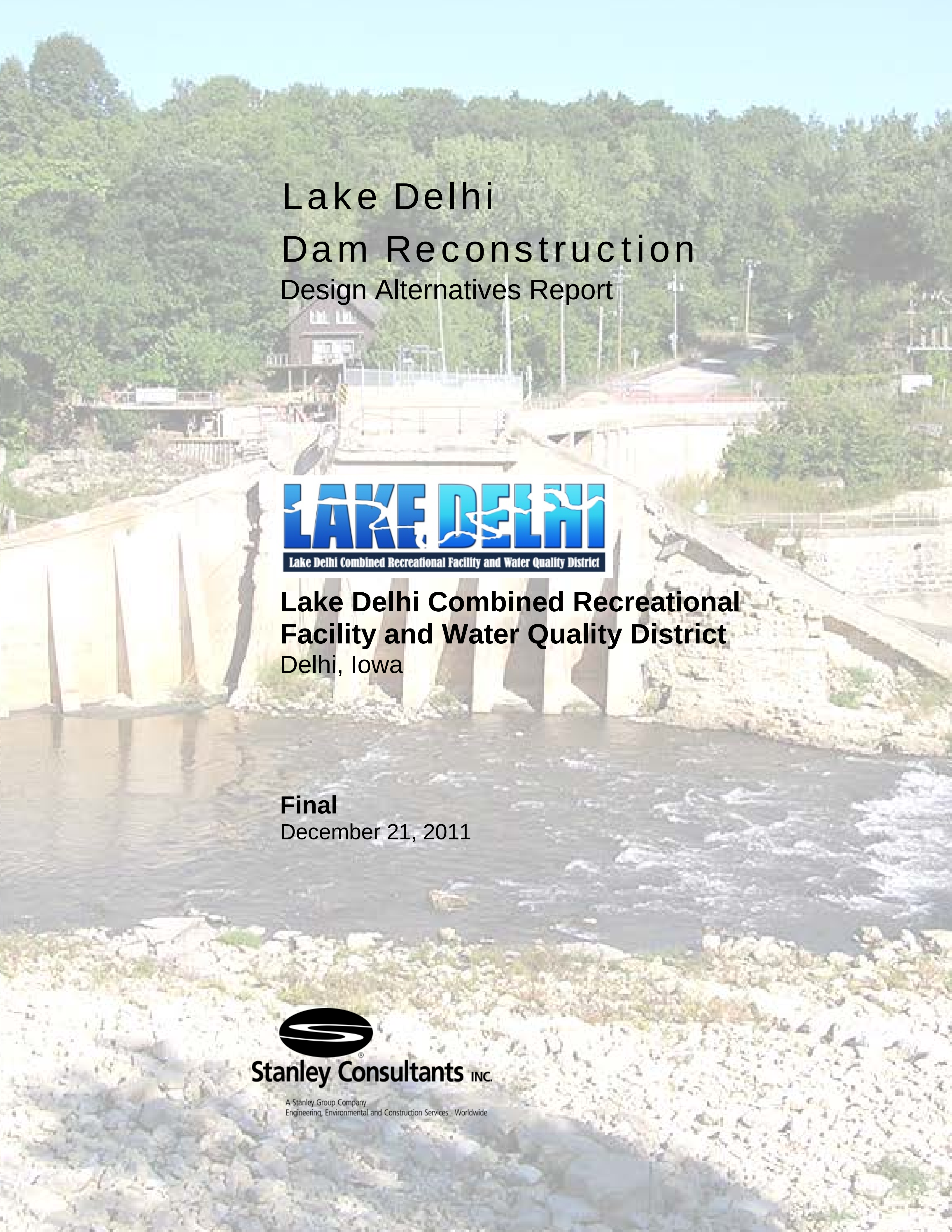

Lake Delhi
Dam Reconstruction
Design Alternatives Report

Lake Delhi Combined Recreational Facility and Water Quality District
Delhi, Iowa

Final
December 21, 2011

Stanley Consultants INC.
A Stanley Group Company
Engineering, Environmental and Construction Services - Worldwide

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Final

December 21, 2011

LICENSED PROFESSIONAL ENGINEER
WILLIAM E. HOLMAN
20164
IOWA

I hereby certify that this engineering document was prepared by me or under my direct personal supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Iowa.

William E. Holman 12/21/11

William E. Holman 12/21/2011

License number 20164

My license renewal date is December 31, 2013.

Pages or sheets covered by this seal: All

Stanley Consultants INC.

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Executive Summary

Lake Delhi Dam is located on the Maquoketa River in Delaware County, Iowa. The dam is maintained and managed by the Lake Delhi Combined Recreational Facility and Water Quality District (District). During the flood event of July 23–24, 2010, the dam's southern earthen embankment was overtopped and fully eroded and the concrete spillway gates were damaged. Floodwaters also infiltrated and seeped through a section of the northern embankment.

Stanley Consultants was selected by the District to perform analysis and preliminary design for the dam's reconstruction and restoration of the Lake Delhi Dam pool.

The objectives of this phase of the project are:

- Provide documentation of the existing condition of Lake Delhi Dam.
- Collect sufficient data to perform technical analysis and preliminary design of the dam reconstruction.
- Review regulatory requirements for dam reconstruction and present findings from the archaeological survey of the lake area.
- Develop and review alternatives for reconstructing Lake Delhi Dam and bringing the dam into compliance with current dam safety standards.
- Provide recommendations for final design and construction.
- Provide a preliminary estimate of construction costs and schedule.

This report provides a summary of findings from the surveys, research, inspection, technical analysis, and preliminary design performed to satisfy the project objectives.

Lake Delhi Dam consists of a series of distinct structures and features; all of which were evaluated in consideration of reconstruction of the dam, restoration of the lake, and future maintenance and operation of the dam.

Field investigation and data collection programs were completed to obtain the information and data required for assessment of the condition of existing structures and equipment and developing and evaluating conceptual designs.

Engineering analyses were completed to establish design requirements for reconstruction. These analyses established engineering parameters that will be utilized in design of repair and construction features, as well as minimum loading conditions for meeting current dam safety and design standards.

A detailed hazard classification analysis was completed and results indicate that a reconstructed Lake Delhi Dam most closely matches the DNR's Moderate Hazard Classification.

The Stanley Consultants design team met with District representatives for an Alternatives Development "Brainstorming" Session and objectives for the reconstruction project were established.

Project objectives were used as the framework for development of potential reconstruction alternatives for Lake Delhi Dam. Conceptual designs were used to estimate construction costs and evaluate each alternative relative to project objectives.

Based on the project objective evaluation and cost comparison, reconstruction alternatives were selected for incorporation into the "recommended project." A preliminary construction cost estimate and construction schedule were then developed for the recommended project.

It is recommended that construction be split into two phases. The first phase would involve restoration and upgrading of the existing powerhouse and gated spillway structure (north side). The second phase would involve reconstruction of the eroded southern embankment and construction of a new spillway to increase discharge capacity.

From the preliminary scheduling it was determined that construction could be accomplished in one construction season but assumes normal weather conditions and an experienced contractor with sufficient resources.

The construction cost estimate for the "recommended project" at this conceptual stage of design is approximately $11.9 million.

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Section 1

Project Description

1.1 General

Lake Delhi Dam is located on the Maquoketa River in Delaware County, Iowa. The dam is maintained and managed by the Lake Delhi Combined Recreational Facility and Water Quality District (District). During the flood event of July 23–24, 2010, the dam's southern earthen embankment was overtopped and fully eroded and the concrete spillway gates were damaged. Floodwaters also infiltrated and seeped through a section of the northern embankment.

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1.2 Description of Features

Exhibit 1 in Appendix F displays the layout of the dam features. A description of the dam's major structures and features evaluated during the alternatives analysis is provided by the following:

Lake Area

Prior to the 2010 dam breach, Lake Delhi extended approximately 7 miles upstream of the dam on the Maquoketa River, with a surface area of approximately 440 acres and a storage volume of 3790 acre-feet. During normal flow conditions, the pool elevation was maintained at elevation 896.3 ft-msl.

North Embankment

The north embankment is located between the north river bank of the Maquoketa River and the Powerhouse Structure and consists of earthen embankment with retaining walls supporting a concrete roadway (230th Avenue). The upstream retaining wall is a precast concrete "bin-type" retaining wall and the downstream retaining wall is a curved reinforced concrete wall. Both walls connect to the Powerhouse Structure.

Powerhouse Structure

The powerhouse structure was built in the 1920s by Interstate Power. Power generation was ceased in 1973, but the wicket gates continued to be used for passing normal flows at the dam. The Powerhouse is a multi-level reinforced concrete structure consisting of three main rooms on three levels. The upper level is the control room, the middle level is the turbine room, and the lower level is the mechanical room. The roof of the powerhouse is a concrete bridge deck with an operator platform separated from the bridge deck by fencing and railing on the upstream side. The top of the bridge deck is at elevation 904.7 ft-msl. Two turbine intakes with a trash rake system are located on the upstream side of the structure. Flow through these intakes is controlled by the wicket gates which were used to discharge normal flows at the dam.

Gated Spillway Structure

The gated spillway structure is located adjacent to the Powerhouse Structure and includes three concrete ogeee spillways separated by concrete spillway piers and abutment walls, with a concrete bridge deck over the top. Three vertical steel slide gates and hoisting equipment are located on a platform on the upstream side of the structure, separated from the bridge deck by fencing and railing. The crest of the ogee spillway is at elevation 879.8 ft-msl, approximately 16.5 feet below the normal pool elevation. The slide gates were usually kept closed and only opened to pass debris and flood magnitude flows at the dam. A large quantity of riprap was deposited upstream of the spillway gates in 2009.

Stilling Basin

The stilling basin is located on the downstream side of the gated spillway structure at the river channel bottom. The concrete stilling basin floor is at an elevation of roughly 849 ft-msl, but is currently buried under approximately 10 feet of silt. The stilling basin is bordered on the north and south by the North Downstream Abutment Wall and the South Buttress Wall, respectively.

South Buttress Wall

The south buttress wall is located on the south side of the gated spillway structure. Prior to the breach it tied into the southern earthen embankment. The wall is on the north side of the breach area and currently approximately 40 feet of the wall is exposed from the channel bottom up to the top of the concrete bridge deck. An abandoned narrow concrete fish ladder is fastened to the top of wall on the downstream side.

North Downstream Abutment Wall

The North Downstream Abutment Wall extends downstream from the Powerhouse Structure. The base and lower portion of the wall is reinforced concrete and the upper portion is masonry block. There is a gravel and grassed access area behind the wall that is even with the turbine room floor of the powerhouse.

South Embankment Area

The South Embankment Area was breached during the 2010 flood. The earthen embankment was almost fully washed away, exposing the river channel bottom. Following the breach, the Iowa Department of Natural Resources (DNR) completed a channel stabilization project that involved installing riprap along the channel bottom and south river bank. There is a concrete cutoff wall that is currently exposed on the south river bank. This cutoff wall used to extend to the South Buttress Wall, but this portion was washed away in the 2010 flood.

Information pertaining to the current condition and reconstruction alternatives for the dam features and structures is provided in subsequent sections of the report.

Section 2

Field Investigations and Data Collection

2.1 General

Field investigation and data collection programs were completed to obtain information for evaluating existing structures and equipment and developing conceptual design. Collected data were utilized in engineering analyses and evaluations to determine upgrades necessary to meet current requirements of regulating agencies and modern dam design standards. These data were also used in the development and evaluation of conceptual designs for dam repair/reconstruction alternatives. The investigation and data collection programs included:

- Topographic Survey.
- Property Research.
- Geotechnical Investigations.
- Structural Investigations.
- Electrical Investigations.
- Mechanical Investigations.
- Archeological Reconnaissance.
- Permitting Requirement Review.

2.2 Topographic Survey

Topographic survey of the dam area was performed by Gibbs Engineering and Surveying from Manchester, Iowa (Gibbs). The survey was performed over several weeks in June and early July 2011. Gibbs issued AutoCAD drawings of the topographic survey which were used in the preliminary design of reconstruction alternatives and development of reconstruction alternative exhibits.

The coordinate system and vertical datum for the topographic survey are:

- Coordinate System: North American Datum of 1983 (NAD 83), State Plane, Iowa North Zone (1401).
- Vertical Datum: North American Vertical Datum of 1988 (NAVD 88).

In addition to NAVD 88, some reference documents and drawings use the National Geodetic Vertical Datum of 1929 (NGVD 29) as well as a local datum which converts to NGVD 29 by adding 774.8 ft to the local datum elevation. NAVD 88 is approximately 0.1 feet lower than NGVD 29 in the Lake Delhi region. Due to the number of references used in this report, the lack of datum definition on several documents, the small difference in vertical datums, and the preliminary stage of design, conversions were not made to a single datum. A single datum (likely NAVD 88) would be used in final design and during construction.

Light Detection and Ranging (LiDAR) data was used for topography outside of the Gibbs survey area (mostly used in hydrologic and hydraulic analysis). LiDAR uses aircraft mounted light-emitting laser scanners to obtain high accuracy elevation data. The Iowa Department of Natural Resources (DNR) has obtained LiDAR data for the entire state of Iowa. LiDAR in the Lake Delhi region was flown post-breach so contains the topography of Lake Delhi below the normal pool which was helpful in developing hydraulic model cross-sections. The DNR LiDAR uses NAVD 88 so is on the same vertical datum as the Gibbs survey.

2.3 Property Research

Gibbs also developed an exhibit of property boundaries in the dam area. The exhibit is not a legal property survey but provides a good depiction of property line locations relative to the dam and surrounding area. Development of the property exhibit included:

- Reviewing Gibb’s previous surveys and extracting plats, deeds, and easements.
- Reviewing Gibb’s previous computer drawings and copying previous line/survey work
- Generating a property line exhibit compiling previous Gibb’s work.
- Conducting research at County courthouse and obtaining nearby plats, surveys, or deeds. Obtaining court cases from the clerk of court referencing the recent Rocky Nook, Lake Delhi Recreation Association property dispute.
- Drawing in the geometry of missing plats into property line exhibit.
- Obtaining coordinates of accessible property pins during topographic survey.
- Comparing surveyed property pins to property line exhibit and adjusting lines as needed to reflect surveyed property pins.

Property and easement requirements for the project will be better defined as design develops. A legal survey will be completed at this time.

2.4 Geotechnical Investigation

A geotechnical investigation was completed by Braun Intertec. A boring plan was developed by Stanley Consultants, advancing 12 borings along the proposed alignment of the dam features. Borings ST-1 and ST-2 were drilled through the north embankment. Borings ST-4 through ST-7 were drilled within the breach limits of the south embankment. Borings ST-8 through ST-10 were drilled through the embankment that remains south of the breach. Borings ST-11 and ST-12 were drilled through the existing powerhouse and spillway bridge deck. All borings were advanced to sufficient depths to allow analysis and evaluation of soil and bedrock foundations for embankment/structural stability and seepage.

Soil samples were collected with split spoon and Shelby tube samplers. Blow counts (N-Values) were recorded by the Braun Intertec drilling crew. Soil samples were classified and tested. Testing on the soil boring samples included moisture content and dry density, Atterberg Limits, unconfined compression testing, and gradations.

Boring ST-11 and ST-12 were advanced through the existing walls and piers of the powerhouse and spillway. Continuous core samples were collected in the concrete and underlying bedrock foundation. Percent recovery and Rock Quality Designation (RQD) were recorded by the Braun Intertec drilling crew for all bedrock core collected. The bedrock core was classified and representative samples tested for unconfined strength.

Borings were also drilled at several properties in the vicinity of the dam to determine the extents and accessibility of loess and till materials for potential use in reconstruction of the earthen portion of the dam. Borings were advanced on the Wilson, Freiburger, and Harbach properties. Soil samples collected from the borings were classified and tested for moisture content, compaction testing, and Atterberg Limits.

Braun Intertec's preliminary geotechnical report describing the geotechnical investigations and presenting results, including boring logs and laboratory test results is included in Appendix C.

2.5 Structural Investigation

A structural inspection/evaluation of Lake Delhi Dam was performed by Stanley Consultants on September 23, 2010 after the July 2010 flood. A copy of the complete report is included in Appendix A.

Additional investigation of concrete structures was completed by Stanley Consultants engineers during their site visit on November 9, 2011. During the investigation concrete deterioration and spalling were observed in many areas of the powerhouse and spillway structures. Concrete on the surface of the spillway and lower portion of the bridge piers was found severely deteriorated. Concrete surrounding the lift gate slots was severely spalled.

As part of the current structural investigation, Stanley Consultants performed a review of the spillway and powerhouse stability analysis completed by Ashton-Barnes Engineering in 1997. In the 1997 Ashton-Barnes report, it was concluded that both the ogee spillway and powerhouse turbine bay section were stable for the normal flow condition (with or without an ice loading).

The dam was not found to have adequate stability for the Probable Maximum Flood (PMF) condition.

In review of the Ashton-Barnes analysis, it was found that:

- Ashton-Barnes categorized Lake Delhi Dam as a low hazard potential and the structures were analyzed accordingly.
- The analysis did not consider rock or silt deposits on the upstream face of the spillway gate structure which applies a lateral driving load on the structure. The Ashton-Barnes report noted but did not analyze the load from significant silt build-up at the upstream face. Currently, a significant amount of riprap stone protection is at the upstream face from a 2009 project.
- The 1997 Ashton-Barnes analysis was based on Federal Energy Regulatory Commission (FERC) criteria.
- Water elevations were based on old hydrology data and hydraulic analysis.
- Bedrock **-** dam concrete bonding strength of 40 pounds per square inch (psi) and a friction angle of 35 degrees were used in the analysis. These were assumed parameters with no bedrock coring verification.

Based upon the field investigations and the results of the Ashton Barnes analyses review, additional investigation and analyses were completed. These included taking the two concrete core borings through the powerhouse and spillway structures to observe the bedrock/concrete interface and to evaluate the condition and strength of the foundation bedrock. In addition, stability analyses were completed of the powerhouse and spillway structures, utilizing recent hydrologic data, dam hazard classification and following the requirements of the FERC and U.S. Army Corps of Engineers (USACE) dam design guidelines.

A concrete coring and testing program was developed to evaluate the subsurface condition of the concrete. Stanley located sites for obtaining concrete cores to evaluate the composition and condition of the concrete below the surface. Braun Intertec completed the concrete coring. Representative cores were selected for unconfined compressive testing and petrographic analyses. Locations of core holes, as well as photographs of the cores and core holes, and laboratory test results are included in Appendix C.

The lower downstream face of the spillway structure and the downstream stilling are not observable due to riprap and silt deposits, respectively. These were found to be in relatively good condition during the 2008 J.F. Brennan Co. underwater inspection; but because the July 2010 flood subjected the dam to conditions that could have undermined the downstream edge of the concrete structures, additional inspection and evaluation will be required during construction when the area is dewatered and riprap and silt removed.

Findings from the 2011 site visit and investigation that were not discussed in the 2010 Stanley Consultants Inspection Report are provided by the following:

- Downstream north abutment wall, upper portion – stone block wall has severely deteriorated.

Downstream North Abutment Wall
Figure 2-1

- Significant seepage through powerhouse concrete slab roof was observed.

Powerhouse Roof Seepage
Figure 2-2

- A hole was observed at the front of the downstream north embankment wall. This indicated that the embankment material behind the retaining wall or foundation for the wall may have been eroded by overtopping flows or piping flows.

Material Loss Downstream of North Embankment Wall
Figure 2-3

2.6 Electrical Investigation

Power Distribution

The Lake Delhi Dam is currently served at 480/277V by three pole-mounted transformers located approximately 150 feet north of the powerhouse structure along 230th Avenue. The transformers and existing service drop to the powerhouse are owned by Alliant Energy. The existing utility meter and service disconnect switch, shown in Figure 2-4 below, are located on the north exterior wall of the powerhouse. The meter and disconnect switch are rated for outdoor environments and appear to be in good condition. The overhead service drop conductors also appear to be in good condition.

Meter and Main Service Disconnect
Figure 2-4

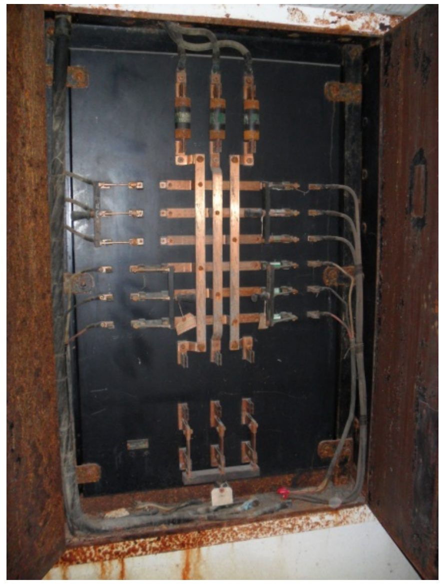
This exterior-mounted disconnect switch feeds an enclosed circuit breaker located in the stairwell of the powerhouse. The enclosed circuit breaker provides service to the main 480-volt distribution panelboard (Panel HP), located in the northwest room in the main floor of the powerhouse. A 30-kVA dry-type transformer steps the voltage down to 208/120-volt distribution for lighting and receptacle circuits in the powerhouse. A 208/120-volt distribution panelboard (Panel LP) is located next to panel HP and the step-down transformer. Figure 2-5 shows panel HP (right), the step-down transformer and panel LP (left). Also shown is an automatic transfer switch (center) not currently connected to the system.

Distribution Panelboards
Figure 2-5

The automatic transfer switch, panelboards, step-down transformer, enclosed circuit breaker, meter and service disconnect switch were installed within the last couple of years to update the electrical service to the dam to 480/277-volts. Prior to the installation of this equipment, the dam was served at 208/120-volts. The panelboards were both installed in NEMA 3R weatherproof enclosures and do not appear to have any water damage due to the flooding.

The 208/120-volt system is still present in the dam, although much of it has been disconnected. The 208/120-volt system was disabled following the 2010 flood event. The service disconnect switch and overcurrent protection, located on a utility pole just north of the powerhouse, is currently in the 'off' position and the service drop cables have been cut.

All of the distribution equipment from the disconnected 208/120-volt system is still present in the powerhouse, including two outdated fuse boxes and a distribution panelboard with no cover plate. One of the outdated fuse boxes is shown in Figure 2-6.



Outdated Fuse Box
Figure 2-6

The lighting and receptacle circuits in the powerhouse are being fed from the new panel LP at 120-volts. Prior to installation of the 480-volt service, these circuits were fed from the distribution panelboard with no cover plate. This panelboard is now being used as a junction box in order to utilize existing wiring in the facility, but to provide power from the new panel LP. The existing distribution panelboard with no cover plate is shown in Figure 2-7.

Distribution Panelboard with No Cover Plate
Figure 2-7

Trash Rake and Hydroelectric Equipment

The trash rake equipment, located on the top of the powerhouse structure, does not appear to have suffered significant water damage during the flooding. The PLC control cabinet does not have any indication of water damage; however, the conveyor equipment was underwater and may be damaged from the flooding. Figure 2-8 shows the existing trash rake system and trash rake PLC control cabinet.

Trash Rake Equipment
Figure 2-8

There does not appear to be any electrical equipment for pool level monitoring. There is an existing control panel, shown in Figure 2-9, labeled as 'Water Level Control Panel', which appears to have been used in conjunction with the hydroelectric generation equipment. There were not any field instruments observed in operation with this control panel.

Water Level Control Panel
Figure 2-9

The existing hydroelectric generation equipment is not operational. The control panels for each hydroelectric unit were inundated with water during the 2010 flood event and significant damage was observed to the battery systems and electronic control devices. Figure 2-10 shows one of the hydroelectric unit control panels. The main PLC cabinet for the hydroelectric system is located on the wall in the generator room and was also damaged by water during the flood event.

Hydroelectric Generator Control Panel
Figure 2-10

Each hydroelectric generator control panel also provides power to the associated wicket gate actuators. Currently, the wicket gates are operated manually, as the control system for the hydroelectric equipment and wicket gates are non-operational.

Lift Gates

The existing lift gates are operated via pushbutton stations located at each lift gate operator. During the 2010 flood event, gate 3 failed to open completely. The existing control equipment has no automatic operation capability and is partially exposed to the elements.

Emergency Generator System

A new automatic transfer switch was installed in the powerhouse when the 480/277-volt service was installed. There was a propane generator located on the site, but it is no longer present.

2.7 Mechanical Investigation

Lift Gates

There are three existing vertical Broome Gates that are original from 1927. During the 2010 flood, one of them did not open completely because the guide in the dam had deteriorated. The cable and drum hoist mechanisms for raising the gates had been recognized as needing replacement. Prior to the flood, the Owner purchased and received new electric motor driven screw actuators for the three existing gates, shown in Figure 2-11. The three existing gates, and gate guides at the dam need to be replaced for the gate system to function adequately.

New Gate Hoisting Mechanism
Figure 2-11

The wicket gates, from the two abandoned hydropower generators, had been maintained to provide minimum flow control. During the flood, the powerhouse was completely inundated and the electronic controls (PLC, battery backup, dc-motor, switches, etc.) were ruined. The wicket gates will require replacement of the existing actuator to bring the system into service. The actual gates and actuator arms will likely need refurbishment as well.

The HydroRake was installed in 2009 to remove the trash that accumulated on the two bar screens protecting the wicket gates. The raking system consists of two bar screens with a hydraulically operated trash raking arm mounted on a traveling chassis that discharges onto belt conveyors that remove the trash to the discharge of the spillway gates. The flood inundated this equipment except for the PLC control cabinet. To make this system operational again, a complete inspection of the equipment is required. At a minimum, the motors would need to be replaced, the local switches and sensors would need to be replaced, the submerged wiring would need to be replaced, and the hydraulic oil would need to be flushed. The belts, bearings, rollers, and hydraulic motor would need to be inspected, oiled, greased, or replaced as necessary. The trash rake is shown on Figure 2-12.

Trash Rake
Figure 2-12

2.8 Document Research

Several sets of historical photos, inspection reports, analyses, and construction drawings were obtained during the course of the reconstruction analysis. Unfortunately a complete set of dam construction drawings was never located. However plan, elevation, and section drawings of the powerhouse, gated spillway, and embankment were located and used to supplement the information obtained by the topographic survey of the dam area.

2.9 Prior Investigations

In addition to the investigations conducted during the alternatives analysis, the following prior investigation documents were reviewed:

- 1997 Ashton-Barnes inspection, stability, and spillway adequacy report.
- 2002 J.F. Brennan Co. underwater inspection report and videos.
- 2004 DNR inspection report.
- 2008 J.F. Brennan Co. underwater inspection report and videos
- 2009 DNR inspection report.
- 2010 Stanley Consultants inspection report.

With no pool, the majority of the dam structure was accessible for inspection during the alternatives analysis phase. However the large riprap installed on the upstream side of the dam and roughly 10 feet of silt deposited in the downstream stilling basin prevented inspection of the upstream spillway structure face and downstream stilling basin. These structures were inspected during the 2008 J.F. Brennan Co. underwater inspection. The underwater inspection found no major issues with the structure and recommended some minor repair of gate piers, the downstream stilling basin wall, and north wing wall of the stilling basin. No evidence of undermining was found at the upstream face of the gated spillway structure. Most of the area upstream of the spillway gates was 12 feet below the spillway crest and covered with riprap size stone. However an area 20 feet below the spillway crest was found near the northernmost gate without revetment stone. This could have been the area near the dam's sluice pipes although the underwater inspection did not locate any pipe intakes on the upstream face. The underwater inspection report recommended filling the deepest portions of the upstream area with riprap due to "scouring."

The 2008 J.F. Brennan Co. and 2011 Stanley Consultants inspection reports have been included in Appendix A.

2.10 Archaeological Reconnaissance Survey

In September 2011, The Louis Berger Group, Inc. (LBG) completed an archaeological reconnaissance survey of the Lake Delhi area. The archaeological studies included a records review to identify potential resources within the former impoundment area followed by a field reconnaissance survey to investigate areas considered to have high potential for unreported archaeological sites. The study area included the Lake Delhi dam and all exposed land areas within the former impoundment area located at or below the former lake elevation level of 897 feet above mean sea level. The study area encompasses an estimated 448 acres.

No archaeological sites had been reported within the project area prior to the July 2010 dam failure. Four sites were recorded within the previous impoundment area during the fall of 2010 by Wapsi Valley Archaeology, Inc. (WVA) during archaeological monitoring for the installation of emergency erosion control structures at the Delhi dam location and upstream at Hartwick bridge.

The four sites included two historic building foundations and two historic artifact scatters associated with the 19th century town site of Hartwick. Prehistoric artifacts with evidence for Early to Middle Archaic, Late Archaic, Middle Woodland, and late prehistoric components were also collected from the four sites.

The LBG study includes a comprehensive records review, a condition assessment of the study area's Quaternary and Holocene valley landforms, and results of a reconnaissance level survey of those landforms.

LBG identified seven additional sites within the study area and redefined one of the sites first identified by WVA to segment one of the historic building foundations at Hartwick as a separate site. As a result, there are a total of 12 archaeological site reported for the study area. These include ten sites with evidence for prehistoric Native American occupations ranging from 8000 to 300 years before present (BP). Most of these sites (7 of 10) appear to be open habitation areas or settlements while one is a smaller habitation site situated within a natural rock shelter. Other prehistoric sites include an apparent fish weir structure and a lithic resource procurement site. Mid-19th century building foundations are represented at two separate locations near the former town site of Hartwick and are believed to be associated with the historic settlement that once existed at that location. One of these is believed to be the Hartwick saw mill which was the first building erected in Hartwick (by John Clark in 1849). Fragments of contemporary historic artifacts were identified at two sites that also produced prehistoric artifacts.

No burial sites were identified within the study area, but potential for unreported human burials is considered possible at the eight prehistoric habitation sites. None of the 12 sites has been evaluated for National Register eligibility. Additional reconnaissance survey is recommended for selected portions of the study area based on the results presented in this report. Additional site investigations are also recommended at all 12 sites as necessary for the purpose of gathering information about the nature, extent, and condition of the archaeological deposits present pursuant to an evaluation of National Register eligibility.

The full Archaeological Reconnaissance Survey Report is included in Appendix E.

2.11 Permitting Requirements

Permitting for the dam reconstruction will be through the USACE/ DNR Joint Permit process. During the detailed design phase an application package will be prepared for submittal to USACE with copies sent simultaneously to both the Floodplain and Sovereign Lands Section at DNR. Included in the submittal will be a separate packet with the forms and information specific to a Dam Construction Permit.

The archaeological survey report will be submitted with the application.

USACE Permit

Conversations with USACE have suggested that the project should qualify for the more streamlined Section 404 Nationwide Permit (NWP). During the design phase, an application will be submitted to the USACE demonstrating that the project meets the conditions for a NWP. USACE review times are typically less than a month. The Section 404 action will trigger the need to obtain Section 401 Water Quality Certification from DNR.

DNR – Section 401

Section 401 Water Quality Certification (aka 401 Cert) specifically addresses the project's potential impacts to water quality that will have to be avoided, minimized, and possibly mitigated.

DNR – Sovereign Lands

A Sovereign Lands Construction Permit will not be required for the project; however, as indicated above the Joint Application process will include a copy of the application to the Sovereign Lands Section for their review. This process will include a review within DNR by threatened & endangered (T&E) species staff and DNR fisheries personnel.

The T&E review will identify any state-listed plant or animal species known from the project area. It will be necessary to assess the likelihood that any of these species will be impacted by the project.

DNR – Floodplain Permit

Construction in a floodplain or floodway always requires a floodplain permit or an evaluation of floodplain issues, but with dam projects it is necessary to complete application forms and provide information specific to dam construction. For this project it will be necessary to obtain a Construction Permit (Floodplain Development Permit). Submittal requirements include:

- Completed and signed Water Storage Permit Application.
- Two sets of certified plans.
- Engineering Design and Hydraulics and Hydrology Report.
- Soil & Foundation investigation report.
- Sedimentation rate assessment.
- Gated low-level outlet design.
- Hazard assessment.
- Summary of Engineering Data.

Cultural Resources

Along with the archaeological survey report, it will be necessary to develop a Programmatic Agreement with the State Historic Preservation Office (SHPO) that will provide a plan for avoiding, minimizing, or mitigating impacts to any significant resources encountered.

U.S. Fish & Wildlife Service (FWS)

FWS will be sent a Public Notice by USACE. FWS will review the project for the potential for the project to impact any federally-listed T&E species. Bald eagles are no longer a listed species but if any potential impacts are identified, application will be made to FWS for a Bald Eagle Permit.

The project area will be reviewed for the potential for federally-listed T&E species to occur in the area. If any potential exists, the Moline, Illinois Field Office of FWS will be contacted during preparation of the application. Any T&E concerns identified by FWS will be addressed in the application and the Moline office will be sent a copy of the application package at the same time it is submitted to the USACE.

Section 3

Engineering Analysis and Preliminary Design

3.1 General

Engineering analyses were completed to establish the requirements for the detailed design of the proposed dam repair and construction concepts. These analyses established engineering parameters that will be utilized in design of repair and construction features, as well as minimum loading conditions for meeting current dam safety and design standards. The engineering design parameters and loading conditions established by engineering analyses were utilized in completion of preliminary design of repair/reconstruction alternatives. Preliminary design established approximate sizing and construction of dam features used in comparative evaluation, preparation of preliminary cost estimates and determination of property/easement acquisition requirements.

Engineering analyses and preliminary design were completed for the following disciplines:

- Geotechnical.
- Structural.
- Hydrology/Hydraulics.

3.2 Geotechnical

Subsurface Investigation

A boring program was established to collect subsurface data necessary to evaluate construction of existing embankments, type and condition of dam foundation materials, and complete analysis of several preliminary design features. Borings were also advanced at several properties near the dam site to evaluate materials for potential use as borrow in earthen embankment construction. The geotechnical investigation was carried out by Braun Intertec. A description of the geotechnical investigation program is provided in Section 2.2.

A copy of the preliminary Braun Intertec Geotechnical Investigation Report is included in Appendix C. The report describes methods used to advance borings and collect and test soil and bedrock samples. The report includes boring logs and laboratory test results.

The results of the geotechnical investigation indicate variable foundation conditions along the dam alignment. A profile sketch of the foundation is included in Appendix C. A description of subsurface conditions encountered is provided below based on existing structure locations:

- **North Embankment** – subsurface consists of up to 28 feet of sand and gravel fill material, underlain by approximately 10 feet of sandy lean clay. The sandy lean clay is underlain by approximately 15 feet of poorly graded sand to approximately elevation 852, where limestone bedrock is encountered.
- **Existing Powerhouse and Spillway** – concrete rests atop limestone bedrock, which is encountered at approximately elevation 848.
- **Dam Breach** – subsurface consists of a varied depth of sand and gravel underlain by limestone bedrock. The bedrock elevation drops off sharply moving from north to south. Bedrock is encountered at elevation 861 at boring location ST-4, elevation 842 at boring location ST-5, and bedrock is not encountered at boring location ST-6 (to elevation 817).
- **South Abutment** – subsurface consists of approximately 20 feet of sandy lean clay fill, underlain by poorly graded sand extending to the limits of the borings at 70 feet in depth. Bedrock was not encountered.

The borings taken at potential borrow sites typically encountered two soil types underlying the topsoil: a silty clay loess overlying a silty clay glacial till soil. The loess soils, while potentially acceptable for embankment construction, were typically encountered at very high moisture contents, requiring excavation and spreading (farming) in order to get the material to an acceptable moisture content for placement and compaction. The till soils typically provide a superior material for embankment construction and have in-situ moisture contents closer to those required for placement and compaction in an earthen embankment. The till soils were encountered at depths of 12 feet or more, under the loess soils, so significant excavation would be required to develop these soils for borrow. Additional future investigations by the Contractor may locate the till soils at shallower depths for borrow development. Both materials indicate acceptable strength and seepage properties for use in earthen dams.

Embankment Seepage Analysis

Seepage analysis was conducted for proposed embankment and seepage control measures using GeoStudio's SEEP/W finite element seepage modeling program. Soil classification and laboratory gradation results were used to develop input seepage parameters. Permeability coefficients were determined according to Hazen's empirical formula using D_{10} values (particle diameter corresponding to 10% passing). The proposed service and auxiliary embankments (located within the current breach) were modeled with various cutoff depths and configurations. Horizontal blanket drains were also included in the model, for safe collection and conveyance of seepage flows, without saturating the downstream slope of the embankments. Exit gradients (exit gradient is defined as the rate of change of total head pressure with distance) and seepage flow rates were analyzed to come up with an optimized and adequate cutoff/drainage system. To achieve the target factor of safety of 1.5, the target exit gradient was assumed as 0.67. This assumes a critical gradient of the material of 1.0. To achieve the target gradient, the sheet pile cutoff was designed as 35 feet below base of the new embankment (into sand foundation). The existing south embankment was also analyzed for seepage, to determine required depth of seepage cut-off beneath this shorter embankment section. Due to uncertainties with the condition and construction of the existing cut-off and embankment, it is conservatively omitted from the analysis. For the existing south embankment, the analyses indicate a 40-foot cut-off is required (20 ft through existing clay embankment and 20 ft into sand foundation).

Soil boring ST-4 encountered poor rock quality and large voids near the north end of the new proposed spillway embankment. To provide a positive seepage cutoff between the new embankment sheet pile cutoff and the steep bedrock slope, a grouting program will be required at this location.

Embankment Stability Analysis

Stability analyses were completed to determine required slopes and footprint of the proposed embankment alternatives so that current dam safety design guidelines are met. Stability analysis was carried out using GeoStudio's SLOPE/W (2007) modeling program. Spencer's Method was used to find minimum factors of safety for various loading conditions. The analyses were completed in general conformance with the requirements for new earth and rock-fill dams presented in the USACE Slope Stability Engineering Manual (EM 1110-2-1920). Table 3-1 summarizes load conditions and required factors of safety.

Table 3-1 Slope Stability Requirements

Load Condition	Required FOS
Total Stress	1.3
Effective Stress	1.5

A maximum surcharge pool was assumed with water to the top of the proposed spillway crest. To account for the decreased water surcharge loading as a result of the labyrinth weir, 50% of the water surcharge load was considered along the width of the new spillway. A rapid drawdown condition was not modeled at this stage in the design because it is unlikely that the pool will ever be rapidly drained.

For proposed new embankment sections, slope stability was analyzed for embankments constructed of locally available borrow materials (identified in Braun Intertec investigation) as well as roller compacted concrete (RCC). It was determined that 3 horizontal on 1 vertical slopes are required for the both the upstream and the downstream faces of embankments constructed of loess or till in order to satisfy all design requirements. Roller compacted concrete faced embankments meet design requirements if constructed with 2.5 horizontal on 1 vertical downstream slopes.

Settlement Analysis

Long-term consolidation settlement is not anticipated as embankment construction will take place on subsurface sands. Given the sand foundation material, a majority of settlement will occur as construction proceeds. Settlement within the embankment fill will be limited by proper placement, moisture control, and compaction of embankment fill.

3.3 Structural

Existing Spillway and Powerhouse Stability

A review of the 1997 Ashton-Barnes stability analysis of the dam indicated several deficiencies when compared to current dam safety design requirements (Discussed in Section 2). As a result, Stanley Consultants conducted a new stability analysis of the ogee spillway and powerhouse structures for sliding and overturning stability in general conformance with the requirements of the USACE *Design of Gravity Dams* (EM 1110-2-2200) and the FERC *Engineering Guidelines for the Evaluation of Hydropower Projects*, Chapter 3. The analyses evaluated the stability of the concrete structures as constructed and investigated options for anchoring the structures to the foundation bedrock in order to meet current dam design criteria. These criteria included:

1. Assuming the dam is a moderate hazard classification (see discussion in Section 3.3) and using applicable FERC structural criteria and design flood headwater and tailwater conditions.
2. The existing ogee spillway and powerhouse structures were checked against both USACE and FERC stability criteria. Rock anchor alternatives were designed to meet either USACE or FERC requirements. In the analysis, it was found that, for these two structures, FERC requirements were more stringent than the USACE. The reason for designing anchors to FERC standards is that, if hydropower generation at the dam is ever rehabilitated, there would be cost savings in adding the additional anchors at this time, versus adding at a later date.
3. Headwater and tailwater elevations reflect the latest hydrology and hydraulic modeling results.
4. A new geotechnical investigation was conducted and foundation parameters were based on new test information and research on similar bedrock founded projects.

Concrete core borings were advanced by Braun Intertec through a powerhouse wall and a central pier of the ogee spillway for the purpose of evaluating bedrock conditions underlying the two structures as well as the potential for concrete to bedrock bond at this interface. The results of the core borings and laboratory testing indicate that the bedrock is of sufficient quality to support the structures and to develop required capacity of future rock anchors. The core borings also indicated a clean interface between concrete and bedrock and that some bonding of concrete to bedrock exists at this interface.

Table 3-2 shows the criteria used in the sliding stability analysis for the existing spillway and powerhouse structures.

Table 3-2 Stability Parameters

Load Condition	USACE Minimum Sliding FOS	FERC Minimum Sliding FOS (Cohesion not Used)
Usual	2.0	1.5
Unusual	1.7	1.5
Extreme/Post Earthquake	1.3	1.5

To satisfy USACE's sliding factors of safety, ten (10) rock anchors are required to stabilize the existing ogee spillway structure, and ten (10) anchors are need for the powerhouse. In order to meet FERC's criteria, thirty (30) would be required for the spillway and twenty (20) for the powerhouse.

Detailed parameters and assumptions used in the analysis are presented in Appendix D.

Repair of Existing Structures

General Concrete Condition

During Stanley Consultants' September 2010 inspection and later site visits, concrete deterioration and spalling were observed in many areas of the powerhouse and spillway structures. Concrete on the surface of the spillway and lower portion of the bridge piers was found severely deteriorated. Concrete around the lift gate slots spalled to prevent a gate to open. Based on the surficial observations, a concrete coring and testing program was developed to evaluate the subsurface condition of the concrete. The coring information and test results are provided in Appendix C.

Based on these results, it is reasonable to assume that the concrete below the spillway and pier/wall surface has acceptable strength. By removing and replacing the deteriorated surface no further structural evaluation of the concrete should be required. If the condition of concrete found during construction differs significantly from the concrete coring and testing results, a further evaluation of the structure will be conducted.

Powerhouse

Stanley Consultants analyzed and designed modifications to the powerhouse assuming the structure acted as one monolith so the powerhouse essentially functions as a water retaining structure. It was assumed that structural upgrades needed for hydropower generation would be completed at a later stage should the facility be restored for generating electricity. Therefore, in this phase of the project, in addition to anchoring the structure to bedrock foundation to meet USACE and/or FERC stability requirements, structural repair work was limited to the portions that were deemed necessary for the powerhouse and spillway to function as a water retaining structure.

During the site visit on November 9, 2010, significant evidence of seepage through the powerhouse roof was observed. There was also evidence of potential corrosion of the reinforcing steel by staining observed along cracks in the ceiling. It is Stanley Consultants' understanding that the County (with assistance from Iowa State University) completed load testing of the bridge deck in response to observed conditions and results of the testing revealed no structural deficiencies. Waterproofing of the roof slab is proposed to minimize further infiltration and degradation. Subsequent use of the bridge deck will be limited to construction and maintenance equipment. However, given the large size and weight of construction vehicles and loads, the bridge will be further analyzed during detailed design when equipment and material weights are known to determine if vehicle restrictions will be required during construction. To minimize further degradation of the bridge deck reinforcement the use of de-icing solutions should be discontinued.

North Downstream Abutment Wall

The masonry block portion of the north downstream abutment wall was observed to be in poor condition. The concrete wall that the blocks were founded on appeared to be in satisfactory condition. Reconstruction of the stone block retaining wall portion of this wall is proposed.

Spillway

The existing ogee spillway will be anchored to bedrock foundation to meet USACE and/or FERC stability requirements. Deteriorated concrete on the ogee sections and piers will be removed and replaced with new concrete. All gate slots will be repaired or replaced to accommodate the new gate system proposed. The bridge structure over the spillway section is relatively new, no significant deterioration was observed during inspections, and the bridge will no longer be utilized for public access, therefore, no significant structural repair work was proposed for the bridge.

Damaged or deteriorated concrete at the spillway training walls and stilling basin will be repaired. The remnants of the existing fish ladder at ogee spillway south training wall will be removed. The training wall will be repaired and modified to accommodate the new spillway structure to be located on the south side of the wall.

Construction of New Spillways

New spillway alternatives were designed to pass 100-year design flood, and have an overall capacity to pass ½ PMF flood.

Construction of new structures, including spillway weir, spillway slab, stilling basin, retaining/training walls, will meet both USACE and FERC requirements for stability and structural strength.

The conceptual structural designs were based upon the following:

Cast-in-Place Concrete Design:

- Conform to “Building Code Requirements for Reinforced Concrete,” ACI 318-08.
- f_c = 4,000 psi for all structural concrete.
- Reinforcing Steel: ASTM A615/A615M, Grade 60.

Structural Steel Design:

- Conform to latest edition of AISC "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings."
- F_y = 36 ksi (yield point), based on using steel conforming to ASTM A36.
- F_y = 50 ksi (yield point), based on using steel conforming to ASTM A572 for sheet-pile.

Loads:

- All loads per ASCE 7-05 and IBC-2006.

Materials and Construction:

- Concrete:
  - Specify that average 28-day compressive strength shall exceed f_c on basis of standard deviation, in accordance with ACI procedures.
  - Use air-entrained concrete for all structures.
  - Allow fly ash in all concrete.
  - Use ASTM C150, Type I cement for all concrete.
- Reinforcing Steel:
  - Deformed billet steel, ASTM A615, Grade 60.
  - Wire fabric, ASTM A185.
- Structural metals:
  - Grade: ASTM A36.
  - ASTM A572, grade 50 for sheet-pile.
  - Protect ferrous metals from corrosion.

Design criteria and parameters used in design are included in Appendix D.

Seismic analysis for the structures is not necessary, since the dam is located in a low seismic zone. S_s = 0.086, S_1 = 0.046.

3.4 Hydrology/Hydraulics

Lake Delhi Dam in its pre-failure condition did not have sufficient hydraulic capacity to pass the new project design flood for a Moderate Hazard Dam. Design of the reconstruction will include significantly increasing Lake Delhi Dam's hydraulic capacity for passing flood flows.

For the alternatives analysis several concepts were developed for reconstructing the dam's spillway(s). Three concepts were taken to preliminary design and evaluated for potential design and construction. Other hydraulic considerations included minimum/low flow passage, lake draining capacity, and cofferdam/bypass during construction. Steps to complete the hydrologic and hydraulic studies for the alternatives analysis included:

- Characterizing Maquoketa River Flows at Lake Delhi Dam.
- Developing a hydrologic model of Lake Delhi Dam watershed.
- Developing a hydraulic model of Maquoketa River upstream and downstream of Lake Delhi Dam.
- Performing hazard classification and design flood analysis for Lake Delhi Dam.
- Developing Lake Delhi Dam spillway concepts.
- Addressing other hydraulic issues.

Maquoketa River Flows

U.S. Geological Survey (USGS) maintains a river flow gage near Manchester, Iowa at the Highway 20 crossing roughly 12 miles upstream of Lake Delhi Dam. USGS performed a frequency analysis of gage flows which were adjusted by USGS regional drainage area ratio methodology to estimate return period flows at Lake Delhi Dam. Results are provided in Table 3-1.

Table 3-3 Return Period Flows

Return Period (yrs)	Annual Exceedance Probability	USGS Gage Flow (cfs)	Lake Delhi Dam Flow (cfs)
1	0.95	1,393	1,491
2	0.5	4,506	4,821
5	0.2	8,636	9,241
10	0.1	12,300	13,161
25	0.04	18,130	19,399
50	0.02	23,420	25,059
100	0.01	29,610	31,683
200	0.005	36,820	39,397
500	0.002	48,150	51,521

Average daily flows at Lake Delhi Dam are in the range of 150 cubic feet per second (cfs).

Hydrologic Model

A HEC-HMS hydrologic model was used to develop a series of design flood hydrographs (i.e. analysis derived) for the Lake Delhi Dam watershed. The flood hydrographs were used as an input for the hydraulic model.

The probable maximum flood (PMF) hydrograph was developed using ArcGIS to establish watershed parameters and NOAA's HMR 51/52 publication to establish rainfall depth-durations. The full and ½ PMF were used in the analysis. The 100-year flood hydrograph was developed using the same ArcGIS watershed parameters and the 100-year/24-hour rainfall was obtained from *Iowa Rainfall Frequencies*.

The peak HEC-HMS derived 100-year flow was checked against the peak 100-year flow established at the USGS streamflow gage at Manchester and the two flows matched closely. Watershed parameters, rainfall depths, and peak flood flows are provided in Table 3-2.

Table 3-4 Lake Delhi Dam Watershed Parameters

Parameter	Value
Drainage Area (mi^2)	349
Infiltration (in/hr)	0.25
Time of Concentration (hrs)	18
Storage Coefficient (hrs)	15
PMF Rainfall Total (in)	25.8
PMF Peak Flow (cfs)	143,900
100-Year Rainfall Total (in)	6.4
100-Year Peak Flow (cfs)	28,100

Hydraulic Model

The starting point for the hydraulic modeling was the HEC-RAS model of the Maquoketa River developed by the DNR to evaluate the 2010 breach of Lake Delhi Dam. The upstream end of the river model is at the Highway 20 Bridge and the model extends approximately 23 miles to just downstream of Hopkinton.

The HEC-RAS model as well as supporting background data was provided to Stanley Consultants by the DNR. The following adjustments were made to the DNR HEC-RAS model:

- River channel topography was updated with post-breach LiDAR data obtained in 2010.
- Bridge structures were added downstream of the dam (Quarter Road., 295th Street and Hopkinton).
- One inflow hydrograph was used at the upstream end of the model (DNR model used two).
- The dam was modified to reflect the proposed condition (working gates, principal/auxiliary spillway.

Hazard Classification

Hydrologic and Hydraulic analysis and design standards for dams in Iowa are specified in *Technical Bulletin 16 - Design Criteria and Guidelines for Iowa Dams*. The standards are defined according to the dam's hazard classification. The state of Iowa has three hazard classifications for dams; Low, Moderate, and High Hazard.

If hydropower is ever redeveloped at Lake Delhi Dam, the reconstructed dam will have to meet FERC criteria. FERC also has three hazard classifications; Low, Significant, and High

Hazard. The FERC and DNR hazard classification definitions are very similar so the classification determined by DNR criteria should correspond to a FERC hazard classification. Table 3-3 provides the agency hazard classification definitions.

Table 3-5 Hazard Classification Definitions

Hazard Class	DNR Definition	FERC Definition
Low	Structures located in areas where damages from a failure would be limited to loss of the dam, loss of livestock, damages to farm outbuildings, agricultural lands, and lesser-used roads, and where loss of human life is considered unlikely.	Structures located in rural or agricultural areas where failure may damage farm buildings, limited agricultural land, or township and country roads. Low hazard potential dams have a small storage capacity, the release of which would be confined to the river channel in the event of a failure and therefore would represent no danger to human life.
Moderate/ Significant	Structures located in areas where failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life.	Structures located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life.
High	Structures located in areas where failure may create a serious threat of loss of human life or result in serious damage to residential, industrial or commercial areas, important public utilities, public buildings, or major transportation facilities.	Structures located where failure may cause serious damage to homes, agricultural, industrial and commercial facilities, important public utilities, main highways, or railroads, and there would be danger to human life.

The hazard classification of Lake Delhi Dam controls several design parameters including the freeboard design flood. For detailed design to proceed, a hazard classification is needed to establish the applicable dam safety and design criteria.

Previous inspections and analyses have identified Lake Delhi Dam as a low, moderate, and high hazard structure, but there has not been a detailed analysis of potential downstream hazard to substantiate the hazard classification. The hazard classification analysis performed for this project provides a more thorough evaluation of risk associated with theoretical dam failure through inundation mapping of a series of flood events with and without dam failure.

The full PMF, ½ PMF, and 100-year flood were modeled in HEC-RAS with and without dam failure (breach). The DNR established dam breach parameters for their original HEC-RAS model based upon the Lake Delhi Dam failure observed in 2010. For the reconstructed dam analysis, the width of the dam breach was reduced from 250 feet to 175 feet to better reflect the reconstructed condition. The breach formation time was left at 1.5 hours. The failure was set to initiate at the peak of the flood hydrograph which yields the highest flood elevation (i.e. worst-case condition).

Failure of the existing powerhouse and gated spillways were also evaluated but the embankment failure provided the most critical dam failure scenario.

Design of the Lake Delhi Dam reconstruction is in the preliminary stage, so the “reconstructed” dam in the HEC-RAS model represents an approximation. Gates will be replaced as part of the reconstruction so they were assumed to be fully operable in the HEC-RAS model with the same opening area as the existing condition.

A representative principal/auxiliary spillway was added to the HEC-RAS model. Hazard classification is focused more on the downstream impact of the dam than the specifics of the spillway so using a principal/auxiliary spillway approximation is reasonable for this analysis. The various spillway options currently being considered have a similar embankment shape so the proposed HEC-RAS model should provide an adequate depiction of the failure condition no matter which alternative is chosen. However, the analysis will be updated once the reconstruction design is established, but a significant change in results is not expected.

The HEC-RAS flood profiles were exported to ArcGIS using HEC-GeoRAS, which uses the profiles to develop inundation extents for each flood/failure event. Inundation maps were created that include geo-referenced aerial imagery so the inundation limits can be viewed relative to downstream buildings and infrastructure. Detailed inundation maps and tables are provided in Appendix B. Results are summarized in Table 3-4.

Table 3-6 Impacted Structure Summary

Event	Scenario	Residential	Comm/Ag	Bridges	Roads
PMF	No Breach	104	30	3	12
	Breach	107	30	3	12
Half PMF	No Breach	27	8	3	8
	Breach	29	8	3	8
100-YR	No Breach	3	1	1	5
	Breach	5	2	1	5
Sunny Day	Breach	0	0	0	1

Hazard classification is based on the potential consequence of dam failure. When analyzing the consequences of dam failure during a flood event it is the increase in consequence (i.e. increase in damage and potential loss of life) due to failure that is evaluated.

Results of the HEC-RAS modeling and inundation mapping indicate that dam failure during flood events does not appear to cause a significant increase in the number of structures inundated. The majority of additional structures that are inundated by a failure event are the homes within 1500 feet downstream of the dam. As the Emergency Action Plan is developed for the reconstructed condition it will be important to have well-defined communication and evacuation procedures defined for these residents.

Hazard classification was reviewed for both the DNR and FERC definitions. Lake Delhi Dam appears to fit the Moderate (DNR), Significant (FERC) Hazard Classification. The reasoning is as follows:

- HEC-RAS modeling and inundation mapping show that a potential failure during a flood would only cause a small increase in the number of structures impacted.
- A potential sunny day failure conditions stays within the limits of the 100-year floodplain (typically non-developed area) so the potential for damage is less than if sunny day failure flooded more habitable or developable lands.
- Much of the area downstream of Lake Delhi Dam is rural and agricultural. Although future development is possible, most development would likely occur closer to the town of Delhi, which is up above the river channel or in Hopkinton which is far enough downstream that the increase in flood elevation due to failure is roughly 1 foot.
- The Maquoketa River downstream of Lake Delhi Dam is widely used for canoeing and fishing activities, however the river does not contain the type of attractions that bring large numbers of people into the river channel area for extended periods of time (i.e. restaurants, resorts, large campgrounds or trailer parks, etc.)
- Therefore, the DNR definition of Moderate hazard where "…failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life." is appropriate for the reconstructed Lake Delhi Dam.
- The FERC definition of Significant hazard for "Structures located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways, or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life." also seems the appropriate classification for Lake Delhi Dam.

Design Flood

Per *Technical Bulletin 16 - Design Criteria and Guidelines for Iowa Dams*, a moderate hazard classification establishes the freeboard design flood as the ½ PMF. FERC uses an incremental analysis to establish the design flood by determining the largest food where failure causes an increase in downstream hazard. An incremental analysis was performed and using the DNR designated ½ PMF as the freeboard design flood should also meet FERC criteria.

Recommendation for Final Design

Based on the analysis Stanley Consultants recommends that design of the Lake Delhi Dam reconstruction proceed with a classification as a Moderate Hazard structure and a freeboard design flood of the ½ PMF. This classification will be verified with an updated analysis once reconstruction design has been established.

A detailed hydrologic and hydraulic studies report, computations are included in Appendix B.

Spillway Concepts

Using the ½ PMF as the design flood, spillway concepts were developed with the objective of the reconstructed Lake Delhi Dam being able to pass the ½ PMF without overtopping the existing powerhouse/gated spillway structure.

Prior to the breach, flood flows were passed by opening the three spillway gates located adjacent to the powerhouse. The gate system has a hydraulic capacity of roughly 30,000 cfs with the gates fully raised and the upstream pool at the top of dam. Reconstructed Lake Delhi Dam will need to pass roughly 69,000 cfs which is more than double the hydraulic capacity of the pre-breach dam.

The new spillway system at Lake Delhi Dam will need to provide roughly 39,000 cfs of additional hydraulic capacity. In performing preliminary design of the spillway alternatives, stage-discharge curves were developed for each spillway alternative.

For the labyrinth weir spillway alternatives a set of empirical equations was used to develop stage-discharge curves. Labyrinth weir hydraulics has been studied in detail so it is possible to predict the discharge rating for a given geometry with reasonable accuracy. *Hydraulic Design of Labyrinth Weirs* was utilized for developing the geometry and estimating the discharge capacity of the labyrinth weir alternatives.

The pneumatic gate spillway alternative essentially acts as a sharp crested weir with an adjustable crest. When flows are low, the crest is kept at or near the normal pool and as flows increased the gate panels are lowered until they are flush with the fixed concrete slab/crest they are mounted to. For preliminary design, the controlling factor is passage of the design flood, so gates were assumed to be down with the weir crest elevation essentially at the fixed concrete slab/crest and stage-discharge curves were computed.

A description of the alternatives analysis is provided in Section 4. A detailed description of hydraulic design and analysis is provided in Appendix B.

Minimum/Low Flow Passage

Minimum/low flow passage was a topic of concern with operation of the pre-breach Lake Delhi Dam. During times of normal and low flows, flow downstream of the dam was controlled by wicket gate discharge. Wicket gate settings and pool elevations were recorded but discharge rates were not quantified. During times of low flow there were concerns that insufficient discharge was being provided to the downstream waterway.

An additional concern was dissolved oxygen levels of the discharge. The wicket gates intake elevation is at 881.3, roughly 15 feet below the normal pool elevation where dissolved oxygen levels are typically low. Discharge through the gates was not aerated so waters in immediate downstream channel frequently did not meet dissolved oxygen requirements.

If the wicket gates are restored as the normal means of discharge, an aeration mechanism will be incorporated into the system. If the labyrinth or pneumatic gates are used as the single principal spillway sufficient aeration will be provided by the pool level discharge and flow down the spillway chute.

In addition to the spillway alternatives, installation of valved openings in two of the new lift gates is being considered. During normal operating conditions the valves would be closed. However the valves could be used to:

- Provide additional discharge capacity prior to gates lifting (roughly 150 cfs for two 30-inch valves at normal pool).
- Provide minimum flow passage if the upstream pool drops below the principal spillway crest.
- Provide bypass flow during potential maintenance work or debris removal at the principal spillway without lifting gates.
- Provide the capability to draw down the pool a small amount or maintain a slightly drawn down pool during low flows. The lift gates are good for passing large flows but not for normal bypass flows or drawing down the pool a few inches.

Unlike the wicket gates, the valves will discharge onto the concrete ogee spillway, so even though the valves would likely be 10 feet below the normal pool, discharge would be aerated by the drop over the concrete spillway.

The previous dam operator indicated the 7Q10 flow (lowest seven-day average flow that occurs once every 10 years) for the Maquoketa River at Lake Delhi Dam is roughly 28 cfs. The 30-inch valves would have the capacity to discharge the 7Q10 flow.

As reconstruction design progresses a detailed operating manual will be developed with DNR input and approval that provides operating protocol and discharge rates for the expected range of flow conditions.

Lake Draining Capacity

DNR requires that “A gated low level outlet shall be provided which is capable of draining at least 50 percent of the permanent storage behind the dam within a reasonable length of time.” The existing lift gates provide sufficient capacity to drain 50 percent of the volume below the normal pool elevation. In addition, existing plans indicate a set of two 37.5-inch diameter sluice pipes were installed through the northernmost spillway pier approximately 20 feet below the crest of the gated spillway.

If they do exist, the sluice pipe intakes are buried under 20 feet of riprap. This riprap will be removed during the dam reconstruction and the feasibility of restoring the existing sluice pipes will be assessed. The sluice pipes are not necessary to meet DNR design requirements but could be useful during construction and for future maintenance and dredging projects.

Section 4

Reconstruction Alternatives Development/Evaluation

4.1 General

The reconstruction project required to restore the Lake Delhi Pool and to bring the facility into compliance with current dam safety and design criteria will require repair work on all of the existing project features described in Section 1. In addition, construction or installation of new features will be required to enhance the safety and performance of the facility. Some of the features are limited to a single option, with no cost effective or practical alternatives available. These features/repairs are called Reconstruction Non-Alternatives and are discussed in Section 5. Other features have one or more alternatives that have enough merit to warrant a preliminary design and cost evaluation to determine the optimum alternative that best meets the District's project objectives. These features are described in this section.

On November 9, 2011, a multi-disciplined team of Stanley Consultants engineers completed a site visit to collect additional data on the equipment and construction of the existing project features, as well as their current condition. Members of the team represented the Civil, Hydrology/Hydraulics, Geotechnical, Structural, Electrical, and Mechanical engineering disciplines. Following the site visit, the Stanley Consultants team met with the District Trustees for an Alternatives Development "Brain Storming" session. The purpose of the session was to:

- Establish District Objectives.
- Review parameters for design development and alternative evaluation.
- Initiate the creative "brain storming" process for alternative development.

The District's Project Objectives were used as the criteria for alternative development and evaluation. Objectives for the reconstruction project include the following:

- Meet requirements of current dam safety and design standards.
- Minimize operation/maintenance requirements.
- Maintain or improve upstream and downstream flow conditions.
- Provide adequate (50+ year) service life.
- Increase public safety at dam site.
- Improve public recreational opportunities.
- Reduce potential for damage from debris flow.
- Provide cost-effective solution.
- Constructability.
- Minimize right-of-way impacts.
- Minimize permitting requirements.
- Provide opportunity for greater pool control.
- Enhance fisheries opportunities.
- Improve water quality.

The Alternatives Kick-Off Meeting provided the Stanley Consultants design team with an understanding of the District's objectives for the reconstruction and performance of the project. Working with the District, Stanley Consultants developed a list of potential alternatives for the repair of existing features and construction of new features. During the alternative concept design and evaluation process, it became apparent that some alternatives were unsuitable due to excessive cost and/or failure to sufficiently meet one or more District's Objectives. Conceptual design and cost estimating was not completed for unsuitable alternatives. The evaluation process for each set of alternatives is described in this section.

4.2 North Embankment

The north embankment consists of the portion of the dam extending north of the powerhouse structure and tying into the north river bank. The existing upstream and downstream walls are showing signs of deterioration, damage, cracking, etc. and will be removed. Available drawings indicate that there is a third concrete wall located within the embankment that was likely the upstream wall prior to the construction of the crib wall and widening of the approach to the dam bridge. This wall will also be removed as part of preparation for the new structure. Site configuration and right of way limits at this location eliminate construction of a full earthen embankment as an alternative at this location. Three separate structural alternatives were considered for reconstruction of the north embankment:

- Double Sheet Pile Wall.
- Cellular Sheet Pile Structure.
- Reinforced Concrete Walls.

Each alternative includes an upstream sheet pile seepage cutoff driven to bedrock. Each alternative also maintains an approximately 25-foot wide roadway atop the embankment. The roadway section will require installation of a guardrail or parapet wall to contain traffic.

The double sheet pile wall alternative consists of two rows of Z-section sheet pile wall driven parallel to one another and tied together with anchor rod. The upstream wall sheet pile will be driven to bedrock to serve as a seepage cutoff. Advantages of the double sheet pile wall include: low-cost alternative, basic construction methods and no temperature restrictions for sheet pile installation. Disadvantages include a non-aesthetic wall face and anchor ties below the upstream water surface would be difficult to inspect.

The cellular sheet pile structure alternative consists of PS-section sheet pile driven in circular "cell" configurations with connector arcs. The upstream sheet pile will be driven to bedrock to serve as a seepage cutoff. Advantages of the cellular sheet pile structure include: single construction operation and no temperature restrictions for sheet pile installation. Disadvantages include a non-aesthetic wall face, high cost of steel sheet pile, and templates required for construction.

The reinforced concrete walls alternative consists of a U-shaped concrete walls/footing structure with anchor ties between wall stems. In addition to the reinforced concrete, an upstream sheet pile will be driven to bedrock to serve as a seepage cutoff. Advantages of the reinforced concrete structure include: options for an aesthetic wall face. Disadvantages include multiple construction operations and temperature restrictions for concrete work.

Given the cost comparison and aesthetics potential, the recommended alternative for the north embankment is the reinforced concrete wall.

Conceptual drawings of the North Embankment Alternatives are provided in Exhibits 2-4 in Appendix F.

(Note: All costs shown in the comparison tables in this section have been adjusted to include markups for contingency and inflation.)

Table 4-1 North Embankment Alternative Cost Comparison

Alternative	Cost
Double Sheet Pile Wall	$469,000
Cellular Sheet Pile Structure	$675,000
Reinforced Concrete Walls	$536,000

4.3 North Downstream Abutment Wall

The North Downstream Abutment Wall extends downstream from the Powerhouse Structure. The base and lower portion of the wall is reinforced concrete and the upper portion is masonry block. The existing masonry block portion of the wall is showing signs of deterioration/damage and replacement is recommended. The following alternatives were considered:

- Leave the existing wall as-is.
- Remove and replace the masonry block portion of the wall with large block or mechanically stabilized earth (MSE) wall.
- Remove and replace the masonry block portion of the wall with a reinforced concrete wall.

Leaving the existing wall in place is not recommended because the existing masonry block wall is showing signs of instability and severe cracking. Even though the masonry block wall's failure would not likely compromise the stability of the concrete portion of the wall or the powerhouse structure its failure could impact access to the downstream entrance to the powerhouse.

The reinforced concrete wall alternative would consist of removing the existing block wall and reconstructing a reinforced concrete wall. The advantages of this alternative include matching construction of adjacent concrete walls and the use of in-place fill soils. Disadvantages include the required excavation to frost depth and temperature restrictions of placing concrete.

The MSE wall alternative would consist of removing the existing wall and reconstructing a modular block wall in its place. The advantages of this alternative include replication of previous construction, aesthetic wall face, less temperature restriction during construction, and the block wall is better-suited to endure any freeze/thaw movement or settlement which may occur. Disadvantages include a requirement for engineered fill which may need to be imported.

Reconstruction of the masonry block portion of the wall would also include removal of the elevated concrete slab at downstream face of the powerhouse and filling of the void. The new wall would extend to the face of the downstream powerhouse wall.

Given the cost comparison and overall aesthetics, the recommended alternative for the north downstream abutment wall is the MSE wall.

Table 4-2 North Downstream Abutment Wall Alternative Cost Comparison

Alternative	Cost
MSE Wall	$106,000
Reinforced Concrete Wall	$181,000

4.4 Powerhouse

Several rehabilitations, replacements, and improvements to the powerhouse structure are being recommended as part of the dam reconstruction project. The major work item is enhancing powerhouse stability to meet current dam safety and design standards. For the alternatives study, both FERC and USACE stability standards were evaluated. The major differences between the two agencies' stability standards include:

- For the sliding stability factor of safety, generally FERC requirements are more stringent than USACE, if the same parameters are used in analysis, i.e. both cohesive bond and sliding friction assumed at the dam foundation interface.

- When cohesive bond at the dam foundation interface cannot be verified by borings or tests, FERC recommends an alternative minimum factor of safety be used in conjunction with a no cohesion assumption. A minimum factor of safety of 1.5 is required for all static load cases.
- For dam stability regarding overturning and foundation bearing pressure, FERC criteria closely resemble the criteria used by USACE.

Table 4-3 Stability Parameters

Load Condition	USACE Minimum Sliding FOS	FERC Minimum Sliding FOS
Usual	2.0	3.0
Unusual	1.7	2.0
Extreme/Post Earthquake	1.3	1.3

To meet the dam design standards of either agency, the stability of the existing powerhouse must be improved to satisfy the structure sliding factor of safety under the design flood condition. The proposed method to increase the stability of the powerhouse structure is installation of post-tensioned rock anchors through the base of the powerhouse and into the underlying bedrock.

Two alternatives were developed: one to satisfy USACE's dam safety criteria, the other is to meet FERC's dam safety requirements. FERC standards would be required should hydropower ever be redeveloped at the dam. USACE standards are considered sufficient for non-hydro generating dams. Installation of the rock anchors during the reconstruction project would be significantly less expensive than mobilizing a contractor to install additional anchors at a later date. The powerhouse structure will need approximately ten (10) rock anchors in order to meet USACE stability requirements. These anchors would be located at the upstream face of the powerhouse. Excavation to bedrock (including removal of upstream riprap) will be required for installation of the anchors. A concrete "bench" would be constructed upstream of the powerhouse and doweled into the powerhouse to provide a location for rock anchor installation.

Meeting FERC criteria will require installation of twenty (20) rock anchors. These anchors will require higher capacity, due to increased load requirements and limited accessibility for installation. Ten (10) anchors will be installed at the upstream face of the powerhouse, and the other ten (10) will be installed through the solid concrete walls of the powerhouse.

The recommended alternative for the powerhouse is to anchor the structure to meet USACE dam design standards. With the future of hydropower development being uncertain and given the significant additional cost, meeting FERC criteria is not recommended.

Conceptual sections of the Powerhouse stabilization are provided in Exhibit 8 in Appendix F.

There are two options for waterproofing the powerhouse roof bridge deck. One alternative would be to clean the deck and epoxy seal any visible cracks in the concrete. The second alternative involves installation of a waterproofing membrane system with asphaltic concrete deck overlay.

The membrane system is recommended for the powerhouse roof rehabilitation since it will provide longer-lasting, more extensive water protection.

Table 4-4 Powerhouse Alternative Cost Comparison

Stabilization Alternatives	Cost
USACE Criteria	$287,000
FERC Criteria	$639,000

Waterproof Alternatives	Cost
Clean Deck/Epoxy Seal	$21,000
Waterproof Membrane System	$37,000

4.5 Existing Spillway

Rehabilitation of the existing gated ogee spillway includes these major items:

- Anchoring the dam structure to the bedrock foundation in order to meet current dam design standards – either USACE or FERC.
- Lift gate repair or replacement.

Similar to the powerhouse, there are two options for increasing the stability of the existing spillway.

To anchor the spillway to meet USACE criteria: approximately ten (10) rock anchors are required. The anchors would be installed either in front of spillway upstream face, or through the spillway crest. The first option would require excavation at upstream face of the spillway to the base of the dam and a new concrete bench doweled into the existing structure. This option would provide relatively easy access for construction. The second option would require drilling anchor holes through existing concrete approximately 30 feet thick, and accessibility for construction would be more difficult.

To anchor the spillway to meet FERC criteria, approximately thirty (30) rock anchors would be required. Twenty (20) of the anchors would be installed as described for the two options above. USACE and an additional ten (10) anchors would be located in the spillway piers.

The recommended alternative for the spillway stabilization is to anchor the structure to meet USACE dam design standards. With the future of hydropower development being uncertain and given the significant additional cost, meeting FERC criteria is not recommended.

A conceptual section of the spillway structure stabilization is provided in Exhibit 8 in Appendix F.

Several gate options were considered for the dam reconstruction; however, the pier and bridge configuration above the spillway is not conducive to different gate systems. The options considered are shown in Table 4-5.

Table 4-5 Spillway Gate System Comparison

Option	Suitable	Explanation
Radial Gates	No	Radial gates are mounted on an arm and are lifted by rotating the arm upwards so have a circular motion. Installing radial gates at the existing spillway would require removing a significant portion of the bridge deck.
Crest Gates	No	Crest gates are mounted to the crest of the spillway and when lowered are flush with the crest. The ogee spillway at Lake Delhi Dam is steep and does not have a wide crest, so installation of crest gates would require removal of large portion of the crest to create a platform for mounting the gates.
Lift Gates	Yes	The existing spillway used lift gates so the configuration is suitable for lift gate installation. The gate guides were damaged and need replacement but that repair would be minor compared to the work required to install other gate systems.

Prior to the 2010 dam failure a project was underway to replace the lift gate hoisting mechanism. The hoisting equipment was received by the dam operator but never installed at the dam so could be installed as part of the reconstruction project. The new hoisting equipment should eliminate previous issues experienced with lifting gates and provides an additional 3 feet of lifting height, so the new gate openings will be 25 feet wide by 20 feet high when the gates are fully lifted.

The recommended alternative is to replace the existing lift gates. Since the structure was originally configured to lift gate operation, there are less modifications required compared to the other gate system alternatives. While replacement of the existing lift gates will involve some structural updates (replacement of the slide inserts, new actuators, etc), the basic structural elements are in place. In addition the hoisting mechanism received for the 2009 upgrade project that was never installed can be installed and used with new lift gates.

Table 4-6 Existing Spillway Alternative Cost Comparison

Spillway Anchoring Alternative	Cost
USACE Criteria	$324,000
FERC Criteria	$607,000
Spillway Gate System Alternative	**Cost**
Replace Lift Gate System	$2,044,000

4.6 New Spillway

The new spillway system at Lake Delhi Dam will need to provide roughly 39,000 cfs of additional hydraulic capacity for the dam to pass the design flood of ½ PMF without overtopping the powerhouse or spillway gate structure. There is roughly 230 feet between the buttress wall at the southern end of the existing powerhouse/spillway structure and the southern riverbank where the new dam will tie into existing ground. With this length, a straight, fixed crest at the normal pool elevation of 899.6 ft-msl could pass approximately 13,500 cfs prior to the powerhouse/spillway structure being overtopped. This is less than half of the hydraulic capacity needed so a more hydraulically effective spillway discharge system will be needed at Lake Delhi Dam. Several spillway systems were reviewed for the alternatives analysis. A summary is provided in Table 4-7.

Table 4-7 New Spillway Option Comparison

Option	Suitable	Explanation
Fuse Plug	No	A fuse plug spillway consists of an earthen embankment overlaying a concrete spillway set several feet below the top of embankment. When the pool reaches the top embankment the earth is eroded away, exposing the concrete spillway. At Lake Delhi Dam, the concrete spillway could not be set low enough to provide sufficient hydraulic capacity.
Additional Lift Gates	No	Additional lift gates would require construction of a new section of spillway structure to essentially extend the existing spillway structure. However, bedrock drops away in this area so in addition to the additional cost of purchasing gates and hoisting equipment, the new concrete ogee spillway and operating platform would be founded on sand which would require expensive stability enhancements to make construction viable.
Pipes Through Embankment	No	In addition to concerns over seepage and maintenance, installing pipes through the dam embankment would not provide sufficient capacity and would require construction of a new intake and operating structure.
Labyrinth Weir	Yes	A labyrinth weir consists of a sharp-crested (vertical wall) in a zigzag pattern that allows a much longer crest length to fit within a shorter length of embankment. The longer crest length significantly increases the hydraulic capacity over a straight weir section. A labyrinth weir is a viable option for meeting hydraulic capacity requirements.
Pneumatic Crest Gates	Yes	A pneumatic gate system would consist of a concrete structure with crest control gates spanning the new spillway. They would consist of bottom mounted gate panels that could be lowered flush with the top of the new spillway. In their raised position they would be at or just above the normal pool elevation, but when lowered could provide an additional 5 to 10 feet of depth for discharging flood magnitude flows. Pneumatic gates are a viable option for meeting hydraulic capacity requirements.

From the initial review of spillway options, three spillway alternatives were developed for preliminary design and comparison. The three spillway alternatives are:

- **Dual Labyrinth Weir Spillway** – consisting of a lower principal labyrinth weir spillway set at the normal pool to discharge normal flows and a higher auxiliary labyrinth weir spillway set several feet above normal pool to discharge the required flood magnitude flows.
- **Single Labyrinth Weir Spillway** – consisting of a single labyrinth spillway set at normal pool to discharge normal flows but with sufficient hydraulic capacity to also discharge the required flood magnitude flows.
- **Pneumatic Gate Spillway** – consisting of a pneumatic gate system set at normal pool when raised to discharge normal flows and when lowered provides sufficient hydraulic capacity to discharge the required flood magnitude flows.

Exhibits showing plans and sections of the spillway alternatives are provided in Exhibits 5-7 Appendix F. All spillway alternatives consist of a concrete spillway slab and chute constructed over an earthen embankment with a concrete stilling basin at the end. All spillway alternatives were sized so with the three existing lift gates and the new spillway, the reconstructed dam could pass the ½ PMF without overtopping the powerhouse/spillway structure.

Dual Labyrinth Weir Spillway

Many dams have both a principal and auxiliary spillway. The principal spillway is designed for continuous use in passing normal flows and then the auxiliary spillway is designed for infrequent use in passing high magnitude flood flows. Because the auxiliary spillway is used infrequently, typically cheaper materials that are stable and safe for occasional but not frequent use can be used to construct portions of the spillway. Theoretically, this provides a cost savings in spillway construction. For the Dual Labyrinth Spillway option a principal labyrinth spillway would be used to discharge normal flows, used in tandem with the lift gates to discharge higher flows, and then the auxiliary spillway would engage at flood magnitude flows.

The Dual Labyrinth Spillway consists of a 120-foot long primary spillway labyrinth weir set at the normal pool elevation of 896.3 ft-msl and a 110-foot long auxiliary spillway labyrinth weir set at an elevation of 900 ft-msl.

The primary spillway discharges to a concrete chute with a concrete stilling basin at the toe. Training walls were kept straight for the preliminary design but could potentially converge slightly to save a small amount of concrete.

DNR design criteria require that at minimum the principal spillway be able to discharge the 50-year flood (~24,000 cfs) without engaging the auxiliary spillway. Combined with the spillway lift gates, the primary labyrinth weir spillway can discharge roughly the 100-year flood (~30,000 cfs). This would mean that the size of the principal labyrinth weir spillway could potentially be reduced so the combined gates and principal spillway discharge the 50-year flood and then the auxiliary spillway engages at flows exceeding the 50-year flood. However, it was determined during design that because the auxiliary spillway crest sits at a higher elevation than the principal spillway crest, the auxiliary spillway would have to be upsized more than the principal could be downsized because the principal spillway can discharge more flow due to its lower crest. So the ½ PMF is controlling the design of both the principal spillway and auxiliary spillway.

The auxiliary spillway discharges to either a roller compacted concrete or articulated concrete block chute. These are cheaper surfacing than a concrete chute but are not meant to have continuous or frequent discharge over them. This is an additional reason for keeping a larger principal spillway because it would reduce the potential frequency of use of the auxiliary spillway. In the past three years a 50-year auxiliary spillway would have been used three times with the 2004, 2008, and 2010 floods whereas a 100-year auxiliary spillway would likely not have been used.

Concrete training walls will be provided between the principal and auxiliary spillways and on the southern edge of the auxiliary spillway to contain flow within the spillway chute.

Single Labyrinth Weir Spillway

With the ½ PMF being the controlling flood, the lower the weir crest elevation, the more flow that can be discharged prior to the upstream pool reaching the top of dam elevation of 906 ft-msl. Using a single labyrinth weir set at the normal pool elevation of 896.3 ft-msl allows a greater length of weir to be at the normal pool elevation, reducing the overall length of spillway required to discharge the ½ PMF.

The Single Labyrinth Spillway consists of a 180-foot long labyrinth weir set at the normal pool elevation of 896.3 ft-msl. The entire spillway uses a concrete chute and stilling basin.

For preliminary design the spillway crest was set at a single elevation. For normal operating conditions a better discharge scenario will likely be to provide a weir segment or series of notches a few inches lower than the rest of the weir crest. This will allow the discharge to be more concentrated rather than a thin film of water going over the entire crest and will help maintain the pool at a more constant elevation. This will be analyzed further and refined in final design. This adjustment will not impact the overall hydraulic capacity of the weir for passing flood flows.

Pneumatic Gate Spillway

Similar to the reasoning for developing the single labyrinth weir option, the pneumatic gates provide ½ PMF discharge capacity by essentially lowering the weir crest below the normal pool elevation during flood flows. Because the pneumatic gates can be lowered they provide an even greater flow depth for discharging floods over the spillway prior to the upstream pool reaching the top of dam.

The range of pneumatic gate settings was set to be from normal pool (896.3 ft-msl) down to 888.3 ft-msl which would be flush with the fixed concrete crest of the spillway. An electronic control system would regulate gate settings for normal flow, maintaining a constant pool elevation of 896.3 ft-msl. The length of the pneumatic gate spillway is 160 feet. Taller gates could reduce the length of spillway but also as the gates get taller the foundation gets larger and the downstream tailwater could impact discharge for floods approaching the ½ PMF magnitude.

Cost and Structural Considerations

Several factors were taken into consideration in the hydraulic design of the spillway alternatives. The ultimate controlling factor is passage of the ½ PMF design flood, but items impacting cost, structural stability and constructability were also evaluated.

The geometry of the labyrinth weir and pneumatic gate spillways were not just controlled by hydraulics but by structural issues as well. Labyrinth weir and gate heights were kept between 8 and 10 feet. A higher weir/gate height could provide more effective discharge, however when the wall or gate starts exceeding 10 feet, the additional structural and foundation requirements to make the overall structure stable start increasing to the point that making the spillway structure longer (i.e. more embankment length) is more cost-effective than trying to achieve a higher weir/gate.

A similar issue influences the steepness of the spillway chute. The steepness of the chute is controlled by the stability of the underlying earthen embankment. Hydraulically, a steeper chute could be used for the new spillway. However, the soil and stability parameters of the embankment and foundation control the steepness of the embankment.

Comparison of Three Spillway Alternatives

All three spillway alternatives have distinct advantages and disadvantages. Without considering cost or operating/maintenance requirements, the pneumatic gates appear to be the best option; they take up the least amount of area and provide normal pool control over a wider range of flows. However, pneumatic gates require additional mechanical and electrical systems that are not required for the labyrinth weir spillways. They also require additional operation and maintenance and have a service life of roughly 25 years, which is less than half of the service life of a concrete structure. With cost comparison between the single and dual labyrinth spillways, pneumatic gates would be more expensive to install and maintain.

The single labyrinth is 30 feet longer than the pneumatic gates but requires no operation. There is a greater sense of security knowing that the principal spillway is not subject to operation and maintenance of equipment. This is not to suggest that a labyrinth spillway will not require maintenance such as debris removal, but over normal day-to-day flows, the fixed labyrinth crest will provide a normal pool within 6 inches of 896.3 ft-msl for river flows up to 500 cfs without operating the lift gates.

With a shorter principal spillway, the hydraulic capacity for discharging flows within 6 inches of the normal pool is 300 cfs, so the lift gates would have to be used more frequently. The dual labyrinth weir is also 50 feet longer than the single labyrinth weir, so additional flow easement acquisition and grading will be needed along the south river bank to fit the dual spillways and chutes within the embankment and channel banks. The potential advantage of the dual labyrinth weir over the single labyrinth would be cost of construction where chute and stilling basin concrete (expensive) could be substituted for articulated concrete block or roller compacted concrete (cheaper) for the auxiliary spillway saving money on the overall construction cost. However after quantifying the additional cost of flow easement acquisition and grading and shaping the embankment and channel area for the larger dual labyrinth weir spillway the single labyrinth weir spillway is more cost effective than the dual labyrinth weir spillway.

The recommendation is to provide a single labyrinth weir spillway to discharge normal and flood flows at the dam. The single labyrinth provides less operation requirements, fits adequately within the channel area, will lower upstream flood elevations compared to the pre-breach dam, and effectively discharges the ½ PMF design flood.

Table 4-8 New Spillway Alternative Cost Comparison

Alternative	Cost
Dual Labyrinth Spillway	$2,805,000
Single Labyrinth Spillway	$2,267,000
Pneumatic Gate Spillway	$2,736,000

4.7 South Spillway Embankment (New)

The alternatives for the new (restored) earthen south embankment include a homogenous clay embankment, a zoned embankment (with a glacial till core and loess slopes), and a roller-compacted concrete (RCC) embankment. Long-term stability analysis and geotechnical recommendations indicate that there is little benefit in terms of stability or size of embankment footprint with a homogeneous embankment as compared to a zoned embankment. This is a result of the loess and till soils sampled in the vicinity of the project having similar composition and plasticity. RCC construction allows the use of steeper side slopes, reducing the size of the embankment footprint. However, the RCC option was not cost-effective due to the lack of on-site granular materials and the need to set up a mixing plant near the site. The recommendation is to construct a zoned embankment, utilizing both types of soils identified in the project vicinity. The central, core portion of the embankment will be constructed with lower-permeability glacial till. The core zone will be tied to the spillway structure seepage cutoff as well as the embankment underseepage cutoff.

Table 4-9 South Spillway Embankment Alternative Cost Comparison

Alternative	Cost
Homogeneous Clay	$1,080,000
Zoned Earth	$1,080,000
Roller Compacted Concrete (RCC)	$1,860,000

4.8 South Dam Embankment (Existing)

Two alternatives were evaluated for the existing south dam embankment, removal and replacement of the existing embankment material, or modification of existing embankment. The first alternative involves removing and replacing all existing in situ material and a portion of the seepage cutoff. Approximately 300 feet of existing embankment would be removed from the exposed face to the south abutment of the dam. The portion of the concrete wall and sheetpile cutoff system located above the embankment subgrade would also be removed to allow for construction of the new embankment. A new sheet pile cutoff would be driven and tied into the south side of the new spillway structure.

The other alternative for the existing south embankment is to tie the new spillway embankment into the existing embankment by “benching” into the existing embankment. For this alternative, the newly constructed spillway embankment section will be benched into the existing embankment with 8-foot horizontal by 2-foot vertical lifts. A small portion of the existing cutoff wall will be removed to allow for proper placement and compaction of new fill against the benches. The construction and integrity of the seepage cutoff within the existing embankment is not known. Therefore, a new sheet pile cutoff will be driven adjacent to the existing cutoff wall for the length of the existing embankment.

Removal and replacement of the existing embankment is expensive due to the large amount of material that will need to be removed and replaced. Considering the observed fair condition of the existing embankment and the substantially lower cost of the benching and modification alternative the recommended alternative is to bench the new embankment section into the existing south embankment with installation of a new sheet pile cutoff.

Table 4-10 South Dam Embankment Alternative Cost Comparison

Alternative	Cost
Remove and Replace Existing Embankment	$1,912,000
Bench Into Existing Embankment	$354,000

4.9 Minimum Flow Passage

One of the dam’s operational requirements is to maintain a minimum flow to the downstream channel during times of very low river flow. Several alternatives were evaluated to maintain minimum flow passage:

- Refurbish or replace existing wicket gates for minimum flow control. (As was used previously).
- Install flow valves on new lift gates.
- Install valve or gate in new service spillway.
- Rehabilitate existing sluiceways at the base of the powerhouse structure.

To restore the operation of the dam to the pre-flood operational status and control minimum flow through the wicket gates would require considerable expense. The repairs would include replacing the old screw actuators on the wicket gates with a new system of hydraulic cylinders and a hydraulic power package. The wicket gates (originals from 1927) themselves would need to be refurbished to maximize the length of time until additional maintenance or replacement would be required. The HydroRake system, which was inundated in the flood, would need to be completely inspected - motors and electrical components replaced, equipment to be rewired, hydraulic oil drained, flushed, and replaced, and condition of the system bearings, belts, wheels, etc., would need to be determined.

Installation of flow valves near the bottom of two of the new lift gates is also an option. Compared to the wicket gates, this alternative will simplify the operation, minimize future maintenance, and is a lower cost alternative. The dam's original lift gates are being replaced regardless of the selected alternative, so valves can be mounted on the new gates for minimal additional cost. The flow through the valves will also provide aeration of the water to help maintain downstream oxygen levels.

The third alternative is to install a minimum flow valve or sluice gate within the concrete weir on the new service spillway. This alternative offers similar advantages as the valves on the lift gate alternative in terms of maintenance, aeration, and low-cost. One disadvantage, however, may be access to the valve as the proposed spillway crest does not include an operator bridge. A separate platform would be required for this alternative. A cost was not developed for this option due to these complications, no benefit over the valved lift gate alternative, and added expense;

The final alternative to pass minimum flow is to pass the flow through the existing sluice pipes at the base of the spillway structure. Part of project construction will be to remove the riprap in the near upstream area of the spillway which should expose the sluice pipes. If located, the condition of the sluice pipes will be evaluated.

With installation of valves, the existing wicket gates and trash rake would not need to be repaired but they would still be available for eventual repair if hydroelectric generation is ever redeveloped. The valve and sluice pipe alternatives require significantly less future maintenance than the wicket gate option. There would be no hydraulic systems to maintain (oil, motors, hoses, controls, etc.) or belt conveyors.

The design alternative recommended for minimum flow passage is providing valves in the new lift gates since this option has the lower "known" cost component and would not have any of the dissolved oxygen issues from bottom discharge that the wicket gates and sluice pipes could have.

Table 4-11 Minimum Flow Passage Alternative Cost Comparison

Alternative	Cost
Refurbish Wicket Gates	$114,000
Valves in Lift Gates	$31,000

4.10 Fish Passage

The State of Iowa requires that fish passage be considered in design of any dam reconstruction. Lake Delhi Dam has an abandoned fish ladder on the south buttress wall. These types of steep concrete structures were typically installed in the 1920s–1940s and are not capable of passing native fish species.

The DNR was consulted on fish passage design at Lake Delhi Dam. The following design criteria for a fish passage system were provided:

- Constructed primarily of native stone materials.
- Sloped at a minimum of 20:1 (horizontal:vertical).
- Maintain a minimum wetted perimeter of 15 feet.

- Maintain a minimum cross-sectional depth of 1.5 feet.
- Incorporate resting structures and channel roughness.

A fish passage channel/structure was designed up the south river bank, crossing the dam on the south side of the new spillway structure. The channel would be a rock ramp/rapids configuration. The channel would be graded with a 5-foot wide bottom, 1.5H:1V side slopes, with a depth varying between 3 feet and 5 feet. Due to the 40-foot dam height, the 20H:1V nominally sloped channel is approximately 800 feet long and would feature approximately 20 rock-formed pools, ascending the riverbank and dam embankment. The channel entrance is approximately 500 feet downstream of the dam.

The rock channel would enter a flat, gate-controlled, open air, rectangular concrete channel at the top of the embankment. The concrete channel sides and bottom could be roughened or filled partially with rock. A sluice gate would be provided at the upstream end of the concrete channel. The flow line of the channel would be set slightly below normal pool to provide a constant flow through fish passage structure and channel. The sluice gate would be kept fully open during normal flow conditions but closed completely during times of high flows due to concerns over scouring and eroding the dam embankment area. Operation of the sluice gate could be manual, but it would be critical that the gate was closed during high flows. To improve the overall safety of the structure an automatic closure system is recommended and was included in concept design and costing.

Additional property will need to be purchased for installation of a fish passage channel.

A preliminary layout and section of the fish passage channel is provided in Exhibit 9 in Appendix F.

Installation of a fish passage channel is not recommended for the Lake Delhi Dam reconstruction for the following reasons:

- **Dam Safety:** The fish passage channel is essentially a small spillway. A gated closure at the upstream end would prevent high flows from scouring out the channel and portions of the dam embankment, but would depend on either automatic or manual closure which is subject to uncertainty.
- **Length of Channel:** Rock ramp/rapid fish passage installations have been successful on low-head dams throughout the Midwest. For the Midwest, Lake Delhi Dam is a relatively tall dam and there would be some uncertainty as to the potential usage of an 800-foot long fish passage channel because few have ever been installed in the region.
- **Invasive Fish:** In its reconstructed state, Lake Delhi Dam would provide an effective barrier to invasive fish such as Asian carp swimming upstream of the dam. A fish ladder would negate the dam acting as a barrier to unwanted invasive fish.
- **Cost:** The cost of the fish passage channel is significant, even relative to other structures being evaluated for the dam reconstruction.

- **Maintenance:** With most of the fish passage channel being constructed from loose, natural stone material, periodic maintenance of the rock pools would be required for the channel to remain effective from year to year. Debris and sediment would need to be removed from pools and rocks would need to be moved and replaced to maintain the channel.
- **Property requirements:** Installation of a fish passage channel will require purchase of additional property south of the dam. This has been incorporated into the alternative cost, but property acquisition could cause delays in starting construction.

Table 4-12 Fish Passage Alternative Cost

Alternative	Cost
Rock Ramp/Rapids Structure	$668,000

4.11 Recreational Amenities

Several recreational and public use amenities have been proposed as part of the dam reconstruction project. Proposed amenities include:

- Handicapped Accessible Fishing Pier.
- Canoe Portage Trail.
- Boat Ramp.
- Observation Deck.

The recommended option for providing a Handicapped Access Fishing Pier and Boat Ramp involve the construction of these amenities at a location separate from the dam. It is anticipated that land will be acquired and dedicated for these public access features. A separate location would enable better access to the lake within an area which is free from the inherent hazards of the dam and spillways.

An asphalt parking area is included at the south embankment area, where an adjacent observation deck will be constructed on the pool side. Preliminary alignments and provisions for a canoe portage across the south embankment were developed. Construction of this trail will require private property easements from landowners on the southwest and southeast quadrants of the dam site.

Since these added amenities are characterized as optional, the alternatives development as it pertains to each recreational amenity is simply whether to include or not include each amenity as part of the dam reconstruction project. The costs in the table below only include the construction costs of those features and do not include assumed property easement and acquisition costs. Those real estate costs are included as a separate item in the cost estimate.

Table 4-13 Recreational Amenity Costs

Amenity	Cost
Canoe Portage Trail	$59,000
Boat Ramp	$76,000
Observation Deck	$7,000
Handicapped Accessible Fishing Pier	$70,000

4.12 Sediment Control and Removal

As with any river, the Maquoketa River carries a sediment load. Sources of sediment are both watershed derived (exposed earth areas, farm fields, ditches, concentrated rural or urban stormwater with no sediment control, etc.) and river channel derived (bank sloughing and bed scouring). When river flow enters the Lake Delhi Pool, its velocity is significantly reduced and sediment is deposited in the upstream end of the lake. Over time, sediment deposition can raise the upstream lake bed to a point where it interferes with boating and recreational activities. Average sedimentation rates and volumes for Lake Delhi will be estimated during final design.

With the no pool, there is an opportunity to remove exposed sediment deposits at a lower cost than with traditional dredging methods used when the pool is up. For future sediment maintenance, a series of sediment control projects could be initiated in the Lake Delhi watershed to reduce the volume of watershed derived sediment that reaches the river with the goal of reducing the frequency of future dredging projects.

Section 5

Reconstruction Non-Alternative Features

5.1 Non-Alternative Features

Several Lake Delhi Dam reconstruction items did not warrant an alternatives analysis. These are major dam features that need repair or installation to return the dam to service but with no flexibility or options with how they are installed or restored so they were identified as Non-Alternative Features. Descriptions are provided in subsequent sections and costs have been included in the recommended project cost estimate which is provided in Appendix G.

5.2 Site Access and Utilities

Vehicular access for the powerhouse and existing spillway will be provided on the north embankment. The access area will be paved with asphalt concrete and will be secured with chain link fencing.

On the south embankment, the fish passage chute and gate control structure, if required, can be accessed from a new asphalt paved parking area, which will accommodate ten parking spaces. Steel beam guardrail will be utilized along the edges of the paved areas, and chain link fence provided to restrict access where necessary.

New storm drainage piping and catch basins will be utilized to drain the paved areas adjacent to the dam and existing gated spillway. Water service will be extended to the powerhouse area.

5.3 Powerhouse/Spillway Concrete Repair

The Structural Inspection/Evaluation performed by Stanley Consultants in September 2010 and subsequent site visits/inspections indicated that portions the powerhouse and spillway structures had experienced significant concrete deterioration, spalling and steel corrosion. Portions of the structures that were not visible, such as the lower upstream face of the powerhouse/spillway structure and stilling basin floor, will need to be inspected and evaluated for concrete repairs during construction. All degraded concrete will require repair prior to returning the structures to service.

For the dam reconstruction, structural repair work was limited to what was necessary for the powerhouse and spillway to function as a water retaining structure and meet current dam design and safety criteria.

Repair work for the powerhouse and spillway includes:

1. Anchoring the existing powerhouse/spillway structure to underlying bedrock.
2. Waterproofing the roof slab of the powerhouse.
3. Removing and replacing deteriorated concrete on surface of ogee spillway sections with new concrete.
4. Removing and replacing deteriorated concrete on piers with new concrete.
5. Repairing/replacing gate slot concrete to accommodate the new gate system.

The bridge slab over existing ogee spillway was reconstructed in early 1990s. No significant deterioration was observed during inspections, and the bridge will no longer be open for general public access. No significant structural repair work was proposed for the spillway bridge but depending upon construction equipment/vehicle and load weights, access to the powerhouse portion of the bridge could be restricted.

The 2008 underwater inspection by J.F. Brennan Co. did not find any major structural issues with the lower upstream face of the dam and the stilling basin floor. These areas are assumed to have remained in good condition. The structures will be inspected once riprap is removed upstream and silt downstream to evaluate if repair is needed. At this stage, no significant item for repair of these areas was included in the cost estimate.

5.4 South Buttress Wall

The existing buttress wall is located on the south side of the gated spillway structure. No signs of instability were observed during the September 2010 inspection and subsequent site visits. Some localized areas showed signs of deteriorated surface concrete. The wall structure was determined to be suitable for rehabilitation to act as a transition between the gated spillway and the new spillway. Repair work for the south buttress wall includes:

1. Removal of the abandoned fish ladder.
2. Removal of the top portion of the upstream wall to allow free flow to the new spillway.
3. Raising portion of the downstream wall to accommodate the new spillway structure on the south side.
4. Repair of deteriorated concrete.

5.5 Electrical Service and Controls

Power Distribution

The existing 480/277-volt service to the dam is sized sufficiently for new mechanical and electrical equipment to be installed at the dam. Additionally, the two distribution panelboards HP and LP and the automatic transfer switch are all in good condition. These pieces of equipment can be removed and salvaged for reuse. The dry-type transformer should be inspected for any signs of water damage before a determination can be made if it should be reused. The main service disconnect switch and utility meter can be reused in their current location.

The remainder of the electrical distribution equipment, including the 208/120-volt distribution equipment, equipment disconnects switches, lighting and power circuits, conduit, wiring, motors, and other electrical devices, should be removed from the powerhouse.

The room on the main floor of the powerhouse currently housing the electrical equipment should be established as an electrical and control room. The room should be refinished to include a floor drain and equipment pads for any floor mounted equipment.

A new 480/277-volt, 200-ampere, 3-phase, 4-wire electrical service should be established in the refinished electrical room. A new service panelboard should be installed to include a main circuit breaker, so that the circuit breaker currently located in the stairwell of the powerhouse can be removed. The existing 208/120-volt panelboard and dry-type transformer can be reused.

Trash Rake and Hydroelectric Equipment

The conveyor equipment associated with the trash rake was underwater during the flooding and the motors, wiring, and control equipment associated with this equipment should be inspected and replaced prior to operating the trash rake. A new power feeder should be installed from the new electrical service to replace any existing wiring that may have been underwater.

The existing pool level monitoring and hydroelectric generator control equipment should all be removed as part of the dam restoration. The existing equipment has been extensively damaged due to the flooding and cannot be reused in its current state. The hydroelectric generators themselves can remain in place for possible use in the future with the addition of a control package, but nothing should be added to operate them at this time. If the wicket gates are repaired as part of the dam restoration, a new hydraulic power package and controls will be installed to operate them.

Lift Gates

As part of the dam restoration, three new lift gates will be installed in place of the existing gates. The dam operator previously purchased three electric motor driven screw actuators for operating the three gates. These three actuators can be modified and reused to operate the three new lift gates. The electric motors are 60 horsepower and rated for operation at 480-volts. The existing service would need to be increased in size to operate all three lift gates, however the existing electrical service to the dam will allow for operation of two lift gates simultaneously. This is sufficient for the required operation of the dam. Three full voltage reversing starters should be purchased and installed in the new electrical room to operate the gates. Two of the new lift gates will be provided with a gate valve integral to the gate to provide for minimum flow.

A new control system should also be installed for operation of the lift gates and minimum flow gate valves. A single submersible level transducer should be installed on the pool side of the dam to monitor pool elevation. Automatic control of the gates can be derived based on existing pool elevation with manual control available at the control panel inside the electrical room.

A data connection should be installed to the dam in order to allow for remote monitoring of the dam and the possibility of remote control in the future. Additionally, the data connection can monitor data from the USGS monitoring station upstream of the dam. An autodialer will also be included with the control system to provide a telephone alarm if there is an issue with the automatic control system.

Emergency Generator System

While commercial power to the site is fairly reliable, it is recommended that an emergency generator be installed on site to operate the lift gates in case of power failure. There is no natural gas service to the site, so the new generator should operate on diesel fuel or propane. The installation of a diesel generator will be less expensive than a propane generator, due to the added cost for a propane tank and associated piping. The diesel generator can be provided with a sub-base diesel fuel tank integral to the generator and enclosure.

The generator should be sized at 125 kilowatts to allow for operation of two lift gates simultaneously. This will provide for a complete backup emergency system for the dam. The generator should be located on top of the powerhouse structure to allow for easy access and to be clear of any discharge path from the dam.

5.6 Safety Features

Safety is always a concern at a dam. In the upstream pool access (whether intentional or accidental) to gate intake areas, weir overflow areas, and spillway chutes need adequate warning and protection systems for all flow conditions. Downstream of the dam discharge and energy dissipation structures can cause rollers, eddies and vortices in the immediate downstream channel area that can be dangerous for recreational users of the downstream waterway. Appropriate warning signage and access control is also needed downstream of the dam.

The overall safety of the immediate upstream pool and downstream channel will be a major factor in the detailed design of dam reconstruction. Warning signage will be installed both upstream and downstream of the dam. The dam, spillway, and warning signage will be fully lit at all times. A buoy system, tie-off system, and boat restraining barrier will be installed upstream of the spillway. Warning lights and potentially sirens will be provided to indicate when lift gates are being operated. Fencing will be installed on the north embankment area and south embankment area to control access to the powerhouse and spillway structures.

Dam safety measures will be coordinated with DNR Dam Safety as design proceeds.

5.7 Archaeological Mitigation

A reconnaissance level archaeological survey was completed by Louis Berger Group (LBG) for the project site and upstream pool area. The survey identified 12 known archeological sites for supplemental investigation. In addition, LBG recommended that a supplemental, reconnaissance level survey be completed for portions of the pool area not studied as part of their original reconnaissance. Based on their findings, LBG recommended that the District enter into a Programmatic Agreement with the regulating state and federal agencies, detailing scope of additional investigations, so that permitting of the dam can proceed without delay.

If the investigations indicate sites are eligible for inclusion in the National Register, and/or burial sites are identified, mitigation of potential adverse effects of the project construction and restoration of the original pool level will need to be mitigated. At this time, it is not known if or what mitigation may be required, so a budgetary amount has been included in the recommended project cost estimate.

5.8 Property/Easement Acquisition

The District will need to acquire temporary construction access easements, perpetual easements, and/or property for the construction and future maintenance and operation of the repaired Lake Delhi Dam. Iowa DNR Technical Bulletin 16, "Design Criteria and Guidelines for Iowa Dams," outlines property ownership and easement requirements for dams. The bulletin states that "The determination of lands, easements, and rights-of-way required for the construction, operation and maintenance of a dam project are considered part of the design process."

Temporary easements are required to allow access for construction of the project. These areas will be restored, if damaged, and easement canceled upon completion of construction. Perpetual easement or ownership is required for areas occupied by the dam structures. This is to ensure the District is allowed access to all areas of the project for inspection and maintenance. Perpetual easement or Ownership also places decision authority with the District regarding any construction or modification to these parcels. Perpetual easement or ownership is also required for areas that may be inundated during all flows, up to and including the design flood event. This is to ensure that no construction on, or modifications to any parcels required for safely passing predicted flows are completed without District approval.

The property research completed by Gibbs Engineering and Survey, combined with the preliminary design completed for this study have indentified approximate limits of temporary, and perpetual easement/ownership requirements for the proposed project. As the project moves to final design, design refinements and legal boundary surveys will determine the required easement and ownership boundaries. A budgetary amount was included in the recommended project cost estimate for property/easement acquisition.

Section 6

Construction Sequencing

6.1 Construction Sequencing

Construction of the Lake Delhi Dam Reconstruction Project will be broken into two phases. The two phases of work may be let as a single project or bid under separate construction contracts. Phase 1 will involve work at the existing powerhouse/spillway and north embankment. The Phase 1 work tasks include:

- Powerhouse/spillway stabilization (rock anchors).
- Powerhouse/spillway concrete rehabilitation.
- Stilling basin silt removal.
- Downstream abutment wall reconstruction.
- Electrical system upgrades.
- Lift gate and hoisting equipment replacement.
- Bridge deck (powerhouse roof) improvements.
- North embankment walls demolition and reconstruction.
- Upstream riprap removal.

It is anticipated that the Phase 1 work would take place during the spring and early summer months of the construction season. Water would continue to flow through the current river channel during Phase 1 construction and an upstream pool would not be maintained. Upstream and downstream cofferdams, along with a dewatering operation would be required to maintain workable site conditions.

Phase 2 will involve all work south of the existing powerhouse/spillway. The Phase 2 work tasks include:

- Buttress wall rehabilitation.
- South embankment construction.
- Seepage cutoff installation.
- New spillway construction.
- New stilling basin construction.
- Channel grading.
- Scour protection.
- Public amenities.

It is anticipated that the Phase 2 work would take place during the late summer and fall months of the construction season, with some site finishing and cleanup performed the following spring. During Phase 2 construction, water would be diverted through the existing gated spillway, requiring construction of a substantial upstream cofferdam to raise an upstream pool and establish flow through the gated spillway. Note that existing sluice pipes at the base of the powerhouse will be investigated during Phase 1 construction. If the sluice pipes are operational, there is the potential to divert flow through the sluice pipes during Phase 2 construction, reducing the size of the upstream pool and substantially reducing cofferdam and dewatering construction costs. Under either scenario, upstream and downstream cofferdams, along with a dewatering operation would be required to maintain workable site conditions during Phase 2 construction.

6.2 Construction Staging

Given the limited amount of space and right-of-way at the dam site, alternate construction staging areas will be required. Potential Phase 1 and Phase 2 construction staging areas are located within close proximity to the site. The Owner may elect to designate specific staging areas for construction, or allow the construction contractor to make arrangements with landowners for construction staging.

6.3 Cofferdams and Dewatering

As discussed in Section 6.1, upstream and downstream cofferdams and dewatering operations will be required for both phases of construction. Cofferdam design and dewatering design will be the responsibility of the construction contractor. For cost-estimating purposes, trapezoidal earthen cofferdams were assumed to be constructed to one foot above the five-year return period flood event for Phase 1 construction and one foot above the two-year return period flood event for Phase 2 construction. See Exhibits 10–11 in Appendix F for conceptual cofferdam layouts and cross sections. A deep-well dewatering system was also assumed for each phase of construction for cost-estimating purposes.

Cofferdam construction may require temporary construction right-of-way easements from adjacent property owners.

6.4 Borrow Material

The construction contractor will be responsible for obtaining borrow material from a private source for embankment construction. Soil borings have been advanced at potential borrow site areas to verify that proper embankment construction materials are available in the area. The upper material (loess) may require farming (drying) prior to placement. For cost estimating purposes, a five-mile round-trip cycle was assumed for material hauling.

6.5 Riprap

It is assumed that a sufficient amount of riprap is available on site for the scour protection requirements of the project. The construction contractor will be responsible for excavating, stockpiling, and placing on-site riprap between the two construction phases. If additional riprap is required, it will likely be imported from a neighboring quarry.

Section 7

Cost Estimate and Construction Schedule

7.1 Cost Estimate

A preliminary cost estimate was developed for alternative concepts to assist with evaluation and selection. Recommended alternatives were then incorporated into a preliminary project estimate, representing the current estimated cost of the repair project. Unit costs for project features were taken from RSMeans and the Stanley Consultants database of recently completed dam construction projects. All costs from previous projects were adjusted for location and inflation.

Several cost items were developed based on visual/surficial inspections, document review, and assumptions of typical conditions. During construction, there is always the possibility that unknown issues or conditions will be encountered, impacting the cost of the project. In addition, this estimate represents a preliminary stage of design. As design progresses, the construction cost estimate will be refined. A 20% contingency has been added to the preliminary cost estimate to account for unknowns and future design development.

Estimates of engineering design fee (assuming 7% of construction cost) and engineering services during construction (assuming 36 weeks of construction) have been included in the recommended project. The total estimated cost was then escalated 5% to account for construction next year. The total estimated preliminary cost for the recommended project is $11,870,000. Both the recommended project cost estimate and the individual cost estimates for comparing the reconstruction alternatives are provided in Appendix G.

7.2 Schedule

A preliminary construction schedule was created using Primavera P7 software, and is presented in bar chart format using the critical path method of scheduling. The two-phase approach from Section 6 was used, with Phase 1 awarded April 1 and construction starting mid-April. Phase 2 is awarded in late June, with construction starting in mid-July.

A preliminary schedule was created for two different scenarios. The first schedule was developed using a five-day workweek calendar with weather and holidays. The second schedule was created using the same construction activity durations, but uses a six-day workweek calendar with weather and holidays. With the six-day workweek calendar, critical construction activities are completed before severe winter weather sets in. In the five-day workweek scenario, the schedule is extended into the winter months, and further extended by severe weather allowances in the calendar. The six-day workweek calendar schedule is provided in Appendix G.

Appendix A

Field Investigations and Data Collection

- Stanley Consultants, Inc.; *Structural Inspection and Evaluation of Lake Delhi Dam*; Lake Delhi Recreation Association (LDRA) – Delhi, Iowa; October 2010.
- J. F. Brennan Co., Inc.; *Underwater Inspection of Lake Delhi Dam Structure*; Delaware County, Delhi, Iowa; November 11, 2008.

Structural Inspection and Evaluation of Lake Delhi Dam

Lake Delhi Recreation Association (LDRA)

Delhi, Iowa

FINAL
October 2010

Stanley Consultants INC.

A Stanley Group Company
Engineering, Environmental and Construction Services - Worldwide

Structural Inspection and Evaluation of Lake Delhi Dam

Lake Delhi Recreation Association (LDRA)
Delhi, Iowa

Final
October 2010

I hereby certify that this engineering document was prepared by me or under my direct personal supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Iowa.

Daniel R. Dreyer 10/14/2010
Daniel R. Dreyer, P.E. 10/14/2010

My license renewal date is December 31, 2011.

Pages or sheets covered by this seal: Entire Report

LICENSED PROFESSIONAL ENGINEER
DANIEL R. DREYER
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IOWA

Stanley Consultants INC.
A Stanley Group Company
Engineering, Environmental and Construction Services - Worldwide

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APPENDICES

Lake Delhi Dam Inspection/Evaluation

General

The Lake Delhi Recreation Association (LDRA) requested that Stanley Consultants, Inc. complete a structural inspection/evaluation of the existing Portland Cement Concrete Dam Structure at Lake Delhi near the Town of Delhi, Iowa. During the flood event of July 23-24, 2010, approximately 240 lineal feet of earthen dam located at the southern end of the Portland Cement Concrete Dam Structure was breached and eroded by the flood. Flood water also infiltrated and seeped through approximately 75 feet of earthen dam located at the north end of the Portland Cement Dam Structure. This section of embankment was not breached during the flood event.

Stanley Consultants performed visual inspections of accessible portions of the concrete dam.

Engineer's Inspection

The structural inspection/evaluation of the Lake Delhi Dam was performed on September 23, 2010 by the following personnel:

William E. Holman, P.E., Stanley Consultants, Inc.

Daniel R. Dreyer, P.E., Stanley Consultants, Inc.

Weather conditions during the inspection/evaluation were overcast with scattered showers and temperatures in the 60s. The use of left and right directions are referred in the text as one faces downstream. The inspection/evaluation was conducted in the presence of Dave Fink (LDRA project manager) and Roger Mohn, P.E. The above water portions of the following dam components (from north to south) were observed:

- Approach roadway and retaining walls (North Embankment)
- Powerhouse Structure
- Gated Spillways
- Inspection Tunnel

Each of these major components was inspected for signs of settlement, movement, seepage, leakage, cracking, erosion, and general signs of deterioration. A detailed photographic log of the major dam components is included in Appendix A.

Approach Roadway and Retaining Walls (North Embankment)

The approach to the north end of the dam consists of earthen embankment with retaining walls supporting a concrete roadway. The original approach roadway curved to the east from the north end of the Powerhouse Structure. The original design drawings indicate that a concrete retaining wall was constructed on each side of this roadway. Sometime in the past, the road was straightened and a precast concrete "bin-type" retaining wall was constructed roughly parallel to the upstream side of the dam (Photo 1.1). The original upstream retaining wall was covered in fill beneath the new road.

A later project installed a storm water intake in the approach roadway (Photo 1.2). This intake discharges via a concrete pipe routed through the retaining wall on the upstream side of the dam. When the drainage pipe was installed, a portion of the original retaining wall on the upstream side of the dam was removed to make room for the pipe. Based on observations made during the inspection, the wall was not patched after pipe was placed.

The upstream retaining wall generally appeared to be stable and in good condition, with a localized area of deterioration below the outlet of the storm pipe (Photo 1.3). The grade along the base of the upstream retaining wall shows signs of significant erosion. The eroded area has been partially filled with concrete rubble and/or large riprap (Photo 1.1)

The approach roadway appears to have been damaged, presumably during the July flood, due to the presence of slab cracking and settlement (Photo 1.4). The cracking and settlement appear to be the result of the loss of material adjacent to and underneath the roadway in the vicinity of the storm drainage pipe. Presumably, the material loss resulted from the seepage of water from the upstream side of the embankment, along the concrete pipe and through the retaining wall on the downstream side of the dam.

The retaining wall on the downstream side of the approach roadway appears to be original construction. The wall is of reinforced concrete construction (Photos 1.5 to 1.7). The wall generally appeared to be stable and in fair condition with no conclusive indications of tipping or differential movement. Efflorescence was observed around multiple cracks in the downstream retaining wall. The efflorescence appeared to be concentrated around a rectangular area near the center of the wall (Photo 1.8). The rectangular portion of concrete appears to have been poured at a different time than the surrounding concrete wall. The efflorescence appears to be result of water behind the wall seeping through the joint around the rectangular area and through cracks that have formed in this area. It was reported that during the July flood event, multiple leaks were observed though cracks in this area of the wall. Water was seen spraying several feet beyond the face of the wall.

A diagonal wall crack was observed near the south end of the wall near the point where the wall ties into the Powerhouse Structure. The crack was visible from the ground surface in front of the wall to a horizontal construction joint approximately 10’ above grade (Photos 1.9 to 1.11). The crack could be an indication of settlement of the retaining wall relative to the Powerhouse Structure. It is unknown whether this crack existed prior to the July flood event. No efflorescence was observed around this crack, possibly indicating the crack developed more recently than some of the other cracks in the wall.

Powerhouse Structure

The Powerhouse Structure is located immediately south (to the right) of the approach roadway and retaining walls (Photos 2.1 and 2.2). The Powerhouse is a multi-level reinforced concrete structure. Various improvements have been performed in recent years including concrete repair and installation of new trash rack cleaning equipment and dewatering gate hoisting equipment. Most of the improvements were to the operating deck level at the top of the dam, and generally did not extend below the upstream waterline or into the turbine room.

The concrete on the upstream face of the Powerhouse below the original waterline appears to be in generally good condition. No significant cracking, differential movement, tilting or alignment changes were observed along this portion of the structure. The concrete at or above the original waterline appeared to be in fair condition. More cracking, spalling and evidence of past repairs were observed in these locations (Photo 2.3). Dave Fink reported that during the dewatering bulkhead hoist installation project, the concrete below the south hoist support columns had to be removed as much as six feet to reach sound concrete. Only three feet of concrete under the remaining columns was removed as needed to install anchor bolts for the new columns. The upstream sides of the turbine bays were packed with wood and other debris remaining from the July flood, and the bar screens and piers were mostly covered in marine growth (Photo 2.4).

Downstream of the structure, the east concrete wall is in fair condition (Photos 2.5, 2.6, and 2.7). No significant cracking, differential movement, tilting or alignment changes were observed along this side of the structure. A deteriorated portion of wall was observed adjacent to the existing metal panel wall near the north end of the wall (Photo 2.8). It is presumed that this deterioration is the result of cracking in the upper portions of the wall, allowing the infiltration of water and de-icing salts from the roadway above the roof of the Powerhouse.

The interior of the Powerhouse consists of three main rooms on three levels. A Control Room is located just below and upstream of the roadway. This room was originally accessed via a floor hatch and ladder (Photo 2.9). The dam operations staff has installed steel stairs enclosed by a concrete masonry block building at roadway level to improve access to this room (Photo 2.10). The concrete roof, walls and floor of this room appear to be in fair condition. Cracks were observed in the north wall of this room (Photo 2.11). These cracks do not appear to be recently formed, due to the presence of paint in the crack in some locations. Due to their random pattern, the cracks do not appear to be a sign of a significant structural defect or differential movement, and were most likely the result of concrete shrinkage. The steel stairs are functional, but are significantly corroded in several locations.

The Turbine Room floor is located approximately 24 feet below the roadway level. The roof of the Turbine Room supports the roadway above. The generators are not currently in operation. Prior to the July flood, the dam operations staff regularly operated the wicket gates to permit the flow of water through this portion of the dam. The original windows on the downstream side of the dam have been replaced with metal panels and, residential type windows and exhaust fans (Photo 2.12). These items appear to be in good condition except for the northernmost section. The exterior door at this location is missing, and a temporary plywood barrier has been installed in its place (Photo 2.13).

The upstream wall of the Turbine Room appears to be in fair condition with no major cracking or other damage. Random cracks were observed near the floor level (Photo 2.14). These cracks do not appear to be a sign of a significant structural defect or differential movement, and were most likely caused by concrete shrinkage. Efflorescence was observed around other cracks and construction joints in this wall (Photo 2.14). The efflorescence is an indication of water seepage through the cracks. A steel stair is located along this wall to provide access between the Turbine Room and the Control Room (Photos 2.14 and 2.15). The stairs are functional, but are heavily corroded in many locations.

The north and south walls of the Turbine Room appear to be in fair condition. Water has been infiltrating through the north wall through various cracks and construction joints (Photo 2.15). The water appears to be infiltrating the wall from the saturated fill behind it, and from cracks in the roof slab adjacent to the wall. The south wall appears to be in better condition than the north wall. The cracking and water infiltration appears to be most prevalent in the upper portions of the wall near the roof slab (Photo 2.16).

Cracks were observed in the northeast corner of Turbine Room roof (Photo 2.17). The cracks appear to extend from the roof slab into the north wall. Dave Fink reported that water regularly leaks through the roof in this location, prompting the dam operation staff to construct wood and metal sheds to keep various pieces of equipment dry (Photo 2.15). The cracks are likely the result of water infiltrating through the roadway above and into the concrete roof structure. Repeated freeze-thaw cycles have likely deteriorated the roof concrete, creating additional cracks and allowing further water infiltration. De-icing chemicals used on the roadway above could have been carried into the roof concrete with the water. These chemicals could have contributed to further deterioration of the roof and walls, by corroding the reinforcing steel. It is likely that this process contributed to the concrete cracking and deterioration observed on the downstream exterior wall (Photo 2.8). The roof leaks have contributed to heavy corrosion observed on all exposed steel in this room, including the overhead bridge crane and runway beams (Photo 2.17).

A Mechanical Room (Photos 2.18 and 2.19) is located below the north end of the Turbine Room floor. This room reportedly housed a boiler and other equipment that have since been removed. Access to an inspection tunnel that runs the full length of the concrete portion of the dam structure is located in the northeast corner of this room. The steel stairs from the Turbine Room to the Mechanical Room are functional, but heavily corroded (Photo 2.20). Embedded steel around the perimeter of removable floor panels between the Turbine Room and the Mechanical Room are significantly corroded (Photo 2.20). The concrete walls and ceiling appear to be in good condition, with no significant cracking or deterioration observed. The majority of the floor was covered in mud and was not observed.

Gated Spillways

The above water inspection of the gated spillways included three concrete spillways, concrete spillway piers and abutment, three vertical lift steel gates (numbered starting with Gate 1 at the left to Gate 3 at the right), the operator's deck and roadway (Photos 3.1 and 3.2).

Concrete Spillways Logs, wood framing, remnants of a pontoon boat, and other debris accumulated upstream of the spillway gates during the July flood (Photo 3.1). The upstream concrete surfaces of the spillways appear to be in fair condition. No significant cracking, differential movement, tilting or alignment changes were observed along this portion of the structure. Several abrasions on the concrete surfaces in this location were observed, presumably caused by debris and other objects impacting the concrete as they passed over the spillway (Photos 3.3 and 3.4). The abrasions may be an indication of deterioration and softening of the outer concrete surface.

The downstream surfaces of the spillway crests appear to be in fair condition. No significant cracking, differential movement, tilting or alignment changes were observed along this portion of the structure. Abrasions similar to those observed on the upstream spillway surfaces were visible on the downstream surfaces, along with apparent areas of concrete surface erosion resulting from the flow of water over the spillways and freeze-thaw action (Photo 3.5). The exposed surface of the spillway concrete was easily chipped with a light hammer, indicating softening of the concrete (Photo 3.6).

Concrete Spillway Piers and Abutment Significant concrete deterioration was observed in the gate piers. Soft concrete that sounded hollow when tapped with a hammer was encountered from the crest of spillway downstream of the gate slots upward as far as could be reached at Gates 1 and 2 (Photo 3.7). This indicates significant concrete deterioration and potential delaminated concrete requiring repair.

Severe concrete deterioration and damage was observed in the piers adjacent to Gate 3 (Photos 3.8 and 3.9). The left embedded steel guide slot for Gate 3 has broken away and moved outward from the pier, spalling the adjacent concrete and exposing the underlying reinforcing. It was reported that Gate 3 seized during the flood and could not be fully opened. The out of plumb guide slots likely caused this situation.

The embedded steel guide slots and seal plates for all three gates are in generally poor condition and are heavily corroded (Photo 3.10). These conditions likely resulted in the significant leakage reported around the gates in the closed position.

The abutment wall appears to be in fair condition. Eroded concrete and exposed reinforcing was visible on the abutment downstream of the spillway (Photo 3.11). The bedrock foundation supporting the downstream end of the abutment appears to be partially eroded away (Photo 3.12). Prior to the earthen dam breach, water was reportedly observed seeping out of the embankment in this area, near the end of an abandoned concrete fish ladder structure.

Minor cracks are present in the top of the downstream abutment wall (Photo 3.13). The top of the upstream abutment wall has been damaged, presumably due to debris, boats and other items that impacted the wall after the earthen dam was breached (Photo 3.14). No excessive cracking beyond that noted above, differential movement, tilting or alignment changes were observed along this portion of the structure.

The abutment wall was also observed from the south bank of the Maquoketa River (Photo 3.15). A reinforced concrete and steel sheet pile cutoff wall tied into the abutment near the upstream face of the spillway. The cutoff wall collapsed during the earthen dam breach. A section of abutment wall near the top of the cutoff wall appears to have been repaired sometime in the in past (Photo 3.16). Cracking with efflorescence and spalling in the concrete repair area was visible from the south bank of the river.

Steel Gates The dam utilizes the steel gates originally constructed with the dam. The gates appear to have multiple coats of paint that appear to be flaking and beyond their useful life. Varying amounts of corrosion are visible on all three gates, along with the degraded coatings (Photos 3.17 and 3.18). The portions of the gates visible at the time of the inspection appear to be in serviceable condition. The gates do not have side seals. The bottom seals consist of timbers attached to the bottom of each gate, which bear on the seal plates embedded in the top of the spillway concrete. The timbers appear to be in poor condition and reportedly do not provide adequate seal at the bottom of the gates (Photo 3.19).

Operator's Deck and Roadway The operator's deck was undergoing partial rehabilitation prior to the July flood. The existing drum and cable hoists were being replaced with vertical stem lifting mechanisms. The tops of the concrete piers were demolished and reconstructed to install anchor bolts for attaching the new lifting mechanisms (Photos 3.20 and 3.21). Cracking was observed on the underside of the concrete beams on the upstream side of the operator's deck (Photo 3.22). Cracking and efflorescence was observed in the concrete beams on the downstream side of the operator's deck (Photo 3.23).

The Roadway appears to be relatively new construction that is in very good condition (Photos 3.23, 3.24 and 3.25). No significant cracking or deterioration was observed in the roadway.

Inspection Tunnel

An inspection tunnel (approximately 4' wide by 8' tall) starts at the north end of the Powerhouse and extends the full length of the concrete structure to the south abutment wall. The tunnel is located on the upstream side of the dam. The floor of the tunnel is approximately 18'-6" below the crest of the gated spillways.

The floor of the tunnel was covered with a thick layer of mud leftover from the July flood (Photo 4.1). The concrete tunnel walls were generally dry and in good condition. No significant cracking or differential movement was observed in this part of the structure.

Moisture on the walls and roof of the tunnel was observed at multiple locations. The moisture appeared to be the result of water seeping into the tunnel through construction joints in the concrete. It appears that this has been happening for an extended period of time, based on the presence of efflorescence and what appear to be lime deposits forming on the concrete adjacent to the construction joints (Photos 4.2 through 4.7).

Recommendations for Repair and Evaluation

Areas in Need of Repair and Evaluation Prior to Returning the Dam to Service

Approach Roadway and Retaining Walls

1. The eroded grade in front of the upstream retaining wall should be filled and protected with additional riprap to reduce the potential for future erosion.
2. The damaged components of the upstream concrete "bin wall" should be repaired or replaced in the dry, while access to the wall is optimal.
3. The north approach roadway between the upstream and downstream retaining walls should be removed and the embankment beneath the roadway evaluated for the presence of voids or other anomalies. Voids or other conditions that could potentially compromise the stability of the roadway and/or the retaining walls should be filled with flowable cementitious fill or compacted soil.
4. The portions of original wall construction removed for the placement of the storm drainage pipe should be patched with concrete to reduce the potential for seepage along the pipe during high water.
5. The grade in front of the downstream retaining wall should be evaluated for the presence of subsurface voids or other anomalies. Voids or other conditions that could compromise the stability of the wall should be filled with flowable cementitious fill or compacted soil.
6. The cracks and holes in the downstream retaining wall should be repaired to reduce the potential for water infiltration into and seepage through the wall.
7. Crack monitoring gauges should be installed across selected existing cracks, so that they can be monitored for differential movement during pool filling and operation.

Powerhouse Structure

1. The debris upstream of the bar screens should be removed and disposed of.
2. The marine growth on the bar screens should be removed.
3. A new, secure, exterior door on the downstream wall of the Powerhouse should be installed.

4. Consideration should be given to performing concrete repairs and repairing or replacing embedded steel items on the upstream side of the structure while the area is dry and access is optimal.

5. Install crack monitoring gauges across selected existing cracks, so that they can be monitored for differential movement during pool filling and operation.

Gated Spillways

1. The debris upstream of the gates should be removed and disposed of.
2. The embedded guide slots in the piers and abutment should be repaired or replaced to ensure they are plumb and adequately anchored to the surrounding concrete.
3. The deteriorated and damaged concrete on the piers, abutment and weirs should be repaired.
4. An Underwater Inspection of the submerged downstream dam apron and adjacent river bottom was reportedly performed in 2008. The July 2010 flood subjected the dam to severe conditions that could have damaged the underwater portions of the structure or eroded the riverbank immediately downstream of the dam. An additional Underwater Inspection should be conducted using a commercial diver and acoustic imaging methods. The primary purpose of this inspection would be to document the current structural condition of the downstream dam apron and to estimate the lateral extent of any apron damage discovered. In addition, the inspection would determine if scour has taken place adjacent to and under the apron, adversely impacting the structural integrity of the dam structure.
5. The embedded gate seal plates at the crest of the weir should be reconditioned or replaced.
6. The deteriorated or damaged concrete observed on the south side of the abutment wall adjacent to cutoff wall tie-in point should be repaired.
7. The wooden seals on the bottom of the gates should be replaced with rubber seals to reduce the amount of leakage under the gates at these locations.
8. Consideration should be given to retrofitting the existing gates with rubber side seals to reduce the amount of leakage past the gates in these locations.

9. Consideration should be given to sandblasting, repairing and recoating the gates while the area is dry and access is optimal.
10. Consideration should be given to repairing the concrete beams supporting the Operator's deck above the spillways while the area is dry and access is optimal.

Inspection Tunnel

1. Remove the mud from the floor of the tunnel and inspect tunnel floor for signs of cracking or differential movement.

Areas in Need of Repair and Evaluation Following Restoration of the Pool

The following items are in need of repair, but could be delayed until after the dam is returned to service. If the dam is not immediately returned to service, these items should be evaluated and performed within the next 5 years to slow the advancement of deterioration in the existing facility.

Powerhouse Structure

1. Perform concrete repairs and repair or replace embedded steel items on the upstream side of the structure.
2. Remove the roadway above the Powerhouse and repair the deteriorated concrete roof structure.
3. The Powerhouse roof should be waterproofed and sloped to drain.
4. The roadway above the Powerhouse roof could be reconstructed, or replaced with an alternative system that will allow access to the remaining structure for maintenance.
5. Cracks and other deterioration observed in the concrete walls should be repaired.
6. The corroded steel stairs and railings in the Powerhouse should be repaired and repainted, or replaced.
7. Metal items embedded in concrete that are significantly corroded should be repaired or replaced.
8. The existing overhead bridge crane should be repaired and repainted.

Gated Spillways

1. If not completed prior to retuning the dam to service, the existing gates should be retrofitted with rubber side seals to reduce the amount of leakage past the gates in these locations.

2. If not completed prior to retuning the dam to service, the existing gates should be sandblasted, repaired and recoated.

3. If not completed prior to retuning the dam to service, the concrete beams supporting the Operator's deck above the spillways should be repaired.

While above recommendations were primarily based upon safety or operational concerns and structures' integrity, other criteria such as preservation of structure, structure lifecycle, environmental, aesthetics, etc. should also be considered when monitoring/ maintenance/repair is programmed.

Conclusions

Stanley Consultants performed visual inspections of accessible portions of the concrete dam. The dam needs repair, but does not show obvious signs of significant structural defects or differential movement.

In our opinion, the remaining concrete dam structure is serviceable and could be returned to service upon completion of the recommended repairs and additional Underwater Inspections. Should the additional Underwater Inspections reveal hidden damage and/or undermining of the dam, repairs should be made prior to returning the dam to service.

The July 2010 flood subjected the dam to severe conditions that could have damaged or undermined the dam in ways that cannot be readily observed from a visual inspection. If the dam is returned to service, it should be monitored while the pool is restored to identify spreading of existing cracks, differential movement within the structure and seepage under the foundation.

A survey of the structure prior to filling the pool should be completed to serve as a reference for monitoring structure movement. Divers could be used to inspect for signs of seepage under the dam during filling.

Respectfully submitted,

Stanley Consultants, Inc.

Prepared by Daniel R. Dreyer P.E.
Daniel R. Dreyer, P.E.

Approved by William E. Holman
William E. Holman, P.E.

Appendix A

Photographic Log

Photo 1.1 – Upstream retaining wall adjacent to north approach roadway – Note eroded area in front of wall partially filled with riprap

Photo 1.2 – Storm water intake in approach roadway

Photo 1.3 – Storm drainage pipe penetration through retaining wall – Note damaged concrete below pipe

Photo 1.4 – Cracking and settlement in the approach roadway – Note the loss of shoulder material beyond the edge of roadway

1.5, 1.6, and 1.7 – Downstream retaining wall – Note diagonal wall crack at location A – Note efflorescence at location B.

1.9 – Efflorescence around cracks in downstream retaining wall

1.9, 1.10, and 1.11 – Diagonal crack in downstream retaining wall

2.1 – Upstream view of Powerhouse Structure

2.2 – Downstream view of Powerhouse Structure

2.3 – Upstream side of Powerhouse – Note apparent spalling and past repairs above original waterline

2.4 – Debris and marine growth on upstream side of turbine bays

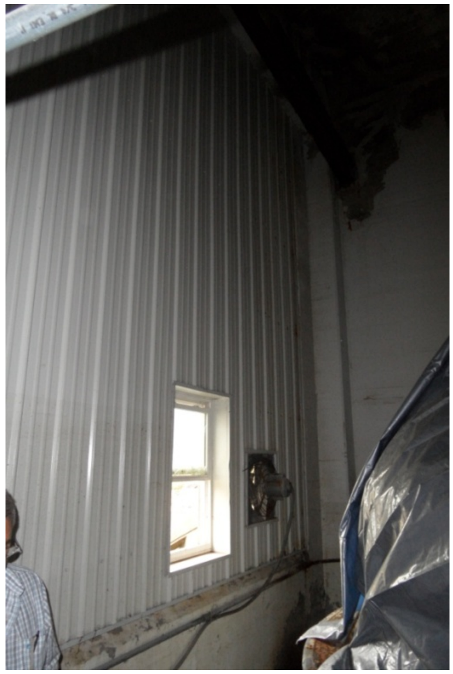
2.5, 2.6, 2,7 – Downstream side of Powerhouse – Concrete deterioration cracking and deterioration observed at location A

2.8 – Deterioration in downstream wall of Powerhouse

2.9 – Control Room access hatch and ladder

2.10 – Control Room access stairs

2.11 – Cracking in north wall of Control Room



2.12 – Metal panels installed along downstream wall

2.13 – Temporary barrier to Turbine Room

2.14 – Random cracking and efflorescence on upstream wall of Turbine Room

2.15 – Cracking in north wall of Turbine Room

2.16 – Cracking near top of south wall of Turbine Room

2.17 – Cracks in Turbine Room roof and north wall

2.18 – Southwest corner of Mechanical Room

2.19 – Southeast corner of Mechanical Room

2.20 – Removable floor panels between Turbine Room and Mechanical Room – Note heavy corrosion on steel stairs and railings on the left

3.1 – Upstream side of Gated Spillways

3.2 – Downstream side of Gated Spillways

3.3 – Abrasions on the upstream side of a typical spillway

3.4 – Abrasions on the upstream side of a typical spillway

3.5 – Abrasions and erosion in downstream surfaces of a typical spillway

3.6 – Soft surfaces of spillway concrete can be chipped away with light hammer

3.7 – Hollow sounding concrete when tapped with hammer at Gates 1 and 2

3.8 – Severe concrete deterioration in left pier at Gate 3

3.9 – Severe concrete deterioration in left pier at Gate 3 – Note how embedded steel guide slot has been pushed away from pier and is out of plumb

3.10 – Typical condition of embedded steel guide slots and seal plates

3.11 – Exposed reinforcing and holes in abutment wall downstream of spillway

3.12 – Erosion of bedrock foundation under downstream end of abutment

3.13 – Top of downstream abutment wall – Note remnants of abandoned fish ladder structure built into wall

3.14 – Damage to top of upstream abutment wall

3.15 – View of abutment wall from south bank – Note area of previously patched concrete adjacent to remnants of cutoff wall

3.16 – Close-up of abutment wall repair area

3.17 – Upstream side of typical gate

3.18 – Downstream side of typical gate

3.19 – Typical timber gate bottom seal

3.20 – Operator's deck rehabilitation work

3.21 – Top of typical reconstructed pier with new equipment anchor bolts

3.22 – Cracking in beams on upstream side of operator’s deck

3.23 – Cracking and efflorescence in concrete beams on downstream side of operator’s deck

3.24 – Roadway (looking north from abutment)

3.25 – Roadway (looking south from north approach roadway)

4.1 – North end of inspection tunnel (looking south)

4.2 – Moisture and lime deposits on tunnel walls

4.3 – Moisture and lime deposits on tunnel walls

4.4 – Moisture and lime deposits on tunnel walls

4.5 – Moisture and lime deposits on tunnel walls

4.6 – South end of tunnel at intersection with abutment wall

4.7 – South end of tunnel (looking north)

Underwater Inspection of Lake Delhi Dam Structure
Delaware County, Delhi, Iowa

November 11, 2008

J. F. Brennan Co., Inc.
820 Bainbridge Street
La Crosse WI. 54603
(608) 784-7173

BRENNAN
MARINE PROFESSIONALS
LA CROSSE, WI

Table of Contents

Underwater Inspection of Lake Delhi Dam

I. Scope of Services

J.F. Brennan Co., Inc. was contracted by the Lake Delhi Recreation Association to perform an underwater inspection of both the upstream and the downstream portions of the Lake Delhi dam structure including the north abutment wall, the hydro flumes, the spillway/ogee section, the south wall, and the downstream baffle wall. Special attention was focused on the extent of lake-bottom scour areas adjacent to the upstream face of the dam and river-bed scour areas adjacent to the downstream baffle wall as well as any damaged areas of the PCC walls of the dam. The inspection involved a visual, tactile, and video inspection of the aforementioned structures.

II. Existing Conditions

Unit Data

Owner:	Lake Delhi Recreation Association
Crossing:	Maquoketa River
Location:	Delhi, Iowa

Water

Visibility:	10'
Water Velocity	Moderate
Water Temperature	35°F-40°F

Site Map

See Attachment 'A'

III. Method of Operation

Procedure

The diver used surface supplied air and was equipped with a Superlite 17 dive helmet with surface communications. The dive supervisor and project engineer was in continuous communication with the diver coordinating the features and areas to be inspected and the conditions encountered at the inspected areas. The project engineer recorded contemporaneous inspection notes. Video of the inspected areas and structures were recorded along with the audio communications of the diver, dive supervisor, and project engineer. All measurements are approximate.

Orientation

For this report, the dam is considered to be running in a north-south direction. Lake Delhi is on the west side of the dam. The Maquoketa River flows eastward from the east side of the dam.

The inspection began on the upstream side of the dam at the south end of the wing wall at the southwest corner of the dam. The diver continued along the wing wall to the upstream spillway/ogee section of the dam and then along the upstream side of the spillway/ogee section of the dam to the power house. The inspection continued along the upstream side of the power house and trash racks of the hydro flume and then along the dam wall at the northwest corner of the dam.

The downstream inspection began at the north end of the baffle wall and progressed south along the baffle wall to the southeast wing wall of the dam. The diver then proceeded to inspect the underwater portion of the southwest wing wall toward the spillway/ogee sections. Once at the spillway/ogee sections, the diver inspected the base of the spillway and along the downstream base of power house. The area under the wicket gates was inspected for PCC concrete damage. The inspection then progressed downstream along the northeast wing wall of the dam. Finally the inspection focused on the condition of a PCC guide wall that runs east-west in line with the south wall of the power house on the downstream side of the dam and on the condition of the PCC floor of the tailrace from the power house and spillway to the baffle wall.

IV. Results of Inspection

Date of Inspection: November 11, 2008

Dive Team: Craig Bartheld-Supervisor, Roger R. Mohn- Project Engineer

Southwest Wing Wall

Fill on the backside (south) side of the wing wall is at same level as the top of the wall. Damage to the PC concrete wall was discovered at one area of the wing wall (see Attachment 'B'). The remainder of the PC concrete wall is in good condition. Revetment stone exists along the base of the wall and appeared to be in good condition. No scouring was evident.

Upstream side of Spillway/ogee Sections and Power House

The PC concrete sill along the upstream edge of the spillway is in good condition. The west edge of the sill is approximately 2 inches from the bulkhead slots that run vertically on each side of each bay of the three spillway sections.

Minor damage to the PC concrete of the bulkhead slots and bull-nose piers was found at approximate waterline at several locations. The damage was ½" –4" deep and extended 12"+/- above to 12"+/- below normal water line. See Attachment 'C' for locations of PC concrete damage.

Revetment stone was found along the upstream side of the spillway sections only. The depth of the top of the revetment stone from the top of the PCC sill along the spillway sections was approximately 11-12 feet. No scouring under the PCC sill was discovered along the spillway sections.

The depth of the lake bed adjacent to the upstream side of the power house was discovered to be 23 feet below the PCC sill. The lake bed substrate was found to be clay without revetment stone. Therefore, the area adjacent to the upstream side of the power house appears to have been scoured out. Scouring did not appear to extend under the power house foundation walls.

Downstream Side of Dam

Baffle Wall

Areas of damage to the PC concrete walls of the baffle wall were found. The wall appears to have been irregularly formed during original construction. The irregularity of the wall makes it difficult to determine extent of damage. Large boulders were used as forms for placement of the concrete for the wall during original construction. The bed of the Maquoketa River is 7-8 feet below the top of the baffle wall as a result of scouring on the downstream side of the baffle wall (see Attachment 'D').

South Wing Wall

The south wing wall on the downstream side of the dam is in good to moderate condition. There are several areas where ½" to 1" of PC concrete is missing from the wall face (see Attachment 'E'). Undermining of the wall appears to have been corrected with the placement of PCC-filled mats in the past.

Ogee Spillway

There is minor damage (1/2"-1") to the PCC of the spillway ogee sections. Due to water flowing over the spillway ogee sections, inspection was minimal. The spillway floor PC concrete appears to be in good condition.

Hydro Discharge/Flume Area

The condition of the PC concrete of the hydro discharge chambers appeared to be good. The exposed PC concrete edge of the south discharge chamber was damaged with exposed reinforcing steel (see Attachment 'F').

North Wing Wall

An area of PC concrete damage was found at the base of the wall approximately 10 feet west of the intersection of wing wall with the baffle wall. An area 10' long by 12" deep by 18" high was found where the concrete was missing. The remainder of the wing wall concrete was in moderate condition.

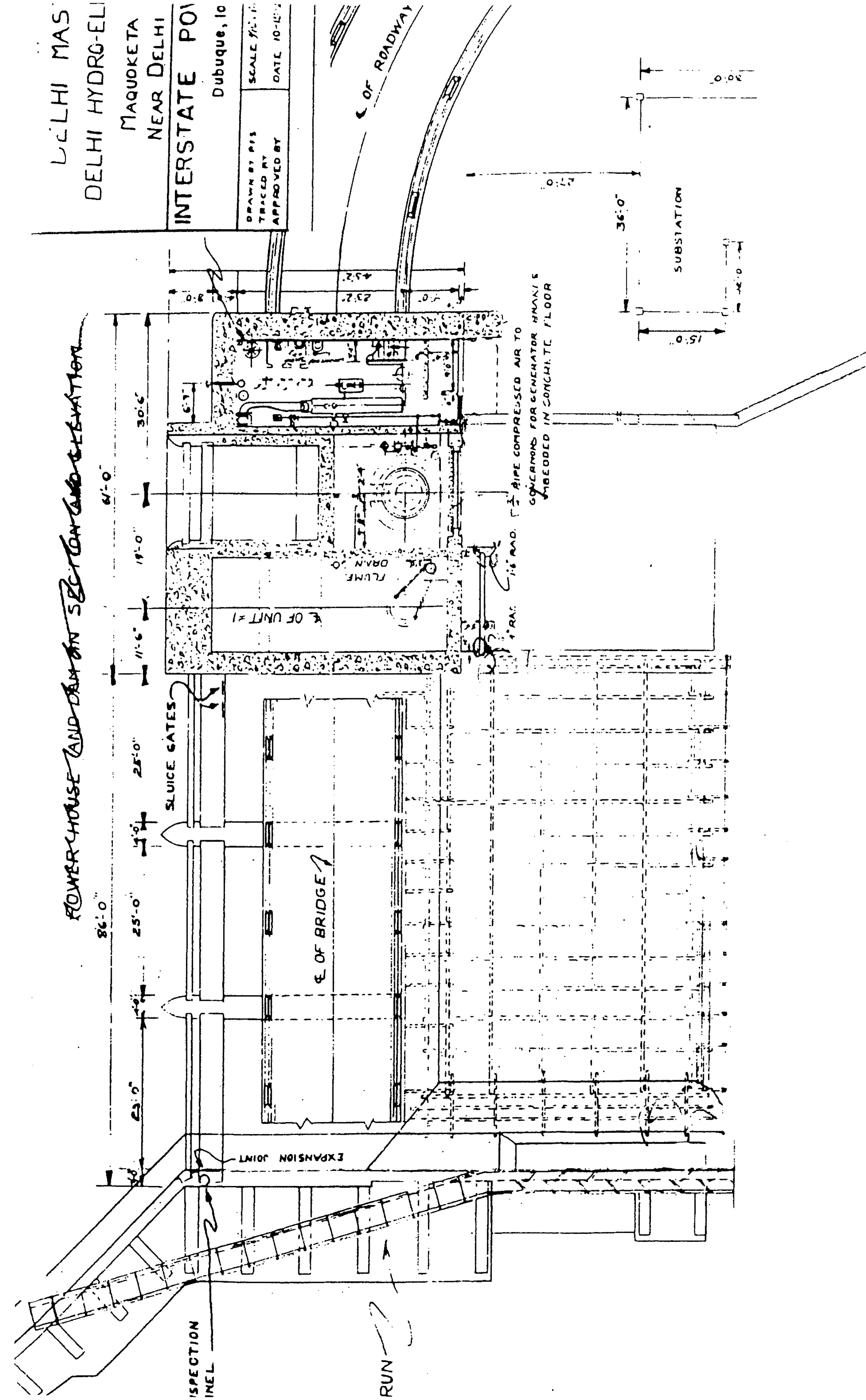
V. Proposed Repairs

All damaged concrete areas would be repaired by removing the existing PC concrete to a depth of 6 inches, placement of reinforcing dowel bars, and placement of new PC concrete.

Revetment stone would be placed in the upstream and downstream scour areas. 12-inch nominal diameter revetment stone would be used and capped with 24-inch diameter derrick stone.

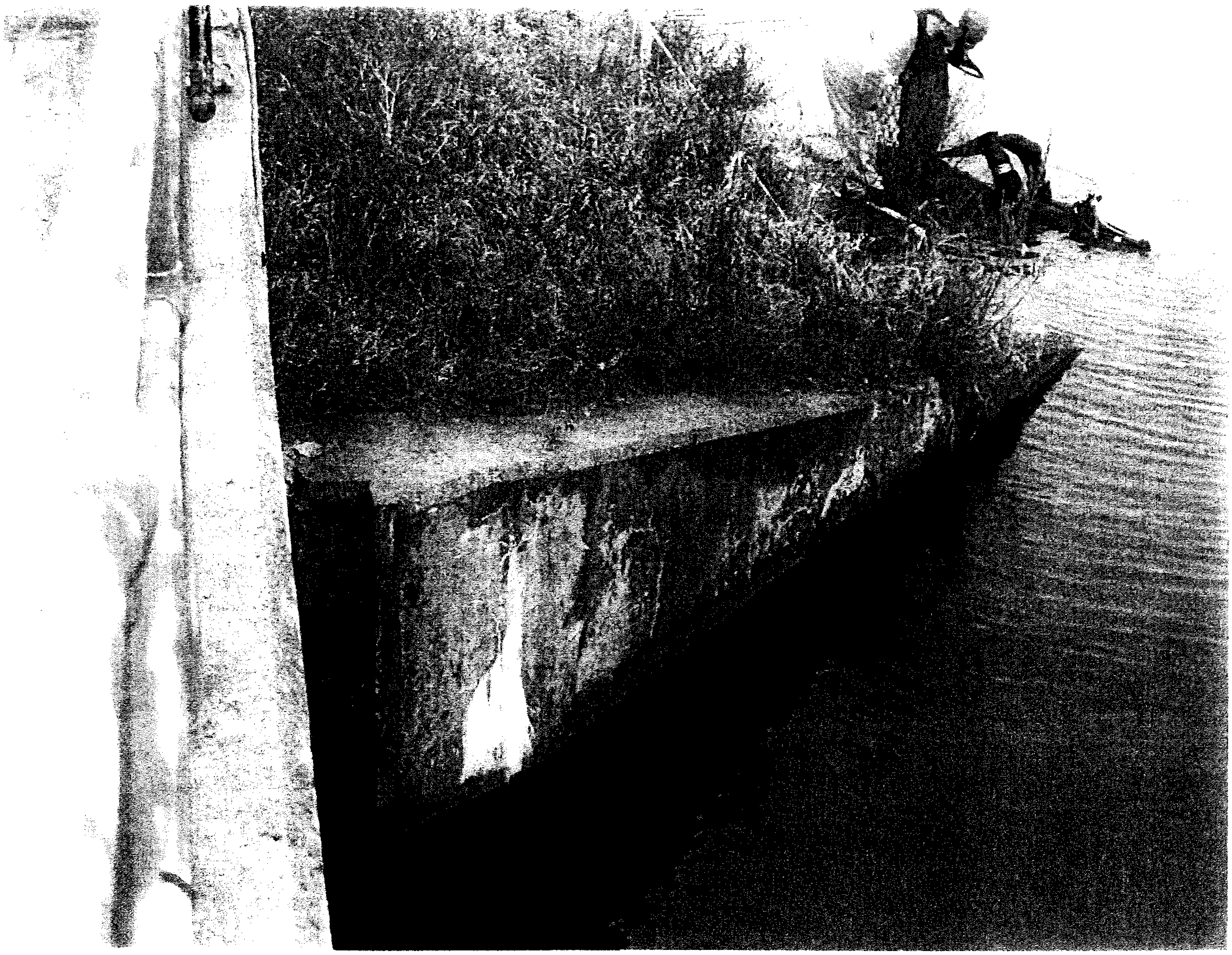
The damaged area of the downstream north wing wall would be repaired using Fabric Formed Grout Filled Mats as a base fore the placement of Fabric Formed Grout Filled Bags. Grout backfill will then be placed in the void area.

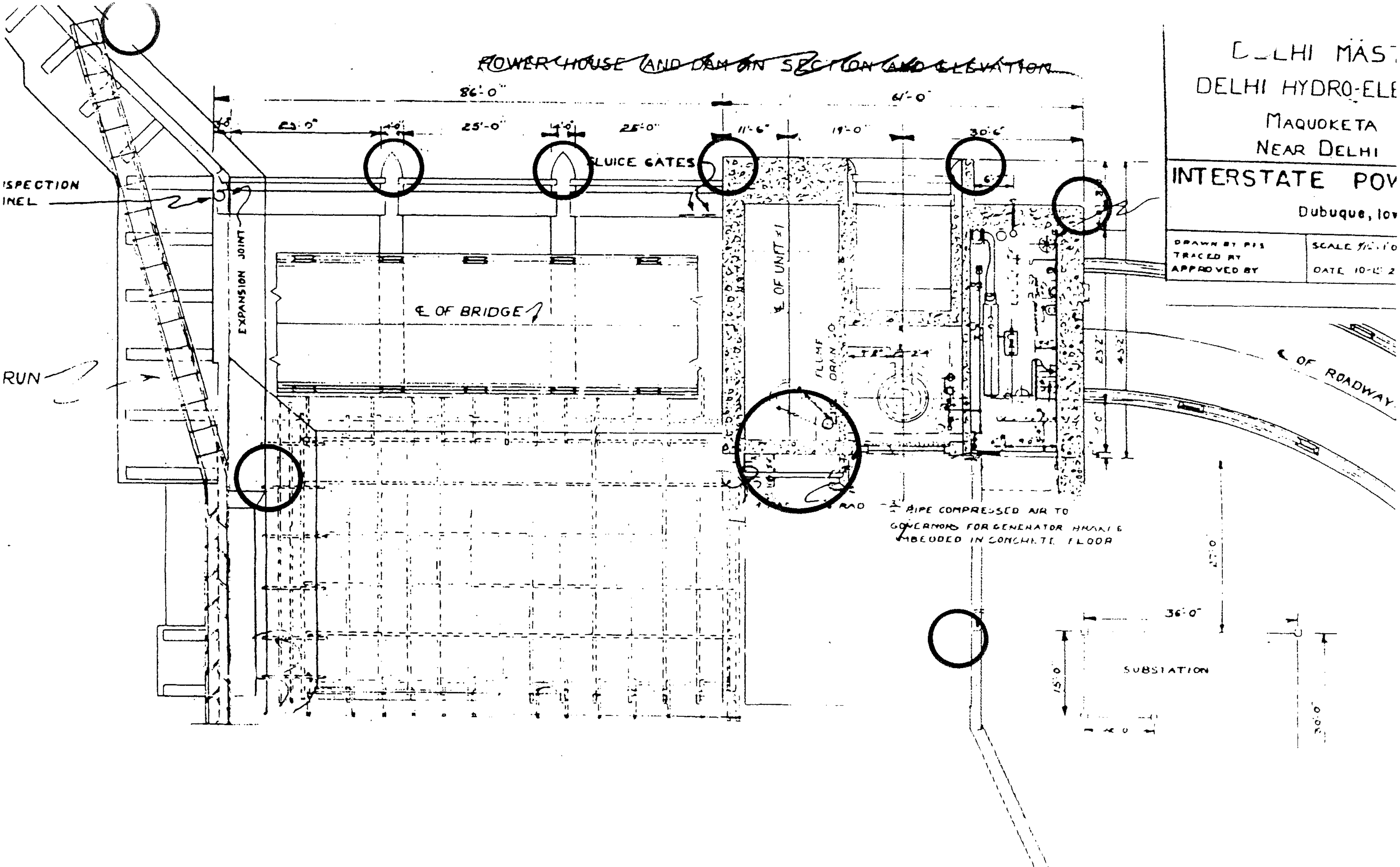
LICENSED PROFESSIONAL ENGINEER ROGER R. MOHN 10165 IOWA	I hereby certify that this engineering document was prepared by me or under my direct personal supervision and that I am a duly licensed Professional Engineer under the laws of the State of Iowa. Roger R. Mohn November 18, 2008 Roger R. Mohn Date My license renewal date is December 31, 2008 Pages or sheets covered by this seal: Page 1 through Page 12



Attachment 'A'

Attachment 'B'

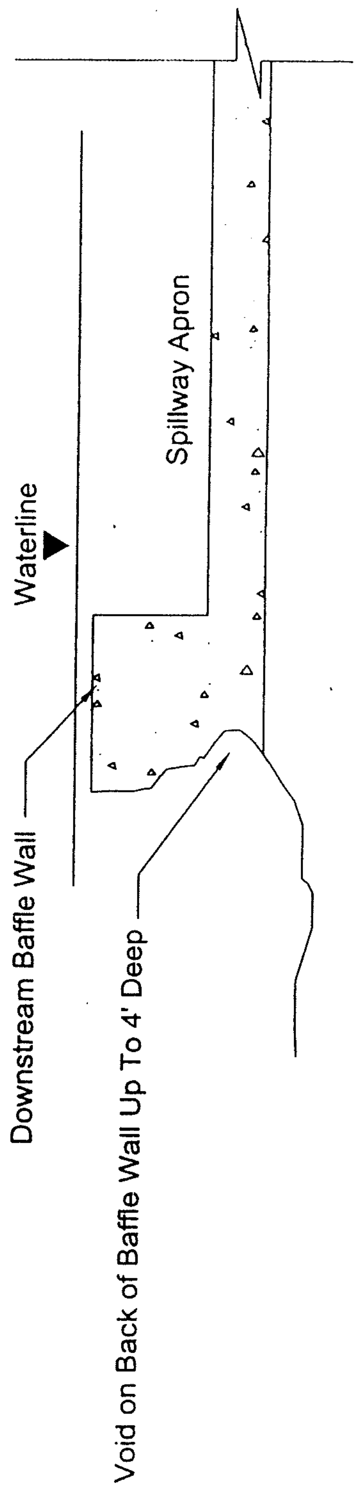




Areas of PC concrete damage

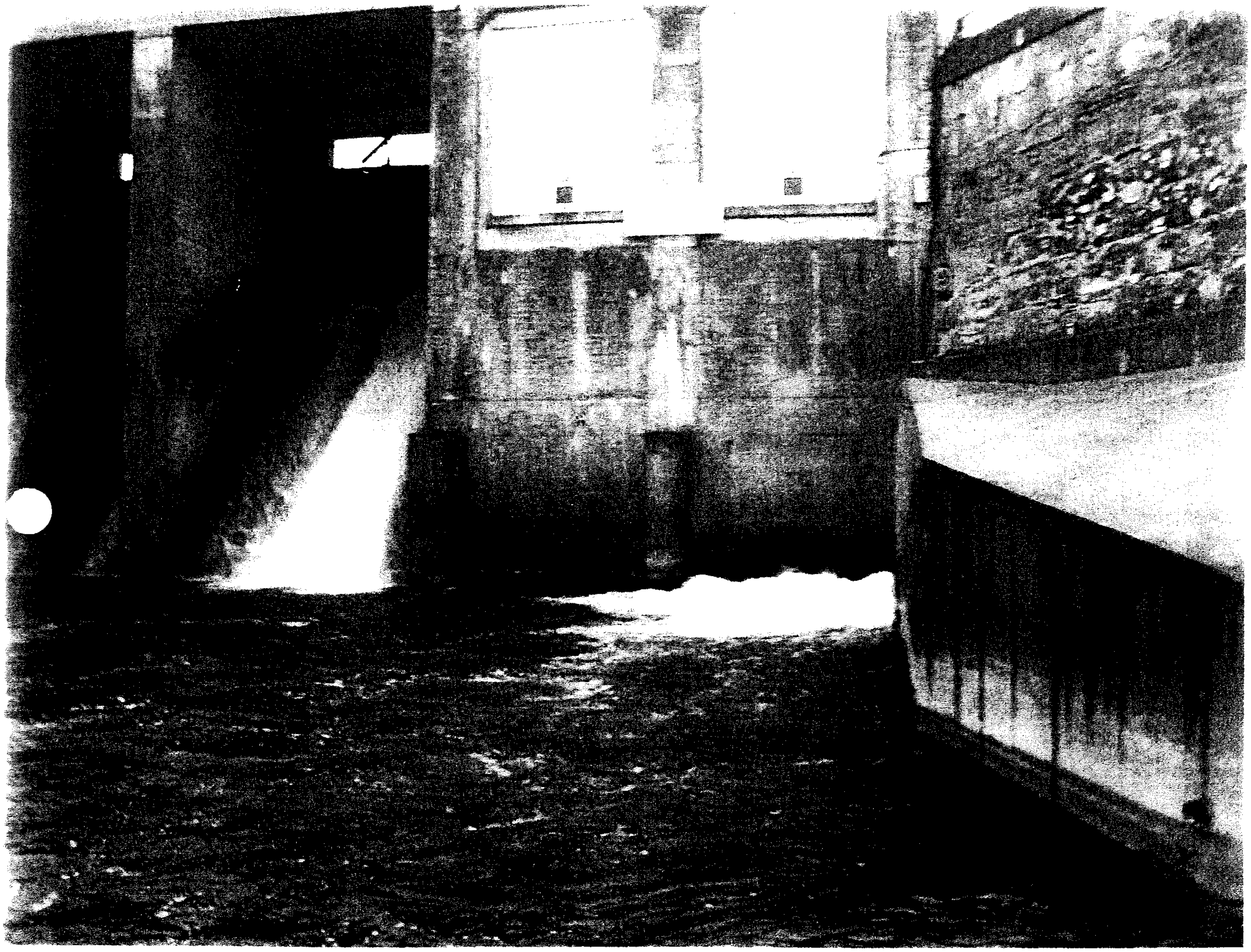
Attachment 'C'

Attachment 'D'



Attachment 'E'

Attachment "F"



Appendix B

Hydrologic and Hydraulic Studies Report

This report summarizes the Hydrologic and Hydraulic Studies completed to analyze and perform preliminary design of alternatives for the reconstruction of Lake Delhi Dam on the Maquoketa River in eastern Iowa. Lake Delhi Dam was breached and failed during a flood on July 24, 2010. The dam in its pre-failure condition did not have sufficient hydraulic capacity to meet current Iowa dam safety criteria. Design of the reconstruction will include significantly increasing Lake Delhi Dam's hydraulic capacity for passing flood flows.

