FIFTH FERC PART 12 INSPECTION REPORT

JACKSON BLUFF HYDROELECTRIC PROJECT

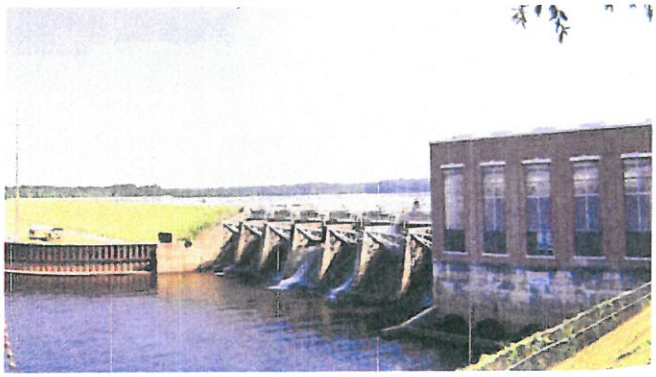
CITY OF TALLAHASSEE, FLORIDA

FERC PROJECT NO. 2891

APRIL 2015

(Due by May 30, 2015)





CITY OF TALLAHASSEE, FLORIDA
JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. P-2891

FIFTH FERC PART 12D INSPECTION REPORT

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City of Tallahassee
Jackson Bluff Hydroelectric Project
FERC Project No. 2891

Fifth FERC Part 12D Inspection Report

General Introduction

This report summarizes the results of the Fifth on-site inspection and PFM Review of the Jackson Bluff Hydroelectric Project No. 2891 Project (Project) located on the Ochlockonee River and the subsequent review of engineering evaluations that are required under the Federal Energy Regulatory Commission (FERC) Regulations for a Part 12D Inspection.

Pursuant to Part 12, Subpart D, of Title 18 of the Code of Federal Regulations, owners of hydropower projects subject to jurisdiction of the FERC are required to retain the services of an Independent Consultant at five-year intervals to inspect the project works and evaluate and identify any actual or potential deficiencies that might endanger public safety. The scope of the field inspection, contents of the inspection report, and the implementation of corrective measure recommended by the Independent Consultant are covered in Part 12, Subpart D of the Commission's regulations. In addition, following the guidelines of Chapter 14, "Dam Safety Performance Monitoring Program" we have reviewed the previously prepared Standard Technical Information Document and the Potential Failure Modes Analysis Report.

This is the Fifth Part 12D Inspection that has been made for the Project and includes the period of record from September 23, 2009 to August 2014. Detailed reviews of the previous two Part 12D Inspection Reports including reviews of the Potential Failure Modes and STID were performed and references from those reports are made, as appropriate, in this Fifth Part 12D Inspection Report (Report). The initial Potential Failure Mode Analysis (PFMA) was conducted between July 15 and July 16, 2003, during the Third Part 12D Inspection in accordance with FERC Guidelines. A review of the Second PFMA occurred in 2009, and the Initial PFMA for the Auxiliary Spillway in 2009 was also performed and referenced as appropriate. A review of the PFMA Report and the Supporting Technical Information Document (STI Document) was performed as a part of this Fifth Part 12D Inspection Report. In addition to the Independent Consultant, representatives of the City of Tallahassee, Florida Department of Environmental Protection (Krystle Hoenstine and Art Sengupta), and FERC were present during the PFMA review. The Category II loading conditions were reviewed and discussed in detailed with step-by-step failure progressions. Data from the original PFMA and the previous PFMA review were considered and updated recommendations for these PFM issues are made in Section 3 of this Report. A number of original PFMs were separated into individual failure modes to reflect the new criteria from FERC.

A detailed site visit was made on August 20, 2014, by W. B. Smith, P.E., Independent Consultant for the Fifth Part 12D Inspection of the Project with representatives of the City of Tallahassee, and FERC.

Presented in the following sections is the Fifth Part 12D Inspection Report in accordance with the revised FERC Guidelines and Regulations, Chapter 14 – Monitoring the Performance of Dam, Appendix H, dated July 1, 2005, as attached to the FERC Letter (undated), a copy of

which is included in *Appendix A*. Minor modifications to the format have been made to accommodate site-specific conditions. Reference documents reviewed or used in the preparation of this report are listed on the reference page. Tables and figures prepared either by the initial Independent Consultant or by HISINC, L.L.C. are provided in this document. The STI Document presents the basic data and analyses used to evaluate the hydraulic capacity, hydrology, stress analyses and structural stability of the Project. The STI Document was reviewed during the preparation of the Fifth Part 12 D Inspection Report.

A copy of the letter from the FERC dated May 5, 2014, approving Mr. W. B. Smith, P.E., CFM as the Independent Consultant is included in *Appendix B*.

All elevations discussed in this report are referenced to North American Vertical Datum of 1988 (NGVD).

1.0 Findings and Recommendations

1.1 Findings

1.1.1 Summary Assessment of PFMA Report

The PFMA Report has been reviewed and is believed to present a true and accurate representation of the results of the discussions, review of technical documents, and consensus of the opinions of the 2003 PFMA Team and updated review in 2009 were performed during the current PFMA review session. The Fifth Part 12D Inspection tour was conducted followed by a PFMA review. The PFMA review was performed and discussions were held with FERC, City of Tallahassee (CITY) Staff and the Independent Consultant. The Jackson Bluff Hydropower Project is considered a safe project. Project structures are sound and meet the functional and various loading requirements of the Project. Further discussion of the PFMA Review is presented in Section 3.0.

1.1.2 Summary Assessment of the Surveillance and Monitoring Plan

The current Surveillance and Monitoring Plan is adequate for the Jackson Bluff Hydropower Project.

1.1.3 Summary of Field Inspection Findings

A pre-site visit was performed by the Independent Consultant on August 19, 2014. The formal site visit with FERC and CITY staff of the outside areas, and a tour of the powerhouse were performed on August 20, 2014. Photographs were taken during both inspections. The August 20th inspection began at the northeast side of the Powerhouse, continued across the gated spillway section, across the southeast end of the non-overflow embankment to the southerly end of the new auxiliary spillway. Then the group traversed down the slope of the embankment near the southerly toe of the auxiliary spillway and continued north along the downstream side of the auxiliary spillway. During this segment of the tour the group observed the existing toe drain system with discharge culverts at various locations (modified in 1997-1998). Upon reaching the north end of the auxiliary spillway the group observed the north abutment area of the non-overflow embankment and then returned along the crest of the embankment and back to the powerhouse. A tour of the powerhouse was performed on August 20, 2014. A boat tour

of the upstream side of the dam and spillway and a portion of the upstream lake area was also performed.

In general, the entire outside perimeter of the Project was found to be in good condition.

The Independent Consultant also performed a detailed visual, but not a 10-Year "close-up" inspection of the tainter gates, and visually observed the operation of two tainter gates – one operated with the standby generator and the other on normal power sources.

The dewatering of the units since the previous Part 12D Inspection has been as follows:

- March 2009
- An upcoming short term dewatering is being scheduled in late spring 2015 for Unit No. 3

There are several areas of minor concern, primarily related to continued monitoring and/or maintenance that were identified by the Independent Consultant which will be discussed in more detail in Section 5. There are no issues that affect the structural integrity of the Project.

1.1.4 Summary of O&M Status

The current operation and maintenance programs are adequate and satisfactory for the Jackson Bluff Hydropower Project and are being performed in accordance with the policies and procedures established at the completion of design/construction phases or as modified during the past years of operation.

CITY personnel are knowledgeable regarding operations of facility and perform the maintenance inspections and remedial repairs in a timely and very satisfactory manner in accordance with the Preventative Maintenance Plan that is in place relating to operation of the Jackson Bluff Hydropower Project. The CITY operating and maintenance personnel walk the entire site each day, including the powerhouse, gated spillway section, auxiliary spillway, toe drains on embankment area and dam abutments. The emergency generator is operated every Monday. On a bi-weekly basis the piezometers are read and a dam checklist is completed. Periodically, general preventative maintenance (PM) of the units is performed. The PM depends on the equipment and some is performed on a monthly, quarterly, or annual basis.

The scope of instrumentation and their reading schedules are appropriate for this Project. As discussed during the PFMA Review, the CITY just began the annual instrument readings in 2014 for the Auxiliary Spillway project, so there is not a trend to compare with this report.

1.1.5 Summary Assessment of Supporting Technical Information Document

The STI Document prepared for the 2014 PFMA session was more than adequate for use during the preparation and review session held on August 20, 2104. Documents in the STID were supplemented by FERC file information that was provided by Mr. John Breslin of ARO based on the initial draft of the PFMA review prepared by the IC. In the Initial STI Document an assembly of the basic data and analyses used to evaluate the

hydraulic capacity, hydrology, stress analyses, and structural stability of the Project are presented. As part of the Fifth Part 12D Inspection, the Independent Consultant reviewed the STI Document and determined some technical corrections were required as well as adding supplemental information from the PFMA review.

1.2 Conclusions

1.2.1 The Jackson Bluff Hydropower Project is suitable for continued safe and reliable operation.

1.2.2 The Project Description is accurate.

1.2.3 The conclusions from the PFMA Report, as amended from the current PFMA review are accurate to the best of our ability. In summary, based on the recent design and construction of the Auxiliary Spillway, the quality control effort during that construction, the location of the Project and the ongoing daily inspections and maintenance issues, the Project has no immediate structural needs that would cause a loss of pool or cause a downstream effect. There are now 56 PFMs identified for the Project; separating previous combined PFMs, and adding the applicable PFMs from the Auxiliary Spillway. There are no specific Highlighted (Category 1) Potential Failure Modes were identified for the Project, except for the three (3) combined Category I/III relating to the current design being finalized by Mead & Hunt for approval by FERC for the stabilization of the downstream slope of the dam embankment as discussed in detail and initiated as an action item during the PFMA review. The design, construction, construction management, documentation and existing operations and maintenance all contribute to the conclusion the Project is safe. Project structures are sound and meet the functional and various loading requirements of the Project. There are fifteen (15) Category II PFMs relating to the ongoing instrumentation and/or monitoring of Project features. There are two (2) stand-alone Category III PFMs. All of the other thirty-six (36) potential failure modes should remain classified as Category IV.

1.2.4 The current Surveillance and Monitoring Plan is adequate for the Project.

1.2.5 The field inspections that were performed were complete and all components of the Project were reviewed.

1.2.6 The Operation and Maintenance Program for the Jackson Bluff Hydropower Project is adequate and is being performed in accordance with the policies and procedures originally established or modified during the course of the Project.

1.3 The STI Document is complete and adequate for use in future Part 12D Inspections. There are additional documentations that should be added to the STI Document, as documented throughout the report.

1.4 Recommendations

1.4.1 The Jackson Bluff Hydropower Project is recommended to continue operation under status quo.

1.4.2 There are no recommended changes relating to the Project Description.

1.4.3 The PFMA Report, as originally filed in March 2004, presented a summary of the potential failure modes in Table 3, as attached. As a result of the more detailed descriptions required by FERC, the original table is recommended to be replaced with the revised Table 3-1 which separates combined PFM conditions into individual PFMs. Three PFMs were changed from Category I to Category I/III since the design is still in progress. One Category I was changed to a Category II based on monitoring. Eight PFMs were changed from a Category II to Category IV. Seven PFMS were changed from a Category III to Category IV. Four PFMs were changed from a Category IV to Category II based on current monitoring. There was one new Category III and four new Category IV PFMs. Based on the PFMA Review, there are no other recommended changes to the current listing.

The two (2) new Failure modes are recommended to be added to the PFMA list as follows:

55. Barge/Boat Impact - This PFM relates to the potential for barge or boat impact on the Gated Spillway section. This failure mode was discussed during the PFMA review. This is recommended to be assigned a Category IV rating since there are no large barges or boats on the reservoir.

56. Lightning Strike of Powerhouse - This PFM relates to the potential for a lightning strike on the powerhouse which would cause an explosion resulting in a failure of controls to operate either the tainter gates or the wicket gates resulting in a "controlled" release (maximum water passage capacity). Though this is a damage issue, it would not result in an uncontrolled release of the reservoir. This failure mode was discussed during the PFMA review. This is recommended to be assigned a Category IV rating.

1.4.4 The current Surveillance and Monitoring Plan is recommended to be continued in the same manner for the Project.

The Field Inspections and Instrument Monitoring are recommended to be continued in the manner currently established. The daily inspections of the entire Project on a daily basis is highly commendable.

1.4.5 The Operation and Maintenance Program for the Jackson Bluff Hydropower Project is recommended to continue in accordance with the policies and procedures originally established or modified during the course of the Project.

1.4.6 The STI Document is recommended to be used in future Part 12D Inspections with the revision of Table 3-1 and the amended pages updated for additional historical analyses that were discussed during the PFMA Review. The Stability Analysis for the powerhouse performed by Ebasco in November 1992, in response to FERC's May 5, 1992 comments, also needs to be added to the STID library. Also the tainter gate remediation project that was performed following the 2004 Part 12D Inspection needs to be addressed in the STID.

In addition to the Recommendations, the following statements are added to this Fifth Independent Part 12D Report:

A. *Corrective Measure Required for the Structure*

1) Complete Design, Obtain FERC Approval, and Implement the Interim Remediation on the approximately 40 foot area of Embankment including installation of Filter and in Area near Culvert No. 7.

2) During the PFMA Review, it was discussed that the City would obtain stockpile of #57 rock for remediation of embankment if needed in an emergency. The City has obtain ASTM C-33 Sand instead of the #57 rock as the sand is more compatible with the base soils.

B. *Corrective Measures Required for the Maintenance or Surveillance Procedures*

No new or additional measures are required for the Maintenance or Surveillance of the Project Works. The Daily walk-throughs of the entire Project area by CITY staff are more than adequate in combination with the survey monuments, piezometer data, and toe drain flow monitoring that are already implemented.

C. *Corrective Measures Required for the Method of Operation of the Project Works*

No new or additional measures are required for the operation of the Project Works

D. *Schedule to Carry Out Each Corrective Measure*

All items should be performed as soon as possible when river flows and weather conditions permit access.

E. *Any New or Additional Monitoring Instruments, Periodic Observations, or Other Methods of Monitoring Project Works or Conditions that may be Required.*

There are no New or Additional Monitoring Instruments, Periodic Observations or Other Methods of Monitoring Project Works or Conditions that are recommended. The Daily walk-throughs of the entire Project area by CITY staff are more than adequate in combination with the survey monuments, piezometer data, and toe drain flow monitoring that are already implemented. ***If*** there are any new cracks or observed changes in existing cracks in the powerhouse area, it would be recommended to add crack monitoring devices to those specific areas and monitor movement.

1.5 Certification

1.5.1 List of Participants

The following CITY and FERC personnel accompanied the Independent Consultant during the inspection, and/or the FERC Annual Inspection.

Triveni Singh, City of Tallahassee, Manager of Power Production
Karl Bauer, City of Tallahassee, Engineer
Dave Mordecai, City of Tallahassee, Plant Manager
David McCranie, City of Tallahassee, Operator
Bobby Stafford, Jr., City of Tallahassee, Operator
John Breslin, FERC, ARO, Engineer
Elise Dombeck, FERC, ARO, Engineer

1.5.2 Statement of Independence

I hereby declare that this report was prepared in accordance with FERC Order 122 dated March 1, 1981 and Paragraph 12.37 (c) (7) to the best of my ability, and that all conclusions and recommendations therein were prepared independently and exclusively by the undersigned consultant.

1.5.3 Signature

W. B. Smith, P.E.



FLORIDA TEMPORARY P.E. PENDING

SIGNED STAMPED 5-14-15

2.0 Project Description

2.1 Physical Features of Project

2.1.1 Dam

Jackson Bluff Dam is located on the Ochlockonee River about 66 miles upstream from its mouth at the Gulf of Mexico and approximately 20 miles west-southwest of the City of Tallahassee. The dam consists of three major structural elements: a powerhouse, a 199-foot-long gated spillway, and a 3,600-foot-long earth embankment. The crest of the embankment is at elevation 77 feet and the typical embankment height is about 40 feet. A 950-foot-long emergency spillway and a 562-foot-long fuse plug, constructed in 1984, that were located on the right end of the earth embankment have been "retired" and replaced with the remote dike. The Auxiliary Spillway project in 2010 included constructing an 825-foot-long concrete auxiliary spillway to replace the existing fuse plug and emergency spillways. Upstream and downstream sheet piling cutoff walls were placed along with a drainage blanket beneath chute slabs that conveys seepage to the gated spillway. A Type III hydraulic-jump stilling basin which is 33 feet in length was constructed. The existing fuse plug and emergency spillway were isolated by a rim dike using on-site materials excavated for spillway construction. The reservoir upstream of the dam, known as Lake Talquin, has a surface area of about 10,200 acres and a gross volume of about 150,000 acre-feet at the normal maximum water surface elevation of 68.5 feet.

For additional description of the Development, see the *Jackson Bluff Supporting Technical Information Document* (STID).

Plan and sectional drawings are provided in *Appendix C*.

2.1.2. Spillway

There is a 199-foot long gated spillway structure for the Project that includes seven (7) tainter gates. There is also a 'new' 825 foot long Auxiliary Spillway located to the west of the gated spillway. Combined these two spillway structures that will pass 1/2 of the PMF equivalent to the IDF. These two structures were included in the scope of this inspection. The Project uses Lake Talquin upstream of the existing dam and powerhouse for a water source for generation.

2.1.3 Powerhouse

The powerhouse joins the bluff on the left abutment. It is an indoor-type plant with a brick superstructure. The powerhouse is 132 feet long and 33 feet wide. The powerhouse was constructed in 1929 and is on the National Register of Historic Places. The substructure containing the water passages is a reinforced concrete structure approximately 62 feet wide (including the intake trashrack piers by approximately 102 feet long (excluding the stairway and slab on the east end). Three (3) new turbine-generator units were installed in 1986. Two units are rated at 4,933 KV at 0.9 power factor and operate at 150 rpm. The small turbine is rated at 3,820 kVA at 0.9 power factor and operates at 180 rpm. The total installed capacity is 12.3 MW. The powerhouse floor is at Elev. 58.5.

Access to the powerhouse is through the operating deck at Elev. 58.5 by way of a vehicular ramp from the Project access road.

The powerhouse founded on firm limestone rock. Limited Plan and sectional drawings of the powerhouse are provided in *Appendix C.*

2.1.4 Intake and Outlet Works

The Project intake and outlet works consist of the powerhouse intake, powerhouse water passages and the downstream river channel as shown on the plan and sectional drawings in *Appendix C.*

2.1.4.1 Powerhouse Intake

The power intake for the three turbine-generator units consists of six submerged openings on the upstream side of the powerhouse. For each of the three units, two openings combine to supply water to the respective unit. A trashrack is located in from of each opening. There are slots for the stoplogs downstream of the trashrack, followed by slots for the head gates.

2.1.4.2 Powerhouse Water Passage

As water passes through the head gate slot it is narrowed in area to enter the distributor section of the turbine unit and passes through the wicket gates. The water falls onto the turbine runner and discharges into the draft tube section of the water passage. After the water passes through the turbines it is bifurcated and discharges through the six draft tube outlets.

2.1.4.3 Draft Tube Gates

There are no draft tube gates for this Project. Tailwater is normally at Elev. 35.0. The bottom of the draft tube is at approximately Elev. 16.0 to 17.0. There is always some water in the draft tube section. If the water passage would need to be dewatered, temporary stop logs would have to be installed.

2.1.4.4 Tailrace Channel Training Walls

The downstream channel receives the powerhouse discharge as well as any discharge through the gated spillway section. There is a South Training Wall that extends approximately 175 feet downstream of the powerhouse to retain the left bank of the river. There is an arched sheetpile training wall on the right downstream bank that guides the water discharge from the gated spillway section.

2.2 Hazard Potential Classification

The Hazard Potential Classification was reviewed. The Jackson Bluff Hydroelectric project has been classified as high hazard based on the findings of breach analyses studies, and the Inflow Design Flood (IDF) has been determined to be equal to one half of

the PMF. No change in classification of the dam has been made since the last Part 12 Inspection and no change is recommended.

2.3 Summary of Standard Operating Procedures

2.3.1 The Jackson Bluff Hydropower Project is a run-of-river project, used primarily by CITY for hydropower generation capability, and is governed by the FERC License issued to the CITY that established procedures for hydropower operation. The upstream reservoir, Lake Talquin, is extensively used for recreation and for some flood control.

2.3.2 Reservoir rule curves by season

The project discharge is regulated to maintain a normal maximum reservoir elevation of 68.5 feet +/- 1 foot. The minimum allowable level of reservoir drawdown is 66.5 feet. In an emergency, the City may request approval to drawdown the reservoir below the 66.5 feet level.

2.3.3 Standard gate operation procedures

Each of the seven tainter gates is operated by its own electricity driven, cable-hoisting gantry mounted on rails on the spillway operating deck. Adjacent hoists can be relocated, as needed, to act as back-up systems. All tainter gates are typically opened the same increment before increasing discharge from a single gate.

The Plant Control System (PCS) provides the status of all generation equipment and components and allows the units to be controlled from the powerhouse. One operator and maintenance person is stationed at the plant site during normal business hours. There is no reservoir rule curve for the Project. The Order Amending the License dated September 23,1983 deleted Article 35 of the License and amended Article 34 of the License to permit operation as a baseload run-of-river mode, rather than a peaking mode as originally proposed in the License application. Article 34 of the Amended License states that the Licensee shall at all times act to minimize the fluctuation of the reservoir surface elevation, i.e. maintain a continuous discharge from the project which approximates the instantaneous inflow to the Project reservoir. The inflow is monitored from upstream USGS gages.

2.4 Modifications Conducted for Project Safety

Since 2009 the auxiliary spillway project has been added to the Project works which replaced the existing fuse plug and emergency spillway. The auxiliary spillway is able to pass a flow of 69,900 cfs when the reservoir level is as its maximum stage of 77.0 feet (NGVD). The project began in August 27, 2010 and was completed on August 18, 2011. The auxiliary spillway changed the downstream slope from 2H:1V to 3H:1v, installed a sheet pile cutoff wall, and installed a blanket drain. A two stage filter system was placed beneath the stilling basin, chute slabs and upper slabs.

2.5 Flood History

2.5.1 Flood of Record, PFM, IDF

The flood of record at the project occurred on September 25, 1969, with a recorded flow of 89,400 cubic feet per second (cfs).

The Project's IDF has been determined to be equal to approximately one-half of the PMF. The PMF has a calculated inflow of 309,000 cfs. The level of the impoundment is projected to reach Elev. 77.0 (NGVD), with the tailwater at Elev. 57.3 (NGVD) during the IDF. The combination of the gated spillway (69,000 cfs) and the Auxiliary Spillway (80,000 cfs) equates to the ability to pass the IDF (148,000 cfs +/- from Mead & Hunt DAMBRK model)

2.5.2 Freeboard Adequacy

The Jackson Bluff Hydroelectric Project had a zero freeboard capacity of 85,315 cfs before the construction of the Auxiliary Spillway; and now has a zero freeboard capacity equivalent to the IDF.

Included in *Appendix C* are the Area-Capacity Curve for the Reservoir, Spillway Discharge Rating Curve and the Tailwater Rating Curve.

2.5.3 Peak spillway discharge during last 5-year period

The peak flow from the Jackson Bluff Project since the previous Consultant's Safety Inspection Report had a flow of approximately 43,900 cfs on April 4, 2009 as a result of an un-named storm that occurred on April 2, 2009.

2.5.4 Peak Reservoir elevation during last 5-year period

The peak level of the impoundment occurred on April 3, 2009 at Elev. 69.75 with a tailwater at Elev. 24.26.

3.0 Discussion of Potential Failure Mode Analysis Report

3.1 Introduction and Background

The PFMA was conducted as part of the 5-year FERC Initial Independent Part 12D Inspection of the Jackson Bluff Hydropower Project in accordance with FERC "Engineering Guidelines for Hydropower Projects," Chapter 14 on Sept 23-25, 2003.

FERC License Project No. 2891 was issued to the CITY on July 14, 1982, for a period of 40 years. Under this authorization the CITY has operated and maintained the Jackson Bluff Hydropower Project. The CITY has been operating and maintaining the Project since 1982. The FERC License Project No. 2891 had previously had the initial PFMA conducted for the Jackson Bluff Hydropower Project in 2004, and its first review in 2009. This review during the Fifth Independent Part 12 D Inspection served as the second opportunity to conduct a review of the PFMA Analysis and to update the PFMs in accordance with the latest FERC guidelines.

The PFMA review was conducted prior to the on-site inspection of the Project on August 19, 2014. The review session was conducted by the Independent Consultant, Mr. W. B. Smith, P.E., CFM and included the following members of the Inspection Team as follows:

FERC Atlanta Regional Representative:

Mr. John Breslin, P.E., FERC Office of Energy Projects, Division of Dam Safety & Inspections, Atlanta Regional Office - 3700 Crestwood Parkway Suite 950, Duluth, Georgia 30096. Phone (678) 245-3026. Ms. Elise Dombeck, FERC Engineer

City of Tallahassee Representatives:

Mr. Triveni Singh, Manager Power Production - 2602 Jackson Bluff Road, Tallahassee, Florida 32304, Phone (850) 891-5532; Mr. Karl Bauer, Engineer - 667 Port Leon Drive, St. Marks, Florida 32335, Phone (850) 891-5977; Mr. David Mordecai, Plant Manager, Mr. David McCranie and Mr. Bobby Stafford, Jr. - Plant Operators.

Florida Department of Environmental Protection:

Mr. Art Sengupta (Dam Safety) and Krystle Hoenstine (Attorney)

3.2 Assessment of Potential Failure Mode Analysis Review

Major Findings and Understandings from PFMA Review

The Jackson Bluff Hydropower Project was originally evaluated as a four-part structure during the PFMA – Powerhouse, Gated Spillway Section, Embankment, and Fuse Plug and Emergency Spillway. In 2011 a supplemental PFMA was performed for the Jackson Bluff Auxiliary Spillway Project.

The review team during the Fifth Part 12D Inspection discussed the on-going monitoring that is performed by CITY. A new PFM spreadsheet was developed that "split" a number of PFMs into individual PFMs to reflect the more defined step-by-step potential failure

mechanisms for each type of failure (sliding, overturning, uplift) on each specific structure (powerhouse, gated spillway section) and combined the original PFM with the Supplemental Auxiliary Spillway Project into one complete set of PFMs. The following paragraphs describe the Major Findings and Understandings that resulted from the PFMA Review session. A new working PFM spreadsheet was developed with all of the updated information on one documents and was submitted for review comments prior to assembling the final Part 12D Inspection Report. A completely new numbering sequence is used in this report.

Table 2

Potential Failure Mode Categories

Category	Description
I. Highlighted Potential Failure Modes	Those potential failure modes of greatest significance considering need for awareness, load probability, magnitude of consequence, and likelihood of adverse response (physical possibility was evident, fundamental flaw or weakness is identified, and conditions and events leading to failure seemed reasonable and credible) are highlighted.
II. Potential Failure Modes Considered But Not Highlighted	These are judged to be of lesser significance and likelihood. Each potential failure mode is described and included with reasons for and against the potential occurrence of the failure mode. The reason for the lesser significance is summarized.
III. More Information or Analyses Needed in Order to Classify	These potential failure modes to some degree lacked information to allow a confident judgment on significance and thus a dam safety investigative action or analyses is recommended. Because these required action before resolution, they may also be highlighted.
IV. Other Consideration (Potential Failure Mode Ruled Out)	Potential failure modes may be ruled out because the physical possibility does not exist, information came to light which eliminated the concern that had generated the development of the potential for the failure mode, or the potential failure mode is clearly so remote or unlikely as to be non-credible or reasonable to postulate.

Due to the conservative nature of the original design, the quality control effort during construction, the location of the Project and the size of the components of the Project, no highly realistic potential failure modes were identified for the Project.

3.2.1 PFM 3.1 – Powerhouse – Sliding Failure Mode - Unusual Condition - Sliding of the powerhouse along a weak foundation plane during flooding conditions -multiple gates are unable to be opened - due to trash or other type of gate operating failure - Reservoir is exceeding normal head above Elev. 69.5 - Unusual Condition - Category II

3.2.1.1 Description of PFM 3.1 – Powerhouse Sliding Failure (***Taken from PFMA Report and enhanced by Review Team***)

Unusual Loading: Due to the complicated geology in the area of the Jackson Bluff Hydroelectric Project, there is the possibility of a weak foundation plane underneath the powerhouse. If the driving forces during flooding conditions are great enough, a sliding failure of the powerhouse could occur at this weak foundation plane. The 1957 failure report describes the porous limestone bedrock with embedded solution cavities - Tampa Foundation. The progression of this failure mode would be during conditions approaching the PMF the powerhouse becomes unstable due to a decrease in the shearing resistance of the limestone layer because over time it is solutioning out and the water continues to work through the layer making discontinuities. At the time of the PMF the hydraulic head on the bedrock increases thus there is less resistance in the bedrock and the powerhouse begins to slide.

Positive Factors (factors suggesting failure mode is more likely): 1) The powerhouse is founded on a possible weak foundation plane underneath the powerhouse. 2) The 1957 failure report describes the porous limestone bedrock which could solution over time, resulting in less resistance in the bedrock and the powerhouse begins to slide.

Negative Factors (factors suggesting failure mode is less likely): 1) Routine daily visual inspection by staff to identify cracking and differential movement that may indicate the potential of a sliding plane, 2) Maintaining the upstream pool to the crest of the tainter gate spillway will reduce sliding forces and likelihood of a failure, and 3) Control points were established in 2006 along the powerhouse, spillway piers, upstream approach wall and along downstream retaining walls to identify any trends in movement of these structures.

