-- *PRINDLE LAKE DAM* --

PHASE II
DAM EVALUATION REPORT

Dam Name:	PRINDLE LAKE DAM
State Dam ID#:	3-14-54-38
NID ID#:	MA01174
Owner:	SANTOS IRREVOCABLE TRUST
Owner Type:	PRIVATE
Town:	CHARLTON
Consultant:	LENARD ENGINEERING, INC.
Date of Inspection:	June 6, 2008

PREFACE

The assessment of the general condition of the dam is based upon available data, visual inspections, subsurface investigations, testing and detailed computational evaluations

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection, along with data available to the inspection team. In cases where an impoundment is lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions, which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is critical to note that the condition of the dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

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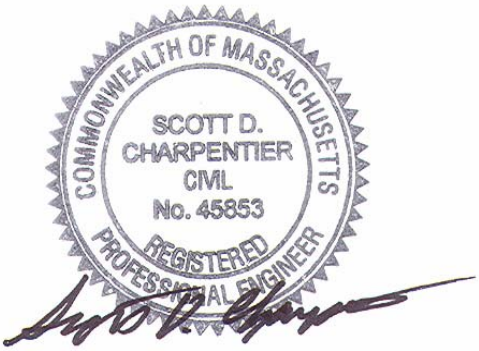

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Appendix B	Hydrologic and Hydraulic Report for Prindle Lake Dam prepared by Lenard Engineering, Inc.
Appendix C	Phase I Inspection/Evaluation Report for Prindle Lake Dam

SECTION 1

1.0 INTRODUCTION

1.1 Location

Prindle Lake Dam is located in the Town of Charlton, Worcester County, Commonwealth of Massachusetts. Latitude and longitude are given as 42.1137 N and 72.0005 W respectively on the USGS Webster quadrangle. It is upstream and easterly of State Route 169 (Southbridge Road) and Harrington Road, Charlton. The dam is situated approximately 2,680 feet south of the intersection of Carpenter Hill Road and Hall Road and is located along the western shore of Prindle Lake near the end of Oak Ridge Road. Vehicular access to the dam is feasible via Oak Ridge Road; access by foot should be undertaken for the last 500± feet.

1.2 Purpose for Performing Phase II

A Phase I Inspection and Evaluation Report dated September 14, 2006 developed by Lenard Engineering Inc (LEI) found Prindle Lake Dam was found to be in POOR condition. Specific concerns include the wet area along greater than half of the toe of the dam, numerous locations of seepage, a lack of effective slope protection (grassy cover), steep downstream face slope, the lack of an emergency spillway, and tree and shrub growth along the upstream and downstream faces and the toe of the dam.

1.3 Authority of Consultant

Mr. Carl Izzo, Trustee for the Santos Irrevocable Trust has retained Lenard Engineering, Inc. (LEI) to perform a Phase II Inspection and Evaluation a report of conditions for Prindle Lake Dam. This Inspection and Evaluation have been performed in accordance with MGL Chapter 253, Sections 44-50 of the Massachusetts General Laws as amended by Chapter 330 of the Acts of 2002.

1.4 Owner/Operator

	Dam Owner	Dam Caretaker
Name	Mr. Carl Izzo, Trustee for the Santos Irrevocable Trust	Mr. Carl Izzo, Trustee for the Santos Irrevocable Trust
Mailing Address	125 Summer Street	125 Summer Street
Town	Boston, MA 02110-1624	Boston, MA 02110-1624
Daytime Phone		
Emergency Phone		
Email Address		

1.5 DCR Size and Hazard Classification

Prindle Lake Dam has a maximum embankment structural height of approximately 11 feet and a maximum storage capacity of approximately 250 acre-feet. Therefore, in accordance with Department of Conservation and Recreation Office of Dam Safety classification, under Commonwealth of Massachusetts dam safety rules and regulations stated in 302 CMR 10.00 as modified amended by Chapter 330 of the Acts of 2002, Prindle Lake Dam is an INTERMEDIATE size structure.

The 2006 Phase 1 inspection and evaluation classified the dam as a "High Hazard". LEI performed a Hydraulic, Hydrologic and Dam Analysis (dated May 31, 2007) of Prindle Pond Dam. The conclusions of the report indicated that a failure of the dam at maximum pool may cause loss of life and/or serious damage to home(s), industrial or commercial facilities, important public utilities, or major transportation arteries main highway(s) or railroad(s). LEI requested reclassification of the dam from "High" to "Significant" based of the report. The Department of Conservation and Recreation reclassification Prindle Lake Dam as a SIGNIFICANT (CLASS II) hazard potential dam in an October 22, 2007 letter to LEI.

1.6 Description of the Dam and Appurtenances

Prindle Lake Dam is an earthen dam built approximately in 1952. The dam is approximately 235 feet long and 16 feet high. The primary spillway is a cast in place concrete box culvert with an opening 25" high by 83" wide, which serves as broad-crested weir. This spillway is approximately 62.5 feet from the left abutment and 173 feet from the right abutment. The crest of the dam has a width ranging from approximately 18 feet in the middle to 25.5 feet at the spillway. The crest elevation varies; the left end of the dam is 1.5 feet higher than the right end of the dam. The downstream face is sloped at 2H:1V. The upstream face was not observable under the vegetation. A timber railing, set on both the upstream and downstream sides on the top of the box culvert, provides for pedestrian safety crossing the spilling. Wire mesh fencing and an 18" picket fence have been placed on the shoulders of the crest to the left of the spillway to limit pedestrian traffic accessing the impoundment.

Remnants of an abandoned spillway inlet were located 105 ft to the right of the existing spillway on the upstream face. The outlet of the abandoned spillway was not observed. However, the old channel for the abandoned spillway was observed on the downstream side of the dam.

1.7 Downstream Area

The immediate area downstream is heavily wooded with trees, saplings, and woody brush growing close to the toe of the dam. At the toe of the dam is a marshy, wet area with evidence of seepage. The primary spillway channel winds to the right and joins with the wider, old channel 50± feet downstream from the dam.

From a visual inspection of the wooded area just upstream of Harrington Road, the brook passes through a steep rocky valley. This valley would only serve to intensify and channel the flood water should the dam suffer a catastrophic failure. The brook passes under Harrington Road through a 4 foot diameter HDPE culvert pipe. The culvert inlet and outlet do not have erosion or scour protection in-place.
Further west downstream the brook passes under State Route 169 (Southbridge Road) and connects with Cady Brook. There are commercial properties, several residences, and a bridge along Cady Brook. These structures may be impacted should the existing dam breach.

SECTION 2

2.0 HYDROLOGIC AND HYDRAULIC ANALYSIS

2.1 Hydrologic Analysis

2.1.1 Input Parameters and Methodology

The drainage area for Prindle Lake is approximately 0.4 square miles and is contained entirely in the Town of Charlton. The topography consists primarily of steep, hilly terrain. The weighted CN value of 75 was derived from SCS methodology in accordance with land uses shown on 2005 MassGIS Orthophotographs and NRCS soil surveys. The time of concentration was calculated with TR-55: Urban Hydrology for Small Watersheds.

2.1.2 Storm Event

The dam is classified by the DCR as being an Intermediate sized structure with a Significant (Class II) hazard potential. Based on the size and hazard classification of this structure, the spillway design flood (SDF) is defined in 302 CMR 10:14 (6) as the 100-year return frequency storm. The 25-year return frequency storm was included in the analysis to indicate the average frequency of use for an emergency spillway if constructed. The Soil Conservation Service (SCS) Type III rainfall distribution for the 25-year and 100-year return frequency storms was modeled using Hydraflow Hydrographs 2004 software. The SCS 24-hour precipitation values for the 25-year and 100-year return frequencies are 5.3 and 6.5 inches respectively.

2.1.3 Results

The hydrologic analysis yielded peak watershed inflows of 514.23 CFS and 710.69 CFS for the 25-year and 100-year design storms respectively. These flows were then used for the dam's hydraulic spillway routing.

2.2 Hydraulic Analysis

2.2.1 Spillway Existing Conditions

The primary spillway is the only functional outlet device at Prindle Lake Dam. There is not defined emergency spillway. The primary spillway is a concrete box culvert with approximate dimensions of eighty-three (83) inches wide by twenty-five (25) inch high. The spillway invert and crown is at elevation 705.00 and 707.00 (NGVD) respectfully. The dam crest ranges from elevation 707 to 708.5. For purposes of the existing analysis, the dam crest was modeled as a secondary spillway weir.

2.2.2 Results

The 25-year and 100-year storm (SDF) peak flows exiting the existing spillway are 16.18 cfs and 24.89 cfs respectively. The spillway capacity was calculated in order to determine the spillway adequacy during the SDF. The maximum flow that the spillway can convey without overtopping the dam is 65.51 CFS. This capacity is 263% greater than that of the SDF. Therefore, the dam would not overtop during the SDF event (see attached hydrologic and hydraulic calculations).

SECTION 3

3.0 GEOTECHNICAL EXPLORATION AND ANALYSIS

3.1 Introduction

A focused program of subsurface exploration and analysis was developed and implemented for the Prindle Lake Dam. The intent of this program was to gather pertinent data necessary for to determine various aspects of dam stability and quantify evidence of seepage. LEI utilized the services of GeoInsight, Inc. (GI) of Manchester, NH as a geotechnical subconsultant. Their role was to derive, implement, and oversee the subsurface exploration and analysis program. The detailed findings of this program are included in the attached report entitled "Phase II Inspection and Investigation, Structural Integrity Analysis, Prindle Lake Dam Charlton, Massachusetts" authored by GI. The following subsections are summaries of the referenced report content.

3.2 Exploration Activates

On April 16, 2008, five soils boring were performed on the dam embankment to assess the soil characteristics within and below the dam embankment. The investigation was performed using a hollow-stem auger drill rig with a depth of drilling range between 10 and 22 feet. Continuous soil samples were recovered through the dam, at which point the sampling interval was increase to every five feet of boring into the native soil. A 1-inch diameter piezometer was installed at boring B-4 in order to measure piezometric head within the dam.

3.3 Subsurface Conditions

The dam was found to consist of sand fill within the dam structure above a natural soil comprised of mainly clay, sand and silt and sand. The soil profile was generally consistent across the investigation area based upon the findings of the borings. Groundwater was generally encountered at a depth of approximately 6 feet. The depth to water in the piezometer as recorded one week after installation was observed at a depth of approximately 5 feet.

3.4 Stability Analysis

Structural stability and seepage analyses were conducted as part of the Phase II Report prepared by LEI. GeoInsight, Inc. provided an engineering analysis and recommendations regarding predicted stability and seepage rates for the dam. Veneer stability and global stability analysis were performed for the dam under the three different loading conditions with a required factor of safety (F.S.) area as follows; case I, steady seepage with maximum storage pool; case II, steady seepage with surcharge pool; and case III, steady seepage and a seismic loading

The slope stability analyses included ponded water being equivalent to two feet below the top of the dam (14 feet of head), one foot below the top of the dam (15 feet of head) and at the top of the dam (16 feet of head) with assessment being conducted of the upstream and downstream face under static and seismic conditions.

The dam stability analysis indicated that adequate resistance forces were available against sliding and overturning forces. However, the stability is based upon conditions that are currently not completely defined, including slope geometry and internal conditions against the spillway. Assessment of the upstream face during rapid drawdown conditions indicated that the veneer surface would most likely slough, leaving behind a steeper face that would in turn slough more until equilibrium between the soil

and retained water within the dam core was reached. However, based on the gradation of the soil within the dam, it does not appear that the dam is significantly susceptile to liquefaction during seismic conditions.

Global stability analysis suggests that the upstream dam slope is currently in a state of failure but because this conflicts with visual observation, the face is likely stabilized by shallower grades than those modeled, vegetation and/or armor is not included in the evaluation. Overall global stability appears to be well below acceptable criteria. Global stability at the downstream face is highly dependent upon the existing fill material and natural soil, which is not readily quantifiable in terms of its integrity. The dam is significantly sensitive to a theoretical seismic loading because of the low density of the embankment materials. It is important to note that degraded, brittle existing structures such as the existing spillway could introduce very significant unknowns into the analysis. These loading scenarios are in accordance with State regulations, 302 CMR 10.14.

Global Stability

Case	Failure Mode	Required	Calculated
I	Overturning	3.0	**
II	Sliding	3.0	12.8
	Overturning	2.0	**
III	Sliding	2.0	11.4
	Overturning	>1.0	**
	Sliding	>1.0	9.9

Slope Stability

Location	Condition*	Required	Calculated
Upstream	Static	1.3	***0.9***
	Seismic	>1.0	***0.6***
Downstream	Static	1.3	3.1
	Seismic	>1.0	2.7

* all analyses considered impoundment at dam crest
** F.S. against overturning was analyzed by inspection and determined that the F.S. for the three conditions would be significantly higher than the required F.S.

3.5 <u>Seepage Analysis</u>

A seepage analysis of the dam was done using an estimated hydraulic gradient based on permeability value of other earthen dam of similar construction. The dam soil permeability was estimated to be 1 x 10^{-4} centimeters per second. Based upon this permeability rate, the total seepage rate through the dam is estimate to be 776 cubic feet per day. This suggests that either the upstream face materials restrict flow into the dam more than modeled in the analysis and/or the internal conductivity is higher and seepages exists into the base and largely out of view. It was also observed that a portion of the water exiting the spillway was traveling off-course from its designated channel, and instead was following a pathway along the base of the dam. It was difficult to determine if seepage was occurring at the base of the dam or if the wet soil conditions at the toe were due to standing water because of this pathway. During a site visit, it was observed that a large amount of tarp Rock has been placed on the northern portion of the dam's downstream face. According to a member of the Santos Irrevocable Trust, this

measure was taken to buttress an area of seepage that has been observed in this area. GeoInsight did not observe slough or other evidence of unstable conditions at the downstream face of the dam.

SECTION 4

4.0 CONCLUSION AND RECOMMENDATION

4.1 Conclusion

The detailed results of the geotechnical exploration and analysis program and the hydrologic and hydraulic analysis have been outlined herein and described in the appendices to this report. These results can be summarized as follows:

1. The dam does not have an emergency means of discharge. The spillway is adequately sized to convey the spillway design flood (100-year storm). In the event of a more significant storm event overtopping may occur and pose significant erosion risk to the crest, downstream face and toe.
2. Variable soil density and gradation exist within the core of the dam fill materials which could result in localized weak zones that may be significantly more prone to failure than other areas.
3. Existing evidence of seepage and internal erosion (loose material) is present, and the extent of potential internal damage or pending damage caused by these conditions has not been assessed in detail and may present situations of localized weak zones that may be significantly more prone to failure than other areas.
4. The lack of low permeability materials within the dam core most likely allows the potential for water to seep through the dam at a relatively fast rate, which could promote rapid failure due to piping if uncontrolled seepage at the downstream face is initiated.
5. A defined drainage filter discharge zone was not evident at the downstream toe of the dam.
6. The sub grade underlying the dam appears to include loose (uncompacted) and/or organic materials that could create unidentified and localized weak zones that may be significantly more prone to stability failure and/or piping than other areas.

In summary, the dam in its current condition is susceptible to moderate physical changes having significant adverse impacts to its stability.

Observations of deficiencies from past reports which still exist are summarized as follows:

1. There is no functional means of impoundment drawdown (low level outlet);
2. Saturated down stream toe;
3. No emergency spillway;
4. Tree growth within 25 of the toe of the embankment.

4.2 Alternatives

Four alternatives are considered viable for correction of the dam deficiencies, three of these alternatives considering maintaining the dam's current geometry and impoundment, and the fourth is to reduce the dam height and impoundment volume. The first three alternatives are presented as being constructed under a cofferdam site condition, therefore without a drawdown of the impoundment.

Complete removal of the dam is not considered a viable alternative due to the significant environmental permit requirements, costs associated with river restoration, and the anticipated resistance from impoundment recreational users.

Alternative One - *Repair Under Load*

Address all deficiencies through conventional dam repair construction methods under hydraulic loading. This alternative would allow in-situ repair of the dam without a complete cofferdam condition. This work would include the following:

1. Installation of a low level outlet with controls north of the existing spillway. The low level outlet could be constructed through open cut method utilizing tiered trench boxes with a maximum cut width of 16-feet and an excavation depth estimated to be between 8 and 12-feet below dam crest. The work area would need to be dewatered through a cofferdam system which would extend perpendicular to dam into the impoundment approximately 30-feet for a length twice that of the cut width, 32-feet. The drawdown conduit would be placed in the cut and secured against buoyancy through anchors and backfill. The control structure and valves could be installed within the dam crest during the backfill process. The outlet pipe would extend approximately 30-feet downstream and discharge through a headway with an energy dissipation pad.

2. Construction of an emergency spillway. The location of the emergency spillway should be south of primary spillway so that safe access to both the low level outlet and the primary spillway is provided during events which trigger activation of the emergency spillway. The emergency spillway should be constructed with a trapezoidal geometry at invert elevation 706.0, thus only being activated during storms equal to or greater than the 100-year return frequency. The discharge channel will be subject to high velocities, therefore must be designed to withstand the resulting shear forces. These construction efforts can occur in the dry without temporary water diversions.

3. Improving the density of the dam core. To increase the overall stability and strength (under both static and seismic conditions) of the fill materials used to construct the dam, a soil densification program could be pursued using carefully controlled compaction grouting. This method is less expensive than chemical grouting but would also help reduce seepage through the dam. The grouting could be performed using conventional drilling equipment from the dam crest, using a carefully sequenced, closely-spaced grout injection scheme combined with confirmatory sampling.

4. Improve seepage control. Seepage control at the dam should be further assessed with regard to conditions at the upstream face, as well as implementing better control at the toe. A formal drain structure design with a filter system should be installed to intercept and collect water from within the dam core, draining it to specific outlets. A drain system could be constructed at the downstream toe of the dam in combination with additional material placed along the toe (see toe buttress construction below).

5. Improve the safety factor against global slope failure. In addition to improving the density of the dam core, a toe buttress should be constructed on the downstream face of the dam to increase overall resistance to circular failure scenarios. The toe buttress will require addressing wetland issues and assessment of ground stability below the toe, but would be straight forward to construct. The buttress should be sized based upon achieving an acceptable factor of safety against rotational failure.

The work of this alternative should result in only minor permanent impacts to wetland resources particularly associated with work along the downstream toe. Implementation of this alternative would require final design, State and local permitting as follows:

1. Prepare complete construction drawings and technical specifications for the proposed work;

2. File for other environmental permits including a Notice of Intent with the Charlton Conservation Commission;

3. Submit an Application for a Dam Safety Permit (Chapter 253) with the Department of Conservation and Recreation.

Alternative Two - *Remove and Replace*

Address all deficiencies through complete removal and replacement of the existing dam. This work would include the following:

1. Installation of a cofferdam for the entire length of the dam approximately 50-feet into the impoundment. Provide temporary discharge diversion through the worksite for the spillway design flood. This can be provided through a defined low point along the cofferdam and a diversion channel. The location of these mechanisms would change as construction progresses.

2. Complete removal of the existing dam to the parent material with proper preparation and compaction of the parent material for acceptance of dam core material placement.

3. Placement and of low permeability dam core material (Zone I) in conjunction with overburden slope material (Zone II) with the Zone I material keyed into the parent material.

4. Installation of the low level outlet, primary and emergency spillways and construction of the discharge waterways would all occur in concert with the dam construction.

The work of this alternative would result in permanent impacts to wetland resources particularly associated with construction of the new downstream slope and discharge waterways. The quantity of impacts is expected to be kept below federal regulatory limits. Implementation of this alternative would require final design, State and local permitting as follows:

1. Prepare complete construction drawings and technical specifications for the proposed work;

2. File for other environmental permits including a Notice of Intent with the Charlton Conservation Commission;

3. Submit an Application for a Dam Safety Permit (Chapter 253) with the Department of Conservation and Recreation.

Alternative Three - *Repair Free of Load*

Address all deficiencies through repair of the existing dam in the dry, therefore under a complete cofferdam condition. This work would include the following:

1. Installation of a cofferdam for the entire length of the dam approximately 50-feet into the impoundment. Provide temporary discharge diversion through the worksite for the spillway design flood. This can be provided through a defined low point along the cofferdam and a diversion channel. The location of these mechanisms would change as construction progresses.

2. Installation of a low level outlet with controls north of the existing spillway. The low level outlet could be constructed through open cut method utilizing tiered trench boxes with a maximum cut width of 16-feet and an excavation depth estimated to be between 8 and 12-feet below dam crest. The drawdown conduit would be placed in the cut and secured against buoyancy through anchors and backfill. The control structure and valves could be installed within the dam crest during the backfill process. The outlet pipe would extend approximately 30-feet downstream and discharge through a headway with an energy dissipation pad.

3. Construction of an emergency spillway. The location of the emergency spillway should be south of primary spillway so that safe access to both the low level outlet and the primary spillway is provided during events which trigger activation of the emergency spillway. The emergency spillway should be constructed with a trapezoidal geometry at invert elevation 706.0, thus only being activated during storms equal to or greater than the 100-year return frequency. The discharge channel will be subject to high velocities, therefore must be designed to withstand the resulting shear forces.

4. Improving the density of the dam core. To increase the overall stability and strength (under both static and seismic conditions) of the fill materials used to construct the dam, a soil densification program could be pursued using carefully controlled compaction grouting or vibro-compaction. Provided the dam core is allowed to drain before implementation, either method could be very effective and could be implemented without as much as of a concern about causing instability with the saturated soils as in Alternative One. Both approaches help reduce seepage through the dam. The grouting could be performed using conventional drilling equipment from the dam crest, using a sequenced, closely-spaced grout injection scheme. Vibrocompaction would require the mobilization of specialized equipment to the crest, but could also be accomplished following a sequenced, closely-spaced scheme. Confirmatory sampling would be used with either approach to assess effectiveness.

5. Improve seepage control. Seepage control at the dam should be further assessed with regard to conditions at the upstream face, as well as implementing better control at the toe. With a coffer dam in place, the upstream face would be lined with a low permeability layer of soil or geosynthetic, and then armored to prevent future sloughing. This effort would significantly reduce the water seeping into the dam core. A formal toe-drain structure designed with a filter system should still be installed to intercept and collect water from within the dam core, draining it to specific outlets. A drain system could be constructed at the downstream toe of the dam in combination with additional material placed along the toe (see toe buttress construction below), as described for Alternative One.

6. Improve the safety factor against global slope failure. In addition to improving the density of the dam core, a toe buttress should be constructed on the downstream face of the dam to increase overall resistance to circular failure scenarios. The toe buttress will require addressing wetland issues and assessment of ground stability below the toe, but would be straight forward to construct. The buttress should be sized based upon achieving an acceptable factor of safety against rotational failure, as described for Alternative One.

The work of this alternative should result in only minor permanent impacts to wetland resources particularly associated with work along the downstream toe. Implementation of this alternative would require final design, State and local permitting as follows:

1. Prepare complete construction drawings and technical specifications for the proposed work;

2. File for other environmental permits including a Notice of Intent with the Charlton Conservation Commission;

3. Submit an Application for a Dam Safety Permit (Chapter 253) with the Department of Conservation and Recreation.

Alternative Four – *Breach the Dam*

Lower the dam impoundment and breach the embankment. The work would include the following:

1. Lower the impoundment by means of a mechanical pump system. The pump system will remain until all construction work is completed;

2. Remove and dispose of sediment from the upstream side of the dam along the entire dam length;

3. Remove the concrete spillway and excavate approximately 60% of the dam embankment length through the complete dam height;

4. Stabilized all disturbed site surfaces.

The work of this alternative would result in permanent impacts to wetland resources. The quantity of impacts is expected to be such that federal regulatory limits would require filing for an Army Corps of Engineer Section 401 Water Quality Certification and a Section 404 Clean Water Act Permit. Implementation of this alternative would require final design, local and State permitting as follows:

1. Prepare complete construction drawings and technical specifications for the proposed work;

2. Perform downstream flood study and likely prepare a Letter of Map Revision to FEMA;

3. Prepare and file an Environmental Notification Form with the Massachusetts Environmental Protection Agency with a waiver request from the preparation of a required Environmental Impact Report, 310 CMR 11.03(a);

4. File for other environmental permits including the Army Corps of Engineers, the Charlton Conservation Commission, and the regional office of the Massachusetts Department of Environmental Protection;

5. Submit an Application for a Dam Safety Permit (Chapter 253) with the Department of Conservation and Recreation.

4.3 Recommendation

LEI recommends Alternative Four – *Breach the Dam* as the preferred alternative. This recommendation is based upon the following factors:

1. Elimination of a manmade water flow impedance can be viewed as an environmental benefit;
2. The dam does not provide any functional service to the dam owner;
3. With an estimated cost of repairs between $325,000 and $375,000 reconstruction of the dam is cost prohibitive.

4.4 Remedial Measures

LEI recommend the following short term repairs and remedial actions:

1. Develop and operations and maintenance manual - Formalize operations and maintenance procedures for the dam. This will develop frequencies for routine inspections, maintenance and monitoring of the condition of the dam.

2. Brush Removal - The removal of brush, shrubs and trees along the embankment. Voids developed by removal of this vegetation would be filled with proper material and compacted.

3. Repair of Spillway - Repair cracked or deteriorated concrete and grout, as necessary, on the spillway and outlet channel

4. Analysis - Evaluate the need for slope protection (riprap) on the upstream side of the embankment

4.5 Opinion of Probable Construction Cost

The following opinion of probable cost has been prepared for recommended Alternative Four – *Breach the Dam*. The values shown herein are based on limited investigation and are provided for general informational purposes only. Actual construction cost may vary significantly dependant upon factors not yet know.

ITEM	ESTIMATED COST
Design Drawings and Technical Specifications	$ 30,000
DCR Chapter 253 Permit	$ 2,000
Local, State and Federal Environmental Permits	$ 25,000
Construction (two months):	
Pumping	*$ 20,000*
Breach	*$ 40,000*
Site Finishes	*$ 15,000*
25% Contingency	*$ 15,000*
Construction Total	*$ 90,000*
Construction and Permit Oversight	$ 15,000
TOTAL	**$ 162,000**

Appendix A: Phase II Inspection and Investigation, Structural Integrity Analysis Prindle Lake Dam Charlton, MA Prepared by GeoInsight , Inc.

Site Location: **Prindle Pond Dam**
Oak Ridge Road
Charlton, MA

Prepared for: **Santos Irrevocable Trust**
125 Summer Street
Boston MA 02110 -1624

Prepared by: **Lenard Engineering, Inc.**
19 Midstate Drive
Suite 200
Auburn, MA 01501

Date: **June 6, 2008**

PHASE II INSPECTION AND INVESTIGATION
STRUCTURAL INTEGRITY ANALYSIS
PRINDLE LAKE DAM
CHARLTON, MASSACHUSETTS

Prepared for:

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June 6, 2008

GeoInsight Project 5482-000
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FIGURES

FIGURE 1	Site Locus
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FIGURE 3	Cross Section A-A' and B-B'

APPENDICES

APPENDIX A	GeoInsight Soil Boring Logs
APPENDIX B	Representative Photographs
APPENDIX C	Analytical Backup Information

PHASE II INSPECTION AND INVESTIGATION
STRUCTURAL INTEGRITY ANALYSIS
PRINDLE LAKE DAM
CHARLTON, MASSACHUSETTS

1.0 INTRODUCTION

GeoInsight, Inc. (GeoInsight) prepared this evaluation report to characterize geotechnical conditions at a small surface water impoundment structure known as the Lake Prindle Dam located off Oak Ridge Drive in Charlton, Massachusetts (the dam). The work performed was described in a Scope of Work, dated March 31, 2008, submitted by GeoInsight to Lenard Engineering, Inc. (LEI). GeoInsight obtained information regarding features associated with the dam and its impoundment based upon an electronic plan set provided by LEI that included a plan view of the dam, local topography, and pertinent dam features. GeoInsight also reviewed correspondence and other information provided by LEI regarding the dam, including a LEI report titled "Prindle Lake Dam Phase I/Inspection Evaluation Report" (Phase I) based upon a September 14, 2006 inspection performed by LEI.

The purpose of GeoInsight's involvement with the project was to develop a geotechnical soil investigation program for the dam, oversee the investigation and characterize subsurface conditions, and provide an engineering analysis and recommendations regarding predicted stability and seepage rates for the dam. GeoInsight's activities were conducted in conjunction with other work performed by LEI to provide a comprehensive Phase II Inspection and Investigation for the dam in accordance with a Certificate of Non-Compliance and Dam Safety Order, dated October 22, 2007, issued by the Massachusetts Department of Conservation and Recreation (DCR) to the owner of the dam.

2.0 PROPERTY DESCRIPTION AND BACKGROUND

The Prindle Lake Dam is located off Oak Ridge Drive in Charlton, Massachusetts. The dam is bordered to the north by Oak Ridge Drive, to the east by Prindle Lake and to the south and west by undeveloped woodlands. The property area and immediate vicinity consist of relatively moderately sloping topography toward the dam.

The dam is an earthen embankment, reportedly built around 1952. The top of the dam includes an approximate 18-foot wide crest and 10-foot wide concrete cover that spans the spillway, allowing vehicle access to the south side of the dam. The dam alignment is approximately 235 feet long in the north to south direction and approximately 16 feet high (maximum) on the downstream face. A Site Locus map based upon the United States Geological Survey (USGS) Quadrangle Map is included as Figure 1.

Based upon the Phase I prepared by LEI, the dam is identified to be in poor condition based upon the following results:

- wet areas along greater than half of the toe of the dam;
- numerous locations of seepage;
- lack of an emergency spillway; and
- uncontrolled tree and shrub growth along the upstream and downstream faces and toe of dam.

The Phase I indicated that the dam has a significant (Class I) hazard potential because its failure at maximum pool elevation could cause loss of life and damage to homes and secondary highways or cause interruption of the use or service of relatively important facilities.

3.0 INVESTIGATION ACTIVITIES

To assess subsurface conditions relative to the dam, GeoInsight coordinated a focused geotechnical drilling program at the dam. Prior to initiating the subsurface investigation, Dig Safe Systems, Inc. and Town of Charlton municipal utility officials were notified to mark known public utilities leading onto the property. A GeoInsight engineer was present during the investigation to observe subsurface conditions encountered at the locations of the borings. Boring locations were designed to preliminarily evaluate subsurface conditions within and beneath the dam, but were also partially dictated by existing property use and access restrictions.

On April 16, 2008, GeoInsight oversaw the advancement of five soil borings (designated as B-1 through B-5) within the dam alignment. The purpose of the boring investigation program was to assess soil characteristics within and below the dam, provide definable subsurface soil strength data, and evaluate apparent soil moisture conditions. The investigation was performed using a hollow-stem auger drill rig (CME-55) mounted to an all-terrain, tracked vehicle operated by Technical Drilling Services, Inc. of Sterling, Massachusetts. GeoInsight observed subsurface conditions at the locations of the five soil borings and used the information to assess the objectives of the boring program. Drilling depths ranged from approximately 10 to 22 feet below ground surface (bgs). Approximate boring locations are depicted on Figure 2, as measured from existing property features.

The soil borings were installed using 4.25-inch inside diameter hollow-stem augers to advance the boreholes through the soil and provide an open hole for sampling. Soil samples were recovered in accordance with the American Society for Testing and Materials specifications for the Standard Penetration Test (SPT). A 24-inch long split spoon sampler was used to recover the samples. The sampler was advanced by blows from a 140-pound weight free falling from a height of 30 inches, with the number of blows needed to advance the sampler in 6-inch increments of penetration being recorded for each 24-inch sample interval. The summation of the blows necessary to drive the second and third increments is

called the Standard Penetration Number (SPT; N-value), which is used as an indicator of the soil's inherent strength and *in situ* density. The SPT samples were recovered continuously at each boring location until penetrating through the dam and into the apparently native underlying material, at which point the sampling interval was increased to every 5 feet for the borings that were further advanced into the native soil. The soil samples retrieved in the split spoon sampler during each SPT were visually classified in general accordance with the Burmister Soil Classification System. Following the soil boring activities, the borings were backfilled with native material. Soil boring logs prepared by GeoInsight to describe conditions encountered during the investigation are included in Appendix A. A generalized cross section provided as Figure 3 was also developed from the boring logs that graphically represents an interpolation of the conditions encountered.

4.0 SUBSURFACE CONDITIONS

Based upon evaluation of the boring data collected by GeoInsight, subsurface conditions at the dam generally included a sand fill within the dam structure above a natural soil comprised mainly of clay, sand and silt, and sand. The soil profile was generally consistent across the investigation area based upon the findings of the boring program.

In the area of B-1 (south side of the existing spillway), a sand fill material was observed to extend to approximately 10 feet bgs. This layer consisted of a loose sand with varying amounts of coarse gravel. Directly beneath the sand and gravel fill layer was an apparently natural layer of very loose organic sand. The natural material extended from approximately 10 to 13 feet bgs before the auger terminated on a refusal surface at the extent of the exploration.

Boring B-2 was located near the northern side of the existing spillway. This area exhibited a very loose to loose sand fill to approximately 10 feet bgs before terminating on a refusal surface.

Boring B-3 was observed to contain a very loose to loose sand fill layer up to approximately 10 feet bgs. This layer consisted of sand with varying amounts of silt in the matrix. A natural sand layer was observed at approximately 10 to 11.3 feet bgs, before a refusal surface was encountered.

Boring B-4 appeared to consist of a loose sand fill layer to approximately 8 feet bgs. A loose, apparently natural sand and silt layer was observed to extend from 8 to 12 feet before encountering a natural organic sand layer. A stiff clay layer was observed from 15 to 22 feet bgs. A piezometer was installed in this location to measure the piezometric head pressure within the dam.

Boring B-5 was located where trap rock had been placed on the northern portion of the dam on the western downstream face. This section of the dam yielded a fill layer consisting of loose sand to approximately 10 feet bgs. Below the fill layer was a medium dense sand to clay layer extending to a depth of 17 feet bgs.

Shallow refusals (i.e., contact with material that prevented advancement of the split spoon or auger) were encountered during the boring program. Although the nature of the refusal surfaces was not identified, bedrock outcrops were not observed at the dam or in the immediate vicinity. Cobbles were observed to be present within the dam fill layer and the natural soil layers. It is important to note that based upon the limited bearing data, the dam is not a rock-filled structure and appears to be entirely constructed of soil.

Ground water was generally encountered at a consistent depth of approximately 6 feet bgs based upon observations of soil moisture or standing water in the boreholes upon completion of drilling. The depth to water in the newly installed piezometer, as recorded approximately one week after installation, was observed to be approximately 5 feet bgs. The differences in observed ground water depths appeared to correlate well with the expected relation of ground water seeping into the dam from the upstream face to potential discharge zones on the downstream face. It should be noted that the range of ground water elevation fluctuation is affected by many factors including pool elevation, season, recent precipitation, and local topography of the dam. Therefore, levels observed in the future may differ from the data indicated in this report.

While the geotechnical investigation of the dam was somewhat limited in scope, zones of loose soil with a relatively high permeability were evident. The investigation revealed a somewhat fine-grained core material, but did not reveal zones of obvious drainage material or internal reinforcement. Moreover, the embankment forming the dam did not appear to have been compacted in a manner producing a consistent density that would promote an even rate of drainage through the dam cross-section.

5.0 EVALUATION

5.1 SLOPE STABILITY EVALUATION

5.1.1 Introduction

A veneer and global slope stability condition was evaluated for a critical section through the earthen embankment that included dam construction materials and underlying subgrade soils. Parameters used in the evaluation included the following information:

- assumed dimensions of the complete earthen embankment (height, width at base, and width at top);
- moist and saturated unit weights for soils within the dam and below, and selecting representative soil friction angles; and
- estimated water profile from the upstream to the downstream side of the dam embankment.