For the alternatives analysis several concepts were developed for reconstructing the dam's spillway(s). Three concepts were taken to preliminary design and evaluated for potential design and construction. Other hydraulic considerations included minimum/low flow passage, lake draining capacity, and cofferdam/bypass during construction. Steps to complete the hydrologic and hydraulic studies for the alternatives analysis included.

- Characterizing Maquoketa River Flows at Lake Delhi Dam
- Developing a hydrologic model of Lake Delhi Dam watershed.
- Developing a hydraulic model of Maquoketa River upstream and downstream of Lake Delhi Dam.
- Performing hazard classification and design flood analysis for Lake Delhi Dam.
- Developing Lake Delhi Dam spillway concepts.
- Addressing other hydraulic issues.

B.1 Maquoketa River Flows

The Maquoketa River is approximately 150 miles long and flows into the Mississippi River. Lake Delhi Dam is located approximately 40 miles downstream from the river's headwaters so is in the upper portion of the river's watershed. The U.S. Geological Survey (USGS) has maintained two gages near Manchester, Iowa at Highway 20 which is approximately 28 miles downstream from the river's headwaters. Gage 05417000 was discontinued in 1973 but has a record of daily flow values from 04/25/33 to 09/30/73. Gage 05416900 is currently in service and has daily flow values from 4/26/00 to the present.

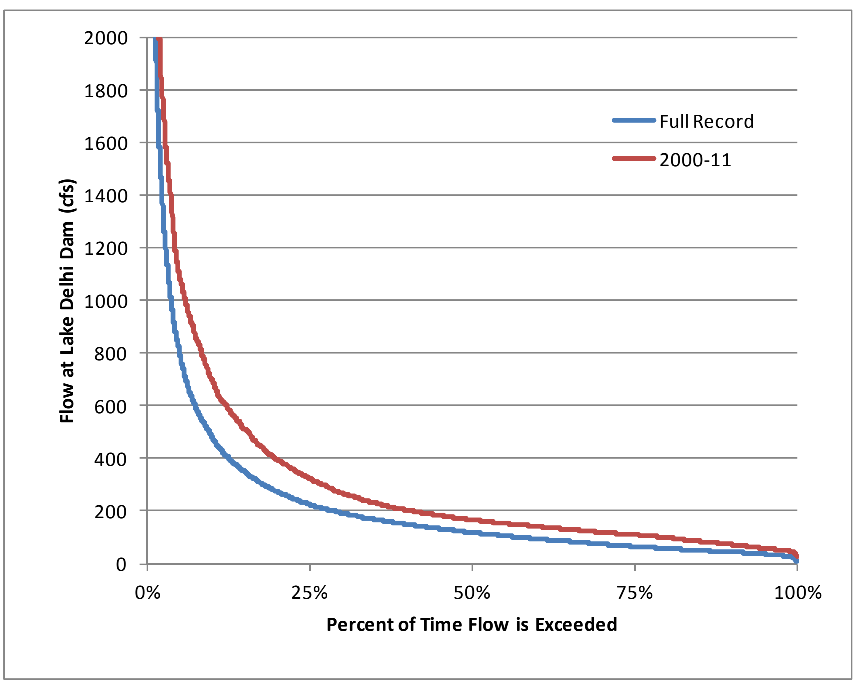
The tributary drainage areas for the Maquoketa River at Highway 20 and at Lake Delhi Dam are 300 square miles and 349 square miles respectively. This means that river flows recorded at the USGS gage are typically going to be smaller than river flows at the dam. *Techniques for Estimating Flood-Frequency Discharges for Streams in Iowa* (USGS, 2001) provides regional empirical equations for translating flows between points on Iowa rivers that have proportional areas. The multiplier for Highway 20 USGS gage flows to Lake Delhi Dam flows was computed to be 1.07. In reality, the difference in flows between the two locations will vary depending on rainfall, groundwater, and snowmelt conditions, but 1.07 provides a reasonable estimate.

Following the 2010 flood, USGS performed a frequency analysis of peak flows at the Highway 20 gages and established new return period discharge estimates. Table B-1 displays the results with Lake Delhi Dam return period flows estimated using the 1.07 multiplier.

Table B-1 Return Period Flows

Return Period (yrs)	Annual Exceedance Probability	USGS Gage Flow (cfs)	Lake Delhi Dam Flow (cfs)
1	0.95	1,393	1,491
2	0.5	4,506	4,821
5	0.2	8,636	9,241
10	0.1	12,300	13,161
25	0.04	18,130	19,399
50	0.02	23,420	25,059
100	0.01	29,610	31,683
200	0.005	36,820	39,397
500	0.002	48,150	51,521

The frequency analysis provides a characterization of peak flood flows for the Maquoketa River. These and larger flow magnitudes were used in designing the hydraulic capacity of the reconstructed Lake Delhi Dam's spillway alternatives. A flow duration analysis provides a characterization of the range of flows that are likely to occur at the dam. For the flow duration analysis the entire 51 year period of daily flow record is (1933-1943, 2000-2011) used to plot a graph showing the percent of time flow a given flow is exceeded over the period of record. The plot is shown in Figure B-1.Similar to the frequency analysis USGS gage flows were adjusted by the 1.07 multiplier.



Flow Duration Curve at Lake Delhi Dam
Figure B-1

The 50% value on the X-axis represents the average flow at the dam. In addition to the full gage record, a depiction of flows over the last 11 years was developed. As shown on the graph, Maquoketa river flows over the last 11 years have been higher relative to the gage record. The average flow for the full gage record is 118 cubic feet per second (cfs) and for the last 11 years the average flow was 167 cfs.

B.2 Hydrologic Model

The hydrologic model was used to develop a series of design flood hydrographs (i.e. analysis derived) for the Lake Delhi Dam watershed. The flood hydrographs were used as an input for the hydraulic model

The watershed area was obtained from the Iowa Department of Natural Resources (DNR) which provided an ArcGIS shapefile of the delineation of the dam's tributary watershed. This delineation was checked against USGS topography and hydrologic unit code (HUC) maps and matched very closely.

The infiltration rate was estimated using ArcGIS mapping software. Hydrologic soil group data for the watershed area was obtained from the Natural Resources Conservation Service (NRCS) Soil Survey Geographic Database. The NRCS has established typical infiltration rates for given hydrologic soil groups. As is typical for flood modeling, initial infiltration losses were ignored and only a constant infiltration rate was used.

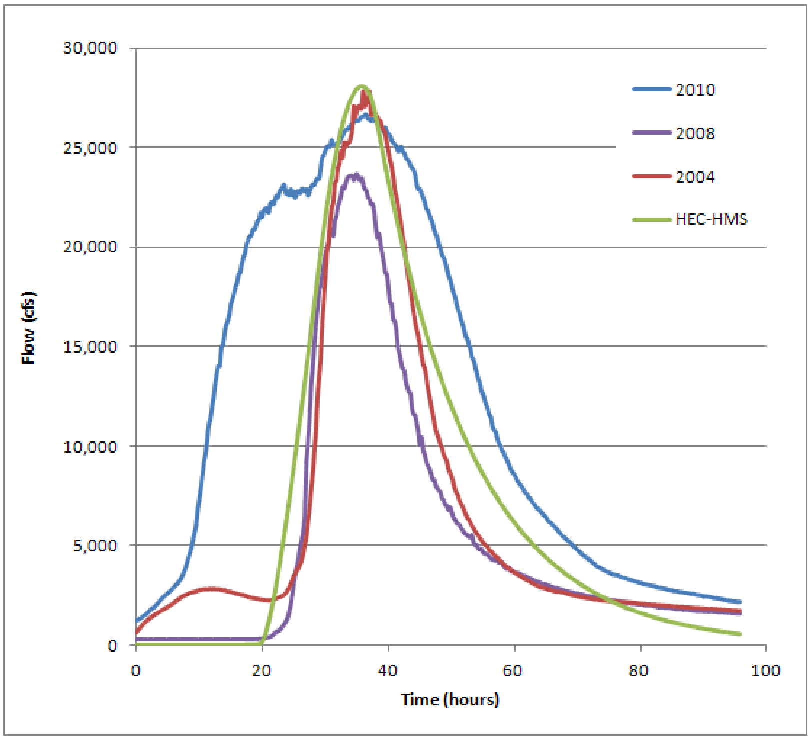
Time of concentration was estimated by measuring the length of the Maquoketa River from the headwaters to the upstream end of Lake Delhi, estimating an appropriate river velocity and computing the travel time. A river velocity of 3feet per second was assumed which matched the velocities computed by the hydraulic model of the Maquoketa River closely.

The storage coefficient was established using flood hydrographs recorded at the USGS stream gage (05416900) located at the Highway 20 Bridge near the upstream end of Lake Delhi. Flow data for the 2004, 2008, and 2010 floods were obtained from the USGS gage and plotted. The storage coefficient was computed from the slope of the descending limb of the flood hydrograph. Each recorded flood had a slightly different storage coefficient so a representative coefficient was estimated.

The rainfall events modeled in HEC-HMS were a 100-year storm and a Probable Maximum Precipitation (PMP). The 100-year/24-hour rainfall for the Lake Delhi area was obtained from *Iowa Rainfall Frequencies* (Waite, 1988). The PMP was established using National Oceanic and Atmospheric Administration's (HMR 51 and HMR 52) guidelines. A 72-hour storm duration provided the probable maximum storm, with the bulk of the rainfall falling within 6 hours on the second day.

The watershed parameters and rainfall events were input into HEC-HMS and the flood hydrographs were computed. The peak HEC-HMS derived 100-year flow was checked against the USGS gage 2004, 2008, and 2010 hydrographs as well as the 100-year flow established at the gage and the flows matched closely. The HEC-HMS derived 100-year flow used the 100-year rainfall from the DNR recommended *Iowa Precipitation Frequencies* (Waite, 1988). Similar to the flow frequency analysis which has increased return period flood flows after incorporating recent floods, the 100-year rainfall value would likely increase with an updated rainfall frequency analysis. So the HEC-HMS 100-year hydrograph could more closely match the updated the 100-year peak flow from the USGS flood frequency analysis with an updated 100-year rainfall.

Figure B-2 displays the hydrograph comparison. USGS gage flow hydrographs were adjusted by the 1.07 multiplier. Watershed parameters, rainfall depths, and peak flood flows are provided in Table B-2.



Flood and HEC-HMS Hydrograph Comparison
Figure B-2

Table B-2 Lake Delhi Dam Watershed Parameters

Parameter	Value
Drainage Area (mi^2)	349
Infiltration (in/hr)	0.25
Time of Concentration (hrs)	18
Storage Coefficient (hrs)	15
PMF Rainfall Total (in)	25.8
PMF Peak Flow (cfs)	143,900
100-Year Rainfall Total (in)	6.4
100-Year Peak Flow (cfs)	28,100

B.3 Hydraulic Model

The starting point for the hydraulic modeling was the HEC-RAS model of the Maquoketa River developed by the DNR to evaluate the 2010 breach of Lake Delhi Dam. The upstream end of the river model is at the Highway 20 Bridge and the model extends approximately 23 miles to just downstream of Hopkinton.

HEC-RAS software was developed by the U.S. Army Corps of Engineers and uses a series of river cross-sections and structures to model a constant flow (steady) or hydrograph (unsteady) through the river reach. All hydraulic modeling performed for the alternatives analysis used unsteady flow modeling which allowed the flood and dam breach hydrographs to be routed through the reservoir, dam, and river system.

The HEC-RAS model of the Maquoketa River as well as supporting background data was provided to Stanley Consultants by the DNR. The DNR HEC-RAS model was created using HEC-GeoRAS, which is an ArcGIS interface with HEC-RAS that allows the model elements to be geographically based and provides improved flood mapping capabilities over the stand-alone HEC-RAS software. The following adjustments were made to the DNR HEC-RAS model:

- River channel topography was updated with post-breach LiDAR data obtained in fall 2010.
- HEC-RAS river cross-sections were extended up to elevations where the design floods were contained within the cross-section.
- Bridge structures were added downstream of the dam (Quarter Road, 295th Street and Hopkinton) using construction drawings provided by Delaware County.
- One inflow hydrograph was used at the upstream end of the model (DNR model used two).
- The dam was modified to reflect the proposed condition (working gates, principal/auxiliary spillway).

The DNR HEC-RAS model was correlated to the 2010 flood using both the approximate time and elevation of high water marks (peak flood levels) at the Lake Delhi Dam and several bridges on the Maquoketa River. The DNR HEC-RAS model provided a good replication of the 2010 flood event. The 2010 flood hydrograph was run through the adjusted model (with the existing dam) and the resulting flood profile did not change from the original DNR model so the adjusted model is also thought to provide a good representation of the Maquoketa River.

B.4 Hazard Classification

Hydrologic and Hydraulic analysis and design standards for dams in Iowa are specified in *Technical Bulletin 16 - Design Criteria and Guidelines for Iowa Dams* (DNR, 1990). The standards are defined according to the dam's hazard classification. The state of Iowa has three hazard classifications for dams; Low, Moderate, and High Hazard.

If hydropower is ever redeveloped at Lake Delhi Dam, the reconstructed dam will have to meet FERC criteria. FERC also has three hazard classifications; Low, Significant, and High Hazard. The FERC and DNR hazard classification definitions are very similar so the classification determined by DNR criteria should correspond to a FERC hazard classification. Table B-3 provides the agency hazard classification definitions.

Table B-3 Hazard Classification Definitions

Hazard Class	DNR Definition	FERC Definition
Low	Structures located in areas where damages from a failure would be limited to loss of the dam, loss of livestock, damages to farm outbuildings, agricultural lands, and lesser-used roads, and where loss of human life is considered unlikely.	Structures located in rural or agricultural areas where failure may damage farm buildings, limited agricultural land, or township and country roads. Low hazard potential dams have a small storage capacity, the release of which would be confined to the river channel in the event of a failure and therefore would represent no danger to human life.
Moderate/ Significant	Structures located in areas where failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life.	Structures located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life.
High	Structures located in areas where failure may create a serious threat of loss of human life or result in serious damage to residential, industrial or commercial areas, important public utilities, public buildings, or major transportation facilities.	Structures located where failure may cause serious damage to homes, agricultural, industrial and commercial facilities, important public utilities, main highways, or railroads, and there would be danger to human life.

The hazard classification of Lake Delhi Dam controls several design parameters including the freeboard design flood. For detailed design to proceed, a hazard classification is needed to establish the applicable dam safety and design criteria.

Previous inspections and analyses have identified Lake Delhi Dam as a low, moderate, and high hazard structure but there has not been a detailed analysis of potential downstream hazard to substantiate the hazard classification. The hazard classification analysis performed for this study provides a more thorough evaluation of risk associated with theoretical dam failure through inundation mapping of a series of flood events with and without dam failure.

The hazard classification analysis consisted of the following steps:

- HEC-HMS generated flood hydrographs (events) for the Lake Delhi Dam were input into the HEC-RAS model of the Maquoketa River.
- The Lake Delhi Dam in the HEC-RAS model was adjusted to reflect the reconstructed condition.
- Dam breach (failure) parameters were developed for Lake Delhi Dam.
- The HEC-RAS model was run under no breach and breach scenarios for a series of HEC-HMS generated flood events.
- Flood profiles (maximum water surface) from the HEC-RAS model were compared and evaluated for the various flood/breach scenarios.
- Flood profiles were exported to ArcGIS and inundation maps created.
- Impacted structures and roadways were tabulated and compared for the no breach and breach scenarios.
- A hazard classification for Lake Delhi Dam was recommended.

HEC-HMS Hydrographs

HEC-HMS derived flood hydrographs were input into the HEC-RAS model. Initially, the full PMF, ½ PMF, and 100-year flood were modeled in HEC-RAS with and without a dam breach. Subsequent flood events modeled included the $^1/_3$, $^2/_3$ and ¾ PMF as well as a "sunny day" event which is a dam failure that occurs during normal flow conditions.

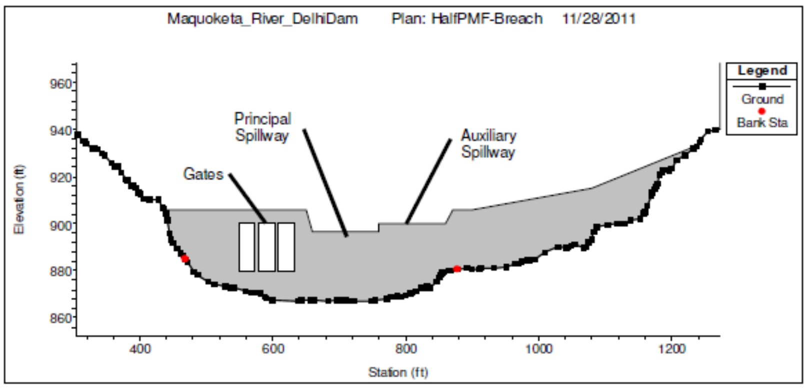
Reconstructed Dam

Discharge from Lake Delhi Dam in the pre-breach condition was provided by two wicket gates and three 25-foot wide by 17-foot high lift gates. When the hydropower facility was in operation, the wicket gates were used to control flow through the turbines. Hydropower generation at the dam was deactivated in 1973, but the wicket gates continued to be used for passing normal flows and maintaining pool elevation. The hydraulic capacity of the two wicket gates was estimated to be roughly 600 cubic feet per second (cfs). The lift gates were used for passing flows that exceeded the capacity of the wicket gates. The gates were difficult to open and close and one of the gate guides cracked during attempted operation during the 2010 flood, contributing to failure of the dam.

The reconstructed dam will have working gates and a new principal/auxiliary spillway to increase hydraulic capacity. The future of hydropower generation and use of the wicket gates is uncertain so wicket gate discharge was excluded from the hydraulic analysis. Design of the Lake Delhi Dam reconstruction is in the preliminary stage, so the "reconstructed" dam in the HEC-RAS model represents an approximation.

Gates will be replaced as part of the reconstruction so they were assumed to be fully operable in the HEC-RAS model. Prior to the breach fabrication plans had been developed for providing a new gate lifting mechanism that would have allowed the gates to be lifted 20-feet above the spillway crest so the gate openings in HEC-RAS were adjusted to reflect a 25-foot wide by 20-foot high opening.

A principal/auxiliary spillway will be added to the dam as part of the reconstruction. The exact dimensions of the spillway have not been established yet. What is known is that the spillway will fit within the roughly 230 foot long southern embankment, the principal spillway crest would likely be at normal pool, and the auxiliary spillway crest would be a few feet above normal pool. To reflect the reconstructed condition a 100-foot long principal spillway with a crest elevation of 896.5 ft-msl and a 100-foot long auxiliary spillway with a crest elevation of 900 ft-msl were added to the HEC-RAS model. The HEC-RAS spillway concept is shown in Figure B-3.



Normal Pool	896.3
Top of Dam	906
Gates	
Gate Crest	879.8
Gate Height	20 ft
Gate Width	25 ft
Principal Spillway	
Spillway Crest	896.5
Spillway Width	100 ft
Weir Coeff.	3.4
Auxiliary Spillway	
Spillway Crest	900
Spillway Width	100 ft
Weir Coeff.	3.4

HEC-RAS Lake Delhi Dam Spillway Concept
Figure B-3

Hazard classification is focused more on the downstream impact of the dam than the specifics of the spillway so using a principal/auxiliary spillway approximation is reasonable for this analysis. The objective is to represent the influence of the spillways on the dam failure and downstream flooding. In this case, the principal and auxiliary spillways increase the amount of flow in the downstream channel for a given Lake Delhi pool elevation and they influence the geometry of the theoretical breach because the spillways will likely be armored (concrete). The various spillway options currently being considered have a similar embankment shape so the proposed HEC-RAS model should provide an adequate depiction of the failure condition no matter which alternative is chosen. However, the analysis will be updated once the reconstruction design is established, but a significant change in results is not expected.

Dam Breach Parameters

The DNR established dam breach parameters for their original HEC-RAS model based upon the Lake Delhi Dam failure of the southern embankment observed in 2010. For the reconstructed dam analysis, the width of the dam breach was reduced from 250 feet to 175 feet to better reflect the reconstructed condition on the southern embankment. The breach formation time was left at 1.5 hours. The failure was set to initiate at the peak of the flood hydrograph which yields the highest flood elevation (i.e. worst-case condition).

The DNR established breach parameters were checked against breach parameters estimated using three empirical methods. The three empirical methods were:

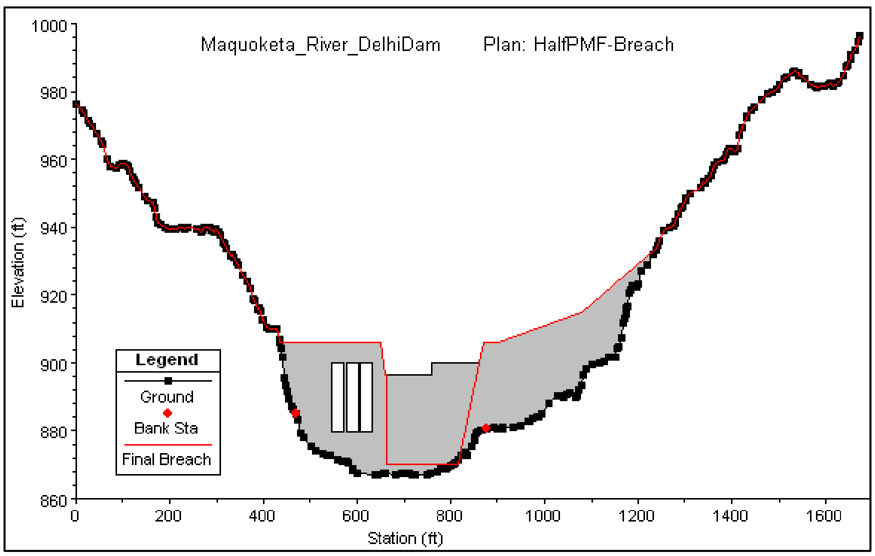
- MacDonald and Langridge-Monopolis – Uses volume of reservoir and height of dam to compute breach parameters.
- FERC – Uses type of dam and height of dam to estimate range of breach parameters.
- Froehlich – Uses type of failure, volume of reservoir and height of dam to compute breach parameters.

The results of the breach parameter computations are shown in Table B-4.

Table B-4 Lake Delhi Breach Parameters

	Method			
Breach Parameter	**M&L-M**	**FERC**	**Froelich**	**Used in HEC-RAS**
Volume Eroded (ft^3)	66781			
Breach Width (ft)	343.4	120	235	175
Side Slopes (h/v)	2	0.5	1	0.1, 1.5
Time to Fail (hrs)	0.9	0.75	1.8	1.5

The empirical computations provided a range of potential breach values. The values used in the HEC-RAS model fell within the range of empirical predictions and were similar to the breach that occurred in 2010 so were considered acceptable. Figure B-4 shows the breach geometry.



HEC-RAS Lake Delhi Dam Breach Limits
Figure B-4

Failure of the existing powerhouse and gated spillways were also considered. The two most likely failure scenarios for these large concrete structures would be tipping/sliding of the structure or undermining of the foundation leading to collapse. Both of these failure scenarios were modeled in HEC-RAS for each structure. The failure time was reduced to 0.5 hours and the geometry of the failure was set to match the extents of the structure being analyzed. Out of the four failure scenarios, tipping/sliding of the gated spillway provided the largest flood wave downstream, but it was not as large as the flood wave created by the failure of the reconstructed southern embankment, so the embankment failure was used as the failure condition for the hazard classification.

HEC-RAS Modeling

Flood events were modeled in HEC-RAS for both failure and non-failure conditions. The dam was assumed to be operating under normal conditions (normal pool of 896.3 ft-msl) prior to the flood. HEC-RAS generates stage (elevation of water surface) and flow hydrographs at each cross-section location in the model. The maximum stages at each cross-section are linked to develop a continuous flood profile of the river segment which is used in creating of inundation (flood) extents along the river channel and surrounding area for a given flood event.

The flood profiles generated by HEC-RAS indicated that the Quarter Road Bridge causes a significant backwater effect for the full and ½ PMF. To analyze the impact of a potential failure of the bridge on flood conditions a HEC-RAS model was created with the Quarter Road Bridge structure removed. Flood scenarios were compared in HEC-RAS with and without the Quarter Road Bridge. Results indicate that Quarter Road Bridge raises flood elevations upstream of the bridge by up to 0.8 feet. Removal of the bridge has minimal impact on the flood elevation or travel time of the floodwave downstream, so this suggests the bridge should not be a significant factor in downstream hazard potential.

Upstream and downstream impacts to flows and water surface elevation were also evaluated. This was analyzed by comparing HEC-RAS models of the pre-breach and reconstructed dam for a series of floods. The HEC-RAS model results show that increasing the dam's hydraulic capacity will reduce upstream pool elevations during a flood event with minimal impact to downstream flood elevations. The analysis will be revisited during detailed design to verify the reconstructed dam will not adversely impact upstream or downstream properties compared to the pre-breach condition.

Inundation Maps

The HEC-RAS flood profiles were exported to ArcGIS using HEC-GeoRAS which uses the profiles to develop inundation extents for each flood/failure event. Inundation maps were created that include geo-referenced aerial imagery so the inundation limits can be viewed relative to downstream buildings and infrastructure. Inundation maps are provided in this Appendix.

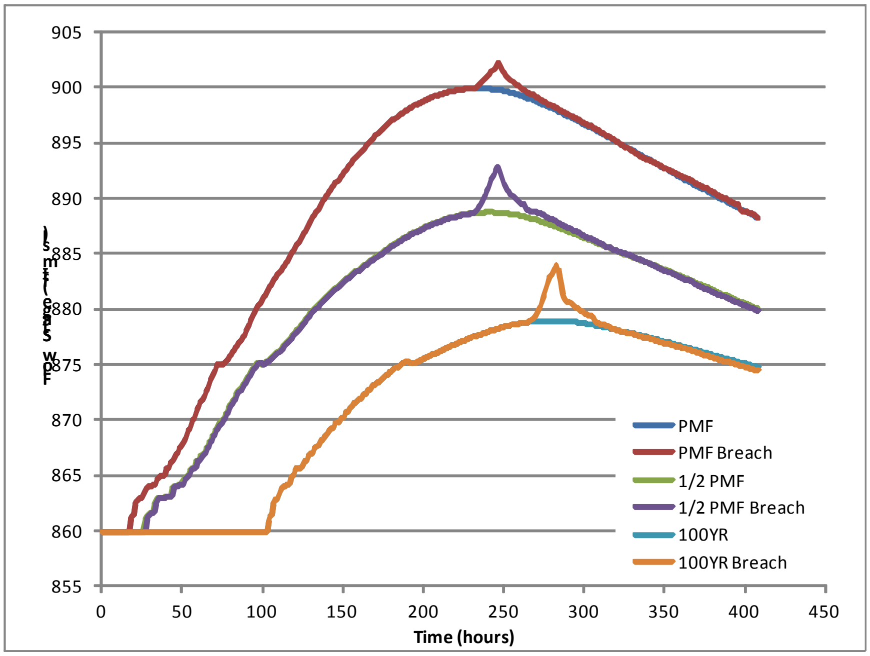
Tabulation of Impacted Structures

The inundation limits were compared for failure and non-failure conditions for the full PMF, ½ PMF, 100-year flood, and sunny day (dam failure during normal flow condition) and buildings and infrastructure inside the inundation limits were tabulated between each set of HEC-RAS cross-sections. The detailed inundation table is provided in this Appendix. A summary of the number of structures impacted is provided in Table B-5.

Table B-5 Impacted Structure Summary

Event	Scenario	Residential	Comm/Ag	Bridges	Roads
PMF	No Breach	104	30	3	12
	Breach	107	30	3	12
Half PMF	No Breach	27	8	3	8
	Breach	29	8	3	8
100-YR	No Breach	3	1	1	5
	Breach	5	2	1	5
Sunny Day	Breach	0	0	0	1

Due to their location and lack of potential warning time, the homes directly downstream of the dam were examined in closer detail for the theoretical failure event. Because of they are located so close to the dam, the nearby downstream properties see the greatest potential increase in flood level due to dam failure. The critical factors for hazard potential are the number of additional properties impacted and the increase in flood level due to breach. The ½ PMF appears to provide the greatest increase in hazard potential from breach due to the fact it raises flood levels by over 4 feet just downstream of the dam. The full PMF inundates more homes but its relative increase in flood level due to breach is 2 feet less than the ½ PMF. The 100-year flood has a greater rise (5 feet) but it impacts fewer buildings and roadways. Exhibits and tables providing an inventory of individual downstream properties impacted are provided in this Appendix. A hydrograph plot comparing the increase in stage (water surface) elevation just downstream of the dam for breach and no breach scenarios is provided in Figure B-5.



HEC-RAS Flood Stage Just Downstream of Lake Delhi Dam
Figure B-5

Hazard Classification

Hazard classification is based on the potential consequence of dam failure. When analyzing the consequences of dam failure during a flood event it is the increase in consequence (i.e. increase in damage and potential loss of life) due to failure that is evaluated. Inundation maps communicate the extent of downstream area that could be impacted by the given flood with and without dam failure.

Results of the HEC-RAS modeling and inundation mapping indicate that dam failure during flood events does not appear to cause a significant increase in the number of structures inundated. The majority of additional structures that are inundated by a failure event are the homes within 1500 feet downstream of the dam. As the Emergency Action Plan is developed for the reconstructed condition it will be important to have well-defined communication and evacuation procedures defined for these residents.

Hazard classification was reviewed for both the DNR and Federal Energy Regulatory Commission (FERC) definitions. Lake Delhi Dam appears to fit the Moderate (DNR), Significant (FERC) Hazard Classification. The reasoning is as follows:

- HEC-RAS modeling and inundation mapping show that a potential failure during a flood would only cause a small increase in the number of structures impacted.
- A potential sunny day failure conditions stays within the limits of the 100-year floodplain (typically non-developed area) so the potential for damage is less than if sunny day failure flooded more habitable or developable lands.

- Much of the area downstream of Lake Delhi Dam is rural and agricultural. Although future development is possible, most development would likely occur closer to the town of Delhi, which is up above the river channel or in Hopkinton which is far enough downstream that the increase in flood elevation due to failure is roughly 1 foot.
- The Maquoketa River downstream of Lake Delhi Dam is widely used for canoeing and fishing activities, however the river does not contain the type of attractions that bring large numbers of people into the river channel area for extended periods of time (i.e. restaurants, resorts, large campgrounds or trailer parks, etc.)
- Therefore, the DNR definition of Moderate hazard where "…failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life." is appropriate for the reconstructed Lake Delhi Dam.
- The FERC definition of Significant hazard for "Structures located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways, or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life." Also seems the appropriate classification for Lake Delhi Dam.

Design Flood

Per *Technical Bulletin 16 - Design Criteria and Guidelines for Iowa Dams* (DNR, 1990), a moderate hazard classification establishes the freeboard design flood as the ½ PMF. FERC uses an incremental analysis to establish the design flood by determining the largest food where failure causes an increase in downstream hazard. The FERC method was analyzed by adding the ¾ PMF, the $^2/_3$ PMF, and $^1/_3$ PMF scenarios to the HEC-RAS model.

FERC recommends using a two-foot increase in flood elevation due to failure as the minimum threshold where hazard potential is increased by dam failure. For all flood events at Lake Delhi Dam, the increase in flood elevation due to failure is greater than two feet just downstream of the dam, but then decreases to less than two feet downstream of the Quarter Road Bridge (located roughly 4 miles downstream of the dam). Besides the immediate homes at the dam, there are no buildings inundated by any of the flood events until the flood is past the Quarter Road Bridge.

The flood that has the greatest overall increase flood elevation due to failure is the 100-year flood. The $^1/_3$ PMF and ½ PMF have a comparable rise in flood elevation due to failure relative to the 100-year flood. For floods greater than the ½ PMF (Full PMF, ¾ PMF and $^2/_3$ PMF) the increase in flood elevation due to failure is discernibly less than the floods greater than ½ PMF, $^1/_3$ PMF and the 100 Year Flood. So given that the greatest increases in flood elevation due to failure are from floods of a lesser magnitude than the ½ PMF, using the DNR designated ½ PMF as the freeboard design flood should also meet FERC criteria.

Recommendation for Final Design

Based on the analysis Stanley Consultants recommends that design of the Lake Delhi Dam reconstruction proceed with a classification as a Moderate Hazard structure and a freeboard design flood of the ½ PMF. This classification will be verified with an updated analysis once reconstruction design has been established.

A detailed summary of the computations performed for the hazard classification is provided with this Appendix.

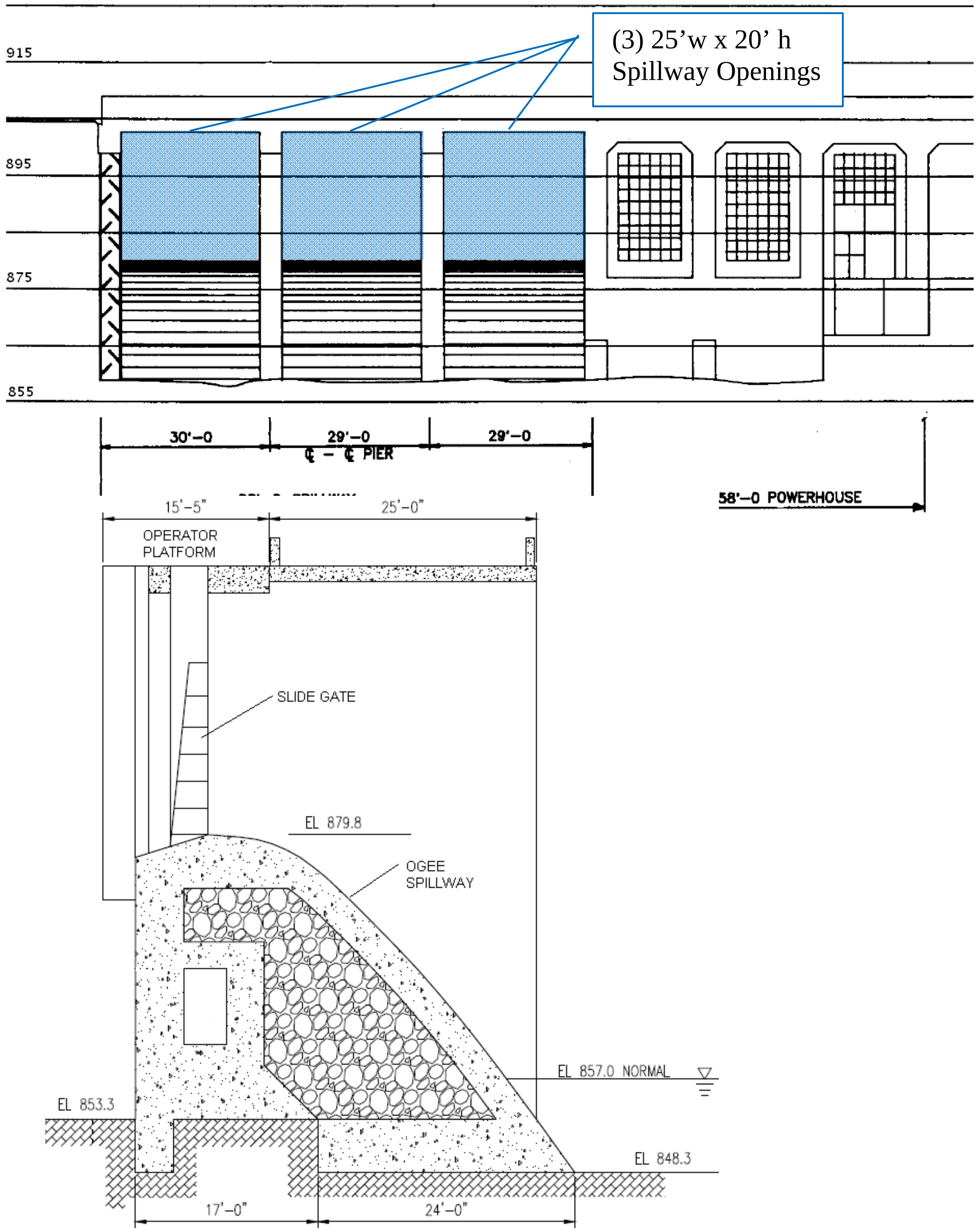
B.5 Spillway Concepts

Using the ½ PMF as the design flood, spillway concepts were developed with the objective of the reconstructed Lake Delhi Dam being able to pass the ½ PMF without overtopping the existing powerhouse/gated spillway structure.

Prior to the breach, river flows exceeding 600 cfs were passed by opening the three spillway gates located adjacent to the powerhouse. If working properly, the gates could raise roughly 17 feet, which provided Lake Delhi Dam with a maximum discharge capacity of approximately 28,000 cfs prior to the dam overtopping. The peak flow of the ½ PMF for the Lake Delhi Dam watershed is close to 72,000 cfs. Using HEC-RAS to route the flood through the reservoir shows that to pass a ½ PMF without overtopping the powerhouse the reconstructed Lake Delhi Dam will need to pass roughly 69,000 cfs through its spillway(s) which is more than double the hydraulic capacity of the pre-breach dam.

Spillway Gates

The existing Lake Delhi Dam spillway is located next to the powerhouse structure. It consists of an ogee spillway with a crest elevation of 879.8 ft-msl. Above the ogee spillway are three 25-foot wide by 20-foot high openings. Flow through the openings was controlled by three lift gates, which were hoisted from an operator platform above the gates. An elevation and section view of the gates are shown in Figure B-6.



Spillway Gates
Figure B-6

Several gate options were considered for the dam reconstruction; however, the pier and bridge configuration above the spillway is not conducive to different gate systems. The options considered are shown in Table B-6.

Table B-6 Spillway Gate System Comparison

Option	Suitable	Explanation
Radial Gates	No	Radial gates are mounted on an arm and are lifted by rotating the arm upwards so have a circular motion. Installing radial gates at the existing spillway would require removing a significant portion of the bridge deck.
Crest Gate	No	Crest gates are mounted to the crest of the spillway and when lowered are flush with the crest. The ogee spillway at Lake Delhi Dam is steep and does not have a wide crest, so installation of crest gates would require removal of large portion of the crest to create a platform for mounting the gates.
Lift Gate	Yes	The existing spillway used lift gates so the configuration is suitable for lift gate installation. The gate guides were damaged and need replacement but that repair would be minor compared to the work required to install other gate systems.

Prior to the 2010 dam failure a project was underway to replace the lift gate hoisting mechanism. The hoisting equipment was received by the dam operator but never installed at the dam so could be installed as part of the reconstruction project. The new hoisting equipment should eliminate previous issues experienced with lifting gates and provides an additional 3 feet of lifting height, so the new gate openings will be 25 feet wide by 20 feet high when the gates are fully lifted.

The new lift gate system will have a hydraulic capacity of roughly 30,000 cfs (40% of design flood) with the gates fully raised and the upstream pool at the top of dam. The wicket gates located on the upstream side of the powerhouse could provide an additional 600 cfs of capacity but their future use is uncertain so they were not included in the spillway analysis.

Potential Spillway Options

The new spillway system at Lake Delhi Dam will need to provide roughly 39,000 cfs of additional hydraulic capacity for the dam to pass the design flood of ½ PMF without overtopping the powerhouse or spillway gate structure. There is roughly 230 feet between the buttress wall at the southern end of the existing powerhouse/spillway structure and the southern riverbank where the new dam will tie into existing ground. With this length, a straight, fixed crest at the normal pool elevation of 899.6 ft-msl could pass approximately 13,500 cfs prior to the powerhouse/spillway structure being overtopped. This is less than half of the hydraulic capacity needed so a more hydraulically effective spillway discharge system will be needed at Lake Delhi Dam. Several spillway systems were reviewed for the alternatives analysis. A summary is provided in Table B-7.

Table B-7 Spillway Option Comparison

Option	Suitable	Explanation
Fuse Plug	No	A fuse plug spillway consists of an earthen embankment overlaying a concrete spillway set several feet below the top of embankment. When the pool reaches the top embankment the earth is eroded away, exposing the concrete spillway. At Lake Delhi Dam, the concrete spillway could not be set low enough to provide sufficient hydraulic capacity.
Additional Lift Gates	No	Additional lift gates would require construction of a new section of a tall concrete ogee spillway structure to essentially extend the existing spillway structure. However, bedrock drops away in this area so in addition to the additional cost of purchasing gates and hoisting equipment, the new concrete ogee spillway and operating platform would be founded on sand which would require expensive stability enhancements to make construction viable.
Pipes Through Embankment	No	In addition to concerns over seepage and maintenance, installing pipes through the dam embankment would not provide sufficient capacity and would require construction of a new intake and operating structure.
Labyrinth Weir	Yes	A labyrinth weir consists of a sharp-crested (vertical wall) in a zigzag pattern that allows a much longer crest length to fit within a shorter length of embankment. The longer crest length significantly increases the hydraulic capacity over a straight weir section. A labyrinth weir is a viable option for meeting hydraulic capacity requirements.
Pneumatic Crest Gates	Yes	Pneumatic crest gates would be installed on top of a new concrete spillway. They would consist of slightly curved, bottom mounted gate panels that could be lowered to be flush with the crest of the new spillway. In their raised position they would be at or just above the normal pool elevation of 896.3 ft-msl, but when lowered could provide an additional 5 to 10 feet of depth for discharging flood magnitude flows. Pneumatic crest gates are also a viable option for meeting hydraulic capacity requirements.

Spillway Alternatives

From the initial review of spillway options, three spillway alternatives were developed for preliminary design and comparison. The three spillway alternatives are:

- **Dual Labyrinth Spillway –** consisting of a lower principal labyrinth weir spillway set at the normal pool to discharge normal flows and a higher auxiliary labyrinth weir spillway set several feet above normal pool to discharge the required flood magnitude flows.
- **Single Labyrinth Spillway** – consisting of a single labyrinth spillway set a normal pool to discharge normal flows but with sufficient hydraulic capacity to also discharge the required flood magnitude flows.
- **Pneumatic Gate Spillway** – consisting of a pneumatic gate system set at normal pool when raised to discharge normal flows and when lowered provides sufficient hydraulic capacity to discharge the required flood magnitude flows

Exhibits showing plans and sections of the spillway alternatives are provided in Appendix F. All spillway alternatives consist of a concrete spillway slab and chute constructed over an earthen embankment with a concrete stilling basin at the end. All spillway alternatives were sized so with the three existing lift gates and the new spillway, the reconstructed dam could pass the ½ PMF without overtopping the powerhouse/spillway structure.

Spillway Hydraulics

In performing preliminary design of the three spillway alternatives, analysis of spillway hydraulics was used to develop stage-discharge curves for each alternative.

Labyrinth weir hydraulics has been studied in detail so it is possible to predict the discharge rating for a given geometry with reasonable accuracy. *Hydraulic Design of Labyrinth Weirs* (Falvey, 2003) was utilized for developing the geometry and estimating the discharge capacity of the labyrinth weir alternatives. Empirical equations have been developed that predict flow for a given weir geometry and depth of flow going over the weir.

The major factors in the hydraulic design of the labyrinth weir are:

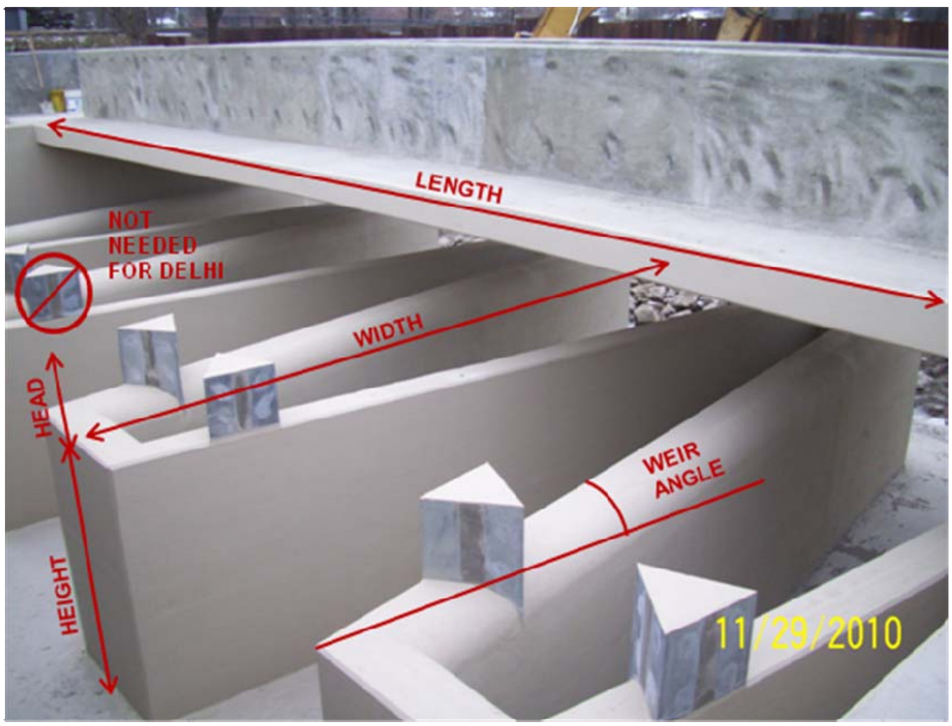
- **Design Head** – the depth of flow going over the labyrinth weir for the design flood. This is the maximum depth of flow the weir would be designed to pass.
- **Weir Height** – the height of the labyrinth weir wall. The higher the design head, the higher the weir height needs to be in order to maintain an effective discharge coefficient from normal flows to the design flood flow.
- **Weir Angle** – the angle of the long-section of the labyrinth weir wall. The overall shape of a labyrinth weir is a series of trapezoids. The weir angle controls how narrow or wide the shape of the trapezoid is. The narrower the shape, the more weir length (i.e. more trapezoids) that can be fit within a given area, but with increasing flow depths the closeness of the weir walls causes interference with flow going over the weir walls so reduces the effectiveness (i.e. reduces discharge coefficient) for higher flows. The wider the shape, the less weir length (i.e. fewer trapezoids) that can be fit within a given area, but there is less interference with increasing flow depths so the

larger weir angle geometries can provide more effective discharge over a larger range of flow depths.

- **Spillway Length** – the length available for the labyrinth weir. The length determines the number of weir cycles (number of trapezoids) that can be fit within the spillway.
- **Weir Cycle Width** – the width available for the labyrinth weir. The width determines how long the angled section of trapezoid can be. The greater the width, the more labyrinth weir that can be fit within the spillway length.

Underlying the labyrinth weir is a concrete spillway slab that provides the weir wall foundation as well as hard surface for flow falling off the weir. The preliminary design exhibits show a flat slab but in reality the labyrinth weir slab will have a slight grade (~2–3 percent) to move flow downstream more effectively.

A photo of a labyrinth weir that was designed by Stanley Consultants and recently constructed as the principal spillway of a dam in central Minnesota is provided in Figure B-7.



Labyrinth Weir
Figure B-7

Pneumatic gates essentially act as a sharp crested weir with an adjustable crest. When flows are low, the crest is kept at or near the normal pool and as flows increased the gate panels are lowered until they are flush with the fixed concrete slab/crest they are mounted to. Typically the gates are operated by a control panel which self-adjusts according to maintenance of a constant normal pool level. For preliminary design, the controlling factor is passage of the design flood, so gates are assumed to be down with the weir crest elevation essentially at the fixed concrete slab/crest. During detailed design the concrete slab/crest will be shaped to provide an efficient shape for discharging high flows when gates are down.

A photo of a pneumatic gate system that was designed by Stanley Consultants and recently installed as the principal spillway of a dam in north-central Iowa is provided in Figure B-8. Note these gates are air bladder controlled; pneumatic gates can also be mechanically controlled.

Pneumatic Gates
Figure B-8

Energy Dissipation

The spillway chute for all options was assumed to be sloped at 3:1, horizontal:vertical. The top of the chute for all alternatives is between elevations 886 ft-msl and 892 ft-msl. The channel elevation downstream of the dam is at approximately 860, so the chute drops roughly 30 feet vertically over 90 feet horizontally. Flow down the concrete chute will be supercritical (i.e. high velocity) so energy dissipation will be needed at the downstream end, likely in the form of a standard United States Bureau of Reclamation (USBR) standard stilling basin.

The energy dissipation will be most critical at lower magnitude flood flows (i.e. 5-year, 10-year, etc.). At high magnitude flood flows (i.e. 100-year, ½ PMF) the tailwater at the downstream end will be high enough to totally submerge any high velocity scouring flows and preventing them from causing downstream damage. At normal flows, the depth of flow down the chute will be shallow enough that it will not have sufficient power to cause scour of the downstream channel. With low magnitude flood flows the tailwater is low enough that the high velocity flows are not submerged prior to entering the downstream channel so a hydraulic jump must be created to dissipate the energy (i.e. slow down) the high velocity flows at the bottom of the chute. Similar to labyrinth weirs, stilling basins have been studied in enough detail so they can be designed using flow parameters and do not require building a physical model For the three spillway alternatives a rough design of a USBR Type III basin was laid out. A 40-foot long basin provided sufficient energy dissipation for a range of low magnitude flood flows. The stilling basin will be analyzed further and refined during detailed design.

Note that the auxiliary spillway portion of the Dual Labyrinth Spillway option does not have a stilling basin. This is because the auxiliary spillway would only discharge flows above the 100-year flood so a stilling basin is not needed at the downstream end. A large riprap apron will be sufficient.

Safety is always a concern at a dam. The hydraulics of supercritical flow, gate discharge and energy dissipation can cause rollers, eddies and vortices in the immediate downstream channel area that can be dangerous for recreational users of the downstream waterway. Appropriate warning signage and access control will be needed downstream of the dam. The overall safety of the downstream area will be a major factor in the detailed design of energy dissipation at the dam.

Cost and Structural Considerations

Several factors were taken into consideration in the hydraulic design of the spillway alternatives. The ultimate controlling factor is passage of the ½ PMF design flood, but items impacting cost, structural stability and constructability were also evaluated.

The geometry of the labyrinth weir and pneumatic gate spillways were not just controlled by hydraulics but structural issues as well. Labyrinth weir and gate heights were kept between 8 and 10 feet. A higher weir/gate height could provide more effective discharge, however when the wall or gate starts exceeding 10 feet, the additional structural and foundation requirements to make the overall structure stable start increasing to the point that making the spillway structure longer (i.e. more embankment length) is more cost-effective and constructable than trying to achieve a higher weir/gate.

A similar issue influences the steepness of the spillway chute. The steepness of the chute is controlled by the stability of the underlying earthen embankment. Hydraulically, a steeper chute could be used for the new spillway. However, the soil and stability parameters of the embankment and foundation are not suitable for increasing the steepness of the embankment.

Dual Labyrinth Weir Spillway

Many dams have both a principal and auxiliary spillway. The principal spillway is designed for continuous use in passing normal flows and then the auxiliary spillway is designed for infrequent use in passing high magnitude flood flows. Because the auxiliary spillway is used infrequently, typically cheaper materials that are stable and safe for occasional but not frequent use can be used to construct portions of the spillway. Theoretically, this provides a cost savings in spillway construction. For the Dual Labyrinth Spillway option a principal labyrinth spillway would be used to discharge normal flows, used in tandem with the lift gates to discharge higher flows, and then the auxiliary spillway would engage at flood magnitude flows.

The Dual Labyrinth Spillway consists of a 120-foot long primary spillway labyrinth weir set at the normal pool elevation of 896.3 ft-msl and a 110-foot long auxiliary spillway labyrinth weir set at an elevation of 900ft-msl. The primary spillway has two labyrinth cycles with a weir wall height of 10 feet, a width of 60 feet and a weir angle of 25 degrees. The auxiliary spillway has four labyrinth cycles with a weir wall height of 8 feet, a width of 40 feet and a weir angle of 15 degrees.

The primary spillway discharges to a concrete chute with a concrete stilling basin at the toe. Training walls were kept straight for the preliminary design but could potentially converge slightly to save a small amount of concrete. The primary spillway weir was kept at a wide angle because of the large depth of flow (head) during the ½ PMF design flood.

DNR design criteria require that at minimum the principal spillway be able to discharge the 50-yr flood (~24,000 cfs) without engaging the auxiliary spillway. Combined with the spillway lift gates, the primary labyrinth weir spillway can discharge roughly the 100-yr flood (~30,000 cfs). This would mean that the size of the principal labyrinth weir spillway could potentially be reduced so the combined gates and principal discharge the 50-yr flood and then the auxiliary spillway engages at flows exceeding the 50-yr flood. However it was determined during design the because the auxiliary spillway crest sits at a higher elevation than the principal spillway crest the auxiliary spillway would have to be upsized more than the principal could be downsized because the principal spillway can discharge more flow due to its lower crest. So the ½ PMF is controlling the design of both the principal spillway and auxiliary spillway.

The auxiliary spillway discharges to either a roller compacted concrete or articulated concrete block chute. These are cheaper surfacing than a concrete chute but are not meant to have continuous or frequent discharge over them. This is an additional reason for keeping a larger principal spillway because it would reduce the potential frequency of use. In the past three years a 50-year auxiliary spillway would have been used three times with the 2004, 2008, and 2010 floods whereas a 100-year auxiliary spillway would likely not have been used. The auxiliary labyrinth weir has a smaller weir angle because the design head is less than the principal spillway. The width of the auxiliary labyrinth weir is also shorter because its slab is at a higher elevation on the embankment so there is less width available.

Concrete training walls will be provided between the principal and auxiliary spillways and on the southern edge of the auxiliary spillway to keep flow contained within the spillway chute.

Single Labyrinth Weir Spillway

With the ½ PMF being the controlling flood, the lower the weir crest elevation, the more flow that can be discharged prior to the upstream pool reaching the top of dam elevation of 906 ft-msl. Using a single labyrinth weir set at the normal pool elevation of 896.3 ft-msl allows a greater length of weir to be at the normal pool elevation so saves on the overall length of spillway required to discharge the ½ PMF.

The Single Labyrinth Spillway consists of a 180-foot long labyrinth weir set at the normal pool elevation of 896.3 ft-msl. The primary spillway has five labyrinth cycles with a weir wall height of 10 feet, a width of 45 feet and a weir angle of 18 degrees. The entire spillway uses a concrete chute and stilling basin.

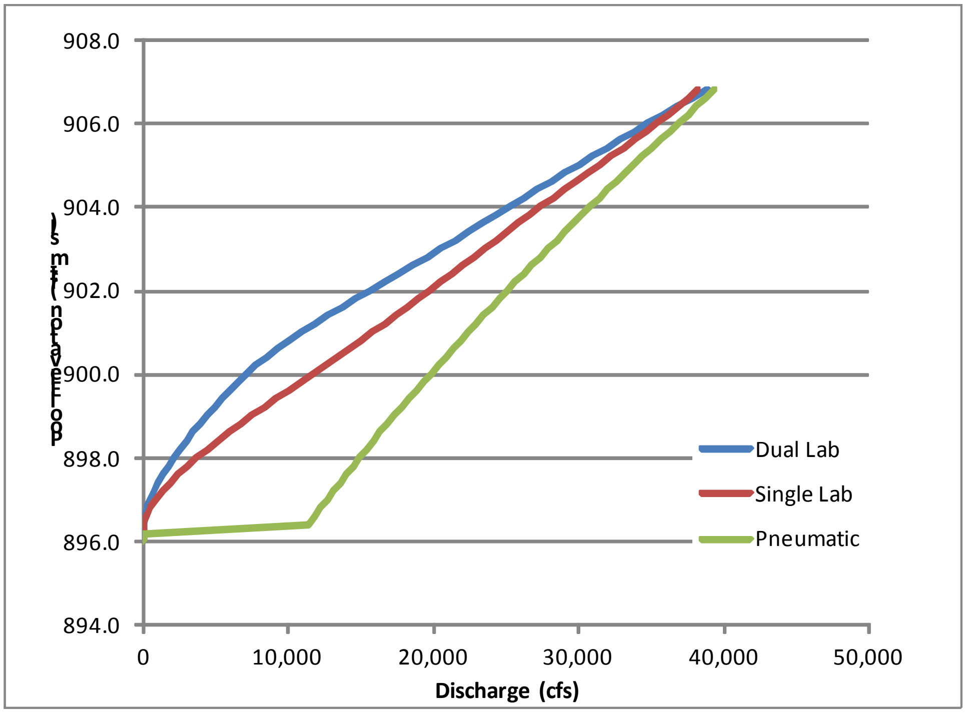
For preliminary design the spillway crest was set at a single elevation. For normal operating conditions a better discharge scenario will likely be to provide a weir segment or series of notches a few inches lower than the rest of the weir crest. This will allow the discharge to be more concentrated rather than a thin film of water going over the entire crest and will help maintain the pool at a more constant elevation. This will be analyzed further and refined in final design. This adjustment will not impact the overall hydraulic capacity of the weir for passing flood flows.

Pneumatic Gate Spillway

Similar to the reasoning for developing the single labyrinth weir option, the pneumatic gates provide ½ PMF discharge capacity by essentially lowering the weir crest below the normal pool elevation during flood flows. Because the pneumatic gates can be lowered they provide an even greater flow depth for discharging floods over the spillway prior to the upstream pool reaching the top of dam.

The range of pneumatic gate settings was set to be from normal pool (896.3 ft-msl) down to 888.3 ft-msl which would be flush with the fixed concrete crest of the spillway. An electronic control system would regulate gate settings for normal flow, maintaining a constant pool elevation of 896.3 ft-msl. The length of the pneumatic gate spillway is 160 feet. Taller gates could reduce the length of spillway but also as the gates get taller the foundation gets larger and the downstream tailwater could impact discharge for floods approaching the ½ PMF magnitude.

Figure B-9 displays Stage-discharge curves were developed for each of the Spillway Alternatives. Combined with the lift gates, all three alternatives can pass the ½ PMF without overtopping the existing powerhouse/spillway which with upstream exterior walls has an overtopping elevation of approximately 906.0 ft-msl.



Spillway Alternative Stage-Discharge Curves
Figure B-9

Comparison of Three Spillway Alternatives

All three spillway alternatives have distinct advantages and disadvantages. Without considering cost or operating/maintenance requirements, the Pneumatic Gates seem to be the best option, they take up the least amount of area, and they provide normal pool control over a wider range of flows. However, pneumatic gates require additional mechanical and electrical systems that are not required for the labyrinth weir spillways. They also require additional operation and maintenance and have a service life of roughly 25 years, which is less than half of the service life of a concrete structure.

The single labyrinth is 30 feet longer than the pneumatic gates but requires no operation. There is a greater sense of security knowing that the principal spillway is not subject to operation and maintenance of equipment. This is not to suggest that a labyrinth spillway will not require maintenance such as debris removal, but over normal day-to-day flows, the fixed labyrinth crest will provide a normal pool within 6 inches of 896.3 ft-msl for river flows up to 500 cfs without operating the lift gates. Reviewing daily flows at the USGS Highway 20 gage between April and December of 2011, lift gates would have been used on approximately 24 days out of the 250-day period. On the flow duration curve for the gage record this translates to roughly 10% of the time (similar to the 4/11-12/11 time period). The amount of time lift gates are used would change year to year depending on rain events that occur but the operation requirement for a single labyrinth weir is significantly less than the pre-breach dam or the pneumatic gate spillway.

Over the same time period with the dual labyrinth weir, lift gates would have been used on approximately 68 days out of the 250-day period. With a shorter principal spillway, the hydraulic capacity for discharging flows within 6 inches of the normal pool is 300 cfs (compared to 500 cfs with the single labyrinth spillway), so the lift gates have to be used more frequently. On the flow duration curve this translates to roughly 25% of the time (similar to the 4/11-12/11 time period). The dual labyrinth weir is also 50 feet longer than the single labyrinth weir, so additional grading will be needed along the south river bank to fit the dual spillways and chutes within the embankment and channel banks. The potential advantage of the dual labyrinth weir over the single labyrinth would be cost of construction where chute and stilling basin concrete (expensive) could be substituted for articulated concrete block or roller compacted concrete (cheaper) for the auxiliary spillway saving money on the overall construction cost. However, if the additional cost of grading and shaping the embankment and channel area for the larger dual labyrinth weir spillway is close to the cost savings of using less concrete, then the single labyrinth weir spillway would be the better option.

Spillway computations are provided in this appendix.

B.6 Other Hydraulic Issues

While the spillway alternatives analysis was the main component of the hydrologic and hydraulic studies, several other hydraulic considerations were evaluated that could impact design and construction. These issues included:

- Minimum/low flow passage
- Lake draining capacity
- Cofferdam/flow bypass during construction.

Minimum/Low Flow Passage

Minimum/low flow passage was a topic of concern with operation of the pre-breach Lake Delhi Dam. During times of normal and low flows, flow downstream of the dam was controlled by wicket gate discharge. Wicket gate settings and pool elevations were recorded but discharge rates were not quantified. During times of low flow there were concerns that insufficient discharge was being provided to the downstream waterway.

An additional concern was dissolved oxygen levels of the discharge. The wicket gates intake elevation is at 881.3, roughly 15 feet below the normal pool elevation where dissolved oxygen levels are typically low. Discharge through the gates was not aerated so waters in immediate downstream channel frequently did not meet dissolved oxygen requirements.

If the wicket gates are restored as the normal means of discharge, an aeration mechanism will be incorporated into the system. If the labyrinth or pneumatic gates are used as the single principal spillway sufficient aeration will be provided by the pool level discharge and flow down the spillway chute.

In addition to the spillway alternatives, installation of valved openings in two of the new lift gates is being considered. During normal operating conditions the valves would be closed. However the valves could be used to:

- Provide additional discharge capacity prior to gates lifting (roughly 150 cfs for two 30-inch valves at normal pool)
- Provide minimum flow passage if the upstream pool drops below the principal spillway crest.
- Provide bypass flow during potential maintenance work or debris removal at the principal spillway without lifting gates.
- Provide the capability to draw down the pool a small amount or maintain a slightly drawn down pool during low flows. The lift gates are good for passing large flows but not for normal bypass flows or drawing down the pool a few inches.

Unlike the wicket gates, the valves will discharge onto the concrete ogee spillway, so even though the valves would likely be 10 feet below the normal pool, discharge would be aerated by the drop over the concrete spillway.

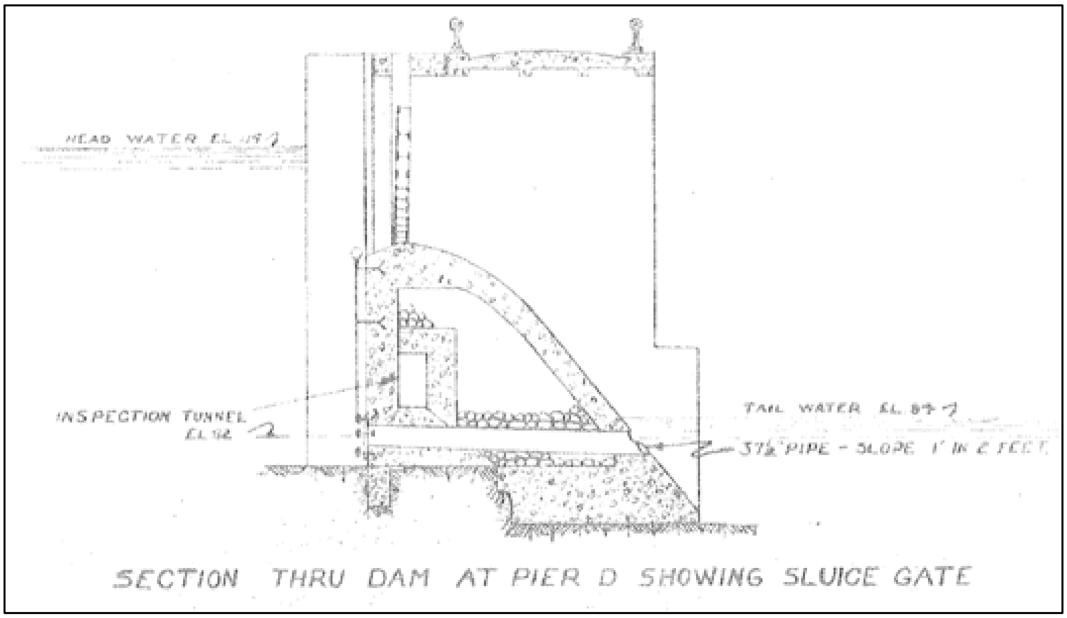
The previous dam operator indicated the 7Q10 flow (lowest 7-day average flow that occurs once every 10 years) for the Maquoketa River at Lake Delhi Dam is roughly 28 cfs. The 30-inch valves would have the capacity to discharge the 7Q10 flow.

As reconstruction design progresses a detailed operating manual will be developed with DNR input and approval that provides operating protocol and discharge rates for the expected range of flow conditions.

Lake Draining Capacity

DNR requires that "A gated low level outlet shall be provided which is capable of draining at least 50 percent of the permanent storage behind the dam within a reasonable length of time." The existing lift gates provide sufficient capacity to drain 50 percent of the volume below the normal pool elevation. In addition, existing plans indicate a set of two 37.5-inch diameter sluice pipes were installed through the northernmost spillway pier approximately 20 feet below the crest of the gated spillway.

If they do exist, the sluice pipe intakes are buried under 20 feet of riprap. This riprap will be removed during the dam reconstruction and the feasibility of restoring the existing sluice pipes will be assessed. The sluice pipes are not necessary to meet DNR design requirements but could be useful during construction and for future maintenance and dredging projects. A copy of the section drawing is shown in Figure B-10.



Potential Existing Sluice Pipes
Figure B-10

Cofferdam/Flow Bypass During Construction

At this stage, the dam reconstruction has been separated into two phases. The first phase would involve restoration of the existing powerhouse/spillway structure and north embankment. The second phase would involve construction of the southern embankment and new spillway.

For the first phase, cofferdams are only needed to prevent high river flows from entering the construction area and for dewatering. Flow bypass is provided by the eroded section of the dam. Relatively short cofferdams would be constructed upstream of the existing spillway and riprap area and downstream of the existing stilling basin.

The size of the second phase cofferdams will be partially controlled by the condition of the existing sluice pipes. If the sluice pipes are restorable, significant bypass capacity can be provided without construction of a tall upstream cofferdam (saves construction time and money). If the sluice pipes are not restorable the cofferdam will have to be constructed several feet above the existing gated spillway crest in order to a means for flow bypass. A taller cofferdam will mean a higher pool during construction (benefit to lake residents) but greater risk and additional cost to the project.

Using the USGS frequency analysis, required cofferdam heights were estimated for given return period flows. Table B-8 provides a summary.

Table B-8 Cofferdam Height Estimates

Return Period (years)	River Flow (cfs)	Phase I Cofferdams Upstream Elev.		Phase I Cofferdams Downstream Elev.		Phase II Cofferdams Upstream Elev.		Phase II Cofferdams Downstream Elev.	
		Elev.	H (ft)	Elev.	H (ft)	Elev.	H (ft)	Elev.	H (ft)
1	1,400	869.5	0	861.5	4	883	18	861.5	2
2	4,500	872	2	866.1	8	886.5	22	865.9	6
5	8,700	874	4	869.5	12	890.5	26	870	10
10	12,300	876	6	872.2	14	893	28	871.9	12

For the cofferdam analysis, sluice pipes were assumed to be inoperable. The cofferdam heights shown in Table B-8 are strictly estimates based upon flow bypass capacity. Ultimately cofferdams will either be designed in subsequent phases of the project or at the discretion of the contractor based upon his assessment of risk.

B.7 References

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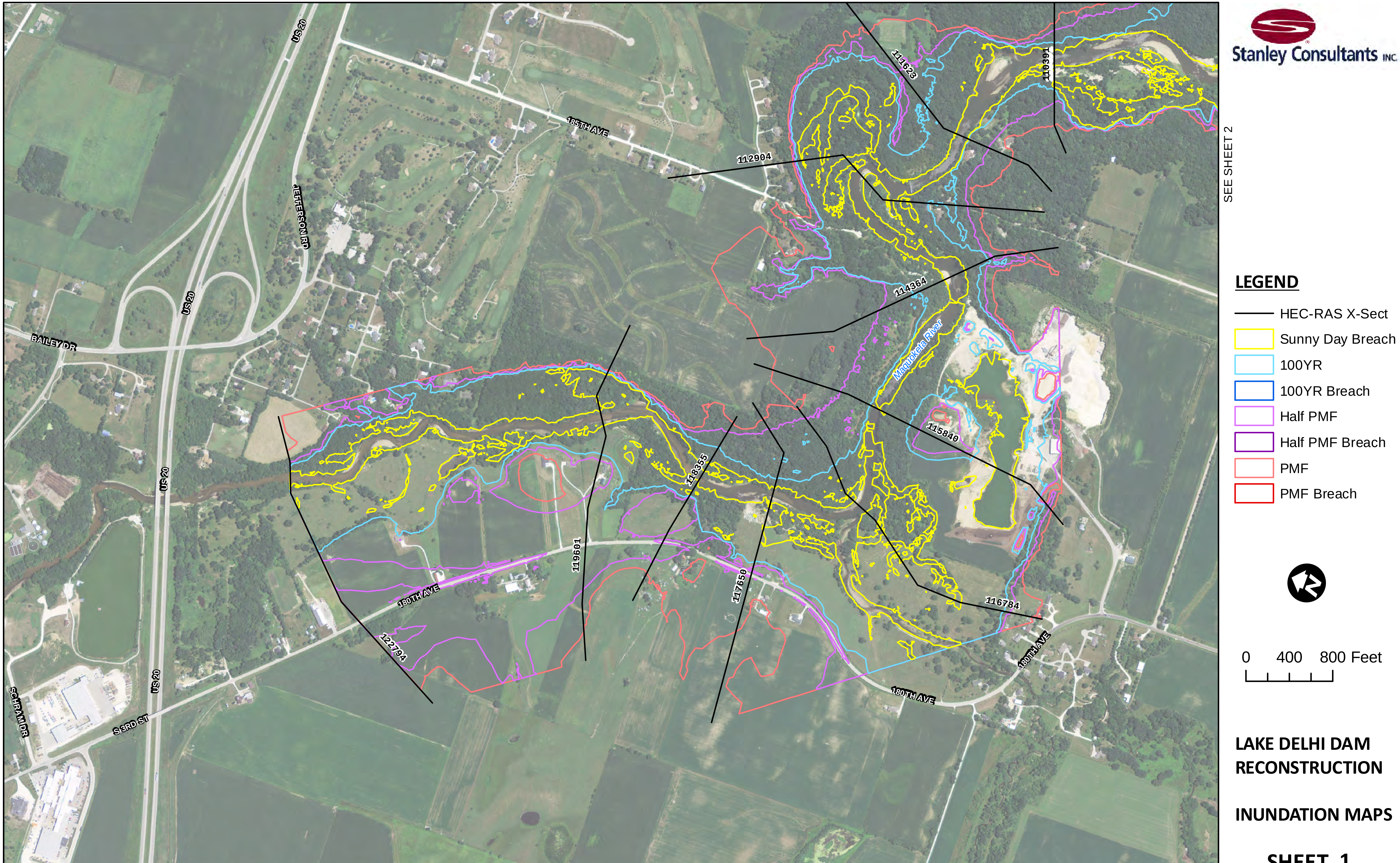
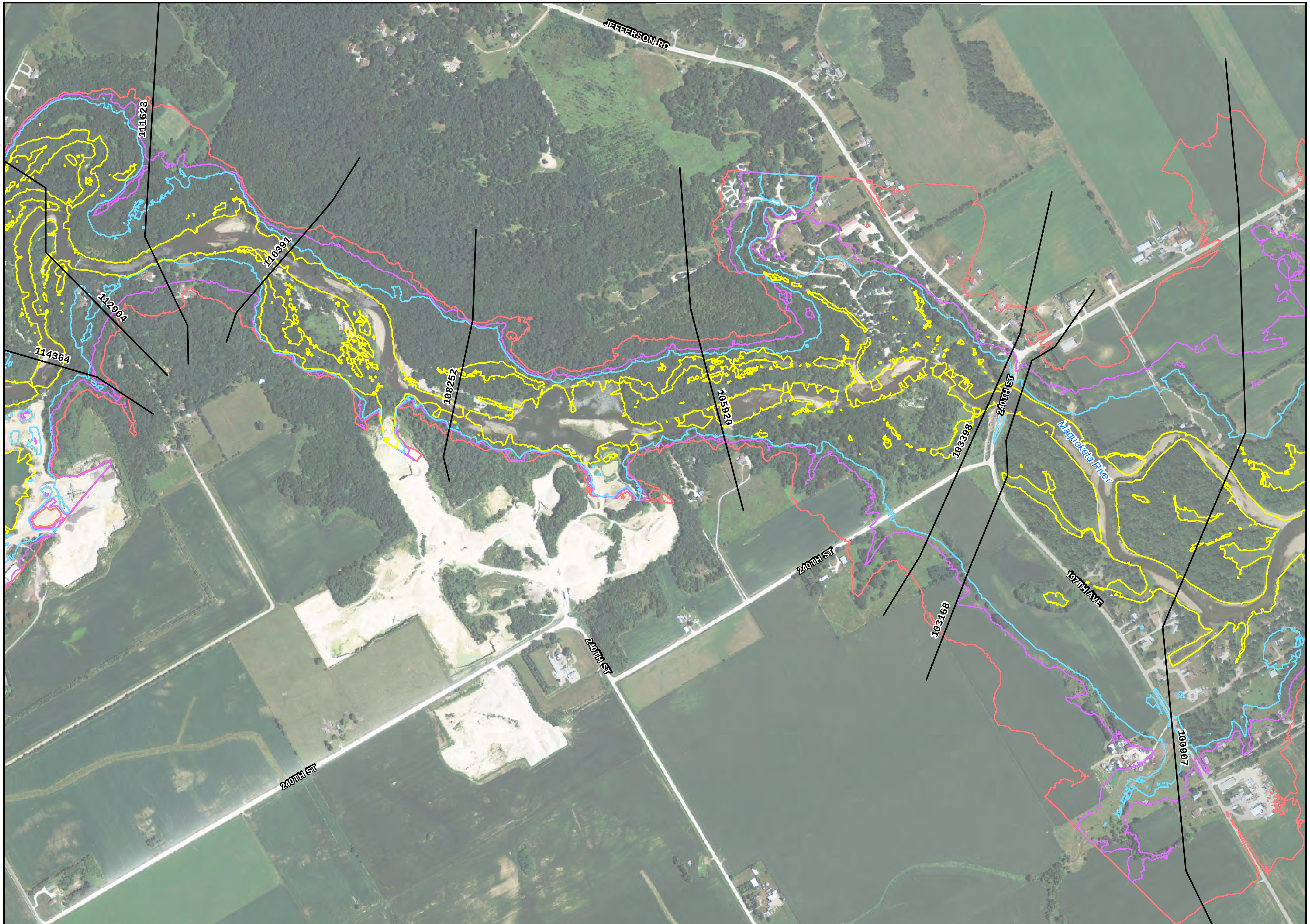


Stanley Consultants INC.
LEGEND
HEC-RAS X-Sect
Sunny Day Breach
100YR
100YR Breach
Half PMF
Half PMF Breach
PMF
PMF Breach
0 400 800 Feet
SEE SHEET 2
LAKE DELHI DAM RECONSTRUCTION
INUNDATION MAPS
SHEET 1
US 20
185TH AVE
JEFFERSON RD
BAILEY DR
180TH AVE
S 3RD ST
SCHRAM DR
Maquoketa River
111623
110391
112904
114364
115840
116784
117650
118355
119601
122794

SEE SHEET 1



SEE SHEET 3

Stanley Consultants INC.

LEGEND

- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

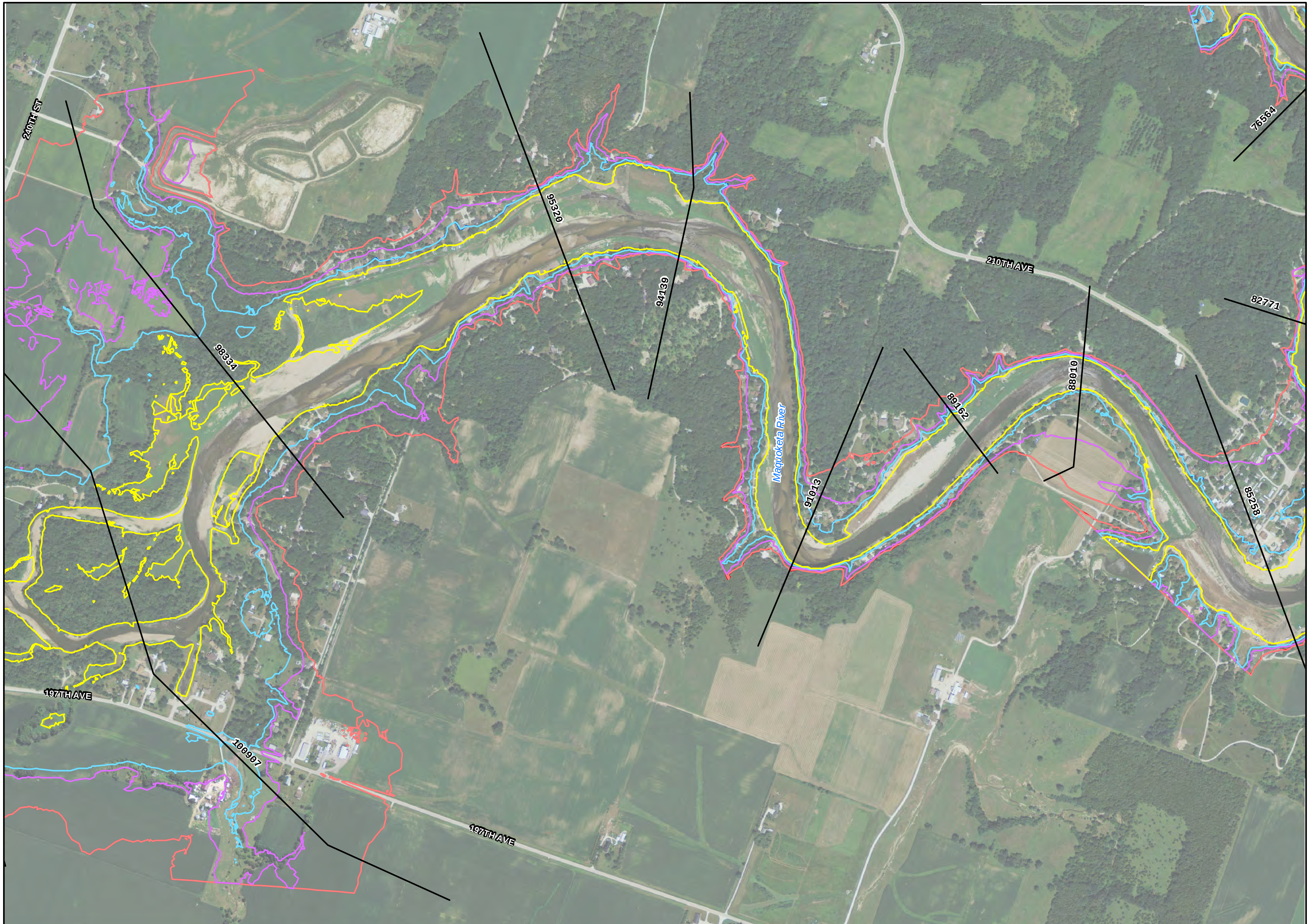
0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 2

SEE SHEET 2



SEE SHEET 4

Stanley Consultants INC.

LEGEND

- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 3

Stanley Consultants INC.

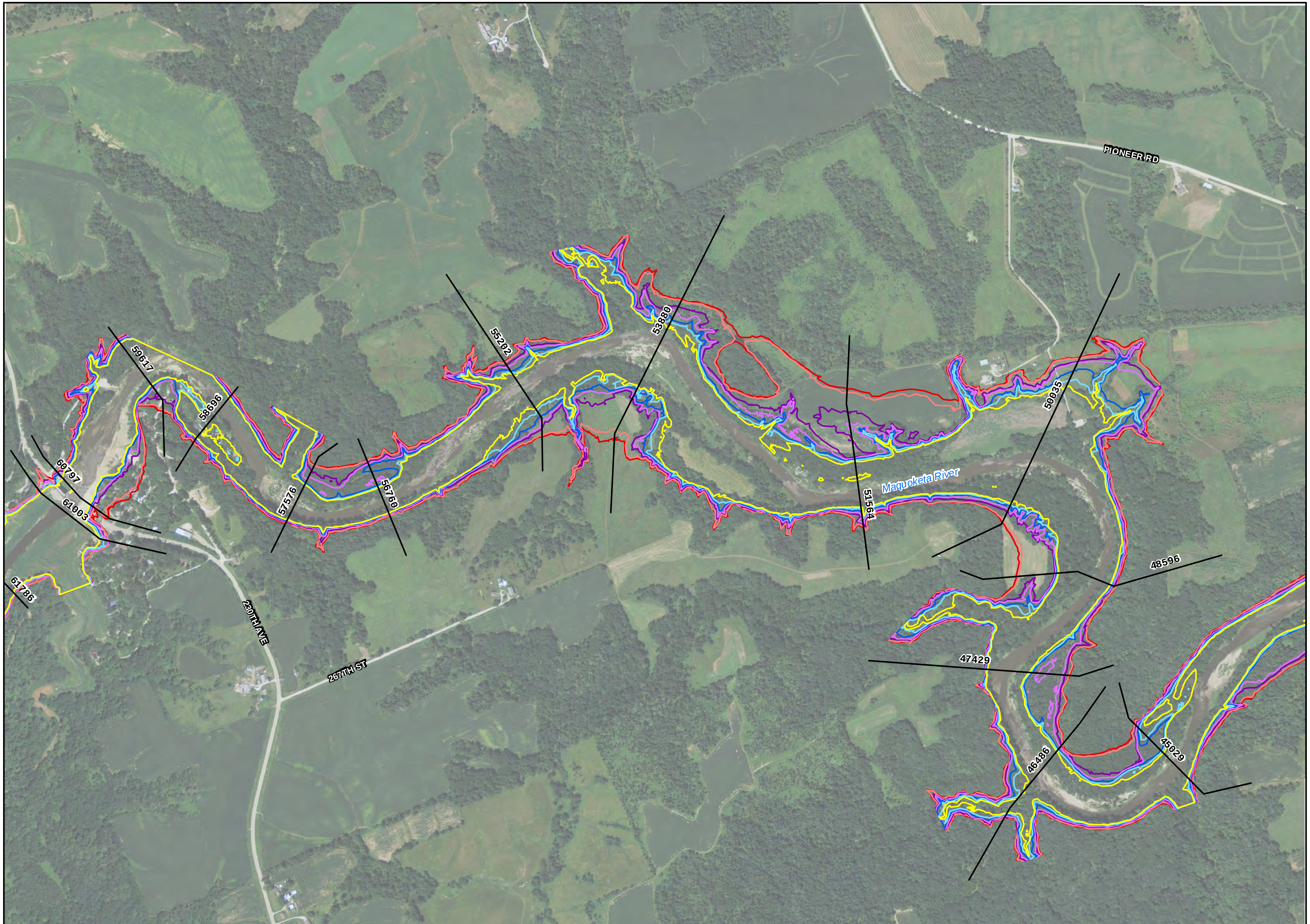
LEGEND

- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 4



Stanley Consultants INC.

LEGEND

HEC-RAS X-Sect
Sunny Day Breach
100YR
100YR Breach
Half PMF
Half PMF Breach
PMF
PMF Breach

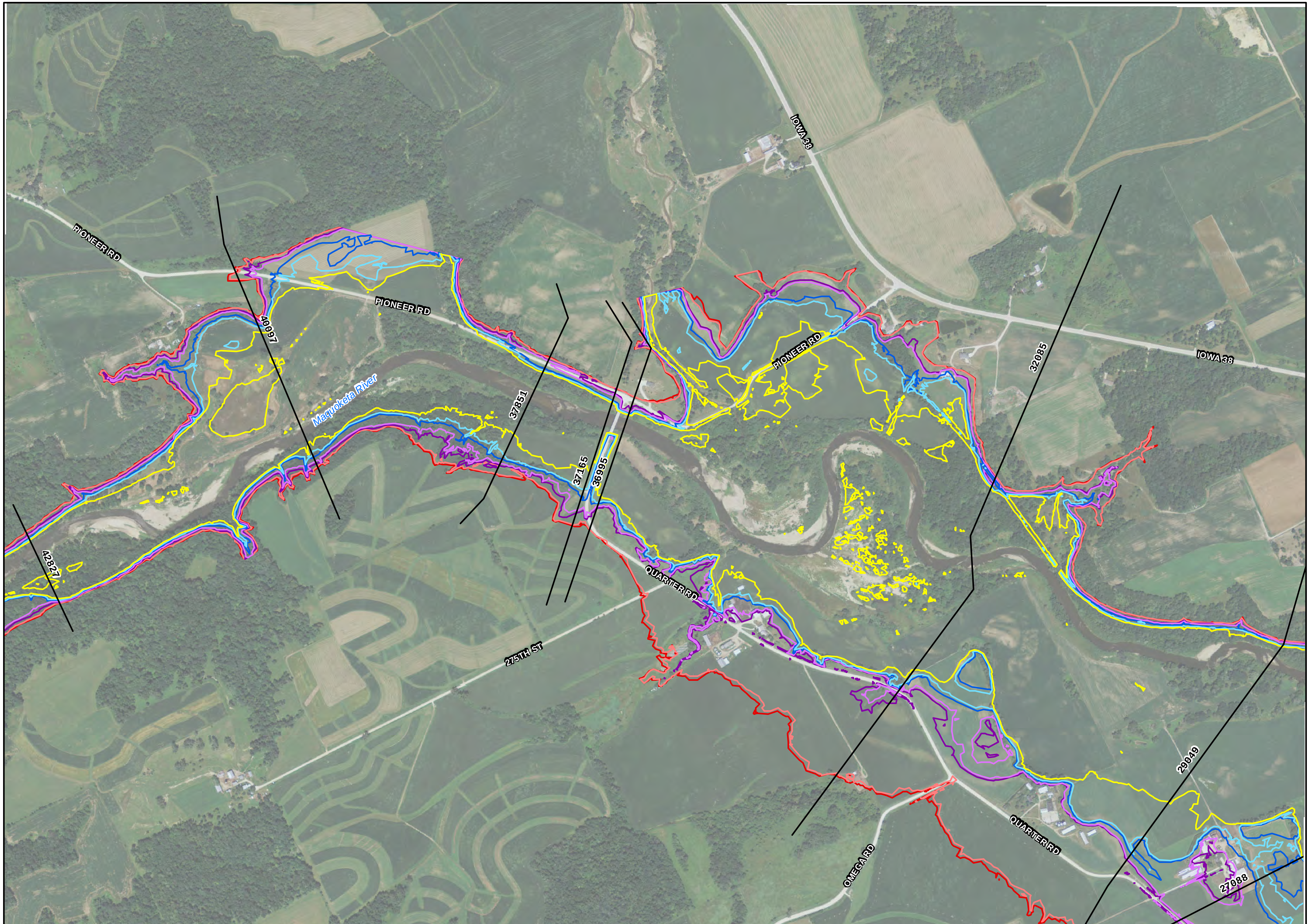
0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 5

SEE SHEET 5



SEE SHEET 7

Stanley Consultants INC.

LEGEND

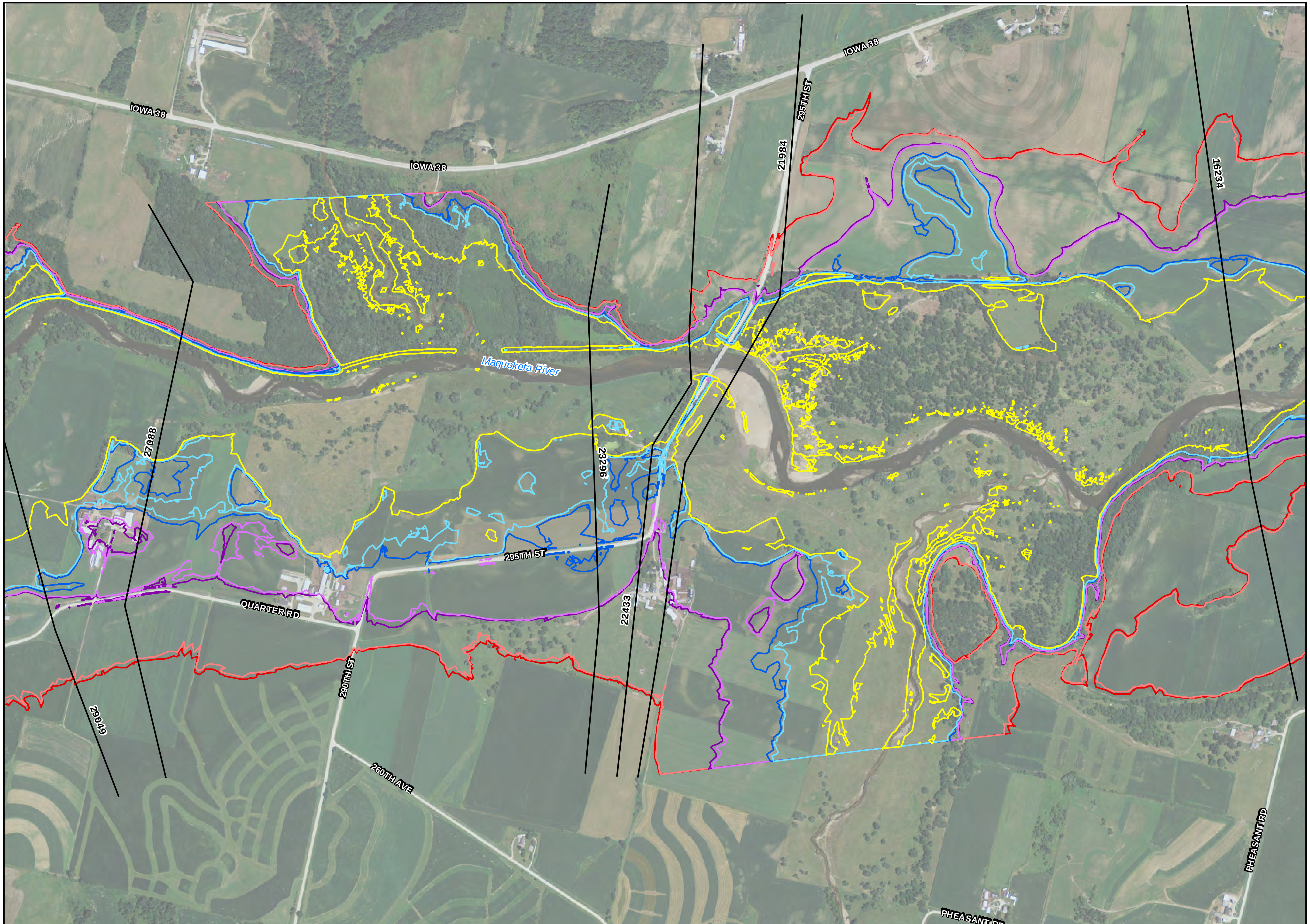
- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 6



Stanley Consultants INC.

LEGEND

- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

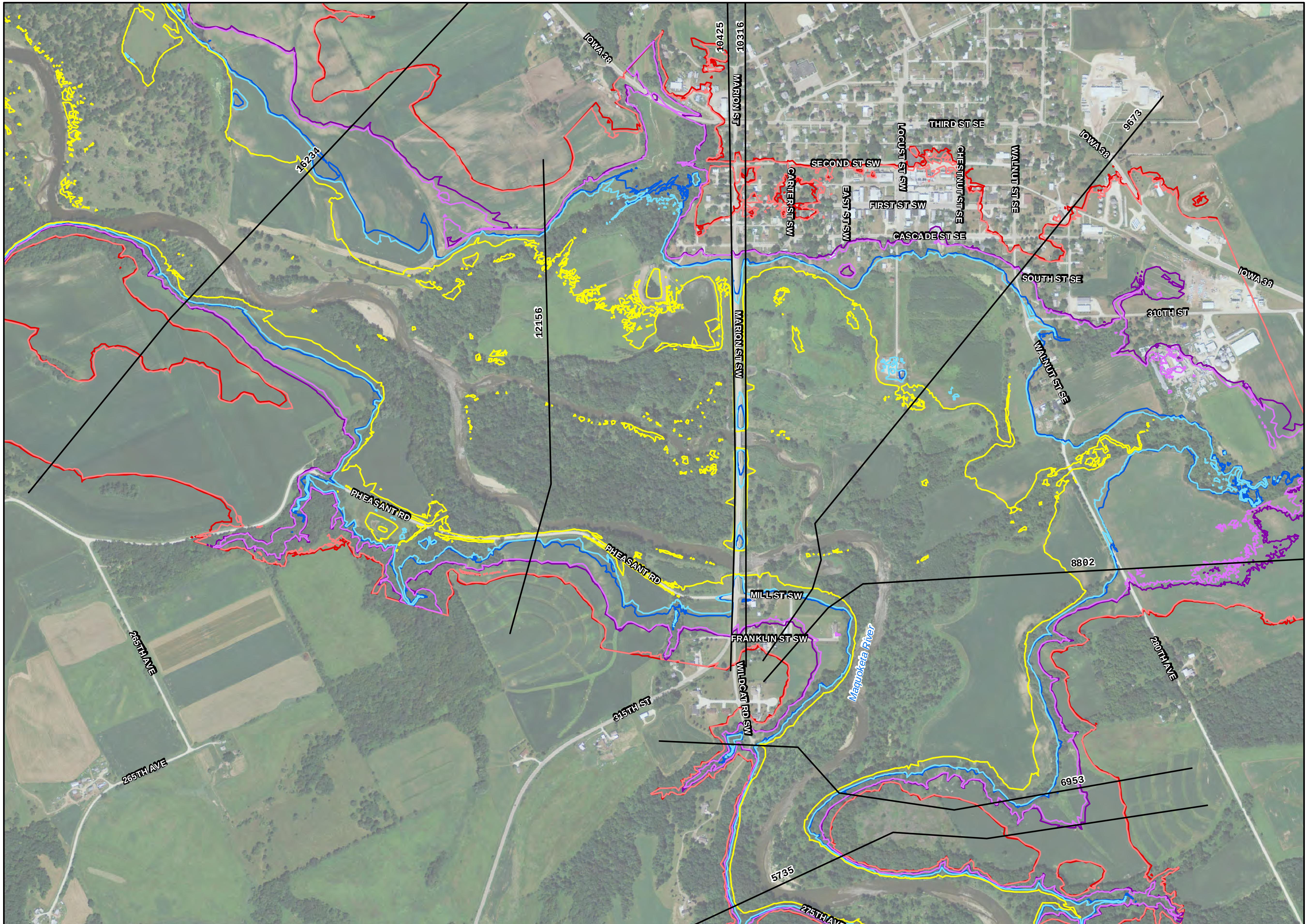
0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 7

SEE SHEET 7



Stanley Consultants INC.

LEGEND

- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

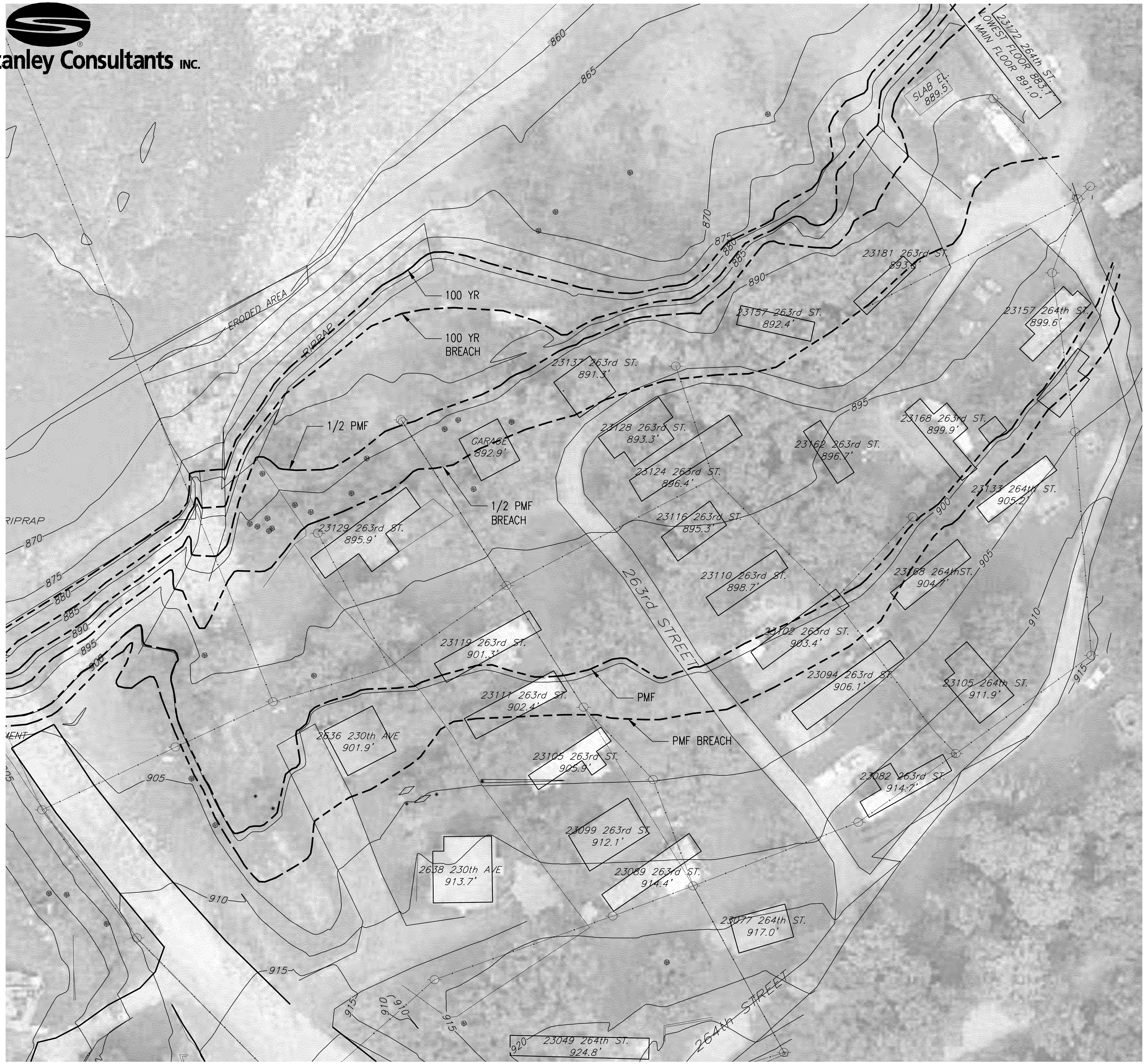
0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 8

Stanley Consultants INC.



PLAN

PROPERTY INUNDATION DEPTH TABLE								
Property			PMF Depth		1/2 PMF Depth		100YR Depth	
Address	Street	F.F. Elev.	No Breach	Breach	No Breach	Breach	No Breach	Breach
2636	230th Ave	901.9	0.0	0.2	0.0	0.0	0.0	0.0
2638	230th Ave	913.7	0.0	0.0	0.0	0.0	0.0	0.0
23082	263rd St	914.7	0.0	0.0	0.0	0.0	0.0	0.0
23089	263rd St	914.4	0.0	0.0	0.0	0.0	0.0	0.0
23094	263rd St	906.1	0.0	0.0	0.0	0.0	0.0	0.0
23099	263rd St	912.1	0.0	0.0	0.0	0.0	0.0	0.0
23102	263rd St	903.4	0.0	0.0	0.0	0.0	0.0	0.0
23105	263rd St	905.9	0.0	0.0	0.0	0.0	0.0	0.0
23110	263rd St	898.7	1.0	3.2	0.0	0.0	0.0	0.0
23111	263rd St	902.4	0.0	0.0	0.0	0.0	0.0	0.0
23116	263rd St	895.3	4.4	6.6	0.0	0.0	0.0	0.0
23119	263rd St	901.3	0.0	0.7	0.0	0.0	0.0	0.0
23124	263rd St	896.4	3.3	5.5	0.0	0.0	0.0	0.0
23128	263rd St	893.3	6.4	8.6	0.0	0.0	0.0	0.0
23129	263rd St	895.9	3.9	6.1	0.0	0.0	0.0	0.0
23137	263rd St	891.3	8.4	10.6	0.0	1.2	0.0	0.0
23157	263rd St	892.4	7.2	9.5	0.0	0.0	0.0	0.0
23162	263rd St	896.7	0.0	0.0	0.0	0.0	0.0	0.0
23168	263rd St	904.7	2.9	5.2	0.0	0.0	0.0	0.0
23181	263rd St	893.3	6.3	8.6	0.0	0.0	0.0	0.0
23049	264th St	924.8	0.0	0.0	0.0	0.0	0.0	0.0
23077	264th St	917	0.0	0.0	0.0	0.0	0.0	0.0
23105	264th St	911.9	0.0	0.0	0.0	0.0	0.0	0.0
23168	264th St	904.7	0.0	0.0	0.0	0.0	0.0	0.0
23133	264th St	905.2	0.0	0.0	0.0	0.0	0.0	0.0
23157	264th St	899.6	0.0	2.2	0.0	0.0	0.0	0.0
23172	264th St	883.1	16.4	18.7	5.1	9.2	0.0	0.2

0' 15' 30' 60'

SCALE: 1" = 60'-0"

LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES
HYDRAULIC ANALYSIS

DOWNSTREAM PROPERTY
INUNDATION EXHIBIT A

CADD D4-R3

Note: Aerial Photo shows Pre-Breach Condition

PROPERTY INUNDATION DEPTH TABLE								
Property			PMF Depth		1/2 PMF Depth		100YR Depth	
Address	Street	F.F. Elev.	No Breach	Breach	No Breach	Breach	No Breach	Breach
26287	231st Ave	889.1	10.0	12.3	0.0	2.7	0.0	0.0
26294	231st Ave	894.4	4.7	7.0	0.0	0.0	0.0	0.0
26299	231st Ave	892.4	6.7	9.0	0.0	0.0	0.0	0.0
26269	232nd Ave	879.8	19.3	21.6	8.0	12.0	0.0	3.0

LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES
HYDRAULIC ANALYSIS

DOWNSTREAM PROPERTY
INUNDATION EXHIBIT B

LAKE DELHI DAM DOWNSTREAM STRUCTURE AND ROADWAY FLOOD INUNDATION TABLE																						
Structure	River Sta	Residences							Commerical/Agricultural Bldgs							Roads						
		PMF		Half PMF		100YR		Sunny Day	PMF		Half PMF		100YR		Sunny Day	PMF		Half PMF		100YR		Sunny Day
		No Breach	Breach	No Breach	Breach	No Breach	Breach	Breach	No Breach	Breach	No Breach	Breach	No Breach	Breach	Breach	No Breach	Breach	No Breach	Breach	No Breach	Breach	Breach
Lake Delhi Dam	60900																					
	60797	9	12	1	2		1									Local	Local					
	59617	4	4	1	2		1											Dwy	Local		Dwy	
	58696																					
	57576																					
	56760																					
	55202																					
	53880																					
	51564																					
	50035															Dwy	Dwy	Dwy	Dwy	Dwy	Dwy	Dwy
	48596																					
	47429																					
	46486																					
	45029																					
	42827																					
	40097															Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd
	37851															Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd
	37165															Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd
Quarter Road Bridge	37080															Overtop	Overtop	Overtop	Overtop			
	36995	1	1						4	4	2	2		1		Pionr/Quarter	Pionr/Quarte	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd
	32085	1	1						1	1						Quarter Rd	Quarter Rd					
	29049	1	1								1	1				Quarter Rd	Quarter Rd					
	27088	3	3	2	2											38/295th	38/295th	38/295th	38/295th	Hwy 38	Hwy 38	
	23296	1	1	1	1	1	1		1	1	1	1	1	1								
	22433	1	1						1	1	1	1										
295th Street Bridge	22180															Overtop	Overtop	Overtop	Overtop			
	21984															260/297th	260/297th	260/297th	260/297th	260th	260th	
	16234	1	1						1	1						Pheasant	Pheasant	Pheasant	Pheasant	Pheasant	Pheasant	
	12156	1	1						2	2	1	1				Pheasant	Pheasant	Pheasant	Pheasant	Pheasant	Pheasant	
	10425	12	12	2	2											Local	Local	Pheasant	Pheasant	Pheasant	Pheasant	
Hopkinton Bridge	10370															Overtop	Overtop	Overtop	Overtop	Overtop	Overtop	
	10316	45	45	12	12	1	1		10	10						Local	Local					
	9673	20	20	7	7	1	1		10	10	2	2				Wilson/38	Wilson/38	Wilson	Wilson	Wilson	Wison	
	8802	3	3	1	1											Local	Local	Local	Local			
	6953																					
	5735																					
	2528																					
	702																					
TOTAL		**103**	**106**	**27**	**29**	**3**	**5**	**0**	**30**	**30**	**8**	**8**	**1**	**2**	**0**	**12**	**12**	**8**	**8**	**5**	**5**	**1**

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Description:

Summary of hydrologic and hydraulic analysis performed for Lake Delhi Dam reconstruction project.

Reference:

(1) Iowa DNR, ***Design Criteria and Guidelines for Iowa Dams***, T.B. 16, 1990.
(2) Ashton-Barnes Engineers, Inc., ***Report of Inspection of Lake Delhi Dam on Maquoketa River***, 1998.
(3) FERC, ***Engineering Guidelines for the Evaluation of Hydropower Projects***, 1993, 2001.
(4) Independent Panel of Engineers, Report on Breach of Delhi Dam, Dec. 2010.
(5) USGS, ***Techniques for Estimating Flood-Frequency Discharges for Streams in Iowa***, WRIR 0-4233, 2001.
(6) NOAA/USACE, ***Probable Maximum Precipitation Estimates/Application***, HMR 51/52, 1978/1982.
(7) Colorado Dam Safety Branch, ***Guidelines for Dam Breach Analysis***, 2010.

Analysis:

Reconstruction of Lake Delhi Dam must meet requirements set forth in Ref 1. In addition, if hydropower generation is to be reinstated at Lake Delhi Dam, the reconstructed dam must meet design standards set forth in Ref 3. The hydrologic and hydraulic analysis was performed in adherence to both Ref 1 and Ref 3. The analysis consisted of the following steps.

	Task	Software	Inputs	Results
1	Establish/Verify Hydrology	ArcGIS	Ref 2/ArcGIS	Drainage Area, Infiltration Rate,Tc
2	Establish/Verify PMP	HMR 52	DA, Location	Hourly PMP Depths
3	Establish/Verify PMF	HEC-HMS	Tasks 1,2	PMF Inflow Hydrograph to Lake Delhi
4	Establish Hydraulic Model	HEC-GeoRAS	LiDAR	HEC-RAS model of Dam and River
5	Design Flood/Hazard Class	HEC-GeoRAS	PMF	Inundation Maps, Hazard Classification

Task 1 **Establish/Verify Hydrology**

As part of their 1997 inspection of Lake Delhi Dam, Ashton-Barnes completed a spillway adequacy analysis that included a development of a Probable Maximum Flood (PMF) and analysis of the dam's spillway capacity.

The analysis is described in Ref 2 and generally follows analysis methodology presented in Ref 1 and Ref 3. The Ref 2 analysis was reviewed and updated to reflect some adjustments to watershed/rainfall parameters.

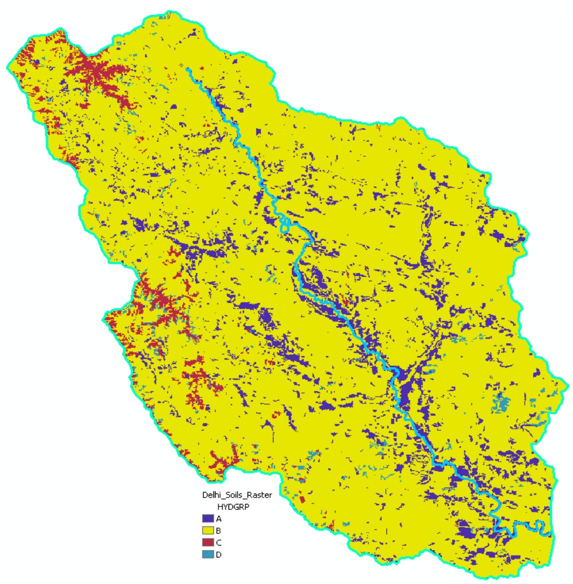
Parameter	Ref 2 Value	2011 Value	2011 Method
DA (mi^2)	347	349.2	ArcGIS - Iowa DNR supplied drainage area checked against HUC shapefiles
Infiltration	CN = 60	0.25 in/hr	See discussion below
Tc (hrs)	18	18	Clarks Unit hydrograph, see discussion below
PMP Total (in)	22.01	25.77	See Task 2

Discussion

Drainage area between the two studies was very close. For infiltration the Ref 2 study categorized the soil hydrologic group as A/B and land use as a 60/40 split between crops/meadow which gave a curve number of 60.

The soil hydrologic group was reviewed in ArcGIS using SSURGO soil data

Group	Area (mi^2)	Percent
A	36.3	10.4%
B	298.1	85.4%
C	10.7	3.1%
D	4.1	1.2%
Total	349.2	



For large watersheds, Ref 3 recommends using an initial and uniform loss rate for infiltration as opposed to the NRCS CN.

Ref 3 - Table 8-8.1

Hydrologic Group	Minimum Infiltration Rates (in./hr)	Soil Description
A	0.30 to 0.45	Deep sand, deep loess, aggregated silts
B	0.15 to 0.30	Shallow loess, sandy loam
C	0.05 to 0.15	Clay loams, shallow loam, soils low in organic content, soils usually high in clay
D	0 to 0.05	Soils that swell significantly when wet, heavy plastic clays, certain saline soils

Because the majority of soil in the watershed is Group B, this range of infiltration rates was used to represent the waterhsed's infiltration.

No initial infiltration amount was assumed (i.e. saturated conditions)

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Clark's unit hydrograph method was used in Ref 2 for hydrograph development which is the method recommended by Ref 3
Clark's method uses time of concentration and a storage parameter "R"

The Time of Concentration was computed in Ref 2 using the river length and an average travel time

River Length	36 miles	(45 miles River - 9 miles of Lake)	Length was verified using ArcGIS
Travel Time	18 hours	(3 ft/sec over 36 miles)	V_{river} was verified as reasonable using HEC-RAS and is in the range of the 4-5 hour travel time between the USGS gage and Delhi Dam established by Ref 4 (12 mi/5 hour = 3.5 ft/sec)

For R, Ref 2 used a ratio of Tc, 0.833*Tc = 15 hours
Ref 3 recommends computing R using the descending limb of a stream gage hydrograph (if available)

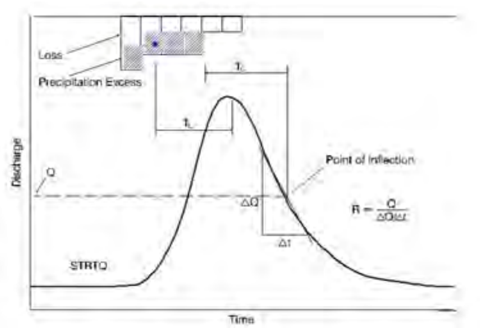


Figure 8-6.2

Stream gage data is available at for USGS Gage 05416900 near Manchester
http://waterdata.usgs.gov/ia/nwis/uv?site_no=05416900

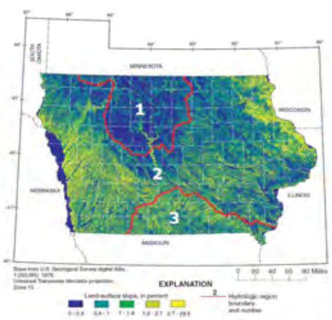
This gage is approximately 12 miles upstream of the Lake Delhi Dam and the drainage areas are

DA_{Delhi} = 349 mi^2
DA_{Gage} = 300 mi^2

Ref 5 recommends estimating flows at Delhi by the following equation

$$Q_{t(aw)} = Q_{t(wg)}\,(DA_u / DA_g)^x, \qquad (6)$$

where $Q_{t(aw)}$ = the area-weighted discharge estimate for an ungaged site on a gaged stream for recurrence interval t;
$Q_{t(wg)}$ = as defined for equation 2;
DA_u, DA_g = as defined for equation 3; and
x = the mean exponent for a hydrologic region; for Region 1, the mean exponent is 0.665; Region 2, 0.446; and Region 3, 0.403.



Multiplier for Gage Flows at Delhi is $(349/300)^{0.446}$ = 1.070

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15-min gage flows were obtained for three recent floods and multiplied to reflect flows at the Dam.

Clark's R was estimated for the floods using the method presented in Ref 3

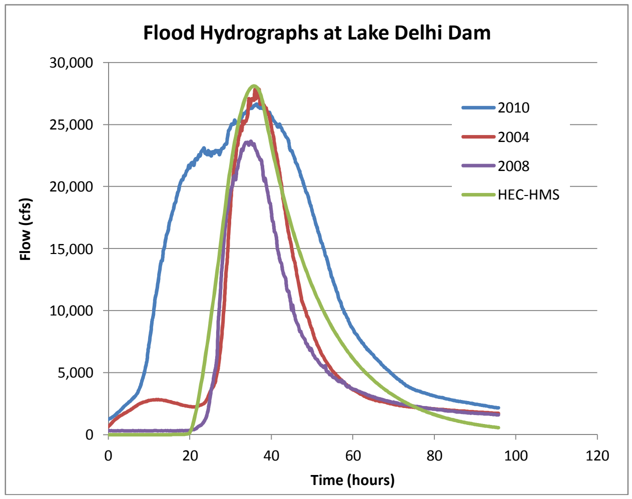
Year	Q (cfs)	ΔQ (cfs)	ΔT (hr)	R (hr)
2010	15100	9900	10	15.3
2008	15200	12960	10	11.7
2004	15200	15550	10	9.8

Both the 2010 and 2004 flood were considered to be close to the 100-year magnitude.

Using HEC-HMS the R and infiltration rate were adjusted to calibrate the 100-year storm (Precip=6.4") and resulting hydrograph to the 2004 hydrograph. The calibrated HEC-HMS hydrograph is shown in the graph.

From calibration the following parameters were developed to represent the Dam's watershed

R =	15 hr
Tc =	18 hr
Infilt =	0.25 in/hr



Task 2 **Establish/Verify PMP**

Probable Maximum Precipitation (PMP) events were developed using methods established by Ref 6.

The PMP is essentially an attempt to quantify the largest rainfall event that could be expected to occur over a given area. It is developed using the area location, area size, and parameters established by Ref 6.

HMR 51 establishes the PMP amount for a given duration and area size for the U.S. east of the 105th Meridian.
HMR 52 establishes a methodology for locating and distributing the PMP event over the given watershed area.

The PMP event is represented by Isohyetals (rainfall intensities) that are rotated and located over the basin to maximize rainfall depth

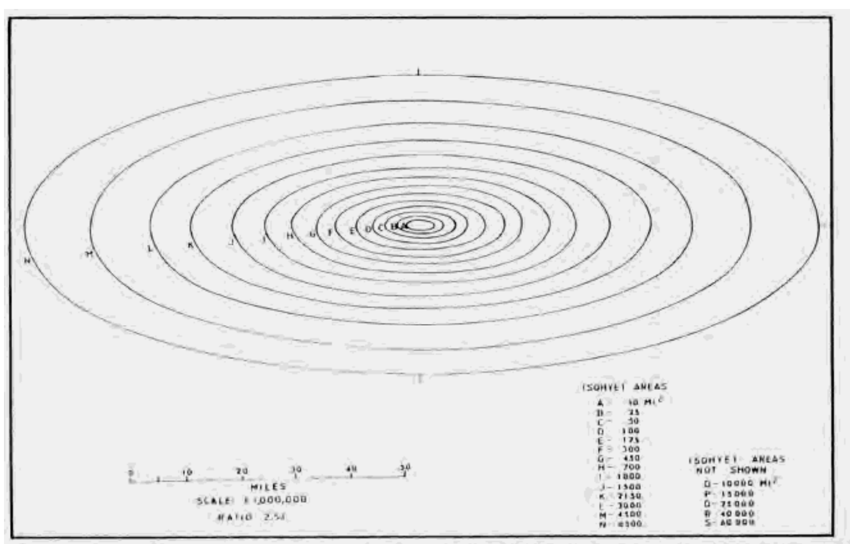


Figure 5.--Standard isohyetal pattern recommended for spatial distribution of PMP east of the 105th meridian (scale 1:1,000,000).

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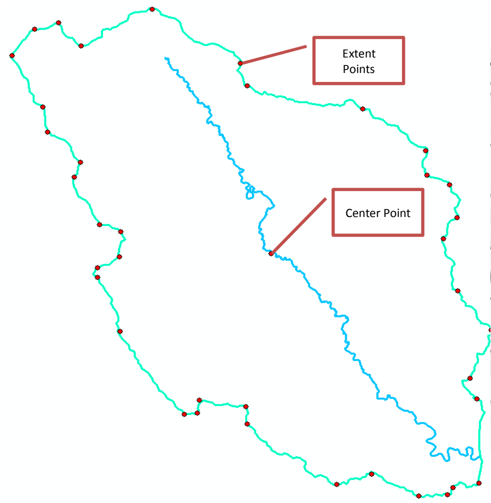
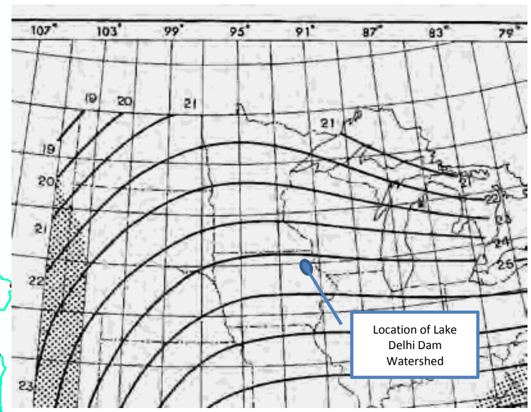
In addition to HMR51/52 manuals, HMR 52 software was developed to automate the development of the PMP for a given area

Inputs to HMR 52 are drainage area extent points, drainage area center, rainfall area/duration depths, and the expected isohyetal orientation which is provided in the HMR 52 manual.

Drainage area extents and center were determined using ArcGIS



Precipitation Depths were obtained from HMR 52 maps



PMP Area/Duration/Depths for Lake Delhi Dam					
Area (mi^2)	6-hour	12-hour	24-hour	48-hour	72-hour
10	25.2	29.5	31	34.4	36.2
200	18.3	21.9	23.8	26.3	28.2
1000	13.5	16.1	18.1	22	22.6
5000	8.3	10.9	12.5	15.2	17.1
10000	6.5	8.7	10.3	13.2	14.9
20000	4.6	6.7	8.2	11.1	12.6

This information is input into HMR 52 software, which then maximizes the rainfall through storm placement and duration

The Output of HMR 52 provides the following precipitation distribution for Delhi Dam's PMP

PROBABLE MAXIMUM STORM FOR DELHI

DAY 1

TIME	PRECIPITATION INCR	TOTAL	TIME	PRECIPITATION INCR	TOTAL	TIME	PRECIPITATION INCR	TOTAL	TIME	PRECIPITATION INCR	TOTAL
0100	0.05	0.05	0700	0.06	0.36	1300	0.08	0.75	1900	0.11	1.25
0200	0.05	0.10	0800	0.06	0.42	1400	0.08	0.82	2000	0.11	1.35
0300	0.05	0.15	0900	0.06	0.49	1500	0.08	0.90	2100	0.11	1.46
0400	0.05	0.20	1000	0.06	0.55	1600	0.08	0.98	2200	0.11	1.57
0500	0.05	0.25	1100	0.06	0.61	1700	0.08	1.06	2300	0.11	1.68
0600	0.05	0.30	1200	0.06	0.67	1800	0.08	1.14	2400	0.11	1.79
6-HR TOTAL	0.30			0.37			0.47			0.65	

DAY 2

TIME	PRECIPITATION INCR	TOTAL	TIME	PRECIPITATION INCR	TOTAL	TIME	PRECIPITATION INCR	TOTAL	TIME	PRECIPITATION INCR	TOTAL
0100	0.15	1.94	0700	0.34	3.18	1300	1.09	6.80	1900	0.32	22.46
0200	0.16	2.10	0800	0.37	3.56	1400	1.86	8.66	2000	0.29	22.74
0300	0.17	2.27	0900	0.42	3.98	1500	2.85	11.51	2100	0.26	23.00
0400	0.18	2.45	1000	0.49	4.46	1600	6.56	18.07	2200	0.24	23.24
0500	0.19	2.64	1100	0.57	5.03	1700	2.45	20.52	2300	0.22	23.47
0600	0.20	2.84	1200	0.67	5.70	1800	1.63	22.14	2400	0.21	23.68
6-HR TOTAL	1.05			2.86			16.44			1.54	

DAY 3

TIME	PRECIPITATION INCR	TOTAL	TIME	PRECIPITATION INCR	TOTAL	TIME	PRECIPITATION INCR	TOTAL	TIME	PRECIPITATION INCR	TOTAL
0100	0.13	23.81	0700	0.09	24.57	1300	0.07	25.09	1900	0.06	25.49
0200	0.13	23.95	0800	0.09	24.66	1400	0.07	25.16	2000	0.06	25.55
0300	0.13	24.08	0900	0.09	24.75	1500	0.07	25.23	2100	0.06	25.60
0400	0.13	24.21	1000	0.09	24.84	1600	0.07	25.30	2200	0.06	25.66
0500	0.13	24.35	1100	0.09	24.93	1700	0.07	25.37	2300	0.06	25.71
0600	0.13	24.48	1200	0.09	25.03	1800	0.07	25.44	2400	0.06	25.77
6-HR TOTAL	0.80			0.54			0.41			0.33	

Note that the HMR 52 method uses an area weighted average (for each isohyetal) to determine the average precipitation depth over the entire Delhi Watershed. Given the uncertainty involved with this magnitude of a rainfall event, this averaging is considered acceptable.

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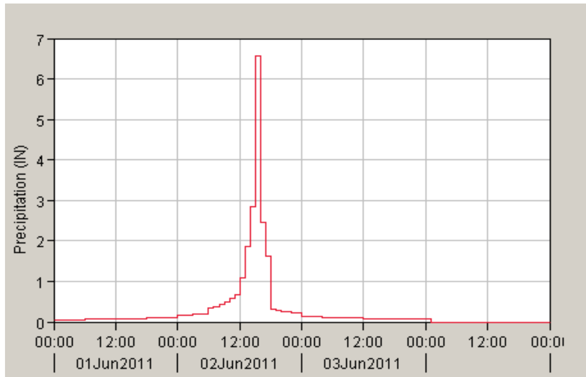
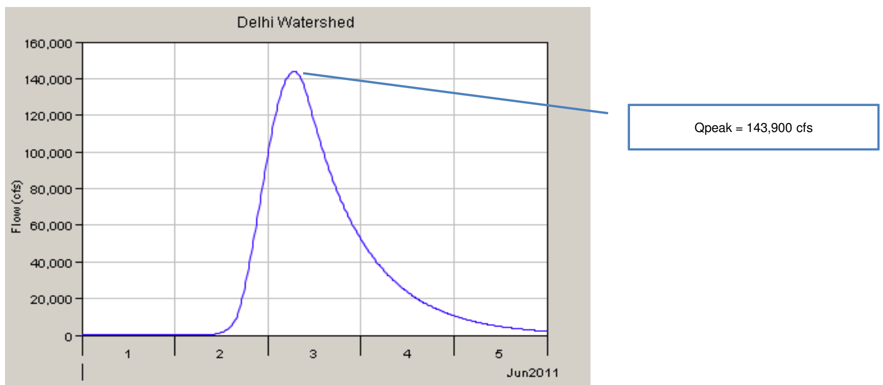
Task 3 **Establish/Verify PMF**

The PMP distribution (aka hyetograph) was then input into HEC-HMS to route the rainfall



The resulting hydrograph is the Probable Maximum Flood (PMF)



As a point of comparison Ref 2 also performed a PMP/PMF analysis

The Ref 2 PMP was very similar but to develop their PMF, Ashton-Barnes only used Day 2 which lowered the peak of the PMF to 132,800 cfs

Ref 2 PMP

Day 1	0.32	0.39	0.50	1.90
Day 2	1.09	2.73	16.63	1.56
Day 3	0.84	0.58	0.44	0.35

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Task 4 **Establish Hydraulic Model**

A hydraulic model of Lake Delhi Dam and the upstream and downstream waterway was used for the following:

- Routing flood hydrographs through Lake Delhi and downstream of the dam
- Analyzing impact of proposed spillway configurations on upstream and downstream peak flood elevations
- Analyzing dam breach scenarios
- Establishing flood elevations for inundation maps

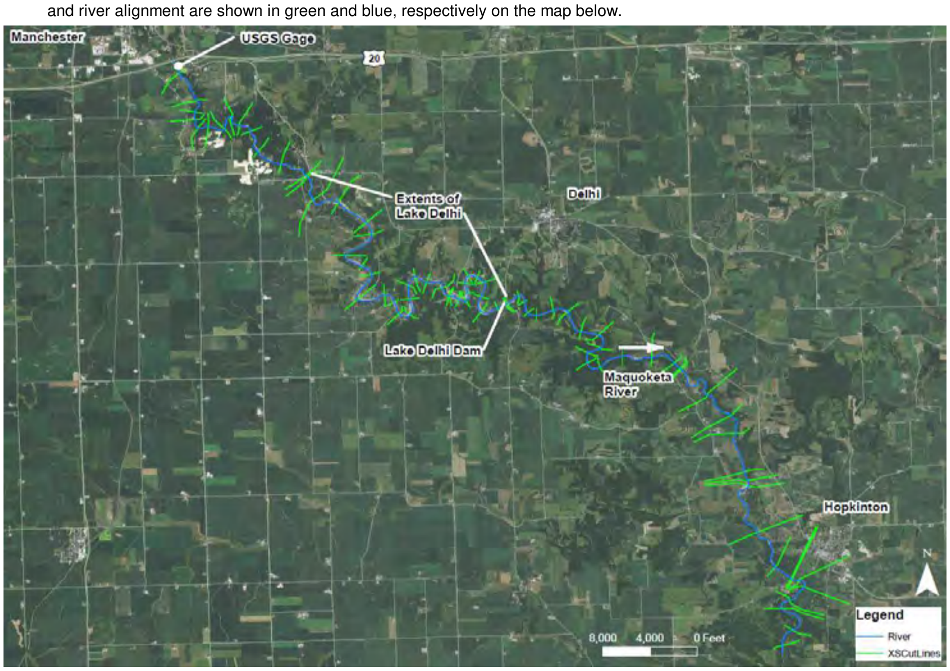
HEC-RAS software was used to establish a hydraulic model of the Maquoketa River and Lake Delhi Dam. HEC-RAS is a USACE developed hydraulic analysis software that uses a series of river/stream cross-sections to model the hydraulics (water surface elevation, velocity, etc.) of a river segment under a user provided flow or hydrograph.

The HEC-RAS model of the Maquoketa River (including Lake Delhi Dam) was obtained from the DNR which had developed the model to analyze the 2010 flood and dam failure. The model was created using HEC-GeoRAS which involves layout out the river geometry in ArcGIS and exporting to HEC-RAS for analysis and importing results back to ArcGIS for mapping. Once the HEC-RAS model was established by the DNR it was checked and calibrated against the 2010 flood/breach event. Ref 4 provides a description of HEC-RAS model development. The model provided a good starting point for the dam reconstruction hydraulic analysis.

The following modifications/additions were made to the DNR HEC-RAS model:

- Cross-sections were regenerated using updated post-breach LiDAR (topography) data
- Cross-sections were extended up to elevations where the PMF was contained within the Cross-Section
- Bridge structures were added at Quarter Road, 295th Street and Hopkinton (using county-supplied Bridge plans)
- The inflow hydrograph near the dam was removed from the model* (see discussion below)
- The HEC-HMS PMF and 100-year hydrographs were input into HEC-RAS
- The unsteady analysis used the Mixed Flow Regime option (to account for critical flows, hydraulic jumps, etc.)
- The starting pool elevation was set to the normal pool of 896.3
- Lake Delhi Dam (represented as inline structure in HEC-RAS) was modified to proposed condition - see task 5

The HEC-RAS model extends from Hwy 20 to downstream of Hopkinton. The model cross-section locations and river alignment are shown in green and blue, respectively on the map below.



*The DNR HEC-RAS model used two inflow hydrographs, one at the upstresam end near the USGS gage and one near Lake Delhi Dam to reflect additional flows coming into the river. Given the relatively small increase in flows estimated (see Task 1 - drainage area ratio multiplier analysis) between the USGS gage and the dam, it was decided to use one inflow hydrograph at the upstream end of the model. This hydrograph represents the tributary drainage area at the dam. Locating the full tributary area hydrograph at the upstream end of the HEC-RAS model allows the hydrograph to be routed through Lake Delhi (9 miles of lake was excluded from travel time estimate in hydrologic analysis) which starts close to the upstream end of the HEC-RAS model.

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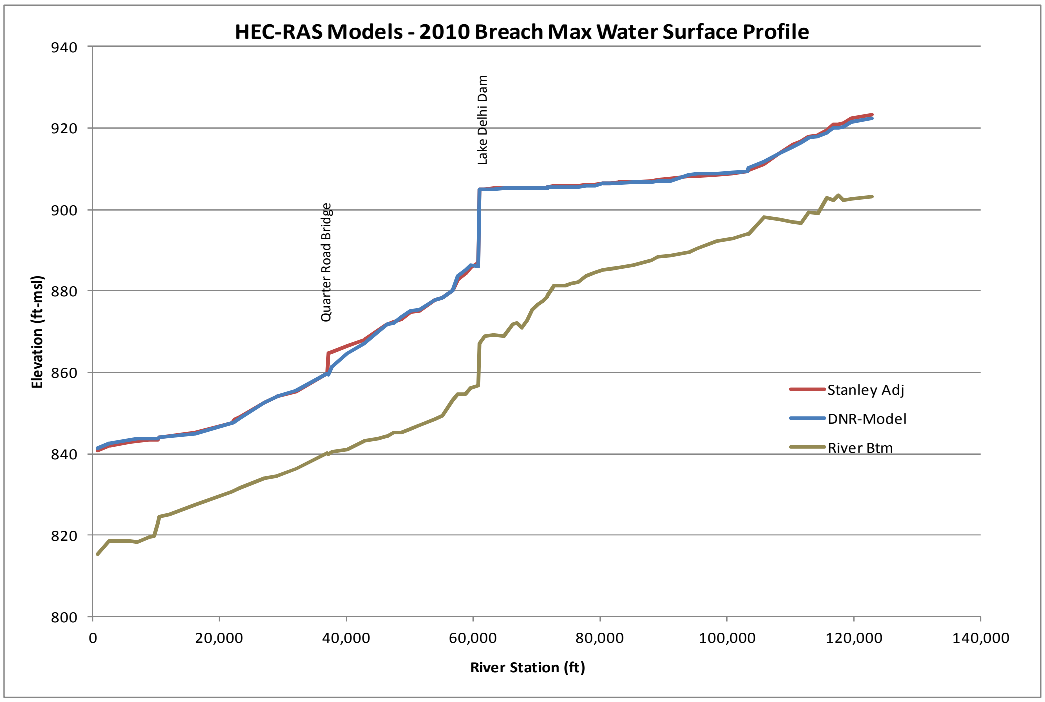
The Stanley Adjusted HEC-RAS model was compared to the DNR model for the 2010 breach and results corresponded fairly closely with the exception of the Quarter Road bridge added to the Stanley Model, which caused a rise in peak water surface.



Task 5 **Design Flood/Hazard Class**

Analysis and Design of the Lake Delhi Dam reconstruction will meet both DNR and FERC standards. The agencies' approach to defining a dam's design flood and hazard classification are shown below:

Iowa DNR (Ref 1)

Freeboard Design Flood

The specified freeboard design flood represents the greatest flood the dam must be designed to accommodate. The flood must be passed without overtopping of the dam and endangering its safety or the dam must be designed to withstand such overflow. Some erosion damage in earth emergency spillways will be tolerated, provided the safety of the dam would not be compromised.

FERC (Ref 3)

Inflow Design Flood (IDF) - The floodflow above which the incremental increase in water surface elevation due to failure of a dam or other water impounding structure is no longer considered to present an unacceptable threat to downstream life or property. The IDF of a dam or other water impounding structure flood hydrograph is used in the design of a dam and its appurtenant works particularly for sizing the spillway and outlet works, and for determining maximum height of a dam, freeboard, and temporary storage requirements.

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The DNR establishes the dam's design flood by its hazard classification

All high hazard dams: the probable maximum flood (PMF).

All moderate hazard dams, and low hazard dams classified as major structures: one-half (0.5) of the probable maximum flood (a flood hydrograph produced by multiplying the ordinates of the PMF hydrograph by a factor of 0.5).

FERC establishes the dam's design flood by a dam breach analysis. A series of floods (fractions of the PMF) are routed through the hydraulic model during a failure and non-failure scenario. The resulting inundations (flooding extents) downstream of the dam are compared and the greatest flood where failure of the dam makes a difference in the hazard (safety and damage) potential downstream is set as the design flood.

The selection of the appropriate IDF for a dam is related to the hazard classification for the dam. The IDF for a dam having a low hazard potential is selected primarily to protect against loss of the dam and its benefits should a failure occur. The IDF for high and significant hazard potential dams is the maximum flood above which there are no significant incremental impacts on downstream life and property.

Lake Delhi Dam is a major structure so it will have to pass either the half PMF or full PMF, depending on its hazard classification.

So, similar to the DNR the design flood is linked to Hazard Classification. However, unlike the DNR, the design flood could be a fraction between the half PMF and full PMF.

The DNR defines Hazard Class by

Low Hazard. Structures located in areas where damages from a failure would be limited to loss of the dam, loss of livestock, damages to farm out-buildings, agricultural lands, and lesser used roads, and where loss of human life is considered unlikely.

Moderate Hazard. Structures located in areas where failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life.

In addition, structures where the dam and its impoundment are of themselves of public importance, such as dams associated with public water supply systems, industrial water supply or public recreation, or which are an integral feature of a private development complex, shall be considered moderate hazard for design and regulatory purposes unless a higher hazard class is warranted by downstream conditions.

High Hazard. Structures located in areas where failure may create a serious threat of loss of human life or result in serious damage to residential, industrial or commercial areas, important public utilities, public buildings, or major transportation facilities.

FERC defines Hazard Class by

Dams conforming to criteria for the low hazard potential category generally are located in rural or agricultural areas where failure may damage farm buildings, limited agricultural land, or township and country roads. Low hazard potential dams have a small storage capacity, the release of which would be confined to the river channel in the event of a failure and therefore would represent no danger to human life.

Significant hazard potential category structures are usually located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life.

Dams in the high hazard potential category are those located where failure may cause serious damage to homes, agricultural, industrial and commercial facilities, important public utilities, main highways, or railroads, and there would be danger to human life.

Definition of hazard classes are very similar between the two guidance documents so a DNR Moderate Hazard Class dam should correspond to a FERC Significant Hazard Class dam.

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The hazard class/design flood evaluation was performed by developing inundation maps of various flow scenarios for a failure (breach) and no failure (non-breach) condition.

Inundation maps were developed using ArcGIS/HEC-GeoRAS/HEC-RAS. The process entailed:

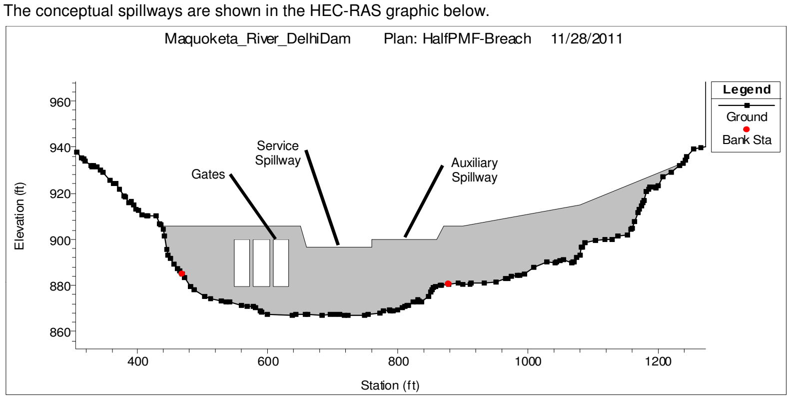
1 Inputting the hydrograph (typically a fraction of the PMF) into HEC-RAS.
2 Running HEC-RAS unsteady flow for a breach and non-breach condition.
3 Exporting breach and non-breach peak water surface profiles to ArcGIS.
4 Using HEC-GeoRAS component of ArcGIS to develop the extent of flooding (i.e. inundation) from the given breach and non-breach event.
5 Comparing inundation extents to determine during what flow conditions a potential breach would increase the downstream hazard.

This evaluation includes several parameters, assumptions, and computations which are discussed below with results following

Service and Auxiliary Spillway Concept
Under previous conditions 3 spillway gates provided flood discharge (ignoring hydro) from Lake Delhi Dam. Once gate capacity was exceeded the dam was overtopped. The addition of a service and auxiliary spillway will supplement both the normal and flood discharge capacity of the dam. Per DNR/FERC guidelines the reconstructed dam will have to provide sufficient hydraulic capacity so the dam can safely passthe design flood.

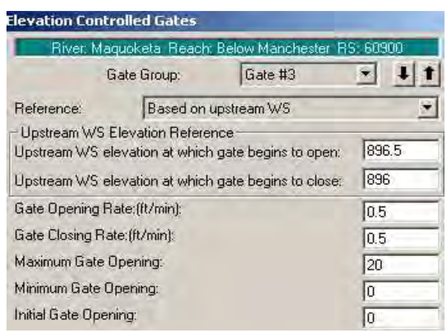
The initial establishiment of hazard class and design flood was performed prior to detailed design of the dam reconstruction. So a conceptual version (rough approximate) of the new service and auxiliary spillways were incorporated into the HEC-RAS model. Evaluation of the design flood/hazard class is focused more on the downstream impact of the dam than the specifics of the spillway so using a spillway approximation is reasonable for this phase of the analysis. The objective is to represent the influence of the spillways on the dam breach and downstream inundation/flooding (i.e. more flow in the downstream channel). Once the spillway designs are established the HEC-RAS model will be updated to reflect, but as long as a decent approximation of the proposed spillways are used the result of the design flood/hazard class evaluation should not change.

The conceptual spillways are shown in the HEC-RAS graphic below.



Normal Pool	896.3
Top of Dam	904.8
Gates	
Gate Crest	879.8
Gate Height	20 ft
Gate Width	25 ft
Service Spillway	
Spillway Crest	896.5
Spillway Width	100 ft
Weir Coeff.	3.4
Auxiliary Spillway	
Spillway Crest	900
Spillway Width	100 ft
Weir Coeff.	3.4

Note that the HEC-RAS model is an unsteady model, which means that a flood hydrograph is run through the dam and waterway. As such the gates are set with with operating parameters to represent how gates would be operated during a flood event. The model starts with the pool at normal elevation and gates closed. As the flows increase and pool elevation rises the gates are fully opened.



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Dam Breach Parameters

The timing and size of the dam breach has a significant impact on the stage and flow increase downstream of the dam. The shorter the time and greater the size, the larger the impact of the breach.

Once the dam was overtopped the 2010 breach took roughly 1.5 hours to fully form (Ref 4)
The extent of the 2010 breach was verified by survey and was roughly 200' wide.

HEC-RAS has the capability to model a dam breach. Inputs for breach modeling are breach geometry, start time, and time to form.
There are a variety of equations that have been developed using past breach case studies to estimate breach parameters.
3 methods for predicting breach parameters were checked against the 2010 breach (FERC from Ref 3, Others from Ref 7).

Empirical Equations (English Units)

Breach Parameters	MacDonald & Langridge-Monopolis (1984)	FERC	Froehlich (2008)
Volume Eroded V_{er} (yd^3)	$V_{er} = 3.264BFF^{0.77}$ (best fit all data) $V_{er} = 0.714BFF^{0.852}$ (rockfill)		
Average Breach Width B_{avg} (ft)	$B_{avg} = \frac{V_{er}}{(H_b \times W_{avg})}$	HD $\leq B_{avg} \leq$ 5HD Earthen, Rockfill.	$B_{avg} = 8.239K_oV_w^{0.32}H_b^{0.04}$ K_o=1.0 for piping K_o=1.3 for overtopping
Breach Side slopes Z_b (H:V)	2.0:1	$\frac{1}{4} \leq Z_b \leq 1$	0.7:1 - piping 1.0:1 - overtopping
Breach Development Time T_f (hr)	$T_f = 0.016V_{er}^{0.364}$	$0.1 \leq T_f \leq 1.0$	$T_f = 3.664\sqrt{\frac{V_w}{gH_b^2}}$

where:

V_w = Full Reservoir Volume
H_w = Full Water Height
H_b = Dam/Breach Height
W_{avg} = Crest Width

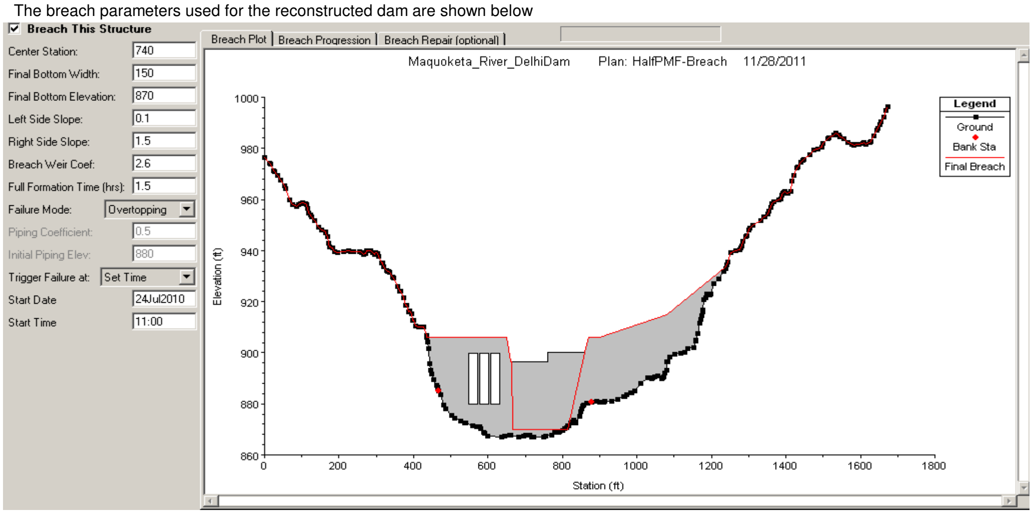
BFF = V_w*H_w (ac-ft^2)

	Delhi
V_w (ac-ft) =	9920
H_w (ft) =	40
H_b (ft) =	35
W_{avg} (ft) =	150

Breach Parameter	Method		
	M&L-M	FERC	Froelich
V_{er}	66781		
B_{avg}	343.4	120	235
Z_b	2	0.5	1
T_f	0.9	0.75	1.8

The Froelich method appears to replicate the parameters of the actual 2010 breach the closest.

The breach parameters used for the reconstructed dam are shown below



For the reconstructed dam the breach parameters were modified slightly to better reflect the proposed condition the breach center was shifted right to center on the service to auxiliary spillway transition. The breach bottom elev. was left as is but the bottom width was shortened to 150', which with a 1.5:1 right side slope gives an average breach width of 175'. This is slightly less than the 200' breach width of 2010, but the geometry fits in better with the proposed condition and still reflects estimated breach parameters. Formation time was left at 1.5 hours. Failure initiation was set to coincide with the peak of the hydrograph which yields the highest flood elevation for the breach scenario.

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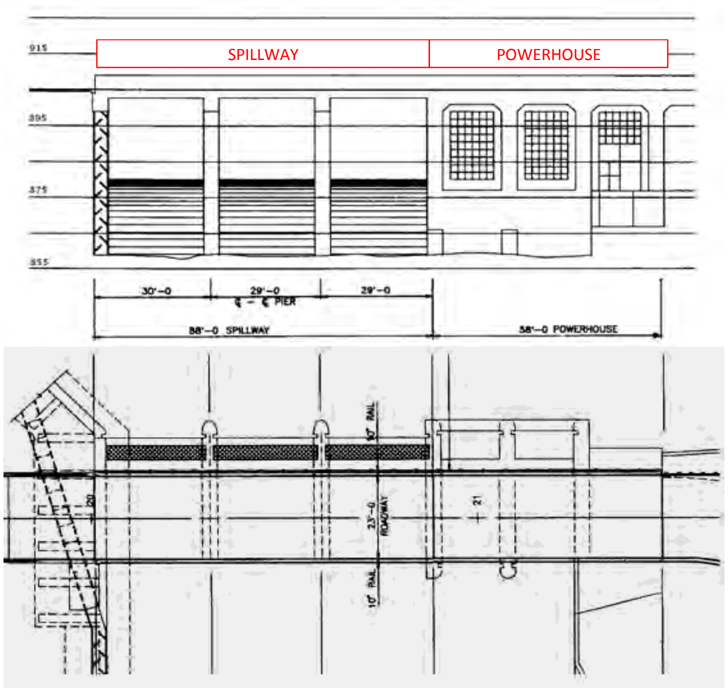
Several forms of failure were considered prior to utilizing the embankment breach. Failure of the powerhouse structure and the spillway structure were also evaluated.

The two most likely failure scenarios of the powerhouse structure would be tipping/sliding of the a section of the structure or an undermining of the foundation and failure/collapse of the above section of structure.



Parameter	Undermine	Tip/Slide
SPILLWAY		
Time to fail	0.5 hrs	0.5 hrs
Width (ft)	88	88
Sides	vertical	vertical
Bottom El.	870	870
POWERHOUSE		
Time to fail	0.5 hrs	0.5 hrs
Width (ft)	58	58
Sides	vertical	vertical
Bottom El.	875	875

Review of reference drawings indicates the powerhouse and spillway are singular structures, although the bridge over the top could be considered an indepedent structure that could remain even with failure of other structures. However to simplify and provide a conservative analysis, both structures were assumed to fully fail, with no debris or remaining structure left in the breach window. The undermine failure was modeled by a "piping failure" in HEC-RAS where the failure starts at the bottom elevation and expands to the top. The Tip/Slide failure was modeled as an overtop where the failure starts at the top and expands to the bottom. Again the failure was timed to occur at the peak of the flood hydrograph.

Flood Profile for 1/2 PMF with Breach

River	Distance D/S	Undermine		Tip/Slide		Embankment
Station	from Dam (ft)	Gated Spwy	Powerhouse	Gated Spwy	Powerhouse	Failure
60797	103	892.58	890.92	892.63	890.94	892.82
59617	1,283	891.54	889.89	891.58	889.91	891.8
58696	2,204	889.98	888.42	890.02	888.44	890.3
57576	3,324	888.22	886.81	888.25	886.83	888.61
56760	4,140	885.35	884.12	885.37	884.13	885.95
55202	5,698	883.95	882.83	883.96	882.84	884.57
53880	7,020	883.39	882.25	883.4	882.25	884.04
51564	9,336	880.92	879.89	880.92	879.9	881.58
50035	10,865	880.72	879.68	880.73	879.68	881.41
48596	12,304	879.12	878.17	879.13	878.17	879.76
47429	13,471	878.31	877.39	878.32	877.39	878.93
46486	14,414	878.1	877.16	878.1	877.16	878.71
45029	15,871	876.5	875.62	876.51	875.62	877.05
42827	18,073	874.68	873.86	874.68	873.87	875.14
40097	20,803	873.48	872.69	873.48	872.7	873.9
37851	23,049	872.25	871.52	872.26	871.52	872.61
37165	23,735	871.89	871.18	871.9	871.18	872.22

Results indicate that the embankment failure would have the highest flood profile (i.e. most critical condition)

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HEC-RAS Flood Modeling

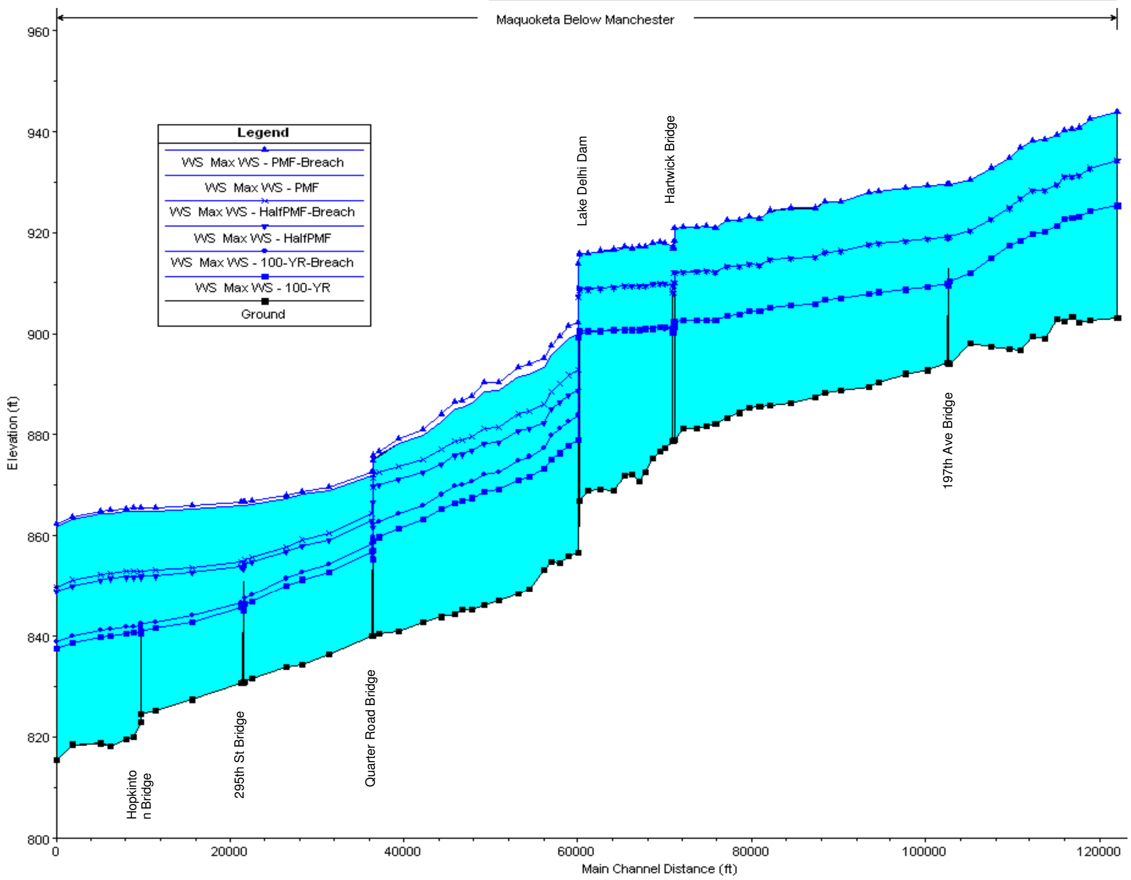
For the hazard class/design flood evaluation, peak flood elevation is the key result. Within HEC-RAS, results can be viewed in several formats. The two most useful for comparing flood scenarios are the river profile and cross-section hydrograph viewer.

The three flood scenarios analyzed initially were

Event	Q_{peak} (cfs)
PMF	143,900
1/2 PMF	71,950
100-YR	31,680

This flow is based off the USGS peak 100-year flow computed for the Manchester Gage, which is 29,610 cfs. To reflect flow at Lake Delhi Dam, this flow was multiplied by the drainage area factor of 1.07 to obtain a 100-year flow of 31,680 cfs at the dam.

The 100-year hydrograph used in HEC-RAS was established by taking the HEC-HMS 100-year hydrograph (developed using the 100-yr/24-hr rainfall) and multiplying by 1.13 to match the peak 100-year flow of 31,860 cfs.