3.2.1.2 Assessment of Risk Reduction Opportunities – Based on the original design and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor the powerhouse sliding.

3.2.1.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category II since CITY does regular and periodic monitoring of the powerhouse.

This failure mode remains classified as Category II

3.2.2 PFM 3.2 – Concrete deterioration and/or reinforcement corrosion in the powerhouse during flooding conditions leading to an overstress condition failure.

3.2.2.1 Description of PFM 3.2 – Concrete Deterioration and/or reinforcement corrosion in powerhouse (***Taken from PFMA Report and enhanced by Review Team***)

Usual Loading: There is a potential for concrete deterioration and/or corrosion of steel reinforcement in the powerhouse structure. Such degradation of the powerhouse structure under flooding conditions could lead to an overstress failure resulting in unsafe conditions. Either an impact or freeze/thaw or crack would create a crack in the concrete, rust color showing out of crack due to corrosion of reinforcing steel; leads to an overloading condition of the reinforcement - tension loading - that causes a portion of the powerhouse to fail.

*Positive Factors (*factors contributing to potential failure*): 1) The age of the powerhouse lends to potential degradation of the concrete and 2) the continuous exposure to surrounding water provides atmosphere for corrosion of exposed reinforcing steel.*

*Negative Factors (*factors prohibiting potential failure*): 1)* Concrete deterioration and/or reinforcement corrosion can be identified by staff during their routine daily inspection of the Project structures and 2) Prompt repair of significantly deteriorated areas can occur in a short time period, avoiding potential failure.

3.2.2.2 Assessment of Risk Reduction Opportunities – Based on the original design and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. The daily monitoring and maintenance practices of the CITY staff are commendable and reduce the potential risk for failure.

3.2.2.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of the Project Review Team and ARO staff that this failure mode should be re-classified as a Category IV since this is a highly unlikely failure mode and the CITY does regular daily walk-throughs of the powerhouse, and especially after a flooding event.

This failure mode is re-classified as Category IV.

3.2.3 PFM 3.3 – Build-up of debris (primarily duck weed) in front of the trashrack resulting in a collapse of the trashrack, triggering high vibrations of the turbine-generator unit, concrete failure, and a loss of the reservoir

3.2.3.1 Description of PFM 3.3 – Powerhouse Trashracks (***Taken from PFMA Report and enhanced by PFM Review Team***)

Usual Loading: Powerhouse Trashracks (4.3.1) – Failure to clean the area in front of the trashrack could lead to the buildup of debris and the eventual collapse of the trashrack. This could trigger high vibrations of the turbine-generator unit, which

might in turn result in a failure of the powerhouse concrete. If this failure were significant, it could result in an uncontrolled release of the reservoir.

*Positive Factors (*factors contributing to potential failure*): 1) The design of the trashrack and lack of ease of removal of trash lends to potential buildup and overloading of the trashrack and 2) the continuous exposure to surrounding water provides atmosphere for corrosion of exposed trashrack steel.*

*Negative Factors (*factors prohibiting potential failure*): 1)* Trash build-up can be identified by staff during their routine daily inspection of the Project structures and 2) Prompt removal of trash on trashrack areas can occur in a relatively short time period, avoiding potential failure.

3.2.3.2 Assessment of Risk Reduction Opportunities – The CITY staff is aware of the difficulties in removal of trash from the trashracks. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the trashracks.

3.2.3.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of the Project Review Team and ARO staff that this failure mode should be re-classified as a Category II since this is a highly unlikely failure mode and the CITY does regular daily walk-throughs of the powerhouse, and especially after a flooding event.

This failure mode is re-classified as Category II.

3.2.4 PFM 3.4 – Tainter Gate Spillway – Sliding of the tainter gate spillway along a weak foundation plane during flooding conditions-multiple gates are unable to be opened - due to trash or other type of gate operating failure - Reservoir is exceeding normal head above Elev. 69.5 - Unusual Condition.

3.2.4.1 Description of PFM 3.4 – Tainter Gate Spillway (***Taken from PFMA Report and Enhanced by PFM Review Team***)

Tainter Gate Spillway Sliding - Unusual Condition: Due to the complicated geology in the area of the Jackson Bluff Project, the existence of a weak foundation plane underneath the tainter gate spillway section is a possibility. If the driving forces during flooding conditions are great enough, a sliding failure of the spillway could occur at this weak foundation plane. If the driving forces during flooding conditions are great enough, a sliding failure of the gated spillway section could occur at this weak foundation plane. The 1957 failure report describes the porous limestone bedrock with embedded solution cavities - Tampa Foundation. The progression of this failure mode would be during conditions approaching the PMF the gated spillway section becomes unstable due to a decrease in the shearing resistance of the limestone layer because over time it is solutioning out and the water continues to work through the layer making discontinuities. At the time of the PMF the hydraulic head on the bedrock increases thus there is less resistance in the bedrock and the tainter gate section begins to slide.

3.2.4.2 *Positive Factors (factors suggesting failure mode is more likely): 1) The gated spillway is founded on a possible weak foundation plane underneath the gated spillway. 2) The 1957 failure report describes the porous limestone bedrock which could solution over time, resulting in less resistance in the bedrock and the gated spillway section begins to slide.*

Negative Factors (factors suggesting failure mode is less likely): 1) Routine daily visual inspection by staff to identify cracking and differential movement that may indicate the potential of a sliding plane, 2) Maintaining the upstream pool to the crest of the tainter gate spillway will reduce sliding forces and likelihood of a failure, and 3) Control points were established in 2006 along the powerhouse, spillway piers, upstream approach wall and along downstream retaining walls to identify any trends in movement of these structures.

3.2.4.3 Assessment of Risk Reduction Opportunities – Based on the original design and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor the powerhouse sliding.

3.2.4.4 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of the Project Review Team and ARO staff that this failure mode should remain classified as a Category II since the CITY does monitor the spillway gate section of the Project.

This failure mode remains classified as Category II.

3.2.5 PFM 3.5 – Gated Spillway - Concrete Deterioration/Reinforcement Corrosion – The fifth potential failure mode Concrete deterioration and/or reinforcement corrosion in the tainter gate spillway leading to an overstress condition failure.

3.2.5.1 Description of PFM 3.5 – Concrete Deterioration and/or reinforcement corrosion in gated spillway section – Unusual Condition (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for concrete deterioration and/or corrosion of steel reinforcement in the tainter gate spillway structure. Such degradations of the spillway structure under flooding conditions could lead to an overstress failure resulting in unsafe conditions. Either an impact or freeze/thaw or crack would create a crack in the concrete, rust color showing out of crack due to corrosion of reinforcing steel; leads to a overloading condition of the reinforcement - tension loading - that causes a portion of the tainter gate spillway to fail.

*Positive Factors (*factors contributing to potential failure*): 1) The age of the gated spillway section lends to potential degradation of the concrete and 2) the continuous exposure to surrounding water provides atmosphere for corrosion of exposed reinforcing steel.*

*Negative Factors (*factors prohibiting potential failure*): 1)* Concrete deterioration and/or reinforcement corrosion can be identified by staff during their routine daily inspection of the Project structures and 2) Prompt repair of significantly deteriorated areas can occur in a short time period, avoiding potential failure.

3.2.5.2 Based on the original design and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. The daily monitoring and maintenance practices of the CITY staff are commendable and reduce the potential risk for failure.

3.2.5.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II with a recommendation that the survey monuments should continue to be monitored to verify satisfactory performance from a prudent design perspective.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be classified as a Category IV since it is highly unlikely to occur even though it is prudent practice that the CITY does regular and periodic monitoring of the gated spillway section.

This failure mode is re-classified as Category IV.

3.2.6 PFM 3.6 – Tainter Gate Debris Blockage – The sixth potential failure mode is build-up of debris in front of one or more of the tainter gates resulting in a decreased gate capacity leading to overtopping of another section of the dam. It is also assumed that the intakes of the turbine-generator would be clogged with debris.

3.2.6.1 Description of PFM 3.6 – Debris blockage in gated spillway section (***Taken from PFMA Report and enhanced by Review Team***)

It is possible that debris could build up in front of the tainter gates during flooding conditions. The presence of this debris would result in a reduction of gate capacity and could lead to the overtopping failure of another section of the dam. this PFM was developed prior to the installation of the Auxiliary Spillway. Now, with the New Auxiliary Spillway which is "self-cleaning", the potential for buildup on the Tainter Gates is significantly reduced, if almost not eliminated as a PFM.

*Positive (*factors contributing to potential failure*): 1) There are historic "islands" of vegetation that float on the reservoir and could block the trashracks.*

*Negative (*factors prohibiting potential failure*): 1) The CITY staff is aware of the "islands" and when observed relocates them to the Auxiliary Spillway area for "self-cleaning".*

3.2.6.2 Assessment of Risk Reduction Opportunities – Based on the revised Auxiliary Spillway design and the operating conditions to date, the CITY staff is able to reduce the risk of potential failure to the greatest extent physically and

economically possible. There are no additional recommendations for risk reduction other than to continue to monitor future floating debris.

3.2.6.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category IV with a recommendation that the gated spillway should continue to be monitored to verify satisfactory performance from a prudent design perspective.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category IV since it is highly unlikely to occur since the CITY does regular and periodic monitoring of the gated spillway section.

This failure mode remains classified as Category IV.

3.2.7 PFM 3.7 – Tainter Gate Mechanical or Electrical Failure – The seventh potential failure mode is mechanical or electrical failure of the gate hoists resulting in an overtopping of the gates or another section of the dam.

3.2.7.1 Description of PFM 3.7 – Gated spillway Electrical or mechanical failure (***Taken from PFMA Report and enhanced by Review Team***)

There is the possibility for a mechanical or electrical failure of the hoists that operate the tainter gates. A prolonged inability to operate the gates could lead to an overtopping failure of the gates or of another section of the dam.

*Positive (*factors contributing to potential failure*): 1) There is always a possibility of an electrical or mechanical failure.*

*Negative (*factors prohibiting potential failure*): 1) There is the ability to pass the 500-Year flood event with two gates inoperable, 2) There is the ability to remove a gate and/or spare parts are available, 3) There are portable generators on-site, 4) Local technicians are available to repair mechanical or electrical failures, and 5) Hoists are portable be being installed on rails; thus once a gate is secured in an open position the hoist may be re-located to other gates.*

3.2.7.2 Assessment of Risk Reduction Opportunities – Based on the maintenance and the operating conditions to date, the CITY staff has attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project electrical and mechanical systems as currently in effect.

3.2.7.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II with a recommendation that the gated spillway should continue to be monitored to verify satisfactory performance from a prudent design perspective.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category II since it has a potential to occur even though the CITY does regular and periodic monitoring of the gated spillway section on a daily basis.

This failure mode remains classified as Category II.

3.2.8 PFM 3.8 – Failure of Operator to properly operate Project – The eighth potential failure mode is failure of the operator to properly operate the Project.

3.2.8.1 Description of PFM 3.8 – Debris blockage in gated spillway section (***Taken from PFMA Report and enhanced by Review Team***)

Failure of the operator to properly operate the Project under flooding conditions and allowing debris to block the gated spillway section could lead to unsafe conditions.

*Positive (*factors contributing to potential failure*): 1) There is always the potential for human error.*

*Negative (*factors prohibiting potential failure*): 1) With the Auxiliary Spillway in place the ability to release water without build-up of additional head is significantly better than prior to the new construction, 2) there are supplemental operators/management on-site during severe events, 3) a transfer of knowledge program has been implemented by the CITY, 4) both Management and Operators observe and discuss operational issues during a storm event, 5) standard procedures for operation of units are documented, and 6) Operators at other power stations are also familiar with emergency operations at this Project in extreme conditions.*

3.2.8.2 Assessment of Risk Reduction Opportunities – Based on the CITY planning and the operating conditions to date, the CITY is able to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.8.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category IV since it is highly unlikely to occur.

This failure mode remains classified as Category IV.

3.2.9 PFM 3.9 – Overtopping of Embankment due to flows in excess of gated spillway capacity – The ninth potential failure mode is overtopping failure of the embankment caused by inflow in excess of the gated spillway capacity that has been reduced by debris blockage.

3.2.9.1 Description of PFM 3.9 – Overtopping of Embankment due to flows in excess of gated spillway capacity (***Taken from PFMA Report and enhanced by Review Team***)

There is the potential for debris to build up in front of the tainter gates, reducing spillway capacity. This reduced capacity could result in an overtopping failure of the embankment.
*Positive (*factors contributing to potential failure*): 1) This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

*Negative (*factors prohibiting potential failure*): 1) This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

3.2.9.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.9.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category IV since it is highly unlikely to occur, especially with the new Auxiliary Spillway and remote dike.

This failure mode remains classified as Category IV.

3.2.10 PFM 3.10 – Overtopping failure of the embankment caused by the reservoir level reaching 77.0 feet without a "failure" of the fuse plug – The tenth potential failure mode is overtopping failure of the embankment caused by inflow in excess of the gated spillway capacity that has been reduced by debris blockage.

3.2.10.1 Description of PFM 3.10 – Overtopping of Embankment due to flows in excess of gated spillway capacity (***Taken from PFMA Report and enhanced by Review Team***)

There is the potential for the reservoir level to reach Elev. 77.0 feet before the failure of the fuse plug section. This would result in the overtopping failure of the embankment. In 2011 the fuse plug was replaced with the new Auxiliary spillway. The embankment area between the Gated Spillway and the Auxiliary Spillway was raised to Elev. 77 and is now not susceptible to overtopping..

*Positive (*factors contributing to potential failure*): 1) This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

*Negative (*factors prohibiting potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

3.2.10.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.10.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category IV since it is highly unlikely to occur, especially with the new Auxiliary Spillway.

This failure mode remains classified as Category IV.

3.2.11 PFM 3.11 – Overtopping failure of the embankment caused by the reservoir level reaching 77.0 feet without a "failure" of the fuse plug – The eleventh potential failure mode is overtopping failure of the embankment caused by inflow in excess of the gated spillway capacity that has been reduced by debris blockage.

3.2.11.1 Description of PFM 3.11 – Overtopping failure of the embankment due to an inflow greater than that of the IDF (***Taken from PFMA Report and enhanced by Review Team***)

For the Category III PFMs that involve the IDF or possible erosion of the fuse plug; D2SI accepted a May 2006 IDF study that determined the IDF to be 1/2 the PMF. The results of the IDF study combined with fuse plug and emergency spillway concerns lead to the conclusion that the Project would experience uncontrollable erosion in the area of the fuse plug and emergency spillway.

*Positive (*factors contributing to potential failure*): 1) This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

*Negative (*factors prohibiting potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place. Dam remediation completed in 2011 included constructing an ogee spillway over the existing embankment near the gated spillway, and installation of a dike across the emergency spillway and fuse plug, which would prevent those structures from serving as functional spillways. These failure modes related to the decommissioned fuse plug and vegetated emergency spillway should be reclassified to Category IV.*

3.2.11.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.11.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be re-classified from a Category III at the 2009 review to a Category IV since it is highly unlikely to occur, especially with the new Auxiliary Spillway.

This failure mode is re-classified as Category IV.

3.2.12 PFM 3.12 – Instability of the downstream slope during flooding conditions caused by tailwater saturation of the downstream slope and drawdown as water recedes.

3.2.12.1 Description of PFM 3.12 – Instability of downstream slope after flooding and tailwater drawdown (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for tailwater saturation of the downstream slope of the embankment during flooding conditions. Drawdown of the tailwater as it recedes could result in the instability of the downstream slope and an eventual slope failure resulting in an uncontrolled release of the reservoir.

*Positive (*factors contributing to potential failure*): 1) Most of the foundation soils are porous which are pipeable and 2) there have been failures in the past.*

*Negative (*factors prohibiting potential failure*): 1) The toe drain was originally installed in 1998 to collect and monitor seepage, 2) the slope failure is more likely in a rapid drawdown, but the actual conditions will be a slow drawdown, 3) Piezometers are installed on the slope, and the toe drain pipes are monitored, 4) the failure would cause a rise in tailwater, 4) tailwater recedes slowly as floodwaters drains in a matter of weeks, and 5) embankment saturation drains through the toe drains. This is not a rapid drawdown condition.*

3.2.12.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect and to repair any slope instability areas as quickly as possible.

3.2.12.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II with a recommendation that the embankment area, especially to the toe of the slope should continue to be monitored to verify satisfactory performance from a prudent design perspective.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category II since it has a potential to occur even though the CITY does regular and periodic monitoring of the embankment section on a daily basis.

This failure mode remains classified as Category II.

3.2.13 PFM 3.13 – Potential sliding failure of the downstream slope during flooding conditions due to higher water levels and hydraulic gradients.

3.2.13.1 Description of PFM 3.13 – Potential sliding failure of downstream slope due to higher water levels and hydraulic gradients (***Taken from PFMA Report and enhanced by Review Team***)

Higher than normal water levels and hydraulic gradients during flooding conditions could cause a slope stability sliding failure of the downstream embankment slope, resulting in an uncontrolled release of the reservoir. The maximum head differential

is 19.5 feet (Elev. 77 - 57.5) if the upper pool is at the maximum level and there is no tailwater, which is an abnormal situation. With normal pool at Elev. 68.5 and the new Auxiliary Spillway at Elev. 69.5 the hydraulic gradient that can occur is significantly less than that contemplated in the original PFM. The piezometric level in the embankment is monitored with piezometers.

*Positive (*factors contributing to potential failure*): 1) Most of the foundation soils are porous which are pipeable and 2) there have been failures in the past.*

*Negative (*factors prohibiting potential failure*): 1) The new Auxiliary Spillway significantly reduces the potential for this originally contemplated condition, 2) the slope failure is more likely in a rapid drawdown, but the actual conditions will be a slow drawdown, 3) Piezometers are installed on the slope, and the toe drain pipes are monitored, 4) the failure would cause a rise in tailwater, 4) tailwater recedes slowly as floodwaters drains in a matter of weeks, and 5) embankment saturation drains through the toe drains. This is not a rapid drawdown condition.*

3.2.13.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect and to repair any slope instability areas as quickly as possible.

3.2.13.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II with a recommendation that the embankment area, especially to the toe of the slope should continue to be monitored to verify satisfactory performance from a prudent design perspective.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category II since it has a potential to occur even though the CITY does regular and periodic monitoring of the embankment section on a daily basis.

This failure mode remains classified as Category II.

3.2.14 PFM 3.14 – Instability of the downstream slope during flooding conditions caused by tailwater saturation of the downstream slope and drawdown as water recedes.

3.2.14.1 Description of PFM 3.14 – instability of downstream slope during flood conditions (Unusual Condition) causes downstream slope saturation by high headwater level and rapid drawdown as tailwater recedes causes slope failure (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for headwater saturation of the upstream slope of the embankment during flooding conditions. Drawdown of the headwater as it recedes could result in the instability of the downstream slope and an eventual slope failure resulting in an uncontrolled release of the reservoir.

*Positive (*factors contributing to potential failure*): 1) Most of the foundation soils are porous which are pipeable.*

*Negative (*factors prohibiting potential failure*): 1) This condition would require 7 feet of additional pool depth and a maximum drawdown of 8.5 feet to normal pool. This is not a rapid drawdown situation and is highly unlikely to occur. The new Auxiliary Spillway will also limit the ability to cause a 7 foot rise in the upstream reservoir.*

3.2.14.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.14.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category IV since it has a potential to occur even though the CITY does regular and periodic monitoring of the Auxiliary Spillway and dam embankment section on a daily basis.

This failure mode remains classified as Category IV.

3.2.15 PFM 3.15 – Overtopping of the fuse plug during flooding conditions leading to erosion of material downstream of the fuse plug and a deep breach resulting in an uncontrolled release of the reservoir.

3.2.15.1 Description of PFM 3.15 – Overtopping of the fuse plug during flooding conditions. (***Taken from PFMA Report and enhanced by Review Team***)

For the Category III PFMs that involve the IDF or possible erosion of the fuse plug; D2SI accepted a May 2006 IDF study that determined the IDF to be 1/2 the PMF. The results of the IDF study combined with fuse plug and emergency spillway concerns lead to the conclusion that the Project would experience uncontrollable erosion in the area of the fuse plug and emergency spillway. Dam remediation completed in 2011 included constructing an ogee spillway over the existing embankment near the gated spillway, and installation of a dike across the emergency spillway and fuse plug, which would prevent those structures from serving as functional spillways. These failure modes related to the decommissioned fuse plug and vegetated emergency spillway should be reclassified to Category IV.

*Positive (*factors contributing to potential failure*): 1) This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

*Negative (*factors prohibiting potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

3.2.15.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.15.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be re-classified as a Category IV since the fuse plug has been decommissioned and the emergency spillway vegetated after the completion of construction of the Auxiliary Spillway and remote dike The original PFM no longer exists.

This failure mode is re-classified as Category IV.

3.2.16 PFM 3.16 – Erosion of the emergency spillway with the pool at elevation 72.0 feet or more resulting in a deep breach and an uncontrolled release of the reservoir.

3.2.16.1 Description of PFM 3.16 – Erosion of the emergency spillway with a pool level at Elev. 72.0 resulting in a deep breach. (***Taken from PFMA Report and enhanced by Review Team***)

For the Category III PFMs that involve the IDF or possible erosion of the fuse plug; D2SI accepted a May 2006 IDF study that determined the IDF to be 1/2 the PMF. The results of the IDF study combined with fuse plug and emergency spillway concerns lead to the conclusion that the Project would experience uncontrollable erosion in the area of the fuse plug and emergency spillway. Dam remediation completed in 2011 included constructing an ogee spillway over the existing embankment near the gated spillway, and installation of a dike across the emergency spillway and fuse plug, which would prevent those structures from serving as functional spillways. These failure modes related to the decommissioned fuse plug and vegetated emergency spillway should be reclassified to Category IV.

*Positive (*factors contributing to potential failure*): 1) This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

*Negative (*factors prohibiting potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

3.2.16.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.16.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be re-classified as a Category IV since the fuse plug has been decommissioned and the emergency spillway vegetated after the completion of construction of the Auxiliary Spillway and remote dike The original PFM no longer exists.

This failure mode is re-classified as Category IV.

3.2.17 PFM 3.17 – Sliding of the powerhouse along a weak foundation plane during normal conditions..

3.2.17.1 Description of PFM 3.17 – Potential sliding failure of powerhouse along a weak foundation plane during Normal Conditions (***Taken from PFMA Report and enhanced by Review Team***)

*Due to the complicated geology in the area of the Jackson Bluff Hydroelectric Project, there is the possibility of a weak foundation plane underneath the powerhouse. If the driving forces during normal conditions are great enough, a sliding failure of the powerhouse could occur at this weak foundation plane. The 1957 failure report describes the porous limestone bedrock with embedded solution cavities - Tampa Foundation. The progression of this failure mode would be during conditions approaching the PMF the powerhouse becomes unstable due to a decrease in the shearing resistance of the limestone layer because over time it is solutioning out and the water continues to work through the layer making discontinuities. At the time of the PMF the hydraulic head on the bedrock increases thus there is less resistance in the bedrock and the powerhouse begins to slide. Positive (*factors contributing to potential failure*): 1) The foundation rock beneath the powerhouse may have areas of porous limestone with embedded solution cavities.*

*Negative (*factors prohibiting potential failure*): 1) There are 20 control points that are monitored between the powerhouse, spillway training walls, and north abutment, 2) the maximum head for this condition would have to be 43.7 feet +/- which is unlikely, and 3) the instrumentation data has been observed for the past five years without measurable movement.*

3.2.17.2 Assessment of Risk Reduction Opportunities – Based on the current monitoring plan the CITY is identifying any risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.17.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II with a recommendation that the powerhouse should continue to be monitored to verify satisfactory performance.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category II since it has a potential to occur even though the CITY does regular and periodic monitoring of the powerhouse on a daily basis.

This failure mode remains classified as Category II.

3.2.18 PFM 3.18 – Gated Spillway - Concrete Deterioration/Reinforcement Corrosion – The eighteenth potential failure mode is concrete deterioration and/or reinforcement corrosion in the powerhouse leading to an overstress condition failure.

3.2.18.1 Description of PFM 3.18 – Concrete Deterioration and/or reinforcement corrosion in the gated spillway section. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for concrete deterioration and/or corrosion of steel reinforcement in the gated spillway structure. Such degradations of the gated spillway structure under normal conditions could lead to an overstress failure resulting in unsafe conditions.

*Positive Factors (*factors contributing to potential failure*): 1) The age of the gated spillway section lends to potential degradation of the concrete and 2) the continuous exposure to surrounding water provides atmosphere for corrosion of exposed reinforcing steel.*

*Negative Factors (*factors prohibiting potential failure*): 1)* Concrete deterioration and/or reinforcement corrosion can be identified by staff during their routine daily inspection of the Project structures, and 2) Prompt repair of significantly deteriorated areas can occur in a short time period, avoiding potential failure.

3.2.18.2 Based on the original design and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. The daily monitoring and maintenance practices of the CITY staff are commendable and reduce the potential risk for failure.

3.2.18.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II with a recommendation that the survey monuments should continue to be monitored to verify satisfactory performance from a prudent design perspective.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be classified as a Category IV since it is highly unlikely to occur even though it is prudent practice that the CITY does regular and periodic monitoring of the gated spillway section.

This failure mode is re-classified as Category IV.

3.2.19 PFM 3.19 – Instability of the powerhouse during normal conditions due to flotation (buoyancy) effects caused by the dewatering of multiple units.

3.2.19.1 Description of PFM 3.19 – Instability of powerhouse during normal conditions due to flotation caused by dewatering of multiple units. (***Taken from PFMA Report and enhanced by Review Team***)

From time to time, it becomes necessary to dewater the powerhouse units for maintenance. There is a potential for failure if more than one of the units is dewatered at one time. Dewatering units results in an overall reduction in structure weight and floatation of the powerhouse becomes a possibility.

*Positive (*factors contributing to potential failure*): 1) There is a potential to dewater at least one unit; however it highly unlikely that multiple units would ever be dewatered concurrently.*

*Negative (*factors prohibiting potential failure*): 1) The Ebasco study in 1992 evaluated two units dewatered condition with a F.S. = 2.5 (See Section 8 of STID). 2) Further there is only one set of intake stoplogs so no more than one unit at a time can be dewatered, and 3) the draft tube is always partially full of water since there are no downstream draft tube gates. Normal conditions would have partial draft tube sections always full.*

3.2.19.2 Assessment of Risk Reduction Opportunities – Based on the original design and the operating and maintenance conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.19.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category IV since it has a potential to occur is highly unlikely with only one set of stoplogs.

This failure mode remains classified as Category IV.

3.2.20 PFM 3.20 – Mechanical shaft failure in addition to the hydraulic system resulting in an uncontrolled release of water and erosion downstream of the powerhouse.

3.2.20.1 Description of PFM 3.20 – Inability to operate wicket gates after turbine shaft failure, leading to uncontrolled release of the reservoir. (***Taken from PFMA Report and enhanced by Review Team***)

It is possible that the turbine shaft may break, leading to an inability to operate the wicket gates. This would result in an uncontrolled release of the reservoir through the damaged unit and possible erosion downstream of the powerhouse.

*Positive (*factors contributing to potential failure*): 1) There is a potential for a turbine shaft failure.*

*Negative (*factors prohibiting potential failure*): 1) Though this is a possible failure, the maximum flow rate through the unit is equivalent to the effective opening of the water passage and would not be a downstream catastrophic event. This would be almost equivalent to operating at full gate.*

3.2.20.2 Assessment of Risk Reduction Opportunities – Based on the original design and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.20.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category IV since the potential to occur is highly unlikely and there would not be an uncontrolled release of water. This condition is very similar to operation at full gate.

This failure mode remains classified as Category IV.

3.2.21 PFM 3.21 – Failure in Potential Failure Mode No. 20 causes flooding of the powerhouse and a failure of the downstream powerhouse wall.

3.2.21.1 Description of PFM 3.21 – Inability to operate wicket gates after turbine shaft break, leading to flooding of the powerhouse and downstream powerhouse wall failure. (***Taken from PFMA Report and enhanced by Review Team***)

If Potential Failure Mode No. 20 were to occur, the powerhouse could become flooded. Resulting loads could cause the downstream powerhouse wall to fail. This is not a physical possibility because the draft tube cannot build up pressure to back water into the powerhouse. There is the ability to mechanically adjust the wicket gate assembly. Also the wicket gates fail to the closed position.

*Positive (*factors contributing to potential failure*): 1) This is a physical impossibility with the current design.*

*Negative (*factors prohibiting potential failure*): 1)* This is not a physical possibility because the draft tube cannot build up pressure to back water into the powerhouse. 2) There is the ability to mechanically adjust the wicket gate assembly, and 3) the wicket gates fail to the closed position.

3.2.21.2 Assessment of Risk Reduction Opportunities – Based on the original design and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.21.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category IV since the potential to occur is highly unlikely and is a physical impossibility.

This failure mode remains classified as Category IV.

3.2.22 PFM 3.22 – Sliding of the tainter gate spillway along a weak foundation plane during normal conditions. Failure mode is similar to PFM 4, but under Normal conditions.

