraulic jump.

For the purposes of cost estimating, conceptual designs for stilling basins were produced based on the flood of record. This criterion was chosen only because it is the highest flow for which the tailwater conditions are known. Any further development of these conceptual designs must first determine the appropriate design flow and include a survey to estimate the tailwater conditions for all flows. The designs should then be adjusted appropriately.

This report does not provide a recommended or approved design for improvements to Rapidan Dam. In order to continue with design, the following aspects of this design must be completed:

1. Thorough study to determine the design criteria based on appropriate standard. FERC is the regulating agency for Rapidan Dam.
2. Determination of complete tailwater rating curve at the dam site. Cross-sectional surveys should be done on the river channel and floodplain downstream of the dam such that a hydraulic model could be created to determine the tailwater rating curve at the toe of the dam for the full range of flows.
3. Down-watering to determine the velocity and depth of the water at the toe of the dam should be done in a manner that accounts for the turbulent boundary layer development losses on the spillway crest.

Appendix 1:

Preliminary Determination of Parameters for Stilling Basin Design

Preliminary calculations for the conceptual design of a standard energy dissipation structure were performed for 1) a standard type II stilling basin and 2) a standard type III baffled stilling basin[6]. These calculations, although preliminary, define the conceptual geometry of energy dissipation structures that would be appropriate for Rapidan Dam for a design flow of 43,100 which is the flood of record and the highest flow for which the tailwater elevation is known. This flow does not represent a selected design flow, and was only used for preliminary and conceptual design purposes. These designs were determined using the same simplifying assumptions stated in Section III for the HEC-RAS modeling. Any further advancement of these conceptual ideas must take these assumptions into account and adjust the design appropriately. These designs were developed following the design procedure from the Bureau of Reclamation (Peterka 1978).

[6] Conceptual designs were determined using methods outlined in Bureau of Reclamation “Hydraulic Design of Stilling Basins and Energy Dissipators” and the hydraulic conditions were verified with HEC-RAS model output

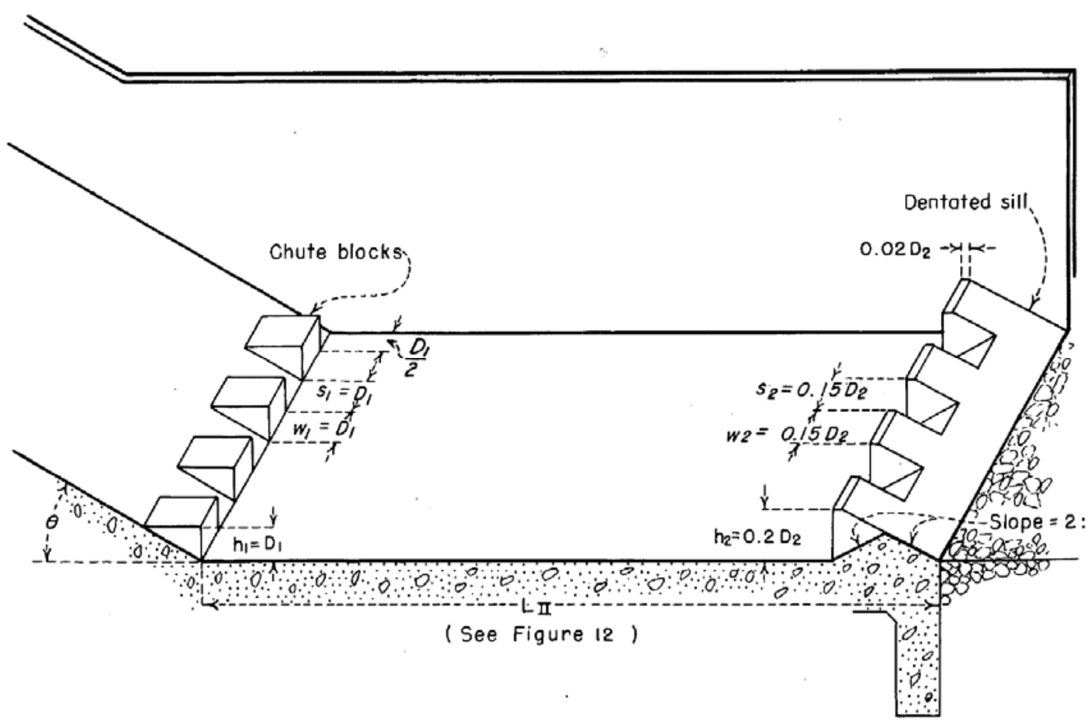
1) *Type II Stilling Basin Design*:

Design Flow: Flood of Record, Q = 43,100 cfs

Type II Stilling Basin Elevation = 802 ft

L_{II} = 113’

D_1 = 3.0’	D_2 = 6.7’
h_1 = 3.0’	h_2 = 1.3’
s_1 = 3.0’	s_2 = 1.0’
w_1 = 3.0’	w_2 = 1.0’



***See Appendix 2 for design calculations

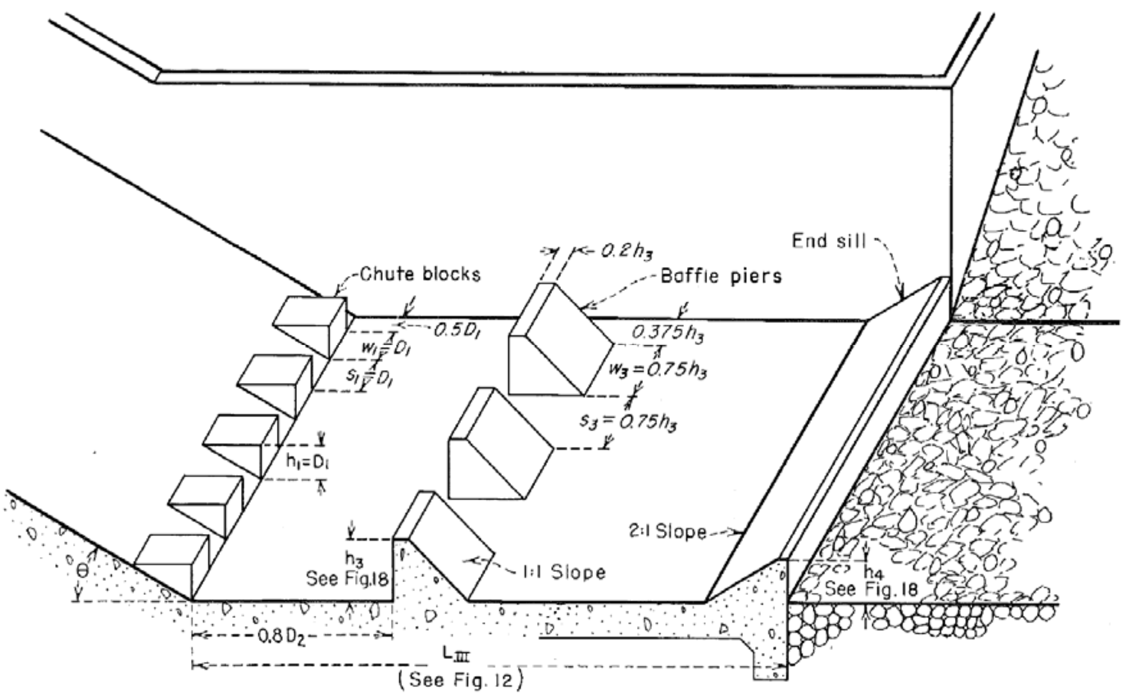
2) Type III Stilling Basin Design

Design Flow: Flood of Record, Q = 43,100 cfs

Type III Stilling Basin Elevation = 805 ft

L_{III} = 67’

D_1 = 3.1’	h_3 = 4.3’
s_1 = 3.1’	s_3 = 3.2’
w_1 = 3.1’	w_3 = 3.2’
	h_4 = 5.2’



***See Appendix 2 for design calculations

References:

Chow, Ven Te (1959), “Open Channel Hydraulics”

Federal Highway Administration (FHWA) (1986) “Bridge Waterways Analysis Model: Research Report” Report No. FHWA/RD-86/108

Federal Emergency Management Agency (FEMA) (1999) “Flood Insurance Study – Blue Earth County, Minnesota (Unincorporated Areas)”

Peterka, A. J. (1978). “Hydraulic Design of Stilling Basins and Energy Dissipators” Department of the Interior—Bureau of Reclamation

US Department of the Army, Corps of Engineers (COE) (1990), “Hydraulic Design of Spillways” Engineering Manual No. 1110-2-1603

US Department of the Army, Corps of Engineers (1991) “Inflow Design Floods for Dams and Reservoirs” ER 1110-8-2 (FR)

US Department of the Army, Corps of Engineers (COE) (2008), “HEC-RAS River Analysis System version 4.0, Hydraulic Reference Manual”

US Department of the Interior, Bureau of Reclamation (1987), “Design of Small Dams”

US Army Corps of Engineers—St Paul District (2008) “ Rapidan Hydrology”

US Army Corps of Engineers—St Paul District (2008) “Rapidan Dam Energy Dissipation, Conceptual Design Calculations”

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Rapidan Dam
Energy Dissipation

Conceptual Design Calculations

August 2008

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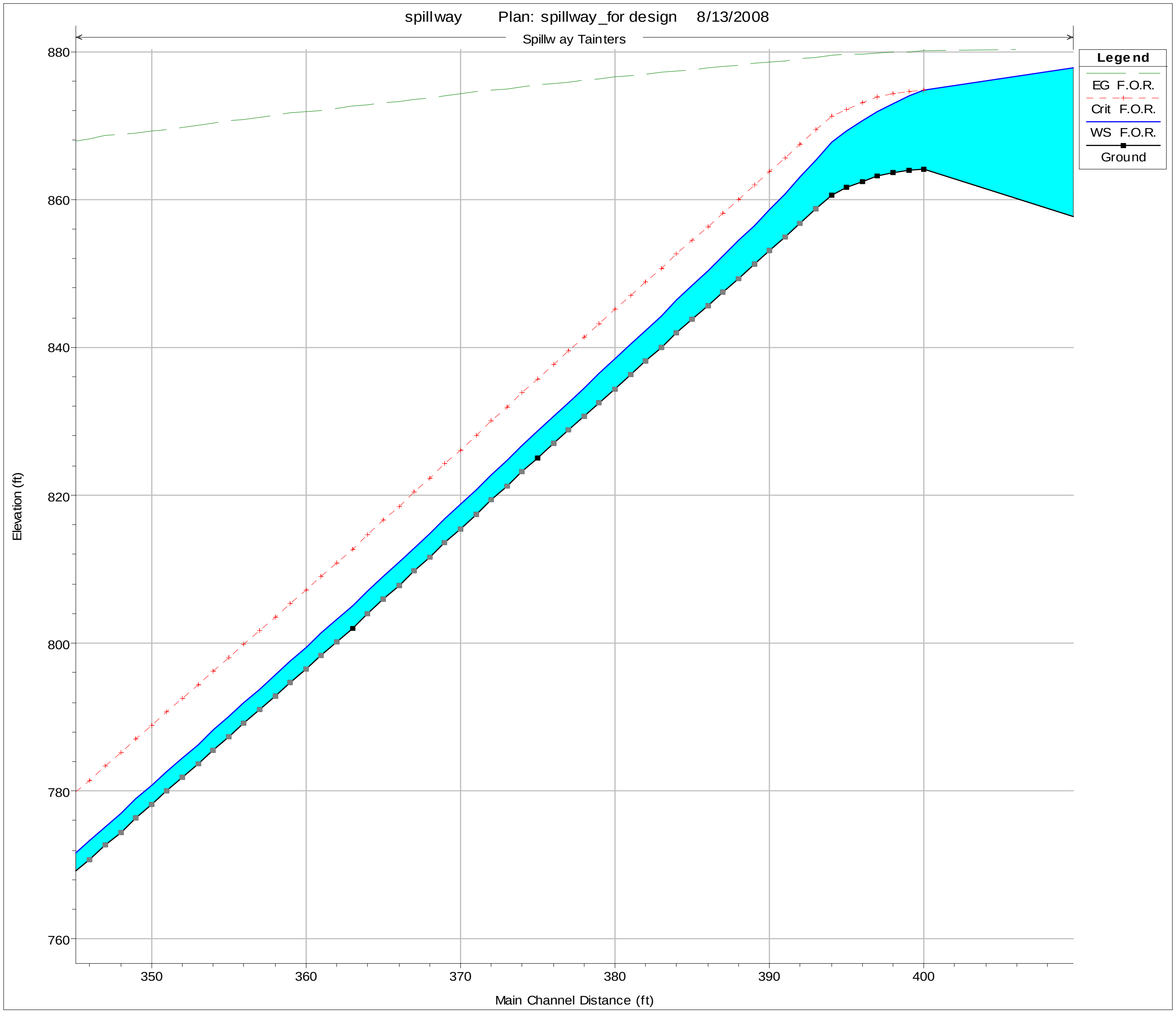
Rapidan Dam: Conceptual Design Calculations for Stilling Basin
Design Flow = 43,100 cfs

Schematic of HEC-RAS Model Results



Rapidan Dam: Conceptual Design Calculations for Stilling Basin
Design Flow = 43,100 cfs