Evaluation of the stability scenarios resulted in estimated factors of safety (F. S.) against overturning, sliding, and global stability of the soil mass forming the embankment. To identify potential critical areas, preliminary global stability analyses were performed for representative sections taken through the embankment. This screening step was limited because of the relative consistency of the cross sections through the dam. The preliminary analyses were performed using generalized slope profiles in the absence of as-built information.

5.1.2 Analytical Methods

Global stability analysis of the embankment was performed using the REAME2008 (REAME) and XSTABL 5.2 (XSTABL) slope stability software packages. Limited, simplified hand calculations were also performed as a general check of the computer modeling output. Performance criteria for evaluation of the dam sections under static and

pseudostatic (i.e., seismic) conditions included comparison of the resulting F. S. to accepted values established by the current standard of practice based upon guidance listed in 302 Code of Massachusetts Regulations (CMR) 10.14.

Stability analyses were performed for the three specific conditions of: 1) steady seepage with maximum storage pool; 2) steady seepage with surcharge pool; and 3) steady seepage and a seismic loading.

The stability analyses included ponded water being equivalent to 2 feet below the top of the dam (14 feet of head), 1 foot below the top of the dam (15 feet of head), and at the top of the dam (16 feet of head) with assessment being conducted of the upstream and downstream face under static and seismic conditions.

The REAME and XSTABL programs are fully-integrated slope stability analytical tools that receive the geometry of a critical cross section as input and perform stability analyses on that cross section. The slope analyses include two-dimensional, limit equilibrium calculations to compute the F. S. for a slope using either the simplified Bishop or modified Janbu Methods for a failure plane. These techniques are based upon the method of slices where the soil above the assumed failure surface is divided into a number of vertical slices. The stability of each slice is computed based upon force or moment equilibrium. Computations for each slice are added to estimate the stability of the slope section. This procedure can be applied to either circular or non-circular failure surfaces. In analyzing a cross section by a method of slices, the software programs divide the critical cross section into many more smaller slices than can practically be developed by hand and produce more accurate calculations of slice dimensions and weight. Therefore, the modeling tools allow rapid searches for hundreds of possible failure surfaces, facilitating a thorough evaluation of an entire slope geometry to identify areas where the lowest F. S. exist. After initial analyses to assess the use of assumed parameters and the critical section, computer modeling was used to search for the most

critical surface (i.e., the surface yielding the lowest F. S.) for the dam embankment under conditions of normal operation, drawdown, and seismic load.

The critical dam cross section was taken as being through the center of the alignment where the downstream face height is the greatest. Modeling of the dam cross section included inputting the soil and moisture parameters obtained during the subsurface investigation, and modeling the phreatic surface as if it ended at the base of the dam rather than through the spillway.

5.1.3 Results of the Stability Analyses

The results of GeoInsight's analyses, including the REAME and XSTABL computer model output, indicated that the earthen dam embankment currently appears stable, but the stability is very dependent upon conditions that are currently not completely defined, including slope geometry and internal conditions against the spillway.

Hand calculations based upon assumed dam geometry determined that an estimated F. S. of 12.8 is applicable to the dam against sliding during normal pool conditions. This F.S. did not change significantly for different pool levels. Sliding involves the lateral translation of a structure due to inadequate resistance to movement at the base. Although this value was based upon assumed geometry and conservative assumptions, it is important to note that localized weak zones may be present within the dam that may have a lower F. S. value.

Assessment of the upstream face during rapid drawdown conditions included establishing an assumed conservatively steep slope on the upstream face (30 degrees), which identified that the veneer surface would most likely slough after a rapid drawdown, leaving behind a steeper face that would, in turn, slough more until equilibrium between the soil and retained water within the dam core was reached. This was indicated in both static and seismic modeling simulations, and an infinite slope evaluation, and is consistent with intuition. However,

based upon the gradation of the soil within the dam, it does not appear that the dam is significantly susceptible to liquefaction during a seismic event.

For the normal pool evaluation and static conditions, the upstream face was assumed to be a slope of approximately 30 degrees. Applying static conditions in the stability analyses indicated F. S. values of 1.05 for the upstream face under full pool and 1.15 for the downstream face under full pool. Because the upstream face is obviously not under a condition of failure based upon visual assessment, it is apparent that the face is likely covered by vegetation and/or rock, creating a stabilized condition not represented by the modeling.

With regard to seismic analysis, the computer simulation of seismic loading incorporates forces from gravitational acceleration during an earthquake as a pseudostatic additional force imparted to the slices. A USGS publication (Miscellaneous Field Studies Map MF-2120, Algermissen, 1991) identifies an estimated maximum horizontal acceleration of 0.15 gravity (i.e., one-fifteenth of the acceleration due to gravity) in rock for the Charlton area (a copy of the relevant portion of the bedrock acceleration map is included in Appendix B). Based upon this datum, a seismic coefficient of 0.075 was input for the stability analyses for a horizontal force and 0.015 input for a vertical force (i.e., 20 percent of the horizontal force). Applying these seismic forces in the stability analysis indicated F. S. values of 0.6 for the upstream face under full pool and 1.6 for the downstream face under full pool.

An overview of the following conditions and calculated F. S. is summarized below:

Global Stability

Case	Failure Mode	Required	Calculated
I	Overturning	3.0	**
II	Sliding	3.0	12.8
	Overturning	2.0	**
III	Sliding	2.0	11.4
	Overturning	>1.0	**
	Sliding	>1.0	9.9

Slope Stability

Location	Condition*	Required	Calculated
Upstream	Static	1.3	1.5
Upstream	Seismic	>1.0	1.1
Downstream	Static	1.3	1.1
Downstream	Seismic	>1.0	0.9

* all analyses considered impoundment at dam crest.

** F. S. against overturning was analyzed by inspection and determined that the F. S. for the three conditions would be significantly higher than the required F. S.

Elevations for Cases I, II, and III were provided by LEI as a normal pool elevation of 705.0 feet, 100-year flood elevation of 706.04 feet, and maximum storage pool elevation of 707.0 feet above sea level.

5.2 LIMITED SEEPAGE ANALYSIS

GeoInsight performed a limited seepage analysis of the dam structure. Based upon measurements obtained from LEI, the crest of the dam is approximately 16 feet in height. A seepage analysis was performed to estimate the hydraulic gradient through the dam using an assumed permeability of the dam material. The dam soil was estimated to have a permeability of 1×10^{-4} centimeter per second (cm/sec) based upon visual assessment of the gradation and information obtained from permeability analyses of other earthen dams of similar construction. The overall permeability of the dam core is likely higher than this value, but it was selected to better represent the influence of vegetation and sediment on the upstream face that likely controls the rate at which water enters the body of the dam. The results of the evaluation indicated that the dam could be seeping as much as approximately 776 cubic feet of water per day (ft^3/day) across its entire alignment. This corresponds to approximately 4 gallons per minute per linear foot of the dam face. However, based upon GeoInsight's site reconnaissance activities, this seepage rate was not consistent with the observed conditions. The downstream side of the dam was observed to be moist during the site reconnaissance activities, but seepage in the form of free flowing water was not observed. This suggests that either the upstream face materials restrict flow into the dam more than

modeled by our analysis and/or that internal conductivity is higher and seepage exits the dam into the base and largely out of view.

It was also observed that a portion of the water exiting the spillway was traveling off-course from its designated channel, and instead was following a pathway along the base of the dam. Because of this pathway, it was difficult to determine if seepage was occurring at the base of the dam or if the wet soil conditions at the toe were due to standing water along the base from the spillway and standing water. During the site reconnaissance, it was also observed that a large quantity of trap rock had been placed on the northern portion of the dam's downstream face. According to a member of the Prindle Lake Association, this measure was conducted to buttress an area of seepage that had been observed in this area. GeoInsight did not observe sloughs or other evidence of unstable conditions at the downstream face of the dam.

Water table readings from the piezometer installed by GeoInsight indicated that water through the middle portion of the dam was at approximately 5 feet bgs. During the boring program, natural material was observed at approximately 10 feet bgs indicating that the constructed portion of the dam is 10 feet. Based upon this condition, this value represents the approximate midpoint height in the transverse section through the dam and where the phreatic surface would be expected to be located with dams of similar construction.

5.3 SENSITIVITY ASSESSMENT

GeoInsight's assessment of the dam with regard to stability and seepage characteristics is dependent upon the parameters and assumptions used in our evaluation. Several different aspects of the dam's physical state make precise theoretical assessment of the dam difficult. The more significant of these aspects include:

- variable soil density and gradation within the body of the dam fill materials, which could result in localized weak zones that may be significantly more prone to failure than other areas;

- existing evidence of seepage and internal erosion (loose material) is present, and the extent of potential internal damage or pending damage caused by these conditions has not been assessed in detail and may present situations of localized weak zones that may be significantly more prone to failure than other areas;

- the lack of low permeability materials within the dam core most likely allows the potential for water to seep through the dam at a relatively fast rate, which could promote rapid failure due to piping if uncontrolled seepage at the downstream face is initiated;

- a defined drainage filter discharge zone was not evident at the downstream toe of the dam; and

- the subgrade underlying the dam appears to include loose (uncompacted) and/or organic materials that could create unidentified and localized weak zones that may be significantly more prone to stability failure and/or piping than other areas.

In summary, the dam in its current condition is susceptible to moderate physical changes having significant adverse overall impacts on its stability. Therefore, GeoInsight agrees with LEI that the structure should be categorized as unsafe and in need of immediate attention.

6.0 RECOMMENDATIONS AND CONCLUSIONS

Based upon the information obtained during the subsurface investigation program and the results of the limited seepage and stability analysis, it appears that the performance of the dam is highly dependent upon several critical criteria. The dam does not meet certain established F. S. values consistent with 302 CMR 10.14. In addition, there are numerous critical defects that, without attention, could lead to rapid failure of the dam during certain conditions such as high pool elevations, heavy precipitation, and/or overtopping. Repairs and modifications to the existing structure should be implemented to ensure the continued stability of the structure.

A piezometer should be installed on the downstream side of the dam within the streambed to observe and record pressure head readings to evaluate uplift pressure that may be distributed at the base of the dam. A piezometer should also be installed directly behind the placed trap rock to observe where water is in relation to the base of the dam to evaluate whether hydrostatic pressure conditions exist within the dam core that warrant relief.

The dam stability analysis indicated that adequate resistance forces were available against overturning and sliding forces. Global stability analysis suggested the upstream dam slope is currently in a state of failure, but because this conflicts with visual observation, the face is likely stabilized by shallower grades than those modeled, vegetation, and/or armor that was not included in our evaluation. Overall global stability of the dam appears to be well below acceptable criteria. Global stability at the downstream face is highly dependent upon the existing fill material and natural soil, which is not readily quantifiable in terms of its integrity. The dam is likely significantly sensitive to a theoretical seismic loading because of the low density of the embankment materials. It is important to note that degraded, brittle existing structures such as the existing spillway could introduce very significant unknowns into the analysis.

Further assessment of the upstream face is needed to identify whether it should be stabilized against surficial failures during drawdown or seismic events using an anchoring technique (such as rip rap) or whether a low permeability facing material (such as geosynthetic clay liner) should be used to further restrict seepage into the dam. Based upon assumptions used in the analyses for this study, a rapid drawdown scenario would most likely result in sloughing of the exposed upstream face. Without knowing the character of upstream sediments and how the dam was constructed, the stability analysis is only an estimation of what actual conditions may be. Seepage conditions appear currently to be more or less at equilibrium, but this may be completely dependent upon the placed traprock. If a low permeability blanket is used on the upstream slope, it can likely be installed over existing soil/sediment as long as it is anchored in place and would further restrict seepage to the point where toe drainage would be less critical given the relatively coarse nature of soils within the dam.

It is evident, based upon observed conditions and limited analysis, that the dam is in need of numerous repairs. Obvious repairs should include installation of a proper filter in the area of the placed traprock to control seepage, maintaining control of vegetation, and the addition of an emergency spillway. To increase the global F. S. against failure, the density of the dam fill materials should be improved and strengthened by dam reconstruction, dynamic compaction, vibro-compaction, grout injection, or other methods of reducing void spaces. These techniques will also decrease permeability. Alternatively, or in addition, the toe of the downstream face could be more heavily buttressed to resist potential failure. However, before modifications are performed on the structure, additional investigation should be performed to evaluate economical options associated with all of the repairs needed. Then repairs should be prioritized to address seepage control and the addition of an emergency spillway.

It is apparent that a combination of relatively straightforward activities can be implemented to attain a solution that is economically feasible to the owner and fulfills the requirements mandated by the Massachusetts DCR.

7.0 LIMITATIONS

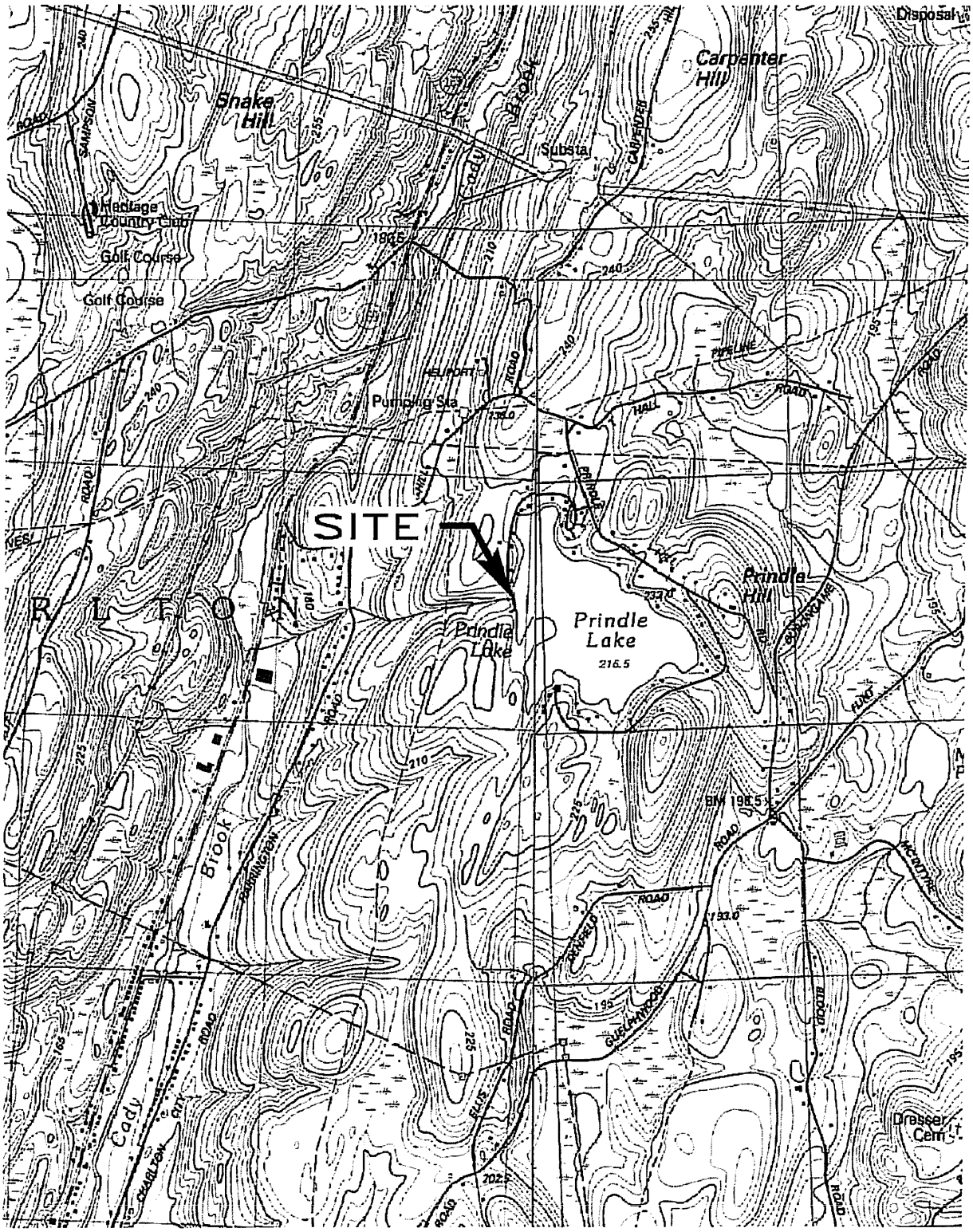
GeoInsight provided the findings and recommendations contained within this report based upon an evaluation of subsurface conditions observed and their interpolated relation to the existing dam, as documented in the report text and attached materials. The evaluation described and recommendations made in this report pertain to the specific areas assessed and investigated. The findings of this report are less likely to apply to areas not investigated as a function of increased distance away from the specific investigation locations. GeoInsight believes the investigation and evaluation described herein were performed in a manner consistent with the services that would have been provided by other geotechnical professionals under similar circumstances. However, given the variable nature of natural soil deposits and man-made structures, we cannot represent that the subsurface conditions identified in the test boring logs and described in this report are exact, nor can we guarantee that our interpolation of data between or extrapolation of data from investigation locations is completely representative of actual conditions. Variations in subsurface conditions are possible laterally and with depth that are not identified on the test boring logs or otherwise in this report.

Should additional information become available regarding the dam structure that is significantly different from that described in this report, or should subsurface conditions be found during the future that vary significantly from those observed during previous investigations and/or assumed and summarized in this report, GeoInsight should be given the opportunity to evaluate the data and modify its recommendations, if warranted.

This report has been prepared for specific application to the property described as the Prindle Lake Dam in Charlton, Massachusetts. No other warranty, expressed or implied, is made. In addition, this report was prepared exclusively for LEI and its client, and the DCR, and the use of this report by other parties without written consent from GeoInsight is hereby prohibited. In preparing this report, GeoInsight relied upon certain verbal information or representations

provided by personnel from LEI. To the extent that the interpretations, findings, and recommendations presented in this report are based in whole, or in part, upon such details, they are contingent on the validity of the information.

FIGURES
GeoInsight



SOURCE:

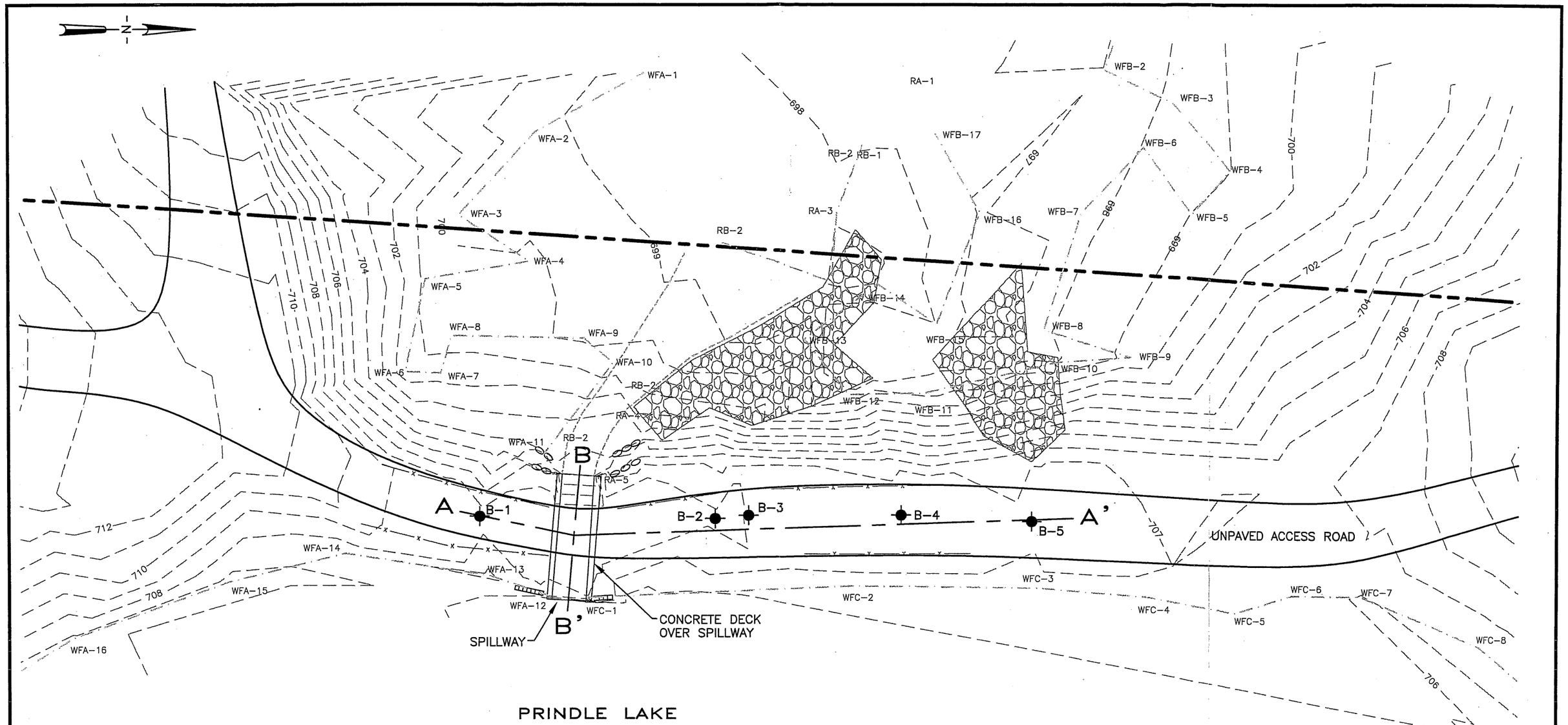
USGS WEBSTER L, MASSACHUSETTS
TOPOGRAPHIC QUADRANGLE DATED 1983

0 2000 4000
APPROX. SCALE IN FEET

CLIENT: LENARD ENGINEERING, INC.			
PROJECT: PRINDLE LAKE DAM CHARLTON, MASSACHUSETTS			
TITLE: SITE LOCUS			
DESIGNED: MRF	DRAWN: NMT	CHECKED: MJR	APPROVED: MCP
SCALE: 1" = 2000'	DATE: 5/21/08	FILE NO.: 5482-LOCUS	PROJECT NO.: 5482-000
FIGURE NO.: 1			

GeoInsight
Practical in Nature

PLOT DATE: 5-21-08
FILE: I:\5482\5482-LOCUS.dwg



LEGEND

- B-1 SOIL BORING LOCATION AND DESIGNATION
- EDGE OF WETLAND
- APPROXIMATE LIMITS OF STREAM CHANNEL
- APPROXIMATE LOCATION OF PROPERTY LINE
- APPROXIMATE LOCATION OF FENCE LINE
- APPROXIMATE EXTENT OF TRAP ROCK PLACED TO STABILIZE SEEPAGE ZONES

NOTES:

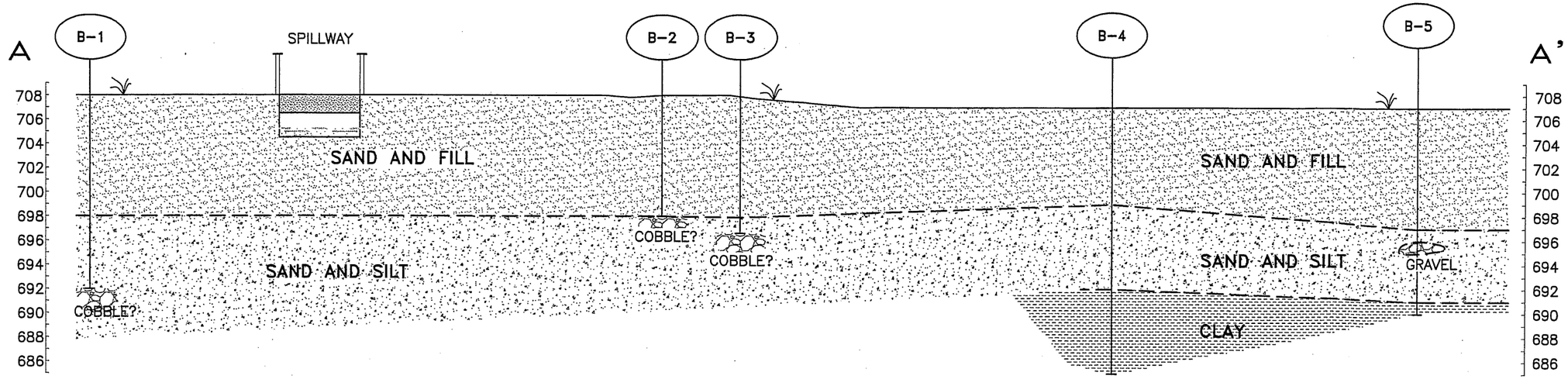
1. THIS FIGURE WAS BASED ON A SURVEY PERFORMED BY FIIELD RESOURCES, INC DATED MAY 1, 2008.

CLIENT: LENARD ENGINEERING, INC			
PROJECT: PRINDLE LAKE DAM CHARLTON, MASSACHUSETTS			
TITLE: SITE PLAN			
DESIGNED: MRF	DRAWN: NMT	CHECKED: MJR	APPROVED: MCP
SCALE: 1" = 20'	DATE: 5/12/08	FILE NO.: 5482D001	PROJECT NO.: 5482-000

GeoInsight
Practical in Nature

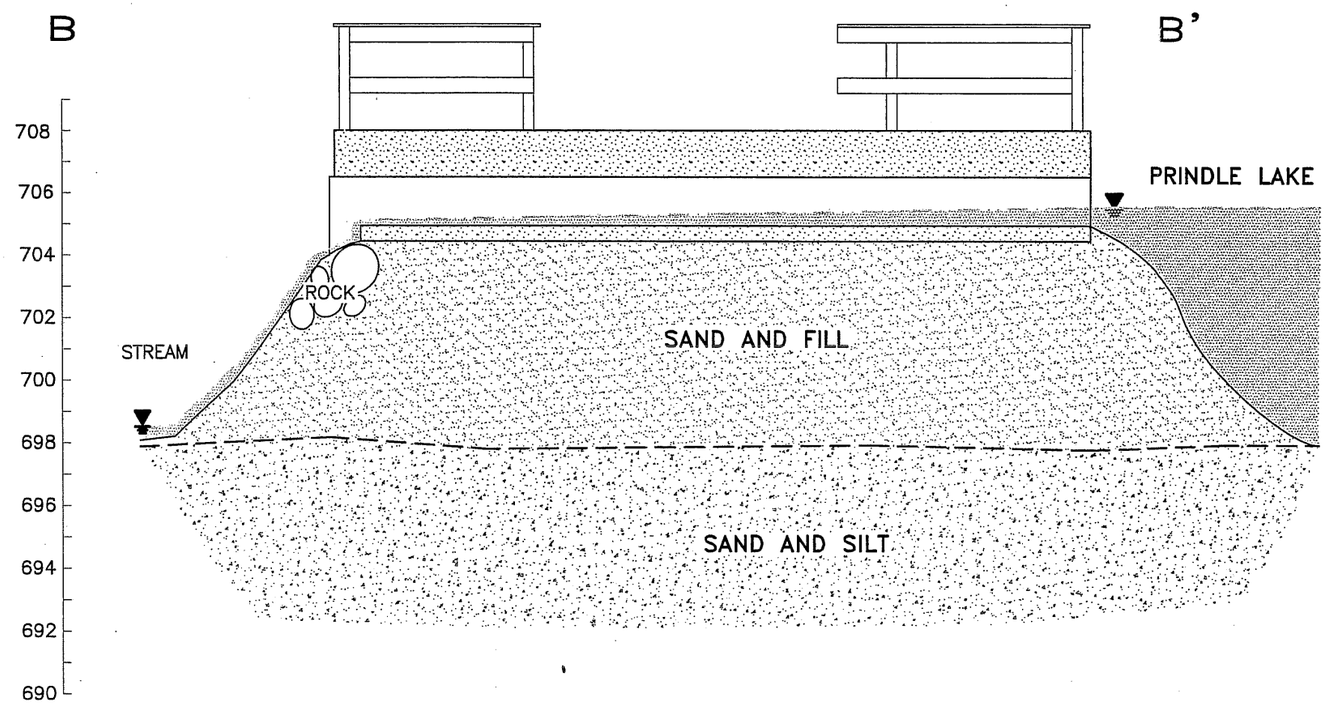
FIGURE NO.: 2

PLOT DATE: 6-5-08
FILE: I:\5482\5482D001.dwg



SECTION A–A'

SCALE 1" = 10'



SECTION B–B'

SCALE 1" = 5'

LEGEND

B-1 — SOIL BORING LOCATION AND DESIGNATION

GROUND SURFACE

GROUND WATER ELEVATION

NOTES:

1. THIS FIGURE WAS BASED ON A SURVEY PERFORMED BY FIELD RESOURCES, INC DATED MAY 1, 2008.

0 5 10

APPROX. SCALE IN FEET

CLIENT: LENARD ENGINEERING, INC				GeoInsight *Practical in Nature*
PROJECT: PRINDLE LAKE DAM CHARLTON, MASSACHUSETTS				
TITLE: CROSS SECTIONS				
DESIGNED: MRF	DRAWN: NMT	CHECKED: MJR	APPROVED: MCP	
SCALE: AS SHOWN	DATE: 5/20/08	FILE NO.: 5482D001	PROJECT NO.: 5482–000	FIGURE NO.: 3

PLOT DATE: 6-9-08
FILE: I:\5482\5482D001.dwg

APPENDIX A

GEOINSIGHT SOIL BORING LOGS

GeoInsight
Practical in Nature

SOIL BORING LOG

Location: Charlton, Massachusetts
Project: Prindle Lake Dam
Drilling Co.: Technical Drilling Services, Inc.
Foreman: Brett Barkly
GeoInsight Eng./Geol: MRF

Boring No: B-1
Sheet: 1 Of: 1
Project Number: 5482-000
Chkd. By: MCP
Boring Location: Refer to Site Plan
Ground Surface Elevation: Not Surveyed Datum: NA
Date Started: 4/16/08 Date Completed: 4/16/08

DRILLING METHOD		SAMPLER	
Vehicle:	ATV	Type:	2' Split Spoon
Model:	CME-55	Hammer(lb):	140
Method:	Hollow Stem Auger	Fall (in):	30

GROUND WATER READINGS			
DATE	DEPTH	REFERENCE	STABILIZATION
4/16/2008	6'	Ground Surface	Upon Completion

DEPTH (ft)	SAMPLE NO.	REC/PEN (in)	DEPTH (ft)	BLOWS/6"	SAMPLE DESCRIPTION	STRATUM DESCRIPTION	FIELD SCREENING (ppm)	NOTE
0	1	16/24	0-2	3	Loose, brown fine SAND and fine GRAVEL, little Silt, dry.	TOP SOIL		
				4	Loose, brown fine SAND, some fine Gravel, little coarse Gravel,	SAND FILL		
				5	trace Silt, moist.			
				4				
5	2	12/24	5-7	2	Loose, brown fine SAND, some Silt, little coarse Gravel, damp.			
				1				
				5	Brown fine SAND, little Silt, trace coarse Gravel, wet.			
				5				
10	3	8/24	10-12	1	Very loose, black fine SAND and SILT, some Organics (Roots,	ORGANIC SAND		
				12	Leaves, Twigs), wet.			1,2
				8				
				2				
								2
					Boring terminated at 13' bgs.			
					Refusal encountered.			
15								

GRANULAR SOILS		COHESIVE SOILS	
BLOWS/ft.	DENSITY	BLOWS/ft.	CONSISTENCY
0-4	V. LOOSE	<2	V. SOFT
4-10	LOOSE	2-4	SOFT
10-30	M. DENSE	4-8	M. STIFF
30-50	DENSE	8-15	STIFF
>50	V. DENSE	15-30	V. STIFF
		>30	HARD

NOTES:
1. High blow count observed due to a cobble being pushed down by the split spoon. Rock was observed in the tip of the split spoon.
2. Cobbles or boulders present based on drill rig behavior.
3. 'bgs = feet below ground surface.

GeoInsight
Practical in Nature

SOIL BORING LOG

Location: Charlton, Massachusetts
Project: Prindle Lake Dam
Drilling Co.: Technical Drilling Services, Inc.
Foreman: Brett Barkly
GeoInsight Eng./Geol: MRF

Boring No: B-2
Sheet: 1 Of: 1
Project Number: 5482-000
Chkd. By: MCP
Boring Location: Refer to Site Plan
Ground Surface Elevation: Not Surveyed Datum: NA
Date Started: 4/16/08 Date Completed: 4/16/08

DRILLING METHOD		SAMPLER	
Vehicle:	ATV	Type:	2' Split Spoon
Model:	CME-55	Hammer(lb):	140
Method:	Hollow Stem Auger	Fall (in):	30

GROUND WATER READINGS			
DATE	DEPTH	REFERENCE	STABILIZATION
4/16/2008	6'	Ground Surface	Upon Completion

DEPTH (ft)	SAMPLE NO.	REC/PEN (in)	DEPTH (ft)	BLOWS/6"	SAMPLE DESCRIPTION	STRATUM DESCRIPTION	FIELD SCREENING (ppm)	NOTE
0	1	12/24	0-2	5	Loose, brown fine SAND and fine GRAVEL, little Silt, dry.	TOPSOIL		
				5	Loose, brown fine SAND, little coarse Sand, little Silt, trace coarse Gravel, dry.	SAND FILL		
				3				
				3				
5	2	8/24	5-7	2	Very loose, brown fine SAND, some Silt, trace coarse Sand, wet.			
				1				
				1				
				2				
10					Boring terminated at 10' bgs. Refusal encountered.			1
15								

GRANULAR SOILS	
BLOWS/ft.	DENSITY
0-4	V. LOOSE
4-10	LOOSE
10-30	M. DENSE
30-50	DENSE
>50	V. DENSE

COHESIVE SOILS	
BLOWS/ft.	CONSISTENCY
<2	V. SOFT
2-4	SOFT
4-8	M. STIFF
8-15	STIFF
15-30	V. STIFF
>30	HARD

NOTES:
1. Cobbles or boulders present based upon drill rig behavior.
2. 'bgs = feet below ground surface.

GeoInsight
Practical in Nature

SOIL BORING LOG

Location:	Charlton, Massachusetts
Project:	Prindle Lake Dam
Drilling Co.:	Technical Drilling Services, Inc.
Foreman:	Brett Barkly
GeoInsight Eng./Geol:	MRF

Boring No:	B-3
Sheet:	1 Of: 1
Project Number:	5482-000
Chkd. By:	MCP
Boring Location:	Refer to Site Plan
Ground Surface Elevation:	Not Surveyed Datum: NA
Date Started: 4/16/08	Date Completed: 4/16/08

DRILLING METHOD		SAMPLER	
Vehicle:	ATV	Type:	2' Split Spoon
Model:	CME-55	Hammer(lb):	140
Method:	Hollow Stem Auger	Fall (in):	30

GROUND WATER READINGS			
DATE	DEPTH	REFERENCE	STABILIZATION
4/16/2008	6'	Ground Surface	Upon Completion

DEPTH (ft)	SAMPLE NO.	REC/PEN (in)	DEPTH (ft)	BLOWS/6"	SAMPLE DESCRIPTION	STRATUM DESCRIPTION	FIELD SCREENING (ppm)	NOTE
0	1	18/24	0-2	54	Loose, brown fine SAND and fine GRAVEL, little Silt, dry.	TOP SOIL		
				5	Loose, brown fine SAND, little Silt, little coarse Sand, trace coarse Gravel, dry.	SAND FILL		
				3				
				4				
	2	4/24	2-4	3	Loose, brown fine SAND, little Silt, little coarse Sand, trace coarse Gravel, moist.			
				4				
				5				
				3				
	3	10/24	4-6	2	Very loose, tan fine SAND, little Silt, little coarse Gravel, trace coarse Sand, wet.			
				1				
5				1				
				1				
	4	12/24	6-8	1	Loose tan fine SAND, some Silt, little coarse Sand, trace fine Gravel, wet.			
				3				
				2				
				1				
	5	12/24	8-10	2	Loose, gray fine SAND, little Silt, little coarse Sand, trace fine Gravel, wet.			
				2				
				5				
				7				
10	6	6/15	10-11.3	13	Medium dense, tan fine SAND, some Silt, little fine Gravel, wet.	SAND		
				8				
				54/3"				1
					Boring terminated at 11.3' bgs. Refusal encountered.			
15								

GRANULAR SOILS		COHESIVE SOILS	
BLOWS/ft.	DENSITY	BLOWS/ft.	CONSISTENCY
0-4	V. LOOSE	<2	V. SOFT
4-10	LOOSE	2-4	SOFT
10-30	M. DENSE	4-8	M. STIFF
30-50	DENSE	8-15	STIFF
>50	V. DENSE	15-30	V. STIFF
		>30	HARD

NOTES:
1. Cobbles or boulders present based upon drill rig behavior.
2. 'bgs = feet below ground surface.