3.2.22.1 Description of PFM 3.22 – Potential sliding failure of Tainter Gate Spillway along a weak foundation plane during Normal Conditions (***Taken from PFMA Report and enhanced by Review Team***)

Due to the complicated geology in the area of the Jackson Bluff Hydroelectric Project, the existence of a weak foundation plane underneath the tainter gate spillway is a possibility. If the driving forces during normal conditions are great enough, a sliding failure of the spillway could occur at this weak foundation plane. The 1957 failure report describes the porous limestone bedrock with embedded solution cavities - Tampa Foundation. The progression of this failure mode would be during conditions approaching the PMF the gated spillway section becomes unstable due to a decrease in the shearing resistance of the limestone layer because over time it is solutioning out and the water continues to work through the layer making discontinuities. At the time of the PMF the hydraulic head on the bedrock increases thus there is less resistance in the bedrock and the tainter gate section begins to slide.

3.2.22.2 *Positive Factors (factors suggesting failure mode is more likely): 1) The gated spillway is founded on a possible weak foundation plane underneath the gated spillway. 2) The 1957 failure report describes the porous limestone bedrock which could solution over time, resulting in less resistance in the bedrock and the gated spillway section begins to slide.*

Negative Factors (factors suggesting failure mode is less likely): 1) Routine daily visual inspection by staff to identify cracking and differential movement that may indicate the potential of a sliding plane, 2) Maintaining the upstream pool to the crest of the tainter gate spillway will reduce sliding forces and likelihood of a failure, and 3) Control points were established in 2006 along the powerhouse, spillway piers, upstream approach wall and along downstream retaining walls to identify any trends in movement of these structures.

3.2.22.3Assessment of Risk Reduction Opportunities – Based on the original design and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor the powerhouse sliding.

It is the opinion of the Project Review Team and ARO staff that this failure mode should remain classified as a Category II since the CITY does monitor the spillway gate section of the Project.

This failure mode remains classified as Category II.

3.2.23 PFM 3.23 – Concrete deterioration and/or reinforcement corrosion in the tainter gate spillway during Normal conditions leading to an overstress condition failure. Failure mode is similar to PFM 5, but under normal conditions.

3.2.23.1 Description of PFM 3.23 – Concrete Deterioration and/or reinforcement corrosion in gated spillway section – Normal Condition (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for concrete deterioration and/or corrosion of steel reinforcement in the tainter gate spillway structure. Such degradations of the spillway structure under normal conditions could lead to an overstress failure resulting in unsafe conditions.

*Positive Factors (*factors contributing to potential failure*): 1) The age of the gated spillway section lends to potential degradation of the concrete and 2) the continuous exposure to surrounding water provides atmosphere for corrosion of exposed reinforcing steel.*

*Negative Factors (*factors prohibiting potential failure*): 1)* Concrete deterioration and/or reinforcement corrosion can be identified by staff during their routine daily inspection of the Project structures and 2) Prompt repair of significantly deteriorated areas can occur in a short time period, avoiding potential failure.

3.2.23.2 Based on the original design and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. The daily monitoring and maintenance practices of the CITY staff are commendable and reduce the potential risk for failure.

3.2.23.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II with a recommendation that the survey monuments should continue to be monitored to verify satisfactory performance from a prudent design perspective.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be classified as a Category IV since it is highly unlikely to occur even though it is prudent practice that the CITY does regular and periodic monitoring of the gated spillway section.

This failure mode is re-classified as Category IV.

3.2.24 PFM 3.24 – Failure of the trunnion anchorage leading to tainter gate failure and an uncontrolled release of the reservoir.

3.2.24.1 Description of PFM 3.24 – Failure of trunnion anchorage on tainter gate leading to failure and uncontrolled release of reservoir. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for failure of the trunnion anchorage. This would lead to the failure of one or more tainter gates and a partial uncontrolled release of the reservoir down to the level of the crest of the gated spillway. The "close-up" structural inspection was last performed in 2004. In 2005 the Mead & Hunt Report recommended significant remediation of the tainter gates. This work was performed by FruCon under contract with the City of Tallahassee. The "close-up" structural inspection will be performed in mid-2015 and will be submitted as a separate document to FERC by December 31, 2015.

*Positive (*factors contributing to potential failure*): 1) There is potential for a failure of a trunnion anchorage - either through mechanical/structural failure of the trunnion shaft or the adjacent concrete structure.*

*Negative (*factors prohibiting potential failure*): 1)* Concrete deterioration and/or cracking in the area of the Trunnion anchorage can be identified during detailed tainter gate inspections and by routine visual inspections by staff, 2) the periodic 10-Year "close-up" inspections by an approved structural engineer will identify any potential issues, and 3) lowering of the upstream pool will reduce the forces on the trunnion and is an appropriate mitigation action if inflows are low.

3.2.24.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect, and to address any identified degradation and/or potential mechanical/structural issues prior to significant failure.

3.2.24.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II with a recommendation that the gated spillway should continue to be monitored to verify satisfactory performance from a prudent design perspective. Since the 10-Year detailed "close-up" structural inspection has yet to be performed this is re-classified to a Category III until the detailed inspection is completed.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be classified as a Category III until the 10-Year structural "close-up" inspection is performed and the report submitted by December 31, 2015. The Independent Consultant will review the report once the inspection and report are available.

This failure mode is re-classified as Category III.

3.2.25 PFM 3.25 – High trunnion pin friction moments leading to tainter gate failure and an uncontrolled release of the reservoir.

3.2.25.1 Description of PFM 3.25 – Failure of trunnion anchorage on tainter gate leading to failure and uncontrolled release of reservoir. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for high trunnion pin friction moments to develop in the tainter gate members during operation. This flexural load combined with the hydrostatic pressure on the gate could result in an eventual failure of the gate and a partial uncontrolled release of the reservoir since only one gate opens to crest of spillway. Direct lubrication of trunnion pins has been implemented.

*Positive (*factors contributing to potential failure*): 1) There is a potential for high trunnion pin friction moments to develop during operation.*

*Negative (*factors prohibiting potential failure*): 1)* Trunnion pin friction is monitored indirectly by amperage and line-to-line voltage measurements taken during gate

operations, and 2) if a tainter gate shows increasing friction, alternate gates can be used while repairs are completed.

3.2.25.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan and daily observations as currently in effect.

3.2.25.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II with a recommendation that the gated spillway should continue to be monitored and maintained to verify satisfactory performance. Since this is a regularly monitored and maintained structure it will remain a Category II.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category II since the CITY monitors and maintains this on a regular basis.

This failure mode remains classified as Category II.

3.2.26 PFM 3.26 – Erosion of the upstream embankment slope during normal conditions due to wave action.

3.2.26.1 Description of PFM 3.26 – Erosion of Upstream Embankment due to wave action. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for erosion of the upstream embankment slope as a result of wave action. There is 7 to 8 feet from normal pool to top of wave wall so any wave action is against riprap. If prolonged erosion of the upstream slope were to go unchecked, a failure of the embankment could eventually occur, resulting in an uncontrolled release of the reservoir.

*Positive (*factors contributing to potential failure*): 1) There is potential for erosion of the upstream embankment of the dam section resulting in a breach in an area of the embankment.*

*Negative (*factors prohibiting potential failure*): 1) During the 2007 construction of the Auxiliary Spillway a new concrete wave wall was constructed along the entire length of the dam embankment to mitigate the potential for this PFM. Riprap was placed on the upstream side of the concrete wave wall on the upstream embankment slope. Some of the riprap is buried under the excavated material from the wave wall excavation.*

3.2.26.2 Assessment of Risk Reduction Opportunities – Based on the original design and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect. The CITY staff needs to be visually monitor the upstream side of the embankment riprap in the future to assure there is no loss of riprap due to sloughing into the reservoir.

3.2.26.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category IV since the potential to occur is highly unlikely and there would not be an uncontrolled release of water.

This failure mode remains classified as Category IV.

3.2.27 PFM 3.27 – Instability of the downstream slope of the embankment caused by seepage and/or rainfall resulting in elevated phreatic surface levels - combined seepage and normal rainfall (non-hurricane rain) event – Unusual Condition

3.2.27.1 Description of PFM 3.27 – Instability of downstream slope of dam embankment by seepage and normal rainfall. Unusual Condition (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for significant combined seepage and rainfall resulting in elevated phreatic surface levels in the embankment. This elevated phreatic surface could result in instability of the downstream slope and an eventual failure of the embankment leading to an uncontrolled release of the reservoir. The normal head is 43.7 feet. The actual event that occurred in 1957 was sequential rainfall events in the 3 to 4 inch per event, resulting in 22 inches of rain total that caused the progressive and sequential small shallow sloughing failures that eventually resulted in the sloughing of the toe of the embankment slope on September 30, 1957. The sequential rainfall events prohibit the Licensee from repairing the small shallow sloughs.

*Positive (*factors contributing to potential failure*): 1) Most of the foundation soils are porous which are pipeable, and 2) there have been historic failures of the embankment area, both major breaches and minor sloughs.*

*Negative (*factors prohibiting potential failure*): 1) An Interim Remediation Plan was discussed during the PFM review to remediate a 40 foot area of the dam embankment where it is planned to install a new filter blanket. It was determined during the PFMA Review that Mead & Hunt had sufficient soils data to perform design to allow construction within next few months - Original Scheduled for Fall 2014. The FERC has reviewed the initial submittal of the remediation plan and provided comments to the CITY, to which Mead & Hunt responded on April 1, 2015. The CITY currently has a stockpile of riprap and CITY was to obtain a stockpile of #57 for remediation in October/November 2014. Remediation Plan is required to increase F.S.*

3.2.27.2 Assessment of Risk Reduction Opportunities – Based on the original design, construction, and the operating conditions to date, the recommendation for risk reduction relates primarily to the implementation of the proposed remediation plan being prepared by the CITY (Mead & Hunt) and approved by FERC. The implementation should proceed once all necessary approvals from FERC are received.

3.2.27.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable. Mead & Hunt should include the proposed mitigation slope stability analyses to the CITY with the approved plans for inclusion in the STID. Once the remediation plan is completed, the Record "As-Built" drawings should replace the design drawings for this area of remediation in the STID.

This failure mode was originally classified as a Category I with a recommendation that the CITY would have Mead & Hunt perform additional stability analyses for this condition and design a remediation plan for implementation in the Fall of 2014.The design is being finalized and will be implemented upon approval by FERC in the Summer of 2015.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be classified as a Category 1/III until the proposed design is approved by FERC and the remediation plan is implemented in the Summer of 2015. The Independent Consultant is available to review the design once the final plan and drawings are available.

This failure mode is re-classified as Category 1/III. Once the Embankment Seepage Mitigation project is completed, this PFM is recommended to become a Category II since it is recommended that the CITY continue to monitor this on a regular and formal basis with recorded data for at least the next 5 year period. Due to the previous dam embankment failure and the need for this current remediation project, this should be a formal checked review line item on the daily check sheets during the daily reviews. Something like "Review of Embankment Seepage: ___ No Change ___ Slight Change ___ Significant Change". After the five year period, during the next Part 12D Inspection is may again be re-classified as a Category IV based on the success of the remediation and updated calculations with the improved FOS as prepared by Mead & Hunt, as presented to FERC for approval of design and construction installation.

3.2.28 PFM 4.38 – Instability of the downstream slope of the embankment caused by seepage resulting in elevated phreatic surface levels – Normal Condition

3.2.28.1 Description of PFM 3.28 – Instability of downstream slope of dam embankment by seepage resulting in elevated phreatic surface levels. Unusual Condition (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for significant seepage resulting in elevated phreatic surface levels in the embankment. This elevated phreatic surface could result in instability of the downstream slope and an eventual failure of the embankment leading to an uncontrolled release of the reservoir. This condition could increase additional 7 to 8 feet without tailwater if the lake goes to Elev. 77. Mead & Hunt analysis showed FS 1.3 vs. 1.5 required with IDF at normal lake elevation. Mead & Hunt analysis FS 1.1 vs. 1.4 required at Sta. 32+00 with IDF at Elev. 77. Additional data and borings are to be obtained at Sta. 30+00 at Culvert 7 for the updated analyses. Downstream side of dam embankment is higher the further north. Remediation Plan is required to increase F.S. - Mead & Hunt is working on analyses where low F.S. is predicted, using piezometer readings as better data to determine exit gradients.

*Positive (*factors contributing to potential failure*): 1) Most of the foundation soils are porous which are pipeable, and 2) there have been historic failures of the embankment area, both major breaches and minor sloughs.*

*Negative (*factors prohibiting potential failure*): 1) An Interim Remediation Plan was discussed during the PFM review to remediate a 40 foot area of the dam embankment where it is planned to install a new filter blanket. It was determined during the PFMA Review that Mead & Hunt had sufficient soils data to perform design to allow construction within next few months - Original Scheduled for Fall 2014. The FERC has reviewed the initial submittal of the remediation plan and provided comments to the CITY, to which Mead & Hunt responded on April 1, 2015. The CITY currently has a stockpile of riprap and CITY was to obtain a stockpile of #57 for remediation in October/November 2014. Remediation Plan is required to increase F.S.*

3.2.28.2 Assessment of Risk Reduction Opportunities – Based on the original design, construction, and the operating conditions to date, the recommendation for risk reduction relates primarily to the implementation of the proposed remediation plan being prepared by the CITY (Mead & Hunt) and approved by FERC. The implementation should proceed once all necessary approvals from FERC are received.

3.2.28.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable. Mead & Hunt should include the proposed mitigation slope stability analyses to the CITY with the approved plans for inclusion in the STID. Once the remediation plan is completed, the Record "As-Built" drawings should replace the design drawings for this area of remediation in the STID.

This failure mode was originally classified as a Category I with a recommendation that the CITY would have Mead & Hunt perform additional stability analyses for this condition and design a remediation plan for implementation in the Fall of 2014.The design is being finalized and will be implemented upon approval by FERC in the Summer of 2015.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be classified as a Category 1/III until the proposed design is approved by FERC and the remediation plan is implemented in the Summer of 2015. The Independent Consultant is available to review the design once the final plan and drawings are available.

This failure mode is re-classified as Category 1/III. Once the Embankment Seepage Mitigation project is completed, this PFM is recommended to become a Category II since it is recommended that the CITY continue to monitor this on a regular and formal basis with recorded data for at least the next 5 year period. Due to the previous dam embankment failure and the need for this current remediation project, this should be a formal checked review line item on the daily check sheets during the daily reviews. Something like "Review of Embankment Seepage: ___ No Change ___ Slight Change ___ Significant Change". After the five year period, during the next Part 12D Inspection is may again be re-classified as a Category IV based on the success of the remediation and updated calculations with the improved FOS as prepared by Mead & Hunt, as presented to FERC for approval of design and construction installation.

Then add a comment section such as - Significant Overnight Rainfall and increase in Reservoir Elevation, etc.

3.2.29 PFM 3.29 – Instability of the downstream slope of the embankment caused by rainfall resulting in elevated phreatic surface levels - Hurricane – Extreme Condition

3.2.29.1 Description of PFM 3.29 – Instability of downstream slope of dam embankment by seepage resulting in elevated phreatic surface levels caused by a hurricane. Extreme Condition (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for significant rainfall resulting in elevated phreatic surface levels in the embankment. Under this PFM the water surface of the reservoir could rise faster than normal, but the elevated phreatic surface could create a rise condition resulting in instability of the downstream slope and an eventual failure of the embankment leading to an uncontrolled release of the reservoir. This is the same conditions as PFM 28. After Mead & Hunt analyses, revise category to match PFM 28.

*Positive (*factors contributing to potential failure*): 1) Most of the foundation soils are porous which are pipeable, and 2) there have been historic failures of the embankment area, both major breaches and minor sloughs.*

*Negative (*factors prohibiting potential failure*): 1) An Interim Remediation Plan was discussed during the PFM review to remediate a 40 foot area of the dam embankment where it is planned to install a new filter blanket. It was determined during the PFMA Review that Mead & Hunt had sufficient soils data to perform design to allow construction within next few months - Original Scheduled for Fall 2014. The FERC has reviewed the initial submittal of the remediation plan and provided comments to the CITY, to which Mead & Hunt responded on April 1, 2015. The CITY currently has a stockpile of riprap and CITY was to obtain a stockpile of #57 for remediation in October/November 2014. Remediation Plan is required to increase F.S.*

3.2.29.2 Assessment of Risk Reduction Opportunities – Based on the original design, construction, and the operating conditions to date, the recommendation for risk reduction relates primarily to the implementation of the proposed remediation plan being prepared by the CITY (Mead & Hunt) and approved by FERC. The implementation should proceed once all necessary approvals from FERC are received.

3.2.29.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable. Mead & Hunt should include the proposed mitigation slope stability analyses to the CITY with the approved plans for inclusion in the STID. Once the remediation plan is completed, the Record "As-Built" drawings should replace the design drawings for this area of remediation in the STID.

This failure mode was originally classified as a Category I with a recommendation that the CITY would have Mead & Hunt perform additional stability analyses for this condition and design a remediation plan for implementation in the Fall of 2014.The

design is being finalized and will be implemented upon approval by FERC in the Summer of 2015.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be classified as a Category 1/III until the proposed design is approved by FERC and the remediation plan is implemented in the Summer of 2015. The Independent Consultant is available to review the design once the final plan and drawings are available.

This failure mode is re-classified as Category 1/III until the analyses are completed and then should be re-classified again to match the classification of PFM 28.

3.2.30 PFM 3.30 – Boils/piping caused by uplift pressures at the toe of the embankment due to windows in the Wakefield piling that would lead to failure of the embankment.

3.2.30.1 Description of PFM 3.30 – Boils/piping caused by uplift pressures at toe of embankment due to (potential) window in Wakefield piling. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for uplift pressures to occur at the toe of the embankment as a result of "windows" in the Wakefield piling originally installed in 1958 and/or pervious zones in the limestone foundation. The 1959 Black and Veatch remediation showed sheet piling added in the failed section. The 1967 Inspection report shows new sheet piling for approximately 3000 feet during the rebuilding of the dam in same area as possible seepage compared with 3600 total length of embankment dam. In 1980s some chemical grouting performed (McGuire Bebbe drawing) in this area. All of failed area was north of Auxiliary Spillway. North of the midwall, seepage occurred south of Auxiliary spillway, north of the Spillway Gates near area of original maintenance shed. These uplift pressures could result in boils/piping that could eventually lead to the failure of the embankment and an uncontrolled release of the reservoir.

*Positive (*factors contributing to potential failure*): 1) Most of the foundation soils/limestone bedrock are porous/pervious which could result in embankment soils that are pipeable.*

*Negative (*factors prohibiting potential failure*): 1) Over the years there has been a substantial amount of sheetpiling added to the embankment area - beginning in 1959 and with the new Auxiliary Spillway, 2) the new Toe Drain System will alleviate any uplift pressure over most of the embankment dam, 3) there is also drainage piping in the stilling basin with 18-inch and 12-inch pipes combining into a 24-inch pipe, similar to a French drain to prevent uplift on the Auxiliary Spillway. 4) Piezometers are monitoring seepage in the embankment with one survey monument located on the embankment. 5) There is no historical evidence of seepage from the Gated Spillway to the north to the toe drains. 6) The "windows" in the Wakefield pilling may have been a theoretical PFM, not reality. 7) With sheet piling going from the north end of the gated spillway to Sta. 0+80; the new Auxiliary Spillway sheet pile "box" from Sta. 1+25 to Sta. 9+25; the rebuild of the dam with sheet piling from approximately Sta. 8+90 to Sta. 32+00 results in minimal "windows" either vertically or horizontally.*

3.2.30.2 Assessment of Risk Reduction Opportunities – Based on the original and updated design and construction installation of various sheetpiling projects over the years, the original and updated designs attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.30.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category III due to a lack of information at the time of the original PFM.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be re-classified as a Category IV since it is highly unlikely that there is any "window" in the Wakefield piling after the construction of the Auxiliary Spillway and there is no historical evidence to support any seepage in the area identified. The additional research of historical data that was performed during the PFMA review needs to be added to the STID.

This failure mode is re-classified as Category IV.

3.2.31 PFM 3.31 – Progressive seepage failure caused by uplift pressures at the toe of the embankment due to windows in the Wakefield piling that would lead to failure of the embankment.

3.2.31.1 Description of PFM 3.31 – Progressive seepage caused by uplift pressures at toe of embankment due to (potential) window in Wakefield piling. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for uplift pressures to occur at the toe of the embankment as a result of "windows" in the Wakefield piling originally installed in 1958 and/or pervious zones in the limestone foundation. The 1959 Black and Veatch remediation showed sheet piling added in the failed section. The 1967 Inspection report shows new sheet piling for approximately 3000 feet during the rebuilding of the dam in same area as possible seepage compared with 3600 total length of embankment dam. In 1980s some chemical grouting performed (McGuire Bebbe drawing) in this area. All of failed area was north of Auxiliary Spillway. North of the midwall, seepage occurred south of Auxiliary spillway, north of the Spillway Gates near area of original maintenance shed. These uplift pressures could result in boils/piping that could eventually lead to the failure of the embankment and an uncontrolled release of the reservoir.

*Positive (*factors contributing to potential failure*): 1) Most of the foundation soils/limestone bedrock are porous/pervious which could result in embankment soils that are pipeable.*

*Negative (*factors prohibiting potential failure*): 1) Over the years there has been a substantial amount of sheetpiling added to the embankment area - beginning in 1959 and with the new Auxiliary Spillway, 2) the new Toe Drain System will alleviate any uplift pressure over most of the embankment dam, 3) there is also drainage piping in*

the stilling basin with 18-inch and 12-inch pipes combining into a 24-inch pipe, similar to a French drain to prevent uplift on the Auxiliary Spillway. 4) Piezometers are monitoring seepage in the embankment with one survey monument located on the embankment. 5) There is no historical evidence of seepage from the Gated Spillway to the north to the toe drains. 6) The "windows" in the Wakefield pilling may have been a theoretical PFM, not reality. 7) With sheet piling going from the north end of the gated spillway to Sta. 0+80; the new Auxiliary Spillway sheet pile "box" from Sta. 1+25 to Sta. 9+25; the rebuild of the dam with sheet piling from approximately Sta. 8+90 to Sta. 32+00 results in minimal "windows" either vertically or horizontally.

3.2.31.2 Assessment of Risk Reduction Opportunities – Based on the original and updated design and construction installation of various sheetpiling projects over the years, the original and updated designs attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.31.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category III due to a lack of information at the time of the original PFM.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be re-classified as a Category IV since it is highly unlikely that there is any "window" in the Wakefield piling after the construction of the Auxiliary Spillway and there is no historical evidence to support any seepage in the area identified. The additional research of historical data that was performed during the PFMA review needs to be added to the STID.

This failure mode is re-classified as Category IV.

3.2.32 PFM 3.32 – Boils/piping caused by uplift pressures at the toe of the embankment due to pervious zones in the lime rock that would lead to failure of the embankment.

3.2.32.1 Description of PFM 3.32 – Boils/Piping causing uplift pressures at toe of embankment due to pervious zones in the lime rock. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for uplift pressures to occur at the toe of the embankment as a result of "windows" in the Wakefield piling originally installed in 1958 and/or pervious zones in the limestone foundation. The 1959 Black and Veatch remediation showed sheet piling added in the failed section. The 1967 Inspection report shows new sheet piling for approximately 3000 feet during the rebuilding of the dam in same area as possible seepage compared with 3600 total length of embankment dam. In 1980s some chemical grouting performed (McGuire Bebbe drawing) in this area. All of failed area was north of Auxiliary Spillway. North of the midwall, seepage occurred south of Auxiliary spillway, north of the Spillway Gates near area of original maintenance shed. These uplift pressures could result in boils/piping that could eventually lead to the failure of the embankment and an uncontrolled release of the reservoir.

*Positive (*factors contributing to potential failure*): 1) Most of the foundation soils/limestone bedrock are porous/pervious which could result in embankment soils that are pipeable.*

*Negative (*factors prohibiting potential failure*): 1) Over the years there has been a substantial amount of sheetpiling added to the embankment area - beginning in 1959 and with the new Auxiliary Spillway, 2) the new Toe Drain System will alleviate any uplift pressure over most of the embankment dam, 3) there is also drainage piping in the stilling basin with 18-inch and 12-inch pipes combining into a 24-inch pipe, similar to a French drain to prevent uplift on the Auxiliary Spillway. 4) Piezometers are monitoring seepage in the embankment with one survey monument located on the embankment. 5) There is no historical evidence of seepage from the Gated Spillway to the north to the toe drains. 6) The "windows" in the Wakefield pilling may have been a theoretical PFM, not reality. 7) With sheet piling going from the north end of the gated spillway to Sta. 0+80; the new Auxiliary Spillway sheet pile "box" from Sta. 1+25 to Sta. 9+25; the rebuild of the dam with sheet piling from approximately Sta. 8+90 to Sta. 32+00 results in minimal "windows" either vertically or horizontally.*

3.2.32.2 Assessment of Risk Reduction Opportunities – Based on the original and updated design and construction installation of various sheetpiling projects over the years, the original and updated designs attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.32.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category III due to a lack of information at the time of the original PFM.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be re-classified as a Category IV since it is highly unlikely that there is any "window" in the Wakefield piling after the construction of the Auxiliary Spillway and there is no historical evidence to support any seepage in the area identified. The additional research of historical data that was performed during the PFMA review needs to be added to the STID.

This failure mode is re-classified as Category IV.

3.2.33 PFM 3.33 – Progressive seepage failure caused by uplift pressures at the toe of the embankment due to pervious zones in the lime rock that would lead to failure of the embankment.

3.2.33.1 Description of PFM 3.33 – Progressive seepage failure caused by uplift pressures at toe of embankment due to pervious zone in the lime rock. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for uplift pressures to occur at the toe of the embankment as a result of "windows" in the Wakefield piling originally installed in 1958 and/or pervious

zones in the limestone foundation. The 1959 Black and Veatch remediation showed sheet piling added in the failed section. The 1967 Inspection report shows new sheet piling for approximately 3000 feet during the rebuilding of the dam in same area as possible seepage compared with 3600 total length of embankment dam. In 1980s some chemical grouting performed (McGuire Bebbe drawing) in this area. All of failed area was north of Auxiliary Spillway. North of the midwall, seepage occurred south of Auxiliary spillway, north of the Spillway Gates near area of original maintenance shed. These uplift pressures could result in boils/piping that could eventually lead to the failure of the embankment and an uncontrolled release of the reservoir.

*Positive (*factors contributing to potential failure*): 1) Most of the foundation soils/limestone bedrock are porous/pervious which could result in embankment soils that are pipeable.*

*Negative (*factors prohibiting potential failure*): 1) Over the years there has been a substantial amount of sheetpiling added to the embankment area - beginning in 1959 and with the new Auxiliary Spillway, 2) the new Toe Drain System will alleviate any uplift pressure over most of the embankment dam, 3) there is also drainage piping in the stilling basin with 18-inch and 12-inch pipes combining into a 24-inch pipe, similar to a French drain to prevent uplift on the Auxiliary Spillway. 4) Piezometers are monitoring seepage in the embankment with one survey monument located on the embankment. 5) There is no historical evidence of seepage from the Gated Spillway to the north to the toe drains. 6) The "windows" in the Wakefield pilling may have been a theoretical PFM, not reality. 7) With sheet piling going from the north end of the gated spillway to Sta. 0+80; the new Auxiliary Spillway sheet pile "box" from Sta. 1+25 to Sta. 9+25; the rebuild of the dam with sheet piling from approximately Sta. 8+90 to Sta. 32+00 results in minimal "windows" either vertically or horizontally.*

3.2.33.2 Assessment of Risk Reduction Opportunities – Based on the original and updated design and construction installation of various sheetpiling projects over the years, the original and updated designs attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.33.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category III due to a lack of information at the time of the original PFM.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be re-classified as a Category IV since it is highly unlikely that there is any "window" in the Wakefield piling after the construction of the Auxiliary Spillway and there is no historical evidence to support any seepage in the area identified. The additional research of historical data that was performed during the PFMA review needs to be added to the STID.

This failure mode is re-classified as Category IV.

3.2.34 PFM 3.34 – Burrowing of animals leading to a seepage path and the failure of the embankment.

3.2.34.1 Description of PFM 3.34 – Burrowing of animals in embankment area creating a seepage path resulting in failure of the embankment. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for burrowing animals especially gopher turtles to dig holes in the embankment. These holes may eventually create a seepage path for the water, and in time, lead to the failure of the embankment, resulting in an uncontrolled release of the reservoir. Fire Ants are also an issue for penetrations.

*Positive (*factors contributing to potential failure*): 1) There is an ongoing issue at the Project with burrowing animals, especially the gopher turtles and fire ants.*

*Negative (*factors prohibiting potential failure*): 1) With the extensive sheet piling in the embankment, the flow pattern is very long and resulting gradients should be low. 2) The CITY staff monitors the embankments on the daily basis and are able to remediate burrow holes before they become an issue.*

3.2.34.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the embankments when burrow holes are observed. Continued trapping and/or extermination of the varmints is recommended. The Gopher Tortoise in Florida is classified at "Threatened" and needs to be addressed in accordance with State regulations.

3.2.34.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be changed from a Category IV and re-classified as a Category II since there are gopher turtle issues that could realistically create a potential failure. The CITY does regular and periodic monitoring of the embankment area on a daily basis to identify and remediate these potential burrowing areas before they become an issue.

This failure mode is re-classified as Category II.

3.2.35 PFM 4.35 – Failure of the right upstream approach wall due to structural failure of sheetpiling due to corrosion resulting in uncontrolled release.

3.2.35.1 Description of PFM 4.35 – Failure of right upstream approach wall due to structural failure due to corrosion. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for failure of the spillway's right upstream approach wall due to a structural failure. The failure mechanism would be corrosion of the sheetpiling so the bottom 1/3 would deteriorate, then a piping mechanism or undercurrent wave action would erode around the north end of the gated spillway; then the discharge would occur near the drain pipe, resulting in a failure path through the embankment between the gated spillway and the Auxiliary Spillway.