RAS Downwatering Output						Type II Basin Design			Type III Basin Design		
Station Dist. D/S of crest (ft)	Min Ch El (ft)	W.S. Elev (ft)	Vel Chnl (ft/s) V_1	Hydr Depth (ft) D_1	Froude #	D_2	Min. TW Depth TW Elev = 830.1		$0.85\ D_2$	Min TW Depth TW Elev = 830.1	
	854.06	879.58	7.75	25.51	0.27						
	854.06	879.57	7.75	25.51	0.27						
0	864.06	874.74	18.51	10.68	1						
1	863.93	873.98	19.68	10.05	1.09						
2	863.6	872.95	21.16	9.35	1.22						
3	863.09	871.81	22.67	8.72	1.35						
4	862.41	870.58	24.21	8.17	1.49						
5	861.56	869.23	25.78	7.67	1.64						
6	860.56	867.79	27.37	7.23	1.79						
7	858.69	865.3	29.9	6.61	2.05						
8	856.82	862.98	32.09	6.16	2.28						
9	854.95	860.76	34.05	5.81	2.49						
10	853.09	858.61	35.84	5.52	2.69						
11	851.22	856.49	37.52	5.27	2.88						
12	849.35	854.41	39.1	5.06	3.06						
13	847.48	852.35	40.61	4.87	3.24						
14	845.61	850.31	42.04	4.7	3.42						
15	843.74	848.3	43.4	4.56	3.58						
16	841.88	846.3	44.71	4.42	3.75						
17	840.01	844.31	45.98	4.3	3.91						
18	838.14	842.33	47.21	4.19	4.06						
19	836.27	840.36	48.4	4.09	4.22						
20	834.4	838.39	49.55	3.99	4.37						
21	832.53	836.43	50.67	3.9	4.52						
22	830.67	834.49	51.76	3.82	4.67						
23	828.8	832.54	52.83	3.74	4.81						
24	826.93	830.6	53.86	3.67	4.95						
25	825.06	828.66	54.88	3.6	5.1						
26	823.14	826.68	55.91	3.54	5.24						
27	821.23	824.71	56.9	3.48	5.38						
28	819.31	822.73	57.9	3.42	5.52	25.0	10.8		21.3	10.8	
29	817.39	820.75	58.85	3.36	5.66	25.3	12.7		21.5	12.7	
30	815.48	818.79	59.79	3.31	5.79	25.5	14.6		21.7	14.6	
31	813.56	816.82	60.69	3.26	5.93	25.7	16.5		21.9	16.5	
32	811.64	814.85	61.59	3.21	6.06	25.9	18.5		22.1	18.5	
33	809.73	812.89	62.49	3.16	6.19	26.1	20.4		22.2	20.4	
34	807.81	810.93	63.37	3.12	6.32	26.4	22.3		22.4	22.3	
35	805.89	808.97	64.24	3.08	6.45	26.6	24.2		22.6	**24.2**	****Min TW is greater than Conj. Depth
36	803.98	807.02	65.1	3.04	6.58	26.8	26.1		22.8	26.1	Basing Elevation = 805'
37	802.06	805.06	65.96	3	6.71	27.0	**28.0**	****Min TW is greater than Conj. Depth	23.0	28.0	
38	800.22	803.18	66.73	2.96	6.83	27.2	29.9	Basin Elevation = 802'	23.1	29.9	**From Figure 12:**
39	798.38	801.31	67.5	2.93	6.95	27.4	31.7		23.3	31.7	L/D_2 = 2.5
40	796.54	799.44	68.28	2.9	7.07	27.6	33.6	**From Figure 12:**	23.4	33.6	L= 67
41	794.7	797.56	69.07	2.86	7.19	27.7	35.4	L/D_2 = 4.2	23.6	35.4	
42	792.86	795.69	69.84	2.83	7.31	27.9	37.2	L= 113	23.7	37.2	**From Figure 18:**
43	791.02	793.82	70.57	2.8	7.43	28.1	39.1		23.9	39.1	h_3/d_1= 1.4
44	789.18	791.96	71.26	2.77	7.54	28.2	40.9		24.0	40.9	h_3= 4.3
45	787.34	790.09	72	2.75	7.66	28.4	42.8		24.2	42.8	h_4/d_1= 1.7
46	785.5	788.22	72.71	2.72	7.77	28.6	44.6		24.3	44.6	h_4= 5.2
47	783.66	786.36	73.37	2.7	7.88	28.7	46.4		24.4	46.4	

Rapidan Dam Investigations
Blue Earth River Feasibility Study
Ecosystem Restoration

Part D
Stilling Basin Conceptual Structural Design

September 2009

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Section 1 Structural Design

1.1 General

The purpose for this structural design was to create a feasibility level design that could be used for cost estimating purposes. A preliminary hydraulic assessment of the spillway capacity was done in August 2008. The information contained in this report was used as the bases for the parameters of the structure.

1.2 Design Criteria

1.2.1 References

1. Rapidan Dam: Hydraulic Assessment of Spillway Capacity & Energy Dissipation Characteristics, USACE, August 2008.
2. EM 1110-2-2104, Strength Design of Reinforced Concrete Hydraulic Structures
3. EM 1110-2-2502, Retailing and Floodwalls

1.2.2 Materials properties

1. Sheetpile: Allowable stresses for permanent sheetpile will not be more than 50% of the yield strength in accordance with EM 1110-2-2504. Minimum thickness for corrosion control shall be 0.375 inches.
2. Concrete T-Wall: Normal weight concrete will be used, 4500 psi
3. Concrete Reinforcement: ASTM A615, Grade 60 deformed steel bars

1.2.3 Geotechnical and Material Weights

1. Water 62.5 pcf
2. Poorly cemented sandstone
    a. Gamma 130 pcf,
    b. Phi 30 degrees
    c. Cohesion 200 psf
3. Concrete 150 pcf

1.2.4 Hydraulic Information

1. Head water Elevation 873.1 ft
2. Tailwater Elevation 830.16 ft
3. Hydraulic Jump – Unknown at this time

1.2.5 Assumptions

1. Wall Elevations: A site layout has not been done at this site. Normally you could assume take the walls as the greater between the tailwater + 5' or the ground surface. Based on pictures of the site, it appears that the ground surface elevation is near the top of the dam around elevation 881.5. As a conservative assumption, the wall elevations were determined by taking the

difference between the head water and tailwater elevation and adding 5' of freeboard. To determine the elevation this was added to the stilling basin elevation of 802' for a final elevation for the top of walls at 850'. If additional surveys and site layout work was done, this height could be reduced. Additionally, it was assumed that there would be backfill on the backsides of the walls to exclude the tailwater. One possible design alternative could be to look at a case where the stilling basin is flooded and there is water on both sides of the walls and there is no backfill.

2. Soil Profile: The soil profile was conservatively taken as 5' below the top of the wall. The profile would then transition down to tie into the banks on either side. For the purpose of these calculations, the soil was assumed to be at 845'. This profile would need to be refined with some site layout work.
3. Uplift pressure: Uplift pressure can be computed from seepage analysis. More refined seepage analysis will need to be done to assume a smaller uplift pressure. At this time, the uplift pressure was assumed using the full hydrostatic head from the headwater. If this were not reduced, the base would need to be 27' thick to meet the floatation criteria. It was assumed that the headwater could be reduced 65% to 845. Without knowing the exact location of the stilling basin or the site constraints, it was not possible to do a line of creep analysis to determine what head should be used. By using 845, the base slab was determined to be 12.5' thick based on floatation. With more refined seepage analysis this thickness could be reduced. Additionally, drains could be used to reduce the uplift further.
4. The slab would be controlled by floatation.
5. The stilling basin would be designed using type III as shown in the Rapidan Dam: Hydraulic Assessment of Spillway Capacity & Energy Dissipation Characteristics, August 2008.
6. This design assumes the use of a T-Wall on either side of the stilling basin slab. This configuration could also be changed to a U-Shaped structure. If this were done, most of the mass of the structure that was designed here would still be used for the U-Shaped structure. The difference would be the layout of the concrete.

1.3 Design Loads

Only one design load case was looked at for the purpose of this analysis: Headwater elevation at 873.1' and tailwater at elevation 830.16.

1. Lateral Loads – The lateral loads were comprised of the soil loads plus the water loads.
2. Vertical Loads – The weight of the soil, water, and concrete were the primary vertical loads for all load cases. Additional vertical loads are discussed separately.
3. Uplift – This load assumes headwater at EL 845 hydrostatic head.

1.4 Design Methodology, T-Walls

1.4.1 Foundation Loads

The foundation loads that were used to design the pile layout were calculated using a MathCAD sheet. These sheets calculated all of the loads for the foundation that were used to determine if the bearing pressure and the base compression checks could be satisfied. It was determined that sliding check for the walls did not need to be satisfied because the walls would be pushing against the stilling basin slab and ultimately against each other. The calculations are included in Appendix B.

1.4.2 Sheetpile

A sheetpile has not been used in this design

1.4.3 Reinforcement Calculations

Reinforcement was not looked at for this stage of the design.

1.5 Base Slab

The base slab was designed assuming that floatation from full head would control the design. The calculation for this is included in Appendix B.

1.6 Summary of Analysis and Design

As shown in the sketches in Appendix A, the stilling basing consists of two T-walls on either side of the stilling basin slab. The slab was reduced in width by the toe length of the T-walls. The walls have a base width of 60’, a total height of 53’, and a length of 72’. For additional dimension of the walls, refer to the page 1 of the calculations.

It was calculated that the base slab would need to be 12.18’. This was rounded up to a thickness of 12.5’. The width of the slab is 238’ and the length is 67’.

Additional features, such as the baffle piers, chute blocks, and end sill were determined by the hydraulic engineer.

Appendix A Sketches

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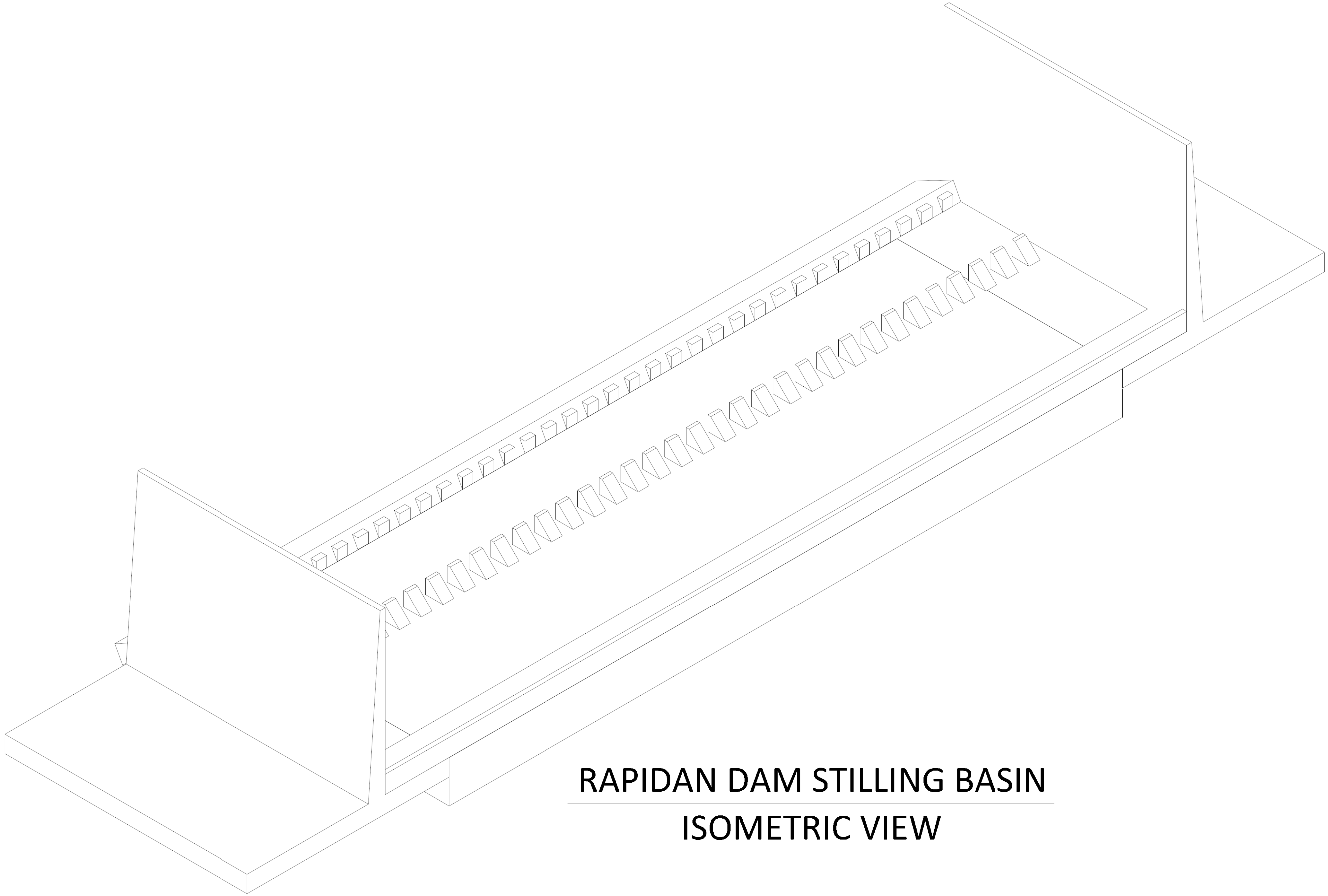
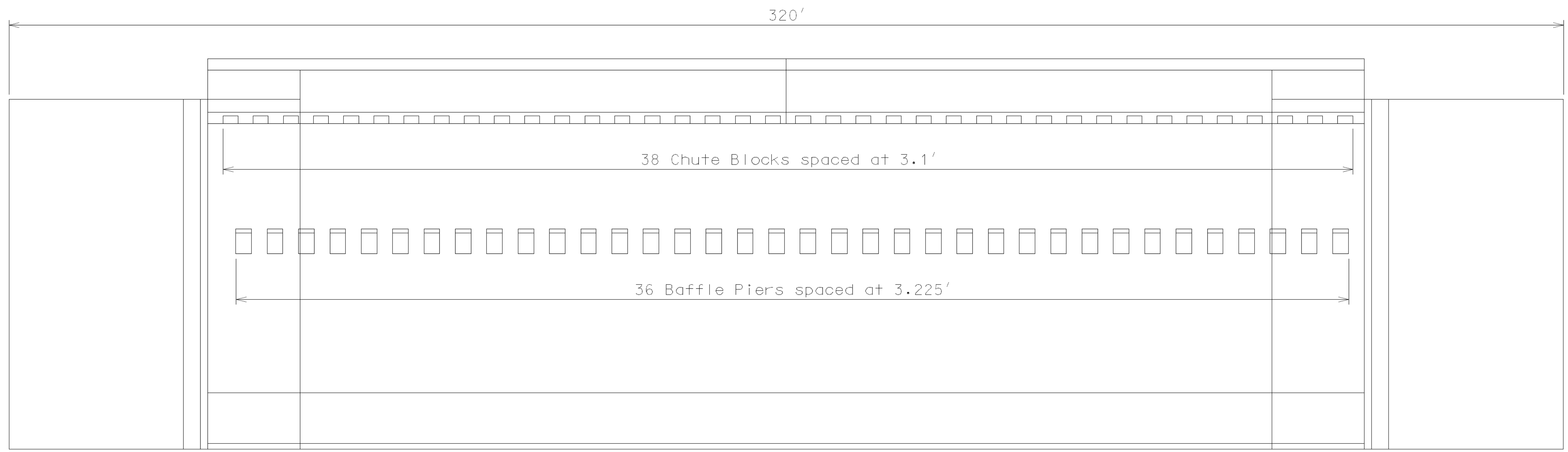