GeoInsight
Practical in Nature

SOIL BORING LOG

Location:	Charlton, Massachusetts	Boring No:	B-4
Project:	Prindle Lake Dam	Sheet:	1 Of: 2
Drilling Co.:	Technical Drilling Services, Inc.	Project Number:	5482-000
Foreman:	Brett Barkly	Chkd. By:	MCP
GeoInsight Eng./Geol:	MRF	Boring Location:	Refer to Site Plan
		Ground Surface Elevation:	Not Surveyed Datum: NA
		Date Started:	4/16/08 Date Completed: 4/16/08

DRILLING METHOD		SAMPLER	
Vehicle:	ATV	Type:	2' Split Spoon
Model:	CME-55	Hammer(lb):	140
Method:	Hollow Stem Auger	Fall (in):	30

GROUND WATER READINGS			
DATE	DEPTH	REFERENCE	STABILIZATION
4/16/2008	6'	Ground Surface	Upon Completion

DEPTH (ft)	SAMPLE NO.	REC/PEN (in)	DEPTH (ft)	BLOWS/6"	SAMPLE DESCRIPTION	STRATUM DESCRIPTION	FIELD SCREENING (ppm)	NOTE
0	1	16/24	0-2	4	Loose, brown fine SAND and fine GRAVEL, little Silt, dry.	TOPSOIL		
				4	Loose, tan fine SAND, little Silt, little fine Gravel, dry.	SAND FILL		
				5				
				3				
	2	8/24	2-4	3	Loose, brown fine SAND, little coarse Gravel, trace Silt, moist.			
				3				
				4				
				3				
	3	0/24	4-6	2	No Recovery.			
				3				
5				3				
				3				
	4	12/24	6-8	1	Loose, gray fine SAND and SILT, little Clay, little coarse Sand, wet.			
				2				
				5	Loose, gray fine SAND, some Silt, trace Roots, wet.			
				5				
	5	6/24	8-10	2	Loose, gray fine SAND and SILT, trace coarse Sand, wet.	SAND AND SILT		
				1				
				3				
				5				
10	6	6/24	10-12	6	Loose, brown coarse SAND and SILT, little fine Gravel, trace Roots, wet.			
				4				
				2				
				1				
	7	6/24	12-14	6	Loose, brown ORGANICS (decaying Roots, Twigs, and Vegetation with Silt), wet.	ORGANIC SAND		1
				6				
				2				
				6				
	8	6/24	14-16	6	Medium dense, brown fine GRAVEL, some fine Sand, some Silt, little Organics, wet.			
				5				
15				6	Gray fine SAND and CLAY, little fine Gravel, wet.	CLAY		
				4				
	9	6/24	16-18	3	Stiff, gray CLAY, some fine Sand, little coarse sand, wet.			
				5				
				5				
				10				
	10	4/24	18-20	8	Very stiff, gray CLAY, some coarse Gravel, little Organics, wet.			
				8				
				8				
				6				

GRANULAR SOILS	
BLOWS/ft.	DENSITY
0-4	V. LOOSE
4-10	LOOSE
10-30	M. DENSE
30-50	DENSE
>50	V. DENSE

COHESIVE SOILS	
BLOWS/ft.	CONSISTENCY
<2	V. SOFT
2-4	SOFT
4-8	M. STIFF
8-15	STIFF
15-30	V. STIFF
>30	HARD

NOTES:

1. Decaying wood observed in sample.

GeoInsight
Practical in Nature

SOIL BORING LOG

Location:	Charlton, Massachusetts	Boring No:	B-4
Project:	Prindle Lake Dam	Sheet:	2 Of: 2
		Project Number:	5482-000
		Chkd. By:	MCP
Drilling Co.:	Technical Drilling Services, Inc.	Boring Location:	Refer to Site Plan
Foreman:	Brett Barkly	Ground Surface Elevation:	Not Surveyed Datum: NA
GeoInsight Eng./Geol:	MRF	Date Started: 4/16/08	Date Completed: 4/16/08

DRILLING METHOD		SAMPLER	
Vehicle:	ATV	Type:	2' Split Spoon
Model:	CME-55	Hammer(lb):	140
Method:	Hollow Stem Auger	Fall (in):	30

GROUND WATER READINGS			
DATE	DEPTH	REFERENCE	STABILIZATION
4/16/2008	6'	Ground Surface	Upon Completion

DEPTH (ft)	SAMPLE NO.	REC/PEN (in)	DEPTH (ft)	BLOWS/6"	SAMPLE DESCRIPTION	STRATUM DESCRIPTION	FIELD SCREENING (ppm)	NOTE
20	11	18/24	20-22	3	Stiff, gray CLAY, some fine Sand, little coarse Sand, wet.	CLAY		
				4				
				4				
				6				
					Boring terminated 22' bgs.			2
					Refusal not encountered.			
25								
30								
35								
				6				

GRANULAR SOILS		COHESIVE SOILS	
BLOWS/ft.	DENSITY	BLOWS/ft.	CONSISTENCY
0-4	V. LOOSE	<2	V. SOFT
4-10	LOOSE	2-4	SOFT
10-30	M. DENSE	4-8	M. STIFF
30-50	DENSE	8-15	STIFF
>50	V. DENSE	15-30	V. STIFF
		>30	HARD

NOTES:

2. A piezometer was installed at this location.
3. 'bgs = feet below ground surface.

GeoInsight
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SOIL BORING LOG

Location:	Charlton, Massachusetts
Project:	Prindle Lake Dam
Drilling Co.:	Technical Drilling Services, Inc.
Foreman:	Brett Barkly
GeoInsight Eng./Geol:	MRF

Boring No:	B-5
Sheet:	1 Of: 1
Project Number:	5482-000
Chkd. By:	MCP
Boring Location:	Refer to Site Plan
Ground Surface Elevation:	Not Surveyed Datum: NA
Date Started:	4/16/08 Date Completed: 4/16/08

DRILLING METHOD		SAMPLER	
Vehicle:	ATV	Type:	2' Split Spoon
Model:	CME-55	Hammer(lb):	140
Method:	Hollow Stem Auger	Fall (in):	30

GROUND WATER READINGS			
DATE	DEPTH	REFERENCE	STABILIZATION
4/16/2008	6'	Ground Surface	Upon Completion

DEPTH (ft)	SAMPLE NO.	REC/PEN (in)	DEPTH (ft)	BLOWS/6"	SAMPLE DESCRIPTION	STRATUM DESCRIPTION	FIELD SCREENING (ppm)	NOTE
0	1	15/24	0-2	6	Brown fine SAND and fine GRAVEL, little Silt, dry.	TOP SOIL		
				7	Brown fine SAND, some coarse Sand, little Silt, dry.	SAND FILL		
				25				
				17				1,2
5	2	6/24	5-7	2	Loose, brown fine SAND, little coarse Sand, little Silt, trace fine Gravel, trace Clay, wet.			
				1				
				3				
				8				
10	3	8/24	10-12	2	Loose, gray fine SAND and SILT, little fine Gravel, wet.	FINE SAND		
				2				
				4	Gray coarse GRAVEL, wet.	GRAVEL		
				8				
						FINE SAND		
15	4	10/24	15-17	5	Medium dense, gray fine SAND, some Clay, some coarse Gravel, wet.			
				7				
				5	Gray fine SAND and CLAY, some coarse Sand, wet.	SAND AND CLAY		
				5				
					Boring terminated at 17' bgs. Refusal not encountered.			

GRANULAR SOILS		COHESIVE SOILS	
BLOWS/ft.	DENSITY	BLOWS/ft.	CONSISTENCY
0-4	V. LOOSE	<2	V. SOFT
4-10	LOOSE	2-4	SOFT
10-30	M. DENSE	4-8	M. STIFF
30-50	DENSE	8-15	STIFF
>50	V. DENSE	15-30	V. STIFF
		>30	HARD

NOTES:

1. High blow count observed due to a cobble being pushed down by the split spoon. Rock was observed in the tip of the split spoon.
2. Cobbles or boulders present based upon drill rig behavior.
3. 'bgs = feet below ground surface.

APPENDIX B

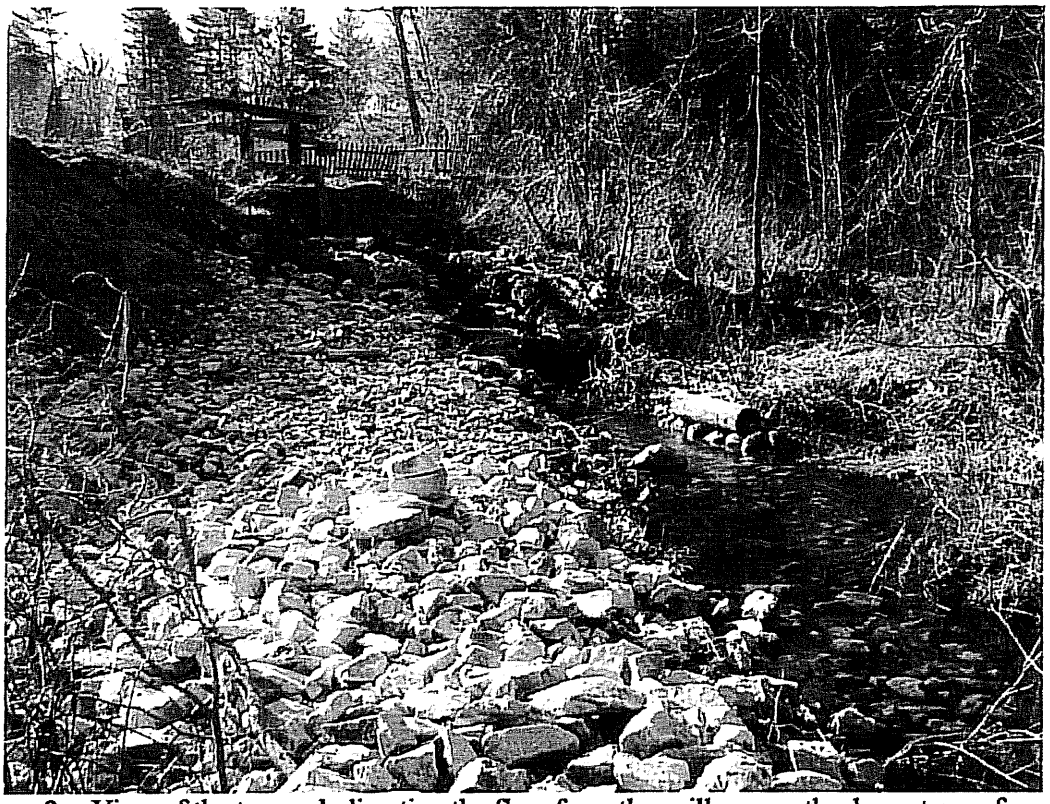
REPRESENTATIVE PHOTOGRAPHS

REPRESENTATIVE PHOTOGRAPHS
PRINDLE LAKE DAM
CHARLTON, MASSACHUSETTS

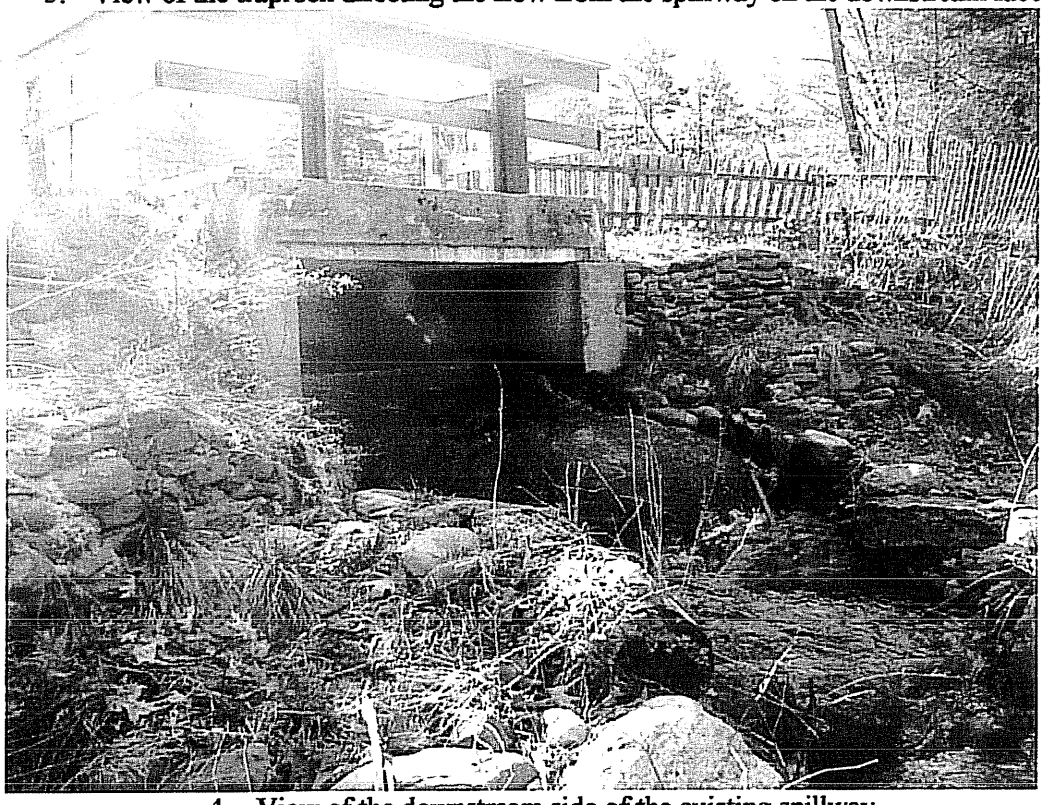
1. View from the north to the south.

2. View of the traprock located on the northern downstream face.

REPRESENTATIVE PHOTOGRAPHS
PRINDLE LAKE DAM
CHARLTON, MASSACHUSETTS



3. View of the traprock directing the flow from the spillway on the downstream face.



4. View of the downstream side of the existing spillway.

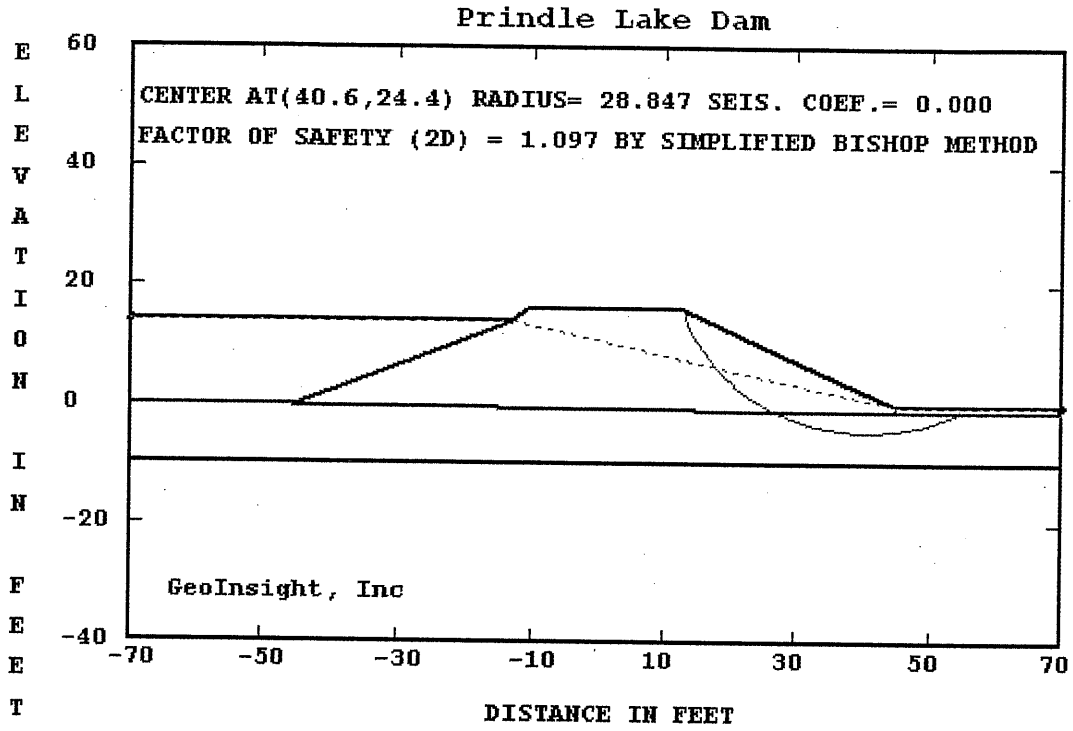
REPRESENTATIVE PHOTOGRAPHS
PRINDLE LAKE DAM
CHARLTON, MASSACHUSETTS

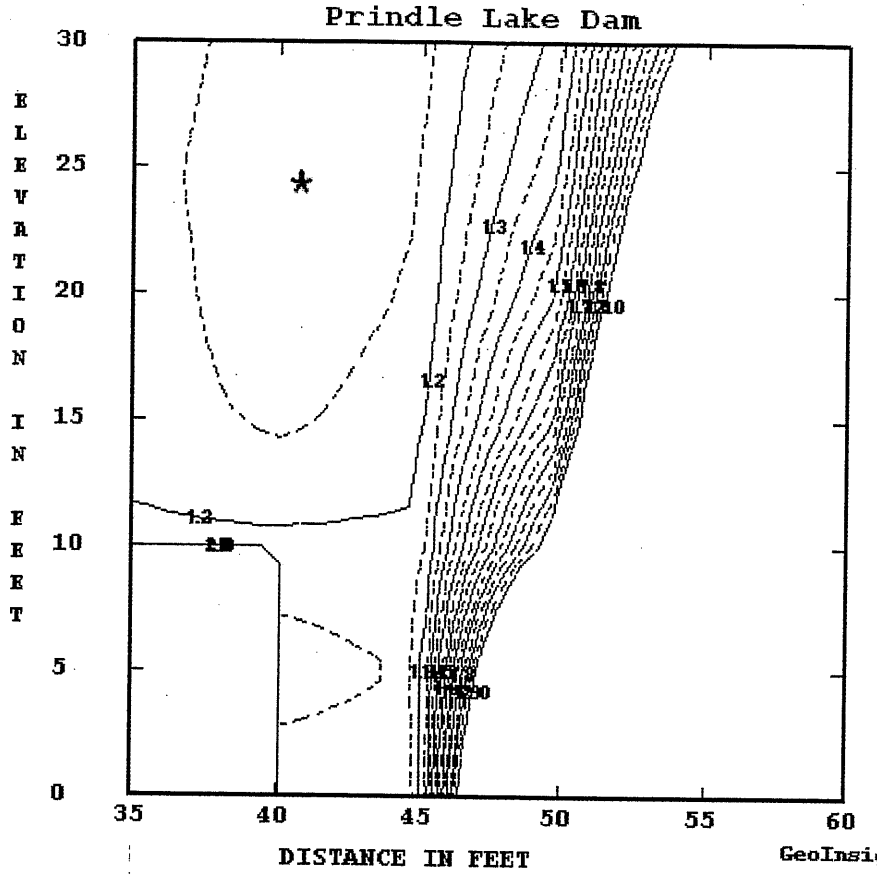
5. View of standing water along the base of the dam.

6. View of the vegetation along the upper face.

APPENDIX C

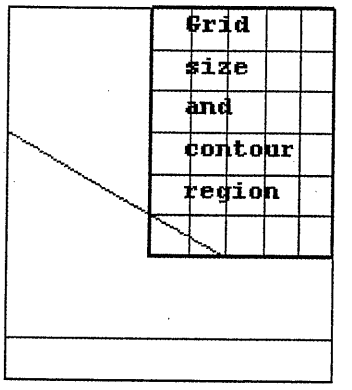
ANALYTICAL BACKUP INFORMATION


Prindle Lake Dam
CENTER AT(40.6,24.4) RADIUS= 28.847 SEIS. COEF.= 0.000
FACTOR OF SAFETY (2D) = 1.097 BY SIMPLIFIED BISHOP METHOD
GeoInsight, Inc
ELEVATION IN FEET
60
40
20
0
-20
-40
-70
-50
-30
-10
10
30
50
70
DISTANCE IN FEET



F.S.(2D) = 1.097
by SIMPLIED BISHOP
SEISMIC COEF. = 0

Number on curves indicates factor of safety.
Graph below shows true shape and location of the contour region.



GeoInsight, Inc

REAME (ROTATIONAL EQUILIBRIUM ANALYSIS OF MULTILAYERED EARTHWORKS)
THIS 2008 VERSION IS LICENSED BY CIVIL ENGINEERING SOFTWARE CENTER TO

GeoInsight, Inc

INPUT FILE NAME -C:\REAME2008\Prindle Lake 3.DAT

TITLE -Prindle Lake Dam

NO. OF STATIC AND SEISMIC CASES (NCASE) = 1

NO. OF NONCIRCULAR FAILURE SURFACES (NNS) = 0

TWO-DIMENSIONAL ANALYSIS (THREED = 0)

ANALYSIS BY DETERMINISTIC METHOD (PROB = 0)

CASE NO. 1 SEISMIC COEFFICIENT (SEIC) =0.000

NO. OF BOUNDARY LINES (NBL) = 5

NO. OF POINTS ON BOUNDARY LINE 1 = 2
1 X COORD.=-70 Y COORD.=-10
2 X COORD.= 70 Y COORD.=-10

NO. OF POINTS ON BOUNDARY LINE 2 = 2
1 X COORD.=-45 Y COORD.= 0
2 X COORD.= 45 Y COORD.=-1

NO. OF POINTS ON BOUNDARY LINE 3 = 3
1 X COORD.=-70 Y COORD.= 0
2 X COORD.=-45 Y COORD.= 0
3 X COORD.=-13 Y COORD.= 14

NO. OF POINTS ON BOUNDARY LINE 4 = 3
1 X COORD.= 45 Y COORD.= 0
2 X COORD.= 45 Y COORD.=-1
3 X COORD.= 70 Y COORD.=-1

NO. OF POINTS ON BOUNDARY LINE 5 = 6
1 X COORD.=-70 Y COORD.= 14
2 X COORD.=-13 Y COORD.= 14
3 X COORD.=-11 Y COORD.= 16
4 X COORD.= 13 Y COORD.= 16
5 X COORD.= 45 Y COORD.= 0
6 X COORD.= 70 Y COORD.= 0

LINE NO. AND SLOPE OF EACH SEGMENT ARE:

1	0.000				
2	-0.011				
3	0.000	0.438			
4	99999.000	0.000			
5	0.000	1.000	0.000	-0.500	0.000

MIN. DEPTH OF TALLEST SLICE (DMIN) = 0

NO. OF RADIUS CONTROL ZONES (NRCZ) = 1

RADIUS DECREMENT (RDEC) FOR ZONE 1 = 0
NO. OF CIRCLES (NCIR) FOR ZONE 1 = 5
NO. OF BOTTOM LINES (NOL) FOR ZONE 1 = 1

LINE NO. (LINO)	BEG. NO. (NBP)	END NO. (NEP)
1	1	2

ENGLISH UNITS ARE USED WITH DISTANCE IN FEET AND FORCE IN POUND.

SOIL No.	ENVELOPE (TSSE)	COHESION (C)	FRIC. ANGLE (PHID)	UNIT WEIGHTT (G)
1	1	20.000	26.000	115.000
2	1	20.000	30.000	128.000
3	1	0.000	0.000	62.400
4	1	0.000	0.000	62.400

USE PHREATIC SURFACE
USE GRID
NO. OF SLICES (NSLI) = 10
NO. OF ADD. CIRCLES (NAC) = 10
ANALYSIS BY SIMPLIFIED BISHOP METHOD (MTHD=2)
NUMBER OF FORCES (NFO)= 0
SOFT SOIL NUMBER (SSN)= 0

NO. OF POINTS ON WATER TABLE (NPWT) = 4

1	X COORD.=-70	Y COORD.= 14
2	X COORD.=-13	Y COORD.= 14
3	X COORD.= 45	Y COORD.= 0
4	X COORD.= 70	Y COORD.= 0

NO. OF SOILS WITH DIFFERENT WATER TABLE (NSDW) = 0
NO. OF SOILS WITH DIFFERENT PORE PRESSURE RATIO (NSDP) = 0

INPUT COORD. OF GRID POINTS 1,2,AND 3

POINT 1 X COORD. = 40 Y COORD. = 25
POINT 2 X COORD. = 40 Y COORD. = 0
POINT 3 X COORD. = 60 Y COORD. = 0

X INCREMENT (XINC) = 2.4 Y INCREMENT (YINC) = 2.4
NO. OF DIVISIONS BETWEEN POINTS 1 AND 2 (ND12) = 5
NO. OF DIVISIONS BETWEEN POINTS 2 AND 3 (ND23) = 4
ONLY A SUMMARY TABLE IS PRINTED (NPRT = 0)
SLICES WILL BE SUBDIVIDED

AUTOMATIC SEARCH WILL FOLLOW AFTER GRID

FACTORS OF SAFETY BASED ON GRID

IN THE FOLLOWING TABLE WARNING INDICATES HOW MANY TIMES THE MAXIMUM RADIUS IS LIMITED BY THE END POINTS OF GROUND LINES

CENTER X COORDINATE	CENTER Y COORDINATE	NO. OF CIRCLE TOTAL	CRITIC.	RADIUS	LOWEST F.S.	WARNING
40.0	25.0	25	3	29.050	1.100	0
40.0	20.0	25	16	24.000	1.110	0

40.0	15.0	25	18	18.718	1.139	0
40.0	10.0	1	1	20.000	1000.000	0
40.0	5.0	1	1	15.000	1000.000	0
40.0	0.0	1	1	10.000	1000.000	0
45.0	25.0	25	14	27.876	1.147	0
45.0	20.0	25	13	23.394	1.160	0
45.0	15.0	25	16	17.839	1.179	0
45.0	10.0	25	21	12.160	1.211	0
45.0	5.0	1	1	15.000	1000.000	0
45.0	0.0	1	1	10.000	1000.000	0
50.0	25.0	24	20	29.723	1.400	1
50.0	20.0	25	16	24.820	1.517	1
50.0	15.0	25	21	19.909	1.726	0
50.0	10.0	24	14	18.182	2.142	0
50.0	5.0	4	1	15.000	3.099	0
50.0	0.0	1	1	10.000	1000.000	0
55.0	25.0	2	1	29.155	2.578	1
55.0	20.0	2	1	25.000	3.085	1
55.0	15.0	2	1	21.213	3.923	1
55.0	10.0	2	1	18.028	5.187	1
55.0	5.0	2	1	15.000	9.299	0
55.0	0.0	1	1	10.000	1000.000	0
60.0	25.0	1	1	26.926	1000.000	1
60.0	20.0	1	1	22.361	1000.000	1
60.0	15.0	1	1	18.028	1000.000	1
60.0	10.0	1	1	14.142	1000.000	1
60.0	5.0	1	1	11.180	1000.000	1
60.0	0.0	1	1	10.000	1000.000	0

GRID IS EXPANDED AS FOLLOWS SO MINIMUM FACTOR OF SAFETY FALLS WITHIN THE GRID

40.0	30.0	25	14	34.399	1.110	0
45.0	30.0	25	19	33.275	1.136	1
50.0	30.0	25	14	34.912	1.329	1
55.0	30.0	2	1	33.541	2.202	1
60.0	30.0	1	1	31.623	1000.000	1
35.0	30.0	25	20	34.869	1.193	0
35.0	25.0	25	20	30.022	1.175	0
35.0	20.0	25	19	25.477	1.181	0
35.0	15.0	1	1	25.000	1000.000	0
35.0	10.0	1	1	20.000	1000.000	0
35.0	5.0	1	1	15.000	1000.000	0
35.0	0.0	1	1	10.000	1000.000	0

LOWEST FACTOR OF SAFETY AT EACH GRID POINT IS TABULATED BELOW

COORDINATE	35.000	40.000	45.000	50.000	55.000	60.000
30.000	1.193	1.110	1.136	1.329	2.202	1000.000
25.000	1.175	1.100	1.147	1.400	2.578	1000.000
20.000	1.181	1.110	1.160	1.517	3.085	1000.000
15.000	1000.000	1.139	1.179	1.726	3.923	1000.000
10.000	1000.000	1000.000	1.211	2.142	5.187	1000.000
5.000	1000.000	1000.000	1000.000	3.099	9.299	1000.000
0.000	1000.000	1000.000	1000.000	1000.000	1000.000	1000.000

ONLY ONE MINIMUM F.S. OF 1.100 EXISTS AT (40.000,25.000)

AT POINT (40.0 , 25.0) RADIUS 29.050
THE MINIMUM FACTOR OF SAFETY IS 1.100

FACTORS OF SAFETY BASED ON SEARCH

IN THE FOLLOWING TABLE WARNING INDICATES HOW MANY TIMES THE MAXIMUM RADIUS IS LIMITED BY THE END POINTS OF GROUND LINES

CENTER X COORDINATE	CENTER Y COORDINATE	NO. OF CIRCLE TOTAL	CRITIC.	RADIUS	LOWEST F.S.	WARNING
40.0	25.0	25	3	29.050	1.100	0
42.4	25.0	25	16	29.228	1.106	0
37.6	25.0	25	13	29.490	1.119	0
40.0	27.4	25	14	31.899	1.104	0
40.0	22.6	25	15	27.017	1.101	0
40.6	25.0	25	3	29.157	1.097	0
41.2	25.0	25	3	29.265	1.098	0
40.6	25.6	25	3	29.732	1.098	0
40.6	24.4	25	15	28.847	1.097	0
40.6	23.8	25	15	28.271	1.098	0
41.2	24.4	25	3	28.690	1.098	0
40.0	24.4	25	3	28.475	1.099	0

AT POINT (40.6 , 24.4) RADIUS 28.847

THE MINIMUM FACTOR OF SAFETY IS 1.097

SUMMARY OF SLICE INFORMATION FOR MOST CRITICAL FAILURE SURFACE

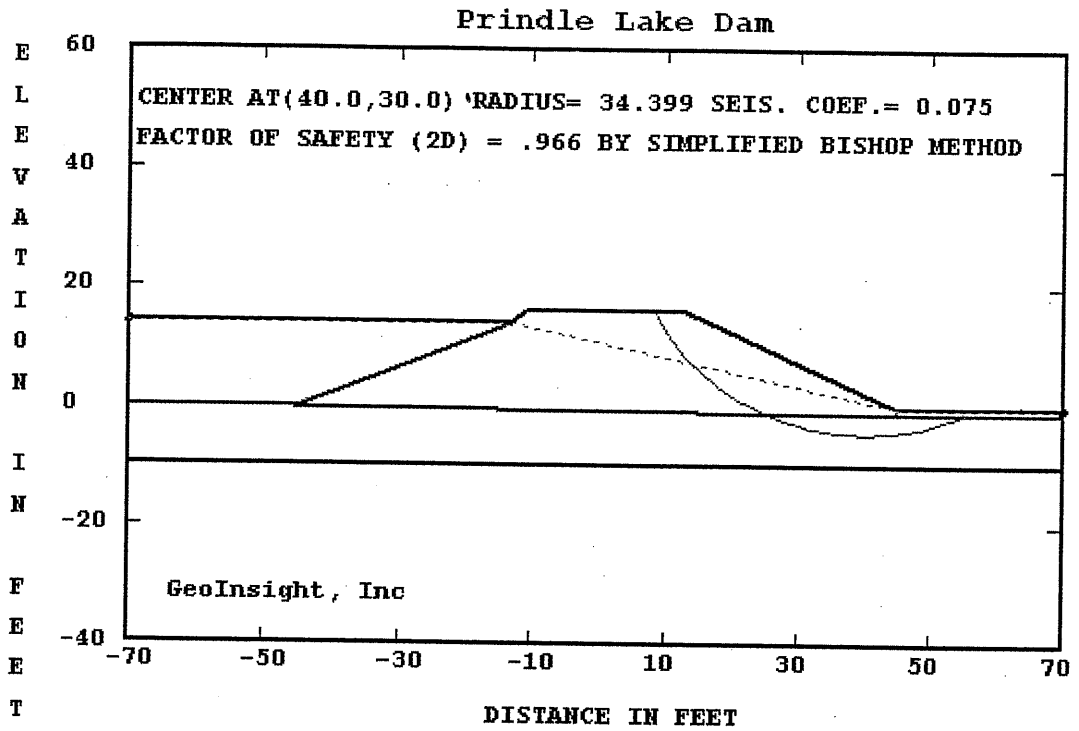
SL. NO.	SOIL NO.	SLICE WIDTH	SLICE HEIGHT	WATER HEIGHT	BOTTOM SINE	TOTAL WEIGHT	EFFEC. WEIGHT	RESIS. MOMENT	DRIVING MOMENT
1	2	4.127	3.991	0.000	-.885	.211E+04	.211E+04	.215E+05	.538E+05
2	2	4.127	7.840	1.167	-.742	.414E+04	.384E+04	.464E+05	.887E+05
3	2	4.127	9.539	3.932	-.599	.504E+04	.403E+04	.567E+05	.871E+05
4	2	1.167	9.977	5.055	-.507	.149E+04	.112E+04	.169E+05	.218E+05
5	1	2.960	10.049	5.661	-.436	.378E+04	.273E+04	.365E+05	.475E+05
6	1	4.127	9.711	6.239	-.313	.501E+04	.341E+04	.480E+05	.453E+05
7	1	4.127	8.677	6.272	-.170	.442E+04	.280E+04	.412E+05	.216E+05
8	1	4.127	7.022	5.685	-.027	.352E+04	.206E+04	.313E+05	.271E+04
9	1	3.107	5.083	4.681	.099	.189E+04	.980E+03	.155E+05	-.537E+04
10	1	1.020	4.026	4.026	.170	.419E+03	.162E+03	.285E+04	-.206E+04
11	1	4.127	3.459	3.459	.259	.142E+04	.534E+03	.972E+04	-.107E+05
12	1	4.127	2.007	2.007	.402	.736E+03	.219E+03	.542E+04	-.932E+04
							SUM	.332E+06	.341E+06

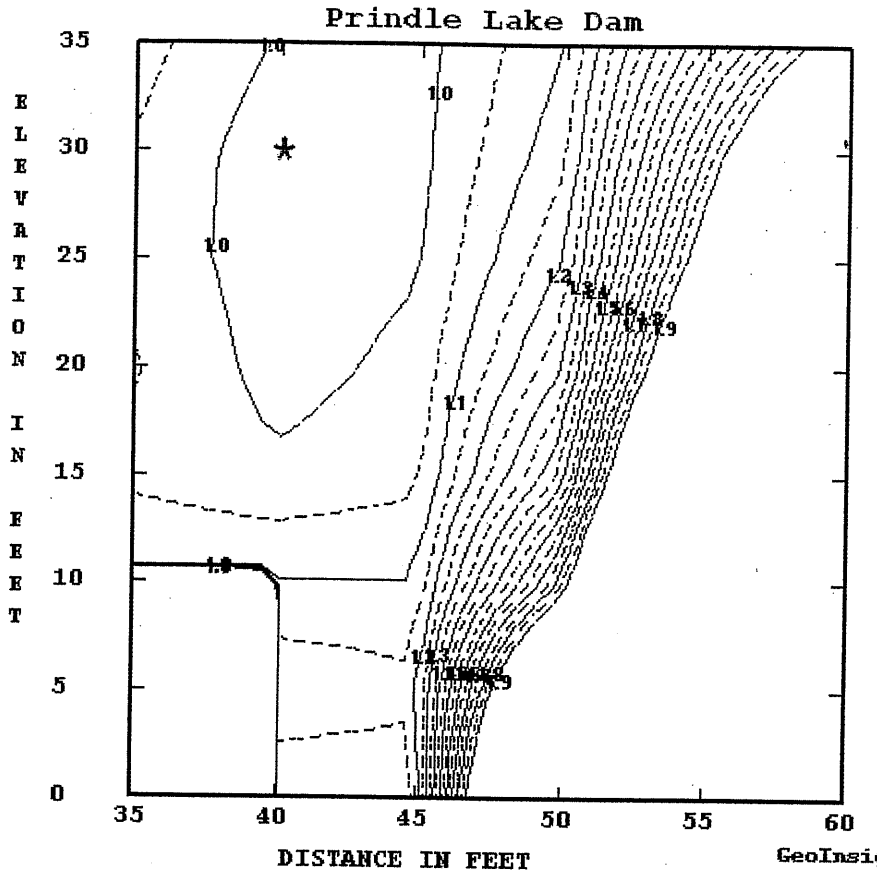
AT CENTER (40.600 , 24.400) WITH RADIUS 28.847 AND SEIS. COEFF. 0.00
FACTOR OF SAFETY BY NORMAL METHOD IS 0.974
FACTOR OF SAFETY BY SIMPLIFIED BISHOP METHOD IS 1.097

SUMMARY OF STABILITY ANALYSIS

FACTOR OF FAFETY IS DETERMINED BY SIMPLIFIED BISHOP METHOD
NUMBER OF CASES = 1

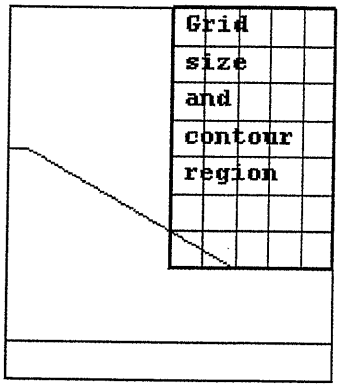
CASE 1 SEISMIC COEFFICIENT = 0
FACTOR OF SAFETY = 1.097


Prindle Lake Dam
CENTER AT(40.0,30.0) RADIUS= 34.399 SEIS. COEF.= 0.075
FACTOR OF SAFETY (2D) = .966 BY SIMPLIFIED BISHOP METHOD
GeoInsight, Inc
ELEVATION IN FEET
60
40
20
0
-20
-40
-70
-50
-30
-10
10
30
50
70
DISTANCE IN FEET



F.S.(2D) = 0.966
by SIMPLIED BISHOP
SEISMIC COEF. = .075

Number on curves indicates factor of safety.
Graph below shows true shape and location of the contour region.