*Positive (*factors contributing to potential failure*): 1) Steel sheetpiling does corrode over time and could cause a 'window' in the current protection wall.*

*Negative (*factors prohibiting potential failure*): 1)* There is significant amount of 80 foot length of sheetpiling blocking the failure path. The sheetpiling was installed in the 1980s, and 2) when diving inspections are performed on an as-needed basis due to other Project evaluations when divers are on-site these walls are reviewed.

3.2.35.2 Assessment of Risk Reduction Opportunities – The CITY will establish a 5-year diving inspection schedule approximately 6 months prior the Part 12D Inspections for upstream and downstream areas, with video and written record.

3.2.35.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable; however it is recommended that these inspections be added to the DSSMP.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category IV, even though the CITY does monitoring, since the potential to occur is highly unlikely and there would not be any immediate uncontrolled release of water.

This failure mode remains classified as Category IV.

3.2.36 PFM 3.36 – Failure of the right upstream approach wall due to erosion/scouring embankment following failure of sheetpiling. (PFM 35).

3.2.36.1 Description of PFM 3.36 – Failure of right upstream approach wall due to erosion/scouring of embankment following failure of sheetpiling (Tied to PFM 35). (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for failure of the spillway's right upstream approach wall due to a structural failure. The failure mechanism would be corrosion of the sheetpiling so the bottom 1/3 would deteriorate, then a piping mechanism or undercurrent wave action would erode around the north end of the gated spillway; then the discharge would occur near the drain pipe, resulting in a failure path through the embankment between the gated spillway and the Auxiliary Spillway.

*Positive (*factors contributing to potential failure*): 1) If the steel sheetpiling does corrode over time and could cause a 'window' in the current protection wall (PFM 35), then erosion/scouring/piping of material could occur through the embankment area.*

*Negative (*factors prohibiting potential failure*): 1)* There is significant amount of 80 foot length of sheetpiling blocking the failure path; thus the ability to create the erosion/scouring piping of the embankment is highly unlikely. The sheetpiling was installed in the 1980s, and 2) when diving inspections are performed on an as-needed basis due to other Project evaluations when divers are on-site these walls are reviewed.

3.2.36.2 Assessment of Risk Reduction Opportunities – The CITY will establish a 5-year diving inspection schedule approximately 6 months prior the Part 12D Inspections for upstream and downstream areas, with video and written record.

3.2.36.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable; however it is recommended that these inspections be added to the DSSMP.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category IV, even though the CITY does monitoring, since the potential to occur is highly unlikely and there would not be any immediate uncontrolled release of water.

This failure mode remains classified as Category IV.

3.2.37 PFM 3.37 – Failure of the right downstream training wall due to the loss of embankment material and the formation of a breach.

3.2.37.1 Description of PFM 3.37 – Failure of right downstream training wall due to loss of embankment and formation of a breach. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for embankment erosion near the right downstream training walls. Erosion could lead to breaching and failure of the right downstream training walls. Erosion without sheetpile impact on upstream side (Wakefield low in ground), breach new and old sheetpiling, with erosion (partial breach) that is no lower than the top of new sheetpiling.

*Positive (*factors contributing to potential failure*): 1) Most of the embankment soils are porous which are pipeable, and 2) heavy rainfall on the downstream slope could occur causing erosion of the embankment. 3) With the erosion, there could be a lake drawdown of 20 to 25 feet resulting in a reservoir elevation of approximately Elev. 47 to 49.*

*Negative (*factors prohibiting potential failure*): 1) There is a 2009 construction survey monitoring base data with 2014 as the first year of survey monitoring, with four (4) monuments on top of new sheetpile wall. On the stairs to the discharge pipe there are two (2) monuments beneath the grating at top of stairs. 2) There are culverts under the access ramps (Elev. 50 and Elev. 47) would provide indication of any water in the top 20 feet of the embankment. 3) The upstream slopes are rip-rap protected.*

3.2.37.2 Assessment of Risk Reduction Opportunities – Based on the remedial design during the Auxiliary Spillway installation and the operating conditions to date, the design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.37.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be changed from a Category IV and re-classified as a Category II since the CITY does regular and periodic monitoring of the embankment area on a daily basis to identify any seepage issues.

This failure mode is re-classified as Category II.

3.2.38 PFM 3.38 – Failure of the left downstream training wall due to the loss of embankment material and the formation of a breach.

3.2.38.1 Description of PFM 3.38 – Failure of left downstream training wall with a serious of subsequent failures, leading to a breach of the training wall. (***Taken from PFMA Report and enhanced by Review Team***)

*There is a potential for embankment erosion near the left downstream training walls. Erosion could lead to breaching and failure of the left downstream training walls. There are two (2) survey monuments, but only one has been found. This failure mode would require corrosion of sheet piling; followed by erosion of material on the south side of the powerhouse; followed by erosion on the backside of the downstream training wall; leading to a breach of the wall. There was a failure if this wall in the past in 1947, due to undermining downstream of the powerhouse. Positive (*factors contributing to potential failure*): 1) Most of the embankment soils are porous which are pipeable, and 2) heavy rainfall on the downstream slope could occur causing erosion of the left downstream embankment. 3) There was a failure of this downstream wall in 1947 due to undermining of the powerhouse.*

*Negative (*factors prohibiting potential failure*): 1) There is a 2009 construction survey monitoring base data with 2014 as the first year of survey monitoring, with two (2) monuments on the downstream training wall. 2) The failure mode is very complex and highly unlikely to occur. 3) A sheetpile wall was constructed with a concrete cap. 4) The historic failure of the downstream wall was not the same condition as this PFM; thus this is a very unlikely PFM.*

3.2.38.2 Assessment of Risk Reduction Opportunities – Based on the remediation following the original training wall failure and the operating conditions to date, the remediation of the training wall attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.38.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable; however the second monument either needs to be found or a new monument is recommended to be added. One monument on the training wall is not acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain as a Category IV since this is a highly unlikely failure due to the numerous sequences of failures that would be required for it to happen. The CITY should continue to monitor the survey monuments as prudent practice without it being a Category II.

This failure mode remains classified as Category IV.

3.2.39 PFM 3.39 – Failure of the right downstream training wall due to the loss of abutment material and the formation of a breach.

3.2.39.1 Description of PFM 3.39 – Failure of right downstream training wall due to loss of abutment material and formation of a breach. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for erosion of the abutment material near the right downstream training walls. Erosion could lead to breaching and failure of the right downstream training walls. Erosion without sheetpile impact on upstream side (Wakefield low), breach new and old sheetpiling, with erosion (partial breach) that is no lower than the top of new sheetpiling.

*Positive (*factors contributing to potential failure*): 1) Most of the abutment soils are porous which are pipeable, and 2) heavy rainfall on the downstream abutment slope could occur causing erosion of the embankment. 3) With the erosion, there could be a lake drawdown of 20 to 25 feet resulting in a reservoir elevation of approximately Elev. 47 to 49.*

*Negative (*factors prohibiting potential failure*): 1) There is a 2009 construction survey monitoring base data with 2014 as the first year of survey monitoring, with four (4) monuments on top of new sheetpile wall. On the stairs to the discharge pipe there are two (2) monuments beneath the grating at top of stairs. 2) There are culverts under the access ramps (Elev. 50 and Elev. 47) would provide indication of any water in the top 20 feet of the abutment. 3) The upstream slopes are rip-rap protected.*

3.2.39.2 Assessment of Risk Reduction Opportunities – Based on the remedial design during the Auxiliary Spillway installation and the operating conditions to date, the design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.39.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be changed from a Category IV and re-classified as a Category II since the CITY does regular and periodic monitoring of the abutment area on a daily basis to identify any seepage issues.

This failure mode is re-classified as Category II.

3.2.40 PFM 3.40 – Failure of the left downstream training wall due to the loss of abutment material and the formation of a breach.

3.2.40.1 Description of PFM 3.40 – Failure of left downstream training wall due to loss of abutment material, leading to a breach of the training wall. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for erosion of the abutment material near the left downstream training walls. Erosion could lead to breaching and failure of the left downstream training walls. There are two (2) survey monuments, but only one was found. This failure mode would require corrosion of sheet piling, followed by erosion of material on the south side of the powerhouse, followed by erosion on the backside of the downstream training wall, leading to a breach of the wall. There was a failure if this wall in the past in 1947, due to undermining downstream of the powerhouse. A sheetpile wall was constructed with a concrete cap. The historic failure of the downstream wall was not the same condition as the PFM. This is a very unlikely PFM.

*Positive (*factors contributing to potential failure*): 1) Most of the abutment soils are porous which are pipeable, and 2) heavy rainfall on the downstream abutment slope could occur causing erosion of the left downstream abutment. 3) There was a failure of this downstream wall in 1947 due to undermining of the powerhouse.*

*Negative (*factors prohibiting potential failure*): 1) There is a 2009 construction survey monitoring base data with 2014 as the first year of survey monitoring, with two (2) monuments on the downstream training wall. 2) The failure mode is very complex and highly unlikely to occur. 3) A sheetpile wall was constructed with a concrete cap. 4) The historic failure of the downstream wall was not the same condition as this PFM; thus this is a very unlikely PFM.*

3.2.40.2 Assessment of Risk Reduction Opportunities – Based on the remediation following the original training wall failure and the operating conditions to date, the remediation of the training wall attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.40.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable; however the second monument either needs to be found or a new monument is recommended to be added. One monument on the training wall is not acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain as a Category IV since this is a highly unlikely failure due to the numerous sequences of failures that would be required for it to happen. The CITY should continue to monitor the survey monuments as prudent practice without it being a Category II.

This failure mode remains classified as Category IV.

3.2.41 PFM 3.41 – Erosion downstream of the tainter gate resulting in undermining of the gate section and a progressive seepage failure.

3.2.41.1 Description of PFM 3.41 – Erosion downstream of tainter gate resulting in undermining of tainter gate section and progressive seepage failure. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for erosion downstream of the tainter gate spillway. This could result in undermining of the gates and a progressive seepage failure. Historically there is no erosion or scouring of rock from the Tainter Gate areas. The downstream island is natural rock outcroppings. When originally taken over by City from the State of Florida, there were two depressions in front of Units No. 6 and No. 7 due to Gate Hoists not being moved periodically. These depressed areas were filled with concrete as a slab as documented in a Mead & Hunt report in early 1980s. Sounding have been performed since the City assumed operation of the facility in 1983.

*Positive (*factors contributing to potential failure*): 1) The foundation rock may have weak limestone or solution cavities and 2) there were historic depressed areas of rock erosion.*

*Negative (*factors prohibiting potential failure*): 1) The operations of the tainter gates that created the historic erosion of the rock has not been repeated and the eroded areas were remediated with concrete. 2) Sheetpiling is located at toe of gated spillway. 3) Soundings have been performed since the CITY assumed operation of the facility in 1983 without any further indication of erosion of the rock.*

3.2.41.2 Assessment of Risk Reduction Opportunities – Based on the operating conditions to date, the remediation of the historic eroded areas has been successful to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.41.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable. *At the next diver inspection, this area is recommended to be reviewed with soundings and diver inspection.*

This failure mode was originally classified as a Category II with a recommendation that the gated spillway area should continue to be monitored, periodically reviewed by divers, and maintained to verify satisfactory performance. Since this is a regularly monitored and maintained structure it will remain a Category II.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category II since the CITY monitors and maintains this on a regular basis.

This failure mode remains classified as Category II.

3.2.42 PFM 3.42 – Erosion downstream of the tainter gate resulting in undermining of the gate section, resulting in a stability failure.

3.2.42.1 Description of PFM 3.42 – Erosion downstream of tainter gate resulting in undermining of tainter gate section resulting in a stability failure. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for erosion downstream of the tainter gate spillway. This could result in undermining of the gates and a stability failure. Historically there is no erosion or scouring of rock from the Tainter Gate areas. The downstream island is natural rock outcroppings. When originally taken over by City from the State of Florida, there were two depressions in front of Units No. 6 and No. 7 due to Gate Hoists not being moved periodically. These depressed areas were filled with concrete as a slab as documented in a Mead & Hunt report in early 1980s. Sounding have been performed since the City assume operation of the facility.

*Positive (*factors contributing to potential failure*): 1) The foundation rock may have weak limestone or solution cavities and 2) there were historic depressed areas of rock erosion.*

*Negative (*factors prohibiting potential failure*): 1) The operations of the tainter gates that created the historic erosion of the rock has not been repeated and the eroded areas were remediated with concrete. 2) Sheetpiling is located at toe of gated spillway. 3) Soundings have been performed since the CITY assume operation of the facility in 1983 without any further indication of erosion of the rock.*

3.2.42.2 Assessment of Risk Reduction Opportunities – Based on the operating conditions to date, the remediation of the historic eroded areas has been successful to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.42.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable. *At the next diver inspection, this area is recommended to be reviewed with soundings and diver inspection.*

This failure mode was originally classified as a Category II with a recommendation that the gated spillway area should continue to be monitored, periodically reviewed by divers, and maintained to verify satisfactory performance. Since this is a regularly monitored and maintained structure it will remain a Category II.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category II since the CITY monitors and maintains this on a regular basis.

This failure mode remains classified as Category II.

3.2.43 PFM 3.43 – Downstream erosion causing undermining and failure of the powerhouse due to sliding.

3.2.43.1 Description of PFM 3.43 – Erosion downstream of powerhouse resulting in a potential sliding failure. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for erosion downstream of the powerhouse through periodic discharges from draft tube section causing a lowering of the tailrace channel. This erosion could lead to undermining and failure of the powerhouse because of sliding.

*Positive (*factors contributing to potential failure*): 1) Some of the foundation rock may be weak limestone with solution cavities. 2) Powerhouse discharges could erode the downstream rock causing a lowering of the tailrace area.*

*Negative (*factors prohibiting potential failure*): 1) The flow pattern resulting in erosion of the limestone is likely a very long time frame. 2) The periodic monitoring by CITY staff and diving inspections would identify erosion areas in an early enough stage to allow for mitigation. 3) Soundings have been performed since the CITY took over the License in 1983 without any further indication of erosion of the rock.*

3.2.43.2 Assessment of Risk Reduction Opportunities – Based on the operating conditions to date, the limestone rock is not eroding thus the risk of potential failure is minimal. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.43.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II with a recommendation that the area downstream of the powerhouse should continue to be monitored and maintained to verify satisfactory performance. Since this is a regularly monitored and maintained structure it will remain a Category II.

It is the opinion of PFM Review Team and ARO staff that this failure mode should remain classified as a Category II since the CITY monitors and maintains this on a regular basis.

This failure mode remains classified as Category II.

3.2.44 PFM 3.44 – Downstream erosion causing undermining and failure of the powerhouse due to seepage.

3.2.44.1 Description of PFM 3.44 – Erosion downstream of powerhouse resulting in a potential seepage failure. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for erosion downstream of the powerhouse through periodic seepage/leakage from draft tube section causing a lowering of the tailrace channel.. This erosion could lead to undermining and failure of the powerhouse because of seepage. This is highly unlikely since the amount and velocity of water from seepage would not cause a scouring of the downstream sandstone.

*Positive (*factors contributing to potential failure*): 1) Some of the foundation rock may be weak limestone with solution cavities.*

*Negative (*factors prohibiting potential failure*): 1) The seepage flow pattern resulting in erosion of the limestone is likely a very long time frame and is highly unlikely due*

to the quantity and velocity of seepage. 2) The periodic monitoring by CITY staff and diving inspections would identify erosion areas in an early enough stage to allow for mitigation. 3) Soundings have been performed since the CITY took over the License in 1983 without any further indication of erosion of the rock.

3.2.44.2 Assessment of Risk Reduction Opportunities – Based on the operating conditions to date, the limestone rock is not eroding thus the risk of potential failure by seepage is very unlikely. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.44.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II with a recommendation that the area downstream of the powerhouse should continue to be monitored and maintained to verify satisfactory performance. However, this failure mode is highly unlikely and even though this is a regularly monitored and maintained structure it is recommended to be re-classified as a Category IV.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be re-classified as a Category IV even though the CITY monitors this on a regular basis.

This failure mode is re-classified as Category IV.

3.2.45 PFM 3.45 – Downstream erosion causing undermining and failure of the powerhouse due to overturning.

3.2.45.1 Description of PFM 3.45 – Erosion downstream of powerhouse resulting in a potential overturning failure. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for erosion downstream of the powerhouse. This erosion could lead to undermining and failure of the powerhouse because of overturning. There have been no analyses performed to date on the Overturning Stability of the Powerhouse. The Ebasco Report dated 1992 does not identify any factors of safety for overturning analysis. If an overturning analysis has not already been performed, it is recommended to be prepared and submitted to FERC.

*Positive (*factors contributing to potential failure*): 1) Some of the foundation rock may be weak limestone with solution cavities. 2) Powerhouse discharges could erode the downstream rock causing a lowering of the tailrace area, resulting in a potential overturning of the structure.*

*Negative (*factors prohibiting potential failure*): 1) The flow pattern resulting in erosion of the limestone is likely a very long time frame. 2) The periodic monitoring by CITY staff and diving inspections would identify erosion areas in an early enough stage to allow for mitigation. 3) Soundings have been performed since the CITY took over the License in 1983 without any further indication of erosion of the rock.*

3.2.45.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect. It is recommended that either a historic overturning analysis needs to be located, reviewed, and submitted to FERC or a new overturning analysis needs to be performed and submitted to FERC.

3.2.45.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

This failure mode was originally classified as a Category II in combination with PFM 43 and PFM 44, with a recommendation that the powerhouse should continue to be monitored and maintained to verify satisfactory performance. However, this failure mode was apparently never fully evaluated as determined by review of the original Ebasco Report of 1992, and therefore if being classified as a Category III until such time as a previous overturning evaluation is either found and submitted to FERC, or a new overturning analysis is performed and submitted and approved by FERC.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be re-classified as a Category III until such time as an overturning analysis is located or performed.

This failure mode is re-classified as Category III.

3.2.46 PFM 3.46 – Failure of the dam caused by the presence of sinkholes.

3.2.46.1 Description of PFM 3.46 – Failure of dam by presence of sinkholes. (***Taken from PFMA Report and enhanced by Review Team***)

The formation of sinkholes could lead to localized erosion and failure of the dam due to piping of material.

*Positive (*factors contributing to potential failure*): 1) Most of the foundation soils are porous which are pipeable, which could be susceptible to the creation of sinkholes.*

*Negative (*factors prohibiting potential failure*): 1) The flow pattern is very long and through numerous layers of sheetpiling. 2) The toe drain system alleviates most of the hydraulic head that could lead to the creation of sinkholes. 3) There has been no observation of sinkholes in the past 18 years.*

3.2.46.2 Assessment of Risk Reduction Opportunities – Based on the design and the operating conditions to date, the risk of potential failure is highly unlikely. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.46.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be remain classified as a Category IV since there is no historical evidence of sinkholes.

This failure mode remains classified as Category IV.

3.2.47 PFM 3.47 – There is a potential for failure of the dam as a result of seismic loading.

3.2.47.1 Description of PFM 3.47 – Failure of dam by seismic loading. (***Taken from PFMA Report and enhanced by Review Team***)

There is a potential for failure of the dam as a result of seismic loading.
*Positive (*factors contributing to potential failure*): 1) There is always a potential for a seismic event, especially in today's oil and gas exploration with the use of fracking.*

*Negative (*factors prohibiting potential failure*): 1) The closest identified earthquake is in Northern Virginia and 2) there is very little oil and gas production in the immediate area.*

3.2.47.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.47.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode should be remain classified as a Category IV since the closest known seismic event is in Northern Virginia.

This failure mode remains classified as Category IV.

3.2.48 PFM 3.48 – Overtopping of embankment which causes embankment failure - Construction Condition Phase 3 for the Auxiliary Spillway – Unusual Conditions.

3.2.48.1 Description of PFM 3.48 – Overtopping of embankment during construction of Auxiliary Spillway – Unusual Conditions. (***Taken from PFMA Report and enhanced by Review Team***)

Overtopping of embankment due to gate failure causes overtopping, erosion and an uncontrolled release of the reservoir. This would require a reservoir rise of greater than 7 foot rise and would overtop the wave barrier wall, all flows through gated and Auxiliary spillway and resultant overtops the embankment (PMF) situation - highly unlikely.

*Positive (*factors contributing to potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

*Negative (*factors prohibiting potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

3.2.48.2 Assessment of Risk Reduction Opportunities –There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.48.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode which was originally classified as a Category I in the 2009 Auxiliary Spillway PFM, should be re-classified as a Category IV since it was a construction phase PFM for the Auxiliary Spillway and is no longer applicable.

This failure mode is re-classified as Category IV.

3.2.49 PFM 3.49 – Overtopping of embankment with Auxiliary Spillway in place which causes embankment failure - Normal Conditions.

3.2.49.1 Description of PFM 3.49 – Overtopping of embankment during construction of Auxiliary Spillway – Normal Conditions. (***Taken from PFMA Report and enhanced by Review Team***)

Overtopping of embankment during Phase 3 construction (remove cofferdam) of Auxiliary Spillway due to high flows causes overtopping, erosion and an uncontrolled release of reservoir. This would require a reservoir rise of greater than 7 foot rise and would overtop the wave barrier wall. Since this was a construction condition and is no longer applicable.

*Positive (*factors contributing to potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

*Negative (*factors prohibiting potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

3.2.49.2 Assessment of Risk Reduction Opportunities –There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.49.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode which was originally classified as a Category I in the 2009 Auxiliary Spillway PFM, should be re-classified as a Category IV since it was a construction phase PFM for the Auxiliary Spillway and is no longer applicable.

This failure mode is re-classified as Category IV.

3.2.50 PFM 3.50 – During construction, heavy rain causes soil erosion and embankment failure beneath Auxiliary Spillway, which leads to an uncontrolled release of the reservoir.

3.2.50.1 Description of PFM 3.50– Heavy rains cause erosion and embankment failure during construction of Auxiliary Spillway – Normal Conditions. (***Taken from PFMA Report and enhanced by Review Team***)

An unusual weather event erodes exposed embankment soils during construction which leads to an uncontrolled release of the reservoir. Since this was a construction condition and is no longer applicable.

*Positive (*factors contributing to potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

*Negative (*factors prohibiting potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

3.2.50.2 Assessment of Risk Reduction Opportunities –There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.50.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode which was originally classified as a Category I in the 2009 Auxiliary Spillway PFM, should be re-classified as a Category IV since it was a construction phase PFM for the Auxiliary Spillway and is no longer applicable.

This failure mode is re-classified as Category IV.

3.2.51 PFM 3.51 – During Normal Operation, heavy rain causes soil erosion and embankment failure beneath Auxiliary Spillway, which leads to an uncontrolled release of the reservoir.

3.2.51.1 Description of PFM 3.51– Heavy rains cause erosion and embankment failure during operation of Auxiliary Spillway – Normal Conditions. (***Taken from Auxiliary Spillway PFMA Report and enhanced by Review Team***)

An unusual weather event causes a rise in the normal pool and creates a flow path beneath the Auxiliary Spillway leading to erosion (boil) and exposes embankment soils which lead to a failure of a part of the Auxiliary Spillway leading to an uncontrolled release of the reservoir.

*Positive (*factors contributing to potential failure*): 1) Most of the embankment soils are porous and pervious which are pipeable.*

*Negative (*factors prohibiting potential failure*): 1) There is a sheetpile cutoff wall encapsulating the Auxiliary Spillway, and 2) there are sub-surface drains to drain water. 3) There are also three (3) piezometers in the each wall (north/south) and four (4) flush mount in the middle of the Auxiliary Spillway; two are flush mount on*

top plateau and two are flush mount in stilling basin. 4) This is an extreme event, but monitoring with piezometers is prudent and will continue.

3.2.51.2 Assessment of Risk Reduction Opportunities – Based on the original design of the Auxiliary Spillway and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.51.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this is a new failure mode which is highly unlikely to occur due to the physical components of the new Auxiliary spillway is should be classified as a Category IV.

This failure mode is classified as Category IV.

3.2.52 PFM 4.52 – Loss of embankment cofferdam during construction of Auxiliary Spillway leads to erosion of embankment with an uncontrolled release of the reservoir.

3.2.52.1 Description of PFM 4.52– Loss of embankment cofferdam during construction of Auxiliary Spillway leads to erosion of embankment. (***Taken from PFMA Report and enhanced by Review Team***)

Both the primary and secondary cofferdams fail which would lead to overtopping of the dam. Since this was a construction condition and is no longer applicable.

*Positive (*factors contributing to potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

*Negative (*factors prohibiting potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

3.2.52.2 Assessment of Risk Reduction Opportunities –There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.52.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode which was originally classified as a Category I in the 2009 Auxiliary Spillway PFM, should be re-classified as a Category IV since it was a construction phase PFM for the Auxiliary Spillway and is no longer applicable.

This failure mode is re-classified as Category IV.

3.2.53 PFM 3.53 – Construction loading causes a slope failure during construction of Auxiliary Spillway.

3.2.53.1 Description of PFM 3.53– Construction Loading during construction of Auxiliary Spillway leads to slope failure. (***Taken from PFMA Report and enhanced by Review Team***)

Construction traffic on crest causes slope instability, which leads to an uncontrolled release of the reservoir. Since this was a construction condition and is no longer applicable.

*Positive (*factors contributing to potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

*Negative (*factors prohibiting potential failure*): This PFM was developed prior to the installation of the Auxiliary Spillway. This PFM is now not applicable since the Auxiliary Spillway in now in place.*

3.2.53.2 Assessment of Risk Reduction Opportunities –There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.53.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode which was originally classified as a Category I in the 2009 Auxiliary Spillway PFM, should be re-classified as a Category IV since it was a construction phase PFM for the Auxiliary Spillway and is no longer applicable.

This failure mode is re-classified as Category IV.

3.2.54 PFM 3.54 – Seepage along new sheet pile interface causes internal erosion and piping of the embankment leading to an uncontrolled release of reservoir.

3.2.54.1 Description of PFM 3.54 – Seepage along new sheetpile interface causes internal erosion and piping of embankment. (***Taken from PFMA Report and enhanced by Review Team***)

The failure mode would require seepage around the sheet pile perimeter boundary of the Auxiliary Spillway. This PFM was not clearly defined during the initial PFM during the 2009 PFM for the Auxiliary Spillway. This was discussed as a "new" PFM during this current review.

*Positive (*factors contributing to potential failure*): 1) Most of the embankment soils are porous and pervious which could lead to potential piping of materials.*

*Negative (*factors prohibiting potential failure*): 1) The flow pattern is very long and resulting gradients should be low. 2) Seepage would have to occur bypassing the*

wing walls and around the extensive length of the sheetpiling. This is a "tortuous path" and is highly unlikely to occur.

3.2.54.2 Assessment of Risk Reduction Opportunities – Based on the original design of the Auxiliary Spillway and the operating conditions to date, the original design attempted to reduce the risk of potential failure to the greatest extent physically and economically possible. There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.54.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode which was originally classified as a Category II in the 2009 Auxiliary Spillway PFM, should be re-classified as a Category IV since it is a "tortuous path" and is highly unlikely to occur.

This failure mode is re-classified as Category IV.

3.2.55 PFM 3.55 – Barge/Boat Impact on Gated Spillway.

3.2.55.1 Description of PFM 3.55 – Barge/Boat impact on Gated Spillway. (***New PFM by Review Team***)

The failure mode would be for impact of a floating object on the reservoir that impacts the gated spillway.

*Positive (*factors contributing to potential failure*): 1) There have been instances on other FERC projects where runaway barges or boats have struck critical structures.*

*Negative (*factors prohibiting potential failure*): 1) There are no large barges or boats or other large structures that could impact the gated spillway. 2) The plant operators are observing the conditions of the reservoir on a daily basis and would be able to identify any potential runaway vessel in time to divert it from the powerhouse or gated spillway. This is not a realistic PFM.*

3.2.55.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.55.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode is highly unlikely and should be classified as a Category IV.

This failure mode is classified as Category IV.

3.2.56 PFM 3.56 – Lightning Strikes Powerhouse.

3.2.56.1 Description of PFM 3.56 – Lightning Strikes on Powerhouse. (***New PFM by Review Team***)

This failure mode could strike the powerhouse and causes an explosion resulting in a failure of all controls to be able to operate the spillway gates or wicket gates, resulting in a "controlled" release (maximum capacity of water passage or spillway gate).

*Positive (*factors contributing to potential failure*): 1) Lighting strikes of structures are not unrealistic and can cause damage and/or electrical/mechanical failures.*

*Negative (*factors prohibiting potential failure*): 1) The Plant has a lightning elimination system. 2)This PFM is a damage issue, not a PFM that could cause an uncontrolled release of the reservoir. Most storms are tropical, very little lightning with hurricanes. This is not a realistic PFM.*

3.2.56.2 Assessment of Risk Reduction Opportunities – There are no additional recommendations for risk reduction other than to continue to monitor and maintain the overall Project monitoring plan as currently in effect.

3.2.56.3 Assessment of Surveillance and Monitoring Plan – The current Surveillance and Monitoring Plan is adequate and acceptable.

It is the opinion of PFM Review Team and ARO staff that this failure mode is highly unlikely and should be classified as a Category IV.

This failure mode is classified as Category IV.

The entire PFMA Report and the PMFA Report in 2009 for the Auxiliary Spillway, including the Case Evaluations is included in the STI Document. The attached Table 3-1 (revised) should be substituted in the STI Document.