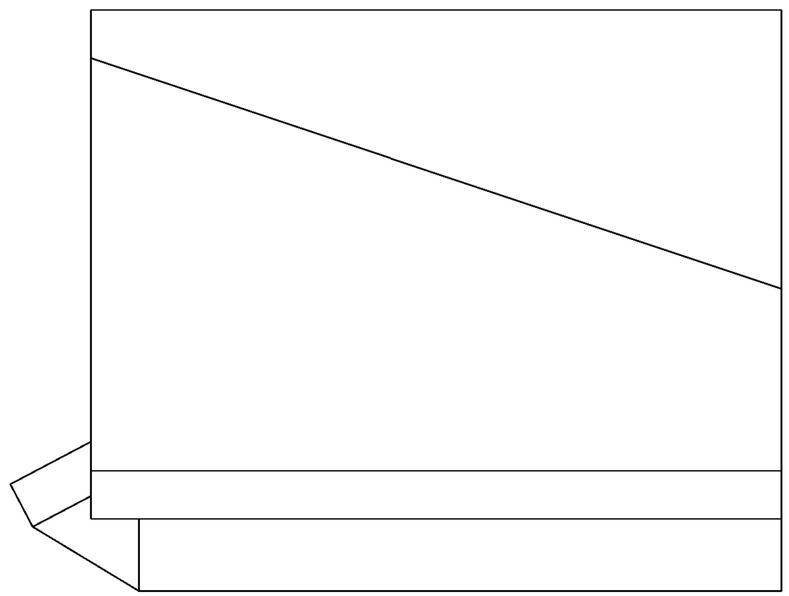
RAPIDAN DAM STILLING BASIN

ISOMETRIC VIEW



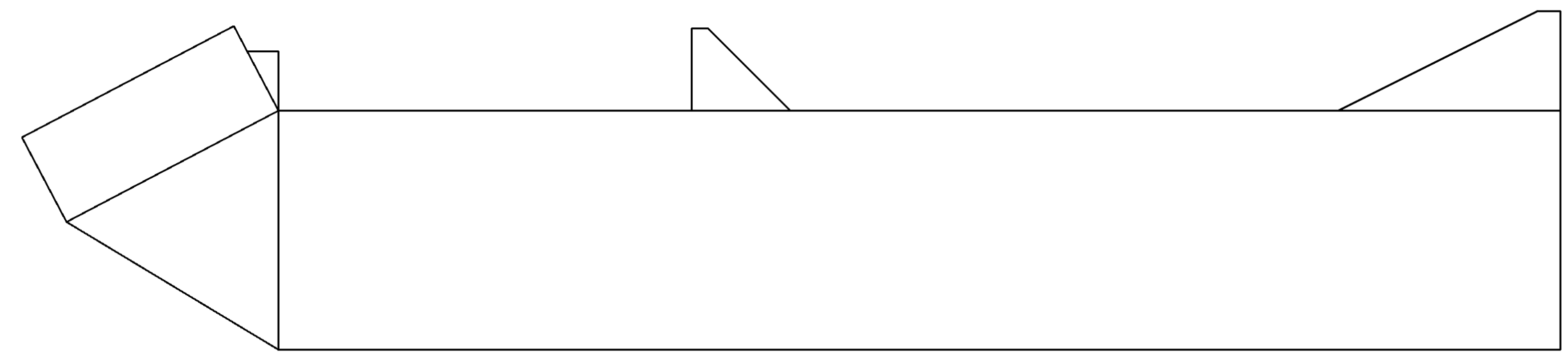
STILLING BASIN
TOP VIEW

STILLING BASIN
FRONT VIEW



ELEVATION VIEW

LEFT SIDE WITH SOIL PROFILE



ELEVATION

INSIDE OF WALLS

STILLING BASIN

FRONT VIEW (BEHIND END SILL)

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Appendix B Calculations

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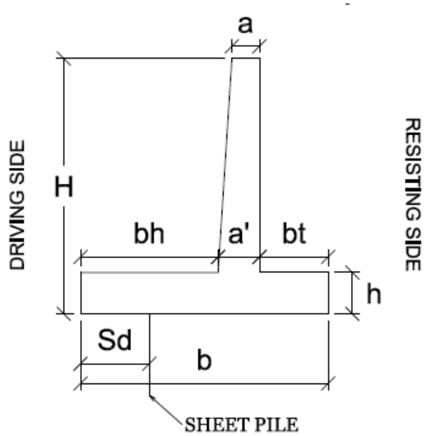
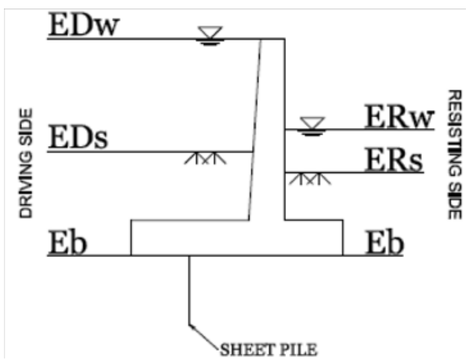
Retaining T-WALL Design





Geometry Input Parameters

Base Width $b := 60ft$

Base Height $h := 5ft$

Width of Stem at top $a := 1.5ft$

Width of Stem at base $a' := 5ft$

Width of toe $b_t := 19ft$

Distance from heel to Sheet Pile ($Sd < b_h$) $Sd := 60ft$

Monolith Length $ML := 72 \cdot ft$

Width of Heel

$b_h := b - b_t - a'$ $b_h = 36\,ft$

Elevation Input Parameters

EL Top of Wall $ED_T := 850ft$

EL Driving Water $ED_W := 845ft$

EL Driving Soil $ED_S := 845ft$

EL Base $E_b := 802ft - h$

EL Resisting Water $ER_W := 830.16ft$

EL Resisting Soil $ER_S := 792ft$

Soil and water properties

Moist unit weight of soil $\gamma := 130pcf$

Saturated unit weight $\gamma_{sat} := 130pcf$

Friction Angle $\phi := 30deg$

Cohesion $c := 200psf$

At rest coefficient $K_0 := 1 - \sin(\phi)$

Unit weight of water $\gamma_w := 62.5pcf$

Site Info. (Defined, Ordinary, Limited) $Site := \text{"Limited"}$

Base Tilt Angle $\alpha := 0rad$

Ground Slope Angle $\beta := 0rad$

Effectiveness of sheet pile (0 or 100) $SP_{eff} := 0\%$

Buoyant Soil Weight

$\gamma_b := \gamma_{sat} - \gamma_w$ $\gamma_b = 67.5 \cdot pcf$

Concrete Properties

Unit weight of concrete $w_c := 150pcf$

Concrete Strength $f_{c'} := 4500psi$

Reinforcement Strength $f_y := 60000psi$

Additional Information

Surcharge Load (Positive = Driving Surcharge) $P := 0plf$

Wind Load Positive = Driving Wind $Wind := 0psf$

Case type (Usual, Unusual, Ext ream) $Case := \text{"Usual"}$

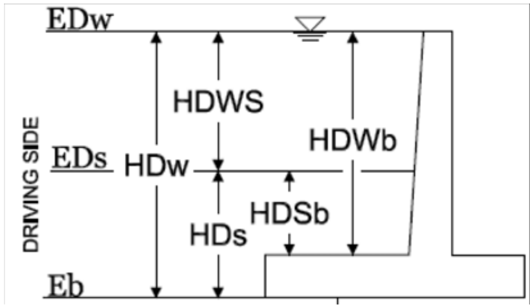
Reduction factor (unusual or extreme cases only)

$$R_f := \begin{vmatrix} 1 & \text{if } Case = \text{"Usual"} \\ .75 & \text{otherwise} \end{vmatrix}$$

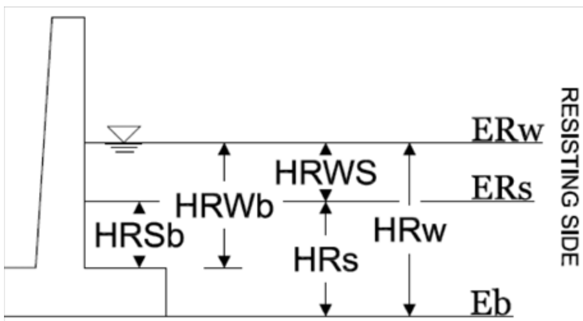
$R_f = 1$

Total Height of wall $H := ED_T - E_b$ $H = 53\,ft$

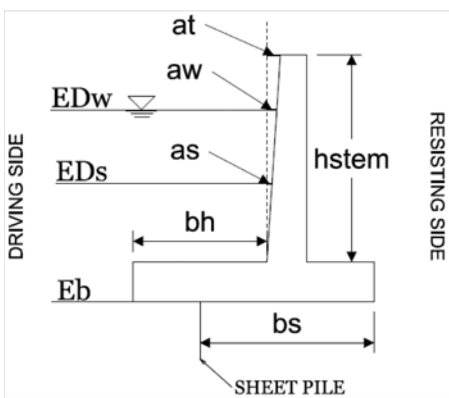
Calculated Driving Side Heights



Calculated Resisting Side Heights



Calculated T-Wall Geometry



Height Driving Water	$HD_W := \max\left[\left(ED_W - E_b\right), 0\right]$	$HD_W = 48\ ft$
Height Driving Water above Soil	$HD_{WS} := \max\left[\left(ED_W - ED_S\right), 0\right]$	$HD_{WS} = 0\ ft$
Height Driving Soil	$HD_S := \max\left[\left(ED_S - E_b\right), 0\right]$	$HD_S = 48\ ft$
Height Driving Soil above base	$HD_{Sb} := \max\left[\left(ED_S - E_b - h\right), 0\right]$	$HD_{Sb} = 43\ ft$
Height Driving Water above base	$HD_{Wb} := \max\left[\left(ED_W - E_b - h\right), 0\right]$	$HD_{Wb} = 43\ ft$
Height Driving Dry Soil	$HD_{Dry} := \max\left(HD_S - HD_W, 0\right)$	$HD_{Dry} = 0\ ft$
Height Driving Wet Soil	$HD_{Wet} := HD_S - HD_{Dry}$	$HD_{Wet} = 48\ ft$
Height Resisting Water	$HR_W := \max\left[\left(ER_W - E_b\right), 0\right]$	$HR_W = 33.16\ ft$
Height Resisting Water above Soil	$HR_{WS} := \max\left[\left(ER_W - ER_S\right), 0\right]$	$HR_{WS} = 38.16\ ft$
Height Resisting Soil	$HR_S := \max\left[\left(ER_S - E_b\right), 0\right]$	$HR_S = 0\ ft$
Height Resisting Water above base	$HR_{Wb} := \max\left[\left(ER_W - E_b - h\right), 0\right]$	$HR_{Wb} = 28.16\ ft$
Height Resisting Soil above base	$HR_{Sb} := \max\left[\left(ER_S - E_b - h\right), 0\right]$	$HR_{Sb} = 0\ ft$
Height Resisting Dry Soil	$HR_{Dry} := \max\left(HR_S - HR_W, 0\right)$	$HR_{Dry} = 0\ ft$
Height Resisting Wet Soil	$HR_{Wet} := HR_S - HR_{Dry}$	$HR_{Wet} = 0\ ft$
Stem Height (h_{stem})	$h_{stem} := H - h$	$h_{stem} = 48\ ft$
Additional Stem Thickness (a_t)	$a_t := a' - a$	$a_t = 3.5\ ft$
Additional Water Thickness (a_w)	$a_w := HD_{Wb} \cdot \dfrac{a_t}{h_{stem}}$	$a_w = 3.14\ ft$
Additional Soil Thickness (a_s)	$a_s := HD_{Sb} \cdot \dfrac{a_t}{h_{stem}}$	$a_s = 3.14\ ft$
With of base on Resisting side of the Sheet pile	$b_s := b - Sd$	$b_s = 0\ ft$