GeoInsight, Inc

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REAME (ROTATIONAL EQUILIBRIUM ANALYSIS OF MULTILAYERED EARTHWORKS)
THIS 2008 VERSION IS LICENSED BY CIVIL ENGINEERING SOFTWARE CENTER TO

                              GeoInsight, Inc

INPUT FILE NAME  -C:\REAME2008\Prindle Lake 3.DAT

TITLE -Prindle Lake Dam

NO. OF STATIC AND SEISMIC CASES (NCASE) = 1

NO. OF NONCIRCULAR FAILURE SURFACES (NNS) = 0

TWO-DIMENSIONAL ANALYSIS ( THREED = 0 )

ANALYSIS BY DETERMINISTIC METHOD   ( PROB = 0 )


CASE NO.  1   SEISMIC COEFFICIENT (SEIC) =0.075

NO. OF BOUNDARY LINES (NBL) = 5

NO. OF POINTS ON BOUNDARY LINE 1 = 2
 1  X COORD.=-70            Y COORD.=-10
 2  X COORD.= 70            Y COORD.=-10

NO. OF POINTS ON BOUNDARY LINE 2 = 2
 1  X COORD.=-45            Y COORD.= 0
 2  X COORD.= 45            Y COORD.=-1

NO. OF POINTS ON BOUNDARY LINE 3 = 3
 1  X COORD.=-70            Y COORD.= 0
 2  X COORD.=-45            Y COORD.= 0
 3  X COORD.=-13            Y COORD.= 14

NO. OF POINTS ON BOUNDARY LINE 4 = 3
 1  X COORD.= 45            Y COORD.= 0
 2  X COORD.= 45            Y COORD.=-1
 3  X COORD.= 70            Y COORD.=-1

NO. OF POINTS ON BOUNDARY LINE 5 = 6
 1  X COORD.=-70            Y COORD.= 14
 2  X COORD.=-13            Y COORD.= 14
 3  X COORD.=-11            Y COORD.= 16
 4  X COORD.= 13            Y COORD.= 16
 5  X COORD.= 45            Y COORD.= 0
 6  X COORD.= 70            Y COORD.= 0

LINE NO. AND SLOPE OF EACH SEGMENT ARE:
 1       0.000
 2      -0.011
 3       0.000        0.438
 4   99999.000        0.000
 5       0.000        1.000        0.000       -0.500        0.000

MIN. DEPTH OF TALLEST SLICE (DMIN) = 0
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NO. OF RADIUS CONTROL ZONES (NRCZ) = 1

RADIUS DECREMENT (RDEC) FOR ZONE 1 = 0
NO. OF CIRCLES (NCIR) FOR ZONE 1 = 5
NO. OF BOTTOM LINES (NOL) FOR ZONE 1 = 1

LINE NO. (LINO)	BEG. NO. (NBP)	END NO. (NEP)
1	1	2

ENGLISH UNITS ARE USED WITH DISTANCE IN FEET AND FORCE IN POUND.

SOIL No.	ENVELOPE (TSSE)	COHESION (C)	FRIC. ANGLE (PHID)	UNIT WEIGHTT (G)
1	1	20.000	26.000	115.000
2	1	20.000	30.000	128.000
3	1	0.000	0.000	62.400
4	1	0.000	0.000	62.400

USE PHREATIC SURFACE
USE GRID
NO. OF SLICES (NSLI) = 10
NO. OF ADD. CIRCLES (NAC) = 10
ANALYSIS BY SIMPLIFIED BISHOP METHOD (MTHD=2)
NUMBER OF FORCES (NFO)= 0
SOFT SOIL NUMBER (SSN)= 0

NO. OF POINTS ON WATER TABLE (NPWT) = 4

1	X COORD.=-70	Y COORD.= 14
2	X COORD.=-13	Y COORD.= 14
3	X COORD.= 45	Y COORD.= 0
4	X COORD.= 70	Y COORD.= 0

NO. OF SOILS WITH DIFFERENT WATER TABLE (NSDW) = 0
NO. OF SOILS WITH DIFFERENT PORE PRESSURE RATIO (NSDP) = 0

INPUT COORD. OF GRID POINTS 1,2,AND 3

POINT 1 X COORD. = 40 Y COORD. = 25
POINT 2 X COORD. = 40 Y COORD. = 0
POINT 3 X COORD. = 60 Y COORD. = 0

X INCREMENT (XINC) = 2.4 Y INCREMENT (YINC) = 2.4
NO. OF DIVISIONS BETWEEN POINTS 1 AND 2 (ND12) = 5
NO. OF DIVISIONS BETWEEN POINTS 2 AND 3 (ND23) = 4
ONLY A SUMMARY TABLE IS PRINTED (NPRT = 0)
SLICES WILL BE SUBDIVIDED

AUTOMATIC SEARCH WILL FOLLOW AFTER GRID

FACTORS OF SAFETY BASED ON GRID

IN THE FOLLOWING TABLE WARNING INDICATES HOW MANY TIMES THE MAXIMUM RADIUS IS LIMITED BY THE END POINTS OF GROUND LINES

CENTER X COORDINATE	CENTER Y COORDINATE	NO. OF CIRCLE TOTAL	CRITIC.	RADIUS	LOWEST F.S.	WARNING
40.0	25.0	25	13	29.861	0.969	0
40.0	20.0	25	16	24.000	0.979	0

40.0	15.0	25	19	18.467	1.010	0
40.0	10.0	1	1	20.000	1000.000	0
40.0	5.0	1	1	15.000	1000.000	0
40.0	0.0	1	1	10.000	1000.000	0
45.0	25.0	25	14	27.876	0.995	0
45.0	20.0	25	16	22.513	1.016	0
45.0	15.0	25	15	18.260	1.037	0
45.0	10.0	15	5	11.155	1.102	0
45.0	5.0	1	1	15.000	1000.000	0
45.0	0.0	1	1	10.000	1000.000	0
50.0	25.0	24	5	30.397	1.201	1
50.0	20.0	25	17	24.669	1.301	1
50.0	15.0	25	22	19.727	1.465	0
50.0	10.0	24	15	17.818	1.813	0
50.0	5.0	4	1	15.000	2.571	0
50.0	0.0	1	1	10.000	1000.000	0
55.0	25.0	2	1	29.155	2.016	1
55.0	20.0	2	1	25.000	2.327	1
55.0	15.0	2	1	21.213	2.834	1
55.0	10.0	2	1	18.028	3.566	1
55.0	5.0	2	1	15.000	5.588	0
55.0	0.0	1	1	10.000	1000.000	0
60.0	25.0	1	1	26.926	1000.000	1
60.0	20.0	1	1	22.361	1000.000	1
60.0	15.0	1	1	18.028	1000.000	1
60.0	10.0	1	1	14.142	1000.000	1
60.0	5.0	1	1	11.180	1000.000	1
60.0	0.0	1	1	10.000	1000.000	0

GRID IS EXPANDED AS FOLLOWS SO MINIMUM FACTOR OF SAFETY FALLS WITHIN THE GRID

40.0	30.0	25	14	34.399	0.966	0
45.0	30.0	25	19	33.275	0.989	1
50.0	30.0	25	15	34.785	1.138	1
55.0	30.0	2	1	33.541	1.823	1
60.0	30.0	1	1	31.623	1000.000	1
40.0	35.0	25	16	38.338	0.994	0
45.0	35.0	25	16	38.116	0.986	1
50.0	35.0	23	13	39.080	1.101	1
55.0	35.0	2	1	38.079	1.579	1
60.0	35.0	1	1	36.401	1000.000	1
35.0	35.0	35	19	40.045	1.067	0
35.0	30.0	25	24	33.586	1.044	0
35.0	25.0	25	11	29.711	1.034	0
35.0	20.0	25	13	24.271	1.053	0
35.0	15.0	1	1	25.000	1000.000	0
35.0	10.0	1	1	20.000	1000.000	0
35.0	5.0	1	1	15.000	1000.000	0
35.0	0.0	1	1	10.000	1000.000	0

LOWEST FACTOR OF SAFETY AT EACH GRID POINT IS TABULATED BELOW

COORDINATE	35.000	40.000	45.000	50.000	55.000	60.000
35.000	1.067	0.994	0.986	1.101	1.579	1000.000
30.000	1.044	0.966	0.989	1.138	1.823	1000.000
25.000	1.034	0.969	0.995	1.201	2.016	1000.000
20.000	1.053	0.979	1.016	1.301	2.327	1000.000

15.000	1000.000	1.010	1.037	1.465	2.834	1000.000
10.000	1000.000	1000.000	1.102	1.813	3.566	1000.000
5.000	1000.000	1000.000	1000.000	2.571	5.588	1000.000
0.000	1000.000	1000.000	1000.000	1000.000	1000.000	1000.000

MINIMUM FACTORS OF SAFETY OCCUR AT THE FOLLOWING 2 CENTERS

FACTOR OF SAFETY = 0.986 AT (45.000,35.000)
FACTOR OF SAFETY = 0.966 AT (40.000,30.000)

AUTOMATIC SEARCH WILL BE MADE ONLY ON THE CENTER WITH THE SMALLEST F.S.

AT POINT (40.0 , 30.0) RADIUS 34.399
THE MINIMUM FACTOR OF SAFETY IS 0.966

FACTORS OF SAFETY BASED ON SEARCH

IN THE FOLLOWING TABLE WARNING INDICATES HOW MANY TIMES THE
MAXIMUM RADIUS IS LIMITED BY THE END POINTS OF GROUND LINES

CENTER X COORDINATE	CENTER Y COORDINATE	NO. OF CIRCLE TOTAL	CRITIC.	RADIUS	LOWEST F.S.	WARNING
40.0	30.0	25	14	34.399	0.966	0
42.4	30.0	25	19	33.226	0.969	0
37.6	30.0	25	14	34.009	1.001	0
40.0	32.4	25	3	36.137	0.980	0
40.0	27.6	25	12	32.642	0.970	0
40.6	30.0	25	3	33.946	0.971	0
39.4	30.0	25	3	33.731	0.976	0
40.0	30.6	25	17	33.851	0.973	0
40.0	29.4	25	3	33.264	0.972	0

AT POINT (40.0 , 30.0) RADIUS 34.399

THE MINIMUM FACTOR OF SAFETY IS 0.966

SUMMARY OF SLICE INFORMATION FOR MOST CRITICAL FAILURE SURFACE

SL. NO.	SOIL NO.	SLICE WIDTH	SLICE HEIGHT	WATER HEIGHT	BOTTOM SINE	TOTAL WEIGHT	EFFEC. WEIGHT	RESIS. MOMENT	DRIVING MOMENT
1	2	4.421	4.167	0.000	-.849	.236E+04	.236E+04	.275E+05	.717E+05
2	2	0.212	7.394	0.000	-.782	.201E+03	.201E+03	.249E+04	.566E+04
3	2	4.633	8.911	1.289	-.711	.528E+04	.491E+04	.679E+05	.137E+06
4	2	4.633	10.521	4.097	-.577	.624E+04	.505E+04	.816E+05	.133E+06
5	2	2.151	10.936	5.389	-.478	.301E+04	.229E+04	.400E+05	.539E+05
6	1	2.482	10.928	5.980	-.411	.345E+04	.253E+04	.392E+05	.538E+05
7	1	4.633	10.520	6.493	-.307	.612E+04	.425E+04	.695E+05	.736E+05
8	1	4.633	9.352	6.523	-.173	.536E+04	.348E+04	.600E+05	.394E+05
9	1	4.633	7.527	5.896	-.038	.426E+04	.255E+04	.458E+05	.110E+05
10	1	3.990	5.265	4.749	.087	.252E+04	.134E+04	.252E+05	-.492E+04
11	1	0.643	3.985	3.985	.155	.261E+03	.101E+03	.214E+04	-.121E+04
12	1	4.633	3.465	3.465	.231	.160E+04	.601E+03	.133E+05	-.115E+05
13	1	4.633	2.011	2.011	.366	.828E+03	.246E+03	.739E+04	-.108E+05
							SUM	.482E+06	.551E+06

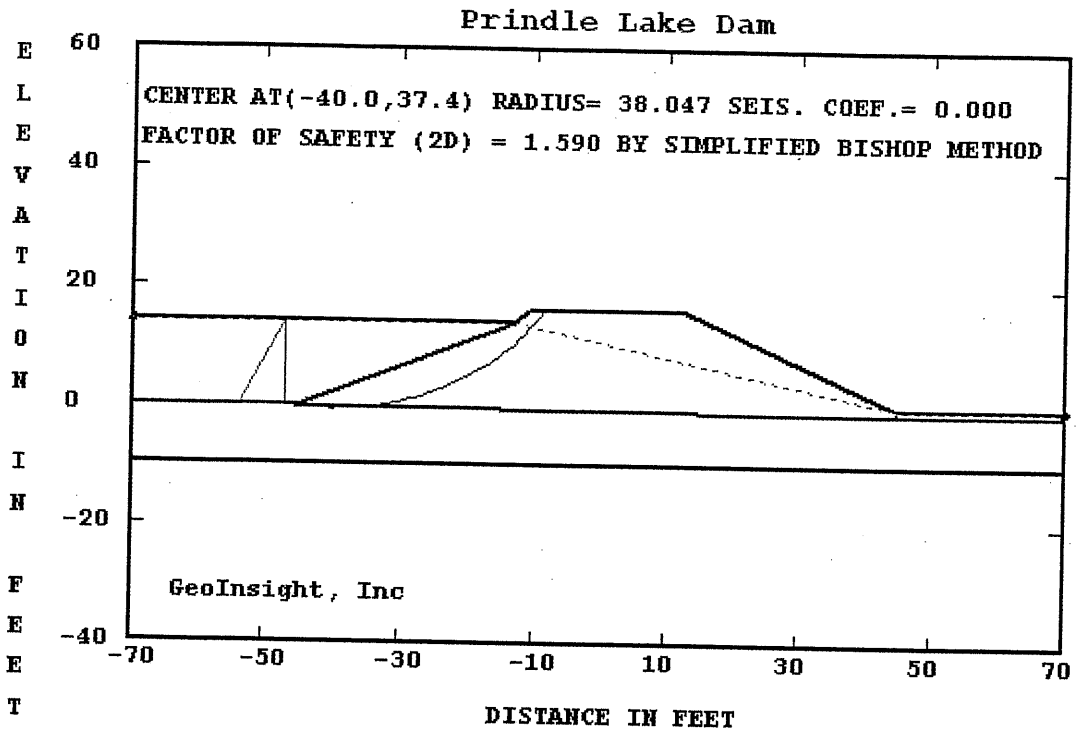
AT CENTER (40.000 , 30.000) WITH RADIUS 34.399 AND SEIS. COEFF. 0.08
FACTOR OF SAFETY BY NORMAL METHOD IS 0.875

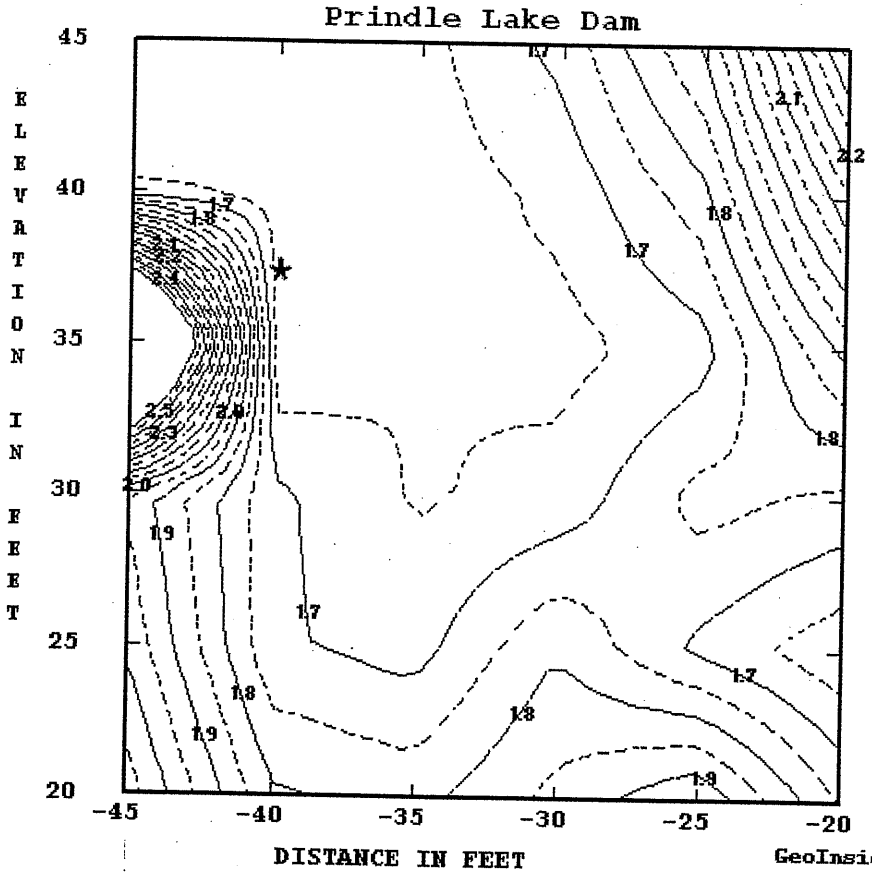
FACTOR OF SAFETY BY SIMPLIFIED BISHOP METHOD IS 0.971

SUMMARY OF STABILITY ANALYSIS

FACTOR OF FAFETY IS DETERMINED BY SIMPLIFIED BISHOP METHOD
NUMBER OF CASES = 1

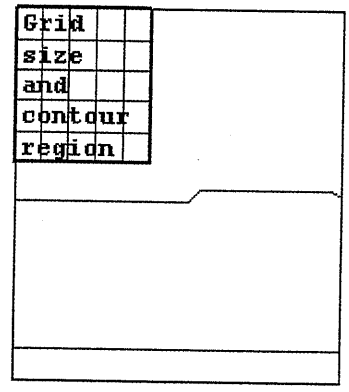
CASE 1 SEISMIC COEFFICIENT = 0.075
FACTOR OF SAFETY = 0.966


Prindle Lake Dam
CENTER AT(-40.0,37.4) RADIUS= 38.047 SEIS. COEF.= 0.000
FACTOR OF SAFETY (2D) = 1.590 BY SIMPLIFIED BISHOP METHOD
GeoInsight, Inc
ELEVATION IN FEET
60
40
20
0
-20
-40
-70
-50
-30
-10
10
30
50
70
DISTANCE IN FEET



F.S.(2D) = 1.590
by SIMPLIED BISHOP
SEISMIC COEF. = 0

Number on curves indicates factor of safety.
Graph below shows true shape and location of the contour region.



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REAME (ROTATIONAL EQUILIBRIUM ANALYSIS OF MULTILAYERED EARTHWORKS)
THIS 2008 VERSION IS LICENSED BY CIVIL ENGINEERING SOFTWARE CENTER TO

                          GeoInsight, Inc

INPUT FILE NAME  -C:\REAME2008\prindle(test).DAT

TITLE -Prindle Lake Dam

NO. OF STATIC AND SEISMIC CASES (NCASE) = 1

NO. OF NONCIRCULAR FAILURE SURFACES (NNS) = 0

TWO-DIMENSIONAL ANALYSIS ( THREED = 0 )

ANALYSIS BY DETERMINISTIC METHOD   ( PROB = 0 )


CASE NO.  1   SEISMIC COEFFICIENT (SEIC) =0.000

NO. OF BOUNDARY LINES (NBL) = 5

NO. OF POINTS ON BOUNDARY LINE 1 = 2
 1  X COORD.=-70            Y COORD.=-10
 2  X COORD.= 70            Y COORD.=-10

NO. OF POINTS ON BOUNDARY LINE 2 = 2
 1  X COORD.=-45            Y COORD.= 0
 2  X COORD.= 45            Y COORD.=-1

NO. OF POINTS ON BOUNDARY LINE 3 = 3
 1  X COORD.=-70            Y COORD.= 0
 2  X COORD.=-45            Y COORD.= 0
 3  X COORD.=-13            Y COORD.= 14

NO. OF POINTS ON BOUNDARY LINE 4 = 3
 1  X COORD.= 45            Y COORD.= 0
 2  X COORD.= 45            Y COORD.=-1
 3  X COORD.= 70            Y COORD.=-1

NO. OF POINTS ON BOUNDARY LINE 5 = 6
 1  X COORD.=-70            Y COORD.= 14
 2  X COORD.=-13            Y COORD.= 14
 3  X COORD.=-11            Y COORD.= 16
 4  X COORD.= 13            Y COORD.= 16
 5  X COORD.= 45            Y COORD.= 0
 6  X COORD.= 70            Y COORD.= 0

LINE NO. AND SLOPE OF EACH SEGMENT ARE:
 1       0.000
 2      -0.011
 3       0.000       0.438
 4   99999.000       0.000
 5       0.000       1.000       0.000       -0.500       0.000

MIN. DEPTH OF TALLEST SLICE (DMIN) = 0
```

NO. OF RADIUS CONTROL ZONES (NRCZ) = 1

RADIUS DECREMENT (RDEC) FOR ZONE 1 = 0
NO. OF CIRCLES (NCIR) FOR ZONE 1 = 5
NO. OF BOTTOM LINES (NOL) FOR ZONE 1 = 1

LINE NO. (LINO)	BEG. NO. (NBP)	END NO. (NEP)
1	1	2

ENGLISH UNITS ARE USED WITH DISTANCE IN FEET AND FORCE IN POUND.

SOIL No.	ENVELOPE (TSSE)	COHESION (C)	FRIC. ANGLE (PHID)	UNIT WEIGHTT (G)
1	1	20.000	26.000	115.000
2	1	20.000	30.000	128.000
3	1	0.000	0.000	62.400
4	1	0.000	0.000	62.400

USE PHREATIC SURFACE
USE GRID
NO. OF SLICES (NSLI) = 10
NO. OF ADD. CIRCLES (NAC) = 3
ANALYSIS BY SIMPLIFIED BISHOP METHOD (MTHD=2)
NUMBER OF FORCES (NFO)= 0
SOFT SOIL NUMBER (SSN)= 0

NO. OF POINTS ON WATER TABLE (NPWT) = 4

1	X COORD.=-70	Y COORD.= 14
2	X COORD.=-13	Y COORD.= 14
3	X COORD.= 45	Y COORD.= 0
4	X COORD.= 70	Y COORD.= 0

NO. OF SOILS WITH DIFFERENT WATER TABLE (NSDW) = 0
NO. OF SOILS WITH DIFFERENT PORE PRESSURE RATIO (NSDP) = 0

INPUT COORD. OF GRID POINTS 1,2,AND 3

POINT 1 X COORD. =-40 Y COORD. = 45
POINT 2 X COORD. =-40 Y COORD. = 20
POINT 3 X COORD. =-20 Y COORD. = 20

X INCREMENT (XINC) = 2.4 Y INCREMENT (YINC) = 2.4
NO. OF DIVISIONS BETWEEN POINTS 1 AND 2 (ND12) = 5
NO. OF DIVISIONS BETWEEN POINTS 2 AND 3 (ND23) = 4
ONLY A SUMMARY TABLE IS PRINTED (NPRT = 0)
SLICES WILL BE SUBDIVIDED

AUTOMATIC SEARCH WILL FOLLOW AFTER GRID

FACTORS OF SAFETY BASED ON GRID

IN THE FOLLOWING TABLE WARNING INDICATES HOW MANY TIMES THE MAXIMUM RADIUS IS LIMITED BY THE END POINTS OF GROUND LINES

CENTER X COORDINATE	CENTER Y COORDINATE	NO. OF CIRCLE TOTAL	CRITIC.	RADIUS	LOWEST F.S.	WARNING
-40.0	45.0	2	1	43.139	1.616	1
-40.0	40.0	2	1	39.699	1.631	1

-40.0	35.0	3	1	36.620	1.598	1
-40.0	30.0	3	1	34.000	1.711	1
-40.0	25.0	9	7	26.715	1.708	1
-40.0	20.0	6	3	20.400	1.809	0
-35.0	45.0	10	3	40.453	1.635	1
-35.0	40.0	10	3	36.560	1.621	1
-35.0	35.0	10	3	32.890	1.651	1
-35.0	30.0	10	9	31.739	1.644	1
-35.0	25.0	10	9	27.800	1.678	0
-35.0	20.0	10	9	22.800	1.780	0
-30.0	45.0	11	9	37.862	1.719	1
-30.0	40.0	7	5	33.598	1.663	1
-30.0	35.0	7	5	29.400	1.633	0
-30.0	30.0	7	4	25.600	1.667	0
-30.0	25.0	10	10	21.800	1.797	0
-30.0	20.0	10	9	22.800	1.863	0
-25.0	45.0	13	12	34.547	1.911	1
-25.0	40.0	13	5	30.800	1.783	0
-25.0	35.0	6	5	25.800	1.678	0
-25.0	30.0	6	5	20.800	1.769	0
-25.0	25.0	11	10	18.200	1.688	0
-25.0	20.0	11	4	15.600	1.949	0
-20.0	45.0	14	12	34.060	2.376	0
-20.0	40.0	14	13	28.069	2.142	0
-20.0	35.0	14	13	23.400	1.917	0
-20.0	30.0	14	13	18.400	1.734	0
-20.0	25.0	7	6	14.600	1.612	0
-20.0	20.0	7	5	10.800	1.747	0

GRID IS EXPANDED AS FOLLOWS SO MINIMUM FACTOR OF SAFETY FALLS WITHIN THE GRID

-45.0	45.0	1	1	39.825	1000.000	1
-45.0	40.0	1	1	36.069	1000.000	1
-45.0	35.0	1	1	32.650	3.268	1
-45.0	30.0	4	1	29.682	1.934	1
-45.0	25.0	2	1	27.313	1.986	1
-45.0	20.0	5	3	20.782	2.080	1

LOWEST FACTOR OF SAFETY AT EACH GRID POINT IS TABULATED BELOW

COORDINATE	-45.000	-40.000	-35.000	-30.000	-25.000	-20.000
45.000	1000.000	1.616	1.635	1.719	1.911	2.376
40.000	1000.000	1.631	1.621	1.663	1.783	2.142
35.000	3.268	1.598	1.651	1.633	1.678	1.917
30.000	1.934	1.711	1.644	1.667	1.769	1.734
25.000	1.986	1.708	1.678	1.797	1.688	1.612
20.000	2.080	1.809	1.780	1.863	1.949	1.747

MINIMUM FACTORS OF SAFETY OCCUR AT THE FOLLOWING 6 CENTERS

FACTOR OF SAFETY = 1.616 AT (-40.000,45.000)
FACTOR OF SAFETY = 1.621 AT (-35.000,40.000)
FACTOR OF SAFETY = 1.598 AT (-40.000,35.000)
FACTOR OF SAFETY = 1.633 AT (-30.000,35.000)
FACTOR OF SAFETY = 1.644 AT (-35.000,30.000)
FACTOR OF SAFETY = 1.612 AT (-20.000,25.000)

AUTOMATIC SEARCH WILL BE MADE ONLY ON THE CENTER WITH THE SMALLEST F.S.

AT POINT (-40.0 , 35.0) RADIUS 36.620
THE MINIMUM FACTOR OF SAFETY IS 1.598

FACTORS OF SAFETY BASED ON SEARCH

IN THE FOLLOWING TABLE WARNING INDICATES HOW MANY TIMES THE MAXIMUM RADIUS IS LIMITED BY THE END POINTS OF GROUND LINES

CENTER X COORDINATE	CENTER Y COORDINATE	NO. OF CIRCLE TOTAL	CRITIC.	RADIUS	LOWEST F.S.	WARNING
-40.0	35.0	3	1	36.620	1.598	1
-37.6	35.0	9	6	35.969	1.611	1
-42.4	35.0	2	1	34.681	1.768	1
-40.0	37.4	2	1	38.047	1.590	1
-40.0	39.8	2	1	39.568	1.632	1
-37.6	37.4	3	1	39.966	1.643	1
-42.4	37.4	2	1	36.185	1.807	1
-39.4	37.4	3	1	38.522	1.599	1
-40.6	37.4	2	1	37.576	1.639	1
-40.0	38.0	2	1	38.419	1.602	1
-40.0	36.8	3	1	37.681	1.596	1

AT POINT (-40.0 , 37.4) RADIUS 38.047

THE MINIMUM FACTOR OF SAFETY IS 1.590

SUMMARY OF SLICE INFORMATION FOR MOST CRITICAL FAILURE SURFACE

SL. NO.	SOIL NO.	SLICE WIDTH	SLICE HEIGHT	WATER HEIGHT	BOTTOM SINE	TOTAL WEIGHT	EFFEC. WEIGHT	RESIS. MOMENT	DRIVING MOMENT
1	1	3.844	14.308	14.308	-.133	.349E+04	.624E+02	.410E+04	-.218E+06
2	1	3.844	14.627	14.627	-.032	.406E+04	.546E+03	.131E+05	-.494E+04
3	1	3.844	14.556	14.556	.069	.445E+04	.958E+03	.207E+05	.117E+05
4	1	1.730	14.260	14.260	.142	.208E+04	.543E+03	.113E+05	.113E+05
5	2	2.114	13.933	13.933	.193	.258E+04	.739E+03	.176E+05	.189E+05
6	2	3.844	13.222	13.222	.271	.467E+04	.149E+04	.346E+05	.481E+05
7	2	3.844	11.914	11.914	.372	.445E+04	.159E+04	.355E+05	.629E+05
8	2	3.844	10.118	10.118	.473	.399E+04	.156E+04	.335E+05	.718E+05
9	2	3.844	7.749	7.749	.574	.324E+04	.139E+04	.285E+05	.709E+05
10	2	3.231	4.940	4.940	.667	.189E+04	.897E+03	.180E+05	.481E+05
11	2	0.614	3.400	3.019	.718	.267E+03	.151E+03	.299E+04	.729E+04
12	2	1.386	3.329	1.707	.744	.591E+03	.443E+03	.808E+04	.167E+05
13	2	2.458	1.703	0.000	.795	.536E+03	.536E+03	.102E+05	.162E+05
							SUM	.238E+06	.161E+06

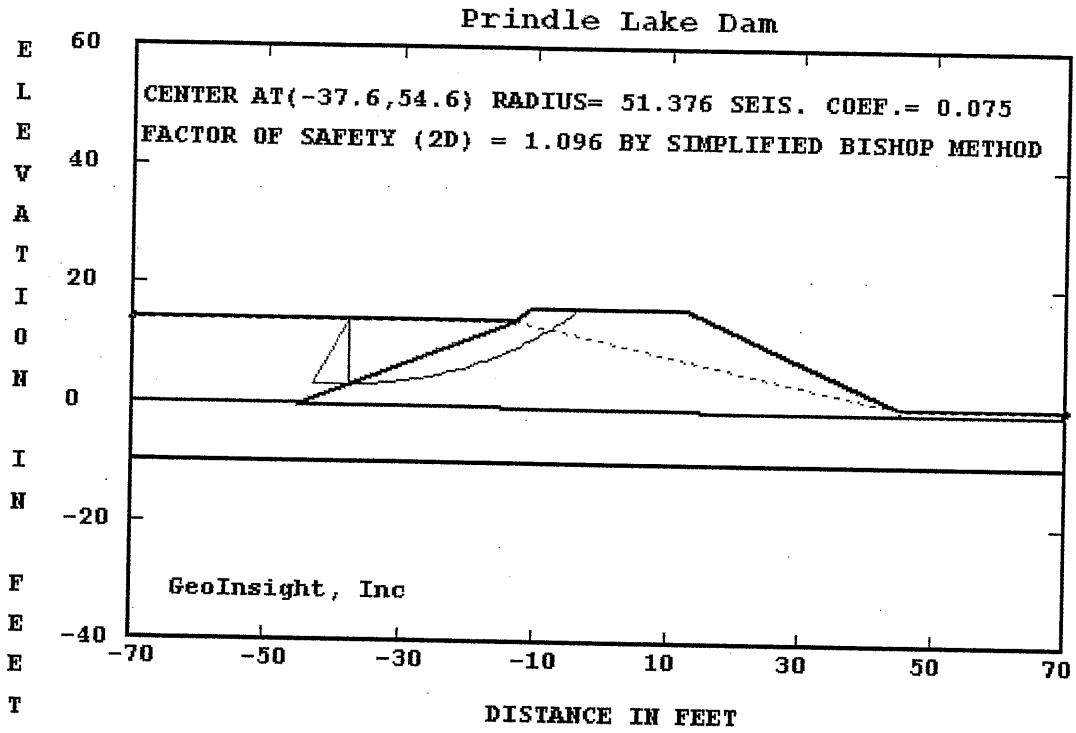
AT CENTER (-40.000 , 37.400) WITH RADIUS 38.047 AND SEIS. COEFF. 0.00
FACTOR OF SAFETY BY NORMAL METHOD IS 1.479
FACTOR OF SAFETY BY SIMPLIFIED BISHOP METHOD IS 1.590

SUMMARY OF STABILITY ANALYSIS

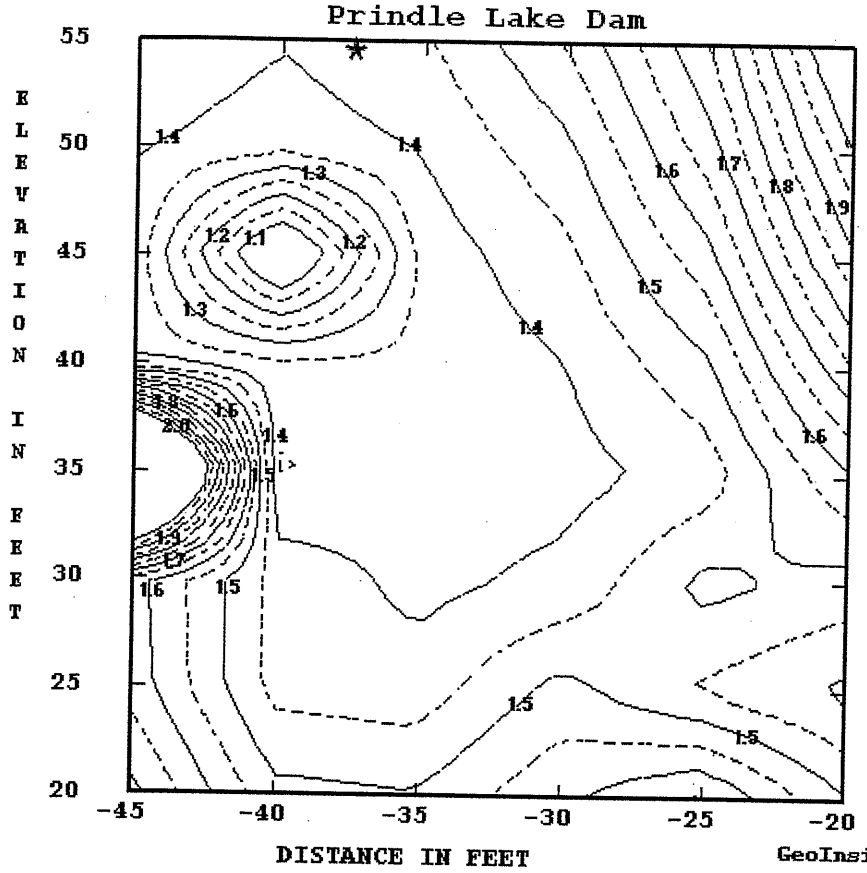
FACTOR OF FAFETY IS DETERMINED BY SIMPLIFIED BISHOP METHOD
NUMBER OF CASES = 1

CASE 1 SEISMIC COEFFICIENT = 0

FACTOR OF SAFETY = 1.590

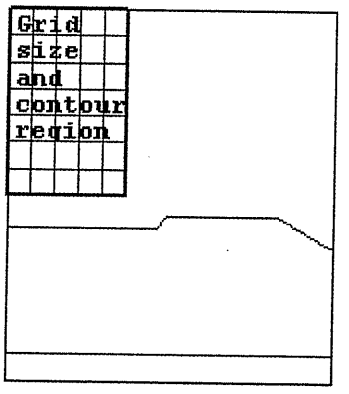

Prindle Lake Dam
CENTER AT(-37.6,54.6) RADIUS= 51.376 SEIS. COEF.= 0.075
FACTOR OF SAFETY (2D) = 1.096 BY SIMPLIFIED BISHOP METHOD
GeoInsight, Inc
ELEVATION IN FEET
60
40
20
0
-20
-40
-70
-50
-30
-10
10
30
50
70
DISTANCE IN FEET

Prindle Lake Dam



F.S.(2D) = 1.096
by SIMPLIED BISHOP
SEISMIC COEF. = .075

Number on curves indicates factor of safety.
Graph below shows true shape and location of the contour region.

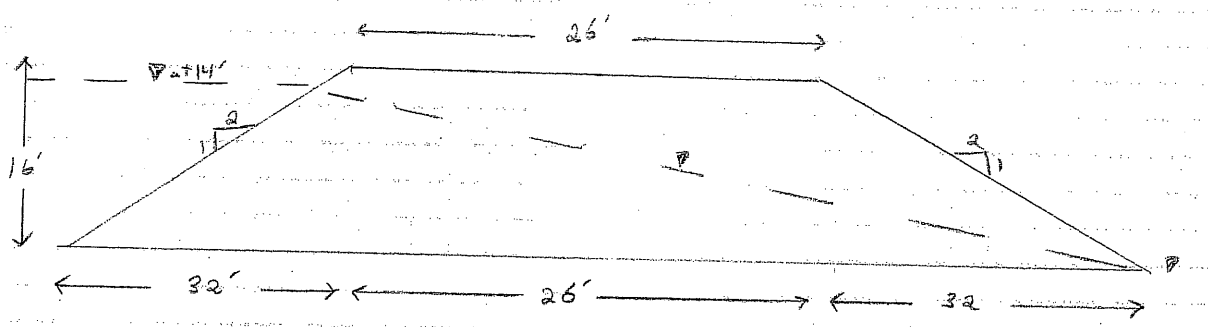


GeoInsight, Inc