Viable Potential Failure Modes

There were no sole Category I - Highlighted Potential Failure Modes identified during the PFMA. There were three (3) Category I/III Highlighted Potential Failure Modes that are currently being addressed with the proposed embankment slope remediation that is being reviewed for approval by FERC. There are two (2) Category III – Considered, but not Highlighted Potential Failure Modes The first relating to the "close-up" inspection of the Tainter Gates for which a detailed structural inspection is being schedule and for which the report is due to FERC on December 31, 2015. The second relating to the lack of analysis for overturning of the powerhouse which is recommended to be performed and filed for approval with FERC.

During the PFMA Review there was discussion concerning the re-classification of certain Category IV Failure Modes to Category II due to the ongoing monitoring of the Project instrumentation that was initiated during the design and construction phases of the Project. The scope and schedule of instrumentation and their reading schedules are appropriate for this Project with the addition of 5-year diving inspections performed prior to each future Part 12D Inspection.

There were a number of Category II and Category III Failure Modes that were changed to Category IV either due to the construction of the Auxiliary Spillway, or due to additional information that was located during the PFM Review, or due to the highly unlikely failure event, but still being monitored by the CITY. Even though the Category IV ratings do not require monitoring of the installed instrumentation, CITY intends to continue monitoring instrumentation on various project features as a part of its prudent operation and maintenance of the Project. The Independent Consultant concurs with this decision and recommends instrumentation monitoring continues in accordance with the original Project Instrumentation Manual. ARO staff concurs during the PFMA Review that all instrumentation monitoring does not necessitate a Category II status.

Similarly, there were some Category IV Failure Modes that were changed to Category II because of the ongoing monitoring and presentation of data results in the DSSMP.

Finally there were Category IV Failure Modes identified in the Initial PFMA and subsequent PFMA reviews that retained the Category IV status.

As documented above, the following additional PFM cases were discussed during the PFMA Review. These cases were new issues that were raised by the Independent Consultant during the PFMA Review preparation for discussion at the PFMA Review.

PFM 3.55 – Barge/Boat impact on Gated Spillway or Powerhouse. This would require the barge or boat to impact the gated spillway or powerhouse. This scenario was deemed to be very unlikely since there are no barges or large boat on the reservoir. This was classified as a Category IV by the PFMA review team.

PFM 3.56 – Lightning Strikes on Powerhouse - This would involve a lighting strike to the powerhouse that would cause an explosion resulting in a failure of all controls to be able to operate the spillway gates or wicket gates, resulting in a "controlled" release (maximum capacity of water passage or spillway gate). This is a damage issue, not a resulting uncontrolled release of water. This was classified as a Category IV by the PFMA review team.

Additional/Continued Monitoring or Performance Related Items Discussed

The Jackson Bluff Hydropower Project was determined to have a low potential for failure in all of the modes that were evaluated. There were no areas that required any immediate or long term physical changes or enhancements as performance related items. There were four PFMs that require additional evaluation.

The PFMA review team recommends that monitoring of all project instrumentation continue in accordance with the current Instrumentation Program.
There were no new instrumentation that was recommended unless the one survey monument on the left downstream training wall is unable to be located.

PFMA SUMMARY AND CONCLUSIONS

In summary, due to the conservative nature of the original design and the Auxiliary Spillway design, the ongoing maintenance activities by CITY staff, the location of the Project and the size of the components of the Project, the only Highlighted (Category

1/III) Potential Failure Modes were related to the ongoing design remediation for the 40 foot area of embankment slope erosion. The original and updated design, construction, documentation and existing operations all contribute to the Jackson Bluff Hydropower Project being considered a safe project. Project structures are considered sound and meet the functional and various loading requirements of the Project.

Table 3-1

Updated Assessment and Categorization of Potential Failure Modes (2015)

Potential Failure Mode	Description	Category
3-1	Sliding of the powerhouse along a weak foundation plane during flooding conditions - Unusual Condition	II
3-2	Concrete deterioration and/or reinforcement corrosion in the powerhouse during flooding conditions leading to an overstress condition failure	IV
3-3	Build-up of debris in front of the trash rack resulting in a collapse of the trashrack, triggering high vibrations of the turbine-generator unit, concrete failure, and a loss of the reservoir	Ii
3-4	Sliding of the tainter gate spillway along a weak foundation plane during flooding conditions - Unusual Condition	II
3-5	Concrete deterioration and/or reinforcement corrosion in the tainter gate spillway leading to an overstress condition failure - Unusual Conditions	IV
3-6	Build-up of debris in front of one or more of the tainter gates resulting in a decreased gate capacity leading to overtopping of another section of the dam	IV
3-7	Mechanical or electrical failure of the gate hoists resulting in an overtopping of the gates or another section of the dam	II
3-8	Failure of the operator to properly operate the project	IV
3-9	Overtopping failure of the embankment caused by inflow in excess of the gated spillway capacity that has been reduced by debris blockage	IV
3-10	Overtopping failure of the embankment caused by the reservoir level reaching 77.0 feet without a "failure" of the fuse plug	IV
3-11	Overtopping failure of the embankment due to an inflow greater than that of the IDF	IV
3-12	Instability of the downstream slope during flooding conditions caused by tailwater saturation of the downstream slope and drawdown as water recedes	II
3-13	Potential sliding failure of the downstream slope during flooding conditions due to higher water levels and hydraulic gradients	II
3-14	Failure of the upstream slope during flooding conditions due to drawdown conditions from receding headwater	IV
3-15	Overtopping of the fuse plug during flooding conditions leading to erosion of material downstream of the fuse plug and a deep breach resulting in an	IV

Table 3-1

Updated Assessment and Categorization of Potential Failure Modes (2015)

Potential Failure Mode	Description	Category
	uncontrolled release of the reservoir	
3-16	Erosion of the emergency spillway with the pool at elevation 72.0 feet or more resulting in a deep breach and an uncontrolled release of the reservoir	IV
3-17	Sliding of the powerhouse along a weak foundation plane during Normal Conditions	II
3-18	Concrete deterioration and/or reinforcement corrosion in the powerhouse during normal conditions leading to an overstress condition failure	IV
3-19	Instability of the powerhouse during normal conditions due to flotation effects caused by the dewatering of multiple units	IV
3-20	Mechanical shaft failure in addition to the hydraulic system resulting in an uncontrolled release of water and erosion downstream of the powerhouse	IV
3-21	Failure in Potential Failure Mode No. 20 causes flooding of the powerhouse and a failure of the downstream powerhouse wall	IV
3-22	Sliding of the tainter gate spillway along a weak foundation plane during Normal Conditions. Similar to PFM 4.	II
3-23	Concrete deterioration and/or reinforcement corrosion in the tainter gate spillway during normal conditions leading to an overstress condition failure under Normal Conditions. Similar to PFM 5.	IV
3-24	Failure of the trunnion anchorage leading to tainter gate failure and an uncontrolled release of the reservoir	III
3-25	High trunnion pin friction moments leading to tainter gate failure and an uncontrolled release of the reservoir	II
3-26	Erosion of the upstream embankment slope during Normal Conditions due to wave action	IV
3-27	Instability of the downstream slope of the embankment caused by combined seepage and rainfall (non-hurricane event) resulting in elevated phreatic surface levels - Unusual Condition	I/III
3-28	Instability of the downstream slope of the embankment caused by combined seepage resulting in elevated phreatic surface levels - Normal Condition	I/III
3-29	Instability of the downstream slope of the embankment caused by rainfall (Hurricane event) resulting in elevated phreatic surface levels - Extreme	I/III

Table 3-1

Updated Assessment and Categorization of Potential Failure Modes (2015)

Potential Failure Mode	Description	Category
	Condition	
3-30	Boils/piping caused by uplift pressures at the toe of the embankment due to windows in the Wakefield piling that would lead to failure of embankment	IV
3-31	Progressive seepage failure caused by uplift pressures at the toe of the embankment due to windows in the Wakefield piling that would lead to failure of embankment	IV
3-32	Boils/piping caused by uplift pressures at the toe of the embankment due to pervious zones in the lime rock that would lead to failure of embankment	IV
3-33	Progressive seepage failure caused by uplift pressures at the toe of the embankment due pervious zones in the lime rock that would lead to failure of embankment	IV
3-34	Burrowing of animals leading to a seepage path and the failure of the embankment	II
3-35	Failure of the right upstream approach wall due to structural failure of sheetpiling due to corrosion resulting in uncontrolled release	IV
3-36	Failure of the right upstream approach wall due to erosion scouring of embankment following failure of sheetpiling (PFM 35)	IV
3-37	Failure of the right downstream training wall due to loss of embankment material and formation of breach	II
3-38	Failure of the left downstream training wall due to loss of embankment material and formation of breach	IV
3-39	Failure of the right downstream training wall due to loss of abutment material and formation of breach	II
3-40	Failure of the left downstream training wall due to loss of abutment material and formation of breach	IV
3-41	Erosion downstream of the tainter gate resulting in undermining of the gate section and a progressive seepage failure	II
3-42	Erosion downstream of the tainter gate resulting in undermining of the gate section and a progressive stability failure	II
3-43	Downstream erosion causing undermining and failure of the powerhouse due to sliding	II

Table 3-1

Updated Assessment and Categorization of Potential Failure Modes (2015)

Potential Failure Mode	Description	Category
3-44	Downstream erosion causing undermining and failure of the powerhouse due to seepage	IV
3-45	Downstream erosion causing undermining and failure of the powerhouse due to overturning	III
3-46	Failure of the dam caused by the presence of sinkholes	IV
3-47	Failure of the dam caused by seismic loading	IV
3-48	Overtopping of embankment with Auxiliary Spillway in place which causes embankment failure - Construction Condition - Phase 3	IV
3-49	Overtopping of embankment with Auxiliary Spillway in place which causes embankment failure - Normal Conditions (Same as PFM 11)	IV
3-50	During construction, heavy rain causes soil erosion and embankment failure beneath Auxiliary Spillway, which leads to uncontrolled release of reservoir	IV
3-51	During Normal Operation, heavy rain causes soil erosion and embankment failure beneath Auxiliary Spillway, which leads to uncontrolled release of reservoir	IV
3-52	Loss of embankment cofferdam during construction leads to erosion and piping of embankment with an uncontrolled release of the reservoir	IV
3-53	Construction loading causes slope failure	IV
3-54	Seepage along new sheet pile interface causes internal erosion and piping of the embankment leading to an uncontrolled release of reservoir	IV
3-55	Barge/boat impact on Gated Spillway	IV
3-56	Lightning Strikes Powerhouse	IV

4.0 Surveillance and Monitoring with Respect to Potential Failure Modes

Additional or continued monitoring was recommended from the PFMA for the following areas:

- Powerhouse - relating to sliding and overturning potential failures
- Powerhouse - relating to seepage for underlying limestone rock conditions
- Powerhouse - relating to changes in or new cracking in floors and/or walls
- Tainter Gate Spillway - relating to sliding and overturning potential failures
- Tainter Gate Spillway - relating to seepage for underlying limestone rock conditions
- Embankment - relating to adequacy of toe drain system, localized sloughing areas, burrowing of animals, upstream embankment erosion/scouring from wave action, riprap slope protection, undermining of concrete wave wall
- Auxiliary Spillway - relating to sliding and seepage
- Remote Dike - Periodic observations during mowing for animal burrowing

- Continued design-related monitoring of Project Instrumentation

4.1 Operator's Surveillance Program

The Project is a manned power project from an operational standpoint; with a full time maintenance person at the Project during normal business hours. CITY staff monitor the powerhouse and the operation of the hydroelectric equipment and operate the tainter gates, as necessary. Plant personnel maintain a daily log and note any unusual conditions.

There are also surveillance cameras at the front gate, powerhouse and spillway that are monitored by plant personnel.

4.2 Active Instrumentation

Instrumentation at the Project currently being monitored by CITY includes the following:

4.2.1 Monitoring Wells

There are 22 active open-end monitoring wells installed within the embankments. These monitoring wells measure phreatic surface water levels. Most of monitoring wells are in groups oriented in lines perpendicular to the center line of the embankment.

4.2.2 Piezometers

There are thirteen piezometers located at the Project. There are five piezometers within the tainter gate spillway, and one piezometer in the left abutment of the powerhouse. In the Auxiliary Spillway there are also three (3) piezometers at each training wall (north/south) and four (4) flush mount in the middle of the Auxiliary Spillway; two are flush mount on top plateau and two are flush mount in stilling basin.

4.2.3 Survey Control Points

There are 16 control points located at the project. There are four points associated with the powerhouse, eight locations along the spillway (including one at each pier), two points along upstream right approach wall, and three points along the downstream sheet pile training wall. There are nineteen (19) control points on the Auxiliary Spillway as shown on Sheet 12 of 44 in *Appendix D*.

4.2.4 Diving Inspections

Periodic underwater diving inspections are performed at the Jackson Bluff Hydroelectric Project in order to investigate, evaluate, define, and map any concrete cracking, deterioration, leakage and other anomalies. An underwater diving inspection has not been performed since the 2004 Part 12D Inspection. Following the PFMA Review it was agreed that the CITY would initiate a regular 5-Year diving program to be performed prior to each future Part 12D Inspection.

4.2.5 River Gauges

In the normal operation of the project and for the control of flood flows, the Licensee relies on river flow gauges operated and maintained by the U.S. Geological Survey (USGS) both in the reservoir and downstream.

Locations of the instrumentation are presented in *Appendix D*.

CITY performs upstream trashrack and tailrace profile surveys on an as-available basis when divers are called to the site for other reasons. These monitoring surveys are recommended to be revised to a regular 5-year interval and also be performed coincident with any other special diving activities in between the 5-year interval. It is understood that these special diving activities generally occur about every two years.

4.3 Threshold and Action Levels

4.3.1 Monitoring Wells

There is currently are threshold values established for monitoring wells. The data is reviewed to identify any trends.

4.3.2 Piezometers

There are threshold values established piezometers. The data is reviewed to identify any trends.

4.3.3 Survey Control Points

There are threshold values established for the deformation survey. The data is reviewed to identify any trends.

4.3.4 River gauges

There are no threshold or action levels for the river gages. Instead, information from these gauges is monitored by plant personnel to help manage flow rates. Headwater and tailwater gauges are used for SOP, which includes operating gates and activating the Emergency Action Plan.

For the collimation survey monuments, the acceptable range of variation is as follows:

Allowable deviations in horizontal position for the survey monuments are Initial Order, Class I accuracy as established by the 1984 FGCS Horizontal Control Survey Accuracy Standards.

For the vertical survey readings, allowable deviations in vertical elevations for the level survey of the monuments are Fifth Order, Class II accuracy as established by the USACE Classification System.

There is no other survey monument instrumentation at the Project site.

4.4 Reading Procedures/Frequency

Since 2009, CITY has installed (2006) and only performed one (1) set of alignment surveys (2014) and the information is included in ***Appendix D*** in graphical forms. Settlement readings are recorded at the same time the alignment survey is performed.

4.4.1 Monitoring wells and piezometers

The monitoring wells and piezometers are read manually every two weeks by measuring the water level in the wells (or stand pipes). After field readings, the data is processed and evaluated monthly by a qualified engineer.

4.4.2 Survey control points

The structure deformation survey (horizontal and vertical deformation) of the water-retaining structures was initially performed in June 2006 when the control panels were installed, and then again in 2014 prior to the Part 12D Inspection.

The settlement monument data have not been plotted versus time because there are only the original set of data and one additional set of readings. In July 2014, surveyors used GPS equipment to complete surveys, However the points did not match known points both on and off the dam. The surveyors then ran a bench-run from the dam to known KDOT control points to confirm vertical and horizontal control. These confirmed benchmarks and control point locations will be used in future surveys relating to water-retaining structures at the Project. (2014 DSSMR)

4.4.3 River Gauges

Data is obtained either automatically and continuously via a Supervisory Control and Data Acquisition (SCADA) system, or manually as needed via a dial-up system.

4.4.4 Diving inspections

Diving inspections are generally performed at least every 10 years. It is recommended that regular diving inspection be performed once every 5 years prior to the Part 12D Inspection.

4.5 Assessment of Instrumentation Data and Surveillance and Monitoring Plans Relative to Potential Failure Modes

A. The instrumentation section from the DSSMR in the initial STID that was created in July 2014 is enclosed in Appendix D for reference.

B. The Dam Safety Surveillance and Monitoring Report (DSSMR) dated December 2014 is the last data that was provided. The settlement and alignment surveys were established in 2006 to monitor Project structures for both horizontal and vertical movements. Additional control points were established within the Auxiliary Spillway and surveyed for the first time in July 2011. The 2006 and 2011 control points were re-surveyed on July 15, 2014. It is recommended that settlement and alignment surveys be performed on an annual basis until the next Part 12D Inspection in 2020 and plot those results to determine any trends in possible movement or settlement. The data that is currently available is unable to provide sufficient baseline data for such an analysis with this report.

C. It is recommended that the DSSMR annual reports be included in the STID at the end of each year after being filed with FERC.

4.5.1 Monitoring Wells and Piezometers

Embankment monitoring wells

The purpose of these wells is to monitor phreatic surface levels within the earth embankment. A plan view showing locations of the monitoring wells, along with a graphical representation of data since the last Part 12 inspection is enclosed in Appendix D. The following is a summary of each monitoring well along the earthen embankment from the DSSMR dated December 2013, since the DSSMR for 2014 had not yet been prepared:

- Well Nos. P25, P26, & P27 - These wells are located in the earthen embankment between the gated spillway and auxiliary spillway and have somewhat elevated phreatic readings. Peak readings at wells P26 and P27 were near established upper threshold values, and both P25 and P26 show phreatic conditions are sensitive to changes in reservoir levels. All readings at well P25 were well below upper threshold values, and readings reflect changes in tailwater levels.

- Well Nos. P22, P23, & P24 – These wells are located along the south training wall of the auxiliary spillway. Similar to wells P26 and P27, wells P22 and P23 have elevated readings which were also observed during construction of the auxiliary spillway. The levels at these wells are approximately one foot lower than the protective granular filter beneath the spillway concrete. Average readings at P22 and peak readings at P23 were at the upper threshold values. One reading at P22 reached the upper action level in March 2013, and then fell to normal levels for the rest of the review period. Phreatic levels drop significantly at P24, which is downstream of P22 and P23. Readings at this well closely followed changes in tailwater levels. Upper threshold and action levels were exceeded only when tailwater levels reached these stages. These events are considered normal and are not indicative of adverse conditions that may compromise the structural integrity of the dam.

- Well Nos. P20 & P21 – These wells are located in the southern portion of the auxiliary spillway. They have shown relatively constant readings during the review period. Readings indicate very little influence from changes in reservoir or tailwater

levels. All readings at P20 were well below upper threshold levels, and average readings at P21 were near upper threshold levels. No readings exceeded the upper action levels. Erratic readings in P20 and P21 observed during the previous review period appear to have been the result of errors made during collection or logging of data.

- Well Nos. P18 & P19 -- These wells are located in the northern portion of the auxiliary spillway. They have shown relatively constant readings with exception to July 2013 where P19 spiked higher and P18 was lower. No significant changes in reservoir or tailwater levels were noted during the spikes. All readings at P18 were well below upper threshold levels, and average readings at P19 were near upper threshold levels. Erratic readings in P18 and P19 observed during the previous review period appear to have been the result of errors made during collection or logging of data.

- Well Nos. P15, P16, & P17 – These wells are located along the north training wall of the auxiliary spillway. They appear to slightly respond to changes in tailwater elevation. Water levels in all three wells have all been relatively uniform, with one spike at P16 during January 2013. This spike did not coincide with changes in reservoir or tailwater levels and is likely the result of errors made during collection or logging of data. All readings at wells P15 and P16 were well below upper threshold values. One reading at P17 exceeded the upper action level when tailwater was also high. This event is considered normal and not indicative of adverse conditions that may compromise the structural integrity of the dam.

- Well Nos. P12, P13, & P14 – These wells are located in the earthen embankment north of the auxiliary spillway and have relatively low phreatic readings. These wells appear to be reading accurately and react to changes in tailwater. Water levels in upper wells P12 and P13 were well below upper threshold levels, and lower well P14 approached the upper threshold level during periods of high tailwater.

- Well Nos. 15, 3, & 4 – These wells are located near the middle of the earthen embankment. Well 15 reacts to changes in reservoir elevations, with slightly less reaction witnessed in well 3 while well 4 has remained stable. No trends in increasing phreatic surface are evident. All readings were well below upper threshold levels.

- Well No. 11 - This well is located near the toe of the slope in the northern portion of the earthen embankment. This well appears to be functioning properly with small reaction to changes in reservoir levels. All readings were below upper threshold levels.

- Well Nos. 14, 5A, 6A, 7A, & 9 – These wells are located in the slope near the northern portion of the earthen embankment. Readings have all been below upper threshold levels and monitoring data does not indicate any significant increases or trends in the phreatic surface. Well 7A appears to be plugged, with one spike occurring in October 2012. This spike was not witnessed in other nearby wells and is likely an error made during collection or logging of data. Recommend cleaning/redevelopment of Well 7A.

- Well Nos. 5, 6, 7, & 8 – These wells are located in the slope near the northern portion of the earthen embankment. Readings at wells 5, 6, & 8 have all been below their upper threshold levels. Well 7 briefly exceeded the upper threshold limit but returned to lower levels at the next reading. No readings of these wells exceeded the upper action levels. These wells appear to be functioning properly with no increasing trends in the phreatic water level. Phreatic levels react to small changes in reservoir levels.

- Well Nos. 12 & 13 – These wells are located near the toe of the slope in the northern portion of the earthen embankment. No upper threshold levels were reached and these wells do not show a trend of increasing phreatic level.

Spillway and Powerhouse Piezometers

The purpose of these piezometers is to monitor pore water pressures within specific soils layers which can be used to asses uplift of the structures. A plan view showing locations of the piezometers, along with a graphical representation of data since the last Part 12 inspection is enclosed in Appendix D. The following is a summary of each piezometer by the powerhouse or within the spillway since the last Part 12 inspection:

Bay Nos. 1A, 4B, & 7B – These piezometers measure uplift pressures within the foundation rock. They both show some fluctuations in pore water pressures that react to varying tailwater levels. Upper threshold values were reached, but are within the range of historic values recorded at these locations. Readings during high tailwater were not obtained since the piezometers were inaccessible.

Road – This piezometer previously showed very large fluctuation in pore water pressures. Since maintenance in 2010, this piezometer has stabilized and now reacts to changes in tailwater. Fluctuations within this review period span a range of 10 feet. The well appears to be functioning properly with no increasing trends in pore water pressures.

For a better understanding of how tailwater and rainfall affects the phreatic water level and pore water pressures, the tailwater elevation and the amount of rainfall have been plotted with the monitoring well and piezometer readings and that documentation has been included in the DSSMR and in the STID. Most of the piezometers tend to track the downstream river elevation very closely. Some do not seen to have correlation to either the reservoir level or the downstream river elevations and therefore indicate there is little or no seepage in those areas.

In addition it is recommended that threshold and action levels should be established for the existing instrumentation so that plant personnel will know when additional actions are needed, such as increase of the schedule of readings.

4.5.2 Settlement/alignment monuments

Powerhouse

Control points were placed and surveyed in 2006 and a follow-up surveys was completed on July 15, 2014 survey. It is recommended that the Licensee perform a

survey for the next years to identify if any movement and the trends that are occurring. These surveys should be conducted annually for the next five years to develop baseline data.

Spillway

Since the control points were placed and surveyed in 2006, no follow up surveys have been performed except for the July 15, 2014 survey. It is recommended that the Licensee perform a survey for the next years to identify if any movement and the trends that are occurring. These surveys should be conducted annually for the next five years to develop baseline data.

See *Appendix D* for available Instrumentation Plots from the 2014 DSSMR.

5.0 Field Inspection

5.1 Field Inspection Observations

The Independent Consultant and the accompanying individuals, Triveni Singh, City of Tallahassee, Manager of Power Production, Karl Bauer, City of Tallahassee, Engineer, Dave Mordecai, City of Tallahassee, Plant Manager, David McCranie, City of Tallahassee, Operator, Bobby Stafford, Jr., City of Tallahassee, Operator, John Breslin, FERC, ARO, Engineer, Elise Dombeck, FERC, ARO, Engineer and Florida Department of Environmental Protection (Krystle Hoenstine and Art Sengupta) made a site inspection of the outside facilities followed by a powerhouse inspection on August 20, 2014. Photographs were taken by the Independent Consultant on a pre-tour on August 19, 2014, and during the inspection on August 20, 2014. The inspection began at the southwest side of the Powerhouse access stairway, continued across the gated spillway section, then observed steel sheetpile wall on the north end of the gated spillway section south of the Auxiliary Spillway, across the downstream side of the Auxiliary Spillway, along the toe of the embankment and toe drain system to the north end of the embankment section, viewed the decommissioned fuse plug and emergency spillway area, returned across the top of the embankment and wave barrier wall. Then a boat tour was conducted, viewing the powerhouse trashrack area, gated spillway area, embankment areas, and emergency spillway area from the lake, including some probing of the depth of the riprap slope protection on the upstream side of the embankment. Returning to the powerhouse, the powerhouse gallery was inspected as was the gated spillway gallery. Then a detailed visual review, not a "close-up 10-Year structural inspection" of the gated spillway tainter gates was performed with operation of two of the gates - one on diesel standby power. Finally the inside of the powerhouse was toured. along with a review of the embankment area for the sub-station.

In general, the entire outside perimeter of the Project was found to be in very good condition.

With the relatively new Auxiliary Spillway there are few concerns that were identified during this Fifth Part 12D Inspection. The original facility features are well maintained for their age. The staff at the Plant is very familiar with the facility and by walking the grounds each day, any issues are quickly identified and remediated. The areas that were identified by the Independent Consultant, as described in the following paragraphs, are primarily related to continued monitoring requirements. There were no areas of concern that immediately affect the structural integrity of the Project.

1. The toe drain leakage has been present for many years and the newly installed toe drain and monitoring system is more than adequate when combined with the embankment piezometers. During the daily walks the water clarity should continue to be monitored to observe for any "piping of materials", especially following any storm event that would cause a significant inflow into the reservoir.

2. The areas of repair on the downstream embankment should continue to be monitored until grass is established. Continue to monitor the toe of the entire embankment for evidence of moisture - cattails, bog areas, etc. - especially identified when mowing.

3. Weed control around the toe drain outlet pipes and structures to provide good visibility to operators on their daily rounds.

4. Continue to monitor the north abutment of the embankment where the drainage enters from the northwest. Potential for weakening of the toe of the north abutment with overland and subsurface water flows exists and needs to be monitored. The previous riprap that was placed seems to be rising to the surface.

5. Continue to observe the upstream side of the powerhouse intake piers and gated spillway piers. Unless this is observed from a boat, concrete spalling is not easily discernible.

6. There were some areas of riprap on the upstream side of the boat tour that appeared to have slightly subsided into the reservoir area. Continue to observe the riprap-waterline interface from the boat to monitor potential sloughing of riprap from wave action.

7. Continue to monitor the powerhouse floors and walls for any new cracks or expansion of existing cracks.

8. Monitor any leakage in the powerhouse or spillway galleries.

5.1.1 Settlement

There were no observed indications of settlement of the Powerhouse or the Upstream or Downstream Training Walls which tie the Powerhouse and Gated Spillway to the right and left abutments. There were no observed indications of settlement of the Gated Spillway or Auxiliary Spillway sections. There were no indications of settlement on the embankment section.

5.1.2 Movement

There were no indications of structural movement observed in Powerhouse, the Upstream or Downstream Training Walls, Gated Spillway, or Auxiliary Spillway sections.

5.1.3 Erosion

There was no visual evidence of erosion/sloughing of embankment.

5.1.4 Seepage

Seepage was observed on a few areas of the downstream slope and toe of the embankment between Sta. 27+00 and 32+00 during the Part 12D Field Inspection. These areas are being addressed in the 2015 Embankment Mitigation Seepage Plan prepared by Mead & Hunt.

5.1.5 Leakage

There were generally no areas that could be classified as leakage observed during inspection in the Powerhouse structure or the gallery sections of the powerhouse and gated spillway section. There was one roof leakage between Gallery Area 2A and 2B

which was identified as having been present for some time. This leakage could be derived from the 3" diameter air holes as shown on Maguire/Beebe Spillway Dam Plan View Drawing C12 dated July 1982. Since its has been ongoing for some time, it will just continue to be observed.

5.1.6 Cracking

No significant cracks were noted during this inspection.

5.1.7 Deterioration

There was no deterioration of any of the concrete structures observed during this inspection.

5.1.8 Geological Conditions

A detailed description of the Project geology is presented in the STI document. There was nothing observed during this inspection that would change the information that was developed during the design and construction. No seismic events have been reported since completion of construction of the Auxiliary Spillway and none are known to have existed prior to that time period.

5.1.9 Foundation Deterioration

There was no evidence of foundation deterioration observed during this inspection.

5.1.10 Condition of Spillway, Gates, Flashboards and Outlets

The Project does have a new Auxiliary Spillway. The Emergency Spillway and Fuse Plug were decommissioned while completing the Auxiliary Spillway and by constructing the Remote Dike. Headwater is controlled by the CITY with its gated spillway and Auxiliary Spillway as well as flow through the Powerhouse.

Overall the upstream approach and downstream sheetpile training walls are in good condition. The downstream sheetpile training wall is stepped, with a lower and upper section. The upper sheetpile section is set back about 10 feet from the lower section. The lower section is capped with a concrete slab which has been repaired from previous cracking, settlement and holes. There have been additional drains added upstream to provide controlled seepage outlets.