Quantity of Horizontal concrete

$Q_{HC} := ML \cdot (b \cdot h)$ $\quad Q_{HC} = 800.00 \cdot yd^3$

Quantity of Vertical concrete

$Q_{VC} := ML \cdot \left[a \cdot h_{stem} + \frac{1}{2} \cdot (a' - a) \cdot h_{stem}\right]$ $\quad Q_{VC} = 416.00 \cdot yd^3$

Height Driving Dry Soil above base $\quad HD_{Dryb} := \max(HD_{Sb} - HD_{Wb}, 0)$ $\quad HD_{Dryb} = 0\,ft$

Height Driving Wet Soil above base $\quad HD_{Wetb} := HD_{Sb} - HD_{Dryb}$ $\quad HD_{Wetb} = 43\,ft$

Height Resist Dry Soil above base $\quad HR_{Dryb} := \max(HR_{Sb} - HR_{Wb}, 0)$ $\quad HR_{Dryb} = 0\,ft$

Height Resist Wet Soil above base $\quad HR_{Wetb} := HR_{Sb} - HR_{Dryb}$ $\quad HR_{Wet} = 0\,ft$

Uplift Forces

Figure used when SPeff = 0%

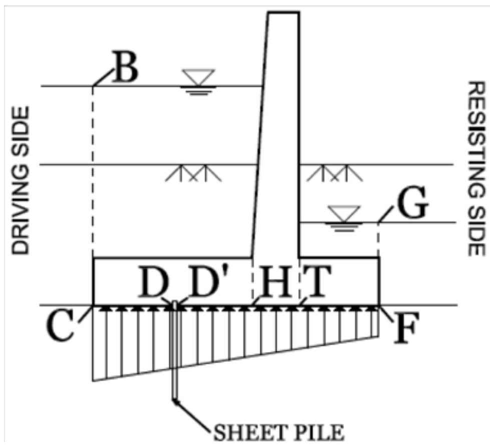
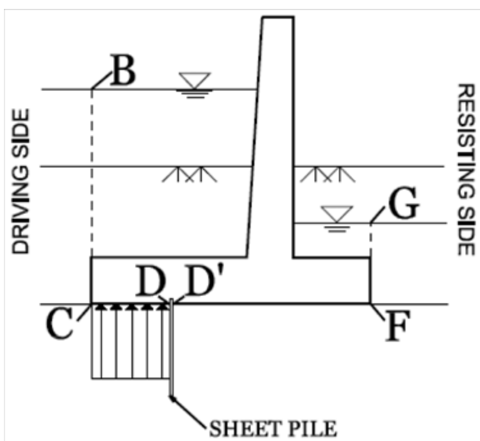


Figure used when SPeff = 100%



EM 1110-2-2502 - Section 3-19 (SPeff = 0%) and pg 7-6 (SPeff > 0%)

The effectiveness of the sheet pile cutoff can range from 0 - 50% per the requirement in EM 1110-2-2502 (pg 7-6), which states that the pressure on the protected side of the cutoff should equal the pressure at point C (see figure below) reduced by up to 50 percent of the difference between the full head value on the unprotected side and the pressure head at the end of the toe of the wall. When the effectiveness of the sheet pile cutoff is 0, the uplift pressure varies uniformly as though a cutoff was not present and the uplift is calculated as though no sheet pile is present.

In these calculations, the sheet pile effectiveness was taken as 0% and 100% effective. Although the effectiveness cannot be taken greater than 50% per the requirements of EM 1110-2-2502, it is more conservative to do so.

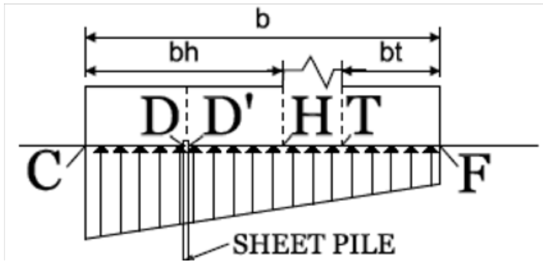
Variables used in calculations

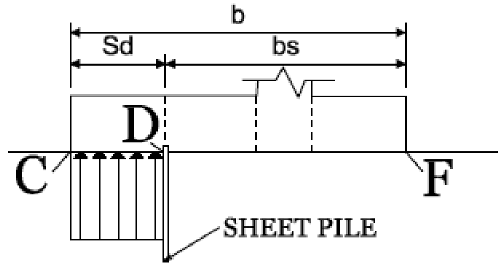
$HD_W = 48\,ft$ $\quad HR_W = 33.16\,ft$ $\quad b = 60\,ft$ $\quad b_s = 0\,ft$ $\quad SP_{eff} = 0$

Distance BC shown in the figures to the left is equal to HDw
Distance GF shown in the figures to the left is equal to HRw

Seepage Path $\quad Seep := HD_W + b + HR_W$ $\quad Seep = 141.16\,ft$

Initial Head $\quad H_0 := HD_W - HR_W$ $\quad H_0 = 14.84\,ft$





Variables used in calculations

$HD_W = 48\,ft$ $HR_W = 33.16\,ft$ $b = 60\,ft$ $b_s = 0\,ft$

$SP_{eff} = 0$ $Seep = 141.16\,ft$ $H_0 = 14.84\,ft$ $b_t = 19\,ft$

$\gamma_w = 62.5 \cdot pcf$ $Sd = 60\,ft$ $a' = 5\,ft$

Pressure Head **Water Pressure**

AT **C**

$$PH_C := \begin{cases} HD_W - \frac{H_0}{Seep} \cdot HD_W & \text{if } SP_{eff} = 100 \\ HD_W & \text{otherwise} \end{cases} \qquad PH_C = 48\,ft \qquad P_C := PH_C \cdot \gamma_w \qquad P_C = 3000 \cdot psf$$

AT **F**

$$PH_F := \begin{cases} HD_W - \frac{H_0}{Seep} \cdot (HD_W + b) & \text{if } SP_{eff} = 0 \\ 0 & \text{otherwise} \end{cases} \qquad PH_F = 36.65\,ft \qquad P_F := PH_F \cdot \gamma_w \qquad P_F = 2290.38 \cdot psf$$

AT **D**

$$PH_D := \begin{cases} HD_W - \frac{H_0}{Seep} \cdot (HD_W + Sd) & \text{if } SP_{eff} = 0 \\ PH_C & \text{otherwise} \end{cases} \qquad PH_D = 36.65\,ft \qquad P_D := PH_D \cdot \gamma_w \qquad P_D = 2290.38 \cdot psf$$

AT **D'**

$$PH_{D'} := \begin{cases} PH_D & \text{if } SP_{eff} = 0 \\ 0 & \text{otherwise} \end{cases} \qquad PH_{D'} = 36.65\,ft \qquad P_{D'} := PH_{D'} \cdot \gamma_w \qquad P_{D'} = 2290.38 \cdot psf$$

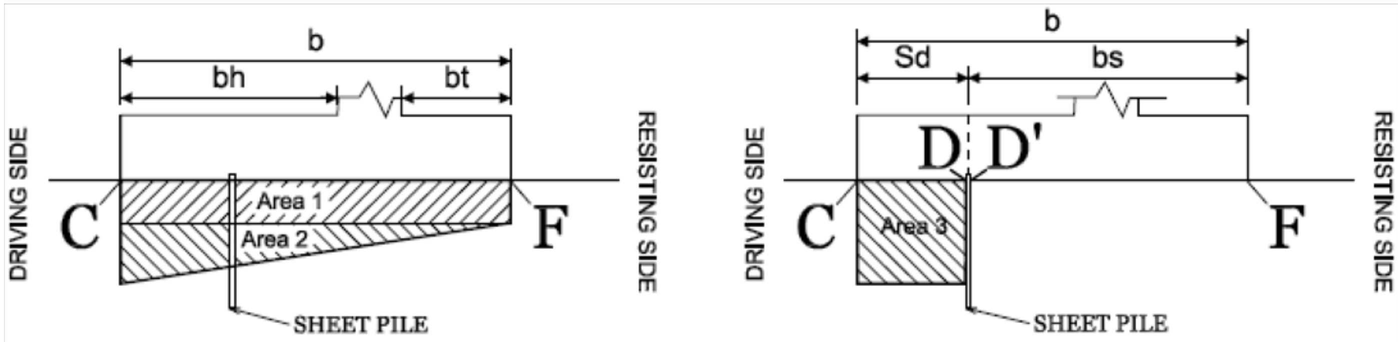
Water Pressure at points H and T are calculated here for use in concrete design. These points are calculated using similar triangles.

AT **H**

$$PH_H := \frac{PH_{D'} - PH_F}{b_s} \cdot (b_t + a') + PH_F \qquad PH_H = 36.65\,ft \qquad P_H := PH_H \cdot \gamma_w \qquad P_H = 2290.38 \cdot psf$$

AT **T**

$$PH_T := \left(\frac{PH_{D'} - PH_F}{b_s}\right) \cdot b_t + PH_F \qquad PH_T = 36.65\,ft \qquad P_T := PH_T \cdot \gamma_w \qquad P_T = 2290.38 \cdot psf$$



Assume axes runs along the heel at point c "into the board". The moment at the heel is positive using the right hand rule, therefore, how the picture is drawn, the moment is positive clockwise.

Variables used in calculations

$P_C = 3000 \cdot psf$ $P_D = 2290.38 \cdot psf$ $P_{D'} = 2290.38 \cdot psf$ $P_F = 2290.38 \cdot psf$ $SP_{eff} = 0$ $b = 60\,ft$ $Sd = 60\,ft$ $b_s = 0\,ft$

	Uplift Forces (in 1' section)		Arm From Heel (C)		Moment at Heel
Uplift Area 1	$U1 := \begin{cases} -P_F \cdot b \cdot 1ft & \text{if } SP_{eff} = 0 \\ 0 & \text{if } SP_{eff} > 0 \end{cases}$	$U1 = -137.42 \cdot kip$	$A_{U1} := \frac{b}{2}$	$A_{U1} = 30\,ft$	$M_{U1} := U1 \cdot A_{U1}$ $M_{U1} = -4122.68 \cdot ft \cdot kip$
Uplift Area 2	$U2 := \begin{cases} \frac{-1}{2} \cdot (P_C - P_F) \cdot b \cdot 1ft & \text{if } SP_{eff} = 0 \\ 0 & \text{if } SP_{eff} > 0 \end{cases}$	$U2 = -21.29 \cdot kip$	$A_{U2} := \frac{b}{3}$	$A_{U2} = 20\,ft$	$M_{U2} := U2 \cdot A_{U2}$ $M_{U2} = -425.77 \cdot ft \cdot kip$
Uplift Area 3	$U3 := \begin{cases} 0 & \text{if } SP_{eff} = 0 \\ -P_C \cdot Sd \cdot 1ft & \text{if } SP_{eff} > 0 \end{cases}$	$U3 = 0 \cdot kip$	$A_{U3} := \frac{Sd}{2}$	$A_{U3} = 30\,ft$	$M_{U3} := U3 \cdot A_{U3}$ $M_{U3} = 0 \cdot ft \cdot kip$
Uplift Total	$U := U1 + U2 + U3$	$U = -158.71 \cdot kip$			$M_U := M_{U1} + M_{U2} + M_{U3}$ $M_U = -4548.46 \cdot ft \cdot kip$
Location	$L_U := \begin{cases} 0 & \text{if } U = 0 \\ \frac{M_U}{U} & \text{otherwise} \end{cases}$			$L_U = 28.66 \cdot ft$	

Weight of Concrete, Soil, and Water

Variables used in calculations

$w_c = 150 \cdot pcf$	$\gamma = 130 \cdot pcf$
$\gamma_w = 62.5 \cdot pcf$	$\gamma_{sat} = 130 \cdot pcf$
$b = 60\,ft$	
$b_h = 36\,ft$	$b_t = 19\,ft$
$h_{stem} = 48\,ft$	$h = 5\,ft$
$a = 1.5\,ft$	$a_t = 3.5\,ft$
$a' = 5\,ft$	$a_w = 3.14\,ft$
$HD_{Wb} = 43\,ft$	$HR_{Wb} = 28.16\,ft$
$HD_{Sb} = 43\,ft$	$HR_{Sb} = 0\,ft$
$HD_{Wetb} = 43\,ft$	$HR_{Wetb} = 0\,ft$
$HD_{Dryb} = 0\,ft$	$HR_{Dryb} = 0\,ft$