```
Prindle Lake Dam
 1                (2) NCASE
 0.075            (3) SEIC
 0   0   0        (5) NNS THREED PROB
 5                (6) NBL
 2                (7) NPBL  ( LINE 1 )
-70  -10           (8) XBL YBL
 70  -10           (8) XBL YBL
 2                (7) NPBL  ( LINE 2 )
-45   0            (8) XBL YBL
 45  -1            (8) XBL YBL
 3                (7) NPBL  ( LINE 3 )
-70   0            (8) XBL YBL
-45   0            (8) XBL YBL
-13   14           (8) XBL YBL
 3                (7) NPBL  ( LINE 4 )
 45   0            (8) XBL YBL
 45  -1            (8) XBL YBL
 70  -1            (8) XBL YBL
 6                (7) NPBL  ( LINE 5 )
-70   14           (8) XBL YBL
-13   14           (8) XBL YBL
-11   16           (8) XBL YBL
 13   16           (8) XBL YBL
 45   0            (8) XBL YBL
 70   0            (8) XBL YBL
 0   1             (11) DMIN NRCZ
 0   5             (13) RDEC NCIR  ( ZONE 1 )
 1                 (14) NOL
 1   1   2         (15) LINO NBP NEP
 0                 (18) UNIT
 1  20  26  115              (21) TSSE C PHID G  ( SOIL 1 )
 1  20  30  128              (21) TSSE C PHID G  ( SOIL 2 )
 1   0  0   62.4             (21) TSSE C PHID G  ( SOIL 3 )
 1   0  0   62.4             (21) TSSE C PHID G  ( SOIL 4 )
 1                 (27) NSPG
 0   10   3   2   0   0      (28) NSRCH NSLI NAC MTHD NFO SSN
 4                 (31) NPWT
-70   14           (32) XWT YWT
-13   14           (32) XWT YWT
 45   0            (32) XWT YWT
 70   0            (32) XWT YWT
 0                 (34) NSDW
 0                 (41) NSDP
 0                 (45) NLRU
-40   45           (61) X Y
-40   20           (61) X Y
-20   20           (61) X Y
 2.4   2.4          (63) XINC YINC
 5   4             (64) ND12 ND23
 0                 (67) NPRT
 1                 (68) NSUB
```

GeoInsight	PROJ. NO: 5482-000	PAGE	OF
CLIENT: LET	DESIGNED BY: MRF	DATE: 4/28/08	
PROJECT: Prindle Lake Dam	CHECKED BY:	DATE:	
	APPROVED BY:	DATE:	

From 9.7.2 of Hydrology and Hydraulic Systems by Ram S. Gupta



$L = 90$

$i = \frac{H}{L} = \frac{16}{90} = .178$

$q = (\text{permeability})(i)(\text{pervious stratum})$

Assumed permeability $= 1 \times 10^{-4}$ cm/sec $\Rightarrow 2.11 \times 10^{-5} \frac{ft}{sec}$

pervious stratum $\approx 10'$

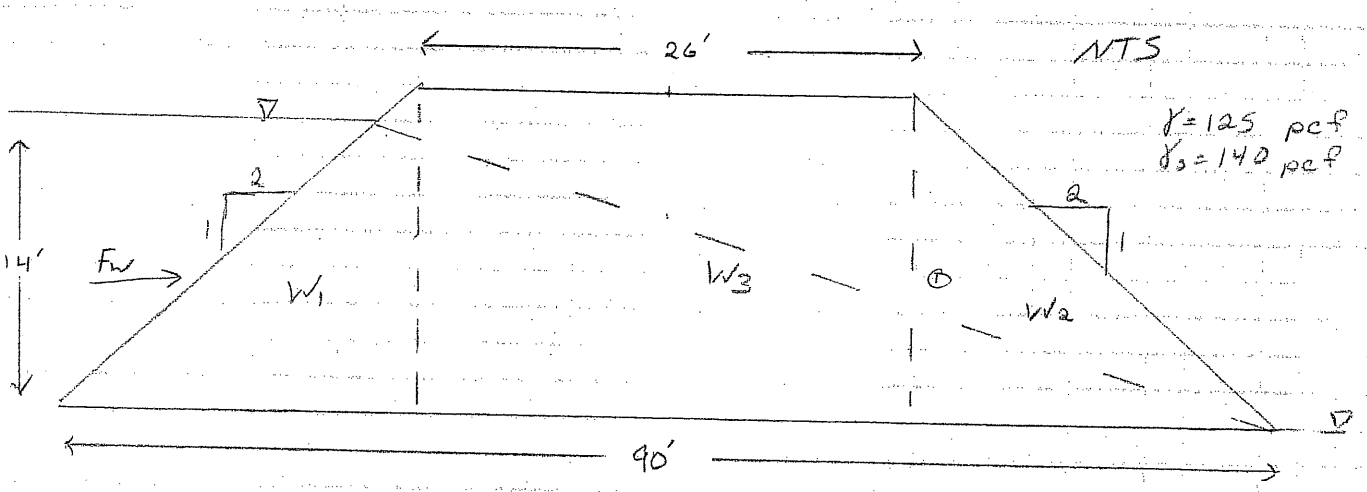
$$q = (2.11 \times 10^{-5})(.18)(10') = 3.798 \times 10^{-5} \text{ cfs} \Rightarrow \boxed{3.3 \frac{ft^2}{day}}$$

$$\left(3.3 \frac{ft^2}{day}\right)(\text{Length of dam})$$

$$\left(3.3 \frac{ft^2}{day}\right)(235 \text{ ft}) = \boxed{775.5 \frac{ft^3}{day}}$$

GeoInsight	PROJ. NO: 5482-000	PAGE	OF
CLIENT: LEI	DESIGNED BY: MRF	DATE: 4/28/08	
PROJECT: Prindle Lake Dam	CHECKED BY:	DATE:	
	APPROVED BY:	DATE:	

Sliding Evaluation - Normal Pool



$F_w = \frac{1}{2}\gamma(h^2) = \frac{1}{2}(62.4)(14^2) = 6{,}115 \frac{lb}{ft}$ - 1 ft of width

$W_1 = (\frac{1}{2})(32)(16)(140) = 71{,}680$ lb

$W_2 = \frac{(\frac{1}{2})(32)(16)(140)}{2} + \frac{(\frac{1}{2})(32)(16)(125)}{2} = 33{,}920$ lb

① Water x at 4' bgs. Assume 8'

$W_3 = (\frac{1}{2})(26)(16)(140) + (\frac{1}{2})(26)(16)(125) = 55{,}120$ lb

$W_T = 160{,}720$ acting over effective length of 90'

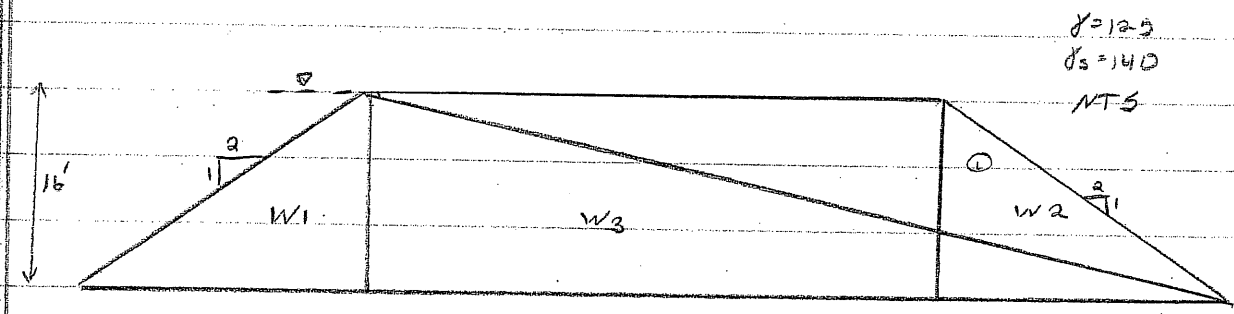
Assume $c = 0$

$f_r = (w)\tan\phi$ $\quad \phi = 26°$ $\quad f_r = (160{,}720)(\tan 26°)$

$f_r = 78{,}388 \frac{lb}{ft}$ - ft of width

F.S. Sliding $= \frac{f_r}{f_w} = \frac{78{,}388}{6{,}115} = \boxed{12.8 > 1.5 \text{ OK}}$

Sliding Elevation - Max pool



$F_w = \frac{1}{2}\gamma(h^2) = \frac{1}{2}(62.4)(16^2) = 7{,}987.2$ · 1 ft of width

$W_1 = \frac{1}{2}(32)(16)(140) = 71{,}680$ lb

$W_2 = 33{,}920$ from Normal Pool elev

$W_3 = (26)(9)(140) + \frac{1}{2}(26)(7)(140) + \frac{1}{2}(26)(7)(125) = 56{,}875$

$WT = 162{,}475$ acting over effective length of 90'

Assume $c = 0$

$f_r = (w)(\tan \phi^\circ) \quad \phi = 28^\circ = (162{,}475)(\tan 28^\circ)$

$f_r = 79{,}244 \frac{lb}{ft}$ · ft of width

$F.S \text{ sliding} = \frac{f_r}{f_w} = \frac{79{,}244}{7987} = 9.92 > 1.5$ OK

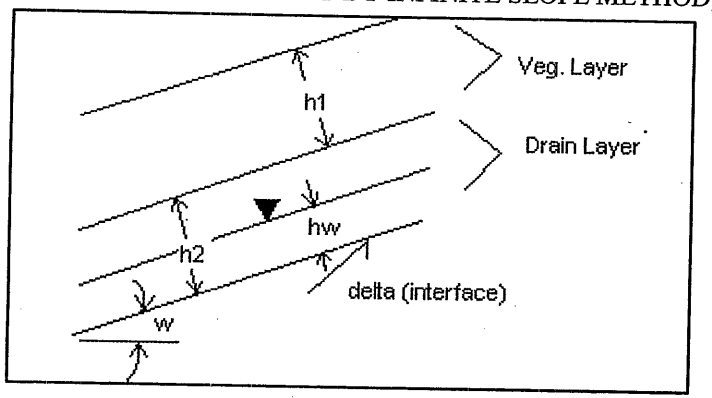
Sliding Elevation - 100 year Flood

12.8	normal
9.92	max
22.72/2 = 11.36	

≈ 11.36 F.S. against sliding

SLOPE STABILITY EVALUATION

LAYER INTERFACE STABILITY BY INFINITE SLOPE METHOD, SATURATED LAYER ON INTERFACE



$$FS = \frac{C + ((H_{cs} * gamma_{sat}) - (Hw*gamma_w)) * TAN(delta_i)}{H_{cs} * gamma_{sat} * TAN(w)}$$

Where:

- FS = Factor of Safety against sliding at discrete layer interface
- C = cohesion of material above layer interface
- H = height of soil above layer interface
- gamma = unit weight of soil above layer interface
- w = angle of slope
- $delta_i$ = interface friction angle above layer interface
- Hw = height of water above layer interface, if any
- 62.4 = unit weight of water

	(enter values)	
H1 =	10	ft
H2 =	6	ft
H3 =		ft
H4 =		ft
H5 =		ft
H6 =		ft
Htot =	16	ft

	(saturated values)	
gamma1 =	135	lb/ft3
gamma2 =	140	lb/ft3
gamma3 =		lb/ft3
gamma4 =		lb/ft3
gamma5 =		lb/ft3
gamma6 =		lb/ft3
$gamma_{avg}$ =	136.9	lb/ft3

rise	1	
run	2	
angle:	26.6	degrees

(calculates the average saturated overburden density)

	(enter values)	
C =	0	lb/ft2
Hw max =	1	ft
Hw avg =	0.5	ft (assume entire drain layer full)
w =	26.6	degrees
$delta_i$ =	28	degrees
w =	0.464022	radians
$delta_i$ =	0.488444	radians

Layer Description:

FS = 1.05 Less than 1.5 !!

Factor of safety less than 1.5 okay only for short term condition.

From Geosynthetics '93 Conference Proceedings
Geosynthetic Landfill Cover Design Methodology and Construction, pg 1135

Appendix B: Hydrologic and Hydraulic Calculations

Site Location: **Prindle Pond Dam**
Oak Ridge Road
Charlton, MA

Prepared for: **Santos Irrevocable Trust**
125 Summer Street
Boston MA 02110 -1624

Prepared by: **Lenard Engineering, Inc.**
19 Midstate Drive
Suite 200
Auburn, MA 01501

Date: **June 6, 2008**

Content: **Spillway Capacity Computations**
Hydraflow Hydrographs Output for Existing Spillway
Hydraflow Hydrographs Output for Proposed Spillway

Spillway Capacity Computations

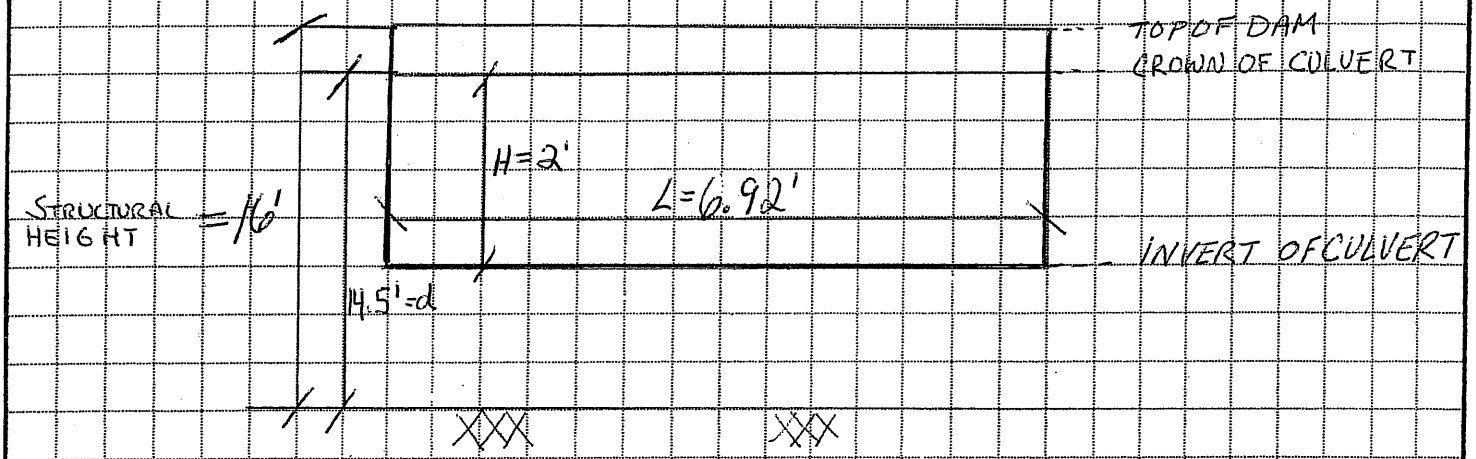
Lenard Engineering, Inc.
Civil, Environmental and Hydrogeological Consultants

JOB 08-709 PRINDLE LAKE DAM
SHEET NO. ____ OF ____
CALCULATED BY CMB DATE 03/31/08
CHECKED BY RJD DATE 03/31/08
SCALE N/A

SPILLWAY CAPACITY

$$Q = \underbrace{CLH^{3/2}}_{\text{WEIR FLOW}} \underbrace{\left[1 + C_1\left(\frac{H}{d}\right)^2\right]}_{\text{VELOCITY HEAD}}$$

GIVEN:
$C = 3.33$
$C_1 = 0.0233\,C^2 = 0.258$
$L = 83'' = 6.92'$
$H = 24'' = 2'$
$d = 14.5'$



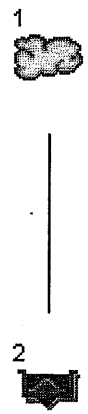
$$Q = 3.33 \times 6.92' \times 2'^{3/2}\left[1 + 0.258\left(\frac{2}{14.5}\right)^2\right]$$
$= 65.18$ CFS $[1.005]$
$= 65.51$ CFS

SPILLWAY CAPACITY = 65.51 CFS (PHASE I REPORT SHOWS 58 cfs)

* 100-YEAR FLOW (SCS) = 24.89 CFS = SPILLWAY DESIGN FLOOD
*(SEE ATTACHED HYDRAFLOW MODEL) DESIGN STORM FLOW

SPILLWAY CAPACITY > SPILLWAY DESIGN FLOOD

Hydraflow Hydrographs Output for Existing Spillway



Legend

Hyd.	Origin	Description
1	SCS Runoff	Prindle Lake
2	Reservoir	Prindle Dam Outflow

Hydrograph Summary Report

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to peak (min)	Volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Maximum storage (cuft)	Hydrograph description
1	SCS Runoff	514.23	6	738	2,678,245	----	------	------	Prindle Lake
2	Reservoir	16.18	6	1194	2,595,062	1	705.78	2,079,550	Prindle Dam Outflow

PRINDLE-EXISTING SPILLWAY.gpw Return Period: 25 Year Thursday, Jun 19 2008, 3:18 PM

Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

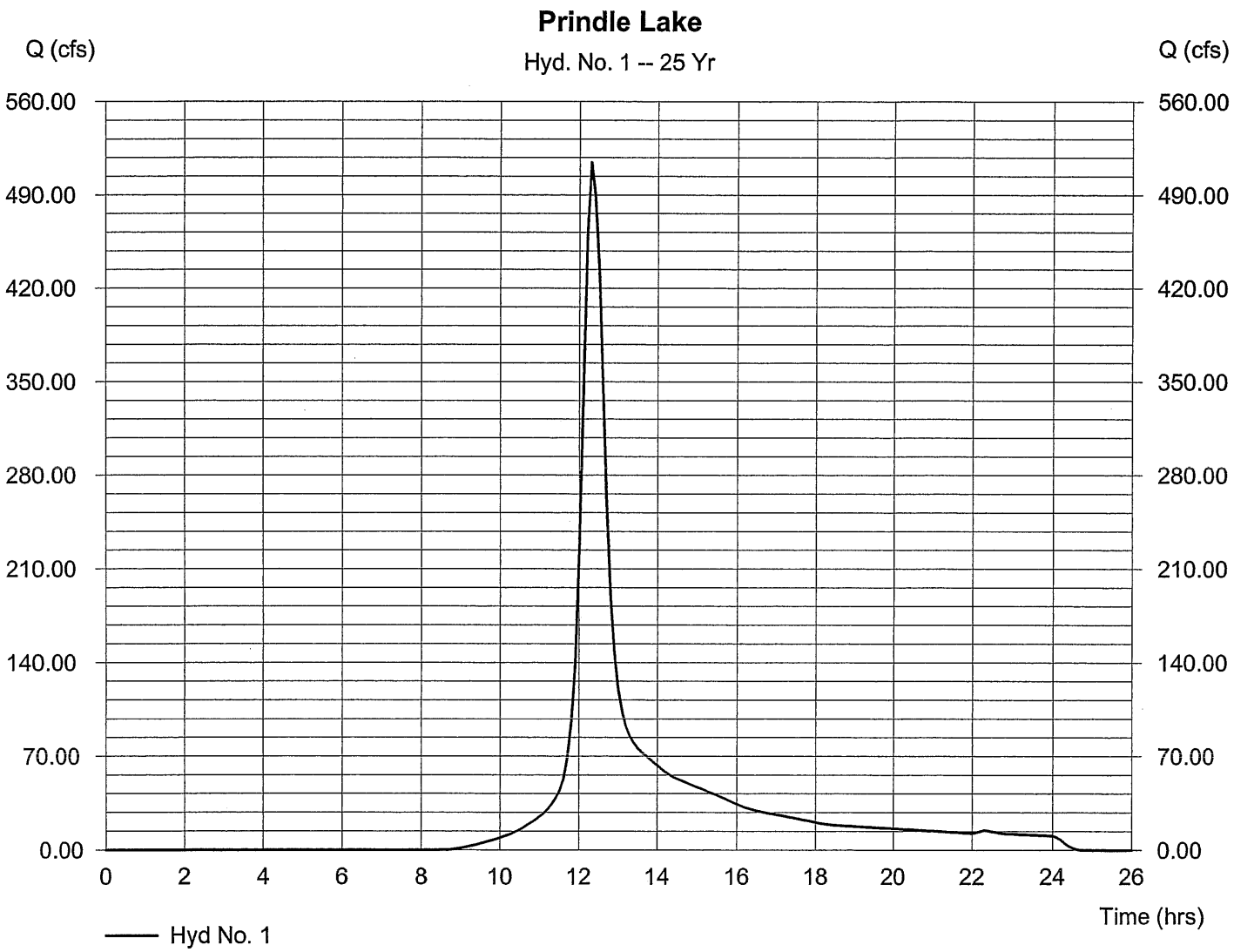
Thursday, Jun 19 2008, 3:18 PM

Hyd. No. 1

Prindle Lake

Hydrograph type	= SCS Runoff	Peak discharge	= 514.23 cfs
Storm frequency	= 25 yrs	Time interval	= 6 min
Drainage area	= 273.800 ac	Curve number	= 75
Basin Slope	= 2.0 %	Hydraulic length	= 17 ft
Tc method	= TR55	Time of conc. (Tc)	= 20.90 min
Total precip.	= 5.30 in	Distribution	= Type III
Storm duration	= 24 hrs	Shape factor	= 484

Hydrograph Volume = 2,678,245 cuft



Hydrograph Plot

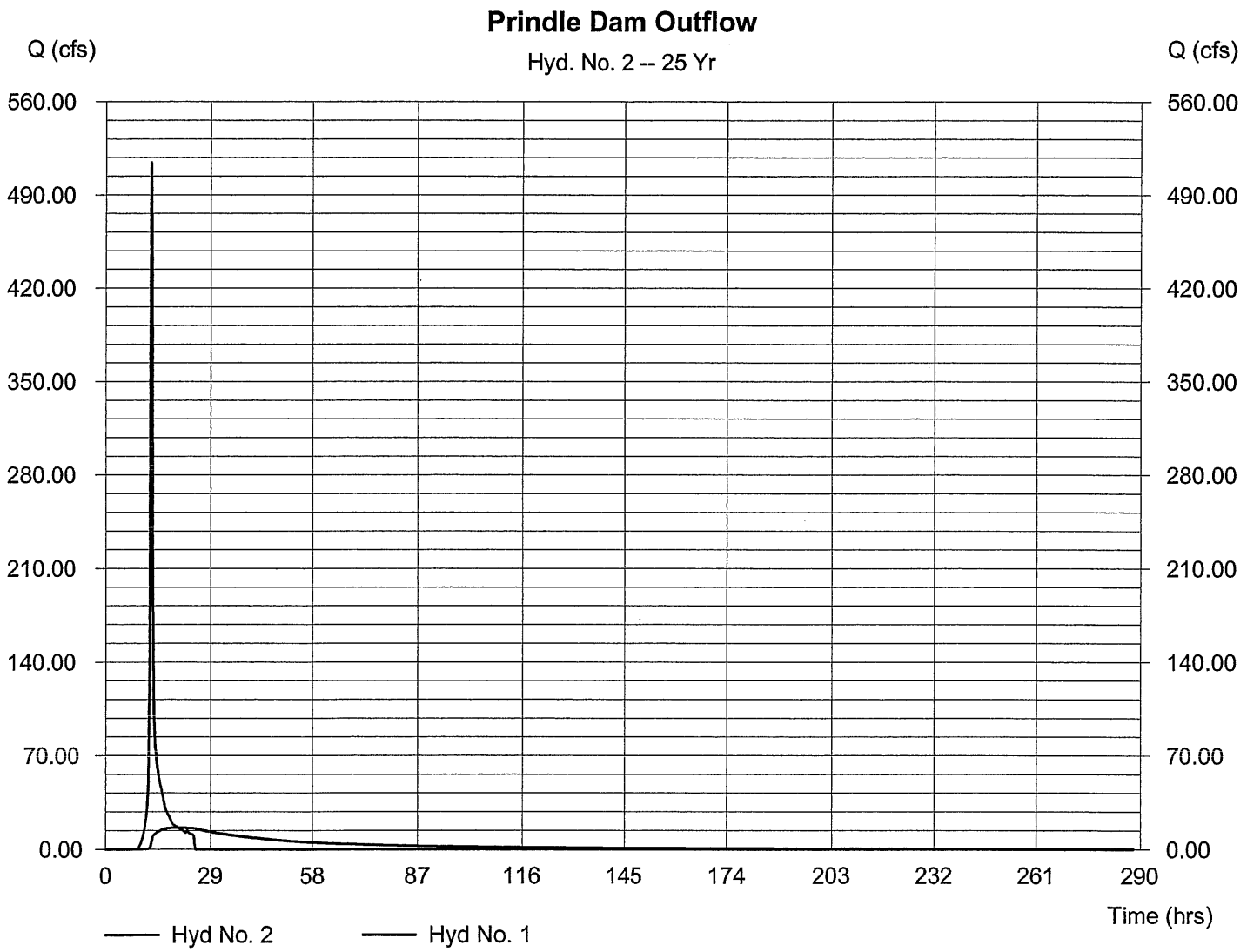
Hydraflow Hydrographs by Intelisolve | Thursday, Jun 19 2008, 3:18 PM

Hyd. No. 2

Prindle Dam Outflow

Hydrograph type	= Reservoir	Peak discharge	= 16.18 cfs
Storm frequency	= 25 yrs	Time interval	= 6 min
Inflow hyd. No.	= 1	Max. Elevation	= 705.78 ft
Reservoir name	= Prindle Lake	Max. Storage	= 2,079,550 cuft

Storage Indication method used. | Hydrograph Volume = 2,595,062 cuft



Pond Report

Hydraflow Hydrographs by Intelisolve

Thursday, Jun 19 2008, 3:18 PM

Pond No. 1 - Prindle Lake

Pond Data

Pond storage is based on known contour areas. Average end area method used.

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	705.00	2,500,000	0	0
0.50	705.50	2,700,000	1,300,000	1,300,000
1.00	706.00	2,900,000	1,400,000	2,700,000
1.50	706.50	3,100,000	1,500,000	4,200,000
2.00	707.00	3,350,000	1,612,500	5,812,500
2.50	707.50	4,004,540	1,838,635	7,651,135
3.00	708.00	4,149,640	2,038,545	9,689,680
3.50	708.50	4,409,340	2,139,750	11,829,430
4.00	709.00	4,593,560	2,250,720	14,080,150
4.50	709.50	4,808,560	2,350,530	16,430,680

Culvert / Orifice Structures

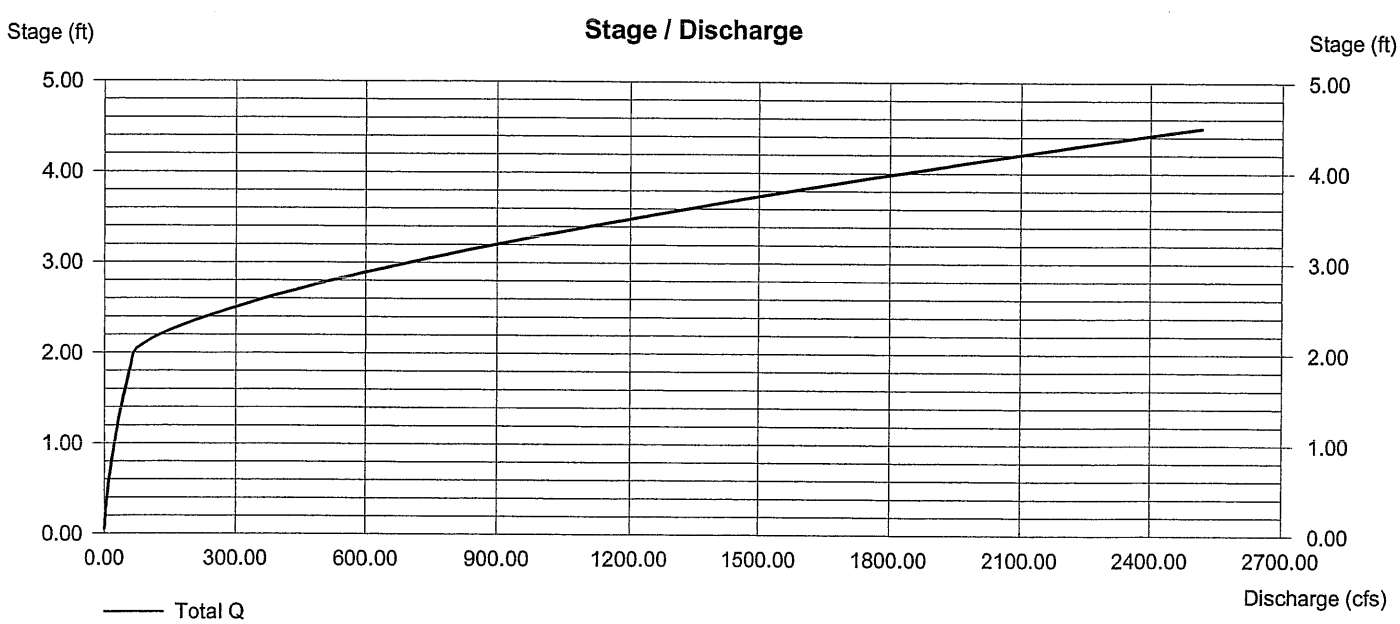
	[A]	[B]	[C]	[D]
Rise (in)	= 24.00	0.00	0.00	0.00
Span (in)	= 83.00	0.00	0.00	0.00
No. Barrels	= 1	0	0	0
Invert El. (ft)	= 705.00	0.00	0.00	0.00
Length (ft)	= 50.00	0.00	0.00	0.00
Slope (%)	= 2.00	0.00	0.00	0.00
N-Value	= .013	.000	.000	.000
Orif. Coeff.	= 0.60	0.00	0.00	0.00
Multi-Stage	= n/a	No	No	No

Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 230.00	0.00	0.00	0.00
Crest El. (ft)	= 707.00	0.00	0.00	0.00
Weir Coeff.	= 2.63	3.33	0.00	0.00
Weir Type	= Broad	---	---	---
Multi-Stage	= No	No	No	No

Exfiltration = 0.000 in/hr (Contour) Tailwater Elev. = 0.00 ft

Note: Culvert/Orifice outflows have been analyzed under inlet and outlet control.



Hydrograph Summary Report

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to peak (min)	Volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Maximum storage (cuft)	Hydrograph description
1	SCS Runoff	710.69	6	738	3,689,445	----	------	------	Prindle Lake
2	Reservoir	24.89	6	1098	3,600,337	1	706.04	2,812,289	Prindle Dam Outflow

PRINDLE-EXISTING SPILLWAY.gpw Return Period: 100 Year Thursday, Jun 19 2008, 3:18 PM

Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

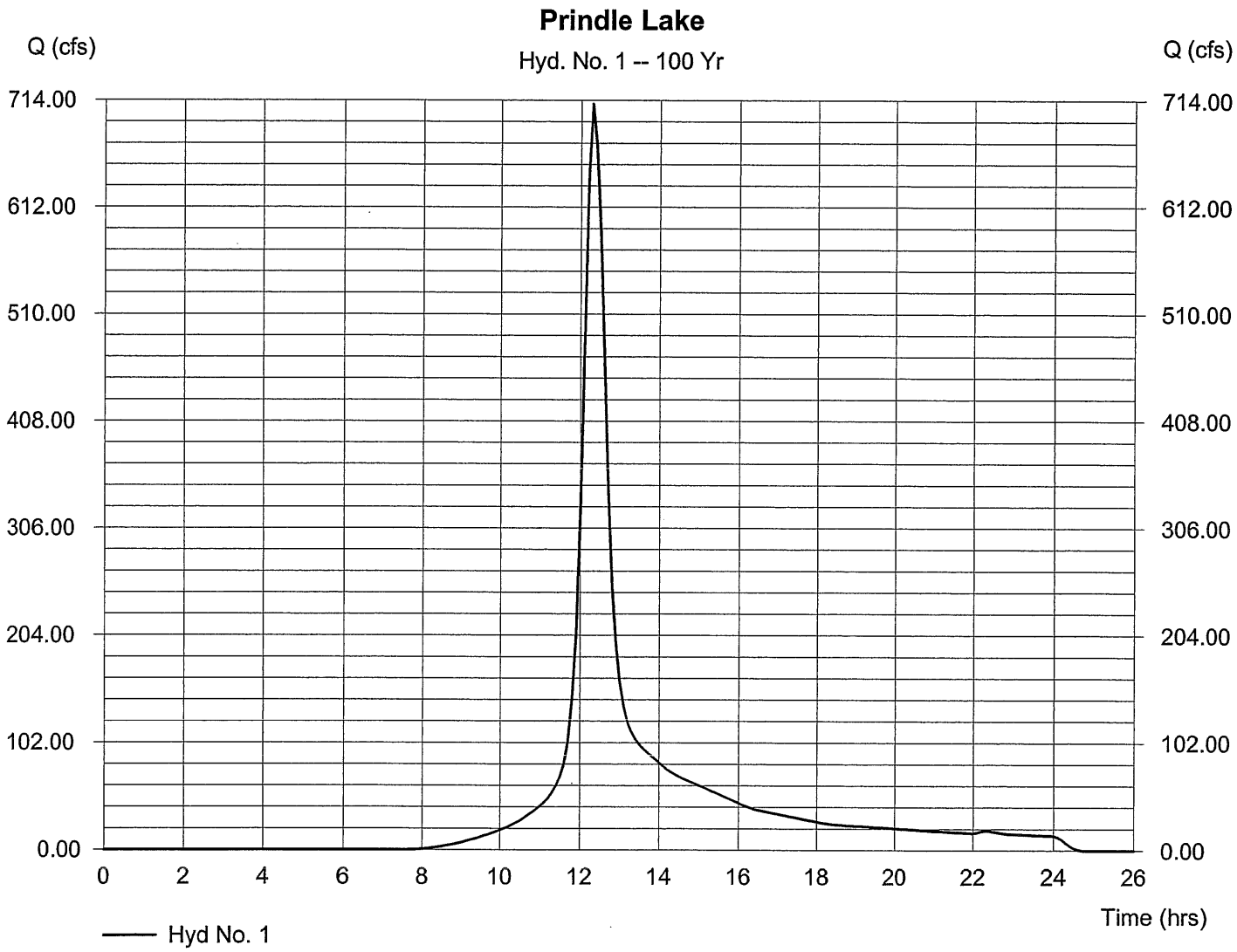
Thursday, Jun 19 2008, 3:18 PM

Hyd. No. 1

Prindle Lake

Hydrograph type	= SCS Runoff	Peak discharge	= 710.69 cfs
Storm frequency	= 100 yrs	Time interval	= 6 min
Drainage area	= 273.800 ac	Curve number	= 75
Basin Slope	= 2.0 %	Hydraulic length	= 17 ft
Tc method	= TR55	Time of conc. (Tc)	= 20.90 min
Total precip.	= 6.50 in	Distribution	= Type III
Storm duration	= 24 hrs	Shape factor	= 484

Hydrograph Volume = 3,689,445 cuft



Hydrograph Plot

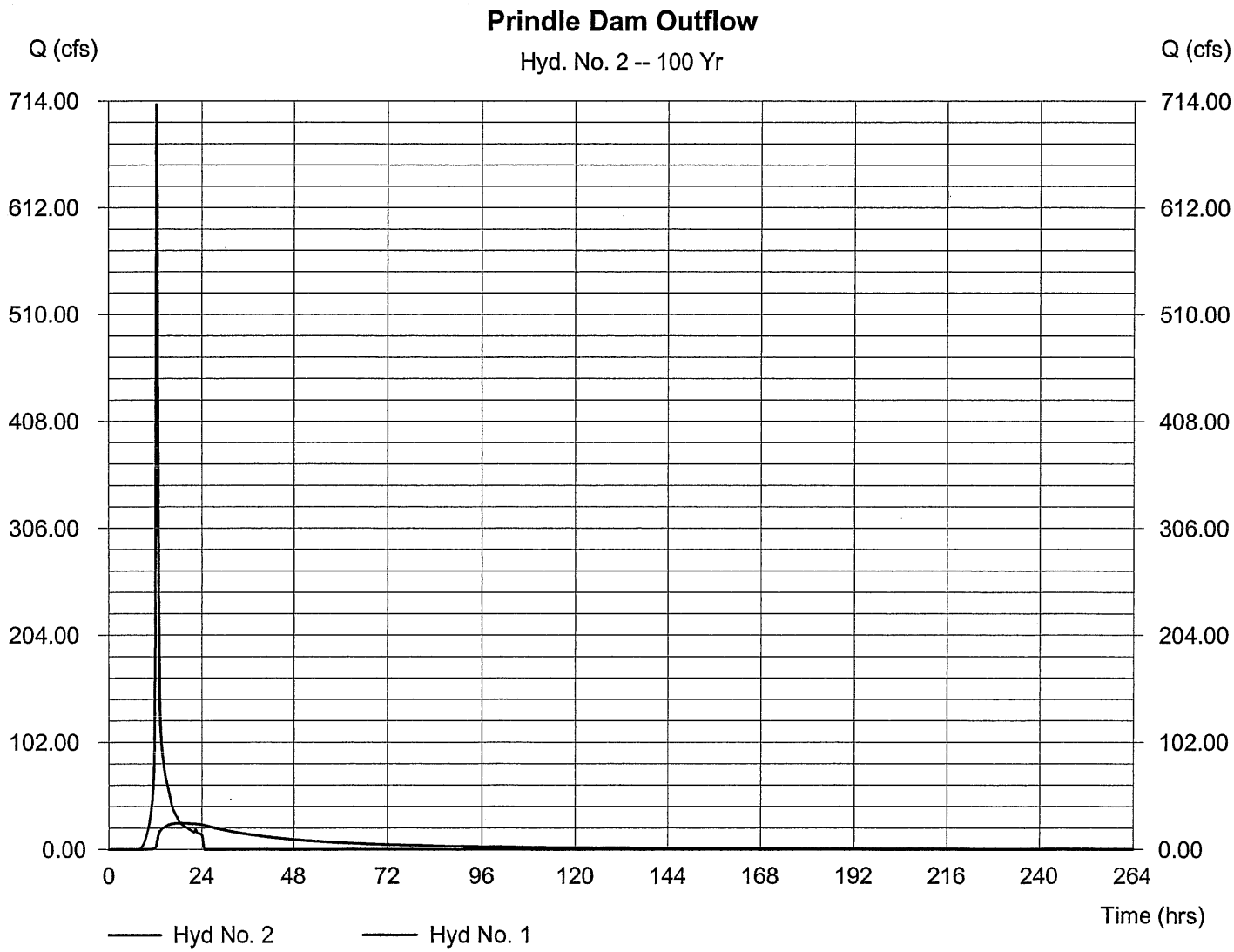
Hydraflow Hydrographs by Intelisolve | Thursday, Jun 19 2008, 3:18 PM

Hyd. No. 2

Prindle Dam Outflow

Hydrograph type	= Reservoir	Peak discharge	= 24.89 cfs
Storm frequency	= 100 yrs	Time interval	= 6 min
Inflow hyd. No.	= 1	Max. Elevation	= 706.04 ft
Reservoir name	= Prindle Lake	Max. Storage	= 2,812,289 cuft

Storage Indication method used. | Hydrograph Volume = 3,600,337 cuft



Pond Report

Hydraflow Hydrographs by Intelisolve — Thursday, Jun 19 2008, 3:18 PM

Pond No. 1 - Prindle Lake

Pond Data

Pond storage is based on known contour areas. Average end area method used.

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	705.00	2,500,000	0	0
0.50	705.50	2,700,000	1,300,000	1,300,000
1.00	706.00	2,900,000	1,400,000	2,700,000
1.50	706.50	3,100,000	1,500,000	4,200,000
2.00	707.00	3,350,000	1,612,500	5,812,500
2.50	707.50	4,004,540	1,838,635	7,651,135
3.00	708.00	4,149,640	2,038,545	9,689,680
3.50	708.50	4,409,340	2,139,750	11,829,430
4.00	709.00	4,593,560	2,250,720	14,080,150
4.50	709.50	4,808,560	2,350,530	16,430,680

Culvert / Orifice Structures

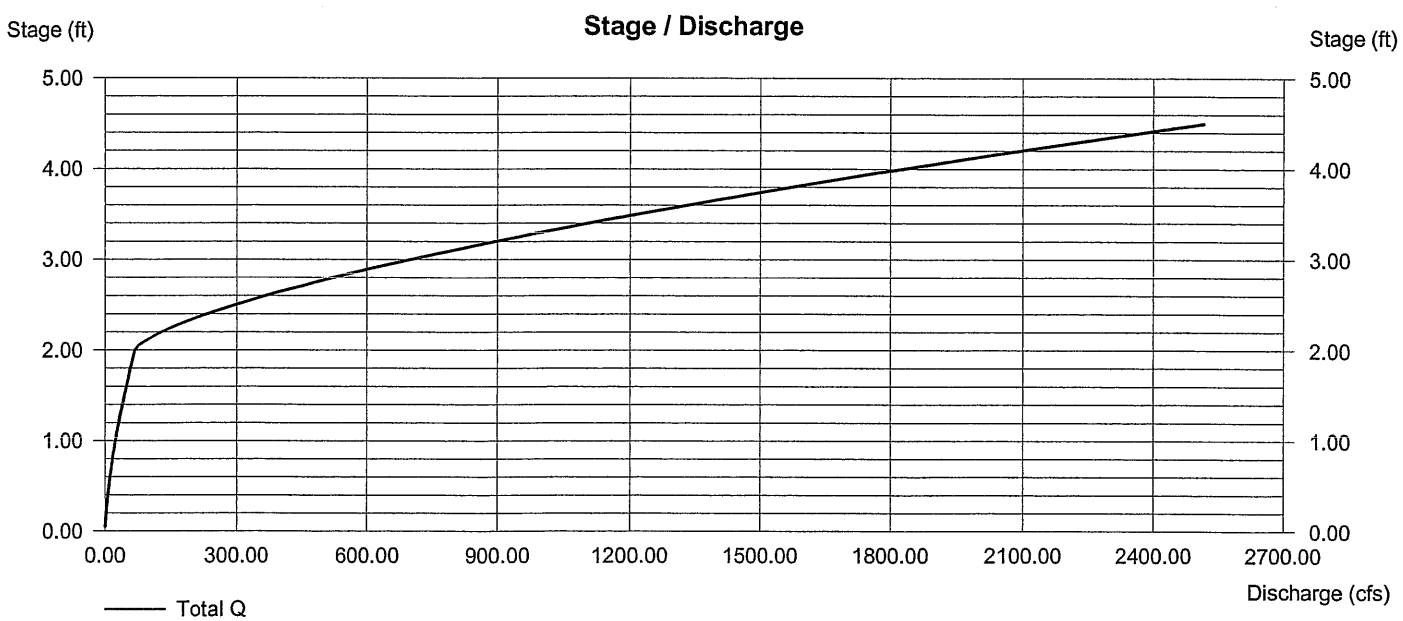
	[A]	[B]	[C]	[D]
Rise (in)	= 24.00	0.00	0.00	0.00
Span (in)	= 83.00	0.00	0.00	0.00
No. Barrels	= 1	0	0	0
Invert El. (ft)	= 705.00	0.00	0.00	0.00
Length (ft)	= 50.00	0.00	0.00	0.00
Slope (%)	= 2.00	0.00	0.00	0.00
N-Value	= .013	.000	.000	.000
Orif. Coeff.	= 0.60	0.00	0.00	0.00
Multi-Stage	= n/a	No	No	No

Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 230.00	0.00	0.00	0.00
Crest El. (ft)	= 707.00	0.00	0.00	0.00
Weir Coeff.	= 2.63	3.33	0.00	0.00
Weir Type	= Broad	---	---	---
Multi-Stage	= No	No	No	No

Exfiltration = 0.000 in/hr (Contour) Tailwater Elev. = 0.00 ft

Note: Culvert/Orifice outflows have been analyzed under inlet and outlet control.



Hydraflow Hydrographs Output for Proposed Spillway

Legend

Hyd.	Origin	Description
1	SCS Runoff	Prindle Lake
2	Reservoir	Prindle Dam Outflow

Hydrograph Return Period Recap

Hyd. No.	Hydrograph type (origin)	Inflow Hyd(s)	Peak Outflow (cfs)								Hydrograph description
			1-Yr	2-Yr	3-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	
1	SCS Runoff	-------	-------	-------	-------	-------	-------	514.23	-------	710.69	Prindle Lake
2	Reservoir	1	-------	-------	-------	-------	-------	16.18	-------	25.18	Prindle Dam Outflow

Proj. file: PRINDLE-PROPOSED SPILLWAY.gpw

Thursday, Jun 19 2008, 3:17 PM

Hydrograph Summary Report

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to peak (min)	Volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Maximum storage (cuft)	Hydrograph description
1	SCS Runoff	514.23	6	738	2,678,245	----	------	------	Prindle Lake
2	Reservoir	16.18	6	1194	2,595,062	1	705.78	2,079,550	Prindle Dam Outflow

Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

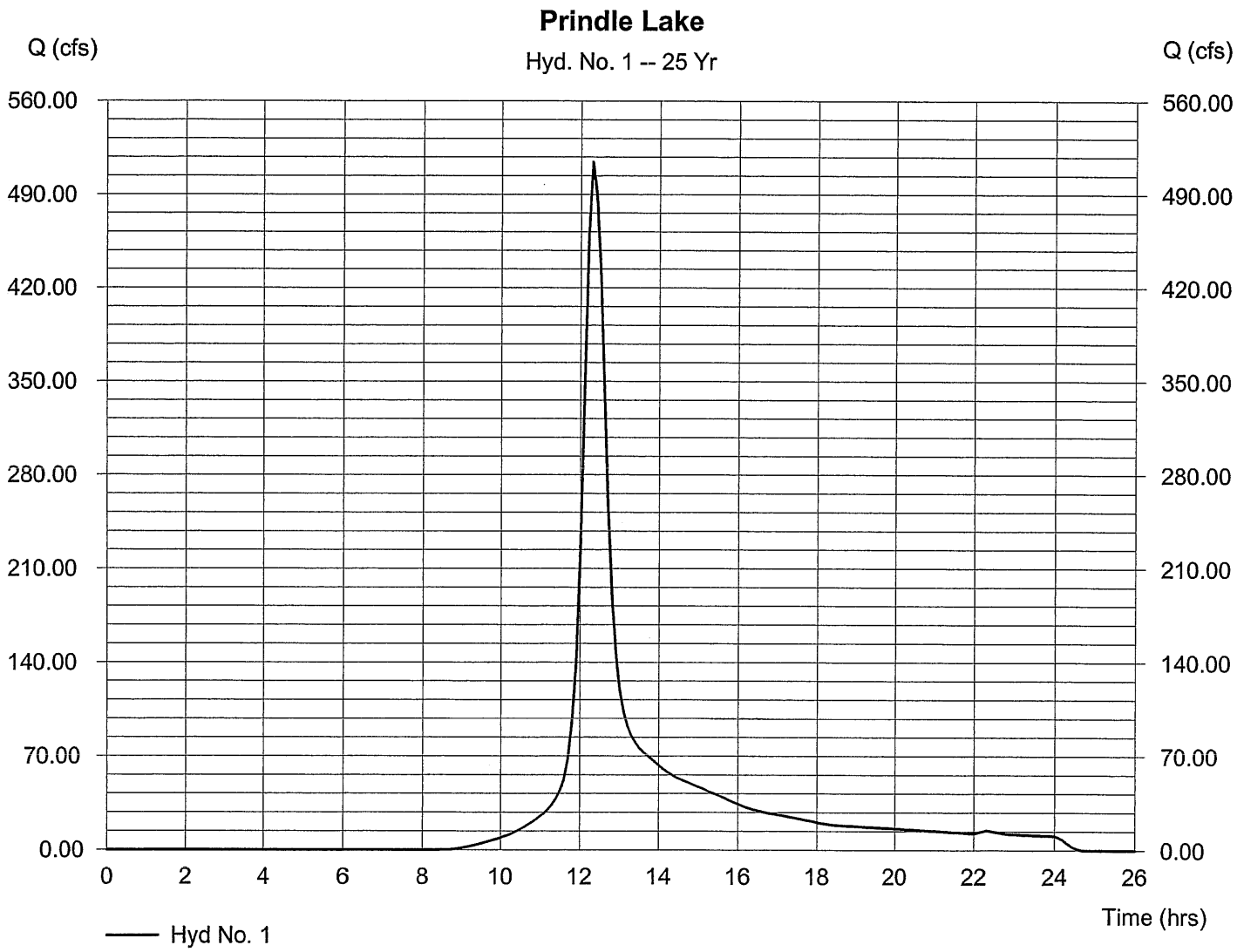
Thursday, Jun 19 2008, 3:17 PM

Hyd. No. 1

Prindle Lake

Hydrograph type	= SCS Runoff	Peak discharge	= 514.23 cfs
Storm frequency	= 25 yrs	Time interval	= 6 min
Drainage area	= 273.800 ac	Curve number	= 75
Basin Slope	= 2.0 %	Hydraulic length	= 17 ft
Tc method	= TR55	Time of conc. (Tc)	= 20.90 min
Total precip.	= 5.30 in	Distribution	= Type III
Storm duration	= 24 hrs	Shape factor	= 484

Hydrograph Volume = 2,678,245 cuft



Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

Thursday, Jun 19 2008, 3:17 PM

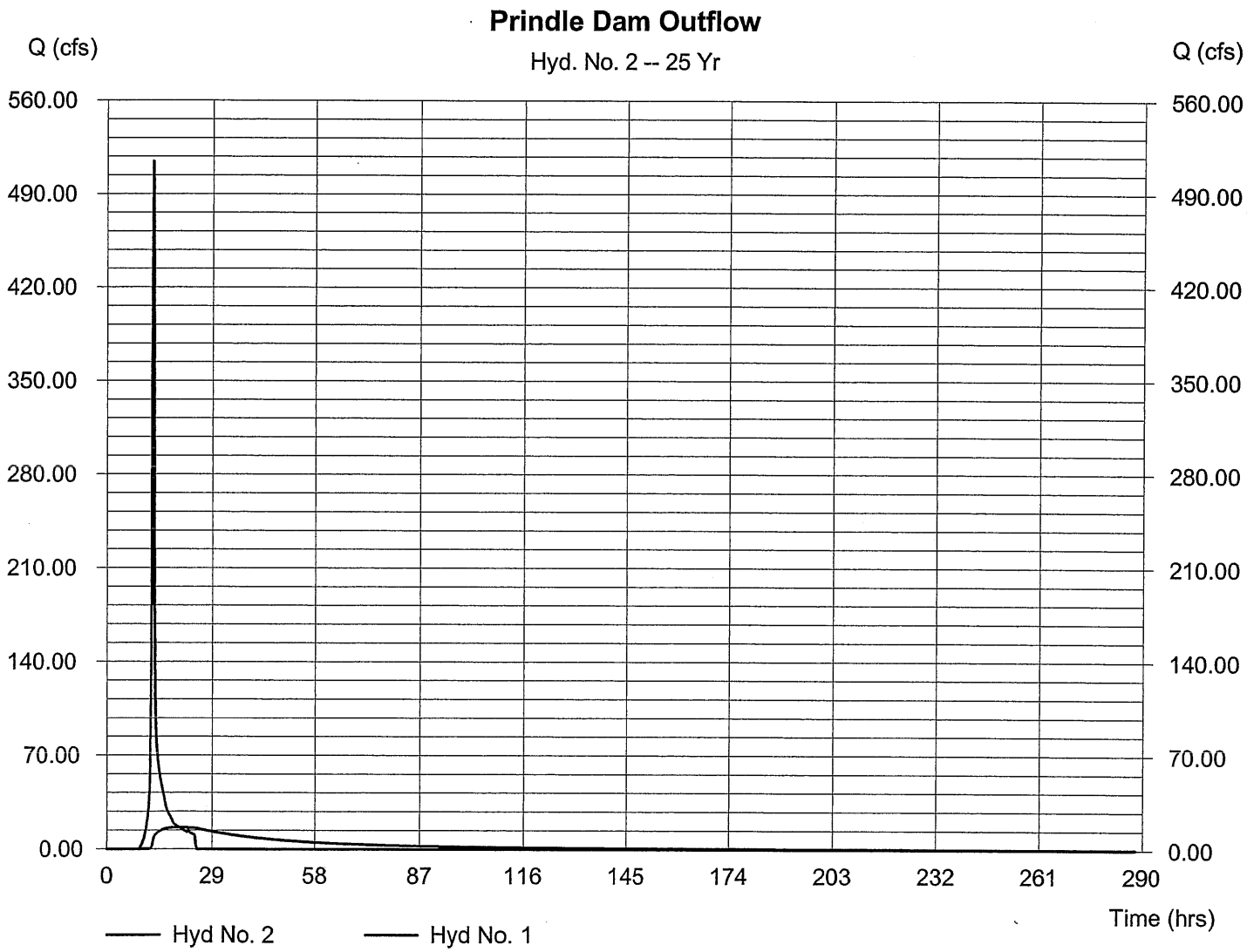
Hyd. No. 2

Prindle Dam Outflow

Hydrograph type	= Reservoir	Peak discharge	= 16.18 cfs
Storm frequency	= 25 yrs	Time interval	= 6 min
Inflow hyd. No.	= 1	Max. Elevation	= 705.78 ft
Reservoir name	= Prindle Lake	Max. Storage	= 2,079,550 cuft

Storage Indication method used.

Hydrograph Volume = 2,595,062 cuft



Pond Report

Hydraflow Hydrographs by Intelisolve

Thursday, Jun 19 2008, 3:17 PM

Pond No. 1 - Prindle Lake

Pond Data

Pond storage is based on known contour areas. Average end area method used.

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	705.00	2,500,000	0	0
0.50	705.50	2,700,000	1,300,000	1,300,000
1.00	706.00	2,900,000	1,400,000	2,700,000
1.50	706.50	3,100,000	1,500,000	4,200,000
2.00	707.00	3,350,000	1,612,500	5,812,500
2.50	707.50	4,004,540	1,838,635	7,651,135
3.00	708.00	4,149,640	2,038,545	9,689,680
3.50	708.50	4,409,340	2,139,750	11,829,430
4.00	709.00	4,593,560	2,250,720	14,080,150
4.50	709.50	4,808,560	2,350,530	16,430,680

Culvert / Orifice Structures

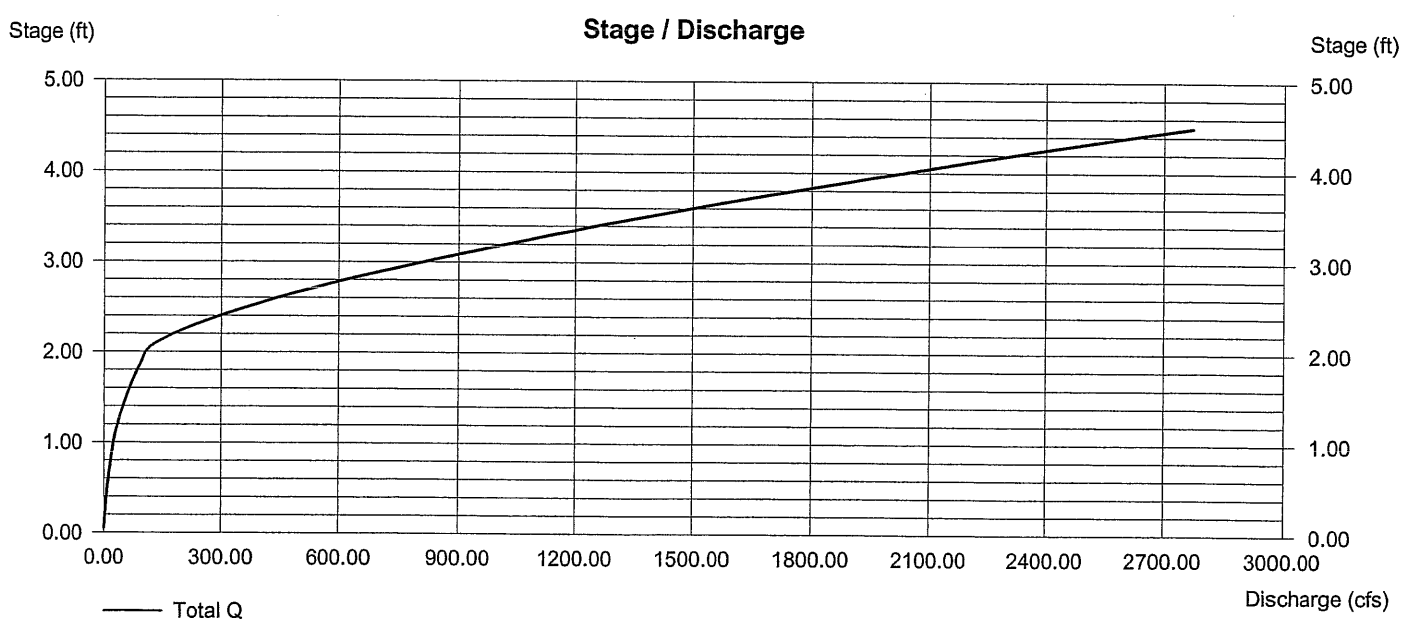
	[A]	[B]	[C]	[D]
Rise (in)	= 24.00	0.00	0.00	0.00
Span (in)	= 83.00	0.00	0.00	0.00
No. Barrels	= 1	0	0	0
Invert El. (ft)	= 705.00	0.00	0.00	0.00
Length (ft)	= 50.00	0.00	0.00	0.00
Slope (%)	= 2.00	0.00	0.00	0.00
N-Value	= .013	.000	.000	.000
Orif. Coeff.	= 0.60	0.00	0.00	0.00
Multi-Stage	= n/a	No	No	No

Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 230.00	15.00	0.00	0.00
Crest El. (ft)	= 707.00	706.00	0.00	0.00
Weir Coeff.	= 2.63	2.60	0.00	0.00
Weir Type	= Broad	Broad	---	---
Multi-Stage	= No	No	No	No

Exfiltration = 0.000 in/hr (Contour) Tailwater Elev. = 0.00 ft

Note: Culvert/Orifice outflows have been analyzed under inlet and outlet control.



Hydrograph Summary Report

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to peak (min)	Volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Maximum storage (cuft)	Hydrograph description
1	SCS Runoff	710.69	6	738	3,689,445	----	------	------	Prindle Lake
2	Reservoir	25.18	6	1098	3,600,380	1	706.04	2,810,179	Prindle Dam Outflow

PRINDLE-PROPOSED SPILLWAY.gpw Return Period: 100 Year Thursday, Jun 19 2008, 3:17 PM

Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

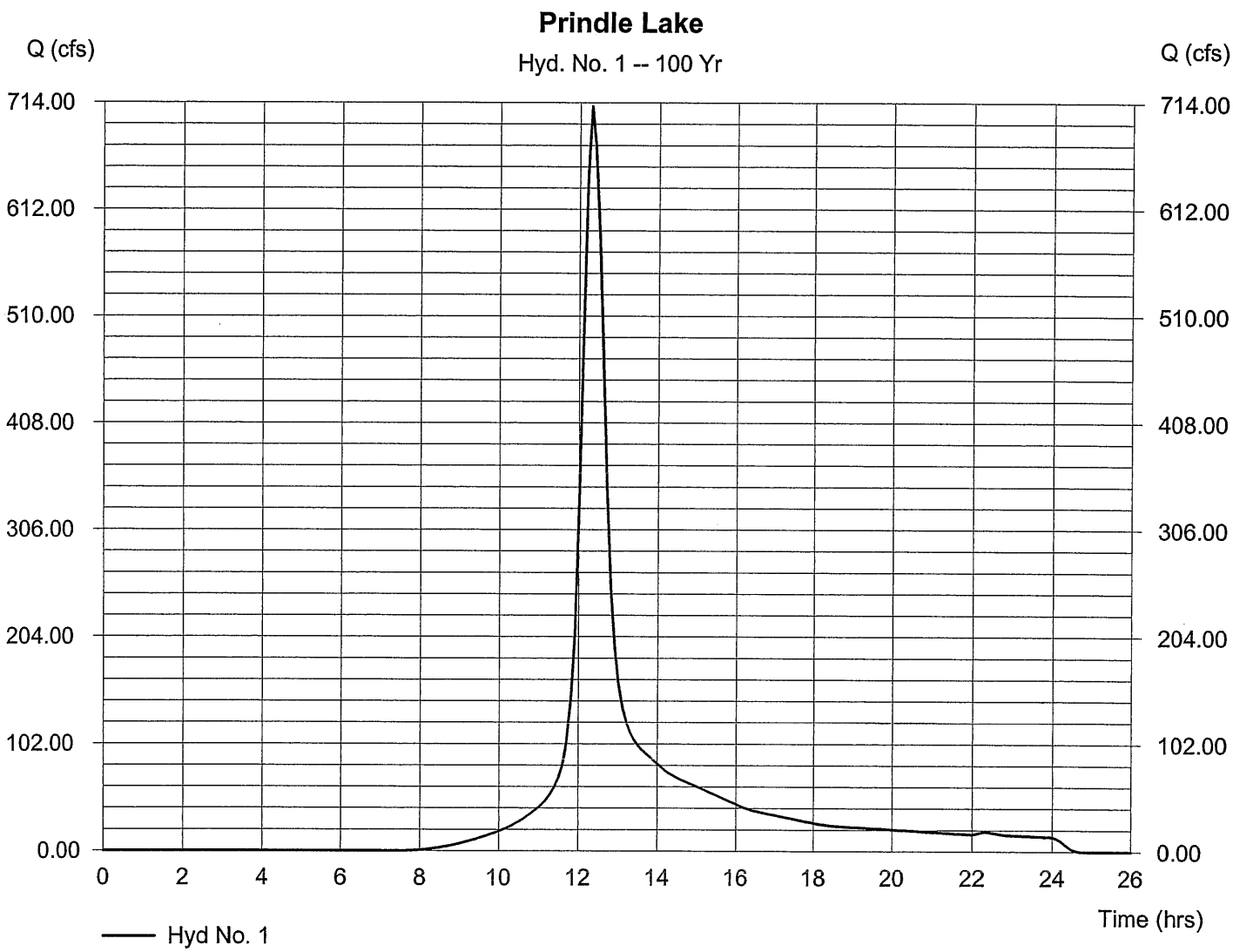
Thursday, Jun 19 2008, 3:17 PM

Hyd. No. 1

Prindle Lake

Hydrograph type	= SCS Runoff	Peak discharge	= 710.69 cfs
Storm frequency	= 100 yrs	Time interval	= 6 min
Drainage area	= 273.800 ac	Curve number	= 75
Basin Slope	= 2.0 %	Hydraulic length	= 17 ft
Tc method	= TR55	Time of conc. (Tc)	= 20.90 min
Total precip.	= 6.50 in	Distribution	= Type III
Storm duration	= 24 hrs	Shape factor	= 484

Hydrograph Volume = 3,689,445 cuft



Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

Thursday, Jun 19 2008, 3:17 PM

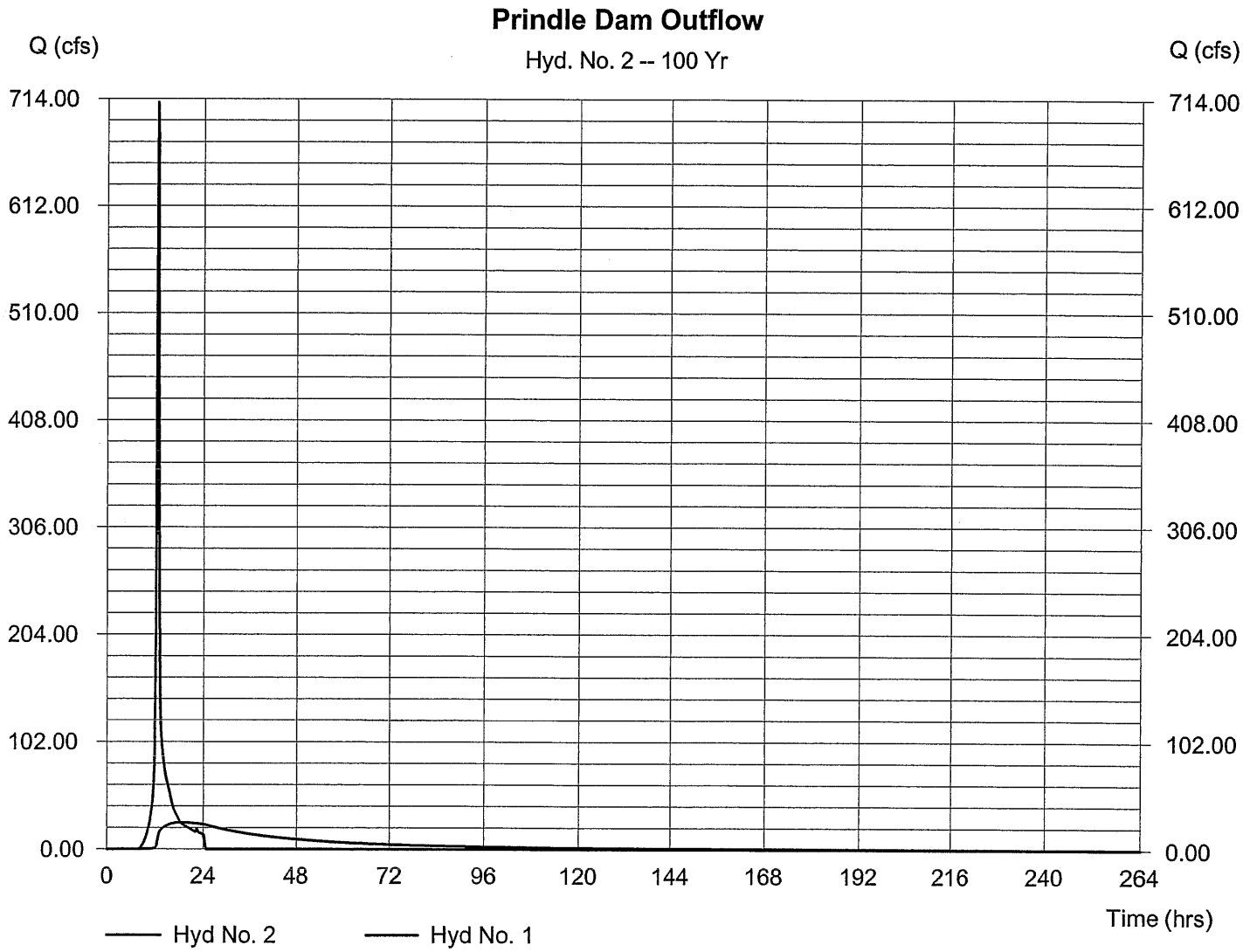
Hyd. No. 2

Prindle Dam Outflow

Hydrograph type	= Reservoir	Peak discharge	= 25.18 cfs
Storm frequency	= 100 yrs	Time interval	= 6 min
Inflow hyd. No.	= 1	Max. Elevation	= 706.04 ft
Reservoir name	= Prindle Lake	Max. Storage	= 2,810,179 cuft

Storage Indication method used.

Hydrograph Volume = 3,600,380 cuft



Pond Report

Hydraflow Hydrographs by Intelisolve

Thursday, Jun 19 2008, 3:17 PM

Pond No. 1 - Prindle Lake

Pond Data

Pond storage is based on known contour areas. Average end area method used.

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	705.00	2,500,000	0	0
0.50	705.50	2,700,000	1,300,000	1,300,000
1.00	706.00	2,900,000	1,400,000	2,700,000
1.50	706.50	3,100,000	1,500,000	4,200,000
2.00	707.00	3,350,000	1,612,500	5,812,500
2.50	707.50	4,004,540	1,838,635	7,651,135
3.00	708.00	4,149,640	2,038,545	9,689,680
3.50	708.50	4,409,340	2,139,750	11,829,430
4.00	709.00	4,593,560	2,250,720	14,080,150
4.50	709.50	4,808,560	2,350,530	16,430,680

Culvert / Orifice Structures

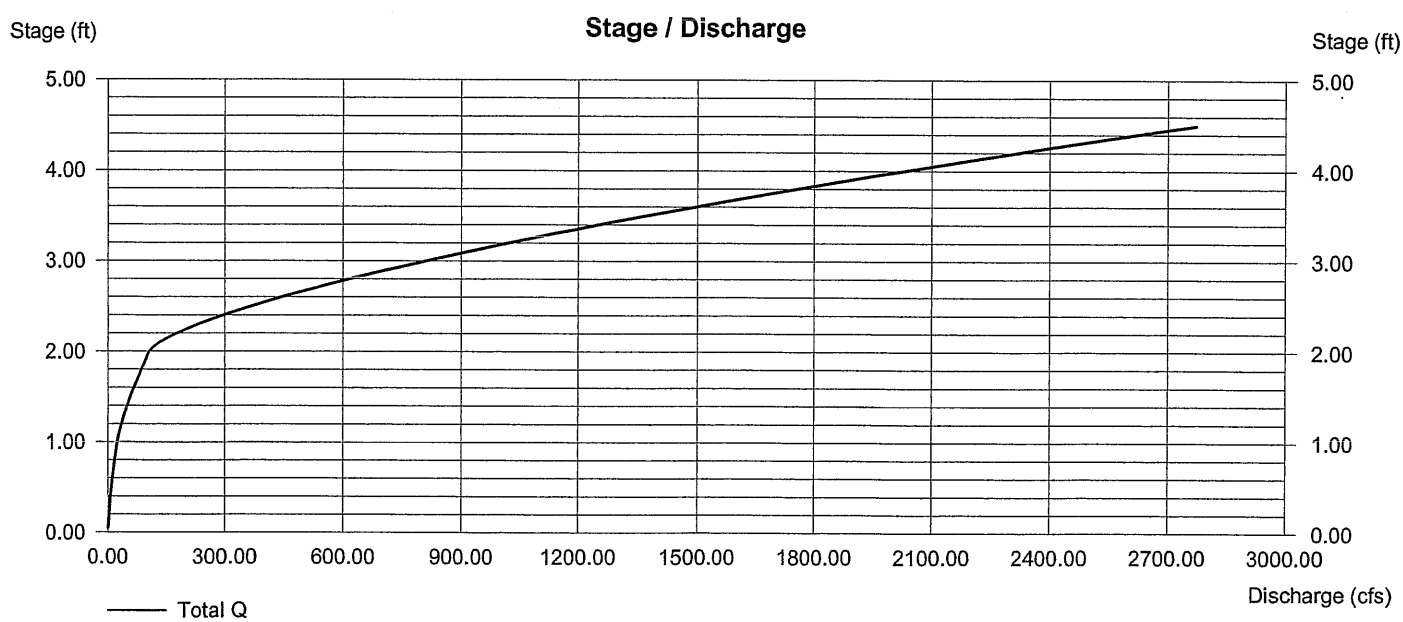
	[A]	[B]	[C]	[D]
Rise (in)	= 24.00	0.00	0.00	0.00
Span (in)	= 83.00	0.00	0.00	0.00
No. Barrels	= 1	0	0	0
Invert El. (ft)	= 705.00	0.00	0.00	0.00
Length (ft)	= 50.00	0.00	0.00	0.00
Slope (%)	= 2.00	0.00	0.00	0.00
N-Value	= .013	.000	.000	.000
Orif. Coeff.	= 0.60	0.00	0.00	0.00
Multi-Stage	= n/a	No	No	No

Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 230.00	15.00	0.00	0.00
Crest El. (ft)	= 707.00	706.00	0.00	0.00
Weir Coeff.	= 2.63	2.60	0.00	0.00
Weir Type	= Broad	Broad	---	---
Multi-Stage	= No	No	No	No

Exfiltration = 0.000 in/hr (Contour) Tailwater Elev. = 0.00 ft

Note: Culvert/Orifice outflows have been analyzed under inlet and outlet control.