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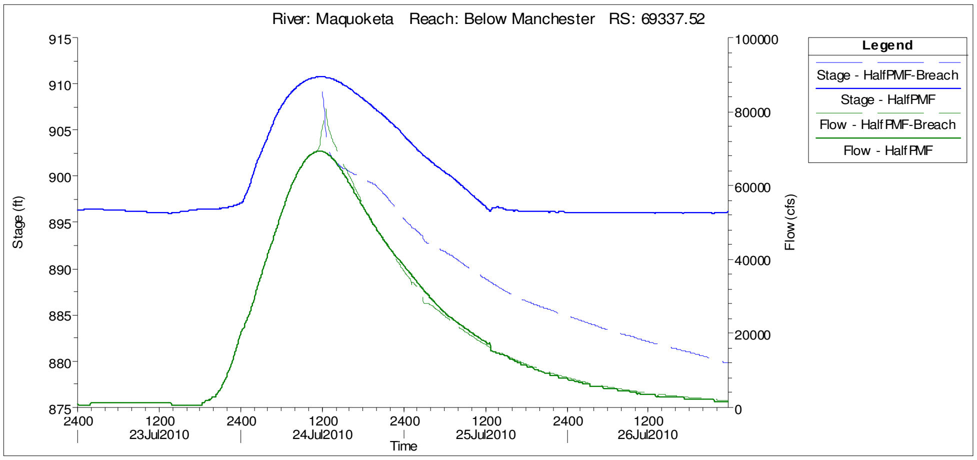
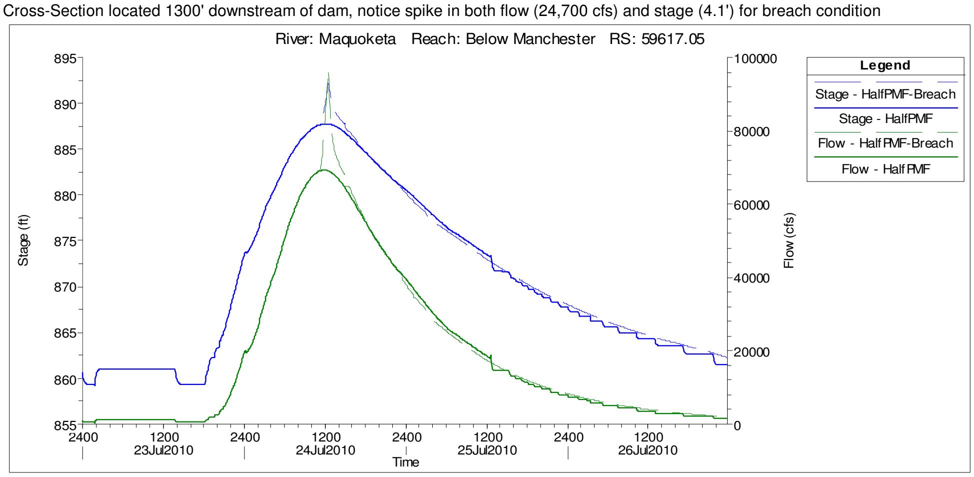
The Max WS profiles indicate that the greatest difference in flood profile for breach vs. non-breach is just downstream of the dam. The breach flood wave dissipates as it flows downstream. The Quarter Road Bridge also provides dissipation of the breach floodwave. Differences in breach vs. non-breach Max WS profiles are less downstream of the bridge. This dissipation can also be viewed in the cross-section stage/flow hydrographs

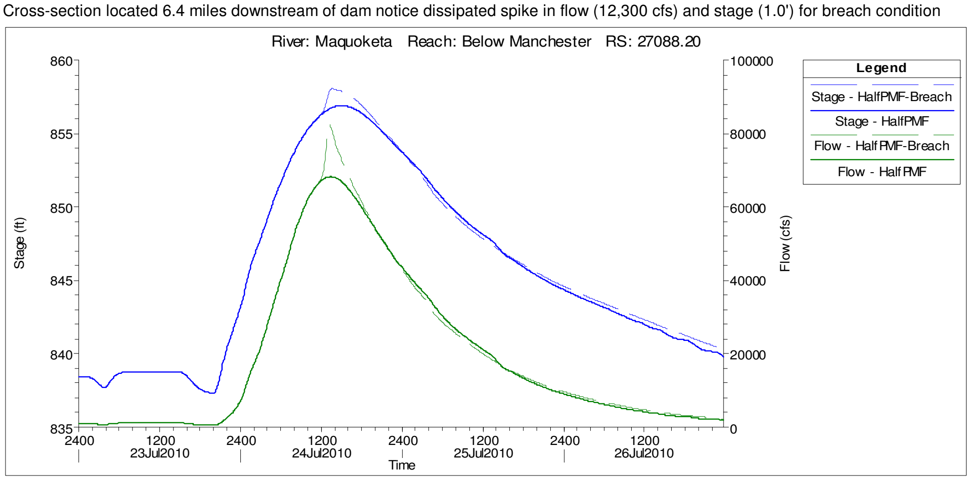
Cross-Section located 8400' upstream of dam, note the drop in WS and spike in flow (10,000 cfs) for the breach scenario.



Cross-Section located 1300' downstream of dam, notice spike in both flow (24,700 cfs) and stage (4.1') for breach condition



Cross-section located 6.4 miles downstream of dam notice dissipated spike in flow (12,300 cfs) and stage (1.0') for breach condition



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Tables summarizing the Flow and Max WS results for the 3 flood conditions are provided on the following pages
River Stations are listed in Upstream to Downstream Order.

HEC-RAS Geometry			PMF					
River Sta	Channel El	Distance (ft)	Q_{peak} (cfs)			Max W.S. Elev (ft)		
			No Breach	Breach	Difference	No Breach	Breach	Difference
122794	903.12	3193	143,285	143,285	0	943.79	943.79	0.00
119601	902.66	1247	143,113	143,113	0	942.39	942.39	0.00
118355	902.13	705	142,854	142,854	0	940.79	940.79	0.00
117650	903.32	866	142,798	142,798	0	940.56	940.56	0.00
116784	902.32	944	142,768	142,768	0	940.42	940.42	0.00
115840	902.85	1476	142,631	142,631	0	939.47	939.47	0.00
114364	898.99	1460	142,497	142,497	0	938.37	938.37	0.00
112904	899.45	1281	142,470	142,470	0	938.21	938.21	0.00
111623	896.76	1232	142,278	142,278	0	936.89	936.89	0.00
110391	896.88	2138	141,909	141,909	0	934.63	934.63	0.00
108252	897.43	2332	141,492	141,492	0	932.9	932.9	0.00
105920	898.14	2522	140,643	140,643	0	930.47	930.47	0.00
103398	893.84	230	140,092	140,092	0	929.44	929.44	0.00
103297					197th Ave Bridge			
103168	894.11	2261	140,144	140,144	0	929.49	929.49	0.00
100907	892.75	2573	140,043	140,043	0	929.22	929.22	0.00
98334	892.14	3014	139,999	139,999	0	928.83	928.83	0.00
95320	890.47	1181	139,928	139,928	0	928.23	928.23	0.00
94139	889.57	3127	139,909	139,909	0	927.85	927.85	0.00
91013	888.71	1851	139,797	139,797	0	926.14	926.14	0.00
89162	888.37	1152	139,778	139,778	0	926	926	0.00
88010	887.45	2752	139,725	139,725	0	924.9	924.9	0.00
85258	886.26	2488	139,706	139,706	0	924.92	924.92	0.00
82771	885.77	1206	139,695	139,695	0	924.3	924.3	0.00
81565	885.49	1142	139,661	139,635	-27	922.71	922.71	0.00
80423	885.18	1135	139,671	139,670	-1	923.24	923.24	0.00
79288	884.41	1483	139,661	139,588	-73	922.62	922.62	0.00
77805	883.49	1240	139,668	139,602	-65	922.47	922.47	0.00
76564	882.22	984	139,644	139,570	-75	920.99	920.99	0.00
75581	881.74	1122	139,649	139,566	-83	921.32	921.32	0.00
74458	881.24	1722	139,643	139,572	-71	921.1	921.11	0.01
72737	881.25	952	139,645	139,569	-76	921.1	921.1	0.00
71785	878.98	151	139,641	139,570	-72	920.78	920.78	0.00
71710					Hartwick Bridge			
71635	878.67	802	139,615	139,569	-46	917.08	917.1	0.02
70833	877.38	669	139,626	139,566	-59	917.92	917.93	0.01
70164	876.67	826	139,625	139,566	-60	918.14	918.15	0.01
69338	875.48	874	139,621	139,548	-73	917.74	917.75	0.01
68463	872.58	743	139,617	139,526	-91	917.12	917.13	0.01
67720	870.83	788	139,620	139,523	-98	917.06	917.07	0.01
66932	872.17	755	139,620	139,497	-123	916.87	916.88	0.01
66177	871.93	1387	139,617	139,494	-124	917.01	917.02	0.01
64790	868.85	1488	139,618	139,462	-156	916.53	916.55	0.02
63302	869.17	1516	139,616	139,431	-185	916.42	916.43	0.01
61786	868.81	783	139,615	139,396	-219	915.92	915.94	0.02
61003	866.93	206	139,615	139,364	-251	916.05	916.06	0.01
60900					Lake Delhi Dam			
60797	856.62	1180	139,586	160,629	21,043	899.92	902.16	2.24
59617	856.09	921	139,580	160,376	20,797	899.13	901.42	2.29
58696	854.52	1119	139,573	159,878	20,305	897.47	899.58	2.11
57576	854.78	816	139,554	159,822	20,269	895.7	897.68	1.98
56760	853.11	1557	139,513	159,407	19,893	893.27	895.01	1.74
55202	849.4	1322	139,503	157,626	18,123	892.1	893.83	1.73
53880	848.44	2316	139,492	157,564	18,072	891.5	893.23	1.73
51564	847.04	1528	139,451	155,601	16,150	888.76	890.41	1.65
50035	846.1	1439	139,456	155,537	16,081	888.65	890.32	1.67
48596	845.13	1168	139,445	155,059	15,614	886.14	887.69	1.55
47429	845.27	943	139,444	154,851	15,407	885.21	886.67	1.46
46486	844.35	1457	139,440	154,850	15,410	885	886.47	1.47
45029	843.77	2202	139,436	154,701	15,265	882.74	884.03	1.29
42827	842.98	2729	139,430	154,415	14,985	879.97	881.02	1.05
40097	841.02	2246	139,425	154,141	14,716	878.28	879.22	0.94
37851	840.48	686	139,422	154,102	14,681	876	876.66	0.66
37165	839.98	170	139,422	154,088	14,666	875.3	875.86	0.56
37080					Quarter Road Bridge			
36995	840.16	4909	135,554	152,346	16,791	871.84	872.68	0.84
32085	836.41	3037	131,454	138,369	6,916	868.9	869.48	0.58
29049	834.42	1961	130,095	135,201	5,105	868.07	868.66	0.59
27088	834.04	3792	129,017	133,684	4,667	867.42	868.02	0.60
23296	831.71	864	127,553	131,734	4,181	866.32	866.95	0.63
22433	830.86	449	127,351	131,380	4,029	866.09	866.73	0.64
22180					295th Street Bridge			
21984	830.78	5750	127,283	131,380	4,097	866.06	866.7	0.64
16234	827.37	4078	126,742	130,627	3,884	865.3	865.96	0.66
12156	825.22	1731	126,606	130,463	3,857	864.94	865.61	0.67
10425	824.55	109	126,540	130,422	3,882	864.84	865.51	0.67
10370					Hopkinton Bridge			
10316	823.07	643	126,568	130,455	3,887	864.84	865.51	0.67
9673	819.82	871	126,562	130,447	3,885	864.82	865.49	0.67
8802	819.52	1849	126,539	130,418	3,878	864.77	865.45	0.68
6953	818.29	1218	126,534	130,397	3,863	864.53	865.2	0.67
5735	818.68	3207	126,526	130,395	3,869	864.32	864.99	0.67
2528	818.58	1826	126,525	130,393	3,868	863.16	863.83	0.67
702	815.37	702	126,524	130,391	3,867	861.75	862.41	0.66

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HEC-RAS Geometry			Half PMF					
River Sta	Channel El	Distance (ft)	Q_{peak} (cfs)			Max W.S. Elev (ft)		
			No Breach	Breach	Difference	No Breach	Breach	Difference
122794	903.12	3193	71,700	71,700	0	934.19	934.19	0.00
119601	902.66	1247	71,624	71,624	0	932.91	932.91	0.00
118355	902.13	705	71,535	71,535	0	931.4	931.4	0.00
117650	903.32	866	71,524	71,524	0	931.26	931.26	0.00
116784	902.32	944	71,502	71,502	0	931.1	931.1	0.00
115840	902.85	1476	71,423	71,423	0	929.51	929.51	0.00
114364	898.99	1460	71,358	71,358	0	928.44	928.44	0.00
112904	899.45	1281	71,357	71,357	0	928.26	928.26	0.00
111623	896.76	1232	71,269	71,269	0	926.74	926.74	0.00
110391	896.88	2138	71,111	71,111	0	924.8	924.8	0.00
108252	897.43	2332	70,803	70,803	0	922.79	922.79	0.00
105920	898.14	2522	70,243	70,243	0	920.36	920.36	0.00
103398	893.84	230	69,872	69,872	0	919.17	919.17	0.00
103297					197th Ave Bridge			
103168	894.11	2261	69,821	69,821	0	919.09	919.09	0.00
100907	892.75	2573	69,736	69,736	0	918.77	918.77	0.00
98334	892.14	3014	69,655	69,655	0	918.34	918.34	0.00
95320	890.47	1181	69,598	69,598	0	917.91	917.91	0.00
94139	889.57	3127	69,597	69,597	0	917.65	917.65	0.00
91013	888.71	1851	69,495	69,494	-1	916.44	916.44	0.00
89162	888.37	1152	69,493	69,492	-2	916.27	916.27	0.00
88010	887.45	2752	69,405	69,433	28	915.24	915.24	0.00
85258	886.26	2488	69,401	69,446	45	915.14	915.14	0.00
82771	885.77	1206	69,372	69,439	67	914.73	914.73	0.00
81565	885.49	1142	69,347	69,435	87	913.7	913.7	0.00
80423	885.18	1135	69,354	69,430	76	913.95	913.94	-0.01
79288	884.41	1483	69,332	69,427	95	913.4	913.39	-0.01
77805	883.49	1240	69,332	69,417	86	913.33	913.32	-0.01
76564	882.22	984	69,318	69,386	68	912.23	912.22	-0.01
75581	881.74	1122	69,317	69,382	65	912.44	912.42	-0.02
74458	881.24	1722	69,317	69,362	45	912.34	912.32	-0.02
72737	881.25	952	69,315	69,336	21	912.3	912.28	-0.02
71785	878.98	151	69,312	69,333	20	912.1	912.08	-0.02
71710					Hartwick Bridge			
71635	878.67	802	69,282	69,255	-27	909.45	909.42	-0.03
70833	877.38	669	69,290	69,250	-40	909.88	909.85	-0.03
70164	876.67	826	69,286	69,247	-39	909.96	909.94	-0.02
69338	875.48	874	69,282	69,212	-70	909.76	909.73	-0.03
68463	872.58	743	69,282	69,208	-74	909.51	909.48	-0.03
67720	870.83	788	69,279	69,168	-111	909.42	909.39	-0.03
66932	872.17	755	69,284	69,165	-119	909.33	909.3	-0.03
66177	871.93	1387	69,283	69,121	-162	909.37	909.33	-0.04
64790	868.85	1488	69,281	69,072	-208	909.14	909.1	-0.04
63302	869.17	1516	69,279	69,024	-255	909.02	908.98	-0.04
61786	868.81	783	69,279	68,971	-308	908.82	908.78	-0.04
61003	866.93	206	69,278	68,921	-357	908.85	908.81	-0.04
60900					Lake Delhi Dam			
60797	856.62	1180	69,242	94,152	24,910	888.74	892.82	4.08
59617	856.09	921	69,226	93,950	24,723	887.75	891.8	4.05
58696	854.52	1119	69,216	93,757	24,541	886.4	890.3	3.90
57576	854.78	816	69,204	93,356	24,151	885.03	888.61	3.58
56760	853.11	1557	69,154	92,912	23,758	882.57	885.95	3.38
55202	849.4	1322	69,114	91,109	21,995	881.37	884.57	3.20
53880	848.44	2316	69,098	91,026	21,928	880.77	884.04	3.27
51564	847.04	1528	69,032	87,871	18,839	878.55	881.58	3.03
50035	846.1	1439	69,030	87,810	18,780	878.31	881.41	3.10
48596	845.13	1168	68,996	86,933	17,937	876.87	879.76	2.89
47429	845.27	943	68,982	86,658	17,676	876.14	878.93	2.79
46486	844.35	1457	68,975	86,638	17,663	875.88	878.71	2.83
45029	843.77	2202	68,956	86,007	17,052	874.35	877.05	2.70
42827	842.98	2729	68,923	85,435	16,511	872.54	875.14	2.60
40097	841.02	2246	68,905	85,226	16,321	871.29	873.9	2.61
37851	840.48	686	68,902	85,120	16,218	870.06	872.61	2.55
37165	839.98	170	68,902	85,110	16,209	869.7	872.22	2.52
37080					Quarter Road Bridge			
36995	840.16	4909	68,697	84,251	15,554	862.95	864.52	1.57
32085	836.41	3037	67,965	81,349	13,385	859.19	860.44	1.25
29049	834.42	1961	67,493	80,245	12,752	858.01	859.15	1.14
27088	834.04	3792	66,942	79,206	12,264	856.91	857.92	1.01
23296	831.71	864	65,703	71,331	5,628	854.81	855.71	0.90
22433	830.86	449	65,437	70,627	5,190	854.41	855.33	0.92
22180					295th Street Bridge			
21984	830.78	5750	65,033	69,906	4,873	853.89	854.87	0.98
16234	827.37	4078	64,374	68,874	4,499	852.61	853.66	1.05
12156	825.22	1731	64,214	68,614	4,400	852.06	853.14	1.08
10425	824.55	109	64,195	68,563	4,368	851.9	852.99	1.09
10370					Hopkinton Bridge			
10316	823.07	643	64,181	68,563	4,382	851.85	852.94	1.09
9673	819.82	871	64,178	68,559	4,381	851.81	852.91	1.10
8802	819.52	1849	64,174	68,555	4,380	851.75	852.84	1.09
6953	818.29	1218	64,158	68,537	4,379	851.45	852.55	1.10
5735	818.68	3207	64,161	68,535	4,373	851.24	852.33	1.09
2528	818.58	1826	64,157	68,529	4,372	850.23	851.32	1.09
702	815.37	702	64,155	68,528	4,372	848.88	849.96	1.08

Stanley Consultants INC.

				Job No.	23601
Computed by	A. Judd	Date	9/28/2011	Subject	Lake Delhi Dam
Checked by	M. Weber	Date	10/12/2011		Hydrology/Hydraulics
Approved by		Date			Computation Summary

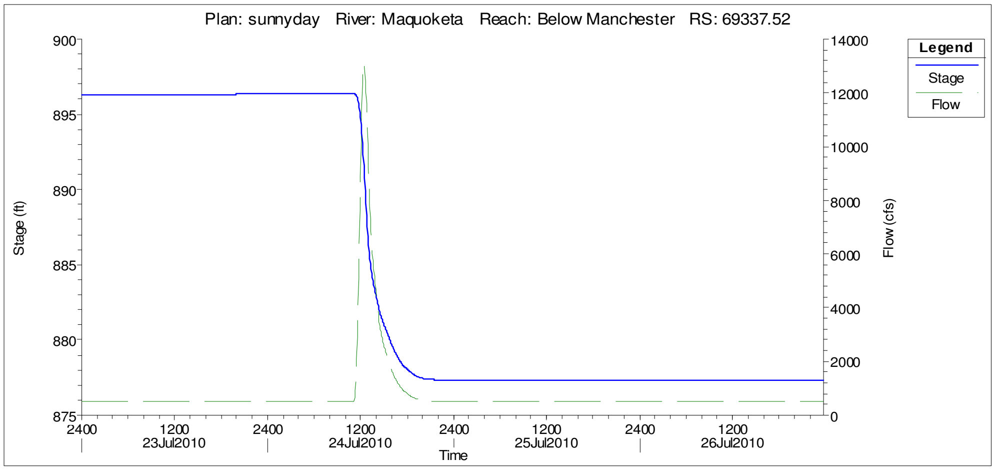
HEC-RAS Geometry			100-YR					
River Sta	Channel El	Distance (ft)	Q_{peak} (cfs)			Max W.S. Elev (ft)		
			No Breach	Breach	Difference	No Breach	Breach	Difference
122794	903.12	3193	31,699	31,699	0	925.41	925.41	0.00
119601	902.66	1247	31,687	31,687	0	924.34	924.34	0.00
118355	902.13	705	31,667	31,667	0	923.14	923.14	0.00
117650	903.32	866	31,663	31,663	0	922.88	922.88	0.00
116784	902.32	944	31,661	31,661	0	922.68	922.68	0.00
115840	902.85	1476	31,649	31,649	0	921.29	921.29	0.00
114364	898.99	1460	31,639	31,639	0	920.06	920.06	0.00
112904	899.45	1281	31,638	31,638	0	919.82	919.82	0.00
111623	896.76	1232	31,632	31,632	0	918.39	918.39	0.00
110391	896.88	2138	31,626	31,626	0	917.31	917.31	0.00
108252	897.43	2332	31,580	31,580	0	914.95	914.95	0.00
105920	898.14	2522	31,444	31,444	0	912.07	912.07	0.00
103398	893.84	230	31,231	31,231	0	910.26	910.26	0.00
103297					197th Ave Bridge			
103168	894.11	2261	31,096	31,096	0	909.82	909.82	0.00
100907	892.75	2573	30,967	30,967	0	909.22	909.22	0.00
98334	892.14	3014	30,899	30,899	0	908.65	908.65	0.00
95320	890.47	1181	30,868	30,867	-1	908.21	908.21	0.00
94139	889.57	3127	30,848	30,860	12	908	908	0.00
91013	888.71	1851	30,812	30,813	0	907.04	907.04	0.00
89162	888.37	1152	30,794	30,808	13	906.72	906.72	0.00
88010	887.45	2752	30,757	30,803	45	905.91	905.91	0.00
85258	886.26	2488	30,737	30,800	64	905.59	905.59	0.00
82771	885.77	1206	30,730	30,794	64	905.22	905.22	0.00
81565	885.49	1142	30,702	30,785	83	904.46	904.45	-0.01
80423	885.18	1135	30,711	30,783	73	904.46	904.44	-0.02
79288	884.41	1483	30,683	30,771	89	903.75	903.73	-0.02
77805	883.49	1240	30,682	30,762	80	903.63	903.6	-0.03
76564	882.22	984	30,646	30,733	87	902.58	902.54	-0.04
75581	881.74	1122	30,655	30,721	66	902.72	902.68	-0.04
74458	881.24	1722	30,654	30,704	50	902.65	902.6	-0.05
72737	881.25	952	30,653	30,683	30	902.56	902.51	-0.05
71785	878.98	151	30,650	30,659	9	902.39	902.34	-0.05
71710					Hartwick Bridge			
71635	878.67	802	30,623	30,586	-37	900.94	900.88	-0.06
70833	877.38	669	30,626	30,583	-43	901.14	901.08	-0.06
70164	876.67	826	30,629	30,580	-48	901.17	901.11	-0.06
69338	875.48	874	30,625	30,552	-73	901.05	900.98	-0.07
68463	872.58	743	30,625	30,522	-103	900.94	900.87	-0.07
67720	870.83	788	30,623	30,520	-103	900.85	900.78	-0.07
66932	872.17	755	30,622	30,489	-133	900.78	900.71	-0.07
66177	871.93	1387	30,624	30,488	-136	900.78	900.71	-0.07
64790	868.85	1488	30,622	30,421	-200	900.65	900.58	-0.07
63302	869.17	1516	30,620	30,384	-236	900.52	900.44	-0.08
61786	868.81	783	30,619	30,344	-275	900.43	900.35	-0.08
61003	866.93	206	30,619	30,305	-314	900.43	900.35	-0.08
60900					Lake Delhi Dam			
60797	856.62	1180	30,612	51,053	20,442	878.96	883.94	4.98
59617	856.09	921	30,608	50,306	19,698	877.76	882.75	4.99
58696	854.52	1119	30,603	50,174	19,571	876.28	881.2	4.92
57576	854.78	816	30,599	49,765	19,166	875.14	879.9	4.76
56760	853.11	1557	30,583	49,470	18,886	873.28	877.38	4.10
55202	849.4	1322	30,568	48,656	18,088	871.78	875.76	3.98
53880	848.44	2316	30,558	48,159	17,601	871	874.98	3.98
51564	847.04	1528	30,531	46,175	15,644	869.08	872.63	3.55
50035	846.1	1439	30,519	45,688	15,169	868.63	872.23	3.60
48596	845.13	1168	30,506	45,159	14,653	867.42	870.79	3.37
47429	845.27	943	30,504	44,687	14,183	866.89	870.14	3.25
46486	844.35	1457	30,501	44,663	14,162	866.45	869.72	3.27
45029	843.77	2202	30,494	44,258	13,764	865.29	868.24	2.95
42827	842.98	2729	30,473	42,349	11,876	863.18	865.97	2.79
40097	841.02	2246	30,452	40,997	10,545	861.33	864.31	2.98
37851	840.48	686	30,448	40,949	10,501	859.68	862.84	3.16
37165	839.98	170	30,448	40,922	10,475	859.07	862.32	3.25
37080					Quarter Road Bridge			
36995	840.16	4909	30,430	40,689	10,259	856.85	858.48	1.63
32085	836.41	3037	30,304	38,885	8,581	852.81	854.25	1.44
29049	834.42	1961	30,240	38,329	8,090	851.26	852.75	1.49
27088	834.04	3792	30,206	38,123	7,917	850.06	851.5	1.44
23296	831.71	864	30,070	37,554	7,484	846.87	848.19	1.32
22433	830.86	449	30,031	37,449	7,417	846.27	847.54	1.27
22180					295th Street Bridge			
21984	830.78	5750	29,904	37,024	7,119	845.57	846.7	1.13
16234	827.37	4078	29,284	33,965	4,681	842.91	844.06	1.15
12156	825.22	1731	28,944	32,815	3,871	841.59	842.91	1.32
10425	824.55	109	28,893	32,646	3,753	841.11	842.53	1.42
10370					Hopkinton Bridge			
10316	823.07	643	28,848	32,483	3,635	840.76	842.06	1.30
9673	819.82	871	28,831	32,457	3,626	840.65	841.97	1.32
8802	819.52	1849	28,829	32,437	3,608	840.56	841.89	1.33
6953	818.29	1218	28,814	32,422	3,608	840.15	841.5	1.35
5735	818.68	3207	28,815	32,412	3,597	839.9	841.26	1.36
2528	818.58	1826	28,811	32,402	3,591	838.75	840.15	1.40
702	815.37	702	28,810	32,402	3,591	837.48	838.88	1.40

Stanley Consultants INC.

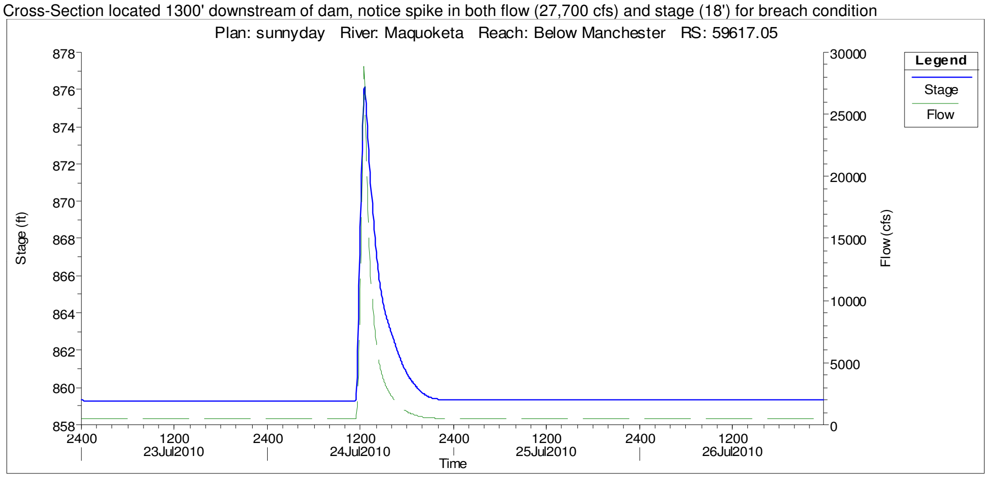
				Job No.	23601
Computed by	A. Judd	Date	9/28/2011	Subject	Lake Delhi Dam
Checked by	M. Weber	Date	10/12/2011		Hydrology/Hydraulics
Approved by		Date			Computation Summary

A Sunny Day Breach was also included in the analysis. This represents a dam failure during normal flow conditions. The HEC-RAS model was set to the normal pool with a constant flow of 500 cfs with the dam breach/failure set to occur midway through the model's time period. The hydrograph results are shown below.

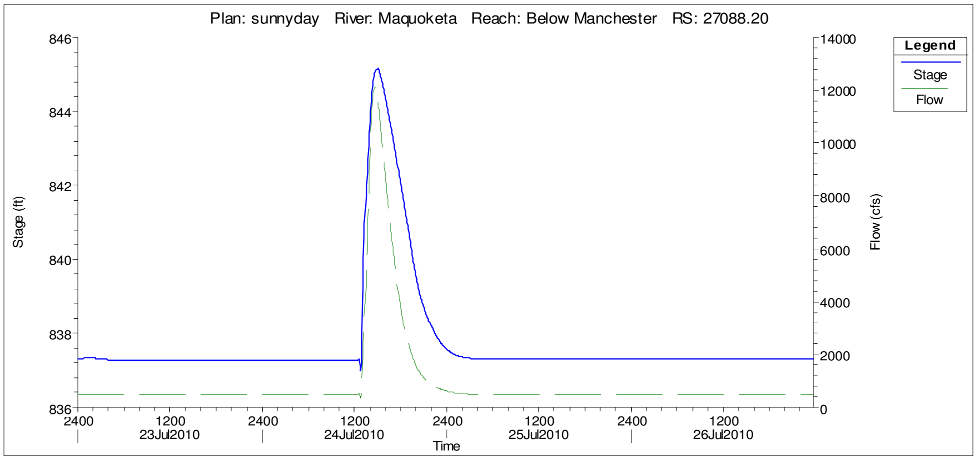
Cross-Section located 8400' upstream of dam, note the drop in WS and spike in flow (13,000 cfs) for the breach scenario.



Cross-Section located 1300' downstream of dam, notice spike in both flow (27,700 cfs) and stage (18') for breach condition



Cross-section located 6.4 miles downstream of dam notice dissipated spike in flow (12,300 cfs) and stage (8') for breach condition



Several breach scenarios were analyzed for the Sunny Day, both a piping failure (failure starts within the embankment) and an overtopping failure (failure starts at the top of the embankment) were used with a 1.5 hour and 1 hour breach formation time. The piping failure resulted in a lower Max W.S. and the 1 hour vs. 1.5 hour did not make an appreciable difference in Max W.S., so the breach/failure was left consistent between the Sunny Day and Flood events.

Something to note, the spike in flow was roughly the same between Sunny Day and Half PMF, but the stage increase for the Sunny Day was much greater due to the narrower channel width at the bottom. This quick and significant increase could be of concern if structures or facilities are located within the Sunny Day Breach's inundation limits. A Table of HEC-RAS results is provided.

Stanley Consultants INC.

				Job No.	23601
Computed by	A. Judd	Date	9/28/2011	Subject	Lake Delhi Dam
Checked by	M. Weber	Date	10/12/2011		Hydrology/Hydraulics
Approved by		Date			Computation Summary

HEC-RAS Geometry			Sunny Day Breach			
River Sta	Channel El	Distance (ft)	Q_{peak} (cfs)	Max W.S. (ft) No Breach	Max W.S. (ft) Breach	Difference
122794	903.12	3193	500	908.44	908.44	0.0
119601	902.66	1247	500	908.27	908.27	0.0
118355	902.13	705	500	908.14	908.14	0.0
117650	903.32	866	500	907.98	907.98	0.0
116784	902.32	944	500	907.83	907.83	0.0
115840	902.85	1476	500	906.32	906.32	0.0
114364	898.99	1460	458	904.17	904.17	0.0
112904	899.45	1281	510	904.01	904.01	0.0
111623	896.76	1232	502	903.88	903.88	0.0
110391	896.88	2138	520	903.83	903.83	0.0
108252	897.43	2332	518	902.73	902.73	0.0
105920	898.14	2522	507	899.8	899.81	0.0
103398	893.84	230	523	897.23	897.22	0.0
103297				197th Ave Bridge		
103168	894.11	2261	523	897.14	897.12	0.0
100907	892.75	2573	499	896.71	896.64	-0.1
98334	892.14	3014	499	896.53	896.43	-0.1
95320	890.47	1181	498	896.49	896.39	-0.1
94139	889.57	3127	498	896.49	896.39	-0.1
91013	888.71	1851	497	896.48	896.38	-0.1
89162	888.37	1152	497	896.48	896.37	-0.1
88010	887.45	2752	497	896.48	896.37	-0.1
85258	886.26	2488	496	896.47	896.37	-0.1
82771	885.77	1206	496	896.47	896.37	-0.1
81565	885.49	1142	495	896.47	896.36	-0.1
80423	885.18	1135	495	896.47	896.36	-0.1
79288	884.41	1483	495	896.47	896.36	-0.1
77805	883.49	1240	494	896.47	896.36	-0.1
76564	882.22	984	494	896.47	896.36	-0.1
75581	881.74	1122	494	896.47	896.36	-0.1
74458	881.24	1722	494	896.47	896.36	-0.1
72737	881.25	952	494	896.47	896.36	-0.1
71785	878.98	151	493	896.47	896.36	-0.1
71710				HartwickBridge		
71635	878.67	802	493	896.46	896.36	-0.1
70833	877.38	669	493	896.46	896.36	-0.1
70164	876.67	826	492	896.46	896.36	-0.1
69338	875.48	874	492	896.46	896.36	-0.1
68463	872.58	743	492	896.46	896.36	-0.1
67720	870.83	788	492	896.46	896.36	-0.1
66932	872.17	755	492	896.46	896.36	-0.1
66177	871.93	1387	491	896.46	896.36	-0.1
64790	868.85	1488	491	896.46	896.36	-0.1
63302	869.17	1516	490	896.46	896.36	-0.1
61786	868.81	783	490	896.46	896.36	-0.1
61003	866.93	206	490	896.46	896.36	-0.1
60900				Lake Delhi Dam		
60797	856.62	1180	29,244	859.93	877.44	17.5
59617	856.09	921	28,027	859.35	876.04	16.7
58696	854.52	1119	27,193	859.02	874.29	15.3
57576	854.78	816	27,124	857.47	872.94	15.5
56760	853.11	1557	26,465	855.49	870.6	15.1
55202	849.4	1322	25,007	852.96	868.53	15.6
53880	848.44	2316	23,841	851.79	867.28	15.5
51564	847.04	1528	20,444	850.31	864.93	14.6
50035	846.1	1439	19,662	849.46	864.29	14.8
48596	845.13	1168	18,699	848.52	862.92	14.4
47429	845.27	943	18,558	848.03	862.42	14.4
46486	844.35	1457	18,309	847.7	861.9	14.2
45029	843.77	2202	17,917	847.17	860.89	13.7
42827	842.98	2729	16,590	845.91	858.59	12.7
40097	841.02	2246	15,614	844.23	856.67	12.4
37851	840.48	686	15,472	843.46	855.19	11.7
37165	839.98	170	15,466	843.1	854.57	11.5
37080				Quarter Road Bridge		
36995	840.16	4909	15,460	842.9	853.48	10.6
32085	836.41	3037	13,234	839.3	848.67	9.4
29049	834.42	1961	12,185	838.21	846.59	8.4
27088	834.04	3792	11,298	837.33	845.22	7.9
23296	831.71	864	11,072	834.2	841.95	7.8
22433	830.86	449	10,662	833.64	841.41	7.8
22180				295th Street Bridge		
21984	830.78	5750	10,662	833.37	840.9	7.5
16234	827.37	4078	9,055	830.31	837.9	7.6
12156	825.22	1731	9,079	829.53	836.16	6.6
10425	824.55	109	9,143	827.19	832.61	5.4
10370				Hopkinton Bridge		
10316	823.07	643	9,168	827.12	832.68	5.6
9673	819.82	871	8,944	827.08	832.38	5.3
8802	819.52	1849	8,898	827.07	832.24	5.2
6953	818.29	1218	8,817	827.05	831.75	4.7
5735	818.68	3207	8,779	827.03	831.44	4.4
2528	818.58	1826	8,736	821.77	829.34	7.6
702	815.37		8,731	818.26	827.74	9.5

Stanley Consultants INC.

				Job No.	23601
Computed by	A. Judd	Date	9/28/2011	Subject	Lake Delhi Dam
Checked by	M. Weber	Date	10/12/2011		Hydrology/Hydraulics
Approved by		Date			Computation Summary

The time for the peak flood stage to travel downstream was computed by determining the time difference between peaks between cross-sections. Travel times at HEC-RAS model cross-sections are shown in the following table.

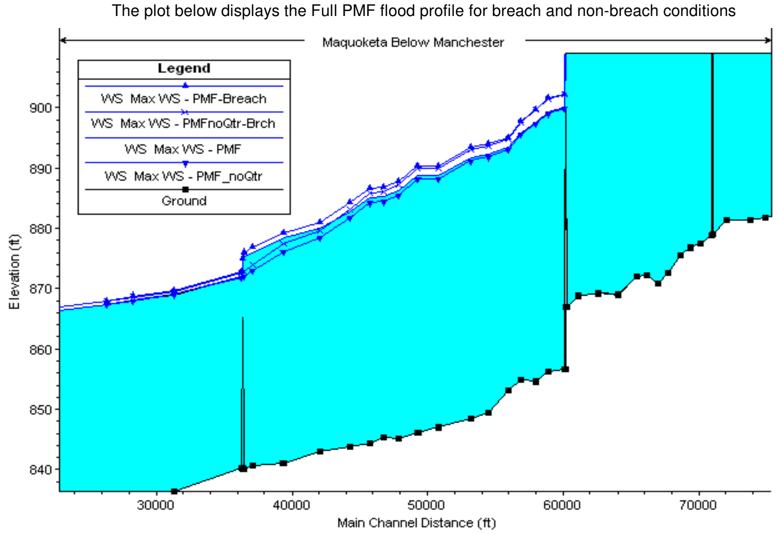
Maquoketa River Station	Distance Downstream (mi)	Peak Flood Stage Travel Time (Hours) PMF No Breach	PMF Breach	1/2 PMF No Breach	1/2 PMF Breach	100 YR No Breach	100 YR Breach
60900	Lake Delhi Dam						
60797	0.0	0.0	0.0	0.0	0.0	0.0	0.0
59617	0.2	0.0	0.0	0.1	0.0	0.0	0.0
58696	0.4	0.0	0.0	0.1	0.0	0.1	0.0
57576	0.6	0.1	0.0	0.2	0.0	0.1	0.0
56760	0.8	0.1	0.0	0.2	0.0	0.1	0.0
55202	1.1	0.2	0.1	0.3	0.1	0.3	0.1
53880	1.3	0.2	0.1	0.3	0.1	0.3	0.1
51564	1.7	0.3	0.2	0.5	0.2	0.5	0.2
50035	2.0	0.3	0.2	0.5	0.2	0.5	0.2
48596	2.3	0.3	0.2	0.6	0.3	0.6	0.3
47429	2.5	0.3	0.2	0.7	0.3	0.7	0.3
46486	2.7	0.3	0.2	0.7	0.3	0.7	0.3
45029	3.0	0.3	0.3	0.8	0.3	0.8	0.3
42827	3.4	0.3	0.3	0.8	0.3	0.8	0.4
40097	3.9	0.3	0.3	0.8	0.3	0.9	0.6
37851	4.3	0.4	0.3	0.8	0.4	1.0	0.6
37165	4.5	0.4	0.3	0.8	0.4	1.0	0.6
37080	Quarter Road Bridge						
36995	4.5	2.3	0.5	1.5	0.5	1.3	0.7
32085	5.4	3.2	1.8	2.3	0.8	1.8	1.2
29049	6.0	3.5	2.2	2.6	1.0	2.0	1.3
27088	6.4	3.7	2.4	2.9	1.1	2.1	1.4
23296	7.1	4.0	2.8	3.7	2.6	2.6	1.8
22433	7.3	4.1	2.8	3.8	2.8	2.8	1.8
22180	295th Street Bridge						
21984	7.4	4.1	2.8	4.1	3.1	3.0	2.0
16234	8.4	4.3	3.0	4.4	3.4	4.1	3.2
12156	9.2	4.3	3.0	4.6	3.5	4.6	3.6
10425	9.5	4.3	3.1	4.6	3.6	4.7	3.8
10370	Hopkinton Bridge						
10316	9.6	4.3	3.1	4.6	3.6	4.8	3.8
9673	9.7	4.3	3.1	4.6	3.6	4.8	3.8
8802	9.8	4.3	3.1	4.6	3.6	4.8	3.8
6953	10.2	4.3	3.1	4.6	3.6	4.8	3.9
5735	10.4	4.3	3.1	4.6	3.6	4.8	3.9
2528	11.0	4.3	3.1	4.7	3.6	4.9	3.9
702	11.4	4.3	3.1	4.7	3.6	4.9	3.9

Overall, the breach peak stage (flood wave) traveled downstream more rapidly than the no breach condition flood wave. Once the breach occurs, HEC-RAS shows it would take between 3-4 hours for the breach floodwave to reach Hopkinton. The Quarter Road bridge appears to provide significant attenuation of the floodwave.

Impact of Quarter Road Bridge

The HEC-RAS flood hydrographs/profiles indicated the Quarter Road Bridge has a significant impact on water surface (backwater) el. upstream of the bridge. The bridge is overtopped for both the Full and Half PMF but not the 100YR (both breach and non-breach) so the impact of a potential failure of the bridge was evaluated by removing the bridge from the HEC-RAS model and comparing bridge and no bridge conditions.

The plot below displays the Full PMF flood profile for breach and non-breach conditions



Max. W.S. Difference (Breach - NonBreach) River Sta	Qtr Rd Bridge	No Qtr Rd
60900	Lake Delhi Dam	
60797	2.24	2.28
59617	2.29	2.33
58696	2.11	2.14
57576	1.98	2.02
56760	1.74	1.79
55202	1.73	1.79
53880	1.73	1.78
51564	1.65	1.74
50035	1.67	1.75
48596	1.55	1.65
47429	1.46	1.58
46486	1.47	1.6
45029	1.29	1.46
42827	1.05	1.27
40097	0.94	1.21
37851	0.66	0.94
37165	0.56	0.8
37080	Quarter Road Bridge	
36995	0.84	0.81
32085	0.58	0.58
29049	0.59	0.58
27088	0.6	0.59
23296	0.63	0.62
22433	0.64	0.63

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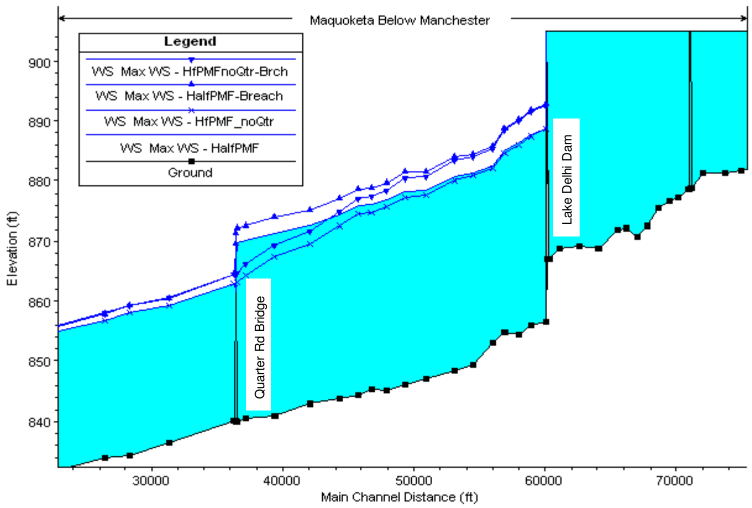
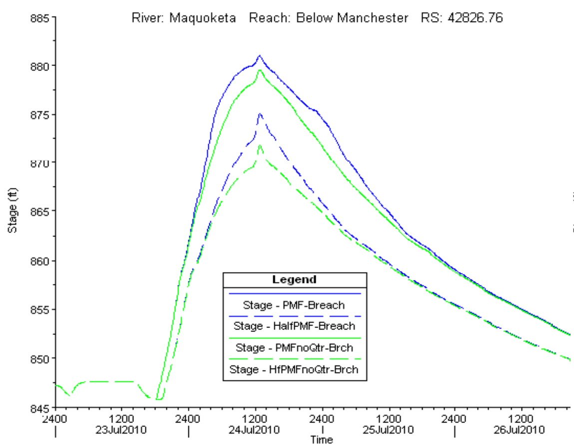
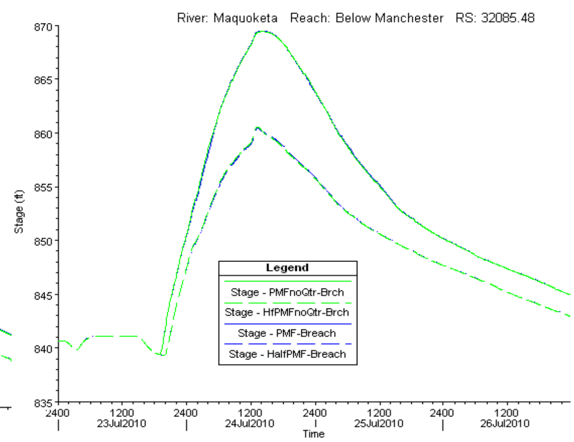
The plot below displays the Half PMF flood profile for breach and non-breach conditions



Max. W.S. Difference (Breach - NonBreach)		
River Sta	Qtr Rd Bridge	No Qtr Rd
60900	Lake Delhi Dam	
60797	4.08	4.15
59617	4.05	4.14
58696	3.9	4
57576	3.58	3.68
56760	3.38	3.4
55202	3.2	3.26
53880	3.27	3.33
51564	3.03	3.05
50035	3.1	3.12
48596	2.89	2.87
47429	2.79	2.7
46486	2.83	2.76
45029	2.7	2.49
42827	2.6	2.21
40097	2.61	2.13
37851	2.55	1.83
37165	2.52	1.71
37080	Quarter Road Bridge	
36995	1.57	1.68
32085	1.25	1.37
29049	1.14	1.26
27088	1.01	1.14
23296	0.9	0.98
22433	0.92	0.97

Timing of the peak stage was also reviewed for the No Quarter Road Bridge model. Removal (failure) of the Quarter Road Bridge did not impact the timing of the peak of the floodwave significantly. As shown in the stage hydrographs below, river segments just upstream of the Quarter Road bridge saw the greatest impact to stage and a slight impact to timing of the peak, but once downstream of the bridge, the impact was minimal.





Results show that a failure of the Quarter Road Bridge during a flood event would decrease the overall flood elevation and would reduce the potential increase in flood elevation due to breach for floods of a Half PMF magnitude. For floods of a full PMF magnitude failure of the Quarter Road Bridge would likely add to the potential increase in flood elevation due to breach, HOWEVER the flood profile would drop by roughly 1.5' with failure of the bridge so the overall flood profile would still be lower for a full PMF with failure of the Quarter Road Bridge.

So the analysis indicates that failure Quarter Road Bridge does not increase the downstream hazard.

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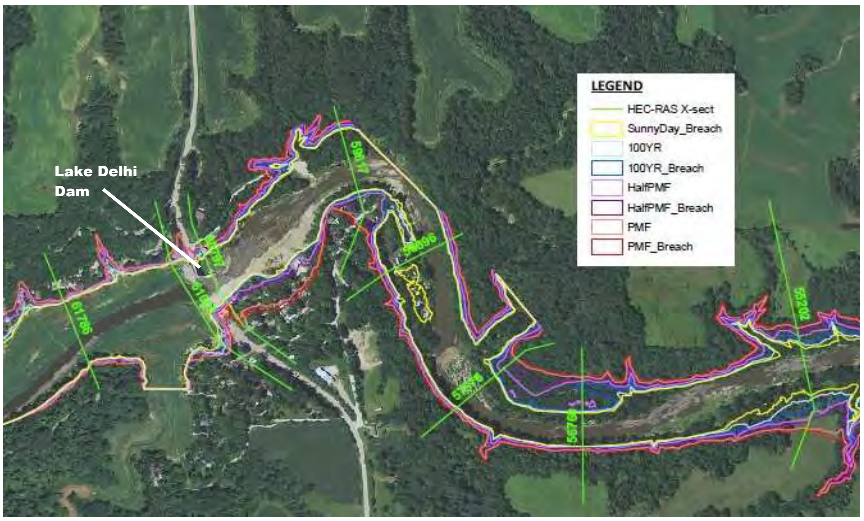
Inudation Mapping

HEC-RAS results are exported to ArcGIS to develop inundation limits for each flood condition (using HEC-GeoRAS).

HEC-GeoRAS establishes inundation limits by the following process

- Results of HEC-RAS are read and each river cross-section in ArcGIS is assigned a Max W.S. Elev for a given flood.
- A 3-D surface of the Max W.S. is created in ArcGIS along the River alignment (i.e. a 3-D version of the Max WS profile).
- The Max W.S. is then laid over the LiDAR topography of the river valley and shapefiles and rasters are created representing the inundation limits and depths, respectively for the given flood.

Note that upstream of the dam the inundation limits are identical for breach/non breach conditions. Downstream of the dam the breach condition causes a rise in flood elevation which translates to an expanded inundation area.



The results of the inundation mapping are provided in a set of maps following these computations.

Hazard Classification

Hazard Classification involves reviewing the properties/structures inundated for the various flood scenarios and evaluating the increase in hazard (i.e. damage/safety risk) caused by failure of the dam during that flood event. Theoretically, the less increase in flood profile, the less increase in hazard. But inundation maps provide a more quantifiable method for evaluating what additional properties and structures would be impacted by a breach during a flood event.

Due to their location and lack of potential warning time, the homes directly downstream of the dam were examined in closer detail for the theoretical breach (failure) event. Flood impacts to individual homes downstream of the dam are provided in the following tables.

Property				PMF Event: No Breach		PMF Event: Breach		Depth
Street Number	Street Name	F.F. Elev.	Dist. from Dam	W.S. Elev.	Flood Depth	W.S. Elev.	Flood Depth	Increase
2636	230th Ave	901.9	150	899.8	0.0	902.1	0.2	0.2
2638	230th Ave	913.7	150	899.8	0.0	902.1	0.0	0.0
23082	263rd St	914.7	340	899.7	0.0	901.9	0.0	0.0
23089	263rd St	914.4	200	899.8	0.0	902.0	0.0	0.0
23094	263rd St	906.1	340	899.7	0.0	901.9	0.0	0.0
23099	263rd St	912.1	200	899.8	0.0	902.0	0.0	0.0
23102	263rd St	903.4	340	899.7	0.0	901.9	0.0	0.0
23105	263rd St	905.9	200	899.8	0.0	902.0	0.0	0.0
23110	263rd St	898.7	340	899.7	1.0	901.9	3.2	2.3
23111	263rd St	902.4	200	899.8	0.0	902.0	0.0	0.0
23116	263rd St	895.3	340	899.7	4.4	901.9	6.6	2.3
23119	263rd St	901.3	200	899.8	0.0	902.0	0.7	0.7
23124	263rd St	896.4	340	899.7	3.3	901.9	5.5	2.3
23128	263rd St	893.3	340	899.7	6.4	901.9	8.6	2.3
23129	263rd St	895.9	200	899.8	3.9	902.0	6.1	2.2
23137	263rd St	891.3	340	899.7	8.4	901.9	10.6	2.3
23157	263rd St	892.4	430	899.6	7.2	901.9	9.5	2.3
23162	263rd St	896.7	430	899.6	0.0	901.9	0.0	0.0
23168	263rd St	904.7	490	899.6	2.9	901.9	5.2	2.3
23181	263rd St	893.3	490	899.6	6.3	901.9	8.6	2.3
23049	264th St	924.8	150	899.8	0.0	902.1	0.0	0.0
23077	264th St	917	200	899.8	0.0	902.0	0.0	0.0
23105	264th St	911.9	430	899.6	0.0	901.9	0.0	0.0
23168	264th St	904.7	430	899.6	0.0	901.9	0.0	0.0
23133	264th St	905.2	490	899.6	0.0	901.9	0.0	0.0
23157	264th St	899.6	540	899.6	0.0	901.8	2.2	2.2
23172	264th St	883.1	630	899.5	16.4	901.8	18.7	2.3
26287	231st Ave	889.1	1180	899.1	10.0	901.4	12.3	2.3
26294	231st Ave	894.4	1180	899.1	4.7	901.4	7.0	2.3
26299	231st Ave	892.4	1180	899.1	6.7	901.4	9.0	2.3
26269	232nd Ave	879.8	1180	899.1	19.3	901.4	21.6	2.3

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				Half PMF Event				
Property				No Breach		Breach		Depth
Street Number	Street Name	F.F. Elev.	Dist. from Dam	W.S. Elev.	Flood Depth	W.S. Elev.	Flood Depth	Increase
2636	230th Ave	901.9	150	888.6	0.0	892.7	0.0	0.0
2638	230th Ave	913.7	150	888.6	0.0	892.7	0.0	0.0
23082	263rd St	914.7	340	888.5	0.0	892.5	0.0	0.0
23089	263rd St	914.4	200	888.6	0.0	892.6	0.0	0.0
23094	263rd St	906.1	340	888.5	0.0	892.5	0.0	0.0
23099	263rd St	912.1	200	888.6	0.0	892.6	0.0	0.0
23102	263rd St	903.4	340	888.5	0.0	892.5	0.0	0.0
23105	263rd St	905.9	200	888.6	0.0	892.6	0.0	0.0
23110	263rd St	898.7	340	888.5	0.0	892.5	0.0	0.0
23111	263rd St	902.4	200	888.6	0.0	892.6	0.0	0.0
23116	263rd St	895.3	340	888.5	0.0	892.5	0.0	0.0
23119	263rd St	901.3	200	888.6	0.0	892.6	0.0	0.0
23124	263rd St	896.4	340	888.5	0.0	892.5	0.0	0.0
23128	263rd St	893.3	340	888.5	0.0	892.5	0.0	0.0
23129	263rd St	895.9	200	888.6	0.0	892.6	0.0	0.0
23137	263rd St	891.3	340	888.5	0.0	892.5	1.2	1.2
23157	263rd St	892.4	430	888.4	0.0	892.4	0.0	0.0
23162	263rd St	896.7	430	888.4	0.0	892.4	0.0	0.0
23168	263rd St	904.7	490	888.3	0.0	892.4	0.0	0.0
23181	263rd St	893.3	490	888.3	0.0	892.4	0.0	0.0
23049	264th St	924.8	150	888.6	0.0	892.7	0.0	0.0
23077	264th St	917	200	888.6	0.0	892.6	0.0	0.0
23105	264th St	911.9	430	888.4	0.0	892.4	0.0	0.0
23168	264th St	904.7	430	888.4	0.0	892.4	0.0	0.0
23133	264th St	905.2	490	888.3	0.0	892.4	0.0	0.0
23157	264th St	899.6	540	888.3	0.0	892.4	0.0	0.0
23172	264th St	883.1	630	888.2	5.1	892.3	9.2	4.1
26287	231st Ave	889.1	1180	887.8	0.0	891.8	2.7	2.7
26294	231st Ave	894.4	1180	887.8	0.0	891.8	0.0	0.0
26299	231st Ave	892.4	1180	887.8	0.0	891.8	0.0	0.0
26269	232nd Ave	879.8	1180	887.8	8.0	891.8	12.0	4.0

				100YR Flood Event				
Property				No Breach		Breach		Depth
Street Number	Street Name	F.F. Elev.	Dist. from Dam	W.S. Elev.	Flood Depth	W.S. Elev.	Flood Depth	Increase
2636	230th Ave	901.9	150	878.8	0.0	883.8	0.0	0.0
2638	230th Ave	913.7	150	878.8	0.0	883.8	0.0	0.0
23082	263rd St	914.7	340	878.6	0.0	883.6	0.0	0.0
23089	263rd St	914.4	200	878.8	0.0	883.7	0.0	0.0
23094	263rd St	906.1	340	878.6	0.0	883.6	0.0	0.0
23099	263rd St	912.1	200	878.8	0.0	883.7	0.0	0.0
23102	263rd St	903.4	340	878.6	0.0	883.6	0.0	0.0
23105	263rd St	905.9	200	878.8	0.0	883.7	0.0	0.0
23110	263rd St	898.7	340	878.6	0.0	883.6	0.0	0.0
23111	263rd St	902.4	200	878.8	0.0	883.7	0.0	0.0
23116	263rd St	895.3	340	878.6	0.0	883.6	0.0	0.0
23119	263rd St	901.3	200	878.8	0.0	883.7	0.0	0.0
23124	263rd St	896.4	340	878.6	0.0	883.6	0.0	0.0
23128	263rd St	893.3	340	878.6	0.0	883.6	0.0	0.0
23129	263rd St	895.9	200	878.8	0.0	883.7	0.0	0.0
23137	263rd St	891.3	340	878.6	0.0	883.6	0.0	0.0
23157	263rd St	892.4	430	878.5	0.0	883.5	0.0	0.0
23162	263rd St	896.7	430	878.5	0.0	883.5	0.0	0.0
23168	263rd St	904.7	490	878.5	0.0	883.4	0.0	0.0
23181	263rd St	893.3	490	878.5	0.0	883.4	0.0	0.0
23049	264th St	924.8	150	878.8	0.0	883.8	0.0	0.0
23077	264th St	917	200	878.8	0.0	883.7	0.0	0.0
23105	264th St	911.9	430	878.5	0.0	883.5	0.0	0.0
23168	264th St	904.7	430	878.5	0.0	883.5	0.0	0.0
23133	264th St	905.2	490	878.5	0.0	883.4	0.0	0.0
23157	264th St	899.6	540	878.4	0.0	883.4	0.0	0.0
23172	264th St	883.1	630	878.3	0.0	883.3	0.2	0.2
26287	231st Ave	889.1	1180	877.8	0.0	882.8	0.0	0.0
26294	231st Ave	894.4	1180	877.8	0.0	882.8	0.0	0.0
26299	231st Ave	892.4	1180	877.8	0.0	882.8	0.0	0.0
26269	232nd Ave	879.8	1180	877.8	0.0	882.8	3.0	3.0

The two critical items being evaluated for these properties are

- Number of additional properties being flooded due to breach
- Increase in depth of flooding at properties being impacted

	Full PMF	Half PMF	100YR Flood
Non-Breach Flooded	13	2	0
Additional by Breach	3	2	2
Greatest Increase in Flood Depth	2.3'	4.1'	3.0'

Results indicate the PMF is a hazardous condition without breach and the breach would impact an additional 3 homes with the 2.3' increase in flood level. The half PMF without breach impacts 2 homes, which would increase to 4 with the 4.1' increase in flood level. The 100 YR flood does not impact any downstream homes, but results show if a breach occurs two downstream homes could be flooded by the breach flood wave.

For increase in flood hazard due to breach, the 100YR has a definite increase in hazard due to the fact that prior to breach no homes are impacted and then with breach the flood level jumps by 5'. The flooding due to the full PMF is widespread and the breach increases the flood level by 2.3' which is significant but relative to the flooding already occurring likely does not constitute as great of an increase in flood hazard as at the Half PMF when flood levels are also high but then increase by over 4' due to a breach event. Qualitatively the Half PMF appears to provide the greatest increase in downstream hazard potential due to the larger increase in flood depth that could catch more properties if the downstream area is further developed.

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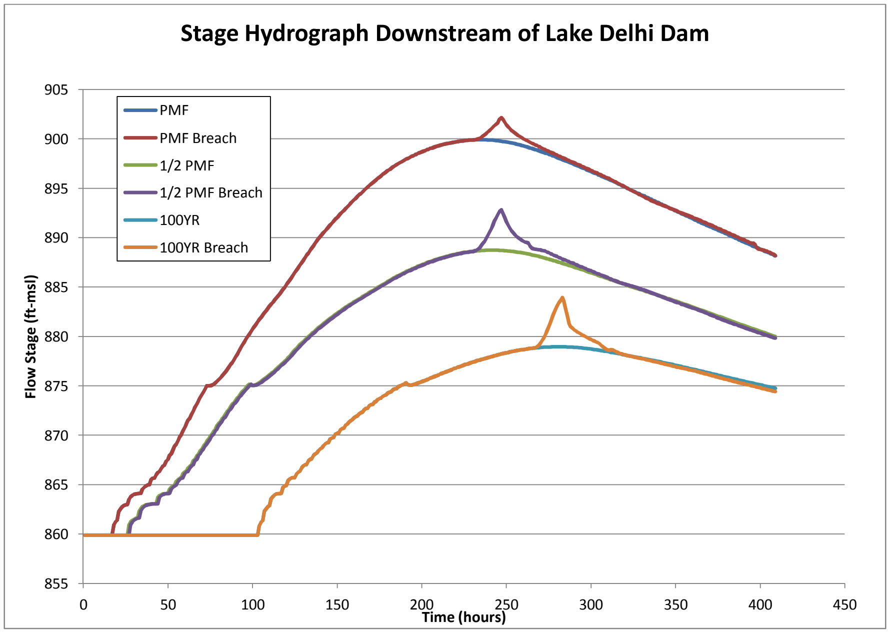
The graph below displays the stage hydrograph during the theoretical design floods downstream of the proposed dam. The rise is roughly 1.5 ft/hr with slight variations between events. With the breach, the peak flood/flow occurs within roughly an hour of the initiation of breach, so the rise would be roughly 2 ft/hr for the PMF and 4 ft/hr with the 1/2 PMF during breach.



	RISE (ft/hr)
PMF	1.7
1/2 PMF	1.3
100YR	1.5

Downstream impacts were also evaluated for the entire downstream reach of the Maquoketa River from the Dam to just downstream of Hopkinton. The table below provides a summary of the structures impacted for the flood events. Structures were inventoried using the inundation maps referencing 2011 aerial photos. The summary table is based off of a detailed inundation table where structures were counted cross-section to cross-section.

Buildings and Infrastructure Inundated					
Event	Scenario	Residential	Comm/Ag	Bridges	Roads
PMF	No Breach	104	30	3	12
	Breach	107	30	3	12
Half PMF	No Breach	27	8	3	8
	Breach	29	8	3	8
100-YR	No Breach	3	1	1	5
	Breach	5	2	1	5
Sunny Day	Breach	0	0	0	1

Dam breach during the flood events does not appear to cause a significant increase in structures inundated. The majority of additional structures inundated are the homes wihin 1500' of the dam which should be evacuated during extreme flood events.

Downstream of the near-dam residential area, homes and buildings that are inundated for the breach condition are generally also inundated for the non-breach condition. This can be seen on the inundation maps provided.

The Sunny Day Breach did not appear to cause significant hazard to downstream permanent structures.

Hazard classification is based on the additional hazard created by failure of the dam. So by this criteria, given the number of additional buildings inundated over the range of failure events, and the low density, rural setting of the majority of the downstream channel, Lake Delhi Dam is best classified as a

MODERATE/SIGNIFICANT HAZARD STRUCTURE

DNR

Low Hazard. Structures located in areas where damages from a failure would be limited to loss of the dam, loss of livestock, damages to farm out-buildings, agricultural lands, and lesser used roads, and where loss of human life is considered unlikely.

Moderate Hazard. Structures located in areas where failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life.

High Hazard. Structures located in areas where failure may create a serious threat of loss of human life or result in serious damage to residential, industrial or commercial areas, important public utilities, public buildings, or major transportation facilities.

FERC

Dams conforming to criteria for the low hazard potential category generally are located in rural or agricultural areas where failure may damage farm buildings, limited agricultural land, or township and country roads. Low hazard potential dams have a small storage capacity, the release of which would be confined to the river channel in the event of a failure and therefore would represent no danger to human life.

Significant hazard potential category structures are usually located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life.

Dams in the high hazard potential category are those located where failure may cause serious damage to homes, agricultural, industrial and commercial facilities, important public utilities, main highways, or railroads, and there would be danger to human life.

Stanley Consultants INC.

				Job No.	23601
Computed by	A. Judd	Date	9/28/2011	Subject	Lake Delhi Dam
Checked by	M. Weber	Date	10/12/2011		Hydrology/Hydraulics
Approved by		Date			Computation Summary

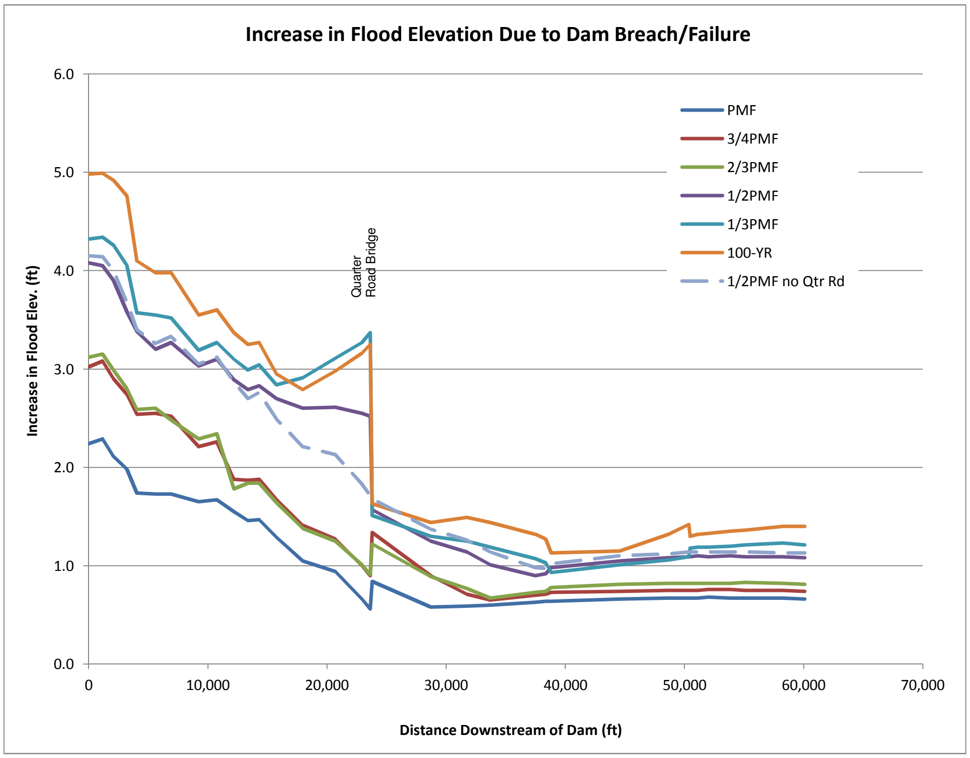
Design Flood

The DNR specifies that a Moderate Hazard dam be able to safely pass the Half PMF for its design flood.

FERC uses an incremental analysis to determine the largest flood where failure causes an increase in downstream hazard.

From the inundation analysis, dam failure did not suggest a significant increase in downstream hazard, mostly due to the lack of structures located in the river valley and the minimal difference in additional structures inundated by the dam's failure/breach during the given flood. The exception is homes immediately downstream of the dam, which do experience additional inundation/hazard due to dam failure. It is assumed that these homes would be evacuated during an extreme flood.

To further analyze the incremental hazard, the rise in flood elevation due to breach was evaluated for a series of floods.



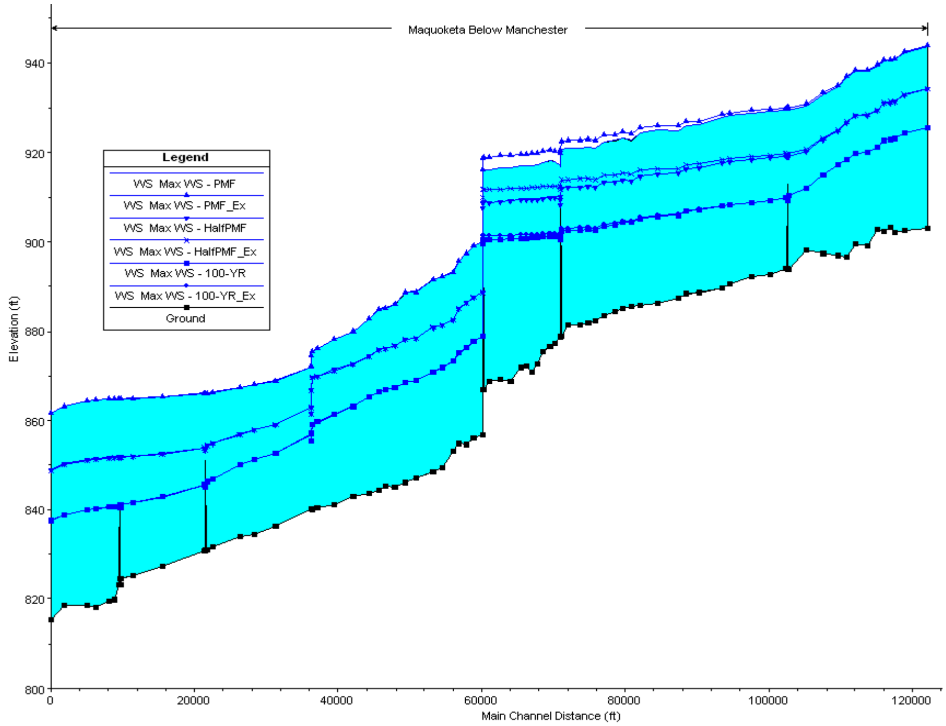
As illustrated in the graph above, the 100YR results in the greatest increase in flood elevation due to breach. This increase is over 5 feet just downstream of the dam and diminishes to less than 2 feet by Hopkinton. FERC recommends using 2 feet as the minimum flood rise where hazard potential is considered to be impacted by breach, however downstream of the immediate homes at the dam, there are no buildings inundated by any of the flood events until the flood is past the Quarter Road Bridge. By the Quarter Road Bridge, the rise due to breach for floods greater than the 1/2 PMF has decreased to less than 2 feet. Floods less than the1/2 PMF have a rise greater than 2 feet at the bridge, but the bridge dissipates the rise due to breach downstream.

To analyze the impact of the bridge a HEC-RAS run was developed with the Quarter Road Bridge removed. This is also shown on the graph. HEC-RAS shows that the bridge causes a rise in flood elevation due to breach upstream of the bridge. Removing the bridge causes a slight increase in flood elevation due to breach downstream of the bridge, but the bridge looks to have more impact on the upstream flood elevation increases.

So given that the greatest increases in flood elevation due to breach are from floods of a lesser magnitude than the1/2 PMF, using the **1/2 PMF** as the design flood should also meet FERC criteria.

Upstream/Downstream Impacts

Hazard classification compares a failure to non-failure condition. Another important comparison is the impact of the reconstructed dam relative to the pre-breach condition. The objective is to provide a reconstructed dam that minimizes negative impacts upstream and downstream. This was analyzed by comparing a series of floods for the pre-breach and reconstrcucted dam HEC-RAS model.



Without the principal/auxiliary spillway and gate system fully defined this is a preliminary evaluation, so will require updating once the auxiliary spillway and gates are designed for reconstruction.

As can be expected with an increase in hydraulic capacity, the reconstructed dam provides lower peak flood elevations upstream, with minimal increase in flood elevation downstream. So HEC-RAS results indicate that the reconstructed dam will improve upstream flood conditions by

The following table provides flood elevation differences at a series of HEC-RAS cross-sections

Stretch of Maquoketa River	HEC-RAS X-SECT ID	Change in Flood Elev Due w/ New Dam PMF (ft)	1/2 PMF (ft)	100-YR (ft)
Upstream of 197th Ave Bridge	122794	-0.06	-0.03	0
	103398	-0.59	-0.64	-0.04
Between 197th Ave Bridge and Hartwick Bridge	103168	-0.6	-0.65	-0.06
	100907	-0.61	-0.69	-0.08
	94139	-0.71	-0.81	-0.12
	82771	-1.08	-1.23	-0.27
	77805	-1.32	-1.49	-0.43
	71785	-1.61	-1.76	-0.58
Between Hartwick Bridge and Lake Delhi Dam	71635	-2.65	-2.73	-0.77
	67720	-2.65	-2.75	-0.79
	63302	-2.83	-2.87	-0.85
	61003	-2.9	-2.92	-0.87
Between Lake Delhi Dam and Quarter Road Bridge	60797	0.05	0.06	0.11
	56760	0.04	0.05	0.11
	50035	0.05	0.06	0.11
	45029	0.03	0.06	0.1
	37165	0.02	0.07	0.1
Between Quarter Road Bridge and 295th Street Bridge	36995	0.05	0.05	0.06
	27088	0.06	0.07	0.07
	22433	0.07	0.09	0.06
Between 295th Street Bridge and Hopkinton Bridge	21984	0.07	0.1	0.05
	10425	0.07	0.11	0.06
Downstream of Hopkinton Bridge	10316	0.07	0.12	0.07
	702	0.07	0.11	0.07

Stanley Consultants INC.

				Job No.	23601
Computed by	A. Judd	Date	11/10/2011	Subject	Lake Delhi Dam
Checked by	M Weber	Date	12/9/2011		Hydraulics
Approved by		Date			Spillway

Description:

Summary of spillway hydraulic analysis for Lake Delhi reconstruction project.

Reference:

(1) Iowa DNR, *Design Criteria and Guidelines for Iowa Dams* , T.B. 16, 1990.
(2) USBR, *Design of Small Dams,* 1987.
(3) Chow Ven Te; *Open Channel Hydraulics* ; McGraw-Hill, 1958.
(4) Henry T. Falvey, *Hydraulic Design of Labyrinth Weirs* , ASCE Press, 2003.

Analysis:

Lake Delhi Dam is classified as a moderate hazard structure. Per Ref 1, the dam must pass the 1/2 PMF safely. Hydrologic analysis has established the 1/2 PMF as roughly 70,000 cfs at the dam. The following analysis evaluates the hydraulics of spillway alternatives for the reconstructed dam to pass the 1/2 PMF safely.

Option 1 **Use two, tiered labyrinth weirs + gates**

A labyrinth weir spillway consists of a sharp-crested weir (e.g. concrete wall) set in a series of trapezoidal folds which fits a longer crest length into a shorter spillway breadth.

Several methods are shown in Ref 4 for estimating flow over a Labyrinth Spillway
This analysis uses the recommended method proposed by Tullis in 1995.
The geometry (i.e. variables) of a Labyrinth Spillway is given by the following

Flow over a Labyrinth Spillway is given by:

$$Q_L = C_D \cdot L_c \cdot \frac{2}{3} \cdot \sqrt{2g} \cdot H^{1.5}$$ Ref 4 5.15

where:

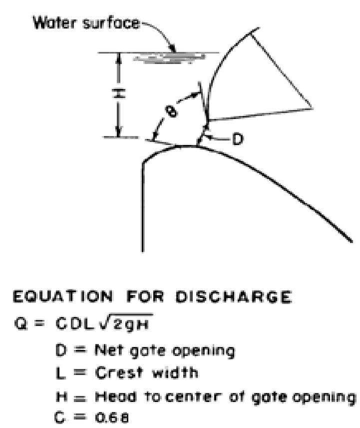
$$C_D = A_1 + A_2\left(\frac{H}{P}\right) + A_3\left(\frac{H}{P}\right)^2 + A_4\left(\frac{H}{P}\right)^3 + A_5\left(\frac{H}{P}\right)^4$$

(1) 5.16

The parameters are defined by the following table (1) table 5.1

a	A_1	A_2	A_3	A_4	A_5
6	0.49	-0.24	-1.20	2.17	-1.03
8	0.49	1.08	-5.27	6.79	-2.83
12	0.49	1.06	-4.43	5.18	-1.97
15	0.49	1.00	-3.57	3.82	-1.38
18	0.49	1.32	-4.13	4.24	-1.50
25	0.49	1.51	-3.83	3.40	-1.05
35	0.49	1.69	-4.05	3.62	-1.10
90	0.49	1.46	-2.56	1.44	0.00

Flow through Gates are given by:



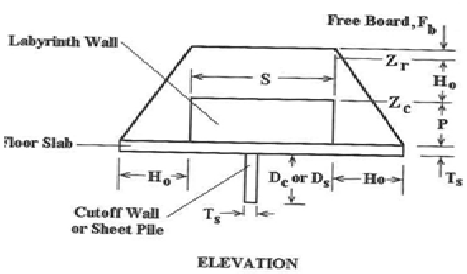
Existing spillway gates at Lake Delhi

Crest Elev.	879.8 ft
# of Gates	3
Open Width	25.0 ft
Open Height	20.0 ft
C	0.68

DESIGN PARAMETER	LABYRINTH SPILLWAY PRINCIPAL	AUX.	EQUATION
Design Pool	904	904	
Crest Elev.	896.3	900	
Floor Elev.	886.3	892	
Wc	124	110	
α	25	15	
a	1.5	1.5	
n	2	4	
$T_{weir\ wall}$	1.5	1.5	
$T_{end\ wall}$	1.5	1.5	
T_s	2	2	
H_0 (ft)	7.7	4	(4) 8.1
P (ft)	10	8	El_{crest}-El_{floor}
W (ft)	62.0	27.5	(4) 8.5
L (ft)	138.5	89.1	(4) 5.2
L_c (ft)	277.0	356.3	(4) 8.6
B (ft)	66.3	41.5	(4) 8.8
S (ft)	60.0	40.1	(1) 8.9
A_1	0.49	0.49	Table 5.1
A_2	1.51	1	Table 5.1
A_3	-3.83	-3.57	Table 5.1
A_4	3.4	3.82	Table 5.1
A_5	-1.05	-1.38	Table 5.1
PARAMETER CHECK			
$H_0/P < 0.7$	0.8	0.5	
2< L/W <9.5	2.2	3.2	
$L_{de}/B \leq 0.35$	0.14	0.20	

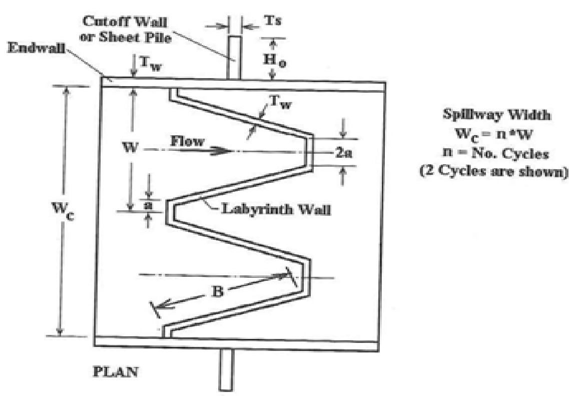
$$\frac{L_{de}}{B} = \frac{h}{B} \cdot 6.1 \cdot e^{-0.052 \cdot \alpha}$$

h is the head at the weir, which is close to critical depth (~2/3H)
0.75*H is used for a conservative *h*



L = crest length of weir in 1 cycle

L_c = total crest length of weir



Gates assumed completely open

POOL ELEVATION	GATE FLOW 3 GATES		SPILLWAY FLOW						TOTAL
			SERVICE			AUX.			
(ft-msl)	H_{gates} (ft)	Q_{gates} (cfs)	H_{princ} (ft)	C_D	Q_{princ} (cfs)	H_{aux} (ft)	C_D	Q_{aux} (cfs)	Q_{total} (cfs)
896	16.2	17,116	0	0.00	0	0	0.0	0	17,116
896.2	16.4	18,430	0	0.00	0	0	0.00	0	18,430
896.4	16.6	17,247	0.1	0.50	24	0	0.00	0	17,270
896.6	16.8	17,559	0.3	0.53	130	0	0.00	0	17,689
896.8	17	17,874	0.5	0.56	292	0	0.00	0	18,165
897	17.2	18,190	0.7	0.58	502	0	0.00	0	18,692
897.2	17.4	18,508	0.9	0.60	756	0	0.00	0	19,264
897.4	17.6	18,828	1.1	0.61	1,050	0	0.00	0	19,878
897.6	17.8	19,150	1.3	0.63	1,381	0	0.00	0	20,531
897.8	18	19,474	1.5	0.64	1,746	0	0.00	0	21,220
898	18.2	19,799	1.7	0.65	2,141	0	0.00	0	21,940
898.2	18.4	20,126	1.9	0.66	2,564	0	0.00	0	22,690
898.4	18.6	20,455	2.1	0.67	3,011	0	0.00	0	23,467
898.6	18.8	20,786	2.3	0.67	3,480	0	0.00	0	24,266
898.8	19	21,119	2.5	0.68	3,967	0	0.00	0	25,086
899	19.2	21,453	2.7	0.68	4,470	0	0.00	0	25,923
899.2	19.4	21,789	2.9	0.68	4,986	0	0.00	0	26,776
899.4	19.6	22,127	3.1	0.68	5,514	0	0.00	0	27,641
899.6	19.8	22,467	3.3	0.68	6,050	0	0.00	0	28,516
899.8	20	22,808	3.5	0.68	6,592	0	0.00	0	29,400
900	20.2	26,142	3.7	0.68	7,140	0	0.00	0	33,282
900.2	20.4	26,397	3.9	0.67	7,690	0.2	0.51	87	34,175
900.4	20.6	26,650	4.1	0.67	8,243	0.4	0.53	256	35,149
900.6	20.8	26,900	4.3	0.67	8,795	0.6	0.55	484	36,179
900.8	21	27,148	4.5	0.66	9,347	0.8	0.56	761	37,256
901	21.2	27,394	4.7	0.66	9,897	1	0.57	1,079	38,371
901.2	21.4	27,637	4.9	0.65	10,445	1.2	0.57	1,433	39,516
901.4	21.6	27,879	5.1	0.64	10,991	1.4	0.57	1,815	40,684
901.6	21.8	28,118	5.3	0.64	11,533	1.6	0.58	2,220	41,871
901.8	22	28,355	5.5	0.63	12,072	1.8	0.57	2,643	43,071
902	22.2	28,591	5.7	0.63	12,608	2	0.57	3,079	44,278
902.2	22.4	28,824	5.9	0.62	13,141	2.2	0.57	3,524	45,489
902.4	22.6	29,056	6.1	0.61	13,672	2.4	0.56	3,973	46,701
902.6	22.8	29,285	6.3	0.61	14,200	2.6	0.55	4,424	47,909
902.8	23	29,513	6.5	0.60	14,726	2.8	0.55	4,874	49,113
903	23.2	29,739	6.7	0.59	15,252	3	0.54	5,320	50,311
903.2	23.4	29,964	6.9	0.59	15,777	3.2	0.53	5,761	51,502
903.4	23.6	30,186	7.1	0.58	16,303	3.4	0.52	6,195	52,684
903.6	23.8	30,408	7.3	0.58	16,830	3.6	0.51	6,622	53,859
903.8	24	30,627	7.5	0.57	17,359	3.8	0.50	7,041	55,027
904	24.2	30,845	7.7	0.56	17,891	4	0.49	7,453	56,189
904.2	24.4	31,062	7.9	0.56	18,427	4.2	0.48	7,858	57,346
904.4	24.6	31,277	8.1	0.56	18,967	4.4	0.47	8,257	58,500
904.6	24.8	31,490	8.3	0.55	19,511	4.6	0.46	8,651	59,653
904.8	25	31,702	8.5	0.55	20,062	4.8	0.45	9,042	60,806
905	25.2	31,913	8.7	0.54	20,618	5	0.44	9,430	61,961
905.2	25.4	32,122	8.9	0.54	21,180	5.2	0.43	9,818	63,121
905.4	25.6	32,330	9.1	0.53	21,749	5.4	0.43	10,208	64,286
905.6	25.8	32,537	9.3	0.53	22,323	5.6	0.42	10,599	65,459
905.8	26	32,742	9.5	0.53	22,903	5.8	0.41	10,995	66,639
906	26.2	32,946	9.7	0.52	23,487	6	0.41	11,396	67,829
906.2	26.4	33,149	9.9	0.52	24,075	6.2	0.40	11,802	69,026
906.4	26.6	33,350	10.1	0.52	24,666	6.4	0.40	12,215	70,230
906.6	26.8	33,550	10.3	0.52	25,256	6.6	0.39	12,633	71,439
906.8	27	33,750	10.5	0.51	25,846	6.8	0.39	13,055	72,651

Option 2 Use single labyrinth weir + gates

Step 1 Enter Design Flow Data/Limitations

Design Pool	904	ft
Weir Crest	896.3	ft

Step 2 Enter, Compute, and Check Geometry Data

LABYRINTH WEIR

Enter			Compute			eqn	Check		
								Recommended Ratio	Result
Floor Elev.	886.3	ft	H_0	5.00	ft	(1) 8.1	Headwater	$H_0/P < 0.7$	0.5
Wc	180	ft	P	10.00	ft	El_{crest}-El_{floor}	Magnification	2< L/W <9.	2.9
α	18	°	W	36.00	ft	(1) 8.5	Interference Lengt	$L_{de}/B \le 0.35$	0.14
a	1.5	ft	L	103.08	ft	(1) 5.2			
n	5	cycles	L_c	515.41	ft	(1) 8.6			
$T_{weir\ wall}$	1.5	ft	B	48.54	ft	(1) 8.8			
$T_{end\ wall}$	1.5	ft	S	46.17	ft	(1) 8.9			
T_s	2	ft	q_0	51.8	cfs/ft	Q/Lc			

SPILLWAY GATES

Crest Elev.	879.8	ft
# of Gates	3	
Gate Width	25.0	ft
Gate Height	20.0	ft
C	0.68	

where:

$$\frac{L_{de}}{B} = \frac{h}{B} \cdot 6.1 \cdot e^{-0.052 \cdot \alpha} \qquad (1)\ 8.4$$

h = head on the weir

head on a labyrinth weir varies over the length of the weir. To simplify, critical depth was assumed, i.e.

$h = h_{crit}$ this condition suggests

critical depth eqn's from (2) App. C

$$v_{crit} = \sqrt[3]{g q_0} = 11.9 \text{ ft/s}$$

$$h_{vel} = \frac{v_{crit}^2}{2g} = 2.2 \text{ ft}$$

$$h_{crit} = H_0 - h_{vel} = 2.8 \text{ ft}$$

Due to the variability of h, this comp. is an estimate
Also, h_{crit} could be solved for directly with q_0 but the method used is more conservative (i.e. higher h_{crit})

Step 3 **Structure Rating Curve**

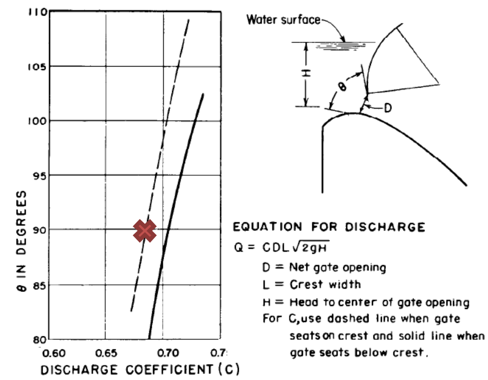
Coefficients for (1) 5.16				
A_1	A_2	A_3	A_4	A_5
0.49	1.32	-4.13	4.24	-1.5

POOL	**LABYRINTH WEIR**					**GATES**			**TOTAL**
Elevation (ft)	Head [H] (ft)	H/P	C_D	Q_L (cfs)	q (cfs/ft)	Open [D] (ft)	Head [H] (ft)	Q_{gates} (cfs)	Q_{total} (cfs)
896.0	0	0.00	0.49	0.00	0.00	0.00	16.20	0.0	0
896.2	0	0.00	0.49	0.00	0.00	0.00	16.40	0.0	0
896.4	0.1	0.01	0.50	43.84	0.09	0.00	16.60	0.0	44
896.6	0.3	0.03	0.53	238.32	0.46	0.00	16.80	0.0	238
896.8	0.5	0.05	0.55	532.49	1.03	0.00	17.00	0.0	532
897.0	0.7	0.07	0.56	910.14	1.77	0.10	17.15	169.5	1080
897.2	0.9	0.09	0.58	1361.61	2.64	0.20	17.30	340.5	1702
897.4	1.1	0.11	0.59	1878.99	3.65	0.30	17.45	512.9	2392
897.6	1.3	0.13	0.60	2455.10	4.76	0.40	17.60	686.8	3142
897.8	1.5	0.15	0.61	3083.13	5.98	0.50	17.75	862.1	3945
898.0	1.7	0.17	0.61	3756.52	7.29	1.00	17.70	1721.9	5478
898.2	1.9	0.19	0.62	4468.99	8.67	2.00	17.40	3414.4	7883
898.4	2.1	0.21	0.62	5214.54	10.12	3.00	17.10	5077.3	10292
898.6	2.3	0.23	0.62	5987.50	11.62	4.00	16.80	6710.1	12698
898.8	2.5	0.25	0.62	6782.51	13.16	5.00	16.50	8312.4	15095
899.0	2.7	0.27	0.62	7594.64	14.74	6.00	16.20	9883.8	17478
899.2	2.9	0.29	0.62	8419.34	16.34	7.00	15.90	11423.8	19843
899.4	3.1	0.31	0.61	9252.49	17.95	8.00	15.60	12932.0	22184
899.6	3.3	0.33	0.61	10090.42	19.58	9.00	15.30	14407.9	24498
899.8	3.5	0.35	0.61	10929.93	21.21	10.00	15.00	15851.1	26781
900.0	3.7	0.37	0.60	11768.25	22.83	10.00	15.20	15956.4	27725
900.2	3.9	0.39	0.59	12603.09	24.45	10.00	15.40	16061.0	28664
900.4	4.1	0.41	0.59	13432.60	26.06	10.00	15.60	16165.0	29598
900.6	4.3	0.43	0.58	14255.37	27.66	10.00	15.80	16268.3	30524
900.8	4.5	0.45	0.57	15070.44	29.24	10.00	16.00	16370.9	31441
901.0	4.7	0.47	0.57	15877.23	30.81	10.00	16.20	16472.9	32350
901.2	4.9	0.49	0.56	16675.55	32.35	10.00	16.40	16574.3	33250
901.4	5.1	0.51	0.55	17465.56	33.89	10.00	16.60	16675.1	34141
901.6	5.3	0.53	0.54	18247.76	35.40	10.00	16.80	16775.2	35023
901.8	5.5	0.55	0.53	19022.92	36.91	15.00	14.50	23377.0	42400
902.0	5.7	0.57	0.53	19792.06	38.40	15.00	14.70	23537.6	43330
902.2	5.9	0.59	0.52	20556.43	39.88	15.00	14.90	23697.2	44254
902.4	6.1	0.61	0.51	21317.40	41.36	15.00	15.10	23855.7	45173
902.6	6.3	0.63	0.51	22076.50	42.83	15.00	15.30	24013.2	46090
902.8	6.5	0.65	0.50	22835.28	44.31	15.00	15.50	24169.6	47005
903.0	6.7	0.67	0.49	23595.33	45.78	15.00	15.70	24325.1	47920
903.2	6.9	0.69	0.49	24358.16	47.26	15.00	15.90	24479.5	48838
903.4	7.1	0.71	0.48	25125.21	48.75	15.00	16.10	24633.0	49758
903.6	7.3	0.73	0.48	25897.72	50.25	15.00	16.30	24785.5	50683
903.8	7.5	0.75	0.47	26676.69	51.76	20.00	14.00	30627.2	57304
904.0	7.7	0.77	0.47	27462.83	53.28	20.00	14.20	30845.2	58308
904.2	7.9	0.79	0.46	28256.48	54.82	20.00	14.40	31061.6	59318
904.4	8.1	0.81	0.46	29057.53	56.38	20.00	14.60	31276.6	60334
904.6	8.3	0.83	0.45	29865.33	57.94	20.00	14.80	31490.1	61355
904.8	8.5	0.85	0.45	30678.67	59.52	20.00	15.00	31702.2	62381
905.0	8.7	0.87	0.45	31495.63	61.11	20.00	15.20	31912.8	63408
905.2	8.9	0.89	0.44	32313.54	62.69	20.00	15.40	32122.1	64436
905.4	9.1	0.91	0.44	33128.92	64.28	20.00	15.60	32330.0	65459
905.6	9.3	0.93	0.43	33937.31	65.85	20.00	15.80	32536.6	66474
905.8	9.5	0.95	0.43	34733.29	67.39	20.00	16.00	32741.8	67475
906.0	9.7	0.97	0.43	35510.31	68.90	20.00	16.20	32945.8	68456
906.2	9.9	0.99	0.42	36260.63	70.35	20.00	16.40	33148.6	69409
906.4	10.1	1.01	0.42	36975.22	71.74	20.00	16.60	33350.1	70325
906.6	10.3	1.03	0.41	37643.67	73.04	20.00	16.80	33550.4	71194
906.8	10.5	1.05	0.41	38254.10	74.22	20.00	17.00	33749.5	72004
907.0	10.7	1.07	0.40	38793.04	75.27	20.00	17.20	33947.5	72741

Option 3 **Use Obermeyer Gates**

Obermeyer gates consist of a series of steel gates, held up by a series of air bladders that sit on the downstream side of the gate and control the gates position by the level of their inflation. When fully inflated, the gates are up, when deflated the gates are down.

The gates act as a sharp crested weir with discharge defined by:



$$Q = C_D \cdot L_c \cdot H^{1.5}$$ Ref 1

where:

C_D = Discharge Coefficient
L_c = Crest Length
H = Head

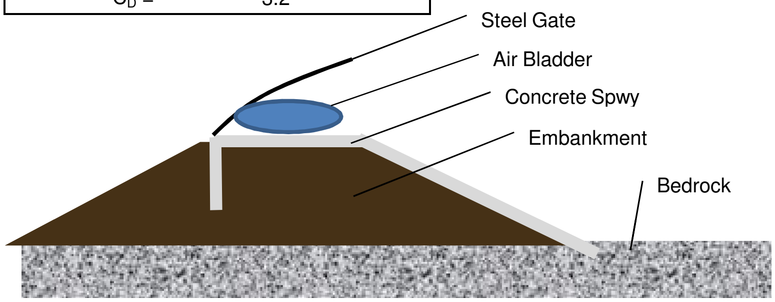
Step 1 **Enter Geometry Data**

OBERMEYER GATES

Raised Elev.	896.3	ft-msl
Lowered Elev	888.3	ft-msl
L_c	155	ft
C_D =	3.2	

SPILLWAY GATES

Crest Elev.	879.8	ft
# of Gates	3	
Gate Width	25.0	ft
Gate Height	20.0	ft
C	0.68	



Step 2 **Structure Rating Curve**

Obermeyer gates assumed to be completely down

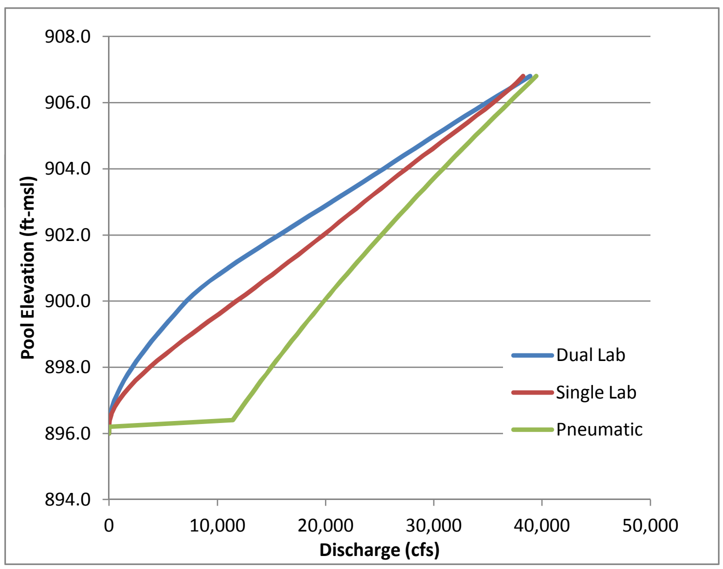
POOL	OBERMEYER			SLIDE GATES				TOTAL
Elevation (ft)	[H] (ft)	C	Q (cfs)	[D] (ft)	[H] (ft)	C	Q_{gates} (cfs)	Q_{total} (cfs)
896.0	0	3.2	0.00	0.00	16.20	0.7	0.0	0
896.2	0	3.2	0.00	0.00	16.40	0.7	0.0	0
896.4	8.1	3.2	11434.29	0.00	16.60	0.7	0.0	11434
896.6	8.3	3.2	11860.39	0.00	16.80	0.7	0.0	11860
896.8	8.5	3.2	12291.65	0.00	17.00	0.7	0.0	12292
897.0	8.7	3.2	12728.01	0.10	17.15	0.7	169.5	12898
897.2	8.9	3.2	13169.42	0.20	17.30	0.7	340.5	13510
897.4	9.1	3.2	13615.82	0.30	17.45	0.7	512.9	14129
897.6	9.3	3.2	14067.15	0.40	17.60	0.7	686.8	14754
897.8	9.5	3.2	14523.36	0.50	17.75	0.7	862.1	15386
898.0	9.7	3.2	14984.40	1.00	17.70	0.7	1721.9	16706
898.2	9.9	3.2	15450.21	2.00	17.40	0.7	3414.4	18865
898.4	10.1	3.2	15920.76	3.00	17.10	0.7	5077.3	20998
898.6	10.3	3.2	16395.99	4.00	16.80	0.7	6710.1	23106
898.8	10.5	3.2	16875.85	5.00	16.50	0.7	8312.4	25188
899.0	10.7	3.2	17360.30	6.00	16.20	0.7	9883.8	27244
899.2	10.9	3.2	17849.31	7.00	15.90	0.7	11423.8	29273
899.4	11.1	3.2	18342.82	8.00	15.60	0.7	12932.0	31275
899.6	11.3	3.2	18840.80	9.00	15.30	0.7	14407.9	33249
899.8	11.5	3.2	19343.21	10.00	15.00	0.7	15851.1	35194
900.0	11.7	3.2	19850.00	10.00	15.20	0.7	15956.4	35806
900.2	11.9	3.2	20361.14	10.00	15.40	0.7	16061.0	36422
900.4	12.1	3.2	20876.60	10.00	15.60	0.7	16165.0	37042
900.6	12.3	3.2	21396.33	10.00	15.80	0.7	16268.3	37665
900.8	12.5	3.2	21920.31	10.00	16.00	0.7	16370.9	38291
901.0	12.7	3.2	22448.50	10.00	16.20	0.7	16472.9	38921
901.2	12.9	3.2	22980.86	10.00	16.40	0.7	16574.3	39555
901.4	13.1	3.2	23517.36	10.00	16.60	0.7	16675.1	40192
901.6	13.3	3.2	24057.98	10.00	16.80	0.7	16775.2	40833
901.8	13.5	3.2	24602.67	15.00	14.50	0.7	23377.0	47980
902.0	13.7	3.2	25151.42	15.00	14.70	0.7	23537.6	48689
902.2	13.9	3.2	25704.19	15.00	14.90	0.7	23697.2	49401
902.4	14.1	3.2	26260.94	15.00	15.10	0.7	23855.7	50117
902.6	14.3	3.2	26821.66	15.00	15.30	0.7	24013.2	50835
902.8	14.5	3.2	27386.32	15.00	15.50	0.7	24169.6	51556
903.0	14.7	3.2	27954.88	15.00	15.70	0.7	24325.1	52280
903.2	14.9	3.2	28527.33	15.00	15.90	0.7	24479.5	53007
903.4	15.1	3.2	29103.63	15.00	16.10	0.7	24633.0	53737
903.6	15.3	3.2	29683.75	15.00	16.30	0.7	24785.5	54469
903.8	15.5	3.2	30267.69	20.00	14.00	0.7	30627.2	60895
904.0	15.7	3.2	30855.40	20.00	14.20	0.7	30845.2	61701
904.2	15.9	3.2	31446.87	20.00	14.40	0.7	31061.6	62509
904.4	16.1	3.2	32042.06	20.00	14.60	0.7	31276.6	63319
904.6	16.3	3.2	32640.97	20.00	14.80	0.7	31490.1	64131
904.8	16.5	3.2	33243.57	20.00	15.00	0.7	31702.2	64946
905.0	16.7	3.2	33849.82	20.00	15.20	0.7	31912.8	65763
905.2	16.9	3.2	34459.72	20.00	15.40	0.7	32122.1	66582
905.4	17.1	3.2	35073.24	20.00	15.60	0.7	32330.0	67403
905.6	17.3	3.2	35690.35	20.00	15.80	0.7	32536.6	68227
905.8	17.5	3.2	36311.05	20.00	16.00	0.7	32741.8	69053
906.0	17.7	3.2	36935.30	20.00	16.20	0.7	32945.8	69881
906.2	17.9	3.2	37563.08	20.00	16.40	0.7	33148.6	70712
906.4	18.1	3.2	38194.39	20.00	16.60	0.7	33350.1	71544
906.6	18.3	3.2	38829.19	20.00	16.80	0.7	33550.4	72380
906.8	18.5	3.2	39467.47	20.00	17.00	0.7	33749.5	73217

Comparison of the 3 Alternatives

Excludes 3 spillway gates which are constant in the three alternatives

Pneumatic gates assumed to be fully lowered at 896.4

POOL Elevation (ft)	Discharge (cfs) Dual Labyrinth	Single Labyrinth	Pneum Gates
896.0	0	0	0
896.2	0	0	0
896.4	24	44	11,434
896.6	130	238	11,860
896.8	292	532	12,292
897.0	502	910	12,728
897.2	756	1,362	13,169
897.4	1,050	1,879	13,616
897.6	1,381	2,455	14,067
897.8	1,746	3,083	14,523
898.0	2,141	3,757	14,984
898.2	2,564	4,469	15,450
898.4	3,011	5,215	15,921
898.6	3,480	5,987	16,396
898.8	3,967	6,783	16,876
899.0	4,470	7,595	17,360
899.2	4,986	8,419	17,849
899.4	5,514	9,252	18,343
899.6	6,050	10,090	18,841
899.8	6,592	10,930	19,343
900.0	7,140	11,768	19,850
900.2	7,778	12,603	20,361
900.4	8,499	13,433	20,877
900.6	9,279	14,255	21,396
900.8	10,108	15,070	21,920
901.0	10,977	15,877	22,448
901.2	11,878	16,676	22,981
901.4	12,806	17,466	23,517
901.6	13,753	18,248	24,058
901.8	14,715	19,023	24,603
902.0	15,687	19,792	25,151
902.2	16,665	20,556	25,704
902.4	17,645	21,317	26,261
902.6	18,624	22,076	26,822
902.8	19,600	22,835	27,386
903.0	20,572	23,595	27,955
903.2	21,538	24,358	28,527
903.4	22,498	25,125	29,104
903.6	23,452	25,898	29,684
903.8	24,400	26,677	30,268
904.0	25,344	27,463	30,855
904.2	26,285	28,256	31,447
904.4	27,223	29,058	32,042
904.6	28,162	29,865	32,641
904.8	29,103	30,679	33,244
905.0	30,048	31,496	33,850
905.2	30,999	32,314	34,460
905.4	31,956	33,129	35,073
905.6	32,922	33,937	35,690
905.8	33,898	34,733	36,311
906.0	34,883	35,510	36,935
906.2	35,877	36,261	37,563
906.4	36,880	36,975	38,194
906.6	37,889	37,644	38,829
906.8	38,901	38,254	39,467



Stanley Consultants INC.

Computed	A. Judd	Date	10/12/11	Subject	Lake Delhi Dam Reconstructin
Checked b	M. Weber	Date	12/9/2011		Dam Hydraulics - Sluice Pipes
Approved by		Date			

Description:

Analyze stage-discharge for Lake Delhi Sluice Pipes

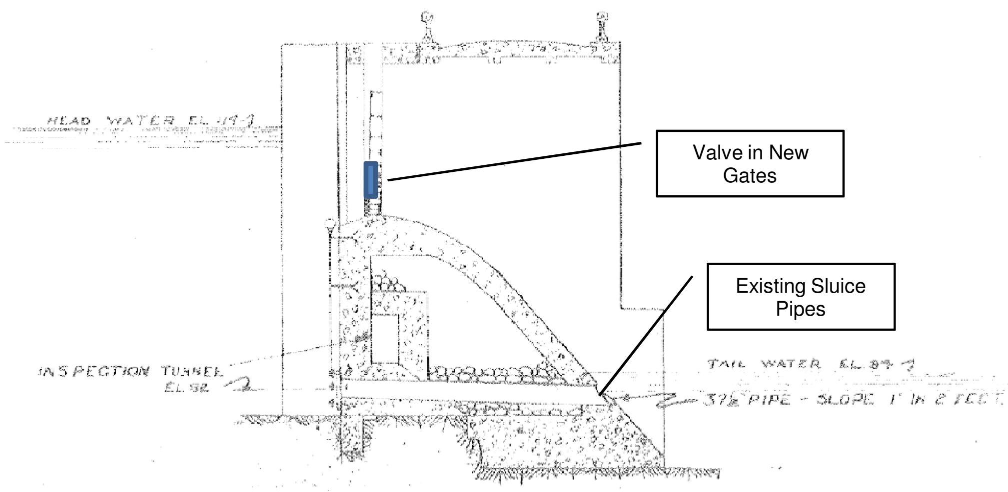
Reference:

(1) USBR, *Design of Small Dams,* 1987.
(2) Chow Ven Te; *Open Channel Hydraulics*; McGraw-Hill, 1958.

Analysis:

Existing drawings show that there may be dual sluice pipes through one of the spillway piers

In addition valves are being considered for installation within the new spillway gates



SECTION THRU DAM AT PIER D SHOWING SLUICE GATE

Elevations provided are on local datum which converts to NGVD 29
by Local Datum + 774.8

Sluice Pipe Info.		
Number	2	
Center Elev	856.8	'
Dia.	37.5	"
Length	40	'+/-

Valve info.		
Number	2	
Center Elev	884	'
Dia.	30	"
Length	2	'+/-

Discharge through the sluice pipe is defined by:

$$Q = CA\sqrt{2gH}$$

where:

A = area of the opening,
H = difference between the upstream and downstream water levels, and,
C = discharge coefficient for the submerged orifice or the tube flow.

Where C = 0.62

Stage-Discharge

Normal Tailwater assumed to be 860 ft

Headwater El. (ft)	Tailwater El. (ft)	Q_{sluice} (cfs)	Q_{valves} (cfs)	
862	860	104	0	
863	860	128	0	
864	860	148	0	
865	860	165	0	
866	860	181	0	
867	860	195	0	
868	860	209	0	
869	860	221	0	
870	860	233	0	
871	860	245	0	
872	860	256	0	
873	860	266	0	
874	860	276	0	
875	860	286	0	
876	860	295	0	
877	860	304	0	
878	860	313	0	
879	860	322	0	
880	860	330	0	
881	860	338	0	
882	860	346	0	
883	860	354	0	
884	860	362	0	
885	860	369	47	
886	860	376	67	
887	860	384	82	
888	860	391	94	
889	860	398	106	
890	860	404	116	
891	860	411	125	
892	860	418	134	
893	860	424	142	
894	860	430	149	
895	860	437	157	
896	860	443	164	Normal Pool
897	862	437	170	
898	864	430	177	
899	866	424	183	
900	868	418	189	

Stanley Consultants INC.

Computed by	A. Judd	Date	10/27/2011	Job No.	23601
Checked by	M. Weber	Date	12/9/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydraulics
					Cofferdam Reqm'ts

Description:

Review cofferdam reqm'ts using 3 gates as flow bypass

Reference:

(1) Iowa DNR, *Design Criteria and Guidelines for Iowa Dams*, T.B. 16, 1990.
(2) USBR, *Design of Small Dams,* 1987.
(3) Chow Ven Te; *Open Channel Hydraulics*; McGraw-Hill, 1958.
(4) Henry T. Falvey, *Hydraulic Design of Labyrinth Weirs*, ASCE Press, 2003.

Analysis:

First stage of construction will be repair of the powerhouse, a D/S cofferdam will be provided at the stilling basin. Once the powerhouse repairs are complete, construction of the south embankment and spillways will commence. Cofferdams will be constructed both upstream and downstream of the embankment

Task 1 **Estimate height of cofferdams required for given flows**

Return Period	Flow (cfs)
1	1400
2	4500
5	8700
10	12300

PHASE I - Open river bypass, small cofferdam U/S and D/S of Gates/Powerhouse

PHASE II - Gate bypass, larger cofferdam around south embankment/spwy construction

Spwy Gates		
Crest Elev.	879.8	ft
# of Gates	3	
Open Width	25.0	ft
Open Height	20.0	ft
C	0.68	

	PHASE I		PHASE II		
U/S W.S.E (ft-msl)	FLOW Q_{river} (cfs)	D/S W.S.E. (ft-msl)	GATE FLOW 3 GATES H_{gates} (ft)	Q_{gates} (cfs)	D/S W.S.E. (ft-msl)
868.0	276	859.1	0	0	858.5
868.5	603	859.8	0	0	858.5
869.0	991	860.6	0	0	858.5
869.5	1,440	861.5	0	0	858.5
870.0	1,952	862.4	0	0	858.5
870.5	2,524	863.3	0	0	858.5
871.0	3,159	864.2	0	0	858.5
871.5	3,855	865.2	0	0	858.5
872.0	4,612	866.1	0	0	858.5
872.5	5,432	867.0	0	0	858.5
873.0	6,312	867.9	0	0	858.5
873.5	7,255	868.7	0	0	858.5
874.0	8,259	869.5	0	0	858.5
874.5	9,324	870.3	0	0	858.5
875.0	10,452	871.0	0	0	858.5
875.5	11,641	871.6	0	0	858.5
876.0	12,891	872.2	0	0	858.5
876.5	14,203	872.8	0	0	858.5
877.0	15,577	873.4	0	0	858.5
877.5	17,012	874.0	0	0	858.5
878.0	18,509	874.6	0	0	858.5
878.5	20,067	875.2	0	0	858.5
879.0	21,687	875.8	0	0	858.5
879.5	23,369	876.4	0	0	858.5
880.0	25,112	877.1	0.2	23	858.6
880.5	26,917	877.8	0.7	149	858.9
881.0	28,783	878.5	1.2	335	859.3
881.5	30,711	879.2	1.7	565	859.8
882.0	32,701	879.9	2.2	832	860.3
882.5	34,752	880.6	2.7	1,131	860.9
883.0	36,865	881.2	3.2	1,460	861.5
883.5	39,039	881.8	3.7	1,815	862.1
884.0	41,275	882.4	4.2	2,195	862.8
884.5	43,573	882.9	4.7	2,598	863.4
885.0	45,932	883.5	5.2	3,024	864.0
885.5	48,353	884.0	5.7	3,470	864.7
886.0	50,835	884.5	6.2	3,937	865.3
886.5	53,379	885.1	6.7	4,422	865.9
887.0	55,985	885.8	7.2	4,927	866.5
887.5	58,652	886.5	7.7	5,448	867.1
888.0	61,381	887.3	8.2	5,988	867.6
888.5			8.7	6,544	868.1
889.0			9.2	7,116	868.6
889.5			9.7	7,704	869.1
890.0			10.2	8,307	869.6
890.5			10.7	8,925	870.0
891.0			11.2	9,558	870.4
891.5			11.7	10,205	870.8
892.0			12.2	10,866	871.2
892.5			12.7	11,541	871.5
893.0			13.2	12,229	871.9
893.5			13.7	12,931	872.2
894.0			14.2	13,645	872.6
894.5			14.7	14,372	872.9
895.0			15.2	15,111	873.2

PHASE I FLOW estimated from HEC-RAS model of post-breach river channel conditions at dam

D/S W.S.E.'s estimated from HEC-RAS model of river channel conditions just downstream of Lake Delhi Dam

GATE FLOW computed using Ref 2 Equations, assuming open gates

Stanley Consultants INC.

				Job No.	23601
Computed	A. Judd	Date	11/30/2011	Subject	Lake Delhi Dam
Checked b	M. Weber	Date	12/9/2011		Hydraulics
Approved by		Date			Headwater/Tailwater

Description:

Estimate Headwater and Tailwater elevations at Lake Delhi Dam for given flows

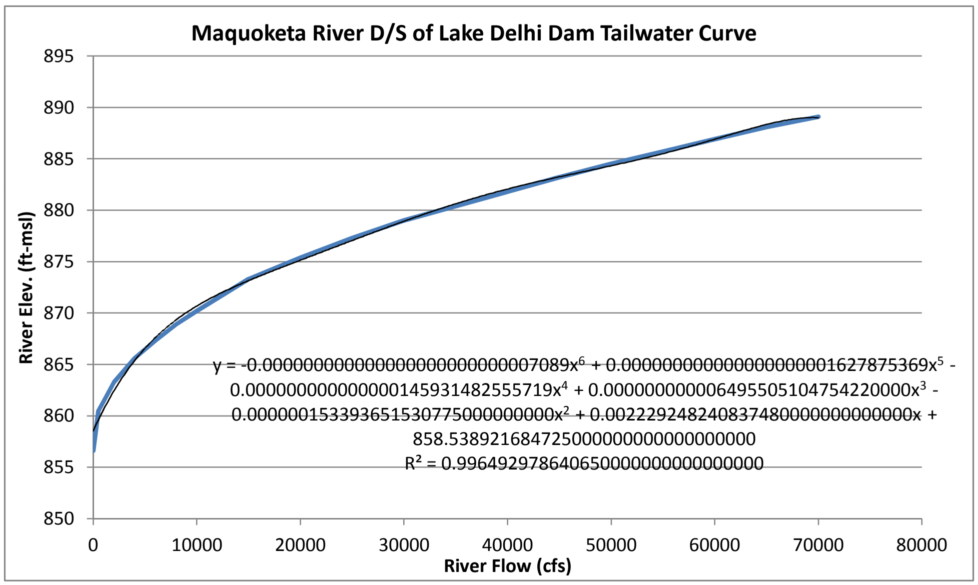
Analysis:

A HEC-RAS model was developed of the proposed lake delhi dam which was used to establish tailwater/headwater curves for a given dam discharge or river flow

Tailwater Curve
From HEC-RAS Model
At River Sta. 60797

Just downstream of dam
W/ Proposed Dam

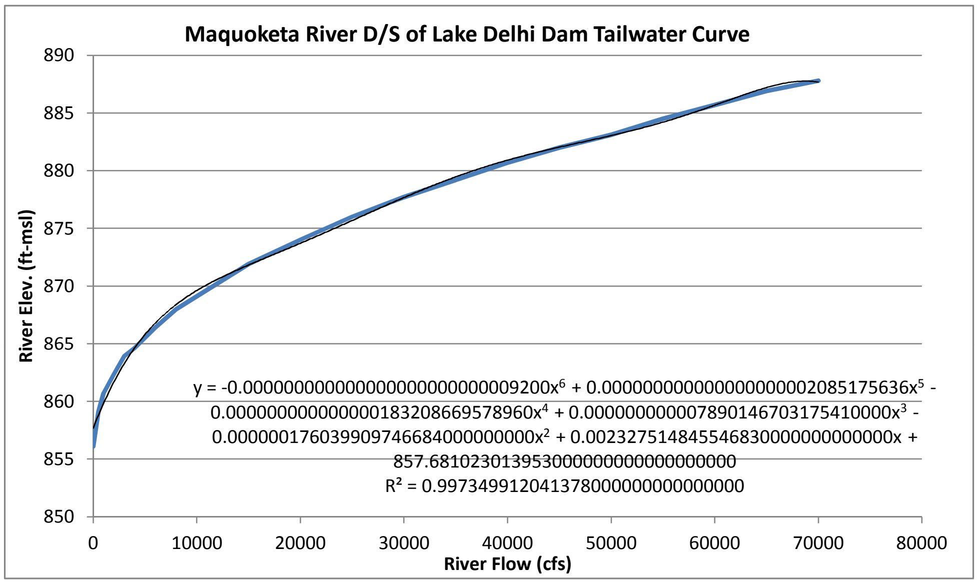
Q (cfs)	W.S.E. (ft)
0	856.6
500	860.5
1000	861.4
2000	863.3
4000	865.6
6000	867.3
8000	868.9
10000	870.2
15000	873.3
20000	875.4
25000	877.3
30000	879
35000	880.4
40000	881.8
45000	883.2
50000	884.5
55000	885.7
60000	886.9
65000	888.1
70000	889.1



Tailwater Curve
From HEC-RAS Model
At River Sta. 59617

~1200' downstream of dam
W/ Proposed Dam

Q (cfs)	W.S.E. (ft)
0	856.1
500	859.1
1000	860.7
2000	862.4
3000	863.9
4000	864.6
6000	866.4
8000	868
10000	869.1
15000	871.9
20000	874
25000	876
30000	877.7
35000	879.2
40000	880.7
45000	882
50000	883.1
55000	884.5
60000	885.7
65000	886.9
70000	887.8



Stanley Consultants INC.

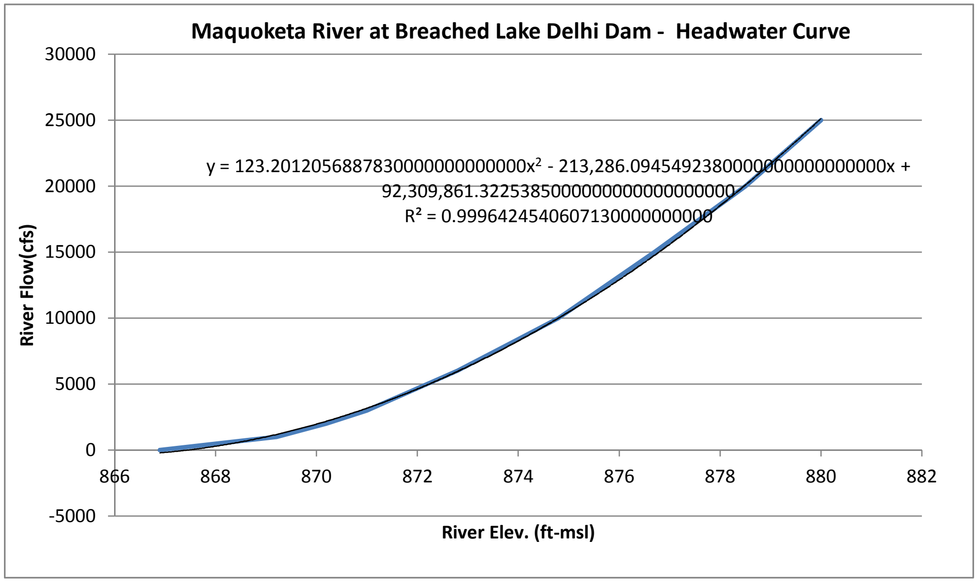
				Job No.	23601
Computed	A. Judd	Date	11/30/2011	Subject	Lake Delhi Dam
Checked b	M. Weber	Date	12/9/2011		Hydraulics
Approved by		Date			Headwater/Tailwater

Headwater Curve
From HEC-RAS Model
At River Sta. 61003.11

This curve is used to develop cofferdam height requirements on the upstream side of the dam during Phase I construction

Just Upstream of Dam
Existing (breached dam) Condition

Q (cfs)	W.S.E. (ft)
0	866.9
992	869.2
2000	870.2
3000	871
4000	871.6
6000	872.8
8000	873.8
10000	874.8
15000	876.7
20000	878.5
25000	880



Appendix C

Geotechnical Analysis and Design

C.1 Subsurface Investigation

Geotechnical Boring Program

A boring program was established to collect subsurface data necessary to evaluate the construction of existing embankments, type and condition of dam foundation materials and complete the analysis of several preliminary design features. Borings were also advanced at several properties near the dam site to evaluate materials for potential use as borrow, for earthen embankment construction. The geotechnical investigation was carried out by Braun Intertec.

Borings ST-1 and ST-2 were drilled through the north embankment. Borings ST-4 through ST-7 were drilled within the breach limits of the earthen dam. Borings ST-8 through ST-10 were drilled through the embankment that remains south of the breach. Borings ST-11 and ST-12 were drilled through the existing powerhouse and spillway bridge deck. All borings were advanced to sufficient depths to allow analysis and evaluation of soil and bedrock foundations for embankment/structural stability and seepage.

Soil samples were collected with split spoon and Shelby tube samplers. Blow counts (N-Values) were recorded by the Braun drilling crew. Soil samples were classified and tested. Testing on the soil boring samples included moisture content and dry density, Atterburg Limits, unconfined compression testing, and gradations.

Boring ST-11 and ST-12 were advanced through the existing walls and piers of the powerhouse and spillway. Continuous core samples were collected in the concrete and underlying bedrock foundation. Percent recovery and Rock Quality Designation (RQD) were recorded by the Braun drilling crew for all bedrock core collected. The bedrock core was classified and representative samples tested for unconfined strength.

Borings were also drilled at several properties in the vicinity of the dam to determine the extents and accessibility of loess and till materials for potential use in reconstruction of the earthen portion of the dam. Borings were advanced on the Wilson, Freiburger, and Harbach properties. Soil samples collected from the borings were classified and tested for moisture content, compaction testing and Atterberg Limits.

Borrow and Subsurface Soil Conditions

The north embankment subsurface material consists of up to 28 feet of sand and gravel fill material, underlain by approximately 10 feet of sandy lean clay. The sandy lean clay is underlain by approximately 15 feet of poorly graded sand to approximately elevation 852, where limestone bedrock is encountered.

At the center of the channel, boring ST-5 indicates that the bedrock has dropped off substantially (approximate elevation 842). A thick (20–25 ft) gravely sand (SP) layer lies between the base of the channel and the weathered limestone. At the south end of the channel, Borings ST-6 and ST-7 indicate a thin layer of sandy lean clay underlain by 70 to 80 ft of gravely sand material. No bedrock was encountered. Likewise, borings ST-8 through ST-10, advanced through the south embankment indicate clay fill materials underlain by gravely sand.

Sandy gravels encountered were primarily medium dense to dense with very little fines and high gravel faction. Bedrock encountered was generally moderately hard highly weathered limestone. Rock encountered at the north end of the power house was good in quality with high recoveries and RQD values. Boring ST-11 drilled directly through the power house showed moderately hard to hard vuggy limestone with relatively high recovery and RQD values as well as 3,300 psi to 11,000 psi unconfined compressive strength. From the south end of the powerhouse extending south into the channel, rock samples recovered showed low quality rock with numerous voids. The rock showed very low recovery and RQD values.

The borrow evaluation borings typically encountered two soil types underlying the topsoil. A silty clay loess, overlying a silty clay glacial till soil. The loess soils, while potentially acceptable for embankment construction were typically encountered at very high moisture contents, requiring excavation and spreading (farming) in order to get the material to an acceptable moisture content for placement and compaction. The till soils typically provide a superior material for embankment construction and have in-situ moisture contents closer to those required for placement and compaction in an earthen embankment. The till soils were encountered at depths of 12 feet or more, under the loess soils, so significant excavation would be required to develop these soils for borrow. Additional future investigations by the Contractor may locate the till soils at shallower depths for borrow development. Both materials indicate acceptable strength and seepage properties for use in earthen dams.

Design Parameters

Parameters were developed for use in the seepage and slope stability analysis based on published tables in EM 1110-2-2504 [6]. Laboratory test Atterberg Limits were used to determine effective (drained) parameters for fill materials (empirical correlation between friction angle and plasticity index from triaxial tests on normally consolidated clays) [6]. Refer to Table C-1 below for design parameters.

Table C-1 Geotechnical Design Parameters

Material Type	Unit Weight (pcf)	Undrained Shear Strength, c (psf)	Friction Angle, φ (deg)	Drained Friction Angle, φ' (deg)
Glacial Till (CL)	110	2000	0	28
Loess (CL)	110	1000	0	28
Foundation (SP)	125	0	30	30

At the time of this report, Braun was in the process of drilling one additional borrow site boring and one additional boring through the powerhouse. The boring logs and corresponding subsurface data for the two additional borings will be provided in subsequent submittals.

The full preliminary geotechnical report developed by Braun Intertec [1] is included within this appendix.

C.2 Seepage Analysis

Methods

Seepage analysis was conducted for proposed embankment and seepage control measures using GeoStudio's SEEP/W finite element seepage modeling program. Soil classification and laboratory gradation results were used to develop input seepage parameters. Permeability coefficients were determined according to Hazen's empirical formula using D_{10} values (particle diameter corresponding to 10% passing). The proposed service and auxiliary embankments (located within the current breach) were modeled with various cutoff depths and configurations. Horizontal blanket drains were also included in the model, for safe collection and conveyance of seepage flows, without saturating the downstream slope of the embankments.

Results

Exit gradients (exit gradient is defined as the rate of change of total head pressure with distance) and seepage flow rates were analyzed to come up with an optimized and adequate cutoff/drainage system. To achieve the target factor of safety of 1.5, the target exit gradient was assumed as 0.67. This assumes a critical gradient of the material of 1.0. To achieve the target gradient, the sheet pile cutoff was designed as 35 feet below base of the new embankment (into sand foundation).

All seepage computations are provided following the narrative portion of this appendix.

C.3 Stability Analysis

Methods

Stability analysis was carried out using GeoStudio's SLOPE/W (2007) modeling program [3]. Spencer's Method was used to find minimum factors of safety for various loading conditions. The United States Army Corps of Engineers' Slope Stability Engineering Manual [4] was consulted for required loading conditions and factors of safety for new earth and rock-fill dams. The Iowa Department of Natural Resources Technical Bulletin on Dam Design Criteria [5] was used as a baseline for the determination of embankment slopes. An end-of-construction case was analyzed on the downstream embankment slope using total stress (undrained) soil parameters. The minimum required factor of safety for the total stress condition is 1.3. Also, long-term steady state seepage conditions were modeled using effective stress soil parameters (drained parameters) with a required minimum factor of safety of 1.5. For the long-term steady state condition, a maximum surcharge pool was assumed with water to the top of the spillway crest (top of labyrinth weir). To account for the decreased water surcharge loading as a result of the labyrinth weir, 50% of the water surcharge load was considered along the width of the weir. A rapid drawdown condition was not modeled because it is unlikely that the pool will ever be rapidly drained.

Results

For proposed new embankment sections, slope stability was analyzed for embankments constructed of locally available borrow materials (identified in Braun Intertec investigation of borrow areas) as well as roller compacted concrete (RCC). It was determined that 3 horizontal on 1 vertical slopes are required for the both the upstream and the downstream faces of embankments constructed of loess or till in order to satisfy all design requirements. Roller compacted concrete faced embankments meet design requirements if constructed with 2.5 horizontal on 1 vertical downstream slopes.

Results of the stability analysis for the various modeled cases are provided in Table C-2 below.

Table C-2 Stability Analysis Results

Spillway	Design Case	Min. FOS (FS_{min})	Required FOS (FS_{req})
Service	Total Stress	1.995	1.3
	Effective Stress	1.539	1.5
Auxiliary	Total Stress	1.916	1.3
	Effective Stress	1.607	1.5
Auxiliary (RCC)[1]	Total Stress	1.817	1.3
	Effective Stress	1.498	1.5

[1] Note that RCC option for aux. spillway was modeled with 2.5:1 slopes.

All stability computations are provided following the narrative portion of this appendix.

C.4 Settlement Analysis

General

Long -term consolidation settlement is not anticipated as embankment construction will take place on subsurface sands. Given the sand foundation material, a majority of settlement will occur as construction proceeds. Settlement within the embankment fill will be limited by proper placement, moisture control, and compaction of embankment fill. As mentioned previously, some of the loess borrow material may require drying prior to placement in order to achieve desired compaction and density values.

C.5 References

1. Braun Intertec Corporation. *Lake Delhi Dam Restoration Geotechnical Services Report*. December 5, 2011.
2. Seepage Modeling with SEEP/W. GeoStudio 2004. GEO-SLOPE International Ltd.
3. Stability Modeling with SLOPE/W. GeoStudio 2007. GEO-SLOPE International Ltd.
4. U.S. Army Corps of Engineers. EM 1110-2-1902. *Slope Stability*. October 2003.
5. Iowa Department of Natural Resources. Technical Bulletin No. 16. *Design Criteria and Guidelines for Iowa Dams*. December 1990.
6. U.S. Army Corps of Engineers. EM 1110-2-2504. *Engineering and Design of Sheet Pile Walls*. March 1994.
7. U.S. Army Corps of Engineers. EM 1110-2-1913. *Design and Construction of Levees*. April 2000.
8. U.S. Army Corps of Engineers. EM 1110-2-1902. *Slope Stability*. October 2003.
9. U.S. Army Corps of Engineers. EM 1110-2-2300. *Engineering and Design – General Design and Construction Considerations for Earth and Rock-Fill Dams*. July 2004.
10. U.S. Army Corps of Engineers. EM 1110-2-1901. *Seepage Analysis and Control for Dams*. April 1993.
11. U.S. Army Corps of Engineers. EM 1110-2-2006. *Roller-Compacted Concrete*. January 2000.

Geotechnical Computations and Reference Material

Braun Intertec Corp. Geotechnical Report

BRAUN INTERTEC

Braun Intertec Corporation
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Suite 100
Cedar Rapids, IA 52404

Phone: 319.365.0961
Fax: 319.365.1306
Web: braunintertec.com

December 5, 2011 Project CR-11-00665

Mr. Bill Holman, PE
Stanley Consultants Inc.
5775 Wayzata Boulevard, Suite 300
Minneapolis, MN 55416

Re: Geotechnical Services
Soil Boring, Rock Coring, Concrete Coring, and Laboratory Testing
Lake Delhi Dam Restoration
Delhi, Iowa

Dear Mr. Holman:

The purpose of this submittal is to transmit to you the results of our soil borings, rock coring, concrete coring and laboratory testing to assist Stanley Consultants with their analysis and concept development for restoration of the for of the existing levee along the Mississippi River in Clinton, Iowa.

Procedures

Site Access and Utility Clearance

Prior to drilling, our drillers staked the boring locations in the field using a cloth tape and measuring from existing site features based on the plan that you provided to us showing recent survey information. Elevations on the boring logs were interpolated from the topographic lines on these plans. Prior to drilling, we contacted Iowa One Call to request they clear public utilities in the areas to be explored.

Penetration Test Borings

We mobilized an ATV-mounted drill rig to drill ten borings (ST1 to ST10) to depths of about 40 to 80 feet below existing grade or below the existing water level in the Maquoketa River along the centerline of the previously existing dam. Two borings were cored through the existing dam and into the underlying bedrock. These two borings (ST11 and ST12) extended approximately 107 feet below the top of the dam with the upper 57 feet consisting of concrete. Seven borings (B1 through B7) were drilled in farm fields south of the dam site for the purpose of evaluating potential borrow material for new dam construction. The locations of all of these borings are shown on the attached location plans.

The borings were advanced using 2¼-inch hollow stem augers or wash boring methods below the water table. Rock coring was performed with a standard NQ size core barrel. Coring through the concrete dam was performed with a larger HQ core barrel. In general, penetration test samples or thin-walled tube samples of the soils were taken at 2½- to 5-foot intervals. Actual sample intervals and corresponding depths are shown on the boring logs. In the borrow borings, samples were obtained from the augers.

Water level observations are recorded on the boring logs where possible. In some cases, the addition of drilling fluid during wash boring methods obscured water levels. After completion the boreholes that were drilled into sand below the water table were allowed to collapse. Borrow borings and abutment borings above the water table were backfilled with cuttings. The concrete dam borings were sealed with cement/bentonite grout.

Concrete Coring

Concrete coring was performed at 17 locations on the existing concrete portion of the Lake Delhi Dam at the locations shown on the two core location sketches that were provided to us. Cores were obtained with an electric portable coring machine using a diamond-tipped core barrel 2 to 3 inches in diameter.

Sample Review and Laboratory Testing

The geologic materials encountered were visually and manually classified in accordance with ASTM Standard Practice D 2488 by a geotechnical engineer. A chart explaining the classification system is attached. Samples were placed in jars or bags and returned to our facility for review, storage and laboratory testing.

After initial classification, draft boring logs were provided and a testing schedule was established. The testing program included: mechanical analyses (ASTM D422), unconfined compression tests on rock cores (ASTM D2983), plastic and liquid limits tests (ASTM D4318), laboratory compaction tests (ASTM D698), moisture content measurements (ASTM D2216), and hand penetrometer tests. The results of our tests are provided on the boring logs at the corresponding sample depths. Grain size curves for the mechanical analyses and the moisture density curves for laboratory compaction tests are included on separate sheets.

Results

The results of our field and laboratory work are included with this submittal in the following order:

- Dam Centerline Boring Location Plan and Logs of Borings ST1 through ST12
- Borrow Boring Location Plan and Logs of Borings B1 through B7
- Grain Size Curves
- Moisture-Density Relationship Curves
- Concrete Core Location Plans
- Table of Concrete Core Compressive Strengths and Pictures of Fractured Cores

Use of Report & Standard of Care

This report is for the exclusive use of the parties to which it has been addressed. Without written approval, we assume no responsibility to other parties regarding this report. Our evaluation, analyses and recommendations may not be appropriate for other parties or projects.

This report is based on information obtained from the borings and our experience with these types of soils. Conditions could exist in places other than our borings that could adversely affect levee performance. In performing its services, Braun Intertec used that degree of care and skill ordinarily exercised under similar circumstances by reputable members of its profession currently practicing in the same locality. No warranty, express or implied, is made.

To have questions answered or schedule a time to meet and discuss these results, please feel free to call us at 319.310.6213.

Sincerely,

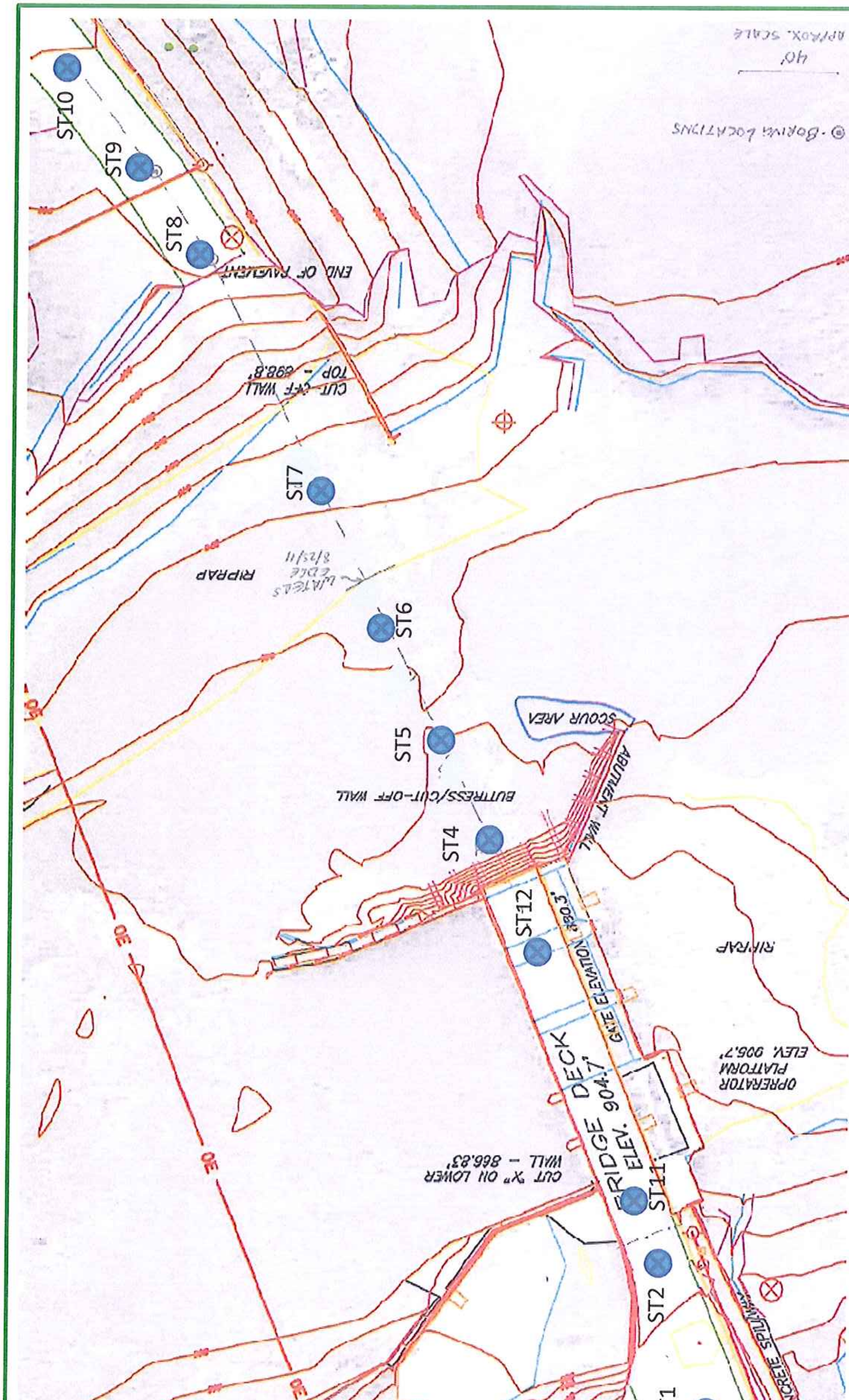
BRAUN INTERTEC CORPORATION

Timothy T. Wiles
Principal/Senior Engineer

CR-11-00665report

BRAUN INTERTEC

Dam Centerline Boring Location Plan and Logs of Borings ST1 through ST12



Source: Stanley Consultants

Approximate scale as shown

Lake Delhi Dam Restoration Delhi, Iowa	Project: CR-11-00665	Dam Centerline Boring Location Plan	BRAUN INTERTEC

LOG OF BORING

BRAUN INTERTEC

Braun Project CR-11-00665 **Geotechnical Evaluation** **Lake Dehli** **Delhi, Iowa**	BORING: **ST-1** LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: NQ Wireline Core/ Power Auger	DATE: 10/14/11	SCALE: 1" = 4'

Elev. feet 905.0	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
904.2	0.8	PAV	Portland Cement Concrete			
		FILL	FILL: Poorly Graded Gravel with Sand, gray to brown	17		
898.0	7.0	FILL	FILL: Lean Clay with Sand and Gravel, brown to dark brown	5		
892.0	13.0	LS	Very highly weathered LIMESTONE, gray	50/5		
889.0	16.0	LS	Auger Refusal at 16 feet Highly weathered LIMESTONE, gray, moderately hard, highly fractured, numerous small voids, clay seams from 26 to 36 feet			REC = 41% RQD = 12% REC = 10% RQD = 0%

(See Descriptive Terminology sheet for explanation of abbreviations)

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LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-1 (cont.) LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: NQ Wireline Core/ Power Auger	DATE: 10/14/11	SCALE: 1" = 4'

Elev. feet 873.0	Depth feet 32.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
865.0	40.0		Highly weathered LIMESTONE, gray, moderately hard, highly fractured, numerous small voids, clay seams from 26 to 36 feet *(continued)*			REC = 25% RQD = 0%
			END OF BORING. Water not observed while drilling. Boring then backfilled.			

(See Descriptive Terminology sheet for explanation of abbreviations)

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BRAUN INTERTEC

LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-2 LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: NQ Wireline Core./HSA	DATE: 10/13/11	SCALE: 1" = 4'

(See Descriptive Terminology sheet for explanation of abbreviations)

Elev. feet 905.0	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
904.5	0.5	PAV	Portland Cement Concrete				
903.0	2.0	AIR	Air Void				
892.5	12.5	FILL	FILL: Poorly Graded Sand with Gravel and occasional Clay seams	6			
				7			
877.0	28.0	FILL	FILL: Poorly Graded Gravel with Sand, gray	7			
				50			
				11			
873.0	32.0	CL	SANDY LEAN CLAY, gray, wet, very soft (Alluvium)	WH		19	

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Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-2 (cont.) LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: NQ Wireline Core./HSA	DATE: 10/13/11	SCALE: 1" = 4'

Elev. feet 873.0	Depth feet 32.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
		CL	SANDY LEAN CLAY, with LIMESTONE fragments, brown, moist, medium (Alluvium)	6		20	
868.0	37.0	SP	POORLY GRADED SAND with GRAVEL, gray, wet, medium dense to dense (Residuum)	31			
				11			
				39	▽		
852.0	53.0	LS	Auger Refusal at 53 feet Highly weathered LIMESTONE, gray, highly fractured, moderately hard, small voids				REC = 100% RQD = 70%
							REC = 95%

(See Descriptive Terminology sheet for explanation of abbreviations)

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LOG OF BORING

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-2 (cont.)**

LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: NQ Wireline Core./HSA	DATE: 10/13/11	SCALE: 1" = 4'

Elev. feet 841.0	Depth feet 64.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
832.0	73.0		Highly weathered LIMESTONE, gray, highly fractured, moderately hard, small voids *(continued)*				RQD = 53%
			END OF BORING. Wate observed at 49 feet while drilling. Boring then backfilled.				

(See Descriptive Terminology sheet for explanation of abbreviations)

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BRAUN℠ INTERTEC

LOG OF BORING

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-4**

LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer | METHOD: NQ Wireline Core./HSA | DATE: 9/9/11 | SCALE: 1" = 4'

(See Descriptive Terminology sheet for explanation of abbreviations)

Elev. feet 868.0	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
865.0	3.0	WATER	Water			
862.0	6.0	GP	POORLY GRADED GRAVEL, with SAND and SILT, brown, wet			
861.0	7.0	CONC	Portland Cement Concrete			REC = 67% RQD = 6%
857.0	11.0	LS	Highly Weathered LIMESTONE, gray, moderately hard, slightly to moderately fractured, small voids			
853.5	14.5		VOID			
852.0	16.0	LS	Highly Weathered LIMESTONE, gray, moderately hard, highly fractured			
842.0	26.0	LS	Highly Weathered LIMESTONE, gray, moderately hard, very fractured, small voids			REC = 10% RQD = 5%
839.5	28.5	LS	Highly Weathered LIMESTONE, gray, moderately hard, highly fractured			REC = 20% RQD = 8%
837.0	31.0		VOID			
		LS				

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INTERTEC

LOG OF BORING

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-4 (cont.)**
LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer | METHOD: NQ Wireline Core./HSA | DATE: 9/9/11 | SCALE: 1" = 4'

Elev. feet 836.0	Depth feet 32.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
			Highly Weathered LIMESTONE,gray, moderately hard, highly fractured *(continued)*			
832.0	36.0	LS	Highly weathered LIMESTONE, gray, moderately hard, moderately fractured, porous, small voids			REC = 98% RQD = 51%
822.0	46.0	LS	Highly weathered LIMESTONE, moderately hard, moderately factured, vuggy			REC = 90% RQD = 72%
817.0	51.0		END OF BORING.			

(See Descriptive Terminology sheet for explanation of abbreviations)

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BRAUN
INTERTEC

LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-5 LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: NQ Wireline Core/Mud Rotary	DATE: 9/13/11	SCALE: 1" = 4'

Elev. feet 868.0	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL ▽	Tests or Notes
		WATER	Water			
866.5	1.5	SP	POORLY GRADED SAND with GRAVEL, fine- to coarse-grained, loose to medium dense, more gravel at 20 feet (Alluvium)	10 22 27		
842.0	26.0	LS	Auger Refusal at 26 feet Highly weathered LIMESTONE, gray, moderately hard, highly fractured, porous lost circulation at 31 feet			REC = 40% RQD = 22%

(See Descriptive Terminology sheet for explanation of abbreviations)

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BRAUN INTERTEC

LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-5 (cont.) LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: NQ Wireline Core/Mud Rotary	DATE: 9/13/11	SCALE: 1" = 4'

Elev. feet 836.0	Depth feet 32.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
			Highly weathered LIMESTONE, gray, moderately hard, highly fractured, porous lost circulation at 31 feet *(continued)*			
832.0	36.0	LS	Highly weathered LIMESTONE, gray, moderately hard, highly fractured, vuggy regained circulation at 44 feet			REC = 20% RQD = 3%
822.0	46.0	LS	Highly weathered LIMESTONE, gray, highly fractured			REC = 0% RQD = 0%
812.0	56.0		END OF BORING.			

(See Descriptive Terminology sheet for explanation of abbreviations)

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LOG OF BORING

BRAUN INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-6**

LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer | METHOD: Mud Rotary | DATE: 9/29/11 | SCALE: 1" = 4'

Elev. feet 867.0	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
		SP	POORLY GRADED SAND with GRAVEL, fine- to coarse-grained, moist, dense (Alluvium)	31		
860.0	7.0	CL	LEAN CLAY with SAND and GRAVEL, dark gray, moist, rather soft (Alluvium)	5		
857.5	9.5	SP	POORLY GRADED SAND, with GRAVEL, fine- to coarse-grained, wet, dense (Alluvium)		▽	
853.0	14.0	GP	POORLY GRADED GRAVEL, with SAND, wet, medium dense to dense (Alluvium)	41		No Recovery
				49		
				11		
841.0	26.0	SW	WELL-GRADED SAND, with GRAVEL, fine- to coarse-grained, wet, medium dense to dense (Alluvium)	36		

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:06

LOG OF BORING

BRAUN INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-6 (cont.)**

LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer | METHOD: Mud Rotary | DATE: 9/29/11 | SCALE: 1" = 4'

Elev. feet 835.0	Depth feet 32.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
			WELL-GRADED SAND, with GRAVEL, fine- to coarse-grained, wet, medium dense to dense (Alluvium) *(continued)*	20		
				15		
				22		
				13		
817.0	50.0		END OF BORING. Water observed at 10 feet while drilling. Boring then backfilled.			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:06

BRAUN℠ INTERTEC

LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-7 LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Mud Rotary/HSA	DATE: 9/30/11	SCALE: 1" = 4'

Elev. feet 870.0	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
		SP	POORLY GRADED SAND with SILTY and GRAVEL, trace Clay, brown, fine- to coarse-grained, moist, loose (Alluvium)			
				9		
				7		
859.0	11.0	SP	POORLY GRADED SAND with GRAVEL, brown, fine- to coarse-grained, wet, loose to medium dense (Alluvium)		▽	
				10		
				6		
				7		
				9		

(See Descriptive Terminology sheet for explanation of abbreviations)

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BRAUN INTERTEC

LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-7 (cont.) LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Mud Rotary/HSA	DATE: 9/30/11	SCALE: 1" = 4'

Elev. feet 838.0	Depth feet 32.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
			POORLY GRADED SAND with GRAVEL, brown, fine- to coarse-grained, wet, loose to medium dense (Alluvium) *(continued)*	3		
				13		
				12		
				18		
820.0	50.0		END OF BORING.			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:07

BRAUN INTERTEC

LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-8 LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Mud Rotary/HSA	DATE: 10/3/11	SCALE: 1" = 4'

Elev. feet 905.0	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
904.5	0.5	AGG	3" Asphaltic Concrete, 3" Portland Cement Concrete				
		FILL	FILL: Very Sandy Lean Clay, brown, moist				
				13		17	
				4		20	
892.0	13.0	FILL	FILL: Lean Clay with Sand and Gravel, brown, moist	10			
887.0	18.0	SP	POORLY GRADED SAND with Gravel trace Silt, fine- to coarse-grained, brown, loose	8			
				9			
878.0	27.0	SW	WELL-GRADED SAND, trace Gravel, fine- to coarse-grained, brown, medium dense (Alluvium)	16			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:07

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-8 (cont.) LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Mud Rotary/HSA	DATE: 10/3/11	SCALE: 1" = 4'

Elev. feet 873.0	Depth feet 32.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
			WELL-GRADED SAND, trace Gravel, fine- to coarse-grained, brown, medium dense (Alluvium) *(continued)*				
				12			
				22			
				22			
				16			
					∇		
				25			
				18			
				19			

(See Descriptive Terminology sheet for explanation of abbreviations)

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LOG OF BORING

BRAUN INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-8**

LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer | METHOD: Mud Rotary/HSA | DATE: 10/3/11 | SCALE: 1" = 4'

(See Descriptive Terminology sheet for explanation of abbreviations)

Elev. feet 905.0	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
904.5	0.5	AGG	3" Asphaltic Concrete, 3" Portland Cement Concrete				
		FILL	FILL: Very Sandy Lean Clay, brown, moist				
				13		17	
				4		20	
892.0	13.0	FILL	FILL: Lean Clay with Sand and Gravel, brown, moist				
				10			
887.0	18.0	SP	POORLY GRADED SAND with Gravel trace Silt, fine- to coarse-grained, brown, loose				
				8			
				9			
878.0	27.0	SW	WELL-GRADED SAND, trace Gravel, fine- to coarse-grained, brown, medium dense (Alluvium)				
				16			

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:19

BRAUN[SM]
INTERTEC

LOG OF BORING

(See Descriptive Terminology sheet for explanation of abbreviations)

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-8 (cont.) LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Mud Rotary/HSA	DATE: 10/3/11	SCALE: 1" = 4'

Elev. feet 873.0	Depth feet 32.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
			WELL-GRADED SAND, trace Gravel, fine- to coarse-grained, brown, medium dense (Alluvium) *(continued)*				
				12			
				22			
				22			
				16			
					▽		
				25			
				18			
				19			

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:19

LOG OF BORING

BRAUN INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-8 (cont.)**

LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer | METHOD: Mud Rotary/HSA | DATE: 10/3/11 | SCALE: 1" = 4'

Elev. feet 841.0	Depth feet 64.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
			WELL-GRADED SAND, trace Gravel, fine- to coarse-grained, brown, medium dense (Alluvium) *(continued)*				
837.0	68.0	SP	POORLY GRADED SAND, trace Silt and Gravel, fine- to coarse-grained, medium dense	22			
				21			
				27			
825.0	80.0		END OF BORING. Water observed at 53 feet while drilling. Boring then backfilled.				

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:19

BRAUN INTERTEC

LOG OF BORING

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: ST-9

LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer | METHOD: Mud Rotary/HSA | DATE: 10/6/11 | SCALE: 1" = 4'

Elev. feet 907.5	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
907.0	0.5	AGG	3" Asphaltic Concrete, 3" Portland Cement Concrete				
		FILL	FILL: Very Sandy Lean Clay with some Gravel trace Silt, brown, moist				
				21		12	
				5		18	
895.5	12.0	FILL	FILL: Lean Clay with Sand and Gravel, brown, moist				
				14		17	
890.5	17.0	SP	POORLY GRADED SAND with GRAVEL trace Silt, fine- to coarse-grained, loose to medium dense (Alluvium)				
				3			
				14			
				14			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:07

BRAUN INTERTEC

LOG OF BORING

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-9 (cont.)**

LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer | METHOD: Mud Rotary/HSA | DATE: 10/6/11 | SCALE: 1" = 4'

Elev. feet 875.5	Depth feet 32.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
			POORLY GRADED SAND with GRAVEL trace Silt, fine- to coarse-grained, loose to medium dense (Alluvium) *(continued)*	22	∇		
870.5	37.0	SM	SILTY SAND, fine- to coarse-grained, brown, medium dense	20			
866.5	41.0	SP	POORLY GRADED SAND with GRAVEL trace Silt, fine- to coarse-grained, medium dense to dense	32			
				25			
				24			
				23			
				30			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:07

BRAUN INTERTEC

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-9 (cont.) LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Mud Rotary/HSA	DATE: 10/6/11	SCALE: 1" = 4'

Elev. feet 843.5	Depth feet 64.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
			POORLY GRADED SAND with GRAVEL trace Silt, fine- to coarse-grained, medium dense to dense *(continued)*				
				26			
837.5	70.0						
			END OF BORING. Water observed at 36 feet while drilling. Boring then backfilled.				

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:07

LOG OF BORING

BRAUN INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-10**

LOCATION: See attached sketch

DRILLER: R. Hunt/D.Dyer | METHOD: Mud Rotary/HSA | DATE: 10/10/11 | SCALE: 1" = 4'

Elev. feet 909.5	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
909.0	0.5	PAV	3" Aggergate, 3" Portland Cement Concrete				
		FILL	FILL: Very Sandy Lean Clay, brown, moist				
				8		18	
				7		15	
897.5	12.0	FILL	FILL: Lean Clay with Sand and Gravel, brown, moist				
				13		12	
				12		20	
887.5	22.0	SP	POORLY GRADED SAND trace GRAVEL, trace Silt, brown to gray, wet, loose to dense (Alluvium)				
				8			
					▽		
				13			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:06

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-10 (cont.) LOCATION: See attached sketch

DRILLER: R. Hunt/D.Dyer	METHOD: Mud Rotary/HSA	DATE: 10/10/11	SCALE: 1" = 4'

Elev. feet 877.5	Depth feet 32.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
			POORLY GRADED SAND trace GRAVEL, trace Silt, brown to gray, wet, loose to dense (Alluvium) *(continued)*	19			
				37			
				27			
				32			
				27			
				35			
				33			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:06

BRAUN[SM]
INTERTEC

LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-10 (cont.) LOCATION: See attached sketch

DRILLER: R. Hunt/D.Dyer	METHOD: Mud Rotary/HSA	DATE: 10/10/11	SCALE: 1" = 4'

Elev. feet 845.5	Depth feet 64.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
			POORLY GRADED SAND trace GRAVEL, trace Silt, brown to gray, wet, loose to dense (Alluvium) *(continued)*				
				26			
839.5	70.0						
			END OF BORING. Water observed at 28 feet while drilling. Boring then backfilled.				