	Weight (in 1' section)	Arm From Heel (C)	Moment at Heel
Concrete Area 1 (Base)	$C1 := w_c \cdot (b \cdot h) \cdot 1ft$	$A_{C1} := \frac{b}{2}$	$M_{C1} := C1 \cdot A_{C1}$
	$C1 = 45 \cdot kip$	$A_{C1} = 30\,ft$	$M_{C1} = 1350 \cdot ft \cdot kip$
Concrete Area 2 (Stem w/o taper)	$C2 := w_c \cdot (a \cdot h_{stem}) \cdot 1ft$	$A_{C2} := b_h + a_t + \frac{a}{2}$	$M_{C2} := C2 \cdot A_{C2}$
	$C2 = 10.8 \cdot kip$	$A_{C2} = 40.25\,ft$	$M_{C2} = 434.7 \cdot ft \cdot kip$
Concrete Area 3 (Stem taper)	$C3 := w_c \cdot \left(\frac{1}{2} \cdot a_t \cdot h_{stem}\right) \cdot 1ft$	$A_{C3} := b_h + \frac{2}{3} \cdot a_t$	$M_{C3} := C3 \cdot A_{C3}$
	$C3 = 12.6 \cdot kip$	$A_{C3} = 38.33\,ft$	$M_{C3} = 483 \cdot ft \cdot kip$
Water Area 1 (Resisting Side)	$W1 := \gamma_w \cdot (b_t \cdot HR_{Wb}) \cdot 1ft$	$A_{W1} := b_h + a' + \frac{b_t}{2}$	$M_{W1} := W1 \cdot A_{W1}$
	$W1 = 33.44 \cdot kip$	$A_{W1} = 50.5\,ft$	$M_{W1} = 1688.72 \cdot ft \cdot kip$
Water Area 2 (Driving Side)	$W2 := \gamma_w \cdot (b_h \cdot HD_{Wb}) \cdot 1ft$	$A_{W2} := \frac{b_h}{2}$	$M_{W2} := W2 \cdot A_{W2}$
	$W2 = 96.75 \cdot kip$	$A_{W2} = 18\,ft$	$M_{W2} = 1741.5 \cdot ft \cdot kip$
Water Area 3 (Stem taper)	$W3 := \gamma_w \cdot \left(\frac{1}{2} a_w \cdot HD_{Wb}\right) \cdot 1ft$	$A_{W3} := b_h + \frac{a_w}{3}$	$M_{W3} := W3 \cdot A_{W3}$
	$W3 = 4.21 \cdot kip$	$A_{W3} = 37.05\,ft$	$M_{W3} = 156.08 \cdot ft \cdot kip$

Soil Area 1 (Resisting Side)

$$S1 := \begin{cases} \gamma_{sat} \cdot (b_t \cdot HR_{Wetb}) \cdot 1ft & \text{if } HR_{Wb} \ge HR_{Sb} \\ \gamma \cdot (b_t \cdot HR_{Dryb}) \cdot 1ft & \text{if } HR_{Wb} = 0 \\ \left[\gamma_{sat} \cdot (b_t \cdot HR_{Wetb}) + \gamma \cdot (b_t \cdot HR_{Dry})\right] \cdot 1ft & \text{if } HR_{Sb} \ge HR_{Wb} \wedge HR_{Wb} \ne 0 \end{cases}$$

$A_{S1} := b_h + a' + \frac{b_t}{2}$ $\quad M_{S1} := S1 \cdot A_{S1}$

$S1 = 0 \cdot kip$ $\quad A_{S1} = 50.5\,ft$ $\quad M_{S1} = 0 \cdot ft \cdot kip$

Weight (in 1' section)	Arm From Heel (C)	Moment at Heel

Soil Area 2 (Driving Side)

$$S2 := \begin{cases} \gamma_{sat}\cdot(b_h\cdot HD_{Wetb})\cdot 1ft & \text{if } HD_{Wb} \geq HD_{Sb} \\ \gamma\cdot(b_h\cdot HD_{Dryb})\cdot 1ft & \text{if } HD_{Wb} = 0 \\ \left[\gamma_{sat}\cdot(b_h\cdot HD_{Wetb}) + \gamma\cdot(b_h\cdot HD_{Dry})\right]\cdot 1ft & \text{if } HD_{Sb} \geq HD_{Wb} \wedge HD_{Wb} \neq 0 \end{cases}$$

$A_{S2} := \frac{b_h}{2}$

$M_{S2} := S2\cdot A_{S2}$

$S2 = 201.24\cdot kip$ $A_{S2} = 18\,ft$ $M_{S2} = 3622.32\cdot ft\cdot kip$

Soil Area 3 (Stem taper)

$$S3 := \begin{cases} \gamma_{sat}\cdot\left(\frac{1}{2}\cdot a_s\cdot HD_{Wetb}\right)\cdot 1\cdot ft & \text{if } HD_{Wb} \geq HD_{Sb} \\ \gamma\cdot\left(\frac{1}{2}\cdot a_s\cdot HD_{Dryb}\right)\cdot 1\cdot ft & \text{if } HD_{Wb} = 0 \\ \left[\gamma_{sat}\cdot\left(\frac{1}{2}\cdot a_w\cdot HD_{Wetb}\right) + \gamma\cdot HD_{Dryb}\cdot\left[a_w + \frac{1}{2}\cdot(a_s - a_w)\right]\right]\cdot 1ft & \text{if } HD_{Sb} \geq HD_{Wb} \wedge HD_{Wb} \neq 0 \end{cases}$$

$A_{S3} := b_h + \frac{a_s}{3}$

$M_{S3} := S3\cdot A_{S3}$

$S3 = 8.76\cdot kip$ $A_{S3} = 37.05\,ft$ $M_{S3} = 324.64\cdot ft\cdot kip$

Weight Concrete

$WC := C1 + C2 + C3$ $MC := M_{C1} + M_{C2} + M_{C3}$

$WC = 68.4\cdot kip$ $MC = 2267.7\cdot ft\cdot kip$

Weight Water

$WW := W1 + W2 + W3$ $MW := M_{W1} + M_{W2} + M_{W3}$

$WW = 134.4\cdot kip$ $MW = 3586.3\cdot ft\cdot kip$

Weight Soil

$WS := S1 + S2 + S3$ $MS := M_{S1} + M_{S2} + M_{S3}$

$WS = 210\cdot kip$ $MS = 3946.96\cdot ft\cdot kip$

Total Weight

$W := WC + WW + WS$ $M_W := MC + MW + MS$

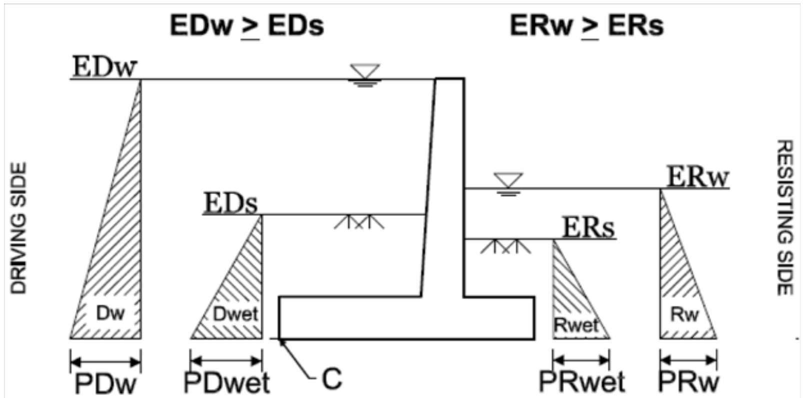
$W = 412.81\cdot kip$ $M_W = 9800.96\cdot ft\cdot kip$

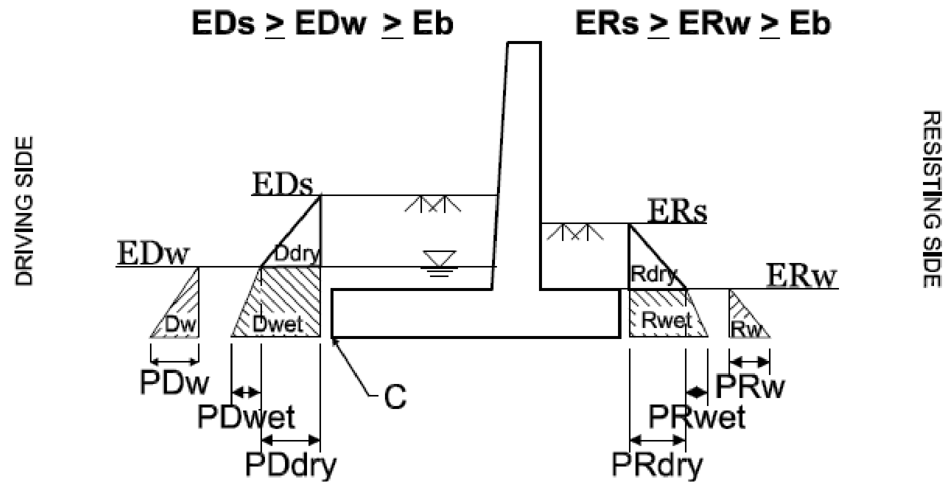
Location

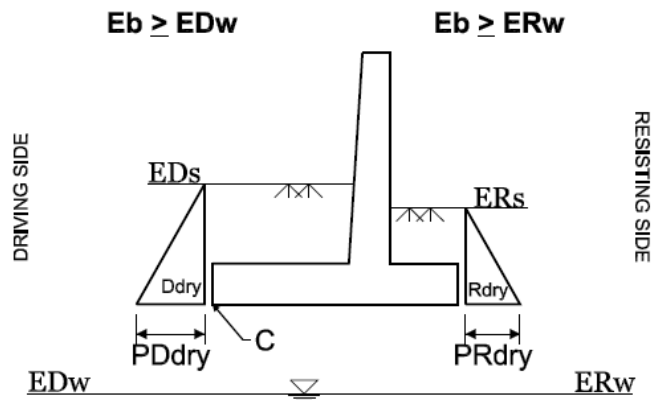
$$L_W := \begin{cases} 0 & \text{if } W = 0 \\ \frac{M_W}{W} & \text{otherwise} \end{cases}$$

$L_W = 23.74\,ft$

Lateral Loads from Water and Soil







Calculations to determine pressures due to water or soil at base Elevation

Pressure due to driving water	$P_{Dw} := \gamma_w \cdot HD_W$	$P_{Dw} = 3 \cdot ksf$
Pressure due to driving dry soil	$P_{Ddry} := K_0 \cdot \gamma \cdot HD_{Dry}$	$P_{Ddry} = 0 \cdot ksf$
Pressure due to driving wet soil	$P_{Dwet} := K_0 \cdot \gamma_b \cdot HD_{Wet}$	$P_{Dwet} = 1.62 \cdot ksf$
Pressure due to resisting water	$P_{Rw} := \gamma_w \cdot HR_W$	$P_{Rw} = 2.07 \cdot ksf$
Pressure due to resisting dry soil	$P_{Rdry} := K_0 \cdot \gamma \cdot HR_{Dry}$	$P_{Rdry} = 0 \cdot ksf$
Pressure due to resisting wet soil	$P_{Rwet} := K_0 \cdot \gamma_b \cdot HR_{Wet}$	$P_{Rwet} = 0 \cdot ksf$

Variables used in calculations

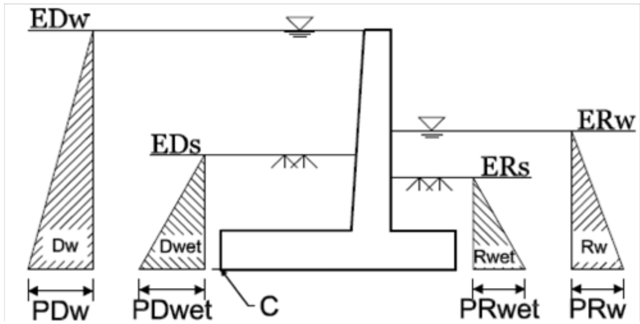
$\gamma_w = 62.5 \cdot pcf$ $\quad \gamma = 130 \cdot pcf$ $\quad \gamma_b = 67.5 \cdot pcf$ $\quad K_0 = 0.5$

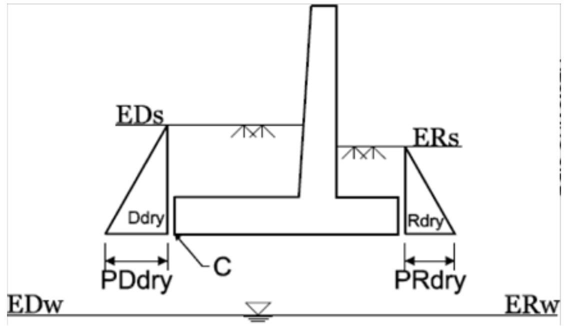
$HD_W = 48\,ft$ $\quad HD_{Dry} = 0\,ft$ $\quad HD_{Wet} = 48\,ft$ $\quad ED_W = 845\,ft$ $\quad ED_S = 845\,ft$