Hydrograph Return Period Recap

Hyd. No.	Hydrograph type (origin)	Inflow Hyd(s)	Peak Outflow (cfs)								Hydrograph description
			1-Yr	2-Yr	3-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	
1	SCS Runoff	-------	-------	-------	-------	-------	-------	514.23	-------	710.69	Prindle Lake
2	Reservoir	1	-------	-------	-------	-------	-------	16.18	-------	24.89	Prindle Dam Outflow

Proj. file: PRINDLE-EXISTING SPILLWAY.gpw

Thursday, Jun 19 2008, 3:18 PM

Appendix C: Phase I Inspection/Evaluation Report for Prindle Lake Dam

Site Location: **Prindle Pond Dam**
Oak Ridge Road
Charlton, MA

Prepared for: **Santos Irrevocable Trust**
125 Summer Street
Boston MA 02110 -1624

Prepared by: **Lenard Engineering, Inc.**
19 Midstate Drive
Suite 200
Auburn, MA 01501

Date: **June 6, 2008**

-- PRINDLE LAKE DAM --

PHASE I

INSPECTION / EVALUATION REPORT

Dam Name:	PRINDLE LAKE DAM
State Dam ID#:	3-14-54-38
NID ID#:	MA01174
Owner:	THE SANTOS IRREVOCABLE TRUST
Owner Type:	PRIVATE
Town:	CHARLTON
Consultant:	LENARD ENGINEERING, INC.
Date of Inspection:	JUNE 6, 2008

EXECUTIVE SUMMARY

Representatives of Lenard Engineering, Inc. visually inspected Prindle Lake Dam in Charlton, MA on June 6, 2008. In general, the condition of the dam is POOR. Specific concerns include the wet area along greater than half of the toe of the dam, numerous locations of seepage, a lack of effective slope protection (grassy cover), loose sandy core material, the lack of an emergency spillway and a low level outlet, and tree and shrub growth along the upstream and downstream faces and the toe of the dam.

The Prindle Lake Dam is classified as an INTERMEDIATE size structure with a SIGNIFICANT (Class II) hazard classification. A request was made to the Department of Conservation and Recreation (DCR), Office of Dam Safety (ODS) to review the then current hazard classification of High. In accordance with M.G.L. Chapter 253 s.44-48 and 302 10.00 Dam Safety Rules and Regulations, the DCR determined in a letter dated October 22, 2007 that the classification of the dam shall be changed from High Hazard Potential to Significant Hazard Potential.

In addition to regular maintenance and as-needed repair, the following activities are recommended to improve the condition of the dam:

- Removal of tree growth from upstream and downstream dam faces and within 25 feet of the toe of the dam
- Improvements to the earth embankments and crest, including removing the apparent abandon spillway
- Installation of a low-level outlet structure and an emergency spillway
- Installation of toe drains

Dam Evaluation Summary Detail Sheet

1. NID ID: MA01174	**2. Dam Name:** Prindle Lake Dam	**3. Dam Location:** Charlton; off of Oak Ridge Drive
4. Inspection Date: 06/06/08	**5. Last Insp. Date:** 09/14/06	**6. Next Inspection:** 09/14/08
7. Inspector: Scott D. Charpentier	**8. Consultant:** Lenard Engineering, Inc.	
9. Hazard Code: Significant (Class II)	**10. Insp. Frequency:** 5 yrs.	**11. Insp. Condition:** Poor

E1. Design Methodology:	1	**E7. Low-Level Discharge Capacity:**	1
E2. Level of Maintenance:	2	**E8. Low-Level Outlet Physical Condition:**	1
E3. Emergency Action Plan:	1	**E9. Spillway Design Flood Capacity:**	5
E4. Embankment Seepage:	3	**E10. Overall Physical Condition of the Dam:**	2
E5. Embankment Condition:	3	**E11. Estimated Repair Cost (in thousand $):**	**235**
E6. Concrete Condition:	4		

Evaluation Description

E1: DESIGN METHODOLOGY

1. Unknown Design – no design records available
3. Some standard design features
5. State of the art design – design records available

E2: LEVEL OF MAINTENANCE

1. No evidence of maintenance, no O&M manual
2. Very little maintenance, no O&M manual
3. Some level of maintenance and standard procedures
4. Adequate level of maintenance and standard procedures
5. Detailed maintenance plan that is executed

E3: EMERGENCY ACTION PLAN

1. No plan or idea of what to do in the event of an emergency
2. Some idea but no written plan
3. No formal plan but well thought out
4. Available written plan that needs updating
5. Detailed, updated written plan available and filed with MADCR

E4: EMBANKMENT SEEPAGE

1. Severe piping and/or seepage with no monitoring
2. Evidence of monitored piping and seepage
3. No piping but uncontrolled seepage
4. Controlled seepage
5. No seepage or piping

E5: EMBANKMENT CONDITION

1. Severe erosion and/or large trees
2. Significant erosion or significant woody vegetation
3. Brush and exposed embankment soils, or moderate erosion
4. Unmaintained grass, rodent activity and maintainable erosion
5. Well maintained healthy uniform grass cover

E6: CONCRETE CONDITION

1. Major cracks, misalignment, discontinuities causing leaks, seepage or stability concerns
2. Cracks with misalignment inclusive of transverse cracks with no misalignment
3. Significant longitudinal cracking and minor transverse cracking
4. Spalling and minor surface cracking
5. No apparent deficiencies

E7: LOW LEVEL OUTLET DISCHARGE CAPACITY

1. No low level outlet
2. Outlet with insufficient drawdown capacity
3. Inoperable gate with potentially sufficient drawdown capacity
4. Operable gate with sufficient drawdown capacity
5. Operable gate with capacity greater than necessary

E8: LOW LEVEL OUTLET PHYSICAL CONDITION

1. Outlet inoperative needs replacement, non-existent or inaccessible
2. Outlet inoperative needs repair
3. Outlet operable but needs repair
4. Outlet operable but needs maintenance
5. Outlet and operator operable and well maintained

E9: SPILLWAY DESIGN FLOOD CAPACITY

1. 0 - 20% of the SDF
2. 21- 40% of the SDF
3. 41- 60% of the SDF
4. 61- 80% of the SDF
5. 81- 100% of the SDF

E10: OVERALL PHYSICAL CONDITION OF THE DAM

1. *UNSAFE* – Major structural, operational, and maintenance deficiencies exist under normal operating conditions
2. *POOR* - Significant structural, operation and maintenance deficiencies are clearly recognized under normal loading conditions
3. *FAIR* - Significant operational and maintenance deficiencies, no structural deficiencies. Potential deficiencies exist under unusual loading conditions that may realistically occur. Can be used when uncertainties exist as to. critical parameters
4. *SATISFACTORY* - Minor operational and maintenance deficiencies. Infrequent hydrologic events would probably result In deficiencies.
5. *GOOD* - No existing or potential deficiencies recognized. Safe performance is expected under all loading including SDF

E11: ESTIMATED REPAIR COST

Estimation of the total cost to address all identified structural, operational, maintenance deficiencies. Cost shall be developed utilizing standard estimating guides and procedures

Changes/Deviations to Database Information since last inspection

The hazard potential classification has been changed from High Hazard (Class I) to Significant Hazard (Class II).

PREFACE

The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigations and analyses involving topographic mapping, subsurface investigations, testing and detailed computational evaluations are beyond the scope of this report.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection, along with data available to the inspection team. In cases where an impoundment is lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions, which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is critical to note that the condition of the dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Scott D. Charpentier
Massachusetts License No.: 45853
Project Manager
Lenard Engineering, Inc.

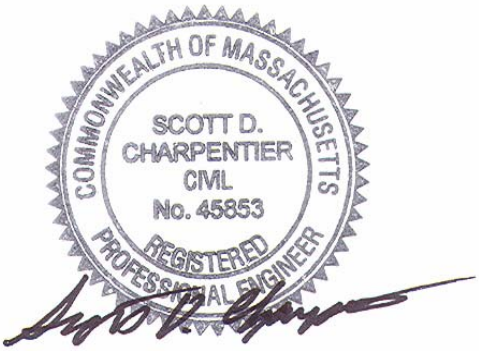


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Figure 1: Locus Plan
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Appendix D: Definitions and Abbreviations

SECTION 1

1.0 DESCRIPTION OF PROJECT

1.1 General

1.1.1 Authority

The Santos Irrevocable Trust retained Lenard Engineering, Inc. (LEI) to perform Phase II Dam Inspection and Evaluation. As part of this work an updated visual inspection and report of conditions for the dam at Prindle Lake along a tributary of Cady Brook in Charlton, Massachusetts has been prepared. This inspection and report were performed in accordance with MGL Chapter 253, Sections 44-50 of the Massachusetts General Laws as amended by Chapter 330 of the Acts of 2002.

1.1.2 Purpose of Work

The purpose of this investigation is to inspect and evaluate the present condition of the dam and appurtenant structures in accordance with 302 CMR10.07 to provide information that will assist in both prioritizing dam repair needs and planning/conducting maintenance and operation.

The investigation is divided into four parts: 1) obtain and review available reports, investigations, and data previously submitted to the owner pertaining to the dam and appurtenant structures; 2) perform a visual inspection of the site; 3) evaluate the status of an emergency action plan for the site and; 4) prepare and submit a final report presenting the evaluation of the structure, including recommendations and remedial actions, and opinion of probable costs.

1.1.3 Definitions

To provide the reader with a better understanding of the report, definitions of commonly used terms associated with dams are provided in Appendix D. Many of these terms may be included in this report. The terms are presented under common categories associated with dams which include: 1) orientation; 2) dam components; 3) size classification; 4) hazard classification; and 5) miscellaneous.

1.2 Description of Project

1.2.1 Location

Prindle Lake Dam is located in the Town of Charlton, Worcester County, Commonwealth of Massachusetts. Latitude and longitude are given as 42.1137 N and 72.0005 W respectively on the USGS Webster quadrangle. The dam is situated approximately 2,680 feet south of the intersection of Carpenter Hill Road and Hall Road and is located along the western shore of Prindle Lake near the end of Oak Ridge Road. Vehicular access to the dam is feasible via the unpaved way of Oak Ridge Road; access by foot should be undertaken for the last 500± feet.

1.2.2 Owner/Operator

	Dam Owner	Dam Caretaker
Name	The Santos Irrevocable Trust Mr. Carl Izzo, Trustee Fiduciary Real Estate Advisors	Nature's Classroom, Inc. Dr. John G. Santos, Director
Mailing Address	125 Summer Street	19 Harrington Rd.
Town	Boston, MA 02110-1624	Charlton, MA 01507
Daytime Phone	1-617-345-3600	1-508-248-2741
Emergency Phone	1-781-771-8356	
Email Address		

1.2.3 Purpose of the Dam

Prindle Lake Dam impounds Prindle Lake, which is used for boating, fishing, and other recreational purposes. The partial owner and caretaker Nature's Classroom, Inc., a private not-for-profit environmental educational center, provides campgrounds, trails, and both indoor and outdoor educational activities in the area immediately adjacent to and downstream of the dam. There is an unpaved walking path on the crest of the dam.

1.2.4 Description of the Dam and Appurtenances

Prindle Lake Dam is an earthen dam built approximately in 1952. The dam is approximately 235 feet long and 16 feet high. The primary spillway is a cast in place concrete box culvert with an opening 25" high by 83" wide, which serves as broad-crested weir. This spillway is centered approximately 62.5 feet from the left abutment and 173 feet from the right abutment. The crest of the dam has a width ranging from approximately 18 feet in the middle to 25.5 feet at the spillway. The crest elevation varies; the left end of the dam is 1.5 feet higher than the right end of the dam. The downstream face is sloped at 2H:1V. The upstream face was not observable under the vegetation. A timber railing, set on both the upstream and downstream sides on the top of the box culvert, provides for pedestrian safety crossing the spilling. Wire mesh fencing and an 18" picket fence have been placed on the shoulders of the crest to the left of the spillway to limit pedestrian traffic accessing the impoundment.

Remnants of an apparent abandon spillway inlet were located 105 ft to the right of the existing spillway on the upstream face. The outlet of the apparent abandon spillway was not observed. However, the old channel was observed on the downstream side of the dam.

1.2.5 Operations and Maintenance

One of the owner Trustees, Dr. John G. Santos is responsible for the operations and maintenance of the dam. No formal written procedures for operations or maintenance exist.

The dam cannot be "operated" in the normal sense since there are neither gates, nor stoplogs, nor low level outlets. The only maintenance of the dam appears to be mowing of the crest and recently dumped rip rap stone along portions of the downstream face.

1.2.6 DCR Size Classification

Prindle Lake Dam has a maximum structural height of approximately 11 feet and a maximum storage capacity of approximately 250 acre-feet. Therefore, in accordance with Department of Conservation and Recreation Office of Dam Safety classification, under Commonwealth of Massachusetts dam safety rules and regulations stated in 302 CMR 10.00 as amended by Chapter 330 of the Acts of 2002, Prindle Lake Dam is an INTERMEDIATE size structure.

1.2.7 DCR Hazard Classification

A request was made to the Department of Conservation and Recreation (DCR), Office of Dam Safety (ODS) to review the then current hazard classification of High. In accordance with M.G.L. Chapter 253 s.44-48 and 302 10.00 Dam Safety Rules and Regulations, the DCR determined in a letter dated October 22, 2007 that the classification of the dam shall be changed from High Hazard Potential to Significant Hazard Potential. Therefore, in accordance with Department of Conservation and Recreation classification procedures, under Commonwealth of Massachusetts dam safety rules and regulations stated in 302 CMR 10.00 as amended by Chapter 330 of the Acts of 2002, Prindle Lake Dam is currently classified as a SIGNIFICANT (CLASS II) hazard potential dam.

1.3 Pertinent Engineering Data

Data contained in this report was taken from previous inspection/evaluation reports including work of the 2008 Phase II Report prepared by LEI which includes complete hydrologic, hydraulic and stability computations.

1.3.1 Drainage Area

The drainage area for Prindle Lake is approximately 0.4 square miles and extends through the community of Charlton. The drainage area consists primarily of steep, hilly terrain, dropping an estimated 807 feet from the upstream end of the watershed northwest of Prindle Lake to normal lake level.

1.3.2 Reservoir

	Length (feet)	Width (feet)	Surface Area (acres)	Storage Volume (acre-feet)
Normal Pool	2,400	1,140	80	150
Maximum Pool	2,680	1,380	85	250
SDF Pool	2,550	1,260	82.6	206

1.3.3 Discharges at the Dam Site

The 1955 flood resulting from Hurricane Diane caused a flood flow of 1,240 CFS at the USGS Gaging Station located on Upper Sibley Pond Outlet in Charlton, Massachusetts, approximately 2-miles from Prindle Lake Dam. Through a watershed comparison, this storm event caused an

approximate flow at the Prindle Lake Dam of 222 CFS. The dam was overtopped during this storm event and the hurricane of 1938.

1.3.4 General Elevations (Mean Sea Level (MSL), 1998)

A.	Top of Dam ……	707 – 708.5
B.	Spillway Design Flood Pool ……..…	706.04
C.	Normal Pool ……..	705
D.	Spillway Crest ……..	705
E.	Upstream Water at Time of Inspection ……..	spillway crest
F.	Streambed at Toe of the Dam ……	698
G.	Low Point along Toe of the Dam …….....……	698 (approx.)
H.	Top of Abutments ……..	712.3

1.3.5 Main Spillway (Mean Sea Level (MSL), 1998)

A.	Type ……	Broad-crested weir (concrete box culvert)
B.	Length ……	6.92 FT(83 inches)
C.	Invert Elevation ……..	705
D.	Upstream Channel ……..	Not applicable
E.	Downstream Channel …….	692 (approx.)
F.	Downstream Water …….	Normal stream flow
G.	SDF …….	711 CFS (100-year inflow from watershed) 24.89 CFS (100-year spillway discharge)
H.	Spillway Capacity ……	65.51 CFS

1.3.6 Design and Construction Records

No records pertaining to design and construction of the dam were provided to LEI for this inspection.

1.3.7 Operating Records

No operating records were available at the time of inspection.

1.4 <u>Other General Information</u>

The referenced safety inspection from April 2008 depicts a downstream face condition without broad leaf vegetation (Photo 011-R). This report also depicts an active flow from the apparent abandon spillway (Photo 012-R).

The 1998 Inspection Report gives the National ID number as MA01826. However, current DCR data give the National ID as MA01174; this is the National ID used in this report.

SECTION 2

2.0 INSPECTION

2.1 Visual Inspection

Representatives of Lenard Engineering, Inc. visually inspected Prindle Lake Dam on June 6, 2008. At the time of the inspection, the weather was overcast with temperatures in the 60's. Photographs to document the current conditions of the dam were taken during the inspection and are included in Appendix A. Underwater areas were not inspected. A copy of the inspection checklist is included in Appendix B.

2.1.1 General Findings

In general, Prindle Lake Dam was found to be in POOR condition. General concerns include:

1) Saturated downstream toe
2) Multiple locations of minor seepage
3) Tree growth on the dam and areas within 25 feet of the toe
4) The lack of a low level outlet and/or an emergency spillway

Specific concerns are identified in more detail within the sections below.

2.1.2 Dam

Abutments

Some trees and shrubs were present within 25 feet of the abutments (Photos 001 and 008).

Upstream Face

The upstream face is covered with broad leaf vegetation, which made observation difficult (Photo 002). The face is possibly covered with earth and riprap. Some brushy vegetation was observed on the right side with a 12" tree stump located on the left side.

Crest

The crest exhibits minor erosion near the abutments and the spillway (Photo 001) apparently due to recreational foot traffic.

Shoulders of the crest are covered in broad leaf vegetation (Photo 007). The crest is paralleled along its limits with wood picket and chain link fence which deter foot traffic from accessing the dam faces.

Downstream Face

The downstream face is partially covered by broad leaf vegetation (Photos 006 and 010), which made observation difficult. Riprap has been dumped on two areas of the face to the right of the spillway (Photo 010). Some seepage was observed through the riprap on the face. A 12" stump had been left in place to

the right of the spillway. The condition of the earth underneath the riprap was not possible to assess.

The toe of the dam was saturated. Seeps (up to 1/2" gallons per minute) were observed to the left of the spillway along the toe.