The upper section of the downstream training wall is supported by steel kickers at its upstream end and a 130 L.F. sheet pile anchor wall acting as a deadman securing the entire length of the training wall as shown on Maguire/Beebe North Training Wall Site Plan Drawing C9 dated July 1982.

The water passages associated with the three units were not dewatered during the inspection.

5.1.11 Observation of Operation of Representative Number of Spillway Gates and Standby Power

There are seven (7) spillway Tainter gates associated with the Project. Standby power for gate operation is not an issue. Two of the six gates on the spillway were operated during the inspection with all seven in good operating condition.

A general observation was made of the condition of the spillway Tainter gates during the Part 12D Inspection walk-through. These gates were remediated following the 2004 Part 12D Inspection and there was no observations of corrosion, or weld failures. A detailed "close-up" structural inspection is due every 10-year period and this will be performed by a structural engineering firm under contract with the CITY prior to December 31, 2015. The Independent Consultant will review the report and provide a supplement to this report upon completion.

The emergency diesel generator was operated during this inspection and ran one of the Tainter Gates. The other Tainter gate was started on normal power and operated without incident. CITY personnel state that the generator is checked, started, and run each week as part of the normal operation and maintenance program. A record of this operation is maintained in the Project logbook.

5.1.12 Reservoir Rim Stability

The Project reservoir is relatively long and shallow as the upstream pool formed by Jackson Bluff Dam. The surrounding terrain is not conducive to massive slides and there has only been one embankment dam failure over the nearly 90 years that the reservoir has been in operation.

5.1.13 Uplift Pressures in Structures, Foundations and Abutments

There were no observed indications of excessive uplift during this inspection that might affect the stability of the Project.

5.1.14 Functioning of Foundation Drains and Relief Wells

The Project has a system of toe drains and drainage culverts along the downstream side of the embankment wells. There is a 12-inch thick, 25-foot long sand toe drain. A 6-inch PVC perforated pipe collection system is present between Sta. 27+00 and 30+00 at the downstream toe. The perforated pipe was removed between Sta. 1+25 and 9+50 as the Auxiliary Spillway was constructed. This collection system and downstream channel/culvert system transfers seepage from within the dam embankment to the downstream river. The piezometers in this area of the embankment document that this drain system is functioning properly.

5.1.15 Other Significant Conditions

There were no other significant conditions observed during this inspection.

5.2 Status of Response(s) to Recommendation(s) in Last Part 12D Report

The previous (Fourth) Part 12D Inspection Report made several recommendations relating to surveillance and monitoring. First, the Licensee should submit surveillance and monitoring data as outlined in Chapter 14, Appendix J and K of the Engineering Guidelines which were issued as a draft in 2007 and updated in 2008. A thorough

review of all monitoring data should be performed by a qualified engineer each month. The CITY submits its DSSMR reports on a timely basis, but these need to be added to the STID.

Further, a recommendation was made relating to the erratic readings of Monitoring Well Nos. 6, 7, 8, 12, and 13 within the earthen embankments and should have a falling head test performed and/or flushed to verify if the wells are functioning properly. Piezometers Nos. 1B, 5A, 7B, and Road located within the spillway and by the powerhouse should also have a falling head test performed and/or flushed. Monitoring Well Nos. 6, 7, and 8 were re-developed in December 2010. Monitoring Well Nos. 12 and 13 were slug tested in December 2010. The Piezometer Road was slug tested - Piezometers 1B and 5A were determine by a qualified engineer to be inaccurate in showing uplift pressures on the gated spillway structure, this are no longer be used (although they remain in place if needed at a later date).

Tailwater elevation and rainfall amounts have been plotted along with the monitoring well and piezometer readings to aid in the evaluation of unusual data for the period 2011 to 2013. At the time of the Part 12D Inspection the 2014 data had not been collected and/or plotted or filed as the 2014 DSSMR. The 2013 DSSMR Report plots are included in Appendix D. The DSSMR Reports are recommended to be added to the STID as they are filed with FERC.

Threshold and action levels should be established for the monitoring wells, piezometers, and alignment surveys. The following threshold and action levels were presented in the STID:

Earth embankments: Upper threshold values for the monitoring wells are set 3 to 5 feet lower than the action levels.

Table 2-4. Monitoring Well Action Levels
Earth Embankment and Spillway

Monitoring Well	Upper Action Value (feet, NGVD)	Lower Action Value (feet, NGVD)
Station 0+75		
P25	68.5	None
P26	65.6	None
P27	59.3	None
Station 1+26		
P22	63.2	None
P23	63.2	None
P24	43.0	None
Station 4+11		
P20	63.2	None
P21	43.0	None
Station 6+63		
P18	63.2	None
P19	43.0	None
Station 9+49		
P15	63.2	None

Monitoring Well	Upper Action Value (feet, NGVD)	Lower Action Value (feet, NGVD)
P16	63.2	None
P17	43.0	None
Station 10+25		
P12	68.5	None
P13	59.5	None
P14	46.4	None

Table 2-4. Monitoring Well Action Levels
Earth Embankment and Spillway

Monitoring Well	Upper Action Value (feet, NGVD)	Lower Action Value (feet, NGVD)
Station 18+02		
15	68.5	None
3	68.5	None
4	60.6	None
Station 28+49		
14	68.5	None
5A	68.5	None
6A	66.1	None
7A	63.35*	None
9	52.04*	None
Station 27+00+/-		
11	52.66*	None
Station 28+91		
5	68.5	None
6	68.5	None
7	59.8	None
Station 29+91 +/-		
8	52.01*	None
Station 30+00+/-		
12	53.26*	None
Station 32+00+/-		
13	53.89*	None

* Top of Casing

Earth embankments: Slope stability analyses were completed for the embankments using the UTEXAS4 slope stability software (Wright, 1999) with TexGraf4 graphical output. The Spencer's Method of Slices (1967) was used for determining and identifying critical slope stability factors of safety. Sensitivity analyses were performed by raising the phreatic level within the monitoring wells until the factor of safety was determined to be at the FERC's minimum requirement (FOS 1.5), or the ground surface (whichever is lower) for a normal pool elevation. The corresponding phreatic levels were then used to establish action levels within each of the wells.

Auxiliary spillway: The upper action levels were set at the elevation of the bottom of concrete. Lower action levels were not set since this instrumentation is primarily to measure uplift forces on the paved spillway.

A settlement and alignment survey of the control points which were installed in 2006 have only been surveyed once for updated control in July 2014 since installation. It was recommended that a survey be performed within the next year, and annually thereafter for five years to develop baseline data. The only survey was performed prior to the Fifth Part 12D Inspection in 2014.

An underwater inspection should be performed prior to the next Part 12 Inspection. The latest underwater inspection was performed in 2010 and summarized in the July 2010 DSSMR.

Since the monitoring wells and piezometers associated with the Auxiliary Spillway project were installed, the STID document was updated in 2014 to show the new wells (See page 188/360) to reflect the changes. There are no survey monuments show on the Instrumentation exhibit for the Auxiliary Spillway area; but Mead & Hunt plans to update the DSSMP. The STID was initially developed in 2004 and the Auxiliary Spillway has been updated in the STID in 2014.

5.3 Field Observations with Respect to Potential Failure Modes

No observations were made during the field inspections that would change the conclusions of the PFMA Report.

5.4 Adequacy/Operation of Public Alert System

The Emergency Action Plan (EAP) dated December 2010 is posted and was reviewed. The EAP for the Project is adequate and includes the Auxiliary Spillway. All other safety alert systems are in proper working order and are adequate. Flood levels are controlled by the CITY through powerhouse discharge and floodgate operation. The CITY is responsible to alert County Emergency Management Officers, Sheriff's Departments and FERC of potential flooding situations. The County Emergency Management Officers and Sheriff's Departments then notify downstream and lake residents. The Public Safety Plan was also reviewed and is found to be adequate.

5.5 Conclusions

The field inspection of the Jackson Bluff Hydropower Project was conducted on August 19th (pre-inspection visit) and the formal inspection was held on August 20, 2014. The inspection consisted of visual observations of the Powerhouse, Gated Spillway, Auxiliary Spillway, intake and outlet channels, embankments and the remote dike area. The inspection did not identify any significant conditions or deficiencies in the operation and maintenance of the Project. All components are in proper working order.

6.0 Operation and Maintenance Programs Relative to Potential Failure Modes

6.1 Summary of PFMA Identified O & M Issues

The PFMA did not identify any significant issues or deficiencies in the operation and maintenance of the Jackson Bluff Hydroelectric Project.

6.2 Summary of Operation and Maintenance Procedures:

The Project is operated locally by the CITY. There is a full-time operator and maintenance person at the Jackson Bluff Hydropower Project and additional personnel visit the Project, as necessary. Operation and maintenance of the Project are documented with checklists and logbook entries.

Project surveillance is satisfactory. Checklists and logbooks are used for maintenance and a "dam safety" checklist has been developed. Surveillance of public safety features is satisfactory. The CITY is responsible for monitoring all instrumentation.

6.2.1 Communication/Response

The CITY is responsible for maintenance of the Jackson Bluff Dam. The CITY adequately maintains the water retaining components of the hydropower project.

6.2.2 Electrical/Mechanical Systems

The CITY is responsible for maintenance of the electrical and mechanical systems related to the Jackson Bluff Dam structures. The CITY adequately maintains the electrical and mechanical systems related to the hydropower project.

6.2.3 Human Factors

The facilities are well maintained, including public safety features such as fencing, gates, and signs. The Powerhouse is well maintained. The CITY staff are well trained in the operation and maintenance of the Project. The CITY staff walk the entire Project site on a daily basis to identify any potential issues. The CITY staff is commended for their diligence and dedication. Allowing the Operators and Maintenance personnel to participate in the PFMA review was very beneficial. The CITY staff stated following the PFMA Review that they understand more fully the issues to observe and identify as potential failure modes.

6.3 Assessment of O&M Procedures Relative to Potential Failure Modes

Project surveillance is generally satisfactory. Checklists and logbooks are used for maintenance and "dam safety". Surveillance of public safety features is satisfactory. CITY has the responsibility for monitoring the external movement of monuments and conducting various surveys. CITY personnel are knowledgeable of the potential failure modes and adequately monitor the conditions of the Project in relation to the identified potential failure modes.

6.4 Conclusions

Overall, the Project operation and maintenance is adequate and satisfactory. The current surveillance and monitoring program is adequate to identify the early warning signs of the potential failure modes as determined by the PFMA Report.

The CITY is responsible for operation and maintenance of the Jackson Bluff Dam. The Project's Powerhouse, Gated Spillway, and Auxiliary Spillway abutment connections are a part of the dam's overflow system and are adequately maintained by CITY.

7.0 Assessment of Supporting Technical Information Document

7.1 Potential Failure Modes Analysis Study Report

The Initial PFMA Report was prepared in conjunction with the Third Part 12D Inspection and reviewed for the Fourth and Fifth Part 12D Inspection Report. The PFMA report and review is discussed in detail in Section 3.0 of this Report.

7.2 Project Description and Drawings

The Project features generally consist of the powerhouse, gated spillway, auxiliary spillway and embankment structures related to the Jackson Bluff Hydropower Project from the south abutment to the north end of the embankment. Lake Talquin created by Jackson Bluff Dam is used primarily for public recreation.

The summary of the Project description in the STID is adequate.

Project Drawings are presented in *Appendix C*.

7.3 Construction History

The summary of construction history and operation for the Project is based on all available data sources including, but not limited to, the following: Project Documents, Geotechnical Investigation Reports, Design Drawings and Specifications, Construction Reports, Operation and Maintenance Reports, and DSSMR and EAP Reports, as well as some FERC file records. The construction outlined in the STI document is complete. Some details of the history of the project are lacking because the City of Tallahassee was not the original developer and some documents were destroyed in a fire. The Construction History in the STI Document presents minor information the original construction of the Project, but describes further all upgrades performed or installed to date. There has been the completion of the Auxiliary Spillway as a major construction feature performed since the previous Part 12D Inspection Report in 2009.

7.4 Standard Operating Procedures

The Project is a run-of-river, non-Federal, FERC Licensed Project. The Project is operated as a manned plant, and does have a full time (8 hours per weekday) employee on-site.

Additional personnel visit the Project, as necessary. Operation and maintenance of the Project are documented with checklists and logbook entries.

During normal operation, the Project will generate with one, two or three units up to 24 hours per day when sufficient river flow and power head are available.

The summary of the standard operation procedures in the STID is adequate.

7.5 Assessment of Geology and Seismicity

The STI Document presents a thorough description of regional, local, and site geology. The evaluation of the Seismicity in the STID states that the states of Florida, Georgia, and Alabama are not considered to be active earthquake or seismic areas.

The summary of the geology and seismicity in the STID is adequate.

7.6 Assessment of Hydrology and Hydraulics

7.6.1 Floods of Record

The flood of record at the Project occurred on September 25, 1969, with a recorded flow of 89,400 cubic feet per second (cfs).

7.6.2 Inflow Design Flood

7.6.2.1 Determination of Probable Maximum Flood

The Project's IDF has been determined to be equal to approximately one-half of the PMF. The PMF has a calculated inflow of 309,000 cfs. The level of the impoundment is projected to reach Elev. 77.0, with the tailwater at Elev. 57.3 during the IDF.

7.6.2.2 Freeboard Adequacy

The Jackson Bluff Hydropower Project has a zero freeboard capacity of 85,315 cfs.

7.6.2.3 Peak Spillway Discharge during last 5-Year Period

The peak flow from the Jackson Bluff Project since the previous Consultant's Safety Inspection Report, according to USGS gage records occurred on April 4, 2009 and had a flow of approximately 43,900 cfs.

7.6.2.4 Peak Reservoir Elevation during last 5-Year Period

The peak level of the impoundment occurred on April 3, 2009 at Elev. 69.75 with a tailwater at Elev. 24.26.

7.6.3 Hydrology Summary

The total drainage area above the Jackson Bluff Dam is 1,720 square miles. The PMF inflow hydrograph was originally developed by the U.S. Army Corps of Engineers (USACE) in its 1978 Inspection Report as a part of the National Dam Safety Program. The USACE estimated the peak PMF inflow to the reservoir to be 309,000 cfs. The USACE prepared the probable maximum precipitation (PMP) rainfall sequence following the procedure outline in Hydrometeorological Report No. 51 (National Weather Service, June 1978). The unit hydrograph and loss rate optimization routines from the HEC-I computer program (USACE January 1973) were used by the USACE to obtain the basin unit hydrograph. The flood of 1948, which had a peak discharge of 55,900 cfs at the

stream gage above Lake Talquin (drainage area 1,140 square miles), was selected by the USACE as the runoff event from which the unit hydrograph was derived.

The summary of the hydrology in the STID is adequate.

7.7 Assessment of Instrumentation

Instrumentation at the Project currently being monitored includes twenty-two (22) wells thirteen (13) piezometers; and sixteen (16) survey monuments. Locations of the surface monuments and alignment targets are presented in *Appendix D.*

The Independent Consultant performed a detailed review of the monitoring data as presented in the 2013 DSSMR for this Part 12D Inspection for the period of 2011 to 2013. Well data and piezometer readings are now performed on an annual basis. Alignment and settlement surveys are now performed annually. The CITY needs to continue monitoring and recording and evaluating data so that there are trend graphs available for the next Part 12D Inspection.

The DSSMR Reports need to be added to the STID.

7.8 Assessment of Stability and Stress Analyses of Project Structures

The summary of the stability and stress analyses of Project structures included in the STID is adequate. Free body diagrams for each loading case are included for each water-retaining structure. The final analysis of design of the Auxiliary Spillway Project also meets or exceeds the FERC guidelines. It is recommended that an overturning analysis for the Powerhouse be performed and included in the STID once it has been approved by FERC.

7.8.1 Method of Analysis

Structural stability studies for the Jackson Bluff Hydropower Project were performed in the initial design for three features of the facility: 1) the Spillway and Powerhouse, 2) the Embankment Dam, and 3) the Radial (Tainter) Gates. In 2010 the Auxiliary Spillway was designed and has also had structural stability analyses performed.

Stability analyses of the dam were performed using the gravity method of analysis. Stress analyses were performed in previous reports using the finite element and working stress methods. The stability analyses of record are summarized in the Initial Part 12D report. Finite element analyses for the dam in its upgraded condition have not been performed.

- Gravity Method of Analysis

  The gravity method of analysis includes calculations for sliding safety factor, determination of maximum and minimum stresses along the postulated slide plane, and location of the resultant of all forces. Calculated factors of safety and foundation stresses were then compared to allowable values specified by the FERC guidelines (FERC 1991).

The results were presented in the Initial Part 12D Report. The basic data and analyses used in the original design are summarized in the STI Document and briefly described herein.

7.8.1.1 Spillway and Powerhouse

The structure has a relatively broad base with a relatively low overall height and is founded into sandstone or limestone. A stability analysis was initially performed by Ebasco Services, Inc. from 1986 to 1992 for all loading conditions identified in the FERC Engineering Guidelines. The Spillway was the critical component and was the subject of the detailed analyses.

From the finite element analysis of the dam. it was found that all components of the dam as it was designed, including the slabs, beams, and buttresses, are capable of carrying the applied loads for the Usual Loading Condition. The Usual Loading Condition was found to be critical for the crest slab, upstream slabs and beams, since the PMF loads produce a high tailwater condition, reducing the net loading on the dam.

In response to FERC comments made during an August 24, 1994, meeting with representatives from Raytheon, the City of Tallahassee, and FERC, a September 30, 1994, report titled "Spillway Stability Analyses for Jackson Bluff Hydroelectric Project" was published by Raytheon Engineers & Constructors, Inc.

Included in this report were revised analyses including the following assumptions, which had resulted from discussions during the FERC meeting:

- The resistive force associated with the upstream shear key was eliminated.
- Sand foundation shear properties were substituted for the rock shear properties used previously.
- The increased soil weight in the spillway bays was accounted for in the stability calculations.
- Side friction was used to compute additional resistive forces.
- Uplift applied only to the base of the buttress walls and not to the soil infill.

Stability analyses were computed for various combinations of these assumptions, with the most realistic sliding scenario judged to be that which includes side friction and the soil load on the footings. Sliding resistance of the captive soil within the spillway bay was considered, but ultimately neglected in the final computations.

These computations demonstrated a factor of safety (FS) of 2.2 for an assumed failure plane at Elevation 15. Those calculations assumed (i) full uplift existed only at the base of the walls, and that (ii) no uplift exists at the base of the soil. Raytheon concluded that the first assumption is conservative and that the second assumption is reasonable based on the following:

At FERC's direction, the spillway structure is assumed to be sliding on sand. This assumption is considered conservative since all evidence indicates the spillway to be placed on rock. Also, it is logical to assume if the spillway is placed on sand, any resultant uplift would relieve itself into the spillway bays. There is no evidence of any

water being relieved into the spillway bays. The water level in these bays follows the tailrace level.

Raytheon published its report "Modification Studies. Volume I of II", for the Jackson Bluff Hydroelectric Project in September 1995. The report summarized modification studies and evaluations of alternative measures to achieve the recommendations made in previous reports, as cited above. The report recommended the following features to address structural deficiencies cited in previous reports:

1. Increase the Structure Weight: The weight of the structure would be increased by filling the spillway bays with sand up to Elevation 34.0 feet.

2. Strengthen the Crest Slab: The crest slab would be strengthened by the addition of a cast-in-place reinforced concrete beam and post on the underside of the crest slab, i.e. inside the spillway.

3. Strengthen the Downstream Wall: The downstream wall should be strengthened by constructing a new cast-in-place reinforced concrete wall on the inside of the spillway located just upstream of the existing wall.

4. Grout the Cracks: It was recommended that continued inspection of the spillway be performed in order to identify potential areas to be grouted.

7.8.1.2 Embankment Dam

For the embankment dam analyses R.W. Beck and Associates assumed two phreatic surfaces for steady state seepage at normal maximum pool. Pore pressure within saturated zones of the embankment was determined by use of a theoretical phreatic surface at normal maximum reservoir Elev. 68.5. This is judged to be a conservative approach since observed piezometer data are less severe than the theoretical calculation. The theoretical phreatic surface calculation assumed the 25-foot granular drain to be a pervious free-draining material. The ratio of the horizontal to vertical permeability of the embankment fill is assumed to be $k_1 / k_2 = 9$. The resulting phreatic surface for both typical sections is shown in Exhibits VIII-3 and VIII-4 in the Initial Part 12D Report.

Rapid drawdown of the reservoir was assumed to be instantaneous reservoir lowering from the normal maximum reservoir, Elev. 68.5, to the crest of the spillway, Elev. 52.0. Under this condition of instantaneous reservoir lowering, drawdown is assumed to occur so rapidly that drainage or reduction in pore pressures cannot occur. In general, the reservoir is not operated under these conditions making this an extreme loading case. Actual factors of safety for the upstream slope drawdown stability case would be expected to be somewhat higher.

Reservoir surcharge from extreme flooding is assumed to raise the reservoir from normal maximum reservoir Elev. 68.5 to 77.0. This loading case assumes the steady seepage conditions existing in the embankment prior to the flood are not significantly changed except in the upstream portion of the dam. This case represents a transient

seepage case in which steady state pore pressures in the downstream portion of the dam do not develop due to the short duration of flood storage.

In the supplement to the initial safety inspection report, R.W. Beck and Associates considered two cases with a higher phreatic surface. Both cases result from heavy precipitation and corresponding flooding. The first case (Case I) assumes the reservoir water surface level is maintained at normal maximum Elev. 68.5 and tailwater rises to Elev. 50. This case typifies an operating condition of releasing floodwaters through the spillway, thereby increasing downstream water surface levels. The embankment materials above the theoretical phreatic surface within the dam in this case are assumed to be in a near saturated state from rainwater infiltration. Pore pressures within the dam arc calculated from the theoretical phreatic surface. The second case (Case II) assumes spillway capacity is exceeded during approximately a 50% PMF event. In this case, the reservoir water surface level rises to the crest of the dam at Elev. 77 and tailwater rises to Elev. 60. For purposes of this conservative analysis, It was assumed the high-water levels allow seepage through the embankment dam to reach steady state condition.

7.8.1.3 Radial Tainter Gates

The structural adequacy of the radial gates at the Jackson Bluff Hydroelectric Project were evaluated in accordance with the FERC's request included in their February 11, 1998, letter to the City of Tallahassee. In that letter, the FERC requested that the 1999 Part 12 Inspection report include: "An evaluation of the forces and moments due to trunnion friction that can be sustained with a temporary overstress allowance of 1.33 on the steel structure. In the absence of a measured value, a friction coefficient of 0.3 shall be assumed with the gate under full hydrostatic pressure. If calculations indicate that any gates cannot satisfy the loading, recommendations for necessary remedial measures should be included in the Part 12-D Inspection Report."

Raytheon conducted an independent analysis using a three dimensional Finite Element Method (FEM) model of the gate, using the NISA II finite element computer software published by the Engineering Mechanics Research Corporation. The finite element model used for the analyses is shown in Exhibit VIII-3 of the referenced report. The calculations supporting these analyses are included in Appendix E the Initial Part 12D Report.

The main members in the radial arms were double angles built up from L6x4x3/8 back to back members with 3/8-in. thick spacer plates between the angles, with the long leg aligned with the depth of the members (vertically). The secondary members are 3x3x1/4, 3x3x3x5/16, and 4x3x5/16 double angle members. Bracing members include some 3x3x5/16 and 4x3x5/16. The double angle sections were assumed to resist tension and compression, while the single angles were assumed to resist tension only.

All compression members in the gate were checked against the code requirements in the AISC Ninth Edition for combined axial loads plus biaxial bending. The maximum allowable interaction value was specified in the FERC letter quoted above as 1.33 for the critical load condition.

Compression members are governed by AISC Sections H1-1, H1-2, and H1 -3, which govern combined stresses due to biaxial bending plus axial load, expressed as allowable stresses in terms of interaction ratios (summation of the ratios of the actual to allowable stresses for axial load and each orthogonal monoaxial bending moment). As stated previously the FERC allows a temporary overstress of one-third for the critical load case.

7.8.1.4 Auxiliary Spillway

The 825-foot long auxiliary spillway is a fixed-crest ogee spillway. The top of the ogee is at Elev. 69.5, with a toe radius of 10 feet that transitions into concrete spillway slab and into chute slab. A Type III hydraulic-jump stilling basin 33 feet in length dissipates flow velocities at the toe of the spillway chute. The auxiliary spillway contains both upstream and downstream sheet piling cutoff walls. The auxiliary spillway contains a graded filter beneath the stilling basin and chute slabs with an 18-inch drain where the chute slab meets the stilling basin and a 12-inch drain that transverses the downstream edge of the outlet sill to relieve uplift pressures. Training walls consist of PZ-13 steel sheet pile with the top of piling capped with MC18x42.7 channel. The proposed auxiliary spillway was designed to pass a flow of 69,900 cubic feet per second (cfs) when the reservoir level is at its maximum stage of 77.0 feet. The project also included a remote dike across the existing fuse plug and emergency spillway areas, which would prevent those structures from serving as functional spillways.

During the design of the auxiliary spillway, soil borings were taken within the footprint of the structure. Embankment soils generally consisted of very loose to loose silty to clayey sand, with occasional shell fragments. Foundation soils consisted of poorly graded sand with layers of silty to clayey sand. A weathered limestone was encountered between Elev. 17 to Elev. 25 and extended to the boring termination depths.

Walls – For the sheet pile training walls, only the extreme load cases were evaluated. In the case of the upper walls subject to backfilling, the loading condition was that of spillway drawdown with the backfill saturated to the top of the wall, immediately post-IDF. In the case of the lower walls not subject to backfill, the IDF flow level was the controlling load condition. Water to the top of the wall was not investigated due to the geometry of the channel.

The sheet-piling design was based on the Rankine model of lateral earth pressure using the free-earth support model for soil-structure interaction, with a requested factor of safety reduction in available passive soil resistance. Some walls are designed as cantilever I-type walls without anchors above the dredge line. Others use the concrete spillway for support at the dredge line. The safety factor for embedment depth used in all cases was 1.5. In some cases, the piling may be deeper than that required structurally, in order to effectively cut off potential seepage paths under the spillway and limit uplift forces.

Stilling basin – The stilling basin was evaluated for sliding and uplift stability using full hydrostatic uplift under flooded conditions. The structural design was based on ACI 207.2-95, Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete. Due to hydraulic jump, actual headwater elevation is lower than the tailwater elevation, but for the purpose of analysis, the higher water surface (whether phreatic or free) was considered to be the headwater in all cases. The reaction of the top of the

downstream cutoff sheet piling was included in the horizontal forces on the stilling basin under the scoured condition.

Spillway slab – The spillway slab was evaluated for sliding and uplift stability using 100-percent hydrostatic uplift under seepage flow conditions. Drainage will be provided under the slab to mitigate the possibility of accumulated uplift pressures remaining under it. The drainage invert is at elevation 49.0 feet, and the drain and filter system is designed with a maximum two-foot headloss. Therefore, the upper limit of seepage head is at elevation 51.0 feet. A seepage head elevation of 53.5 feet was used as the design headwater elevation for the slab uplift analysis to account for loss of effectiveness of the drains.

Uplift forces – A seepage analysis was conducted for the purpose of determining the uplift forces that would be present under the spillway slab and stilling basin under flood and drawdown conditions. The uplift under the stilling basin under the IDF and post-IDF conditions was reduced from the tailwater elevation of 57.3 feet based on the presence of the sheet pile cutoff that completely surrounds the paved spillway, and the design of the underdrain system beneath the basin and the lower spillway slab.

7.8.2 Properties of Materials Based on Site Specific Data

The foundation material for the spillway and powerhouse is predominantly sandstone. The subsurface studies have also indicated the presence of discontinuous sand seams which can be conservatively interpreted as existing in continuous seams beneath the structures. Laboratory analyses have previously been performed by others on sandstone samples obtained from borings beneath the structures.

The table the material properties used for analyses conducted since the previous Part 12 Inspection report is summarized in the Initial Part 12D Report.

7.8.3 Uplift Assumptions (Concrete Structures)

7.8.3.1 Gated Spillway and Powerhouse

Ambursen dams rely upon an upstream cutoff for control of seepage and, due to the relatively narrow width of the buttress bases relative to the entire width of the dam uplift is usually neglected. However, during meetings between the City of Tallahassee, the FERC, and Raytheon in August/September 1994, the following uplift assumptions were agreed upon:

1. Full uplift exists at the base of the buttress walls.
2. No uplift exists at the base of the soil infill.

Raytheon concluded that the first assumption is conservative and that the second assumption is reasonable. The basis for this opinion is that all evidence shows the spillway to be placed on rock. If the spillway were founded upon sand, any uplift acting upon the buttresses would relieve itself into the spillway bays. There is no evidence of any water being relieved into the spillway bays. The water level in these bays varies with tailwater elevation.

7.8.4 Phreatic Surface Assumptions (Embankment)

For the embankment dam analyses R.W. Beck and Associates assumed two phreatic surfaces for steady state seepage at normal maximum pool. Pore pressure within saturated zones of the embankment was determined by use of a theoretical phreatic surface at normal maximum reservoir Elev. 68.5. This is judged to be a conservative approach since observed piezometer data are less severe than the theoretical calculation. The theoretical phreatic surface calculation assumed the 25-foot granular drain to be a pervious free-draining material. The ratio of the horizontal to vertical permeability of the embankment fill is assumed to be $k_1/k_2 = 9$. The resulting phreatic surface for both typical sections is shown in Exhibits VIII-3 and VIII-4 in the1999 Part 12D Report.