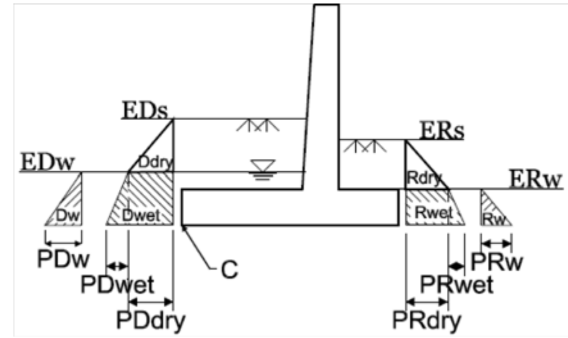
$HR_W = 33.16\,ft$ $\quad HR_{Dry} = 0\,ft$ $\quad HR_{Wet} = 0\,ft$ $\quad ER_W = 830.16\,ft$ $\quad ER_S = 792\,ft$ $\quad E_b = 797\,ft$ $\quad HD_W = 48\,ft$ $\quad HR_W = 33.16\,ft$

	Lateral Pressure (in 1' section)		Arm From Heel (C)		Moment at Heel	
Driving Water	$Dw := \frac{1}{2} \cdot P_{Dw} \cdot HD_W \cdot 1ft$	$Dw = 72 \cdot kip$	$A_{Dw} := \frac{1}{3} \cdot HD_W$	$A_{Dw} = 16\,ft$	$M_{Dw} := Dw \cdot A_{Dw}$	$M_{Dw} = 1152 \cdot ft \cdot kip$
Resisting Water	$Rw := \frac{-1}{2} \cdot P_{Rw} \cdot HR_W \cdot 1ft$	$Rw = -34.36 \cdot kip$	$A_{Rw} := \frac{1}{3} \cdot HR_W$	$A_{Rw} = 11.05\,ft$	$M_{Rw} := Rw \cdot A_{Rw}$	$M_{Rw} = -379.82 \cdot ft \cdot kip$

Variables used $P_{Ddry} = 0 \cdot ksf$ $P_{Dwet} = 1.62 \cdot ksf$ $HD_{Dry} = 0\,ft$ $HD_{Wet} = 48\,ft$ $P_{Rdry} = 0 \cdot ksf$ $P_{Rwet} = 0 \cdot ksf$ $HR_{Dry} = 0\,ft$ $HR_{Wet} = 0\,ft$







Driving Wet Soil

$Dw1 := P_{Ddry} \cdot HD_{Wet} \cdot 1ft$ $Dw1 = 0 \cdot kip$ $A_{Dw1} := \frac{1}{2} \cdot HD_{Wet}$ $A_{Dw1} = 24\,ft$ $M_{Dw1} := Dw1 \cdot A_{Dw1}$ $M_{Dw1} = 0 \cdot ft \cdot kip$

$Dw2 := \frac{1}{2} \cdot P_{Dwet} \cdot HD_{Wet} \cdot 1ft$ $Dw2 = 38.88 \cdot kip$ $A_{Dw2} := \frac{1}{3} \cdot HD_{Wet}$ $A_{Dw2} = 16\,ft$ $M_{Dw2} := Dw2 \cdot A_{Dw2}$ $M_{Dw2} = 622.08 \cdot ft \cdot kip$

$Dwet := Dw1 + Dw2$ $Dwet = 38.88 \cdot kip$ $A_{Dwet} := \begin{cases} 0 & \text{if } Dwet = 0 \\ \frac{M_{Dwet}}{Dwet} & \text{otherwise} \end{cases}$ $A_{Dwet} = 16\,ft$ $M_{Dwet} := M_{Dw1} + M_{Dw2}$ $M_{Dwet} = 622.08 \cdot ft \cdot kip$

Driving Dry Soil

$Ddry := \frac{1}{2} \cdot P_{Ddry} \cdot HD_{Dry} \cdot 1ft$ $Ddry = 0 \cdot kip$ $A_{Ddry} := \frac{1}{3} HD_{Dry} + HD_{Wet}$ $A_{Ddry} = 48\,ft$ $M_{Ddry} := Ddry \cdot A_{Ddry}$ $M_{Ddry} = 0 \cdot ft \cdot kip$

Resisting Wet Soil

$Rw1 := -P_{Rdry} \cdot HR_{Wet} \cdot 1ft$ $Rw1 = 0 \cdot kip$ $A_{Rw1} := \frac{1}{2} \cdot HR_{Wet}$ $A_{Rw1} = 0\,ft$ $M_{Rw1} := Rw1 \cdot A_{Rw1}$ $M_{Rw1} = 0 \cdot ft \cdot kip$

$Rw2 := \frac{-1}{2} \cdot P_{Rwet} \cdot HR_{Wet} \cdot 1ft$ $Rw2 = 0 \cdot kip$ $A_{Rw2} := \frac{1}{3} \cdot HR_{Wet}$ $A_{Rw2} = 0\,ft$ $M_{Rw2} := Rw2 \cdot A_{Rw2}$ $M_{Rw2} = 0 \cdot ft \cdot kip$

$Rwet := Rw1 + Rw2$ $Rwet = 0 \cdot kip$ $A_{Rwet} := \begin{cases} 0 & \text{if } Rwet = 0 \\ \frac{M_{Rwet}}{Rwet} & \text{otherwise} \end{cases}$ $A_{Rwet} = 0\,ft$ $M_{Rwet} := M_{Rw1} + M_{Rw2}$ $M_{Dwet} = 622.08 \cdot ft \cdot kip$

Resisting Dry Soil

$Rdry := \frac{-1}{2} \cdot P_{Rdry} \cdot HR_{Dry} \cdot 1ft$ $Rdry = 0 \cdot kip$ $A_{Rdry} := \frac{1}{3} HR_{Dry} + HR_{Wet}$ $A_{Rdry} = 0\,ft$ $M_{Rdry} := Rdry \cdot A_{Rdry}$ $M_{Rdry} = 0 \cdot ft \cdot kip$

Soil & Water Lateral Loads

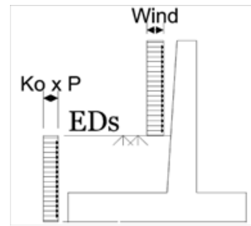
$SW := Dw + Dwet + Ddry + Rw + Rwet + Rdry$ $L_{SW} := \begin{cases} 0 & \text{if } SW = 0 \\ \frac{M_{SW}}{SW} & \text{otherwise} \end{cases}$ $M_{SW} := M_{Dw} + M_{Dwet} + M_{Ddry} + M_{Rw} + M_{Rwet} + M_{Rdry}$

$SW = 76.52 \cdot kip$ $L_{SW} = 18.22\,ft$ $M_{SW} = 1394.26 \cdot ft \cdot kip$

Surcharge and Wind Loads

Wind Height Resisting $WH_R := H - HR_S$ $WH_R = 53\,ft$

Wind Height Driving $WH_D := H - HD_S$ $WH_D = 5\,ft$



Variables used in calculations

$H = 53\,ft$ $HR_S = 0\,ft$ $HD_S = 48\,ft$

Surcharge Load ("+" = Driving Surcharge) $P = 0 \cdot plf$

Wind Load ("+" = Driving Wind) $Wind = 0 \cdot psf$

	Force (in 1' section)	Arm From Heel (C)	Moment at Heel
Compaction Driving	$ComD := \begin{cases} K_0 \cdot P \cdot HD_S & \text{if } P > 0 \\ 0 & \text{otherwise} \end{cases}$	$A_{ComD} := \frac{1}{2} \cdot HD_S$	$M_{ComD} := ComD \cdot A_{ComD}$
	$ComD = 0 \cdot kip$	$A_{ComD} = 24\,ft$	$M_{ComD} = 0 \cdot ft \cdot kip$
Compaction Resisting	$ComR := \begin{cases} K_0 \cdot P \cdot HR_S & \text{if } P < 0 \\ 0 & \text{otherwise} \end{cases}$	$A_{ComR} := \frac{1}{2} HR_S$	$M_{ComR} := ComR \cdot A_{ComR}$
	$ComR = 0 \cdot kip$	$A_{ComR} = 0\,ft$	$M_{ComR} = 0 \cdot ft \cdot kip$
Wind Driving	$Wind_D := \begin{cases} Wind \cdot WH_D \cdot 1ft & \text{if } Wind > 0 \\ 0 & \text{otherwise} \end{cases}$	$A_{WindD} := HD_S + \frac{1}{2} \cdot WH_D$	$M_{WindD} := Wind_D \cdot A_{WindD}$
	$Wind_D = 0 \cdot kip$	$A_{WindD} = 50.5\,ft$	$M_{WindD} = 0 \cdot ft \cdot kip$
Wind Resisting	$Wind_R := \begin{cases} Wind \cdot WH_R \cdot 1ft & \text{if } Wind < 0 \\ 0 & \text{otherwise} \end{cases}$	$A_{WindR} := HR_S + \frac{1}{2} \cdot WH_R$	$M_{WindR} := Wind_R \cdot A_{WindR}$
	$Wind_R = 0 \cdot kip$	$A_{WindR} = 26.5\,ft$	$M_{WindR} = 0 \cdot ft \cdot kip$
Driving Wind or Compaction	$D_{WC} := ComD + Wind_D$	$L_{DWC} := \begin{cases} 0 & \text{if } D_{WC} = 0 \\ \frac{M_{DWC}}{D_{WC}} & \text{otherwise} \end{cases}$	$M_{DWC} := M_{ComD} + M_{WindD}$
	$D_{WC} = 0 \cdot kip$	$L_{DWC} = 0$	$M_{DWC} = 0 \cdot ft \cdot kip$
Resisting Wind or Compaction	$R_{WC} := ComR + Wind_R$	$L_{RWC} := \begin{cases} 0 & \text{if } R_{WC} = 0 \\ \frac{M_{RWC}}{R_{WC}} & \text{otherwise} \end{cases}$	$M_{RWC} := M_{ComR} + M_{WindR}$
	$R_{WC} = 0 \cdot kip$	$L_{RWC} = 0$	$M_{RWC} = 0 \cdot ft \cdot kip$

Summary of Loads

The direction of the driving force and the weight is positive. The moments are determined based on an arm length from point C (in the figures above). The moments are positive clockwise around point C.

Vertical Loads	Force (in 1' section)		Arm From Heel (C)		Moment at Heel	
Weight (Concrete, water, and soil)		$W = 412.81 \cdot kip$		$L_W = 23.74\ ft$		$M_W = 9800.96 \cdot ft \cdot kip$
Uplift		$U = -158.71 \cdot kip$		$L_U = 28.66\ ft$		$M_U = -4548.46 \cdot ft \cdot kip$
Driving Compaction	$DC := \begin{cases} P \cdot b_h & \text{if } P > 0 \\ 0 & \text{otherwise} \end{cases}$	$DC = 0 \cdot kip$	$L_{DC} := \frac{b_h}{2}$	$L_{DC} = 18\ ft$	$M_{DC} := DC \cdot L_{DC}$	$M_{DC} = 0 \cdot ft \cdot kip$
Resisting Compaction	$RC := \begin{cases} P \cdot b_t & \text{if } P < 0 \\ 0 & \text{otherwise} \end{cases}$	$RC = 0 \cdot kip$	$L_{RC} := \frac{b_t}{2} + a' + b_h$	$L_{RC} = 50.5\ ft$	$M_{RC} := RC \cdot L_{RC}$	$M_{RC} = 0 \cdot ft \cdot kip$
Total Vertical Load	$V := W + U + DC + RC$	$V = 254.1 \cdot kip$	$L_V := \frac{M_V}{V}$	$L_V = 20.67\ ft$	$M_V := M_W + M_U + M_{DC} + M_{RC}$	$M_V = 5252.51 \cdot ft \cdot kip$

Lateral Loads						
Total Lateral Loads from water and Soil		$SW = 76.52 \cdot kip$		$L_{SW} = 18.22\ ft$		$M_{SW} = 1394.26 \cdot ft \cdot kip$
Driving Wind or Compaction		$D_{WC} = 0 \cdot kip$		$L_{DWC} = 0$		$M_{DWC} = 0 \cdot ft \cdot kip$
Resisting Wind or Compaction		$R_{WC} = 0 \cdot kip$		$L_{RWC} = 0$		$M_{RWC} = 0 \cdot ft \cdot kip$
Total Lateral Load	$L := SW + D_{WC} + R_{WC}$	$L = 76.52 \cdot kip$	$L_L := \frac{M_L}{L}$	$L_L = 18.22\ ft$	$M_L := M_{SW} + M_{DWC} + M_{RWC}$	$M_L = 1394.26 \cdot ft \cdot kip$
Position of Vertical Force due toTotal Moment (measured from the toe or "F")		$V = 254.1 \cdot kip$	$X_R := b - \frac{M_R}{V}$	$X_R = 33.84\ ft$	$M_R := M_V + M_L$	$M_R = 6646.77 \cdot ft \cdot kip$

Check Limits on Resultant Location (% of Base in Compression)