Drains

None were observed.

Instrumentation

A piezometer was installed on the dam crest in conjunction with the work of the references Phase II evaluation. The water elevation within the peizometer was measured at 5-feet below dam crest at the time of inspection.

Access Roads and Gates

The dam is located on private property. Vehicular traffic is controlled with an unlocked steel gate and signage. There are no controls for pedestrian or ATV traffic.

2.1.3 Appurtenant Structures

Primary Spillway

The primary spillway, which is the only defined dam discharge, consists of a cast in place concrete slab supported on 16" wide cast-in-place concrete abutments (Photo 003). The top of the slab is spalled and the condition of the inside of the culvert is satisfactory. No trash rack was observed. However, the culvert appeared to be clear of debris.

Wingwalls built of concrete block exist on both sides of the entrance to the spillway. It is unknown if the blocks are a facing to a mechanically stabilized earth wall system or if the blocks are individual units. The grout between the blocks is cracked.

Downstream, the spillway discharges over a large piece of ledge. However, because the ledge sits lower than the floor of the spillway, it does not interfere with outflow (Photo 003). The right limit of the spillway is undermined approximately 6-inches (Photo 004).

On both sides of the spillway discharge, the downstream slope is terraced with stone masonry walls which rise to the crest. Stones have been placed on both sides of the channel immediately adjacent to the spillway discharge however, a clear running seep was found just left of the channel.

Further downstream, riprap was dumped alongside the right of the downstream channel.

Remnants of an apparent abandon spillway were located 150 ft to the right of the existing spillway. Erosion had occurred at the sides of the apparent abandon spillway. The discharge for this spillway was not able to be located. The channel that served this spillway was defined and wet. It is possible that existing

seepage at the toe of dam drains towards the right along the toe and finds it way into this old channel.

Low Level Outlet

No low level outlet exists.

Auxiliary/Emergency Spillway

No auxiliary/emergency spillway exists.

2.1.4 Downstream Area

The immediate area downstream is heavily wooded with trees, saplings, and woody brush growing close to the toe of the dam (Photos 009 and 010). At the toe of the dam is a marshy, wet area with evidence of seepage. The primary spillway channel winds to the right and joins with the wider, old channel 100± feet downstream from the dam. The old channel is partially blocked by the placement of cut timbers (Photo 009) which has resulted in ponding at the right toe of the dam.

Located further downstream, the brook passes under Harrington Road through a 4 foot diameter HDPE culvert pipe. The culvert inlet and outlet do not have erosion or scour protection in-place. From a visual inspection of the wooded area just upstream of Harrington road, the brook passes through a steep rocky valley.

Further west downstream is State Route 169 (Southbridge Road), along which is located an active commercial property, several residences, and a bridge.

2.1.5 Reservoir Area

The inspection was limited to the immediate dam area and downstream area.

2.2 Caretaker Interview

Dr. John G. Santos, director of the Nature's Campground is the caretaker and was not interviewed at the time of the inspection.

2.3 Operation and Maintenance Procedures

No information pertaining to formal operation and maintenance procedures was provided to LEI for this inspection.

2.4 Emergency Warning System

No information pertaining to a formal emergency warning system was provided to LEI for this inspection.

2.5 Hydrologic/Hydraulic Data

Based on an INTERMEDIATE size classification and a SIGNIFICANT hazard classification, the spillway design flood for this dam is the 100-year return frequency storm. The Phase II Report prepared by LEI contains the test flood analysis and hydrology and hydraulic calculations for this dam. This analysis determines that the current spillway will convey the spillway design flood without overtopping the dam.

2.6 Structural Stability/Overtopping Potential

2.6.1 Structural Stability

Structural stability and seepage analyses were conducted as part of the Phase II Report prepared by LEI. GeoInsight, Inc. provided an engineering analysis and recommendations regarding predicted stability and seepage rates for the dam. Veneer stability and global stability analysis were performed for the dam under the three different loading conditions with a required factor of safety (F.S.) area as follows; case I, steady seepage with maximum storage pool; case II, steady seepage with surcharge pool; and case III, steady seepage and a seismic loading

The slope stability analyses included ponded water being equivalent to two feet below the top of the dam (14 feet of head), one foot below the top of the dam (15 feet of head) and at the top of the dam (16 feet of head) with assessment being conducted of the upstream and downstream face under static and seismic conditions.

The dam stability analysis indicated that adequate resistance forces were available against sliding and overturning forces. However, the stability is based upon conditions that are currently not completely defined, including slope geometry and internal conditions against the spillway. Assessment of the upstream face during rapid drawdown conditions indicated that the veneer surface would most likely slough, leaving behind a steeper face that would in turn slough more until equilibrium between the soil and retained water within the dam core was reached. However, based on the gradation of the soil within the dam, it does not appear that the dam is significantly susceptile to liquefaction during seismic conditions.

Global stability analysis suggests that the upstream dam slope is currently in a state of failure but because this conflicts with visual observation, the face is likely stabilized by shallower grades than those modeled, vegetation and/or armor is not included in the evaluation. Overall global stability appears to be well below acceptable criteria. Global stability at the downstream face is highly dependent upon the existing fill material and natural soil, which is not readily quantifiable in terms of its integrity. The dam is significantly sensitive to a theoretical seismic loading because of the low density of the embankment materials. It is important to note that degraded, brittle existing structures such as the existing spillway could introduce very significant unknowns into the analysis. These loading scenarios are in accordance with State regulations, 302 CMR 10.14.

Global Stability

Case	Failure Mode	Required	Calculated
I	Overturning	3.0	**
II	Sliding	3.0	12.8
	Overturning	2.0	**
III	Sliding	2.0	11.4
	Overturning	>1.0	**
	Sliding	>1.0	9.9

Slope Stability

Location	Condition*	Required	Calculated
Upstream	Static	1.3	***0.9***
	Seismic	>1.0	***0.6***
Downstream	Static	1.3	3.1
	Seismic	>1.0	2.7

* all analyses considered impoundment at dam crest
** F.S. against overturning was analyzed by inspection and determined that the F>S. for the three conditions would be significantly higher than the required F.S.

GeoInsight, Inc. performed a limited seepage analysis of the dam structure. The results of the evaluation indicated that the dam could be seeping as much as 776 cubic feet per day across its entire alignment. The dam soil was estimated to have a seepage rate of $1x10^{-4}$ centimeter per second. However, this seepage rate was not consistent with observed conditions. The downstream side of the dam was observed to be moist, but seepage in the form of free flowing water was not observed. This suggests that either the upstream face materials restrict flow into the dam more than modeled in the analysis and/or the internal conductivity is higher and seepages exists into the base and largely out of view. It was also observed that a portion of the water exiting the spillway was traveling off-course from its designated channel, and instead was following a pathway along the base of the dam. It was difficult to determine if seepage was occurring at the base of the dam or if the wet soil conditions at the toe were due to standing water because of this pathway. During a site visit, it was observed that a large amount of tarp Rock has been placed on the northern portion of the dam's downstream face. According to a member of the Prindle Lake Association, this measure was taken to buttress an area of seepage that has been observed in this area. GeoInsight did not observe slough or other evidence of unstable conditions at the downstream face of the dam.

2.6.2 Overtopping Potential

The 1998 Inspection Report relates the previous flooding experiences. During Hurricane Diane in 1955, the entire Quinebaug drainage area suffered from severe flooding (see USGS Water Supply Paper 1420 "Floods of August – October 1955 New England to North Carolina"). The flood of record for the dam occurred on August 19, 1955 at 3:00 PM. The flow recorded at the USGS Gaging Station located on the Upper Sibley Pond Outlet at Charlton City, Massachusetts, about 2 miles from Prindle Lake Dam was 1240 CFS from a drainage area of 2.23 sq. mi. or

approximately 556 CFS / sq. mi. The estimated flow at Prindle Lake Dam during this event was 222 cfs. The dam was overtopped during both the 1955 and 1938 hurricanes.

The hydraulic analysis for the dam prepared as part of the 2008 Phase II Inspection indicates the dam will not be overtopped during the 100-year return frequency storm event. The dam does not have an emergency spillway therefore an overtopping potential exists. This potential is considered likely but low, with an occurrence frequency less that once per century.

SECTION 3

3.0 ASSESSMENTS AND RECOMMENDATIONS

3.1 Assessments

In general, Prindle Lake Dam was found to be in POOR condition. General concerns include:

1) Saturated downstream toe
2) Multiple locations of minor seepage
3) Loose sandy core material
4) Lack of effective slope treatments
5) Tree growth on the dam and areas within 25 feet of the toe
6) The lack of an emergency spillway and a low level outlet

Some of recommendations from the 2006 report have been executed. A detailed geotechnical evaluation and a hydrologic and hydraulic study have been performed, the results of which are presented with the Phase II Report.

In 2006, the condition of the dam was rated as poor. The current rating is also POOR. A safe means to convey storm flow in excess of the SDF needs to be provided. There are no controls to lower the water level should there be a need for a rapid drawdown of the lake. There is no positive mechanism of seepage control or conveyance (i.e., there are no toe drains).

The following recommendations and remedial measures generally describe the recommended approach to address current deficiencies at the dam. Prior to undertaking recommended maintenance, repairs and remedial measure, the applicability of environmental permits needs to be obtained prior to undertaking activities that may occur within resource areas under the jurisdiction of local conservation commissions, MADEP, or other regulatory agencies.

3.2 Studies and Analyses

1) A toe drain system, emergency spillway, slope stabilization system, and mechanism of dam core compaction should be designed by a registered profession engineer.
2) Prepare an Operations and Maintenance Plan for the dam.

3.3 Yearly Recommendations

1) Reseed areas of thin vegetation on slopes with grassy cover. Fill low spots on crest with gravel or crushed stone and cover with loam and seed.
2) Mow grass surfaces regularly (at least three times per year).
3) Monitor seepage to look for changing conditions.
4) Inspect the dam as required by Massachusetts General Law (by a qualified, registered professional engineer).

3.4 Recommendations, Maintenance, and Minor Repairs

1) Remove brush and debris from the crest and embankments (do not grub stumps). Remove fallen trees and pulled stumps away from the dam area to prevent rodent inhabitation.
2) Fill low spots and small depressions with appropriate fill material; gravel or crushed stone on the crest; topsoil covered with seed on the embankments.
3) Remove riprap from the downstream slope. Establish a grassy cover on both upstream and downstream slopes.

3.5 Remedial Measures

1) Clear and grub the dam and areas within 25 feet of the abutments and downstream toe. Regrade and install slope protection for upstream and downstream slopes.
2) Remove apparent abandon spillway and reconstruct that portion of the dam.
3) Repair cracked or deteriorated concrete and grout, as necessary, on the spillway and outlet channel.
4) Construct a defined waterway (minimum 50 feet downstream of toe) to safely convey discharge away from the toe of the dam. Regrade the immediate downstream area so that the channel under the existing spillway is utilized and the old channel becomes abandoned.
5) Install toe drains along the downstream face.
6) In-situ compact the dam core material.
7) Construct an emergency spillway and a low level outlet.

3.6 Alternatives

The preferred alternative identified within the Phase II report should be implemented. These include:

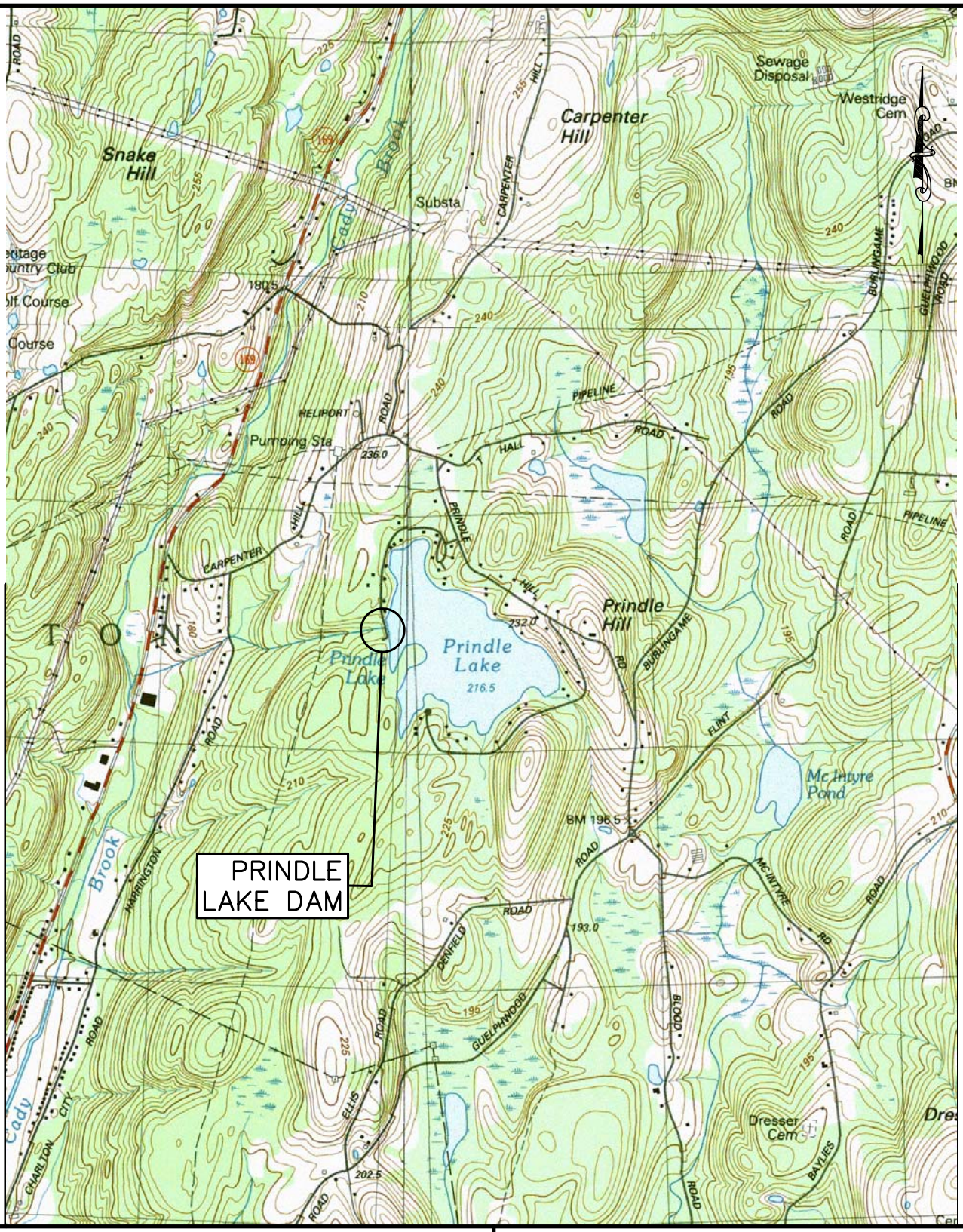
1) Installation of a low level outlet with controls;
2) Construction of an emergency spillway;
3) In-situ densification of the dam core material;
4) Installation of a toe drain;
5) Buttress downstream toe.

3.7 Opinion of Probable Construction Costs

Item	Opinion of Probable Cost
Engineering	$ 47,000
Permits	$ 7,500
Yearly Recommendations	$2,000 per year
Maintenance and Minor Repairs	$5,000
Remedial Measures	$288,000

TOTAL REPAIRS: $342,500 (does not include yearly recommendations)

FIGURES



Source:
USGS TOPOGRAPHIC MAP
WEBSTER, MA QUADRANGLE

Lenard Engineering, Inc.
Auburn, MA

FIGURE 1 - LOCUS PLAN

State ID# 3-14-54-68 NID# MA01174

PRINDLE LAKE DAM

CHARLTON, MA

Scale 1"=2000'

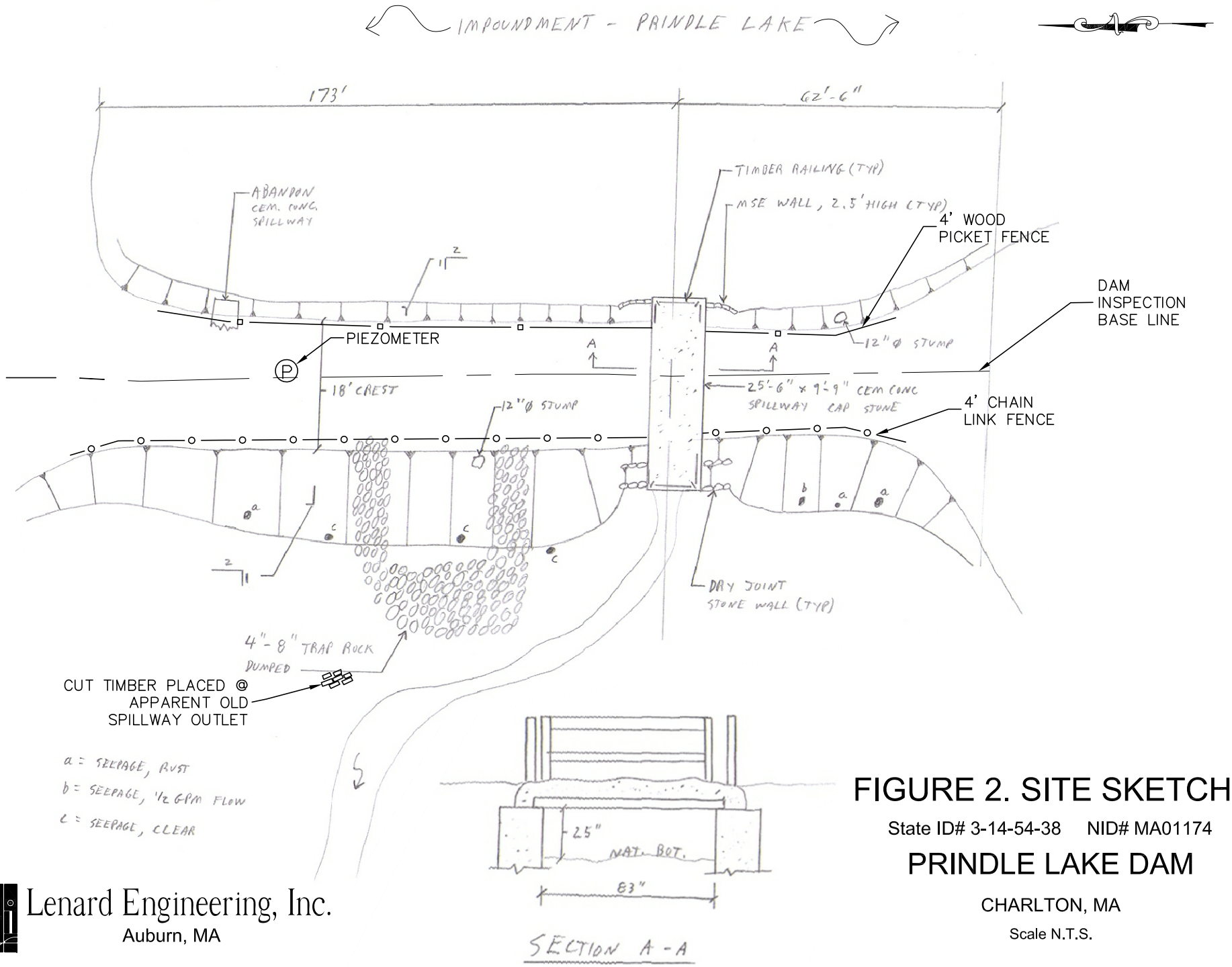


FIGURE 2. SITE SKETCH

State ID# 3-14-54-38 NID# MA01174

PRINDLE LAKE DAM

CHARLTON, MA

Scale N.T.S.

Lenard Engineering, Inc.
Auburn, MA

APPENDIX A
Photographs

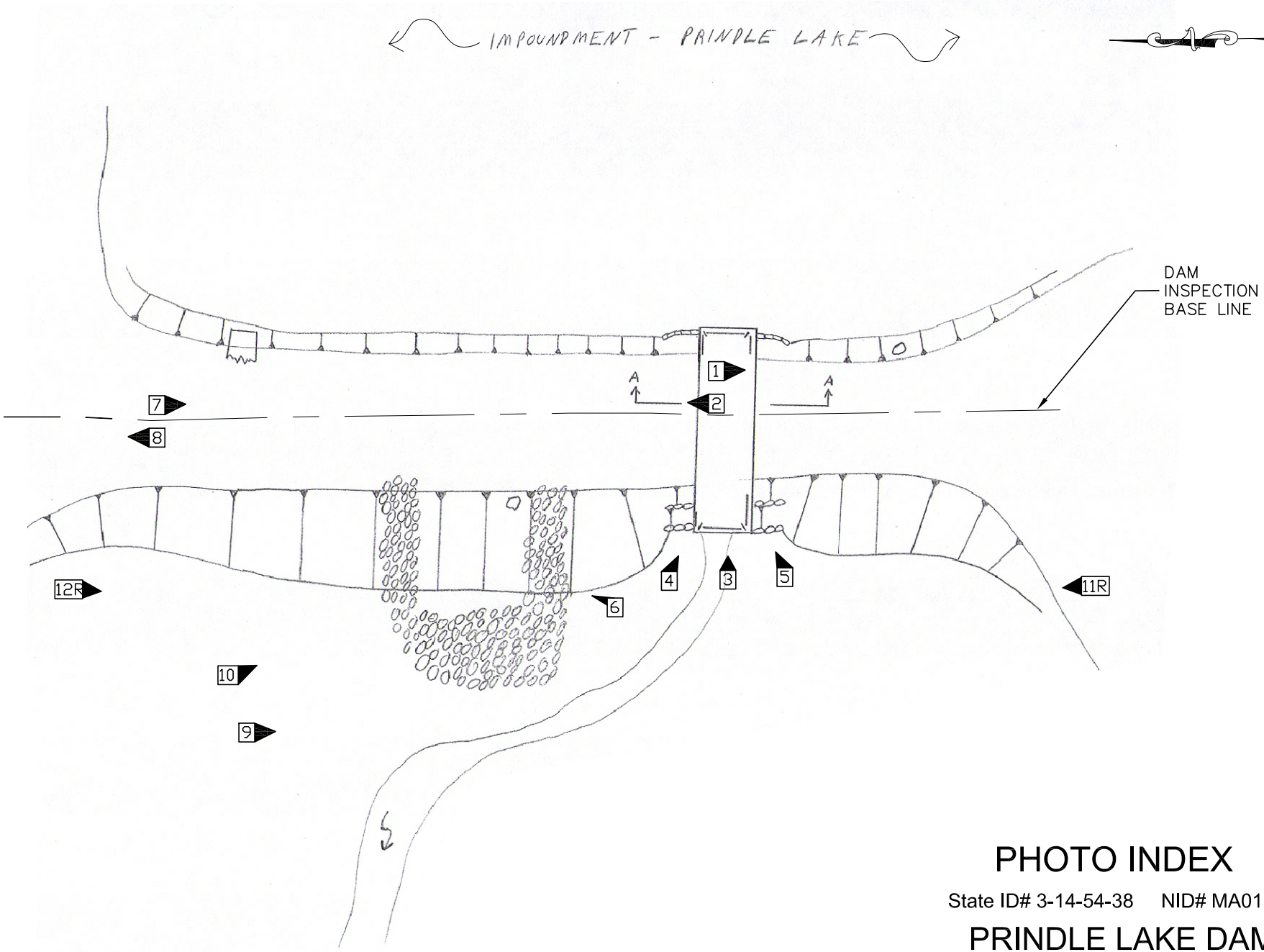


PHOTO INDEX

State ID# 3-14-54-38 NID# MA01174

PRINDLE LAKE DAM

CHARLTON, MA

Scale N.T.S.

Lenard Engineering, Inc.
Auburn, MA

Photo 001. Standing on spillway looking at left abutment

Photo 002. Standing on spillway looking right along crest

Photo 003. Spillway downstream opening

Photo 004. Spillway outlet undermining

Photo 005. Right of spillway outlet

Photo 006. Downstream face looking right

Photo 007. Crest looking right with piezometer location identified by stone

Photo 008. Right abutment

Photo 009. Cut timber placement at apparent old spillway outlet

Photo 010. Downstream face with apparent out spillway outlet left of center

Photo 011-R. Downstream face from April 2008 Inspection

Photo 012-R. Apparent abandon spillway seep from April 2008 Inspection (active spillway in left background)

APPENDIX B

Inspection Checklist

DAM SAFETY INSPECTION CHECKLIST INSTRUCTION PAGE

The checklist includes sections applicable to a variety of dam structure types. Complete those pages pertaining to each structure and omit pages that are not relevant. Checklist should be signed by the inspecting engineer and a clean, neat copy included in the final inspection report.

E1: DESIGN METHODOLOGY
1. Unknown Design – no design records available
3. Some standard design features
5. State of the art design – design records available

E2: LEVEL OF MAINTENANCE
1. No evidence of maintenance, no O&M manual
2. Very little maintenance, no O&M manual
3. Some level of maintenance and standard procedures
4. Adequate level of maintenance and standard procedures
5. Detailed maintenance plan that is executed

E3: EMERGENCY ACTION PLAN
1. No plan or idea of what to do in the event of an emergency
2. Some idea but no written plan
3. No formal plan but well thought out
4. Available written plan that needs updating
5. Detailed, updated written plan available and filed with MADCR

E4: EMBANKMENT SEEPAGE
1. Severe piping and/or seepage with no monitoring
2. Evidence of monitored piping and seepage
3. No piping but uncontrolled seepage
4. Controlled seepage
5. No seepage or piping

E5: EMBANKMENT CONDITION
1. Severe erosion and/or large trees
2. Significant erosion or significant woody vegetation along lower
3. Brush and exposed embankment soils, or moderate erosion
4. Unmaintained grass, rodent activity and maintainable erosion
5. Well maintained healthy uniform grass cover

E6: CONCRETE CONDITION
1. Major cracks, misalignment, discontinuities causing leaks, seepage or stability concerns
2. Cracks with misalignment inclusive of transverse cracks with no misalignment
3. Significant longitudinal cracking and minor transverse cracking
4. Spalling and minor surface cracking
5. No apparent deficiencies

E7: LOW LEVEL OUTLET DISCHARGE CAPACITY
1. No low level outlet
2. Outlet with insufficient drawdown capacity
3. Inoperable gate with potentially sufficient capacity
4. Operable gate with sufficient drawdown capacity
5. Operable gate with capacity greater than necessary

E8: LOW LEVEL OUTLET PHYSICAL CONDITION
1. Outlet inoperative needs replacement, non-existent or inaccessible
2. Outlet inoperative needs repair
3. Outlet operable but needs repair
4. Outlet operable but needs maintenance
5. Outlet and operator operable and well maintained

E9: SPILLWAY DESIGN FLOOD CAPACITY
1. 0 - 20% of the SDF
2. 21- 40% of the SDF
3. 41- 60% of the SDF
4. 61- 80% of the SDF
5. 81- 100% of the SDF

E10: OVERALL PHYSICAL CONDITION OF THE DAM
1. *UNSAFE* – Major structural, operational, and maintenance deficiencies exist under normal operating conditions
2. *POOR* - Significant structural, operation and maintenance deficiencies are clearly recognized for normal loading conditions
3. *FAIR* - Significant operational and maintenance deficiencies, no structural deficiencies. Potential deficiencies exist under unusual loading conditions that may realistically occur. Can be used when uncertainties exist as to. critical parameters
4. *SATISFACTORY* - Minor operational and maintenance deficiencies. Infrequent hydrologic events would probably result In deficiencies.
5. *GOOD* - No existing or potential deficiencies recognized. Safe performance is expected under all loading including SDF

E11: ESTIMATED REPAIR COST