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/23/11 14:06

LOG OF BORING

BRAUN INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-11**

LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer | METHOD: NQ Wireline Core/ HQ | DATE: 11/7/11 | SCALE: 1" = 4'

Elev. feet 904.5	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
904.0	0.5	PAV	Asphaltic Concrete			
		CONC	Portland Cement Concrete			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 12/1/11 10:16

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-11 (cont.)**
LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer | METHOD: NQ Wireline Core/ HQ | DATE: 11/7/11 | SCALE: 1" = 4'

Elev. feet 872.5	Depth feet 32.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
			Portland Cement Concrete *(continued)*			
847.5	57.0	LS	Highly weathered LIMESTONE, gray, moderately hard, highly fractured, vuggy			REC = 100% RQD = 82% UC = 3300 psi REC = 79% RQD = 23%

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 12/1/11 10:16

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: ST-11 (cont.) LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: NQ Wireline Core/ HQ	DATE: 11/7/11	SCALE: 1" = 4'

Elev. feet 840.5	Depth feet 64.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
			Highly weathered LIMESTONE, gray, moderately hard, highly fractured, vuggy *(continued)*			
						REC = 79% RQD = 36%
						REC = 95% RQD = 36% UC = 11,720 psi
						REC = 100% RQD = 51%

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **ST-11 (cont.)**

LOCATION: See attached sketch

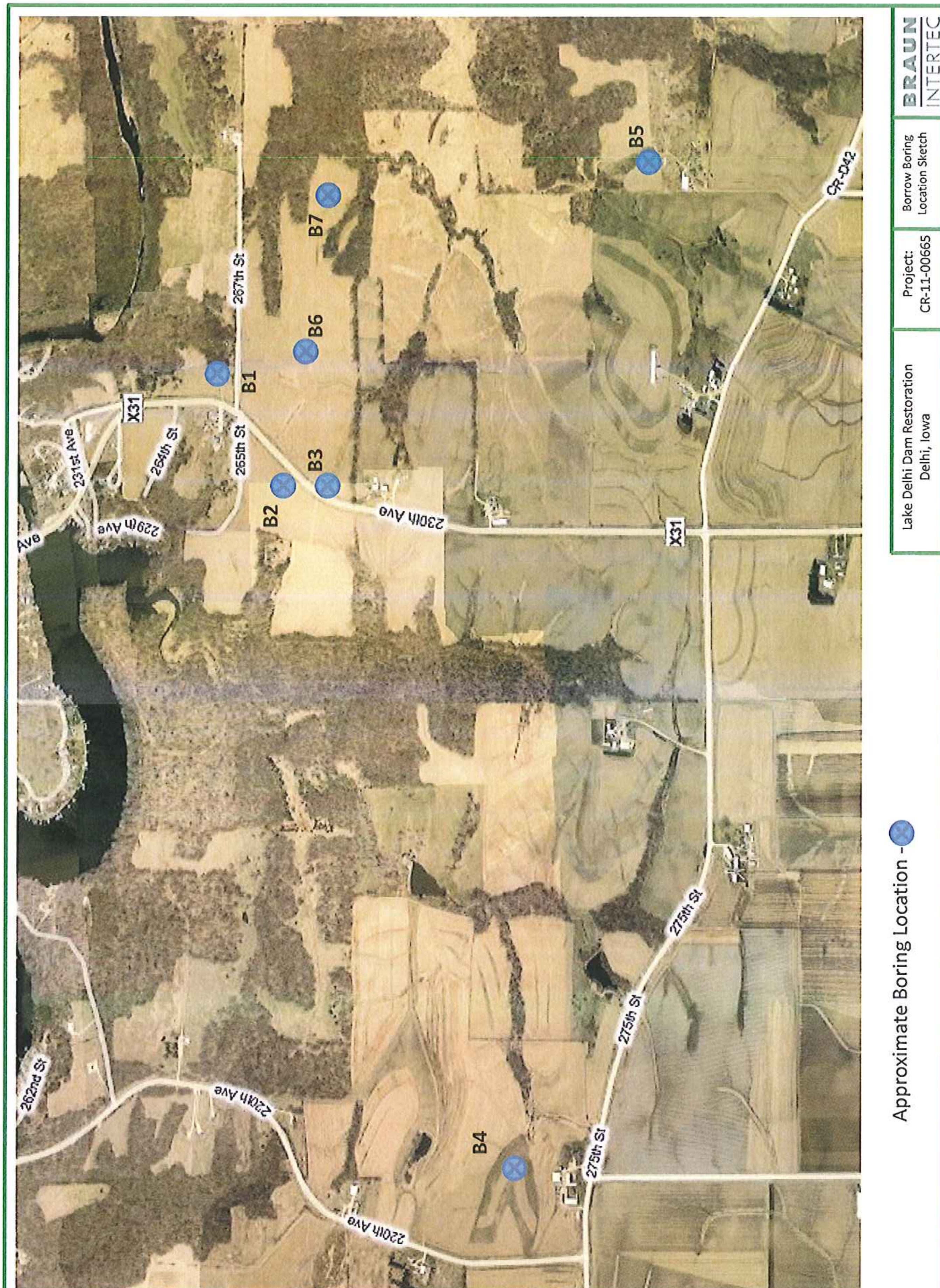
DRILLER: R.Hunt/D.Dyer | METHOD: NQ Wireline Core/ HQ | DATE: 11/7/11 | SCALE: 1" = 4'

Elev. feet 808.5	Depth feet 96.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
			Highly weathered LIMESTONE, gray, moderately hard, highly fractured, vuggy *(continued)*			REC = 100% RQD = 58% UC = 4,390 psi
797.5	107.0					
			END OF BORING. Boring then grouted.			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 12/1/11 10:16

Borrow Boring Location Plan and Logs of Borings B1 through B7


B1
B2
B3
B4
B5
B6
B7
267th St
265th St
264th St
231st Ave
229th Ave
230th Ave
X31
CR-D42
262nd St
220th Ave
275th St
Approximate Boring Location -
Lake Delhi Dam Restoration
Delhi, Iowa
Project:
CR-11-00665
Borrow Boring
Location Sketch
BRAUN
INTERTEC

BRAUN INTERTEC

LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: B-1 LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Power Auger	DATE: 10/10/11	SCALE: 1" = 4'

Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
1.0	TS	Topsoil			
25.0	CL	LEAN CLAY, brown, moist, medium (Loess)			LL = 32 PL = 23 PI = 9 Max DD = 108.8 pcf Optimum Moisture = 10.5% Natural Moisture = 26%
		END OF BORING. Water not observed while drilling. Boring then backfilled.			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/29/11 11:18

BRAUN INTERTEC

LOG OF BORING

Braun Project CR-11-00665 **Geotechnical Evaluation** **Lake Dehli** **Delhi, Iowa**	BORING: **B-2** LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Power Auger	DATE: 10/10/11	SCALE: 1" = 4'

Depth feet 0.0	Symbol		Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
1.0	TS		Topsoil			
25.0	CL		LEAN CLAY, brown, moist, medium (Loess)			LL = 34 PL = 23 PI = 11 Max DD = 107.6 pcf Optimum Moisture = 17.7% Natural Moisture = 25%
			END OF BORING. Water not observed while drilling. Boring then backfilled.			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/29/11 11:18

BRAUN℠ INTERTEC

LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: B-3 LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Power Auger	DATE: 10/10/11	SCALE: 1" = 4'

Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
1.0	TS	Topsoil			
25.0	CL	LEAN CLAY, brown, moist, medium (Loess)			LL = 32 PL = 23 PI = 9 Max DD = 110.8 pcf Optimum Moisture = 14.3% Natural Moisture = 30%
		END OF BORING. Water not observed while drilling. Boring then backfilled.			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/29/11 11:18

BRAUN[SM]
INTERTEC

LOG OF BORING

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: **B-4**

LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer | METHOD: Power Auger | DATE: 10/10/11 | SCALE: 1" = 4'

Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
1.0	TS	Topsoil			
18.0	CL	LEAN CLAY, brown, moist, medium (Loess)			LL = 34 PL = 23 PI = 11 Max DD = 106.2 pcf Optimum Moisture = 13.9% Natural Moisture = 35 %
25.0	CL	SANDY LEAN CLAY, trace Gravel, brown, moist, rather stiff (Glacial Till)			LL = 30 PL = 19 PI = 16 Max DD = 119.0 pcf Optimum Moisture = 10.7% Natural Moisture = 16%
		END OF BORING. Water not observed while drilling. Boring then backfilled.			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/29/11 11:18

BRAUN[SM]
INTERTEC

LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: B-5 LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Power Auger	DATE: 10/10/11	SCALE: 1" = 4'

Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
1.0	TS	Topsoil			
12.0	CL	LEAN CLAY, brown, moist, medium (Loess)			LL = 29 PL = 20 PI = 9 Max DD = 112.7 pcf Optimum Moisture = 14.8 % Natural Moisture = 24%
25.0	CL	SANDY LEAN CLAY, trace Gravel, brown, moist, rather stiff (Glacial Till)			LL = 33 PL = 16 PI = 17 Max DD = 116.3 pcf Optimum Moisture = 12.6% Natural Moisture = 15%
		END OF BORING. Water not observed while drilling. Boring then backfilled.			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/29/11 11:18

BRAUN℠ INTERTEC

LOG OF BORING

Braun Project CR-11-00665 Geotechnical Evaluation Lake Dehli Delhi, Iowa	BORING: B-6 LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Power Auger	DATE: 10/10/11	SCALE: 1" = 4'

Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
1.0	TS	Topsoil			
25.0	CL	LEAN CLAY, brown, moist, medium (Loess)			LL = 38 PL = 23 PI = 15 Natural Moisture = 27%
		END OF BORING. Water not observed while drilling. Boring then backfilled.			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/30/11 11:06

BRAUN℠ INTERTEC

LOG OF BORING

Braun Project CR-11-00665 **Geotechnical Evaluation** **Lake Dehli** **Delhi, Iowa**	BORING: **B-7** LOCATION: See attached sketch

DRILLER: R.Hunt/D.Dyer	METHOD: Power Auger	DATE: 10/25/11	SCALE: 1" = 4'

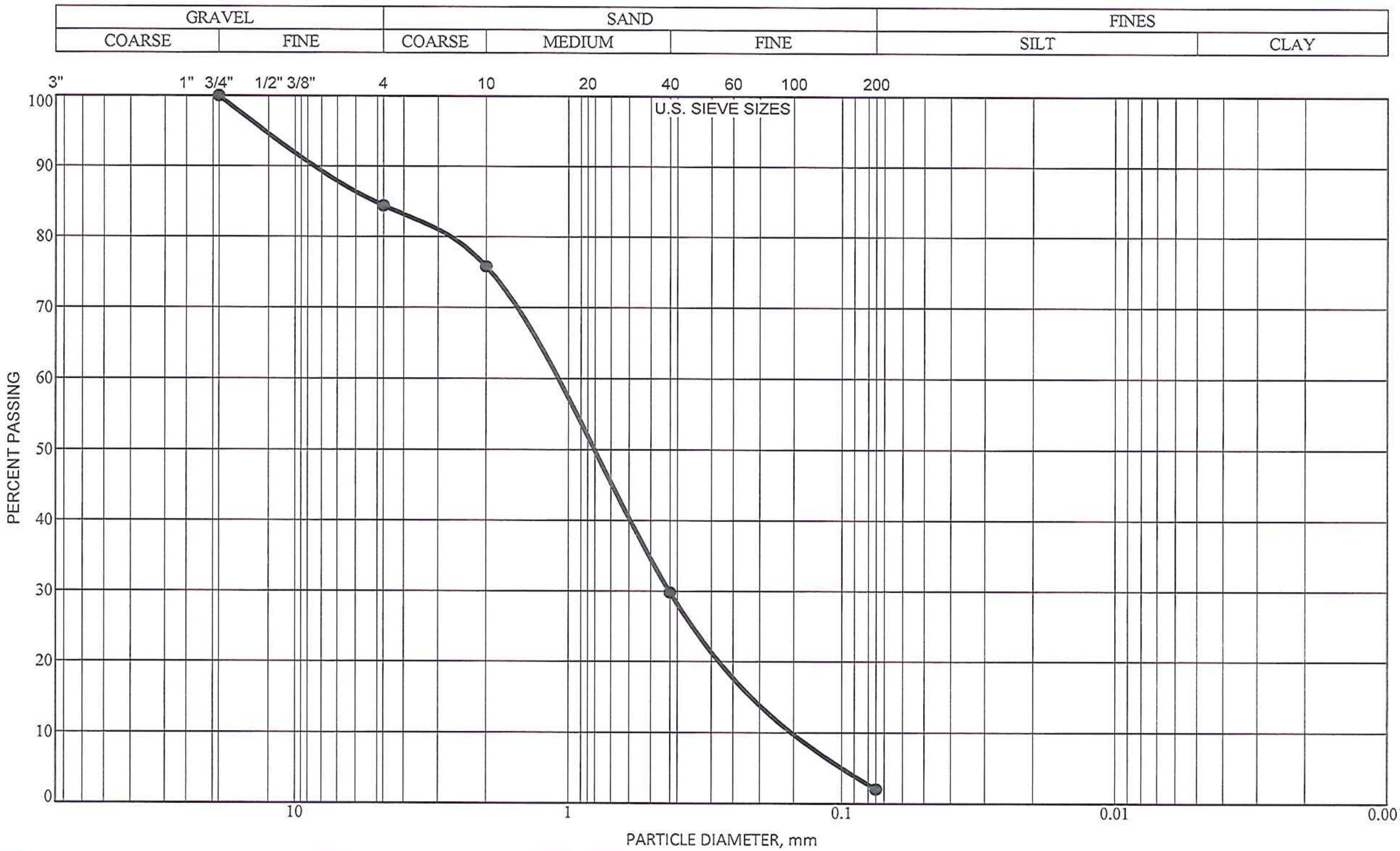
Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	Tests or Notes
1.0	TS	Topsoil			
25.0	CL	LEAN CLAY, brown, moist, medium (Loess)			LL = 39 PL = 23 PI = 16 Natural Moisture = 26%
		END OF BORING. Water not observed while drilling. Boring then backfilled.			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/30/11 11:02

Grain Size Curves

GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN[SM] INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: ST-6 DEPTH: 28.5'

GRAVEL	15.6%
SAND	82.4%
FINES	2.0%
D60=1.175	Cu=9.5
D30=0.428	Cc=1.3
D10=0.124	

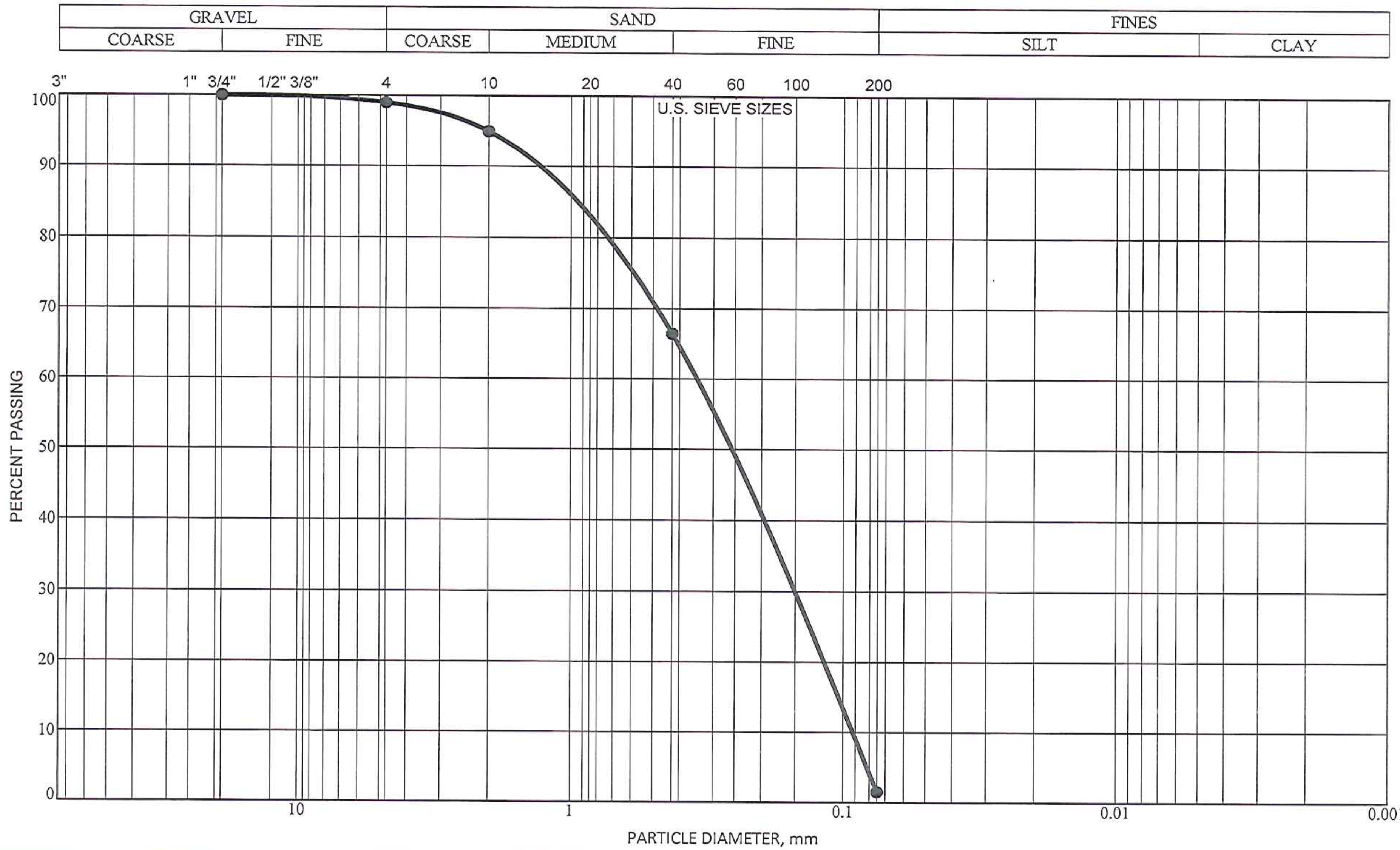
CLASSIFICATION:
WELL-GRADED SAND with GRAVEL(SW)

CR-11-00665

Braun Intertec Corporation

GS ASTM N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/22/11 14:41

GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN℠ INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: ST-7 DEPTH: 33.5'

GRAVEL	1.0%
SAND	97.5%
FINES	1.5%
D60=0.358	Cu=3.8
D30=0.161	Cc=0.8
D10=0.094	

CLASSIFICATION:
POORLY GRADED SAND(SP)

CR-11-00665

Braun Intertec Corporation

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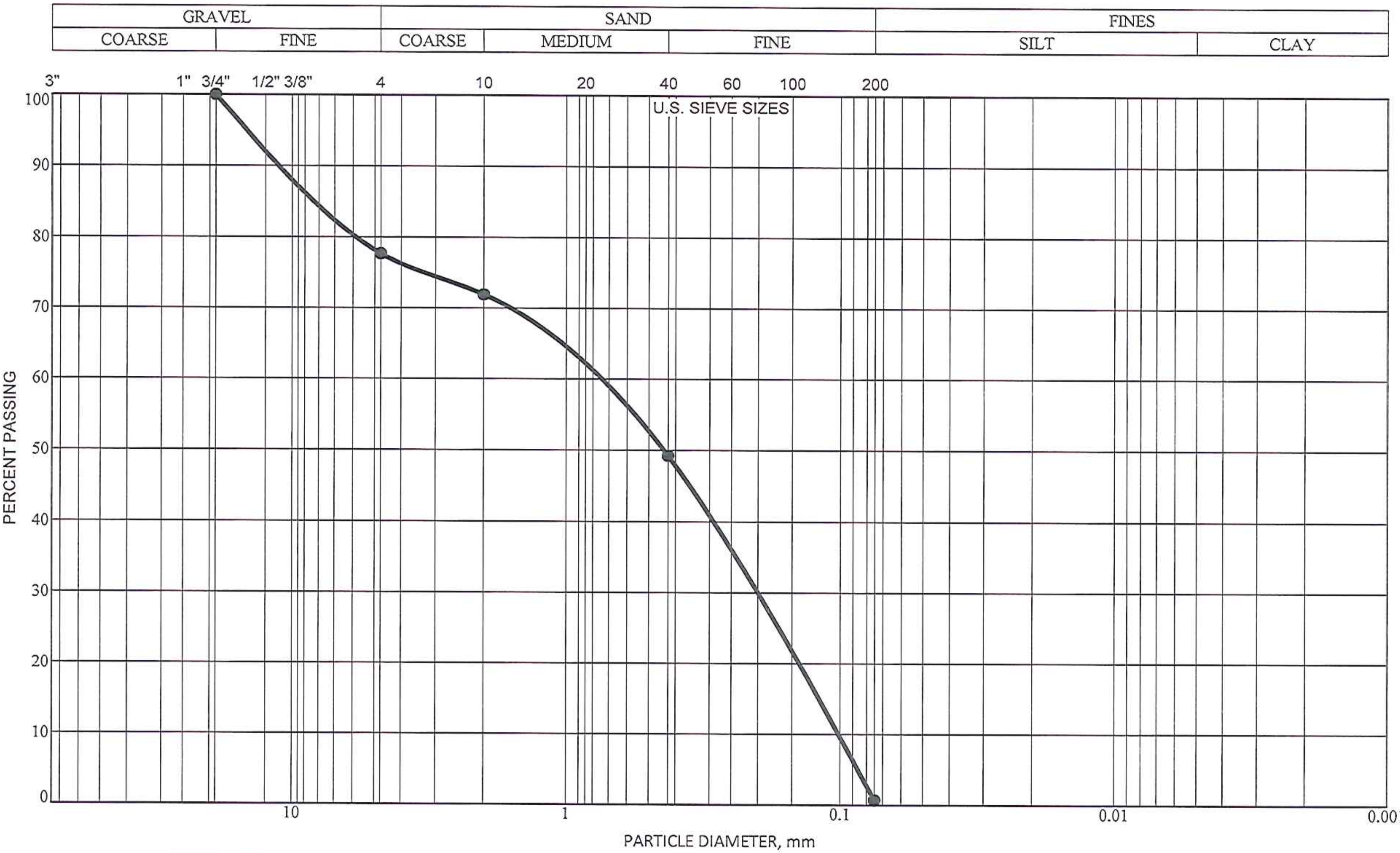
GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: ST-8 DEPTH: 18.5'

GRAVEL	22.3%
SAND	77.0%
FINES	0.7%
D60=0.888	Cu=8.5
D30=0.214	Cc=0.5
D10=0.105	

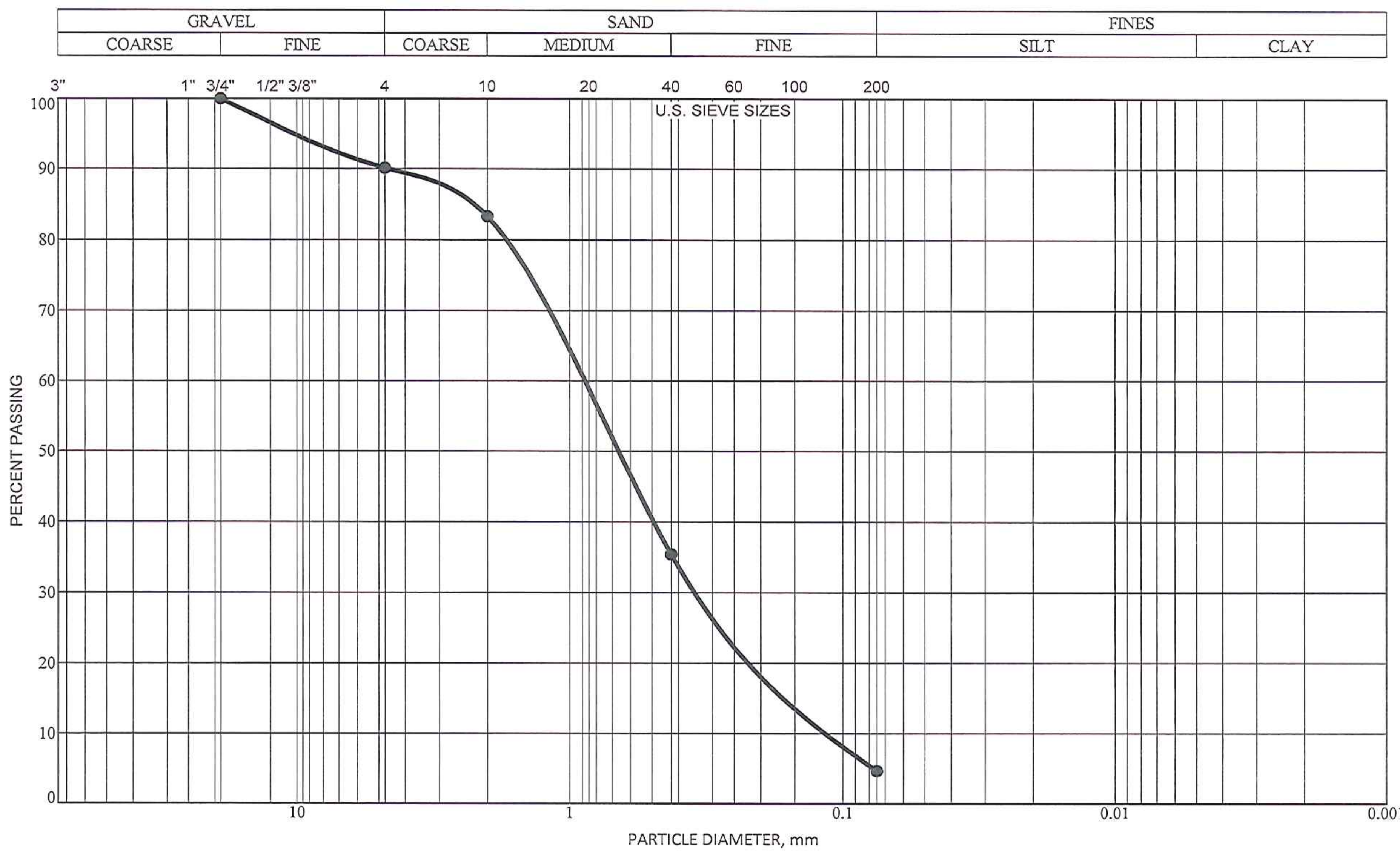
CLASSIFICATION:
POORLY GRADED SAND with GRAVEL(SP)

CR-11-00665

Braun Intertec Corporation

GS ASTM N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/22/11 14:41

GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: ST-8 DEPTH: 33.5'

GRAVEL	9.9%
SAND	85.4%
FINES	4.7%
D60=0.942	Cu=9.3
D30=0.313	Cc=1.0
D10=0.101	

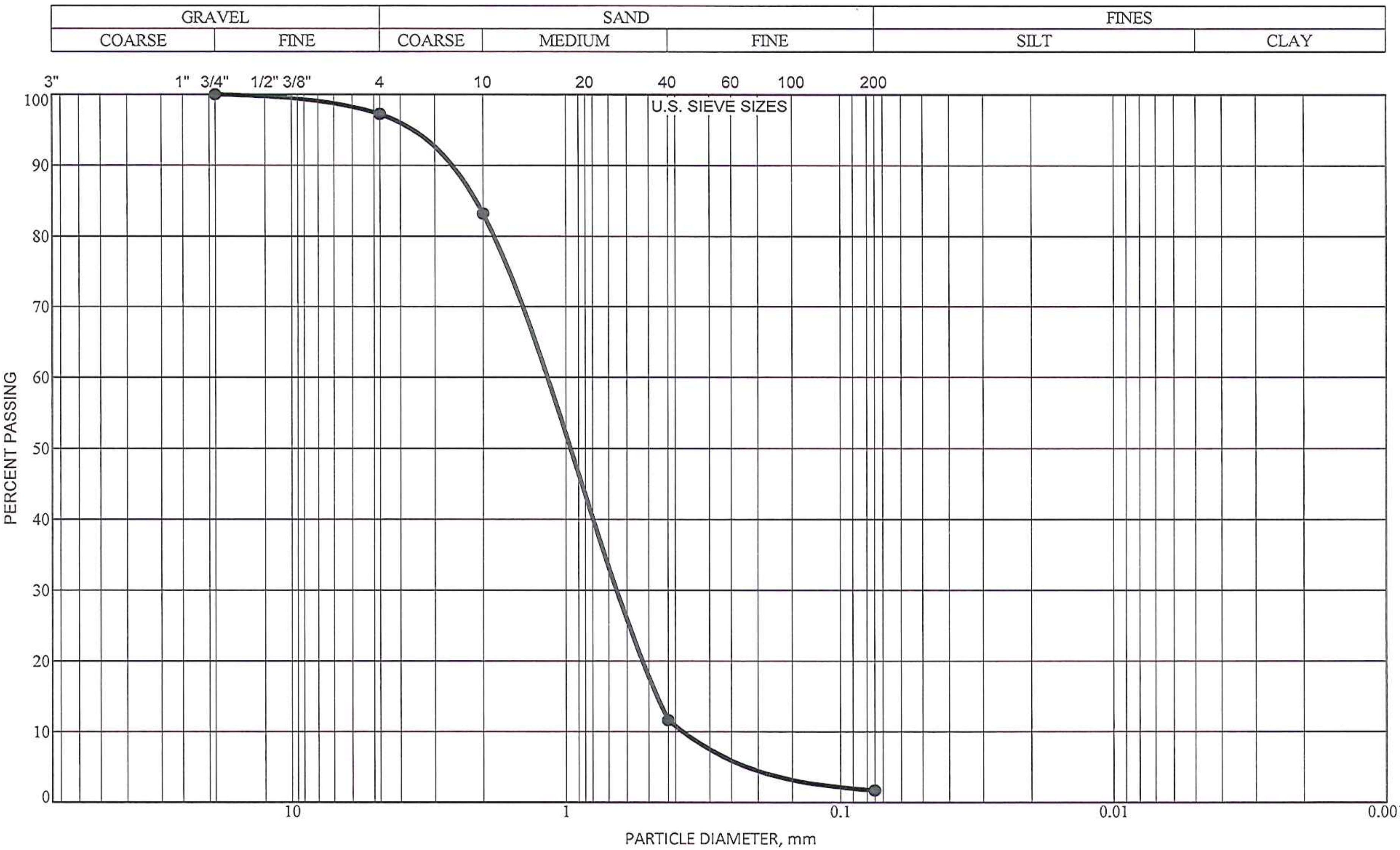
CLASSIFICATION:
WELL-GRADED SAND(SW)

CR-11-00665

Braun Intertec Corporation

GS ASTM N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/22/11 14:41

GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN[SM] INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: ST-8 DEPTH: 68.5'

GRAVEL	2.8%
SAND	95.5%
FINES	1.7%
D60=1.211 D30=0.633 D10=0.321	Cu=3.8 Cc=1.0

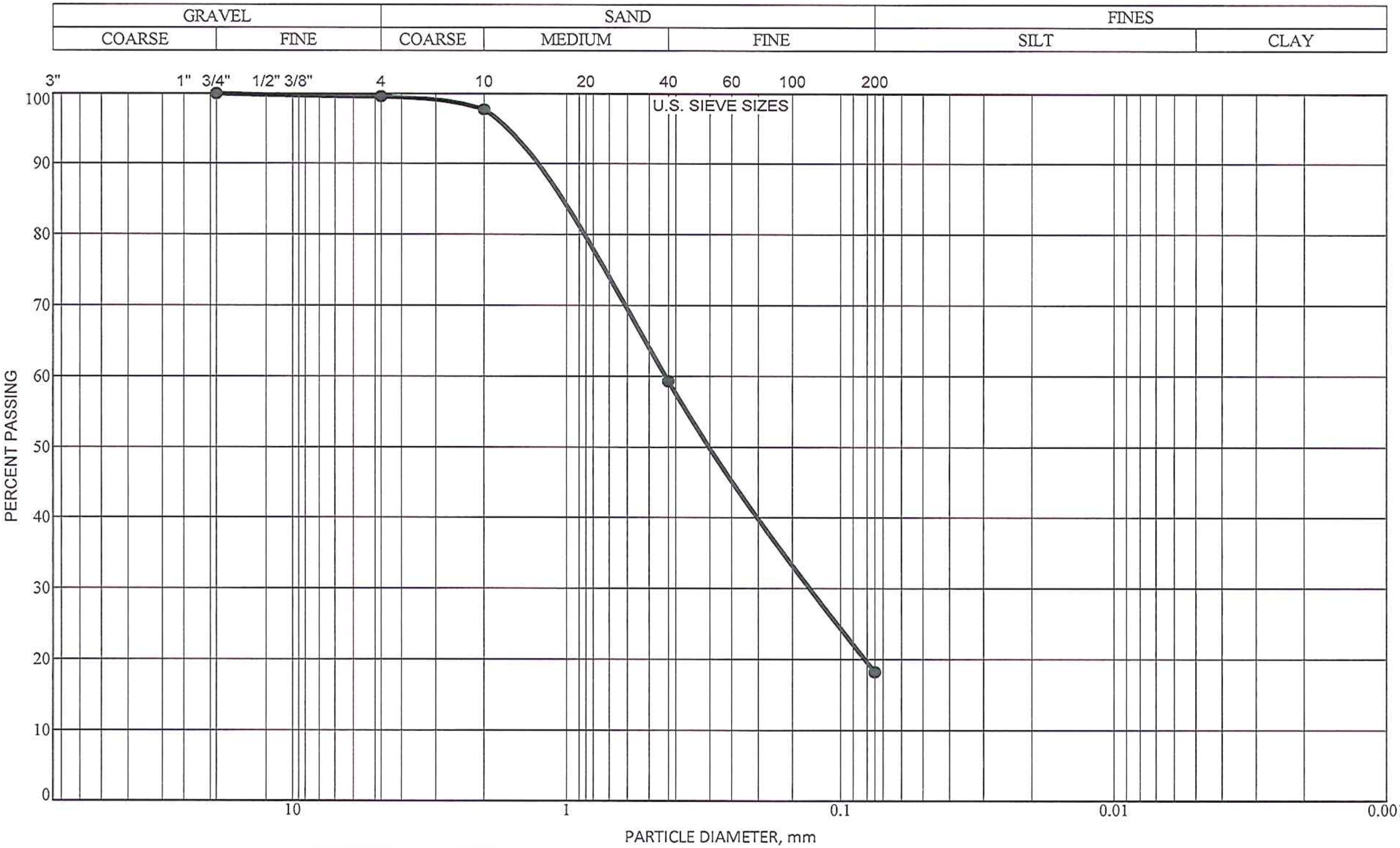
CLASSIFICATION:
POORLY GRADED SAND(SP)

CR-11-00665

Braun Intertec Corporation

GS ASTM N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/22/11 14:41

GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN[SM] INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: ST-9 DEPTH: 38.5'

GRAVEL	0.4%
SAND	81.4%
FINES	18.2%
D60=0.437	Cu=
D30=0.123	Cc=
D10=	

CLASSIFICATION:

CR-11-00665

Braun Intertec Corporation

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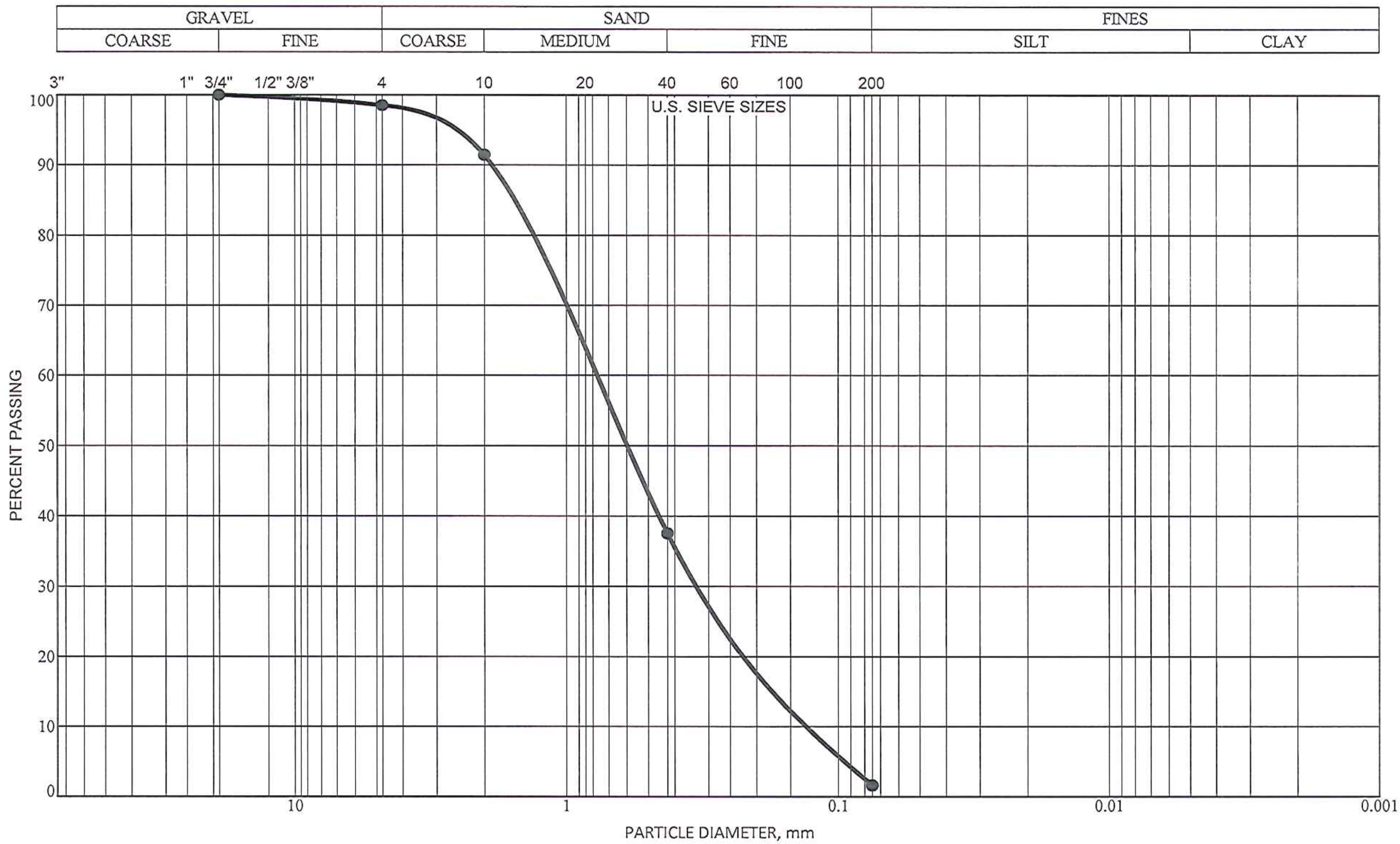
GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: ST-10 DEPTH: 23.5'

GRAVEL	1.5%
SAND	96.9%
FINES	1.6%
D60=0.811	Cu=7.2
D30=0.296	Cc=1.0
D10=0.113	

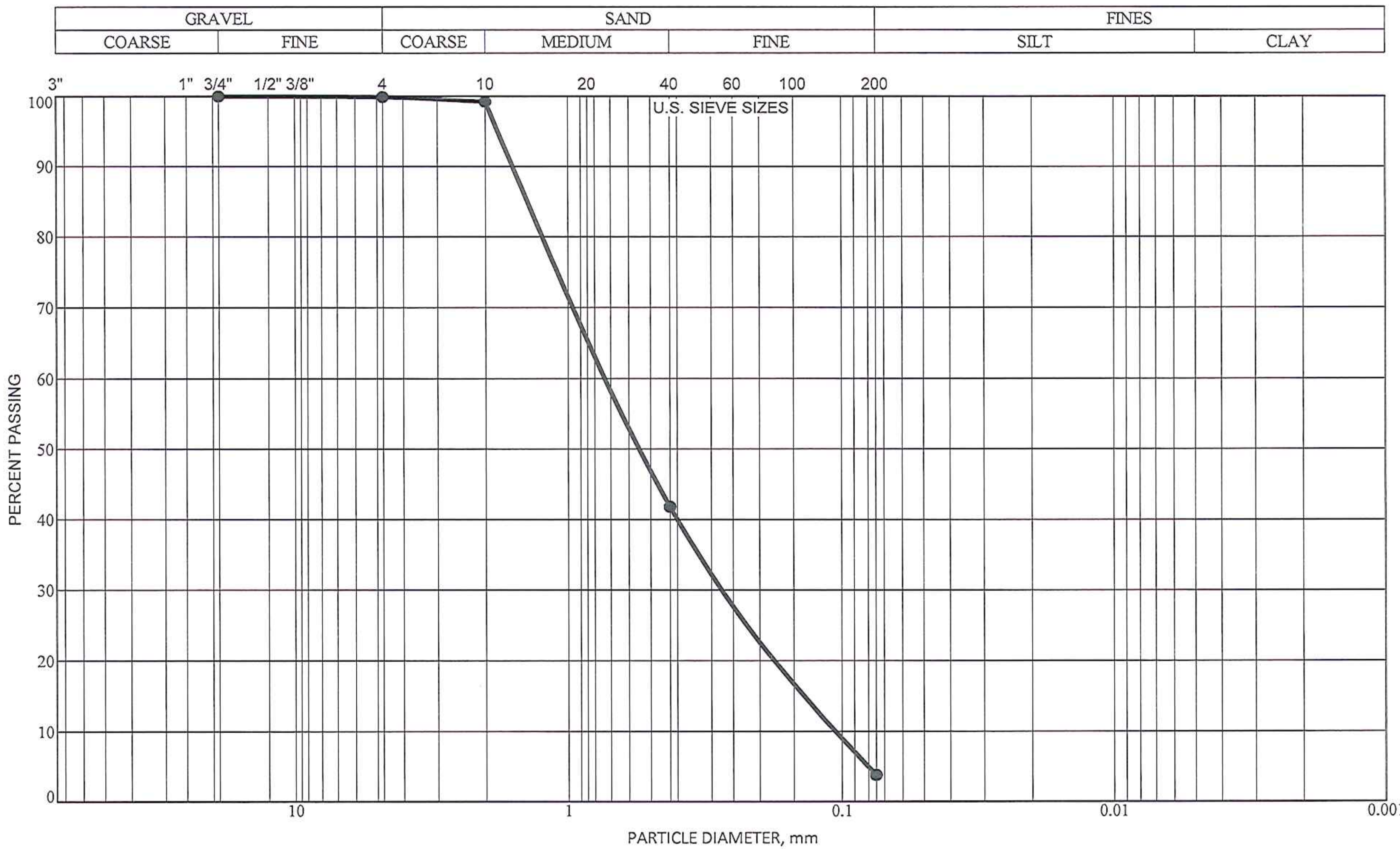
CLASSIFICATION:
POORLY GRADED SAND(SP)

CR-11-00665

Braun Intertec Corporation

GS ASTM N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/22/11 14:40

GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN[SM] INTERTEC

Braun Project CR-11-00665
Geotechnical Evaluation
Lake Dehli
Delhi, Iowa

BORING: ST-10 DEPTH: 48.5'

GRAVEL	0.1%
SAND	96.1%
FINES	3.8%
D60=0.694	Cu=7.0
D30=0.248	Cc=0.9
D10=0.100	

CLASSIFICATION:
POORLY GRADED SAND(SP)

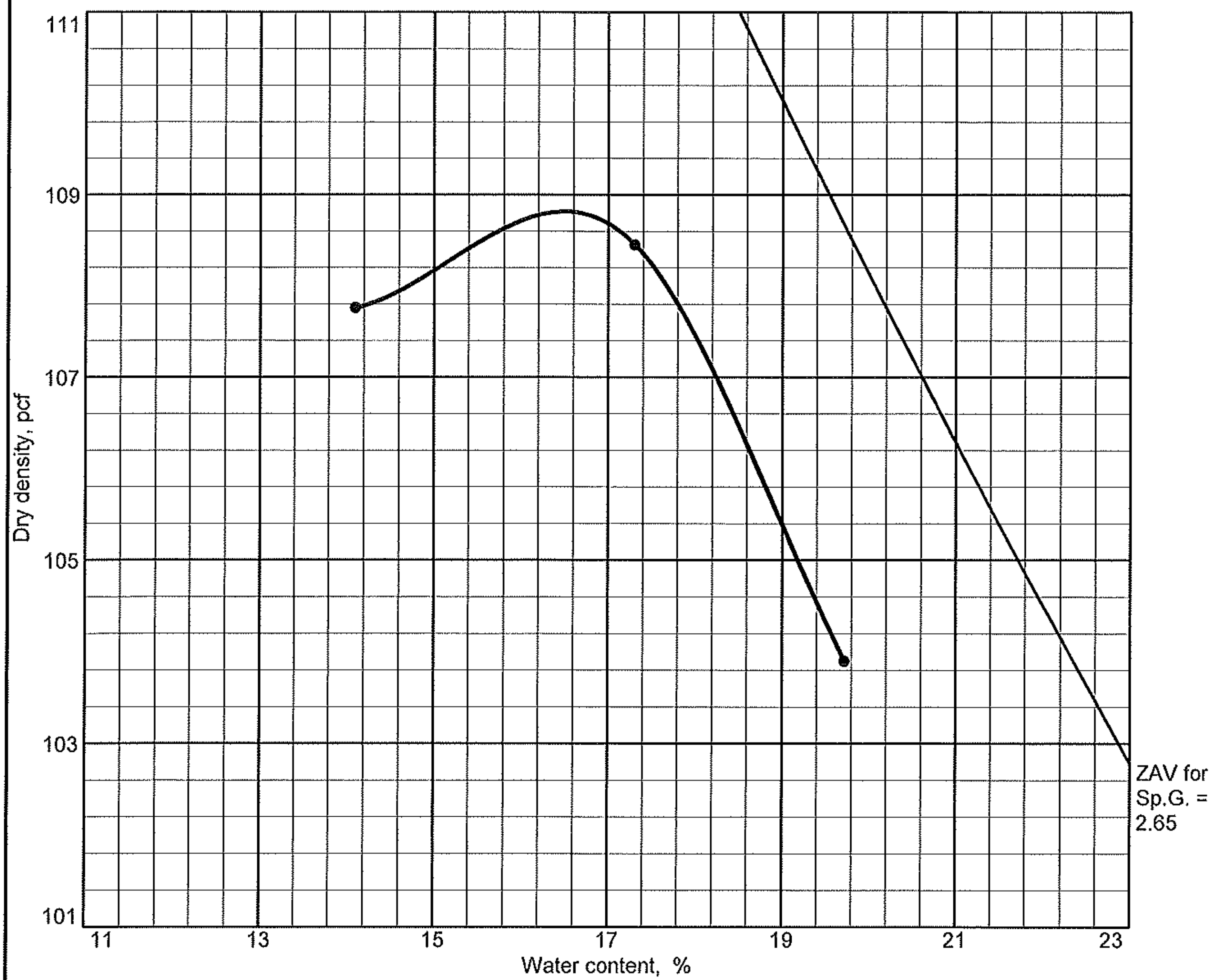
CR-11-00665

Braun Intertec Corporation

GS ASTM N:\GINT\PROJECTS\CEDARRAPIDS\2011\00665.GPJ BRAUN_V8_CURRENT.GDT 11/22/11 14:40

Moisture-Density Relationship Curves

Moisture-Density Relationship



Test specification: ASTM D 698-07e1 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
0-25'	CL		26		32	9		

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 108.8 pcf Optimum moisture = 16.5 %	Lean Clay

Project No.: CR-11-00665 **Client:**

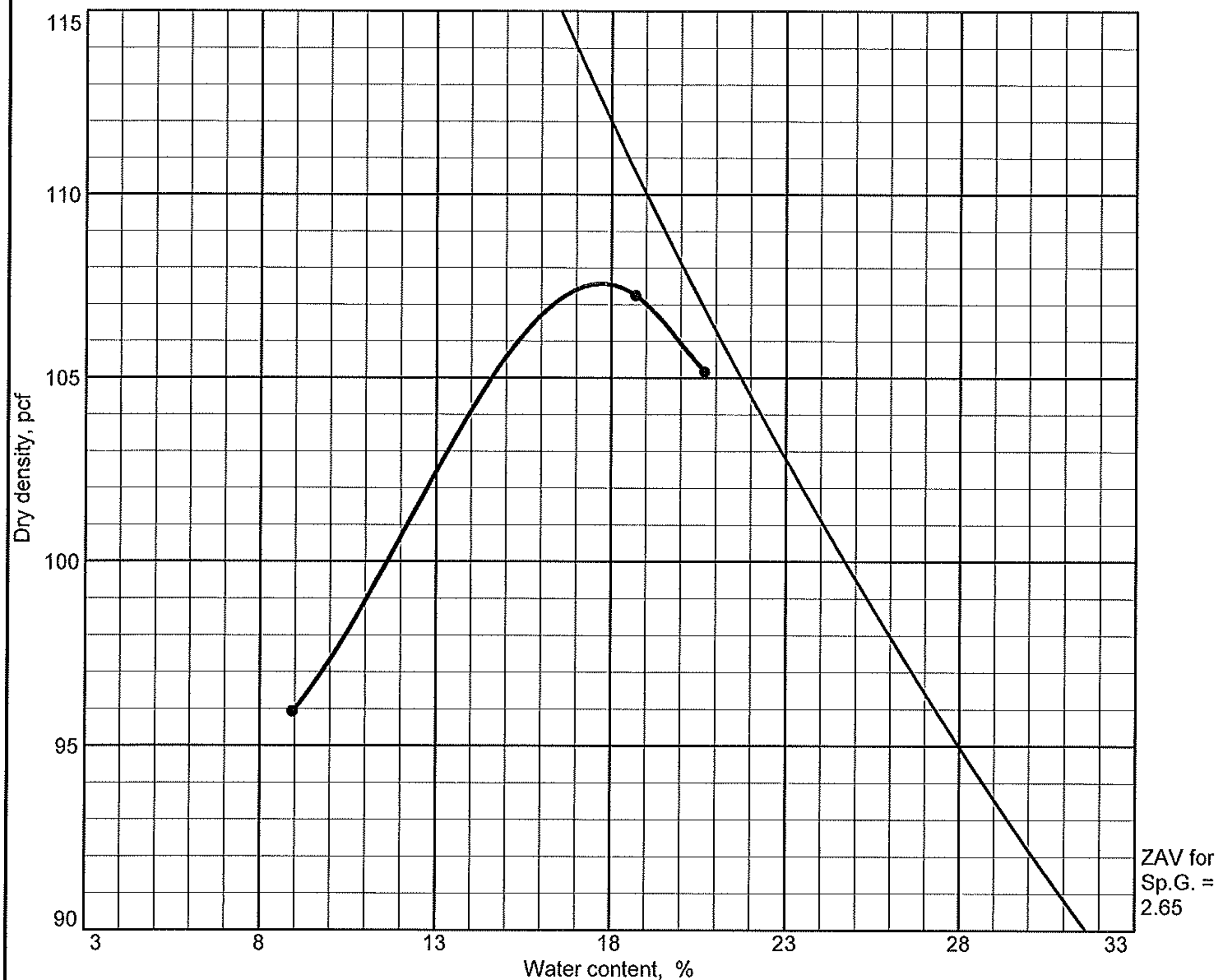
Project: Lake Delhi Dam Repairs

● **Location:** Boring B-1

Remarks:

BRAUN INTERTEC

Moisture-Density Relationship



Test specification: ASTM D 698-07e1 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
0-25'	CL		25		34	11		

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 107.6 pcf Optimum moisture = 17.7 %	Lean Clay

Project No.: CR-11-00665 **Client:**

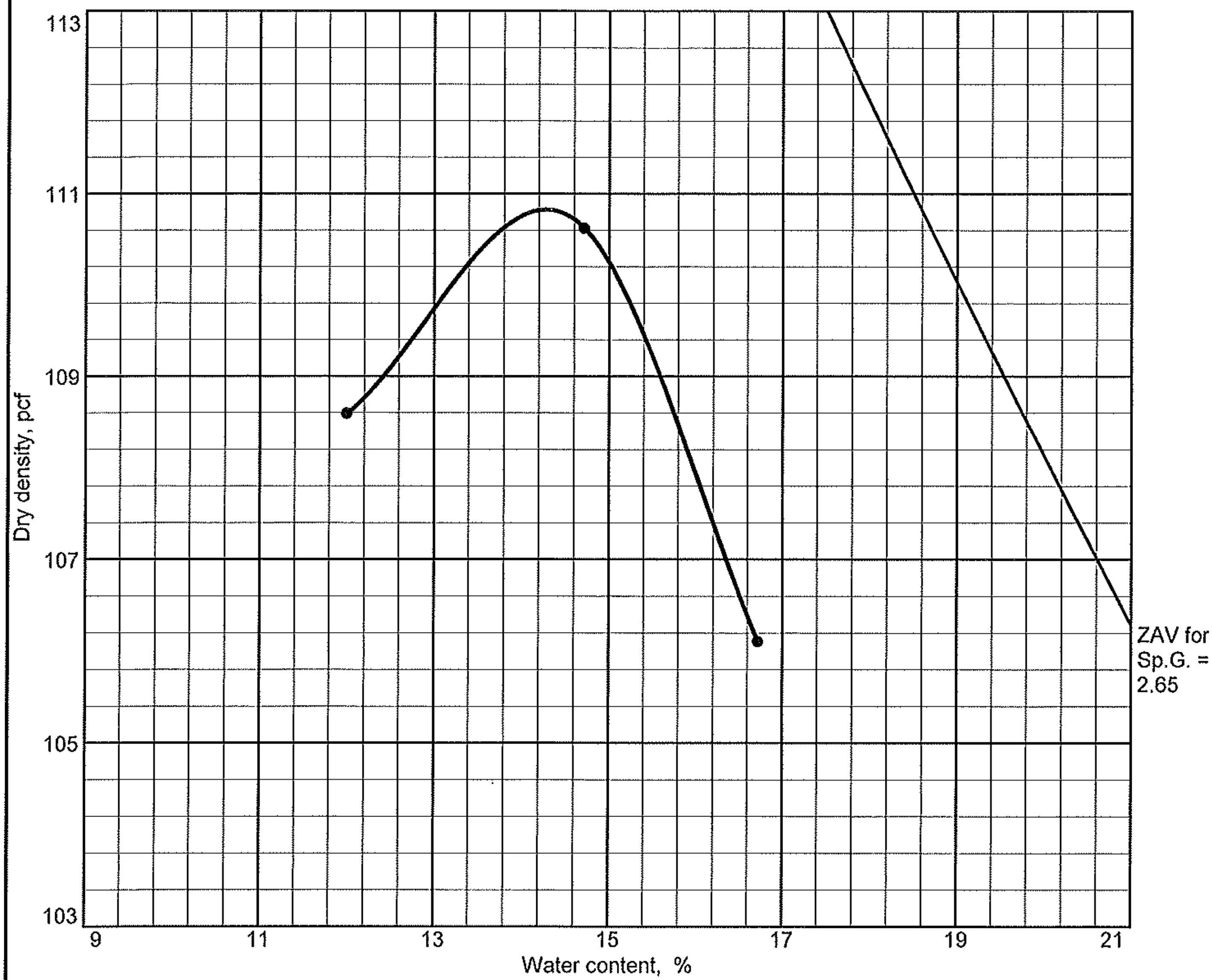
Project: Lake Delhi Dam Repairs

● **Location:** Boring B-2

Remarks:

BRAUN INTERTEC

Moisture-Density Relationship



Test specification: ASTM D 698-07e1 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
0-25'	CL		30		32	9		

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 110.8 pcf Optimum moisture = 14.3 %	Lean Clay

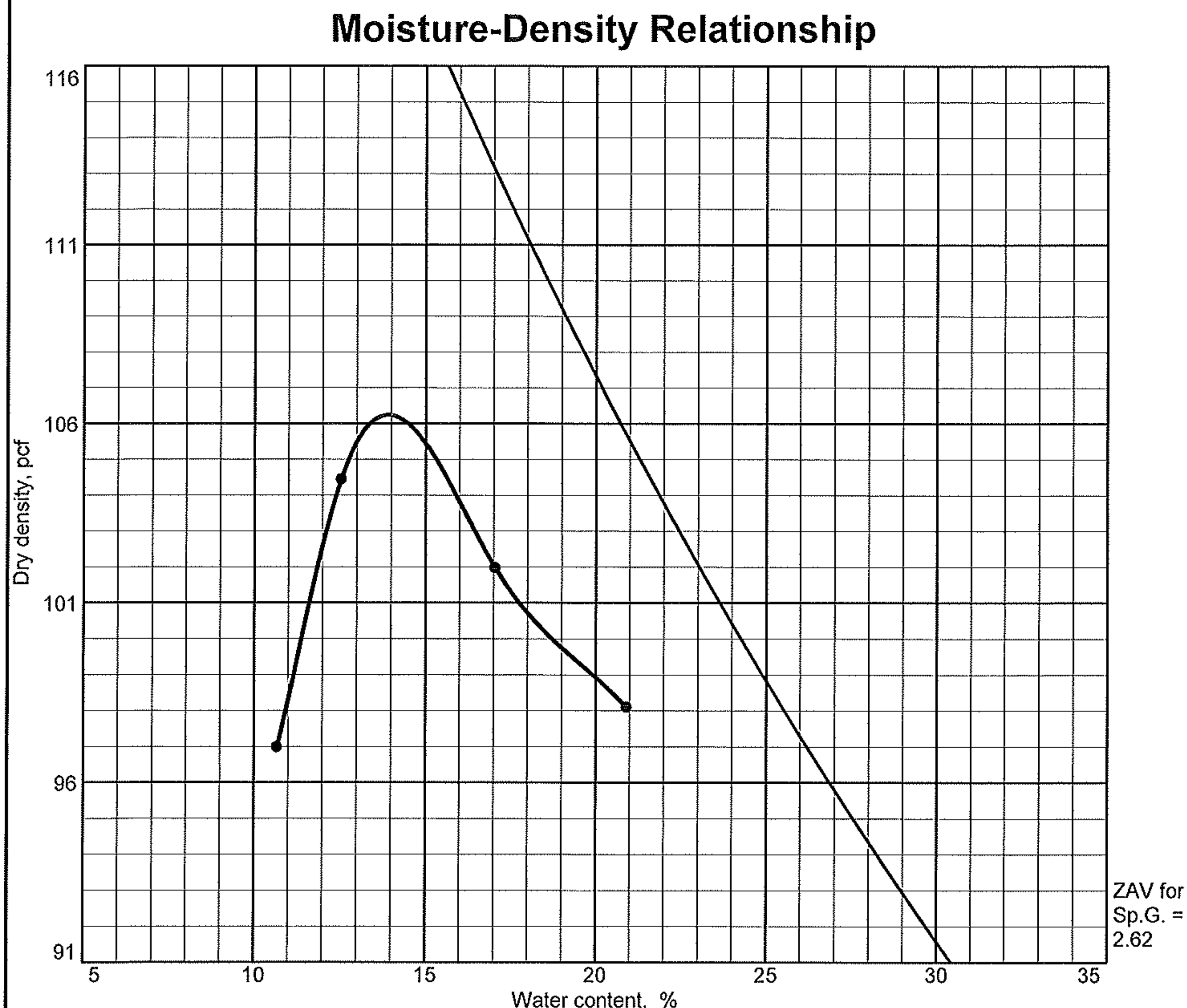
Project No.: CR-11-00665 **Client:**

Project: Lake Delhi Dam Repairs

● **Location:** Boring B-3

BRAUN INTERTEC

Remarks:



Test specification: ASTM D 698-07e1 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
0' - 18'	CL		35		34	11		

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 106.2 pcf Optimum moisture = 13.9 %	Lean Clay

Project No.: CR-11-00665 **Client:**

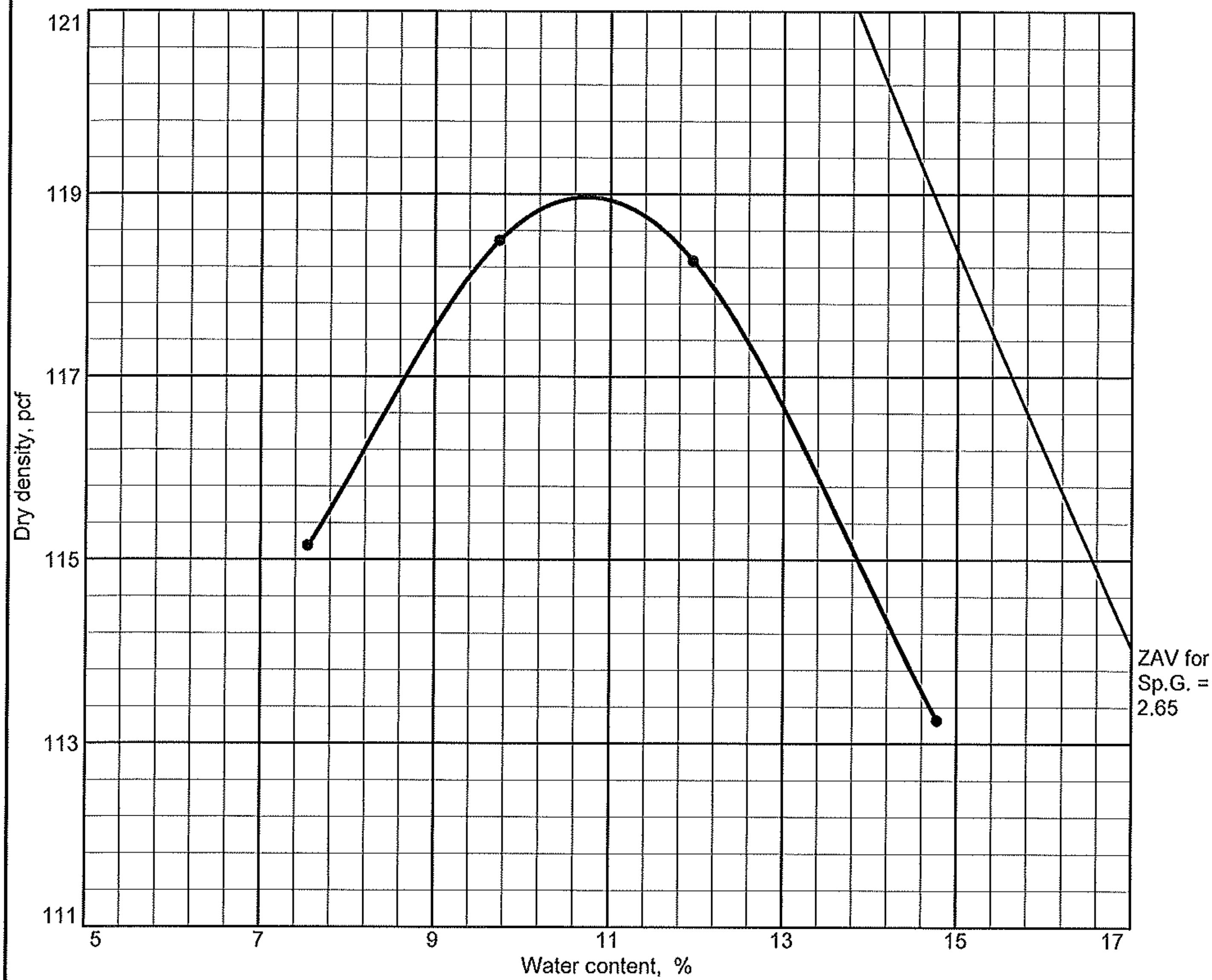
Project: Lake Delhi Dam Repairs

● **Location:** Boring B-4

Remarks:

BRAUN INTERTEC

Moisture-Density Relationship



Test specification: ASTM D 698-07e1 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
19 - 25'	CL		16		30	16		

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 119.0 pcf Optimum moisture = 10.7 %	Sandy Lean Clay

Project No.: CR-11-00665 **Client:**

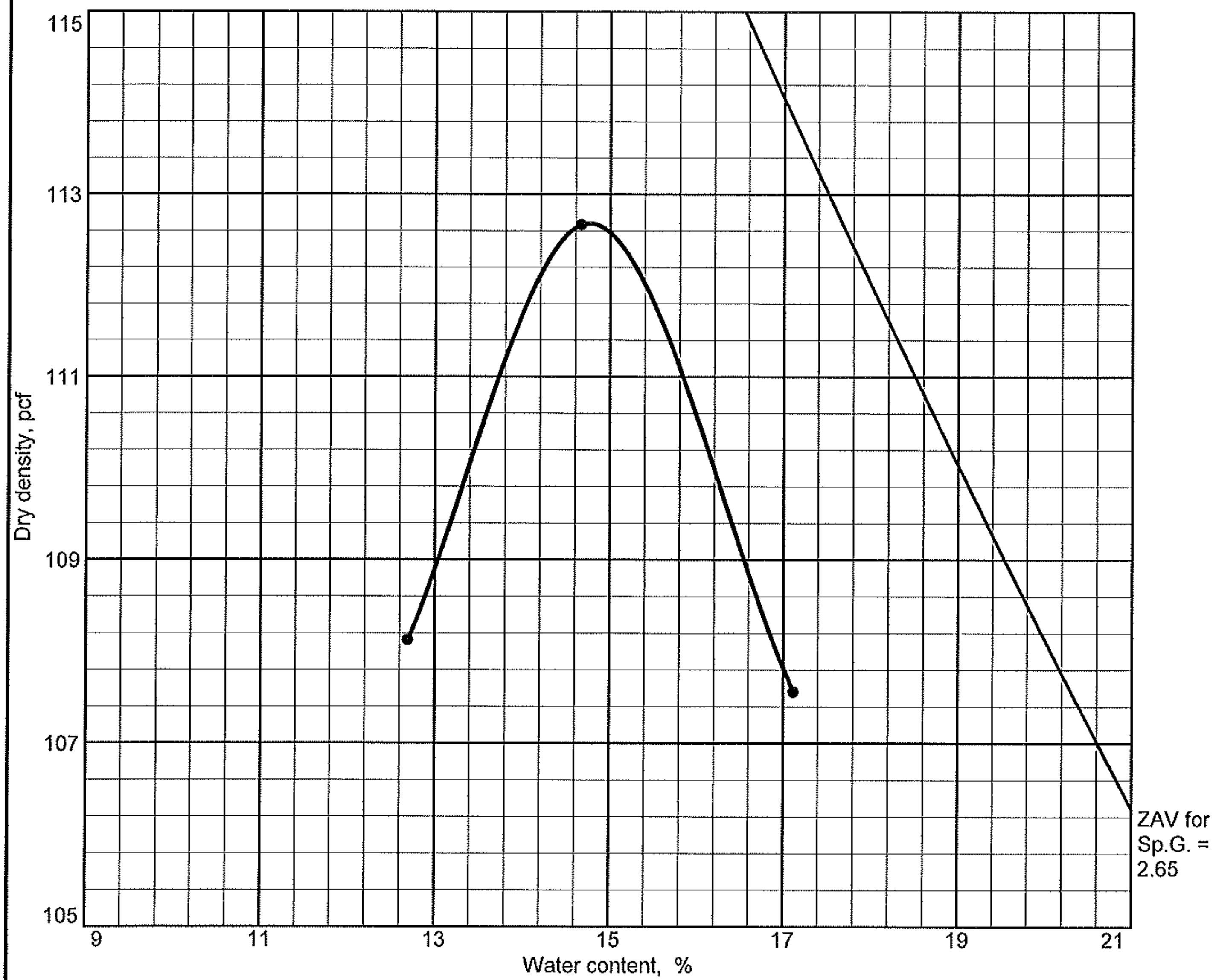
Project: Lake Delhi Dam Repairs

● **Location:** Boring B-4

Remarks:

BRAUN INTERTEC

Moisture-Density Relationship



Test specification: ASTM D 698-07e1 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
0' - 12'	CL		24		29	9		

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 112.7 pcf Optimum moisture = 14.8 %	Lean Clay

Project No.: CR-11-00665 **Client:**

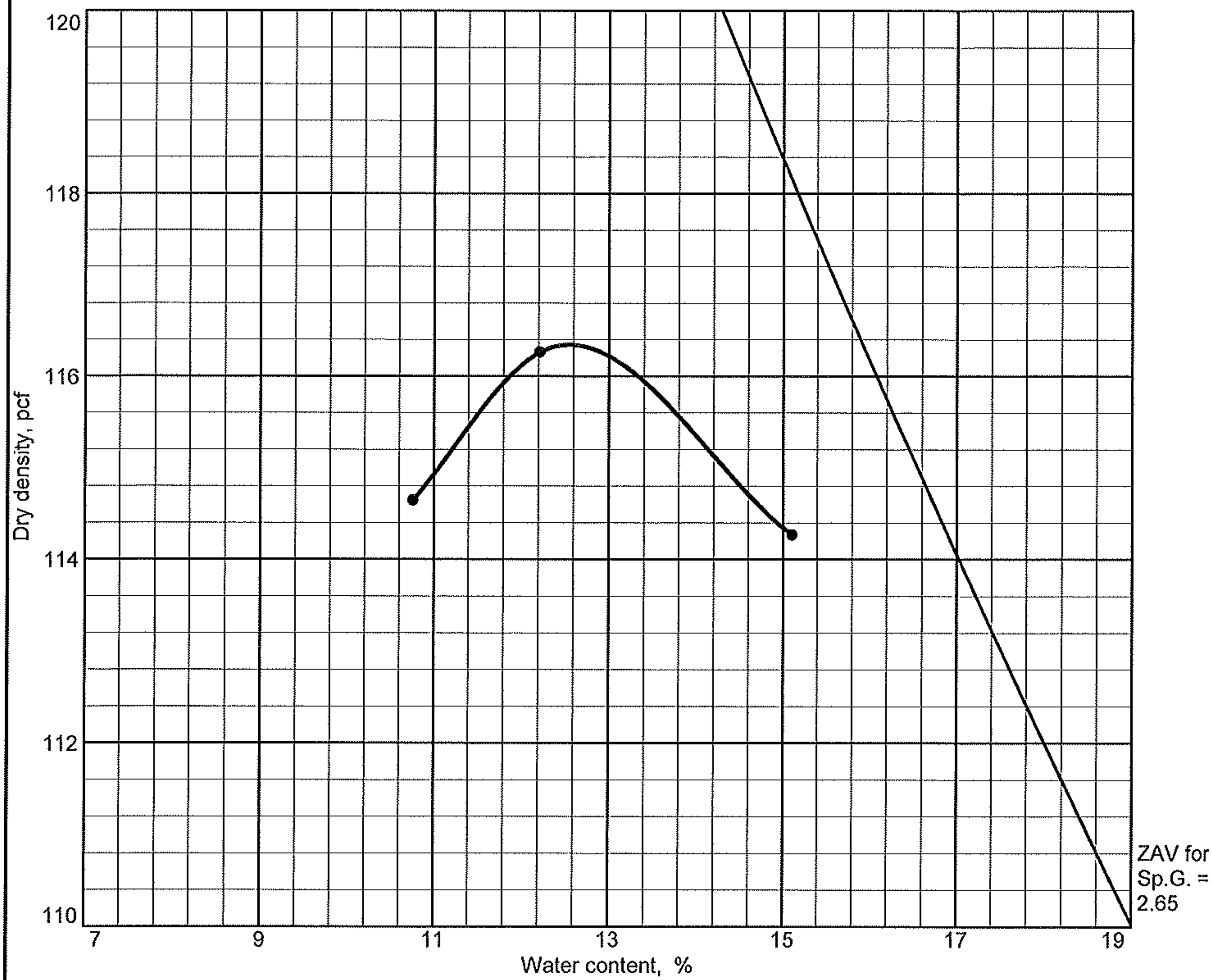
Project: Lake Delhi Dam Repairs

● **Location:** Boring B-5

Remarks:

BRAUN INTERTEC

Moisture-Density Relationship



Test specification: ASTM D 698-07e1 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
13 - 25'	CL		15		33	17		

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 116.3 pcf Optimum moisture = 12.6 %	Sandy Lean Clay

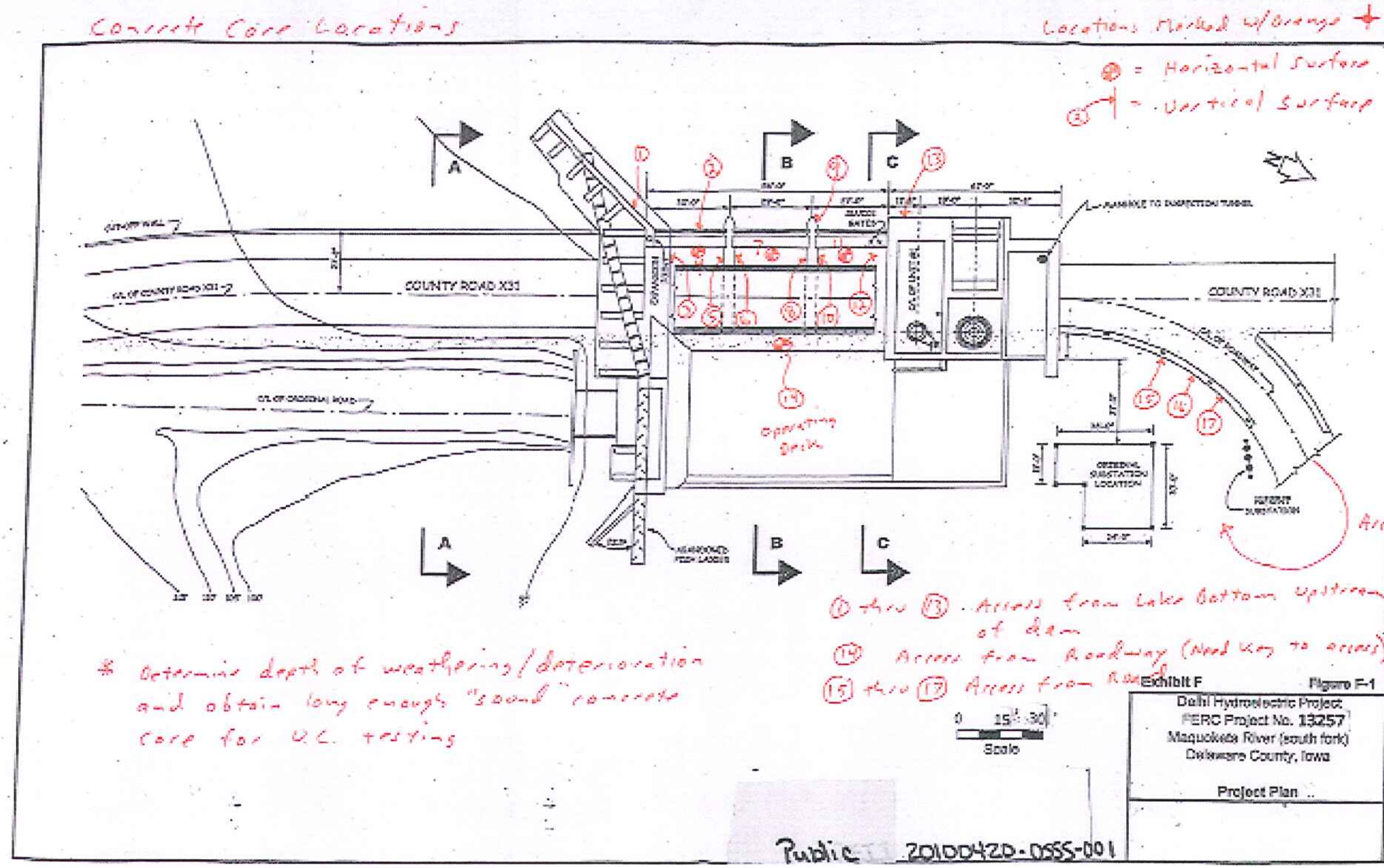
Project No.: CR-11-00665 **Client:**
Project: Lake Delhi Dam Repairs

● **Location:** Boring B-5

BRAUN INTERTEC

Remarks:

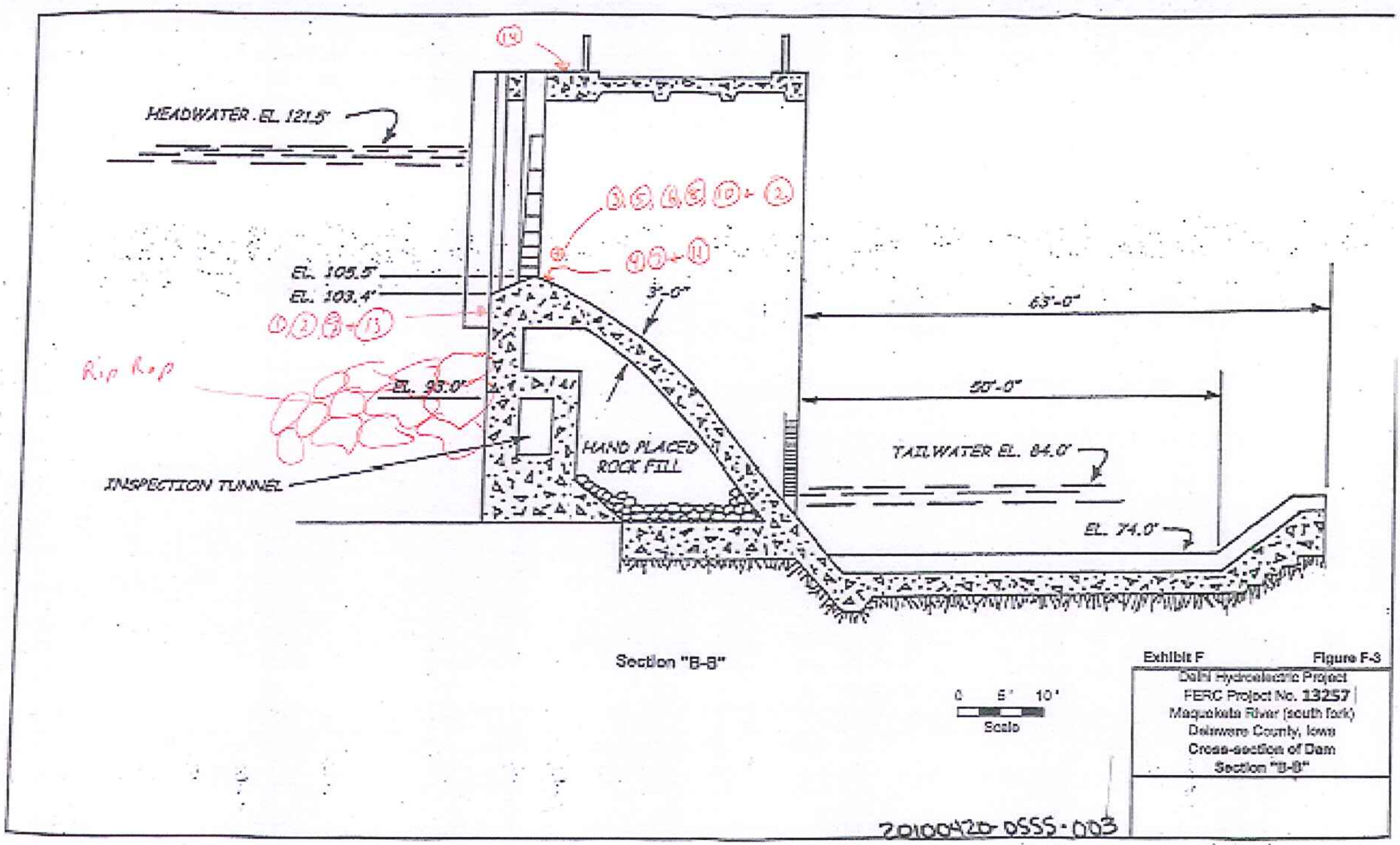
Concrete Core Location Plans



Source: Stanley Consultants

Approximate scale as shown

Lake Delhi Dam Restoration Delhi, Iowa	Project: CR-11-00665	Concrete Coring Location Sketch Page 1	BRAUN INTERTEC



Source: Stanley Consultants

Approximate scale as shown

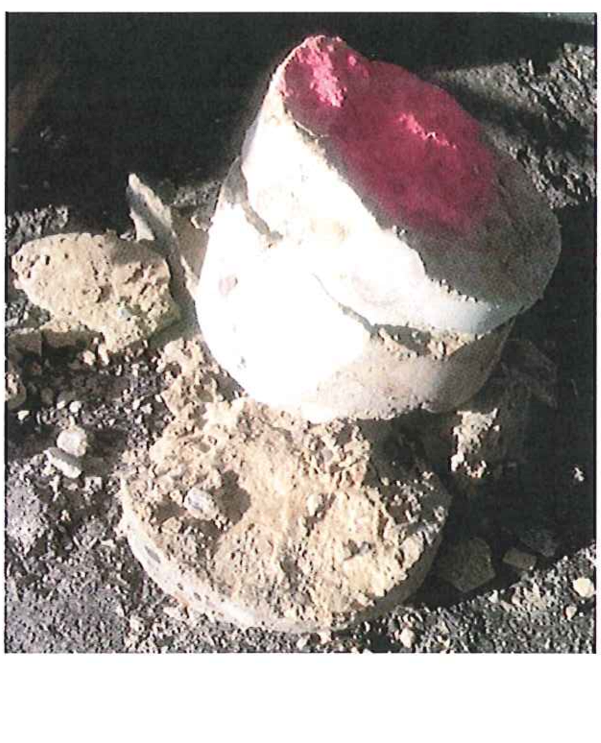
Lake Delhi Dam Restoration Delhi, Iowa	Project: CR-11-00665	Concrete Coring Location Sketch Page 2	BRAUN INTERTEC

Table of Concrete Core Compressive Strengths and Pictures of Fractured Cores

The following table shows the results of compressive strength testing where possible.

Core Number	Compressive Strength (psi)
1	6590
2	1480
3	2680
4	4180
7	4310
11	5120
15	3860
16	6470
17	1940

In general, the concrete did not respond well to coring and many of the cores broke or turned to rubble during the coring operation. The cores taken from vertical surfaces were typically in poorer condition than those taken from horizontal surfaces. Three examples of cores that were too short to test are in the following pictures. More than one attempt was made at several locations to obtain intact cores. The results of the petrographic analysis will be forwarded under separate cover.



Core Number 5

Core Number 6

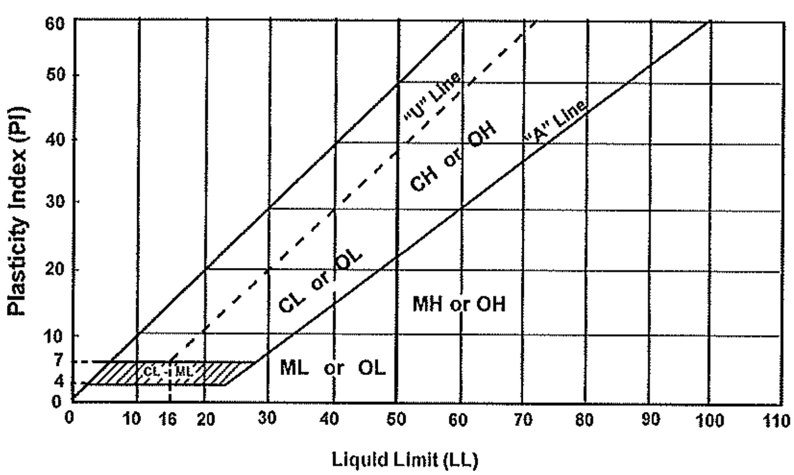
Core Number 8

Descriptive Terminology of Soil

ASTM INTERNATIONAL
Standard D 2487 - 00
Classification of Soils for Engineering Purposes
(Unified Soil Classification System)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [a]				Soils Classification: Group Symbol	Soils Classification: Group Name [b]
Coarse-grained Soils more than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels 5% or less fines [e]	$C_u \geq 4$ and $1 \leq C_c \leq 3$ [c]	GW	Well-graded gravel [d]
			$C_u < 4$ and/or $1 > C_c > 3$ [c]	GP	Poorly graded gravel [d]
		Gravels with Fines More than 12% fines [e]	Fines classify as ML or MH	GM	Silty gravel [d f g]
			Fines classify as CL or CH	GC	Clayey gravel [d f g]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands 5% or less fines [i]	$C_u \geq 6$ and $1 \leq C_c \leq 3$ [c]	SW	Well-graded sand [h]
			$C_u < 6$ and/or $1 > C_c > 3$ [c]	SP	Poorly graded sand [h]
		Sands with Fines More than 12% [i]	Fines classify as ML or MH	SM	Silty sand [f g h]
			Fines classify as CL or CH	SC	Clayey sand [f g h]
Fine-grained Soils 50% or more passed the No. 200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line [j]	CL	Lean clay [k l m]
			PI < 4 or plots below "A" line [j]	ML	Silt [k l m]
		Organic	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay [k l m n]
				OL	Organic silt [k l m o]
	Silts and clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay [k l m]
			PI plots below "A" line	MH	Elastic silt [k l m]
		Organic	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay [k l m p]
				OH	Organic silt [k l m q]
Highly Organic Soils	Primarily organic matter, dark in color and organic odor			PT	Peat

a. Based on the material passing the 3-in (75mm) sieve.
b. If field sample contained cobbles or boulders, or both, add "with cobbles or boulders or both" to group name.
c. $C_u = D_{60} / D_{10}$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
d. If soil contains ≥15% sand, add "with sand" to group name.
e. Gravels with 5 to 12% fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay
f. If fines classify as CL-ML, use dual symbol GC-GM or SC-SM.
g. If fines are organic, add "with organic fines" to group name.
h. If soil contains ≥ 15% gravel, add "with gravel" to group name.
i. Sands with 5 to 12% fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay
j. If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
k. If soil contains 10 to 29% plus No. 200, add "with sand" or "with gravel" whichever is predominant.
l. If soil contains ≥ 30% plus No. 200, predominantly sand, add "sandy" to group name.
m. If soil contains ≥ 30% plus No. 200 predominantly gravel, add "gravelly" to group name.
n. PI ≥ 4 and plots on or above "A" line.
o. PI < 4 or plots below "A" line.
p. PI plots on or above "A" line.
q. PI plots below "A" line.



Laboratory Tests

DD	Dry density, pcf	OC	Organic content, %
WD	Wet density, pcf	S	Percent of saturation, %
MC	Natural moisture content, %	SG	Specific gravity
LL	Liquid limit, %	C	Cohesion, psf
PL	Plastic limit, %	Ø	Angle of internal friction
PI	Plasticity index, %	qu	Unconfined compressive strength, psf
P200	% passing 200 sieve	qp	Pocket penetrometer strength, tsf

Particle Size Identification

Boulders .... over 12"
Cobbles .... 3" to 12"
Gravel
Coarse .... 3/4" to 3"
Fine .... No. 4 to 3/4"
Sand
Coarse .... No. 4 to No. 10
Medium .... No. 10 to No. 40
Fine .... No. 40 to No. 200
Silt .... < No. 200, PI < 4 or below "A" line
Clay .... < No. 200, PI ≥ 4 and on or above "A" line

Relative Density of Cohesionless Soils

Very loose .... 0 to 4 BPF
Loose .... 5 to 10 BPF
Medium dense .... 11 to 30 BPF
Dense .... 31 to 50 BPF
Very dense .... over 50 BPF

Consistency of Cohesive Soils

Very soft .... 0 to 1 BPF
Soft .... 2 to 3 BPF
Rather soft .... 4 to 5 BPF
Medium .... 6 to 8 BPF
Rather stiff .... 9 to 12 BPF
Stiff .... 13 to 16 BPF
Very stiff .... 17 to 30 BPF
Hard .... over 30 BPF

Drilling Notes

Standard penetration test borings were advanced by 3 1/4" or 6 1/4" ID hollow-stem augers unless noted otherwise, Jetting water was used to clean out auger prior to sampling only where indicated on logs. Standard penetration test borings are designated by the prefix "ST" (Split Tube). All samples were taken with the standard 2" OD split-tube sampler, except where noted.

Power auger borings were advanced by 4" or 6" diameter continuous-flight, solid-stem augers. Soil classifications and strata depths were inferred from disturbed samples augered to the surface and are, therefore, somewhat approximate. Power auger borings are designated by the prefix "B."

Hand auger borings were advanced manually with a 1 1/2" or 3 1/4" diameter auger and were limited to the depth from which the auger could be manually withdrawn. Hand auger borings are indicated by the prefix "H."

BPF: Numbers indicate blows per foot recorded in standard penetration test, also known as "N" value. The sampler was set 6" into undisturbed soil below the hollow-stem auger. Driving resistances were then counted for second and third 6" increments and added to get BPF. Where they differed significantly, they are reported in the following form: 2/12 for the second and third 6" increments, respectively.

WH: WH indicates the sampler penetrated soil under weight of hammer and rods alone; driving not required.

WR: WR indicates the sampler penetrated soil under weight of rods alone; hammer weight and driving not required.

TW indicates thin-walled (undisturbed) tube sample.

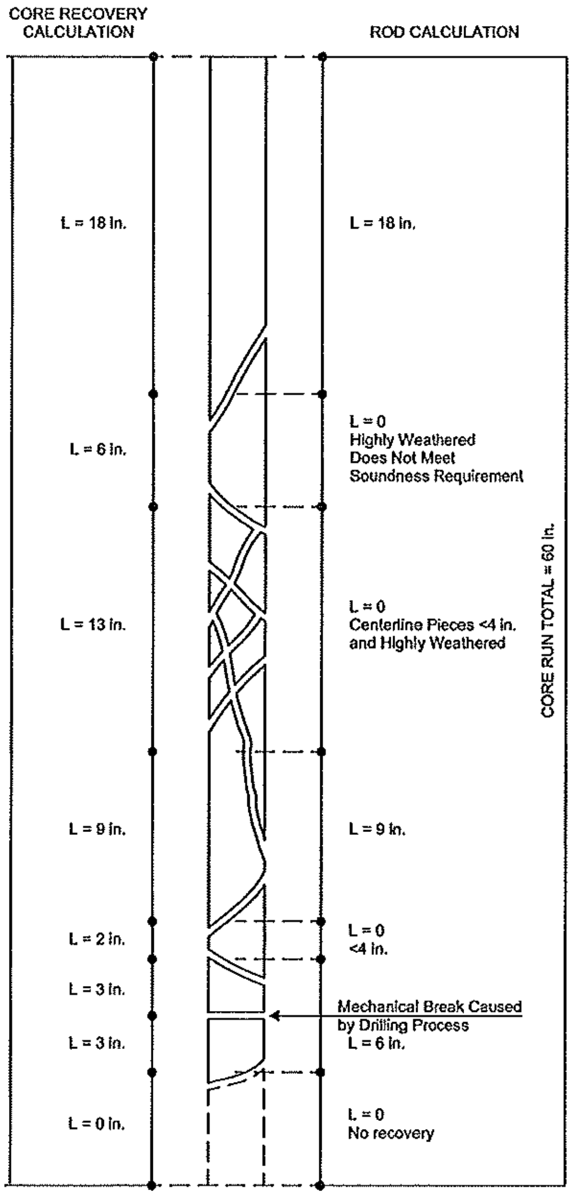
Note: All tests were run in general accordance with applicable ASTM standards.

Rev. 7/07

BRAUN INTERTEC

Descriptive Terminology of Rock

Based on U.S. Army Corps of Engineers EM 1110-1-2908



Example Calculations

Core recovery, CR = $\frac{\text{Total length of rock recovered}}{\text{Total core run length}}$

Example: $CR = \frac{(18 + 6 + 13 + 9 + 2 + 3 + 3)}{(60)}$

CR = 90%

RQD = $\frac{\text{Sum of sound piece longer than 4 inches}}{\text{Total core run length}}$

RQD Percent	Rock Quality
< 25	very poor
25 < 50	poor
50 < 75	fair
75 < 90	good
90 < 100	excellent

Example: $RQD = \frac{(18 + 9 + 6)}{(60)}$

RQD = 55%

Weathering

Unweathered: No evidence of chemical or mechanical alteration.

Slightly weathered: Slight discoloration on surface, slight alteration along discontinuities, less than 10% of rock volume altered.

Moderately Weathered: Discoloration evident, surface pitted and altered with alteration penetrating well below rock surfaces, weathering halos evident, 10% to 50% of the rock altered.

Highly Weathered: Entire mass discolored, alteration pervading nearly all of the rock, with some pockets of slightly weathered rock noticeable, some mineral leached away.

Hardness

Very soft:	Can be deformed by hand
Soft:	Can be scratched with a fingernail
Moderately hard:	Can be scratched easily with a knife
Hard:	Can be scratched with difficulty with a knife
Very hard:	Cannot be scratched with a knife

Thickness of Bedding

Massive:	3 ft. thick or greater
Thick bedded:	1 to 3 ft. thick
Medium bedded:	4 in. to 1 ft. thick
Thin bedded:	4 in. thick or less

Degree of Fracturing (jointing)

Unfractured:	Fracture spacing 6 ft. of more
Slightly fractured:	Fracture spacing 2 to 6 ft.
Moderately fractured:	Fracture spacing 8 in. to 2 ft.
Highly fractured:	Fracture spacing 2 to 8 in.
Intensely fractured:	Fracture spacing 2 in. or less

2/08

Concrete Core Photographs

Core #1

Core #1 Break

Core #2

Core #2 Break

Core #3

Core #3 Break – Before

Core #3 Break – After

Core #4

Core #4 Break


Core #5

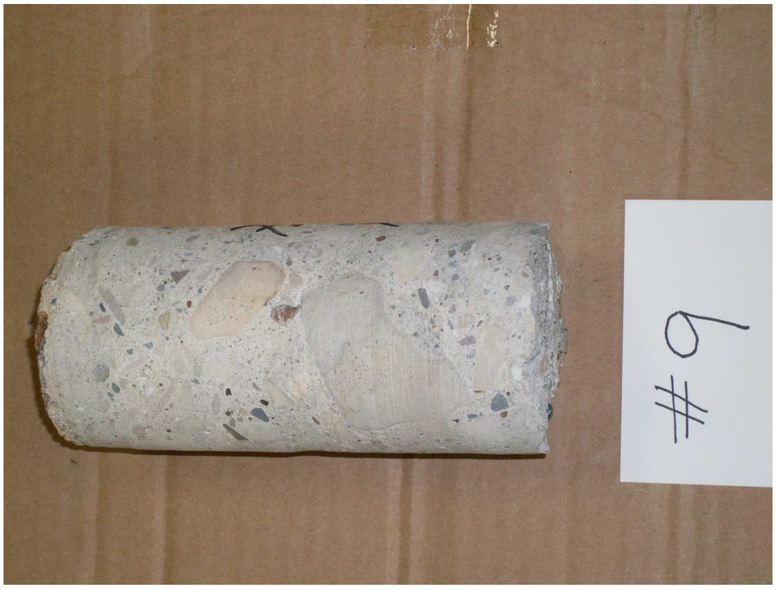
Core #6

Core #7

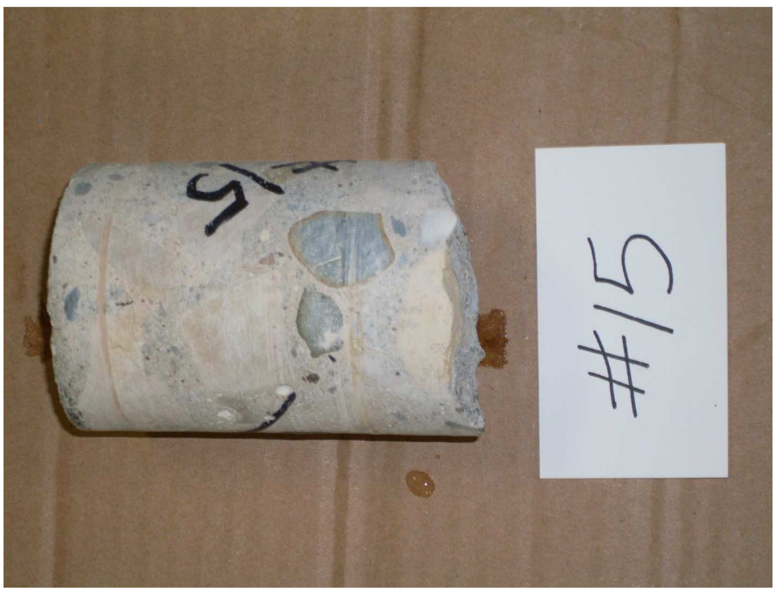
Core #7 Break

Core #8

Core #9



Core #9 Dry



Core #15 Dry

Core #16 Dry

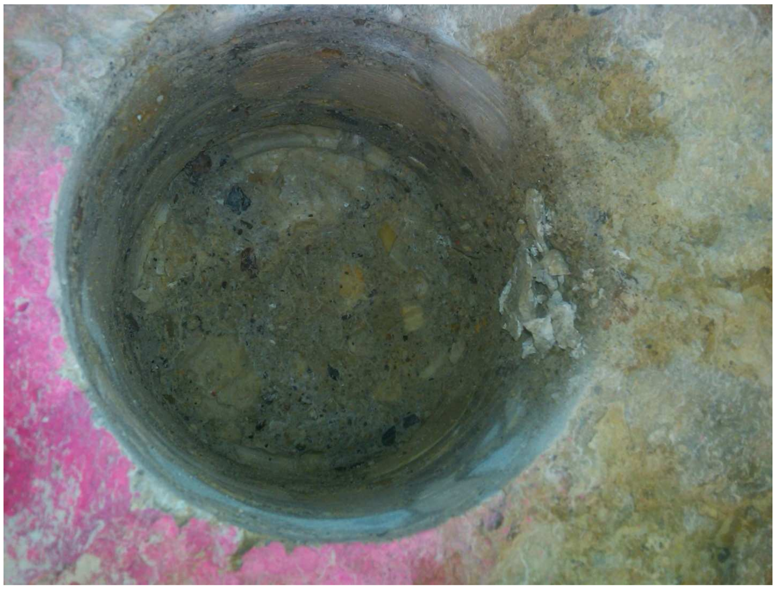
Core #17 Dry


Hole #2

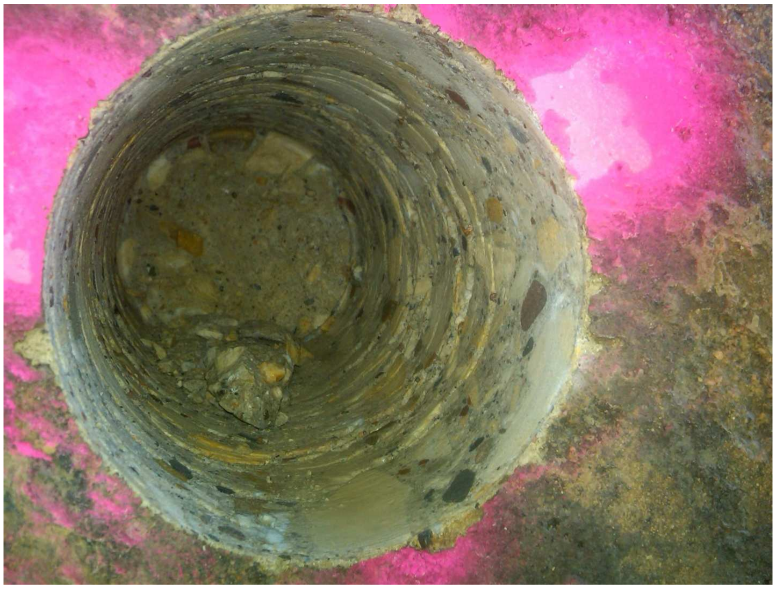

Hole #3


Hole #4


Hole #5 – 1


Hole #5 – 2


Hole #6



Hole #8

Hole #9

ST-11 Core Photographs

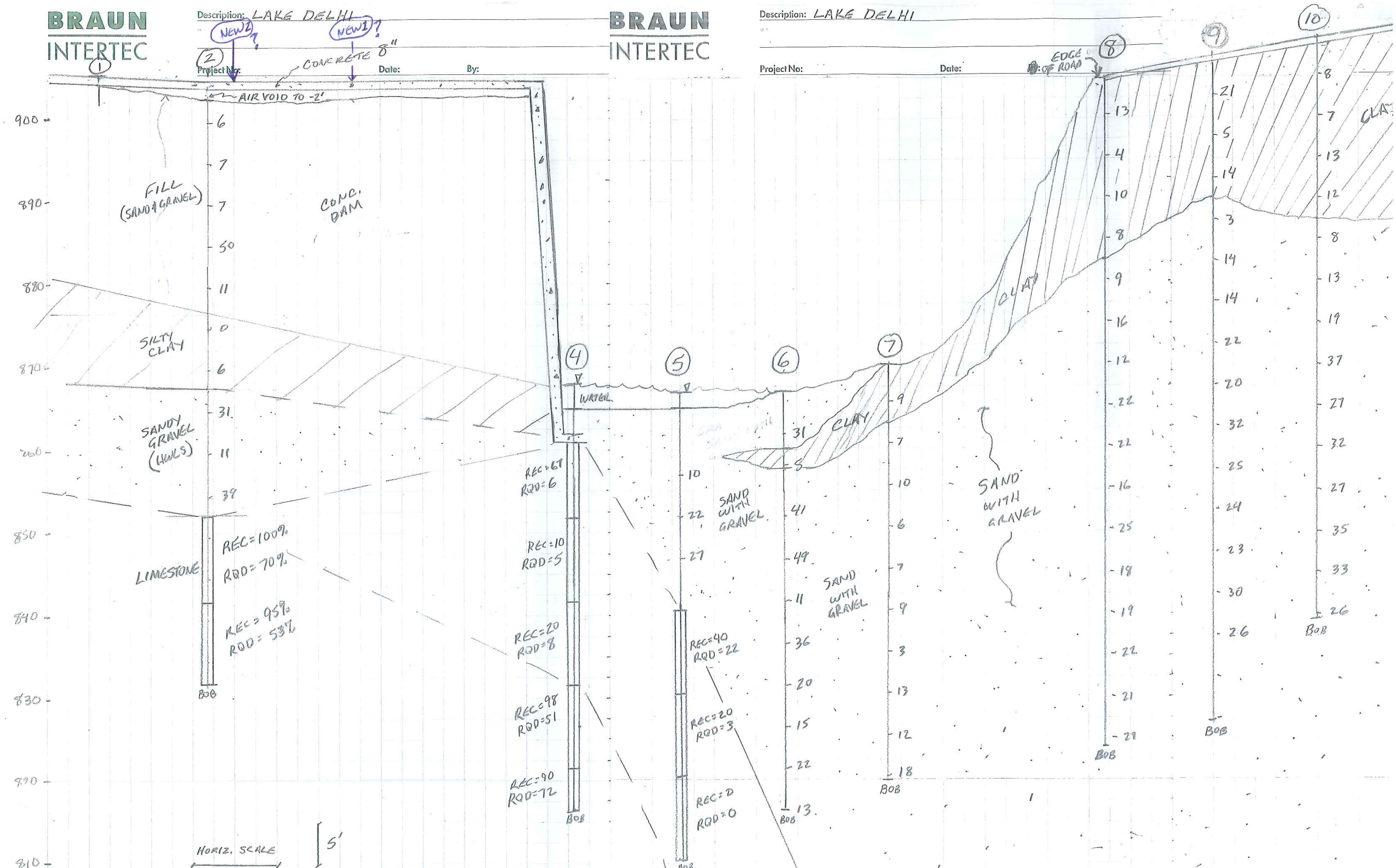
ST-11 50' to 60'

ST-11 50' to 60' – Interface 1

ST-11 50' to 60' – Interface 2

ST-11 87' to 97'

Geotechnical Parameters


BRAUN INTERTEC
Description: LAKE DELHI
Project No:
Date:
By:
NEW2 ?
NEW1 ?
CONCRETE 8"
AIR VOID TO -2'
FILL (SAND & GRAVEL)
CONC. DAM
SILTY CLAY
SANDY GRAVEL (HWLS)
LIMESTONE
REC=100% RQD=70%
REC=95% RQD=53%
WATER
REC=67 RQD=6
REC=10 RQD=5
REC=20 RQD=8
REC=98 RQD=51
REC=90 RQD=72
REC=40 RQD=22
REC=20 RQD=3
REC=0 RQD=0
SAND WITH GRAVEL
CLAY
EDGE OF ROAD
BOB
HORIZ. SCALE
5'
900
890
880
870
860
850
840
830
820
810

Clay Borrow: Total Stress Parameters

Table 3-2
Ratio of ϕ/δ (After Allen, Duncan, and Snacio 1988)

Soil Type	Steel	Wood	Concrete
Sand	$\delta/\phi = 0.54$	$\delta/\phi = 0.76$	$\delta/\phi = 0.76$
Silt & Clay	$\delta/\phi = 0.54$	$\delta/\phi = 0.55$	$\delta/\phi = 0.50$

Table 3-3
Values of δ for Various Interfaces
(after U.S. Department of the Navy 1982)

Soil Type	δ (deg)
(a) Steel sheet piles	
Clean gravel, gravel sand mixtures, well-graded rockfill with spalls	22
Clean sand, silty sand-gravel mixture, single-size hard rockfill	17
Silty sand, gravel or sand mixed with silt or clay	14
Fine sandy silt, nonplastic silt	11
(b) Concrete sheet piles	
Clean gravel, gravel sand mixtures, well-graded rockfill with spalls	22-26
Clean sand, silty sand-gravel mixture, single-size hard rockfill	17-22
Silty sand, gravel or sand mixed with silt or clay	17
Fine sandy silt, nonplastic silt	14

Table 3-4
Correlation of Undrained Shear Strength of Clay (q_u=2c)

Consistency	qu (psf)	SPT (blows/ft)	Saturated Unit Weight (psf)	
Very Soft	0-500	0-2	<100-110	
Soft	500-1,000	3-4	100-120	
Medium	1,000-2,000	5-8	110-125	Loess
Stiff	2,000-4,000	9-16	115-130	Till
Very Stiff	4,000-8,000	16-32	120-140	
Hard	>8,000	>32	>130	

(5) Since an undrained condition may be expected to occur under "fast" loading in the field, it represents a "short-term" condition; in time, drainage will occur, and the drained strength will govern (the "long-term" condition). To model these conditions in the laboratory, three types of tests are generally used; unconsolidated undrained (Q or UU), consolidated undrained (R or CU), and consolidated drained (S or CD). Undrained shear strength in the laboratory is determined from either Q or R tests and drained shear strength is established from S tests or from consolidated undrained tests with pore pressure measurements ($\bar{R}$).

(6) The undrained shear strength, S_u, of a normally consolidated clay is usually expressed by only a cohesion intercept; and it is labeled c_u to indicate that ϕ was taken as zero. c_u decreases dramatically with water content; therefore, in design it is common to consider the fully saturated condition even if a clay is partly saturated in the field. Typical undrained shear strength values are presented in Table 3-4. S_u increases with depth (or effective stress) and this is commonly expressed with the ratio "S_u/p" (p denotes the effective vertical stress). This ratio correlates roughly with plasticity index and overconsolidation ratio (Figures 3-2, 3-3, respectively). The undrained shear strength of many overconsolidated soils is further complicated due to the presence of fissures; this leads to a lower field strength than tests on small laboratory samples indicate.

(7) The drained shear strength of normally consolidated clays is similar to that of loose sands ($c' = O$), except that ϕ is generally lower. An empirical correlation of the effective angle of internal friction, ϕ', with plasticity index for normally consolidated clays is shown in Figure 3-4. The drained shear strength of over-consolidated clays is similar to that of dense sands (again with lower ϕ'), where there is a peak strength (c' nonzero) and a "residual" shear strength ($c' = O$).

(8) The general approach in solving problems involving clay is that, unless the choice is obvious, both undrained and drained conditions are analyzed separately. The more critical condition governs the design. Total stresses are used in an analysis with undrained shear strength (since pore pressures are "included" in the undrained shear strength) and effective stresses in a drained case; thus such analyses are usually called total and effective stress analyses, respectively.

(9) At low stress levels, such as near the top of a wall, the undrained strength is greater than the drained

Loess: assume q_u = 2000 psf; c = 1000 psf (compacted)
Till: assume q_u = 4000 psf; c = 2000 psf (compacted)

CLAY Borrow: Effective Strength Parameters

Assume
$\phi_{EFF} = 28°$
$c = 0$

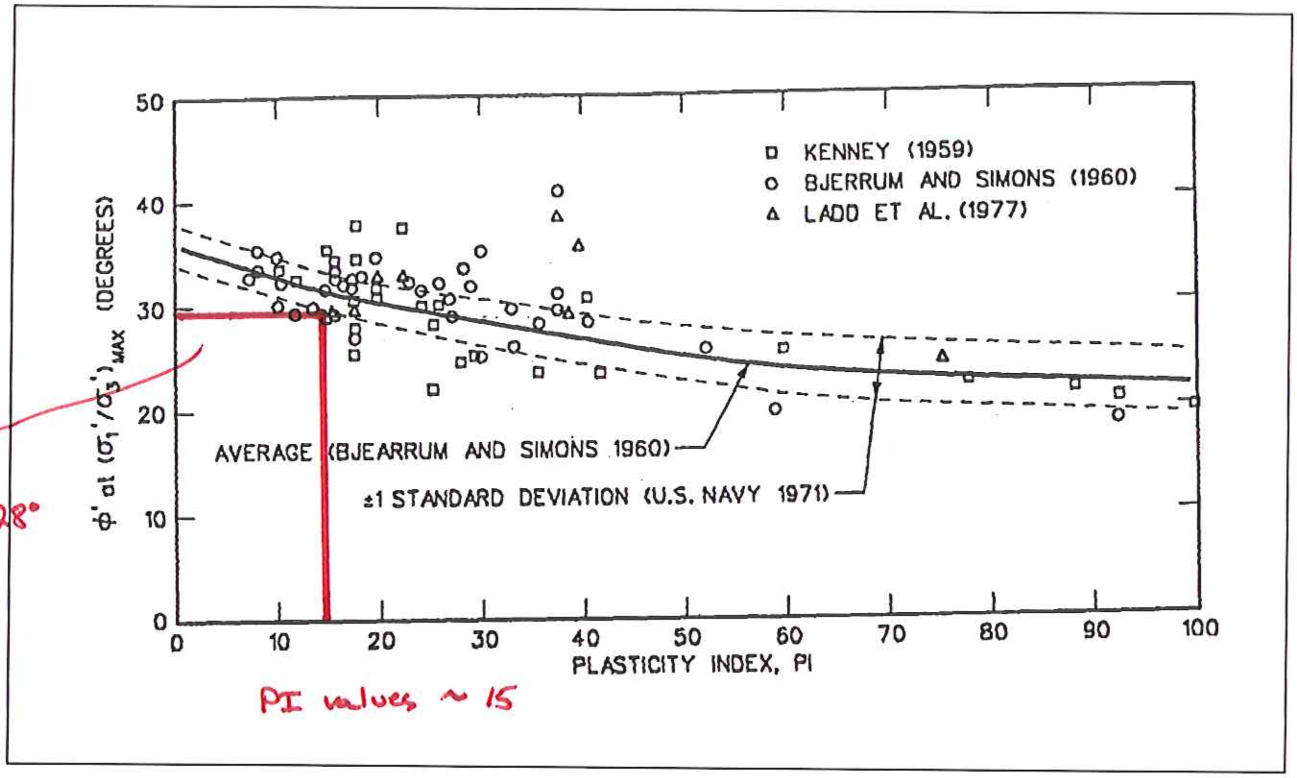


Figure 3-4. Empirical correlation between friction angle and PI from triaxial tests on normally consolidated clays

stress of 1 ton per square foot using an equation of the form:

$$N' = C_N N \tag{3-1}$$

where

N' = corrected resistance

C_N = correction factor

N = measured resistance

Table 3-5 and Figure 3-5 summarize the some most commonly proposed values for C_N. Whitman and Liao (1984) developed the following expression for C_N:

$$C_N = \sqrt{\frac{1}{\sigma'_{vo}}} \tag{3-2}$$

where effective stress due to overburden, σ'_{vo}, is expressed in tons per square foot. The drained friction angle ϕ' can be estimated from N' using Figure 3-6. The relative density of normally consolidated sands can be estimated from the correlation obtained by Marcuson and Bieganousky (1977):

$$D_r = 11.7 + 0.76[|222(N) + 1600 - 53(p'_{vo}) - 50(C_u)^2|]^{1/2} \tag{3-3}$$

where

p'_{vo} = effective overburden pressure in pounds per square inch

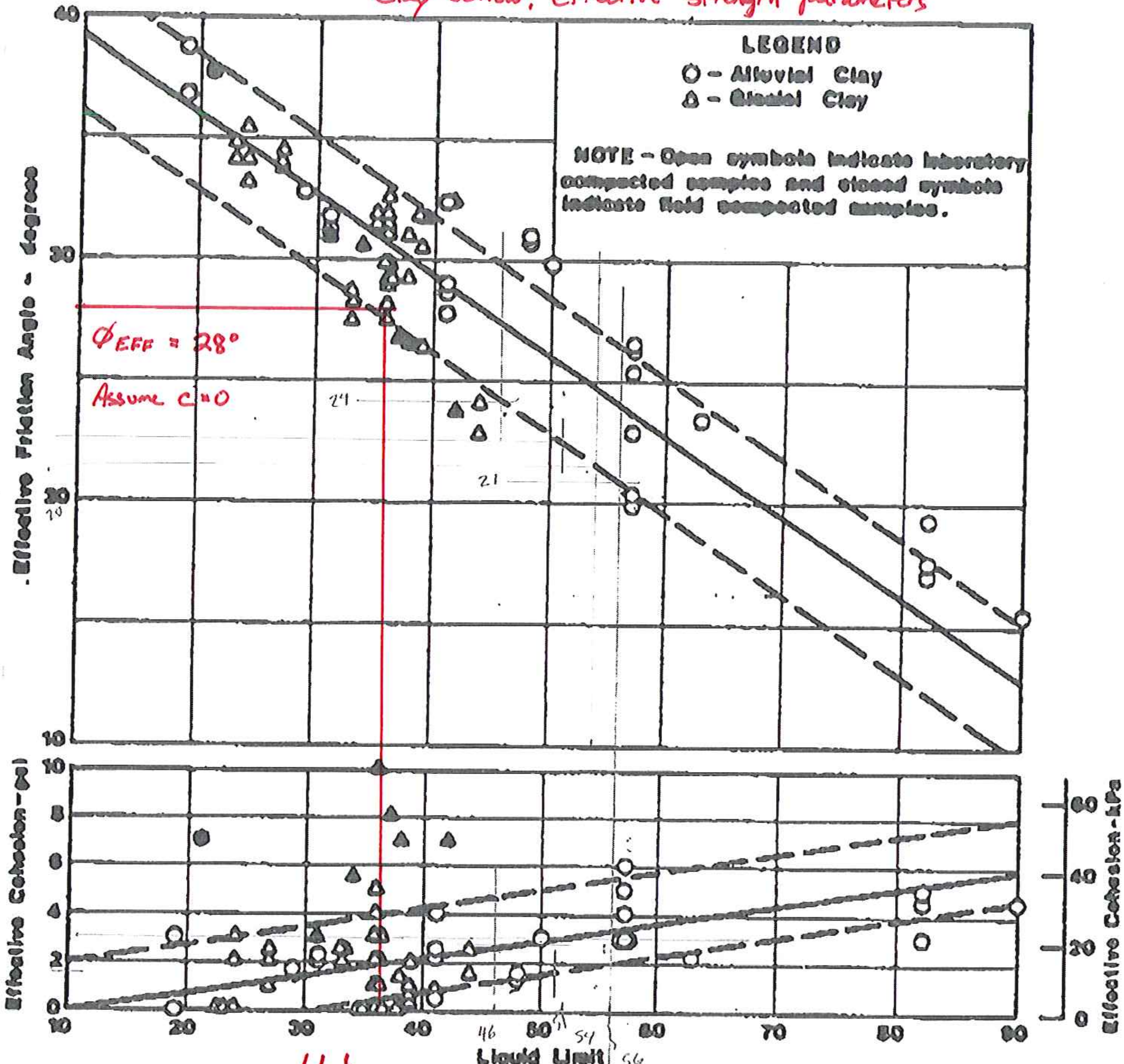
C_u = coefficient of uniformity (D_{60}/D_{10})

Correlations have also been proposed between the SPT and the undrained strength of clays (see Table 3-4). However, these are generally unreliable and should be used for very preliminary studies only and for checking the reasonableness of SPT and lab data.

c. Cone penetration test. The CPT (ASTM D 3441-79 (1986a)) is widely used in Europe and is gaining

From Brown. Ref. unknown

Clay borrow: effective strength parameters



LL's typically mid to upper 30's

FIG. 11. Effective friction angle and cohesion vs. liquid limit for consolidated undrained tests on compacted clays.

Sand/Gravel Strength Parameters

Table 3-1
Granular Soil Properties (after Teng 1962)

Compactness	Relative Density (%)	SPT N (blows per ft)	Angle of Internal Friction (deg)	Unit Weight	
				Moist (pcf)	Submerged (pcf)
Very Loose	0-15	0-4	<28	<100	<60
Loose	16-35	5-10	28-30	95-125	55-65
Medium	36-65	11-30	31-36	110-130	60-70
Dense	66-85	31-50	37-41	110-140	65-85
Very Dense	86-100	>51	>41	>130	>75

Blow Count Range

Assume $\phi = 30°$

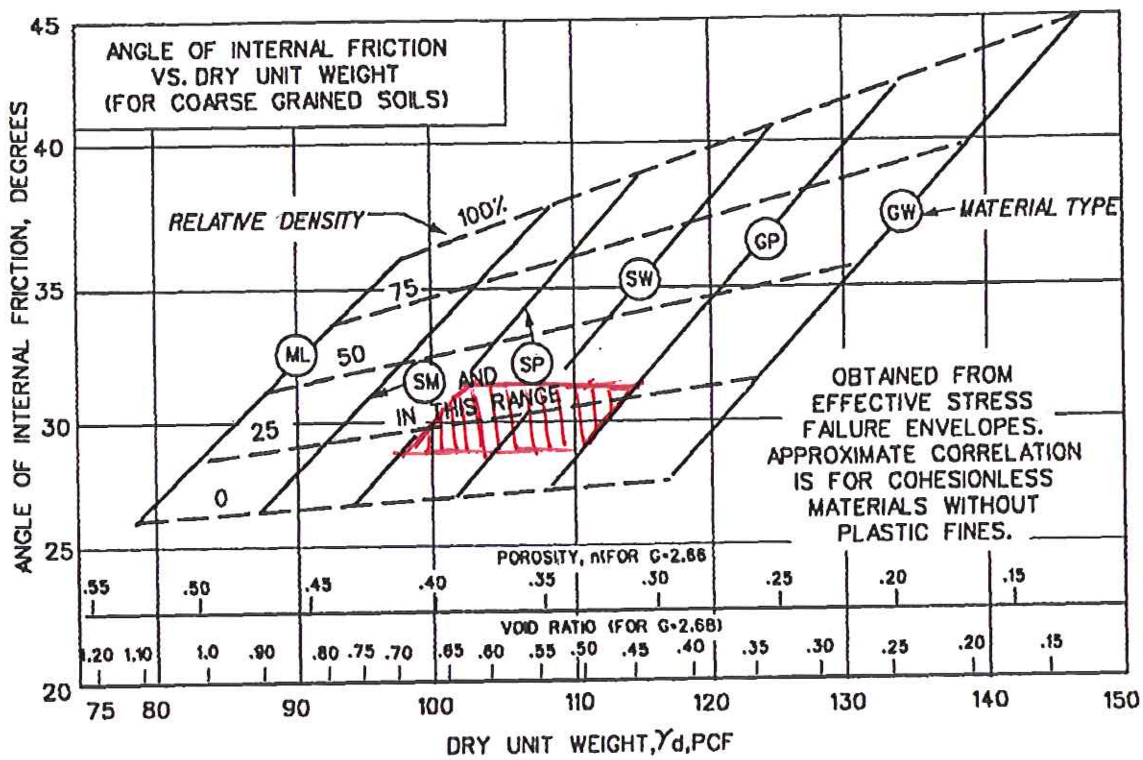


Figure 3-1. Cohesionless Soil Properties (after U.S. Department of the Navy 1971)

SEEPAGE PARAMETERS

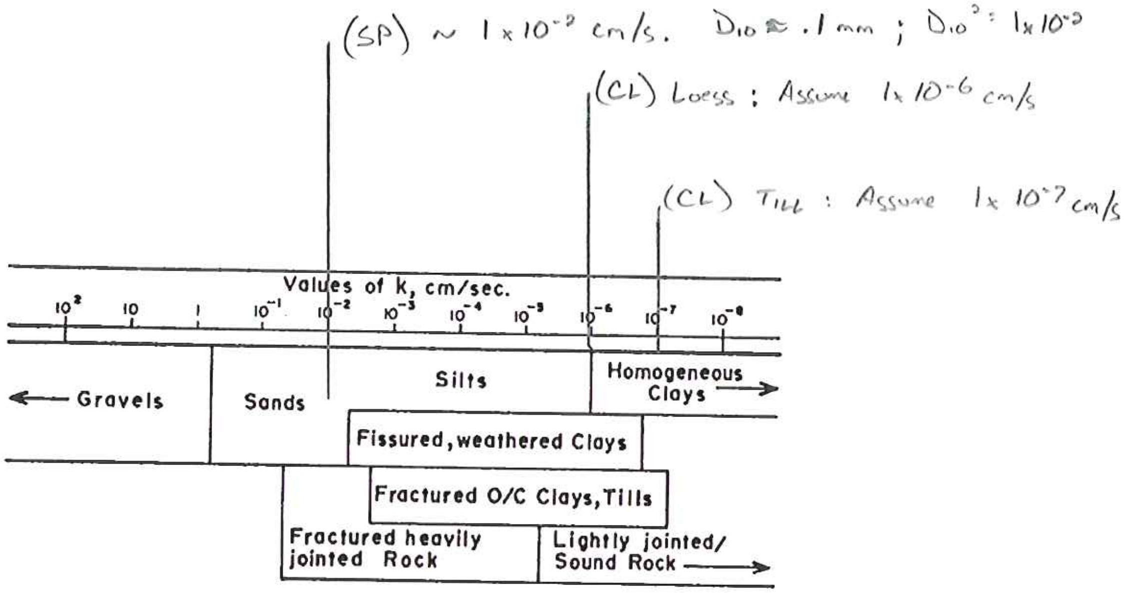


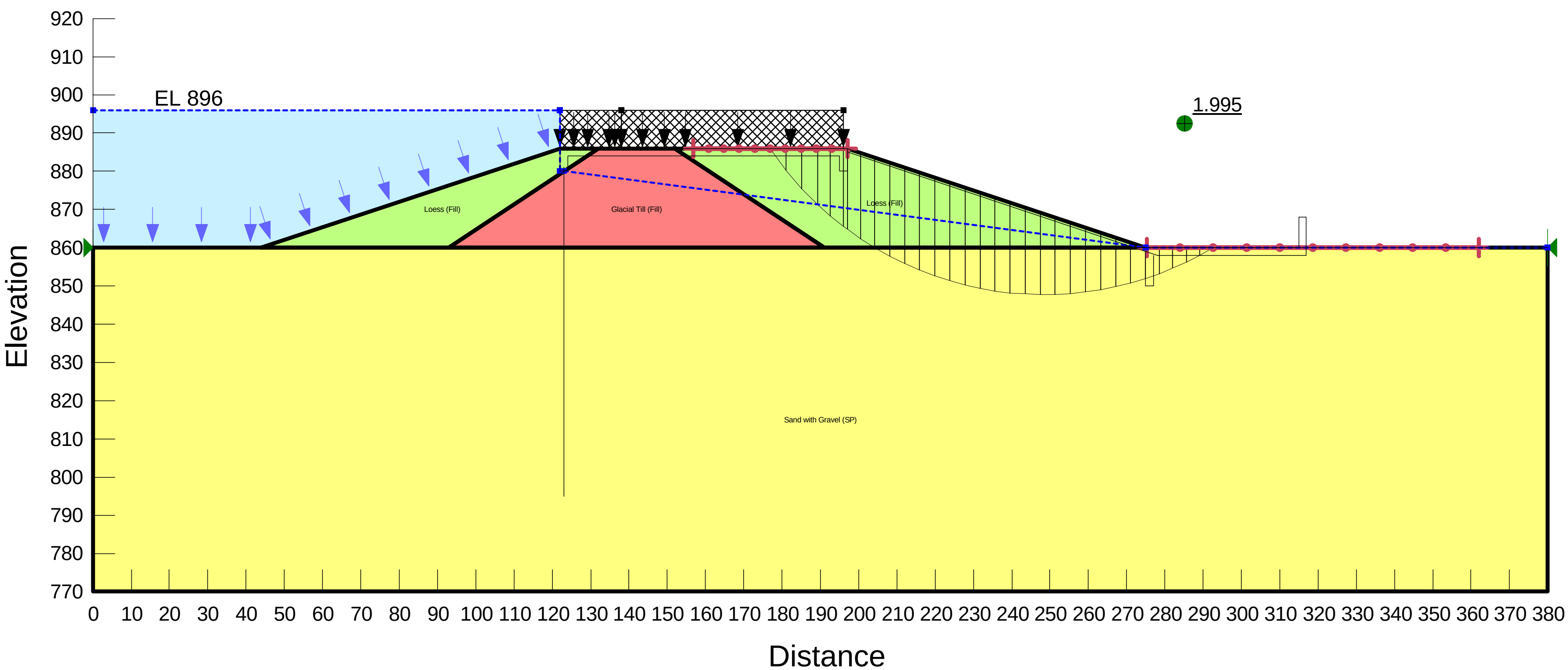
Figure 2-5. Approximate range in coefficient of permeability of soils and rocks (from Milligan[224])

ordinary gravel or soil and have tremendous influence on the watertightness of dam foundations and abutments as shown in figure 2-6 (Cedergren 1977). Figure 2-6a shows a soil profile surmised from several drill holes. The grain size analysis of soil samples taken at frequent intervals erroneously indicated that the deposit was composed of relatively uniform sandy gravels. Laboratory permeability tests on disturbed samples produced coefficients of permeability of about 1×10^{-6} cm/sec. Using this value of permeability, the probable seepage loss beneath the proposed dam was estimated to be 3 cu ft/day, which is an insignificant quantity. However, the design engineer had observed many openwork streaks in which the fines fraction of the material was almost completely absent along the banks of the river and noted that the ground-water table was level for several hundred feet away from the river and fluctuated rapidly with changes in river stage. Field pumping tests were conducted which indicated somewhat variable permeabilities but none approaching the magnitude of openwork gravels. Based upon the available data, the dam was designed with a cutoff trench to bedrock. During the excavation of the cutoff trench, streaks of openwork gravel were found throughout the foundation. A revised seepage computation based on a permeability of 30 cm/sec indicated that without the cutoff trench, the theoretical underseepage would be about 1,000,000 cu ft/day. If openwork gravel or other important discontinuities in earth dam foundations remain undetected, serious problems from excessive seepage and hydrostatic pressures will develop. This example illustrates the potential serious effects of deviations between the design assumptions and the as-built dam (Cedergren 1977). Also, thin continuous seams of cohesive soil can drastically alter the vertical flow through what would otherwise be a highly permeable site.

Slope Stability Analysis

Slope Stability Analysis
Service Spillway
Total Stress Condition
File Name: Service_Emb_Slope_Total.gsz

Computed By: L. Karels Date: 12/2/2011
Checked By: J. Jacks Date: 12/2/2011



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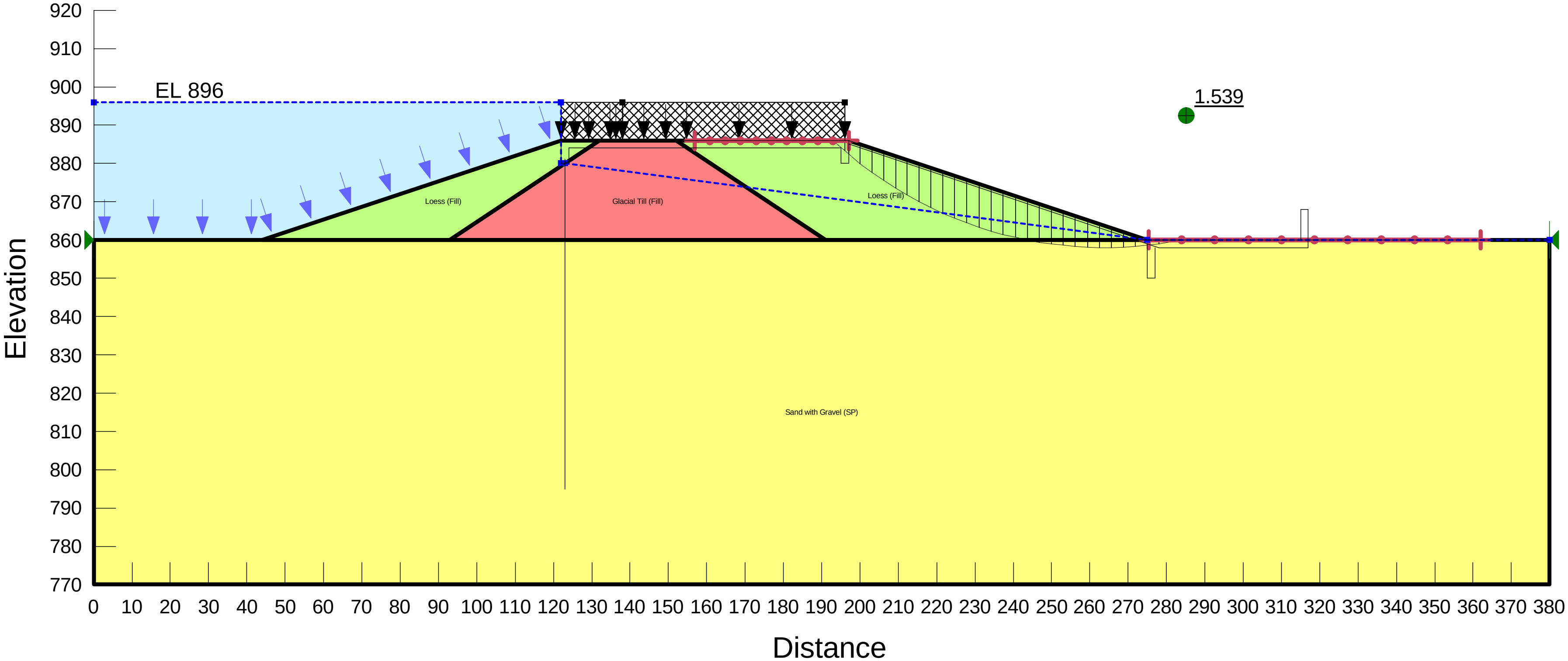
Slope Stability Analysis
Service Spillway
Effective Stress Condition
File Name: Service_Emb_Slope_Drained.gsz

Computed By: L. Karels Date: 12/2/2011
Checked By: J. Jacks Date: 12/2/2011



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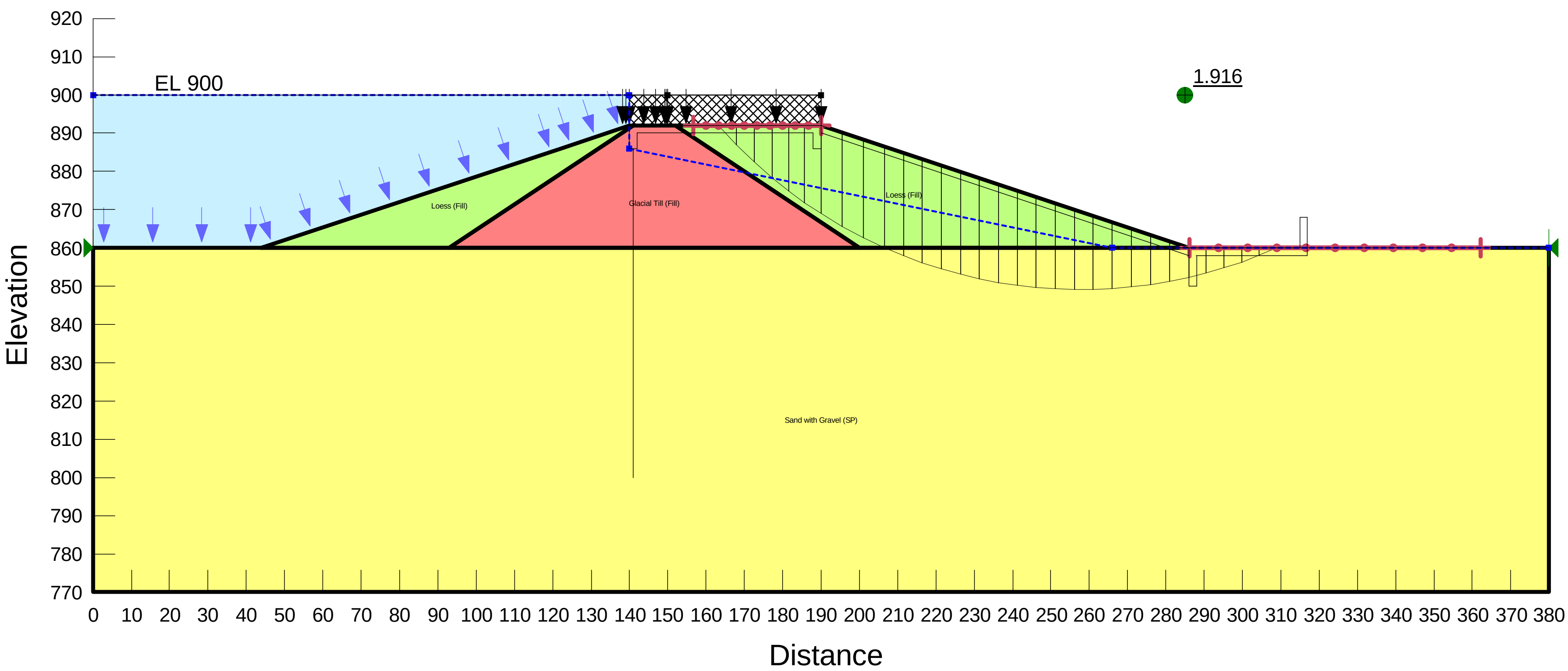
Slope Stability Analysis
Auxiliary Spillway
Total Stress Condition
File Name: Aux_Emb_Slope_Total.gsz

Computed By: L. Karels Date: 12/2/2011
Checked By: J. Jacks Date: 12/2/2011



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Slope Stability Analysis
Auxiliary Spillway
Total Stress Condition
File Name: Aux_Emb_Slope_Total_RCC.gsz

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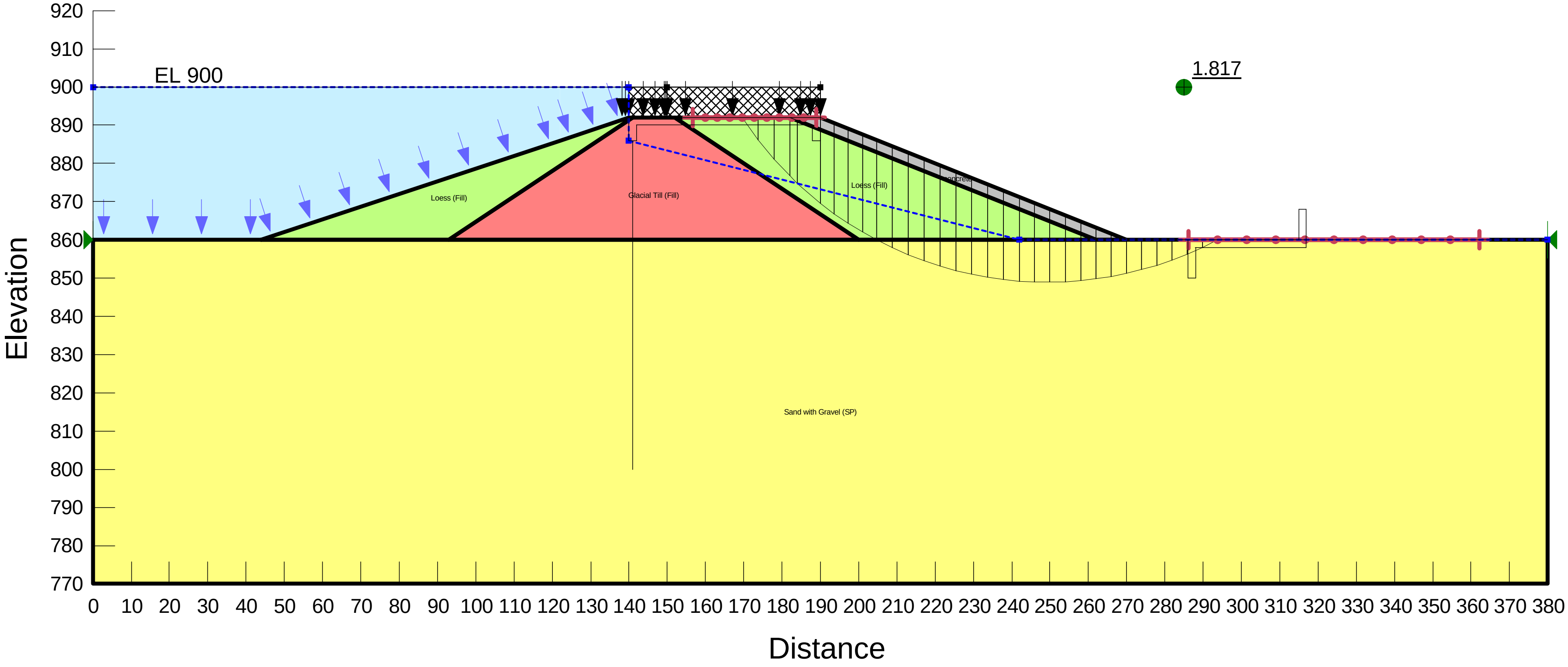
Checked By: J. Jacks Date: 12/2/2011



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Slope Stability Analysis
Auxiliary Spillway
Effective Stress Condition
File Name: Aux_Emb_Slope_Drained.gsz

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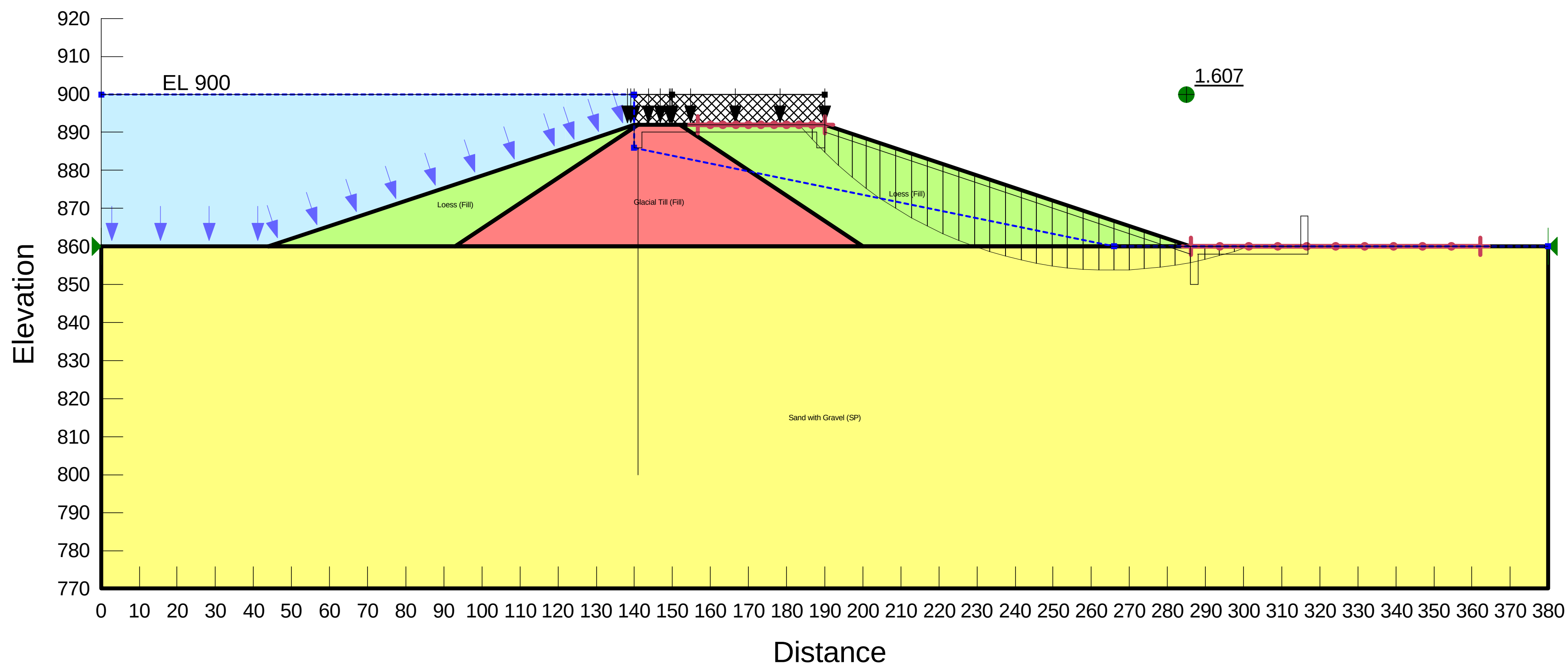
Checked By: J. Jacks Date: 12/2/2011



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Auxiliary Spillway
Effective Stress Condition
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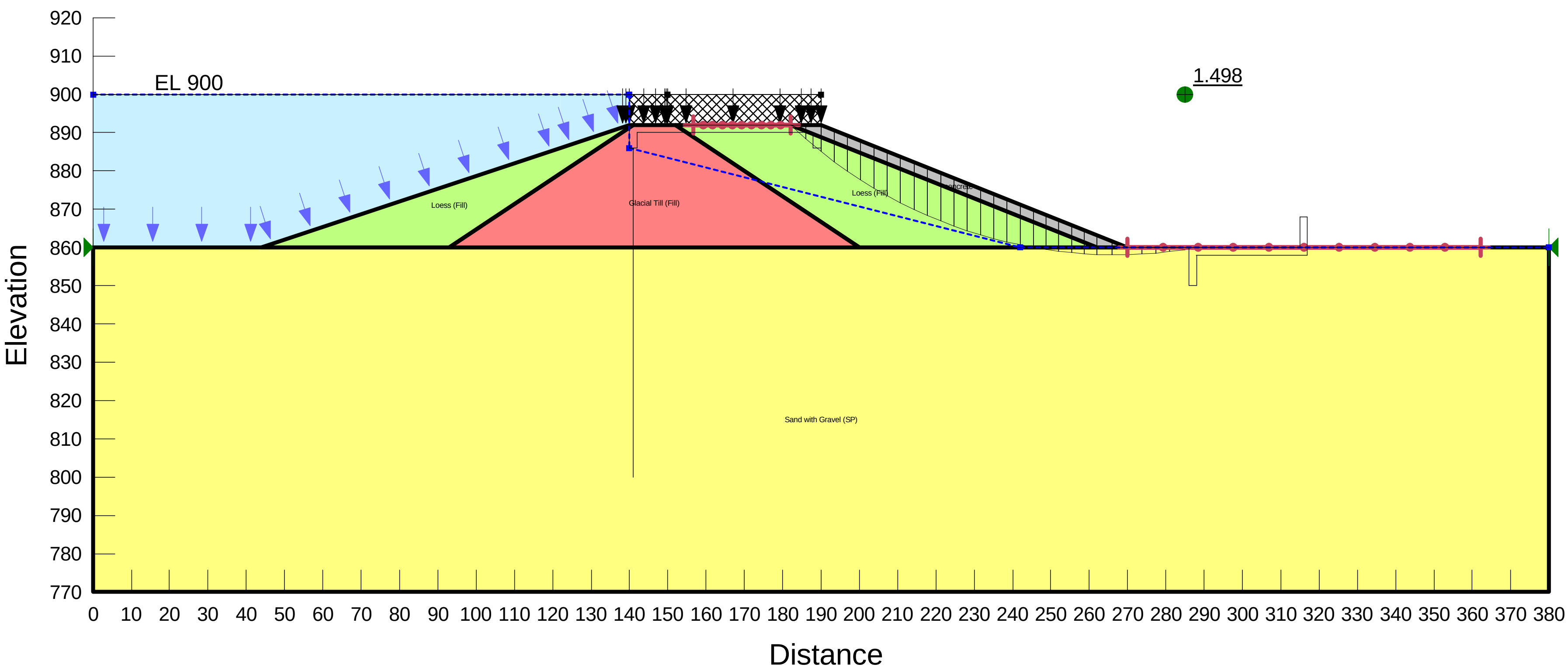

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Section 1

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Revision Number: 72
Last Edited By: Karels, Lucas
Date: 12/2/2011
Time: 1:23:55 PM
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Last Solved Date: 12/2/2011
Last Solved Time: 1:23:58 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Section 1

Description: Cut bank along Mill Creek (1)
Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

Slip Surface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Entry and Exit
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No
- Tension Crack

Tension Crack Option: (none)

FOS Distribution

FOS Calculation Option: Constant

Advanced

Number of Slices: 30

Optimization Tolerance: 0.01

Minimum Slip Surface Depth: 0.1 ft

Optimization Maximum Iterations: 2000

Optimization Convergence Tolerance: 1e-007

Starting Optimization Points: 8

Ending Optimization Points: 16

Complete Passes per Insertion: 1

Driving Side Maximum Convex Angle: 5 °

Resisting Side Maximum Convex Angle: 1 °

Materials

Glacial Till (Fill)

Model: Mohr-Coulomb

Unit Weight: 110 pcf

Cohesion: 2000 psf

Phi: 0 °

Phi-B: 0 °

Pore Water Pressure

Piezometric Line: 1

Loess (Fill)

Model: Mohr-Coulomb

Unit Weight: 110 pcf

Cohesion: 1000 psf

Phi: 0 °

Phi-B: 0 °

Pore Water Pressure

Piezometric Line: 1

Sand with Gravel (SP)

Model: Mohr-Coulomb

Unit Weight: 125 pcf

Cohesion: 0 psf

Phi: 30 °

Phi-B: 0 °

Pore Water Pressure

Piezometric Line: 1

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (156.77031, 886) ft
Left-Zone Right Coordinate: (197, 886) ft
Left-Zone Increment: 10
Right Projection: Range
Right-Zone Left Coordinate: (275.30048, 860) ft
Right-Zone Right Coordinate: (362.05151, 860) ft
Right-Zone Increment: 10
Radius Increments: 10

Slip Surface Limits

Left Coordinate: (0, 860) ft
Right Coordinate: (380, 860) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	0	896
	122	896
	122	880
	123	880
	275	860
	380	860

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 62.4 pcf
Direction: Vertical

Coordinates

	X (ft)	Y (ft)
	122	896
	138	896

Surcharge Load 2

Surcharge (Unit Weight): 31.2 pcf

Direction: Vertical

Coordinates

	X (ft)	Y (ft)
	138	896
	196	896

Regions

	Material	Points	Area (ft²)
Region 1	Sand with Gravel (SP)	2,6,9,12,11,3,4,5	34200
Region 2	Loess (Fill)	6,7,8,9	767
Region 3	Glacial Till (Fill)	8,9,12,10	1534
Region 4	Loess (Fill)	11,13,10,12	1677

Points

	X (ft)	Y (ft)
Point 1	16	5591
Point 2	0	860
Point 3	380	860
Point 4	380	770
Point 5	0	770
Point 6	44	860
Point 7	122	886
Point 8	132	886
Point 9	93	860
Point 10	152	886
Point 11	275	860
Point 12	191	860
Point 13	197	886

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	633	1.995	(248.41, 933.743)	85.996	(176.885, 886)	(292.651, 860)

Slices of Slip Surface: 633

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
1	633	178.95575	883.1571	-656.43613	17.792044	0	1000
2	633	183.0969	877.8928	-361.94322	584.82478	0	1000
3	633	187.2381	873.3733	-113.92154	1087.9055	0	1000
4	633	190.9815	869.7734	79.976495	1499.6872	0	1000

5	633	194.32715	866.91775	230.69581	1834.2888	0	1000
6	633	196.5	865.18535	320.96352	1764.7851	0	1000
7	633	198.79255	863.53985	404.81957	1903.7818	0	1000
8	633	202.3776	861.13645	525.36269	2078.105	0	1000
9	633	206.1376	858.8889	634.72605	2258.6281	937.56027	0
10	633	210.0726	856.7975	732.92334	2414.2661	970.72372	0
11	633	214.0076	854.95805	815.38725	2542.5102	997.15493	0
12	633	217.9426	853.35325	883.22017	2644.1711	1016.6855	0
13	633	221.8776	851.9692	937.28675	2720.6093	1029.6017	0
14	633	225.8126	850.7948	978.25032	2771.6724	1035.4328	0
15	633	229.7476	849.82125	1006.6934	2798.2312	1034.3448	0
16	633	233.6826	849.0416	1023.0509	2799.9261	1025.8794	0
17	633	237.6176	848.45055	1027.6147	2777.0985	1010.0649	0
18	633	241.5526	848.04415	1020.6783	2729.0199	986.31152	0
19	633	245.4876	847.81975	1002.3702	2655.6485	954.52071	0
20	633	249.42255	847.776	972.79345	2555.8635	913.9859	0
21	633	253.3575	847.9126	931.9494	2428.7566	864.18204	0
22	633	257.2925	848.23035	879.81736	2273.1374	804.43373	0
23	633	261.2275	848.73135	816.24058	2087.2912	733.84139	0
24	633	265.1625	849.41885	741.04705	1869.1186	651.2924	0
25	633	269.0975	850.29745	653.90431	1615.9157	555.41754	0
26	633	273.0325	851.37335	554.46353	1324.2406	444.43102	0
27	633	276.76505	852.5781	463.12874	1088.1306	360.84498	0
28	633	280.2952	853.89955	380.66846	914.20938	308.03999	0
29	633	283.82535	855.40235	286.90623	705.76611	241.82886	0
30	633	287.35545	857.09715	181.13976	457.66036	159.64924	0
31	633	290.8856	858.997	62.589033	162.94132	57.938421	0

Section 1

File Information

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Comments:
Created By: Karels, Lucas
Revision Number: 72
Last Edited By: Karels, Lucas
Date: 12/3/2011
Time: 3:23:13 PM
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Last Solved Date: 12/3/2011
Last Solved Time: 3:23:16 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Section 1

Description: Cut bank along Mill Creek (1)
Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

Slip Surface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Entry and Exit
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No
- Tension Crack

Tension Crack Option: (none)
FOS Distribution
FOS Calculation Option: Constant
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Glacial Till (Fill)

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 0 psf
Phi: 28 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Loess (Fill)

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 0 psf
Phi: 28 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Sand with Gravel (SP)

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 30 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (156.77031, 886) ft
Left-Zone Right Coordinate: (197, 886) ft
Left-Zone Increment: 10
Right Projection: Range
Right-Zone Left Coordinate: (275.30048, 860) ft
Right-Zone Right Coordinate: (362.05151, 860) ft
Right-Zone Increment: 10
Radius Increments: 10

Slip Surface Limits

Left Coordinate: (0, 860) ft
Right Coordinate: (380, 860) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	0	896
	122	896
	122	880
	123	880
	275	860
	380	860

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 62.4 pcf
Direction: Vertical

Coordinates

	X (ft)	Y (ft)
	122	896
	138	896

Surcharge Load 2

Surcharge (Unit Weight): 31.2 pcf

Direction: Vertical

Coordinates

	X (ft)	Y (ft)
	138	896
	196	896

Regions

	Material	Points	Area (ft^2)
Region 1	Sand with Gravel (SP)	2,6,9,12,11,3,4,5	34200
Region 2	Loess (Fill)	6,7,8,9	767
Region 3	Glacial Till (Fill)	8,9,12,10	1534
Region 4	Loess (Fill)	11,13,10,12	1677

Points

	X (ft)	Y (ft)
Point 1	16	5591
Point 2	0	860
Point 3	380	860
Point 4	380	770
Point 5	0	770
Point 6	44	860
Point 7	122	886
Point 8	132	886
Point 9	93	860
Point 10	152	886
Point 11	275	860
Point 12	191	860
Point 13	197	886

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	1104	1.539	(263.836, 961.759)	103.733	(192.977, 886)	(283.976, 860)

Slices of Slip Surface: 1104

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
1	1104	194.4885	884.6408	-876.55539	310.19868	164.93556	0
2	1104	196.5	882.8549	-781.65707	237.39525	126.22529	0
3	1104	198.5351	881.1841	-694.07331	331.38591	176.20101	0
4	1104	201.6053	878.7886	-569.80131	449.70737	239.11365	0

5	1104	204.6755	876.5711	-456.63933	559.18186	297.32227	0
6	1104	207.7457	874.51805	-353.7239	659.72205	350.78044	0
7	1104	210.8159	872.618	-260.37688	751.20827	399.42452	0
8	1104	213.8861	870.86135	-175.97194	833.48819	443.17353	0
9	1104	216.9563	869.23995	-100.00142	906.45993	481.97329	0
10	1104	220.0265	867.74675	-32.035328	969.97577	515.74527	0
11	1104	223.1427	866.35695	29.102237	1025.4866	529.78698	0
12	1104	226.30495	865.0688	83.517807	1071.522	525.33113	0
13	1104	229.4672	863.8998	130.50263	1105.7173	518.53084	0
14	1104	232.6294	862.84565	170.31729	1128.1202	509.27285	0
15	1104	235.7916	861.90275	203.19263	1138.5972	497.36345	0
16	1104	238.9538	861.06795	229.31873	1137.0614	482.65532	0
17	1104	242.11605	860.3386	248.86477	1123.3718	464.98361	0
18	1104	245.26235	859.7151	261.94041	1105.0955	486.79581	0
19	1104	248.3926	859.19455	268.72245	1075.5406	465.81665	0
20	1104	251.52285	858.77175	269.40174	1032.0125	440.29355	0
21	1104	254.65315	858.4455	264.0595	973.99467	409.88126	0
22	1104	257.78345	858.2149	252.75009	900.93624	374.23045	0
23	1104	260.91375	858.07925	235.5113	812.15969	332.9281	0
24	1104	264.04405	858.03825	212.36691	706.89936	285.51845	0
25	1104	267.1743	858.0918	183.32606	584.24487	231.47058	0
26	1104	270.30455	858.24	148.37769	443.15308	170.18865	0
27	1104	273.43485	858.48325	107.495	282.34218	100.94806	0
28	1104	276.49595	858.81265	74.087761	171.21163	56.074493	0
29	1104	279.4878	859.22495	48.362524	114.02546	37.910515	0
30	1104	282.47965	859.72665	17.056318	41.06997	13.864288	0

Section 1

File Information

Title:
Comments:
Created By: Karels, Lucas
Revision Number: 81
Last Edited By: Karels, Lucas
Date: 12/7/2011
Time: 1:29:04 PM
File Name: Aux_Emb_Slope_Total.gsz
Directory: \\mnp-fs1\Mnp-Projects-1\23601\Active\07-Design\02-Comps\GEO\Slope_Stability\
Last Solved Date: 12/7/2011
Last Solved Time: 1:29:08 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Section 1

Description: Cut bank along Mill Creek (1)
Kind: SLOPE/W
Method: Spencer
Settings
  Apply Phreatic Correction: No
  PWP Conditions Source: Piezometric Line
  Use Staged Rapid Drawdown: No
Slip Surface
  Direction of movement: Left to Right
  Use Passive Mode: No
  Slip Surface Option: Entry and Exit
  Critical slip surfaces saved: 1
  Optimize Critical Slip Surface Location: No
  Tension Crack

Tension Crack Option: (none)

FOS Distribution

FOS Calculation Option: Constant

Advanced

Number of Slices: 30

Optimization Tolerance: 0.01

Minimum Slip Surface Depth: 0.1 ft

Optimization Maximum Iterations: 2000

Optimization Convergence Tolerance: 1e-007

Starting Optimization Points: 8

Ending Optimization Points: 16

Complete Passes per Insertion: 1

Driving Side Maximum Convex Angle: 5 °

Resisting Side Maximum Convex Angle: 1 °

Materials

Glacial Till (Fill)

Model: Undrained (Phi=0)

Unit Weight: 110 pcf

Cohesion: 2000 psf

Pore Water Pressure

Piezometric Line: 1

Loess (Fill)

Model: Undrained (Phi=0)

Unit Weight: 110 pcf

Cohesion: 1000 psf

Pore Water Pressure

Piezometric Line: 1

Sand with Gravel (SP)

Model: Mohr-Coulomb

Unit Weight: 125 pcf

Cohesion: 0 psf

Phi: 30 °

Phi-B: 0 °

Pore Water Pressure

Piezometric Line: 1

Slip Surface Entry and Exit

Left Projection: Range

Left-Zone Left Coordinate: (156.65428, 892) ft

Left-Zone Right Coordinate: (190, 892) ft

Left-Zone Increment: 10

Right Projection: Range
Right-Zone Left Coordinate: (286.15634, 860) ft
Right-Zone Right Coordinate: (362.08136, 860) ft
Right-Zone Increment: 10
Radius Increments: 10

Slip Surface Limits

Left Coordinate: (0, 860) ft
Right Coordinate: (380, 860) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	0	900
	140	900
	140	886
	266	860
	380	860

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 62.4 pcf
Direction: Normal

Coordinates

	X (ft)	Y (ft)
	140	900
	150	900

Surcharge Load 2

Surcharge (Unit Weight): 31.2 pcf
Direction: Normal

Coordinates

	X (ft)	Y (ft)
	150	900
	190	900

Regions

	Material	Points	Area (ft²)
Region 1	Sand with Gravel (SP)	2,6,9,12,11,3,4,5	34200
Region 2	Loess (Fill)	6,7,14,8,9	800
Region 3	Glacial Till (Fill)	8,9,12,10	1888
Region 4	Loess (Fill)	11,13,10,12	1984

Points

	X (ft)	Y (ft)
Point 1	16	5591
Point 2	0	860
Point 3	380	860
Point 4	380	770
Point 5	0	770
Point 6	44	860
Point 7	122	886
Point 8	141	892
Point 9	93	860
Point 10	152	892
Point 11	286	860
Point 12	200	860
Point 13	190	892
Point 14	140	892

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	280	1.916	(257.757, 974.417)	125.34	(163.323, 892)	(308.934, 860)

Slices of Slip Surface: 280

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
1	280	165.6541	889.47285	-547.0347	46.614012	0	1000
2	280	170.3155	884.67475	-307.6551	544.83056	0	1000
3	280	174.9769	880.3527	-97.97876	1005.4583	0	1000
4	280	179.423	876.6074	78.479144	1413.9861	0	1000
5	280	183.6538	873.362	226.51645	1775.6115	0	1000
6	280	187.8846	870.3899	357.49407	2113.5709	0	1000
7	280	192.7634	867.2924	487.96078	2157.3361	0	1000
8	280	198.2902	864.12585	614.38411	2346.2443	0	1000
9	280	203.817	861.31795	718.43435	2498.2133	0	1000
10	280	209.0562	858.9557	798.38589	2619.6048	1051.4812	0
11	280	214.0078	856.989	857.34124	2724.5749	1078.0479	0

12	280	218.95945	855.26045	901.44862	2802.6847	1097.6791	0
13	280	223.9111	853.7598	931.32744	2854.1056	1110.1165	0
14	280	228.8627	852.47865	947.52056	2879.2897	1115.3074	0
15	280	233.81435	851.4101	950.44846	2877.9483	1112.8425	0
16	280	238.766	850.5486	940.43393	2850.1869	1102.5964	0
17	280	243.71765	849.8899	917.77916	2795.7015	1084.2189	0
18	280	248.6693	849.43075	882.67006	2714.1571	1057.4095	0
19	280	253.6209	849.16895	835.24815	2604.5872	1021.5284	0
20	280	258.57255	849.1033	775.58564	2466.2028	976.07827	0
21	280	263.5242	849.23345	703.70751	2298.2157	920.58972	0
22	280	268.5	849.5626	651.29347	2095.258	833.67332	0
23	280	273.5	850.0943	618.12391	1853.5979	713.30125	0
24	280	278.5	850.8305	572.17662	1574.398	578.63275	0
25	280	283.5	851.77485	513.25504	1254.854	428.16234	0
26	280	288.2934	852.8758	444.54594	1010.1452	326.5489	0
27	280	292.88015	854.1216	366.80563	848.23409	277.95285	0
28	280	297.4669	855.5574	277.21372	653.16647	217.05642	0
29	280	302.0537	857.1902	175.3284	421.49053	142.12178	0
30	280	306.64045	859.0284	60.6256	148.9583	50.99891	0

Section 1

File Information

Title:
Comments:
Created By: Karels, Lucas
Revision Number: 82
Last Edited By: Karels, Lucas
Date: 12/7/2011
Time: 1:26:34 PM
File Name: Aux_Emb_Slope_Total_RCC.gsz
Directory: \\mnp-fs1\Mnp-Projects-1\23601\Active\07-Design\02-Comps\GEO\Slope_Stability\
Last Solved Date: 12/7/2011
Last Solved Time: 1:26:38 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Section 1

Description: Cut bank along Mill Creek (1)
Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

Slip Surface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Entry and Exit
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No
- Tension Crack

Tension Crack Option: (none)

FOS Distribution

FOS Calculation Option: Constant

Advanced

Number of Slices: 30

Optimization Tolerance: 0.01

Minimum Slip Surface Depth: 0.1 ft

Optimization Maximum Iterations: 2000

Optimization Convergence Tolerance: 1e-007

Starting Optimization Points: 8

Ending Optimization Points: 16

Complete Passes per Insertion: 1

Driving Side Maximum Convex Angle: 5 °

Resisting Side Maximum Convex Angle: 1 °

Materials

Glacial Till (Fill)

Model: Undrained (Phi=0)

Unit Weight: 110 pcf

Cohesion: 2000 psf

Pore Water Pressure

Piezometric Line: 1

Loess (Fill)

Model: Undrained (Phi=0)

Unit Weight: 110 pcf

Cohesion: 1000 psf

Pore Water Pressure

Piezometric Line: 1

Sand with Gravel (SP)

Model: Mohr-Coulomb

Unit Weight: 125 pcf

Cohesion: 0 psf

Phi: 30 °

Phi-B: 0 °

Pore Water Pressure

Piezometric Line: 1

concrete

Model: Mohr-Coulomb

Unit Weight: 150 pcf

Cohesion: 5000 psf

Phi: 0 °

Phi-B: 0 °

Pore Water Pressure
Piezometric Line: 1

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (156.65428, 892) ft
Left-Zone Right Coordinate: (188.97878, 892) ft
Left-Zone Increment: 10
Right Projection: Range
Right-Zone Left Coordinate: (286.16396, 860) ft
Right-Zone Right Coordinate: (362.08281, 860) ft
Right-Zone Increment: 10
Radius Increments: 10

Slip Surface Limits

Left Coordinate: (0, 860) ft
Right Coordinate: (380, 860) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	0	900
	140	900
	140	886
	242	860
	380	860