Variables used in calculations

Case Type: Case = "Usual"

Location of Vertical force: $X_R = 33.84\ ft$

Minimum allowable compression (EM 1110-2-2100 Section 3-9)

$$Com_{Min} := \begin{cases} 100\% & \text{if } Case = \text{"Usual"} \\ 75\% & \text{if } Case = \text{"Unusual"} \\ \text{"Resultant in Base"} & \text{if } Case = \text{"Extream"} \\ \text{"Error in Minimum Compression"} & \text{otherwise} \end{cases}$$

$Com_{Min} = 100 \cdot \%$

Percentage of base in Compression

$$base_{com} := \frac{3X_R}{b}$$

$base_{com} = 169.21 \cdot \%$

Base Compression Check

Base_Compression_Check := if$(Com_{Min} \le base_{com}$, "OK" , "Not Satisfied"$)$

Base_Compression_Check = "OK"

Check Sliding Stability

Variables used in calculations

Case Type: Case = "Usual"

Site Information: Site = "Limited"

Total Lateral Load: $L = 76.52 \cdot kip$

Total Vertical Load: $V = 254.1 \cdot kip$

Friction Angle: $\phi = 30 \cdot deg$

Cohession: $c = 1.39\ psi$

Base Width: $b = 60\ ft$

Minimum Factor of Safety for sliding (EM 1110-2-2100 Section 3-7)

$$FS_S := \begin{cases} 1.4 & \text{if } (Site = \text{"Defined"} \wedge Case = \text{"Usual"}) \\ 1.2 & \text{if } (Site = \text{"Defined"} \wedge Case = \text{"Unusual"}) \\ 1.1 & \text{if } (Site = \text{"Defined"} \wedge Case = \text{"Extream"}) \\ 1.5 & \text{if } (Site = \text{"Ordinary"} \wedge Case = \text{"Usual"}) \\ 1.3 & \text{if } (Site = \text{"Ordinary"} \wedge Case = \text{"Unusual"}) \\ 1.1 & \text{if } (Site = \text{"Ordinary"} \wedge Case = \text{"Extream"}) \\ 3.0 & \text{if } (Site = \text{"Limited"} \wedge Case = \text{"Usual"}) \\ 2.6 & \text{if } (Site = \text{"Limited"} \wedge Case = \text{"Unusual"}) \\ 2.2 & \text{if } (Site = \text{"Limited"} \wedge Case = \text{"Extream"}) \\ \text{"Error in Factor of Safety for Sliding"} & \text{otherwise} \end{cases}$$

$FS_S = 3$

Required Shear Force to cause failure

$$\tau_{req} := \frac{V \cdot \tan(\phi) + c \cdot b \cdot 1ft}{FS_S}$$

$\tau_{req} = 52.9 \cdot kip$

Sliding Stability Check

Sliding_Stability_Check := if$(|L| \le \tau_{req}$, "OK" , "Not Satisfied"$)$

Sliding_Stability_Check = "Not Satisfied"

Check Bearing Capacity

Bearing Capacity Factors

$$N_q := \left[e^{(\pi \cdot \tan(\phi))}\right] \cdot \left(\tan\left(45 \cdot deg + \frac{\phi}{2}\right)\right)^2$$

$N_q = 18.4$

$$N_c := \begin{cases} \left[(N_q - 1) \cdot \cot(\phi)\right] & \text{if } \phi > 0 \\ 5.14 & \text{if } \phi = 0 \end{cases}$$

$N_c = 30.14$

$$N_\gamma := \begin{cases} (-2 \cdot \sin(\beta)) & \text{if } \phi = 0 \\ \left[(N_q - 1) \cdot \tan(1.4 \cdot \phi)\right] & \text{otherwise} \end{cases}$$

$N_\gamma = 15.67$

Embedment Factors

$$\xi_{cd} := 1 + 0.2 \cdot \frac{HR_S}{B_{bar}} \cdot \tan\left(45deg + \frac{\phi}{2}\right)$$

$\xi_{cd} = 1$

$$\xi_{qd} := \begin{cases} 1 + 0.1 \cdot \frac{HR_S}{B_{bar}} \cdot \tan\left(45 \cdot deg + \frac{\phi}{2}\right) & \text{if } \phi > 10deg \\ 1 & \text{if } \phi = 0 \\ \text{"(RE EM 1110-2-2502 pg 5-4)"} & \text{otherwise} \end{cases}$$

$\xi_{qd} = 1$

$$\xi_{\gamma d} := \xi_{qd}$$

$\xi_{\gamma d} = 1$

Inclination Factors

$$\xi_{qi} := \left(1 - \frac{\delta}{90deg}\right)^2$$

$\xi_{qi} = 0.66$

$$\xi_{ci} := \xi_{qi}$$

$\xi_{ci} = 0.66$

$$\xi_{\gamma i} := \begin{cases} 0 & \text{if } \delta > \phi \\ \left(1 - \frac{\delta}{\phi}\right)^2 & \text{otherwise} \end{cases}$$

$\xi_{\gamma i} = 0.19$

Vertical Load eccentricity

$$E := \frac{b}{2} - X_R$$

$E = -3.84\,ft$

Effective width

$$B_{bar} := b - 2E$$

$B_{bar} = 67.68\,ft$

Load Inclination

$$\delta := \operatorname{atan}\left(\frac{L}{V}\right)$$

$\delta = 16.76 \cdot deg$

Effective Unit Weight

$$\gamma' := \begin{cases} \gamma & \text{if } HR_{Dry} > HR_{Wet} \\ \gamma_b & \text{otherwise} \end{cases}$$

$\gamma' = 67.5 \cdot pcf$

Overburden Pressure

$$q_0 := HR_{Dry} \cdot \gamma + HR_{Wet} \cdot \gamma_b$$

$q_0 = 0\,psi$

Variables used in calculations

Case Type	Case = "Usual"
Total Lateral Load	$L = 76.52 \cdot kip$
Total Vertical Load	$V = 254.1 \cdot kip$
Friction Angle	$\phi = 30 \cdot deg$
Resisting Dry Soil	$HR_{Dry} = 0\,ft$
Resisting Wet Soil	$HR_{Wet} = 0\,ft$
Unit Weight of Soil	$\gamma = 130 \cdot pcf$
Boyant unit weight	$\gamma_b = 67.5 \cdot pcf$
Height resisting soil	$HR_S = 0\,ft$

Base Tilt Factors

$$\xi_{qt} := (1 - \alpha \cdot \tan(\phi))^2 \qquad \xi_{qt} = 1$$

$$\xi_{\gamma t} := \xi_{qt} \qquad \xi_{\gamma t} = 1$$

$$\xi_{ct} := \begin{cases} \left[1 - \left(\dfrac{2 \cdot \alpha}{\pi + 2}\right)\right] & \text{if } \phi = 0 \cdot deg \\ \left(\left(\xi_{qt} - \dfrac{1 - \xi_{qt}}{N_c \cdot \tan(\phi)}\right)\right) & \text{if } \phi > 0.deg \end{cases} \qquad \xi_{ct} = 1$$

Ground Slope Factors

$$\xi_{\gamma g} := (1 - \tan(\beta))^2 \qquad \xi_{\gamma g} = 1$$

$$\xi_{qg} := \xi_{\gamma g} \qquad \xi_{qg} = 1$$

$$\xi_{cg} := \begin{cases} \left[1 - \left(\dfrac{2 \cdot \beta}{\pi + 2}\right)\right] & \text{if } \phi = 0 \cdot deg \\ \left(\left(\xi_{qg} - \dfrac{1 - \xi_{qg}}{N_c \cdot \tan(\phi)}\right)\right) & \text{if } \phi > 0.deg \end{cases} \qquad \xi_{cg} = 1$$

Normal Component of the ultimate bearing capacity of the foundation

$$Q := B_{bar} \cdot \left[\left(\xi_{cd} \cdot \xi_{ci} \cdot \xi_{ct} \cdot \xi_{cg} \cdot c \cdot N_c\right) + \left(\xi_{qd} \cdot \xi_{qi} \cdot \xi_{qt} \cdot \xi_{qg} \cdot q_0 \cdot N_q\right) + \frac{\left(\xi_{\gamma d} \cdot \xi_{\gamma i} \cdot \xi_{\gamma t} \cdot \xi_{\gamma g} \cdot B_{bar} \cdot \gamma' \cdot N_\gamma\right)}{2}\right] \cdot 1ft \qquad Q = 742.08 \cdot kip$$

Minimum Factor of Safety for bearing capacity (EM 1110-2-2502)

$$FS_B := \begin{cases} 3.0 & \text{if Case = "Usual"} \\ 2.0 & \text{if (Case = "Unusual"} \vee \text{Case = "Extream")} \\ \text{"Error calculating Bearing FS"} & \text{otherwise} \end{cases} \qquad FS_B = 3$$

Calculated Factor of Safety for bearing capacity

$$FS_{Bearing} := \frac{Q}{V} \qquad FS_{Bearing} = 2.92$$

Sliding Stability Check

Bearing_Capacity_Check := if(FS_B ≤ FS_Bearing , "OK" , "Not Satisfied")

Bearing_Capacity_Check = "Not Satisfied"

Minimum and Maximum Bearing Pressure (EM 1110-2-2502 page 3-12)

Maximum Bearing Pressure

$$qf := \begin{cases} \dfrac{V}{b} \cdot \left(1 + \dfrac{6 \cdot E}{b}\right) & \text{if } E < \dfrac{b}{6} \\ \dfrac{4}{3}\left(\dfrac{V}{b - 2 \cdot E}\right) & \text{otherwise} \end{cases} \qquad qf = 2.61 \frac{1}{ft} \cdot kip$$

Minimum Bearing Pressure

$$qc := \begin{cases} \dfrac{V}{b} \cdot \left(1 - \dfrac{6 \cdot E}{b}\right) & \text{if } E < \dfrac{b}{6} \\ 0 & \text{otherwise} \end{cases} \qquad qc = 5.86 \frac{1}{ft} \cdot kip$$

Bearing Base Width

$$b_B := \begin{cases} b & \text{if } E < \dfrac{b}{6} \\ \left[\dfrac{3}{2} \cdot (b - 2E)\right] & \text{otherwise} \end{cases} \qquad b_B = 60\,ft$$

Check Floatation - determine slab thickness

Slab WIdth $Slab := 238ft$

Factor of Safety for Floatation (assume normal conditions) $SF_f := 1.3$

SF.Floation = (WGT_Slab + Water in Basin)/Uplift

Therefore, WGT_Slab = SF.Floation*Uplift - Water in Basin

$Uplift := HD_W \cdot Slab \cdot \gamma_w \cdot 1ft$ $Uplift = 714 \cdot kip$

$Water\_in := (ER_W - E_b) \cdot Slab \cdot \gamma_w \cdot 1ft$ $Water\_in = 493.26 \cdot kip$

$WGT\_Slab := SF_f \cdot Uplift - Water\_in$ $WGT\_Slab = 434.95 \cdot kip$

Slab Thickness $ST := \frac{WGT\_Slab}{w_c \cdot Slab \cdot 1ft}$ $ST = 12.18\,ft$

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Rapidan Dam Investigations
Blue Earth River Feasibility Study
Ecosystem Restoration

Part E
Life-Cycle Cost/Benefit Analysis

September 2009

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Feasibility Study: Blue Earth River Basin in Minnesota and Iowa
Rapidan Dam Assessment and
Life-Cycle Benefit-Cost Analysis

Study Background

The Blue Earth River Basin feasibility study was undertaken to assess the potential for federal ecosystem restoration activities in the Blue Earth River basin. The study was initiated on October 5, 2007 with the execution of a cost-share agreement between the Department of the Army and Blue Earth County. The study is authorized by a May 10, 1962 resolution of the Committee on Public Works, U.S. House of Representatives.

Initial study efforts focused on the Rapidan Dam, because the dam blocks fish passage from the Minnesota River to the Blue Earth River and Watonwan River watersheds upstream and affects the fishery in the study area. In order to assess the expected “future without project” conditions, it was necessary to assess the current and likely future condition of the dam. To that end, the Corps conducted geotechnical investigations and preliminary hydraulic and hydrologic analyses. Those efforts identified a lack of energy dissipation and the need for a stilling basin below the dam to reduce the risk of failure during large flood events. A conceptual stilling basin design was developed for cost estimating purposes. A construction estimate and operation and maintenance estimate were prepared, and finally a life-cycle cost analysis was conducted. This report presents the preliminary information generated during the study from its inception through September 2009.

Summary of Findings

- The estimated cost of a proposed stilling basin and necessary repairs to the dam is $10.4 million
- Net revenue for Blue Earth County is negative for all alternatives evaluated in this study. Estimates of annual net costs to the County ranged from $394,000 to $696,000 per year for 50 years to construct a stilling basin and maintain the dam to support continued hydropower generation

Blue Earth River Basin Setting

The Blue Earth River basin is located in south-central Minnesota and northern Iowa. The Blue Earth River joins the Minnesota River near Mankato, Minnesota. The Blue Earth River ecosystem has been degraded by land use changes in the watershed that have altered the hydrologic and sediment transport regimes. Extensive artificial drainage made up of public and private ditch and tile systems facilitates the movement of water throughout the watershed. Approximately 86 percent of wetlands once present in the watershed have been lost through drainage. Predominant land use within the watershed is agricultural. Much of the land in the watershed is highly erodible, and the intensive agricultural land use and steep slopes in the lower reaches of the watershed result in considerable bank erosion along stream channels and high suspended sediment concentrations in the river. The Blue Earth River is a major contributor of sediment and nutrients to the Minnesota River.