Rapid drawdown of the reservoir was assumed to be instantaneous reservoir lowering from the normal maximum reservoir, Elev. 68.5, to the crest of the spillway, Elev. 52.0. Under this condition of instantaneous reservoir lowering, drawdown is assumed to occur so rapidly that drainage or reduction in pore pressures cannot occur. In general, the reservoir is not operated under these conditions making this an extreme loading case. Actual factors of safety for the upstream slope drawdown stability case would be expected to be somewhat higher.

Reservoir surcharge from extreme flooding is assumed to raise the reservoir from normal maximum reservoir Elev. 68.5 to 77.0. This loading case assumes the steady seepage conditions existing in the embankment prior to the flood are not significantly changed except in the upstream portion of the dam. This case represents a transient seepage case in which steady state pore pressures in the downstream portion of the dam do not develop due to the short duration of flood storage.

In the supplement to the initial safety inspection report, R.W. Beck and Associates considered two cases with a higher phreatic surface. Both cases result from heavy precipitation and corresponding flooding. The first case (Case I) assumes the reservoir water surface level is maintained at normal maximum Elev. 68.5 and tailwater rises to Elev. 50. This case typifies an operating condition of releasing floodwaters through the spillway, thereby increasing downstream water surface levels. The embankment materials above the theoretical phreatic surface within the dam in this case are assumed to be in a near saturated state from rainwater infiltration. Pore pressures within the dam are calculated from the theoretical phreatic surface. The second case (Case II assumes spillway capacity is exceeded during approximately a 50% PMF event In this case, the reservoir water surface level rises to the crest of the dam at Elev. 77 and tailwater rises to Elev. 60. For purposes of this conservative analysis, it was assumed the high-water levels allow seepage through the embankment dam to reach steady state condition.

7.8.5 Factors of Safety

Since the maximum driving force on the dam occurs for the Usual Operating Condition, stability analyses for only that load case were performed initially. In addition, a factor of safety of 2.0 was applied to the analyses of record, based upon Section 3-5.5 of the FERC guidelines, which stares "... *it is acceptable to apply the factor of safety criteria for the unusual loading combination ... to the usual loading*

combination if the normal headwater and tailwater elevations result in the maximum net static forces tending to indicate overturning of the structure."

Analysis of reinforced concrete members was according to ACI 318, Appendix A - Alternate Design Method (Working Stress Method). Analysis of plain concrete members was according to ACI 318, Chapter 22, which governs the analysis and design of structural plain concrete.

All structures were determined by the original designers and subsequent reviewers to have acceptable factors of safety. The results of the analyses and tables listing the factors of safety are presented in the STI Document and in Appendix D.

7.8.6 Stress Analysis and Evaluation

As slated in the previous section, working stress methods were used for proportioning of new members and acceptance criteria for existing members. Through specification of the ACI code for analysis and design, stresses in all new and existing and new members are less than those allowed by the code. A tabulation of calculated stresses in every structural member, both new and existing is provided in the September 1995 Raytheon report "Modification Studies - Volume I of II - Jackson Bluff Hydroelectric Project" for calculations supporting the rehabilitation design features.

7.8.7 Loading Diagrams and Summary of Results

Loading diagrams and a summary of results for the Spillway and Powerhouse sections, are presented as Exhibits VIII-1 and VIII-2 in the 1999 Part 12D Report in the STI Document. Loading conditions and results of the embankment dam stability analyses are shown on Exhibits VIII-3 through VIII-6 in the 1999 Part 12D Report in the STI Document.

7.8.8 Seismic Stability (Earthfill Structures and Soil Foundation)

For the embankment dam, R .W. Beck and Associates used a low-risk seismic Zone I as defined by the U.S. Corps of Engineers and others at the time of the original design. Due to the low potential for seismic events in this area, simplified methods of dynamic stability analysis are generally appropriate. When evaluating seismic stability of a dam, it must first be determined whether soils at the site are susceptible to liquefaction. The low density sandy materials in the dam and foundation are the type of material usually suspected of having potential for liquefaction. The empirical method of analysis given in Reference 24 and blow-count data from Reference 9 were used for an estimate of liquefaction potential.

If liquefaction is not a significant problem, then pseudo static methods of analysis were used to reasonably predict dynamic stability at the time of original design. Two methods were used to evaluate this case. The first method followed procedures and criteria established by the U.S. Army Corps of Engineers in which factors of safety were computed for the slopes and evaluated with standards established by the Corps. In the second method, the potential for permanent displacement was evaluated . The permanent displacement procedure is based on the concept that earthquake acceleration varies with time. For periods when the ground acceleration exceeds the yield acceleration of the slope, factor of safety =

1.0, the slope will incur a displacement. For this study, the yield acceleration of the downstream slope was calculated and compared to an estimated peak acceleration.
The current USGS Peak Ground Acceleration Maps are included in *Appendix D*. These include the Peak Acceleration with 2% Probability of Exceedance in 50 Years.

7.8.9 Seismic Stability (Concrete Structures)

Seismic stability is not an issue at the Jackson Bluff Hydropower Project Development, which is located in Seismic Zone 0 on the FERC seismic zone map. The current USGS Seismic Peak Acceleration with 2% Probability of Exceedance in 50 years is 4. The 2014 USGS Seismic Hazard Map shows the Project to be in the lowest hazard zone in the United States.

7.8.10 Summary of Results

The results for the Project structures indicate that they all meet the required factors of safety for stability and stress analyses, and are acceptable based on previous performance. Summary of results tables are presented in the 1999 Part 12D Report and the table prepared by the Independent Consultant.

7.9 Assessment of Spillway Gates

The detailed "close-up" inspection of the Spillway Tainter Gates was last performed as part of the 2004 Part 12D Inspection. There were significant comments provided at that time. The CITY implemented a remediation program using FruCon in 2005. A detailed report of the remediation was unable to be located by the Independent Consultant. The FERC is requiring a detailed "close-up" inspection to be performed and a report submitted to FERC by December 31, 2015. The Independent Consultant will review that report and provide a supplemental letter following review. The Independent Consultant did perform a general review of the Spillway Tainter Gates and observe operation of two of the gates - one using normal power and one using emergency power during the field inspection. All gates were observed to be in similar condition for operation and structural integrity.

7.10 Pertinent Correspondence Related to Safety of Project Works

All pertinent correspondence provided by CITY are presented in the STI Document with the exception of the latest Dam Embankment Seepage project approval.

7.11 Status of Studies in Process and Outstanding Issues

The only studies in progress at the Project relate to the 1) final design of the Embankment Seepage Mitigation Project of 2015, and 2) the recommended analysis of the Powerhouse overturning.

7.12 References

The References listed in the STI Document provide adequate documentation to assess the structures, stability, spillway, and Project operation and maintenance.

7.13 Conclusions

The information and analyses contained in the STI Document are current with the exception of the addition of Instrumentation and Monitoring data, Auxiliary Spillway final design, and structural stability data. As stated, when these analyses are complete and accepted by FERC, they need to be added to the STI Document.

8.0 Recommendations

8.1 Project Description

The Project is consists of the Powerhouse, Gated Spillway, Auxiliary Spillway and Embankment to create an integral structure at the Project site.

8.2 Potential Failure Mode Analysis Report

The PFMA Review identified two (2) additional potential failure modes.

PFM 3.55 – Barge/Boat impact on Gated Spillway or Powerhouse. This would require the barge or boat to impact the gated spillway or powerhouse. This scenario was deemed to be very unlikely since there are no barges or large boat on the reservoir. This was classified as a Category IV by the PFMA review team.

PFM 3.56 – Lightning Strikes on Powerhouse - This would involve a lighting strike to the powerhouse that would cause an explosion resulting in a failure of all controls to be able to operate the spillway gates or wicket gates, resulting in a "controlled" release (maximum capacity of water passage or spillway gate). This is a damage issue, not a resulting uncontrolled release of water. This was classified as a Category IV by the PFMA review team.

ARO staff believes all instrumentation monitoring is not required to be associated with Category II Failure Modes. There were no sole Category I - Highlighted Potential Failure Modes identified during the PFMA. There were three (3) Category I/III Highlighted Potential Failure Modes that are currently being addressed with the proposed embankment slope remediation that is being reviewed for approval by FERC. There are two (2) Category III – Considered, but not Highlighted Potential Failure Modes The first relating to the "close-up" inspection of the Tainter Gates for which a detailed structural inspection is being schedule and for which the report is due to FERC on December 31, 2015. The second relating to the lack of analysis for overturning of the powerhouse which is recommended to be performed and filed for approval with FERC.

During the PFMA Review there was discussion concerning the re-classification of certain Category IV Failure Modes to Category II due to the ongoing monitoring of the Project instrumentation that was initiated during the design and construction phases of the Project. The scope and schedule of instrumentation and their reading schedules are appropriate for this Project with the recommended addition of 5-year diving inspections performed prior to each future Part 12D Inspection.

There were a number of Category II and Category III Failure Modes that were changed to Category IV either due to the construction of the Auxiliary Spillway, or due to additional information that was located during the PFM Review, or due to the highly unlikely failure event, but still being monitored by the CITY. Even though the Category IV ratings do not require monitoring of the installed instrumentation, CITY intends to continue monitoring instrumentation on various project features as a part of its prudent operation and maintenance of the Project. The Independent Consultant concurs with this decision and recommends instrumentation monitoring continues in accordance with

the original Project Instrumentation Manual. ARO staff concurs during the PFMA Review that all instrumentation monitoring does not necessitate a Category II status.

Similarly, there were some Category IV Failure Modes that were recommended to be changed to Category II because of the ongoing monitoring and presentation of data results in the DSSMP.

Finally there were Category IV Failure Modes identified in the Initial PFMA and subsequent PFMA reviews that are recommended to retain the Category IV status.

The powerhouse stability analysis for overturning either needs to be recovered from archives or a new powerhouse stability analysis for overturning needs to be performed and submitted to FERC.

8.3 Performance Monitoring and Visual Assessment with Respect to Potential Failure Modes

The current monitoring instruments and surface movement monuments are adequate and one additional survey monument is recommended at this time if it is not located on the left tailrace wall.

The following recommendation is made with respect to monitoring:

- The Surveillance and Monitoring Plan needs to continue in the current manner with annual survey monuments, well monitoring and piezometers read on an annual basis and plots derived for each set of periodic to develop trends for the next Part 12D Inspection in 2020.

- The periodic results of instrumentation need to be added to the STID on an annual basis so they are readily available for review prior to the next PFMA review.

- The left bank tailrace channel wall survey monument that has not been located either needs to be found and surveys updated or a new monument needs to be established for base line data as soon as possible to have a trend of readings prior to the next Part 12D Inspection.

- A graphic is recommended to be created to collectively assemble all relative data relating to sheetpile walls, both in plan view and, to the best of one's ability, in elevation for the Wakefield piling, the Auxiliary Spillway and the replacement sheetpiling installed following the dam failure in 1957. This graphic would begin at the north end of the gated spillway and proceed to the north end of the dam embankment. The stationing and elevations for this graphic were discussed during the PFMA.

- The Spillway rating curve for the combination of the Gated Spillway and the Auxiliary Spillway needs to be updated to remove the Fuse Plug reference that is no longer applicable, and also be clearly identified as a separate document in the STID.

8.4 Field Inspection

It is recommended that the following items, requiring corrective action, be completed as follows:

The completion of the seepage remediation program on the embankment that is currently in final design and review approval by FERC.

8.5 Operations and Maintenance Programs Relative to Potential Failure Modes

This inspection did not identify any corrective measures required for the methods of Project works operation. The operation of the Project is satisfactory.

8.6 Assessment of Supporting Technical Information Document

The STI Document adequately describes the Project features, history, instrumentation, and operations and maintenance. The STI Document is complete and adequate for use in future Part 12D Inspections. There are several items that were discussed during the PFMA Review that are documented elsewhere in this report that are to be included in the STI Document. In addition, the CITY needs to contract with an outside engineering firm to review the Powerhouse overturning analysis to reflect current FERC engineering guidelines. After this review has been completed the overturning analysis needs to be added to the STI Document.

Appendices for Part 12D Inspection Report

Appendix A –FERC Letter Requiring Part 12D Inspection

Appendix B–FERC Letter Approving Part 12D Consultant

Appendix C – Project Figures

Appendix D – Instrumentation Monitoring Data Plots

Appendix E – Inspection Photographs

Appendix F – Inspection Checklist (Optional)

Appendix G – Operation and Maintenance Documentation

Appendix H – Supplemental Stability Evaluation Documentation

APPENDIX A

FERC Letter Requiring Part 12D Inspection

FEDERAL ENERGY REGULATORY COMMISSION
Office of Energy Projects
Division of Dam Safety and Inspections - Atlanta Regional Office
Gwinnett Commerce Center
3700 Crestwood Parkway, Suite 950
Duluth, Georgia 30096
(678) 245-3000 Office - (678) 245-3010 Facsimile

In reply refer to:
P-2891
NATDAM No. FL00108
Jackson Bluff

Mr. Triveni Singh, P.E.
Manager – Power Production
Electric Production Division
City of Tallahassee, Electric Utility
2602 Jackson Bluff Road
Tallahassee, Florida 32304

Subject: Fifth Independent Consultant's Safety Inspection Report for the Jackson Bluff Project (P-2891) due by March 1, 2015, with Revised PFMA Requirements

Dear Mr. Singh:

Three copies of the subject Independent Consultant's Safety Inspection Report (Report) are to be submitted to this office by the subject due date. CFR 18, Part 12, Subpart D, of the Commission's regulations prescribes the scope of the Independent Consultant (IC) evaluations and field inspection, as well as the information that must be contained in the Report. Your responsibilities, as well as those for your IC, are discussed in more detail in Enclosure 1. I encourage you to take time and review those responsibilities. The Report outline to be used by the IC is also included as Enclosure 2.

Potential Failure Modes Analysis

The Commission's goal to incorporate Risk-Informed Decision Making into our dam safety program continues to evolve with important advances starting with Chapter 14, Dam Safety Performance Monitoring Program (DSPMP) of the FERC publication, Engineering Guidelines for the Evaluation of Hydropower Projects (Guidelines). An important feature of

the DSPMP is the Potential Failure Modes Analyses (PFMA). Section 1 of your Supporting Technical Information (STI) document should be a PFMA report completed during a previous submittal under the Part 12D process. Incorporating the potential failure mode process and development of the PFMA report have been significant enhancements to our dam safety program.

During a recent FERC-wide re-evaluation of all PFMs, we discovered that many of them still do not meet the expectations that we have for complete PFMs. Review and, if necessary, revision of the PFMs is required during each new Part 12D inspection to ensure they are fully described and each PFM is developed from initiation through to potential loss of the reservoir. Therefore, you and your staff should be available for a PFMA review to be scheduled in conjunction with the IC's field inspection. Your IC is required to: review the PFMA report for completeness and clarity prior to performing the PFMA review; evaluate the adequacy and completeness of the previously identified and described PFMs, category assignments and risk reduction measures; and recommend any revisions to these items or new PFMs, if needed. Any resulting supplements to the PFMA report require review by this office and should follow the guidance provided in Chapter 14.

The PFM review is a critically important function for the safety of the dam. Depending on the quality of the original PFMA, the review may require a smaller resource commitment than the original PFMA, or it may require more time and resources than the original PFMA session. Recently conducted reviews for relatively complex dams have lasted as long as two days, and could potentially last longer for large, complicated projects. Your IC should direct a group review of identified potential failure modes for completeness; lead a group evaluation of the current assigned categorizations and risk reduction measures; and facilitate the revision of existing PFMs, as required, and the full development of any new potential failure modes that may be identified during the review. Alternately, you may wish to retain a facilitator to complete the PFMA review and revision. You should be aware that it is likely many of the existing PFMs may require revision to more fully describe the actual mode of failure. We request that you coordinate your IC's field inspection and PFMA review with this office well in advance so that we may attend both. You are also requested to set up a telephone conference or a face-to-face meeting at least 90 days prior to the PFMA review to discuss our expectations for this review. We will conduct our annual dam safety inspection in conjunction with the IC's field inspection.

Our review of the original PFMA for the Jackson Bluff Dam found that many of the PFMs include multiple loading conditions and/or locations. Each unique loading condition and location constitutes a PFM that should be evaluated separately. PFM No. 27, instability of the downstream slope by seepage and/or rainfall, combined seepage and rainfall activity. This PFM should be broken out to consider each condition. PFM No. 28 combined seepage failures from windows in the Wakefield piling and pervious zones in the lime rock and

should be broken out to consider each scenario separately. PFM No. 32, also combined stability and seepage issues and should be broken out and evaluated separately. Additional. consideration should be given to the development of detailed step-by-step descriptions for all failure modes. For more information on how to complete well-developed PFMs, please refer to the D2SI website at:

http://www.ferc.gov/industries/hydropower/safety/initiatives/pfms.asp

Project Features

Commission Regulations require that the project works of a development subject to Part 12, Subpart D, of the Commission's Regulations be inspected and analyzed periodically by an IC. This includes all dams and all principal works of the development. If applicable, the IC's inspection should also include inspecting the spillway apron for undermining. The following dams and associated appurtenant water-retaining structures require inspection at your project:

Dam No.	Dam Name	Report No.	Due Date	NATDAM ID.
P-2891-01-01	Main Dam	Fifth	March 1, 2015	FL00108
P-2891-01-02	Remote Dike	Fifth	March 1, 2015	FL00108

The following inspection, which was outstanding from previous Part 12 Reports, will also need to be provided to this office for review together with this Report:

1. The Tainter gates are Category 1 gates and as such require a close-up detailed inspection every 10 years. The last detailed inspection was in 1999. Your IC should carefully inspect the gates including all structural members and trunnions, and specifically address the findings of this inspection in the fifth Part 12D report.

IC Approval

You must obtain approval of your proposed IC(s) prior to the initiation of the field inspection. You should send three copies of your letter requesting approval of the IC (together with the proposed IC's detailed résumé) to:

Mr. William Allerton, P.E., Director
Division of Dam Safety and Inspections (D2SI), OEP
Federal Energy Regulatory Commission
888 First Street, N.E., Room 6N-01
Washington, D.C. 20426

One copy of the letter and résumé should also be sent to this office. By regulation, the request for the approval of the IC and the résumé are to be filed at least 60 days prior to the initiation of the safety inspection. **In order to allow your IC adequate time to inspect your project and prepare the Report, we request that you submit the request letter and résumé at least six months before the Report is due; that is, by September 1, 2014.**

The First Report for new constructed projects or projects where a major dam safety remediation has recently been completed may be done by the design engineer or an engineer from the design engineer's firm. The next Report must be completed by a different engineer not associated with either the design or construction firm. Subsequent Reports may be completed by an engineer associated with the design, construction, or remediation work. However, an engineer or engineers from the same firm will not be approved as the IC for more than two consecutive Reports for any project.

Once the IC has been approved, it is your responsibility to provide the IC with copies of, or access to, all project files well in advance of the field inspection. You should include file review in your scope of work and strongly encourage the IC to adequately prepare for the field inspection by adequately reviewing all the pertinent background information for the project in advance. Inadequate preparation of your IC may result in the need to reschedule the inspection until they are properly prepared to perform a thorough inspection.

Report

The Report outline included in Enclosure 2 of this letter establishes the format to be followed by the IC. Stability and stress analyses of representative structural sections must be evaluated and commented on specifically with regards to current state of practice input parameters, assumptions, and analysis. It is critically important that the IC review, evaluate, and comment on the appropriateness and current validity of all the previous analyses located in Section 8 of the STI. Section 7 of the Report should contain your IC's detailed assessment of the STI including the PFMA report. Acceptable technical criteria are prescribed in FERC's Engineering Guidelines. If needed, this publication can be ordered from the Commission's Division of Public and Intergovernmental Affairs, at (202) 502-6088, or downloaded from our website at:

http://www.ferc.gov/industries/hydropower/safety/guidelines/eng-guide.asp

As discussed in Enclosure 1, the IC must provide a clear and comprehensive statement of concurrence or non-concurrence with the methodology, assumptions, and conclusions of previous reports and studies summarized in Section 8 of the STI. The statement of concurrence must be specific to each item and include a detailed explanation, not merely a repetitive general statement. Reasons for non-concurrence must be explained in detail and

may require a simple independent analysis by the IC to clarify the effects on factors of safety of the structures. **Please ensure that the Report meets the Commission Regulations and Guidelines including adequately developed PFMs prior to submittal to this office as incomplete Reports may be returned as deficient and not accepted for further review. It is recommended that you review Enclosure 1 to become familiar with the requirements contained therein and to assist you in providing an acceptable Report by the due date.**

Report Follow-Up

If the IC makes specific recommendations in the Report, **Section 12.39 of the Commission's regulations requires you to submit to us, within 60 days of the date the Report is filed, your plan of action and schedule to satisfy these recommendations.** It is also necessary to confirm your agreement with the IC's recommendations to continue any ongoing measures (ex. annual settlement survey) specifically identified in the Report. Your plan of action may include any proposal, including taking no action, that you consider a preferable alternative to any corrective measures recommended by the IC in the Report. However, any proposed alternative must be supported by complete justification and detailed analysis and evaluation in support of that alternative.

Unresponsive Reports Will Be Returned

We have noted several instances lately where an IC did not make "*a clear statement that they have reviewed the pertinent analyses and evaluations along with the underlying assumptions and that they have concluded that the assumptions and methods of analysis or evaluation were appropriate for the structure, were applied correctly and are appropriate given current guidelines and the state of dam safety practice*" as is required by the Commission's Guidelines. A general statement is not acceptable. The Report should indicate in each section that this review and concurrence has been completed. **Please ensure that the Report fulfills this requirement, as unresponsive Reports received by the D2SI will likely be returned for resubmittal.**

The Commission's dam safety program is a cooperative process that includes the licensee, the IC, and the FERC. The most important of the three elements is the licensee, as they operate the dam, see the dam on a regular basis, and are responsible for the surveillance and monitoring plan used to determine if a potential failure mode is developing. It is the licensee's responsibility to submit the IC's Report to the FERC and ensure that the Report meets the requirements of the Commission's Regulations and Guidelines before it is submitted. Although the Report is a FERC requirement, it is also a valuable resource for you as the dam owner. Enclosure 1 provides a more complete discussion of the requirements of the Commission's Regulations and Guidelines.

If you have any questions regarding this letter or Enclosures, please do not hesitate to call me at (678) 245-3075. Your support is critically important and I am available to discuss any concerns or comments that you may have.

Sincerely,

Wayne B. King, P.E.
Regional Engineer

Enclosures:
1. Licensee and IC Responsibilities
2. IC's Safety Inspection Report Outline

cc: D2SI
eLibrary

JFB/jfb:jls

922

C:\Users\jfbpj13\Desktop\02891 Jackson Bluff Part12D Post-PFMA Reminder Letter final.doc

City of Tallahassee
Your Own Utilities℠

2602 Jackson Bluff Road, Tallahassee, Florida 32304, (850) 891-4YOU (4968), talgov.com

TRANSMITTED BY E-MAIL
NO HARDCOPY SUBMITTED

February 27, 2015

Mr. Wayne King, Regional Engineer
Federal Energy Regulatory Commission
Office of Energy Projects
Division of Dam Safety and Inspections
Gwinnett Commerce Center
3700 Crestwood Pkwy, Suite 950
Duluth, GA 30096

Re: Part 12 Safety Inspection Report– Request for Extension on Submittal
Jackson Bluff Dam Project P-2891, NATDAM No. FL00108

Mr. King:

The City is requesting a ninety (90) day extension on the submission of the Part 12 Safety Inspection report that is currently due on or before March 1, 2015. William Smith, Hydropower International Services Inter-National Consultancy, L.L.C. (HISINC), is the Independent Consultant currently drafting the report. In completing the on-site work in August 2014, HISINC did not include a close-up detailed gate inspection. As a result, there has been communications between the City, HISINC and FERC regarding the gate inspection and the discussions have caused some delays in completing the Part 12 report.

As a result, the City of Tallahassee is proposing submission of the Part 12/PFMA report on or before May 30, 2015, and to follow-up with performing a close-up detailed gate inspection by December 31, 2015.

If you have any questions regarding this request, please do not hesitate to contact me at 891-5535.

Very Truly,

Triveni Singh
Manager, Power Production
Electric Production Division

cc: Dave Mordecai, COT, Karl Bauer, COT
Martin Kemps, Mead and Hunt

APPENDIX B

FERC Letter Approving Consultant

FEDERAL ENERGY REGULATORY COMMISSION
Office of Energy Projects
Division of Dam Safety and Inspections
888 First Street, NE Routing Code: PJ-13
Washington, DC 20426
(202) 502-6025 Office – (202) 219-2731 Facsimile

RECEIVED
CITY OF TALLAHASSEE
2014 MAY -7 P 1:0
ELECTRIC UTILITIES

MAY 05 2014

In reply refer to: P-2891-FL
NATDAM #FL00108

Mr. Triveni Singh
Manager, Power Production
Electric Production Division
City Of Tallahassee
2602 Jackson Bluff Road
Tallahassee, FL 32304

Dear Mr. Singh:

By letter dated March 19, 2014, you proposed Mr. William B. Smith, P.E., CFM, of HISINC, L.L.C., as the independent consultant to be responsible for the fifth Part 12D safety inspection of the Jackson Bluff Project. Mr. Smith's resume confirms that he meets the Commission's independent consultant qualifications specified in Section 12.31(a) of the regulations. Mr. Smith is therefore approved as the independent consultant for this inspection. In accordance with Part 12, Subpart D, the approved independent consultant must either personally inspect the project or be present during the inspection to supervise those individuals that conduct the inspection. You are also reminded to instruct your consultant that should any condition be discovered that requires emergency corrective measures, he must immediately notify you, since you are required to submit a report to the Regional Engineer in accordance with Section 12.36.

Three copies of the inspection report must be filed with the Division of Dam Safety and Inspections-Atlanta Regional Office by March 1, 2015. The consultant's report must be formatted in accordance with Enclosure 1, Part 12D Safety Inspection Report Outline (Outline), revised July 1, 2005. You should remind your consultant that it is important to submit reports that are formatted in accordance with the Outline and which adequately address all the information requested. The inspection and project features analyses are to include a thorough evaluation of all dam safety aspects of the project in accordance with the Commission's Part 12D Dam Safety Regulations

consistent with the requirements covered in the Commission's Engineering Guidelines for the Evaluation of Hydropower Projects (Engineering Guidelines).

As a part of addressing the information requested above, the independent consultant should specifically review the Potential Failure Modes Analysis (PFMA) for any appropriate changes in Potential Failure Mode (PFM) category classification. This includes reviewing all reports and investigations performed or ongoing since the previous Part 12D report and determining whether the information in these reports and investigations have changed a PFM category classification or introduced any additional PFMs. Recently we updated our website http://www.ferc.gov/industries/hydropower/safety/initiatives/pfms.asp providing information about fully developed PFMs. Your consultant should be instructed to review this site before beginning work on the Part 12D inspection and report. The independent consultant should also address those items specific to the project that were included in the Part 12D reminder letter sent to you by the Regional Office.

The first Part 12D Independent Consultant Inspections for new constructed projects or projects where a major dam safety remediation has recently been completed may be done by the design engineer or an engineer from the design engineer's firm. The next Part 12D inspection must be performed by a different engineer not associated with either the design or construction firm. Subsequent Part 12D inspections may be performed by an engineer associated with the design, construction, or remediation work. In addition, an engineer or engineers from the same firm will not be approved as the independent consultant for more than two consecutive Part 12D Safety Inspections of any project.

You as the licensee are responsible for preparing the Supporting Technical Information document (STI) or having it prepared for you. The STI replaces Appendix D of the Part 12D report and will be submitted with the Part 12D Report. Enclosure 2 contains the outline for the STI. Appendix I of Chapter 14, Dam Safety Performance Monitoring Program, of the Engineering Guidelines includes the STI outline, guidance on what is to be included in the STI, and examples of the document. Much of the information required for the STI was previously asked for in Appendix D of the Part 12D report. A Part 12D report submitted that has an incomplete, significantly deficient, or unreadable STI will be returned to you as an unresponsive report.

A new feature of the Commission's Dam Safety Program is the Dam Safety Performance Monitoring Program (DSPMP). A complete description of this program may be found in Chapter 14 of the Engineering Guidelines. An important part of the DSPMP is the PFMA. The PFMA formally identifies PFMs of structures at the project. An item of utmost importance in conducting the PFMA is the need for a person to fulfill the responsibilities of a facilitator. The facilitator will oversee and direct the PFMA process and PFMA report preparation by the independent consultant.

The PFMA facilitator should be a civil engineer with a broad background and experience in dam safety engineering and experience in performing a PFMA similar to that described in the guidance in Chapter 14 of the Engineering Guidelines. A basic recommended qualification for the facilitator is that the proposed facilitator for a project should have participated in an actual PFMA of the nature described in these guidelines.

Qualifying experience for a facilitator is participation as a core team member of a PFMA or actually facilitating a PFMA. This ensures that the person leading the PFMA process knows not only how the process is carried out, but is also aware of what can be accomplished. This is especially critical if the other core team members have not been through a PFMA, which may often be the case. As an alternative to actual experience participating in or facilitating a PFMA, the proposed facilitator should have attended a FERC sponsored Dam Safety Performance Monitoring Training Workshop.

It is important to understand that if the PFMA facilitator does not accomplish the goals of the PFMA, which is identifying and obtaining a clear understanding of each dam site's specific PFMs, we may require that the PFMA be supplemented or redone entirely.