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 62.4 pcf
Direction: Normal

Coordinates

	X (ft)	Y (ft)
	140	900
	150	900

Surcharge Load 2

Surcharge (Unit Weight): 31.2 pcf
Direction: Normal

Coordinates

	X (ft)	Y (ft)
	150	900
	190	900

Regions

	Material	Points	Area (ft^2)
Region 1	Sand with Gravel (SP)	2,6,9,12,11,16,3,4,5	34200
Region 2	Loess (Fill)	6,7,14,8,9	800
Region 3	Glacial Till (Fill)	8,9,12,10	1888
Region 4	Loess (Fill)	11,13,10,12	1472
Region 5	concrete	13,15,16,11	256

Points

	X (ft)	Y (ft)
Point 1	16	5591
Point 2	0	860
Point 3	380	860
Point 4	380	770
Point 5	0	770
Point 6	44	860
Point 7	122	886
Point 8	141	892
Point 9	93	860
Point 10	152	892
Point 11	262	860
Point 12	200	860
Point 13	182	892
Point 14	140	892
Point 15	190	892
Point 16	270	860

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	501	1.817	(249.216, 944.086)	95.154	(169.584, 892)	(293.756, 860)

Slices of Slip Surface: 501

	Slip	X (ft)	Y (ft)	PWP (psf)	Base Normal	Frictional	Cohesive

	Surface				Stress (psf)	Strength (psf)	Strength (psf)
1	501	171.6534	889.082	-695.7943	-70.648679	0	1000
2	501	175.79205	883.64995	-422.66258	487.98525	0	1000
3	501	179.9307	878.93525	-194.28914	992.4504	0	1000
4	501	183.00865	875.75725	-44.942044	1355.8086	0	1000
5	501	187.00865	872.1916	113.9305	1815.1018	0	1000
6	501	191.83455	868.2158	285.26066	2036.6911	0	1000
7	501	195.5037	865.5732	391.80304	2211.6695	0	1000
8	501	199.17285	863.1833	482.57604	2360.4968	0	1000
9	501	202.84195	861.02425	558.93773	2485.3311	0	1000
10	501	206.75005	858.96555	625.224	2587.5505	1132.9497	0
11	501	210.8971	857.0185	680.76697	2702.5422	1167.2725	0
12	501	215.04415	855.30775	721.54502	2790.9432	1194.7676	0
13	501	219.1912	853.8199	748.435	2853.9376	1215.6125	0
14	501	223.33825	852.54405	762.09442	2891.9352	1229.6641	0
15	501	227.4853	851.47135	763.06183	2904.6776	1236.4624	0
16	501	231.63235	850.5948	751.7956	2892.5788	1235.9818	0
17	501	235.7794	849.909	728.61406	2855.302	1227.8439	0
18	501	239.92645	849.4097	693.81382	2792.678	1211.7798	0
19	501	244	849.0964	680.37502	2704.1762	1168.4421	0
20	501	248	848.96105	688.84369	2588.7884	1096.9336	0
21	501	252	848.99405	686.78071	2445.4271	1015.355	0
22	501	256	849.19555	674.20426	2272.6746	922.87726	0
23	501	260	849.5667	651.02647	2068.754	818.52535	0
24	501	264	850.1095	617.16001	1793.913	679.39866	0
25	501	268	850.82695	572.40468	1442.1873	502.16921	0
26	501	271.97965	851.71765	516.82685	1196.5615	392.44495	0
27	501	275.93895	852.78495	450.20936	1066.6323	355.89197	0
28	501	279.89825	854.03895	371.97251	903.47793	306.8648	0
29	501	283.85755	855.4875	281.59174	702.76793	243.16619	0
30	501	287.81685	857.1404	178.43618	458.89063	161.92045	0
31	501	291.77615	859.00965	61.796703	164.34246	59.204819	0

Section 1

File Information

Title:
Comments:
Created By: Karels, Lucas
Revision Number: 78
Last Edited By: Karels, Lucas
Date: 12/7/2011
Time: 1:17:49 PM
File Name: Aux_Emb_Slope_Drained.gsz
Directory: \\mnp-fs1\Mnp-Projects-1\23601\Active\07-Design\02-Comps\GEO\Slope_Stability\
Last Solved Date: 12/7/2011
Last Solved Time: 1:17:52 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Section 1

Description: Cut bank along Mill Creek (1)
Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

Slip Surface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Entry and Exit
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No
- Tension Crack

Tension Crack Option: (none)
FOS Distribution
FOS Calculation Option: Constant
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Glacial Till (Fill)

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 0 psf
Phi: 28 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Loess (Fill)

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 0 psf
Phi: 28 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Sand with Gravel (SP)

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 30 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (156.65428, 892) ft
Left-Zone Right Coordinate: (190, 892) ft
Left-Zone Increment: 10
Right Projection: Range
Right-Zone Left Coordinate: (286.15634, 860) ft
Right-Zone Right Coordinate: (362.08136, 860) ft
Right-Zone Increment: 10
Radius Increments: 10

Slip Surface Limits

Left Coordinate: (0, 860) ft
Right Coordinate: (380, 860) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	0	900
	140	900
	140	886
	266	860
	380	860

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 62.4 pcf
Direction: Normal

Coordinates

	X (ft)	Y (ft)
	140	900
	150	900

Surcharge Load 2

Surcharge (Unit Weight): 31.2 pcf
Direction: Normal

Coordinates

	X (ft)	Y (ft)
	150	900
	190	900

Regions

	Material	Points	Area (ft²)
Region 1	Sand with Gravel (SP)	2,6,9,12,11,3,4,5	34200
Region 2	Loess (Fill)	6,7,14,8,9	800
Region 3	Glacial Till (Fill)	8,9,12,10	1888
Region 4	Loess (Fill)	11,13,10,12	1984

Points

	X (ft)	Y (ft)
Point 1	16	5591
Point 2	0	860
Point 3	380	860
Point 4	380	770
Point 5	0	770
Point 6	44	860
Point 7	122	886
Point 8	141	892
Point 9	93	860
Point 10	152	892
Point 11	286	860
Point 12	200	860
Point 13	190	892
Point 14	140	892

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	995	1.607	(265.335, 960.815)	107.052	(183.331, 892)	(301.341, 860)

Slices of Slip Surface: 995

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
1	995	184.99815	890.1057	-835.59885	284.12699	151.073	0
2	995	188.3327	886.4842	-652.55651	552.74959	293.90217	0
3	995	191.8223	883.03475	-482.23519	615.98003	327.52239	0
4	995	195.46685	879.74205	-323.70249	796.91005	423.72459	0
5	995	199.1114	876.73635	-183.07731	964.65046	512.91375	0

6	995	202.756	873.9877	-58.490831	1119.1321	595.05311	0
7	995	206.6408	871.32165	57.850784	1273.9011	646.58542	0
8	995	210.7658	868.74655	165.42405	1424.5075	669.46654	0
9	995	214.8908	866.42185	257.3605	1554.2333	689.55951	0
10	995	219.0158	864.3295	334.8107	1663.7819	706.62651	0
11	995	223.14085	862.45465	398.70434	1753.5996	720.41056	0
12	995	227.2659	860.78495	449.7639	1824.0689	730.73094	0
13	995	231.3657	859.31815	488.50648	1882.8562	805.02818	0
14	995	235.4403	858.0437	515.56413	1934.367	819.14623	0
15	995	239.51495	856.94465	531.67419	1965.7393	827.95785	0
16	995	243.5896	856.01545	537.19734	1976.7084	831.10209	0
17	995	247.6642	855.2517	532.39694	1966.8793	828.19879	0
18	995	251.7388	854.6498	517.49056	1935.7575	818.83677	0
19	995	255.81345	854.20695	492.66129	1882.6437	802.50675	0
20	995	259.8881	853.92115	458.02481	1806.731	778.6759	0
21	995	263.9627	853.7912	413.67287	1706.962	746.6808	0
22	995	268	853.8149	385.95526	1580.0848	689.431	0
23	995	272	853.9895	375.04572	1423.5818	605.37261	0
24	995	276	854.3146	354.7755	1237.4108	509.58972	0
25	995	280	854.7915	325.0057	1019.0974	400.73404	0
26	995	284	855.4223	285.65485	765.68498	277.14553	0
27	995	287.91765	856.19045	237.71677	573.24014	193.71451	0
28	995	291.753	857.0928	181.4096	449.08969	154.54518	0
29	995	295.58835	858.1463	115.66929	294.51913	103.25901	0
30	995	299.42365	859.3557	40.204892	105.53074	37.715894	0

Section 1

File Information

Title:
Comments:
Created By: Karels, Lucas
Revision Number: 85
Last Edited By: Karels, Lucas
Date: 12/7/2011
Time: 1:30:20 PM
File Name: Aux_Emb_Slope_Drained_RCC.gsz
Directory: \\mnp-fs1\Mnp-Projects-1\23601\Active\07-Design\02-Comps\GEO\Slope_Stability\
Last Solved Date: 12/7/2011
Last Solved Time: 1:30:24 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Section 1

Description: Cut bank along Mill Creek (1)
Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

Slip Surface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Entry and Exit
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No
- Tension Crack

Tension Crack Option: (none)
FOS Distribution
FOS Calculation Option: Constant
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Glacial Till (Fill)

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 0 psf
Phi: 28 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Loess (Fill)

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 0 psf
Phi: 28 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Sand with Gravel (SP)

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 30 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

concrete

Model: Mohr-Coulomb

Unit Weight: 150 pcf
Cohesion: 5000 psf
Phi: 0 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (156.65428, 892) ft
Left-Zone Right Coordinate: (182, 892) ft
Left-Zone Increment: 10
Right Projection: Range
Right-Zone Left Coordinate: (270, 860) ft
Right-Zone Right Coordinate: (362.08281, 860) ft
Right-Zone Increment: 10
Radius Increments: 10

Slip Surface Limits

Left Coordinate: (0, 860) ft
Right Coordinate: (380, 860) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	0	900
	140	900
	140	886
	242	860
	380	860

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 62.4 pcf
Direction: Normal

Coordinates

	X (ft)	Y (ft)
	140	900
	150	900

Surcharge Load 2

Surcharge (Unit Weight): 31.2 pcf
Direction: Normal

Coordinates

	X (ft)	Y (ft)
	150	900
	190	900

Regions

	Material	Points	Area (ft^2)
Region 1	Sand with Gravel (SP)	2,6,9,12,11,16,3,4,5	34200
Region 2	Loess (Fill)	6,7,14,8,9	800
Region 3	Glacial Till (Fill)	8,9,12,10	1888
Region 4	Loess (Fill)	11,13,10,12	1472
Region 5	concrete	13,15,16,11	256

Points

	X (ft)	Y (ft)
Point 1	16	5591
Point 2	0	860
Point 3	380	860
Point 4	380	770
Point 5	0	770
Point 6	44	860
Point 7	122	886
Point 8	141	892
Point 9	93	860
Point 10	152	892
Point 11	262	860
Point 12	200	860
Point 13	182	892
Point 14	140	892
Point 15	190	892
Point 16	270	860

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	1236	1.498	(266.928, 981.486)	123.371	(182, 892)	(288.417, 860)

Slices of Slip Surface: 1236

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
1	1236	184	890.18345	-960.90161	315.71107	167.86655	0
2	1236	188	886.70145	-807.24191	631.45802	335.75218	0
3	1236	191.73335	883.70305	-679.52902	677.10515	360.0232	0
4	1236	195.2	881.1312	-574.18961	790.32915	420.22546	0
5	1236	198.66665	878.7407	-480.17503	893.45953	475.06086	0
6	1236	202.13335	876.51915	-396.6901	986.36192	524.45794	0
7	1236	205.6	874.456	-323.06757	1068.8911	568.3395	0
8	1236	209.06665	872.5422	-258.80963	1140.9125	606.63395	0
9	1236	212.53335	870.77	-203.35211	1202.2639	639.25508	0
10	1236	216	869.1327	-156.32693	1252.8014	666.1263	0
11	1236	219.46665	867.62445	-117.35233	1292.3001	687.12815	0
12	1236	222.93335	866.2402	-86.118893	1320.5652	702.15695	0
13	1236	226.4	864.97555	-62.344449	1337.342	711.07733	0
14	1236	229.86665	863.82665	-45.790685	1342.3277	713.72828	0
15	1236	233.33335	862.7901	-36.249111	1335.2218	709.95005	0
16	1236	236.8	861.86295	-33.53633	1315.6625	699.55016	0
17	1236	240.26665	861.0427	-37.492392	1283.2244	682.30254	0
18	1236	243.72005	860.3295	-20.559823	1237.6207	658.05457	0
19	1236	247.09605	859.73025	16.831678	1189.9976	677.32767	0
20	1236	250.408	859.23675	47.62885	1125.3131	622.20131	0
21	1236	253.72	858.83465	72.719915	1045.2287	561.47821	0
22	1236	257.032	858.523	92.163083	949.1988	494.8098	0
23	1236	260.344	858.3012	106.00549	836.49032	421.74561	0
24	1236	264	858.1652	114.49021	654.6154	311.84142	0
25	1236	268	858.1351	116.37061	394.33511	160.48288	0
26	1236	271.84165	858.2259	110.70365	246.49249	78.397726	0
27	1236	275.52495	858.42795	98.09633	223.08655	72.163135	0
28	1236	279.20825	858.74085	78.572007	182.65932	60.094837	0
29	1236	282.89155	859.16545	52.076351	123.87786	41.454619	0
30	1236	286.5749	859.7029	18.539321	45.176343	15.378892	0

Seepage Analysis

Lake Delhi Dam Reconstruction
Seepage Analysis

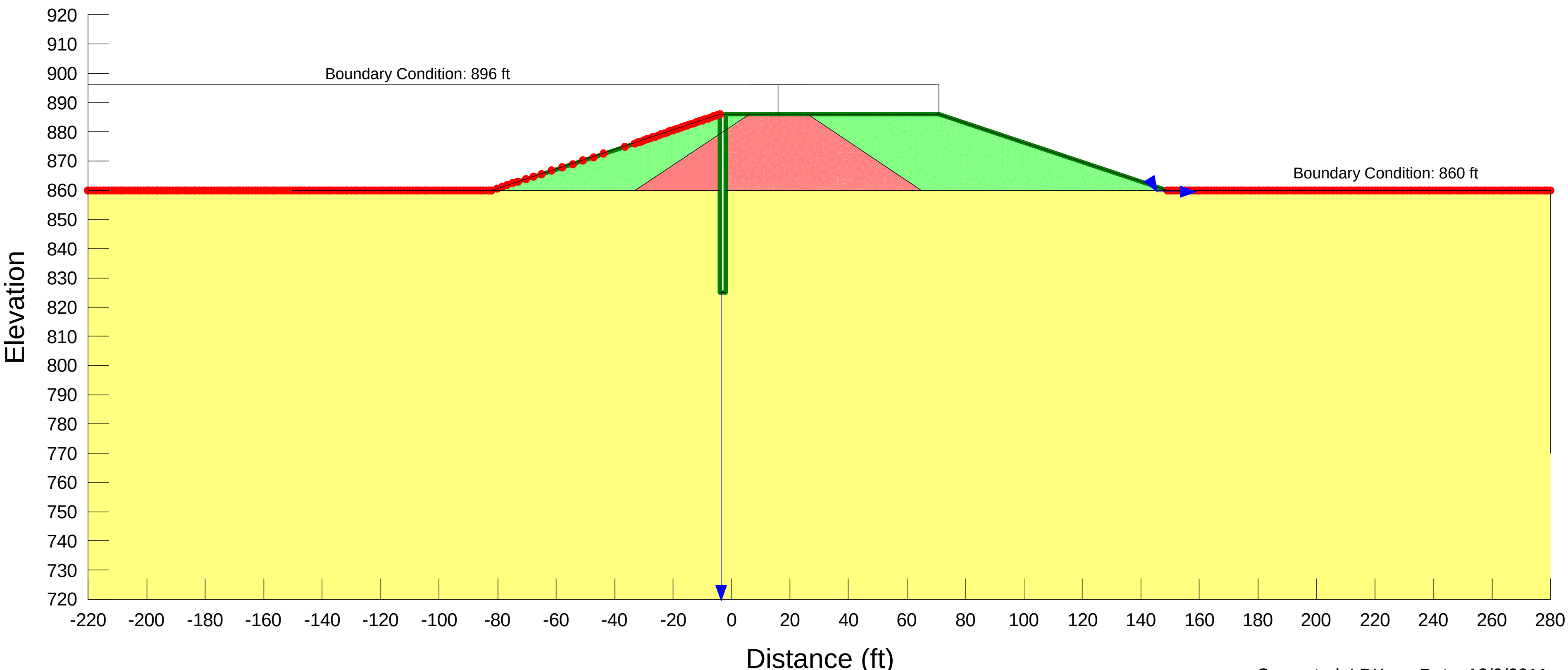
Material Parameters

Assume kv = 0.33*kh

Poorly Graded Sand with Gravel (SP): k = 1.1 x 10^-2 cm/s = 3.61 x 10^-4 ft/s

Non-Levee Fill (Loess): k = 1.0 x 10^-6 cm/s = 3.28 x 10^-8 ft/s

Levee Fill (Till): k = 1.0 x 10^-7 cm/s = 3.28 x 10^-9 ft/s



Computed: LDK Date: 12/9/2011
Checked: JRJ Date: 12/9/2011

Lake Delhi Dam Reconstruction
Seepage Analysis - Total Head

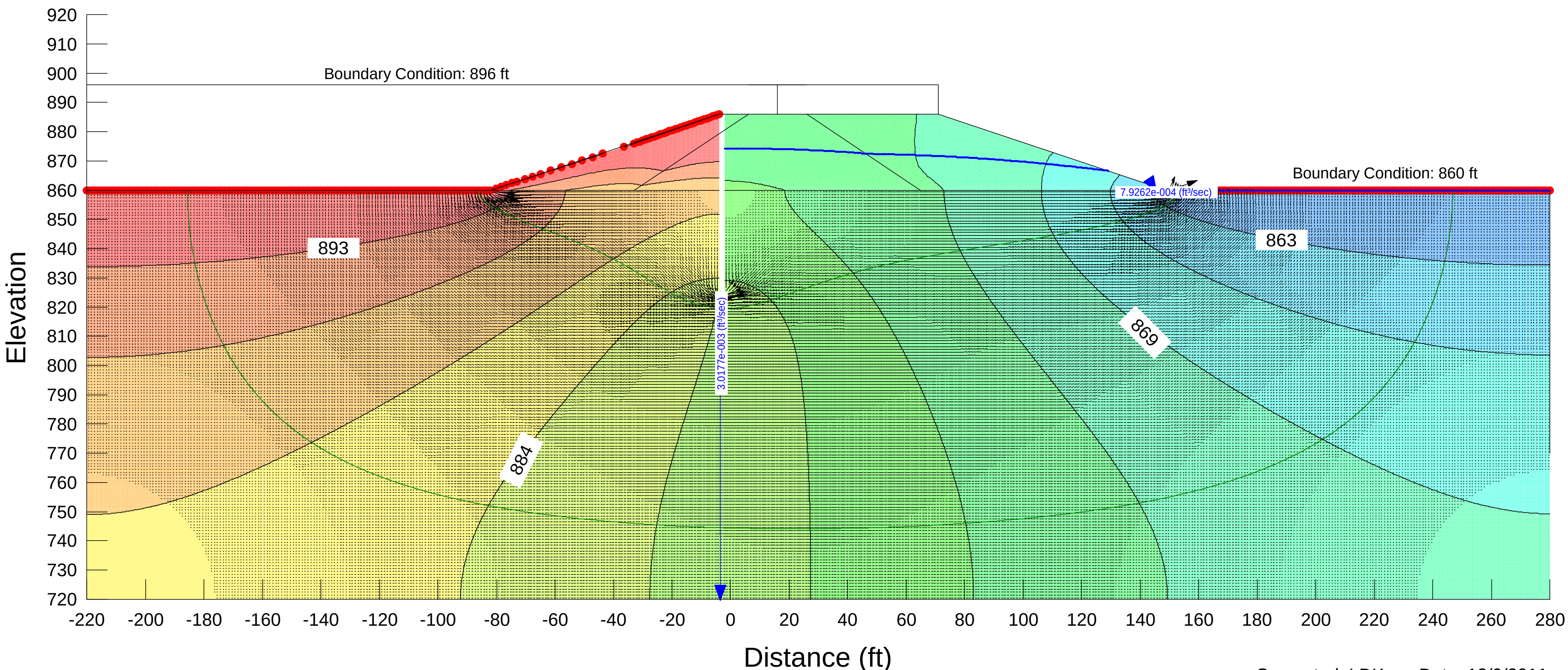
Material Parameters

Assume kv = 0.33*kh

Poorly Graded Sand with Gravel (SP): k = 1.1 x 10^-2 cm/s = 3.61 x 10^-4 ft/s

Non-Levee Fill (Loess): k = 1.0 x 10^-6 cm/s = 3.28 x 10^-8 ft/s

Levee Fill (Till): k = 1.0 x 10^-7 cm/s = 3.28 x 10^-9 ft/s



Computed: LDK Date: 12/9/2011
Checked: JRJ Date: 12/9/2011

Lake Delhi Dam Reconstruction
Seepage Analysis - Exit Gradient

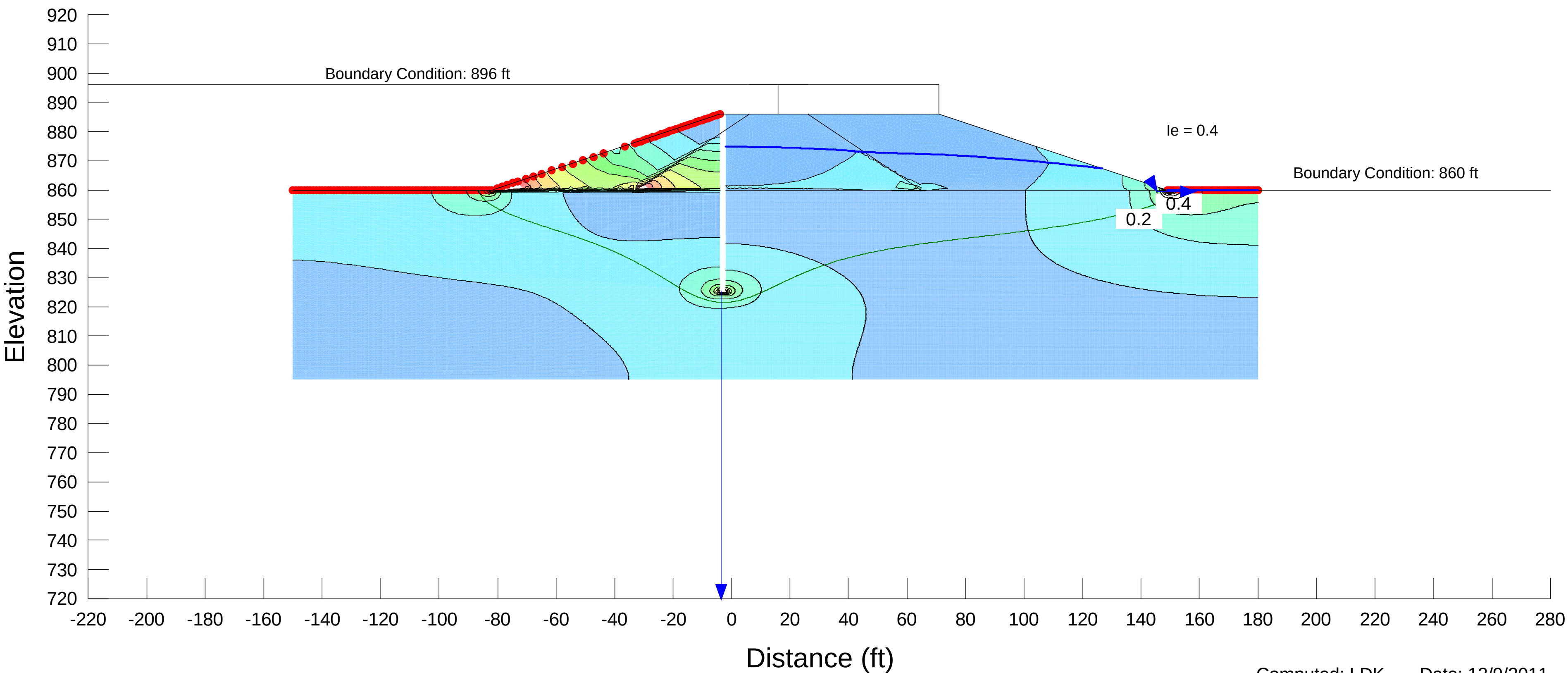
Material Parameters

Assume kv = 0.33*kh

Poorly Graded Sand with Gravel (SP): k = 1.1 x 10^-2 cm/s = 3.61 x 10^-4 ft/s

Non-Levee Fill (Loess): k = 1.0 x 10^-6 cm/s = 3.28 x 10^-8 ft/s

Levee Fill (Till): k = 1.0 x 10^-7 cm/s = 3.28 x 10^-9 ft/s



Computed: LDK Date: 12/9/2011
Checked: JRJ Date: 12/9/2011

North Embankment Sheet Pile Design Alternatives

Stanley Consultants INC.

Computed by	JRS	Date	12-3-11
Checked by	LDK	Date	12/21/11
Approved by		Date	

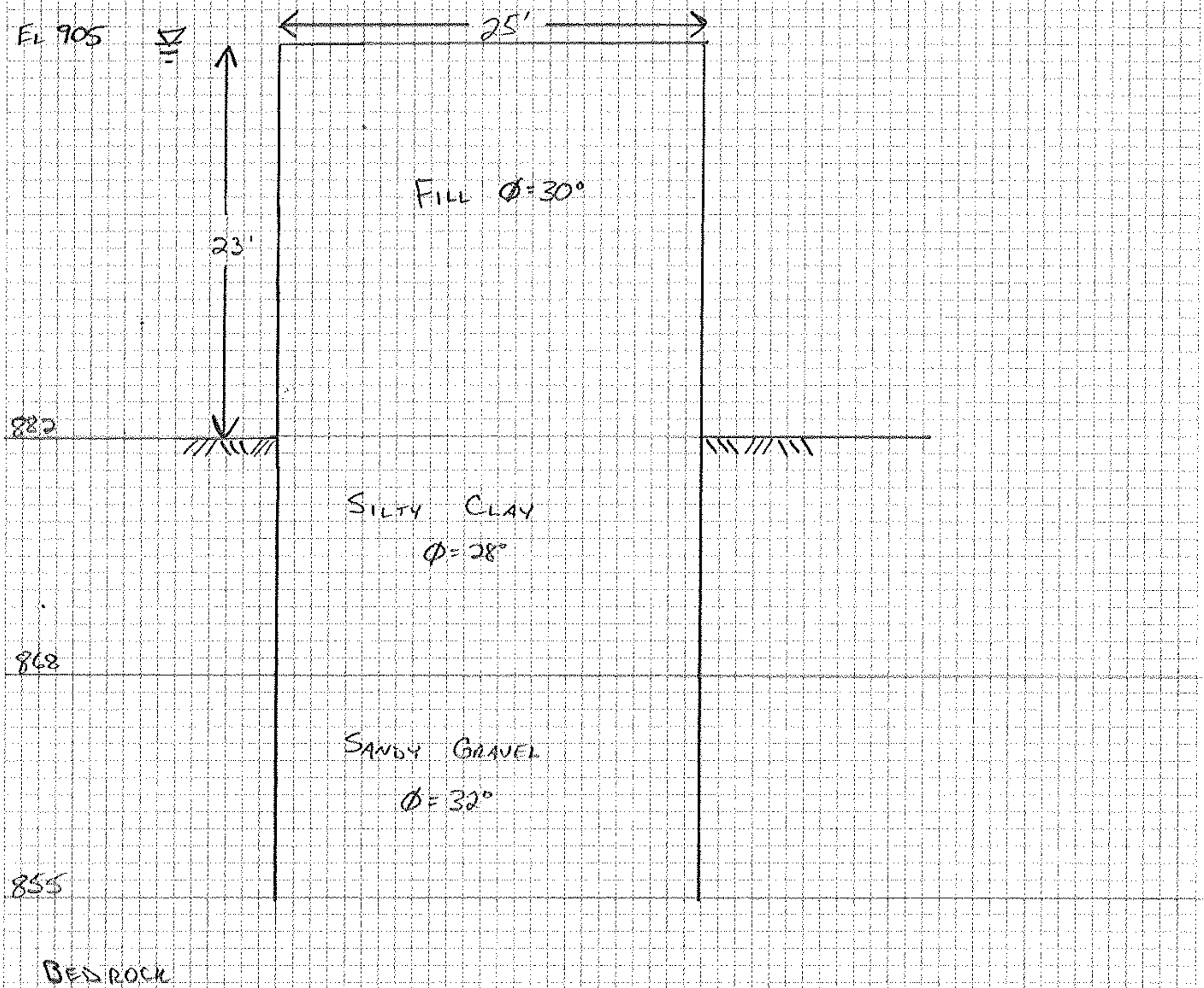
Project No. Page No.

Subject LAKE DELHI

N. EMBANKMENT

Sheet No. of

NORTH EMBANKMENT



Stanley Consultants INC.

Project No. Page No.

Subject LAKE DELHI

N. EMBANKMENT

Computed by JRJ Date 12-3-11

Checked by LDK Date 12/21/11

Approved by Date

Sheet No. of

SHEET PILE TO 853 for ANCHOR AT EL 900

$M = 1.04 \times 10^5$ lb-ft max

$\sigma = 2.52 \times 10^{10}$ LB-IN^3

SHEET PILE TO 855 for ANCHOR AT EL 895

$M = 6.9 \times 10^4$ lb-ft max

$\sigma = 1.2 \times 10^{10}$ LB-IN^3

Anchor force = 1.3×10^4 LB

★ SHEET PILE TO 859 For ANCHOR AT EL 890 → Assume drive to bedrock @ 855.

$M = 3.2 \times 10^4$ LB-FT max

$\sigma = 2.6 \times 10^9$ LB-IN^3

Achor force = 1.6×10^4 LB → Assume 2 rows of anchors

PZC 18 SHEET PILE OK ✓

CWALSHT "5a" File

SC 3020 R2 199

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Computed By:	JRJ	**Date:**	12/3/2011	**Job No.**	23601.01.00 Lake Delhi Dam
Checked By:	LDK	**Date:**	12/21/2011	**Subject**	Design of Cellular Structure - North Abutment
Approved By:		**Date:**		**Sheet No.**	1 of 3

Diagrams and Variables, Sliding Stability: ***Notes:***

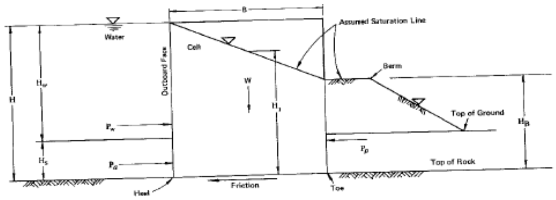


Fig. 69

Driving Forces:

Full water pressure, $P_W = \gamma_W H^2/2$, per foot of wall

Active earth pressure, $P_a = \gamma' K_a H_S^2/2$, per foot of wall

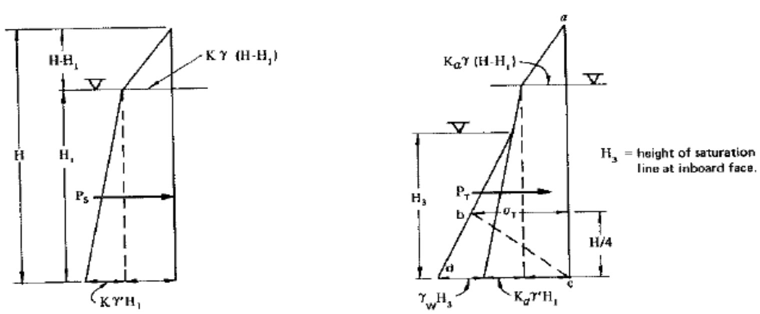
where K_a = active earth pressure coefficient
γ' = submerged unit weight of soil on the outboard side of the cofferdam
γ_W = unit weight of water = 62.4 pounds per cubic foot
H and H_S = height of cofferdam and soil, respectively
H_B = height from toe of cofferdam to top of berm

Resisting Forces:

Friction force along bottom of the cell, W tanδ';

$W \tan\delta' = B[\gamma(H-H_1) + \gamma' H_1] \tan\delta'$

where W = effective weight of cell fill
B = equivalent width of cofferdam
γ = unit weight of cell fill above saturation line
H = total height of cofferdam
H_1 = average height of saturation line
γ' = submerged unit weight of cell fill
tan δ' = coefficient of friction of cell fill on rock, for smooth rock = 0.5
δ' = ϕ-angle of the soil for other types



Vertical Plane On Centerline of Cofferdam (a)

Inboard Sheeting (b)

System:		
Top El	905	
Base El	882	
B	25	
Wet Water	905	High Water Case
Dry Water	882	
Wet Soil	882	
Dry Soil	882	
Soil:		
Unit Weight	120	
Friction Ang.	30	Fill
Friction Ang.	28	Foundation
Water:		
Unit Weight	62.5	
		Radians(45-phi/2)
Ka	0.333333	0.523598776
		Radians(45+phi/2)
Kp	3	1.047197551
γ'	57.5	
H	23	
Hs	0	
Hb	0	
H1	14.66667	(assume 1.5 : 1 sat line)
H3	6.333333	
Tan(δ')	0.531709	= tan(34)
W	46.08333	k
Friction	24.50294	=W*tan(δ')
Pw(drive)	16.53125	k
Pa	0	k
Pw(resist)	0	
Pp	0	
FOS	**1.48222**	

Overturning about Toe: ***Notes:***

Driving Moment: 126.7395833

Resist Moment: 576.0416667

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Checked By:	LDK	**Date:**	12/21/2011	**Subject**	Design of Cellular Structure - North Abutment
Approved By:		**Date:**		**Sheet No.**	2 of 3

Shear Failure on Centerline of Cell (Vertical Shear): ***Notes:***

Q= 3M / 2B where M = net overturning moment about the base

M= 126.7395833 k-ft/ft

Q= 7.604375 k/ft

K= cos(phi)^2 / (2 - cos(phi)^2)
= 0.6

Ps= ((1/2)*K*unit weight*(H-H1)^2 + K*unitweight*(H-H1)*H1 + (1/2)*K*unitweighteff*H1^2)/1000
= 15.01066667

S= Ps*tan(phi)
= 8.666412441

f= steel on steel coef of friction = 0.3

From Figure:
Pt= (1/2)(Ka)(Unitweight)(H-H1)^2 + (Ka)(Unitweight)(H-H1)(H1) + (1/2)(ka)(Unitweighteff)(H1)^2 +
(1/2)(Unitweightwater)(H3)^2 - (1/2)(H/4)[(Unitweightwater)(H3)+(Ka)(Unitweighteff)(H1)+(Ka)(Unitweight)(H-H1)]

Pt= 6.68818287
St= S+f*Pt
= 10.6728673

FOS= St / Q
= **1.4035167**

Horizontal Shear (Cummings' Methods): ***Notes:***

Mr= (a*c^2*(unitweight_weighted)/2 + c^3*(unitweight_weighted)/3)/1000

Unitweight_weighted =
76.9 pcf

Note: Weighted unit weight computed based on percentages of cofferdam above A-T line that are saturated and unsaturated.

c= B*tan(phi)
= 14.43375673

a= H-c
= 8.56624327

Mr= 145.6994482

Mi= Pt*f*B
= 50.16137153

FOS= $\frac{\text{Mr + Mi + Pp*Hb/3}}{\text{(1/3)*(PwHw + PaHs)}}$

FOS= **1.545380019**

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Checked By:	LDK	**Date:**	12/21/2011	**Subject**	Design of Cellular Structure - North Abutment
Approved By:		**Date:**		**Sheet No.**	3 of 3

Interlock Tension: ***Notes:***

t= sigma(t)*R/12 lbs per linear inch
sigma(t) in lbs/ft, R in ft

R= radius= 13

sigma(t)= (Ka(unitweight)(H-H1) + Ka(unitweighteff)*(H1-H/4) + unitweightwater*(H3-H/4)), use K rather than Ka
= 944.0833333 lbs/ft

t= 1022.756944 lbs/in

tmax= 8000 lbs/in

Inboard Sheeting Penetration: ***Notes:***

F1= Pt*tan(d) where tan(d)= 0.3 steel on soil coef of friction
= 2.006454861 k/ft

FOS= Fr / F1 where Fr= (Pl+Pr)*tan(d) D= 14 ft

Pl= 24.045
Pr= 3.92

Fr= 8.3895

FOS= **4.181255289**

Outboard Sheeting Pull-out: ***Notes:***

FOS= Qu/Qp

Qu= (1/2)*Ka*unitweighteff*D^2*tan(d)*perimeter where D=embedment depth D= 27 ft
where perimeter = "2" sides of sheet pile

Qu= 4.19175

Qp= M / B*(1+B/4L) where L= 13
Qp= 3.423614719

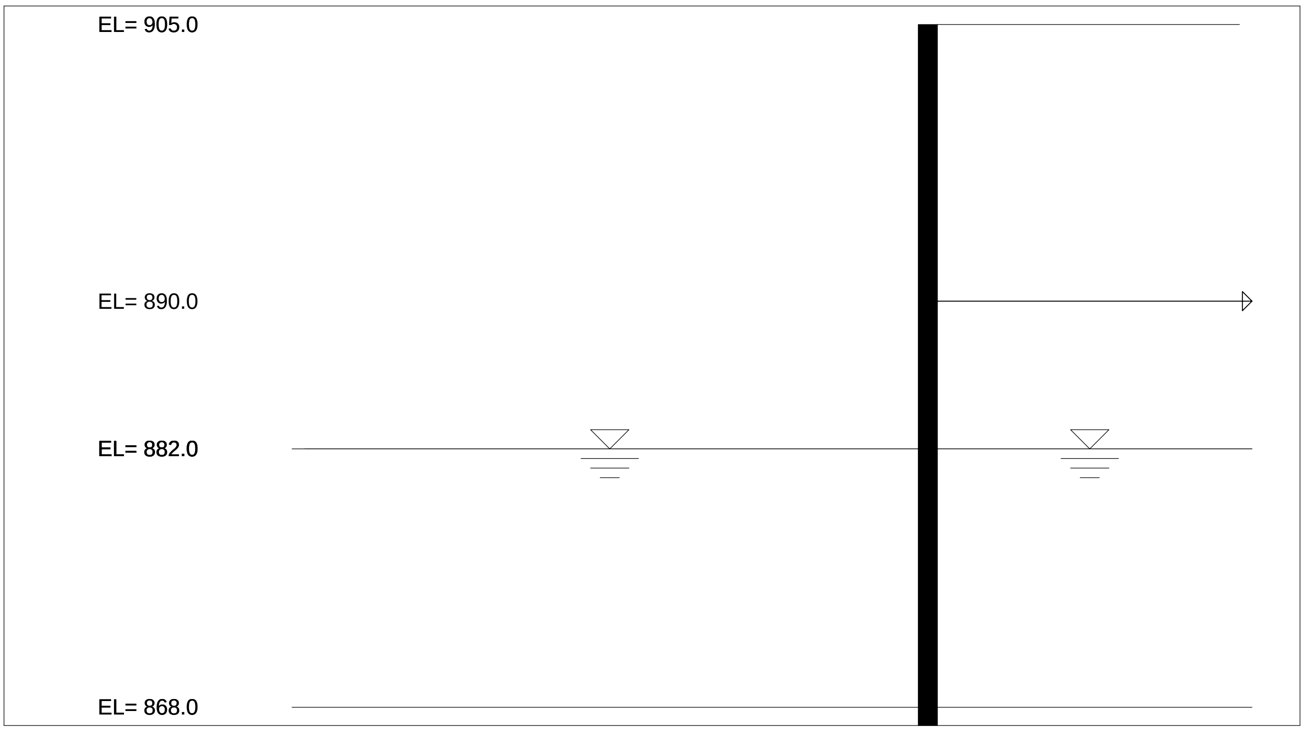
FOS= **1.224363821**

Bearing Capacity: ***Notes:***

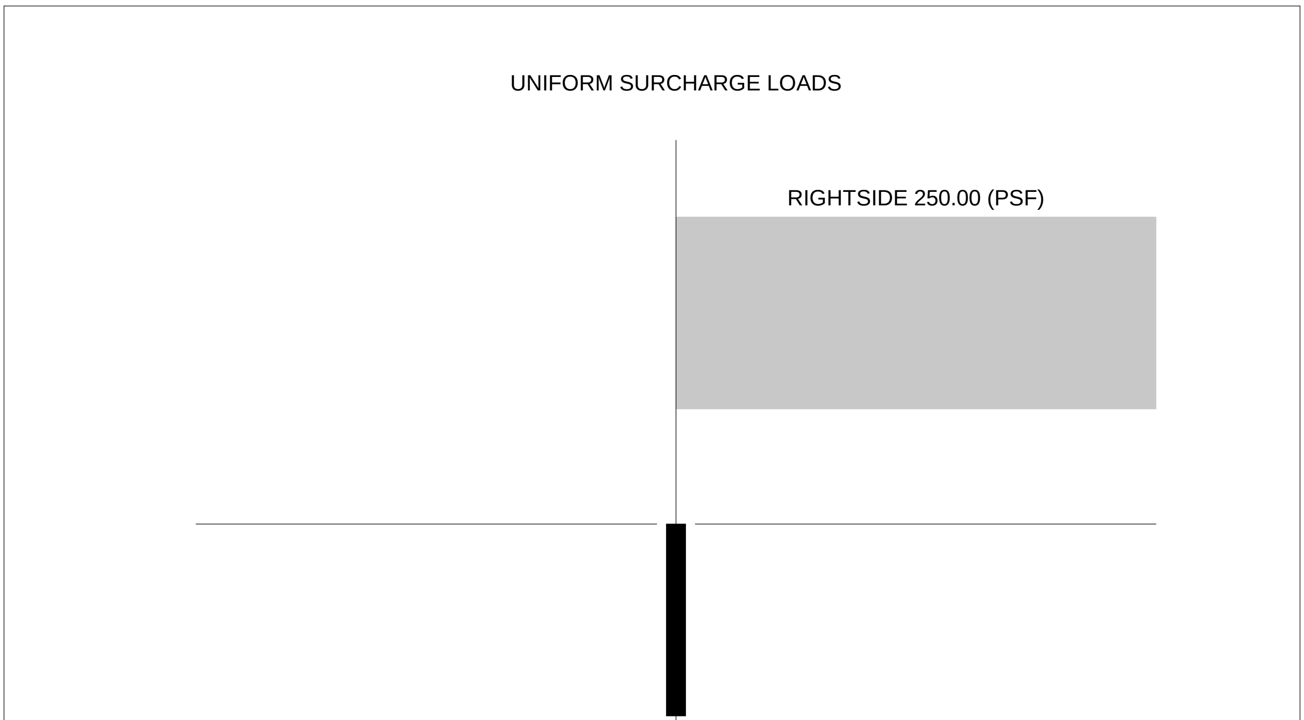
FOS= (1/2)*(unitweighteff)*B*Ng / ((6*M)/B^2 + (unitweight)*H) Ng= 30 for phi equals 32 degrees

= **5.42220937**

'LAKE DELHI DAM - NORTH ABUTMENT



'LAKE DELHI DAM - NORTH ABUTMENT



```
'Lake Delhi Dam - North Abutment
CONTROL  ANCHORED  DESIGN  1.00  1.50
WALL   905    890
SURFACE RIGHTSIDE   2    0    905
                  25    905
SURFACE LEFTSIDE   2    0    882
                  50    882
SOIL RIGHTSIDE  STRENGTHS   3    1   1.5
   120    120    30    0    0    0    882   0
   120    120    28    0    0    0    868   0
   125    125    32    0    0    0
SOIL LEFTSIDE  STRENGTHS   2    1   1.5
   120    120    28    0    0    0    868   0
   125    125    32    0    0    0
WATER ELEVATIONS   62.4    882    882
VERTICAL UNIFORM   250    0
FINISHED
```

```
PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                          BY CLASSICAL METHODS
DATE: 2-DECEMBER-2011                                       TIME: 17:02:12

                            ****************
                            *  INPUT DATA  *
                            ****************

     I.--HEADING
    'Lake Delhi Dam - North Abutment

    II.--CONTROL
       ANCHORED WALL DESIGN
       FACTOR OF SAFETY FOR ACTIVE PRESSURES  = 1.00
       FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

   III.--WALL DATA
       ELEVATION AT TOP OF WALL     = 905.00 FT.
       ELEVATION AT ANCHOR          = 890.00 FT.

    IV.--SURFACE POINT DATA

         IV.A.--RIGHTSIDE
            DIST. FROM     ELEVATION
            WALL (FT)         (FT)
                 0.00        905.00
                25.00        905.00

         IV.B.--LEFTSIDE
            DIST. FROM     ELEVATION
            WALL (FT)         (FT)
                 0.00        882.00
                50.00        882.00

     V.--SOIL LAYER DATA

        V.A.--RIGHTSIDE
            LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = 1.00
            LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = 1.50

              ANGLE OF          ANGLE OF                       <-SAFETY->
 SAT.   MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM--> <-FACTOR->
 WGHT.  WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE ACT. PASS.
 (PCF)  (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
120.00 120.00    30.00    0.00     0.00    0.00  882.00   0.00  DEF  DEF
120.00 120.00    28.00    0.00     0.00    0.00  868.00   0.00  DEF  DEF
125.00 125.00    32.00    0.00     0.00    0.00                 DEF  DEF

        V.B.--LEFTSIDE
            LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = 1.00
            LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = 1.50

              ANGLE OF          ANGLE OF                       <-SAFETY->
 SAT.   MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM--> <-FACTOR->
 WGHT.  WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE ACT. PASS.
 (PCF)  (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
120.00 120.00    28.00    0.00     0.00    0.00  868.00   0.00  DEF  DEF
125.00 125.00    32.00    0.00     0.00    0.00                 DEF  DEF

    VI.--WATER DATA
       UNIT WEIGHT         = 62.40 (PCF)
       RIGHTSIDE ELEVATION = 882.00 (FT)
       LEFTSIDE ELEVATION  = 882.00 (FT)
       NO SEEPAGE

   VII.--VERTICAL SURCHARGE LOADS

       VII.A.--VERTICAL LINE LOADS
            NONE

       VII.B.--VERTICAL UNIFORM LOADS
                 LEFTSIDE     RIGHTSIDE
                   (PSF)        (PSF)
                    0.00       250.00

       VII.C.--VERTICAL STRIP LOADS
            NONE

       VII.D.--VERTICAL RAMP LOADS
```

NONE

VII.E.--VERTICAL TRIANGULAR LOADS
NONE

VII.F.--VERTICAL VARIABLE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 2-DECEMBER-2011 TIME: 17:02:15

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*    SOIL PRESSURES FOR   *
*  ANCHORED WALL DESIGN   *
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I.--HEADING
'Lake Delhi Dam - North Abutment

II.--SOIL PRESSURES

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

	NET	<---LEFTSIDE--->		<------NET------> (SOIL + WATER)	<--RIGHTSIDE--->	
ELEV.	WATER	PASSIVE	ACTIVE	ACTIVE	ACTIVE	PASSIVE
(FT)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)
905.0	0.0	0.0	0.0	83.3	83.3	530.3
904.0	0.0	0.0	0.0	123.2	123.2	784.9
903.0	0.0	0.0	0.0	163.2	163.2	1039.5
902.0	0.0	0.0	0.0	203.2	203.2	1294.1
901.0	0.0	0.0	0.0	243.1	243.1	1548.7
900.0	0.0	0.0	0.0	283.1	283.1	1803.2
899.0	0.0	0.0	0.0	323.1	323.1	2057.8
898.0	0.0	0.0	0.0	363.0	363.0	2312.4
897.0	0.0	0.0	0.0	403.0	403.0	2566.9
896.0	0.0	0.0	0.0	443.0	443.0	2821.5
895.0	0.0	0.0	0.0	482.9	482.9	3076.1
894.0	0.0	0.0	0.0	522.9	522.9	3330.7
893.0	0.0	0.0	0.0	562.9	562.9	3585.2
892.0	0.0	0.0	0.0	602.8	602.8	3839.8
891.0	0.0	0.0	0.0	642.8	642.8	4094.4
890.0	0.0	0.0	0.0	682.8	682.8	4349.0
889.0	0.0	0.0	0.0	722.7	722.7	4603.5
888.0	0.0	0.0	0.0	762.7	762.7	4858.1
887.0	0.0	0.0	0.0	802.7	802.7	5112.7
886.0	0.0	0.0	0.0	842.6	842.6	5367.3
885.0	0.0	0.0	0.0	882.6	882.6	5621.8
884.0	0.0	0.0	0.0	922.6	922.6	5876.4
883.0	0.0	0.0	0.0	962.6	962.6	6131.0
882.0	0.0	0.0	0.0	1039.8	1039.8	6173.3
881.0	0.0	115.4	20.8	992.1	1107.5	6146.0
880.0	0.0	230.8	41.6	897.5	1128.3	6261.4
879.0	0.0	346.2	62.4	802.9	1149.1	6376.8
878.0	0.0	461.6	83.2	708.3	1169.9	6492.2
877.0	0.0	577.0	104.0	613.7	1190.7	6607.6
876.0	0.0	692.4	124.8	519.1	1211.5	6723.0
875.0	0.0	807.8	145.6	424.5	1232.3	6838.4
874.0	0.0	923.2	166.4	329.9	1253.1	6953.8
873.0	0.0	1038.6	187.2	235.2	1273.9	7069.2
872.0	0.0	1154.0	208.0	140.6	1294.7	7184.6
871.0	0.0	1269.4	228.8	46.0	1315.5	7300.0
870.5	0.0	1325.6	238.9	0.0	1325.6	7356.2
870.0	0.0	1384.8	249.5	-48.6	1336.3	7415.4

869.0	0.0	1500.2	270.3	-143.2	1357.1	7530.8
868.0	0.0	1722.1	268.6	-449.4	1272.7	8126.3
867.0	0.0	1956.8	266.0	-769.2	1187.6	8734.6
866.0	0.0	2097.7	285.2	-891.0	1206.7	8875.5
865.0	0.0	2238.7	304.4	-1012.8	1225.9	9016.5
864.0	0.0	2379.7	323.5	-1134.6	1245.1	9157.4
863.0	0.0	2520.6	343.0	-1256.4	1264.2	9298.4
862.0	0.0	2661.6	362.8	-1378.2	1283.4	9439.4
861.0	0.0	2802.5	382.4	-1500.0	1302.6	9580.3
860.0	0.0	2943.5	401.6	-1621.8	1321.7	9721.3
859.0	0.0	3084.5	420.9	-1743.6	1340.9	9862.2
858.0	0.0	3225.4	440.1	-1865.4	1360.1	10003.2
857.0	0.0	3366.4	459.4	-1984.7	1381.7	10144.2
856.0	0.0	3507.3	478.6	-2104.0	1403.4	10285.1
855.0	0.0	3647.9	497.8	-2225.2	1422.7	10426.1
854.0	0.0	3787.6	517.1	-2345.7	1441.9	10567.0
853.0	0.0	3927.6	536.3	-2466.4	1461.1	10708.0
852.0	0.0	4068.4	555.5	-2588.0	1480.4	10849.0
851.0	0.0	4209.3	574.8	-2709.7	1499.6	10989.9
850.0	0.0	4350.1	594.0	-2831.3	1518.8	11130.9
849.0	0.0	4491.0	613.2	-2952.9	1538.1	11271.8
848.0	0.0	4631.9	632.5	-3074.6	1557.3	11412.8
847.0	0.0	4772.7	651.7	-3196.2	1576.5	11553.8
846.0	0.0	4913.6	670.9	-3317.8	1595.8	11694.7
845.0	0.0	5054.5	690.2	-3439.5	1615.0	11835.7
844.0	0.0	5195.3	709.4	-3561.1	1634.2	11976.6
843.0	0.0	5336.2	728.6	-3682.7	1653.5	12117.6
842.0	0.0	5477.1	747.9	-3804.3	1672.7	12258.6
841.0	0.0	5617.9	767.1	-3926.0	1691.9	12399.5
840.0	0.0	5758.8	786.3	-4047.6	1711.2	12540.5
839.0	0.0	5899.7	805.6	-4169.2	1730.4	12681.4
838.0	0.0	6040.5	824.8	-4290.9	1749.7	12822.4
837.0	0.0	6181.4	844.0	-4412.5	1768.9	12963.4
836.0	0.0	6322.2	863.3	-4534.1	1788.1	13104.3
835.0	0.0	6463.1	882.5	-4655.8	1807.4	13245.3
834.0	0.0	6604.0	901.7	-4777.4	1826.6	13386.2
833.0	0.0	6744.8	921.0	-4899.0	1845.8	13527.2
832.0	0.0	6885.7	940.2	-5020.6	1865.1	13668.2
831.0	0.0	7026.6	959.4	-5142.3	1884.3	13809.1
830.0	0.0	7167.4	978.7	-5263.9	1903.5	13947.5
829.0	0.0	7308.3	997.9	-5385.5	1922.8	14083.7
828.0	0.0	7449.2	1017.1	-5507.2	1942.0	14222.4
827.0	0.0	7590.0	1036.4	-5628.8	1961.2	14363.2
826.0	0.0	7730.9	1055.6	-5750.4	1980.5	14504.1
825.0	0.0	7871.8	1074.9	-5872.1	1999.7	14645.0
824.0	0.0	8012.6	1094.1	-5993.7	2018.9	14785.8
823.0	0.0	8153.5	1113.3	-6115.3	2038.2	14926.7
822.0	0.0	8294.4	1132.6	-6236.9	2057.4	15067.5
821.0	0.0	8435.2	1151.8	-6358.6	2076.6	15208.4
820.0	0.0	8576.1	1171.0	-6480.2	2095.9	15349.3
819.0	0.0	8716.9	1190.3	-6601.8	2115.1	15490.1
818.0	0.0	8857.8	1209.5	-6723.5	2134.3	15631.0
817.0	0.0	8998.7	1228.7	-6845.1	2153.6	15771.9
816.0	0.0	9139.5	1248.0	-6966.7	2172.8	15912.7
815.0	0.0	9280.4	1267.2	-7088.4	2192.0	16053.6
814.0	0.0	9421.3	1286.4	-7210.0	2211.3	16194.5
813.0	0.0	9562.1	1305.7	-7331.6	2230.5	16335.3
812.0	0.0	9703.0	1324.9	-7453.2	2249.7	16476.2

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 2-DECEMBER-2011 TIME: 17:02:15

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*  SUMMARY OF RESULTS FOR  *
*   ANCHORED WALL DESIGN   *
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I.--HEADING
'Lake Delhi Dam - North Abutment

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

METHOD		FREE EARTH	FIXED EARTH
WALL BOTTOM ELEVATION (FT)	:	864.79	859.61
PENETRATION (FT)	:	17.21	22.39
MAXIMUM BENDING MOMENT (LB-FT)	:	3.1850E+04	3.1850E+04
AT ELEVATION (FT)	:	890.00	890.00
MAXIMUM SCALED DEFLECTION (LB-IN^3)	:	2.6340E+09	1.7810E+09
AT ELEVATION (FT)	:	877.00	878.00
ANCHOR FORCE (LB)	:	1.5712E+04	1.5160E+04

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF
ELLASTICITY IN PSI TIMES PILE MOMENT
OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
IN INCHES.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHOREDOR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 2-DECEMBER-2011 TIME: 17:02:15

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* COMPLETE OF RESULTS FOR  *
*   ANCHORED WALL DESIGN   *
*   BY FREE EARTH METHOD   *
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I.--HEADING
'Lake Delhi Dam - North Abutment

II.--RESULTS (ANCHOR FORCE= 15712. (LB))

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
905.00	0.0000E+00	0.	-4.2690E+08	83.33
904.00	4.8317E+01	103.	-4.4780E+08	123.23
903.00	2.1988E+02	247.	-4.6861E+08	163.20
902.00	5.5464E+02	430.	-4.8901E+08	203.17
901.00	1.0926E+03	653.	-5.0842E+08	243.14
900.00	1.8736E+03	916.	-5.2591E+08	283.10
899.00	2.9378E+03	1219.	-5.4012E+08	323.07
898.00	4.3250E+03	1562.	-5.4921E+08	363.04
897.00	6.0753E+03	1945.	-5.5078E+08	403.01
896.00	8.2286E+03	2368.	-5.4178E+08	442.97
895.00	1.0825E+04	2831.	-5.1851E+08	482.94
894.00	1.3904E+04	3334.	-4.7645E+08	522.91
893.00	1.7506E+04	3877.	-4.1030E+08	562.88
892.00	2.1671E+04	4460.	-3.1382E+08	602.84
891.00	2.6439E+04	5083.	-1.7980E+08	642.81
890.00+	3.1850E+04	5745.	0.0000E+00	682.78
890.00-	3.1850E+04	-9967.	0.0000E+00	682.78
889.00	2.2231E+04	-9264.	2.3041E+08	722.75
888.00	1.3335E+04	-8521.	4.9933E+08	762.71
887.00	5.2013E+03	-7739.	7.9141E+08	802.68
886.00	-2.1294E+03	-6916.	1.0926E+09	842.65
885.00	-8.6174E+03	-6053.	1.3902E+09	882.62
884.00	-1.4223E+04	-5151.	1.6731E+09	922.58
883.00	-1.8906E+04	-4208.	1.9315E+09	962.55
882.00	-2.2620E+04	-3207.	2.1574E+09	1039.82
881.00	-2.5315E+04	-2191.	2.3443E+09	992.10
880.00	-2.7026E+04	-1246.	2.4877E+09	897.50
879.00	-2.7839E+04	-396.	2.5844E+09	802.89
878.00	-2.7849E+04	359.	2.6332E+09	708.28
877.00	-2.7152E+04	1020.	2.6340E+09	613.67
876.00	-2.5840E+04	1587.	2.5879E+09	519.07
875.00	-2.4009E+04	2059.	2.4973E+09	424.46

874.00	-2.1754E+04	2436.	2.3652E+09	329.85
873.00	-1.9169E+04	2718.	2.1956E+09	235.24
872.00	-1.6349E+04	2906.	1.9929E+09	140.64
871.00	-1.3388E+04	3000.	1.7619E+09	46.03
870.51	-1.1925E+04	3011.	1.6409E+09	0.00
870.00	-1.0382E+04	2998.	1.5079E+09	-48.58
869.00	-7.4234E+03	2902.	1.2359E+09	-143.19
868.00	-4.6436E+03	2606.	9.5100E+08	-449.40
867.00	-2.3154E+03	1997.	6.5805E+08	-769.21
866.00	-7.2352E+02	1167.	3.6099E+08	-891.00
865.00	-2.2593E+01	215.	6.2546E+07	-1012.80
864.79	0.0000E+00	0.	0.0000E+00	-1038.31

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELLASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

III.--WATER AND SOIL PRESSURES

		<-------------SOIL PRESSURES------------->			
	WATER	<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
905.00	0.	0.	0.	83.	530.
904.00	0.	0.	0.	123.	785.
903.00	0.	0.	0.	163.	1040.
902.00	0.	0.	0.	203.	1294.
901.00	0.	0.	0.	243.	1549.
900.00	0.	0.	0.	283.	1803.
899.00	0.	0.	0.	323.	2058.
898.00	0.	0.	0.	363.	2312.
897.00	0.	0.	0.	403.	2567.
896.00	0.	0.	0.	443.	2822.
895.00	0.	0.	0.	483.	3076.
894.00	0.	0.	0.	523.	3331.
893.00	0.	0.	0.	563.	3585.
892.00	0.	0.	0.	603.	3840.
891.00	0.	0.	0.	643.	4094.
890.00	0.	0.	0.	683.	4349.
889.00	0.	0.	0.	723.	4604.
888.00	0.	0.	0.	763.	4858.
887.00	0.	0.	0.	803.	5113.
886.00	0.	0.	0.	843.	5367.
885.00	0.	0.	0.	883.	5622.
884.00	0.	0.	0.	923.	5876.
883.00	0.	0.	0.	963.	6131.
882.00	0.	0.	0.	1040.	6173.
881.00	0.	115.	21.	1108.	6146.
880.00	0.	231.	42.	1128.	6261.
879.00	0.	346.	62.	1149.	6377.
878.00	0.	462.	83.	1170.	6492.
877.00	0.	577.	104.	1191.	6608.
876.00	0.	692.	125.	1211.	6723.
875.00	0.	808.	146.	1232.	6838.
874.00	0.	923.	166.	1253.	6954.
873.00	0.	1039.	187.	1274.	7069.
872.00	0.	1154.	208.	1295.	7185.
871.00	0.	1269.	229.	1315.	7300.
870.51	0.	1326.	239.	1326.	7356.
870.00	0.	1385.	250.	1336.	7415.
869.00	0.	1500.	270.	1357.	7531.
868.00	0.	1722.	269.	1273.	8126.
867.00	0.	1957.	266.	1188.	8735.
866.00	0.	2098.	285.	1207.	8876.
865.00	0.	2239.	304.	1226.	9016.
864.00	0.	2380.	324.	1245.	9157.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHOREDOR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 2-DECEMBER-2011 TIME: 17:02:15

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* COMPLETE OF RESULTS FOR  *
*   ANCHORED WALL DESIGN   *
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*   BY FIXED EARTH METHOD  *
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I.--HEADING
'Lake Delhi Dam - North Abutment

II.--RESULTS (ANCHOR FORCE= 15160. (LB))

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
905.00	0.0000E+00	0.	9.8177E+08	83.33
904.00	4.8317E+01	103.	8.6696E+08	123.23
903.00	2.1988E+02	247.	7.5224E+08	163.20
902.00	5.5464E+02	430.	6.3793E+08	203.17
901.00	1.0926E+03	653.	5.2460E+08	243.14
900.00	1.8736E+03	916.	4.1320E+08	283.10
899.00	2.9378E+03	1219.	3.0508E+08	323.07
898.00	4.3250E+03	1562.	2.0208E+08	363.04
897.00	6.0753E+03	1945.	1.0660E+08	403.01
896.00	8.2286E+03	2368.	2.1687E+07	442.97
895.00	1.0825E+04	2831.	-4.8948E+07	482.94
894.00	1.3904E+04	3334.	-1.0081E+08	522.91
893.00	1.7506E+04	3877.	-1.2857E+08	562.88
892.00	2.1671E+04	4460.	-1.2599E+08	602.84
891.00	2.6439E+04	5083.	-8.5887E+07	642.81
890.00+	3.1850E+04	5745.	0.0000E+00	682.78
890.00-	3.1850E+04	-9415.	0.0000E+00	682.78
889.00	2.2783E+04	-8712.	1.3665E+08	722.75
888.00	1.4439E+04	-7969.	3.1278E+08	762.71
887.00	6.8570E+03	-7187.	5.1397E+08	802.68
886.00	7.8271E+01	-6364.	7.2712E+08	842.65
885.00	-5.8579E+03	-5501.	9.4053E+08	882.62
884.00	-1.0911E+04	-4599.	1.1439E+09	922.58
883.00	-1.5042E+04	-3656.	1.3286E+09	962.55
882.00	-1.8204E+04	-2655.	1.4875E+09	1039.82
881.00	-2.0348E+04	-1639.	1.6150E+09	992.10
880.00	-2.1507E+04	-694.	1.7075E+09	897.50
879.00	-2.1768E+04	156.	1.7630E+09	802.89
878.00	-2.1226E+04	911.	1.7810E+09	708.28
877.00	-1.9977E+04	1572.	1.7624E+09	613.67
876.00	-1.8113E+04	2139.	1.7093E+09	519.07
875.00	-1.5731E+04	2611.	1.6251E+09	424.46
874.00	-1.2924E+04	2988.	1.5137E+09	329.85
873.00	-9.7869E+03	3270.	1.3800E+09	235.24
872.00	-6.4148E+03	3458.	1.2295E+09	140.64
871.00	-2.9021E+03	3551.	1.0679E+09	46.03
870.51	-1.1705E+03	3563.	9.8706E+08	0.00
870.00	6.5667E+02	3550.	9.0126E+08	-48.58
869.00	4.1668E+03	3454.	7.3577E+08	-143.19
868.00	7.4985E+03	3158.	5.7746E+08	-449.40
867.00	1.0379E+04	2549.	4.3204E+08	-769.21
866.00	1.2522E+04	1719.	3.0445E+08	-891.00
865.00	1.3775E+04	767.	1.9837E+08	-1012.80
864.00	1.4015E+04	-307.	1.1595E+08	-1134.59
863.00	1.3121E+04	-1502.	5.7582E+07	-1256.39
862.00	1.0970E+04	-2820.	2.1706E+07	-1378.18
861.00	7.4407E+03	-4259.	4.5872E+06	-1499.98
860.00	2.4116E+03	-5820.	1.0992E+05	-1621.77
859.00	0.0000E+00	-6466.	0.0000E+00	-1669.60

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELLASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

III.--WATER AND SOIL PRESSURES

		<-------------SOIL PRESSURES-------------->			
		<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	WATER PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
905.00	0.	0.	0.	83.	530.
904.00	0.	0.	0.	123.	785.
903.00	0.	0.	0.	163.	1040.
902.00	0.	0.	0.	203.	1294.
901.00	0.	0.	0.	243.	1549.
900.00	0.	0.	0.	283.	1803.
899.00	0.	0.	0.	323.	2058.

898.00	0.	0.	0.	363.	2312.
897.00	0.	0.	0.	403.	2567.
896.00	0.	0.	0.	443.	2822.
895.00	0.	0.	0.	483.	3076.
894.00	0.	0.	0.	523.	3331.
893.00	0.	0.	0.	563.	3585.
892.00	0.	0.	0.	603.	3840.
891.00	0.	0.	0.	643.	4094.
890.00	0.	0.	0.	683.	4349.
889.00	0.	0.	0.	723.	4604.
888.00	0.	0.	0.	763.	4858.
887.00	0.	0.	0.	803.	5113.
886.00	0.	0.	0.	843.	5367.
885.00	0.	0.	0.	883.	5622.
884.00	0.	0.	0.	923.	5876.
883.00	0.	0.	0.	963.	6131.
882.00	0.	0.	0.	1040.	6173.
881.00	0.	115.	21.	1108.	6146.
880.00	0.	231.	42.	1128.	6261.
879.00	0.	346.	62.	1149.	6377.
878.00	0.	462.	83.	1170.	6492.
877.00	0.	577.	104.	1191.	6608.
876.00	0.	692.	125.	1211.	6723.
875.00	0.	808.	146.	1232.	6838.
874.00	0.	923.	166.	1253.	6954.
873.00	0.	1039.	187.	1274.	7069.
872.00	0.	1154.	208.	1295.	7185.
871.00	0.	1269.	229.	1315.	7300.
870.51	0.	1326.	239.	1326.	7356.
870.00	0.	1385.	250.	1336.	7415.
869.00	0.	1500.	270.	1357.	7531.
868.00	0.	1722.	269.	1273.	8126.
867.00	0.	1957.	266.	1188.	8735.
866.00	0.	2098.	285.	1207.	8876.
865.00	0.	2239.	304.	1226.	9016.
864.00	0.	2380.	324.	1245.	9157.
863.00	0.	2521.	343.	1264.	9298.
862.00	0.	2662.	363.	1283.	9439.
861.00	0.	2803.	382.	1303.	9580.
860.00	0.	2943.	402.	1322.	9721.
859.00	0.	3084.	421.	1341.	9862.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 2-DECEMBER-2011 TIME: 17:02:17