History and Description of Rapidan Dam

The Rapidan Dam is located on the Blue Earth River approximately 12 miles upstream of Mankato in Blue Earth County, Minnesota. The dam, built in 1910, supports hydroelectric power generation, but it also blocks fish passage between the Minnesota River and the 1,200 miles of perennial tributary streams above the dam. The Federal Energy Regulatory Commission has classified the dam as having a significant downstream hazard potential based on the environmental damage that would be caused by an uncontrolled release of the agriculturally impacted sediments behind the dam.

The dam is an Ambursen Dam consisting of concrete structures founded on friable sandstone bedrock in a steep U-shaped valley. The overall length of the dam is approximately 475 feet and the maximum height is approximately 90 feet. The reservoir upstream of the dam provides storage for power generation, recreation and conservation value. The reservoir also serves as a sedimentation basin and is essentially full of sediment. Therefore, sediment in current runoff passes downstream. The tainter gates and the upstream bay of the powerhouse intake structure walls were damaged during the 1965 flood. All of the tainter gates were lost or damaged beyond repair. The loss of the tainter gates lowered the normal pool elevation of the reservoir by approximately 7 feet. The upstream forebay wall of the powerhouse bay nearest the spillway was severely damaged and a portion of the right (east) pier was destroyed.

The dam served as an electric power generating facility for Northern States Power Company until 1965, when it was substantially damaged during a flood. Blue Earth County obtained ownership of the structure in 1970. The dam historically supported a county bridge over the river channel. A new county bridge, located a short distance upstream of the dam, was installed during the 1980’s. Under an agreement with the county, Rapidan Redevelopment, Ltd. redeveloped the dam for producing hydroelectric power in 1984. As part of the redevelopment, the powerhouse, draft tubes, and penstocks were modified; new turbines, new tainter gates, and a low-flow valve were installed; the upstream tainter gate piers and upstream forebay wall were repaired, and the corbels were post-tensioned. In 2002, extensive undermining of the dam’s foundation was discovered, and emergency repairs were required to prevent a dam failure. Additional apron and abutment repairs have been conducted since 2002. North American Hydro operates the hydroelectric generation equipment at the dam under a lease agreement with Blue Earth County.

Prior Studies

A study and report *“Rapidan Dam Feasibility Study, Dam Repair Option”* (Barr Engineering; November, 2002) evaluated four courses of action for the dam. The four options for the facility included: (1) dam removal option, (2) dam rehabilitation option, (3) monitor and long-term sustain plan, and (4) do-nothing option. The objective of the study was to further develop the option to repair and maintain the existing dam. The study was undertaken to assist the Blue Earth County Board of Commissioners to determine a future course of action for the Rapidan Dam.

Current Study Report

This study report is a preliminary product of the Blue Earth River feasibility study. The report presents a life-cycle benefit-cost analysis of the hydropower operation and a dam maintenance option that includes improvements to dissipate hydraulic energy below the Rapidan Dam along with the associated operation and maintenance costs. This study updates and expands on the 2002 Barr report.

The 2002 Barr report included an alternative that involved monitoring of the dam with continued maintenance on an as-needed basis without the construction of a stilling basin. For this study, the Corps of Engineers did not analyze continued operation without the addition of a stilling basin. Without additional measures to dissipate energy downstream of the dam, a large flood event could cause catastrophic erosion leading to loss of the dam. Such an event could cause an uncontrolled release of large volumes of sediment causing significant environmental damage downstream. Smaller flood events could cause significant erosion that would need to be repaired. The uncertainty regarding future maintenance needs and potential financial impacts from dam failure makes it impossible to reliably estimate future costs. It would take analyses beyond the scope of this effort to quantify the risk and costs associated with continued operation of the dam without additional energy dissipation measures.

NOTE: It should be noted that the analyses presented in this report have undergone a quality control review within the St. Paul District office, but no independent technical review has been conducted. This report is being prepared only to document the preliminary work of the study team. The results are **preliminary** in nature, and not intended to serve as final recommendations of the U.S. Army Corps of Engineers. The Rapidan Dam is not required to meet and does not currently meet Corps of Engineers dam safety criteria. The improvements described in this report were not intended to bring the dam into compliance with any accepted dam safety standards. However, the conceptual features described herein would significantly reduce the risk of scour below the dam that could lead to the kind of undermining observed in 2002.

Cost Estimates

The Corps worked with the Blue Earth County Engineer to identify necessary repairs to the dam and likely operation and maintenance activities. These activities are only associated with the dam itself; all costs to operate and maintain the power generating equipment are borne by the operator in accordance with the lease agreement. Cost estimates were developed for these activities. In addition, an estimate was prepared for a conceptual stilling basin design that would be adequate to prevent scour downstream of the dam for all flows up to the flood of record (the 1965 event). Both the conceptual design and the cost estimate should be considered preliminary approximations of what would be needed to address the scour issue at the dam. Considerably more detail would be needed to support final design and budgeting for such a project. The cost estimates are shown in Tables 1-3.

Table 1: Cost Estimate Summary

ITEM DESCRIPTION	AMOUNT $K	CONTINGENCIES		TOTAL AMOUNT $K
		%	AMOUNT $K	
04 Rapidan Dam Features	6,646		1,994	8,639
30 Engineering and Design	797	30%	239	1,037
31 Supervision and Administration	532	30%	159	691
PROJECT COST				10,367
OMR&R (Present Value)				1,640
PROJECT COST Plus OMR&R				12,007
AVERAGE ANNNUAL COST				620

Table 2: Stilling Basin Cost

ITEM DESCRIPTION	QUANTITY	UNITS	UNIT COST $K	AMOUNT $K	CONTINGENCIES		TOTAL AMOUNT $K
					%	AMOUNT $K	
04 Rapidan Dam Features				**6,646**		**1,994**	**8,639**
Stilling Basin				6,481		1,944	8,425
Mob/Demob	1	LS	294	294	30%	88	382
Cofferdam and Dewatering	1	LS	500	500	30%	150	650
Site Work	1	LS	1,077	1,077	30%	323	1,400
Concrete				4,310		1,293	5,602
T-Walls				1,144		343	1,487
Horizontal	1,600	CY	0.400	640	30%	192	832
Vertical	840	CY	0.600	504	30%	151	655
Slab	6,300	CY	0.350	2,205	30%	662	2,867
Chute Blocks	30	CY	1.200	36	30%	11	47
Baffle Piers	60	CY	1.200	72	30%	22	94
End Sill	530	CY	0.250	133	30%	40	172
Chute	900	CY	0.800	720	30%	216	936
Left Abut Retaining Wall	1	LS	300	300	30%	90	390
Concrete Repairs(1)	1	LS	132	132	30%	40	172
Interior hand Railing(1)	500	LF	0.066	33	30%	10	43
30 Engineering and Design		**12%**		**797**	**30%**	**239**	**1,037**
31 Supervision and Administration		**8%**		**532**	**30%**	**159**	**691**

NOTES

1 From 2002 Report

Concrete Repairs	1	LS	100
Interior hand Railing	500	LF	0.050

2 EM 1110-2-1304 Revised 31 Mar 08, TABLE A-2, YEARLY COST INDEXES BY CWBS FEATURE CODE

Jun-09	700.37	1.323	Index Factor
Nov-02	529.45		

3 Economic Adjustment Factors

interest rate	4.625%	
period years	50	

Table 3: Operation and Maintenance Costs

ITEM DESCRIPTION	QUANTITY	UNITS	UNIT COST $K	AMOUNT $K	CONTINGENCIES		TOTAL AMOUNT $K	Year	Present Value $K
					%	AMOUNT $K			
OPERATIONS MAINTENANCE REPAIR & REHABILITATION (OMR&R)									**1,640**
Overflow Spillway Overlay									**349**
04 Dams	2,300	SY	0.300	690	30%	207	897	25	290
30 Engineering and Design		LS	12%	83	30%	25	108	24	36
31 Supervision and Administration		LS	8%	55	30%	17	72	25	23
Right Abutment Rock Replenishment									**159**
04 Dams	1	LS	200	200	30%	60	260	15	132
30 Engineering and Design		LS	12%	24	30%	7	31	14	17
31 Supervision and Administration		LS	8%	16	30%	5	21	15	11
Right Abutment Rock Replenishment									**81**
04 Dams	1	LS	200	200	30%	60	260	30	67
30 Engineering and Design		LS	12%	24	30%	7	31	29	8
31 Supervision and Administration		LS	8%	16	30%	5	21	30	5
Right Abutment Rock Replenishment									**41**
04 Dams	1	LS	200	200	30%	60	260	45	34
30 Engineering and Design		LS	12%	24	30%	7	31	44	4
31 Supervision and Administration		LS	8%	16	30%	5	21	45	3
Rehabilitation									**506**
04 Dams	1	LS	1,000	1,000	30%	300	1,300	25	420
30 Engineering and Design		LS	12%	120	30%	36	156	24	53
31 Supervision and Administration		LS	8%	80	30%	24	104	25	34
Routine Annual O & M	1	LS	20	20	30%	6	26	Annual	**504**

Economic Analyses

Throughout this analysis, price levels are stated as of June 2009, with a Federal discount rate of 4 5/8 percent for water resource projects being used to amortize costs and to discount benefits to a common period of time, and a 50-year period of analysis.

The repair option involves the construction of a stilling basin. It assumes that Rapidan Dam will continue to generate hydroelectric power. This will result in revenues based on the county’s lease agreement with the operator of the hydroelectric generating facilities and Minnesota’s Renewable Energy Production Incentive Program. Records of revenues from 2002-2008 were obtained from Blue Earth County and compared to the projections in the 2002 Barr report. Although total annual revenues have varied significantly, the average annual revenue figures used in the 2002 Barr Report appear to be a reasonable estimate for future revenues. Based on the lease agreement, the county receives approximately 5 percent of the total revenues generated at the dam. The county's annual revenues from the facility are estimated at $37,000. The county receives an additional $189,000 (approximately) in annual revenue from Minnesota's Renewable Energy Production Incentive Program. This program has an end date and is currently set to expire in 2013. However, the County plans to work with the State legislature to renew the agreement and is confident that the program will be extended. For this analysis, future revenue streams were calculated both with and without an extension of the incentive program.

The estimated initial cost for the proposed stilling basin and other immediate repairs/improvements to the Rapidan Dam is $10,367,000. This is an annual cost of $535,000. Average annual operation and maintenance costs for this option total $85,000. A sensitivity analysis was also conducted assuming that the dam would be removed in year 50 at a cost of $29,081,000 (based on the 2002 Barr report figure of $22,300,000 in November 2002 dollars updated to June 2009 dollars using the ENR Construction Cost Index 8578.28/6578.03 = 1.304). Note that the cost of removal at year 50 is not included in Tables 1-3 above, but is reflected in the annualized maintenance costs for Options 3 and 4 in Table 4 below. This analysis concludes that the average annual revenue to the County is negative for all of the alternatives studied; it would cost Blue Earth County between $394,000 and $696,000 per year for 50 years to construct a stilling basin and maintain the dam to support continued hydropower generation.

The current life-cycle benefit-cost analysis is summarized in Table 4.

Table 4

Corps of Engineers Necessary Improvements Option: Equivalent Annual Cost 4 5/8% Interest, 50 Years - June 2009 Price Levels

	A	B	C	D	E	F	G	H
							G = D + F - B - C	H = D + E - B - C
Alternative	Initial Construction Cost	Annualized Construction Cost	Annualized Maintenance Cost	Annualized Renewable Energy Incen-tive Payment	Total Annual Revenue for Power Production	Blue Earth County Annual Revenues	Blue Earth County Annual Net (Cost)/Revenue	Total Annualized (Cost)/Revenue
Option #1	$10,367,000	$535,000	$85,000	$43,000	$721,000	$37,000	-$540,000	$144,000
Option #2	$10,367,000	$535,000	$85,000	$189,000	$721,000	$37,000	-$394,000	$290,000
Option #3	$10,367,000	$535,000	$241,000	$43,000	$721,000	$37,000	-$696,000	-$12,000
Option #4	$10,367,000	$535,000	$241,000	$189,000	$721,000	$37,000	-$550,000	$134,000

Revenue assumptions:

The Corps necessary improvements option will continue to generate hydroelectric power.

Average annual revenue is $721,000. From Barr Report.

The county receives approximately 5% of the total revenues generated at the dam ($37,000). From Barr Report.

Option assumptions:

Potential revenues based on Minnesota's Renewable Energy Production Incentive Program include $189,000 per year thru the year 2013. (Option's 1 & 3)

Potential revenues based on Minnesota's Renewable Energy Production Incentive Program include $189,000 per year thru the entire period of analysis. (Option's 2 & 4)

Option's 1 & 2 don't include removal of the dam at the end of the period of analysis.

Option's 3 & 4 include removal of the dam at the end of the period of analysis.