Estimation of the total cost to address all identified structural, operational, maintenance deficiencies. Cost shall be developed utilizing standard estimating guides and procedures

See Appendix D for a complete listing of dam orientation and terminology definitions.

Upstream – Shall mean the side of the dam that borders the impoundment.
Downstream – Shall mean the high side of the dam, the side opposite the upstream side.
Right – Shall mean the area to the right when looking in the downstream direction.
Left – Shall mean the area to the left when looking in the downstream direction.
Height of Dam – Shall mean the vertical distance from the lowest portion of the natural ground, including any stream channel, along the downstream toe of the dam to the crest of the dam.
Embankment – Shall mean the fill material, usually earth or rock, placed with sloping sides, such that it forms a permanent barrier that impounds water.
Crest – Shall mean the top of the dam, usually provides a road or path across the dam.
Abutment – Shall mean that part of a valley side against which a dam is constructed. An artificial abutment is sometimes constructed as a concrete gravity section, to take the thrust of an arch dam where there is no suitable natural abutment.
Appurtenant Works – Shall mean structures, either in dams or separate therefrom. including but not be limited to, spillways; reservoirs and their rims; low level outlet works; and water conduits including tunnels, pipelines, or penstocks, either through the dams or their abutments.
Spillway – Shall mean a structure over or through which water flows are discharged. If the flow is controlled by gates or boards, it is a controlled spillway; if the fixed elevation of the spillway crest controls the level of the impoundment, it is an uncontrolled spillway.

DAM SAFETY INSPECTION CHECKLIST

NAME OF DAM: Prindle Lake Dam

STATE ID #: 3-14-54-38

REGISTERED: ☐ YES ☑ NO

NID ID #: MA01174

STATE SIZE CLASSIFICATION: INTERMEDIATE

STATE HAZARD CLASSIFICATION: SIGNIFICANT

LOCATION INFORMATION

CITY/TOWN: Charlton

COUNTY: Worcester

DAM LOCATION: Charlton; end of Oak Ridge Drive

AKA NAME:

USGS QUAD.: Webster

LAT.: 42.11365571

LONG.: -72.0005309

DRAINAGE BASIN: 9 - Quinebaug

RIVER: Tributary of Cady Brook

IMPOUNDMENT NAME(S): Prindle Lake

GENERAL DAM INFORMATION

TYPE OF DAM: Earthen with box-culvert spillway

OVERALL LENGTH (FT): 235.5

PURPOSE OF DAM: Recreation

NORMAL POOL STORAGE (ACRE-FT): 150

YEAR BUILT: 1952

MAXIMUM POOL STORAGE (ACRE-FT): 250

STRUCTURAL HEIGHT (FT): 11

EL. NORMAL POOL (FT): 705 msl

HYDRAULIC HEIGHT (FT): 7

EL. MAXIMUM POOL (FT): 707 msl

FOR INTERNAL MADCR USE ONLY

FOLLOW-UP INSPECTION REQUIRED: ☐ YES ☐ NO

CONDITIONAL LETTER: ☐ YES ☐ NO

NAME OF DAM: Prindle Lake Dam

STATE ID #: 3-14-54-38

NID ID #: MA01174

INSPECTION SUMMARY

DATE OF INSPECTION: June 6, 2008

DATE OF PREVIOUS INSPECTION: April 7, 1998

TEMPERATURE/WEATHER: Overcast, drizzle, 60's

ARMY CORP PHASE I: ☐ YES ☑ NO If YES, date

CONSULTANT: Lenard Engineering, Inc.

PREVIOUS DCR PHASE I: ☑ YES ☐ NO If YES, date 2006

BENCHMARK/DATUM:

OVERALL CONDITION: POOR

DATE OF LAST REHABILITATION: Unknown

EL. POOL DURING INSP.: 705

EL. TAILWATER DURING INSP.: 692.5

PERSONS PRESENT AT INSPECTION

NAME	TITLE/POSITION	REPRESENTING
Scott D. Charpentier, P.E.	Project Manager	Lenard Engineering, Inc.
Karen Fung	Project Engineer	Lenard Engineering, Inc.

EVALUATION INFORMATION

E1) TYPE OF DESIGN	1	E8) LOW-LEVEL OUTLET COND.	1
E2) LEVEL OF MAINTENANCE	2	E9) SPILLWAY DESIGN FLOOD	5
E3) EMERGENCY ACTION PLAN	1	E10) GENERAL CONDITIONS	2
E4) EMBANKMENT SEEPAGE	3	E11) ESTIMATED REPAIR COST ($000)	150
E5) EMBANKMENT CONDITION	3	ROADWAY OVER CREST	☐ YES ☑ NO
E6) CONCRETE CONDITION	4	BRIDGE NEAR DAM	☐ YES ☑ NO
E7) LOW-LEVEL OUTLET CAP	1		

SIGNATURE OF INSPECTING ENGINEER: Scott D. Charpentier

NAME OF DAM: Prindle Lake Dam

STATE ID #: 3-14-54-38

NID ID #: MA01174

MA01174

OWNER:		CARETAKER:	
ORGANIZATION	The Santos Irrevocable Trust	ORGANIZATION	Nature's Classroom
NAME/TITLE	Carl Izzo, Trustee; Fiduciary Real Estate Advisors	NAME/TITLE	Dr. John G. Santos, Director
STREET	125 Summer Street	STREET	19 Harrington Rd.
TOWN, STATE, ZIP	Boston, MA 02110-1624	TOWN, STATE, ZIP	Charlton, MA 01507
PHONE	1-617-345-3600	PHONE	1-508-248-2741
FAX	1-617-345-3640	FAX	1-508-248-2745
EMAIL		EMAIL	
OWNER TYPE	Private		

PRIMARY SPILLWAY TYPE: Concrete broad crested weir (box culvert)

SPILLWAY LENGTH (FT): 6.92

SPILLWAY CAPACITY (CFS): 65.5

AUXILIARY SPILLWAY TYPE: No auxiliary spillway

AUX. SPILLWAY CAPACITY (CFS): N/A

NUMBER OF OUTLETS: None

OUTLET(S) CAPACITY (CFS): N/A

TYPE OF OUTLETS: N/A

TOTAL DISCHARGE CAPACITY (CFS): 65.5

DRAINAGE AREQ (SQ MI): 0.4

SPILLWAY DESIGN FLOOD (PERIOD/CFS): 100-YR - 711 cfs

HAS DAM BEEN BREACHED OR OVERTOPPED ☑ YES ☐ NO

IF YES, PROVIDE DATE(S): Overtopped 1938 Hurricane & 1955 Hurricane Diane

FISH LADDER (LIST TYPE IF PRESENT): None

DOES CREST SUPPORT PUBLIC ROAD? ☐ YES ☑ NO

IF YES, ROAD NAME:

PUBLIC BRIDGE WITHIN 50' OF DAM? ☐ YES ☑ NO

IF YES, ROAD/BRIDGE NAME:

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

EMBANKMENT

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
CREST	SURFACE TYPE	Earthen	X		
	SURFACE CRACKING	None observed	X		
	SINKHOLES, ANIMAL BURROWS	None observed	X		
	VERTICAL ALIGNMENT (DEPRESSIONS)	Some minor depressions; crest has vertical radius, higher at left abut. than right		X	
	HORIZONTAL ALIGNMENT	Satisfactory	X		
	RUTS AND/OR PUDDLES	None observed	X		
	VEGETATION (PRESENCE/CONDITION)	Grassy vegetation, some spots of bare earth (see below).		X	X
	ABUTMENT CONTACT	Satisfactory	X		

ADDITIONAL COMMENTS: Shoulders exhibit broad leaf vegetation. Some brushy vegetation on shoulder near the upstream side of right abutment.

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

EMBANKMENT

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
D/S SLOPE	WET AREAS (NO FLOW)	1 spot to rt of spway; unable to observe most of sl due to vegetation; toe saturated		X	X
	SEEPAGE	seepage along toe with measuable flow to lt of spillway (1/2 gpm)		X	X
	SLIDE, SLOUGH, SCARP	Riprap nearer to right abutment has slid		X	
	EMB.-ABUTMENT CONTACT	Satisfactory	X		
	SINKHOLE/ANIMAL BURROWS	Unable to observe		X	
	EROSION	Unable to observe		X	
	UNUSUAL MOVEMENT	Unable to observe		X	
	VEGETATION (PRESENCE/CONDITION)	Broad leaf vegetation		X	X

ADDITIONAL COMMENTS: Remove vegetation and inspect again. Regrade and reseed
Saturated toe for most of dam length
Rip rap dumped at two locations, unable to determine why, possibly to dress erosion
Slope of 2H:1V

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

EMBANKMENT

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
U/S SLOPE	SLIDE, SLOUGH, SCARP	Unable to observe		X	
	SLOPE PROTECTION TYPE AND COND.	Broad leaf vegetation, some brushy vegetation, bare soil below		X	X
	SINKHOLE/ANIMAL BURROWS	Unable to observe		X	
	EMB.-ABUTMENT CONTACT	Satisfactory	X		
	EROSION	Unable to observe		X	
	UNUSUAL MOVEMENT	Unable to observe		X	
	VEGETATION (PRESENCE/CONDITION)	Broad leaf vegetation, some brushy vegetation, bare soil below		X	X

ADDITIONAL COMMENTS: 12" stump left near left abutment
Upstream face below water surface appears to be partially protected by rip rap

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

EMBANKMENT

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
INSTR.	PIEZOMETERS	one installed, water elevation found to be 5-feet below dam crest		X	
	OBSERVATION WELLS	None observed			
	STAFF GAGE AND RECORDER	None observed			
	WEIRS	None observed			
	INCLINOMETERS	None observed			
	SURVEY MONUMENTS	None observed			
	DRAINS	None observed			
	FREQUENCY OF READINGS	None observed			
	LOCATION OF READINGS	None observed			

ADDITIONAL COMMENTS:

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

UPSTREAM AND/OR DOWNSTREAM MASONRY WALLS

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
D/S WALLS	WALL TYPE	Stone masonry terraced walls at either side of spillway outlet	X		
	WALL ALIGNMENT	Fair		X	
	WALL CONDITION	Fair		X	
	HEIGHT: TOP OF WALL TO MUDLINE	Walls appear to go from crest to midway down slope	X		
	SEEPAGE OR LEAKAGE	None		X	
	ABUTMENT CONTACT	Not applicable	X		
	EROSION/SINKHOLES BEHIND WALL	None		X	
	ANIMAL BURROWS	None		X	
	UNUSUAL MOVEMENT	None		X	
	WET AREAS AT TOE OF WALL	None		X	

ADDITIONAL COMMENTS: Grout missing in many places

NAME OF DAM: Prindle Lake Dam STAE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

DOWNSTREAM AREA

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
D/S AREA	ABUTMENT LEAKAGE	Toe is saturated		X	X
	FOUNDATION SEEPAGE	Toe is saturated		X	X
	SLIDE,SLOUGH,SCARP	Unable to determine	X		
	WEIRS	None observed	X		
	DRAINAGE SYSTEM	None observed	X		
	INSTRUMENTATION	None observed	X		
	VEGETATION	Woody plants, brush, saplings and trees		X	X
	ACCESSIBILITY	By foot	X		
	DOWNSTREAM HAZARD DESCRIPTION	Harrington Rd: private residences and a culvert; SR 169 (Southbridge St.): L&P Paper Inc., private residences, and a bridge (near Sherwood Ln).			
	DATE OF LAST EAP UPDATE	Unknown			X

ADDITIONAL COMMENTS: Unable to determine slide due to recently dumped rip rap.

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

MISCELLANEOUS

AREA INSPECTED	CONDITION	OBSERVATIONS
MISC.	RESERVOIR DEPTH (AVG)	8-10 feet
	RESERVOIR SHORELINE	Vegetated w/ grass, brush, trees. Private residences.
	RESERVOIR SLOPES	Unknown
	ACCESS ROADS	Dirt road (Oak Ridge Drive) and dirt foot paths
	SECURITY DEVICES	No special devices; timber rail along spway top; fencing along left shores
	VANDALISM OR TRESPASS	YES: ☐ NO: ☑ WHAT:
	AVAILABILITY OF PLANS	YES: ☐ NO: ☑ * DATE:
	AVAILABILITY OF DESIGN CALCS	YES: ☐ NO: ☑ * DATE:
	AVAILABILITY OF EAP/LAST UPDATE	YES: ☐ NO: ☑ * DATE:
	AVAILABILITY OF O&M MANUAL	YES: ☐ NO: ☑ * DATE:
	CARETAKER/OWNER AVAILABLE	YES: ☐ NO: ☑ DATE:
	CONFINED SPACE ENTRY REQUIRED	YES: ☐ NO: ☑ PURPOSE:

* UNKNOWN

ADDITIONAL COMMENTS:

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

PRIMARY SPILLWAY

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
SPILLWAY	SPILLWAY TYPE	Broad crested weir; cast in place concrete box culvert (slab-over design)	X		
	WEIR TYPE	Broad crested weir	X		
	SPILLWAY CONDITION	Satisfactory; culvert top slab is spalled.		X	X
	TRAINING WALLS	7' long curved MSE wingwalls U/S; cracked grout		X	X
	SPILLWAY CONTROLS AND CONDITION	No controls observed	X		
	UNUSUAL MOVEMENT	None observed	X		
	APPROACH AREA	Satisfactory	X		
	DISCHARGE AREA	Channel sides defined w/ rocks/riprap		X	
	DEBRIS	Some debris		X	X
	WATER LEVEL AT TIME OF INSPECTION	18" deep at entrance; 1" over spway at exit	X		

ADDITIONAL COMMENTS: A large piece of ledge sits at the exit of the spillway but does not appear to be elevated above the spillway floor.

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

AUXILIARY SPILLWAY

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
SPILLWAY	SPILLWAY TYPE	N/A			
	WEIR TYPE				
	SPILLWAY CONDITION				
	TRAINING WALLS				
	SPILLWAY CONTROLS AND CONDITION				
	UNUSUAL MOVEMENT				
	APPROACH AREA				
	DISCHARGE AREA				
	DEBRIS				
	WATER LEVEL AT TIME OF INSPECTION				

ADDITIONAL COMMENTS:

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

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OUTLET WORKS

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
OUTLET WORKS	TYPE				
	INTAKE STRUCTURE				
	TRASHRACK				
	PRIMARY CLOSURE				
	SECONDARY CLOSURE				
	CONDUIT				
	OUTLET STRUCTURE/HEADWALL				
	EROSION ALONG TOE OF DAM				
	SEEPAGE/LEAKAGE				
	DEBRIS/BLOCKAGE				
	UNUSUAL MOVEMENT				
	DOWNSTREAM AREA				
	MISCELLANEOUS				

N/A

ADDITIONAL COMMENTS:

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

CONCRETE/MASONRY DAMS

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
GENERAL	TYPE	N/A			
	AVAILABILITY OF PLANS				
	AVAILABILITY OF DESIGN CALCS				
	PIEZOMETERS				
	OBSERVATION WELLS				
	INCLINOMETERS				
	SEEPAGE GALLERY				
	UNUSUAL MOVEMENT				

ADDITIONAL COMMENTS:

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

CONCRETE/MASONRY DAMS

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
U/S FACE	TYPE SURFACE CONDITIONS CONDITIONS OF JOINTS UNUSUAL MOVEMENT ABUTMENT CONTACTS	N/A			

ADDITIONAL COMMENTS:

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

CONCRETE/MASONRY DAMS

AREA INSPECTED	CONDITION	OBSERVATIONS	NO ACTION	MONITOR	REPAIR
D/S FACE	TYPE	N/A			
	SURFACE CONDITIONS				
	CONDITIONS OF JOINTS				
	UNUSUAL MOVEMENT				
	ABUTMENT CONTACTS				
	DRAINS				
	LEAKAGE				

ADDITIONAL COMMENTS:

NAME OF DAM: Prindle Lake Dam STATE ID #: 3-14-54-38

INSPECTION DATE: June 6, 2008 NID ID #: MA01174

CONCRETE/MASONRY DAMS

AREA INSPECTED	CONDITION	OBSERVATIONS
CREST	TYPE SURFACE CONDITIONS CONDITIONS OF JOINTS UNUSUAL MOVEMENT HORIZONTAL ALIGNMENT VERTICAL ALIGNMENT	N/A

ADDITIONAL COMMENTS:

APPENDIX C

Previous Reports and References

PREVIOUS REPORTS AND REFERENCES

The following is a list of reports that were located during the file review, or were referenced in previous reports:

1. Department of Environmental Management, Office of Dam Safety, Inspection/Evaluation Report – Prindle Lake Dam, prepared by Fay Engineering Services for the DEM, Thorndike, MA, 1998.

2. Prindle Lake Dam Phase I Inspection / Evaluation Report, prepare by Lenard Engineering, Inc., September 2006

3. Prindle Lake Dam Poor and Unsafe Condition Dam Follow-up Inspection, prepare by Lenard Engineering, Inc., April 2008

4. Prindle Lake Dam Phase II Dam Evaluation Report, prepared by Lenard Engineering, Inc., April 2008

APPENDIX D

Definitions and Abbreviations

COMMON DAM SAFETY DEFINITIONS

For a comprehensive list of dam engineering terminology and definitions refer to 302 CMR10.00 Dam Safety, or other reference published by FERC, Dept. of the Interior Bureau of Reclamation, or FEMA. Please note should discrepancies between definitions exits, those definitions included within 302 CMR 10.00 govern for dams located within the Commonwealth of Massachusetts.

Orientation

Upstream – Shall mean the side of the dam that borders the impoundment.

Downstream – Shall mean the high side of the dam, the side opposite the upstream side.

Right – Shall mean the area to the right when looking in the downstream direction.

Left – Shall mean the area to the left when looking in the downstream direction.

Dam Components

Dam – Shall mean any artificial barrier, including appurtenant works, which impounds or diverts water.

Embankment – Shall mean the fill material, usually earth or rock, placed with sloping sides, such that it forms a permanent barrier that impounds water.

Crest – Shall mean the top of the dam, usually provides a road or path across the dam.

Abutment – Shall mean that part of a valley side against which a dam is constructed. An artificial abutment is sometimes constructed as a concrete gravity section, to take the thrust of an arch dam where there is no suitable natural abutment.

Appurtenant Works – Shall mean structures, either in dams or separate therefrom. including but not be limited to, spillways; reservoirs and their rims;low level outlet works; and water conduits including tunnels, pipelines, or penstocks, either through the dams or their abutments.

Spillway – Shall mean a structure over or through which water flows are discharged. If the flow is controlled by gates or boards, it is a controlled spillway; if the fixed elevation of the spillway crest controls the level of the impoundment, it is an uncontrolled spillway.

Size Classification

(as listed in Commonwealth of Massachusetts, 302 CMR 10.00 *Dam Safety*)

Large – structure with a height greater than 40 feet or a storage capacity greater than 1,000 acre-feet.

Intermediate – structure with a height between 15 and 40 feet or a storage capacity of 50 to 1,000 acre-feet.

Small – structure with a height between 6 and 15 feet and a storage capacity of 15 to 50 acre-feet.

Non-Jurisdictional – structure less than 6 feet in height or having a storage capacity of less than 15 acre-feet.

Hazard Classification

(as listed in Commonwealth of Massachusetts, 302 CMR 10.00 *Dam Safety*)

High Hazard (Class I) – Shall mean dams located where failure will likely cause loss of life and serious damage to home(s), industrial or commercial facilities, important public utilities, main highway(s) or railroad(s).

Significant Hazard (Class II) – Shall mean dams located where failure may cause loss of life and damage to home(s), industrial or commercial facilities, secondary highway(s) or railroad(s), or cause the interruption of the use or service of relatively important facilities.

Low Hazard (Class III) – Dams located where failure may cause minimal property damage to others.Loss of life is not expected.

General

EAP – Emergency Action Plan - Shall mean a predetermined plan of action to be taken to reduce the potential for property damage and/or loss of life in an area affected by an impending dam break.

O&M Manual – Operations and Maintenance Manual; Document identifying routine maintenance and operational procedures under normal and storm conditions.

Normal Pool – Shall mean the elevation of the impoundment during normal operating conditions.

Acre-foot – Shall mean a unit of volumetric measure that would cover one acreto a depth of one foot. It is equal to 43,560 cubic feet. On million U.S. gallons = 3.068 acre feet
Height of Dam – Shall mean the vertical distance from the lowest portion of the natural ground, including any stream channel, along the downstream toe of the dam to the crest of the dam.

Spillway Design Flood (SDF) – Shall mean the flood used in the design of a dam and its appurtenant works particularly for sizing the spillway and outlet works, and for determining maximum temporary storage and height of dam requirements.

Condition Rating

Unsafe - Major structural, operational, and maintenance deficiencies exist under normal operating conditions.

Poor - Significant structural, operation and maintenance deficiencies are clearly recognized for normal loading conditions.

Fair - Significant operational and maintenance deficiencies, no structural deficiencies. Potential deficiencies exist under unusual loading conditions that may realistically occur. Can be used when uncertainties exist as to critical parameters.

Satisfactory - Minor operational and maintenance deficiencies. Infrequent hydrologic events would probably result in deficiencies.

Good - No existing or potential deficiencies recognized. Safe performance is expected under all loading including SDF.

Abbreviations

Abut – abutment
U/S – upstream
Lt - left
D/S – downstream
LL – low level
Rt - right