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*******************************
* PRELIMINARY DESIGN DATA FOR *
*  FREE EARTH DESIGN IN SAND  *
*******************************
```

I.--HEADING
'Lake Delhi Dam - North Abutment

II.--DESIGN PARAMETERS

WALL HEIGHT RATIO (ALPHA) = 0.57
ANCHOR HEIGHT RATIO (BETA) = 0.37

SHEET PILE DATA:

	<SECTION PROPERTIES> (PER FOOT OF WALL)			
SHEET PILE NAME	SECTION MODULUS (IN^3)	MOMENT OF INERTIA (IN^4)	ALLOWABLE STRESS (PSI)	MODULUS OF ELLASTICITY (PSI)
PZ40	60.70	490.80	2.40E+04	2.90E+07
PZ38	46.80	280.80	2.40E+04	2.90E+07
PZ35	48.50	361.20	2.40E+04	2.90E+07
PZ32	38.30	220.40	2.40E+04	2.90E+07
PZ27	30.20	184.20	2.40E+04	2.90E+07

PZ22	18.10	84.40	2.40E+04	2.90E+07
PLZ25	32.80	223.25	2.40E+04	2.90E+07
PLZ23	30.20	203.75	2.40E+04	2.90E+07

III.--PRELIMINARY DESIGN DATA

SHEET PILE NAME	LOG(H^4/EI)	ROWE'S MOMENT REDUCTION COEF.	RATIO OF ALLOWABLE MOMENT TO FREE EARTH MOMENT
PZ40	-3.74	1.0 (***)	3.81
PZ38	-3.49	1.0 (***)	2.94
PZ35	-3.60	1.0 (***)	3.05
PZ32	-3.39	1.0 (***)	2.41
PZ27	-3.31	1.0 (***)	1.90
PZ22	-2.97	1.0 (***)	1.14
PLZ25	-3.39	1.0 (***)	2.06
PLZ23	-3.35	1.0 (***)	1.90

*** REDUCTION NOT APPLICABLE DUE TO ALPHA LESS THAN 0.6.

*** REDUCTION NOT APPLICABLE DUE TO BETA GREATER THAN 0.3.

Reference Material

Embankment Slopes

O) An adequate energy dissipation structure (stilling basin) or an alternative acceptable method should be incorporated at the outlet of all structural spillways. On major structures, uplift analysis and arching requirements should be considered.

CHAPTER V
EMBANKMENTS

The earth embankment of a dam should be designed and built according to the following criteria and guidelines.

A) Foreslopes and Backslopes. Embankments should be built of suitable materials and with stable slopes. Foreslopes should not be steeper than 3:1 (horizontal to vertical) in till or loess soils below the permanent water level. Above the permanent water level, foreslopes should not be steeper than 2.5:1 in till soil or 3:1 in loess soil. Backslopes should not be steeper than 2.5:1 in till soils or 3:1 in loess soils. Steeper foreslopes or backslopes may be used if justified by soil tests and stability analysis.

*

3:1 slopes used in embankments.

B) Settlement Allowance. A minimum vertical settlement allowance of five percent of the depth of fill should be provided unless a lesser amount is justified by soil tests.

C) Top Width. The minimum embankment top width should be 14 feet for dams 25 feet high or higher. As a rule of thumb, the top width could be reduced by two feet for every five feet of reduction in the height of the dam to a minimum width of eight feet. The top width of dams with roads across the crest should be consistent with normal roadway design practices, including roadway and shoulders.

D) Core Trench. Core trenches should be located approximately along the centerline (axis) of the earth fill. It should be continuous across the base of the fill extending into and up the side slopes of the dam abutments to normal reservoir level. The core trench should be excavated to a minimum depth of five feet or until a suitable base material is reached. The base width should be that which will accommodate excavating equipment, but not less than eight feet. The side slopes of the core trench should not be steeper than 1:1, regardless of depth or width of base. Impervious material shall be used in backfilling the core trench.

E) Wave Erosion Protection. On the upstream face of the dam, a horizontal bench or berm at least 10 feet wide usually should be provided at the normal pool elevation to limit damage from wave erosion. On larger impoundments, riprap or other physical means of protection should be considered whether or not the berm is provided.

F) Site Preparation. All vegetation, sod, stumps, and large roots should be removed from the embankment site, and the ground surface scarified to provide bond with the earth fill. Overhanging banks, pits, or holes should be sloped and graded so slopes do not exceed 1:1, and any other sharp discontinuities in the ground surface shall be smoothed. Special consideration should be given to the removal of sandy or mucky deposits unless otherwise provided for in the design.

In till soils, topsoil should be saved and placed as a surface layer over the finished embankment to provide an adequate seed base in establishing vegetation.

G) Fill. Fill material should be clean earth containing no appreciable amounts of vegetation, large rock, frozen material or other foreign substances. Fill should not be placed on a frozen foundation or in freezing weather.

Moisture content of the fill should be sufficient to assure adequate compaction. A moisture content slightly higher than optimum is recommended. An above optimum moisture content is desirable from the standpoint of providing a more plastic embankment capable of resisting greater differential settlement without experiencing

Drains / Riprap

potentially hazardous cracking. Unless otherwise specified after soil testing, fill should be placed in horizontal lifts not exceeding eight inches extending over the entire fill area and compacted by not less than four overlapping passes by sheepsfoot or rubber tired rollers. Smooth steel rollers or passes by caterpillar tracks are not considered adequate for compaction of earth-fill dams. The surface of the fill should be scarified or roughened if sufficient time elapses between lifts for a crust to develop.

Backfill adjacent to spillway structures and anti-seep collars should be carefully placed and compacted by hand equipment. Heavy equipment shall not pass over conduit structures until two feet of compacted earth cover is in place.

* Gravel seepage drain included in embankment costs.

H) Drains. Internal seepage control drains are recommended for all dams and are normally required on major structures unless soils investigation finds they are not needed. Drains should be capable of preventing saturation of the downstream portion of the embankment by intercepting any seepage through the fill or the foundation and any seepage along structural spillways or conduits.

If springs are encountered during site preparation, drains should be provided to allow a controlled outlet.

I) Seeding. As soon as possible after earth fill for the embankment is completed, the embankment and any other exposed areas should be seeded. Mulch or other means of erosion control can be placed as part of seeding and maintained until vegetation is established. Grass or vegetative species selected for use should be appropriate for the soils and conditions expected at the site. Crown vetch is generally not acceptable for dam embankments or emergency spillways.

* Riprap (on site) and bedding/filter fabric included in alternative costs.

J) Riprap. Riprap shall be designed for its expected use and anticipated water velocities. All riprap should be placed on a properly designed bedding unless the gradation of the underlying base material is such that it will not infiltrate through the riprap, or an acceptable filter fabric is used.

K) Groins. Where the embankment foreslopes and backslopes intersect the natural or modified abutment slopes, appropriate groin design and erosion control recommendations should be provided.

Embankments composed of concrete, rock, or other materials, as associated with gravity, rockfill and arch dams, designed in accordance with standards by the U.S. Army Corps of Engineers, U.S.D.A. Soil Conservation Service, or the U.S. Bureau of Reclamation are generally acceptable. These types of dams are not normally constructed in Iowa.

CHAPTER VI
SPECIAL REQUIREMENTS FOR MAJOR DAM STRUCTURES

Because of the size, public importance, or potential hazard of a major dam structure, a higher level of investigation, design and assurance of proper construction is needed. A major dam structure is defined as a dam meeting any of the following criteria.

1. Any high hazard dam.

2. Any moderate hazard dam with a permanent storage exceeding one hundred (100) acre-feet or a total of permanent and temporary storage exceeding two hundred fifty (250) acre-feet at the top of the dam elevation.

3. Any dam, including low hazard dams, where the height of the emergency spillway crest measured above the elevation of the channel bottom at the centerline of the dam (in feet) multiplied by the total storage volume (in acre-feet) to the emergency spillway crest elevation exceeds 30,000. For dams without emergency spillways, these measurements shall be taken to the top of dam elevation.

As a condition of permit approval, the following items will be required for major dam structures.

A) A soils and foundation investigation shall be made which includes the evaluation of slope stability requirements, anticipated vertical

Reference

IOWA DEPARTMENT OF NATURAL RESOURCES

Technical Bulletin No. 16

DESIGN CRITERIA AND GUIDELINES FOR IOWA DAMS

December, 1990

SLOPE STABILITY FACTOR OF SAFETY REQUIREMENTS

Table 3-1
Minimum Required Factors of Safety: New Earth and Rock-Fill Dams

Analysis Condition[1]	Required Minimum Factor of Safety	Slope
End-of-Construction (including staged construction)[2]	1.3	Upstream and Downstream
Long-term (Steady seepage, maximum storage pool, spillway crest or top of gates)	1.5	Downstream
Maximum surcharge pool[3]	1.4	Downstream
Rapid drawdown	1.1-1.3[4,5]	Upstream

[1] For earthquake loading, see ER 1110-2-1806 for guidance. An Engineer Circular, "Dynamic Analysis of Embankment Dams," is still in preparation.

[2] For embankments over 50 feet high on soft foundations and for embankments that will be subjected to pool loading during construction, a higher minimum end-of-construction factor of safety may be appropriate.

[3] Pool thrust from maximum surcharge level. Pore pressures are usually taken as those developed under steady-state seepage at maximum storage pool. However, for pervious foundations with no positive cutoff steady-state seepage may develop under maximum surcharge pool.

[4] Factor of safety (FS) to be used with improved method of analysis described in Appendix G.

[5] FS = 1.1 applies to drawdown from maximum surcharge pool; FS = 1.3 applies to drawdown from maximum storage pool.

For dams used in pump storage schemes or similar applications where rapid drawdown is a routine operating condition, higher factors of safety, e.g., 1.4-1.5, are appropriate. If consequences of an upstream failure are great, such as blockage of the outlet works resulting in a potential catastrophic failure, higher factors of safety should be considered.

(1) During construction of embankments, materials should be examined to ensure that they are consistent with the materials on which the design was based. Records of compaction, moisture, and density for fill materials should be compared with the compaction conditions on which the undrained shear strengths used in stability analyses were based.

(2) Particular attention should be given to determining if field compaction moisture contents of cohesive materials are significantly higher or dry unit weights are significantly lower than values on which design strengths were based. If so, undrained (UU, Q) shear strengths may be lower than the values used for design, and end-of-construction stability should be reevaluated. Undisturbed samples of cohesive materials should be taken during construction and unconsolidated-undrained (UU, Q) tests should be performed to verify end-of-construction stability.

d. Pore water pressure. Seepage analyses (flow nets or numerical analyses) should be performed to estimate pore water pressures for use in long-term stability computations. During operation of the reservoir, especially during initial filling and as each new record pool is experienced, an appropriate monitoring and evaluation program must be carried out. This is imperative to identify unexpected seepage conditions, abnormally high piezometric levels, and unexpected deformations or rates of deformations. As the reservoir is brought up and as higher pools are experienced, trends of piezometric levels versus reservoir stage can be used to project piezometric levels for maximum storage and maximum surcharge pool levels. This allows comparison of anticipated actual performance to the piezometric levels assumed during original design studies and analysis. These projections provide a firm basis to assess the stability of the downstream slope of the dam for future maximum loading conditions. If this process indicates that pore water pressures will be higher than those used in design stability analyses, additional analyses should be performed to verify long-term stability.

e. Loads on slopes. Loads imposed on slopes, such as those resulting from structures, vehicles, stored materials, etc. should be accounted for in stability analyses.

SEEPAGE GRADIENT

$$\Delta q = k \frac{\Delta h}{\Delta \ell} a$$

where a is the area of the rectangle perpendicular to the flow direction. If one side of the rectangle is one unit of length perpendicular of the plane of the flow net, and the other dimension is $\Delta \ell$, thus a = $\Delta \ell$ (1) . This leads to:

$$\Delta q = k \frac{\Delta h}{\Delta \ell} \Delta \ell \ (1) = k \Delta h \ (1)$$

$$= k \frac{h}{N_d} \ (1)$$

Then:

$$q = \Delta q N_f$$

$$q = k \frac{h}{N_d} N_f \ (1)$$

$$q = k \not{s} h \ (1) \text{ or } k \not{s} h$$

which gives the quantity of seepage flow for each unit of thickness of porous media perpendicular to the plane of the flow net. Figure 4-10(b) gives an example of this calculation for anisotropic seepage conditions in a dam foundation. The permeability, k' , used for anisotropic conditions, $k' = \sqrt{k_v k_h}$, is derived by Casagrande (1937).

* b. Escape and Critical Gradients. The escape or exit gradient, i_e , is the rate of dissipation of head per unit of length in the area where seepage is exiting the porous media. For confined flow, the area of concern is usually along the uppermost flow line near the flow exit, e.g., at the downstream edge of a concrete or other impermeable structure, figure 4-15. Escape gradients for flow through embankments may also be studied by choosing squares from the area of interest in the flow net (usually at or near the exit face and downstream toe) and calculating gradients. If the gradient is too great where seepage is exiting, soil particles may be removed from this area. This phenomenon, called flotation, can cause piping (the removal of soil particles by moving water) which can lead to undermining and loss of the structure. The gradient at which flotation of particles begins is termed the critical gradient, i_{cr} . Critical gradient is determined by the in-place

unit weight of the soil and is the gradient at which upward drag forces on the soil particles equal the submerged weight of the soil particles, figure 4-16. The critical gradient is dependent on the specific gravity and density of the soil particles and can be defined in terms of specific gravity of solids, G_s, void ratio, e, and porosity, n:

$$i_{cr} = \frac{\gamma'_m}{\gamma_w} \quad \frac{G_s(1 - n)\gamma_w + n\gamma_w - \gamma_w}{\gamma_w}$$

$$i_{ca} = \frac{\gamma'}{\gamma_w} = \frac{125\,pcf - 62.4\,pcf}{62.4\,pcf} = 1.0$$

$$= G_s(1 - n) + n - 1 = G_s\ (1 - n) - (1 - n)$$

$$i_{cr} = (G_s - 1)(1 - n)$$

or, since $e = \frac{n}{1-n}$ and $n = \frac{e}{1 + e}$

$$i_{cr} = (G_s - 1)\left(\frac{n}{e}\right) = (G_s - 1)\ \frac{\frac{e}{1+e}}{e}$$

$$FS = \frac{i_{ca}}{i_e} = \frac{1.0}{0.4} = 2.5$$

$$i_{cr} = \frac{G_s - 1}{1 + e}$$

$i_e = 0.4$

If typical values of G_s, e, and n for sand are used in the above equations, i_{cr} will be approximately 1. Investigators have recommended ranges for factor of safety for escape gradient, $FS_G = \frac{i_{cr}}{i_e}$ from 1.5 and 15, depending on knowledge of soil and possible seepage conditions. Generally, factors of safety in the range of 4-5 (Harr 1962, 1977) or 2.5-3 (Cedergren 1977) have been proposed.

c. Heave. In some cases, movement of soil at the downstream seepage exit may not occur as flotation followed by particle-by-particle movement. A mass of soil may be lifted initially, followed by piping. This phenomenon is called heave and occurs when the upward seepage force due to differential head equals the overlying buoyant weight of soil. Heave occurs under conditions of critical hydraulic gradient. For field conditions, the point at which minimum differential head offsets the overlying buoyant weight must be determined by judgment and calculations. Terzaghi and Peck (1967) have evaluated the factor of safety with respect to heave for a row of sheet piles. Resistance to heave

SEEPAGE Cutoff Design.

from the standpoint of failure due to piping. The path of this upper flow line is called the *creep path* and the phenomenon associated with this type of piping is called *roofing*.

From his analysis of the action of actual dams, Lane concluded that the creep path along contact surfaces with slopes less than 45° from the horizontal offer only one-third the resistance to roofing as do those with slopes greater than 45°. On the basis of this criterion he developed the equation for the *weighted creep ratio* R_c:

$$R_c = \frac{\frac{1}{3}H + V}{h} \tag{3}$$

where H = horizontal contacts ($\leqq 45°$)
V = vertical contacts ($> 45°$)
h = head loss through the system

For the section illustrated in Fig. 5-24, the creep path is $BCDEFG$, $H = 66$ ft, $V = 83$ ft, and $h = 15$ ft; hence the weighted creep ratio is

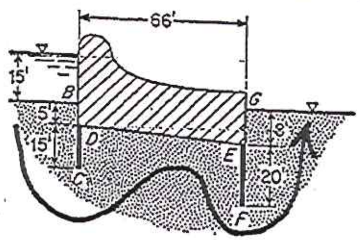

Fig. 5-24

$R_c = 7.0$. For safety Lane recommended that R_c should not be less than the values given in Table 5-2 for the particular foundation material.

Table 5-2. Recommended Weighted Creep Ratios*

Material	*Safe weighted creep-head ratios, Rc*
Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel, including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

Structure cutoff creep ratio range.

* From E. W. Lane, Security from Under-seepage: Masonry Dams on Earth Foundations, *Trans. Am. Soc. Civil Eng.*, vol. 100, p. 1257, 1935.

We see from this table that the section of Fig. 5-24, according to Lane, would be safe for all materials except very fine sand or silt.*

The values given in Table 5-2 are recognized as being very conservative, tending toward maximum rather than average values. Not a single failure was found by Lane where the dam evidenced a weighted creep-head ratio as large as those given.

Whereas roofing is a consideration only for those cases where the possibility of imperfect contact exists between the base of the structure and the surface of the foundation material, provisions which would ensure this contact, such as grouting under the structure, are strongly recommended.

Blind application of any piping criterion without cognizance of subsurface soil conditions can lead to dangerous results. For example, for the condition illustrated in Fig. 5-25*a*, the critical location for piping will

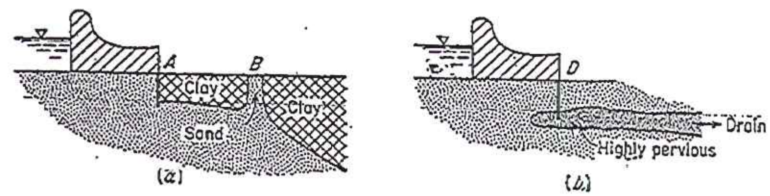

Fig. 5-25

probably not occur at point A but at point B, whereas for the subsurface condition in Fig. 5-25*b*, it is highly probable that the tail water may never reach point D. Finally, with respect to Lane's criterion, we note that although the weighted creep ratios of both sections in Fig. 5-26 may be

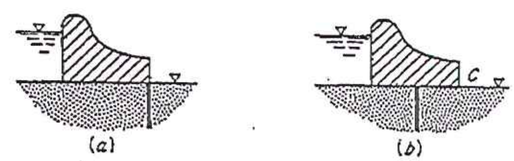

Fig. 5-26

identical, the design shown in (*b*) would have a high probability of failure due to incipient piping in the vicinity of point C.

In summary, the following procedure is recommended (assuming adequate exploration of the subsurface) for design considerations with respect to piping:

* R_c, in Eq. (3), is somewhat equivalent to the reciprocal of the gradient. Hence, assuming the critical gradient is approximately equal to unity, the values in Table 5-2 can be thought of as quasi-factors-of-safety with respect to piping.

Groundwater and Seepage

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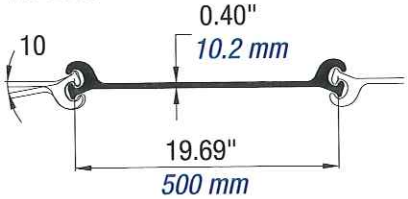
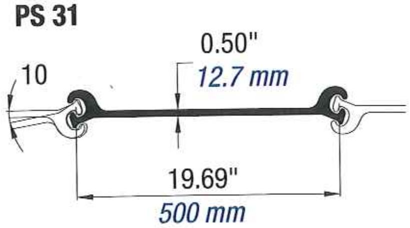
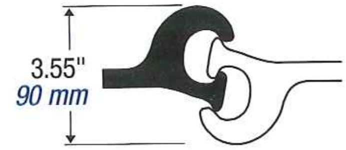
PS (FLAT SHEET) PILING PROPERTIES

PS 27.5



PS 31



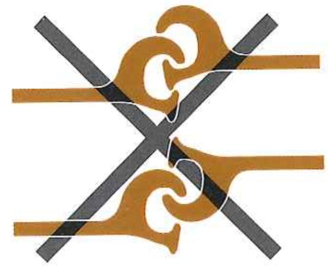


Section	Nominal Width in. (mm)	Depth (Height) in. (mm)	Wall Depth (Height) in. (mm)	Web Thickness in. (mm)	Per Single Section: Area in.^{2} (cm^2)	Per Single Section: Weight lbs/ft (kg/m)	Per Single Section: Moment of Inertia in.^{4} (cm^4)	Per Single Section: Section Modulus in.^{3} (cm^3)	Per Single Section: Total Surface Area ft^2/ft (m^2/m)	Per Single Section: Nominal Coating Area* ft^2/ft (m^2/m)	Per Unit of Wall: Area in.^{2}/ft (cm^2/m)	Per Unit of Wall: Weight lbs/ft^2 (kg/m^2)	Per Unit of Wall: Moment of Inertia in.^{4}/ft (cm^4/m)	Per Unit of Wall: Section Modulus in.^{3}/ft (cm^3/m)
PS 27.5	19.69	2.83	3.55	0.40	13.26	45.1	5.0	3.2	4.50	3.64	8.08	27.5	3.0	1.9
	500	*72*	*90*	*10.2*	*85.5*	*67.1*	*207*	*52*	*1.37*	*1.11*	*171.0*	*134.2*	*414*	*103*
PS 31	19.69	2.83	3.55	0.50	14.96	50.9	5.0	3.2	4.50	3.64	9.11	31.0	3.0	1.9
	500	*72*	*90*	*12.7*	*96.5*	*75.7*	*207*	*52*	*1.37*	*1.11*	*192.9*	*151.4*	*414*	*103*

*Both sides of sheet; excludes interior of interlock.

All dimensions given are nominal. Actual web thickness varies due to mill rolling practices; however, permitted variations for such dimension are not addressed.

Proper Interlock



Improper Interlock

Grade	Minimum Interlock Strength(1)	Minimum Swing(2)
A328	16 kips/in. (2,800 kN/m)	10 degrees
A572-50	20 kips/in. (3,500 kN/m)	10 degrees
A572-60	24 kips/in. (4,200 kN/m)	10 degrees

Higher interlock strengths are available but obtainable swing may be reduced in interlock strengths above 24 kips/in (4,200 kN/m).

(1) These minimum ultimate interlock strengths assume proper interlocking of sheets. To verify the strength of PS Sheet Piling, both yielding of the web and failure of the interlock should be considered.
(2) Swing reduces 1.5 degrees for each 10 feet (3 meters) in length over 70 feet (21 meters).

NOTE: *INTERLOCKING OF GERDAU PS SECTIONS WITH ANOTHER PRODUCER'S SECTION SHOULD NEVER BE CONSIDERED UNLESS APPROVED IN ADVANCE BY GERDAU. PS and Z-Piling sections should not be interlocked together. Gerdau PS 27.5 and PS 31 can be interlocked with each other.*

GERDAU

300 Ward Road, Midlothian, Texas 76065 USA
972.775.8241 • 800.527.7979 • www.sheet-piling.com

PZC SHEET PILING PROPERTIES

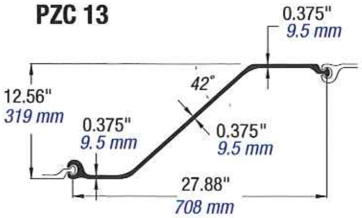
PZC sections are the "latest generation" of sheet piling profiles and were developed to be lighter, wider, and stronger than the older traditional PZ sections. PZC profiles are named for their strength in metric designations. For example, PZC 18 has a Section Modulus of 1,800 cm³/meter. **PZC profiles should always be the designer's first choice in order to provide the end user the most efficient retention wall with the most efficient ratio of section modulus to weight.**

					Per Single Section						Per Unit of Wall			
Section	Nominal Width	Wall Depth (Height)	Web Thickness	Flange Thickness	Area	Weight	Moment of Inertia	Section Modulus	Total Surface Area	Nominal Coating Area*	Area	Weight	Moment of Inertia	Section Modulus
	in. (mm)	in. (mm)	in. (mm)	in. (mm)	in.² (cm²)	lbs/ft (kg/m)	in.⁴ (cm⁴)	in.³ (cm³)	ft²/ft (m²/m)	ft²/ft (m²/m)	in.²/ft (cm²/m)	lbs/ft² (kg/m²)	in.⁴/ft (cm⁴/m)	in.³/ft (cm³/m)
PZC 13	27.88	12.56	0.375	0.375	14.82	50.4	353.0	56.2	6.10	5.60	6.38	21.7	152.0	24.2
	708	*319*	*9.5*	*9.5*	*95.6*	*75.1*	*14,690*	*920*	*1.86*	*1.71*	*135.1*	*106.0*	*20,760*	*1,300*
PZC 14	27.88	12.60	0.420	0.420	16.15	55.0	381.6	60.5	6.10	5.60	6.95	23.7	164.3	26.0
	708	*320*	*10.7*	*10.7*	*104.2*	*81.8*	*15,890*	*990*	*1.86*	*1.71*	*147.2*	*115.5*	*22,440*	*1,400*
PZC 18	25.00	15.25	0.375	0.375	14.82	50.4	532.2	69.8	6.10	5.60	7.12	24.2	255.5	33.5
	635	*387*	*9.5*	*9.5*	*95.6*	*75.1*	*22,150*	*1,145*	*1.86*	*1.71*	*150.6*	*118.2*	*34,890*	*1,800*
PZC 19	25.00	15.30	0.420	0.420	16.16	55.0	576.3	75.3	6.10	5.60	7.75	26.4	276.6	36.1
	635	*388*	*10.7*	*10.7*	*104.2*	*81.8*	*23,990*	*1,235*	*1.86*	*1.71*	*164.1*	*128.8*	*37,780*	*1,945*
PZC 25	27.88	17.66	0.485	0.560	20.40	69.4	938.7	106.3	6.65	6.15	8.78	29.9	404.1	45.7
	708	*449*	*12.3*	*14.2*	*131.6*	*103.3*	*39,070*	*1,740*	*2.03*	*1.87*	*185.9*	*145.9*	*55,190*	*2,455*
PZC 26	27.88	17.70	0.525	0.600	21.72	73.9	994.3	112.4	6.65	6.15	9.35	31.8	428.1	48.4
	708	*450*	*13.3*	*15.2*	*140.1*	*110.0*	*41,390*	*1,840*	*2.03*	*1.87*	*197.9*	*155.4*	*58,460*	*2,600*
PZC 28	27.88	17.75	0.570	0.645	23.22	79.0	1,057	119.1	6.65	6.15	10.00	34.0	455.1	51.3
	708	*451*	*14.5*	*16.4*	*149.8*	*117.6*	*44,000*	*1,950*	*2.03*	*1.87*	*211.6*	*166.1*	*62,150*	*2,755*
PZC 37	22.50	21.02	0.488	0.563	20.45	69.6	1,349	128.4	6.65	6.15	10.91	37.1	719.6	68.5
	572	*534*	*12.4*	*14.3*	*132.0*	*103.6*	*56,160*	*2,100*	*2.03*	*1.87*	*230.9*	*181.2*	*98,270*	*3,680*
PZC 39	22.50	21.05	0.525	0.600	21.76	74.0	1,429	135.6	6.65	6.15	11.61	39.5	762.1	72.3
	572	*535*	*13.3*	*15.2*	*140.4*	*110.2*	*59,480*	*2,220*	*2.03*	*1.87*	*245.6*	*192.8*	*104,100*	*3,890*
PZC 41	22.50	21.09	0.561	0.636	23.03	78.4	1,507	142.7	6.65	6.15	12.28	41.8	803.6	76.1
	572	*536*	*14.2*	*16.2*	*148.6*	*116.6*	*62,720*	*2,340*	*2.03*	*1.87*	*260.0*	*204.1*	*109,700*	*4,090*

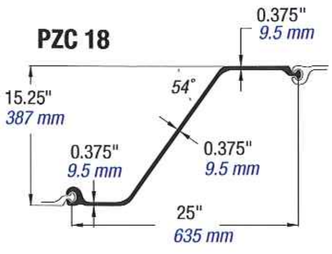
*Both sides of sheet; excludes socket interior and ball of interlock.

All dimensions given are nominal. Actual flange and web thicknesses vary due to mill rolling practices; however, permitted variations for such dimensions are not addressed.

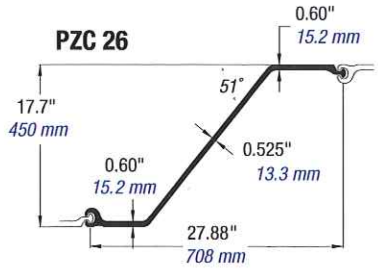
PZC 13



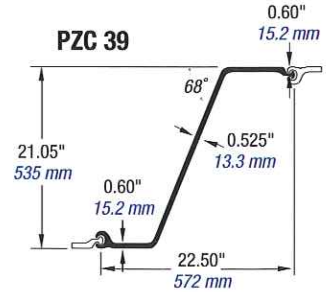
PZC 18

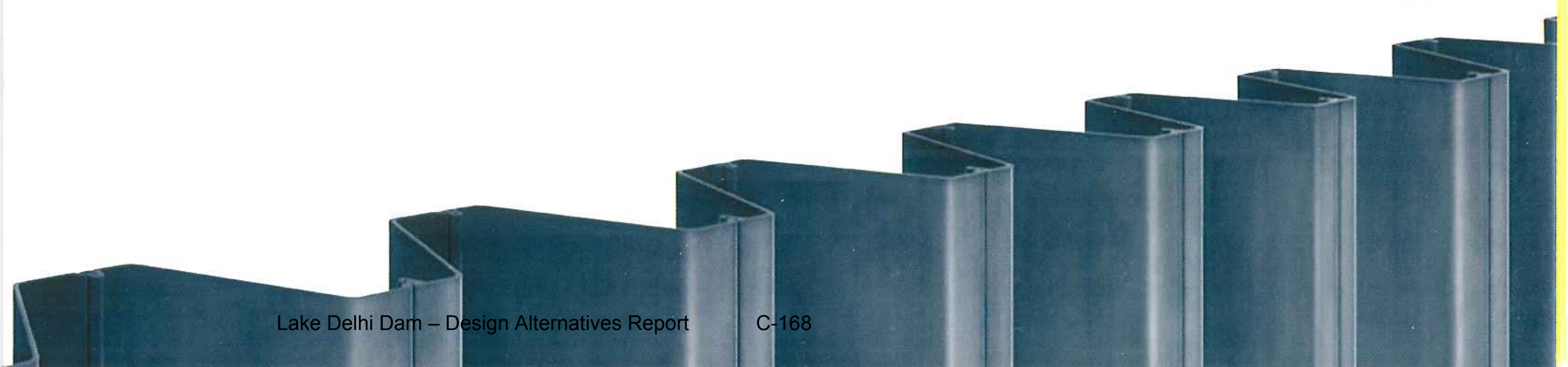


PZC 26



PZC 39





Geotechnical Quantities

Stanley Consultants INC.

Project No. _______ Page No. _______

Subject LAKE DELHI

SHEET PILE QUANTITY

Computed by JRS Date 12-5-11

Checked by _______ Date _______

Approved by _______ Date _______

Sheet No. _______ of _______

SPILLWAY STRUCTURE SHEET PILE QNTY'S

① SINGLE LABYRINTH

Length = 180'

Pile Length = 35' (860 to 825)

TOTAL = 6300 s.f. PZC-13 @ 21.7 lbs/s.f. = 136,710 lbs

Cutoff below structure: 10' x 180' = 1800 s.f. = 39,060 lbs.

② DOUBLE LABYRINTH

$\text{LENGTH}_{\text{SERVICE}}$ = 120' Pile Length = 35' (860 to 825)

$\text{Length}_{\text{AUX}}$ = 110' Pile Length assume reduced length moving south up the bank

$\text{Length}_{\text{AVG}}$ = 20'

TOTAL: 120' x 35' + 110' x 20' = 6400 s.f. = 138,880 lbs

Cutoff below structure: on service spillway only

10' x 120' = 1200 s.f. PZC-13 @ 21.7 lbs/s.f. = 26040 lbs

③ PNEUMATIC GATES:

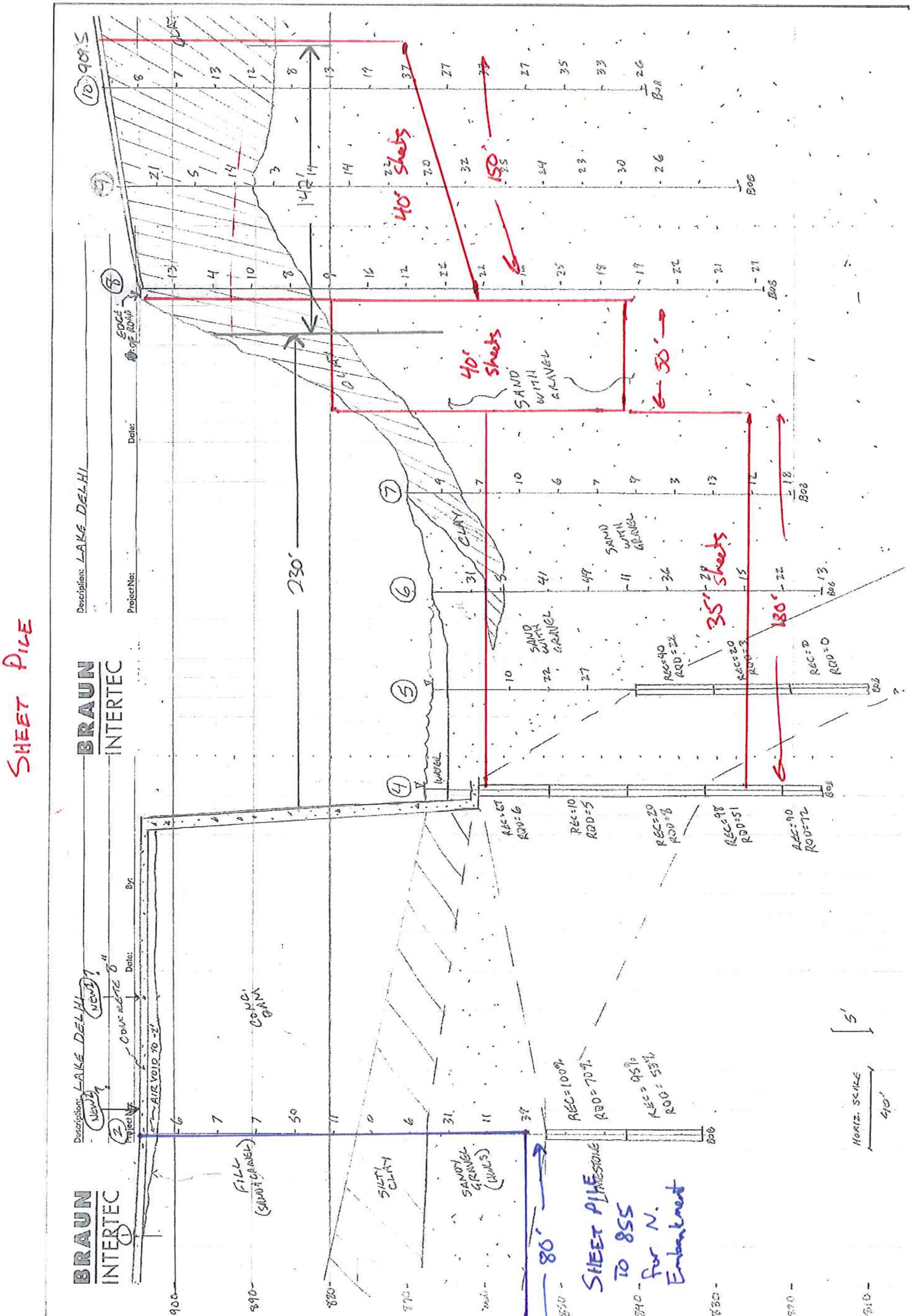
Length = 160'

Pile Length = 45' (860 to 820, added length due to smaller embankment)

TOTAL = 7200 s.f. = 156,240 lbs

Cutoff below structure: 10' x 160' = 1600 s.f. = 34,720 lbs

SC 3020 R2 199


SHEET PILE
BRAUN INTERTEC
Description: LAKE DELHI
CONC DAM
FILL (SAND & GRAVEL)
SILTY CLAY
SANDY GRAVEL (WACS)
LIMESTONE
SAND WITH GRAVEL
CLAY
230'
142'
40' sheets
40' Sheets
35' sheets
150'
50'
180'
80'
SHEET PILE TO 855 for N. Embankment
HORIZ. SCALE 40'

Stanley Consultants INC.

Computed by JRS Date 12-3-11
Checked by Date
Approved by Date

Project No. Page No.
Subject LAKE DELHI

Sheet No. of

CELLULAR SHEET PILE FOR N. ABUTMENT

D = 28'

Avg 2.4 sheets per foot of length

Length = 80'

Sheet pile length = 50' on west side, 38' on east side
Avg = 44'

2.4 sheets per ft of cell

Total sheets = 2.4 × 80 = 192

Sheet length = 19.67" = 1.64'

Total sheet pile sq. ft = 1.64' × 44' × 192 = 13,855 ft²

PS 27.5
@ 45.1 lb/ft
= 381,000 lbs

Other Items:

FILL: use in-place material

Excavate: 25' × 23' × 80' = 1704 CY

Place: 1704 CY + IMPORT ADD'L 200 CY

GUARDRAIL 160'

SC 3020 R2 199

Stanley Consultants INC.

Project No. Page No.
Subject

Computed by Date
Checked by Date
Approved by Date
Sheet No. of

Z-section Sheet Pile: — NORTH ABUTMENT

Both E + W sides = 50' Length, Add 10% for walers, tie-backs, etc.

Assume PZC 18 @ 50.4 lbs/ft

160' x 50' = 8,000 sq ft

Need 77 sheets x 50' length x 50.4 lb/ft = 194,000 lbs

Add 10% = 214,000 lbs

SC 3020 R2 199

Cofferdams

Jacks, Jason

From: Judd, Andrew
Sent: Monday, December 05, 2011 11:23 AM
To: Karels, Lucas; Jacks, Jason
Subject: Cofferdam headwater and tailwater elev's

RETURN PERIOD	RIVER FLOW (CFS)	PHASE I		PHASE II	
		U/S W.S.E.	D/S W.S.E	U/S W.S.E.	D/S W.S.E
1	1400	869.5	861.5	883	861.5
2	4500	872	866.1	886.5	865.9
5	8700	874	869.5	890.5	870
10	12300	876	872.2	893	871.9

*

Here are river elev's for given return periods for sizing upstream and downstream cofferdams. Phase I is powerhouse repair with flow bypass through the breached section. Phase II is South Embankment construction with flow bypass through the spillway gates only (no sluice pipes or hydro included).

I would use 5 year for PHASE I and 2 year for PHASE II. PHASE II will be constructed in the fall, and over 10 years, the largest flows ever recorded during that period are 5,600 and then the next highest is 2,500 cfs, so 2 year cofferdams should provide adequate protection during the fall construction period.

* Use cofferdam height 1' above river elevations noted in Table.

Cofferdam qnty's measured in contour CADD file

Appendix D

Structural Analysis and Design

Major structures of the project include:

1. North embankment retaining structure.
2. Existing powerhouse and downstream retaining walls.
3. Existing Ogee spillway with vertical lift gates.
4. Existing training wall south of Ogee spillway.
5. Service labyrinth spillway south of existing Ogee spillway.
6. Auxiliary labyrinth spillway south of service spillway.
7. South embankment and training/retaining structures.

A structural inspection/evaluation of items 1 through 4 above, after the July 2010 flood, was performed on September 23, 2010 by Stanley Consultants. The report is included in Appendix A. Even though these inspected structures remained after the flood, and no noticeable movement or differential settlement were observed during the inspection, significant repair work was recommended before putting the Dam back to service.

In this stage of the project, following structural analysis was performed.

D.1 Existing Powerhouse and Spillway Stability

After review of 1997 Ashton Barnes stability analysis of the dam, a new analysis for existing powerhouse and spillway structures was conducted based on these information and assumptions:

1. Design information, previous analysis, and inspection reports available to Stanley Consultants.

2. Parameters for the foundation soil and bedrock used in analysis and design were based on the newly obtained boring data, recommended values in USACE Engineering Manuals, and researches on similar projects.
3. Water elevations were obtained from hydrology and hydraulic analysis using latest rainfall data.
4. The structures were checked against both USACE and FERC criteria.
5. Upstream seepage cutoff efficiency for the both structures was assumed to be no less than 50%.

Other design parameters used in analysis are listed the following computations.

Based on above information and assumptions, existing powerhouse and spillway do not satisfy current USACE or FERC requirements for overall stability. Without considering contributions from downstream soil pressure and north embankment soil friction effect for stability of powerhouse structure, analysis indicated that the powerhouse structure was unstable for overturning stability.

Both the existing powerhouse and spillway are required to be anchored to bedrock foundation in order to meets current design criteria.

D.2 Anchorage Design for Existing Powerhouse and Spillway

Existing powerhouse and spillway structure were designed to be anchored to bedrock foundation using pretensioned steel rods. Two options were provided: one is to satisfy USACE's safety criteria, the other is to meet FERC's requirements. The latter option is such that, if Owner of the dam chooses to rehabilitate the hydropower facility to generate electricity at a later time, major structures of the dam would not need significant repair work in order to meet FERC standards for overall stability. Conceptual design and computations are presented in the following pages. Cost estimates for these two options are discussed in Section 7.

For the spillway-USACE case, approximately ten (10) rock anchors are required. The anchors would be installed 1) in front of spillway upstream face, or 2) at spillway crest. First option would require excavating at upstream to bottom of dam and new concrete doweled into existing structure. This option would provide relatively easy access for construction. Second option would require drilling anchor holes through existing concrete approximately 30 feet, and accessibility for construction may be more difficult.

For the spillway-FERC case, approximately thirty (30) rock anchors would be required. Both anchor options in the USACE case would be required and additional ten (10) anchors would be located in the bridge piers.

The powerhouse structure would need approximately ten (10) anchors in order to meet USACE stability requirements. These anchors are proposed to be located at upstream face of the powerhouse. Excavation to bedrock would be required for installation of the anchors.

Meeting FERC criteria would need about twenty (20) rock anchors. These anchors have higher capacity, due to limited accessibility for installation. Ten (10) anchors would be installed at upstream face of the powerhouse, and the other ten (10) would be installed through the solid concrete walls.

Should the dam structures be anchored to meet FERC requirements, installation of some anchors would be performed on the existing bridge, therefore, the bridge and powerhouse roof structures should be investigated for construction equipment loading conditions.

D.3 Design of New Spillways

New spillways were designed to pass 100-year design flood, and have an overall capacity to pass ½ PMF flood.

Construction of new structures, including spillway weir, spillway slab, stilling basin, retaining/training walls, would meet both USACE and FERC requirements for stability and structural strength.

Seismic analysis for the structures is not necessary, since the dam is located in a low seismic zone. $S_s = 0.086$, $S_1 = 0.046$.

The conceptual structural design computations for new spillway stability are presented in the following.

Delhi Dam

Structural Computations

&

Reference Material

Structural Design Criteria

Delhi Dam Reconstruction- Design Criteria

Reference:K:\Technical_Programs\Structural\ST084 ACI 318-2005 Mathcad Electronic Book.mcd

Units:

$k := 1000 \cdot lbf \quad kpf := \frac{k}{ft} \quad ksf := \frac{k}{ft^2} \quad ksi := \frac{k}{in^2} \quad kcf := \frac{k}{ft^3} \quad ppf := \frac{lbf}{ft} \quad psf := \frac{lbf}{ft^2} \quad psi := \frac{lbf}{in^2} \quad pcf := \frac{lbf}{ft^3}$

Description

Documentaion of the design criteria, codes and loads used in the design of the existing powerhouse, existing gated spillway, new labyrinth spillways, and earth retaining structures.

References

1. USCOE EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures
2. USCOE EM 1110-2-2105 Design of Hydraulic Steel Structures
3. USCOE EM 1110-2-1612 Engineering and Design - Ice Engineering
4. USCOE EM 1110-2-2200 Gravity Dam Design
5. USCOE ETL 1110-2-256 Sliding Stability for Concrete Structures
6. FERC Engineering Guidelines for the Evaluation of Hydropower Projects (2005)
7. American Society Of Civil Engineers - Minimum Design Loads (ASCE-7)
8. American Concrete Institute (ACI 318) - See note below
9. American Institute of Steel Construction (AISC - ASD Manual 13th ed)

**Note: Although the current edition of ACI 318 will be used for design, the older ACI Load Factors of 1.4 and 1.7 will be used in lieu of the the current recommended ACI Load Factors of 1.2 and 1.6 for "Hydraulic Structures" (as documented in the USCOE design manuals).

Design Criteria

Materials

Concrete weight $\gamma_{conc} := 150 \cdot pcf$

Saturated Soil Weight $\gamma_{soil} := 115 \cdot pcf$

Submerged Soil Weight $\gamma_{soil\_sub} := 115 \cdot pcf$

Water Weight $\gamma_{water} := 62.4 \cdot pcf$

Concrete Strength $f_c := 4 \cdot ksi$

Reinforcing Strength $f_y := 60 \cdot ksi$

Steel Framing Strength $F_y := 50 \cdot ksi$

Steel Modulus of Elasticity $E := 29000 \cdot ksi$

Lateral Earth Pressure Coeff (use "at-rest") $k_o := 0.8$ for clay fill

$k_o := 0.5$ for granular fill

Existing mass concrete - bed rock interface bonding $c := 20psi$

sliding friction angle $\alpha := 35°$

Embankment soil cohesion $c := 1000psf$

Concrete slab - embankment soil sliding friction angle $\alpha := 28°$

Dead Loads

Dead loads:

1. Self-weight of structure.
2. Soil/pavement Weight

Design Dead Loads were computed for each structure. Actual values can be found in the design calculations for the specific structure.

Live Loads

Live Loads:

1. Lateral Earth Pressure
2. Traffic Loads (not applicable)
3. Surcharge
4. Walkway Live Loads, if applicable
5. Hydrostatic Loads
6. Ice Forces.
7. Snow Loads
8. Wind Loads
9. Seismic Loads (not applicable)

Design Live Loads were computed for each structure. Actual values can be found in the design calculations for the specific structure.

Lateral Earth Pressures

Lateral Earth Pressures vary for each structure, so design values were computed seperately for each. Use an "at-rest" lateral soil pressure coefficient of 0.5 or 0.8, depending on the soil types, for computing lateral soil pressures.

Surcharge

Assume a 200 psf surcharge to account for compaction equipt or approx 2-ft of soil. Use on earth retaining walls and Spillway walls.

Surcharge Lateral Pressure

Assume a 200 psf surcharge to account for compaction equipt or approx 2-ft of soil. Use "at-rest" lateral soil pressure coefficient of 0.5 or 0.8 for computing lateral soil pressures.

Walkway Live Loads (ASCE - 7)

Use 100 psf uniform Live load or 1000 lb concentrate load.

Hydrostatic Loads

Hydraulic Loads (standing water or ground water) were based on water heights. See individual structure design calcs for specific information.

Ice Loads

EM 1110-2-1612 states that a 5,000 psf load be used over the ice contact area. Assume a 1-ft thick layer of ice = 5 klf.

Note: Apply ice load to top of applicable walls (in addition to lateral water loads)

Snow Loads (ASCE - 7)

Only applies to Walkway and Bridge. Does not control over 100 psf Liveload, so ignore.

Snow loads will accumulate on walkway. Although it is not a "building", use same approach to compute a base snow load.

$p_g := 50 \cdot psf$ ground snow load from Figure 7-1

$C_e := 1.0$ Table 7-2 Exposure C, partially exposed

$C_t := 1.2$ Table 7-3 Unheated structures

$I_{snow} := 1.0$ Table 7-4 Category II

$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_{snow} \cdot p_g$ $p_f = 42 \cdot psf$ Snow Load is less than Live Load (100 psf). Unlikely that walkway could have full LL and snow load at same time. So LL will govern over Snow Load

Wind Loads

See Wind Loads computed below. Will not be used in actual design as combined 0.75 (LL + WIND) does not control.

$K_z := 0.9$ Table 6-3 for 20-ft and Exposure C

$K_{zt} := 1.0$ Section 6.5.7.2

$K_d := 0.85$ Table 6-4 for "solid sign"

$V_{wind} := 90$

$I_{wind} := 1.0$

$$q_z := 0.00256\cdot(K_z)\cdot(K_{zt})\cdot(K_d)\cdot(V_{wind})^2\cdot(I_{wind})\cdot psf = 15.863\cdot psf$$

$G := 0.85$ Section 6.5.8

$C_f := 1.7$ Figure 6-20 for sign with clearance ratio = 0.5, and Aspect Ratio = 10

$F_{wind} := q_z\cdot(G)\cdot(C_f) = 22.922\cdot psf$ wind pressure

Reference Material

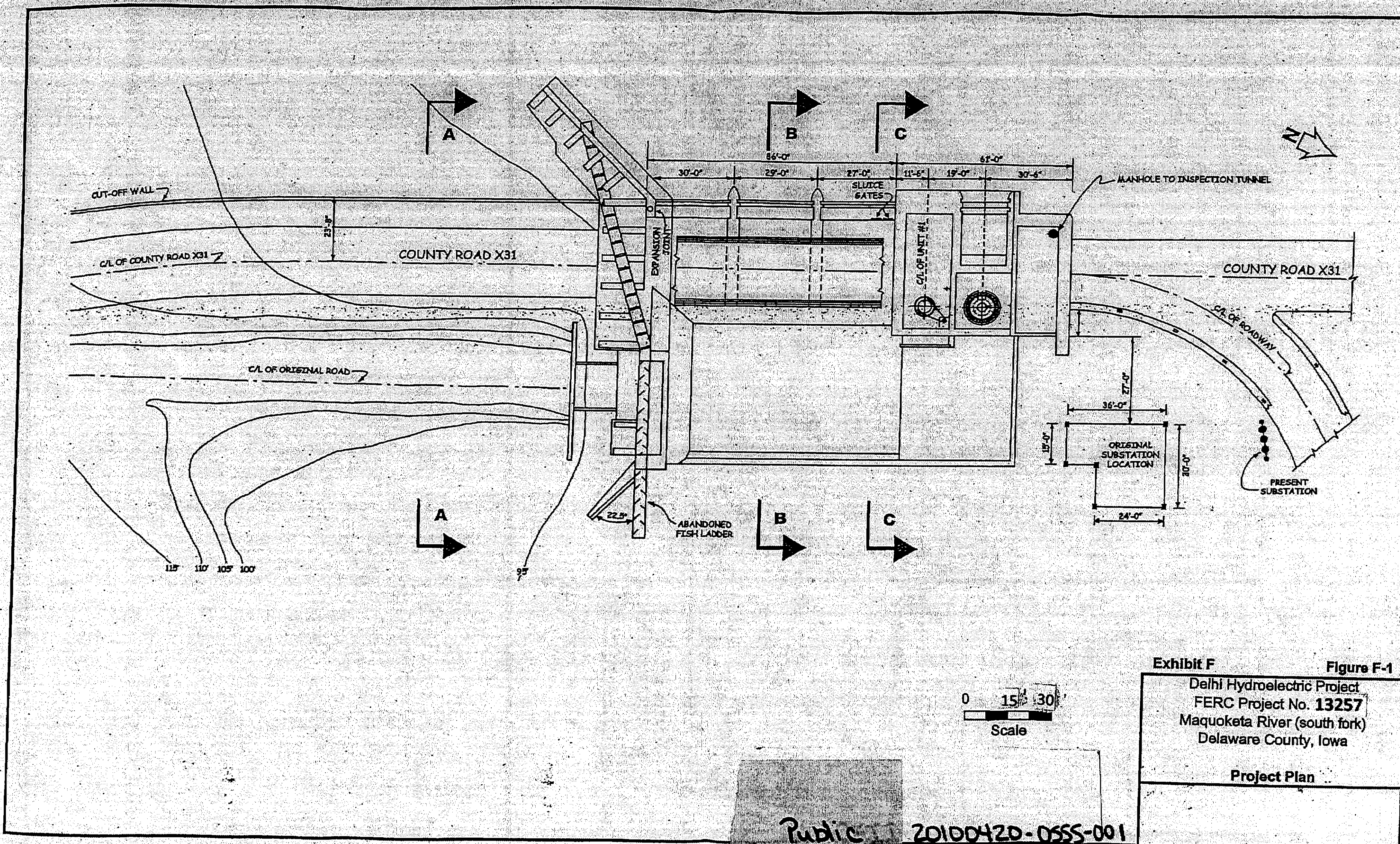


Exhibit F

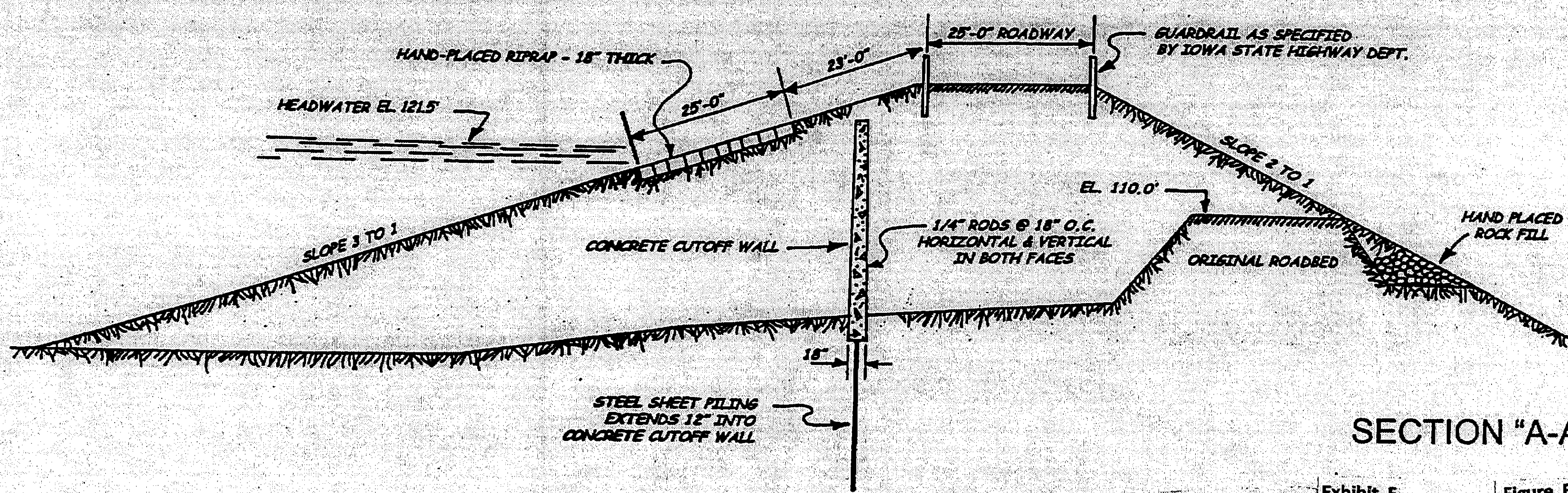
Figure F-1

Delhi Hydroelectric Project
FERC Project No. 13257
Maquoketa River (south fork)
Delaware County, Iowa

Project Plan

Public 20100420-0555-001

EXHIBIT F-2
CROSS-SECTIONS OF DAM



SECTION "A-A"

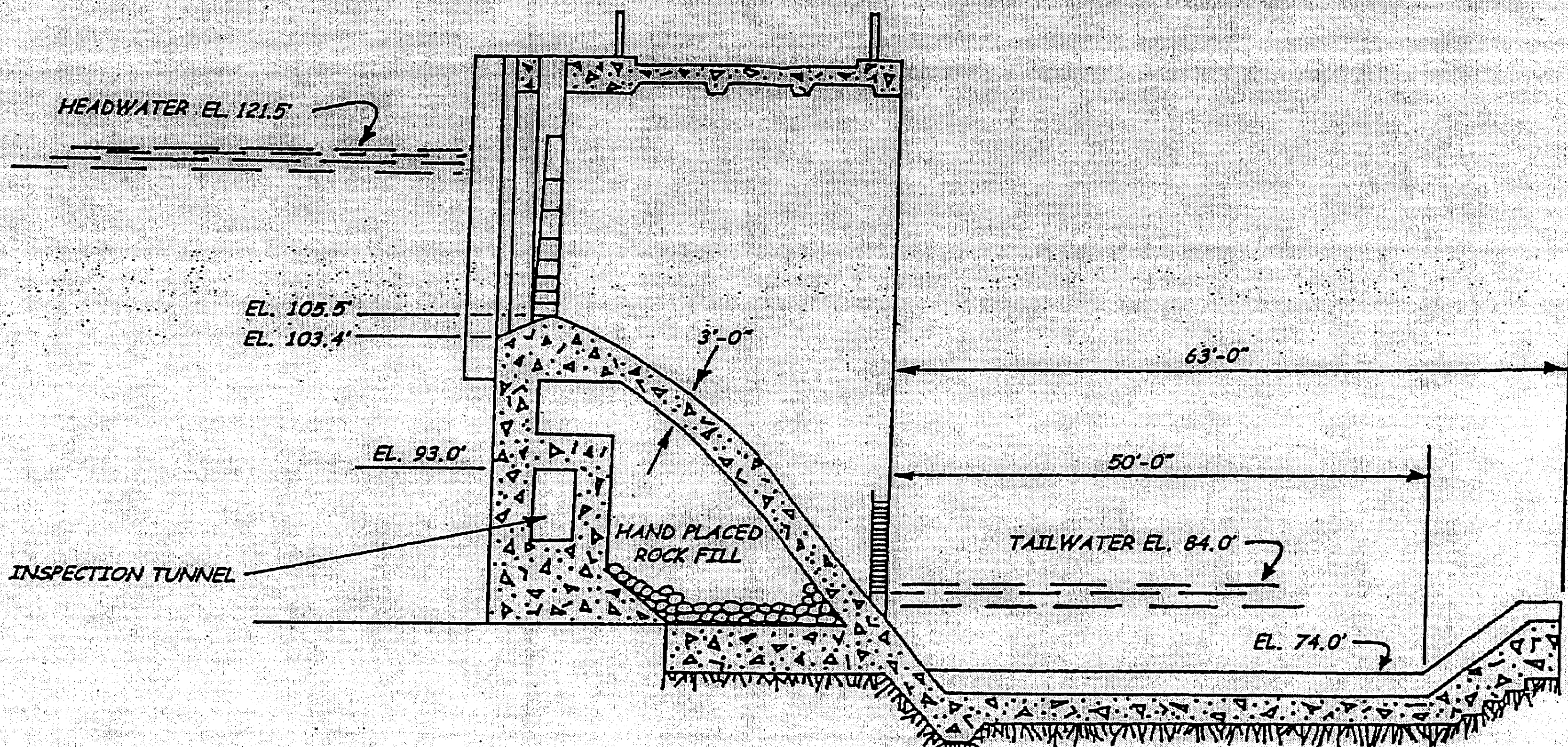
0 12
Scale

Exhibit F	Figure F-2

Delhi Hydroelectric Project
FERC Project No. 13257
Maquoketa River (south fork)
Delaware County, Iowa

Project Plan

20100420-0555-002

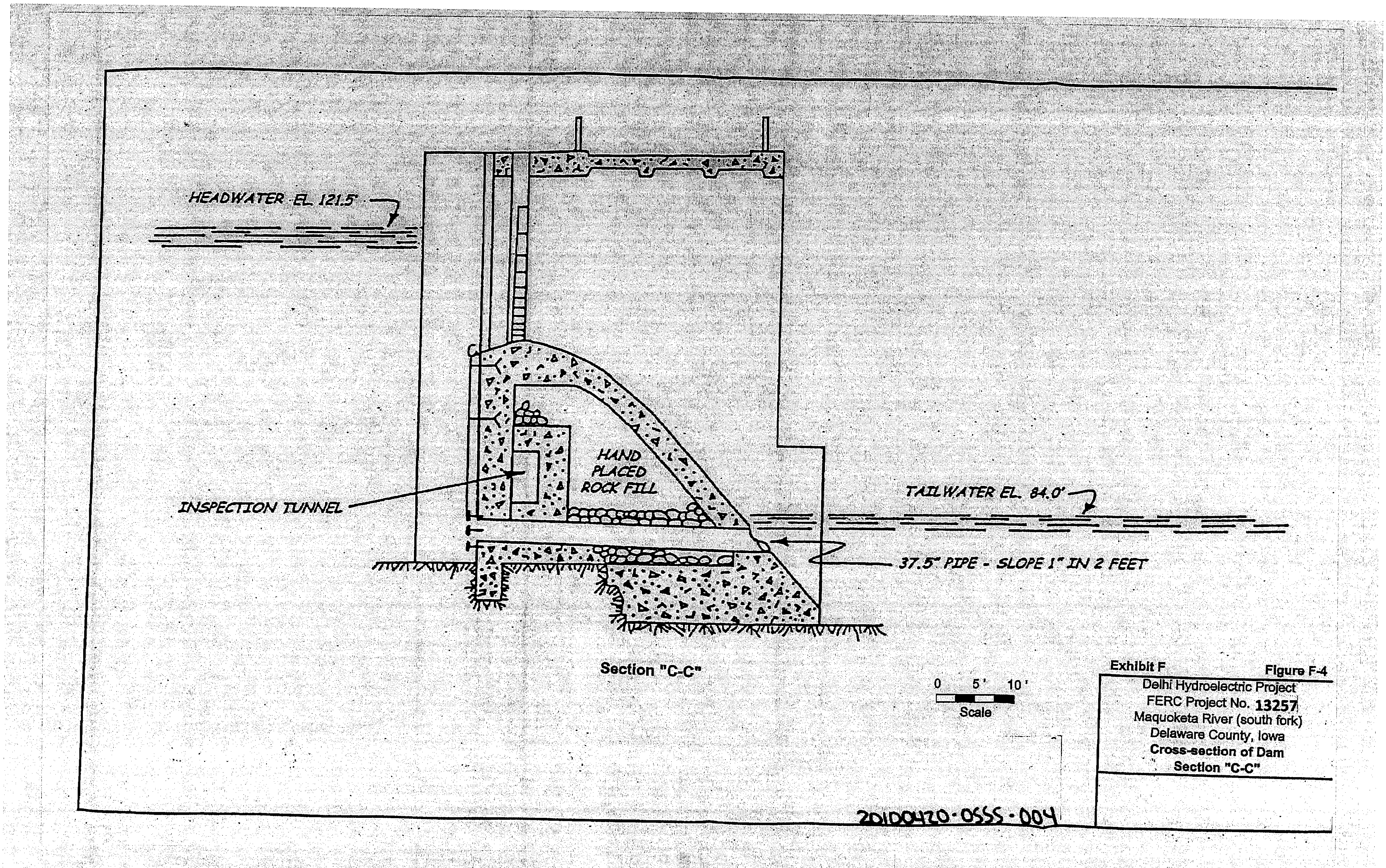


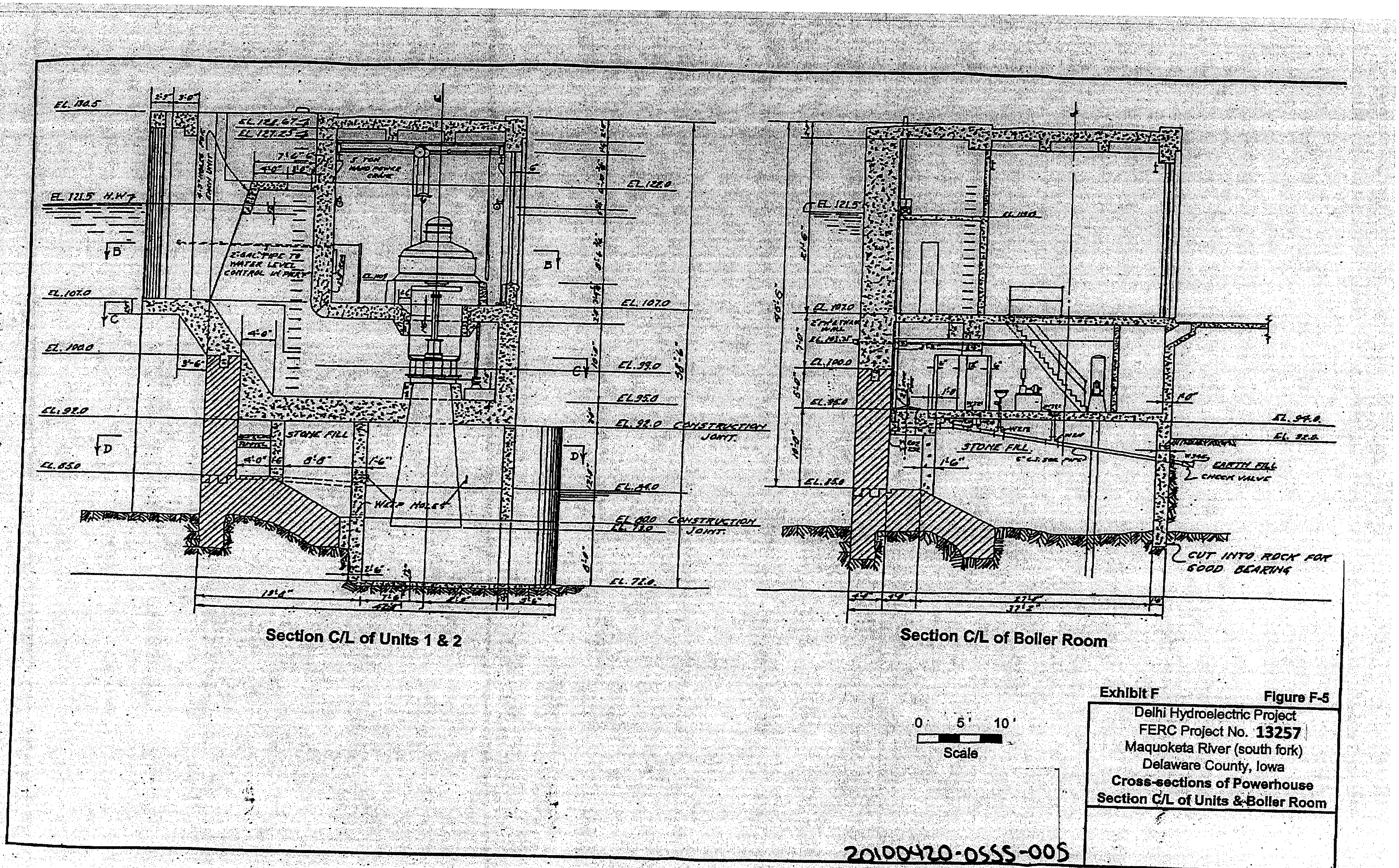
Section "B-B"

Exhibit F Figure F-3

Delhi Hydroelectric Project
FERC Project No. 13257
Maquoketa River (south fork)
Delaware County, Iowa
Cross-section of Dam
Section "B-B"

20100420-0555-003


HEADWATER EL 121.5'
HAND PLACED ROCK FILL
INSPECTION TUNNEL
TAILWATER EL. 84.0'
37.5" PIPE - SLOPE 1" IN 2 FEET
Section "C-C"
0 5' 10'
Scale
Exhibit F
Figure F-4
Delhi Hydroelectric Project
FERC Project No. 13257
Maquoketa River (south fork)
Delaware County, Iowa
Cross-section of Dam
Section "C-C"
20100420-0555-004


Section C/L of Units 1 & 2
Section C/L of Boiler Room
0 5' 10'
Scale
Exhibit F
Figure F-5
Delhi Hydroelectric Project
FERC Project No. 13257
Maquoketa River (south fork)
Delaware County, Iowa
Cross-sections of Powerhouse
Section C/L of Units & Boiler Room
20100420-0555-005

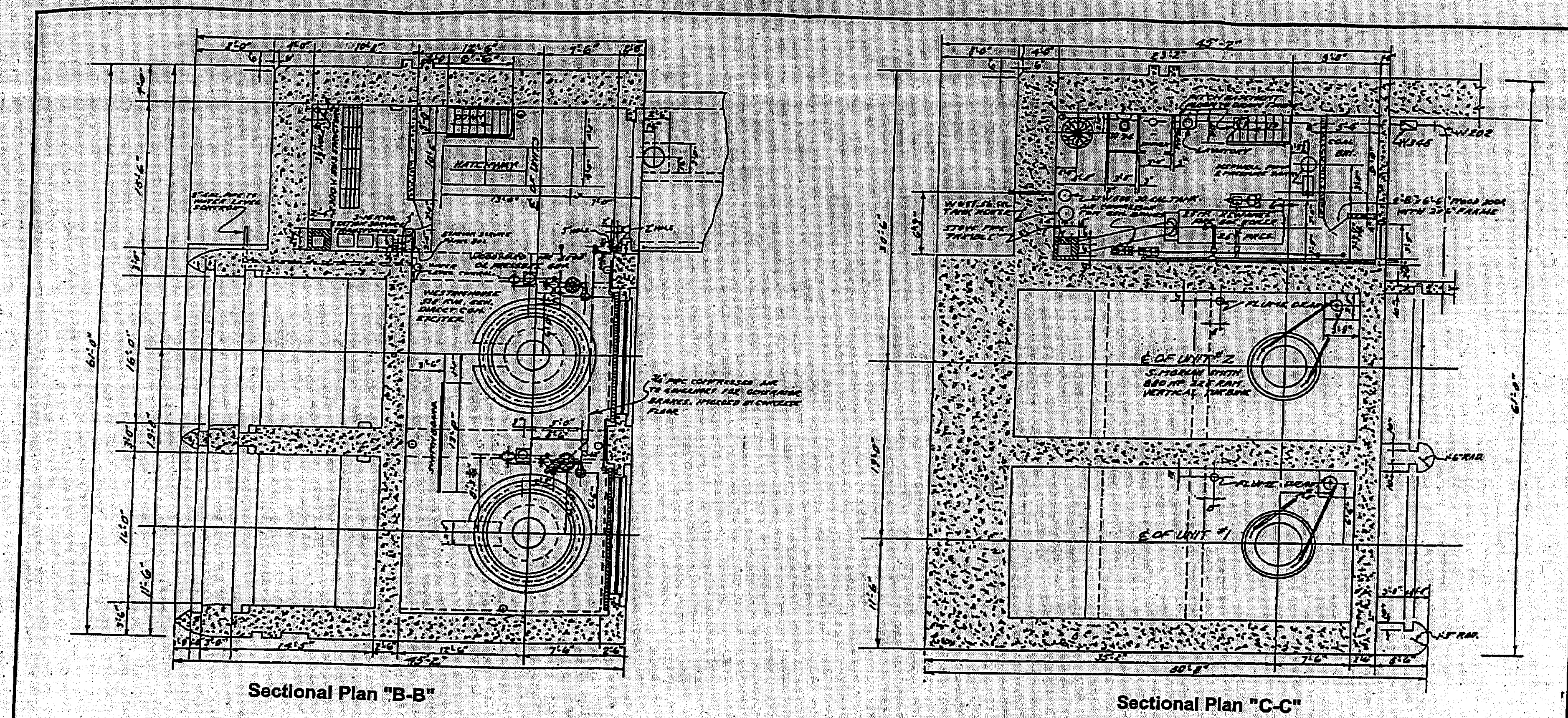
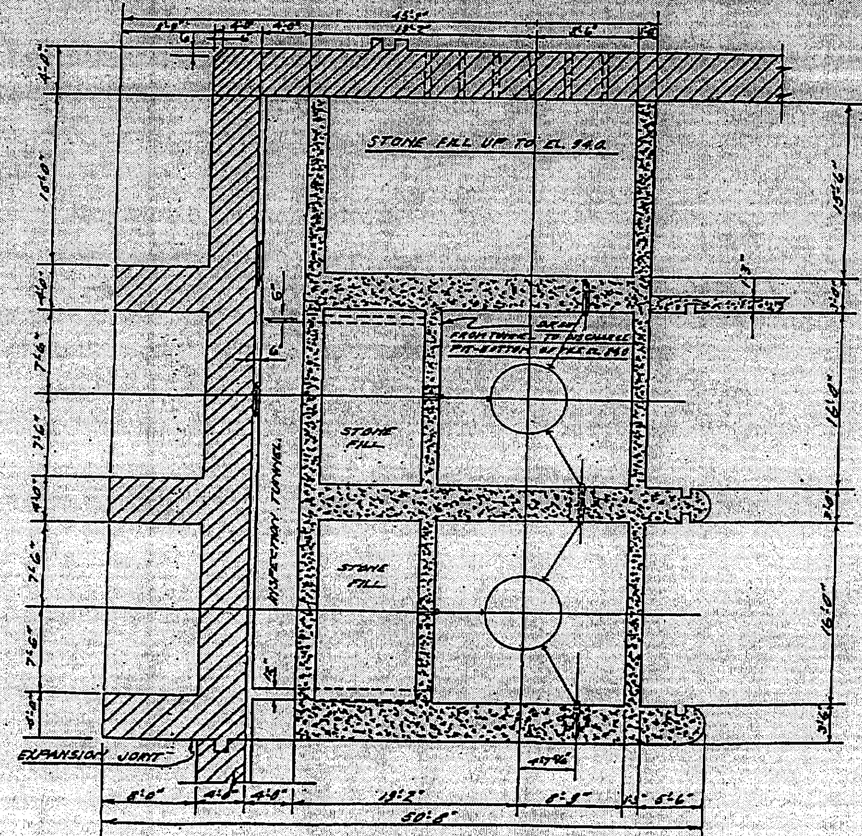


Exhibit F Figure F-6

Delhi Hydroelectric Project
FERC Project No. 13257
Maquoketa River (south fork)
Delaware County, Iowa
Sectional Plans of Powerhouse
Sectional Plans "B-B" & "C-C"

20100420-0555-006



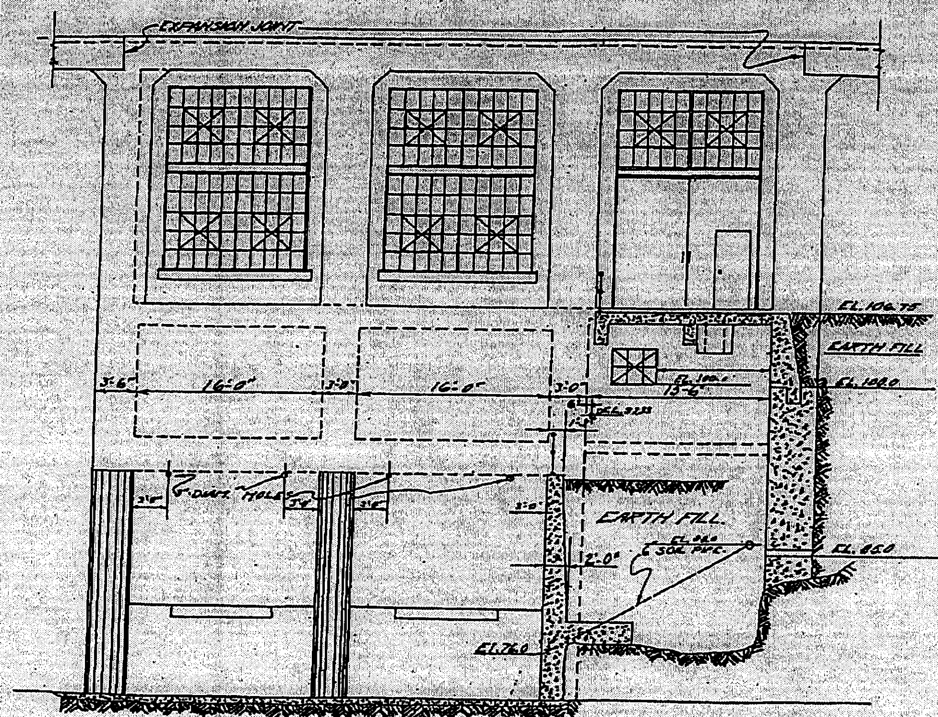
Sectional Plan "D-D"

0 5' 10'
Scale

Exhibit F **Figure F-7**

Delhi Hydroelectric Project
FERC Project No. **13257**
Maquoketa River (south fork)
Delaware County, Iowa
Sectional Plan of Powerhouse
Sectional Plan "D-D"

20100420-0555-007



Elevation

Exhibit F

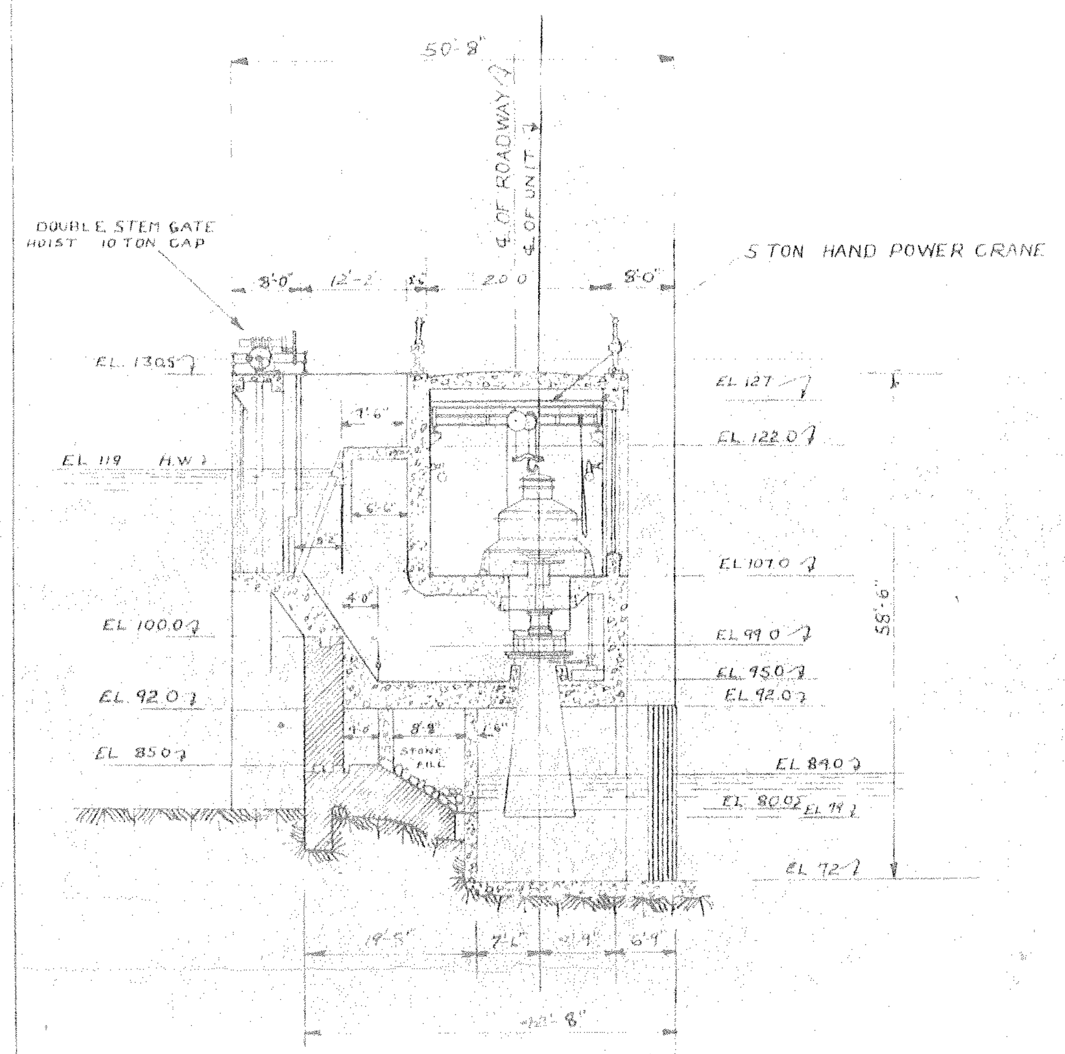
Figure F-8

Delhi Hydroelectric Project
FERC Project No. 13257
Maquoketa River (south fork)
Delaware County, Iowa

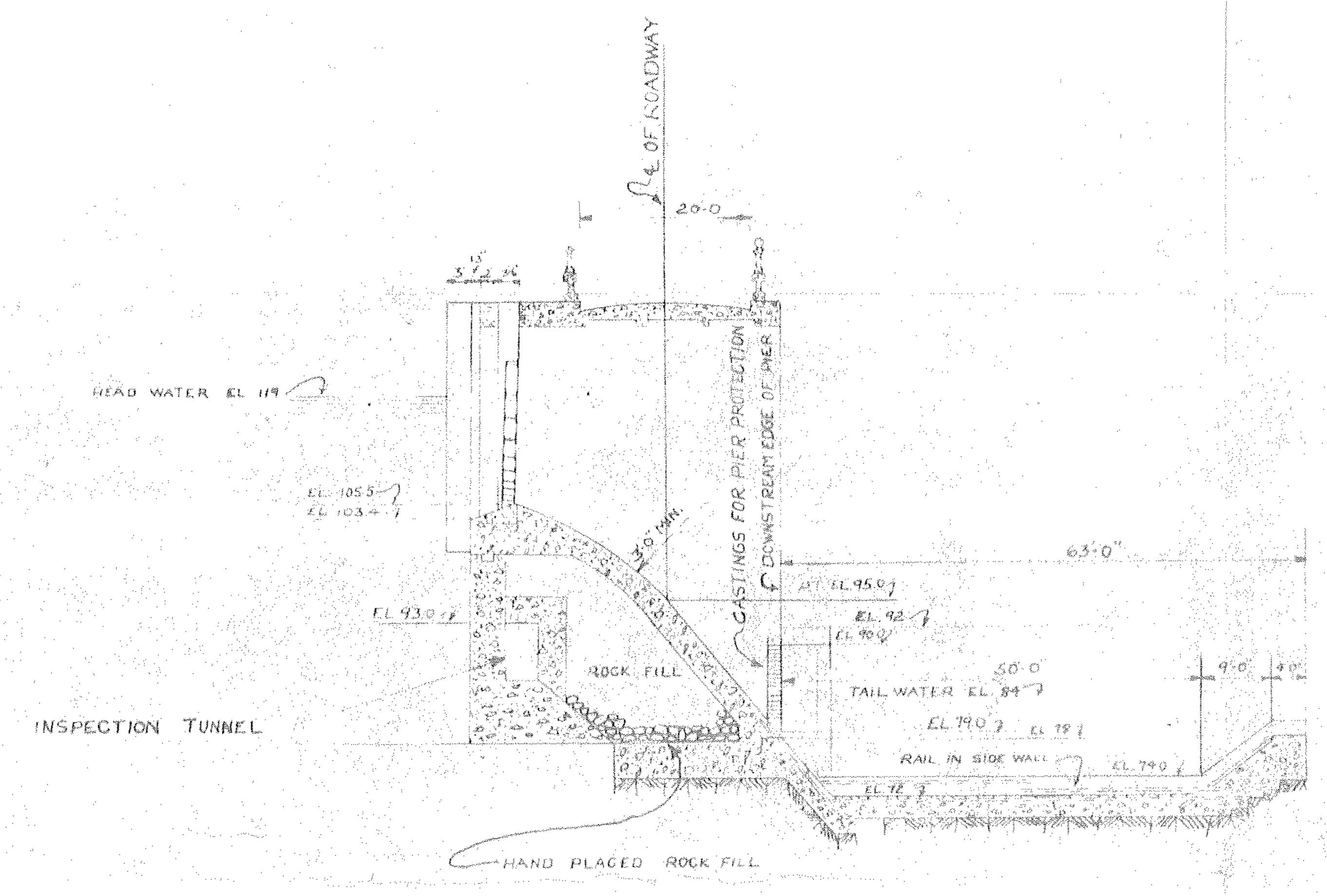
Elevation of Powerhouse

20100420-DSSS-008

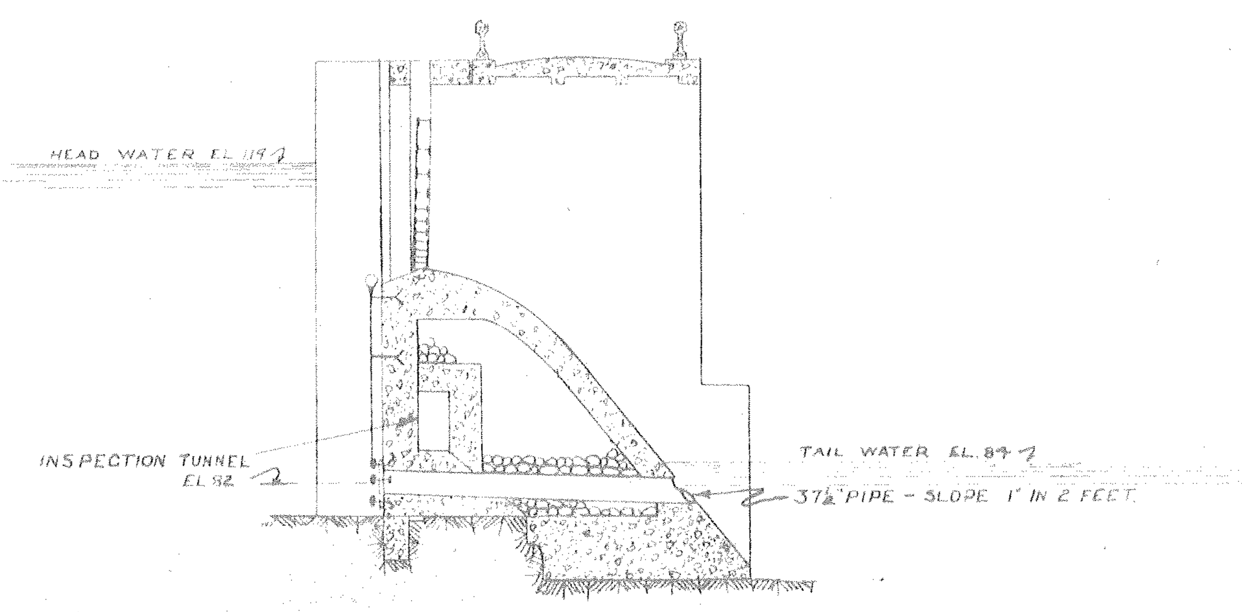
REVISED	GENERAL DRAWING POWER HOUSE DELHI HYDRO-ELECTRIC PLANT MAQUOKETA RIVER NEAR DELHI IOWA INTERSTATE POWER COMPANY DUBUQUE IOWA		
	UTILITIES POWER & LIGHT CORP ENGINEERS — CONSTRUCTORS 327 SO. LASALLE ST CHICAGO, ILL.		
	DRAWN BY ERLANDZON TRACED BY O.H.M. APPROVED BY	SCALE: 1/8" = 1'-0" DATE: 2-2-27	1-1751



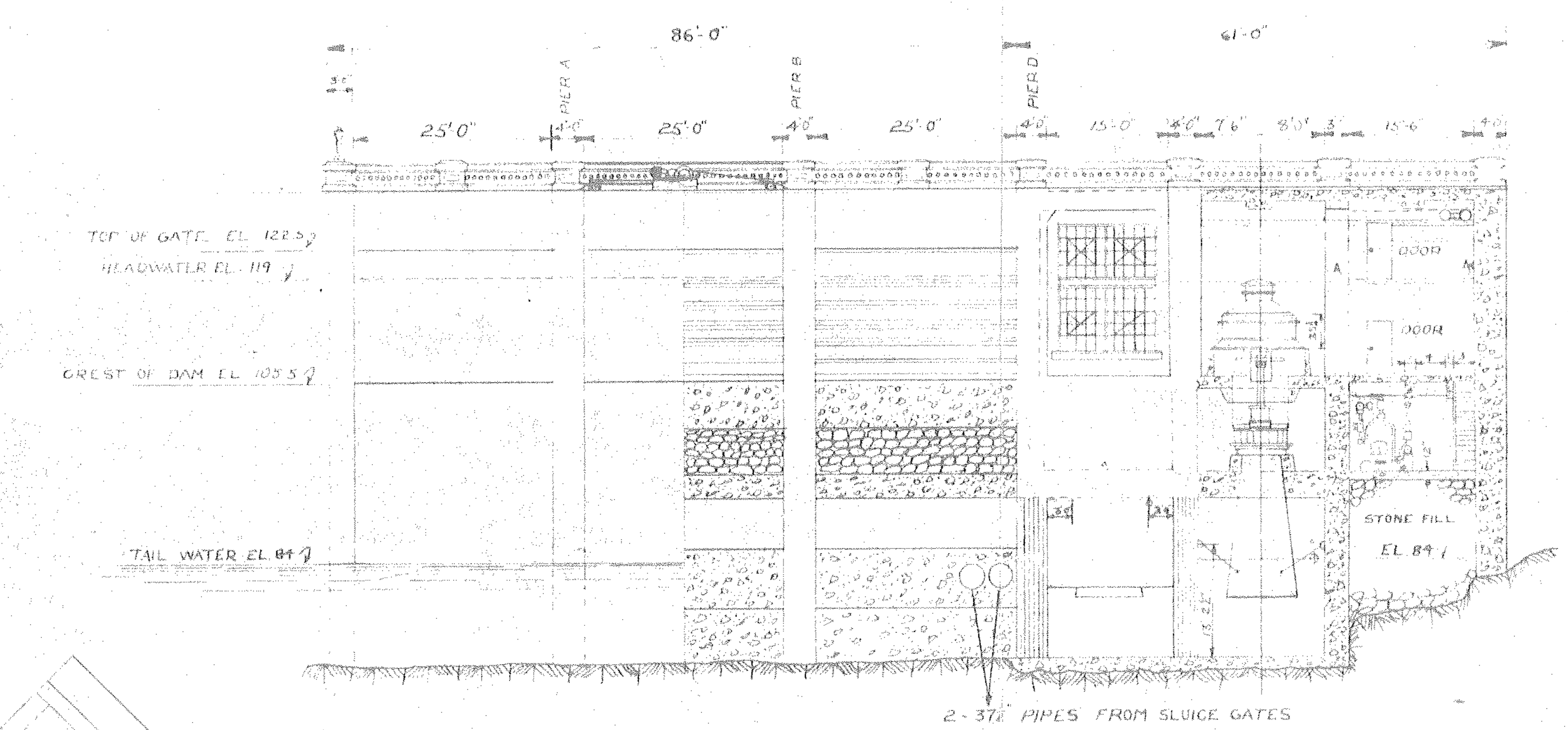
SECTION THRU POWER HOUSE ON ℄ OF UNITS 1&2



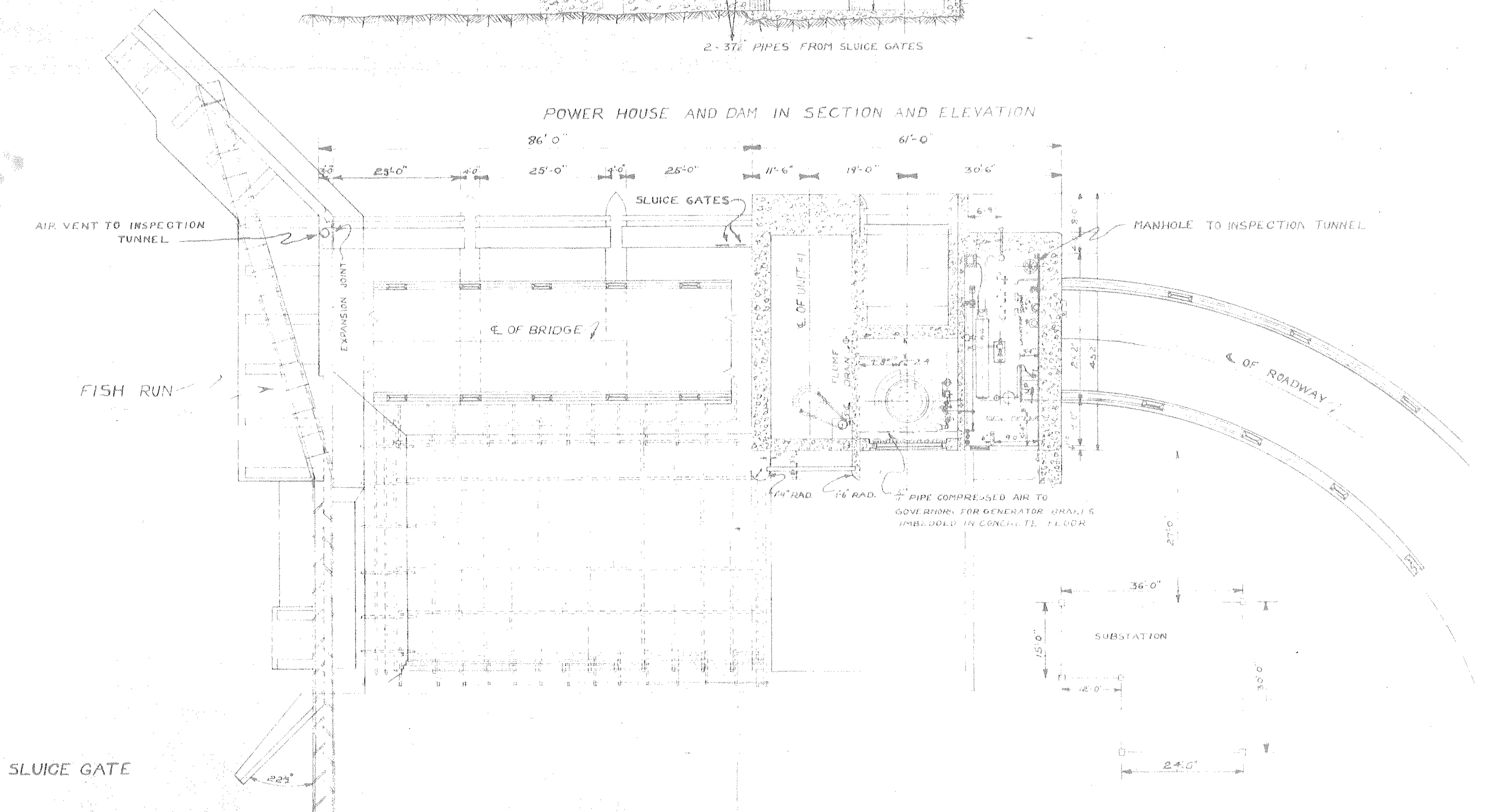
SECTION THRU DAM AT PIERS A&B



SECTION THRU DAM AT PIER D SHOWING SLUICE GATE



POWER HOUSE AND DAM IN SECTION AND ELEVATION



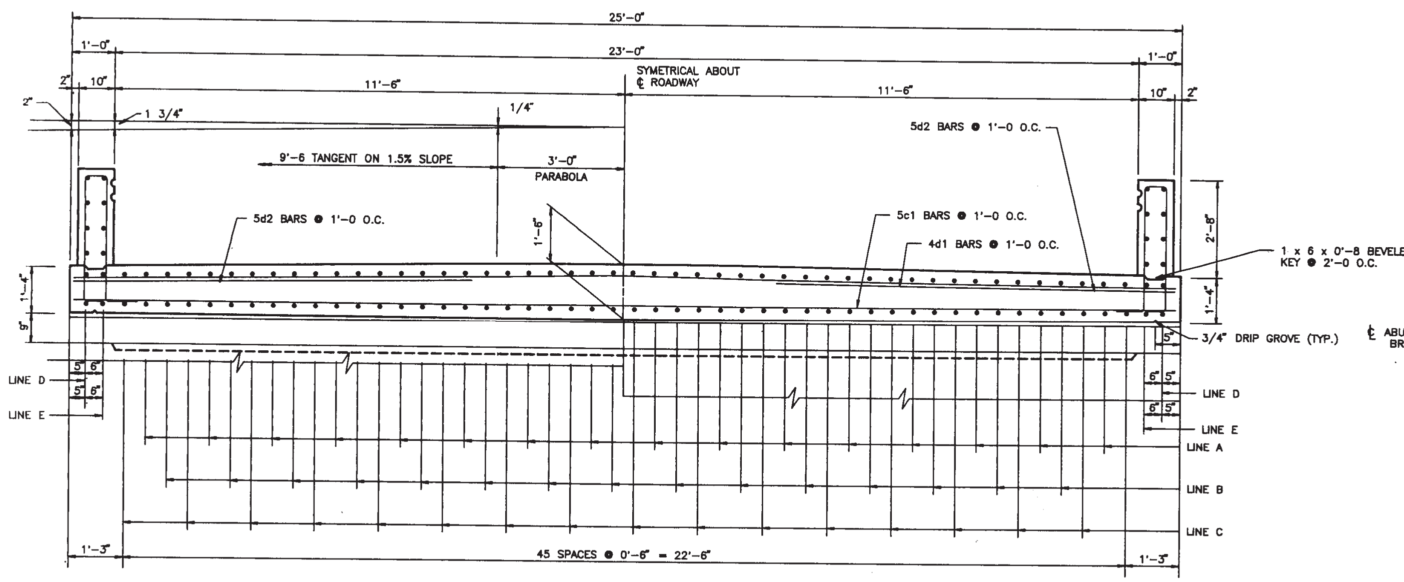
PLAN OF POWER PLANT - DELHI.

THIS MAP IS PART OF THE APPLICATION
FOR A LICENSE MADE BY THE UNDERSIGNED
THIS 8th DAY OF FEBUARY, 1963.
INTERSTATE POWER COMPANY
BY [signature]
EXECUTIVE VICE PRESIDENT

EXHIBIT L

INTERSTATE POWER COMPANY
DELHI HYDRO-ELECTRIC GENERATING STATION
LOCATION NEAR DELHI, DELAWARE COUNTY, IOWA
POND ACREAGE 538
ALL PROPERTY OWNED IN FEE

DRAWN BY W.R.P.	SCALE 1/16"=1'-0"	A-3610
CHECKED BY [initials]	DATE 1/14/63	
APPROVED BY [initials]	REVISED	



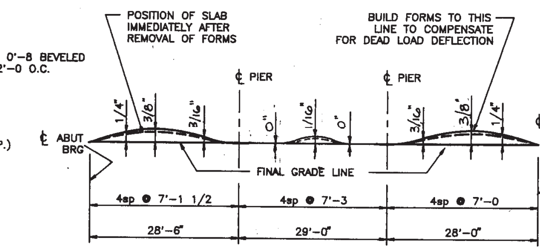
HALF SECTION NEAR PIER

HALF SECTION NEAR ABUTMENT

NOTE: MATCH EXISTING CROWN AT POWERHOUSE WALL

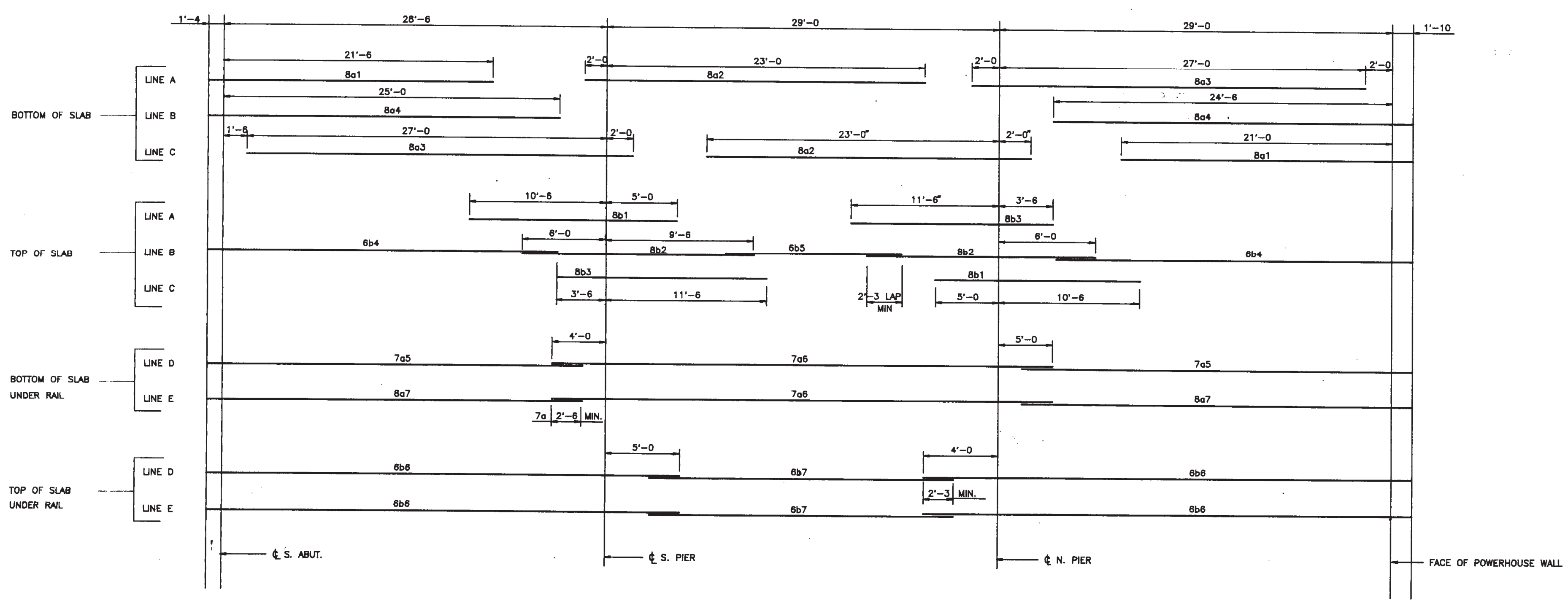
NOTE: TOP LONGITUDINAL REINFORCING STEEL IS TO BE PARALLEL TO AND 2 1/2" CLEAR BELOW TOP OF SLAB. BOTTOM LONGITUDINAL REINFORCING STEEL IS TO BE PARALLEL TO AND 1 1/2" CLEAR ABOVE BOTTOM OF SLAB. REINFORCING STEEL IS TO BE SECURELY WIRED IN PLACE AND ADEQUATELY SUPPORTED ON METAL BAR CHAIRS BEFORE CONCRETE IS PLACED.

NOTE: POUR RAIL & CURE BEFORE FALSEWORK IS STRIPPED.



FORM CAMBER DIAGRAM

THIS DIAGRAM SHOWS THE CAMBER REQUIRED FOR THE ANTICIPATED ULTIMATE DEAD LOAD DEFLECTION AND INCLUDES PLASTIC FLOW. THESE FIGURES DO NOT INCLUDE ANY ALLOWANCE FOR SETTLEMENT OF FORMS

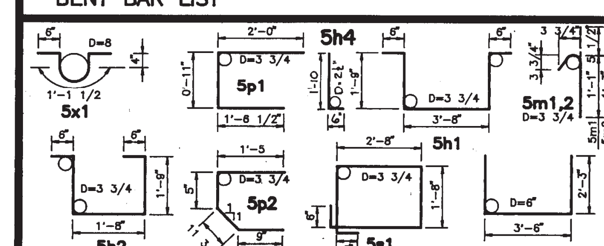


LONGITUDINAL REINFORCING BAR LAYOUT

REINFORCING BAR LIST

BAR	LOCATION	SHAPE	NO.	LENGTH	WEIGHT
8a1	SLAB LONGITUDINAL BOTTOM		32	22'-10	1951
8a2	SLAB LONGITUDINAL BOTTOM		32	25'-0	2136
8a3	SLAB LONGITUDINAL BOTTOM		32	29'-0	2478
8a4	SLAB LONGITUDINAL BOTTOM		30	26'-4	2109
7a5	SLAB LONGITUDINAL BOTTOM UNDER RAIL		4	28'-4	232
7a6	SLAB LONGITUDINAL BOTTOM UNDER RAIL		4	38'-0	311
8a7	SLAB LONGITUDINAL BOTTOM UNDER RAIL		4	28'-4	303
*8b1	SLAB LONGITUDINAL TOP		32	15'-6	1324
*8b2	SLAB LONGITUDINAL TOP		30	15'-6	1242
*6b4	SLAB LONGITUDINAL TOP		30	27'-1	1220
*8b3	SLAB LONGITUDINAL TOP		32	15'-0	1282
*6b5	SLAB LONGITUDINAL TOP		15	14'-6	327
*6b6	SLAB LONGITUDINAL TOP		8	34'-10	419
*6b7	SLAB LONGITUDINAL TOP		4	24'-6	147
5c1	SLAB TRANSVERSE BOTTOM		105	24'-8	2701
5c2	PIER TOP		15	24'-8	386
*4d1	SLAB TRANSVERSE TOP		94	24'-8	1549
*5d2	SLAB TRANSVERSE, TOP, UNDER RAIL		174	9'-6	1724
5h1	PIER HAUNCH STIRRUPS		48	8'-2	409
7h2	PIER HAUNCH ENDS		4	8'-0	65
5h3	PIER HAUNCH STIRRUPS		24	6'-2	154
5h4	SLAB, VERT. AT JOINT		48	2'-4	117
5m1	DOWELS TO EXIST. CONCRETE		192	1'-7	317
5m2	DOWELS AT EXP. JOINT		24	1'-9	44
5p1	PAVING SUPPORT TRANSVERSE		24	4'-6	113
5p2	PAVING SUPPORT TRANSVERSE		24	3'-7	90
*5s1	ABUT. HOOPS		24	9'-8	242
5x1	PAVING BLOCK HOOPS		8	2'-10	24
5n1	PIER, TOP, TRANSVERSE		72	3'-6	263
5n2	PIER, TOP, TRANVERSE		24	2'-0	50
*	CONCRETE RAIL REINFORCING				6,211
	REINFORCING STEEL – EPOXY COATED			TOTAL (LBS)	15,687
	REINFORCING STEEL – NONCOATED			TOTAL (LBS)	14,253

BENT BAR LIST



NOTE: ALL DIMENSIONS ARE OUT TO OUT. D=PIN DIAMETER

EPOXY COATING NOTE

ALL TOP OF SLAB REINFORCING STEEL, BOTH LONGITUDINAL AND TRANSVERSE BARS, SHALL BE EPOXY COATED IN ACCORDANCE WITH CURRENT SUPPLEMENTAL SPECIFICATIONS.
THE BARS MARKED (*) ARE TO BE EPOXY COATED

LAKE DELHI BRIDGE REPAIRS

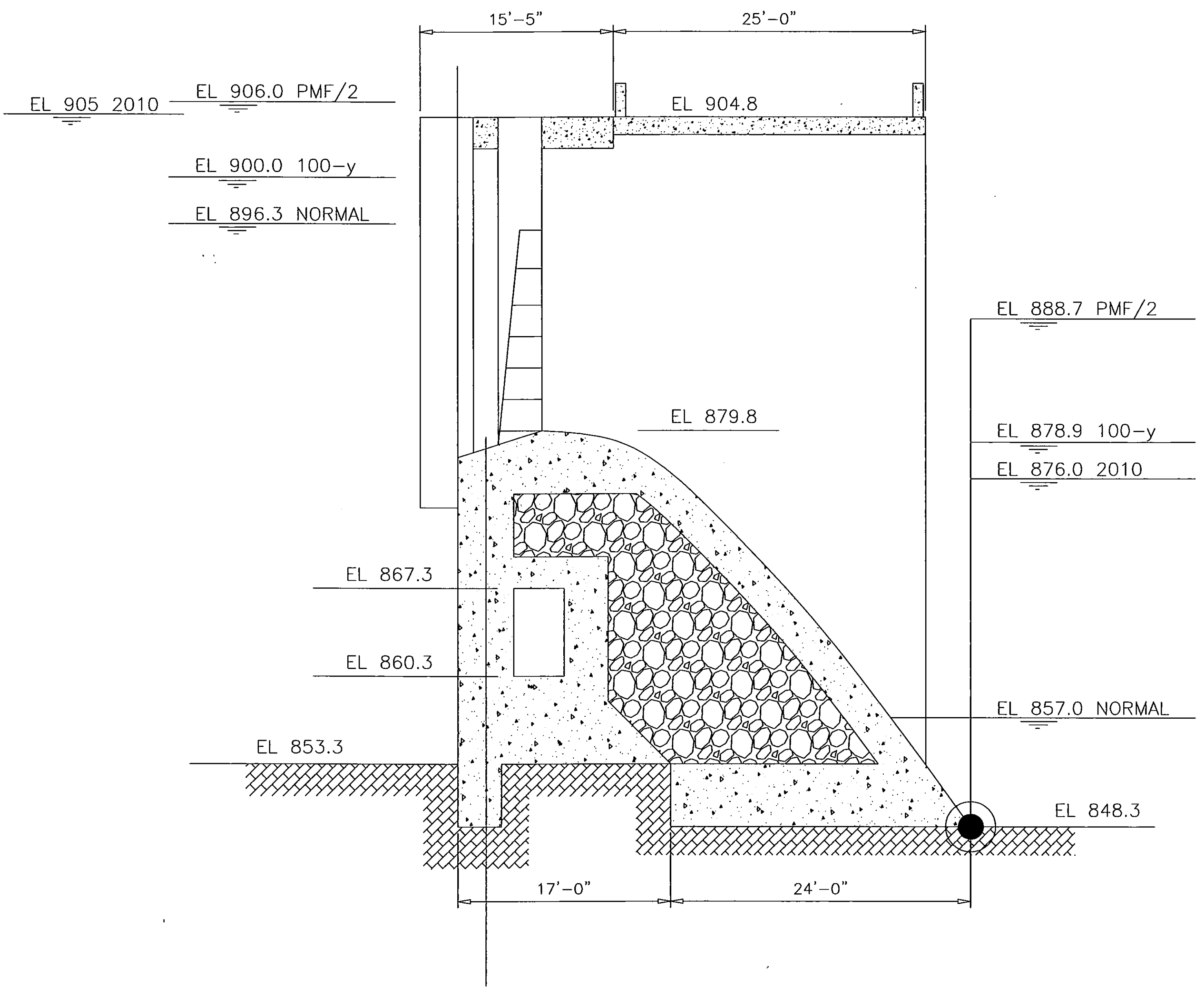
SUPERSTRUCTURE DETAILS

DELAWARE COUNTY IOWA

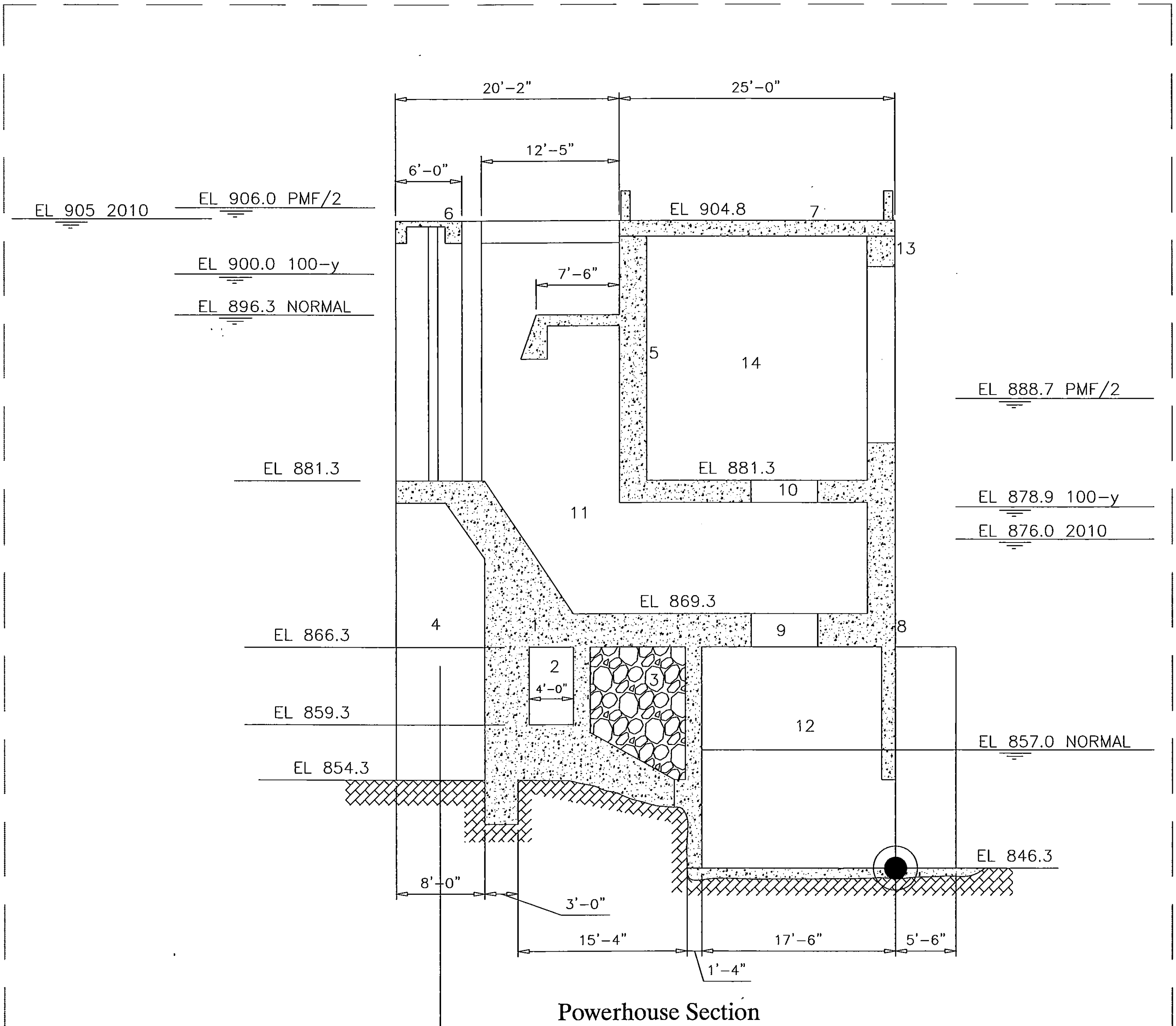
SHEET 9 OF 16

BRITSON CONSULTANTS INC. STRUCTURAL ENGINEERS 929 NINTH STREET WEST DES MOINES, IOWA (515) 224-4949

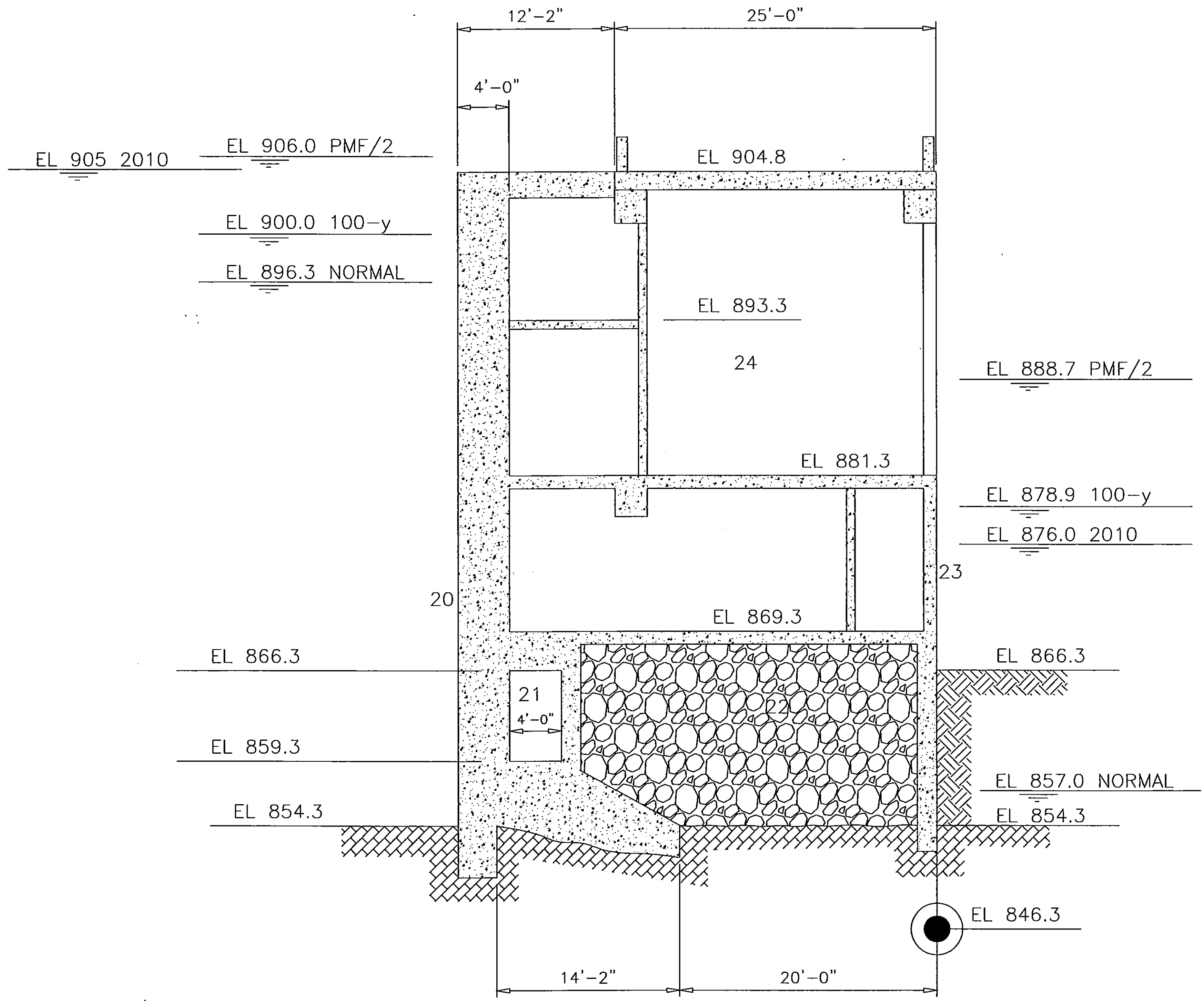
B.C.I. NO. 11888 DESIGN NO. 5190 DELAWARE COUNTY PROJECT NO. BHS-4886(1)--63-28 FILE NO. 54139



Spillway Section



Powerhouse Section
(Turbine Centerline)



Powerhouse Section
(Boiler Room Centerline)

TABLE 1
Ultimate Friction Factors and Adhesion for Dissimilar Materials

Interface Materials	Friction factor, tan [delta]	Friction angle [delta] degrees
Mass concrete on the following foundation materials:		
Clean sound rock................................	0.70	35
Clean gravel, gravel-sand mixtures, coarse sand...	0.55 to 0.60	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel...................	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine to medium sand..	0.35 to 0.45	19 to 24
Fine sandy silt, nonplastic silt.................	0.30 to 0.35	17 to 19
Very stiff and hard residual or preconsolidated clay..	0.40 to 0.50	22 to 26
Medium stiff and stiff clay and silty clay........	0.30 to 0.35	17 to 19
(Masonry on foundation materials has same friction factors.)		
Steel sheet piles against the following soils:		
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls..........................	0.40	22
Clean sand, silty sand-gravel mixture, single size hard rock fill................................	0.30	17
Silty sand, gravel or sand mixed with silt or clay	0.25	14
Fine sandy silt, nonplastic silt.................	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls..........................	0.40 to 0.50	22 to 26
Clean sand, silty sand-gravel mixture, single size hard rock fill................................	0.30 to 0.40	17 to 22
Silty sand, gravel or sand mixed with silt or clay	0.30	17
Fine sandy silt, nonplastic silt.................	0.25	14
Various structural materials:		
Masonry on masonry, igneous and metamorphic rocks:		
Dressed soft rock on dressed soft rock..........	0.70	35
Dressed hard rock on dressed soft rock..........	0.65	33
Dressed hard rock on dressed hard rock..........	0.55	29
Masonry on wood (cross grain)....................	0.50	26
Steel on steel at sheet pile interlocks..........	0.30	17

Interface Materials (Cohesion)	Adhesion c+a, (psf)
Very soft cohesive soil (0 - 250 psf)	0 - 250
Soft cohesive soil (250 - 500 psf)	250 - 500
Medium stiff cohesive soil (500 - 1000 psf)	500 - 750
Stiff cohesive soil (1000 - 2000 psf)	750 - 950
Very stiff cohesive soil (2000 - 4000 psf)	950 - 1,300

STATISTICAL ANALYSIS OF THE STRENGTH OF THE CONTACT BETWEEN CONCRETE DAMS AND ROCK FOUNDATIONS

K.Y. Lo and A. Hethy

Geotechnical Research Centre, Civil Engineering Department
The University of Western Ontario, London, Ontario, N6A 5B9

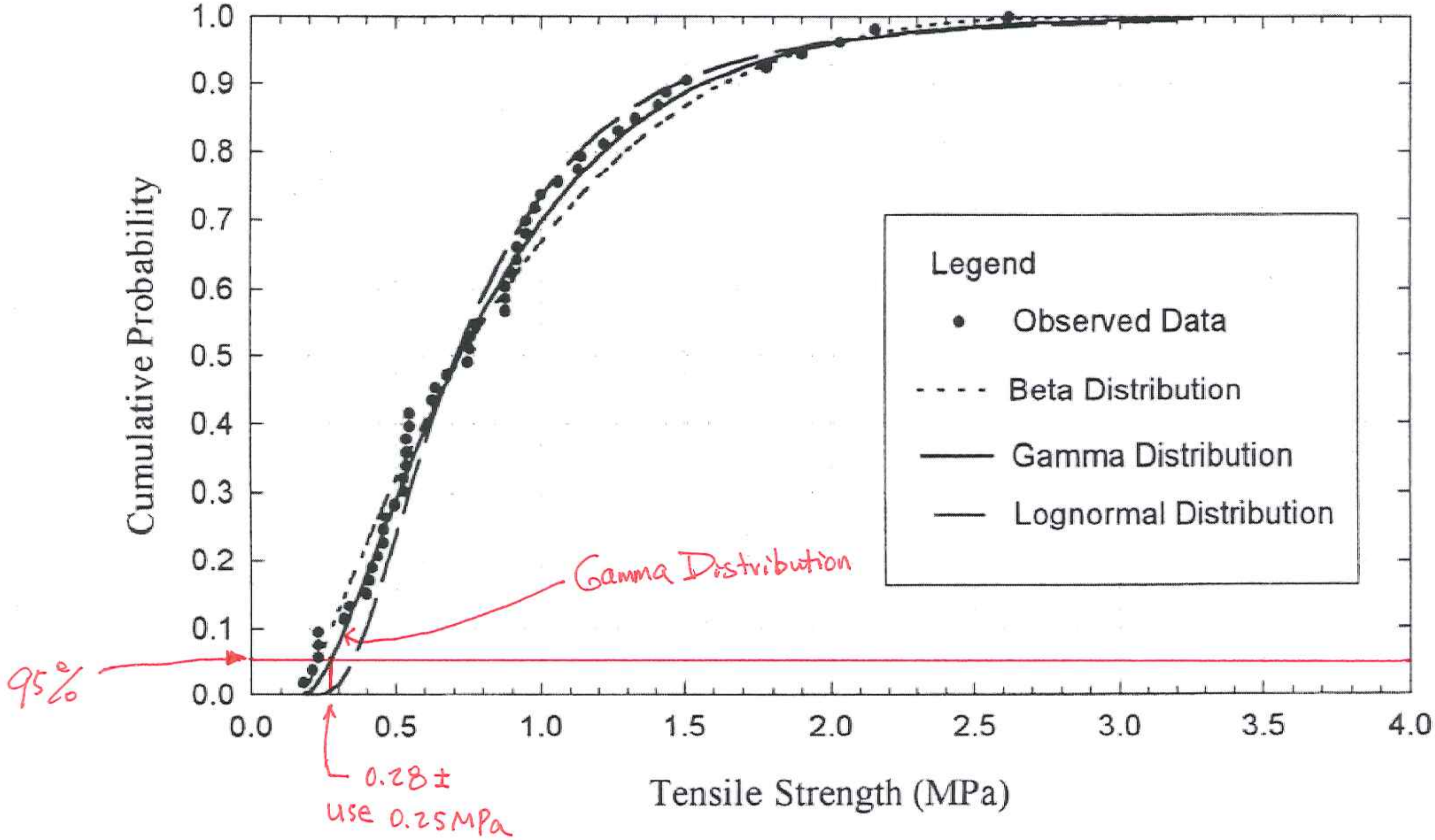


FIGURE 5. Beta, gamma, and lognormal distributions fitted to the tensile strength of intact interface of concrete dams on rock foundations

THE CANADIAN DAM ASSOCIATION
P.O. Box 4490 S. Edmonton Postal Sta.
Edmonton, Alberta, T6E 4X7
Tel: (403) 422-1356, http://www.cda.ca

1[ST] ANNUAL CONFERENCE
System Stewardship for Dams & Reservoirs
Halifax NS, Sept.27- Oct.1, 1998

Existing Spillway

Stability Check

Stanley Consultants INC.

				Job No.	23601 Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Dam / Powerhouse Stability
Checked by:	E. Daly	Date:	12/19/11		Spillway Stability LC_1
Approved by:		Date:		Sheet No.	of

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Mass concrete	798.8	1	75	150	8985.9	298.2	223301
Less Placed Stone	299.1	1	75	-35	-785.2	271.2	-17746
Tunnel	28.0	1	83	-150	-348.6	414.0	-12027
Piers (to bottom)	1966.9	1	11	150	3245.4	279.0	75456
Bridge (thru)	39.9	1	73.5	150	439.5	193.0	7068
Gate Platform (thru)	14.2	1	73.5	150	156.2	377.0	4907
Platform BM (thru)	5.0	1	73.5	150	55.1	465.0	2136
Total					**11748.3**		**283095**

Top of Bridge =	904.8 ft		20 psi
Total Length of Spillway =	83.0 ft	Ice Load =	5.0 klf
Bottom of Dam EL =	848.3 ft	L1 =	17.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	24.0 ft
Dam / Foundation Bonding=	2880 psf	step =	5.0 ft
Length of Seepage Path =	46 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	879.8 ft	Foundation Width =	41.0 ft

2. Case I: Normal Operating Condition

Head Water EL = 896.3 ft

Tail Water EL = 857.0 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
	521.2	37.6	19611
	142.4	2.1	300
Total	**663.6**		**19912**

Uplift	39.3	8.7	39.3	0%
Upstream	Head 1 = 43.0	ft		Crack input
Downstream	Head 2 = 8.7	ft		0.00 ft
	Seep Grade = 0.854	ft/ft		29.2

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E. Daly	Date:	12/19/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Spillway Stability LC_1
				Sheet No.	of

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	41	0
uplift 1 (total rectangular_US)	-3786.0	32.5	-123045
uplift 2 (add back triangular_US)	639.4	29.7	18969
uplift 3 (rectangular_DS)	-1081.4	12.0	-12977
uplift 4 (triangular_DS)	-1274.4	16.0	-20390
Total	**-5502.4**		**-137443**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream 1	4788.2	19.3	-92571
Upstream 2 at Step	756.3	2.5	-1891
Downstream	-196.0	2.9	568
Total	**5348.4**		**-93894**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	1226.4	13.8	-16965
Upstream - Ice	415.0	48.0	-19920
Total	**1641.4**		**-36885**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	11748.3		283095
Weight of Water	663.6		19912
Driving of Water		5348.4	-93894
Uplift at Efficiency = 0	-5502.4		-137443
Silt & Ice		1641.4	-36885
Rock Anchor US @ 38.75 ft.	**0.0**		**0**
Total	**6909.6**	**6989.8**	**34785.4**

Stanley Consultants INC.

				Job No.	23601 Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Dam / Powerhouse Stability
Checked by:	E. Daly	Date:	12/19/11		Spillway Stability LC_1
Approved by:		Date:		Sheet No.	of

ΣV =	6909.6	kips
ΣH =	6989.8	kips
ΣM =	34785.4	k-ft

USACE Stability

Force Resultant Location @ Base L =	5.03	ft		NG	Crack
Force Resultant Location Offset e =	15.47	ft			25.90 ft
Foundation Bearing p_{max} =	11024	psf	10000	NG	
Foundation Bearing p_{min} =	0	psf			For Usual LC, Rock Foundation.
Sliding FOS =	1.21		2.00 Reqr'd	NG	

FERC Stability

Force Resultant Location @ Base L =	5.03	ft			
Force Resultant Location Offset e =	15.47	ft			
Foundation Bearing p_{max} =	11024	psf	10000	NG	
Foundation Bearing p_{min} =	0	psf			
Sliding FOS =	1.21		3.00 Reqr'd	NG	For Usual LC, Rock Foundation.
Sliding FOS (No Cohesion) =	0.69		1.50 Reqr'd	NG	

For Usual LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/19/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Spillway Stability LC_2
				Sheet No.	of

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Mass concrete	798.8	1	75	150	8985.9	298.2	223301
Less Placed Stone	299.1	1	75	-35	-785.2	271.2	-17746
Tunnel	28.0	1	83	-150	-348.6	414.0	-12027
Piers (to bottom)	1966.9	1	11	150	3245.4	279.0	75456
Bridge (thru)	39.9	1	73.5	150	439.5	193.0	7068
Gate Platform (thru)	14.2	1	73.5	150	156.2	377.0	4907
Platform BM (thru)	5.0	1	73.5	150	55.1	465.0	2136
Total					**11748.3**		**283095**

Top of Bridge =	904.8 ft		
Total Length of Spillway =	83.0 ft	Ice Load =	0.0 klf
Bottom of Dam EL =	848.3 ft	L1 =	17.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	24.0 ft
Dam / Foundation Bonding=	2880 psf	step =	5.0 ft
Length of Seepage Path =	46 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	879.8 ft	Foundation Width =	41.0 ft

2. Case II: Unusual Flood Discharge Condition

Head Water EL = 900.0 ft

Tail Water EL = 878.9 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
	3705.0	20.0	74100
Total	**3705.0**		**74100**

Uplift	21.1	30.6	21.1	0%
Upstream	Head 1 = 46.7	ft		Crack input
Downstream	Head 2 = 30.6	ft		0.00 ft
	Seep Grade = 0.459	ft/ft		41.6

Stanley Consultants INC.

				Job No. 23601	Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Dam / Powerhouse Stability
Checked by:	E.Daly	Date:	12/19/11		Spillway Stability LC_2
Approved by:		Date:		Sheet No.	of

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	41	0
uplift 1 (total rectangular_US)	-4111.8	32.5	-133632
uplift 2 (add back triangular_US)	343.3	29.7	10184
uplift 3 (rectangular_DS)	-3803.6	12.0	-45643
uplift 4 (triangular_DS)	-684.2	16.0	-10947
Total	**-8256.3**		**-180039**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream on Ogee_Rectangular	2772.4	18.3	-50597
Upstream on Ogee_Triangular	1818.5	13.8	-25157
Upstream on Piers_Triangular	101.8	38.2	-3894
Upstream 2 at Step	1077.5	2.5	-2694
Downstream on Ogee_Rectangular	0.0	15.8	0
Downstream on Ogee_Triangular	-2424.8	10.2	24733
Downstream on Piers_Triangular	0.0	31.2	0
Total	**3345.5**		**-57608**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	1226.4	13.8	-16965
Upstream - Ice	0.0	51.7	0
Total	**1226.4**		**-16965**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	11748.3		283095
Weight of Water	3705.0		74100
Driving of Water		3345.5	-57608
Uplift at Efficiency = 0	-8256.3		-180039
Silt & Ice		1226.4	-16965
Rock Anchor US @ 38.75 ft.	**0.0**		**0**
Total	**7197.0**	**4571.9**	**102583.6**

Stanley Consultants INC.			Job No.	23601	Page No.
			Subject:	Delhi Lake Dam Reconstruction	
Computed by:	Y.Ding	Date: 12/05/11		Dam / Powerhouse Stability	
Checked by:	E.Daly	Date: 12/19/11		Spillway Stability LC_2	
Approved by:		Date:	Sheet No.	of	

ΣV = 7197.0 kips

ΣH = 4571.9 kips

ΣM = 102583.6 k-ft

USACE Stability

Force Resultant Location @ Base L =	14.25 ft	41	**OK**	**Crack**
Force Resultant Location Offset e =	6.25 ft			**0.00 ft**
Foundation Bearing p_{max} =	4048 psf	10000	**OK**	
Foundation Bearing p_{min} =	182 psf			
Sliding FOS =	3.25	1.70 Reqr'd	**OK**	

For Unusual LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	14.25 ft		
Force Resultant Location Offset e =	6.25 ft		
Foundation Bearing p_{max} =	4048 psf	10000	**OK**
Foundation Bearing p_{min} =	182 psf		
Sliding FOS =	3.25	2.00 Reqr'd	**OK**
Sliding FOS (No Cohesion) =	1.10	1.50 Reqr'd	**NG**

For Unusual LC, Rock Foundation.

For Unusual LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/19/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Spillway Stability LC_3
				Sheet No.	of

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Mass concrete	798.8	1	75	150	8985.9	298.2	223301
Less Placed Stone	299.1	1	75	-35	-785.2	271.2	-17746
Tunnel	28.0	1	83	-150	-348.6	414.0	-12027
Piers (to bottom)	1966.9	1	11	150	3245.4	279.0	75456
Bridge (thru)	39.9	1	73.5	150	439.5	193.0	7068
Gate Platform (thru)	14.2	1	73.5	150	156.2	377.0	4907
Platform BM (thru)	5.0	1	73.5	150	55.1	465.0	2136
Total					**11748.3**		**283095**

Top of Bridge =	904.8 ft		
Total Length of Spillway =	83.0 ft	Ice Load =	0.0 klf
Bottom of Dam EL =	848.3 ft	L1 =	17.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	24.0 ft
Dam / Foundation Bonding=	2880 psf	step =	5.0 ft
Length of Seepage Path =	46 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	879.8 ft	Foundation Width =	41.0 ft

2. Case III: Extreme Flood Discharge Condition

Head Water EL = 906.0 ft

Tail Water EL = 888.7 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
	5551.0	20.0	111020
Total	**5551.0**		**111020**

Uplift	17.3	40.4	17.3		0%
Upstream	Head 1 =	52.7	ft		**Crack input**
Downstream	Head 2 =	40.4	ft		**0.00 ft**
	Seep Grade =	0.376	ft/ft		49.4

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	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	41	0
uplift 1 (total rectangular_US)	-4640.0	32.5	-150801
uplift 2 (add back triangular_US)	281.5	29.7	8350
uplift 3 (rectangular_DS)	-5021.8	12.0	-60261
uplift 4 (triangular_DS)	-561.0	16.0	-8976
Total	**-9941.3**		**-211688**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream on Ogee_Rectangular	3595.9	18.3	-65626
Upstream on Ogee_Triangular	1818.5	13.8	-25157
Upstream on Piers_Rectangular	41.6	42.8	-1777
Upstream on Piers_Triangular	126.4	39.0	-4928
Upstream on Gate_Triangular	35.5	55.2	-1958
Upstream 2 at Step	1279.9	2.5	-3200
Downstream on Ogee_Rectangular	-1452.0	15.8	22869
Downstream on Ogee_Triangular	-2569.5	10.5	26980
Downstream on Piers_Triangular	-19.8	34.5	681
Total	**2856.5**		**-52114**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	1226.4	13.8	-16965
Upstream - Ice	0.0	57.7	0
Total	**1226.4**		**-16965**

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Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	11748.3		283095
Weight of Water	5551.0		111020
Driving of Water		2856.5	-52114
Uplift at Efficiency = 0	-9941.3		-211688
Silt & Ice		1226.4	-16965
Rock Anchor US @ 38.75 ft.	**0.0**		**0**
Total	**7358.0**	**4082.8**	**113347.9**

ΣV = 7358.0 kips

ΣH = 4082.8 kips

ΣM = 113347.9 k-ft

USACE Stability

Force Resultant Location @ Base L =	15.40 ft	41	**OK**	**Crack**
Force Resultant Location Offset e =	5.10 ft			**0.00 ft**
Foundation Bearing p_{max} =	3774 psf	13300	**OK**	
Foundation Bearing p_{min} =	550 psf			
Sliding FOS =	3.66	1.30 Reqr'd	**OK**	

For Extreme LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	15.40 ft		
Force Resultant Location Offset e =	5.10 ft		
Foundation Bearing p_{max} =	3774 psf	13300	**OK**
Foundation Bearing p_{min} =	550 psf		
Sliding FOS =	3.66	2.00 Reqr'd	**OK**
Sliding FOS (No Cohesion) =	1.26	1.50 Reqr'd	**NG**

For Extreme LC, Rock Foundation.

For Extreme LC, Rock Foundation. No Cohesion.

Existing Powerhouse

Stability Check

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		Subject:	Delhi Lake Dam Reconstruction
Computed by: Y.Ding	Date: 12/05/11		Dam / Powerhouse Stability
Checked by: E.Daly	Date: 12/19/11		Powerhouse Stability LC_1
Approved by:	Date:	Sheet No.	of

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
1,5,6,7,8,13	560.1	1	41.5	150	3486.4	318.8	92614
2 tunnel	28.0	1	41.5	-150	-174.3	374.0	-5432
3 stone fill	87.7	1	32	-35	-98.2	276.0	-2258
4	191.3	1	12	150	344.3	495.6	14216
9, 10	30.0	1	29.5	150	132.8	120.0	1328
11,12	1111.9	1	9.5	150	1584.5	269.5	35582
14	481.5	1	3	150	216.7	138.8	2507
Equipments					200.0	120.0	2000
20 wall + roof	399.6	1	12.5	150	749.3	361.0	22543
21 tunnel	28.0	1	12.5	-150	-52.5	374.0	-1636
22 stone fill	350.1	1	12.5	115	503.2	169.1	7091
23	140.4	1	12.5	150	263.3	135.0	2961
24 side wall	1988.4	1	4	150	1193.1	224.4	22309
Total					**8348.5**		**193824**

Top of Bridge =	904.8	ft		20	psi
Total Length of Powerhouse =	61.0	ft	Ice Load =	**5.0**	klf
Bottom of Dam EL =	846.3	ft	L1 =	20.0	ft
Dam / Foundation Friction Angle =	35	degrees	L2 =	17.2	ft
Dam / Foundation Bonding=	2880	psf	step =	8.0	ft
Length of Seepage Path =	45.2	ft	Allowable Bearing =	10000	psf
Top Of Crest EL =	881.3	ft	Foundation Width =	37.2	ft

2. Case I: Normal Operating Condition - Dewatered

Head Water EL =	896.3	ft
Tail Water EL =	857.0	ft

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Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	119.8	43.0	5152
Top Water weight 2			
Upward pressure	-254.6	41.0	-10438
Upward pressure at US piers	-251.6	41.2	-10366
Total	**-386.4**		**-15652**

Uplift	39.3	10.7	39.3	0%
Upstream	Head 1 = 42.0	ft		Crack input
Downstream	Head 2 = 10.7	ft		0.00 ft
	Seep Grade = 0.869	ft/ft		25.7

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	37	0
uplift 1 (total rectangular_US)	-3197.4	27.2	-86969
uplift 2 (add back triangular_US)	661.9	23.9	15798
uplift 3 (rectangular_DS)	-700.5	8.6	-6025
uplift 4 (triangular_DS)	-489.5	11.5	-5613
Total	**-3725.5**		**-82809**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream 1	3357.2	22.0	-73859
Upstream 2 at Step	781.2	4.0	-3125
Downstream	-217.9	3.6	777
Total	**3920.6**		**-76207**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	935.6	17.0	-15906
Upstream - Ice	305.0	50.0	-15250
Total	**1240.6**		**-31156**

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Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	8348.5		193824
Weight of Water	-386.4		-15652
Driving of Water		3920.6	-76207
Uplift at Efficiency = 0	-3725.5		-82809
Silt & Ice		1240.6	-31156
Rock Anchor US @ 33.0 ft.	**0.0**		**0**
Total	**4236.6**	**5161.2**	**-12000.2**

ΣV = 4236.6 kips
ΣH = 5161.2 kips
ΣM = -12000.2 k-ft

USACE Stability

Force Resultant Location @ Base L =	-2.83 ft		**NG**	**Crack**
Force Resultant Location Offset e =	21.43 ft			**45.70 ft**
Foundation Bearing p_{max} =	-16346 psf	10000	**NG**	
Foundation Bearing p_{min} =	0 psf			
Sliding FOS =	0.29	2.00 Reqr'd	**NG**	For Usual LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	-2.83 ft			
Force Resultant Location Offset e =	21.43 ft			
Foundation Bearing p_{max} =	-16346 psf	10000	**NG**	
Foundation Bearing p_{min} =	0 psf			
Sliding FOS =	0.29	3.00 Reqr'd	**NG**	For Usual LC, Rock Foundation.
Sliding FOS (No Cohesion) =	0.57	1.50 Reqr'd	**NG**	

For Usual LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.

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1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
1,5,6,7,8,13	560.1	1	41.5	150	3486.4	318.8	92614
2 tunnel	28.0	1	41.5	-150	-174.3	374.0	-5432
3 stone fill	87.7	1	32	-35	-98.2	276.0	-2258
4	191.3	1	12	150	344.3	495.6	14216
9, 10	30.0	1	29.5	150	132.8	120.0	1328
11,12	1111.9	1	9.5	150	1584.5	269.5	35582
14	481.5	1	3	150	216.7	138.8	2507
Equipments					200.0	120.0	2000
20 wall + roof	399.6	1	12.5	150	749.3	361.0	22543
21 tunnel	28.0	1	12.5	-150	-52.5	374.0	-1636
22 stone fill	350.1	1	12.5	115	503.2	169.1	7091
23	140.4	1	12.5	150	263.3	135.0	2961
24 side wall	1988.4	1	4	150	1193.1	224.4	22309
Total					**8348.5**		**193824**

Top of Bridge =	904.8 ft		
Total Length of Powerhouse =	61.0 ft	Ice Load =	0.0 klf
Bottom of Dam EL =	846.3 ft	L1 =	20.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	17.2 ft
Dam / Foundation Bonding=	2880 psf	step =	8.0 ft
Length of Seepage Path =	45.2 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	881.3 ft	Foundation Width =	37.2 ft

2. Case II: Unusual Flood Discharge Condition

Head Water EL =	900.0 ft
Tail Water EL =	878.9 ft

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Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	2028.6	22.5	45555
Top Water weight 2			
Upward pressure	-310.0	41.0	-12710
Upward pressure at US piers	-273.8	41.2	-11279
Total	**1444.8**		**21566**

Uplift	21.1	32.6	21.1	0%
Upstream	Head 1 =	45.7	ft	Crack input
Downstream	Head 2 =	32.6	ft	0.00 ft
	Seep Grade =	0.467	ft/ft	40.6

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	37	0
uplift 1 (total rectangular_US)	-3479.0	27.2	-94630
uplift 2 (add back triangular_US)	355.4	23.9	8482
uplift 3 (rectangular_DS)	-2134.3	8.6	-18355
uplift 4 (triangular_DS)	-262.8	11.5	-3014
Total	**-5520.8**		**-107518**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream 1	3974.8	23.2	-92348
Upstream 2 at Step	1237.2	4.0	-4949
Downstream	-2022.6	10.9	21979
Total	**3189.4**		**-75318**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	935.6	17.0	-15906
Upstream - Ice	0.0	53.7	0
Total	**935.6**		**-15906**

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Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	8348.5		193824
Weight of Water	1444.8		21566
Driving of Water		3189.4	-75318
Uplift at Efficiency = 0	-5520.8		-107518
Silt & Ice		935.6	-15906
Rock Anchor US @ 33.0 ft.	**0.0**		**0**
Total	**4272.5**	**4125.0**	**16649.3**

$\Sigma V =$ 4272.5 kips

$\Sigma H =$ 4125.0 kips

$\Sigma M =$ 16649.3 k-ft

USACE Stability

Force Resultant Location @ Base L =	3.90 ft		NG	Crack
Force Resultant Location Offset e =	14.70 ft			25.51 ft
Foundation Bearing p_{max} =	11982 psf	10000	NG	
Foundation Bearing p_{min} =	0 psf			
Sliding FOS =	1.22	1.70 Reqr'd	NG	

For Unusual LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	3.90 ft		
Force Resultant Location Offset e =	14.70 ft		
Foundation Bearing p_{max} =	11982 psf	10000	NG
Foundation Bearing p_{min} =	0 psf		
Sliding FOS =	1.22	2.00 Reqr'd	NG
Sliding FOS (No Cohesion) =	0.73	1.50 Reqr'd	NG

For Unusual LC, Rock Foundation.

For Unusual LC, Rock Foundation. No Cohesion.

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Checked by:	E.Daly	Date:	12/19/11		Powerhouse Stability LC_3		
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1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
1,5,6,7,8,13	560.1	1	41.5	150	3486.4	318.8	92614
2 tunnel	28.0	1	41.5	-150	-174.3	374.0	-5432
3 stone fill	87.7	1	32	-35	-98.2	276.0	-2258
4	191.3	1	12	150	344.3	495.6	14216
9, 10	30.0	1	29.5	150	132.8	120.0	1328
11,12	1111.9	1	9.5	150	1584.5	269.5	35582
14	481.5	1	3	150	216.7	138.8	2507
Equipments					200.0	120.0	2000
20 wall + roof	399.6	1	12.5	150	749.3	361.0	22543
21 tunnel	28.0	1	12.5	-150	-52.5	374.0	-1636
22 stone fill	350.1	1	12.5	115	503.2	169.1	7091
23	140.4	1	12.5	150	263.3	135.0	2961
24 side wall	1988.4	1	4	150	1193.1	224.4	22309
Total					**8348.5**		**193824**

Top of Bridge =	904.8 ft		
Total Length of Powerhouse =	61.0 ft	Ice Load =	0.0 klf
Bottom of Dam EL =	846.3 ft	L1 =	20.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	17.2 ft
Dam / Foundation Bonding=	2880 psf	step =	8.0 ft
Length of Seepage Path =	45.2 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	881.3 ft	Foundation Width =	37.2 ft

2. Case III: Extreme Flood Discharge Condition

Head Water EL = 906.0 ft

Tail Water EL = 888.7 ft

Stanley Consultants INC.

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Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	2220.3	22.5	49860
Top Water weight 2	7.2	44.2	316
Upward pressure	-399.9	41.0	-16394
Upward pressure at US piers	-309.7	41.2	-12760
Total	**1517.9**		**21022**

Uplift	17.3	42.4	17.3	0%
Upstream	Head 1 = 51.7	ft		Crack input
Downstream	Head 2 = 42.4	ft		0.00 ft
	Seep Grade = 0.383	ft/ft		49.0

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	37	0
uplift 1 (total rectangular_US)	-3935.8	27.2	-107054
uplift 2 (add back triangular_US)	291.4	23.9	6954
uplift 3 (rectangular_DS)	-2775.9	8.6	-23873
uplift 4 (triangular_DS)	-215.5	11.5	-2471
Total	**-6635.9**		**-126444**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream_Rectangular	-159.9	34.3	5476
Upstream_Triangular	5245.7	25.5	-133765
Upstream 2 at Step	1491.6	4.0	-5966
Downstream	-3421.5	14.1	48357
Total	**3155.9**		**-85899**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	935.6	17.0	-15906
Upstream - Ice	0.0	59.7	0
Total	**935.6**		**-15906**

Stanley Consultants INC.

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Computed by:	Y.Ding	Date:	12/05/11		Dam / Powerhouse Stability
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Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	8348.5		193824
Weight of Water	1517.9		21022
Driving of Water		3155.9	-85899
Uplift at Efficiency = 0	-6635.9		-126444
Silt & Ice		935.6	-15906
Rock Anchor US @ 33.0 ft.	**0.0**		**0**
Total	**3230.5**	**4091.6**	**-13402.6**

$\Sigma V =$ 3230.5 kips

$\Sigma H =$ 4091.6 kips

$\Sigma M =$ -13402.6 k-ft

USACE Stability

Force Resultant Location @ Base L =	-4.15 ft		**NG**	**Crack**
Force Resultant Location Offset e =	22.75 ft			**49.65 ft**
Foundation Bearing p_{max} =	-8510 psf	13300	**NG**	
Foundation Bearing p_{min} =	0 psf			For Extreme LC, Rock Foundation.
Sliding FOS =	0.02	1.30 Reqr'd	**NG**	

FERC Stability

Force Resultant Location @ Base L =	-4.15 ft			
Force Resultant Location Offset e =	22.75 ft			
Foundation Bearing p_{max} =	-8510 psf	13300	**NG**	
Foundation Bearing p_{min} =	0 psf			For Extreme LC, Rock Foundation.
Sliding FOS =	0.02	2.00 Reqr'd	**NG**	
Sliding FOS (No Cohesion) =	0.55	1.50 Reqr'd	**NG**	

For Extreme LC, Rock Foundation. No Cohesion.

Existing Spillway

Anchored to USACE

Criteria

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
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Approved by:		Date:			Dam / Powerhouse Stability
					Spillway Stability LC_1
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1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Mass concrete	798.8	1	75	150	8985.9	298.2	223301
Less Placed Stone	299.1	1	75	-35	-785.2	271.2	-17746
Tunnel	28.0	1	83	-150	-348.6	414.0	-12027
Piers (to bottom)	1966.9	1	11	150	3245.4	279.0	75456
Bridge (thru)	39.9	1	73.5	150	439.5	193.0	7068
Gate Platform (thru)	14.2	1	73.5	150	156.2	377.0	4907
Platform BM (thru)	5.0	1	73.5	150	55.1	465.0	2136
Total					**11748.3**		**283095**

Top of Bridge =	904.8 ft		20 psi
Total Length of Spillway =	83.0 ft	Ice Load =	5.0 klf
Bottom of Dam EL =	848.3 ft	L1 =	17.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	24.0 ft
Dam / Foundation Bonding=	2880 psf	step =	5.0 ft
Length of Seepage Path =	46 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	879.8 ft	Foundation Width =	41.0 ft

2. Case I: Normal Operating Condition

Head Water EL = 896.3 ft

Tail Water EL = 857.0 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
	521.2	37.6	19611
	142.4	2.1	300
Total	**663.6**		**19912**

Uplift	39.3	8.7	39.3	0%
Upstream	Head 1 = 43.0	ft		Crack input
Downstream	Head 2 = 8.7	ft		0.00 ft
	Seep Grade = 0.854	ft/ft		29.2

Stanley Consultants INC.				Job No.	23601	Page No.	
Computed by:	Y.Ding	Date:	12/05/11	Subject:	Delhi Lake Dam Reconstruction		
Checked by:	E.Daly	Date:	12/16/11		Dam / Powerhouse Stability		
Approved by:		Date:			Spillway Stability LC_1		
				Sheet No.	of		

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	41	0
uplift 1 (total rectangular_US)	-3786.0	32.5	-123045
uplift 2 (add back triangular_US)	639.4	29.7	18969
uplift 3 (rectangular_DS)	-1081.4	12.0	-12977
uplift 4 (triangular_DS)	-1274.4	16.0	-20390
Total	**-5502.4**		**-137443**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream 1	4788.2	19.3	-92571
Upstream 2 at Step	756.3	2.5	-1891
Downstream	-196.0	2.9	568
Total	**5348.4**		**-93894**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	1226.4	13.8	-16965
Upstream - Ice	415.0	48.0	-19920
Total	**1641.4**		**-36885**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	11748.3		283095
Weight of Water	663.6		19912
Driving of Water		5348.4	-93894
Uplift at Efficiency = 0	-5502.4		-137443
Silt & Ice		1641.4	-36885
Rock Anchor US @ 38.75 ft.	**2500.0**		**96875**
Total	**9409.6**	**6989.8**	**131660.4**

Stanley Consultants INC.			Job No.	23601	Page No.
			Subject:	Delhi Lake Dam Reconstruction	
Computed by:	Y.Ding	Date: 12/05/11		Dam / Powerhouse Stability	
Checked by:	E.Daly	Date: 12/16/11		Spillway Stability LC_1	
Approved by:		Date:	Sheet No.	of	

$\Sigma V =$ 9409.6 kips

$\Sigma H =$ 6989.8 kips

$\Sigma M =$ 131660.4 k-ft

USACE Stability

Force Resultant Location @ Base L =	13.99 ft		**OK**	**Crack**
Force Resultant Location Offset e =	6.51 ft			**0.00 ft**
Foundation Bearing p_{max} =	5398 psf	10000	**OK**	
Foundation Bearing p_{min} =	132 psf			For Usual LC, Rock Foundation.
Sliding FOS =	2.34	2.00 Reqr'd	**OK**	

FERC Stability

Force Resultant Location @ Base L =	13.99 ft			
Force Resultant Location Offset e =	6.51 ft			
Foundation Bearing p_{max} =	5398 psf	10000	**OK**	
Foundation Bearing p_{min} =	132 psf			
Sliding FOS =	2.34	3.00 Reqr'd	**NG**	For Usual LC, Rock Foundation.
Sliding FOS (No Cohesion) =	0.94	1.50 Reqr'd	**NG**	

For Usual LC, Rock Foundation. No Cohesion.

ANCHORED TO USACE CRITERIA

Stanley Consultants INC.

Computed by: Y.Ding	Date: 12/05/11	Job No. 23601	Page No.
Checked by: E.Daly	Date: 12/16/11	Subject: Delhi Lake Dam Reconstruction	
Approved by:	Date:	Dam / Powerhouse Stability	
		Spillway Stability LC_2	
		Sheet No. of	

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Mass concrete	798.8	1	75	150	8985.9	298.2	223301
Less Placed Stone	299.1	1	75	-35	-785.2	271.2	-17746
Tunnel	28.0	1	83	-150	-348.6	414.0	-12027
Piers (to bottom)	1966.9	1	11	150	3245.4	279.0	75456
Bridge (thru)	39.9	1	73.5	150	439.5	193.0	7068
Gate Platform (thru)	14.2	1	73.5	150	156.2	377.0	4907
Platform BM (thru)	5.0	1	73.5	150	55.1	465.0	2136
Total					**11748.3**		**283095**

Top of Bridge = 904.8 ft

Total Length of Spillway = 83.0 ft — Ice Load = 0.0 klf

Bottom of Dam EL = 848.3 ft — L1 = 17.0 ft

Dam / Foundation Friction Angle = 35 degrees — L2 = 24.0 ft

Dam / Foundation Bonding= 2880 psf — step = 5.0 ft

Length of Seepage Path = 46 ft — Allowable Bearing = 10000 psf

Top Of Crest EL = 879.8 ft — Foundation Width = 41.0 ft

2. Case II: Unusual Flood Discharge Condition

Head Water EL = 900.0 ft

Tail Water EL = 878.9 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
	3705.0	20.0	74100
Total	**3705.0**		**74100**

Uplift	21.1	30.6	21.1		0%
Upstream	Head 1 =	46.7	ft		Crack input
Downstream	Head 2 =	30.6	ft		0.00 ft
	Seep Grade =	0.459	ft/ft		41.6

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Spillway Stability LC_2
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	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	41	0
uplift 1 (total rectangular_US)	-4111.8	32.5	-133632
uplift 2 (add back triangular_US)	343.3	29.7	10184
uplift 3 (rectangular_DS)	-3803.6	12.0	-45643
uplift 4 (triangular_DS)	-684.2	16.0	-10947
Total	**-8256.3**		**-180039**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream on Ogee_Rectangular	2772.4	18.3	-50597
Upstream on Ogee_Triangular	1818.5	13.8	-25157
Upstream on Piers_Triangular	101.8	38.2	-3894
Upstream 2 at Step	1077.5	2.5	-2694
Downstream on Ogee_Rectangular	0.0	15.8	0
Downstream on Ogee_Triangular	-2424.8	10.2	24733
Downstream on Piers_Triangular	0.0	31.2	0
Total	**3345.5**		**-57608**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	1226.4	13.8	-16965
Upstream - Ice	0.0	51.7	0
Total	**1226.4**		**-16965**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	11748.3		283095
Weight of Water	3705.0		74100
Driving of Water		3345.5	-57608
Uplift at Efficiency = 0	-8256.3		-180039
Silt & Ice		1226.4	-16965
Rock Anchor US @ 38.75 ft.	**2500.0**		**96875**
Total	**9697.0**	**4571.9**	**199458.6**

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
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Subject: Delhi Lake Dam Reconstruction
Dam / Powerhouse Stability
Spillway Stability LC_2

$\Sigma V =$ 9697.0 kips
$\Sigma H =$ 4571.9 kips
$\Sigma M =$ 199458.6 k-ft

USACE Stability

Force Resultant Location @ Base L =	20.57 ft	41	**OK**	**Crack**
Force Resultant Location Offset e =	0.07 ft			**0.00 ft**
Foundation Bearing p_{max} =	2878 psf	10000	**OK**	
Foundation Bearing p_{min} =	2821 psf			
Sliding FOS =	3.63	1.70 Reqr'd	**OK**	

For Unusual LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	20.57 ft		
Force Resultant Location Offset e =	0.07 ft		
Foundation Bearing p_{max} =	2878 psf	10000	**OK**
Foundation Bearing p_{min} =	2821 psf		
Sliding FOS =	3.63	2.00 Reqr'd	**OK**
Sliding FOS (No Cohesion) =	1.49	1.50 Reqr'd	**NG**

For Unusual LC, Rock Foundation.

For Unusual LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Spillway Stability LC_3
				Sheet No.	of

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Mass concrete	798.8	1	75	150	8985.9	298.2	223301
Less Placed Stone	299.1	1	75	-35	-785.2	271.2	-17746
Tunnel	28.0	1	83	-150	-348.6	414.0	-12027
Piers (to bottom)	1966.9	1	11	150	3245.4	279.0	75456
Bridge (thru)	39.9	1	73.5	150	439.5	193.0	7068
Gate Platform (thru)	14.2	1	73.5	150	156.2	377.0	4907
Platform BM (thru)	5.0	1	73.5	150	55.1	465.0	2136
Total					**11748.3**		**283095**

Top of Bridge =	904.8 ft		
Total Length of Spillway =	83.0 ft	Ice Load =	0.0 klf
Bottom of Dam EL =	848.3 ft	L1 =	17.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	24.0 ft
Dam / Foundation Bonding=	2880 psf	step =	5.0 ft
Length of Seepage Path =	46 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	879.8 ft	Foundation Width =	41.0 ft

2. Case III: Extreme Flood Discharge Condition

Head Water EL = 906.0 ft

Tail Water EL = 888.7 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
	5551.0	20.0	111020
Total	**5551.0**		**111020**

Uplift	17.3	40.4	17.3		0%
Upstream	Head 1 =	52.7	ft		Crack input
Downstream	Head 2 =	40.4	ft		0.00 ft
	Seep Grade =	0.376	ft/ft		49.4

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Spillway Stability LC_3
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	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	41	0
uplift 1 (total rectangular_US)	-4640.0	32.5	-150801
uplift 2 (add back triangular_US)	281.5	29.7	8350
uplift 3 (rectangular_DS)	-5021.8	12.0	-60261
uplift 4 (triangular_DS)	-561.0	16.0	-8976
Total	**-9941.3**		**-211688**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream on Ogee_Rectangular	3595.9	18.3	-65626
Upstream on Ogee_Triangular	1818.5	13.8	-25157
Upstream on Piers_Rectangular	41.6	42.8	-1777
Upstream on Piers_Triangular	126.4	39.0	-4928
Upstream on Gate_Triangular	35.5	55.2	-1958
Upstream 2 at Step	1279.9	2.5	-3200
Downstream on Ogee_Rectangular	-1452.0	15.8	22869
Downstream on Ogee_Triangular	-2569.5	10.5	26980
Downstream on Piers_Triangular	-19.8	34.5	681
Total	**2856.5**		**-52114**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	1226.4	13.8	-16965
Upstream - Ice	0.0	57.7	0
Total	**1226.4**		**-16965**

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Spillway Stability LC_3
				Sheet No.	of

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	11748.3		283095
Weight of Water	5551.0		111020
Driving of Water		2856.5	-52114
Uplift at Efficiency = 0	-9941.3		-211688
Silt & Ice		1226.4	-16965
Rock Anchor US @ 38.75 ft.	**2500.0**		**96875**
Total	**9858.0**	**4082.8**	**210222.9**

$\Sigma V =$ 9858.0 kips
$\Sigma H =$ 4082.8 kips
$\Sigma M =$ 210222.9 k-ft

USACE Stability

Force Resultant Location @ Base L =	21.33 ft	41	**OK**	**Crack**
Force Resultant Location Offset e =	0.83 ft			**0.00 ft**
Foundation Bearing p_{max} =	3247 psf	13300	**OK**	
Foundation Bearing p_{min} =	2547 psf			
Sliding FOS =	4.09	1.30 Reqr'd	**OK**	For Extreme LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	21.33 ft			
Force Resultant Location Offset e =	0.83 ft			
Foundation Bearing p_{max} =	3247 psf	13300	**OK**	
Foundation Bearing p_{min} =	2547 psf			
Sliding FOS =	4.09	2.00 Reqr'd	**OK**	For Extreme LC, Rock Foundation.
Sliding FOS (No Cohesion) =	1.69	1.50 Reqr'd	**OK**	For Extreme LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.

Computed By:	Y.Ding	**Date:**	11/29/2011	**Job No.**	23601 Delhi Lake Dam Reconstruction
Checked By:	E.Daly	**Date:**	12/19/11	**Subject**	Dam / Powerhouse Stability
					Spillway Rock Anchor Design - USACE

Rock Unit Weight =	152.4	pcf	
Rock Buoyant Unit Weight =	90	pcf	(recommended in Geotechical report)
Anchor Spacing =	6.0	ft	
Total Required Anchor Force =	2500	k	(unfactored required anchor force under normal load condition)
Total Number of Anchors =	10		
Required Effective Anchor Force =	**250**	k	
Grouted Rock Shear Allowable Strength =	5760	psf	
Grout Hole Diameter =	5.5	in.	
Anchor to Fractured Rock Bond Ultimate =	150	psi	(recommended in Geotechical report)

Anchor Length			Total Cone					Overlap Reduction							Net Wt.
Unbonded (ft)	Total L (ft)	Bonding L (ft)	Alpha (deg)	Cone (ft)	r (ft)	Vol (cf)	Wt (kips)	d (ft)	d/D	A/Atotal	A (sf)	h (ft)	Vol (cf)	Wt (kips)	**(kips)**
25	60	35	60	32.5	18.8	11,983	1,078	15.8	0.420	0.374	413	27.3	7,521	677	**402**

EM 1110-1-2908, Page 9-2, (9-2).	D =	28.9	ft	good
EM 1110-1-2908, Page 9-3, (9-5).	D =	26.4	ft	good
EM 1110-1-2908, Page 9-3, (9-6a).				
Grout Hole Perimeter =		1.44	ft	
Bond Surface =		43.2	sf	
Allowable Force by Bonding =		**466.5**	k	

Bonded Length under dam	35
Unbonded Length under dam	15
Unbonded Length above bott. of Dam	10

Anchorage Steel Bar Ultimate Stress =	150	ksi
Initial Prestress =	133%	
Factor of Safety for Steel Bar =	1.25	
required Garranteed Ultimate Tensile Strength (GUTS) =	417	k
Required Steel Bar Area =	2.78	in^2
Minimum Steel Bar Size =	1.88	in

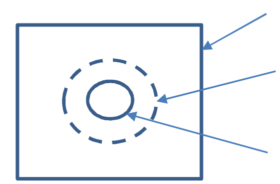
Use 150 ksi All-Thread-Bar (Williams Form Engineering Corp.) **2 1/4** **in.** **Diameter.**

At Concrete/Steel Plate Contact			
Initial Prestress	333	k	
Assumed Concrete Strength	4000	psi	
Allowable Concrete Bearing Pressure	2800	psi	(70% of f_c')
Minimum Bearing Area	119.0	in^2	
Minimum Size of Sqare Steel Pate	12.0	in	

At Depth of OLD Concrete			
Initial Prestress	333	k	
Assumed Concrete Strength	4000	psi	
Allowable Concrete Bearing Pressure	2800	psi	(70% of f_c')
Minimum Bearing Area	119.0	in^2	
Depth of Old Concrete from Plate	0.0	in	
Minimum Size of Sqare Steel Pate	12.0	in	

Use of Square Plate 12.0 in. X 12.0 in. X 4.0 in. Thk

Hole Size for Anchor	2 3/4	in		60.1209
Pressure on Plate	2772	psi		11.8791
Total Force on One Side	167	k		72
Moment Arm	3.36	in		0.57559
Moment	560	k-in		0.42441
Minimum Plate Thickness	4.00	in	(Use **50** ksi steel)	



50ksi Steel Plate
12.0 in.X 12.0 in.X 4.0 in. Thk
Drill Hole in Concrete
Dia.= 5.5 in.
Hole in Steel Plate
Dia.= 2 3/4 in.

Existing Spillway

Anchored to FERC

Criteria

ANCHORED TO FERC CRITERIA

Stanley Consultants INC.

				Job No.	23601 Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Dam / Powerhouse Stability
Checked by:	E.Daly	Date:	12/16/11		Spillway Stability LC_1
Approved by:		Date:		Sheet No.	of

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Mass concrete	798.8	1	75	150	8985.9	298.2	223301
Less Placed Stone	299.1	1	75	-35	-785.2	271.2	-17746
Tunnel	28.0	1	83	-150	-348.6	414.0	-12027
Piers (to bottom)	1966.9	1	11	150	3245.4	279.0	75456
Bridge (thru)	39.9	1	73.5	150	439.5	193.0	7068
Gate Platform (thru)	14.2	1	73.5	150	156.2	377.0	4907
Platform BM (thru)	5.0	1	73.5	150	55.1	465.0	2136
Total					**11748.3**		**283095**

Top of Bridge =	904.8 ft		20 psi
Total Length of Spillway =	83.0 ft	Ice Load =	5.0 klf
Bottom of Dam EL =	848.3 ft	L1 =	17.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	24.0 ft
Dam / Foundation Bonding=	2880 psf	step =	5.0 ft
Length of Seepage Path =	46 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	879.8 ft	Foundation Width =	41.0 ft

2. Case I: Normal Operating Condition

Head Water EL = 896.3 ft

Tail Water EL = 857.0 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
	521.2	37.6	19611
	142.4	2.1	300
Total	**663.6**		**19912**

Uplift	39.3	8.7	39.3	0%
Upstream	Head 1 = 43.0	ft		Crack input
Downstream	Head 2 = 8.7	ft		0.00 ft
	Seep Grade = 0.854	ft/ft		29.2

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Spillway Stability LC_1
				Sheet No.	of

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	41	0
uplift 1 (total rectangular_US)	-3786.0	32.5	-123045
uplift 2 (add back triangular_US)	639.4	29.7	18969
uplift 3 (rectangular_DS)	-1081.4	12.0	-12977
uplift 4 (triangular_DS)	-1274.4	16.0	-20390
Total	**-5502.4**		**-137443**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream 1	4788.2	19.3	-92571
Upstream 2 at Step	756.3	2.5	-1891
Downstream	-196.0	2.9	568
Total	**5348.4**		**-93894**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	1226.4	13.8	-16965
Upstream - Ice	415.0	48.0	-19920
Total	**1641.4**		**-36885**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	11748.3		283095
Weight of Water	663.6		19912
Driving of Water		5348.4	-93894
Uplift at Efficiency = 0	-5502.4		-137443
Silt & Ice		1641.4	-36885
Rock Anchor US @ 38.75 ft.	**8100.0**		**313875**
Total	**15009.6**	**6989.8**	**348660.4**

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Spillway Stability LC_1
				Sheet No.	of

ΣV = 15009.6 kips

ΣH = 6989.8 kips

ΣM = 348660.4 k-ft

USACE Stability

Force Resultant Location @ Base L =	23.23	ft		OK	Crack
Force Resultant Location Offset e =	2.73	ft			0.00 ft
Foundation Bearing p_{max} =	6172	psf	10000	OK	
Foundation Bearing p_{min} =	2649	psf			
Sliding FOS =	2.91		2.00 Reqr'd	OK	

For Usual LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	23.23	ft		
Force Resultant Location Offset e =	2.73	ft		
Foundation Bearing p_{max} =	6172	psf	10000	OK
Foundation Bearing p_{min} =	2649	psf		
Sliding FOS =	2.91		3.00 Reqr'd	NG
Sliding FOS (No Cohesion) =	1.50		1.50 Reqr'd	OK

For Usual LC, Rock Foundation.

For Usual LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.				Job No.	23601	Page No.	
				Subject:	Delhi Lake Dam Reconstruction		
Computed by:	Y.Ding	Date:	12/05/11		Dam / Powerhouse Stability		
Checked by:	E.Daly	Date:	12/16/11		Spillway Stability LC_2		
Approved by:		Date:		Sheet No.	of		

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Mass concrete	798.8	1	75	150	8985.9	298.2	223301
Less Placed Stone	299.1	1	75	-35	-785.2	271.2	-17746
Tunnel	28.0	1	83	-150	-348.6	414.0	-12027
Piers (to bottom)	1966.9	1	11	150	3245.4	279.0	75456
Bridge (thru)	39.9	1	73.5	150	439.5	193.0	7068
Gate Platform (thru)	14.2	1	73.5	150	156.2	377.0	4907
Platform BM (thru)	5.0	1	73.5	150	55.1	465.0	2136
Total					**11748.3**		**283095**

Top of Bridge =	904.8 ft		
Total Length of Spillway =	83.0 ft	Ice Load =	0.0 klf
Bottom of Dam EL =	848.3 ft	L1 =	17.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	24.0 ft
Dam / Foundation Bonding=	2880 psf	step =	5.0 ft
Length of Seepage Path =	46 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	879.8 ft	Foundation Width =	41.0 ft

2. Case II: Unusual Flood Discharge Condition

Head Water EL = 900.0 ft

Tail Water EL = 878.9 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
	3705.0	20.0	74100
Total	**3705.0**		**74100**

Uplift	21.1	30.6	21.1	0%
Upstream	Head 1 = 46.7	ft		Crack input
Downstream	Head 2 = 30.6	ft		0.00 ft
	Seep Grade = 0.459	ft/ft		41.6

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
Approved by:		Date:		Subject:	Delhi Lake Dam Reconstruction Dam / Powerhouse Stability Spillway Stability LC_2
				Sheet No.	of

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	41	0
uplift 1 (total rectangular_US)	-4111.8	32.5	-133632
uplift 2 (add back triangular_US)	343.3	29.7	10184
uplift 3 (rectangular_DS)	-3803.6	12.0	-45643
uplift 4 (triangular_DS)	-684.2	16.0	-10947
Total	**-8256.3**		**-180039**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream on Ogee_Rectangular	2772.4	18.3	-50597
Upstream on Ogee_Triangular	1818.5	13.8	-25157
Upstream on Piers_Triangular	101.8	38.2	-3894
Upstream 2 at Step	1077.5	2.5	-2694
Downstream on Ogee_Rectangular	0.0	15.8	0
Downstream on Ogee_Triangular	-2424.8	10.2	24733
Downstream on Piers_Triangular	0.0	31.2	0
Total	**3345.5**		**-57608**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	1226.4	13.8	-16965
Upstream - Ice	0.0	51.7	0
Total	**1226.4**		**-16965**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	11748.3		283095
Weight of Water	3705.0		74100
Driving of Water		3345.5	-57608
Uplift at Efficiency = 0	-8256.3		-180039
Silt & Ice		1226.4	-16965
Rock Anchor US @ 38.75 ft.	**8100.0**		**313875**
Total	**15297.0**	**4571.9**	**416458.6**

Stanley Consultants INC.

				Job No.	23601 Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Dam / Powerhouse Stability
Checked by:	E.Daly	Date:	12/16/11		Spillway Stability LC_2
Approved by:		Date:		Sheet No.	of

ΣV = 15297.0 kips

ΣH = 4571.9 kips

ΣM = 416458.6 k-ft

USACE Stability

Force Resultant Location @ Base L =	27.22 ft	41	**OK**	**Crack**
Force Resultant Location Offset e =	6.72 ft			**0.00 ft**
Foundation Bearing p_{max} =	8919 psf	10000	**OK**	
Foundation Bearing p_{min} =	71 psf			
Sliding FOS =	4.49	1.70 Reqr'd	**OK**	For Unusual LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	27.22 ft			
Force Resultant Location Offset e =	6.72 ft			
Foundation Bearing p_{max} =	8919 psf	10000	**OK**	
Foundation Bearing p_{min} =	71 psf			
Sliding FOS =	4.49	2.00 Reqr'd	**OK**	For Unusual LC, Rock Foundation.
Sliding FOS (No Cohesion) =	2.34	1.50 Reqr'd	**OK**	

For Unusual LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.		Job No. 23601	Page No.
Computed by: Y.Ding	Date: 12/05/11	Subject: Delhi Lake Dam Reconstruction	
Checked by: E.Daly	Date: 12/16/11	Dam / Powerhouse Stability	
Approved by:	Date:	Spillway Stability LC_3	
		Sheet No. of	

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Mass concrete	798.8	1	75	150	8985.9	298.2	223301
Less Placed Stone	299.1	1	75	-35	-785.2	271.2	-17746
Tunnel	28.0	1	83	-150	-348.6	414.0	-12027
Piers (to bottom)	1966.9	1	11	150	3245.4	279.0	75456
Bridge (thru)	39.9	1	73.5	150	439.5	193.0	7068
Gate Platform (thru)	14.2	1	73.5	150	156.2	377.0	4907
Platform BM (thru)	5.0	1	73.5	150	55.1	465.0	2136
Total					**11748.3**		**283095**

Top of Bridge =	904.8 ft		
Total Length of Spillway =	83.0 ft	Ice Load =	0.0 klf
Bottom of Dam EL =	848.3 ft	L1 =	17.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	24.0 ft
Dam / Foundation Bonding=	2880 psf	step =	5.0 ft
Length of Seepage Path =	46 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	879.8 ft	Foundation Width =	41.0 ft

2. Case III: Extreme Flood Discharge Condition

Head Water EL = 906.0 ft

Tail Water EL = 888.7 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
	5551.0	20.0	111020
Total	**5551.0**		**111020**

Uplift	17.3	40.4	17.3	0%
Upstream	Head 1 = 52.7	ft		Crack input
Downstream	Head 2 = 40.4	ft		0.00 ft
	Seep Grade = 0.376	ft/ft		49.4

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
Approved by:		Date:		Subject:	Delhi Lake Dam Reconstruction Dam / Powerhouse Stability Spillway Stability LC_3
				Sheet No.	of

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	41	0
uplift 1 (total rectangular_US)	-4640.0	32.5	-150801
uplift 2 (add back triangular_US)	281.5	29.7	8350
uplift 3 (rectangular_DS)	-5021.8	12.0	-60261
uplift 4 (triangular_DS)	-561.0	16.0	-8976
Total	**-9941.3**		**-211688**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream on Ogee_Rectangular	3595.9	18.3	-65626
Upstream on Ogee_Triangular	1818.5	13.8	-25157
Upstream on Piers_Rectangular	41.6	42.8	-1777
Upstream on Piers_Triangular	126.4	39.0	-4928
Upstream on Gate_Triangular	35.5	55.2	-1958
Upstream 2 at Step	1279.9	2.5	-3200
Downstream on Ogee_Rectangular	-1452.0	15.8	22869
Downstream on Ogee_Triangular	-2569.5	10.5	26980
Downstream on Piers_Triangular	-19.8	34.5	681
Total	**2856.5**		**-52114**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	1226.4	13.8	-16965
Upstream - Ice	0.0	57.7	0
Total	**1226.4**		**-16965**

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Spillway Stability LC_3
				Sheet No.	of

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	11748.3		283095
Weight of Water	5551.0		111020
Driving of Water		2856.5	-52114
Uplift at Efficiency = 0	-9941.3		-211688
Silt & Ice		1226.4	-16965
Rock Anchor US @ 38.75 ft.	**8100.0**		**313875**
Total	**15458.0**	**4082.8**	**427222.9**

$\Sigma V =$ 15458.0 kips

$\Sigma H =$ 4082.8 kips

$\Sigma M =$ 427222.9 k-ft

<u>USACE Stability</u>

Force Resultant Location @ Base L =	27.64 ft	41	**OK**	**Crack**
Force Resultant Location Offset e =	7.14 ft			**0.91 ft**
Foundation Bearing p_{max} =	9292 psf	13300	**OK**	
Foundation Bearing p_{min} =	0 psf			
Sliding FOS =	5.00	1.30 Reqr'd	**OK**	For Extreme LC, Rock Foundation.

<u>FERC Stability</u>

Force Resultant Location @ Base L =	27.64 ft			
Force Resultant Location Offset e =	7.14 ft			
Foundation Bearing p_{max} =	9292 psf	13300	**OK**	
Foundation Bearing p_{min} =	0 psf			
Sliding FOS =	5.00	2.00 Reqr'd	**OK**	For Extreme LC, Rock Foundation.
Sliding FOS (No Cohesion) =	2.65	1.50 Reqr'd	**OK**	

For Extreme LC, Rock Foundation. No Cohesion.

ANCHORED TO FERC CRITERIA

Stanley Consultants INC.

Computed By:	Y.Ding	**Date:**	11/29/2011	**Job No.**	23601 Delhi Lake Dam Reconstruction
Checked By:	E.Daly	**Date:**	12/19/2011	**Subject**	Dam / Powerhouse Stability
					Spillway Rock Anchor Design - FERC

Rock Unit Weight =	152.4	pcf	
Rock Buoyant Unit Weight =	90	pcf	(recommended in Geotechical report)
Anchor Spacing =	8.0	ft	
Total Required Anchor Force =	8100	k	(unfactored required anchor force under normal load condition)
Total Number of Anchors =	30		
Required Effective Anchor Force =	**270**	k	
Grouted Rock Shear Allowable Strength =	**5760**	psf	
Grout Hole Diameter =	**5.5**	in.	
Anchor to Fractured Rock Bond Ultimate =	**150**	psi	(recommended in Geotechical report)

Anchor Length			**Total Cone**					**Overlap Reduction**							**Net Wt.**
Unbonded (ft)	Total L (ft)	Bonding L (ft)	Alpha (deg)	Cone (ft)	r (ft)	Vol (cf)	Wt (kips)	d (ft)	d/D	A/Atotal	A (sf)	h (ft)	Vol (cf)	Wt (kips)	**(kips)**
25	60	35	**60**	32.5	18.8	11,983	1,078	14.8	0.393	0.365	404	25.6	6,890	620	**458**

EM 1110-1-2908, Page 9-2, (9-2).	D =	23.4	ft	good
EM 1110-1-2908, Page 9-3, (9-5).	D =	23.7	ft	good
EM 1110-1-2908, Page 9-3, (9-6a).				
Grout Hole Perimeter =		1.44	ft	
Bond Surface =		43.2	sf	
Allowable Force by Bonding =		**466.5**	k	

Bonded Length under dam	35
Unbonded Length under dam	15
Unbonded Length above bott. of Dam	10

Anchorage Steel Bar Ultimate Stress =	150	ksi
Initial Prestress =	133%	
Factor of Safety for Steel Bar =	1.25	
required Garranteed Ultimate Tensile Strength (GUTS) =	450	k
Required Steel Bar Area =	3.00	in^2
Minimum Steel Bar Size =	1.95	in

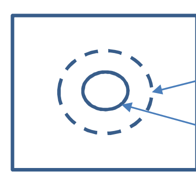
Use 150 ksi All-Thread-Bar (Williams Form Engineering Corp.) **2 1/4** **in.** **Diameter.**

At Concrete/Steel Plate Contact			
Initial Prestress	360	k	
Assumed Concrete Strength	4000	psi	
Allowable Concrete Bearing Pressure	2800	psi	(70% of f_c')
Minimum Bearing Area	128.6	in^2	
Minimum Size of Sqare Steel Pate	13.0	in	

At Depth of OLD Concrete			
Initial Prestress	360	k	
Assumed Concrete Strength	4000	psi	
Allowable Concrete Bearing Pressure	2800	psi	(70% of f_c')
Minimum Bearing Area	128.57	in^2	
Depth of Old Concrete from Plate	0	in	
Minimum Size of Sqare Steel Pate	13.0	in	

Use of Square Plate 14.0 in. X 14.0 in. X 4.0 in. Thk

Hole Size for Anchor	2 3/4	in		86.1209
Pressure on Plate	2090	psi		11.8791
Total Force on One Side	180	k		98
Moment Arm	3.82	in		0.57559
Moment	688	k-in		0.42441
Minimum Plate Thickness	4.00	in	(Use **50** ksi steel)	



50ksi Steel Plate

14.0 in.X 14.0 in.X 4.0 in. Thk

Drill Hole in Concrete

Dia.= 5.5 in.

Hole in Steel Plate

Dia.= 2 3/4 in.

Existing Powerhouse

Anchored to USACE

Criteria

Stanley Consultants INC.				Job No.	23601	Page No.	
				Subject:	Delhi Lake Dam Reconstruction		
Computed by:	Y.Ding	Date:	12/05/11		Dam / Powerhouse Stability		
Checked by:	E.Daly	Date:	12/16/11		Powerhouse Stability LC_1		
Approved by:		Date:		Sheet No.	of		

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
1,5,6,7,8,13	560.1	1	41.5	150	3486.4	318.8	92614
2 tunnel	28.0	1	41.5	-150	-174.3	374.0	-5432
3 stone fill	87.7	1	32	-35	-98.2	276.0	-2258
4	191.3	1	12	150	344.3	495.6	14216
9, 10	30.0	1	29.5	150	132.8	120.0	1328
11,12	1111.9	1	9.5	150	1584.5	269.5	35582
14	481.5	1	3	150	216.7	138.8	2507
Equipments					200.0	120.0	2000
20 wall + roof	399.6	1	12.5	150	749.3	361.0	22543
21 tunnel	28.0	1	12.5	-150	-52.5	374.0	-1636
22 stone fill	350.1	1	12.5	115	503.2	169.1	7091
23	140.4	1	12.5	150	263.3	135.0	2961
24 side wall	1988.4	1	4	150	1193.1	224.4	22309
Total					**8348.5**		**193824**

Top of Bridge =	904.8 ft		20 psi	
Total Length of Powerhouse =	61.0 ft	Ice Load =	**5.0**	klf
Bottom of Dam EL =	846.3 ft	L1 =	20.0	ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	17.2	ft
Dam / Foundation Bonding=	2880 psf	step =	8.0	ft
Length of Seepage Path =	45.2 ft	Allowable Bearing =	10000	psf
Top Of Crest EL =	881.3 ft	Foundation Width =	37.2	ft

2. Case I: Normal Operating Condition - Dewatered

Head Water EL = 896.3 ft

Tail Water EL = 857.0 ft

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
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					Dam / Powerhouse Stability
					Powerhouse Stability LC_1
				Sheet No.	of

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	119.8	43.0	5152
Top Water weight 2			
Upward pressure	-254.6	41.0	-10438
Upward pressure at US piers	-251.6	41.2	-10366
Total	**-386.4**		**-15652**

Uplift	39.3	10.7	39.3	0%
Upstream	Head 1 = 42.0		ft	Crack input
Downstream	Head 2 = 10.7		ft	0.00 ft
	Seep Grade = 0.869		ft/ft	25.7

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	37	0
uplift 1 (total rectangular_US)	-3197.4	27.2	-86969
uplift 2 (add back triangular_US)	661.9	23.9	15798
uplift 3 (rectangular_DS)	-700.5	8.6	-6025
uplift 4 (triangular_DS)	-489.5	11.5	-5613
Total	**-3725.5**		**-82809**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream 1	3357.2	22.0	-73859
Upstream 2 at Step	781.2	4.0	-3125
Downstream	-217.9	3.6	777
Total	**3920.6**		**-76207**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	935.6	17.0	-15906
Upstream - Ice	305.0	50.0	-15250
Total	**1240.6**		**-31156**

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Powerhouse Stability LC_1
				Sheet No.	of

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	8348.5		193824
Weight of Water	-386.4		-15652
Driving of Water		3920.6	-76207
Uplift at Efficiency = 0	-3725.5		-82809
Silt & Ice		1240.6	-31156
Rock Anchor US @ 41.0 ft.	**2300.0**		**94300**
Total	**6536.6**	**5161.2**	**82299.8**

ΣV = 6536.6 kips
ΣH = 5161.2 kips
ΣM = 82299.8 k-ft

USACE Stability

Force Resultant Location @ Base L =	12.59 ft		**OK**	**Crack**
Force Resultant Location Offset e =	6.01 ft			**0.00 ft**
Foundation Bearing p_{max} =	5673 psf	10000	**OK**	
Foundation Bearing p_{min} =	89 psf			
Sliding FOS =	2.15	2.00 Reqr'd	**OK**	For Usual LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	12.59 ft			
Force Resultant Location Offset e =	6.01 ft			
Foundation Bearing p_{max} =	5673 psf	10000	**OK**	
Foundation Bearing p_{min} =	89 psf			
Sliding FOS =	2.15	3.00 Reqr'd	**NG**	For Usual LC, Rock Foundation.
Sliding FOS (No Cohesion) =	0.89	1.50 Reqr'd	**NG**	For Usual LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.

				Job No.	23601 Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Dam / Powerhouse Stability
Checked by:	E.Daly	Date:	12/16/11		Powerhouse Stability LC_2
Approved by:		Date:		Sheet No.	of

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
1,5,6,7,8,13	560.1	1	41.5	150	3486.4	318.8	92614
2 tunnel	28.0	1	41.5	-150	-174.3	374.0	-5432
3 stone fill	87.7	1	32	-35	-98.2	276.0	-2258
4	191.3	1	12	150	344.3	495.6	14216
9, 10	30.0	1	29.5	150	132.8	120.0	1328
11,12	1111.9	1	9.5	150	1584.5	269.5	35582
14	481.5	1	3	150	216.7	138.8	2507
Equipments					200.0	120.0	2000
20 wall + roof	399.6	1	12.5	150	749.3	361.0	22543
21 tunnel	28.0	1	12.5	-150	-52.5	374.0	-1636
22 stone fill	350.1	1	12.5	115	503.2	169.1	7091
23	140.4	1	12.5	150	263.3	135.0	2961
24 side wall	1988.4	1	4	150	1193.1	224.4	22309
Total					**8348.5**		**193824**

Top of Bridge =	904.8 ft			
Total Length of Powerhouse =	61.0 ft		Ice Load =	0.0 klf
Bottom of Dam EL =	846.3 ft		L1 =	20.0 ft
Dam / Foundation Friction Angle =	35 degrees		L2 =	17.2 ft
Dam / Foundation Bonding=	2880 psf		step =	8.0 ft
Length of Seepage Path =	45.2 ft		Allowable Bearing =	10000 psf
Top Of Crest EL =	881.3 ft		Foundation Width =	37.2 ft

2. Case II: Unusual Flood Discharge Condition

Head Water EL =	900.0 ft
Tail Water EL =	878.9 ft

Stanley Consultants INC.				Job No.	23601	Page No.	
Computed by:	Y.Ding	Date:	12/05/11	Subject:	Delhi Lake Dam Reconstruction		
Checked by:	E.Daly	Date:	12/16/11		Dam / Powerhouse Stability		
Approved by:		Date:			Powerhouse Stability LC_2		
				Sheet No.	of		

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	2028.6	22.5	45555
Top Water weight 2			
Upward pressure	-310.0	41.0	-12710
Upward pressure at US piers	-273.8	41.2	-11279
Total	**1444.8**		**21566**

Uplift	21.1	32.6	21.1		0%
Upstream	Head 1 =	45.7	ft		Crack input
Downstream	Head 2 =	32.6	ft		0.00 ft
	Seep Grade =	0.467	ft/ft		40.6

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	37	0
uplift 1 (total rectangular_US)	-3479.0	27.2	-94630
uplift 2 (add back triangular_US)	355.4	23.9	8482
uplift 3 (rectangular_DS)	-2134.3	8.6	-18355
uplift 4 (triangular_DS)	-262.8	11.5	-3014
Total	**-5520.8**		**-107518**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream 1	3974.8	23.2	-92348
Upstream 2 at Step	1237.2	4.0	-4949
Downstream	-2022.6	10.9	21979
Total	**3189.4**		**-75318**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	935.6	17.0	-15906
Upstream - Ice	0.0	53.7	0
Total	**935.6**		**-15906**

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Powerhouse Stability LC_2
				Sheet No.	of

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	8348.5		193824
Weight of Water	1444.8		21566
Driving of Water		3189.4	-75318
Uplift at Efficiency = 0	-5520.8		-107518
Silt & Ice		935.6	-15906
Rock Anchor US @ 41.0 ft.	**2300.0**		**94300**
Total	**6572.5**	**4125.0**	**110949.3**

$\Sigma V =$ 6572.5 kips

$\Sigma H =$ 4125.0 kips

$\Sigma M =$ 110949.3 k-ft

USACE Stability

Force Resultant Location @ Base L =	16.88	ft		**OK**	**Crack**
Force Resultant Location Offset e =	1.72	ft			**0.00 ft**
Foundation Bearing p_{max} =	3699	psf	10000	**OK**	
Foundation Bearing p_{min} =	2093	psf			
Sliding FOS =	2.70		1.70 Reqr'd	**OK**	

For Unusual LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	16.88	ft		
Force Resultant Location Offset e =	1.72	ft		
Foundation Bearing p_{max} =	3699	psf	10000	**OK**
Foundation Bearing p_{min} =	2093	psf		
Sliding FOS =	2.70		2.00 Reqr'd	**OK**
Sliding FOS (No Cohesion) =	1.12		1.50 Reqr'd	**NG**

For Unusual LC, Rock Foundation.

For Unusual LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
Approved by:		Date:		Subject:	Delhi Lake Dam Reconstruction Dam / Powerhouse Stability Powerhouse Stability LC_3
				Sheet No.	of

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
1,5,6,7,8,13	560.1	1	41.5	150	3486.4	318.8	92614
2 tunnel	28.0	1	41.5	-150	-174.3	374.0	-5432
3 stone fill	87.7	1	32	-35	-98.2	276.0	-2258
4	191.3	1	12	150	344.3	495.6	14216
9, 10	30.0	1	29.5	150	132.8	120.0	1328
11,12	1111.9	1	9.5	150	1584.5	269.5	35582
14	481.5	1	3	150	216.7	138.8	2507
Equipments					200.0	120.0	2000
20 wall + roof	399.6	1	12.5	150	749.3	361.0	22543
21 tunnel	28.0	1	12.5	-150	-52.5	374.0	-1636
22 stone fill	350.1	1	12.5	115	503.2	169.1	7091
23	140.4	1	12.5	150	263.3	135.0	2961
24 side wall	1988.4	1	4	150	1193.1	224.4	22309
Total					**8348.5**		**193824**

Top of Bridge =	904.8 ft		
Total Length of Powerhouse =	61.0 ft	Ice Load =	0.0 klf
Bottom of Dam EL =	846.3 ft	L1 =	20.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	17.2 ft
Dam / Foundation Bonding=	2880 psf	step =	8.0 ft
Length of Seepage Path =	45.2 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	881.3 ft	Foundation Width =	37.2 ft

2. Case III: Extreme Flood Discharge Condition

Head Water EL = 906.0 ft

Tail Water EL = 888.7 ft

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Powerhouse Stability LC_3
				Sheet No.	of

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	2220.3	22.5	49860
Top Water weight 2	7.2	44.2	316
Upward pressure	-399.9	41.0	-16394
Upward pressure at US piers	-309.7	41.2	-12760
Total	**1517.9**		**21022**

Uplift	17.3	42.4	17.3	0%
Upstream	Head 1 = 51.7	ft		Crack input
Downstream	Head 2 = 42.4	ft		0.00 ft
	Seep Grade = 0.383	ft/ft		49.0

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	37	0
uplift 1 (total rectangular_US)	-3935.8	27.2	-107054
uplift 2 (add back triangular_US)	291.4	23.9	6954
uplift 3 (rectangular_DS)	-2775.9	8.6	-23873
uplift 4 (triangular_DS)	-215.5	11.5	-2471
Total	**-6635.9**		**-126444**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream_Rectangular	-159.9	34.3	5476
Upstream_Triangular	5245.7	25.5	-133765
Upstream 2 at Step	1491.6	4.0	-5966
Downstream	-3421.5	14.1	48357
Total	**3155.9**		**-85899**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	935.6	17.0	-15906
Upstream - Ice	0.0	59.7	0
Total	**935.6**		**-15906**

Stanley Consultants INC.

Computed by: Y.Ding	Date: 12/05/11	Job No. 23601	Page No.
Checked by: E.Daly	Date: 12/16/11	Subject: Delhi Lake Dam Reconstruction	
Approved by:	Date:	Dam / Powerhouse Stability	
		Powerhouse Stability LC_3	
		Sheet No. of	

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	8348.5		193824
Weight of Water	1517.9		21022
Driving of Water		3155.9	-85899
Uplift at Efficiency = 0	-6635.9		-126444
Silt & Ice		935.6	-15906
Rock Anchor US @ 41.0 ft.	**2300.0**		**94300**
Total	**5530.5**	**4091.6**	**80897.4**

ΣV = 5530.5 kips
ΣH = 4091.6 kips
ΣM = 80897.4 k-ft

USACE Stability

Force Resultant Location @ Base L =	14.63 ft		OK	Crack
Force Resultant Location Offset e =	3.97 ft			0.00 ft
Foundation Bearing p_{max} =	3999 psf	13300	OK	
Foundation Bearing p_{min} =	876 psf			For Extreme LC, Rock Foundation.
Sliding FOS =	2.54	1.30 Reqr'd	OK	

FERC Stability

Force Resultant Location @ Base L =	14.63 ft			
Force Resultant Location Offset e =	3.97 ft			
Foundation Bearing p_{max} =	3999 psf	13300	OK	
Foundation Bearing p_{min} =	876 psf			For Extreme LC, Rock Foundation.
Sliding FOS =	2.54	2.00 Reqr'd	OK	
Sliding FOS (No Cohesion) =	0.95	1.50 Reqr'd	NG	

For Extreme LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.

Computed By:	Y.Ding	**Date:**	11/29/2011	**Job No.**	23601 Delhi Lake Dam Reconstruction
Checked By:	E.Daly	**Date:**	~~12/19/2011~~	**Subject**	Dam / Powerhouse Stability
					Powerhouse Rock Anchor Design - USACE

Rock Unit Weight =	152.4	pcf	
Rock Buoyant Unit Weight =	90	pcf	(recommended in Geotechical report)
Anchor Spacing =	6.0	ft	
Total Required Anchor Force =	2300	k	(unfactored required anchor force under normal load condition)
Total Number of Anchors =	10		
Required Effective Anchor Force =	**230**	k	
Grouted Rock Shear Allowable Strength =	**5760**	psf	
Grout Hole Diameter =	**5.5**	in.	
Anchor to Fractured Rock Bond Ultimate =	**150**	psi	(recommended in Geotechical report)

Anchor Length			**Total Cone**					**Overlap Reduction**							**Net Wt.**
Unbonded (ft)	Total L (ft)	Bonding L (ft)	Alpha (deg)	Cone (ft)	r (ft)	Vol (cf)	Wt (kips)	d (ft)	d/D	A/Atotal	A (sf)	h (ft)	Vol (cf)	Wt (kips)	**(kips)**
25	60	35	**60**	32.5	18.8	11,983	1,078	15.8	0.420	0.374	413	27.3	7,521	677	**402**

EM 1110-1-2908, Page 9-2, (9-2).	D =	26.6 ft	good
EM 1110-1-2908, Page 9-3, (9-5).	D =	25.3 ft	good
EM 1110-1-2908, Page 9-3, (9-6a).			
Grout Hole Perimeter =		1.44 ft	
Bond Surface =		43.2 sf	
Allowable Force by Bonding =		**466.5** k	

Bonded Length under dam	35
Unbonded Length under dam	15
Unbonded Length above bott. of Dam	10

Anchorage Steel Bar Ultimate Stress =	150 ksi
Initial Prestress =	133%
Factor of Safety for Steel Bar =	1.25
required Garranteed Ultimate Tensile Strength (GUTS) =	383 k
Required Steel Bar Area =	2.56 in^2
Minimum Steel Bar Size =	1.80 in

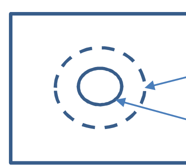
Use 150 ksi All-Thread-Bar (Williams Form Engineering Corp.) **1 3/4** **in.** **Diameter.**

At Concrete/Steel Plate Contact		
Initial Prestress	306.67 k	
Assumed Concrete Strength	4000 psi	
Allowable Concrete Bearing Pressure	2800 psi	0.7 of f_c')
Minimum Bearing Area	109.52 in^2	
Minimum Size of Sqare Steel Pate	12 in	

At Depth of OLD Concrete		
Initial Prestress	306.67 k	
Assumed Concrete Strength	4000 psi	
Allowable Concrete Bearing Pressure	2800 psi	0.7 of f_c')
Minimum Bearing Area	109.52 in^2	
Depth of Old Concrete from Plate	0 in	
Minimum Size of Sqare Steel Pate	12 in	

Use of Square Plate 12.0 in. X 12.0 in. X 3.0 in. Thk

Hole Size for Anchor	2 1/4 in	60.1209
Pressure on Plate	2550 psi	11.8791
Total Force on One Side	153 k	72
Moment Arm	3.36 in	0.57559
Moment	516 k-in	0.42441
Minimum Plate Thickness	3.00 in (Use **50** ksi steel)	



50ksi Steel Plate

12.0 in.X 12.0 in.X 3.0 in. Thk

Drill Hole in Concrete

Dia.= 5.5 in.

Hole in Steel Plate

Dia.= 2 1/4 in.

Existing Powerhouse

Anchored to FERC

Criteria

Stanley Consultants INC.				Job No.	23601	Page No.	
				Subject:	Delhi Lake Dam Reconstruction		
Computed by:	Y.Ding	Date:	12/05/11		Dam / Powerhouse Stability		
Checked by:	E.Daly	Date:	12/16/11		Powerhouse Stability LC_1		
Approved by:		Date:		Sheet No.	of		

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
1,5,6,7,8,13	560.1	1	41.5	150	3486.4	318.8	92614
2 tunnel	28.0	1	41.5	-150	-174.3	374.0	-5432
3 stone fill	87.7	1	32	-35	-98.2	276.0	-2258
4	191.3	1	12	150	344.3	495.6	14216
9, 10	30.0	1	29.5	150	132.8	120.0	1328
11,12	1111.9	1	9.5	150	1584.5	269.5	35582
14	481.5	1	3	150	216.7	138.8	2507
Equipments					200.0	120.0	2000
20 wall + roof	399.6	1	12.5	150	749.3	361.0	22543
21 tunnel	28.0	1	12.5	-150	-52.5	374.0	-1636
22 stone fill	350.1	1	12.5	115	503.2	169.1	7091
23	140.4	1	12.5	150	263.3	135.0	2961
24 side wall	1988.4	1	4	150	1193.1	224.4	22309
Total					**8348.5**		**193824**

Top of Bridge =	904.8	ft		20	psi
Total Length of Powerhouse =	61.0	ft	Ice Load =	**5.0**	klf
Bottom of Dam EL =	846.3	ft	L1 =	20.0	ft
Dam / Foundation Friction Angle =	35	degrees	L2 =	17.2	ft
Dam / Foundation Bonding=	2880	psf	step =	8.0	ft
Length of Seepage Path =	45.2	ft	Allowable Bearing =	10000	psf
Top Of Crest EL =	881.3	ft	Foundation Width =	37.2	ft

2. Case I: Normal Operating Condition - Dewatered

Head Water EL =	896.3	ft
Tail Water EL =	857.0	ft

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
Approved by:		Date:		Subject:	Delhi Lake Dam Reconstruction
					Dam / Powerhouse Stability
					Powerhouse Stability LC_1
				Sheet No.	of

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	119.8	43.0	5152
Top Water weight 2			
Upward pressure	-254.6	41.0	-10438
Upward pressure at US piers	-251.6	41.2	-10366
Total	**-386.4**		**-15652**

Uplift	39.3	10.7	39.3	0%
Upstream	Head 1 = 42.0	ft		Crack input
Downstream	Head 2 = 10.7	ft		0.00 ft
	Seep Grade = 0.869	ft/ft		25.7

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	37	0
uplift 1 (total rectangular_US)	-3197.4	27.2	-86969
uplift 2 (add back triangular_US)	661.9	23.9	15798
uplift 3 (rectangular_DS)	-700.5	8.6	-6025
uplift 4 (triangular_DS)	-489.5	11.5	-5613
Total	**-3725.5**		**-82809**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream 1	3357.2	22.0	-73859
Upstream 2 at Step	781.2	4.0	-3125
Downstream	-217.9	3.6	777
Total	**3920.6**		**-76207**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	935.6	17.0	-15906
Upstream - Ice	305.0	50.0	-15250
Total	**1240.6**		**-31156**

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Powerhouse Stability LC_1
				Sheet No.	of

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	8348.5		193824
Weight of Water	-386.4		-15652
Driving of Water		3920.6	-76207
Uplift at Efficiency = 0	-3725.5		-82809
Silt & Ice		1240.6	-31156
Rock Anchor US @ 33.0 ft.	**7000.0**		**231000**
Total	**11236.6**	**5161.2**	**218999.8**

ΣV = 11236.6 kips
ΣH = 5161.2 kips
ΣM = 218999.8 k-ft

USACE Stability

Force Resultant Location @ Base L =	19.49	ft		**OK**	**Crack**
Force Resultant Location Offset e =	0.89	ft			**0.00 ft**
Foundation Bearing p_{max} =	5663	psf	10000	**OK**	
Foundation Bearing p_{min} =	4241	psf			
Sliding FOS =	2.79		2.00 Reqr'd	**OK**	For Usual LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	19.49	ft			
Force Resultant Location Offset e =	0.89	ft			
Foundation Bearing p_{max} =	5663	psf	10000	**OK**	
Foundation Bearing p_{min} =	4241	psf			
Sliding FOS =	2.79		3.00 Reqr'd	**NG**	For Usual LC, Rock Foundation.
Sliding FOS (No Cohesion) =	1.52		1.50 Reqr'd	**OK**	For Usual LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
Approved by:		Date:		Subject:	Delhi Lake Dam Reconstruction
					Dam / Powerhouse Stability
					Powerhouse Stability LC_2
				Sheet No.	of

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
1,5,6,7,8,13	560.1	1	41.5	150	3486.4	318.8	92614
2 tunnel	28.0	1	41.5	-150	-174.3	374.0	-5432
3 stone fill	87.7	1	32	-35	-98.2	276.0	-2258
4	191.3	1	12	150	344.3	495.6	14216
9, 10	30.0	1	29.5	150	132.8	120.0	1328
11,12	1111.9	1	9.5	150	1584.5	269.5	35582
14	481.5	1	3	150	216.7	138.8	2507
Equipments					200.0	120.0	2000
20 wall + roof	399.6	1	12.5	150	749.3	361.0	22543
21 tunnel	28.0	1	12.5	-150	-52.5	374.0	-1636
22 stone fill	350.1	1	12.5	115	503.2	169.1	7091
23	140.4	1	12.5	150	263.3	135.0	2961
24 side wall	1988.4	1	4	150	1193.1	224.4	22309
Total					**8348.5**		**193824**

Top of Bridge =	904.8 ft		
Total Length of Powerhouse =	61.0 ft	Ice Load =	0.0 klf
Bottom of Dam EL =	846.3 ft	L1 =	20.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	17.2 ft
Dam / Foundation Bonding=	2880 psf	step =	8.0 ft
Length of Seepage Path =	45.2 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	881.3 ft	Foundation Width =	37.2 ft

2. Case II: Unusual Flood Discharge Condition

Head Water EL = 900.0 ft

Tail Water EL = 878.9 ft

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
Approved by:		Date:		Subject:	Delhi Lake Dam Reconstruction
					Dam / Powerhouse Stability
					Powerhouse Stability LC_2
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Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	2028.6	22.5	45555
Top Water weight 2			
Upward pressure	-310.0	41.0	-12710
Upward pressure at US piers	-273.8	41.2	-11279
Total	**1444.8**		**21566**

Uplift	21.1	32.6	21.1	0%
Upstream	Head 1 = 45.7		ft	Crack input
Downstream	Head 2 = 32.6		ft	0.00 ft
	Seep Grade = 0.467		ft/ft	40.6

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	37	0
uplift 1 (total rectangular_US)	-3479.0	27.2	-94630
uplift 2 (add back triangular_US)	355.4	23.9	8482
uplift 3 (rectangular_DS)	-2134.3	8.6	-18355
uplift 4 (triangular_DS)	-262.8	11.5	-3014
Total	**-5520.8**		**-107518**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream 1	3974.8	23.2	-92348
Upstream 2 at Step	1237.2	4.0	-4949
Downstream	-2022.6	10.9	21979
Total	**3189.4**		**-75318**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	935.6	17.0	-15906
Upstream - Ice	0.0	53.7	0
Total	**935.6**		**-15906**

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
Approved by:		Date:		Subject:	Delhi Lake Dam Reconstruction
					Dam / Powerhouse Stability
					Powerhouse Stability LC_2
				Sheet No.	of

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	8348.5		193824
Weight of Water	1444.8		21566
Driving of Water		3189.4	-75318
Uplift at Efficiency = 0	-5520.8		-107518
Silt & Ice		935.6	-15906
Rock Anchor US @ 33.0 ft.	**7000.0**		**231000**
Total	**11272.5**	**4125.0**	**247649.3**

ΣV = 11272.5 kips
ΣH = 4125.0 kips
ΣM = 247649.3 k-ft

USACE Stability

Force Resultant Location @ Base L =	21.97 ft		**OK**	**Crack**
Force Resultant Location Offset e =	3.37 ft			**0.00 ft**
Foundation Bearing p_{max} =	7667 psf	10000	**OK**	
Foundation Bearing p_{min} =	2268 psf			
Sliding FOS =	3.50	1.70 Reqr'd	**OK**	

For Unusual LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	21.97 ft		
Force Resultant Location Offset e =	3.37 ft		
Foundation Bearing p_{max} =	7667 psf	10000	**OK**
Foundation Bearing p_{min} =	2268 psf		
Sliding FOS =	3.50	2.00 Reqr'd	**OK**
Sliding FOS (No Cohesion) =	1.91	1.50 Reqr'd	**OK**

For Unusual LC, Rock Foundation.

For Unusual LC, Rock Foundation. No Cohesion.

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Powerhouse Stability LC_3
				Sheet No.	of

1. Weight Computations

Item	A1 (ft^2)	D2	D3 (ft)	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
1,5,6,7,8,13	560.1	1	41.5	150	3486.4	318.8	92614
2 tunnel	28.0	1	41.5	-150	-174.3	374.0	-5432
3 stone fill	87.7	1	32	-35	-98.2	276.0	-2258
4	191.3	1	12	150	344.3	495.6	14216
9, 10	30.0	1	29.5	150	132.8	120.0	1328
11,12	1111.9	1	9.5	150	1584.5	269.5	35582
14	481.5	1	3	150	216.7	138.8	2507
Equipments					200.0	120.0	2000
20 wall + roof	399.6	1	12.5	150	749.3	361.0	22543
21 tunnel	28.0	1	12.5	-150	-52.5	374.0	-1636
22 stone fill	350.1	1	12.5	115	503.2	169.1	7091
23	140.4	1	12.5	150	263.3	135.0	2961
24 side wall	1988.4	1	4	150	1193.1	224.4	22309
Total					**8348.5**		**193824**

Top of Bridge =	904.8 ft		
Total Length of Powerhouse =	61.0 ft	Ice Load =	0.0 klf
Bottom of Dam EL =	846.3 ft	L1 =	20.0 ft
Dam / Foundation Friction Angle =	35 degrees	L2 =	17.2 ft
Dam / Foundation Bonding=	2880 psf	step =	8.0 ft
Length of Seepage Path =	45.2 ft	Allowable Bearing =	10000 psf
Top Of Crest EL =	881.3 ft	Foundation Width =	37.2 ft

2. Case III: Extreme Flood Discharge Condition

Head Water EL = 906.0 ft

Tail Water EL = 888.7 ft

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Dam / Powerhouse Stability
					Powerhouse Stability LC_3
				Sheet No.	of

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	2220.3	22.5	49860
Top Water weight 2	7.2	44.2	316
Upward pressure	-399.9	41.0	-16394
Upward pressure at US piers	-309.7	41.2	-12760
Total	**1517.9**		**21022**

Uplift	17.3	42.4	17.3	0%
Upstream	Head 1 = 51.7	ft		Crack input
Downstream	Head 2 = 42.4	ft		0.00 ft
	Seep Grade = 0.383	ft/ft		49.0

	U (kip)	L (ft)	M (k-ft)
uplift - crack	0.0	37	0
uplift 1 (total rectangular_US)	-3935.8	27.2	-107054
uplift 2 (add back triangular_US)	291.4	23.9	6954
uplift 3 (rectangular_DS)	-2775.9	8.6	-23873
uplift 4 (triangular_DS)	-215.5	11.5	-2471
Total	**-6635.9**		**-126444**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream_Rectangular	-159.9	34.3	5476
Upstream_Triangular	5245.7	25.5	-133765
Upstream 2 at Step	1491.6	4.0	-5966
Downstream	-3421.5	14.1	48357
Total	**3155.9**		**-85899**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	935.6	17.0	-15906
Upstream - Ice	0.0	59.7	0
Total	**935.6**		**-15906**

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
Approved by:		Date:		Sheet No.	of

Subject: Delhi Lake Dam Reconstruction
Dam / Powerhouse Stability
Powerhouse Stability LC_3

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	8348.5		193824
Weight of Water	1517.9		21022
Driving of Water		3155.9	-85899
Uplift at Efficiency = 0	-6635.9		-126444
Silt & Ice		935.6	-15906
Rock Anchor US @ 33.0 ft.	**7000.0**		**231000**
Total	**10230.5**	**4091.6**	**217597.4**

$\Sigma V =$ 10230.5 kips
$\Sigma H =$ 4091.6 kips
$\Sigma M =$ 217597.4 k-ft

USACE Stability

Force Resultant Location @ Base L =	21.27 ft		OK	Crack
Force Resultant Location Offset e =	2.67 ft			0.00 ft
Foundation Bearing p_{max} =	6450 psf	13300	OK	
Foundation Bearing p_{min} =	2567 psf			
Sliding FOS =	3.35	1.30 Reqr'd	OK	

For Extreme LC, Rock Foundation.

FERC Stability

Force Resultant Location @ Base L =	21.27 ft		
Force Resultant Location Offset e =	2.67 ft		
Foundation Bearing p_{max} =	6450 psf	13300	OK
Foundation Bearing p_{min} =	2567 psf		
Sliding FOS =	3.35	2.00 Reqr'd	OK
Sliding FOS (No Cohesion) =	1.75	1.50 Reqr'd	OK

For Extreme LC, Rock Foundation.

For Extreme LC, Rock Foundation. No Cohesion.

ANCHORED TO FERC CRITERIA

Stanley Consultants INC.

Computed By:	Y.Ding	**Date:**	11/29/2011	**Job No.**	23601 Delhi Lake Dam Reconstruction
Checked By:	E.Daly	**Date:**	12/19/2011	**Subject**	Dam / Powerhouse Stability
					Powerhouse Rock Anchor Design - FERC

Rock Unit Weight =	152.4	pcf	
Rock Buoyant Unit Weight =	90	pcf	(recommended in Geotechical report)
Anchor Spacing =	6.0	ft	
Total Required Anchor Force =	7000	k	(unfactored required anchor force under normal load condition)
Total Number of Anchors =	20		
Required Effective Anchor Force =	**350**	k	
Grouted Rock Shear Allowable Strength =	**5760**	psf	
Grout Hole Diameter =	**5.5**	in.	
Anchor to Fractured Rock Bond Ultimate =	**150**	psi	(recommended in Geotechical report)

Anchor Length			Total Cone					Overlap Reduction							Net Wt.
Unbonded (ft)	Total L (ft)	Bonding L (ft)	Alpha (deg)	Cone (ft)	r (ft)	Vol (cf)	Wt (kips)	d (ft)	d/D	A/Atotal	A (sf)	h (ft)	Vol (cf)	Wt (kips)	**(kips)**
25	70	45	**60**	37.5	21.7	18,408	1,657	18.7	0.431	0.374	550	32.3	11,846	1,066	**591**

EM 1110-1-2908, Page 9-2, (9-2).	D =	40.5	ft	good
EM 1110-1-2908, Page 9-3, (9-5).	D =	31.2	ft	good
EM 1110-1-2908, Page 9-3, (9-6a).				
	Grout Hole Perimeter =	1.44	ft	
	Bond Surface =	57.6	sf	
	Allowable Force by Bonding =	**622.0**	k	

Bonded Length under dam	45
Unbonded Length under dam	15
Unbonded Length above bott. of Dam	10

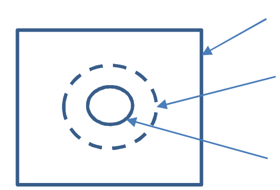
Anchorage Steel Bar Ultimate Stress =	150	ksi
Initial Prestress =	133%	
Factor of Safety for Steel Bar =	1.25	
required Garranteed Ultimate Tensile Strength (GUTS) =	583	k
Required Steel Bar Area =	3.89	in^2
Minimum Steel Bar Size =	2.23	in

Use 150 ksi All-Thread-Bar (Williams Form Engineering Corp.) **2 1/4** **in. Diameter.**

At Concrete/Steel Plate Contact				At Depth of OLD Concrete			
Initial Prestress	466.67	k		Initial Prestress	466.67	k	
Assumed Concrete Strength	4000	psi		Assumed Concrete Strength	4000	psi	
Allowable Concrete Bearing Pressure	2800	psi	0.7 of f_c')	Allowable Concrete Bearing Pressure	2800	psi	0.7 of f_c')
Minimum Bearing Area	166.67	in^2		Minimum Bearing Area	166.67	in^2	
Minimum Size of Sqare Steel Pate	14	in		Depth of Old Concrete from Plate	0	in	
				Minimum Size of Sqare Steel Pate	14	in	

Use of Square Plate 14.0 in. X 14.0 in. X 4.0 in. Thk

Hole Size for Anchor	2 3/4	in	86.1209
Pressure on Plate	2709	psi	11.8791
Total Force on One Side	233	k	98
Moment Arm	3.82	in	0.57559
Moment	892	k-in	0.42441
Minimum Plate Thickness	4.00	in	(Use **50** ksi steel)



50ksi Steel Plate
14.0 in.X 14.0 in.X 4.0 in. Thk

Drill Hole in Concrete
Dia.= 5.5 in.

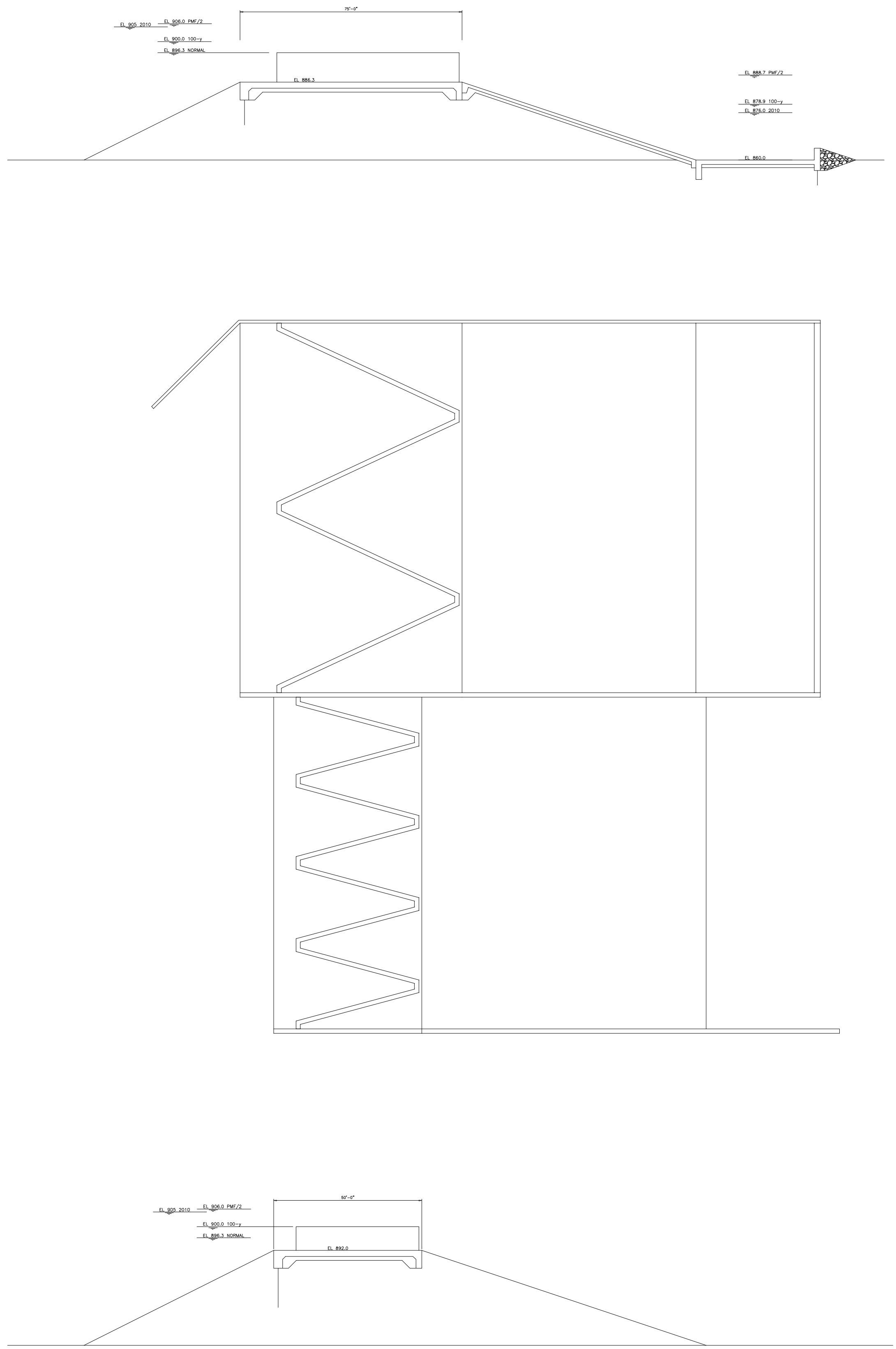
Hole in Steel Plate
Dia.= 2 3/4 in.

New Labyrinth Spillway

Dual – Tiered

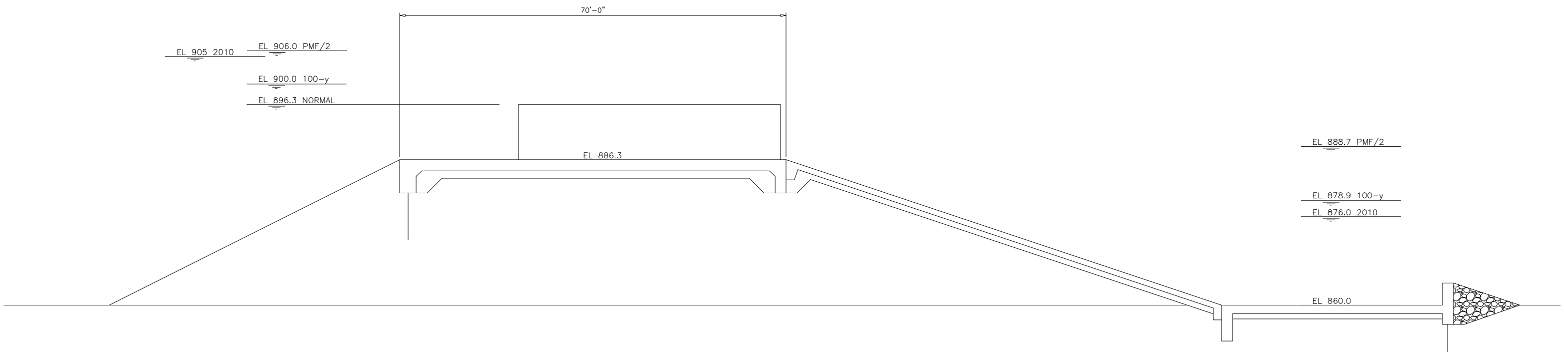
Service Spillway

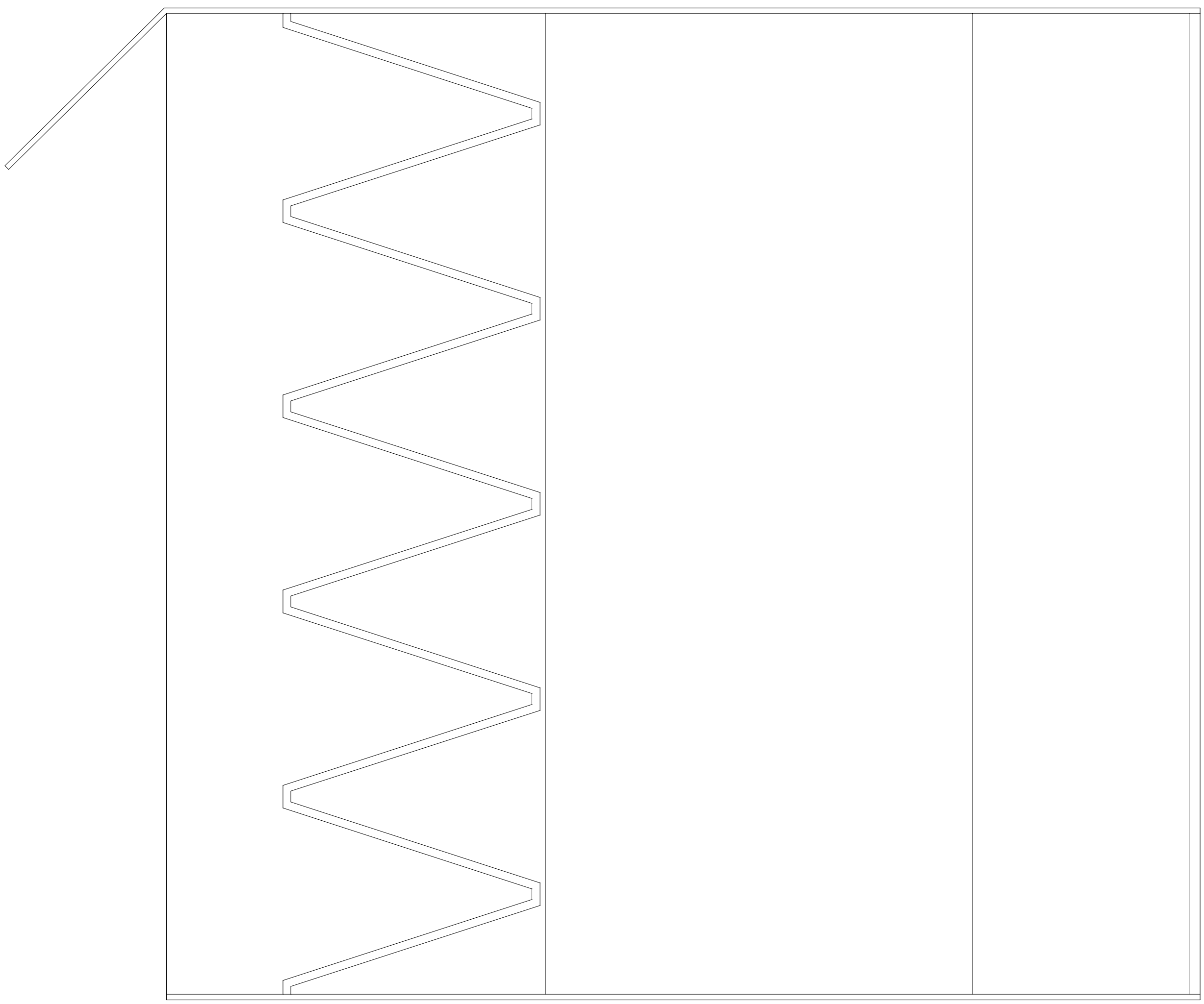
NEW LABYRINTH SPILLWAY



Dual Labyrinth
Spillway Section

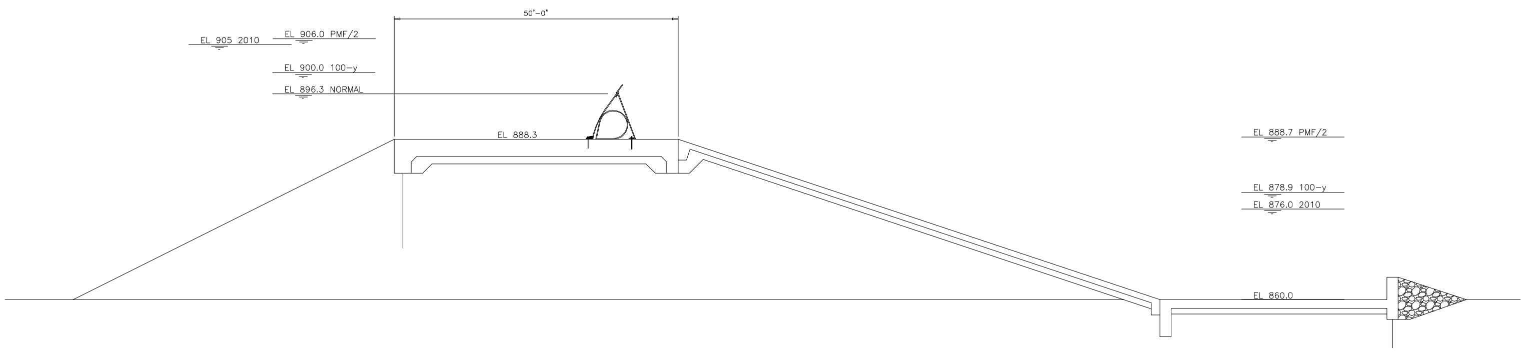
NEW LABYRINTH SPILLWAY

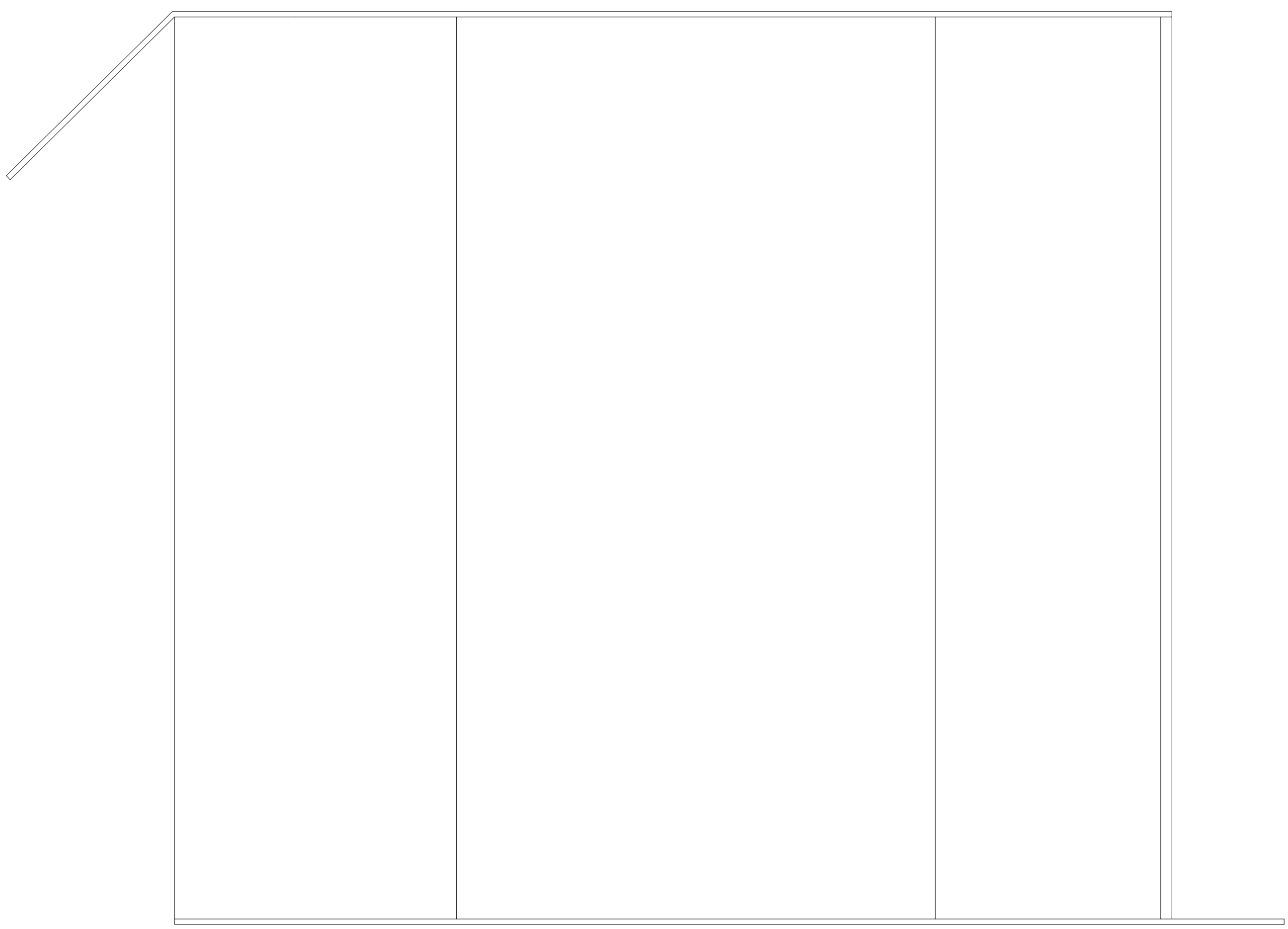




Single Labyrinth
Spillway Section

NEW PNEUMATIC GATE SPILLWAY





Obermeyer Gate
Spillway Section

Stanley Consultants INC.

				Job No.	23601 Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Labyrinth (1) Spillway Stability - 75ft
Checked by:	E.Daly	Date:	12/16/11		Stability LC_1
Approved by:		Date:		Sheet No.	of

1. Weight Computations

Item	D1	D2	D3	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Slab concrete	171.0	1	126	150	3231.9	450.0	121196
Wall concrete	416.0	1	10	150	624.0	381.0	19812
Soil under slab	279.0	1	126	115	4042.7	450.0	151602
Water (used below)	5156.6	50.0					
Slab width (used below)		75					
Total					**7898.6**		**292610**

Top of Slab =	886.3 ft		0 psi
Total Length of spillway =	126.0 ft	Ice Load =	5.0 klf
Bottom of key EL =	880.3 ft	L1 =	75.0 ft
Foundation Friction Angle =	28 degrees	L2 =	0.0 ft
Foundation Bonding=	0 psf	step =	0.0 ft
Length of Seepage Path =	75 ft	Allowable Bearing =	2000 psf
Top Of Crest EL =	896.3 ft	Foundation Width =	75.0 ft

2. Case I: Normal Operating Condition

Head Water EL = 896.3 ft

Tail Water EL = 857.0 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	3217.7	50.0	160795
Water weight 2	0.0	50.0	0
Total	**3217.7**		**160795**

Uplift 50%

Upstream	Head 1 = 8.0	ft
Downstream	Head 2 = 0	ft

Stanley Consultants INC.

				Job No.	23601
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				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Labyrinth (1) Spillway Stability - 75ft
Checked by:	E.Daly	Date:	12/16/11		Stability LC_1
Approved by:		Date:		Sheet No.	of

	U (kip)	L (ft)	M (k-ft)
uplift (rectangular)	0.0	38	0
uplift (triangular)	-2358.7	50	-117936
Total	**-2358.7**		**-117936**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream rectangular	0.0	8.0	0
Upstream triangular	1006.4	5.3	-5367
Downstream	0.0	0.0	0
Total	**1006.4**		**-5367**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	0.0	0.0	0
Upstream - Ice	630.0	16.0	-10080
Total	**630.0**		**-10080**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	7898.6		292610
Weight of Water	3217.7		160795
Driving of Water		1006.4	-5367
Uplift at Efficiency = 0.5	-2358.7		-117936
Silt & Ice		630.0	-10080
Total	**8757.6**	**1636.4**	**320021.8**

$\Sigma V =$ 8757.6 kips

$\Sigma H =$ 1636.4 kips

$\Sigma M =$ 320021.8 k-ft

Stanley Consultants INC.			Job No. 23601	Page No.
			Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date: 12/05/11		Labyrinth (1) Spillway Stability - 75ft
Checked by:	E.Daly	Date: 12/16/11		Stability LC_1
Approved by:		Date:	Sheet No.	of

USACE Stability

Force Resultant Location @ Base L =	36.54 ft		**OK**	**Crack**
Force Resultant Location Offset e =	0.96 ft			**0.00 ft**
Foundation Bearing p_{max} =	998 psf	2000	**OK**	
Foundation Bearing p_{min} =	856 psf			
Sliding FOS =	2.85	2.00 Reqr'd	**OK**	For Usual LC,

FERC Stability

Force Resultant Location @ Base L =	36.54 ft		
Force Resultant Location Offset e =	0.96 ft		
Foundation Bearing p_{max} =	998 psf	2000	**OK**
Foundation Bearing p_{min} =	856 psf		
Sliding FOS (No Cohesion) =	2.85	1.50 Reqr'd	**OK**

For Usual LC, No Cohesion.

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
Approved by:		Date:		Subject:	Delhi Lake Dam Reconstruction
					Labyrinth (1) Spillway Stability - 75ft Stability LC_2
				Sheet No.	of

1. Weight Computations

Item	D1	D2	D3	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Slab concrete	171.0	1	126	150	3231.9	450.0	121196
Wall concrete	416.0	1	10	150	624.0	381.0	19812
Soil under slab	279.0	1	126	115	4042.7	450.0	151602
Water (used below)	5156.6	50.0					
Slab width (used below)		75					
Total					**7898.6**		**292610**

			0 psi
Top of Slab =	886.3 ft	Ice Load =	0.0 klf
Total Length of spillway =	126.0 ft	L1 =	75.0 ft
Bottom of key EL =	880.3 ft	L2 =	0.0 ft
Foundation Friction Angle =	28 degrees	step =	0.0 ft
Foundation Bonding=	0 psf	Allowable Bearing =	2000 psf
Length of Seepage Path =	75 ft	Foundation Width =	75.0 ft
Top Of Crest EL =	896.3 ft		

2. Case II: 100 Year Flood Condition

Head Water EL = 900.0 ft

Tail Water EL = 878.9 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	3217.7	50.0	160795
Water weight 2	595.3	50.0	29747
Total	**3813.0**		**190542**

Uplift 50%

Upstream Head 1 = 9.9 ft

Downstream Head 2 = 0 ft

Stanley Consultants INC.

Job No. 23601 Page No.
Subject: Delhi Lake Dam Reconstruction
Labyrinth (1) Spillway Stability - 75ft
Stability LC_2

Computed by: Y.Ding Date: 12/05/11
Checked by: E.Daly Date: 12/16/11
Approved by: Date: Sheet No. of

	U (kip)	L (ft)	M (k-ft)
uplift (rectangular)	0.0	38	0
uplift (triangular)	-2904.2	50	-145209
Total	**-2904.2**		**-145209**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream rectangular	465.5	8.0	-3724
Upstream triangular	1006.4	5.3	-5367
Downstream	0.0	0.0	0
Total	**1471.8**		**-9091**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	0.0	0.0	0
Upstream - Ice	0.0	19.7	0
Total	**0.0**		**0**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	7898.6		292610
Weight of Water	3813.0		190542
Driving of Water		1471.8	-9091
Uplift at Efficiency = 0.5	-2904.2		-145209
Silt & Ice		0.0	0
Total	**8807.4**	**1471.8**	**328852.6**

$\Sigma V =$ 8807.4 kips
$\Sigma H =$ 1471.8 kips
$\Sigma M =$ 328852.6 k-ft

Stanley Consultants INC.				Job No.	23601	Page No.	
				Subject:	Delhi Lake Dam Reconstruction		
Computed by:	Y.Ding	Date:	12/05/11		Labyrinth (1) Spillway Stability - 75ft		
Checked by:	E.Daly	Date:	12/16/11		Stability LC_2		
Approved by:		Date:		Sheet No.	of		

USACE Stability

Force Resultant Location @ Base L =	37.34 ft		OK	Crack
Force Resultant Location Offset e =	0.16 ft			0.00 ft
Foundation Bearing p_{max} =	944 psf	2000	OK	
Foundation Bearing p_{min} =	920 psf			For Unusual LC,
Sliding FOS =	3.18	1.70 Reqr'd	OK	

FERC Stability

Force Resultant Location @ Base L =	37.34 ft		
Force Resultant Location Offset e =	0.16 ft		
Foundation Bearing p_{max} =	944 psf	2000	OK
Foundation Bearing p_{min} =	920 psf		
Sliding FOS (No Cohesion) =	3.18	1.50 Reqr'd	OK

For Unusual LC, No Cohesion.

Stanley Consultants INC.				Job No.	23601	Page No.	
				Subject:	Delhi Lake Dam Reconstruction		
Computed by:	Y.Ding	Date:	12/05/11		Labyrinth (1) Spillway Stability - 75ft		
Checked by:	E.Daly	Date:	12/16/11		Stability LC_3		
Approved by:		Date:		Sheet No.	of		

1. Weight Computations

Item	D1	D2	D3	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Slab concrete	171.0	1	126	150	3231.9	450.0	121196
Wall concrete	416.0	1	10	150	624.0	381.0	19812
Soil under slab	279.0	1	126	115	4042.7	450.0	151602
Water (used below)	5156.6	50.0					
Slab width (used below)		75					
Total					**7898.6**		**292610**

Top of Slab =	886.3 ft		0 psi
Total Length of spillway =	126.0 ft	Ice Load =	**0.0** klf
Bottom of key EL =	880.3 ft	L1 =	75.0 ft
Foundation Friction Angle =	28 degrees	L2 =	0.0 ft
Foundation Bonding=	0 psf	step =	0.0 ft
Length of Seepage Path =	75 ft	Allowable Bearing =	2000 psf
Top Of Crest EL =	896.3 ft	Foundation Width =	75.0 ft

2. Case III: PMF/2 Flood Condition

Head Water EL = 906.0 ft

Tail Water EL = 888.7 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	3217.7	50.0	160795
Water weight 2	3121.2	50.0	155971
Total	**6338.9**		**316767**

Uplift 50%

Upstream	Head 1 = 17.1	ft
Downstream	Head 2 = 8.4	ft

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Labyrinth (1) Spillway Stability - 75ft
					Stability LC_3
				Sheet No.	of

	U (kip)	L (ft)	M (k-ft)
uplift (rectangular)	-4953.3	38	-185749
uplift (triangular)	-2550.4	50	-127518
Total	**-7503.7**		**-313268**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream rectangular	1220.2	8.0	-9762
Upstream triangular	1006.4	5.3	-5367
Downstream	0.0	0.0	0
Total	**2226.6**		**-15129**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	0.0	0.0	0
Upstream - Ice	0.0	25.7	0
Total	**0.0**		**0**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	7898.6		292610
Weight of Water	6338.9		316767
Driving of Water		2226.6	-15129
Uplift at Efficiency = 0.5	-7503.7		-313268
Silt & Ice		0.0	0
Total	**6733.9**	**2226.6**	**280979.8**

ΣV = 6733.9 kips

ΣH = 2226.6 kips

ΣM = 280979.8 k-ft

Stanley Consultants INC.				Job No. 23601	Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Labyrinth (1) Spillway Stability - 75ft
Checked by:	E.Daly	Date:	12/16/11		Stability LC_3
Approved by:		Date:		Sheet No.	of

USACE Stability

Force Resultant Location @ Base L =	41.73 ft		OK	Crack
Force Resultant Location Offset e =	4.23 ft			0.00 ft
Foundation Bearing p_{max} =	954 psf	2000	OK	
Foundation Bearing p_{min} =	472 psf			For Extreme LC,
Sliding FOS =	1.61	1.30 Reqr'd	OK	

FERC Stability

Force Resultant Location @ Base L =	41.73 ft		
Force Resultant Location Offset e =	4.23 ft		
Foundation Bearing p_{max} =	954 psf	2000	OK
Foundation Bearing p_{min} =	472 psf		
Sliding FOS (No Cohesion) =	1.61	1.50 Reqr'd	OK

For Extremel LC, No Cohesion.

New Labyrinth Spillway

Dual – Tiered

Auxiliary Spillway

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Labyrinth (2) Spillway Stability - 50ft
					Stability LC_1
				Sheet No.	of

1. Weight Computations

Item	D1	D2	D3	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Slab concrete	121.0	1	113.5	150	2060.0	308.9	53025
Wall concrete	536.2	1	8	150	643.5	261.7	14032
Soil under slab	179.0	1	113.5	115	2336.4	294.0	57242
Water (used below)	2874.5	33.4					
Slab width (used below)		50					
Total					**5039.9**		**124299**

Top of Slab =	892 ft		0 psi
Total Length of spillway =	113.5 ft	Ice Load =	5.0 klf
Bottom of key EL =	886.0 ft	L1 =	50.0 ft
Foundation Friction Angle =	28 degrees	L2 =	0.0 ft
Foundation Bonding=	0 psf	step =	0.0 ft
Length of Seepage Path =	50 ft	Allowable Bearing =	2000 psf
Top Of Crest EL =	900 ft	Foundation Width =	50.0 ft

2. Case I: Normal Operating Condition

Head Water EL = 896.3 ft

Tail Water EL = 857.0 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	771.3	33.4	25774
Water weight 2		33.4	0
Total	**771.3**		**25774**

Uplift 50%

Upstream	Head 1 = 5.1	ft
Downstream	Head 2 = 0	ft

Stanley Consultants INC.

				Job No.	23601 Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Labyrinth (2) Spillway Stability - 50ft
Checked by:	E.Daly	Date:	12/16/11		Stability LC_1
Approved by:		Date:		Sheet No.	of

	U (kip)	L (ft)	M (k-ft)
uplift (rectangular)	0.0	25	0
uplift (triangular)	-911.9	33	-30395
Total	**-911.9**		**-30395**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream rectangular	0.0	7.0	0
Upstream triangular	375.7	3.4	-1290
Downstream	0.0	0.0	0
Total	**375.7**		**-1290**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	0.0	0.0	0
Upstream - Ice	567.5	10.3	-5845
Total	**567.5**		**-5845**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	5039.9		124299
Weight of Water	771.3		25774
Driving of Water		375.7	-1290
Uplift at Efficiency = 0.5	-911.9		-30395
Silt & Ice		567.5	-5845
Total	**4899.3**	**943.2**	**112542.2**

$\Sigma V =$ 4899.3 kips

$\Sigma H =$ 943.2 kips

$\Sigma M =$ 112542.2 k-ft

Stanley Consultants INC.				Job No.	23601	Page No.	
				Subject:	Delhi Lake Dam Reconstruction		
Computed by:	Y.Ding	Date:	12/05/11		Labyrinth (2) Spillway Stability - 50ft		
Checked by:	E.Daly	Date:	12/16/11		Stability LC_1		
Approved by:		Date:		Sheet No.	of		

USACE Stability

Force Resultant Location @ Base L =	22.97 ft			OK	Crack
Force Resultant Location Offset e =	2.03 ft				0.00 ft
Foundation Bearing p_{max} =	1074 psf	2000		OK	
Foundation Bearing p_{min} =	653 psf				
Sliding FOS =	2.76	2.00 Reqr'd		OK	For Usual LC,

FERC Stability

Force Resultant Location @ Base L =	22.97 ft		
Force Resultant Location Offset e =	2.03 ft		
Foundation Bearing p_{max} =	1074 psf	2000	OK
Foundation Bearing p_{min} =	653 psf		
Sliding FOS (No Cohesion) =	2.76	1.50 Reqr'd	OK

For Usual LC, No Cohesion.

Stanley Consultants INC.

				Job No.	23601 Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Labyrinth (2) Spillway Stability - 50ft
Checked by:	E.Daly	Date:	12/16/11		Stability LC_2
Approved by:		Date:		Sheet No.	of

1. Weight Computations

Item	D1	D2	D3	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Slab concrete	121.0	1	113.5	150	2060.0	308.9	53025
Wall concrete	536.2	1	8	150	643.5	261.7	14032
Soil under slab	179.0	1	113.5	115	2336.4	294.0	57242
Water (used below)	2874.5	33.4					
Slab width (used below)		50					
Total					**5039.9**		**124299**

Top of Slab =	892 ft		0 psi
Total Length of spillway =	113.5 ft	Ice Load =	0.0 klf
Bottom of key EL =	886.0 ft	L1 =	50.0 ft
Foundation Friction Angle =	28 degrees	L2 =	0.0 ft
Foundation Bonding=	0 psf	step =	0.0 ft
Length of Seepage Path =	50 ft	Allowable Bearing =	2000 psf
Top Of Crest EL =	900 ft	Foundation Width =	50.0 ft

2. Case II: 100 Year Flood Condition

Head Water EL = 900.0 ft

Tail Water EL = 878.9 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	1435.0	33.4	47951
Water weight 2	0.0	33.4	0
Total	**1435.0**		**47951**

Uplift 50%

Upstream	Head 1 = 7.0	ft
Downstream	Head 2 = 0	ft

Stanley Consultants INC.

Computed by: Y.Ding	Date: 12/05/11	Job No. 23601	Page No.
Checked by: E.Daly	Date: 12/16/11	Subject: Delhi Lake Dam Reconstruction	
Approved by:	Date:	Labyrinth (2) Spillway Stability - 50ft	
		Stability LC_2	
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	U (kip)	L (ft)	M (k-ft)
uplift (rectangular)	0.0	25	0
uplift (triangular)	-1239.4	33	-41314
Total	**-1239.4**		**-41314**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream rectangular	0.0	7.0	0
Upstream triangular	694.1	4.7	-3239
Downstream	0.0	0.0	0
Total	**694.1**		**-3239**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	0.0	0.0	0
Upstream - Ice	0.0	14.0	0
Total	**0.0**		**0**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	5039.9		124299
Weight of Water	1435.0		47951
Driving of Water		694.1	-3239
Uplift at Efficiency = 0.5	-1239.4		-41314
Silt & Ice		0.0	0
Total	**5235.5**	**694.1**	**127697.1**

$\Sigma V =$ 5235.5 kips

$\Sigma H =$ 694.1 kips

$\Sigma M =$ 127697.1 k-ft

Stanley Consultants INC.		Job No. 23601	Page No.
Computed by: Y.Ding	Date: 12/05/11	Subject:	Delhi Lake Dam Reconstruction
			Labyrinth (2) Spillway Stability - 50ft
Checked by: E.Daly	Date: 12/16/11		Stability LC_2
Approved by:	Date:	Sheet No.	of

USACE Stability

Force Resultant Location @ Base L =	24.39 ft		**OK**	**Crack**
Force Resultant Location Offset e =	0.61 ft			**0.00 ft**
Foundation Bearing p_{max} =	990 psf	2000	**OK**	
Foundation Bearing p_{min} =	855 psf			For Unusual LC,
Sliding FOS =	4.01	1.70 Reqr'd	**OK**	

FERC Stability

Force Resultant Location @ Base L =	24.39 ft		
Force Resultant Location Offset e =	0.61 ft		
Foundation Bearing p_{max} =	990 psf	2000	**OK**
Foundation Bearing p_{min} =	855 psf		
Sliding FOS (No Cohesion) =	4.01	1.50 Reqr'd	**OK**

For Unusual LC, No Cohesion.

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
Approved by:		Date:		Subject:	Delhi Lake Dam Reconstruction
					Labyrinth (2) Spillway Stability - 50ft
					Stability LC_3
				Sheet No.	of

1. Weight Computations

Item	D1	D2	D3	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Slab concrete	121.0	1	113.5	150	2060.0	308.9	53025
Wall concrete	536.2	1	8	150	643.5	261.7	14032
Soil under slab	179.0	1	113.5	115	2336.4	294.0	57242
Water (used below)	2874.5	33.4					
Slab width (used below)		50					
Total					**5039.9**		**124299**

Top of Slab =	892 ft		0 psi
Total Length of spillway =	113.5 ft	Ice Load =	0.0 klf
Bottom of key EL =	886.0 ft	L1 =	50.0 ft
Foundation Friction Angle =	28 degrees	L2 =	0.0 ft
Foundation Bonding=	0 psf	step =	0.0 ft
Length of Seepage Path =	50 ft	Allowable Bearing =	2000 psf
Top Of Crest EL =	900 ft	Foundation Width =	50.0 ft

2. Case III: PMF/2 Flood Condition

Head Water EL = 906.0 ft

Tail Water EL = 888.7 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	1435.0	33.4	47951
Water weight 2	1076.2	33.4	35964
Total	**2511.2**		**83915**

Uplift 50%

Upstream	Head 1 = 11.4	ft
Downstream	Head 2 = 2.7	ft

Stanley Consultants INC.

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Computed by:	Y.Ding	Date:	12/05/11		Labyrinth (2) Spillway Stability - 50ft
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	U (kip)	L (ft)	M (k-ft)
uplift (rectangular)	-956.1	25	-23903
uplift (triangular)	-1531.6	33	-51052
Total	**-2487.7**		**-74955**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream rectangular	594.9	7.0	-4164
Upstream triangular	694.1	4.7	-3239
Downstream	0.0	0.0	0
Total	**1289.0**		**-7403**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	0.0	0.0	0
Upstream - Ice	0.0	20.0	0
Total	**0.0**		**0**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	5039.9		124299
Weight of Water	2511.2		83915
Driving of Water		1289.0	-7403
Uplift at Efficiency = 0.5	-2487.7		-74955
Silt & Ice		0.0	0
Total	**5063.4**	**1289.0**	**125854.9**

ΣV = 5063.4 kips

ΣH = 1289.0 kips

ΣM = 125854.9 k-ft

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Computed by: Y.Ding	Date: 12/05/11	Labyrinth (2) Spillway Stability - 50ft	
Checked by: E.Daly	Date: 12/16/11	Stability LC_3	
Approved by:	Date:	Sheet No. of	

USACE Stability

Force Resultant Location @ Base L =	24.86	ft		OK	Crack
Force Resultant Location Offset e =	0.14	ft			0.00 ft
Foundation Bearing p_{max} =	908	psf	2000	OK	
Foundation Bearing p_{min} =	877	psf			For Extreme LC,
Sliding FOS =	2.09		1.30 Reqr'd	OK	

FERC Stability

Force Resultant Location @ Base L =	24.86	ft		
Force Resultant Location Offset e =	0.14	ft		
Foundation Bearing p_{max} =	908	psf	2000	OK
Foundation Bearing p_{min} =	877	psf		
Sliding FOS (No Cohesion) =	2.09		1.50 Reqr'd	OK

For Extremel LC, No Cohesion.

New Labyrinth Spillway

Single

Service/Auxiliary Spillway

Stanley Consultants INC.			Job No.	23601	Page No.	
			Subject:	Delhi Lake Dam Reconstruction		
Computed by:	Y.Ding	Date: 12/05/11		Labrinth (3) Spillway Stability - 70ft		
Checked by:	E.Daly	Date: 12/16/11		Stability LC_1		
Approved by:		Date:	Sheet No.	of		

1. Weight Computations

Item	D1	D2	D3	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Slab concrete	161.0	1	181	150	4371.2	429.7	156505
Wall concrete	773.4	1	10	150	1160.0	297.5	28763
Soil under slab	259.0	1	181	115	5391.1	414.0	185992
Water (used below)	7796.9	45.8					
Slab width (used below)		70					
Total					**10922.3**		**371261**

Top of Slab =	886.3 ft		0 psi
Total Length of spillway =	181.0 ft	Ice Load =	5.0 klf
Bottom of key EL =	880.3 ft	L1 =	70.0 ft
Foundation Friction Angle =	28 degrees	L2 =	0.0 ft
Foundation Bonding=	0 psf	step =	0.0 ft
Length of Seepage Path =	70 ft	Allowable Bearing =	2000 psf
Top Of Crest EL =	896.3 ft	Foundation Width =	70.0 ft

2. Case I: Normal Operating Condition

Head Water EL = 896.3 ft

Tail Water EL = 857.0 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	4865.3	45.8	222627
Water weight 2	0.0	45.8	0
Total	**4865.3**		**222627**

Uplift 50%

Upstream	Head 1 = 8.0	ft
Downstream	Head 2 = 0	ft

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601
Checked by:	E.Daly	Date:	12/16/11	Page No.	
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Subject: Delhi Lake Dam Reconstruction
Labrinth (3) Spillway Stability - 70ft
Stability LC_1

	U (kip)	L (ft)	M (k-ft)
uplift (rectangular)	0.0	35	0
uplift (triangular)	-3162.4	47	-147580
Total	**-3162.4**		**-147580**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream rectangular	0.0	8.0	0
Upstream triangular	1445.7	5.3	-7710
Downstream	0.0	0.0	0
Total	**1445.7**		**-7710**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	0.0	0.0	0
Upstream - Ice	905.0	16.0	-14480
Total	**905.0**		**-14480**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	10922.3		371261
Weight of Water	4865.3		222627
Driving of Water		1445.7	-7710
Uplift at Efficiency = 0.5	-3162.4		-147580
Silt & Ice		905.0	-14480
Total	**12625.1**	**2350.7**	**424117.3**

$\Sigma V =$ 12625.1 kips
$\Sigma H =$ 2350.7 kips
$\Sigma M =$ 424117.3 k-ft

Stanley Consultants INC.			Job No.	23601	Page No.
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Computed by:	Y.Ding	Date: 12/05/11		Labrinth (3) Spillway Stability - 70ft	
Checked by:	E.Daly	Date: 12/16/11		Stability LC_1	
Approved by:		Date:	Sheet No.	of	

USACE Stability

Force Resultant Location @ Base L =	33.59 ft		**OK**	**Crack**
Force Resultant Location Offset e =	1.41 ft			**0.00 ft**
Foundation Bearing p_{max} =	1117 psf	2000	**OK**	
Foundation Bearing p_{min} =	876 psf			
Sliding FOS =	2.86	2.00 Reqr'd	**OK**	For Usual LC,

FERC Stability

Force Resultant Location @ Base L =	33.59 ft		
Force Resultant Location Offset e =	1.41 ft		
Foundation Bearing p_{max} =	1117 psf	2000	**OK**
Foundation Bearing p_{min} =	876 psf		
Sliding FOS (No Cohesion) =	2.86	1.50 Reqr'd	**OK**

For Usual LC, No Cohesion.

Stanley Consultants INC.

				Job No.	23601 Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Labrinth (3) Spillway Stability - 70ft
Checked by:	E.Daly	Date:	12/16/11		Stability LC_2
Approved by:		Date:		Sheet No.	of

1. Weight Computations

Item	D1	D2	D3	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Slab concrete	161.0	1	181	150	4371.2	429.7	156505
Wall concrete	773.4	1	10	150	1160.0	297.5	28763
Soil under slab	259.0	1	181	115	5391.1	414.0	185992
Water (used below)	7796.9	45.8					
Total					**10922.3**		**371261**

Top of Slab =	886.3 ft		0 psi
Total Length of spillway =	181.0 ft	Ice Load =	0.0 klf
Bottom of key EL =	880.3 ft	L1 =	70.0 ft
Foundation Friction Angle =	28 degrees	L2 =	0.0 ft
Foundation Bonding=	0 psf	step =	0.0 ft
Length of Seepage Path =	70 ft	Allowable Bearing =	2000 psf
Top Of Crest EL =	896.3 ft	Foundation Width =	70.0 ft

2. Case II: 100 Year Flood Condition

Head Water EL = 900.0 ft

Tail Water EL = 878.9 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	4865.3	45.8	222627
Water weight 2	900.1	45.8	41186
Total	**5765.4**		**263813**

Uplift 50%

Upstream	Head 1 = 9.9	ft
Downstream	Head 2 = 0	ft

Stanley Consultants INC.

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	U (kip)	L (ft)	M (k-ft)
uplift (rectangular)	0.0	35	0
uplift (triangular)	-3893.7	47	-181708
Total	**-3893.7**		**-181708**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream rectangular	668.6	8.0	-5349
Upstream triangular	1445.7	5.3	-7710
Downstream	0.0	0.0	0
Total	**2114.3**		**-13059**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	0.0	0.0	0
Upstream - Ice	0.0	19.7	0
Total	**0.0**		**0**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	10922.3		371261
Weight of Water	5765.4		263813
Driving of Water		2114.3	-13059
Uplift at Efficiency = 0.5	-3893.7		-181708
Silt & Ice		0.0	0
Total	**12793.9**	**2114.3**	**440306.3**

ΣV = 12793.9 kips

ΣH = 2114.3 kips

ΣM = 440306.3 k-ft

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Checked by:	E.Daly	Date:	12/16/11		Stability LC_2
Approved by:		Date:		Sheet No.	of

USACE Stability

Force Resultant Location @ Base L =	34.42	ft		OK	Crack
Force Resultant Location Offset e =	0.58	ft			0.00 ft
Foundation Bearing p_{max} =	1060	psf	2000	OK	
Foundation Bearing p_{min} =	959	psf			
Sliding FOS =	3.22		1.70 Reqr'd	OK	For Unusual LC,

FERC Stability

Force Resultant Location @ Base L =	34.42	ft		
Force Resultant Location Offset e =	0.58	ft		
Foundation Bearing p_{max} =	1060	psf	2000	OK
Foundation Bearing p_{min} =	959	psf		
Sliding FOS (No Cohesion) =	3.22		1.50 Reqr'd	OK

For Unusual LC, No Cohesion.

Stanley Consultants INC.

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Computed by:	Y.Ding	Date:	12/05/11		Labrinth (3) Spillway Stability - 70ft
Checked by:	E.Daly	Date:	12/16/11		Stability LC_3
Approved by:		Date:		Sheet No.	of

1. Weight Computations

Item	D1	D2	D3	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Slab concrete	161.0	1	181	150	4371.2	429.7	156505
Wall concrete	773.4	1	10	150	1160.0	297.5	28763
Soil under slab	259.0	1	181	115	5391.1	414.0	185992
Water (used below)	7796.9	45.8					
Total					**10922.3**		**371261**

Top of Slab =	886.3 ft		0 psi
Total Length of spillway =	181.0 ft	Ice Load =	**0.0** klf
Bottom of key EL =	880.3 ft	L1 =	70.0 ft
Foundation Friction Angle =	28 degrees	L2 =	0.0 ft
Foundation Bonding=	0 psf	step =	0.0 ft
Length of Seepage Path =	70 ft	Allowable Bearing =	2000 psf
Top Of Crest EL =	896.3 ft	Foundation Width =	70.0 ft

2. Case III: PMF/2 Flood Condition

Head Water EL = 906.0 ft

Tail Water EL = 888.7 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	4865.3	45.8	222627
Water weight 2	4719.3	45.8	215948
Total	**9584.6**		**438575**

Uplift 50%

Upstream	Head 1 = 17.1	ft
Downstream	Head 2 = 8.4	ft

Stanley Consultants INC.

Computed by: Y.Ding	Date: 12/05/11	Job No. 23601	Page No.
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		Stability LC_3	
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	U (kip)	L (ft)	M (k-ft)
uplift (rectangular)	-6641.1	35	-232439
uplift (triangular)	-3419.4	47	-159571
Total	**-10060.5**		**-392010**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream rectangular	1752.9	8.0	-14023
Upstream triangular	1445.7	5.3	-7710
Downstream	0.0	0.0	0
Total	**3198.6**		**-21733**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	0.0	0.0	0
Upstream - Ice	0.0	25.7	0
Total	**0.0**		**0**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	10922.3		371261
Weight of Water	9584.6		438575
Driving of Water		3198.6	-21733
Uplift at Efficiency = 0.5	-10060.5		-392010
Silt & Ice		0.0	0
Total	**10446.4**	**3198.6**	**396092.6**

ΣV = 10446.4 kips
ΣH = 3198.6 kips
ΣM = 396092.6 k-ft

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USACE Stability

Force Resultant Location @ Base L =	37.92 ft		OK	Crack
Force Resultant Location Offset e =	2.92 ft			0.00 ft
Foundation Bearing p_{max} =	1031 psf	2000	OK	
Foundation Bearing p_{min} =	618 psf			For Extreme LC,
Sliding FOS =	1.74	1.30 Reqr'd	OK	

FERC Stability

Force Resultant Location @ Base L =	37.92 ft		
Force Resultant Location Offset e =	2.92 ft		
Foundation Bearing p_{max} =	1031 psf	2000	OK
Foundation Bearing p_{min} =	618 psf		
Sliding FOS (No Cohesion) =	1.74	1.50 Reqr'd	OK

For Extremel LC, No Cohesion.

New Pneumatic Spillway

Service Spillway

Stanley Consultants INC.

				Job No.	23601 Page No.
				Subject:	Delhi Lake Dam Reconstruction
Computed by:	Y.Ding	Date:	12/05/11		Obermeyer Spillway Stability - 50ft
Checked by:	E.Daly	Date:	12/16/11		Stability LC_1
Approved by:		Date:		Sheet No.	of

1. Weight Computations

Item	D1	D2	D3	γ (pcf)	Wt (kip)	L (in)	M (k-ft)
Slab concrete	166.0	1	160	150	3984.0	304.8	101207
Gate	1.0	1	160	350	56.0	156.0	728
Soil under slab	134.0	1	160	115	2465.6	294.0	60407
Water (used below)	292.6	31.7					
Slab width (used below)		50					
Total					**6505.6**		**162342**

Top of Slab =	888.3 ft		0 psi
Total Length of spillway =	160.0 ft	Ice Load =	5.0 klf
Bottom of key EL =	882.3 ft	L1 =	50.0 ft
Foundation Friction Angle =	28 degrees	L2 =	0.0 ft
Foundation Bonding=	0 psf	step =	0.0 ft
Length of Seepage Path =	50 ft	Allowable Bearing =	2000 psf
Top Of Crest EL =	896.3 ft	Foundation Width =	50.0 ft

2. Case I: Normal Operating Condition

Head Water EL = 896.3 ft

Tail Water EL = 857.0 ft

Weight of Water	Wt (kips)	L (ft)	M (k-ft)
Water weight 1	2921.5	31.7	92514
Total	**2921.5**		**92514**

Uplift			50%
Upstream	Head 1 = 7.0	ft	
Downstream	Head 2 = 0	ft	

Stanley Consultants INC.

Computed by:	Y.Ding	Date:	12/05/11	Job No.	23601 Page No.
Checked by:	E.Daly	Date:	12/16/11	Subject:	Delhi Lake Dam Reconstruction
Approved by:		Date:			Obermeyer Spillway Stability - 50ft
					Stability LC_1
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	U (kip)	L (ft)	M (k-ft)
uplift (rectangular)	0.0	25	0
uplift (triangular)	-1747.2	33	-58240
Total	**-1747.2**		**-58240**

Driving of Water	H (kips)	L (ft)	M (k-ft)
Upstream rectangular	0.0	7.0	0
Upstream triangular	978.4	4.7	-4566
Downstream	0.0	0.0	0
Total	**978.4**		**-4566**

Silt & Ice	H (kips)	L (ft)	M (k-ft)
Upstream - Silt	0.0	0.0	0
Upstream - Ice	800.0	14.0	-11200
Total	**800.0**		**-11200**

Load Summary

	V (kips)	H (kips)	M (k-ft)
Structure Weight	6505.6		162342
Weight of Water	2921.5		92514
Driving of Water		978.4	-4566
Uplift at Efficiency = 0.5	-1747.2		-58240
Silt & Ice		800.0	-11200
Total	**7679.9**	**1778.4**	**180850.2**

ΣV = 7679.9 kips

ΣH = 1778.4 kips

ΣM = 180850.2 k-ft

Stanley Consultants INC.			Job No.	23601	Page No.
			Subject:	Delhi Lake Dam Reconstruction	
Computed by:	Y.Ding	Date: 12/05/11		Obermeyer Spillway Stability - 50ft	
Checked by:	E.Daly	Date: 12/16/11		Stability LC_1	
Approved by:		Date:	Sheet No.	of	

USACE Stability

Force Resultant Location @ Base L =	23.55 ft		**OK**	**Crack**
Force Resultant Location Offset e =	1.45 ft			**0.00 ft**
Foundation Bearing p_{max} =	1127 psf	2000	**OK**	
Foundation Bearing p_{min} =	793 psf			For Usual LC,
Sliding FOS =	2.30	2.00 Reqr'd	**OK**	

FERC Stability

Force Resultant Location @ Base L =	23.55 ft		
Force Resultant Location Offset e =	1.45 ft		
Foundation Bearing p_{max} =	1127 psf	2000	**OK**
Foundation Bearing p_{min} =	793 psf		
Sliding FOS (No Cohesion) =	2.30	1.50 Reqr'd	**OK**

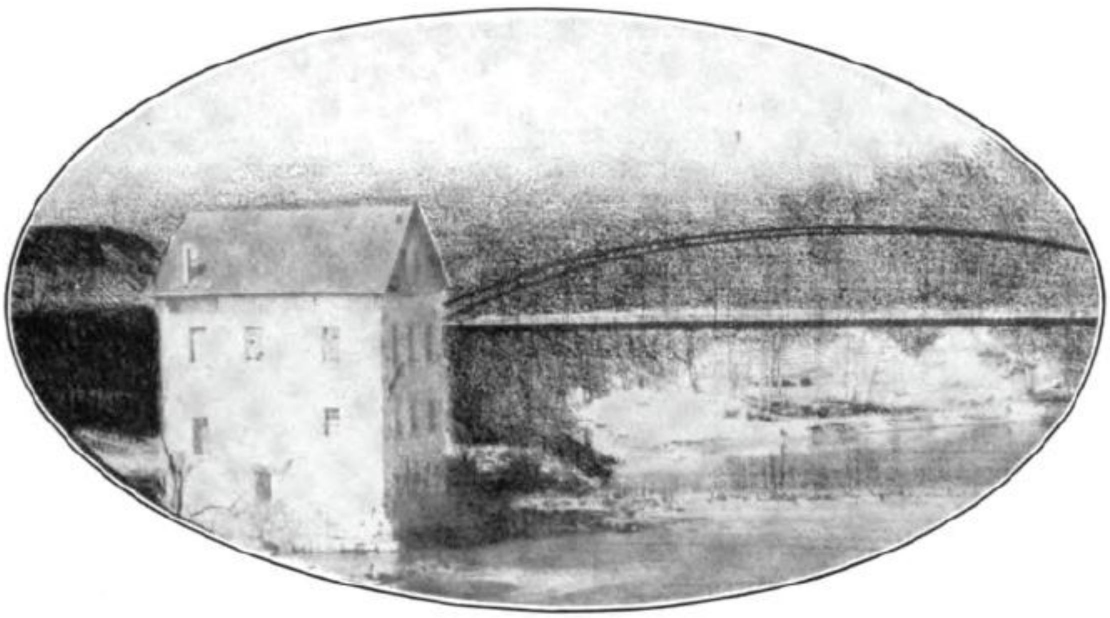
For Usual LC, No Cohesion.

Appendix E

Archaeological Reconnaissance Report

ARCHAEOLOGICAL RECONNAISSANCE SURVEY OF LAKE DELHI

DELAWARE COUNTY, IOWA



Prepared for:

Lake Delhi Combined Recreation Facility
and Water Quality District

Prepared by:

THE Louis Berger Group, INC.

November 2011

ARCHAEOLOGICAL RECONNAISSANCE SURVEY OF LAKE DELHI

DELAWARE COUNTY, IOWA

Prepared for:

Lake Delhi Combined Recreation Facility and Water Quality District

Prepared by:

Randall M. Withrow, RPA
Principal Investigator

The Louis Berger Group, Inc.
950 50th Street
Marion, Iowa 52302

November 14, 2011

ABSTRACT

In September 2011, The Louis Berger Group, Inc. (LBG) completed an archaeological reconnaissance survey for the Lake Delhi portion of the Maquoketa River valley in Delaware County, Iowa. This study was prepared on behalf of the Lake Delhi Combined Recreation Facility and Water Quality District (District) as part of the District's ongoing effort to rebuild the Lake Delhi Dam and restore the lake to its previous level. The Lake Delhi Dam was breached during a flood event that occurred on July 23-24, 2010.

This report presents the findings of an archaeological study that included a records review to identify potential resources within the former impoundment area followed by a field reconnaissance survey to investigate areas considered to have high potential for unreported archaeological sites. The study area included the Lake Delhi dam and all exposed land areas within the former impoundment area located at or below the former lake elevation level of 897 feet above mean sea level. The study area encompasses an estimated 448 acres.

No archaeological sites had been reported within the project area prior to the July 2010 dam failure. Four sites were recorded within the previous impoundment area during the fall of 2010 by Wapsi Valley Archaeology, Inc. during archaeological monitoring for the installation of emergency erosion control structures at the Delhi dam location and upstream at Hartwick bridge. The four sites included two historic building foundations and two historic artifact scatters associated with the 19th century townsite of Hartwick. Prehistoric artifacts with evidence for Early to Middle Archaic, Late Archaic, Middle Woodland and late prehistoric components were also collected from the four sites.

The present study includes a comprehensive records review, a condition assessment of the study area's Quaternary and Holocene valley landforms, and results of a reconnaissance level survey of those landforms. LBG identified seven additional sites within the study area and redefined one of the sites first identified by WVA to segment one of the historic building foundations at Hartwick as a separate site. As a result, there are a total of 12 archaeological site reported for the study area. These include ten sites with evidence for prehistoric Native American occupations ranging from 8000 to 300 years before present (BP). Most of these sites (7 of 10) appear to be open habitation areas or settlements (13DW123, 13DW124, 13DW133, 13DW134, 13DW137, 13DW138, 13DW139) while one is a smaller habitation site situated within a natural rock shelter (13DW141). Other prehistoric sites include an apparent fish weir structure (13DW140) and a lithic resource procurement site (13DW126). Mid-19th century building foundations are represented at two separate locations near the former townsite of Hartwick (13DW125, 13DW136) and are believed to be associated with the historic settlement that once existed at that location. One of these is believed to be the Hartwick saw mill (13DW136) which was the first building erected in Hartwick (by John Clark in 1849). Fragments of contemporary historic artifacts were identified at two sites that also produced prehistoric artifacts (13DW123, 13DW126).

No burial sites were identified within the study area, but potential for unreported human burials is considered possible at the eight prehistoric habitation sites. None of the 12 sites has been evaluated for National Register eligibility. Additional reconnaissance survey is recommended for selected portions of the study area based on the results presented in this report. Additional site investigations are also recommended at all 12 sites as necessary for the purpose of gathering information about the nature, extent, and condition of the archaeological deposits present pursuant to an evaluation of National Register eligibility.

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I. INTRODUCTION

A. PURPOSE OF THE INVESTIGATION

The Louis Berger Group, Inc. (LBG) has completed an archaeological reconnaissance survey for the Lake Delhi portion of the Maquoketa River valley in Delaware County, Iowa. This archaeological review was performed on behalf of the Lake Delhi Combined Recreation Facility and Water Quality District (District) as part of the District's ongoing effort to rebuild the Lake Delhi Dam.

The Lake Delhi Dam was breached during a flood event that occurred on July 23-24, 2010. A portion of the southern earthen embankment failed and was subsequently washed away by flood waters. The concrete spillway's gates were damaged and water also infiltrated and seeped through the dam's north embankment. The District is planning to rebuild and repair the dam and restore the Lake Delhi impoundment to its former elevation of 897 feet above mean sea level (amsl). Completion of the project is expected to require a permit from the United States Army Corps of Engineers. The District may also seek funding from other federal and state agencies.

The present study was conducted to help fulfill state and federal historic preservation compliance requirements including Iowa's state burial protection laws (Iowa Code Chapters 716.5, 523I, 263B; Iowa Administrative Code 685-11) and the requirements of Section 106 of the National Historic Preservation Act of 1966 (as amended) and its implementing regulations at 36 CFR § 800 which state that federal agencies and/or designated applicants must take into account the potential effects of federally funded or regulated undertakings on historic properties (i.e., buildings, structures, sites, districts, or objects) listed in or eligible for listing in the National Register of Historic Places (NRHP). The results presented in this report are intended to provide the project sponsors and other review agencies with information necessary to determine the proposed project's effect on significant cultural resources and potential mitigation alternatives.

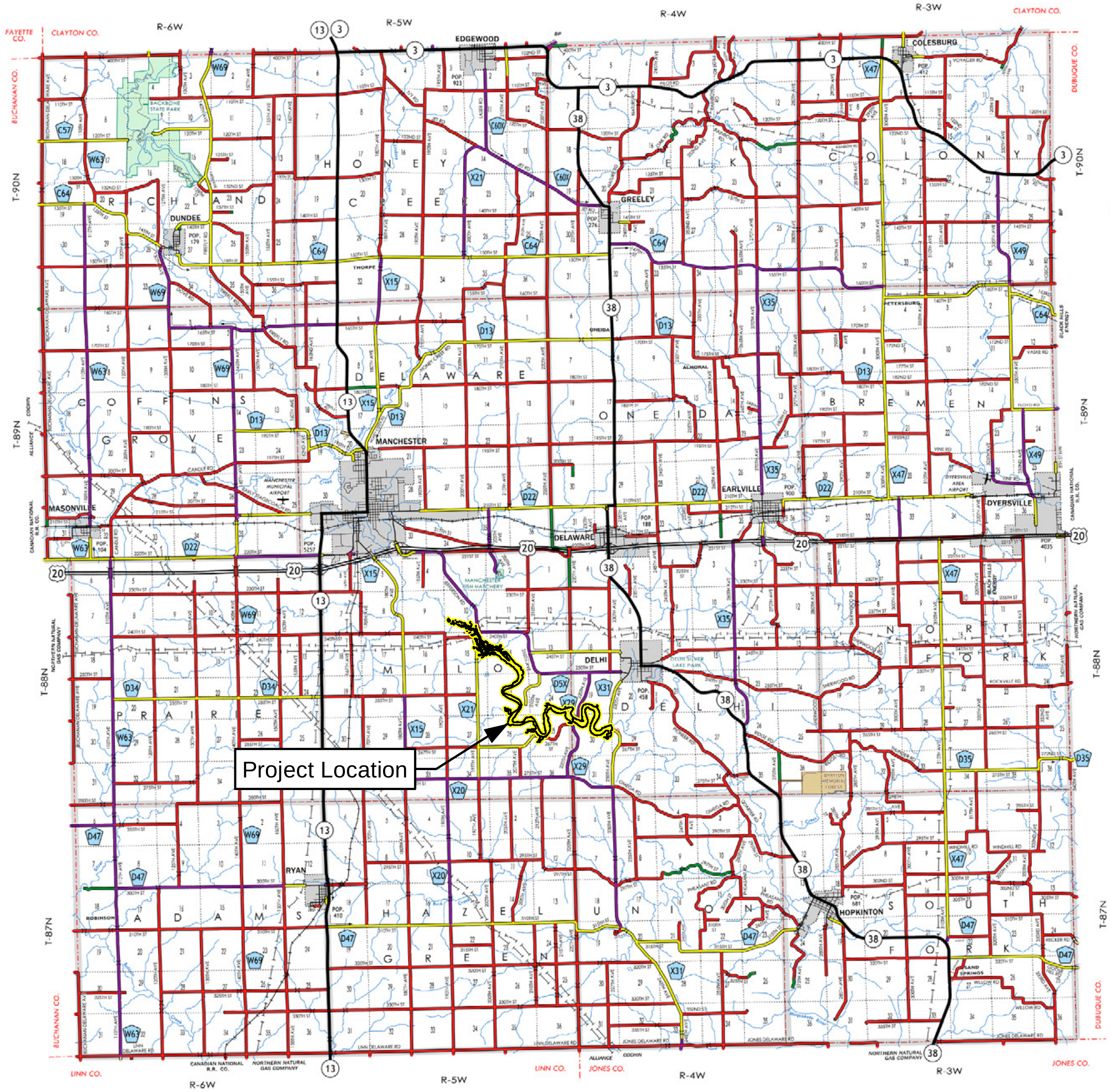
B. PROJECT LOCATION AND AREA OF POTENTIAL EFFECT

The project area is located in the central portion of Delaware County, Iowa approximately 1.5 miles southwest of Delhi, Iowa (Figure 1). The Lake Delhi Dam is located in the NE quarter of Section 30 in Delhi Township (T88N-R4W). The Lake Delhi impoundment, which is federally recognized as Hartwick Lake, includes portions of Sections 19 and 30 in Delhi Township (T88N-R4W) and portions of Sections 14, 23, 24, 25 and 26 of Milo Township (T88N-R5W). The impoundment area behind the dam occupies a nine-mile segment of the Maquoketa River valley and encompasses approximately 448 acres.

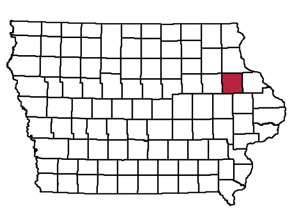
For purposes of this investigation, the study area or area of potential effect (APE) included the area occupied by the Lake Delhi Dam plus all exposed land areas upstream within the former impoundment area that are located at or below the former lake elevation level of 897 feet above mean sea level. A more detailed description of planned improvements is provided in Chapter II.

C. PROJECT AUTHORIZATION AND PERSONNEL

The field investigations and the information presented in this technical report are designed to meet the standards specified in the Secretary of the Interior's *Standards and Guidelines for Archaeology and Historic Preservation* (Federal Register 48:190:44716-44742), and the *Guidelines for Archaeological Investigations in Iowa* (Kaufmann 1999). Assistant Director Randall M. Withrow served as Project



LEGEND



0 1 2 3 4 8 Miles

0 2 4 6 12 Kilometers

N

The Louis Berger Group, Inc.

FIGURE 1: Project Location

SOURCE: IADOT 2010

Manager and Principal Investigator. Field investigations were directed and performed by Mr. Withrow with the assistance of Field Archaeologist Samuel Williams. Mr. Withrow is a Registered Professional Archaeologist (R.P.A.) and has qualifications that meet or exceed the Secretary of the Interior's Professional Qualification Standards (Federal Register 48:190:44738-9). The Phase I reconnaissance survey was conducted from September 28-30, 2011. This report was prepared by Mr. Withrow. The maps and other graphic illustrations included in this report were prepared by GIS Specialist Jackie Horsford. A completed National Archaeological Database Form for this report is provided in Appendix A.

II. PROJECT DESCRIPTION

A. LAKE DELHI DAM RESTORATION

The Lake Delhi Dam was breached on July 24, 2010 following several days of heavy rain within the catchment basin above the dam. According to a report filed by an independent review panel in December 2010 (Fieldler et al. 2010), the breach was caused by seepage in the south earth embankment combined with overtopping flow caused by the rising floodwaters behind the dam. A portion of the southern earthen embankment failed and was subsequently washed away by flood waters. The concrete spillway's gates were damaged and water also infiltrated and seeped through the dam's north embankment.

The review panel's final report includes a detailed description of the dam which includes a powerhouse for generating hydroelectric power. Their description is quoted at length here as it appeared in the report:

> *Delhi Dam, also known as Hartwick Dam, was designed as a concrete dam and earthen embankment. The 704-foot long structure consists of (from left to right looking downstream): a 60-foot long concrete reinforced earthfill section abutting the left limestone abutment; a 61-foot long conventional reinforced concrete powerhouse containing two S. Morgan Smith turbines with two Westinghouse generators (each rated at 750 kW); an 86-foot long gated concrete ogee spillway, with three 25-foot x 17-foot vertical lift gates; and, a 495-foot long embankment section that was originally constructed with 1V:3H upstream slopes and 1V:2H downstream slopes, that extends to the right abutment of the dam (in this report when right and left is used in reference to the dam, the convention is that this is while looking downstream; also the right abutment of the dam is the south abutment and the left abutment is the north abutment)*. *The crest of the south embankment section of the dam is 25 ft wide and the dam crest is at elevation 904.8 ft NGVD29. A general plan of the site is shown on Figure II-2.*
>
> *The maximum section of the concrete portion of the dam has a height of about 59 ft and the embankment section has an estimated maximum height of 43 ft. Lake Delhi, the reservoir behind Delhi Dam has an area of approximately 440 acres and a storage volume of 3790 acre-ft at normal reservoir (elevation 896 ft) and a reservoir volume of about 9920 acre-ft at the crest of the dam (elevation 904.8 ft) [Allen, 2009]. The spillway crest is at elevation 879.8 and the hollow inside of the spillway crest structure is filled with rock.*
>
> *The concrete reinforced earthfill section of the dam at the left abutment was originally constructed with two parallel concrete retaining walls, founded on rock and spaced 20 ft apart. Rock fill was placed between the walls. In 1967, a concrete crib wall and additional fill was placed upstream of the original walls. The area downstream of this section serves as a parking and staging area for performing maintenance in the powerhouse [FERC 2002 Prelicense Inspection] (*Fieldler et al. 2010:6-7)

The District is planning to rebuild and repair the dam and restore the Lake Delhi impoundment to its former elevation of 896 to 897 feet above mean sea level (amsl). Engineering studies are currently underway to consider alternatives for reconstructing the dam's embankment, repairing the gated concrete spillway portion of the dam, and bringing the embankment, spillway and adjoining powerhouse structures into compliance with current dam safety standards.

Plate 1. Lake Delhi Dam Spillway and Powerhouse. Photo by LDRA reproduced from Fiedler et al. 2010.

Plate 2. Aerial View of Breached Embankment. Photo by Iowa Wing Civil Air Patrol.

B. CURRENT LAND USE

Lake Delhi has approximately 18 miles of shoreline. Land use along the shoreline is primarily private and residential although some commercial properties such as the Hartwick Marina are also sited near the shoreline. Public access is provided near the center of the lake at the Turtle Creek Recreation Area administered by the Delaware County Conservation Board and several parcels owned by the Lake Delhi Recreation Association (LDRA) have been improved to accommodate recreational use by non-residents (e.g., Lost Beach). Most of the privately owned lakeshore properties are occupied by permanent residences and seasonal homes and most lakeshore areas suitable for home construction have been landscaped to afford access to the lake. Properties with gentle slopes to the lakeshore typically support lawns with little or no modification of the shoreline, while properties with more steeply sloped shorelines are typically reinforced with rip rap materials or engineered retaining walls designed to both retain the slope and provide safe access to the lake.

With the release of Lake Delhi, there are now vast areas of exposed land that have remained submerged for more than 80 years. At the time of our survey in late September 2011, most of these areas had lain exposed for a period of about 14 months. Point bars formed on inside meanders between Maples and Hickory Hollow tended to be covered with thick deposits of coarse sand and thickly matted patches of weedy vegetation. Floodplain surfaces located upstream from Maples and downstream from Hickory Hollow seemed to support an extensive and more or less continuous mat of tall weedy vegetation.

Several landowners whose parcels extend onto the former lake bottom have proceeded to adapt newly exposed land areas for other forms of light recreational use. We observed a number of areas that were being used for picnicking or evening campfires and one area that had been graded as a track for dirt bikes or other all-terrain recreational vehicles. For the most part however the exposed lake bottom appears to receive little use, recreational or otherwise. The river itself continues to be used for fishing and boating although the class of pleasure craft suitable for use on the river is now limited to canoes or kayaks.

III. RESEARCH DESIGN

A. RESEARCH OBJECTIVES

Prior to initiating this study, the author contacted staff archaeologists at the Iowa State Historic Preservation Office (SHPO – Dan Higginbottom) and the U.S. Army Corps of Engineers (USACE – Brant Vollman) to determine if either agency had specific concerns about the area's potential for unreported archaeological resources and to seek their advice regarding appropriate investigation methods.

The Iowa SHPO archaeologist noted that their office had already provided some guidance to the Governor's Task Force on Rebuilding Lake Delhi in a letter to the Iowa Department of Natural Resources dated November 2, 2010 (Appendix B). This letter cites the project's potential to require federal funding, permits or licensing and advises that participants initiate compliance with Section 106 of the National Historic Preservation Act (NHPA) at an early stage in the planning process. The letter cites the Maquoketa River valley's potential to include a variety of both prehistoric and historic archaeological sites including human burials and remains of a 19th century townsite at Hartwick. The SHPO also called attention to the presence of non-archaeological resources that are likely to be affected by the planned reconstruction, most notably the dam itself, which is part of the Lake Delhi Dam and Powerhouse Historic District. The latter has already been evaluated as eligible for listing in the NRHP. In consideration of the large area that could be affected by the project and the fact that no archaeological surveys had ever been completed within the Lake Delhi impoundment, Mr. Higginbottom advised a reconnaissance-level investigation designed to collect baseline information throughout the area of potential effect.

The USACE archaeologist also advised beginning with a reconnaissance-level investigation. He indicated that a formal area of potential effect had not yet been defined with regard to reconstruction of the Lake Delhi dam, but that initial records review and field reconnaissance should minimally include the area directly affected by dam reconstruction as well as the proposed impoundment area. The USACE archaeologist also cited the area's high potential to include unreported prehistoric and historic archaeological resources.

The present study was therefore designed to gather and compile baseline information regarding the project area's potential to include unreported archaeological resources that may be subject to further investigation or review in compliance with state and/or federal historic preservation laws. The most common among these are the following:

- <u>National Historic Preservation Act of 1966</u>, as amended (16 USC 470-470t, 110). Section 106 of the NHPA requires federal agencies to take into account the effects of federal undertakings on historic properties listed on or eligible for listing on the NRHP. If the current project will require federal funding, permitting, licensing or approval, then it may be subject to review under this portion of the NHPA.
- <u>Iowa Statutes Protecting Human Remains and Burial Sites</u>. All burial sites in Iowa are protected by state law (Iowa Code Chapter 716.5). Those less than 150 years old are governed by the Iowa Cemetery Act (Iowa Code 523I) while those greater than 150 years old are protected by the Office of the State Archaeologist (Iowa Code 263B).

The primary objectives of this study were defined as follows: (1) determine whether any burial sites or other resources listed on or considered eligible for listing on the NRHP are known to exist within the current project area; (2) determine which portions of the project area have already been surveyed for archaeological sites; (3) conduct initial survey work to locate previously unreported archaeological resources identifiable on the ground surface within the project area; and (4) determine what if any additional archaeological resource inventory or evaluation work may be needed within the project boundary prior to project construction in order to establish the project sponsor's compliance with applicable state and/or federal statutes.

In accordance with state *Guidelines for Archaeological Investigations in Iowa* (Kaufmann 1999) our investigation included: (1) a review of current site location and cultural resource survey information on file at the Office of the State Archaeologist, University of Iowa (OSA), and the Iowa State Historic Preservation Office (SHPO) to identify the location of *known* archaeological resources; (2) a review of historic and archival documents (local histories, historic maps and plats, aerial photographs, etc.) and environmental information (topography, surface geology, soils, vegetation, hydrology, etc.) to assess the project area's potential for *unreported* archaeological sites, historic cemeteries, prehistoric burials, or other historic resources; and (3) a field reconnaissance survey designed to investigate areas with high potential for known or unreported sites with the purpose of confirming their presence or absence and/or evaluating the need for additional field investigations.

B. RESEARCH METHODS

The information used to address the research objectives for a Phase I reconnaissance survey is drawn from two sources: written records and on-site field observations. The research methods used in each case are outlined below.

1. Literature Search

Prior to beginning the Phase I field survey, the author reviewed site records on file at the Office of the State Archaeologist at the University of Iowa using the OSA's on-line GIS database: I-Sites (OSA 2011) at http://www.uiowa.edu/~osa/gisatosa/ and the Iowa Department of Transportation GIS project portal (IDOT 2011) at http://www.environmental.iowadot.gov. Available information includes locational data and site forms for previously identified archaeological sites and surveys in and near the project area. Also prior to present fieldwork, pertinent archaeological, environmental, and historical reference materials were consulted, including: regional archaeological overviews, state and county histories, soil surveys, historic plats and aerial photographs on file at the University of Iowa library and the State Historical Society of Iowa in Iowa City, and documents on file at the Louis Berger Group, Inc., Marion, Iowa.

2. Field Investigations

The field investigation consisted of visual inspection of exposed ground surfaces throughout the study area. LBG had access to all private land parcels, but only those portions at or below the former lake elevation. The former lakebed was usually accessed from adjacent public roads or public boat access locations. In some instances, surveyors accessed isolated lakebed areas from private property with landowner consent. The field investigation was limited to pedestrian survey or visual inspection of the ground surface and shoreline. No subsurface testing was performed. When a site was identified, the survey team used a GPS unit equipped with a sub-meter correction beacon to document its location. Surface finds were not collected and no features were tested or excavated, but photographs were take to document representative site materials and site conditions. Representative photographs were also taken to document survey conditions throughout the survey area.

IV. LITERATURE SEARCH

A. SOURCES CONSULTED

Prior to beginning the Phase I field investigation LBG conducted background research for the project by accessing site files on-line via I-Sites (OSA 2011), a statewide database of archaeological site information maintained by the Office of the State Archaeologist, Iowa City, and the Iowa Department of Transportation's GIS Project Portal (IADOT 2011) which is a spatial database that depicts the location of recorded archaeological sites as well as the locations of archaeological investigations that have taken place within the state. A variety of historic documents, maps, and aerial photographs were also researched using a combination of on-line and library sources. Digital copies of various historic plats, atlases, and other maps were accessed and reviewed on line at digital map library websites maintained by the University of Iowa (UIA 2011) and the University of Texas (UTX 2011). Historic maps reviewed for this project included documents prepared in 1838 (GLO 1837-1838), 1875 (Andreas 1875); 1894 (Davis 1894); 1896 (USGS 1896); 1904 (Huebinger 1904); 1930 (Hixson 1930); 1936 (Lovell 1936), and 1973 (USGS 1973a, 1973b). Other historical and environmental information was obtained from the soil survey for Delaware County (Wisner 1986), geological references (Anderson 1998, Prior 1991, Witzke 1995), local and regional histories (Andreas 1875, Merry 1914, Sage 1974; Western Historical Society 1878) and regional archaeological overviews (Alex 2000).

B. NATIONAL REGISTER LISTED PROPERTIES

As of November 1, 2011, the National Park Service database (http://nrhp.focus.nps.gov) identifies 15 properties in Delaware County that are listed on the National Register of Historic Places. These include the Backbone State Park Historic District (91001842) and the Delaware County Courthouse (81000234) in Manchester. None of these listed properties are located within or near the Lake Delhi project area. The nearest listed property is the Bay Settlement Church and Monument (77000506) which is located approximately one mile south of Lake Delhi near the Turtle Creek Recreation Area.

In 2009, Alexa McDowell, AKAY Consulting completed an intensive historic and architectural survey and evaluation of the Lake Delhi Dam and Powerhouse as part of an application by the facility's owner, the LDRA, to the Federal Emergency Management Agency (FEMA) to repair elements of the facility damaged by flooding in 2008. McDowell recommended that the Lake Delhi Dam and Powerhouse be considered eligible for listing on the NRHP under Criterion A (important events) for its association with "the history of the hydroelectric industry in the state of Iowa and in its association with the development of electric service in the state of Iowa (McDowell 2009:1)." She also recommended that the dam/powerhouse facility be considered part of a historic district along with the bridge and roadway (County Road X31) that traversed the dam and the dam operator's house located south of the dam. The present National Register status of the property is obviously in question pending an assessment of its current condition and the potential for further loss of its historic integrity as needed repairs are made.

C. PREVIOUS ARCHAEOLOGICAL INVESTIGATIONS

On September 9, 2011, LBG accessed the Iowa Department of Transportation's GIS Project Portal website (IADOT 2011) to request GIS shapefile information regarding the location of previous archaeological surveys completed within or near the study area. This website, hosted by the Iowa DOT's Office of Location and Environment, is a GIS database with geo-referenced environmental, historical, and cultural information for the state of Iowa. Approved users may also access confidential information regarding the location of recorded archaeological sites, archaeological surveys, historic building locations, and historic map information.

LBG's review of this database indicated that only five professional archaeological surveys had been conducted in the project vicinity (Table 1). Subsequent review of survey reports completed for the project area revealed that the results of two more recent archaeological studies had not yet been included in the Iowa DOT's database. All seven studies were completed within the past 25 years and all but one was performed for purposes of federal agency compliance with Section 106 regulations. The most recent archaeological work was performed on behalf of the Iowa Department of Natural Resources in response to an emergency proclamation by the Governor of Iowa to address erosion and sedimentation concerns following the July 2010 breach event. Three of these studies have included survey work within the limits of the current study area (Figure 2).

In 1990, Bear Creek Archaeology, Inc. completed a Phase I archaeological survey for proposed boat ramp and public access area immediately below the Delhi Dam (Roberts and Stanley 1990). The proposed project included new boat ramp, access road and parking area and encompassed a four-acre parcel situated on the left (north) bank of the Maquoketa River. No sites were reported within the survey area as a result of the investigation.

In 2000, Cultural Resources Management Services completed a Phase I archaeological survey for replacement of the Maquoketa River bridge along 220th Avenue at Hartwick for the Delaware County Secondary Roads Department (Marcucci 2000). The survey examined a 1.8-acre area that included the existing north approach to the Hartwick bridge and a portion of the upland slope at the south side of the valley. No archaeological sites were reported as a result of the survey.

In 2007, the Office of the State Archaeologist completed a Phase I archaeological survey for a proposed communications tower near the 220th Avenue/225th Avenue intersection located approximately one-quarter mile south of Lake Delhi (Peterson 2007). The survey examined approximately three acres on the upland summit to accommodate the tower platform, guy anchors and access road. Two previously unreported sites were discovered within the survey area (13DW99, 13DW100). Both sites were recorded as prehistoric artifact scatters or lithic workshops based on the presence of manufacturing debris associated with chipped-stone tool production. Both sites were recommended as potentially eligible for listing in the National Register and both were avoided by project construction.

In 2008 and 2009, Wapsi Valley Archaeology, Inc. (WVA) completed two Phase I archaeological surveys for expansion of the City of Delhi's wastewater treatment facility situated on the uplands northeast of Lake Delhi. A survey completed in 2008 examined a five-acre parcel adjacent to the existing sewage disposal ponds (Morrow 2008) and recorded two small prehistoric lithic scatters (13DW104, 13DW105). A supplemental Phase I survey in 2009 examined an additional 13 acres north of the facility and recorded one additional prehistoric habitation site (13DW106; Finn 2009). All three sites were evaluated not eligible for listing in the NRHP.

In 2010, WVA completed two archaeological investigations within and around Lake Delhi. One of these was completed prior to the breach event in July 2010 while the second was completed in response to the event. In February 2010, Michael Finn completed a cultural resources assessment for the entire Lake Delhi area as part of a broader study that was being conducted by the LDRA to investigate the feasibility of restoring the Lake Delhi Dam and Powerhouse as a functioning hydroelectric facility. The WVA study included a review of existing site records, historical information, and environmental data along with limited field reconnaissance to determine the proposed project's potential effects on known or unreported archaeological deposits. The study area included the entire Lake Delhi impoundment but field inspections were necessarily limited to exposed shoreline with potential to be affected by minor fluctuations in the

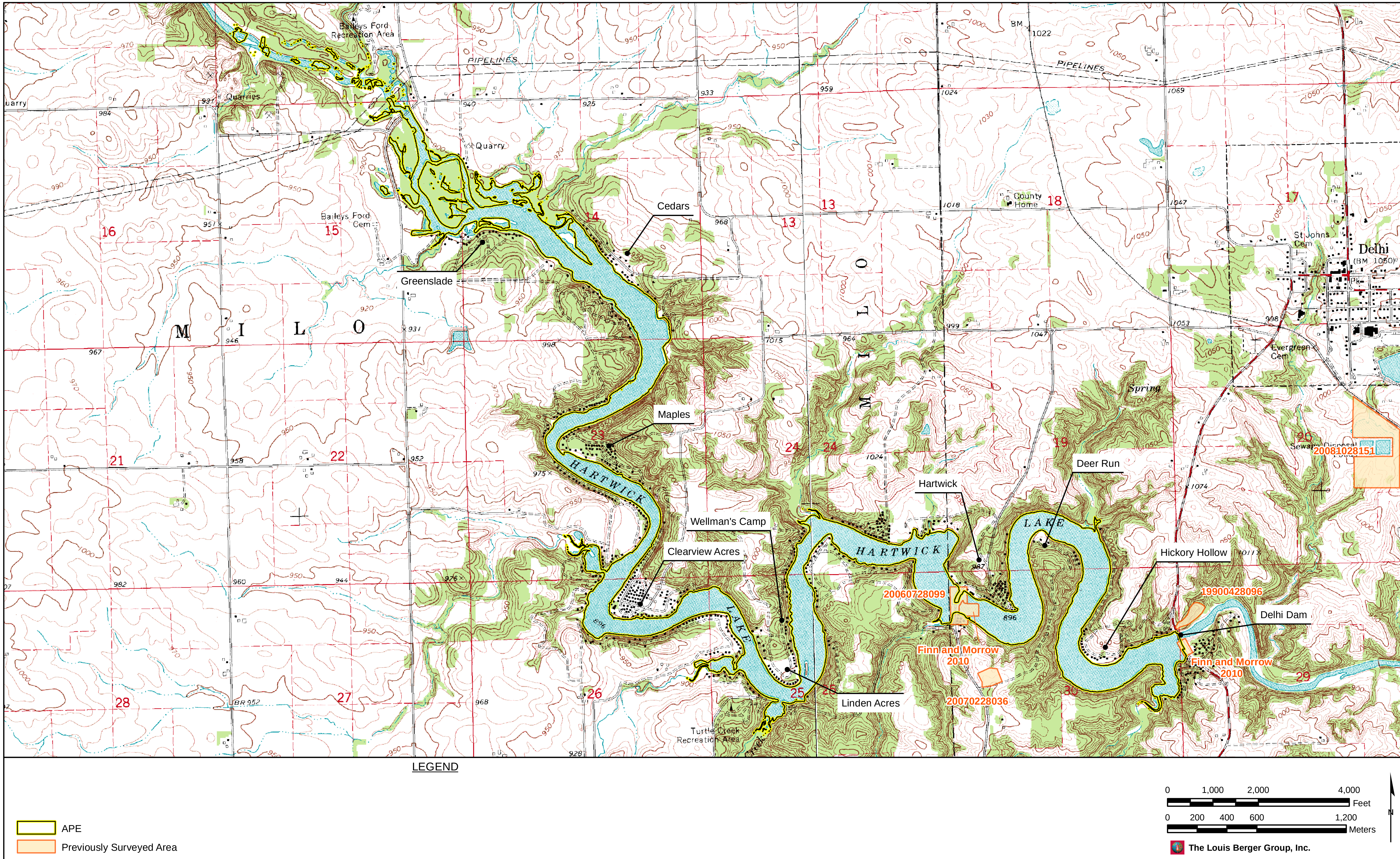


FIGURE 2: Study Area and Location of Previous Surveys

SOURCE: USGS 1973a, 1973b

Table 1. Previous Archaeological Investigations

SHPO R&C Number	Year	Township & Section	Type of Investigation	Result of Investigation	Reference	Survey Coverage Within Current Project Area
19900428096	1990	T88N-R4W Section 29	Phase I Survey for Boat Ramp Below the Delhi Dam (4 acres)	No sites reported	Roberts & Stanley 1990	-
20060728099	2000	T88N-R4W Section 30	Phase I Survey for Replacement of Hartwick Bridge (2 acres)	No sites reported	Marcucci 2000	< 1 acre
20070228036	2007	T88N-R4W Section 30	Phase I Survey for US Cellular Communications Tower (3 acres)	13DW99, 13DW100	Peterson 2007	-
20081028151	2008	T88N-R4W Section 20	Phase I Survey for Delhi Wastewater Treatment Facility (5 acres)	13DW104, 13DW105	Morrow 2008	-
20081028151	2009	T88N-R4W Section 20	Phase I Survey for Delhi Wastewater Treatment Facility (13 acres)	13DW106	Finn 2009	-
20080928021	2010	T88N-R5W Secs 14, 15, 23, 24, 25, 26 T88N-R4W Secs 19, 29, 30	Phase IA Assessment for Lake Delhi Dam Hydroelectric Facility Restoration (1100 acres)	No sites reported	Finn 2010	limited shoreline reconnaissance only
	2010	T88N-R5W Section 25 T88N-R4W Sections 29, 30	Archaeological Monitoring for Installation of Emergency Headcut Structures on the Maquoketa River (12 acres)	13DW123, 13DW124 13DW125, 13DW126	Finn & Morrow 2010	12 acres

level of the reservoir. No new sites were identified as a result of the study; however, Finn's (2010:19-21) review of historic maps and other local references called attention to the potential for the archaeological remains of several 19th century historic sites within the impoundment area that had likely been submerged by creation of the lake in 1927. These included part of the historic village of Hartwick, for which the lake itself is named. The townsite was settled during the 1850s and was formerly home to a sawmill/gristmill/dam complex, general store, tavern, blacksmith shop, brickyard, a cobbler/shoe shop, a paint shop and a wagon shop. Finn also noted that a second sawmill/gristmill/dam complex was once located in the immediate vicinity of the present day dam and powerhouse facility based on information recorded on historic maps and published county histories. The WVA report concluded that stream terraces carried potential for prehistoric sites but that most of these shoreline areas were obscured by riprap and retaining walls that prevented easy detection. Potential for undisturbed prehistoric site deposits within the impoundment area was judged to have been further diminished as a result of extensive dredging operations performed throughout the lake in 2004-2006.

In November 2010, WVA staff performed archaeological monitoring for the installation of erosion control structures at two locations within the newly exposed lake basin. The work was initiated in response to concerns about the rapid erosion of silt deposits from the former lakebed and the effects that accelerated sedimentation were having on the Maquoketa River basin farther downstream. The Iowa Department of Natural Resources (DNR) designed a series of rock riffle structures to be installed at two locations within the former lake to help slow the rate of stream erosion. The two areas selected by the DNR happened to correspond precisely with the two locations identified by Finn (2010) as having highest potential for submerged 19th century historic archaeological sites, i.e, the historic valley crossings at Delhi Dam and Hartwick. Due to the emergency nature of the situation, the DNR retained WVA to monitor the installation of the riffle barriers for potential impacts to unreported archaeological features that might be encountered during construction. No archaeological features or materials were identified within the work zone; however, WVA noticed exposed historic and prehistoric archaeological materials in several places on the nearby river terraces. Four new archaeological sites were reported. Three sites include both prehistoric and historic period features and artifacts (13DW123, 13DW124, 13DW126) while a fourth site marks the location of a former brick and limestone building (13DW125). Site 13DW123 includes a scatter of eroded prehistoric and historic artifacts that include diagnostic materials associated with Middle Woodland (2200-1600 years BP), late prehistoric (800-300 years BP), and mid-19th century occupations. Site 13DW124 includes a light scatter of prehistoric chipped-stone artifacts and what appeared to be a late 19th century limestone and brick building foundation. Site 13DW126 appears to include the remains of a prehistoric stone procurement and stone tool manufacturing site with evidence of use during the Archaic period (10,000-2,800 years BP) along with a comparatively light scatter of mid-19th century and modern domestic items.

D. KNOWN ARCHAEOLOGICAL SITES

The Iowa Site Inventory for Delaware County lists 30 known archaeological sites located within an approximate one mile radius of Lake Delhi (Table 2). Only four of these sites are located within the current study area. Ten sites were recorded by professional archaeologists; the other 20 sites were reported by a local artifact collector and avocational archaeologist who lives in the area.

All but one of the 30 known archaeological sites are associated with prehistoric Native American use of the region and the vast majority of these sites (25 of 29) are located on upland landforms outside the current study area. Most of these prehistoric sites (19 of 29) consist of small artifact scatters ranging in estimated size from 12 to 100 square meters (13DW55, 13DW56, 13DW57, 13DW58, 13DW61, 13DW62, 13DW63, 13DW64, 13DW65, 13DW67, 13DW68, 13DW69, 13DW71, 13DW72, 13DW73, 13DW74, 13DW100, 13DW104, 13DW105). The small size of these sites suggests very short-term and limited use, perhaps most likely associated with procurement of upland resources rather than living sites.

Table 2. Known Archaeological Sites Located Within or Near the Study Area

SITE NUMBER	LANDSCAPE POSITION	CULTURAL AFFILIATION	SITE TYPE	WORK EFFORT/ NRHP EVALUATION	REFERENCE
13DW34	High Terrace Maquoketa River	Prehistoric (Woodland)	Burial Mounds	Not Evaluated	Schermer 1986
13DW55	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1994
13DW56	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1994
13DW57	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1994
13DW58	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1994
13DW59	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1991
13DW60	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1991
13DW61	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW62	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW63	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW64	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW65	Upland Summit	Prehistoric (Late Prehistoric)	Artifact Scatter	Not Evaluated	Martens 1993
13DW66	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW67	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW68	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW69	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW70	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW71	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW72	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW73	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW74	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Martens 1993
13DW99	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Peterson 2007
13DW100	Upland Summit	Prehistoric	Artifact Scatter	Not Evaluated	Peterson 2007
13DW104	Upland Summit	Prehistoric	Artifact Scatter	Not Eligible	Morrow 2008
13DW105	Upland Summit	Prehistoric	Artifact Scatter	Not Eligible	Morrow 2008
13DW106	Upland Summit	Prehistoric	Artifact Scatter	Not Eligible	Finn 2009
13DW123	**High Terrace**	**Prehistoric (Middle Woodland & possible late prehistoric); Historic**	**Artifact Scatter**	**Not Evaluated**	**Finn & Morrow 2010**
13DW124	**Early Holocene Terrace**	**Prehistoric & Historic**	**Artifact Scatter**	**Not Evaluated**	**Finn & Morrow 2010**
13DW125	**Early Holocene Terrace**	**Historic**	**Building Foundation**	**Not Evaluated**	**Finn & Morrow 2010**
13DW126	**High Terrace**	**Prehistoric (Archaic); Historic**	**Lithic Procurement; Artifact Scatter**	**Not Evaluated**	**Finn & Morrow 2010**

Sites listed in **bold font** represent resources located within the study area.

Nine larger artifact scatters have also been recorded (13DW59, 13DW60, 13DW66, 13DW70, 13DW99, 13DW106, 13DW123, 13DW124, 13DW126). Precise dimensions are not provided for every site, but they appear to range in size from 150 to perhaps as much as 9600 square meters in area. While some of these are still comparatively small sites, they are large enough to represent short-term campsites or other more intensively used resource procurement areas. The type and number of artifacts observed or collected from these sites is not always specified on the inventory forms, but most of the sites appear to include small numbers of stone tools such as projectile points or other bifaces and end scrapers along with other items associated with the manufacture and/or maintenance of chipped-stone tools, i.e., cores, flaking debris, hammerstones, etc. It appears that temporally diagnostic artifacts such as projectile points have been recovered from several of the upland sites by private landowners, but no information regarding their number, type, style or other physical attributes is included in the records. Those with documented temporally or culturally diagnostic artifacts include: Site 13DW65, which included a small triangular-shaped projectile point that indicates a late prehistoric temporal association (1200-300 years BP); Site 13DW123, which has produced fragments of Middle Woodland (2200-1600 years BP) pottery and what may be a late prehistoric agricultural implement (bone hoe); and 13DW126, which has produced a Late Archaic stemmed projectile point (3600-3000 years BP) and other unfinished items (i.e., biface preforms) that suggests potential for much earlier use.

One prehistoric mound cemetery (Site 13DW34) has been identified near Lake Delhi. Very little information about this site is known, but it is reported to include mounds constructed in both conical and linear shapes. Burial mounds built in these forms are most commonly associated with the Woodland cultural tradition in east-central Iowa and were constructed between 2800 to 1200 years ago. Burial mounds were also constructed and used by later cultural traditions continuing to the time of European contact in the late 17th century.

Four sites (13DW123, 13DW124, 13DW125, 13DW126) mark the location of mid-19th century European-American habitations and/or artifact scatters. Each is associated with early settlement of the Hartwick area and each site is situated within the impoundment area. Building foundations are present at two locations (13DW124, 13DW125) and light to moderate density scatters of contemporary artifacts (e.g., fragmented dinnerware, glassware, bottle glass, window glass, brick, nails, etc.) are present at all four sites.

Only three of these sites (13DW104, 13DW105, 13DW106) have been evaluated with regard to their eligibility for listing in the National Register of Historic Places (NRHP) and each of them was determined to be not eligible. Sites 13DW99 and 13DW100 are officially considered "potentially eligible" based on preliminary field investigations, but neither site has been fully evaluated. All other 25 sites remain unevaluated.

E. POTENTIAL FOR UNREPORTED ARCHAEOLOGICAL SITES

Information regarding the potential for additional, but unreported archaeological sites can be drawn from three principal sources: (1) local residents already knowledgeable about artifact collection spots or the location of other historic places; (2) historic documents that record the location of early settlements that no longer exist; and (3) observed patterns of site occurrence, especially with regard to the landform types preferred by prehistoric inhabitants. Archaeological sites that represent former trading posts, cabins, farmsteads, mills, schoolhouses, churches, or other buildings constructed after this part of Iowa was opened to Euro-American settlement (1832-1837) can often be identified using historic maps or historic aerial photographs. Predicting the specific location of unreported Native American sites is more difficult, although larger sites can sometimes be identified from aerial photographs. A more common approach involves using information about the location of known prehistoric sites and their patterns of occurrence on the landscape to help predict the locations of similar unreported sites. Both approaches, using historic records research and predictive modeling based on the distribution of sensitive landforms, were used in this study.

Sources Consulted

Aerial photographs taken of the project corridor during the 1930s, 1950s, 1960s, 1980s, 1990s and 2000s were accessed on-line at the *Iowa Geographic Map Server* website developed by the Iowa State University Geographic Information Systems Support and Research Facility in cooperation with the USDA Natural Resources Conservation Service and the Massachusetts Institute of Technology (ISU 2010). These photographs provide a useful record of land development and land use history and reveal information about former building locations and previous land disturbance activities. Unfortunately, none of this aerial photography pre-dates construction of the Delhi Dam in 1927, but these photographs do document post-dam development of the study area including changes to the lakeshore and adjacent areas as a result of those developments and soil erosion.

A collection of historic maps spanning the period 1838 to 1936 was assembled from both on-line and library sources and reviewed for information about the location of potential historic period archaeological sites. Historic maps are available for portions of the project corridor for the following years: 1837 to 1838 (GLO 1837-1838); 1875 (Andreas 1875); 1894 (Davis 1894); 1904 (Huebinger 1904); 1930 (Hixson 1930); and 1936 (Lovell 1936). Other historical information was obtained from local and regional histories (Andreas 1875, Merry 1914, Sage 1974; Western Historical Society 1878) and regional archaeological overviews (Alex 2000).

Important environmental resources consulted for this portion of the study included the soil survey for Delaware County (Wisner 1986). No mapped soils information is available for newly exposed land areas within the study area, but this information provided a basis for interpreting adjacent landforms above the former lake level and was used to help interpret exposed soil and sediment packages observed during the field reconnaissance survey. Soils information was also obtained on-line via *Landmass* (*Landmass* 2007), a website maintained by the Iowa OSA in partnership with the United States Department of Agriculture. The latter provides information about more than 400 soil series mapped within the state of Iowa and includes information about each soil's landscape position, parent material, relative age, and other characteristics that can be used to help assess an area's geological potential to contain prehistoric archaeological resources and determine whether those deposits are likely to be buried by more recent sediment. The project sponsor, through its primary design consultant, also provided post-breach LiDAR coverage for the exposed valley which was used by LBG to generate elevation contours for valley landforms which was used to help interpret the relative ages of alluvial landforms within the valley and correlate them with terrace complexes located outside the study area for which comparative soils information was available.

Environmental Background

The study area is located within a physiographic region known as the Iowan Surface (Prior 1991:68-75). This part of Iowa is believed to have been last glaciated in Pre-Illinoian times or more than 500,000 years ago. Today it is characterized by a relatively open landscape with gently rolling hills, long slopes and low topographic relief and owes its present configuration to several hundred thousand years of intense surface erosion aided by frost action, downslope movement of saturated soil and sediment, and intense glacial winds. These forces had a general leveling effect on the regional landscape which has since been dissected by several prominent northwest to southeast trending drainage systems including the Maquoketa River and its tributaries.

The Maquoketa River valley has a broad low profile in much of Delaware County where it flows through these ancient glacial deposits, but in some places like Backbone State Park north of Manchester and south of Delhi where it encounters bedrock, the river valley has become deeply entrenched with a narrow valley lined by exposed bedrock. Silurian-age bedrock is exposed at the valley floor in several locations, most notably at the point now occupied by the Delhi Dam and also upstream at Hartwick. Local bedrock consists of the mineral dolomite which is a chemically modified form of limestone that is rich in magnesium carbonate. The local bedrock is part of the Hopkinton Formation, named for exposures along the Maquoketa River less than 10 miles downstream from Delhi. The upper portion of the Hopkinton Formation is exposed at the dam and at numerous locations throughout the study area. This includes the fossil rich Farmers Creek Member which is recognized for its cave formations and the overlying Picture Rock Member which is relatively more resistant to erosion and is recognized for its tendency to form overhanging cliffs or ledges (Anderson 1998:119-121; Witzke 1995, 2001).

The sediments found within the Maquoketa River valley itself are associated with geological events that span more than 20,000 years. The oldest deposits are believed to have accumulated as the result of upland erosion during the most recent glacial maximum that occurred between 21,000 and 16,000 years ago. Geologists cite the periglacial conditions of that time including sparse tundra vegetation, high winds and permafrost soil that prevented precipitation and meltwater from penetrating the earth as important factors that contributed to greater runoff and erosion during this period (Bettis 1995:21). A result was the accumulation along valley walls of large quantities of frost-broken rock, silt and sand derived from glacial till and bedrock sources. Large amounts of silt and sand were also washed into valleys from the adjacent uplands. In some places just above the Lake Delhi dam these glacial deposits (i.e., Hickory Hollow) filled the valley to a depth of about 40 feet (the current river channel is close to bedrock in this portion of the valley). The high glacial terrace has an elevation of approximately 920 to 910 feet amsl within the study area (higher elevations are located upstream and lower elevations are located near the dam). Typical soil profiles are characterized by well-developed A, E, and Bt horizons (e.g., Lilah, Tell soil series).

Subsequent downcutting of these full-glacial sediments by the Maquoketa River has removed much of this material from the valley, but substantial remnants remain throughout the study area in the form of high sandy and gravel-rich river terraces. Most of these high glacial terraces are now occupied by modern residential developments like those at Hickory Hollow, Deer Run, Linden Acres, Clear View, and Maples (see Figures 2 and 3).

More recent alluvial fills, deposited at various times over the past 10,000 years, are also present within the valley. These Holocene-age alluvial and colluvial deposits are of particular interest to archaeologists since this period captures more than 75 percent of the time period that humans are believed to have lived in Iowa (Alex 2000:37). As such, these deposits have potential to contain buried archaeological remains and understanding how they've accumulated and been altered over time not only helps us assess which landforms might contain evidence of past human occupation, but it also helps us determine what investigative methods may be required to locate those deposits. Figure 3 illustrates the estimated distribution of these principal fill deposits within the valley based on landform elevations and limited field reconnaissance.

As described above, new influxes of valley sediment during the glacial period appear to be correlated with changes in regional climate (i.e., increases and decreases in temperature and precipitation) that in turn fostered periods of increased erosion and sedimentation. These same patterns apply to the post-glacial Holocene period, but at a much reduced scale. Valley fill deposited during the early to middle Holocene (10,000 to 3,000 years ago), is inset about 20 to 25 feet below the high glacial terraces (approximately 895 to 885 feet amsl within the study area) and lacks the coarse sand, pebbles, and fractured rock material found in the full-glacial deposits. These early to middle Holocene-age sediments typically consist of silty or loamy alluvium overlying stratified sandy alluvium with a basal layer

Plate 3. High Terrace Remnant Above Hartwick. (Note quantities of heavy rock and gravel on the exposed terrace slope. Most of the overlying loamy topsoil has been washed away.)

Plate 4. Early Holocene Alluvium in Terrace Cut Near Hartwick. (Note banded layers of stratified modern alluvium on top of the buried soil.)

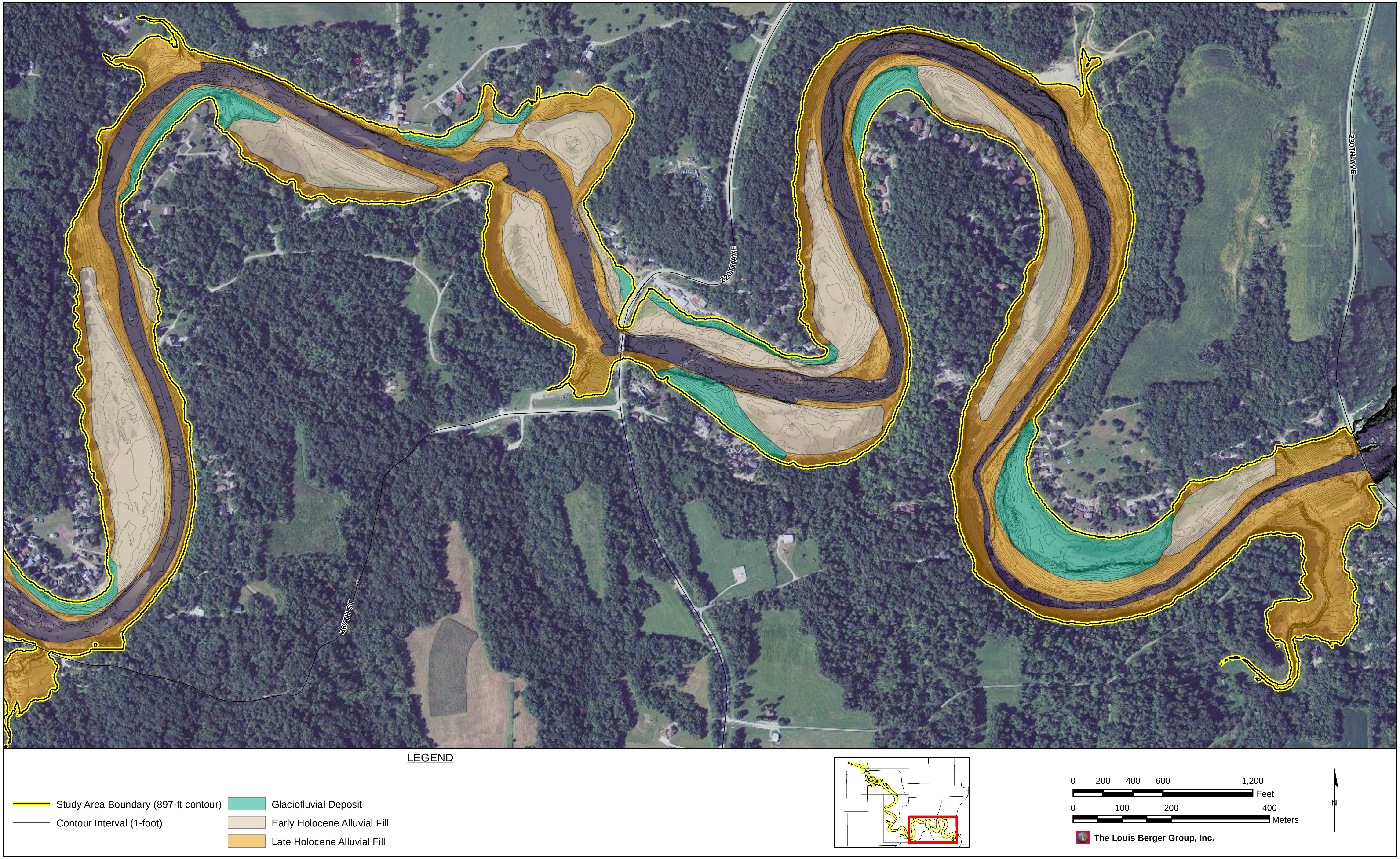


FIGURE 3a: Estimated Age and Distribution of Valley Fill Deposits

SOURCE: ISU 2010

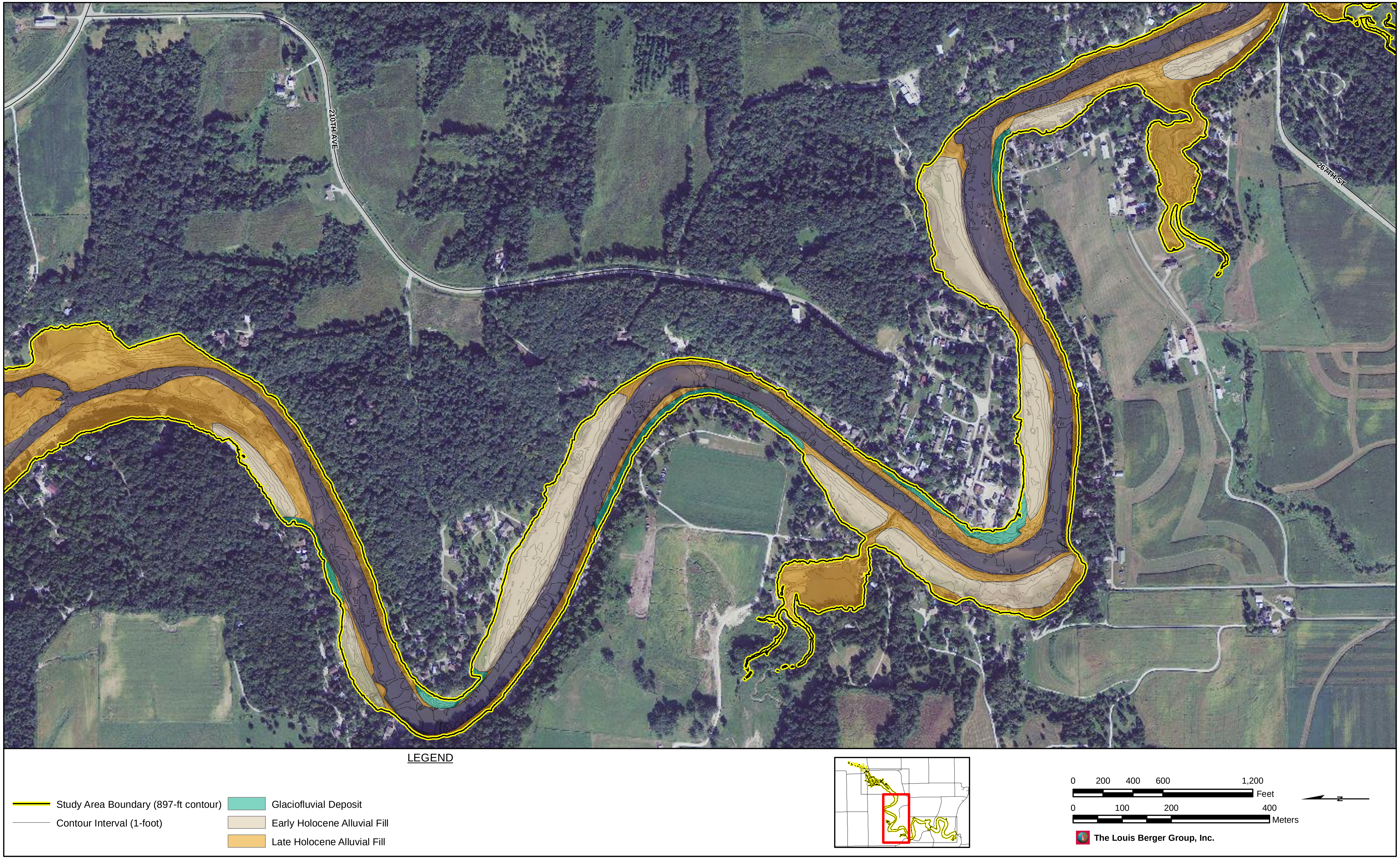


FIGURE 3b: Estimated Age and Distribution of Valley Fill Deposits

SOURCE: ISU 2010

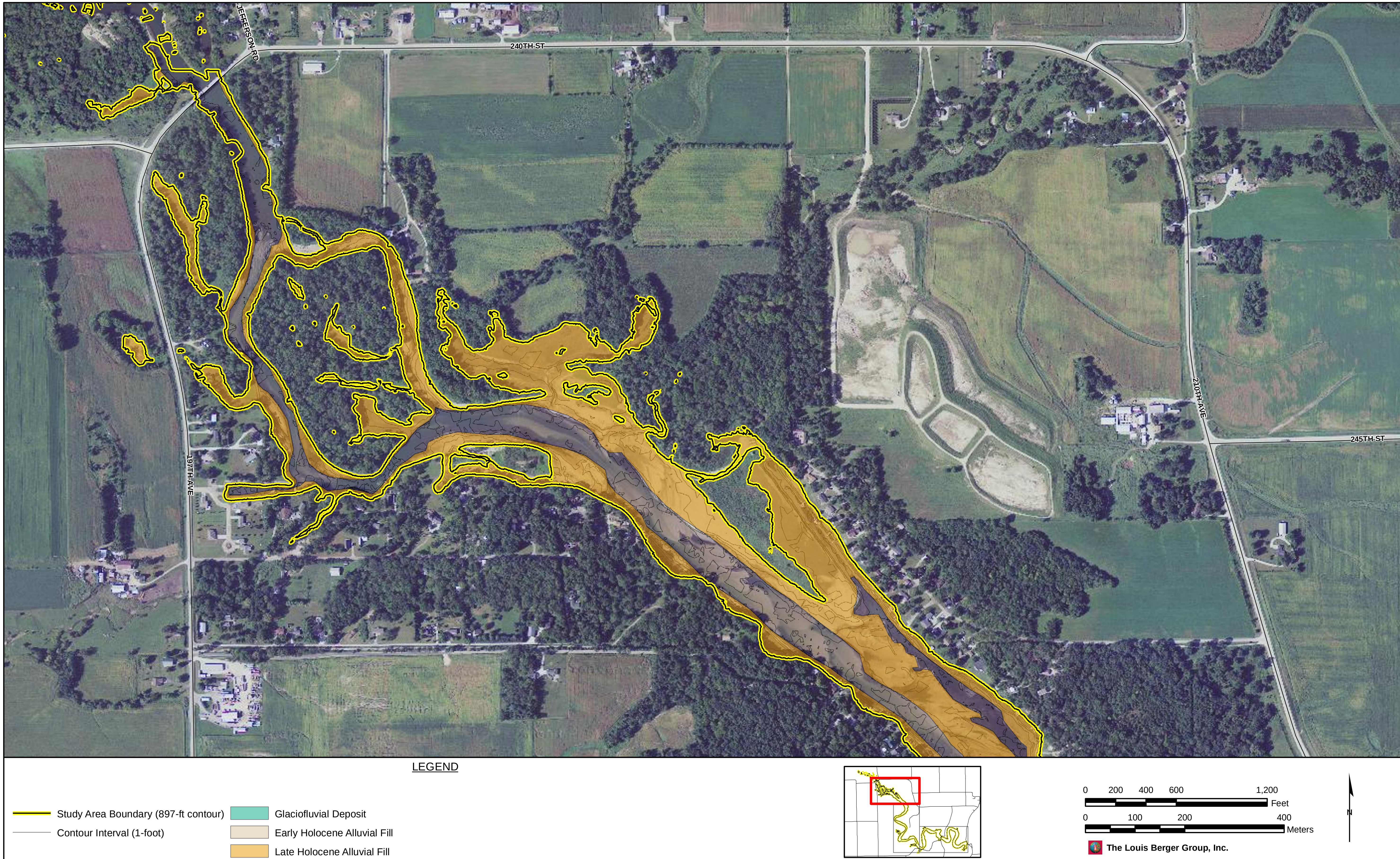


FIGURE 3c: Estimated Age and Distribution of Valley Fill Deposits

SOURCE: ISU 2010

consisting of chert gravel. Soils formed in these deposits typically have well-developed E and B subsoil horizons that extend to depths of five to six feet (e.g., Bertrand series).

Late Holocene-age alluvium deposited between 3,000 and 500 years ago is generally present at still lower elevations within the valley and closest to the active river channel although in some places late Holocene deposits may also overlie portions of older land surfaces. These late Holocene sediments typically consist of dark colored loamy alluvium grading to coarser sand or gravel. Soils formed in these deposits have A over C horizon profiles and may have "overthickened" A horizons measuring one meter or more that reflect relatively slow sedimentation rates compared with rates of pedogenic development (e.g., Spillville, Coland series).

Alluvial materials deposited within the past 400 to 500 years are also present as a distinctive sediment package throughout the study area. The influx of these materials is partially correlated with increases in human activity that introduced large amounts of sediment into drainage systems, particularly with the arrival of European-Americans and clearing of land for intensive agriculture. These sediments usually accumulate close to the modern stream channel and often bury older land surfaces close to the channel. They show very little evidence of soil development and frequently include modern artifacts and other modern debris items. As the site of an artificial lake, the Lake Delhi segment of the Maquoketa River valley is somewhat atypical in the sense that deposits of recent alluvium are found throughout the study area; however, deposits of this recent material are likely to be thickest at the upper end of the impoundment where the river slows as it enters the lake and drops much of its sediment load and at lower elevations as one moves downstream. Historic air photos from the 1930s to 1960s document significant changes in the shoreline of Lake Delhi as soil eroded from surrounding uplands filled the mouths of the many tributary streams feeding into the lake.

Potential for Unreported Prehistoric Sites

Generally speaking, most prehistoric habitation sites tend to be located on well-drained, level ground surfaces located near sources of water. As such, many of the river terraces found throughout the study area would therefore be considered to have potential for unreported prehistoric sites including open habitation sites as well as associated cemeteries or burial mounds. In addition to fresh water, these locations were often favorably situated near a variety of other important material and food resources. High quality stone resources suitable for the manufacture of stone tools are abundant throughout the study area. Layers of Hopkinton chert are exposed in bedrock outcrops along the valley margin and large quantities of this material are exposed in the reworked glacial deposits that fill the valley. Igneous and metamorphic rock derived from eroded glacial till would have provided suitable raw material for a variety of ground-stone tools such as axes, mauls, hammers, and milling stones. Timber for building material and fuel was often most abundant along drainageways where the rivers or streams provided barriers to prairie fires. Fish, mollusks and aquatic animals would also have been readily available nearby, and water sources were also an important attraction for game animals. Prominent landforms (bluff tops, ridge tops, and knolls) located adjacent to these habitation zones were often selected as a preferred location for associated cemeteries.

The local geology found in this part of Iowa also offers potential for additional living sites situated on the steep-sided valley walls. The Silurian bedrock that outcrops throughout the study area includes strata that are variably resistant to erosion. Some formations include caves or other cavities that may have been used by the region's prehistoric inhabitants. Another one of the region's unique features are natural rock ledges or overhangs along the valley margins created where more resistant layers outcrop above softer ones. Rock ledges and overhangs were sometimes used for shelter by the area's prehistoric inhabitants. These natural "rockshelters" may therefore contain prehistoric archaeological deposits and should be considered as another potentially important landscape feature within the study area particularly where these overhangs occur in close proximity to the artificial lake shoreline.

Potential for Unreported Historic Sites

As described in Sections C and D of this chapter, previous researchers have identified two locations within the study area as having high potential for archaeological remains associated with the region's early settlement by European-Americans during the mid-19th century. One of these is located at the Lake Delhi dam which appears to have been constructed on the former site of the "Rockynook Mills", a sawmill/gristmill/dam complex owned and operated by Charles Fleming from 1861 to 1894 (Merry 1914:328-330; Western Historical Company 1878:601) and others after him. The second locale is represented by the former townsite at Hartwick which boasted its own sawmill/gristmill/dam complex plus a variety of other business interests including a general store, tavern, blacksmith, wheelwright or wagon shop, cobbler shop, paint shop, and brickyard.

Table 3. Potential Historic Archaeological Sites Located Within the Study Area

Potential Historic Site	Legal Location	Period of Occupation	Notes
Rockynook Grist Mill (Charles Fleming)	**SE, NE of Section 30, T88N-R4W**	**1861-1894**	**North Side of River at Delhi Dam; Above Former Maquoketa River Bridge**
Rockynook Saw Mill (Charles Fleming)	**SW, NW of Section 29, T88N-R4W**	**1861-1894**	**South Side of River at Delhi Dam; Below Former Maquoketa River Bridge**
Hartwick Saw Mill (John Clark/Russell Furman)	**NW, NW of Section 30, T88N-R4W**	**1849-1907**	**North Side of River; Above Maquoketa River Bridge**
Hartwick Grist Mill (John Clark/Russell Furman)	**NW, NW of Section 30, T88N-R4W**	**1853-1907**	**South Side of River; Below Maquoketa River Bridge**
Hartwick Townsite	**NW, NW of Section 30, T88N-R4W**	**1858-1907**	**North Side of River; Below Maquoketa River Bridge; reported to include sawmill, gristmill, general store, tavern, blacksmith, wheelwright/wagon shop, cobbler/shoe shop, paint shop, brickyard and bridge**

The local geology and topography found in this portion of the Maquoketa River valley made these two locations particularly attractive for early milling operations. The exposed bedrock in the floor of the valley and the bedrock constrained valley walls offered ideal settings for creation of the lowhead dam and mill pond reservoir needed to power the mills.

Hartwick Mills and the Town of Hartwick

According to local histories (Merry 1914; Western Historical Company 1878), the town of Hartwick was founded in 1858 by John W. Clark and his wife Miriam Clark. The Clarks were the first settlers in Hartwick, having built a dam and sawmill on the north bank of the Maquoketa River at this location in 1849. The sawmill at Hartwick was built using materials originally intended for a mill on Spring Branch, farther upstream near Bailey's Ford. The would-be proprietor of the Spring Branch Mill, Leverett

Rexford, had reportedly completed most of the dam, water wheel and gear mechanism for his mill when he died in 1848. John Clark subsequently purchased the framing materials and machinery and used them to build the saw mill at Hartwick which was completed in the spring of 1849 (Western Historical Company 1878:375). Several years later in 1853, Clark began work on a gristmill situated on the south side of the river and had already opened a store and tavern. Clark was joined at Hartwick by a blacksmith named John Whitman in 1855, an unnamed wheelwright whom Clark helped sponsor in 1857, and a cobbler/shopkeeper, a brickmaker (Samuel Stansbury and brother), and a painter/shopkeeper named Jacob Williams by 1859. John Clark, along with John Whitman, are reported to have been the driving forces intent on placing the town of Hartwick on the map, but it appears that they each developed financial problems that led them to sell their interests and leave the area during the late 1850s. The Clark farmstead near Hartwick was leased to the county in 1861 and was used temporarily to house the indigent (Merry 1914:230; Western Historical Company 1878:552-553). Clark's milling operation was eventually purchased by Russell W. Furman in 1869. Furman and later on his sons, Charles and George Furman, appear to have operated and maintained the mill in operating condition until 1907 when the mill dam was washed away in a flood. Charles Furman relocated to Delhi where he became a storeowner and businessman (Merry 1914:327-328), while his brother George remained on the family farm at Hartwick. George Furman proceeded to demolish the vestige houses that remained at Hartwick and converted the abandoned townsite to farmland (Merry 1914:408-409).

The mills at Hartwick are not well recorded on historic maps of the project area. No mills are plotted at this location on the 1875 Andreas map or the 1894 Davis plat (Figures 4 & 5). A grist mill is depicted on the 1904 Huebinger map of Delaware County south of the Maquoketa River and downstream of the road/bridge that crosses the river at this location (Figure 6). This lack of specificity is frustrating but is probably best explained by the fact that mapmakers instead depict the Hartwick townsite at this location and therefore opted not to call out the location of individual buildings. Fortunately, several images of what appears to be the Hartwick gristmill have been preserved in the form of both hand-drawn illustrations and photographs (Plates 5 & 6). The images compare well with each other and details such as the pattern of sunlight and shadow on the buildings confirm that this is indeed the structure located on the south side of the Maquoketa River. The structure's position below the river bridge is consistent with the Huebinger map and the building's three-story construction suggests that it is more likely to be a gristmill as compared to sawmills of the period. Even though its location cannot be confirmed by reference to historic maps or other documents, it seems logical then to assume that the Hartwick sawmill was probably located on the opposite or north side of the river.

Rockynook Mills

Local histories report that Charles F. Fleming was the founder and proprietor the Rockynook Mills, which is apparently how the former milling operation sited at Delhi Dam was known (Merry 1914:328-331). Fleming is reported to have settled near Delhi in 1857 and operated a steam-powered gristmill at Silver Lake before eventually deciding in 1861 to either buy or perhaps build a sawmill on the Maquoketa River south of Delhi. Merry's 1914 *History of Delaware County* states that Fleming built the saw mill at the Rockynook Mills location in one passage (Merry 1914 [V2]:328), but elsewhere suggests that Fleming purchased an already existing mill (Merry 1914 [v1]: 218). Whichever is correct, it appears that Fleming successfully managed the milling operation for more than 30 years before finally selling it to an unnamed party in 1894. Historic photographs taken during dam construction in 1926 strongly suggest that any remains of the former grist mill were obliterated during dam construction. Newspaper accounts describing construction of the Delhi dam in 1926 reported that an eight-foot-high low-head dam was removed prior to construction of the new dam (McDowell 2009).

The location of the Rockynook Mills is better documented on historic maps. All available maps are consistent with regard to placing the gristmill on the north side of the river and the saw mill south of the

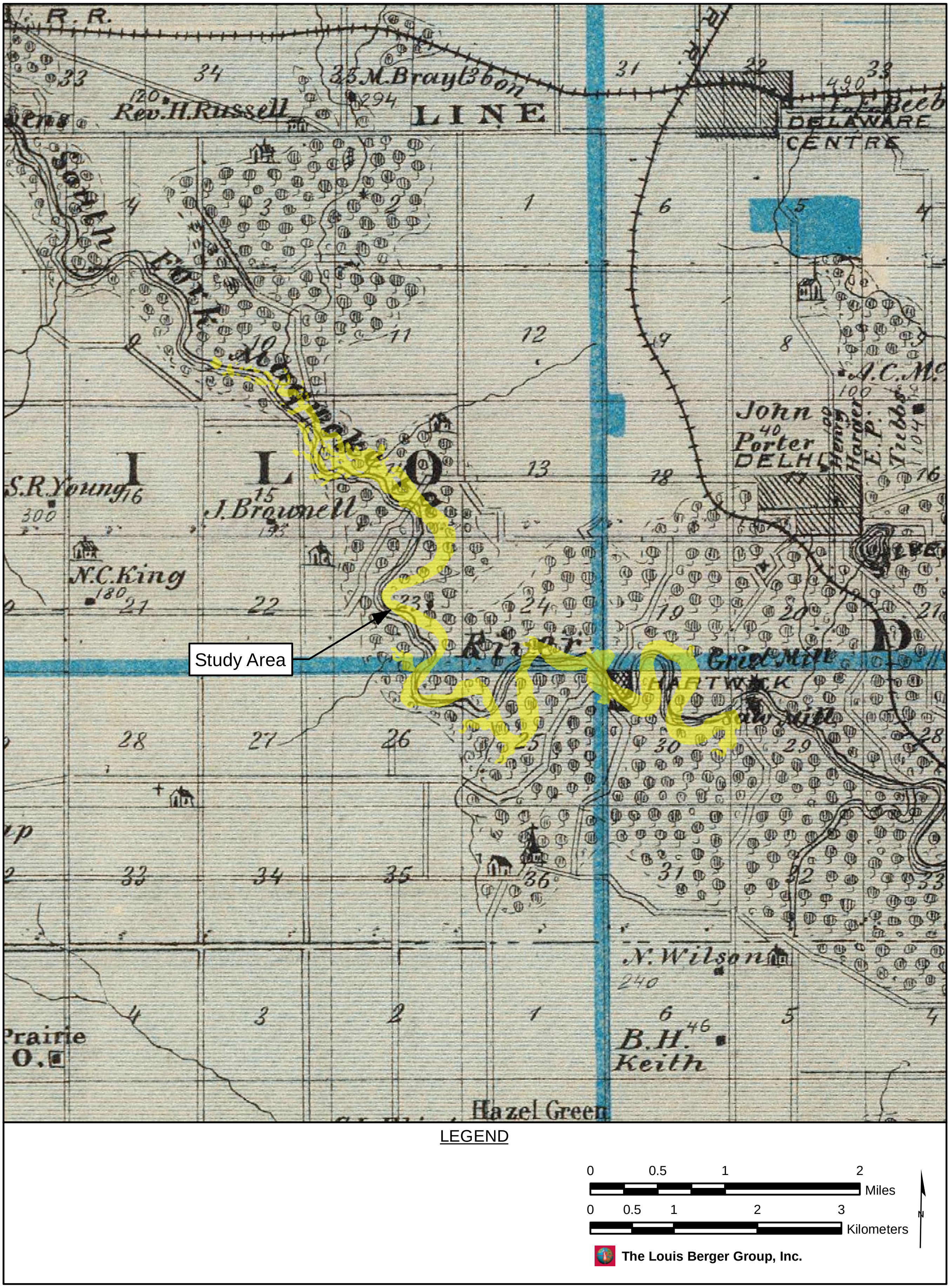


FIGURE 4: 1875 Map of the Study Area

SOURCE: Andreas 1875

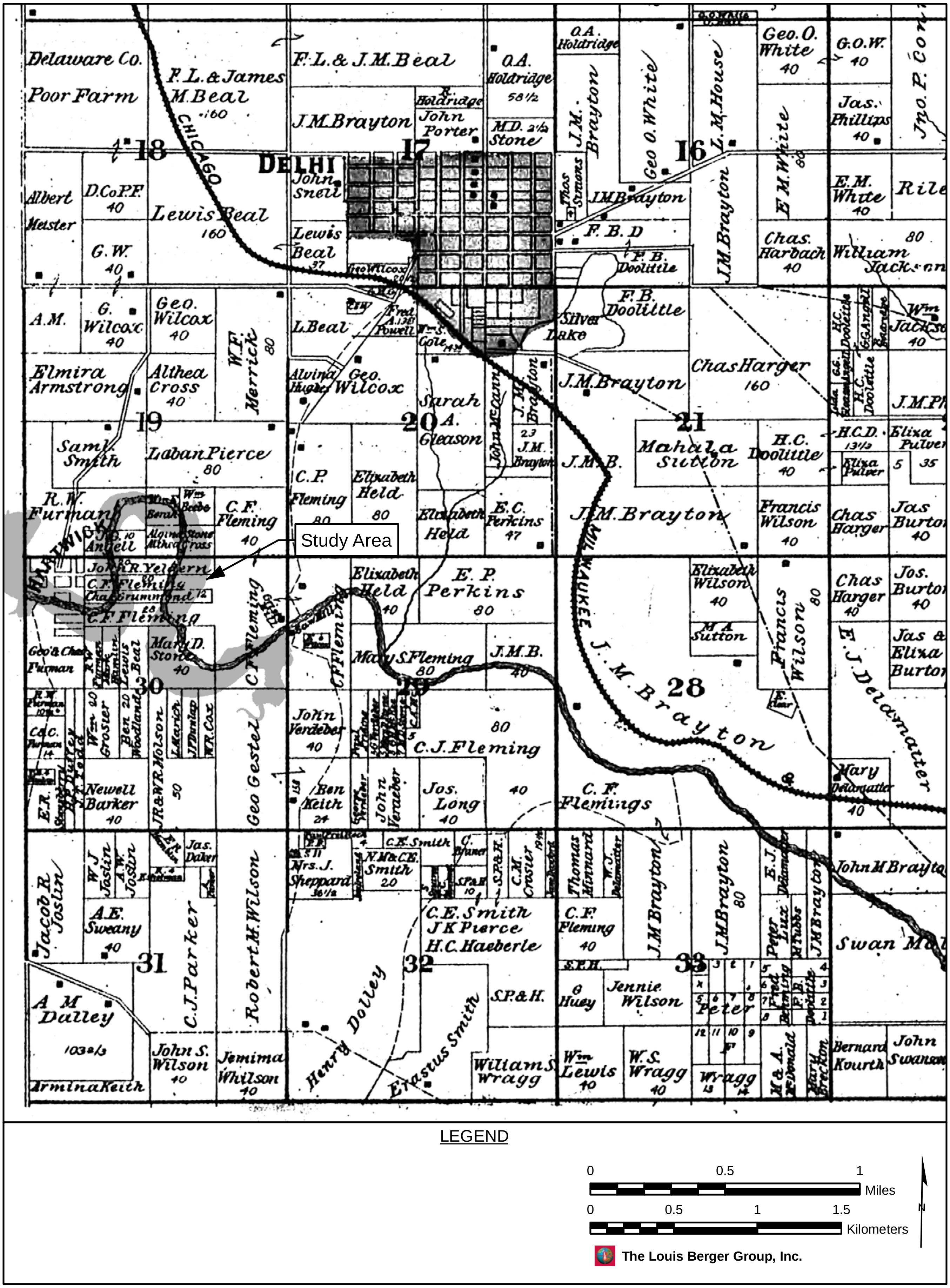


FIGURE 5: 1894 Map of the Study Area

SOURCE: Davis 1894

FIGURE 6: 1904 Map of the Study Area

SOURCE: Huebinger 1904

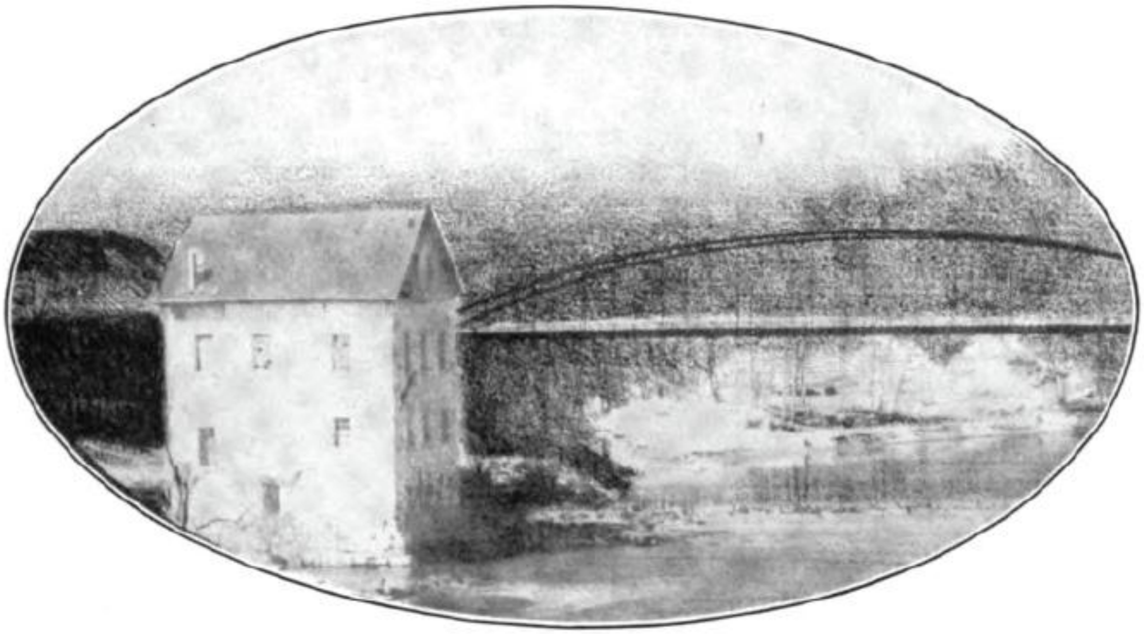


Figure 7. Illustration of Furman's Mill and Hartwick Bridge (Reproduced from Merry 1914:219).

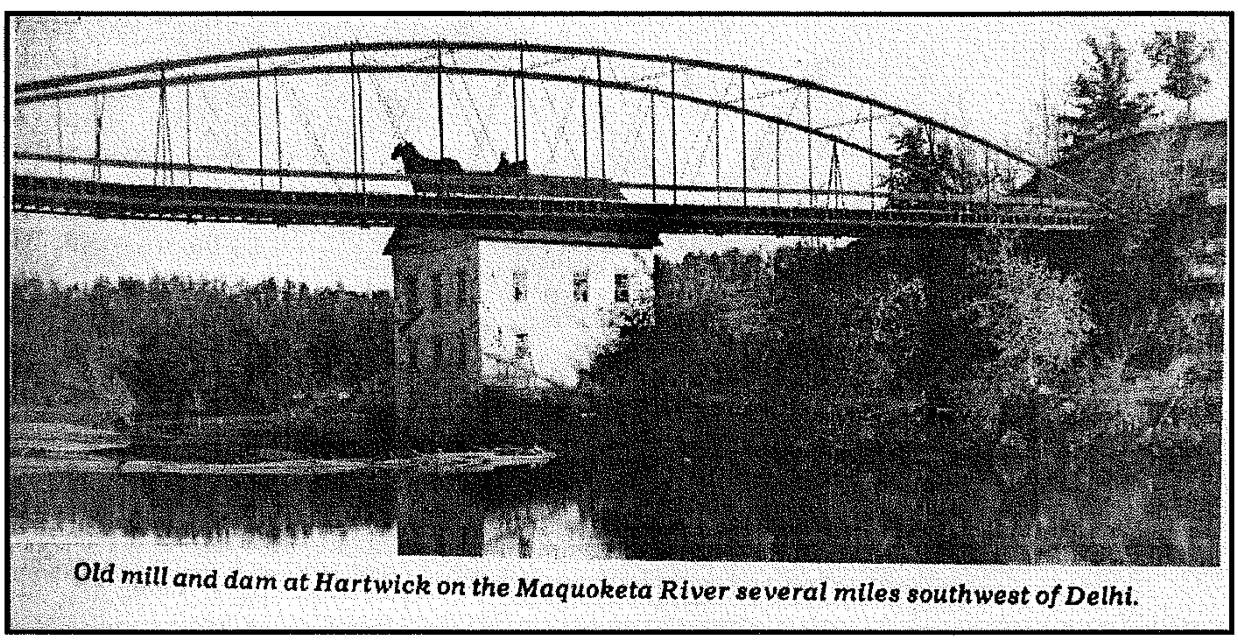


Plate 5. Photograph of Furman's Mill and Hartwick Bridge. (Reproduced from Finn 2010. Original source is Bailey nd.)

River (see Figures 4, 5, 6). There is however some apparent inconsistency with regard to the mill locations and spatial relationships to the road/bridge that crossed the river. The 1875 Andreas map, the 1894 Davis map, and the 1904 Huebinger map each show the Rockynook gristmill on the north side of the Maquoketa River. The 1875 map shows it east of the road that crosses the river while both the 1894 and 1904 maps clearly show the gristmill on the west side of the road/bridge. The Rockynook sawmill is shown south of the river on both the 1875 and 1894 maps, but is absent on the 1904 map suggesting that it may have been abandoned prior to that time. As with the gristmill, the saw mill is depicted east of the road/bridge in 1875 but west of the road/bridge in 1894. If this information is assumed accurate, it would indicate that the original river crossing passed upstream from the mills and behind the mill dam which seems unlikely. In any case, the more recent map information is consistent in showing both mills positioned west of the road/bridge.

One of the Rockynook Mills appears to be depicted in a hand-drawn illustration published in Merry's 1914 *History of Delaware County* (Merry 1914:219; Figure 7); however the sketch is much too generalized differentiate it as either the gristmill or saw mill or confirm its location.

Maquoketa River Bridges at the Hartwick and Rockynook Crossings

Bowstring Arch-Truss bridges were constructed across the Maquoketa River at both the Hartwick and Rockynook locations at an early date, most likely sometime during the 1860s, 1870s, or early 1880s. Images of the former bridges at these locations are preserved in both drawings and photographs (see Plates 5, 6, & 8). The bowstring arch-truss was patented in 1841 and was an especially popular design for this period. It was widely used in Iowa due to its use of inexpensive materials, relative ease of construction, and wide range of span lengths. The structures were apparently mounted on tall rock piers made from locally quarried cut limestone. Remnants of the south bridge pier at the Rockynook crossing was exposed by the 2010 floodwater along the south wall of the gated spillway. The north pier was most likely incorporated into the block retaining wall along the north bank of the river below the dam. The bridge at Hartwick was supported by piers of similar construction judging by the following description preserved in the 1878 county history:

> *"The Maquoketa is bridged at this place by a graceful iron structure, which springs from a high rocky bank on the south side of the stream, and the north side rests on a high pier built of massive magnesian rock* [Western Historical Company 1878:553]."

Figure 8. Illustration of the Fleming Rockynook Mill at the Delhi Dam Location. (Reproduced from Merry 1914:219).

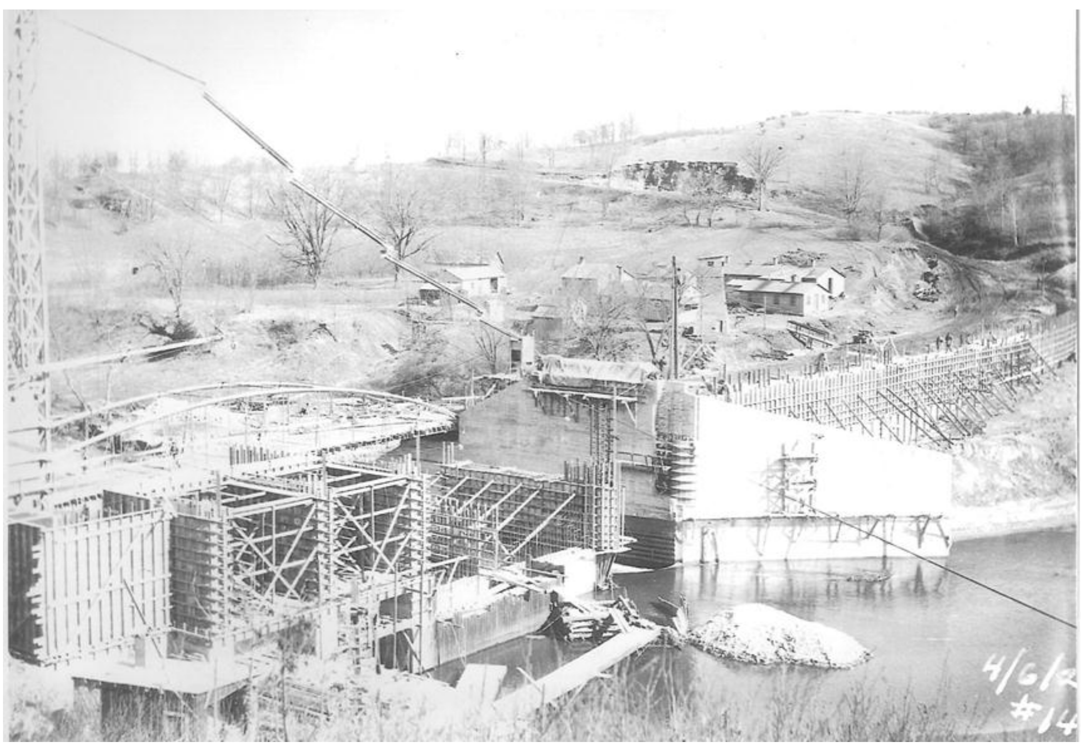


Plate 6. 1926 Photograph of Delhi Dam Under Construction (Reproduced from Fiedler et al. 2010). Note the location of the Bowstring Arch-Truss bridge below the dam in the background.

Plate 7. Photograph of the exposed south wall of the gated spillway at Delhi Dam (Reproduced from Fiedler et al. 2010; This limestone pier once supported the south end of the bowstring arch bridge.)

V. FIELD INVESTIGATIONS

A. PHASE I RECONNAISSANCE

On September 28-30, 2011, LBG conducted Phase I reconnaissance surveys for selected portions of the study area (Figures 9a-9c). The field survey was performed by the author with assistance from Field Archaeologist Sam Williams and was designed to inspect landforms considered to have high potential for unreported archaeological sites. The survey also gathered information on the nature of valley landforms and local geology, patterns of land use and landscape/shoreline modification, and the nature and extent of damage to valley landforms and archaeological resources caused by catastrophic flooding. LBG was given access to the entire study area but was instructed to respect private property beyond the lake shoreline. Public roads and boat ramps provided preferred access points to the lakebed although LBG also gained access to portions of the study area via private property with the consent of local landowners. Public road access was limited to the valley crossings at 220th Avenue, the Turtle Creek Recreation Area along 267th Street, and County Road X31 (which of course remained closed at the Delhi Dam). Boat ramp access was utilized at Linden Acres, Maples and the Cedars (see Figure 2).

Point bar locations with full glacial and early Holocene landforms were the primary focus of the field survey. Surveyors also traversed the steep-sided slopes of outside meanders to inspect the valley walls for potential cave formations and rockshelters along the former shoreline. Eroded surfaces and cutbanks were closely inspected for artifacts, archaeological features and other archaeological materials (e.g., burned rock, charcoal, animal bone, shell concentrations, etc.); however, the field survey was ultimately performed in a largely opportunistic and unsystematic fashion due to the presence of thick and extensive overwash deposits on many of the open terrace landforms and the presence of dense weedy vegetation covering most low lying areas that hid potential resources from view in most places. For this reason, the survey results presented below should not be considered to represent a complete inventory of the archaeological resources present within the study area. To the contrary, we did not examine every landform within the study area that might be considered to have potential for unreported archaeological sites. More importantly, our investigation confirms that there are many Holocene-age landforms within the study area that have geological potential to include buried archaeological deposits that could only be detected through subsurface exploration. As a result and for these reasons, the current investigation is best viewed as a preliminary effort in information gathering designed to provide project planners and agency reviewers with a more complete basis for making future decisions regarding long-term management of the area's known and potential archaeological resources. Some recommendations for future planning are presented in the next chapter.

General Observations

Photos taken shortly after the July 24, 2010 breach event at Lake Delhi show a largely barren landscape of sand and mud littered with boat cradles, boat docks and other debris. The survey described here was performed about 14 months after the event and during the interim nature did its best to reclaim the exposed lakebed. Most low-lying areas within the drained valley were blanketed with a thick layer of silt that represented fertile ground for pioneering weedy plants. At the time of our survey, these locations were completely covered with a thick tangled growth of weedy vegetation about three to four feet tall. Higher terraces tended to have less vegetation, partly because they were covered by less fertile sand deposits but also because some landowners were making efforts to cut it back where possible. Even so, all but the most sandy land surfaces were also becoming overgrown by vegetation (Plates 8-13).

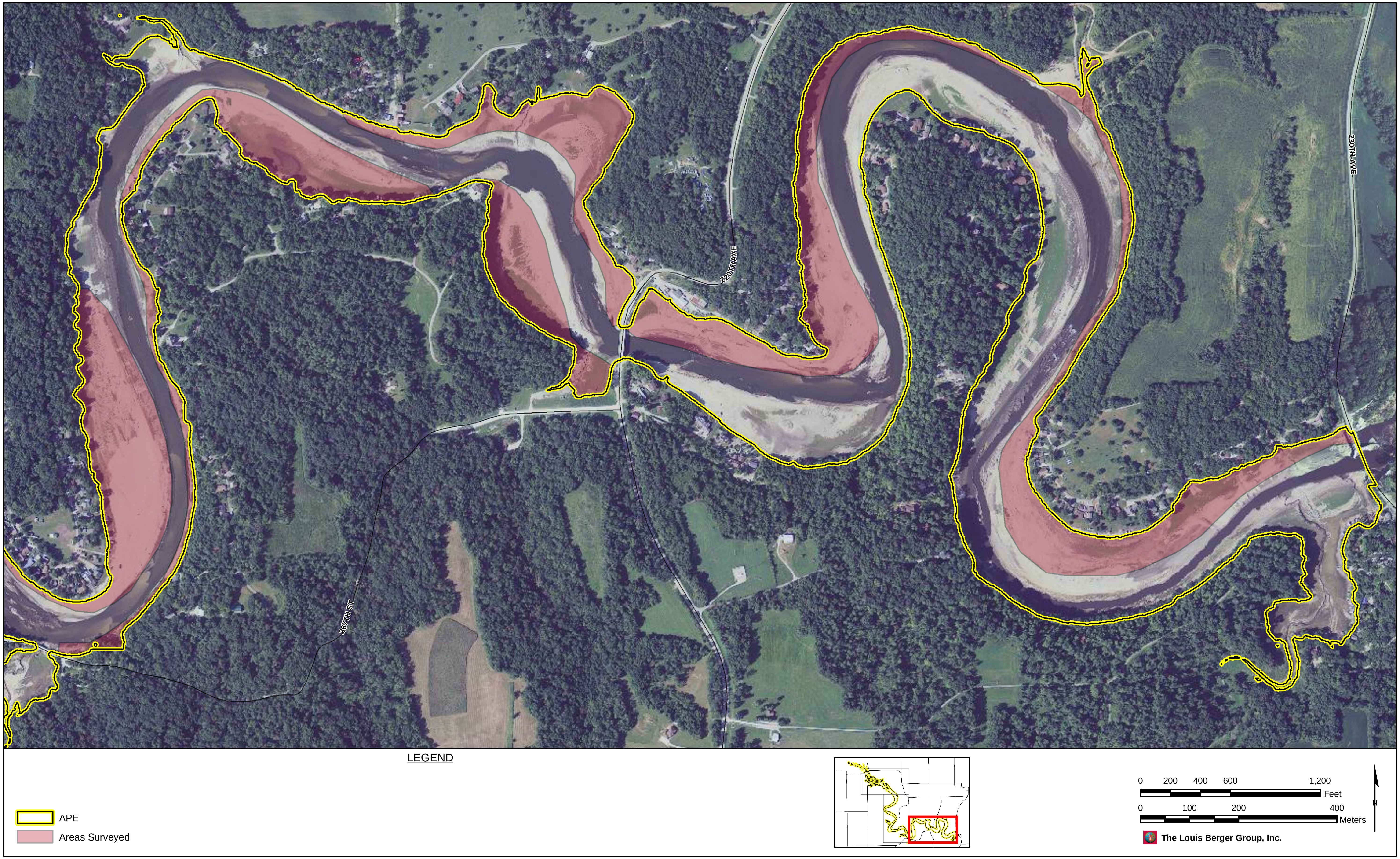


FIGURE 9a: Area Surveyed

SOURCE: ISU 2010

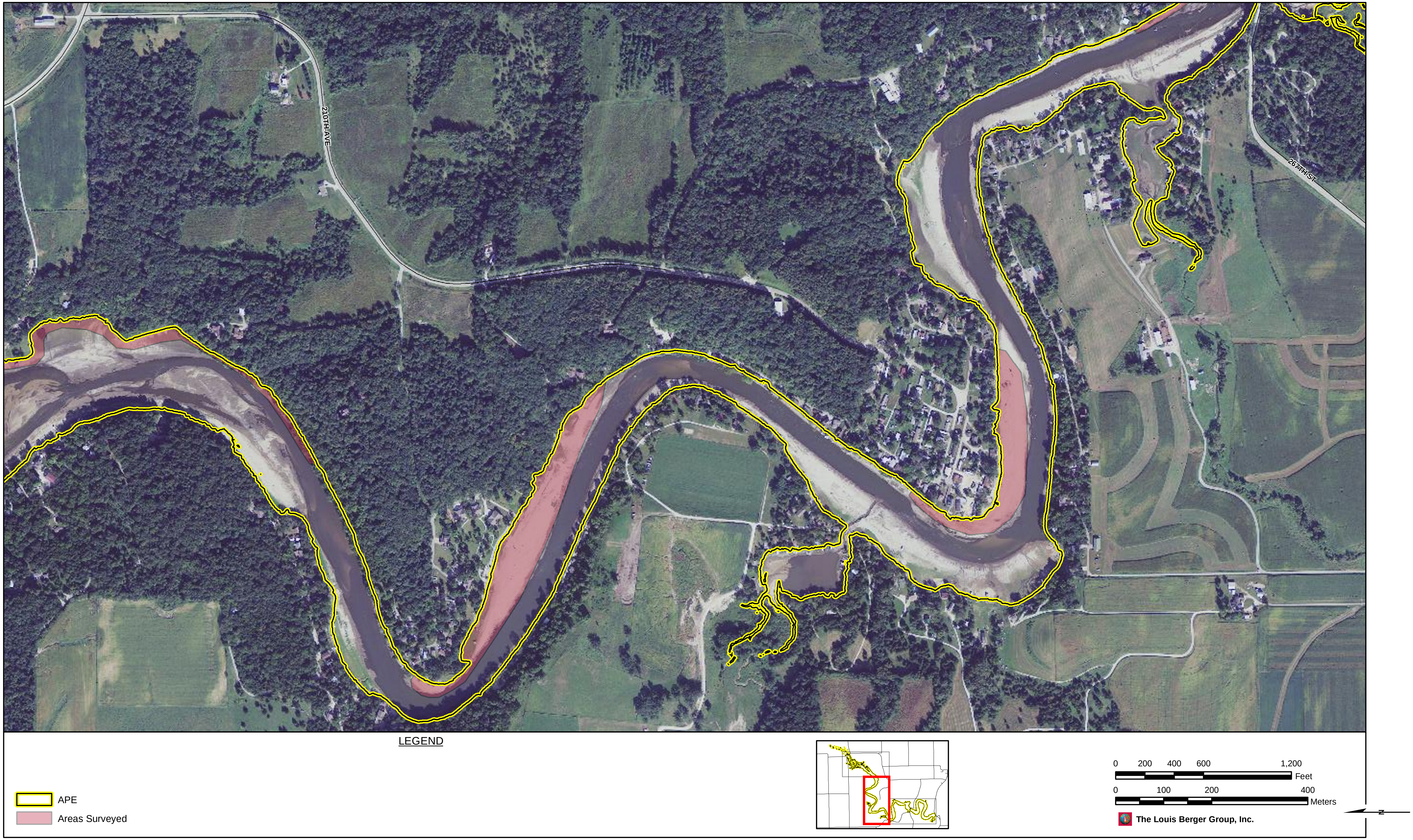


FIGURE 9b: Area Surveyed

SOURCE: ISU 2010

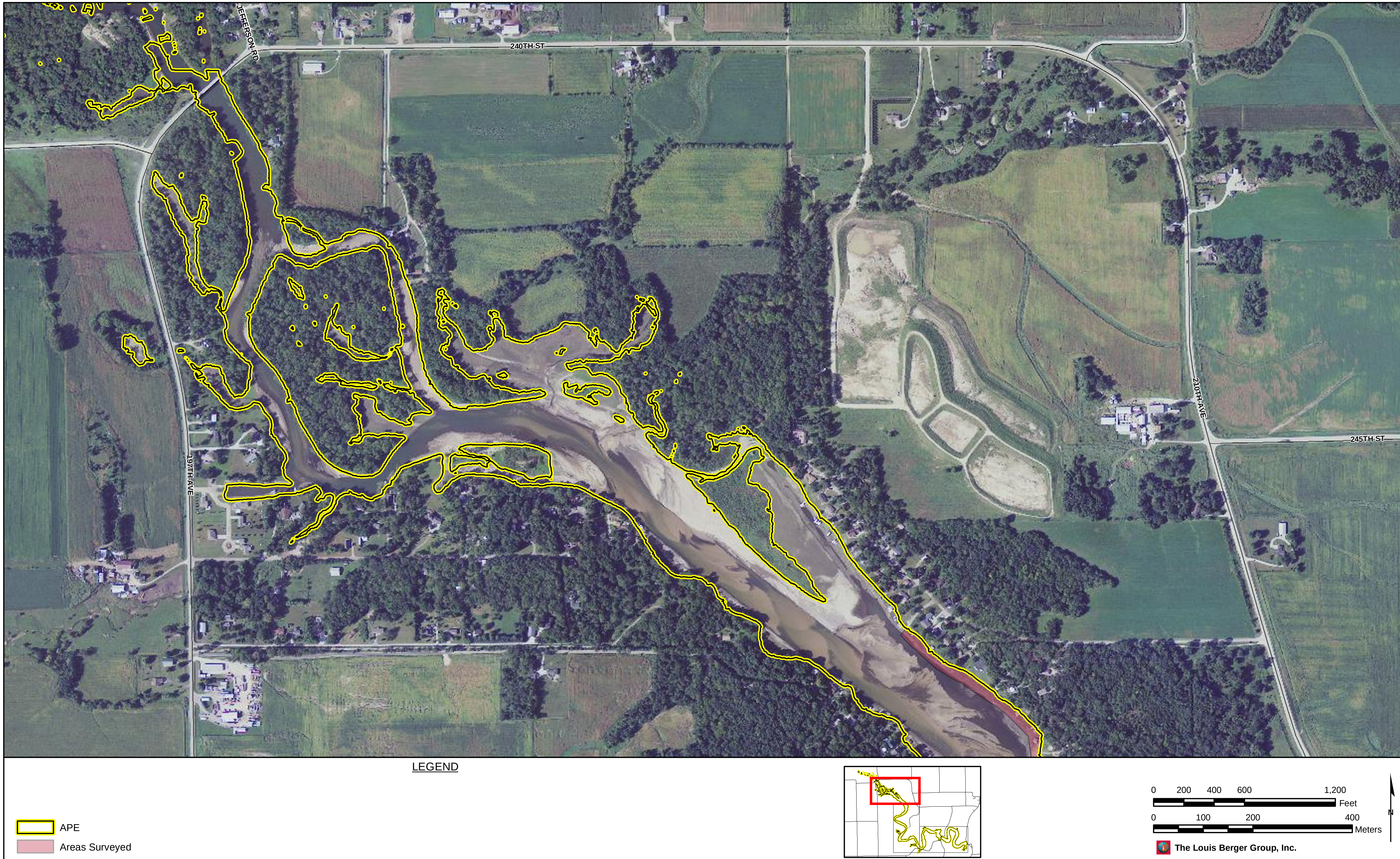


FIGURE 9c: Area Surveyed

SOURCE: ISU 2010

Plate 8. Vegetation Obscures the Lakebed Above Delhi Dam. View downstream.

Plate 9. Hartwick Terrace Downstream from the 220th Avenue Bridge.
(Note new riffle structures and rip-rap material installed during the fall of 2010).

Plate 10. Sand deposits on the Holocene terrace below Linden Acres. View upstream.

Plate 11. Sand Deposits Cover the Holocene Terrace at Clearview Acres. View downstream.

Plate 12. Sand and vegetation on the Holocene terrace below the Maples. View downstream.

Plate 13. View looking upstream toward the Cedars.

The Maquoketa River valley of course continues to be a very active environment with areas of significant erosion and deposition of alluvial materials. The thick layers of recent sediment described above greatly limited our ability to locate artifacts and other evidence of archaeological deposits within the valley. Meandering streams tend to deposit larger particles like sand and pebbles along the inside edge of point bars -- the points of land that form the inside of the stream meander – and over the past eight decades some of these deposits have accumulated in significant quantities. A cursory review of recent aerial photographs of the valley shows this pattern very clearly with large splays of light-colored sand spreading downstream below every point bar. These modern overwash deposits obviously bury and conceal the native surface of terrace landforms throughout the study area and thereby greatly diminish the likelihood of detecting any archaeological deposits without subsurface testing.

We also observed places that have been subject to erosion in the past and others that continue to be actively eroded. The most significant erosion continues to occur along the river channel itself. As mentioned above, concerns about the amount of sediment being carried downstream as the river began to reclaim its historic channel prompted the installation of erosion control structures within the channel at both Hartwick and Delhi dam locations in the fall of 2010 (see Plate 9). The most serious channel and stream bank erosion continues to occur in this lower section of the study area where the gradient is most steep and valley slopes have not yet stabilized. The south or right bank just below the Hartwick bridge is one area in particular where the river continues to cut laterally against high terrace landforms and where stream bank erosion has potential to impact potentially significant archaeological deposits associated with sites 13DW125 and 13DW126 (see Plate 4). Elsewhere in the study area outside meanders are largely constrained by bedrock with the result that there are virtually no exposed vertical cut banks along the river channel upstream from Hartwick.

Sheet erosion along with rill and gully erosion is also developing on many of the high terrace slopes from a point beginning about one-half mile above the Hartwick bridge downstream to the Delhi Dam (Plates 14, 15). The damage is caused by normal rainfall and sheet erosion on unprotected surfaces, but the problem is also exacerbated or amplified in many places by stormwater runoff from nearby homes and other developments above the shoreline. In many places, stormwater that was once diverted by drain tile or landscaping directly into the lake is now being drained onto the surface of the exposed terrace landforms. In places where this surface is unprotected, we observed many rills and large gullies developing along the terrace edge. In several places we found gullies like these cutting through what appear to be potentially significant archaeological deposits (e.g., Site 13DW133).

B. SURVEY RESULTS

LBG identified eight new archaeological sites within the study area. Together with the four sites reported by WVA archaeologists near Hartwick in 2010, there are currently a total of 12 known archaeological sites within the study area (Table 4). Use of the valley by prehistoric Native American cultures is evidenced at 10 of the 12 sites and includes eight probable habitation sites (7 open sites, 1 rockshelter), one lithic resource procurement location, and one fishweir. Mid-19th century building foundations are represented at two separate locations near the former townsite of Hartwick and are believed to be associated with the historic settlement that once existed at that location. Fragments of contemporaneous historic artifacts were also identified at two other nearby locations. A map depicting the locations of these sites along with copies of associated site forms are provided in a separate volume (Volume II) in recognition of legal restrictions regarding the public disclosure of confidential site location information.

Plate 14. Gullies formed by storm water runoff near Hickory Hollow

Plate 15. Gully erosion along the point bar terrace at Deer Run below Hartwick. View to south.

Table 4. List of Known Archaeological Sites Within the Study Area

Site Number	Site Type	Cultural & Temporal Associations	Estimated Dimensions
13DW123	Artifact Scatter	Middle Woodland (2200-1650 BP) Late Prehistoric (800-300 BP) Historic (AD 1855-1907)	140 x 20 meters
13DW124	Habitation	Undetermined Prehistoric	130 x 30 meters
13DW125	Stone Foundation	(AD 1855-1907)	10 x 10 meters
13DW126	Resource Procurement (prehistoric); Artifact Scatter (historic)	Archaic (10,000-2800 BP) Historic (AD 1855-1907)	160 x 60 meters
13DW133	Habitation	Early to Middle Woodland (2800-1500 BP)	130 x 50 meters
13DW134	Habitation	Middle Woodland (2500 to 1650 BP)	100 x 20 meters
13DW136	Stone Foundation (Hartwick Saw Mill)	Historic (AD 1849-1907)	20 x 20 meters
13DW137	Habitation	Undetermined Prehistoric	30 x 10 meters
13DW138	Artifact Scatter	Undetermined Prehistoric	30 x 10 meters
13DW139	Artifact Scatter	Undetermined Prehistoric	30 x 15 meters
13DW140	Fish Weir	Undetermined Prehistoric	20 x 20 meters
13DW141	Rockshelter	Undetermined Prehistoric	20 x 20 meters

Site 13DW123

Site Name: None
Map Source: USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description: (Confidential)
Site Type: Artifact Scatter
Cultural/Temporal Association: Prehistoric (Middle Woodland, 2200-1650 BP; Late Prehistoric 800-300 BP); Historic (Hartwick Townsite, circa AD 1855-1907)
Site Size: 20 Meters (N-S) X 140 Meters (E-W)
Phase I Methods: Pedestrian Survey
Area Excavated: N/A
Cultural Materials Collected: None by LBG; Reported by Finn and Morrow 2010; WVA Collection included: Middle Woodland Ceramic, 3 Bifaces, 4 Debitage, Scapula Hoe (Late Prehistoric), Bottle Glass, Window Glass, Cut Nails, Horseshoe, 19th Century Dinnerware, Brick Fragments
Cultural Materials Observed: LBG did not reinvestigate this site
Landform: High Terrace & Holocene Terrace
Elevation: 890-897 Feet Above Mean Sea Level
Land Use/Surface Visibility: Eroded Shoreline (50-75% Surface Visibility)
Soil Type: Lilah Sandy Loam, 2-9% Slopes
Site Disturbance: Soil Erosion; Gullied Slope
Relation To Study Area: Boundary Is Not Fully Established; Site Likely Extends Above Shoreline
NRHP Eligibility: Not Evaluated
Recommendations: Additional Phase I Testing; Bank Stabilization if Warranted

Site 13DW123 was identified by WVA in the fall of 2010 based on a scatter of prehistoric and historic period artifacts observed along an eroded terrace slope. The artifact scatter consists primarily of fragmented historic period items that include several varieties of glazed stoneware and decorated whiteware ceramics (blue feather and shell-edge, blue, purple and flow blue transfer print, blue sponge-decorated, annular/banded), olive and aqua bottle glass, window glass, and machine-cut nails. All of these items are consistent with manufacture and use dates during the mid-19th century and suggest the presence of a probable domestic use context associated within the Hartwick townsite. A relatively smaller number of prehistoric chipped-stone, ceramic, and bone artifacts were also observed and collected by WVA. These items included a grit-tempered pottery fragment with Middle Woodland characteristics, 3 bifaces, and a bone tool that was identified by WVA as a possible garden hoe manufacture from part of a bison scapula. The bone tool had been modified in a manner that suggested possible association with the later prehistoric Oneota culture (Toby Morrow, personal communication, November 7, 2011). No subsurface tests have been excavated at the site which is believed to extend above the shoreline onto the adjacent terrace.

LBG did not reinvestigate this site and the site remains unevaluated for listing in the NRHP. Based on the observations recorded by WVA, LBG recommends that additional archaeological investigations be conducted to more fully define the nature and vertical and horizontal extent of the archaeological deposits present at 13DW123, including site areas that may extend above the anticipated shoreline elevation of 897 feet amsl.

Site 13DW124

Site Name:	None
Map Source:	USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description:	**(**Confidential)
Site Type:	Artifact Scatter
Cultural/Temporal Association:	Prehistoric (undefined)
Site Size:	20 Meters (N-S) X 100 Meters (E-W)
Phase I Methods:	Pedestrian Survey
Area Excavated:	N/A
Cultural Materials Collected:	None by LBG; Reported by Finn and Morrow 2010; WVA Collection included: 2 Bifaces, 1 utilized flake, 27 Debitage, 1 greenstone axe/celt blank
Cultural Materials Observed:	LBG observed several hundred pieces of additional chipped stone debitage at the site but did not collect them.
Landform:	High Terrace & Holocene Terrace
Elevation:	885-897 Feet Above Mean Sea Level
Land Use/Surface Visibility:	Eroded Shoreline (50-75% Surface Visibility)
Soil Type:	Lilah Sandy Loam**,** 0-2% Slopes; Bertrand silt loam, 0-2% Slopes
Site Disturbance:	Soil Erosion; Gullied Terrace and Terrace Slope
Relation To Study Area:	Boundary Is Not Fully Established; Site Likely Extends Above Shoreline
NRHP Eligibility:	Not Evaluated
Recommendations:	Additional Phase I Testing; Bank Stabilization if Warranted

Site 13DW124 was first reported by WVA in the fall of 2010 based on a scatter of both prehistoric and historic period artifacts and a historic building foundation situated on an eroded river terrace. The building feature was described as a stone foundation measuring 4x6 meters with few associated artifacts. The overall site size was estimated as measuring 60 meters (NW-SE) by 25 meters (SW-NE) which encompassed the more extensive prehistoric artifact scatter.

LBG visited this site on September 29, 2011 and relocated, mapped and photographed the building foundation. LBG archaeologists also identified additional prehistoric artifacts outside the previously reported site boundary which almost doubled the estimated length of the total site area. Based on our field observations, we proposed that the Office of the State Archaeologist consider recording the historic building foundation and the prehistoric artifact scatter as two separate sites since the deposits were close together but did not overlap. The OSA Site Records coordinator (Eck) agreed to assign a new number to the building foundation (13DW136) but recommended that 13DW124 be cross-referenced on the new site record. Site 13DW124 now refers only to the prehistoric artifact scatter at this location.

WVA archaeologists collected a total of 31 prehistoric artifacts from the surface of this site in the fall of 2010 including: two chipped-bifaces, one utilized flake, 27 pieces of debitage or chipping debris, and one percussion-flaked greenstone axe/celt blank. The greenstone artifact was actually found resting on the nearby building foundation and was presumed abandoned there by someone who most likely collected it nearby (Toby Morrow, personal communication, November 7, 2011).

Plate 16. Chipped-Stone Artifacts Exposed on the Eroded Surface of Site 13DW124.

LBG did not collect any additional material from the surface of the site but observed several concentrations of chipped-stone artifacts exposed on the ground surface extending as much as 50 meters north of the site boundary reported by WVA. In each instance, the artifacts appeared to be in-situ and were pedestaled on otherwise eroded subsoil (B horizon) deposits (Plate 16). No modified specimens, tools, or other temporally diagnostic artifacts were observed at the site. Other downstream portions of the site appeared to have experienced less sheet erosion and still retained what appeared to be an intact A horizon; however, surface erosion has progressed to the point where rills and gullies have developed through these deposits, in some places well into subsoil.

No subsurface tests have been excavated at the site which is believed likely to extend above the shoreline onto the adjacent terrace. The site remains unevaluated for listing in the NRHP. Based on the field observations summarized above, LBG recommends that additional archaeological investigations be conducted to more fully define the nature and vertical and horizontal extent of the archaeological deposits present at 13DW124, including site areas that may extend above the anticipated shoreline elevation of 897 feet amsl.

Site 13DW125

Site Name:	None
Map Source:	USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description:	**(**Confidential)
Site Type:	Building Foundation
Cultural/Temporal Association:	Historic (Hartwick Townsite, circa AD 1855-1907)
Site Size:	10 Meters (N-S) X 10 Meters (E-W)
Phase I Methods:	Pedestrian Survey
Area Excavated:	N/A
Cultural Materials Collected:	None by LBG; Identified by Finn and Morrow 2010; WVA did not collect the site
Cultural Materials Observed:	LBG did not reinvestigate this site;
Landform:	Holocene Terrace
Elevation:	885-890 Feet Above Mean Sea Level
Land Use/Surface Visibility:	Eroded Terrace (90-100% Surface Visibility)
Soil Type:	Bertrand silt loam, 0-2% Slopes
Site Disturbance:	Soil Erosion; Sheet Erosion
Relation To Study Area:	Site is Located 100% Within the Study Area
NRHP Eligibility:	Not Evaluated
Recommendations:	Additional Phase I Testing

Plate 17. Exposed Building Foundation at 13DW125. (Reproduced from Finn and Morrow 2010).

Site 13DW125 was identified by WVA in the fall of 2010 based on the discovery of a square stone-foundation with brick-wall superstructure. The foundation appeared to have been exposed by floodwaters that scoured overlying sediments from the area. The stone foundation was constructed with cut blocks of local dolomite and measured approximately six meters by six meters in size. A substantial number of bricks interpreted to be collapsed walls were observed on all sides of the foundation. A small cast-iron stove was observed inside the structure, but no other associated artifacts were observed around the building. WVA speculated that archaeological deposits outside the building foundation were likely removed by floodwaters but that some deposits may still exist within the interior.

LBG did not reinvestigate this site and the site remains unevaluated for listing in the NRHP. Based on the observations recorded by WVA, LBG recommends that additional archaeological investigations be conducted to more fully define the nature and extent of the archaeological deposits present at 13DW125.

Site 13DW126

Site Name:	None
Map Source:	USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description:	**(**Confidential)
Site Type:	Resource Procurement (Lithic); Artifact Scatter
Cultural/Temporal Association:	Prehistoric (Archaic, 10,000-2,800 BP) Historic (Hartwick Townsite circa 1855-1907)
Site Size:	60 Meters (N-S) X 160 Meters (E-W)
Phase I Methods:	Pedestrian Survey
Area Excavated:	N/A
Cultural Materials Collected:	None by LBG; Identified by Finn and Morrow 2010; WVA Collection included: 1 Merom-type projectile point, 1 point mid-section, 9 Bifaces, 1 hammerstone, 152 Debitage and a burned rock concentration plus 12 Bottle Glass, 12 Window Glass, 4 Cut Nails, 24 fragments of 19^{th} Century ceramic, Brick Fragments, 1 deer bone
Cultural Materials Observed:	LBG did not reinvestigate this site
Landform:	High Terrace
Elevation:	890-895 Feet Above Mean Sea Level
Land Use/Surface Visibility:	Eroded Shoreline (90-100% Surface Visibility)
Soil Type:	Lilah Sandy Loam**,** 0-2% Slopes
Site Disturbance:	Soil Erosion
Relation To Study Area:	Site is Located 100% Within The Study Area.
NRHP Eligibility:	Not Evaluated
Recommendations:	Additional Phase I Testing; Bank Stabilization if Warranted

Site 13DW126 was identified by WVA in the fall of 2010 based on the discovery of prehistoric and historic period artifacts an eroded river terrace. The majority of the site material is associated with prehistoric Native American use of the area as an apparent resource procurement site focused on the collection and early stage processing of Hopkinton Formation chert. Morrow feels that the overall assemblage is consistent with an Early Archaic tool kit (Toby Morrow, personal communication, November 7, 2011). He noted the presence of large thinning flakes produced during reduction of large bifaces as well as several bifaces with similar proportions. He also noted that one unfinished biface collected from the site has a distinctive "plano-convex" cross-section suggesting that the original intent may have been to fashion it into a chipped-stone adze, a tool type that has strong Early to Middle Archaic associations elsewhere in Iowa (e.g., Fiedel et al. 2004). In addition to these hints of an Early Archaic presence at 13DW126, WVA collected a single Late Archaic Merom-style projectile point (3600-3000 years BP; Justice 1987:132) from the site and also reported a concentration of burned rock suggesting potential for hearth-like archaeological features that may be associated with these early site components.

WVA also reported finding a light scatter of historic period materials across the site. Temporally diagnostic items found at the site include machine-cut nails and a variety of decorated whiteware ceramics (i.e., hand-painted, light blue, brown and purple transfer-printed, blue sponge-decorated). Other historic items collected from the site included: bottle glass, window glass, and fragments of brick. All of these items are consistent with manufacture and use dates during the mid-19th century and suggest the presence of a probable domestic use context nearby that is associated with the Hartwick townsite. No subsurface tests have been excavated at the site.

LBG did not reinvestigate this site and the site remains unevaluated for listing in the NRHP. Based on the observations recorded by WVA including the presence of early prehistoric materials, temporally diagnostic artifacts, and potential for undisturbed prehistoric hearth features, LBG recommends that additional archaeological investigations be conducted to more fully define the nature and vertical and horizontal extent of the archaeological deposits present at 13DW126.

Site 13DW133

Site Name:	None
Map Source:	USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description:	**(**Confidential)
Site Type:	Habitation
Cultural/Temporal Association:	Prehistoric (Middle Woodland)
Site Size:	50 Meters (N-S) X 130 Meters (E-W)
Phase I Methods:	Pedestrian Survey
Area Excavated:	N/A
Cultural Materials Collected:	Waubesa Contracting Stem Projectile Point; returned to landowner
Cultural Materials Observed:	3 Bifaces, 4 Debitage,
Landform:	High Terrace
Elevation:	890-897 Feet Above Mean Sea Level
Land Use/Surface Visibility:	Eroded Shoreline (50-75% Surface Visibility)
Soil Type:	Lilah Sandy Loam**,** 0-2% Slopes
Site Disturbance:	Soil Erosion; Gullied Slope
Relation To Study Area:	Boundary Is Not Fully Established; Site Likely Extends Above Shoreline
NRHP Eligibility:	Not Evaluated
Recommendations:	Additional Phase I Testing; Bank Stabilization if Warranted

Site 13DW133 was identified by LBG based on field investigations performed on September 28, 2011. The site is represented by a moderate scatter of prehistoric chipped-stone tools, bifaces and other by-products of stone-tool manufacture on the exposed surface of a high terrace overlooking the Maquoketa River. Chipped-stone artifacts were observed on the exposed surface of the terrace and in erosional gullies that had developed along the outer margins of the terrace slope. One of these, a small contracting stem projectile point (cf., Waubesa Contracting Stem) is diagnostic of Early to Middle Woodland period (2800-1500 years BP) assemblages in eastern Iowa. The specimen from 13DW133 is unusual in that it was manufactured from a tan or brownish variety of Cambrian-age Hixton silicified sandstone which is derived from bedrock sources in Jackson County, Wisconsin near Black River Falls. A small fragment of another finished biface was also observed at the site. This appeared to be manufactured from a light-colored Mississippian-age (Burlington Formation) chert most likely derived from sources in southeast Iowa. The overwhelming majority of specimens were otherwise made from a transluscent gray variety of Hopkinton chert which is locally abundant in both bedrock sources and secondary outwash deposits. The latter included a wide range of early reduction bifaces, primary shaping flakes and biface thinning flakes. A concentration of unmodified limestone/dolomite cobbles was observed near the center of the site. The rock concentration must have a cultural origin because stones of this size would not occur naturally in this geologic context, but its purpose or function was not obvious or apparent. No bone or artifacts were observed in direct association with the feature and there was no visible indication of any soil staining or discoloration that might indicate that it was part of a fill deposit. Nonetheless, it may warrant treatment as a cultural feature (additional photos are provided in Appendix C).

Plate 18. Chipped-Stone Artifacts from Site 13DW133.

No subsurface tests have been excavated at the site. Artifacts were observed near the outer margin of the terrace and were also observed close to the former shoreline and there is a very high likelihood that they are more extensive than reported here, i.e., there is high potential for the site deposits to extend onto the adjacent terrace landform above the shoreline. Portions of the site, particularly areas near the outside terrace margin are being severely eroded by stormwater runoff that is creating deep rills and gullies through the site deposits including areas immediately adjacent to the cultural feature described above.

Site 13DW133 remains unevaluated for listing in the NRHP; however, the presence of temporally diagnostic artifacts, high artifact density, evidence for the use of non-local raw materials, and the likely presence of undisturbed cultural features indicates that the site deposits have potential to be considered for National Register eligibility. Based on these observations LBG recommends that additional archaeological investigations be conducted to more fully define the nature and vertical and horizontal extent of the archaeological deposits present at 13DW133, including site deposits that may extend above the anticipated shoreline elevation of 897 feet amsl. These investigations should include consultation with the Office of the State Archaeologist regarding an appropriate methodology for investigating the rock feature observed at the site.

Site 13DW134	
Site Name:	None
Map Source:	USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description:	**(**Confidential)
Site Type:	Artifact Scatter
Cultural/Temporal Association:	Prehistoric (Middle Woodland)
Site Size:	20 Meters (N-S) X 100 Meters (E-W)
Phase I Methods:	Pedestrian Survey
Area Excavated:	N/A
Cultural Materials Collected:	None
Cultural Materials Observed:	Manker Corner Notched and Dickson Contracted Stem projectile points, finished biface fragments, early-stage biface blanks, primary shaping and biface thinning debitage.
Landform:	High Terrace
Elevation:	890-897 Feet Above Mean Sea Level
Land Use/Surface Visibility:	Eroded Shoreline (50-75% Surface Visibility)
Soil Type:	Lilah Sandy Loam**,** 2-9% Slopes
Site Disturbance:	Soil Erosion; Gullied Slope
Relation To Study Area:	Boundary Is Not Fully Established; Site Likely Extends Above Shoreline
NRHP Eligibility:	Not Evaluated
Recommendations:	Additional Phase I Testing; Bank Stabilization if Warranted

Site 13DW134 was identified by LBG based on field investigations performed on September 29, 2011. The site is represented by a moderate scatter of prehistoric chipped-stone tools, bifaces and other by-products of stone-tool manufacture on the exposed surface of a high terrace overlooking the Maquoketa River. Chipped-stone artifacts were observed on the exposed surface of the terrace and in erosional gullies that had developed along the outer margins of the terrace slope. Two broken projectile point discovered on the surface of the site indicate that it was occupied during the Middle Woodland period (2500-1650 years BP). The two specimens resemble Manker corner-notched and Dickson contracting stem types. The overwhelming majority of chipped-stone artifacts appeared to be made from locally available varieties of Hopkinton chert which seemed to be particularly abundant in the high terrace outwash deposits downslope from the site. A high percentage of the chipped-stone material observed at the site also appeared to be heat-altered or heat-treated giving these specimens a distinctive oxidized color (i.e., orange to red) compared with untreated gray specimens. This process is thought to have been applied to improve the fracture qualities of the local stone and is often applied to late-stage biface specimens prior to final thinning and edge finishing.

No subsurface tests have been excavated at the site. Artifacts were observed near the outer margin of the terrace and were also observed close to the former shoreline and there is a very high likelihood that they are more extensive than reported here, i.e., there is high potential for the site deposits to extend onto the adjacent terrace landform above the shoreline. Portions of the site, particularly areas near the outside terrace margin are being severely eroded by stormwater runoff that is creating deep rills and gullies through the site deposits. The surface soil at this site has been eroded over time and appears to be very thin if not altogether absent. Subsurface testing is needed to better evaluate the present condition of the site deposits at 13DW134.

Plate 19. Chipped-Stone Artifacts from Site 13DW134.

Site 13DW134 remains unevaluated for listing in the NRHP; however, assuming there is sufficient surface soil at the site to offer potential for undisturbed subsurface deposits, then the presence of temporally diagnostic artifacts and high artifact density would allow the site deposits to be considered for National Register eligibility. Based on these observations LBG recommends that additional archaeological investigations be conducted to more fully define the nature and vertical and horizontal extent of the archaeological deposits present at 13DW134, including site deposits that almost certainly extend above the anticipated shoreline elevation of 897 feet amsl.

Site 13DW136

Site Name:	None
Map Source:	USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description:	**(**Confidential)
Site Type:	Stone Foundation; Possible Site of the Clark/Furman Saw Mill
Cultural/Temporal Association:	Historic (Hartwick Townsite, circa 1849-1907)
Site Size:	20 Meters (N-S) X 20 Meters (E-W)
Phase I Methods:	Pedestrian Survey
Area Excavated:	N/A
Cultural Materials Collected:	None; Identified by Finn and Morrow 2010 as Site 13DW124
Cultural Materials Observed:	Cut Limestone/Dolomite Blocks, Brick, Machine-Cut Nails; Window Glass
Landform:	Holocene Terrace
Elevation:	882-890 Feet Above Mean Sea Level

Land Use/Surface Visibility:	Eroded Shoreline (50-75% Surface Visibility)
Soil Type:	Bertrand Silt Loam, 0-2% Slopes
Site Disturbance:	Soil Erosion; Gullied Slope
Relation To Study Area:	Site is 100% Within the Study Area
NRHP Eligibility:	Not Evaluated
Recommendations:	Additional Phase I Testing

Site 13DW136 was first reported by WVA in the fall of 2010 as part of 13DW124. Site 13DW124 included a historic building foundation situated on an eroded river terrace and a moderately sized scatter of prehistoric artifacts extending away from the structure. The building feature was described as a stone foundation measuring 4x6 meters with few associated artifacts. The overall site size, which included the prehistoric artifact scatter, was estimated as measuring 60 meters (NW-SE) by 25 meters (SW-NE).

LBG visited this site on September 29, 2011 and relocated, mapped and photographed the building foundation. LBG archaeologists also identified additional prehistoric artifacts outside the previously reported site boundary which almost doubled the estimated length of the total site area for 13DW124. Based on our field observations, we proposed that the Office of the State Archaeologist consider recording the historic building foundation and the prehistoric artifact scatter as two separate sites since the deposits were close together but did not appear to overlap. The OSA Site Records coordinator (Eck) agreed to assign 13DW136 to the building foundation but recommended that the original number assigned by WVA (13DW124) be cross-referenced on the new site record. Site 13DW136 now refers only to the historic building foundation at this location.

Site 13DW136 consists of a stacked limestone or dolomite foundation situated about 40 feet from the outer margin of an early Holocene terrace. The foundation appears to be rectangular in outline and is oriented with its shorter wall parallel to the terrace edge and the present Maquoketa River channel a short distance beyond. The full extent of the foundation is not exposed but the visible portion measures 24 feet wide and at least 30 feet long. The short wall on the west side is fully exposed and both it and the south wall are mostly visible and appear to be largely intact; however, the north west corner of the foundation has been washed out by the river and a 15-foot section is mostly absent. The short wall facing the river is exposed to a height of three feet and includes seven courses of tabular stone. Close inspection of the foundation along the south wall shows that it measures 24 inches thick. The ruins also include a large quantity of brick and many of the individual bricks are still attached to one another with mortar. Comparatively few historic artifacts were observed, although a number of machine-cut nails could be seen along the walls.

The building's position on the north bank of the river close to the former mill dam at Hartwick, its proximity and orientation to the river, its overall size and its thick-walled foundation suggests that it could be Hartwick's very first building, i.e., John Clark's saw mill built in 1849. The interior of the foundation appears to be filled with rubble and demolition debris suggesting that excavation of the interior may yield materials that could be used to determine its historic use and purpose.

No test excavations have been performed at this location and the site remains unevaluated for listing in the NRHP. Based on the historic background research completed for this report and the information gathered about the building's foundation, LBG recommends that additional archaeological investigations be conducted to determine if the structure has potential to be identified as the saw mill built by John Clark and later operated by the Furman family. These investigations should begin with more comprehensive archival research and interviews with local residents who may be knowledgeable about the early history of the Hartwick area. There may be historic photographs of the original mill complex that could be useful in making this determination without additional archaeological testing. If no conclusive evidence is found, then subsurface investigations both within and outside the foundation should be performed to

Plate 20. Stone Foundation at Site 13DW136. View of the west wall.

Plate 21. Plan View of the South Foundation Wall at Site 13DW136.

determine if any undisturbed archaeological materials exist that may help establish the building's historic identity and evaluate its potential eligibility for listing in the NRHP.

Site 13DW137

Site Name:	None
Map Source:	USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description:	**(**Confidential)
Site Type:	Habitation
Cultural/Temporal Association:	Prehistoric
Site Size:	30 Meters (N-S) X 10 Meters (E-W)
Phase I Methods:	Pedestrian Survey
Area Excavated:	N/A
Cultural Materials Collected:	None;
Cultural Materials Observed:	1 finished biface fragment; chert debitage
Landform:	High Terrace
Elevation:	890-897 Feet Above Mean Sea Level
Land Use/Surface Visibility:	Eroded Shoreline (50-75% Surface Visibility)
Soil Type:	Lilah Sandy Loam**,** 2-9% Slopes
Site Disturbance:	Soil Erosion; Eroded Slope
Relation To Study Area:	Boundary Is Not Fully Established; Site Likely Extends Above Shoreline
NRHP Eligibility:	Not Evaluated
Recommendations:	Additional Phase I Testing; Bank Stabilization if Warranted

Site 13DW137 was identified by LBG based on field investigations performed on September 29, 2011. The site is represented by a moderate scatter of prehistoric chipped-stone artifacts including a finished knife-like biface and other by-products of middle to late-stage stone-tool manufacture that were observed eroding from the top of a high terrace slope overlooking the Maquoketa River (Plate 22). Chipped-stone artifacts were observed on the exposed surface of the terrace at and below the former lake shoreline. No temporally diagnostic artifacts were observed, but numerous pieces of chert debitage are present on the eroded slope below the former lake shoreline. Most of these items appear to be middle to late-stage biface thinning flakes manufactured from local variations of Hopkinton chert. The majority of the chipped stone artifacts observed at the site appear to be unaltered with regard to heat treatment, but some heat-treated specimens are present including the finished biface. Heat may have been used to improve the fracture qualities of the local stone and is often applied during the later stages of stone tool manufacture.

No subsurface tests were excavated at the site. Artifacts were observed near the outer edge of the high terrace at and below the former shoreline, and it is reasonable to assume that more extensive site deposits extend onto the adjacent terrace above the shoreline. All of the archaeological materials observed on the terrace scarp below the shoreline are in eroded or secondary context. The potential for undisturbed or intact archaeological deposits exists at the 894 to 897 elevation contours and above. Subsurface testing on the adjacent terrace surface above the shoreline is needed to better evaluate the present condition of the site deposits at 13DW137.

Plate 22. Chipped-Stone Artifacts from Site 13DW137.

Site 13DW137 remains unevaluated for listing in the NRHP but the high terrace adjacent to the site is not heavily developed; therefore, there is potential for well preserved archaeological deposits to be present in the residential yard areas adjacent to the site. Based on these observations LBG recommends that additional archaeological investigations be conducted to more fully define the nature and vertical and horizontal extent of the archaeological deposits present at 13DW137, including site deposits that are likely to extend above the anticipated shoreline elevation of 897 feet amsl.

Site 13DW138

Site Name:	None
Map Source:	USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description:	**(**Confidential)
Site Type:	Artifact Scatter
Cultural/Temporal Association:	Prehistoric
Site Size:	30 Meters (N-S) X 10 Meters (E-W)
Phase I Methods:	Pedestrian Survey
Area Excavated:	N/A
Cultural Materials Collected:	None;
Cultural Materials Observed:	Early-Stage Bifaces, Debitage
Landform:	High Terrace Slope
Elevation:	890-897 Feet Above Mean Sea Level
Land Use/Surface Visibility:	Eroded Shoreline (50-75% Surface Visibility)
Soil Type:	Lilah Sandy Loam**,** 2-9% Slopes
Site Disturbance:	Soil Erosion; Gullied Slope
Relation To Study Area:	Boundary Is Not Fully Established; Site Likely Extends Above Shoreline

NRHP Eligibility: Not Evaluated
Recommendations: Additional Phase I Testing; Bank Stabilization if Warranted

Site 13DW138 was identified by LBG based on field investigations performed on September 30, 2011. The site is represented by a light scatter of prehistoric chipped-stone artifacts that include early to middle-stage bifaces, flakes, and other by-products of stone-tool manufacture (Plate 23). Artifacts were observed on the eroded terrace slope below the surface of a high terrace. No temporally diagnostic artifacts were observed, but pieces of chert debitage are present on the eroded slope below the former lake shoreline. Most of these items appear to be associated with early-stage biface shaping and reduction and all of the items observed appeared to be manufactured from local variations of untreated or un-heat-altered Hopkinton chert.

Plate 23. Chipped-Stone Artifacts from Site 13DW138.

No subsurface tests were excavated at the site. Artifacts were observed below the outer edge of the high terrace below the former shoreline in disturbed contexts; however, it is reasonable to assume that more extensive site deposits extend onto the adjacent high terrace above the shoreline. The potential for undisturbed or intact archaeological deposits along the shoreline exists at the 895 to 897 elevation level and above; however, since most of the shoreline adjacent to this site is already stabilized by two to three-foot tall concrete retaining walls; the potential for continued erosion of in-situ archaeological site deposits at this location appears to be minimal. Subsurface testing on the adjacent terrace surface above the shoreline would be needed to determine the nature and extent of the site deposits at 13DW138.

Site 13DW138 remains unevaluated for listing in the NRHP. The high terrace adjacent to the site is occupied by closely spaced residential structures, but there are also extensive back yard areas adjacent to the shoreline that have potential to be largely undisturbed; therefore, there is potential for well preserved archaeological deposits to be present in the residential yard areas adjacent to the site. Based on these observations LBG recommends that additional archaeological investigations be conducted to define the nature and vertical and horizontal extent of the archaeological deposits present at 13DW138, including site deposits that are likely to extend above the anticipated shoreline elevation of 897 feet amsl. Even though the shoreline adjacent to the present site materials has already been stabilized, it is possible that undisturbed site deposits may extend to other portions of this terrace landform that are unprotected.

Site 13DW139

Site Name:	None
Map Source:	USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description:	(Confidential)
Site Type:	Artifact Scatter
Cultural/Temporal Association:	Prehistoric
Site Size:	30 Meters (E-W) X 15 Meters (N-S)
Phase I Methods:	Pedestrian Survey
Area Excavated:	N/A
Cultural Materials Collected:	None;
Cultural Materials Observed:	Middle-Stage Biface, Chert Debitage
Landform:	High Terrace
Elevation:	890-897 Feet Above Mean Sea Level
Land Use/Surface Visibility:	Eroded Shoreline (80-90% Surface Visibility)
Soil Type:	Lilah Sandy Loam, 0-2% Slopes
Site Disturbance:	Soil Erosion
Relation To Study Area:	Boundary Is Not Fully Established; Site Likely Extends Above Shoreline
NRHP Eligibility:	Not Evaluated
Recommendations:	Additional Phase I Testing; Bank Stabilization if Warranted

Site 13DW139 was identified by LBG based on field investigations performed on September 30, 2011. This site is located on the same high terrace point bar landform as 13DW138, but is situated almost 200 meters downstream. Site 13DW139 is represented by a moderate scatter of prehistoric chipped-stone artifacts that included a middle-stage biface, thinning flakes, and other probable by-products of stone-tool manufacture. Artifacts were observed on the eroded portion of the high terrace below the former shoreline. No temporally diagnostic artifacts were observed, but at least one shaped and thinned biface blank (Plate 24) and numerous pieces of chert debitage were noted on the sandy terrace surface. Most of these items appear to be manufactured from local variations of untreated Hopkinton chert.

No subsurface tests were excavated at the site. Artifacts were observed on an eroded extension of the high terrace surface that had previously been submerged by the lake. The artifacts were concentrated in an area measuring approximately 30 by 10 meters in extent that had the appearance of an outlier or eroded terrace lag deposit surrounded by sandy outwash. The presence of these finds on the same landform as 13DW138 suggests that there may be a much more extensive site area farther inland and that both 13DW138 and 13DW139 may be marginal expressions of that larger site. In any case, it is reasonable to assume that more extensive site deposits extend onto the adjacent high terrace above the shoreline in the immediate vicinity of Site 13DW139. The potential for undisturbed or intact archaeological deposits along the shoreline exists at the 890 to 897 elevation level and above. Similar to the situation at Site 13DW138, there are short retaining wall structures along the shoreline adjacent to 13DW139 and the potential for continued erosion of in-situ archaeological site deposits above the shoreline at this location appears to be minimal. Subsurface testing on the adjacent terrace surface above the shoreline would be needed to determine the nature and extent of the site deposits at 13DW139 and whether they may be linked to the site area at nearby 13DW138.

Site 13DW139 remains unevaluated for listing in the NRHP. The high terrace adjacent to the site is occupied by closely spaced residential structures, but there are also extensive back yard areas adjacent to the shoreline that have potential to be largely undisturbed; therefore, there is potential for preserved archaeological deposits to be present in the residential yard areas adjacent to the site. Based on these observations LBG recommends that additional archaeological investigations be conducted to define the nature and vertical and horizontal extent of the archaeological deposits present at 13DW139, including site deposits that are likely to extend above the anticipated shoreline elevation of 897 feet amsl. Even though the shoreline adjacent to the present site materials has already been stabilized, it is possible that undisturbed site deposits may extend to other portions of this terrace landform that are unprotected.

Plate 24. Middle-Stage Biface from Site 13DW139.

Site 13DW140

Site Name:	Lake Delhi Fish Weir
Map Source:	USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description:	**(**Confidential)
Site Type:	Fish Weir
Cultural/Temporal Association:	Prehistoric
Site Size:	20 Meters (N-S) X 20 Meters (E-W)
Phase I Methods:	Pedestrian Survey
Area Excavated:	N/A
Cultural Materials Collected:	None;
Cultural Materials Observed:	Constructed Rock Barrier
Landform:	Maquoketa River Channel and Adjacent Holocene Terrace
Elevation:	885 Feet Above Mean Sea Level
Land Use/Surface Visibility:	Eroded Shoreline (100% Surface Visibility)
Soil Type:	Spillville Loam, 0-2% Slopes (Adjacent River Bank)
Site Disturbance:	Channel Erosion; Current May Have Removed Part of the Site
Relation To Study Area:	Site is 100% Within the Study Area
NRHP Eligibility:	Not Evaluated
Recommendations:	Detailed Mapping and Documentation

Site 13DW140 was identified by LBG during field reconnaissance on September 30, 2011. The site is represented by an angled alignment of cobble-size natural rock that extends into the present channel of the Maquoketa River. At the present water level (elevation 884-885 feet amsl), the rock alignment protrudes about 6 to 12 inches above the surface. The exposed portion of the alignment that extends into the river channel has a uniform width that measures approximately 4 to five feet across. This segment of the alignment is about 35 feet long and extends at an acute angle into the river channel in a downstream direction. At the upstream end of this alignment, there is a 15-to-20-foot-long dog-leg extension that angles back toward the left bank of the river. The overall dimensions of this bank alignment are variable depending on where one chooses to measure, but the concentration of rock expands from a width of about 6 feet where it joins the upstream end of the channel alignment to an approximate width of 20 feet across where it merges with the terrace sediments on the left bank.

Similarly constructed rock alignments have been documented on the Iowa and Wapsipinicon rivers in eastern Iowa (Jones 2003). The structure located on the Iowa River near Amana, is known as the Amana Fish Weir (13IW100) and was listed on the NRHP in 1988. The Amana Fish Weir is constructed of rock cobbles similar to the alignment described above, but consists of two curved alignments that meet in the middle of the river channel to form a V-shape with the point oriented downstream. It was constructed by Native American tribes and is believed to have functioned as a fish trap. Similar structures, described in historical accounts, typically included wooden poles embedded in rock along with woven branches or brush to form a fencelike barrier across rivers or streams. The structures were designed to allow fish to move upstream by passing through or over the barrier during periods of high water, but it would then serve as a barrier to movement downstream. In the case of V-shaped structures, the barrier might also function to funnel fish toward a central opening where a net or some other form of collection device or even a spearing station might be used (Jones 2003:88).

The rock alignment observed within the study area appears to include only rock. No wood poles or other evidence of an associated structure was observed at the site and no artifacts of any type, prehistoric or historic, appeared to be present or associated. The setting for this structure strongly suggests that it may be part of a prehistoric fish weir. It is situated in a portion of the valley that has a bedrock wall on the outside meander to create a stable shoreline and the talus materials that have accumulated along the base of that slope would have provided a close and convenient supply of building materials. The structure has been submerged for the past 80 years so it is not of recent construction, and there are no records of historic activity in this part of the valley that might indicate 19th century construction of a small overflow dam in this area. The overall dimensions of the alignment are comparable with other barrier structures located on the Wapsipinicon River (e.g., Slide Rock Fish Weir – 13JN313) and the size of rocks used to assemble the Lake Delhi structure are comparable to those used to construct other prehistoric fish weirs, i.e., small enough to be carried by hand

Site 13DW140 remains unevaluated for listing in the NRHP; however, it appears to be a strong candidate to be the first prehistoric fish weir reported on the Maquoketa River. The structure appears to be incomplete, but it is possible that a companion alignment extending from the right bank, if it existed, may have been removed during the 2004-2005 dredging operations that took place within the river channel. Based on the observations described above, LBG recommends that additional archaeological investigations be conducted to carefully map and document the rock alignment that remains. These investigation may include exploration to determine if any evidence exists for wood pole inclusions that might help confirm its design and purpose.

Plate 25. Photograph of Site 13DW140. View looking downstream.

Site 13DW141

Site Name:	None
Map Source:	USGS Earlville, IA, 7.5' Series Topographic Quadrangle (1973)
Legal Description:	**(**Confidential)
Site Type:	Rockshelter
Cultural/Temporal Association:	Prehistoric
Site Size:	20 Meters (N-S) X 20 Meters (E-W)
Phase I Methods:	Pedestrian Survey
Area Excavated:	N/A
Cultural Materials Collected:	None;
Cultural Materials Observed:	Burned rock, Debitage
Landform:	Upland Slope
Elevation:	897-925 Feet Above Mean Sea Level
Land Use/Surface Visibility:	Eroded Shoreline (90-100% Surface Visibility)
Soil Type:	Nordness Rock Outcrop, 25-60% Slope
Site Disturbance:	Soil Erosion at Shoreline
Relation To Study Area:	Site Deposits Eroded at Shoreline; Site Extends Above Shoreline
NRHP Eligibility:	Not Evaluated
Recommendations:	Additional Phase I Testing; Bank Stabilization if Warranted

Site 13DW141 is a rockshelter formation identified by LBG based on field investigations performed on September 28, 2011. The shelter is located above the 897 contour identified as the study area boundary; however, it appears that archaeological deposits may extend beyond the dripline in front of the shelter and down to the proposed lake level where they have been subject to past shoreline erosion. Chipped-stone artifacts and small fragments of burned rock were observed eroding from the exposed face of the shoreline deposits. The floor of the shelter is situated an estimated five feet above the former shoreline.

Plate 26. Overview of Rockshelter Interior at 13DW141.

Plate 27. Photograph of Eroded Shoreline at 13DW141.

The shelter is small but has an estimated floor space of about 40 square meters (Plates 26, 27). The ceiling is approximately 7 to 8 feet above the current floor elevation providing ample space for use as a habitation site. No subsurface tests have been excavated at the site and the floor deposits within the protected portion of the shelter do not appear to have been disturbed. No carvings, petroglyphs, or mineral stains were observed on the walls of the shelter and no features were observed on the floor. Some reddish discoloration was noted on the back wall of the shelter which suggests oxidation from fire and heat. No artifacts were observed on the floor of the shelter, but several pieces of chipped stone debitage and small fragments of burned rock were observed eroding from deposits in front of the shelter within approximately two to three feet of the former high water mark for Lake Delhi.

Site 13DW141 remains unevaluated for listing in the NRHP. Site 13DW141 is located above the 897 foot elevation contour which has been identified as the upper limit for consideration of the proposed project's potential for adverse effects. However, it appears that the potential for below-ground archaeological deposits associated with Site 13DW141 may extend into or very near this zone of effect. Based on the field observations described above including the presence of associated archaeological materials at or near the previous high water mark for Lake Delhi (see Plate 27), LBG recommends that this site be included in the assessment of the project's potential effects on archaeological resources. LBG further recommends that additional archaeological investigations be conducted to define the nature and extent of the archaeological deposits present at 13DW141 including those located outside the dripline in front of the shelter.

VI. SUMMARY AND RECOMMENDATIONS

A. SUMMARY OF SURVEY FINDINGS

LBG has completed a cultural resource assessment and Phase I reconnaissance survey for the Lake Delhi study area. A records search for the area revealed that the area had received very little attention from professional archaeologists prior to the embankment failure at the Delhi Dam in July 2010. However, recent historical and archaeological investigations following this event have identified a number of potentially significant archaeological resources within the drained impoundment area including a variety of prehistoric Native American sites and several sites associated with the valley's earliest non-Indian settlement at Hartwick.

The study area's potential to contain historically or scientifically significant archaeological sites is closely linked to its geological history which not only created an environment attractive for human settlement, but also provides a basis for understanding where that history is most likely to be preserved in the form of archaeological sites. The study area is situated in a gorge-like section of the Maquoketa River valley, and like Backbone State Park upstream and Pictured Rocks County Park downstream, the exposed bedrock, pronounced stream meanders, diverse natural resources, and picturesque setting found in this part of the river valley have offered unique resource and settlement opportunities to human populations for thousands of years. The research findings reported in this study show that Native American cultures have used the valley's stone resources for at least 8000 years. They settled on many of the same high terrace landforms that the area's modern residents also find attractive. Evidence of buildings established by some of the valley's first European-American settlers have also been re-discovered.

The present study was not intended to locate all of the archaeological sites that exist within the lake. Its goal was instead to provide a broader view of the overall situation and based on that to offer some guidance for moving forward with plans to recreate Lake Delhi in a manner that takes into account its potential to have additional adverse effects to the valley's known and unknown archaeological resources.

Without investigating some of the archaeological sites located within the former impoundment area in more detail, i.e., through testing or excavation, it is difficult to know how the deposits may have been affected by placing them under water for the past 80 years. In many places, this action has likely resulted in many sites being covered with a blanket of silt that one might argue has actually provided them with added protection. Dredging operations conducted prior to 2010 presumably damaged some resources. The precise location of these activities is unknown and difficult to judge at this point, but one possible casualty may have been the fish weir structure identified at Site 13DW140. In any case, our review of the area suggests that the most significant damage to archaeological resources situated within the former impoundment area likely occurred in the immediate aftermath of the embankment failure as immense volumes of water drained from the valley causing surface scour on the upstream portions of terrace landforms and downcutting through terrace sediments as the river reclaimed its former channel. Significant erosion of the exposed terrace landforms has also continued in the months since the failure event as rain and storm-water runoff have resulted in extensive sheet erosion and the development of rills and deep gullies along the outer margins of virtually all the high terrace landforms. These conditions are actively affecting archaeological deposits at several reported sites, particularly those located in the lower half of the study area (i.e., 13DW123, 13DW124, 13DW126, 13DW133, 13DW134). Channel erosion of adjacent terrace landforms was also noted as a potential threat to terrace landforms with known and potential unreported archaeological sites, particularly the Holocene age terrace near Hartwick.

As a result of this study, we can confirm the expectation that there are indeed a large number of prehistoric archaeological sites within the impoundment area. Virtually every high terrace landform with exposed surfaces that we inspected within the valley shows evidence of prior use by prehistoric cultures. And although surface exposures were less common on early Holocene landforms within the valley, it is reasonable to assume from this site density that they likewise contain archaeological deposits. Most of the lower Holocene-age landforms are more thickly covered with deposits of alluvial sand and silt which limit their suitability for effective pedestrian survey. In most places, the only way to confirm the presence or absence of archaeological sites on these landforms would be to employ some form of intensive subsurface testing.

To date, we can confirm the presence of 12 archaeological sites within the impoundment area. These include eight new archaeological sites identified as a result of the current survey and four sites reported by previous investigators working near Hartwick in 2010. Ten of these sites contain evidence of past use by prehistoric Native American cultures as much as 8000 years ago. Most of these sites (7 of 10) appear to be open habitation areas or settlements (13DW123, 13DW124, 13DW133, 13DW134, 13DW137, 13DW138, 13DW139) while one is a smaller habitation site situated within a natural rock shelter (13DW141). Other sites include an apparent fishing site that includes a rare fish weir structure (13DW140) and a site used to secure and prepare stone used in the manufacture of everyday tools (13DW126). Mid-19th century building foundations are represented at two separate locations near the former townsite of Hartwick (13DW125, 13DW136) and are believed to be associated with the historic settlement that once existed at that location. Fragments of contemporary historic artifacts were also identified at two other nearby locations that also produced prehistoric artifacts (13DW123, 13DW126).

No prehistoric burial mounds were identified during the Phase I reconnaissance survey; however, the possibility exists that human remains could be interred at any of the habitation sites identified within the study area. Since human remains and human burials are protected by Iowa State Law (Chapters 516 and 716.5, Iowa Code), special consideration must be accorded these site areas in the event that additional site testing is conducted or if some of the protective measures recommended below are implemented at these sites.

None of the 12 known archaeological sites described in this report have been evaluated with regard to their eligibility for listing in the NRHP. No subsurface testing has yet been conducted at any of these sites and this type of investigation would be needed in each instance to gather the information necessary to make those determinations. At present, the reported boundaries of these sites are based solely on the distribution of artifacts exposed at the ground surface and many of the site boundaries are also truncated at the 897-foot elevation contour since no investigations were authorized outside the limits of the study area. In most instances, investigations outside this study area boundary would require subsurface testing because the land surfaces above the 897-foot elevation are obscured by lawns and other vegetation that limit the effectiveness of visual survey. Because of these limitations, the reported boundaries of these 12 sites may not coincide with the full horizontal extent of subsurface archaeological deposits present at these locations. Additional site investigations including subsurface testing would be necessary to delineate the full areal extent of archaeological site deposits in relation to the current project area and to assess their eligibility for listing in the NRHP.

B. RECOMMENDATIONS

In the event that this project qualifies as a federal undertaking, LBG recommends that the District consult with the responsible federal agency as advised by the Iowa SHPO (see Appendix B) to determine the scope of the federal undertaking and define an appropriate area of potential effect (APE). An APE is defined as “a geographic area within which an undertaking may directly or indirectly cause changes in the character of use of historic properties, if any such properties exist." (36 CFR § 800.16(d)). For the purposes of this preliminary investigation, the study area was defined as including all land surfaces upstream from the Delhi dam that are located at or below an elevation of 897 feet amsl; however, as explained in several of the site descriptions it may be necessary to conduct some site investigations above this elevation in order to properly evaluate sites situated on higher landforms whose margins may be affected by shoreline erosion.

Recommendations to fulfill Section 106 requirements are detailed below. These recommendations apply to the entire study area and include potential consideration of resource types other than archaeological sites.

- The *Lake Delhi Dam and Powerhouse Historic District* was recommended eligible for inclusion in the NRHP in March 2009. Key elements of this district were obviously damaged as a result of the embankment failure in July 2010. As a result, the National Register status of the district is in question. LBG recommends that the District consult with the appropriate federal agency and Iowa SHPO to seek an opinion on whether a re-evaluation of the property is warranted.
- LBG recommends that supplemental archaeological reconnaissance survey be completed for portions of the study area that were not inspected as part of the current investigation. The purpose of the survey should be to inspect landforms with high potential for exposed archaeological sites that may be subject to damage or disturbance by ongoing soil erosion. These areas include:
  - High terrace landforms located south of the Maquoketa River between Hartwick and Delhi Dam.
  - Early Holocene terrace landforms upstream from Linden Acres.
- LBG has recommended supplemental investigations at each of the 12 known archaeological sites described in this report. These recommendations are site specific and are intended to gather additional information regarding the nature, extent, and condition of the archaeological deposits present at each of these locations. This information may be needed in order for the federal agency to consult with the Iowa SHPO and other consulting parties regarding National Register eligibility and potential adverse effects associated with re-establishing Lake Delhi at its former levels. A summary of these recommendations is provided in Table 5.
- Pending completion of these supplemental investigations, LBG recommends that the project's consulting parties consider preparation of a cultural resource management plan designed to address any site-specific mitigation measures that may be required to minimize adverse effects to potential historic properties and monitor their long-term effectiveness.

It is important to note that no method of archaeological survey or testing is considered adequate to identify all potential archaeological resources that may exist in a given project area. Therefore, should any unrecorded archaeological resources be discovered during the course of the project construction, all ground disturbing activities in the vicinity of the discovery should be discontinued and the responsible federal agency, if any, and the State Historical Preservation Office (SHPO) at the Historic Preservation Bureau of the State Historical Society of Iowa (a Division of the Iowa Department of Cultural Affairs), should be notified and consulted regarding the need for further evaluation of the discovery. In the event that suspected human remains or unreported human burials are discovered during project construction, Iowa law (Iowa Code Chapters 263B and 716.5) requires that all construction activities in the immediate area be halted immediately pending notification of law enforcement authorities and/or the Office of the State Archaeologist as appropriate.

Table 5. Recommended Site Investigations

SITE #	SITE TYPE	RECOMMENDED INVESTIGATIONS
13DW123	Artifact Scatter	Conduct subsurface testing to sample site deposits and establish site boundaries; may require testing above 897 elevation
13DW124	Habitation	Conduct subsurface testing to sample site deposits and establish site boundaries; may require testing above 897 elevation
13DW125	Stone Foundation	Conduct subsurface testing to sample site deposits and establish site boundaries
13DW126	Resource Procurement (prehistoric); Artifact Scatter (historic)	Conduct subsurface testing to sample site deposits and establish site boundaries; may require testing above 897 elevation
13DW133	Habitation	Conduct subsurface testing to sample site deposits and establish site boundaries; will likely require testing above 897 elevation Consult with OSA prior to investigation of exposed rock feature
13DW134	Habitation	Conduct subsurface testing to sample site deposits and establish site boundaries; will likely require testing above 897 elevation
13DW136	Stone Foundation (Hartwick Saw Mill)	Conduct archival research & interview local historians Conduct subsurface testing within foundation to sample site deposits
13DW137	Habitation	Conduct subsurface testing to sample site deposits and establish site boundaries; will require testing above 897 elevation
13DW138	Artifact Scatter	Conduct subsurface testing to sample site deposits and establish site boundaries; will require testing above 897 elevation
13DW139	Artifact Scatter	Conduct subsurface testing to sample site deposits and establish site boundaries; will require testing above 897 elevation
13DW140	Fish Weir	Map and document rock alignment; Limited exploration to determine presence/absence of wood posts
13DW141	Rockshelter	Prepare detailed map of shelter interior Conduct limited subsurface testing to sample site deposits that may be affected by shoreline erosion; will require testing above 897 elevation

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APPENDIX A

National Archaeological Database Form (NADB)

Database Doc Number:

NATIONAL ARCHAEOLOGICAL DATABASE – REPORTS DATA ENTRY FORM

1. R & C #: __________
2. Authors: Randall M. Withrow

Year of Publication: 2011

3. Title: Archaeological Reconnaissance Survey of Lake Delhi, Delaware County, Iowa

----

4. Report
   - Title:
   - Volume #: ______ Report #: ______ NTIS: ______
   - Publisher:
   - Place:

----

5. Unpublished
   - Sent From: The Louis Berger Group, Inc.
   - Sent to: Lake Delhi Combined Recreation Facility and Water Quality District Trustees
   - Contract #:

----

6. Federal Agency:

7.

State:	Iowa	Iowa			
County:	Delaware	Delaware			
Town:	Delhi	Milo			

----

8. Work Type: 7 86

----

9. Keyword:

0-Types of Resources/Features
1-Generic Terms/Research Questions
2-Taxonomic Names
3-Artifact Types/Material Classes
4-Geogrpahic Names/Locations
5-Time Periods
6-Project Names/Study Unit
7-Other Key Words

450 acres	[7]	rockshelter	[0]
Hartwick Lake	[4]	Fishweir	[0]
Iowan Surface	[4]	Hartwick townsite	[0]
Maquoketa River	[4]		[ ]
Lake Delhi	[4]		[ ]
	[ ]		[ ]

----

10. UTM Zone:

	Easting:		Northing:	
	Easting:		Northing:	
	Easting:		Northing:	
	Easting:		Northing:	

11.

Township:	T88N	T88N			
Range:	R4W	R5W			

Other Publication Types:

12. Monograph
    Name
    :
    Place:

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13. Chapter: In: First: Last:

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14. Journal: Volume: Issue: First: Last:

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15. Dissertation:
    Degree: Ph.D. LL.D M.A. M.S. B.A. B.S. Institute:

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16. Paper: Meeting:
    Place: Date:

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17. Other:
    Reference Line:

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18. Site #:

13DW123	13DW141
13DW124	
13DW125	
13DW126	
13DW133	
13DW134	
13DW136	
13DW137	
13DW138	
13DW139	
13DW140	

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19. Quad Map:

Name:	Date:
Earlville, IA, 7.5' Series	1973
Manchester, IA, 7.5' Series	1973

APPENDIX B

Correspondence

STATE HISTORICAL SOCIETY of IOWA
A Division of the Iowa Department of Cultural Affairs

November 2, 2010

Ms. Pat Boddy, Interim Director
Iowa Department of Natural Resources
Wallace State Office Building
Des Moines, 50319

RE: COE – DELWARE COUNTY – LAKE DELHI – LAKE DELHI RESTORATION INITIATIE – IOWA SHPO RECOMMENDATIONS TO THE GOVERNOR'S TASK FORCE ON REBUILDING LAKE DELHI

Dear Ms. Boddy,

The State Historic Preservation Office has prepared the following for the consideration of the Environment Committee of the Lake Delhi Recover & Rebuild Task Force. It is our sincere hope that the Task Force will incorporate some or all of its content into its final report to the Governor.

Please feel free to contact Ms. Barbara Mitchell, Deputy State Historic Preservation Officer, at (515) 281-4013 or me at (515) 281-8744 if you have any questions or if you would care to discuss this further.

Sincerely,

Dan Higginbottom

Daniel K. Higginbottom, Archaeologist
Iowa State Historic Preservation Office

NATIONAL HISTORIC PRESERVATION ACT

There is every indication that implementation of the Lake Delhi restoration initiative will require some level of Federal involvement in order to accomplish its objectives. The infusion of Federal funds, the issuance of Federal permits or licenses, or the acceptance of Federal assistance will invoke agency compliance with Federal environmental and preservation laws including section 106 of the National Historic Preservation Act of 1966 (16 U.S.C. 470 *et seq.*)

Section 106 of the National Historic Preservation Act directs all Federal agencies to take into account the effects of their undertakings on historic properties. According to the Advisory Council on Historic Preservation's rules implementing section 106, an undertaking means "a project, activity, or program funded in whole or in part under the direct or indirect jurisdiction of a Federal agency, including those carried out by or on behalf of a Federal agency; those carried out with Federal financial assistance; those requiring a Federal permit, license or approval; and those subject to State or local regulation administered pursuant to a delegation or approval by a Federal agency" [36 CFR Part 800.16(y)].

In accordance with the Act, Federal agencies are obligated to complete the Section 106 compliance process *prior to* the approval of the expenditure of any Federal funds on an undertaking or *prior to* the issuance of any license or permit [36 CFR Part 800.1(c)]. With this in mind, the section 106 process requires consideration early in the planning process in order to be completed in a timely, efficient, and cost effective manner. In this respect, section 106 is much like the compliance processes established for other Federal environmental rules and regulations. Many Federal agencies have internal policies and

600 EAST LOCUST STREET, DES MOINES, IA 50319-0290 P: (515) 281-5111

procedures outlining how they approach Section 106 compliance and some maintain professional staff to deal specifically with historic preservation matters. Others do not.

The Maquoketa River drainage has a deep, rich, cultural past spanning a period of around 12,000 years. The area surrounding modern-day Lake Delhi would have been particularly attractive to prehistoric people owing to its plentiful riverine resources including local bedrock outcrops that yield a particularly high-grade of chert used prehistorically in the manufacture of stone tools. Systematic professional archaeological survey of the Lake Delhi basin and surrounding shoreline was not conducted prior to original construction so unfortunately no baseline information on local cultural resources is available. However, nodules of raw and quarried chert, along with the byproducts of stone tool manufacture, have been observed in rock bars along the floor of the drained lake. Of more recent historic interest are the abandoned town of Hartwick, features of which - including building foundations and associated artifact deposits - were inundated by the impoundment of Lake Delhi, and the Lake Delhi Dam and Powerhouse Historic District, which has been evaluated as eligible for listing in the National Register of Historic Places.

The Iowa State Historic Preservation Office (SHPO) offers the following recommendations in anticipation of a federal action(s) that would invoke Section 106 compliance.

Designation of a Lead Federal Agency. As project planning proceeds and federal sponsors are identified, it would be in the project's best interest to designate a lead federal agency in order to eliminate redundancy, improve efficiency and reduce the cost of project compliance. The Council's rules allow for the designation of a lead federal agency to serve on behalf of all agencies involved in fulfilling their collective responsibilities under section 106 [36 CFR Part 800.2(a)(1)]. The SHPO encourages this type of arrangement. However, it is ultimately the agencies' decision on whether or not they will participate in a lead or subordinate capacity.

Coordinate with other Reviews. The agency should coordinate the steps of the section 106 process, as appropriate, with the overall planning schedule for the undertaking and with any reviews required under other authorities such as the National Environmental Policy Act of NEPA, the Native American Graves Protection and Repatriation Act, the American Indian Religious Freedom Act, and the Archeological Resource Protection Act. This too will eliminate redundancy, improve efficiency and reduce costs associated with project compliance.

Early Consultation. The National Historic Preservation Act, along with other applicable federal authorities, require consultation with federally recognized Indian tribes in order to identify any concerns that they may have with respect to project impacts upon sites or objects of cultural patrimony. As recognized sovereign Nations, Tribes enjoy a unique status in the Federal review and compliance process. Consultation with an Indian tribe must recognize the government-to-government relationship between the Federal Government and Indian Tribes. The Act also requires the involvement of the State Historic Preservation Office in the consultation process and further directs the federal agency to identify and invite the consultation of other parties that may have a valid interest in historic properties affected by an undertaking. One such party is the University of Iowa, Office of the State Archaeologist (OSA), which has jurisdictional oversight of the Iowa Burial Laws and has statutory authority over Iowa's archaeological site records. The OSA frequently participates as consulting party on federal undertakings. Consultation is most successfully and expeditiously accomplished through early coordination and under the auspices of a lead agency that has procedures already in place and experienced staff to oversee their execution.

Identification and Evaluation of Historic Properties. 'Historic property' is defined as "any prehistoric or historic district, site, building, structure, or object included in, or eligible for inclusion in, the National Register of Historic Places.' The term also includes artifacts, records and remains that are related to and located with such properties and properties of traditional religious and cultural importance to an Indian tribe. The federal agency is obligated to make a reasonable and good faith effort to identify and evaluate historic properties within a project's area of potential effects (APE), assess the magnitude of project effects, and to consult on ways to avoid, minimize, or mitigate those effects found to be adverse. Some federal funding programs include historic properties investigations as an allowable expense, others do not. When selecting a cultural resource service consultant, it is advisable (but not required) to select a vendor with regional historical expertise, one that employs a multi-disciplinary staff with expertise in archaeology, history, architectural history, and geomorphology, possesses an understanding of applicable state and federal preservation law, and that has a demonstrated history of assisting their clients through the compliance process.

Programmatic agreement. The rules implementing section 106 are found at 36 CFR Part 800. However, agencies are permitted to develop alternative procedures to accommodate the special circumstances and needs of a project provided that those procedures are consistent with the Council's rules (36 CFR Part 800.14, Subpart C). Under its terms, a Programmatic agreement (PA), among other things, outlines roles and responsibilities of the signatories in meeting section 106 compliance, establishes procedures for consultation and action protocols in the event of unanticipated discovery, establishes procedures for historic property identification and evaluation; and outlines mutually-agreed upon mitigation measures. The advantages of a PA over the 36 CFR Part 800 rules - particularly for a complex projects such as Lake Delhi restoration, are numerous. A PA allows its parties to address multiple steps in the process at once and allows for a seamless transition from one step to the next. Coordination of the section 106 process with other federal authorities such as NEPA, can also be facilitated under the terms of a programmatic agreement. One of the principal advantages of a PA is the definition of types of activities for which all parties agree that no further consultation is necessary.

APPENDIX C

SITE MAPS

Provided in Volume II

CONFIDENTIAL

THIS APPENDIX CONTAINS CONFIDENTIAL INFORMATION REGARDING THE LOCATION OF ARCHAEOLOGICAL SITES AND, UNDER SECTION 304 OF THE NATIONAL HISTORIC PRESERVATION ACT OF 1966, IS NOT FOR PUBLIC DISTRIBUTION

Appendix F

Reconstruction Exhibits

Stanley Consultants INC.

PLAN

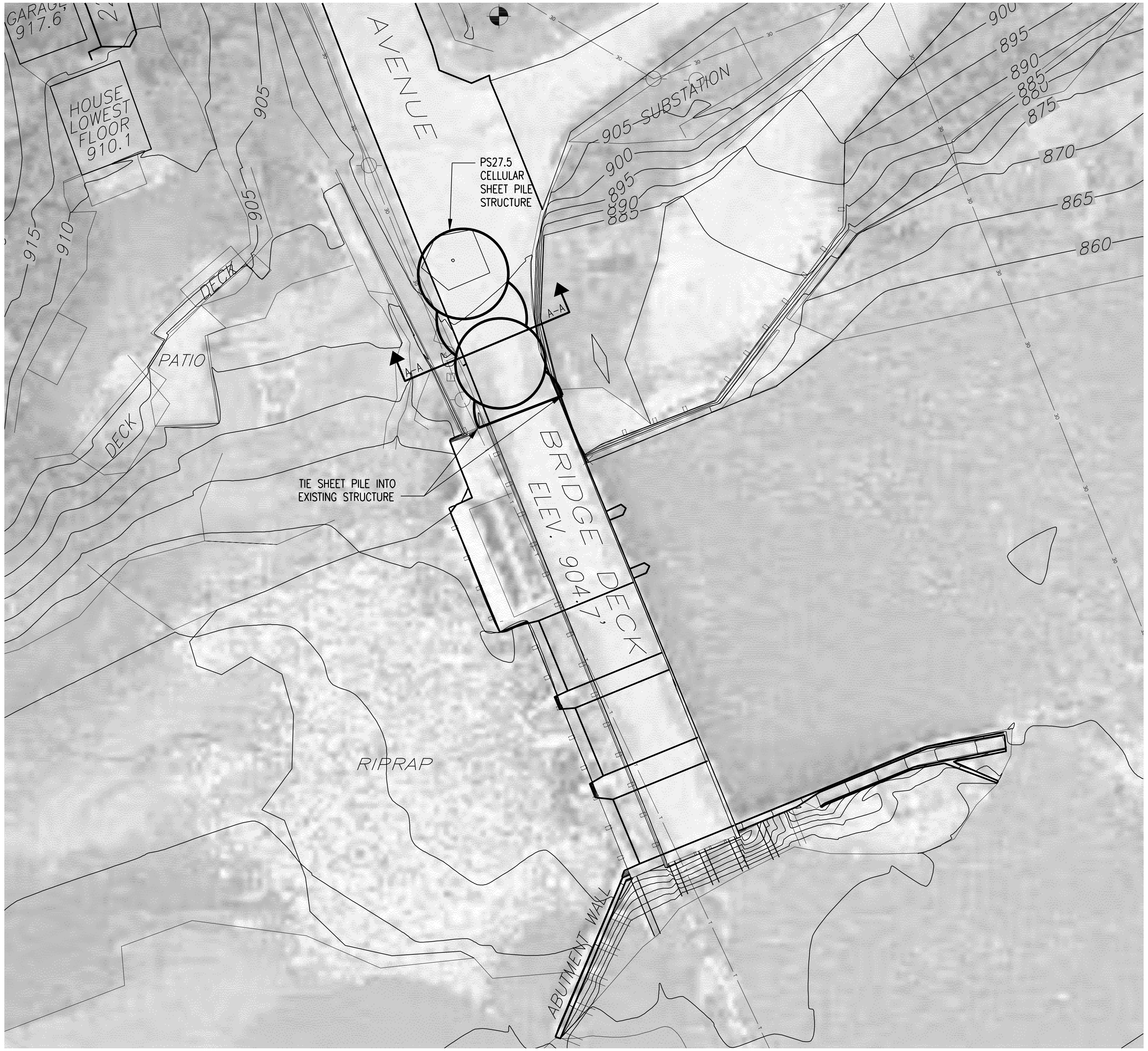
LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES

EXHIBIT 1 – EXISTING SITE

CADD D4-R3

Stanley Consultants INC.

NORTH



PLAN

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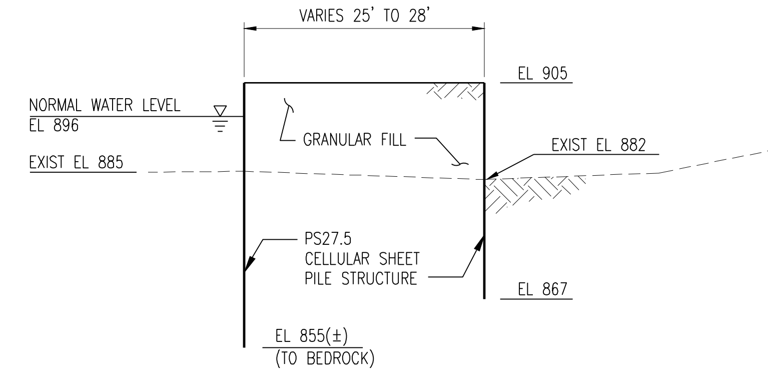
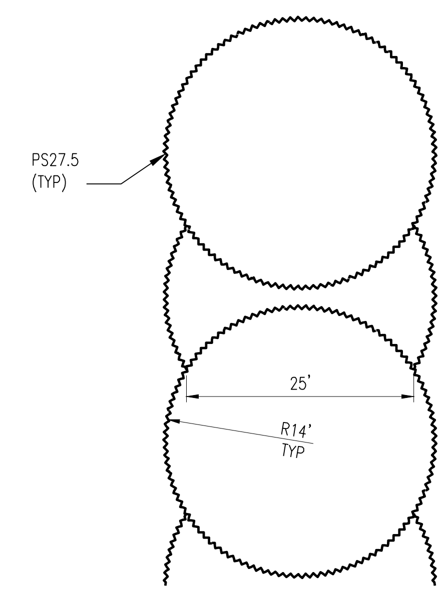
SCALE: 1" = 40'-0"



SECTION A-A



CELLULAR SHEET PILE STRUCTURE DETAIL

LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES

EXHIBIT 2
NORTH EMBANKMENT
CELLULAR SHEET PILE STRUCTURE
PLAN, SECTION, AND DETAIL

CADD D4-R3

Stanley Consultants INC.

NORTH

PLAN

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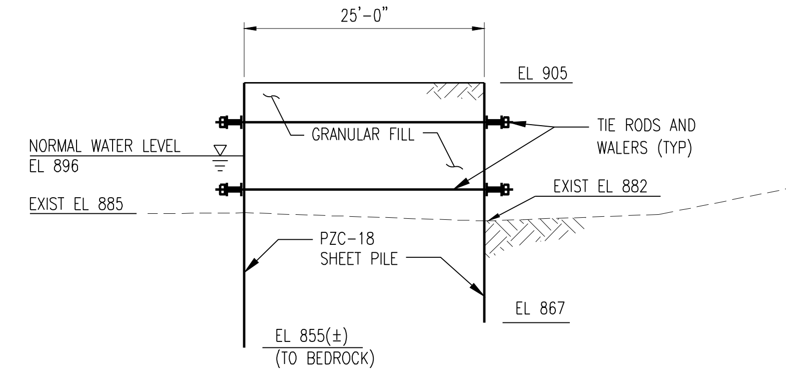
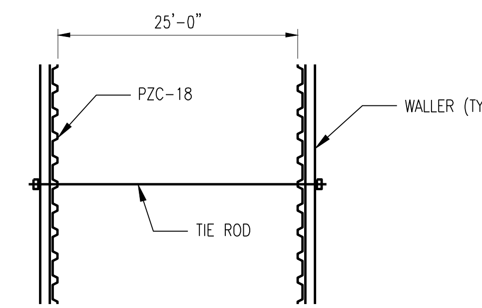


SECTION B–B



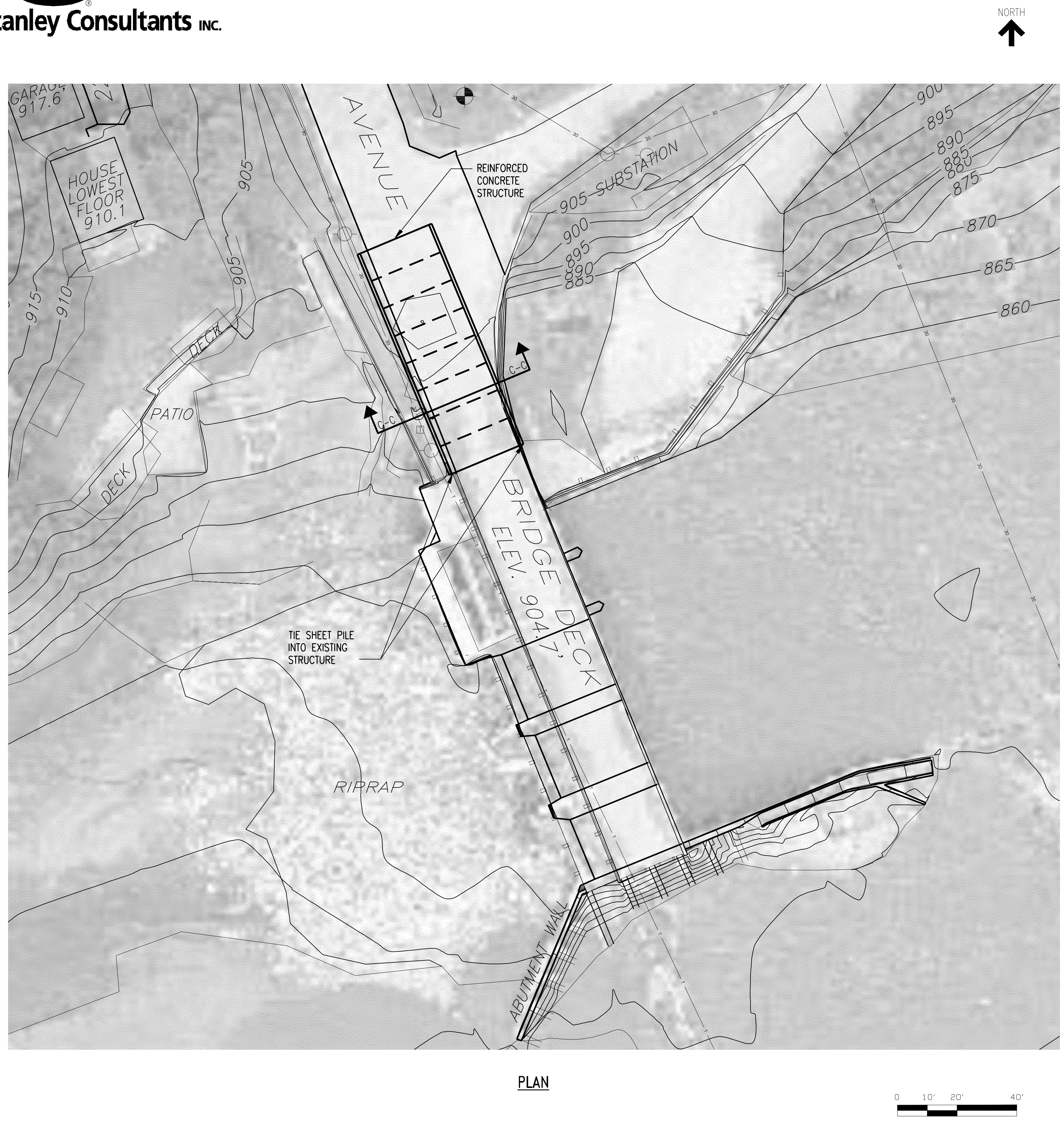
SHEET PILE DOUBLE WALL DETAIL

LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES

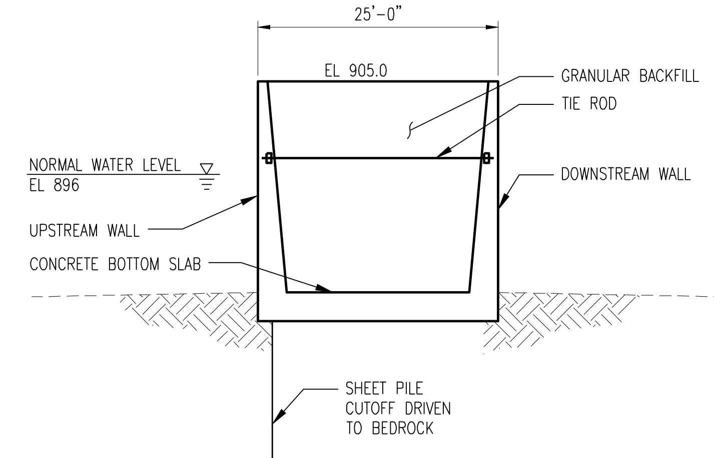
EXHIBIT 3
NORTH EMBANKMENT
SHEET PILE DOUBLE WALL
PLAN, SECTION, AND DETAIL

CADD D4-R3

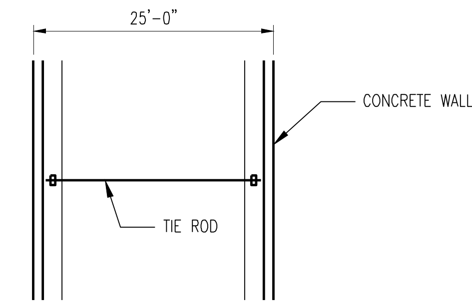
Stanley Consultants INC.



PLAN



SECTION C–C



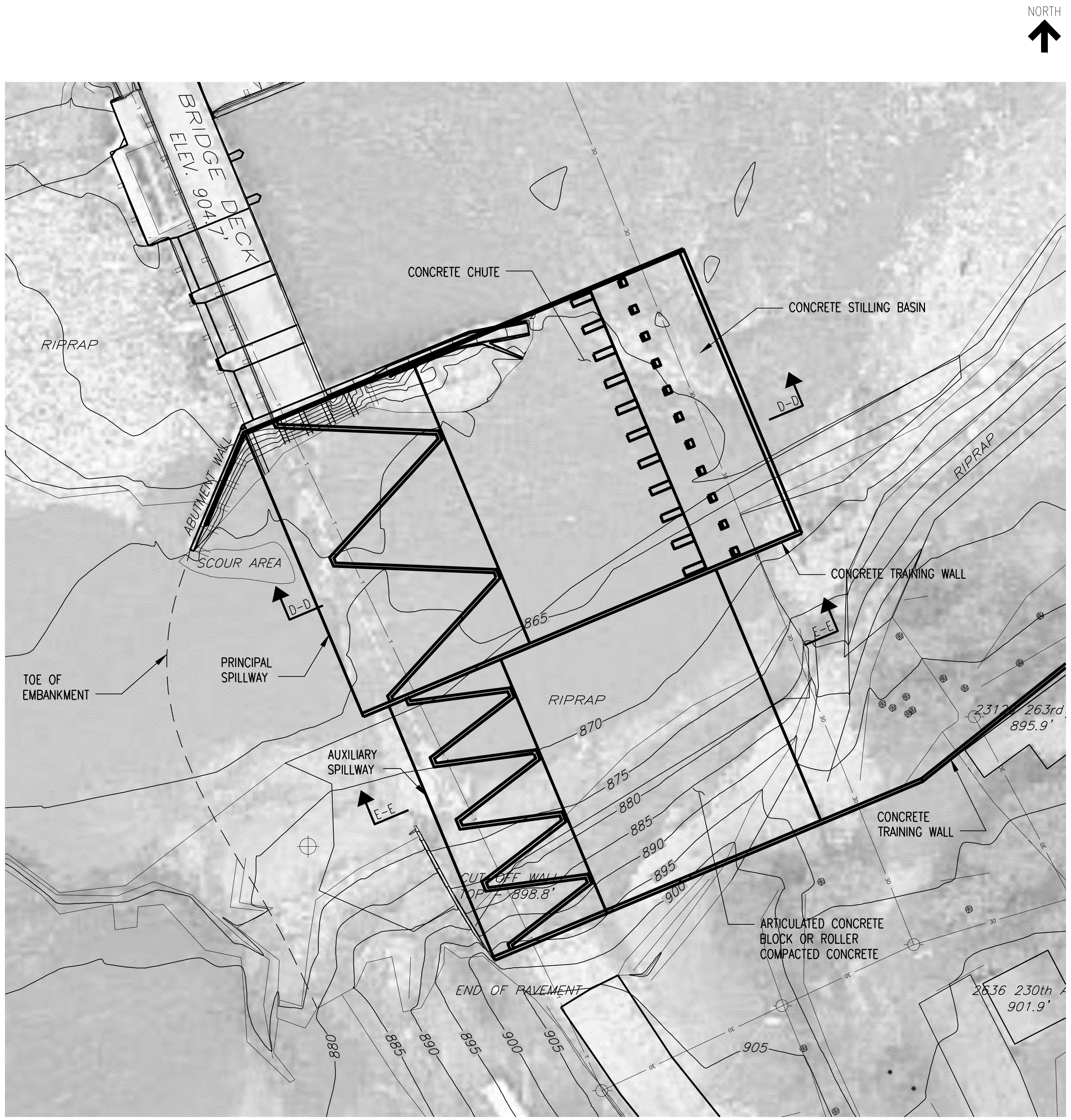
NORTH EMBANKMENT SECTION
REINFORCED CONCRETE WALLS

LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES

EXHIBIT 4
NORTH EMBANKMENT
REINFORCED CONCRETE STRUCTURE
PLAN, SECTION, AND DETAIL

CADD D4-R3

Stanley Consultants INC.



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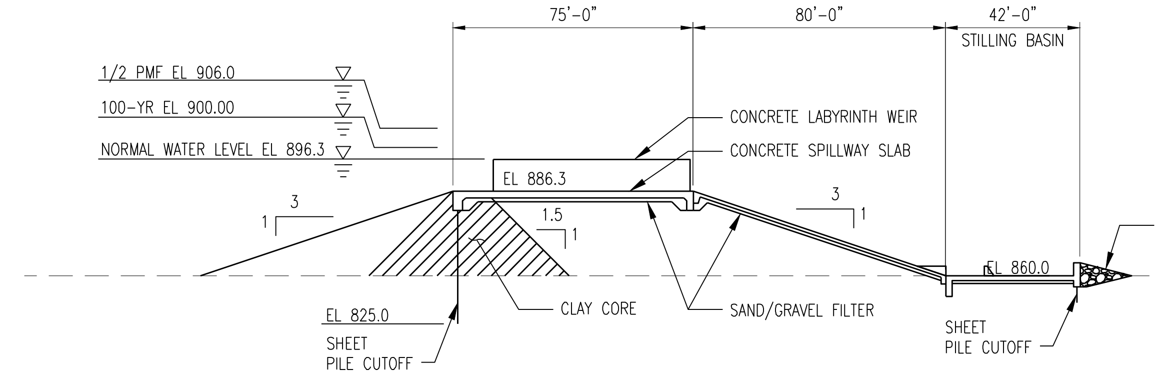
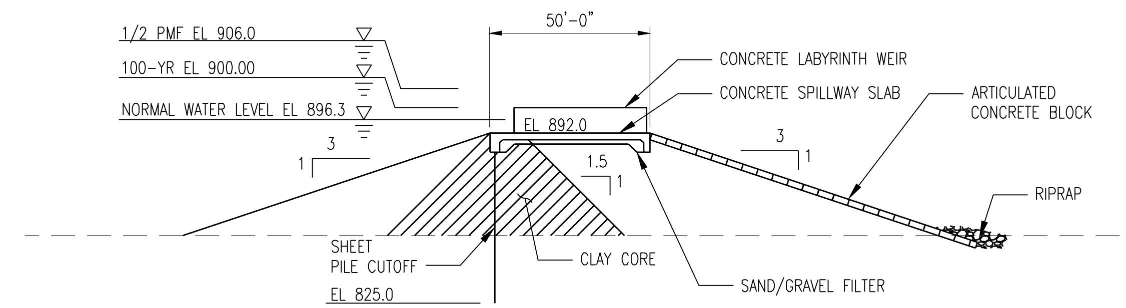
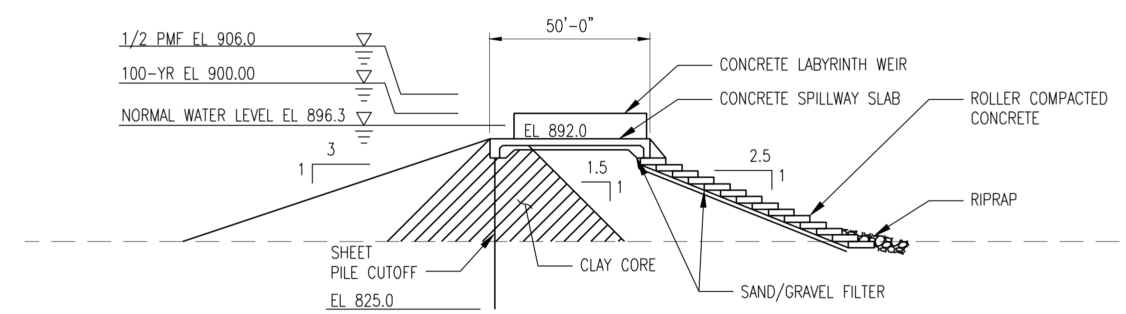
PLAN



PRINCIPAL SPILLWAY
SECTION D-D



AUXILIARY SPILLWAY
ARTICULATED CONCRETE BLOCK OPTION
SECTION E-E



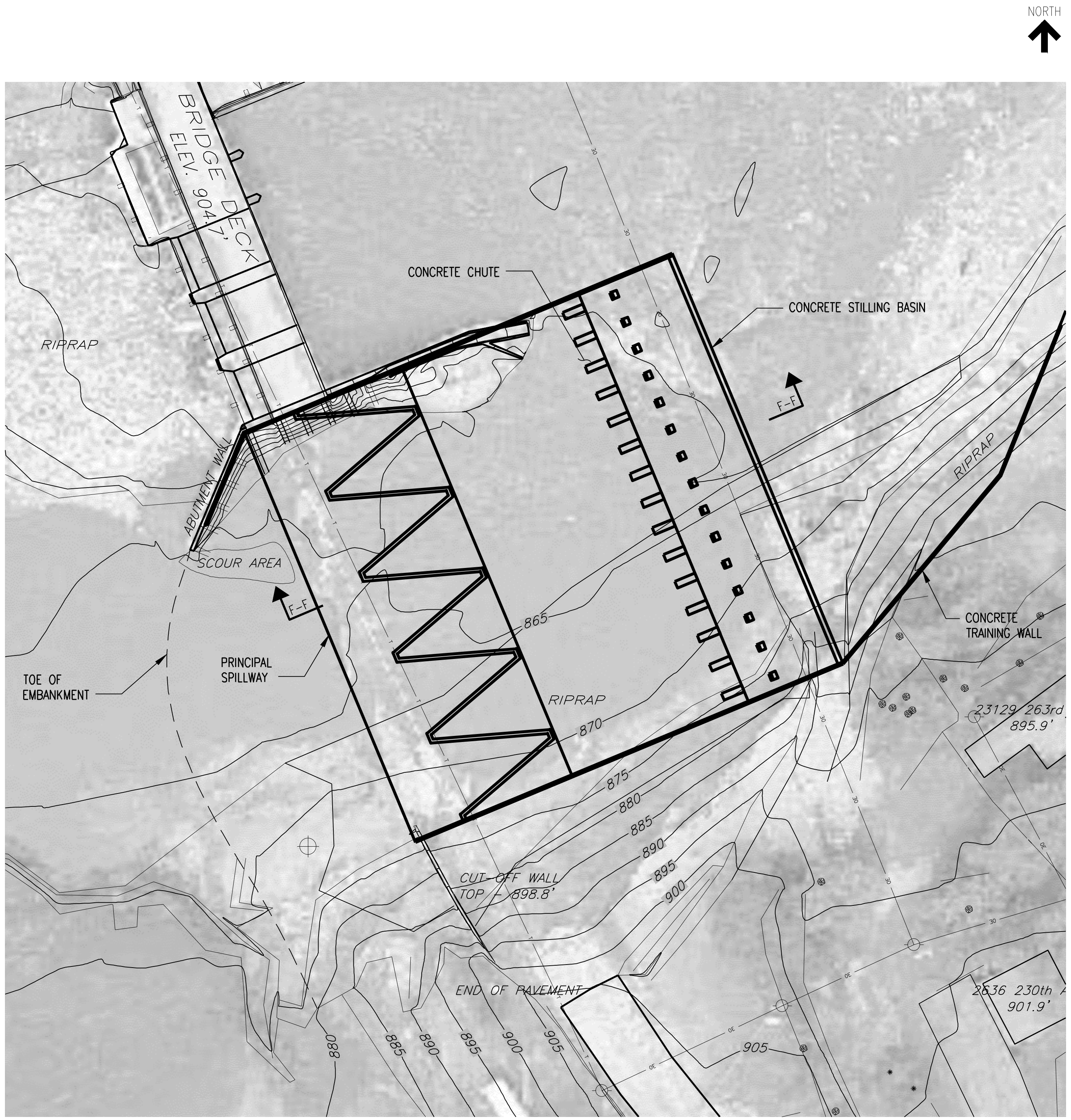
AUXILIARY SPILLWAY
ROLLER COMPACTED CONCRETE OPTION
SECTION E-E

LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES

EXHIBIT 5
DUAL LABYRINTH SPILLWAY

CADD D4-R3

Stanley Consultants INC.



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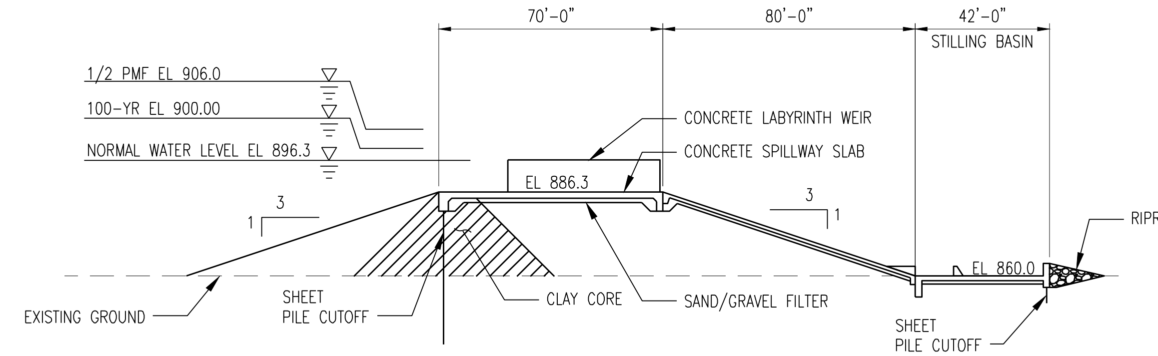
SCALE: 1" = 60'-0"

PLAN



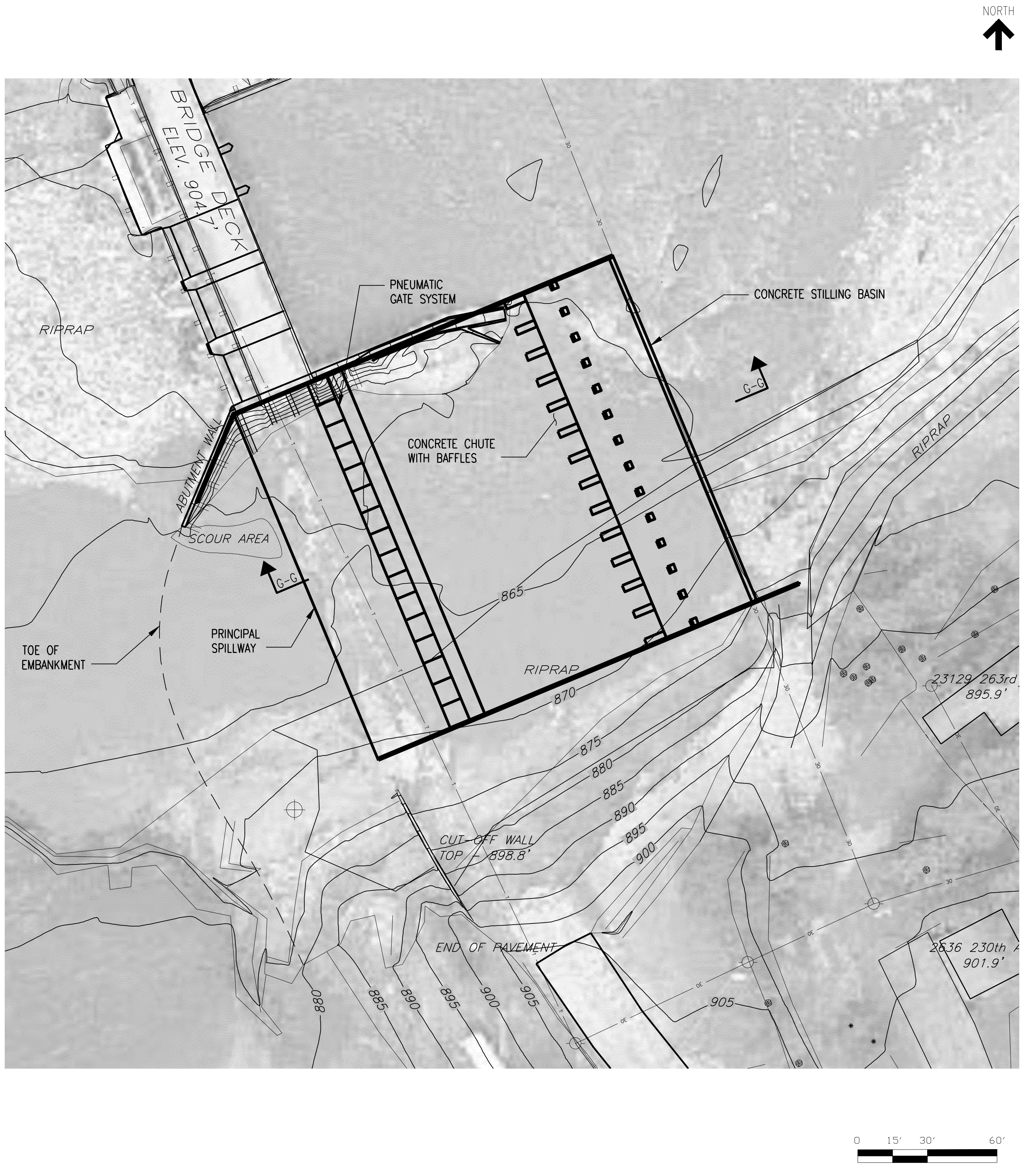
PRINCIPAL SPILLWAY
SECTION F-F

LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES

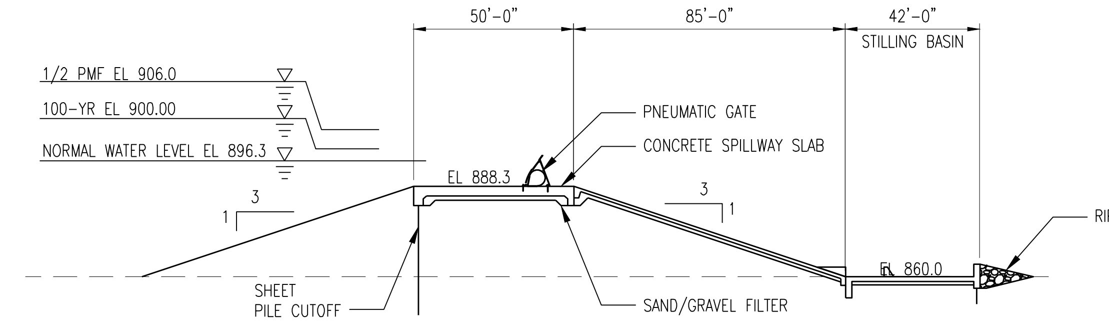
EXHIBIT 6
SINGLE LABYRINTH SPILLWAY

CADD D4-R3

Stanley Consultants INC.



PLAN



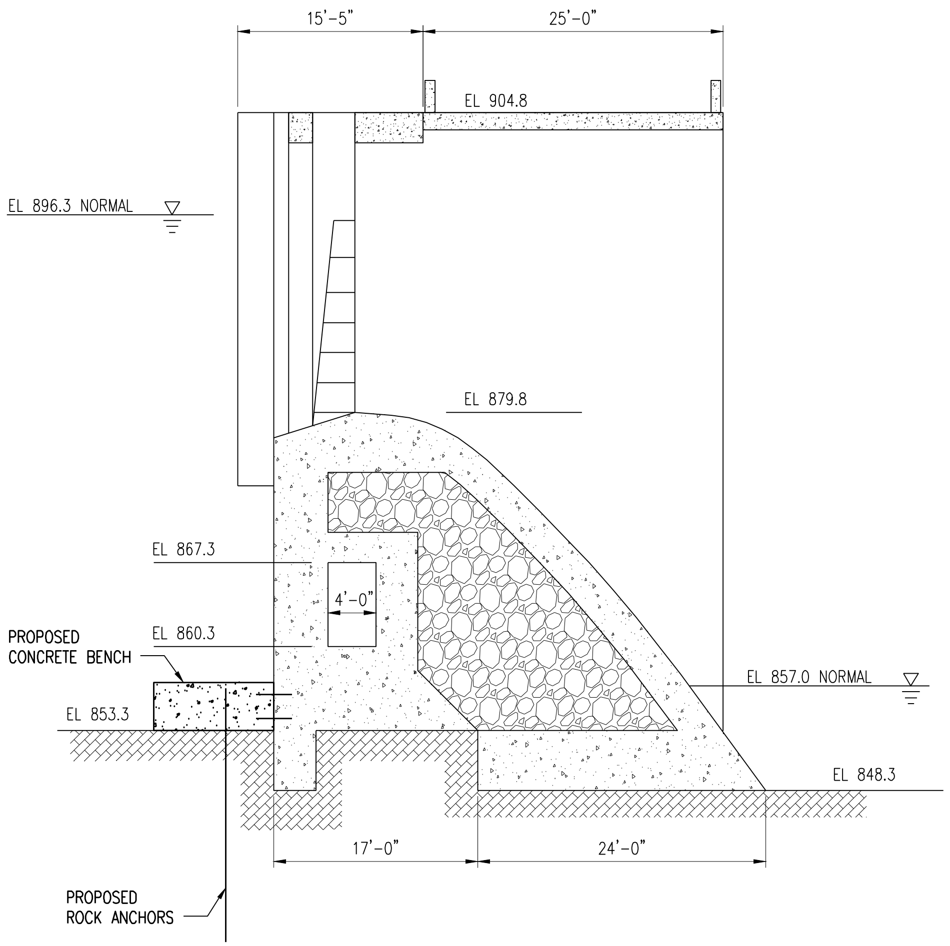
PRINCIPAL SPILLWAY
SECTION G-G

LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES

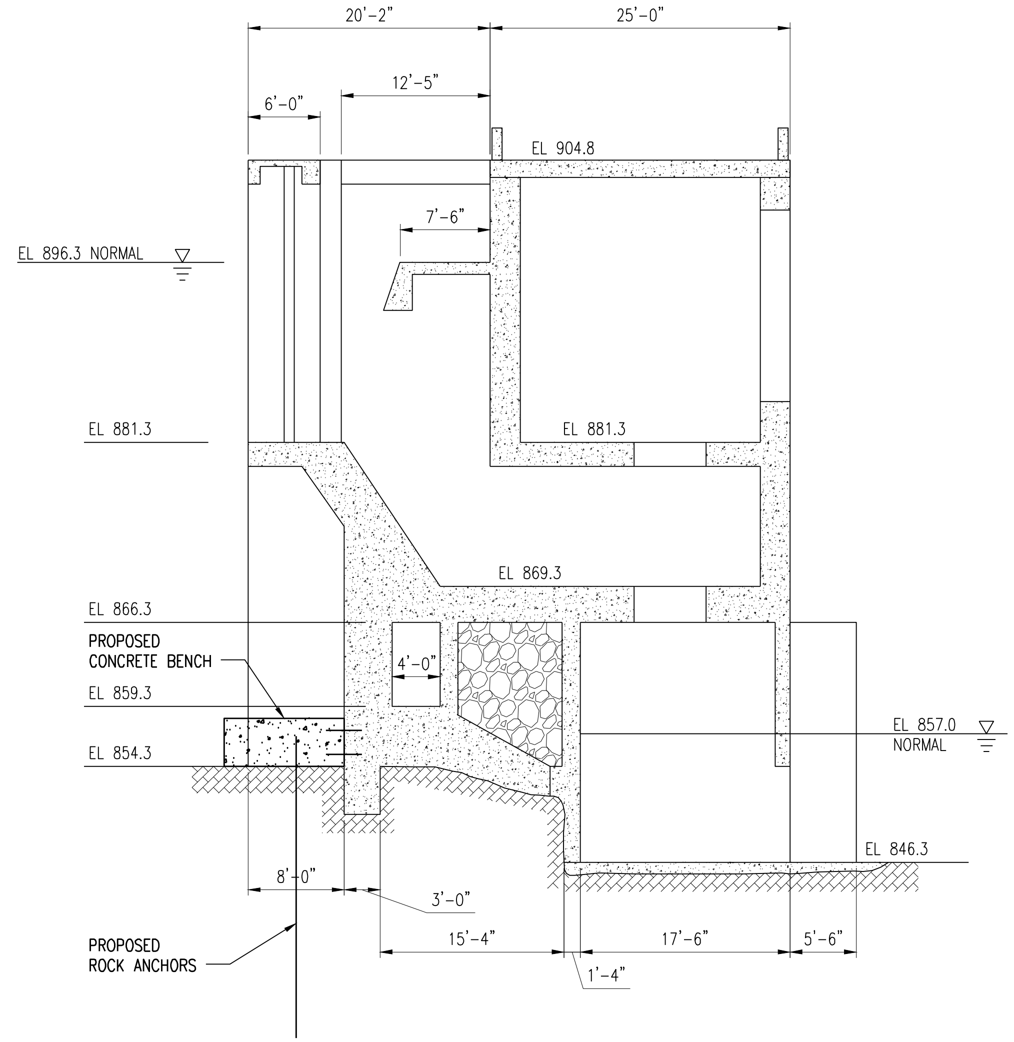
EXHIBIT 7
PNEUMATIC GATE SPILLWAY

CADD D4-R3

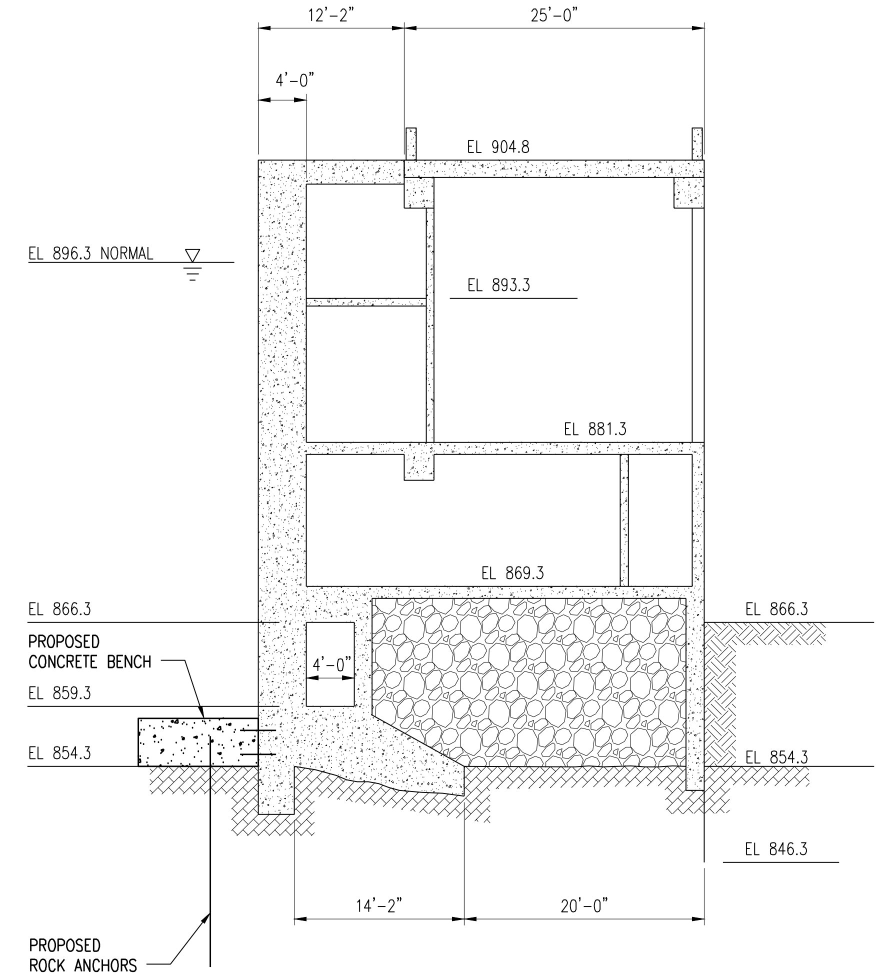
Stanley Consultants INC.



SPILLWAY SECTION



POWERHOUSE SECTION
(TURBINE CENTERLINE)

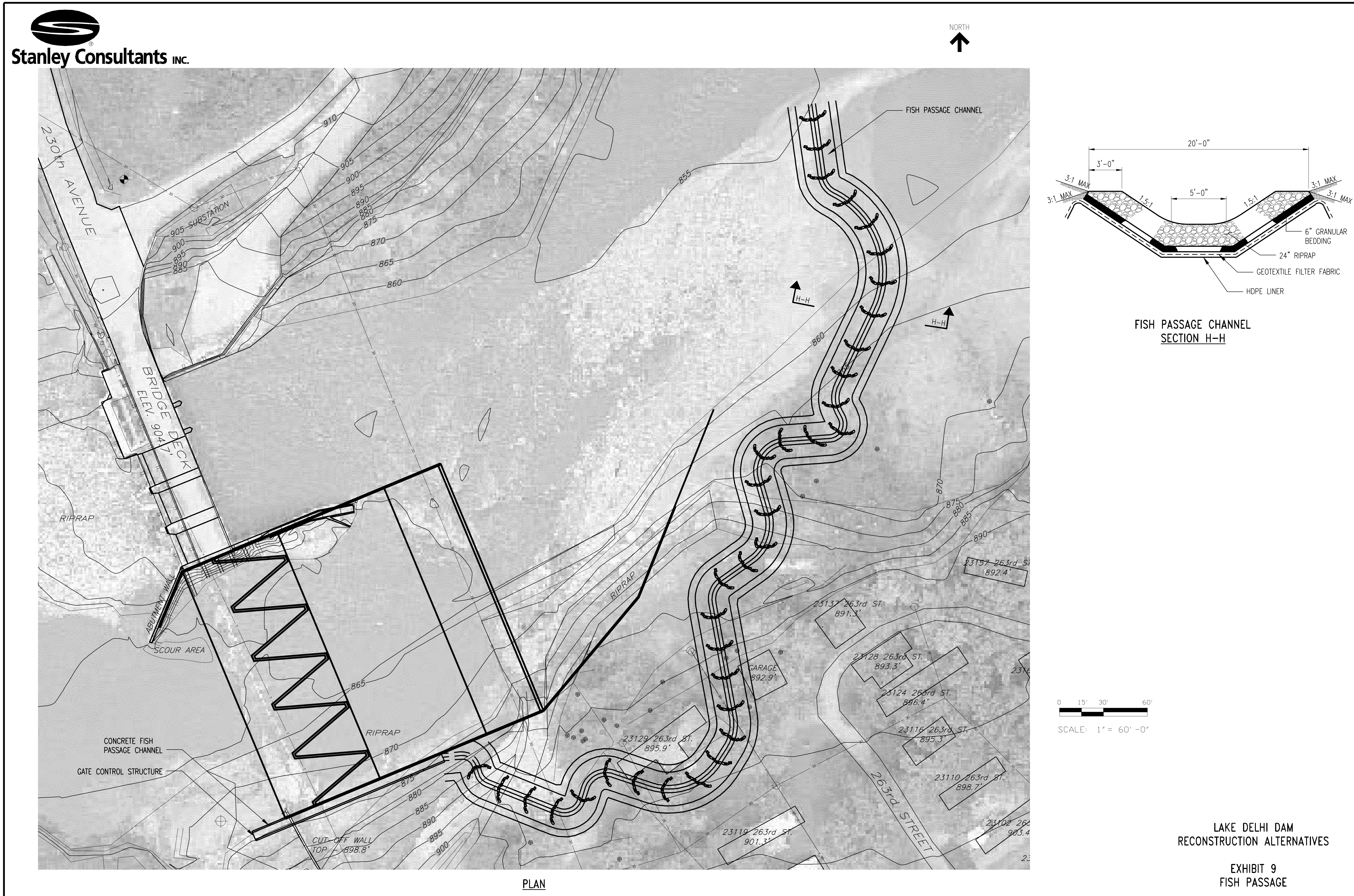


POWERHOUSE SECTION
(BOILER ROOM CENTERLINE)

CADD D4-R3

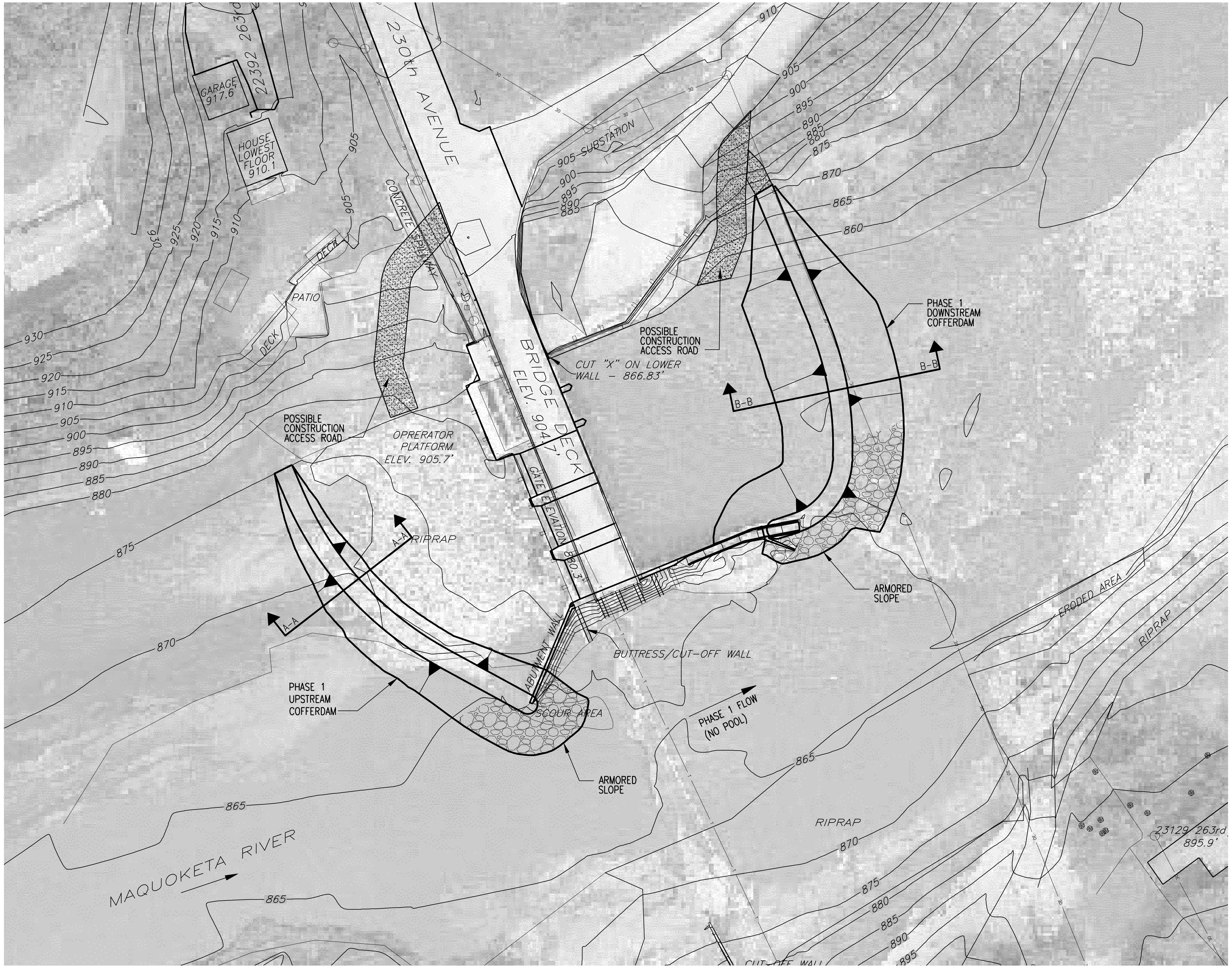
LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES

EXHIBIT 8
SPILLWAY AND POWERHOUSE
STABILIZATION SECTIONS


Stanley Consultants INC.
NORTH
FISH PASSAGE CHANNEL
20'-0"
3'-0"
5'-0"
3:1 MAX
1.5:1
6" GRANULAR BEDDING
24" RIPRAP
GEOTEXTILE FILTER FABRIC
HDPE LINER
FISH PASSAGE CHANNEL
SECTION H-H
230th AVENUE
905 SUBSTATION
BRIDGE DECK
ELEV. 904.7'
RIPRAP
ABUTMENT WALL
SCOUR AREA
CONCRETE FISH PASSAGE CHANNEL
GATE CONTROL STRUCTURE
CUT-OFF WALL TOP 898.8'
H-H
23157 263rd ST. 892.4'
23137 263rd ST. 891.3'
23128 263rd ST. 893.3'
GARAGE 892.9'
23124 263rd ST. 896.4'
23116 263rd ST. 895.3'
23129 263rd ST. 895.9'
23110 263rd ST. 898.7'
23119 263rd ST. 901.3'
263rd STREET
PLAN
SCALE: 1" = 60'-0"
LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES
EXHIBIT 9
FISH PASSAGE
CADD D4-R3

Stanley Consultants INC.

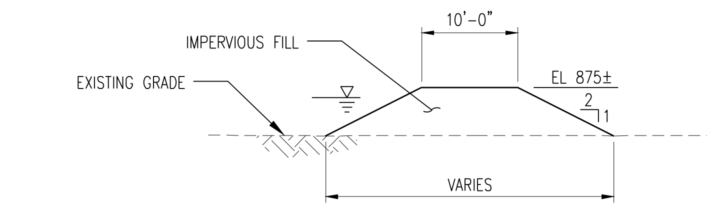
NORTH



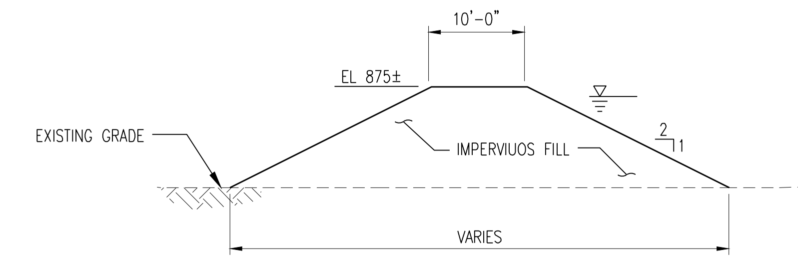
PLAN

0' 15' 30' 60'

SCALE: 1" = 60'-0"



SECTION A-A
PHASE 1 UPSTREAM COFFERDAM



SECTION B-B
PHASE 1 DOWNSTREAM COFFERDAM

NOTES:

1. OFF-SITE TEMPORARY CONSTRUCTION EASEMENTS MAY BE REQUIRED FOR CONSTRUCTION STAGING/ LAYDOWN AND STOCKPILE AREAS.
2. ARMOR SLOPES AT HIGH SHEAR LOCATIONS AS SHOWN.

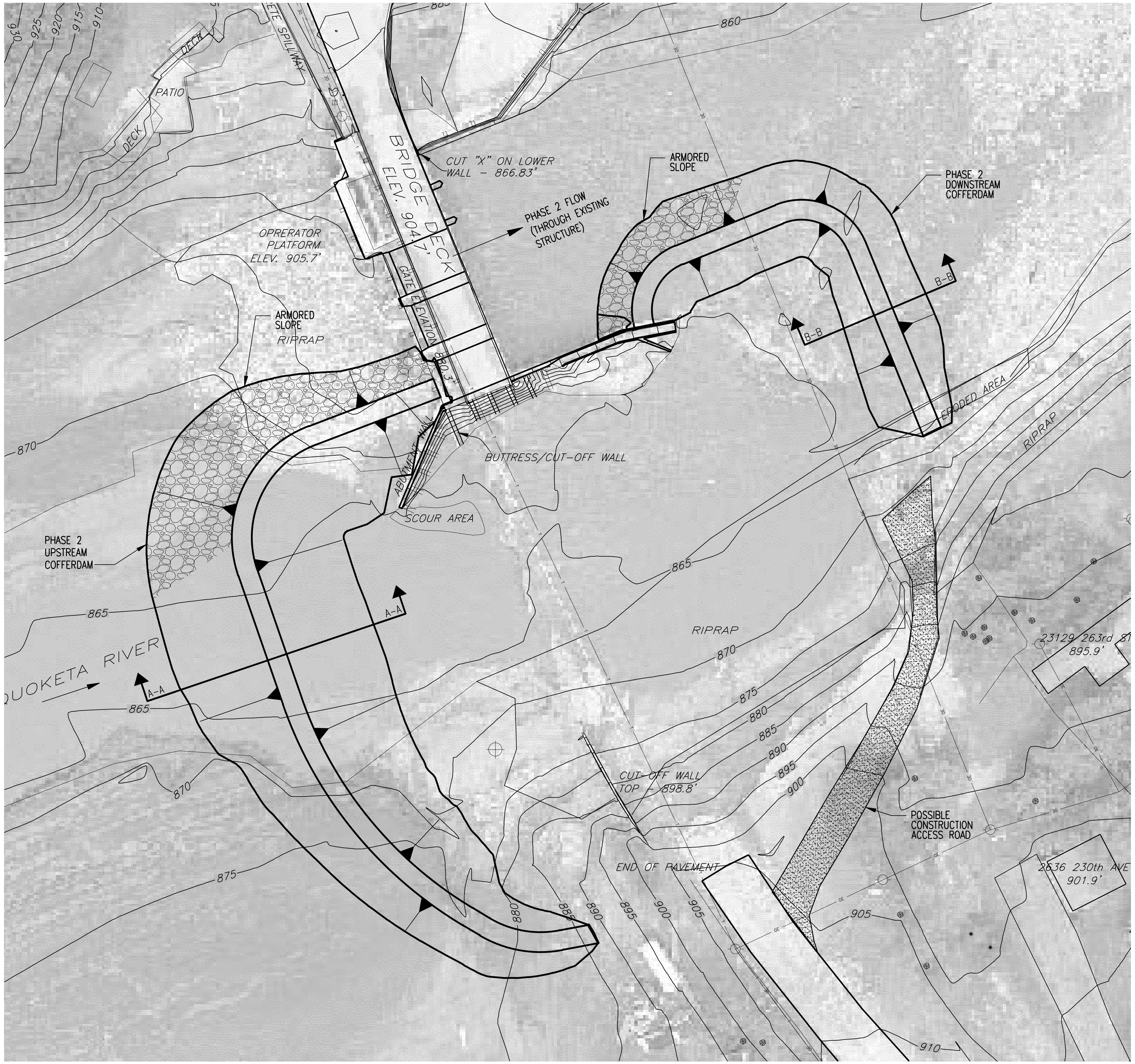
LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES

EXHIBIT 10
CONSTRUCTION STAGING
PHASE 1 COFFERDAM

CADD D4-R3

Stanley Consultants INC.

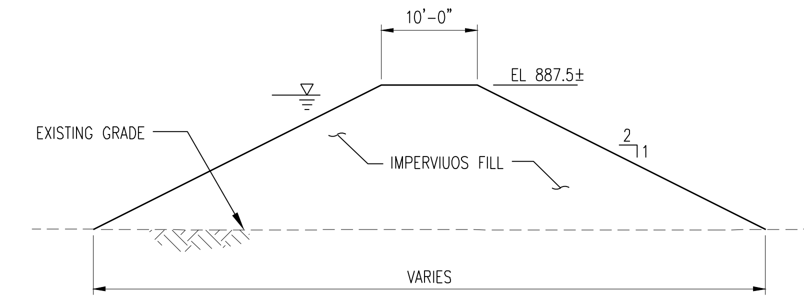
NORTH



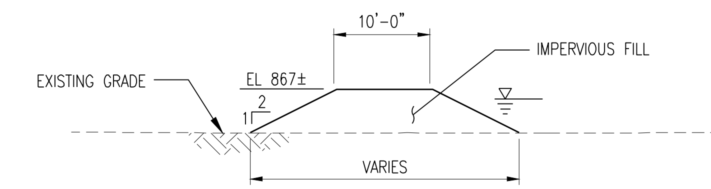
PLAN

0' 15' 30' 60'

SCALE: 1" = 60'-0"



SECTION A-A
PHASE 2 UPSTREAM COFFERDAM



SECTION B-B
PHASE 2 UPSTREAM COFFERDAM

NOTES:

1. OFF-SITE TEMPORARY CONSTRUCTION EASEMENTS MAY BE REQUIRED FOR CONSTRUCTION STAGING/ LAYDOWN AND STOCKPILE AREAS.

2. ARMOR SLOPES AT HIGH SHEAR LOCATIONS AS SHOWN.

LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES

EXHIBIT 11
CONSTRUCTION STAGING
PHASE 2 COFFERDAM

CADD D4-R3

Appendix G

Cost Estimate and Construction Schedule

LAKE DELHI DAM RECONSTRUCTION
OPINION OF PROBABLE COST FOR RECOMMENDED ALTERNATIVES
PRELIMINARY DESIGN
DATE: 12/16/11

Item No.	Title 1	Title 2	Title 3	Units	Quantity	Unit Cost	Extension	Subtotals
	Contractor Costs							
		Mobilization/Demob		LS	1	$ 450,000	$ 450,000	
		Contractor Administration		LS	1	$ 80,000	$ 80,000	
		Pile Rig Mob		LS	2	$ 5,000	$ 10,000	
		Construction Surveying		LS	1	$ 60,000	$ 60,000	
		Regulatory Requirements		LS	1	$ 25,000	$ 25,000	
		Independent Testing		LS	1	$ 50,000	$ 50,000	
		Traffic Control		LS	1	$ 5,000	$ 5,000	
		Temporary Fence		LF	500	$ 20	$ 10,000	
		Trailer Office		MONTH	12	$ 1,000	$ 12,000	
		Cleanup		LS	1	$ 50,000	$ 50,000	
								$ 752,000
	Demolition							
		Concrete Walls						
			North Embankment - East Wall	CY	91	$ 180	$ 16,400	
			North Embankment - West Wall	CY	106	$ 180	$ 19,000	
			North Embankment - Crib Wall	CY	70	$ 180	$ 12,667	
			North Embankment - Block Wall	CY	169	$ 180	$ 30,420	
			South Embankment - Cutoff Wall	CY	120	$ 180	$ 21,600	
		Storm Drainage						
			Catch Basin	EA	1	$ 500	$ 500	
			Storm Drain Pipe	LF	40	$ 20	$ 800	
			Concrete Drainage Spillway	CY	5	$ 100	$ 500	
		Pavement						
			North Embankment - Concrete	SY	395	$ 7	$ 2,844	
			South Embankment - Asphalt	SY	40	$ 5	$ 208	
		Misc. Site Work						
			Site Clearing	ACRE	3	$ 3,000	$ 9,000	
			Remove Topsoil	ACRE	3	$ 3,000	$ 9,000	
			Tree Removal	EA	10	$ 450	$ 4,500	
			Remove Traffic Sign	EA	3	$ 100	$ 300	
			Remove Chain Link Fence	LS	1	$ 4,000	$ 4,000	
			Remove Fish Ladder	CY	5	$ 180	$ 900	
								$ 133,000
	North Embankment							
		Reinforced Conc. Walls						
			Remove existing embankment	CY	1704	$ 10.00	$ 17,040	
			Reinforced concrete footing	CY	278	$ 300.00	$ 83,400	
			Reinforced concrete walls	CY	261	$ 750.00	$ 195,750	
			Drill holes for tie-backs	EA	40	$ 200.00	$ 8,000	
			Steel tension bars (tie-back)	LB	5773	$ 3.00	$ 17,319	
			Steel sheet pile cutoff	SF	2000	$ 35.00	$ 70,000	
			Bolted connection to wall	EA	40	$ 200.00	$ 8,000	
			Soil fill	CY	1711	$ 15.00	$ 25,665	
								$ 425,000
	North Abutment Wall							
		Replace Block Portion						
		(Massive Blocks)						
			Concrete slab removal	CY	42	$ 180.00	$ 7,560	
			Excavation	CY	481	$ 10.00	$ 4,810	
			Geotextile	SY	300	$ 3.00	$ 900	
			Structural backfill	CY	815	$ 45.00	$ 36,675	
			Drain pipe	LF	120	$ 12.00	$ 1,440	
			Massive blocks	SF	1300	$ 25.00	$ 32,500	
								$ 84,000
	Powerhouse							
		USACE Stabilization						
			Slab reinforced concrete	CY	120	$ 300.00	$ 36,000	
			Rock anchors	LF	600	$ 54.00	$ 32,400	
			Drill 6" hole (air rotary - rock)	LF	600	$ 65.00	$ 39,000	
			Re-Drill 6" hole (rock)	LF	600	$ 30.00	$ 18,000	
			Grouting program	CF	600	$ 16.50	$ 9,900	
			Misc structural repair	CY	40	$ 1,800.00	$ 72,000	
			Powerhouse corrosion investigation	LS	1	$ 20,000.00	$ 20,000	
		Powerhouse Waterproofing						
			Membrane System	SY	180	$ 160.00	$ 28,800	
								$ 256,000

LAKE DELHI DAM RECONSTRUCTION
OPINION OF PROBABLE COST FOR RECOMMENDED ALTERNATIVES
PRELIMINARY DESIGN
DATE: 12/16/11

Item No.	Title 1	Title 2	Title 3	Units	Quantity	Unit Cost	Extension	Subtotals
	Existing Spillway							
		Gate Replacement						
			Demo existing gates	EA	3	$ 5,000	$ 15,000	
			Demo existing guide locations	EA	6	$ 3,000	$ 18,000	
			New gates	EA	3	$ 300,000	$ 900,000	
			New guides	EA	6	$ 60,000	$ 360,000	
			New stop log assemblies	EA	3	$ 30,000	$ 90,000	
			New min flow valves (incl. w/ "Min. Flow Passage")	EA	0	$ 8,000	$ -	
			New gate installation	LS	1	$ 202,500	$ 202,500	
			Remaining vendor payment	LS	1	$ 37,000	$ 37,000	
		USACE Stabilization						
			Drill 6" hole (air rotary - rock]	LF	600	$ 65.00	$ 39,000	
			Re-Drill 6" hole (rock)	LF	600	$ 30.00	$ 18,000	
			Rock anchors	LF	600	$ 54.00	$ 32,400	
			Slab reinforced concrete	CY	157	$ 300.00	$ 47,100	
			Grouting program	CF	600	$ 16.50	$ 9,900	
			Concrete Removal	CY	185	$ 180.00	$ 33,300	
			Spillway resurfacing concrete	CY	67	$ 300.00	$ 20,100	
			Spillway piers concrete	CY	33	$ 450.00	$ 14,850	
			Spillway gate concrete	CY	30	$ 450.00	$ 13,500	
			Stilling basin slab repair concrete	CY	20	$ 300.00	$ 6,000	
			South wall repair concrete	CY	30	$ 450.00	$ 13,500	
			Other minor structural repairs	CY	5	$ 1,800.00	$ 9,000	
								$ 1,879,000
	New Spillway (Service/Aux.)							
		Single Labyrinth Weir						
			Spillway slab	CY	1079	$ 300.00	$ 323,700	
			Spillway weir wall	CY	286	$ 750.00	$ 214,500	
			Spillway side wall	CY	178	$ 750.00	$ 133,500	
			Spillway chute and stilling basin	CY	1200	$ 300.00	$ 360,000	
			Spillway stilling basin sheet pile	SF	900	$ 35.00	$ 31,500	
			Spillway filter gravel/sand	CY	1532	$ 45.00	$ 68,940	
			Steel sheet pile seepage cutoff (structure)	SF	1800	$ 35.00	$ 63,000	
			Steel sheet pile seepage cutoff (embankment)	SF	6300	$ 35.00	$ 220,500	
			Downstream Channel Wall	CY	336	$ 750.00	$ 252,000	
			Downstream Channel Excavation	CY	5357	$ 10.00	$ 53,570	
			Riprap	CY	1430	$ 50.00	$ 71,500	
			Geotextile	SY	2000	$ 3.00	$ 6,000	
								$ 1,799,000
	South Spillway Embankment Construction (New)							
		Zoned Earth						
			Remove existing embankment	CY	9700	$ 10.00	$ 97,000	
			Till borrow - material	CY	6600	$ 35.00	$ 231,000	
			Loess borrow - material	CY	12500	$ 35.00	$ 437,500	
			Riprap (included in "New Spillway")					
			Drainage aggregate	CY	300	$ 45.00	$ 13,500	
			Geotextile	SY	1000	$ 2.50	$ 2,500	
			Steel sheet pile cutoff (included in "New Spillway")					
			Grout curtain program (In Rock)	LS	1	$ 75,000.00	$ 75,000	
								$ 857,000
	South Dam Embankment Construction (Existing)							
		Cut into Existing						
			Remove existing fill (included in S. Embankment)	CY	0	$ 10.00	$ -	
			Place new fill (included in S. Embankment)	CY	0	$ 35.00	$ -	
			Torch cut existing sheet pile	LF	50	$ 8.50	$ 425	
			Steel sheet pile cutoff	SF	8000	$ 35.00	$ 280,000	
								$ 280,000
	Minimum Flow Passage							
		Valves in Slide Gate						
			Valves in Slide Gate (Included in Exst Spillway)	EA	3	$ 8,000.00	$ 24,000	
								$ 24,000

LAKE DELHI DAM RECONSTRUCTION
OPINION OF PROBABLE COST FOR RECOMMENDED ALTERNATIVES
PRELIMINARY DESIGN
DATE: 12/16/11

Item No.	Title 1	Title 2	Title 3	Units	Quantity	Unit Cost	Extension	Subtotals
	Misc Site Work							
		Erosion Control						
			Rock Dike	EA	1	$ 24,000	$ 24,000	
			Silt Curtain	LF	400	$ 20	$ 8,000	
			Silt Fence	LF	1200	$ 3	$ 3,600	
			Rock Construction Entrance	EA	4	$ 1,000	$ 4,000	
		Grading		SY	10000	$ 1	$ 10,000	
		Landscaping		LS	2	$ 10,000	$ 20,000	
		Silt Removal		CY	2400	$ 30	$ 72,000	
		Seeding & Fertilizing		SY	10000	$ 1	$ 10,000	
		Scour Protection						
			Geotextile	SY	2000	$ 2	$ 4,500	
			RipRap	CY	450	$ 50	$ 22,500	
								$ 179,000
	Civil Features							
		Surface Drainage						
			Storm Drain Pipe	LF	200	$ 60	$ 12,000	
			Storm Drain Catch Basins	EA	4	$ 3,500	$ 14,000	
		Access and Parking						
			North Embankment - AC Pavement	SY	935	$ 35	$ 32,725	
			North Embankment - Agg Base	CY	312	$ 15	$ 4,680	
			South Abutment - AC Pavement	SY	1375	$ 35	$ 48,125	
			South Embankment - Agg. Base	CY	458	$ 15	$ 6,870	
			South Embankment - Pavement Marking	LF	500	$ 2	$ 1,000	
			Concrete Walk	SF	500	$ 3	$ 1,500	
		Water Supply						
			Water Service Line	LF	1000	$ 30	$ 30,000	
			Water Valves	EA	4	$ 600	$ 2,400	
		Guardrails						
			Salvage and Reinstall Steel Beam Guardrail	LF	200	$ 10	$ 2,000	
			Steel Beam Guardrail	LF	300	$ 20	$ 6,000	
			Guardrail End Terminals	LF	4	$ 2,000	$ 8,000	
								$ 169,000
	Electrical/Controls							
			Demo Existing Electrical Equipment	LS	1	$12,000.00	$ 12,000	
			3/4" RGS Conduit	LF	750	$10.40	$ 7,800	
			1" RGS Conduit	LF	150	$13.20	$ 1,980	
			1-1/2" RGS Conduit	LF	375	$17.30	$ 6,488	
			2" RGS Conduit	LF	250	$21.50	$ 5,375	
			2/C #16 AWG Shielded Cable	LF	900	$1.50	$ 1,350	
			#14 AWG Copper	LF	2000	$0.60	$ 1,200	
			#12 AWG Copper	LF	3500	$0.70	$ 2,450	
			#10 AWG Copper	LF	500	$0.80	$ 400	
			#6 AWG Copper	LF	750	$1.50	$ 1,125	
			#1 AWG Copper	LF	1250	$3.40	$ 4,250	
			3/0 AWG Copper	LF	1000	$6.20	$ 6,200	
			Refinish Electrical Room	LS	1	$15,000.00	$ 15,000	
			Control PLC / Computer / Autodialer	EA	1	$55,000.00	$ 55,000	
			6" PVC Stilling Well	LF	30	$45.00	$ 1,350	
			Submersible Level Transducer	EA	1	$1,000.00	$ 1,000	
			Gate Limit Switches	EA	8	$200.00	$ 1,600	
			Gate Position Indicators	EA	3	$750.00	$ 2,250	
			125 kW Diesel Generator	EA	1	$65,000.00	$ 65,000	
			480V Panelboard	EA	1	$5,500.00	$ 5,500	
			Disconnect Switch - 30A/3P	EA	4	$400.00	$ 1,600	
			Combination Motor Starter - 60 HP	EA	3	$9,500.00	$ 28,500	
			Wiring Devices (Receptacles and Switches)	EA	25	$25.00	$ 625	
			Light Fixtures - Interior	EA	20	$350.00	$ 7,000	
			Light Fixtures - Exterior	EA	5	$575.00	$ 2,875	
			Telecom Connection to Dam	LS	1	$10,000.00	$ 10,000	
								$ 248,000

LAKE DELHI DAM RECONSTRUCTION
OPINION OF PROBABLE COST FOR RECOMMENDED ALTERNATIVES
PRELIMINARY DESIGN
DATE: 12/16/11

Item No.	Title 1	Title 2	Title 3	Units	Quantity	Unit Cost	Extension	Subtotals
	Cofferdams/Dewatering							
		Phase 1 Upstream						
			Loess borrow - material	CY	1015	$ 35	$ 35,525	
			Riprap	CY	150	$ 50	$ 7,500	
		Phase 1 Downsteam						
			Loess borrow - material	CY	2833	$ 35	$ 99,155	
			Riprap	CY	75	$ 50	$ 3,750	
		Phase 1 Dewatering						
			Deep wells	LS	1	$ 50,000	$ 50,000	
		Phase 2 Upstream						
			Loess borrow - material	CY	11893	$ 35	$ 416,255	
			Riprap	CY	150	$ 50	$ 7,500	
		Phase 2 Downstream						
			Loess borrow - material	CY	2322	$ 35	$ 81,270	
			Riprap	CY	75	$ 50	$ 3,750	
		Phase 2 Dewatering						
			Deep wells	LS	1	$ 150,000	$ 150,000	
								$ 855,000
	Safety Features							
		Discharge Warning System		LS	1	$ 10,000	$ 10,000	
		Buoys & Floats		LS	1	$ 60,000	$ 60,000	
		Markers and Signage		LS	1	$ 8,000	$ 8,000	
		Fencing						
			Chain Link (6' Height)	LF	400	$ 20	$ 8,000	
								$ 86,000
	Archaeological Mitigation							
		Allowance	Allowance	LS	1	$ 75,000	$ 75,000	
								$ 75,000
	Recreational Features							
		Canoe Portage Trail		LS	1	$ 46,500	$ 46,500	
		Boat Ramp		LS	1	$ 60,000	$ 60,000	
		Observation Deck		LS	1	$ 5,000	$ 5,000	
		Handicapped Acc. Fishing Pier		LS	1	$ 55,000	$ 55,000	
								$ 167,000
	Property/Easement Acquisition							
		Upstream properties		LS	1	$ 150,000	$ 150,000	
		Downstream properties		LS	1	$ 100,000	$ 100,000	
		Staging areas		LS	1	$ 10,000	$ 10,000	
		Boat Ramp/Fishing Pier		LS	1	$ 100,000	$ 100,000	
								$ 360,000
	Field Engineering & Admin							
		Resident Services						
			Labor	WEEK	36	$ 3,800	$ 136,800	
			Expenses	WEEK	36	$ 500	$ 18,000	
		EDC Office Support		LS	1	$ 50,000	$ 50,000	
								$ 205,000

Construction Cost Items	$ 7,516,000	
Contractor Costs	$ 752,000	
Property Costs	$ 360,000	
Field Engineering & Admin	$ 205,000	
Subtotal	**$ 8,833,000**	
Contingency	$ 1,770,000	**20%**
Subtotal with Contingency	**$ 10,600,000**	
Escalation for 2012/2013	**$ 530,000**	**5%**
Engineering Fee	**$ 740,000**	**7%**
TOTAL	**$ 11,870,000**	

LAKE DELHI DAM RECONSTRUCTION

Reconstruction Alternatives Cost Comparison

Area	Alternative Concept	Item	Units	Quantity	Unit Cost	Extension
North Embankment	Reinforced Conc. Walls					
		Remove existing embankment	CY	1704	$ 10.00	$ 17,040
		Reinforced concrete footing	CY	278	$ 300.00	$ 83,400
		Reinforced concrete walls	CY	261	$ 750.00	$ 195,750
		Drill holes for tie-backs	EA	40	$ 200.00	$ 8,000
		Steel tension bars (tie-back)	LB	5773	$ 3.00	$ 17,319
		Steel sheet pile cutoff	SF	2000	$ 35.00	$ 70,000
		Bolted connection to wall	EA	40	$ 200.00	$ 8,000
		Soil fill	CY	1711	$ 15.00	$ 25,665
					Subtotal 1	*$ 425,174*
					Contingency	*$ 85,035*
					Subtotal 2	*$ 510,209*
					Inflation	*$ 25,510*
					Total	**$ 536,000**
	Cellular Sheet Pile Structure					
		Remove existing embankment	CY	1704	$ 10.00	$ 17,040
		Steel sheet pile	SF	13855	$ 35.00	$ 484,925
		Granular fill	CY	1904	$ 15.00	$ 28,560
		Torch cut sheet pile	LF	44	$ 1.50	$ 66
		Guardrail	LF	160	$ 28.00	$ 4,480
					Subtotal 1	*$ 535,071*
					Contingency	*$ 107,014*
					Subtotal 2	*$ 642,085*
					Inflation	*$ 32,104*
					Total	**$ 675,000**
	Double Sheet Pile Wall					
		Remove existing embankment	CY	1704	$ 10.00	$ 17,040
		Steel sheet pile	SF	8000	$ 35.00	$ 280,000
		Granular fill	CY	1704	$ 15.00	$ 25,560
		Torch cut sheet pile	LF	44	$ 1.50	$ 66
		Torch cut holes for tie-backs	EA	40	$ 50.00	$ 2,000
		Steel tension bars (tie-back)	LB	5773	$ 3.00	$ 17,319
		Steel waler	LB	8000	$ 3.00	$ 24,000
		Bolted connection to waler	EA	40	$ 30.00	$ 1,200
		Guardrail	LF	160	$ 28.00	$ 4,480
					Subtotal 1	*$ 371,665*
					Contingency	*$ 74,333*
					Subtotal 2	*$ 445,998*
					Inflation	*$ 22,300*
					Total	**$ 469,000**
North Abutment Wall	Replace Block Portion					
	MSE (Massive Blocks)					
		Concrete slab removal	CY	42	$ 180.00	$ 7,560
		Excavation	CY	481	$ 10.00	$ 4,810
		Geotextile	SY	300	$ 3.00	$ 900
		Structural backfill	CY	815	$ 45.00	$ 36,675
		Drain pipe	LF	120	$ 12.00	$ 1,440
		Massive blocks	SF	1300	$ 25.00	$ 32,500
					Subtotal 1	*$ 83,885*
					Contingency	*$ 16,777*
					Subtotal 2	*$ 100,662*
					Inflation	*$ 5,033*
					Total	**$ 106,000**
	Replace Block Portion					
	Reinforced Concrete					
		Concrete slab removal	CY	42	$ 180.00	$ 7,560
		Excavation	CY	481	$ 10.00	$ 4,810
		Geotextile	SY	300	$ 3.00	$ 900
		Structural backfill	CY	815	$ 45.00	$ 36,675
		Drain pipe	LF	120	$ 12.00	$ 1,440
		Reinforced concrete walls	CY	123	$ 750.00	$ 92,250
					Subtotal 1	*$ 143,635*
					Contingency	*$ 28,727*
					Subtotal 2	*$ 172,362*
					Inflation	*$ 8,618*
					Total	**$ 181,000**

LAKE DELHI DAM RECONSTRUCTION

Reconstruction Alternatives Cost Comparison

Area	Alternative Concept	Item	Units	Quantity	Unit Cost	Extension
Powerhouse	FERC Stabilization					
		Slab reinforced concrete	CY	195	$ 300.00	$ 58,500
		Rock anchors	LF	1900	$ 54.00	$ 102,600
		Drill 6" hole (core - concrete)	LF	600	$ 165.00	$ 99,000
		Drill 6" hole (air rotary - rock)	LF	1300	$ 65.00	$ 84,500
		Re-Drill 6" hole (rock)	LF	1300	$ 30.00	$ 39,000
		Grouting program	CF	1900	$ 16.50	$ 31,350
		Misc structural repair	CY	40	$ 1,800.00	$ 72,000
		Powerhouse corrosion investigation	LS	1	$ 20,000.00	$ 20,000
					Subtotal 1	*$ 506,950*
					Contingency	*$ 101,390*
					Subtotal 2	*$ 608,340*
					Inflation	*$ 30,417*
					Total	**$ 639,000**
	USACE Stabilization					
		Slab reinforced concrete	CY	120	$ 300.00	$ 36,000
		Rock anchors	LF	600	$ 54.00	$ 32,400
		Drill 6" hole (air rotary - rock)	LF	600	$ 65.00	$ 39,000
		Re-Drill 6" hole (rock)	LF	600	$ 30.00	$ 18,000
		Grouting program	CF	600	$ 16.50	$ 9,900
		Misc structural repair	CY	40	$ 1,800.00	$ 72,000
		Powerhouse corrosion investigation	LS	1	$ 20,000.00	$ 20,000
					Subtotal 1	*$ 227,300*
					Contingency	*$ 45,460*
					Subtotal 2	*$ 272,760*
					Inflation	*$ 13,638*
					Total	**$ 287,000**
	Powerhouse Waterproofing					
		Clean and Epoxy Seal	SF	1625	$ 10.00	$ 16,250
					Subtotal 1	*$ 16,250*
					Contingency	*$ 3,250*
					Subtotal 2	*$ 19,500*
					Inflation	*$ 975*
					Total	**$ 21,000**
		Waterproof Membrane System	SY	180	$ 160.00	$ 28,800
					Subtotal 1	*$ 28,800*
					Contingency	*$ 5,760*
					Subtotal 2	*$ 34,560*
					Inflation	*$ 1,728*
					Total	**$ 37,000**
Existing Spillway	USACE Stabilization					
		Drill 6" hole (air rotary - rock)	LF	600	$ 65.00	$ 39,000
		Re-Drill 6" hole (rock)	LF	600	$ 30.00	$ 18,000
		Rock anchors	LF	600	$ 54.00	$ 32,400
		Slab reinforced concrete	CY	157	$ 300.00	$ 47,100
		Grouting program	CF	600	$ 16.50	$ 9,900
		Concrete Removal	CY	185	$ 180.00	$ 33,300
		Spillway resurfacing concrete	CY	67	$ 300.00	$ 20,100
		Spillway piers concrete	CY	33	$ 450.00	$ 14,850
		Spillway gate concrete	CY	30	$ 450.00	$ 13,500
		Stilling basin slab repair concrete	CY	20	$ 300.00	$ 6,000
		South wall repair concrete	CY	30	$ 450.00	$ 13,500
		Other minor structural repairs	CY	5	$ 1,800.00	$ 9,000
					Subtotal 1	*$ 256,650*
					Contingency	*$ 51,330*
					Subtotal 2	*$ 307,980*
					Inflation	*$ 15,399*
					Total	**$ 324,000**
	FERC Stabilization					
		Drill 6" hole (core - concrete)	LF	950	$ 165.00	$ 156,750
		Drill 6" hole (air rotary - rock)	LF	1600	$ 65.00	$ 104,000
		Re-Drill 6" hole (rock)	LF	1600	$ 30.00	$ 48,000
		Rock anchors	LF	2550	$ 54.00	$ 137,700
		Slab reinforced concrete	CY	157	$ 300.00	$ 47,100
		Grouting program	CF	2550	$ 16.50	$ 42,075
		Concrete Removal	CY	185	$ 180.00	$ 33,300
		Spillway resurfacing concrete	CY	67	$ 300.00	$ 20,100
		Spillway piers concrete	CY	33	$ 450.00	$ 14,850
		Spillway gate concrete	CY	30	$ 450.00	$ 13,500
		Stilling basin slab repair concrete	CY	20	$ 300.00	$ 6,000
		South wall repair concrete	CY	30	$ 450.00	$ 13,500
		Other minor structural repairs concrete	CY	5	$ 300.00	$ 1,500
					Subtotal 1	*$ 481,625*
					Contingency	*$ 96,325*
					Subtotal 2	*$ 577,950*
					Inflation	*$ 28,898*
					Total	**$ 607,000**

LAKE DELHI DAM RECONSTRUCTION

Reconstruction Alternatives Cost Comparison

Area	Alternative Concept	Item	Units	Quantity	Unit Cost	Extension
New Spillway						
(Service and Auxillary)	Dual Labyrinth Weir	Property Acquisition	AC	0.4	$ 160,000.00	$ 64,000
		Structure Demo, Removal, Disposal	EA	2	$ 30,000.00	$ 60,000
		Service spillway slab concrete	CY	798	$ 300.00	$ 239,400
		Service spillway weir wall	CY	154	$ 750.00	$ 115,500
		Service spillway side wall	CY	72	$ 750.00	$ 54,000
		Service spillway chute and stilling basin	CY	867	$ 300.00	$ 260,100
		Service spillway stilling basin sheet pile	SF	700	$ 35.00	$ 24,500
		Service spillway filter gravel/sand	CY	1140	$ 45.00	$ 51,300
		Auxiliary spillway slab	CY	509	$ 300.00	$ 152,700
		Auxiliary spillway weir wall	CY	159	$ 750.00	$ 119,250
		Auxiliary spillway side wall	CY	193	$ 750.00	$ 144,750
		Auxiliary spillway filter gravel/sand	CY	356	$ 45.00	$ 16,020
		Auxiliary spillway filter gravel/sand	CY	500	$ 45.00	$ 22,500
		Aux. spillway erosion protection (see next item)				$ -
		Steel sheet pile seepage cutoff (structure)	SF	1200	$ 35.00	$ 42,000
		Steel sheet pile seepage cutoff (embankment)	SF	6400	$ 35.00	$ 224,000
		Downstream Channel Wall	CY	504	$ 750.00	$ 378,000
		Downstream Channel Excavation	CY	17764	$ 10.00	$ 177,640
		Riprap	CY	1490	$ 50.00	$ 74,500
		Geotextile	SY	2000	$ 3.00	$ 6,000
					Subtotal 1	*$ 2,226,160*
					Contingency	*$ 445,232*
					Subtotal 2	*$ 2,671,392*
					Inflation	*$ 133,570*
					Total	**$ 2,805,000**
	Single Labyrinth Weir					
		Spillway slab	CY	1079	$ 300.00	$ 323,700
		Spillway weir wall	CY	286	$ 750.00	$ 214,500
		Spillway side wall	CY	178	$ 750.00	$ 133,500
		Spillway chute and stilling basin	CY	1200	$ 300.00	$ 360,000
		Spillway stilling basin sheet pile	SF	900	$ 35.00	$ 31,500
		Spillway filter gravel/sand	CY	1532	$ 45.00	$ 68,940
		Steel sheet pile seepage cutoff (structure)	SF	1800	$ 35.00	$ 63,000
		Steel sheet pile seepage cutoff (embankment)	SF	6300	$ 35.00	$ 220,500
		Downstream Channel Wall	CY	336	$ 750.00	$ 252,000
		Downstream Channel Excavation	CY	5357	$ 10.00	$ 53,570
		Riprap	CY	1430	$ 50.00	$ 71,500
		Geotextile	SY	2000	$ 3.00	$ 6,000
					Subtotal 1	*$ 1,798,710*
					Contingency	*$ 359,742*
					Subtotal 2	*$ 2,158,452*
					Inflation	*$ 107,923*
					Total	**$ 2,267,000**
	Pneumatic Gates					
		Gate System	LF	160	$ 6,000.00	$ 960,000
		Obermeyer spillway slab	CY	984	$ 300.00	$ 295,200
		Obermeyer spillway side wall	CY	183	$ 750.00	$ 137,250
		Obermeyer spillway chute and stilling basin	CY	1096	$ 300.00	$ 328,800
		Obermeyer spillway stilling basin sheet pile	SF	800	$ 35.00	$ 28,000
		Obermeyer spillway filter gravel/sand	CY	1193	$ 45.00	$ 53,685
		Steel sheet pile seepage cutoff (structure)	SF	1600	$ 35.00	$ 56,000
		Steel sheet pile seepage cutoff (embankment)	SF	7200	$ 35.00	$ 252,000
		Riprap	CY	1350	$ 40.00	$ 54,000
		Geotextile	SY	2000	$ 3.00	$ 6,000
					Subtotal 1	*$ 2,170,935*
					Contingency	*$ 434,187*
					Subtotal 2	*$ 2,605,122*
					Inflation	*$ 130,256*
					Total	**$ 2,736,000**
Auxillary Spillway Section						
Erosion Protection						
	Articulated Concrete Block					
		Articulated concrete block	SF	16200	$ 12.00	$ 194,400
		Anchoring	LS	1	$ 20,000.00	$ 20,000
					Subtotal 1	*$ 214,400*
					Contingency	*$ 42,880*
					Subtotal 2	*$ 257,280*
					Inflation	*$ 12,864*
					Total	**$ 271,000**
	Roller Compacted Concrete					
		RCC	CY	2600	$ 130.00	$ 338,000
		Drainage aggregate	CY	1100	$ 45.00	$ 49,500
					Subtotal 1	*$ 387,500*
					Contingency	*$ 77,500*
					Subtotal 2	*$ 465,000*
					Inflation	*$ 23,250*
					Total	**$ 489,000**
	Concrete					
		Reinforced concrete slab	CY	650	$ 300.00	$ 195,000
		Drainage aggregate	CY	1100	$ 45.00	$ 49,500
					Subtotal 1	*$ 244,500*
					Contingency	*$ 48,900*
					Subtotal 2	*$ 293,400*
					Inflation	*$ 14,670*
					Total	**$ 309,000**

LAKE DELHI DAM RECONSTRUCTION

Reconstruction Alternatives Cost Comparison

Area	Alternative Concept	Item	Units	Quantity	Unit Cost	Extension
South Spillway Embankment						
Construction (New)						
	Homogeneous Clay					
		Remove existing embankment	CY	9700	$ 10.00	$ 97,000
		Till borrow - material	CY	19100	$ 35.00	$ 668,500
		Riprap (included in "New Spillway")	CY		$ 60.00	$ -
		Drain aggregate	CY	300	$ 45.00	$ 13,500
		Geotextile	SY	1000	$ 2.50	$ 2,500
		Steel sheet pile cutoff (included in "New Spillway")	SF			$ -
		Grout curtain program	LS	1	$ 75,000.00	$ 75,000
					Subtotal 1	*$ 856,500*
					Contingency	*$ 171,300*
					Subtotal 2	*$ 1,027,800*
					Inflation	*$ 51,390*
					Total	**$ 1,080,000**
	Zoned Earth					
		Remove existing embankment	CY	9700	$ 10.00	$ 97,000
		Till borrow - material	CY	6600	$ 35.00	$ 231,000
		Loess borrow - material	CY	12500	$ 35.00	$ 437,500
		Riprap (included in "New Spillway")	CY		$ 60.00	$ -
		Drainage aggregate	CY	300	$ 45.00	$ 13,500
		Geotextile	SY	1000	$ 2.50	$ 2,500
		Steel sheet pile cutoff (included in "New Spillway")	SF			$ -
		Grout curtain program	LS	1	$ 75,000.00	$ 75,000
					Subtotal 1	*$ 856,500*
					Contingency	*$ 171,300*
					Subtotal 2	*$ 1,027,800*
					Inflation	*$ 51,390*
					Total	**$ 1,080,000**
	Roller Compacted Concrete					
	(RCC)					
		Remove existing embankment	CY	9700	$ 10.00	$ 97,000
		RCC	CY	15000	$ 115.00	$ 1,725,000
		Drain aggregate	CY	300	$ 45.00	$ 13,500
		Grout curtain program	LS	1	$ 75,000.00	$ 75,000
		Chute and stilling basin discount	LS	1	$ (360,000.00)	$ (360,000)
		Riprap discount	CY	1490	$ (50.00)	$ (74,500)
					Subtotal 1	*$ 1,476,000*
					Contingency	*$ 295,200*
					Subtotal 2	*$ 1,771,200*
					Inflation	*$ 88,560*
					Total	**$ 1,860,000**
South Dam Embankment						
Construction (Existing)	Remove and Replace					
		Remove existing fill	CY	24700	$ 10.00	$ 247,000
		Place new fill	CY	32200	$ 35.00	$ 1,127,000
		Torch cut existing sheet pile	LF	325	$ 8.50	$ 2,763
		Steel sheet pile cutoff	SF	4000	$ 35.00	$ 140,000
					Subtotal 1	*$ 1,516,763*
					Contingency	*$ 303,353*
					Subtotal 2	*$ 1,820,115*
					Inflation	*$ 91,006*
					Total	**$ 1,912,000**
	Cut into Existing					
		Remove existing fill	CY	0	$ 10.00	$ -
		Place new fill	CY	0	$ 35.00	$ -
		Torch cut existing sheet pile	LF	50	$ 8.50	$ 425
		Steel sheet pile cutoff	SF	8000	$ 35.00	$ 280,000
					Subtotal 1	*$ 280,425*
					Contingency	*$ 56,085*
					Subtotal 2	*$ 336,510*
					Inflation	*$ 16,826*
					Total	**$ 354,000**

LAKE DELHI DAM RECONSTRUCTION

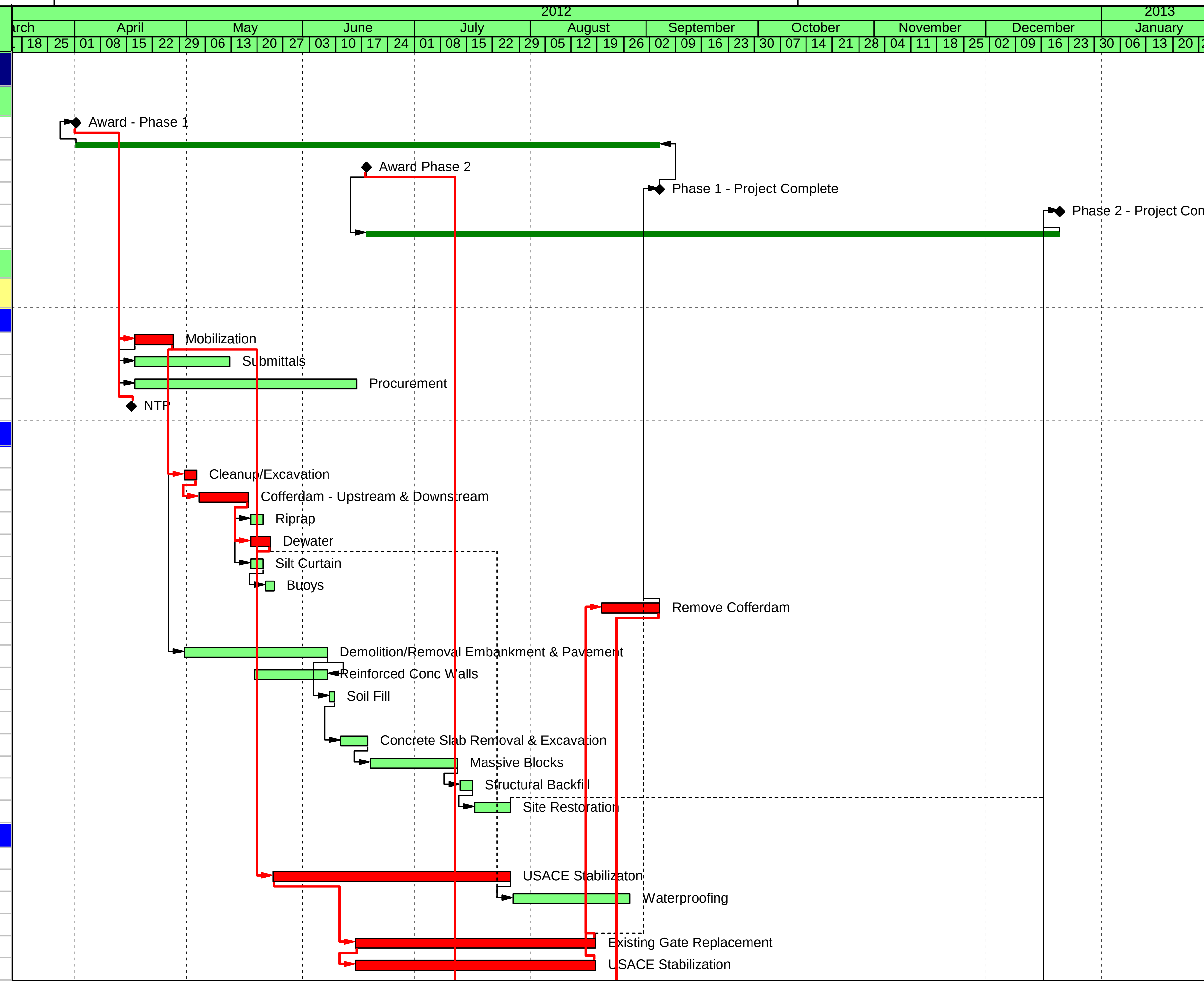
Reconstruction Alternatives Cost Comparison

Area	Alternative Concept	Item	Units	Quantity	Unit Cost	Extension
Minimum Flow Passage						
	Refurbish Wicket Gates					
		Refurbish Wicket Gates	LS	1	$ 85,000.00	$ 85,000
		Downstream Aeration System	LS	1	$ 5,000.00	$ 5,000
					Subtotal 1	*$ 90,000*
					Contingency	*$ 18,000*
					Subtotal 2	*$ 108,000*
					Inflation	*$ 5,400*
					Total	**$ 114,000**
	Valves in Slide Gate					
		Valves in Slide Gate	EA	3	$ 8,000.00	$ 24,000
					Subtotal 1	*$ 24,000*
					Contingency	*$ 4,800*
					Subtotal 2	*$ 28,800*
					Inflation	*$ 1,440*
					Total	**$ 31,000**
Fish Passage						
	Rock Rapids Structure					
		Property Acquisition	AC	0.4	$ 160,000.00	$ 64,000
		Structure Demo, Removal, Disposal	EA	2	$ 30,000.00	$ 60,000
		Pool-Riffle Grading	SY	2800	$ 3.00	$ 8,400
		Excavation / Fill	CY	10900	$ 10.00	$ 109,000
		Rock Channel and Pools	CY	1750	$ 60.00	$ 105,000
		Separation Wall	CY	200	$ 750.00	$ 150,000
		Concrete Bottom Slab	CY	42	$ 300.00	$ 12,600
		Aggregate Fill on Bottom Slab	CY	28	$ 15.00	$ 420
		Gate Valve Control Structure w/ Automatic Control	EA	1	$ 20,000.00	$ 20,000
					Subtotal 1	*$ 529,420*
					Contingency	*$ 105,884*
					Subtotal 2	*$ 635,304*
					Inflation	*$ 31,765*
					Total	**$ 668,000**

Lake Delhi Dam Reconstruction

Preliminary Planning Schedule - 6 day Work Calendar with Weather

Activity ID	Activity Name	Orig Dur	Start	Finish
Lake Delhi Dam Reconstruction		189	01-Apr-12	14-Jan-13
Project		264	01-Apr-12	20-Dec-12
1040	Award - Phase 1	0	01-Apr-12*	
1042	Phase 2 - Construction Duration - Cal Days	157	01-Apr-12	04-Sep-12
1045	Award Phase 2	0	18-Jun-12*	
1047	Phase 1 - Project Complete	0		04-Sep-12
1050	Phase 2 - Project Complete	0		20-Dec-12
1060	Phase 2 - Construction Duration - Cal Days	186	18-Jun-12	20-Dec-12
Construction		180	16-Apr-12	14-Jan-13
Phase 1 Construction		95	16-Apr-12	04-Sep-12
Mobilization		46	16-Apr-12	15-Jun-12
2000	Mobilization	10	17-Apr-12	27-Apr-12
2010	Submittals	20	17-Apr-12	12-May-12
2020	Procurement	45	17-Apr-12	15-Jun-12
2030	NTP	0	16-Apr-12*	
North Dam Structure		84	30-Apr-12	04-Sep-12
Cofferdam		84	30-Apr-12	04-Sep-12
5590	Cleanup/Excavation	4	30-Apr-12	03-May-12
5600	Cofferdam - Upstream & Downstream	10	04-May-12	17-May-12
5605	Riprap	3	18-May-12	21-May-12
5610	Dewater	5	18-May-12	23-May-12
5620	Silt Curtain	3	18-May-12	21-May-12
5630	Buoys	3	22-May-12	24-May-12
5640	Remove Cofferdam	10	20-Aug-12	04-Sep-12
North Embankment		32	30-Apr-12	09-Jun-12
3000	Demolition/Removal Embankment & Pavement	30	30-Apr-12	07-Jun-12
3010	Reinforced Conc Walls	15	19-May-12	07-Jun-12
3012	Soil Fill	2	08-Jun-12	09-Jun-12
North Abutment Wall		27	11-Jun-12	26-Jul-12
3015	Concrete Slab Removal & Excavation	5	11-Jun-12	18-Jun-12
3017	Massive Blocks	15	19-Jun-12	12-Jul-12
3060	Structural Backfill	3	13-Jul-12	16-Jul-12
3070	Site Restoration	4	17-Jul-12	26-Jul-12
Powerhouse & Existing Spillway		60	24-May-12	27-Aug-12
Powerhouse		60	24-May-12	27-Aug-12
4000	USACE Stabilizaton	40	24-May-12	26-Jul-12
4010	Waterproofing	20	27-Jul-12	27-Aug-12
Spillway		40	15-Jun-12	18-Aug-12
4020	Existing Gate Replacement	40	15-Jun-12	18-Aug-12
4030	USACE Stabilization	40	15-Jun-12	18-Aug-12



Remaining Level of Effort ◆ ◆ Milestone
Actual Level of Effort
Second Baseline
Actual Work
Remaining Work
Critical Remaining Work
WBS Summary Activity
WBS Summary Progress

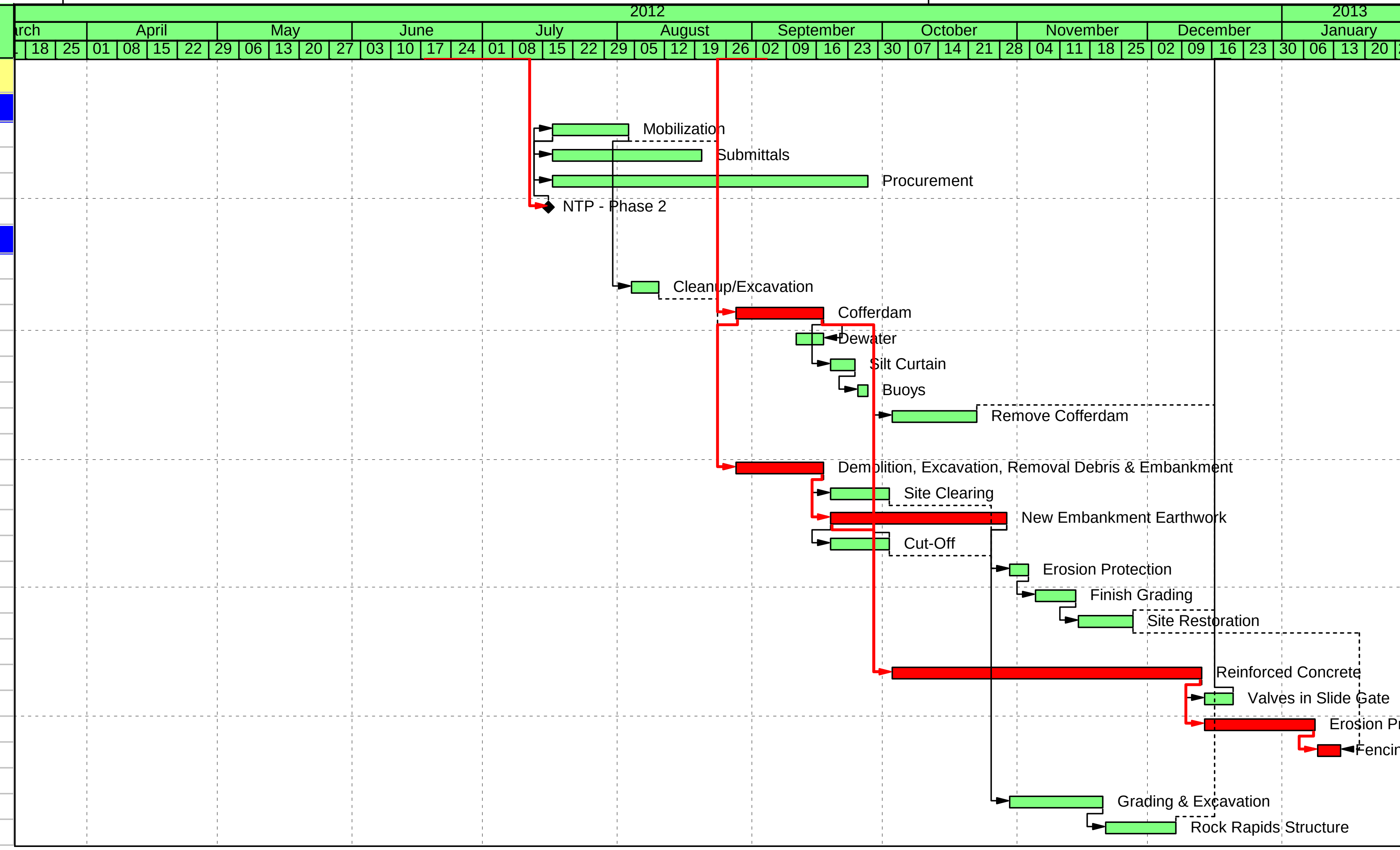
Lake Delhi Dam Reconstructon

Preliminary Planning Schedule

Date	Revision	Checked	Approved
24-Aug-11	Rev 1.00 Peliiminary Planning S...		
25-Aug-11	Rev 1.01 Backcheck 1		
25-Aug-11	Rev 1.02 Backcheck 2		
25-Aug-11	Rev 1.03 Backcheck 3		
12-Dec-11	Rev 2.00 - Phase Construction		
12-Dec-11	Rev 2.01 - Backcheck 1		

Stanley Consultants INC.

Activity ID	Activity Name	Orig Dur	Start	Finish
Phase 2 Construction		115	16-Jul-12	14-Jan-13
Mobilization		46	16-Jul-12	27-Sep-12
5550	Mobilization	10	17-Jul-12	03-Aug-12
5560	Submittals	20	17-Jul-12	20-Aug-12
5570	Procurement	45	17-Jul-12	27-Sep-12
5580	NTP - Phase 2	0	16-Jul-12*	
South Embankment		104	04-Aug-12	14-Jan-13
Cofferdam		54	04-Aug-12	22-Oct-12
5000	Cleanup/Excavation	4	04-Aug-12	10-Aug-12
5010	Cofferdam	15	28-Aug-12	17-Sep-12
5020	Dewater	5	11-Sep-12	17-Sep-12
5030	Silt Curtain	3	19-Sep-12	24-Sep-12
5040	Buoys	3	25-Sep-12	27-Sep-12
5050	Remove Cofferdam	15	03-Oct-12	22-Oct-12
Earthen Dam		65	28-Aug-12	27-Nov-12
5200	Demolition, Excavation, Removal Debris & Emba...	15	28-Aug-12	17-Sep-12
5210	Site Clearing	10	19-Sep-12	02-Oct-12
5213	New Embankment Earthwork	30	19-Sep-12	29-Oct-12
5220	Cut-Off	10	19-Sep-12	02-Oct-12
5225	Erosion Protection	5	30-Oct-12	03-Nov-12
5270	Finish Grading	5	05-Nov-12	14-Nov-12
5280	Site Restoration	10	15-Nov-12	27-Nov-12
New Spillway		65	03-Oct-12	14-Jan-13
5500	Reinforced Concrete	50	03-Oct-12	13-Dec-12
5505	Valves in Slide Gate	6	14-Dec-12	20-Dec-12
5510	Erosion Protection	10	14-Dec-12	08-Jan-13
5540	Fencing	5	09-Jan-13	14-Jan-13
Fish Passage		25	30-Oct-12	07-Dec-12
6100	Grading & Excavation	15	30-Oct-12	20-Nov-12
6110	Rock Rapids Structure	10	21-Nov-12	07-Dec-12



Appendix H

Project Scope

SUPPLEMENTAL AGREEMENT NO. 1
SCOPE OF SERVICES

BACKGROUND:

Lake Delhi Dam is located southwest of the city of Delhi, Iowa and forms an impoundment of the Maquoketa River. The dam is owned and maintained by the Lake Delhi Recreation Association. During the flood event of July 23-24, 2010 a portion of the southern earthen embankment was breached and eroded by the flood and the concrete spillway's gates were damaged. Flood waters also infiltrated and seeped through a section of the northern embankment.

The Lake Delhi Combined Recreation and Water Quality Tax District (CLIENT) have retained Stanley Consultants, Inc. (CONSULTANT) to provide consulting services for:

- Analysis of conditions for reconstruction of Lake Delhi Dam.
- Preparation of regulatory documentation for the reconstruction of Lake Delhi Dam.
- Preparation of construction documents for the reconstruction of Lake Delhi Dam.
- Bidding services
- Engineering services during construction.

The primary objectives of this project are:

- Reconstruct eroded, damaged, or inadequate portions of the dam's embankment.
- Repair damages to the gated concrete spillway structure.
- Bring the structure (embankments, spillway(s) and powerhouse) into compliance with current dam safety standards.
- Restore Lake Delhi to its normal pool elevation.

1.0 BASIC SERVICES

The following Tasks will comprise the Scope of Services for the Lake Delhi Dam Reconstruction Project:

Task 1.1 - Geotechnical Evaluation and Report
Task 1.2 – Spillway/Powerhouse Structure Evaluation and Report
Task 1.3 – Archaeological Survey
Task 1.4 – Hydrologic/Hydraulic Studies
Task 1.5– Reconstruction Design Alternatives
Task 1.7 – Permitting and Agency Coordination

1.1 TASK 1 – GEOTECHNICAL EVALUATION AND REPORT

Geotechnical data will be collected by Braun Intertec, a regional geotechnical consultant. CONSULTANT will review geotechnical data, perform analysis and provide recommendations related to reconstruction design and allow evaluation of the stability of the concrete structures. The geotechnical data, analysis, and recommendations will be summarized in a report submitted to the CLIENT.

1.1.1 *Geotechnical Investigations* – A soil boring plan will be developed for a 240' by 210' area south of the existing concrete spillway structure and a 120' by 120' area north of the spillway structure and a potential borrow area identified by the CLIENT. The program will consist of the following:

- A total of 64 borings (36 in the area south of the structure, 20 north of the structure, 3 through the spillway and powerhouse structures and 5 in a potential borrow area).
    - Dam reconstruction area borings
        - Depth will extend to auger refusal to determine the top of bedrock.
        - 5' deep rock corings will also be taken at 20 of the boring locations.
        - More borings may be cored if rock elevations are found to be erratic.
    - Existing Spillway and Powerhouse borings

        - Depth will extend 20 feet below base of concrete structures
        - Continuous cores of concrete and foundation bedrock will be taken
    - Borrow area borings
        - Depth will extend to +/- 20 feet.
        - Soil samples will be taken at 2½- to 5-ft. intervals above the bedrock.
- Braun Intertec will mobilize a drill rig and field crew to perform the boring program.
- CONSULTANT will mobilize a geotechnical engineer to the site to kick-off and coordinate the boring program.
- Once the boring program has begun collecting initial field data, CONSULTANT will organize a teleconference between the CONSULTANT's Project Manager, Braun Intertec Crew Chief, and CLIENT's Project Manager to determine if adjustments of the boring program are required due to site conditions.
- If boring program adjustments are needed, CONSULTANT will revise the Scope of Services and Compensation of the boring program and submit to the CLIENT for approval.
- A laboratory testing program will be developed to provide design input parameters.
- Data will be summarized in boring logs which will characterize subsurface soil, rock and groundwater conditions.
- Braun Intertec will summarize findings in a geotechnical report certified by an Iowa Professional Engineer.
- The report will include:
    - Field notes and observations
    - Boring logs
    - Laboratory test results
    - Soil/Bedrock profiles
    - Recommended engineering parameters (shear strength, unit weight and permeability) of site soils, bedrock, and proposed borrow materials
- CONSULTANT's geotechnical engineers will review a draft version of the report prior to its final issue.

1.1.2 *Geotechnical Analyses/Evaluations* - CONSULTANT will complete global stability, seepage and settlement analyses of the proposed reconstruction alternatives in accordance with current dam safety criteria and IDNR Technical Bulletin 16.

- Global stability analyses will be completed utilizing the Spencer's Method of analysis as included in the SLOPE/W slope stability analyses software.
- Seepage analyses will be completed using flow net procedures and/or finite element methods as contained in the SEEP/W seepage analysis software.
- Settlement analyses will be completed utilizing the CSETT analysis software developed by the U. S. Army Corps of Engineers.

1.1.3 *Geotechnical Evaluation Report* - The results of the geotechnical investigation will be packaged with the geotechnical analyses and included in a stand-alone geotechnical report. The results of the geotechnical investigation will be included as an appendix to the report. The geotechnical evaluation report will be certified and signed by an Iowa Professional Engineer from CONSULTANT. The report will include:

- Description of subsurface profiles and material parameters used in analyses.
- Global stability, seepage and settlement analyses computations. Discussion of the geotechnical aspects of the reconstruction alternatives.

Recommendations for reconstruction design including geotechnical design parameters, fill material and placement requirements, foundation preparation and seepage control requirements, drain design requirements and erosion and scour protection design.

The *Geotechnical Evaluation Report* and geotechnical analyses and computations completed as part of the final design will be included in the Design Documentation Report (Task 6.3).

1.2 TASK 3 – SPILLWAY STRUCTURE EVALUATION AND REPORT

CONSULTANT will perform a structural evaluation of the spillway structure. For the Scope of Services of this project the spillway structure is assumed to include the gated spillway portion of the dam as well as the water retaining portions of the powerhouse structure.

NOTE: The evaluation of Interior walls, floors and generating equipment are not included in BASIC SERVICES.

The evaluation, including data, analysis, and recommendations will be summarized in a report submitted to the CLIENT. This task will include the following:

1.2.1 *Concrete Coring and Testing* – Braun Intertec will take twenty (20) cores of the concrete spillway structure and powerhouse walls. These cores will be tested to evaluate the condition of the existing concrete. The coring program will include the following:

- Braun Intertec will mobilize a field crew to perform the coring program.
- CONSULTANT will mobilize a structural engineer to the site to kick-off and coordinate the concrete sampling program.
- Once the coring program has begun collecting initial field data, CONSULTANT will organize a teleconference between the CONSULTANT's Project Manager, Braun Intertec Crew Chief, and CLIENT's Project Manager to determine if adjustments of the coring program are required due to site conditions.
- If coring program adjustments are needed, CONSULTANT will revise the Scope of Services and Compensation of the coring program and submit to the CLIENT for approval.
- A laboratory testing program that includes compressive strength testing and petrographic analysis will be developed to evaluate material properties of the existing concrete to verify its strength and durability.
- Braun Intertec will summarize findings in a concrete testing report certified by an Iowa Professional Engineer.
- The report will include:
    - Field notes and observations
    - Coring locations
    - Photographs of cores
    - Laboratory test results
    - Concrete material properties
- CONSULTANT'S structural engineers will review a draft version of the report prior to its final issue.

1.2.2 *Structural Analyses/Evaluation* – CONSULTANT will perform structural analysis of the spillway structure using results of the concrete coring/testing, foundation bedrock coring/testing and available reference drawings and engineering calculations of the dam and spillway. The following evaluations will be performed:

- Structure Stability (sliding, overturning, uplift)
    - Review of Ashton-Barnes stability calculations for concrete spillway and powerhouse
    - Determine if bedrock parameters utilized in the stability analyses are confirmed by the results of the geotechnical investigation.
- Determine if additional investigations are required to support/verify assumptions contained in the stability analyses. Investigations may include additional coring through the structure to obtain samples of the concrete/bedrock interface and bedrock immediately below the base of the structure. If additional investigations are required, CONSUSLTANT will provide the CLIENT with a fee proposal for completing this work as an ADDITIONAL SERVICE.
- Structure Strength
    - Analysis of strength of structural components of spillway structure
        - Abutment walls
        - Spillway concrete

- Spillway platform, equipment supports
- Powerhouse Exterior walls
- Gate piers
- Bridge slab
- Steel gates

1.2.3 *Structural Evaluation Report* – The structural evaluation will be summarized in a stand-alone structural report. The results of the concrete tests will be included as an appendix to the report. The structural evaluation report will be certified and signed by an Iowa Professional Engineer from CONSULTANT. The report will include:

- Description of results of concrete testing and impacts on reconstruction.
- Structural stability and strength computations and summary discussion.
- Discussion of the structural aspects of repairing the spillway structure.
- Recommendations for reconstruction/rehabilitation design including what structural components will be repaired or replaced and any new structures or modifications needed to bring the dam into compliance with current dam safety standards

The *Structural Evaluation Report* and subsequent structural computations completed as part of the final design will be included in the Design Documentation Report (Task 6.3).

1.3 TASK 3 – ARCHAEOLOGICAL SURVEY

Louis Berger Group (LBG) of Marion, IA will complete a Phase I archaeological survey for all land areas located within the impoundment area situated below an elevation of 897 feet above mean sea level. The survey will be performed in accordance with current Guidelines for Archaeological Investigations in Iowa.

1.3.1 *Existing Record Review* – LBG will review existing records currently on file at the Office of the State Archaeologist. If necessary, additional research will be conducted to attempt to identify areas that have been subject to previous ground disturbance and thus have little or no potential to contain undisturbed archaeological resources. LBG will develop an archaeological survey plan delineating survey areas which will be submitted to Stanley Consultants and the Client for review prior to commencing survey activities.

1.3.2 *Field Survey and Report* - LBG will conduct a field survey of land areas delineated in the archaeological survey plan.

1.3.3 *Survey Report* - LBG will prepare a Phase I survey report in accordance with state guidelines that describes the survey methods and findings. The Phase I report will include:

- Appropriate recommendations regarding the eligibility of each affected resource for inclusion in the National Register of Historic Places.
- Recommendations for additional archaeological study or resource mitigation if any.

For fee estimating purposes, the archaeological survey is assumed to include conducting up to 300 subsurface tests and processing up to 500 artifacts from as many as 10 different sites. The fee will decrease/increase based upon how many fewer/additional sites and artifacts are discovered during the survey.

1.4 TASK 4 – HYDROLOGIC/HYDRAULIC STUDIES:

The primary purposes of hydrologic and hydraulic studies are to:

- Establish the design flood for the Maquoketa River at the dam.
- Determine the effects of theoretical failure of Lake Delhi Dam.
- Define the dam's hazard classification.
- Establish the total required spillway capacity of the dam.

- Perform hydraulic design of the proposed reconstruction.

1.4.1 *Hydrologic Analysis* - A previous hydrologic study by Ashton Engineering established a Probable Maximum Flood (PMF) for Lake Delhi Dam. Stanley Consultants will review the criteria and parameters used to develop the PMF and perform a new PMF analysis if inconsistencies are found. If needed, the revised PMF will be established using rainfall depth and distribution methods defined in the Iowa Department of Natural Resources (DNR) Publication, *Technical Bulletin No. 16 - Design Criteria and Guidelines for Iowa Dams.*

Hydrograph routing for the PMF will be performed using HEC-HMS software. Return interval floods (100-year, 50-year, etc.) will be established using the nearby USGS Flow Gaging Station (No. 05416900) at Manchester, Iowa with Maquoketa River flows at Lake Delhi Dam determined by proportioning respective drainage areas.

1.4.2 *Hazard Classification* - The proposed dam reconstruction will meet design standards of FERC and DNR. Both FERC and DNR use hazard classification as a criterion to establish the dam's design flood. Currently the hazard classification of Lake Delhi Dam is uncertain.

CONSULTANT will use inundation mapping of the design flood and/or dam failure scenarios to establish the hazard classification of the proposed dam reconstruction for both FERC and DNR classification methods. Once the hazard classification is established for Lake Delhi Dam, CONSULTANT will use the more conservative of the FERC and DNR design floods in design of the dam reconstruction

The hazard classification analysis will be performed by the following steps:

- Develop HEC-RAS hydraulic model of the Maquoketa River and Lake Delhi Dam (See task 4.3).
- Dam failure scenarios will include:
  - High flow event failure (dam failure + design flow event)
  - Normal conditions failure (sunny day failure).
- Establish failure scenarios' flows and resulting peak water surface elevations in the downstream river channel using HEC-RAS.
- Use HEC-RAS peak water surface elevation to delineate inundation limits using ArcGIS with DNR LiDAR topographic data.
- Reference inundation extents to downstream topography and aerial imagery to assess the potential for loss of human life and property damage.
- Establish hazard classification as low, moderate/significant, or high.

CONSULTANT will present a recommendation for DNR and FERC hazard classifications for Lake Delhi Dam in the Hydrologic and Hydraulic Studies Report (Task 4.4).

NOTE: The determination of the hazard classification will follow DNR methods but will be established through engineering judgment, and will be subject to review and approval by DNR. CONSULTANT is not responsible for opinions or judgments by DNR that differ from CONSULTANT's or the CLIENT's.

Due to the interest in developing hydropower at the site, CONSULTANT will perform analysis and design in compliance with FERC standards and summarize FERC hazard classification and other significant findings in the Hydrologic and Hydraulic Studies Report.

NOTE: BASIC SERVICES do not include additional work by CONSULTANT to assist with hydropower redevelopment.

1.4.3 *Hydraulic Analysis* – CONSULTANT will use the HEC-RAS model of the Lake Delhi Dam and Maquoketa River developed by the DNR for the 2010 Report on the Breach of Delhi Dam as the basis of hydraulic analysis. CONSULTANT will review the model and backup information and revise as necessary. Revisions to the model will be documented. Once the project HEC-RAS model has been established it will be used for:

- Dam failure analysis.
- Hazard Classification.
- Analyzing spillway and dam reconstruction alternatives.
- Design of hydraulic aspects of dam reconstruction.
- Estimating water surface profiles and flood extents for 100-yr and 50-yr floods
- Evaluating hydraulic impacts of upstream bridges.
- Demonstrating proposed dam reconstruction provides sufficient hydraulic capacity to pass the applicable DNR design flood.

1.4.4. *Hydrologic and Hydraulic Studies Report* – CONSULTANT will develop a Hydrologic and Hydraulic Studies Report which will summarize:

- Hazard classification (DNR and FERC)
- Analysis methods and findings
- Hydraulic aspects of the proposed design

The report will be submitted to the CLIENT for review. Following approval by the CLIENT, the report will be submitted to the DNR for approval and other interested agencies.

1.5 TASK 5 –RECONSTRUCTION DESIGN ALTERNATIVES:

CONSULTANT will develop and assist in evaluating a minimum of three dam reconstruction alternatives and the selection of one alternative for final design and construction.

1.5.1 *Project Kick-Off Meeting* – CONSULTANT will attend a Design Alternatives Kick-Off Meeting at the project site. While visiting the project site, CONSULTANT will complete a reconnaissance and inspection of the project features. CONSULTANT's inspection team will consist of the Project Manager and the Mechanical, and Electrical discipline leads. The team will inspect the dam, downstream channel, walls, spillway structure, gates, powerhouse, electrical systems, gate and trash rack motors, access ways, and operating platforms. The inspectors will review these components for any defects or changes since the last inspection and for use in developing reconstruction strategies.

It is assumed that sufficient as-constructed drawings exist to document the spillway and powerhouse construction. If not, a 3-Dimensional radar survey of the existing concrete structures may be required to allow development of repair/rehabilitation alternatives, as well as construction drawings for the selected alternatives. If a structure survey is required, Stanley will provide the Client with a fee proposal for completing this work.

1.5.2 *Alternatives Development Meeting* – This meeting will be held at the project site following completion of initial geotechnical, structural and hydrologic/hydraulic analysis. Attendees will include CONSULTANT, CLIENT, and other stakeholders and representatives from regulating agencies (DNR, USACE, FWS, SHPO and others) designated and invited by CLIENT. The objective of the meeting will be to develop and discuss potential reconstruction and repair alternatives and ultimately select three to four alternatives for further development and evaluation by CONSULTANT.

1.5.3 *Alternatives Refinement* – CONSULTANT will refine and evaluate reconstruction alternatives by developing conceptual designs and cost estimates for each alternative.

Conceptual designs will be shown on engineering exhibits which will include plan, profile, and section views of each reconstruction alternative.

Conceptual estimates of probable construction costs for each alternative will be developed using construction material quantities estimated from the engineering exhibits.

NOTE: Estimates of probable construction costs at the conceptual level are approximate and subject to change based on design refinement, revisions, schedule, and/or material price fluctuations.

CONSULTANT will compare and evaluate the alternatives using a set of project specific parameters. With input from CLIENT, CONSULTANT will develop a set of parameters for a comparative evaluation of the reconstruction alternatives using a scoring matrix. Weights and scores used in the alternatives evaluation will be developed collaboratively with CLIENT. Parameters may include:

- Discharge capacity
- Construction and operation and maintenance costs
- Regulatory compliance
- Day-to-day pool level control
- Frequency of auxiliary spillway activation
- Impacts to hydroelectric potential
- Impacts to recreation
- Constructability
- Construction schedule
- Risks to other stakeholders
- Public perception

1.5.4 *Alternative Selection Meeting* – This meeting will be held at the project site. CONSULTANT will present conceptual level designs and estimated costs for each of the alternatives. A preliminary scoring matrix will be presented and input solicited from the client on scores and weights. The objective of the meeting will be to select the preferred alternative for reconstruction.

1.5.5 *Alternatives Analysis Report* - The alternative designs, cost estimates, analysis, and evaluation will be summarized in an *Alternatives Analysis Report*. A draft version will be submitted to CLIENT for comment and a final version will be provided to the CLIENT for documentation of the alternative selection process.

1.7 TASK 7 – PERMITTING AND AGENCY COORDINATION

CONSULTANT will evaluate permitting implications of the proposed project. Reconstruction of the spillway will require approval from the DNR and the United States Army Corps of Engineers (COE) under the Rivers and Harbors Act (Section 10) and the Clean Water Act (Section 404).

Permitting will be through the COE/DNR Joint Permit process. CONSULTANT will prepare an application package for submittal to COE with copies sent simultaneously to both the Floodplain and Sovereign Lands Section at DNR. Included in the submittal will be a separate packet with the forms and information specific to the Dam Construction Permit. It is understood that the CLIENT will directly pay for any applicable fees associated with necessary permits.

It is assumed that a single set of permit documents will be submitted covering both the embankment reconstruction and the spillway structure rehabilitation.

CONSULTANT will obtain and prepare all necessary permit application forms and exhibits. Completed permit documents will be provided to the CLIENT for review and submittal to regulating agency. The permitting process will include:

1.7.1 COE Section 404 - The application will be prepared with the assumption that COE will require an Individual Section 404 Permit. COE will prepare and send out a Public Notice giving agencies, organizations, and individuals an opportunity to comment on the project. The applicant will be required to respond to any expressed opposition. CONSULTANT will prepare permit application documents and assist with comment responses.

1.7.2 DNR Section 401 - Section 401 Water Quality Certification (aka 401 Cert) specifically addresses the project's potential impacts to water quality that will have to be avoided, minimized and possibly mitigated. CONSULTANT will prepare permit application documents.

1.7.3 DNR Sovereign Lands - It will be necessary to obtain a *Sovereign Lands Construction Permit* for this project. As indicated, the Joint Application process will include a copy of the application to the Sovereign Lands Section for their review. This process will include a review within DNR by threatened & endangered (T&E) species staff and DNR fisheries personnel. The T&E review will identify any state-listed plant or animal species known in the project area. It will be necessary to assess the likelihood that any of these species will be impacted by the project. The fisheries personnel will likely request the proposed project includes consideration for fish passage. It may not be feasible to provide an external "fish ladder" but the issue will likely have to be addressed. Stanley Consultants will prepare permit application documents and assist with DNR coordination.

1.7.4 DNR Floodplain Development Permit - Construction in a floodplain requires a floodplain permit oriented to dam construction which includes complete application forms and providing information specific to dam construction. Stanley Consultants will prepare permit application documents and assist with DNR coordination. Submittal requirements include:

- Completed and signed Water Storage Permit Application (Task 7)
- Two sets of certified plans (Task 6)
- Engineering Design and Hydraulics & Hydrology Report (Tasks 4)
- Soil & Foundation Investigation Report (Task 1)
- Structural Evaluation Report (Task 2)
- Sedimentation rate assessment (Task 4)
- Gated low level outlet design (Task 4 and 6)
- Hazard assessment (Task 4)
- Summary of Engineering Data (Tasks 4 and 5)

1.7.5 State Historic Preservation Office (SHPO) - A Phase I Cultural Resource Survey will be performed that will encompass the entire former lake bottom. An archaeological report will be prepared that will summarize the findings and address potential project impacts to both historic and prehistoric resources (Task 3). It will be necessary to develop a Programmatic Agreement with the State Historic Preservation Office that will provide a plan for avoiding, minimizing or mitigating impacts to any significant resources encountered. Stanley Consultants will assist with development of the Programmatic Agreement and coordination with SHPO.

1.7.6 U.S. Fish & Wildlife Service (FWS) - FWS will be sent a Public Notice by COE. FWS will review the project for potential impact on federally-listed T&E species. Bald eagles are no longer a listed species but if any potential impacts are identified, application will be made to FWS for a Bald Eagle Permit. The project area will be reviewed for the potential for federally-listed T&E species to occur in the area. If any potential exists, the Moline, Illinois Field Office of FWS will be contacted during preparation of the application. Any T&E concerns identified by FWS will be addressed in the application and the Moline office will be sent a copy of the application package at the same time it is submitted to the COE. CONSULTANT will assist with coordination with FWS.

Additional permits that may arise that are not identified in Task 6 are considered ADDITIONAL SERVICES and not included in BASIC SERVICES.

NOTE: Any local permits required for construction will be the Contractor's responsibility.

3.0 ADDITIONAL SERVICES:

Services requested by the CLIENT that are NOT included in BASIC SERVICES, shall constitute ADDITIONAL SERVICES. ADDITIONAL SERVICES shall be authorized by the CLIENT prior to the commencement of services.