If a PFMA has not been conducted for the project prior to the preparation of the Part 12D report, the sections of the Outline that address the PFMA are to include a brief statement that the PFMA has not yet been conducted for the project. If you have any questions concerning the requirements of the Outline, please address your questions to Mr. David Snyder at telephone number 202-502-8968. Your consultant should be prepared to submit, if requested, CDs containing programs with documentation and input files for any of the computer analyses used to reach the conclusions in his report.

Supplemental PFMAs for Subsequent Part 12, Subpart D Reports

The comprehensive PFMA report and the resulting section in the STI appended to the Part 12D report described above are to be regarded as living documents. These documents are to be appended as conditions at the site change and as new information is obtained at any time following the initial comprehensive PFMA or when discovered during subsequent Part 12, Subpart D inspections.

Therefore, for the Jackson Bluff Project Part 12D Inspections and Reports subsequent to the third inspection and report, a formal PFMA will not be required. Instead, as part of the preparation for the inspection, your personnel and your Part 12D consultant should review the existing PFMA, STI and all other project documents relating to the operation and dam safety of the dam since the last Part 12D inspection. During the inspection, your consultant and you should organize and conduct an informal supplemental PFMA session with your representatives and our FERC inspector to examine any revisions or additional failure modes identified during the review.

Please inform the Regional office when the Independent Consultant will inspect the Jackson Bluff Project so they may attend. They will also conduct their Dam Safety inspection of the Jackson Bluff Project in conjunction with the Independent Consultant's inspection.

The supplemental PFMA should be conducted consistent with Chapter 14 of the FERC's Engineering Guidelines. It is important to retain the original PFMA report as prepared so that the findings, discussions and thought processes of the original PFMA session are retained for future evaluations.

You are reminded that not later than 60 days after the report of the independent consultant is filed with the Regional Engineer, you must submit to the Regional Engineer three copies of a plan and schedule for designing and carrying out any proposed corrective measures.

Sincerely,

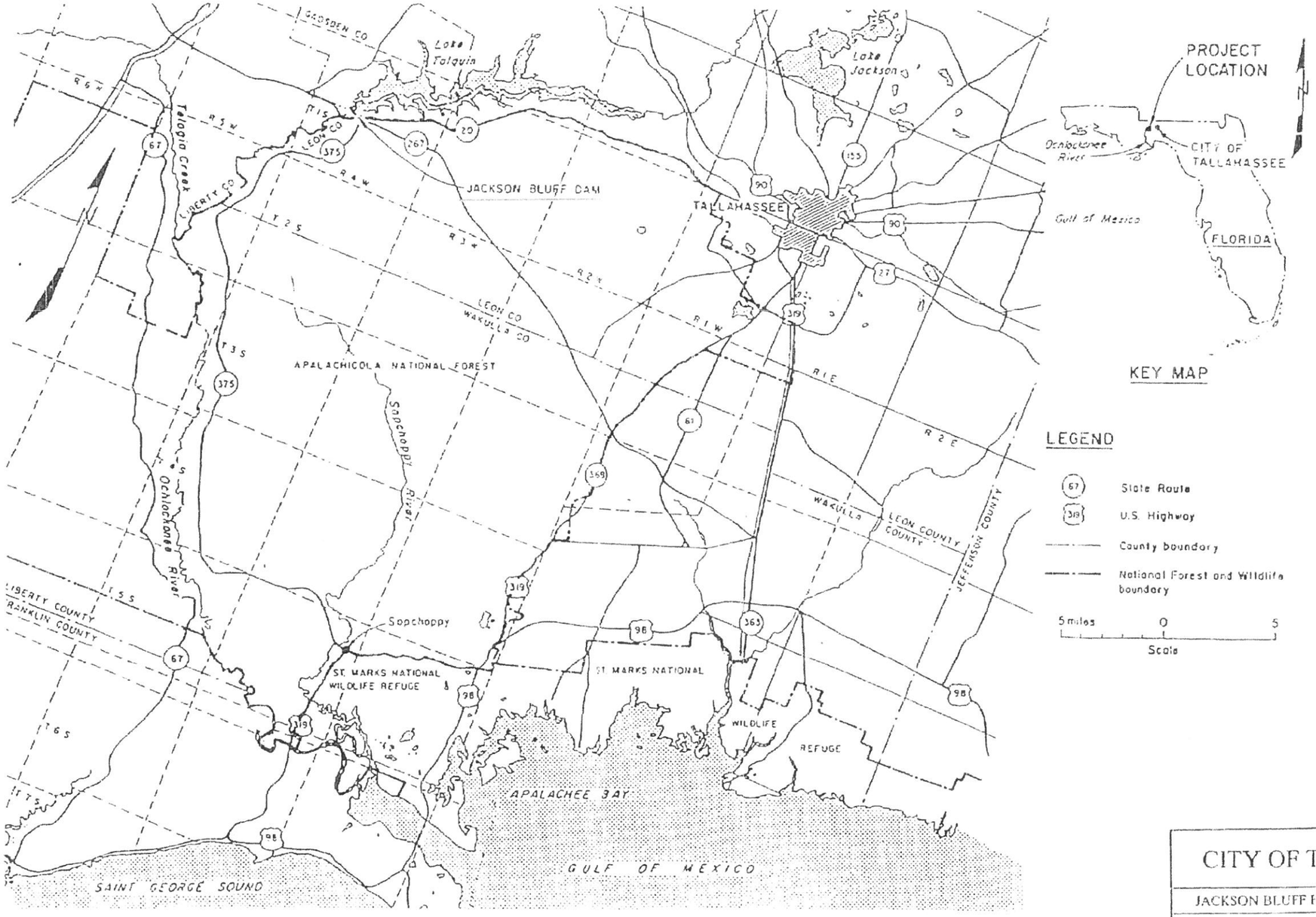
David S. Snyder, P.E.
Senior Civil Engineer
Division of Dam Safety and Inspections

Enclosure

cc: Public Files

APPENDIX C

Project Figures



KEY MAP

LEGEND

- (67) State Route
- (319) U.S. Highway
- County boundary
- National Forest and Wildlife boundary

5 miles 0 5

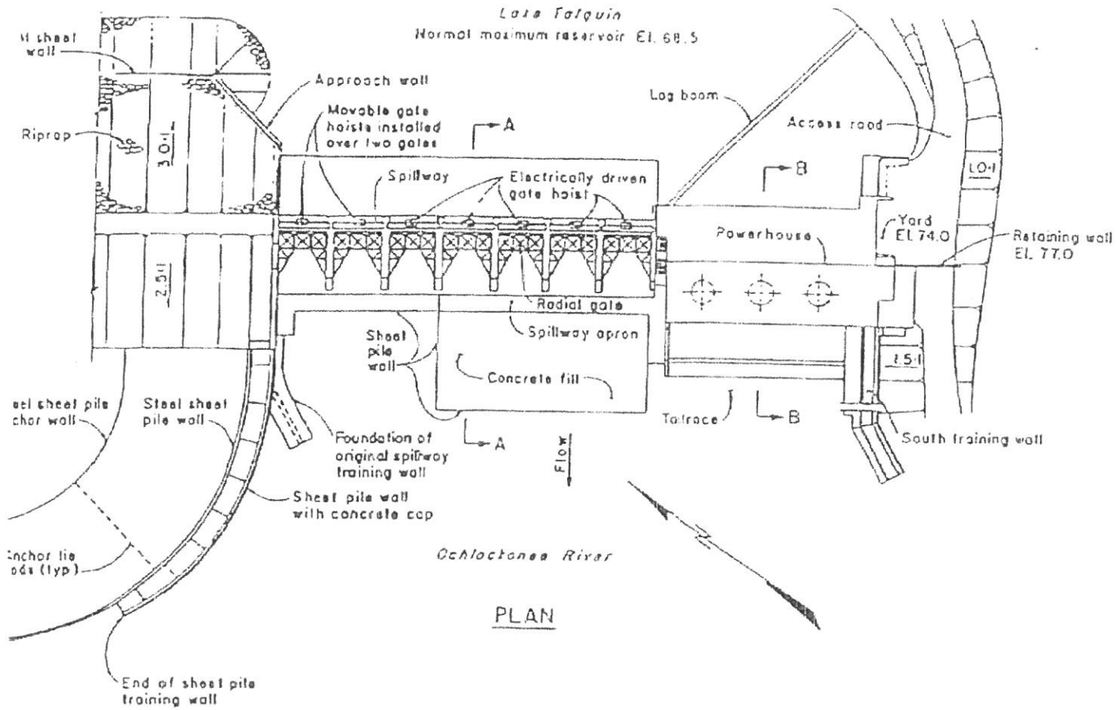
Scale

CITY OF TALLAHASSEE

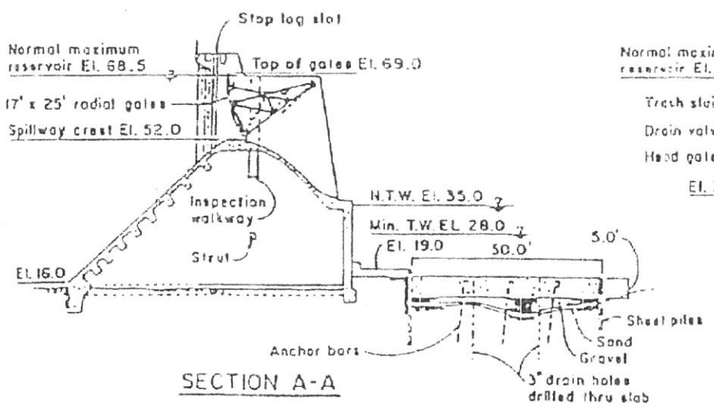
JACKSON BLUFF HYDROELECTRIC PROJECT

VICINITY MAP

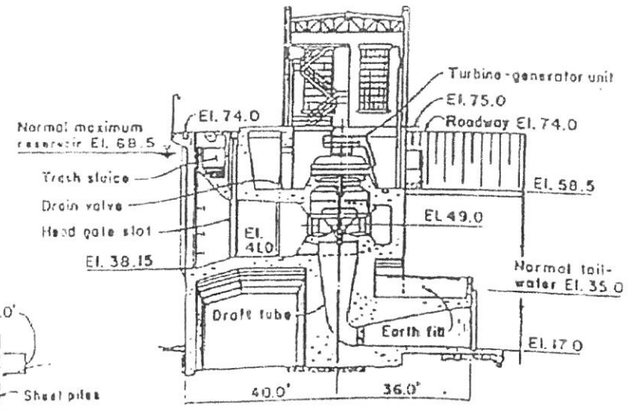
RAYTHEON INFRASTRUCTURE, INC | EXHIBIT II-1



PLAN



SECTION A-A



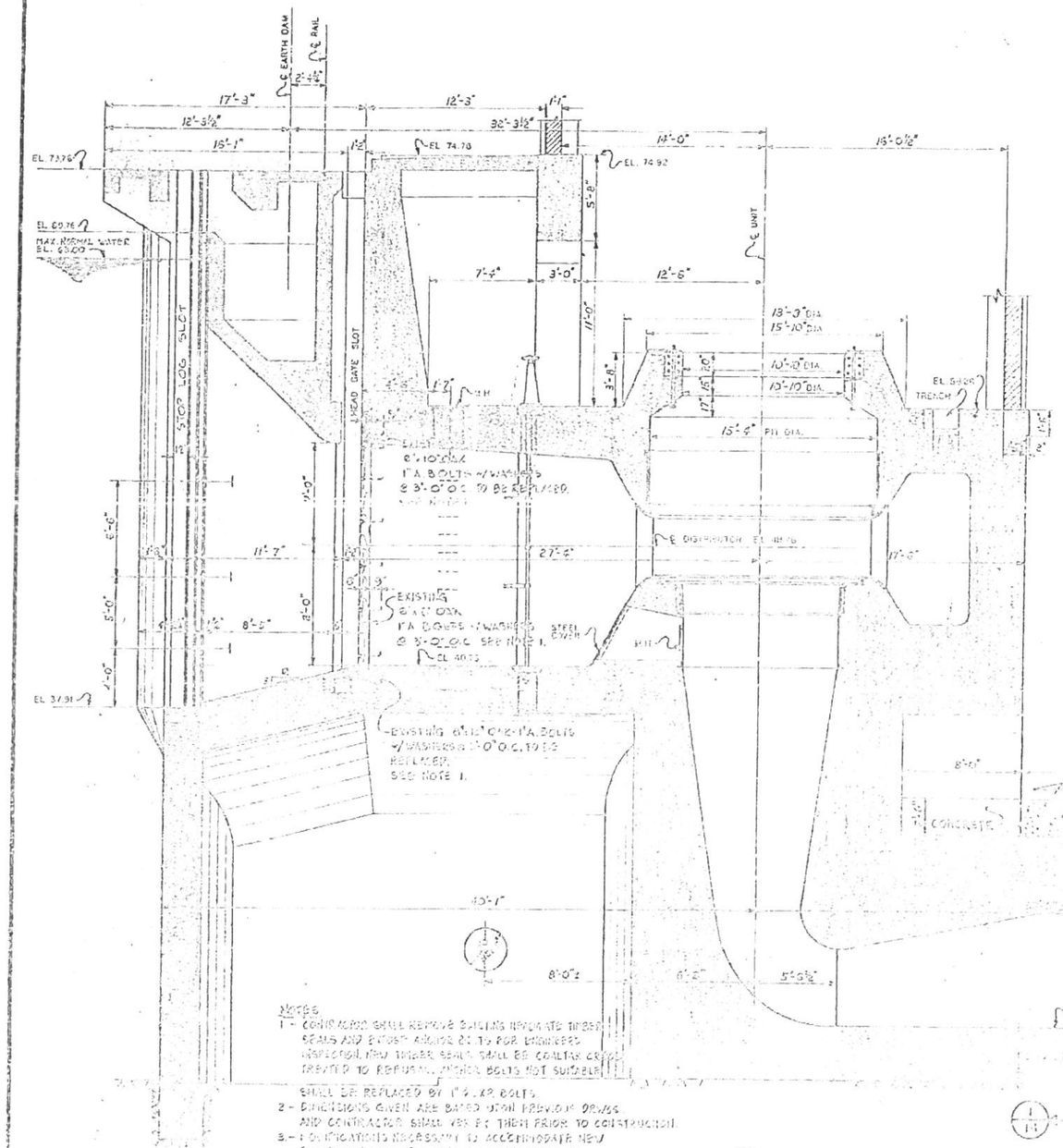
SECTION B-B

CITY OF TALLAHASSEE

JACKSON BLUFF HYDROELECTRIC PROJECT

SERVICE SPILLWAY AND POWERHOUSE PLAN AND PROFILE

RAYTHEON INFRASTRUCTURE, INC. | EXHIBIT II-3



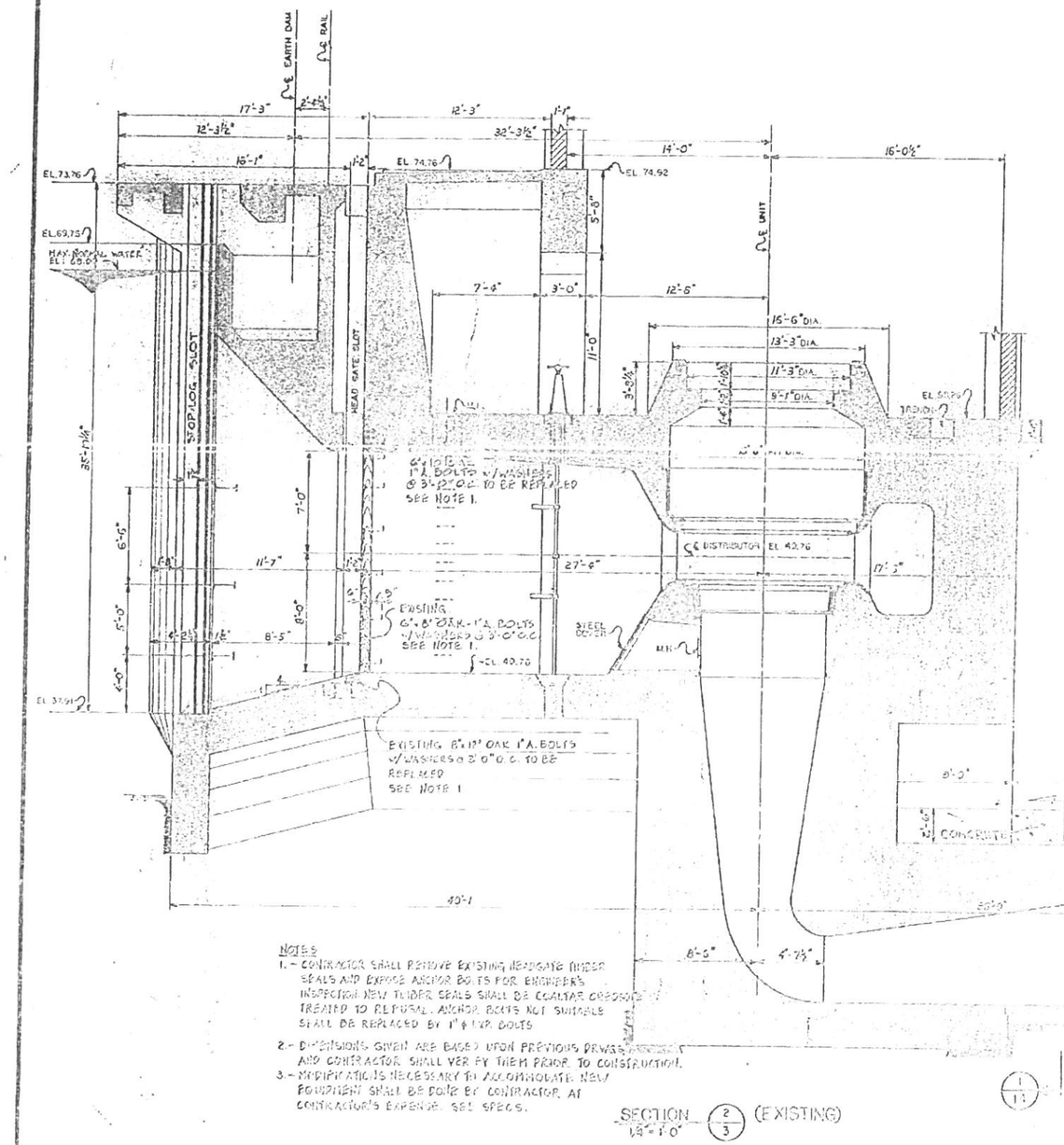
ORIGINAL CONSTRUCTION, FOR INFORMATION ONLY

WILLIAM M. BISHOP CONSULTING ENGINEERS, INC.

DOUGHERTY

CITY OF TALLAHASSEE FLORIDA
FACTORY BLUFF HYDROELECTRIC PLANT
FERC PROJECT NO. 2351

DIVISION I



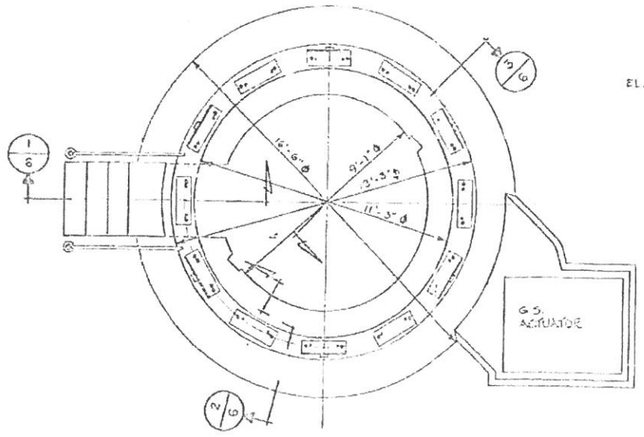
NOTES

1.- CONTRACTOR SHALL REMOVE EXISTING HEADGATE TIMBER SEALS AND EXPOSE ANCHOR BOLTS FOR ENGINEER'S INSPECTION. NEW TIMBER SEALS SHALL BE COASTAL DOUGLAS TREATED TO REFUSAL. ANCHOR BOLTS NOT SUITABLE SHALL BE REPLACED BY 1" ϕ I.V.P. BOLTS

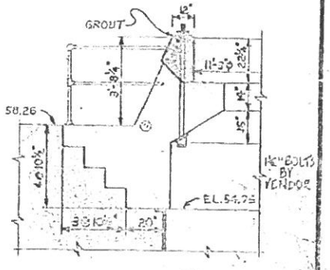
2.- DIMENSIONS GIVEN ARE BASED UPON PREVIOUS DRAWINGS AND CONTRACTOR SHALL VERIFY THEM PRIOR TO CONSTRUCTION.

3.- MODIFICATIONS NECESSARY TO ACCOMMODATE NEW EQUIPMENT SHALL BE DONE BY CONTRACTOR AT CONTRACTOR'S EXPENSE. SEE SPECS.

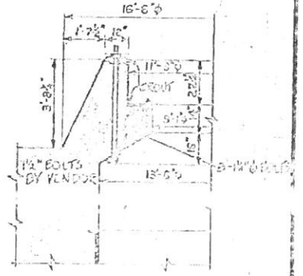
SECTION 1/8" = 1'-0" 2/3 (EXISTING)



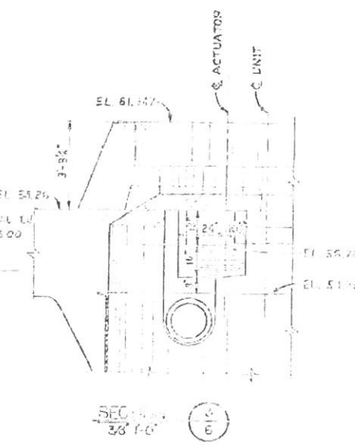
GENERATOR FOUNDATION PLAN
3/8"=1'-0"



SECTION 3/8"=1'-0" 1/6



SECTION 3/8"=1'-0" 2/6



SECTION 3/8"=1'-0" 5/6

NEEDLE TO BE SUPPLIED BY CONTRACTOR FOR POWERING DURING EQUIPMENT INSTALLATION DESIGN TWG EL 45.0' NEEDLES WILL REMAIN AS A PROPERTY OF THE CITY AFTER CONSTRUCTION IS FINISHED FOR FUTURE REPAIR AND MAINTENANCE WORKS.
FOR DETAILS SEE DWG N. 17

WITH AS-BUILT DRAWINGS
SPECIFICATIONS NO. [illegible]

ORIGINAL CONSTRUCTION, FOR INFORMATION ONLY

DIVISION I

WILLIAM M. BISHOP CONSULTING ENGINEERS, INC.

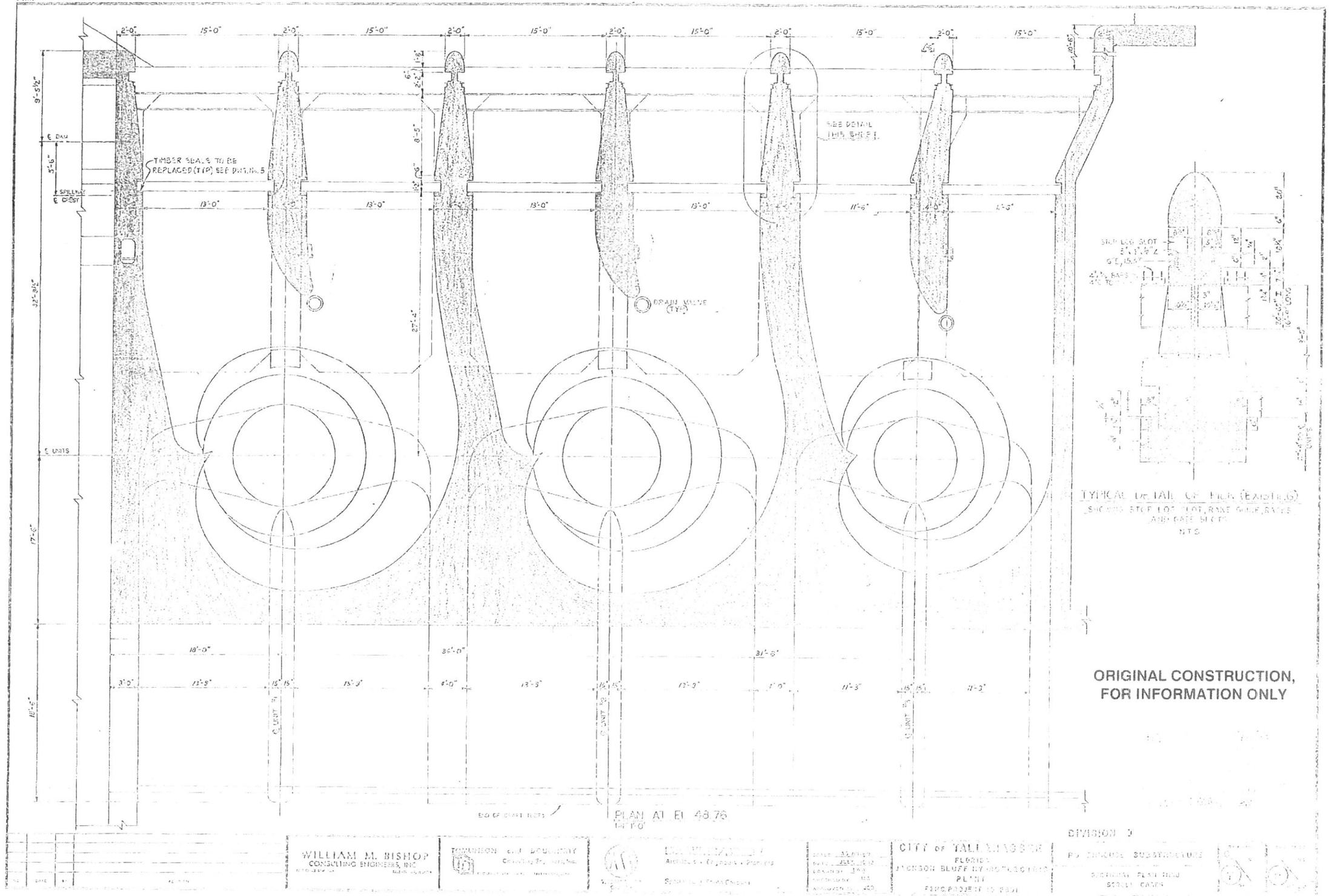
ROBERTSON and DOUGHERTY Consulting Engineers, Inc.

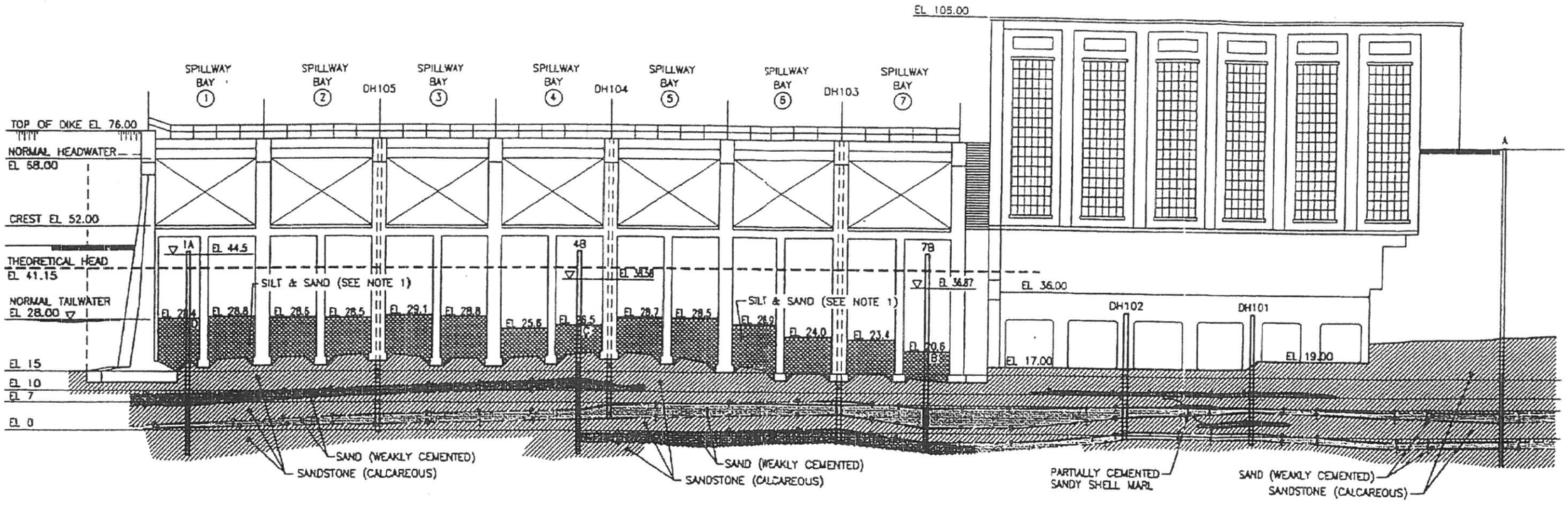
Architects • Engineers • Planners
Specialized Study Centers

SCALE AS NOTED
DATE JULY 1959
DESIGNED BY EWW
DRAWN BY [illegible]
CHECKED BY MS
APPROVED BY JDB

CITY OF TALLAHASSEE
FLORIDA
JACKSON BLUFF HYDROELECTRIC PLANT
REA PROJECT # 2481

POWER HOUSE SUBSTRUCTURE
CROSS SECTION THRU UNIT 3 AND GENERATOR FOUNDATION DETAILS


2'-0"
15'-0"
9'-5½"
℄ DAM
5'-6"
SPILLWAY CREST
TIMBER SEALS TO BE REPLACED (TYP) SEE DWG. No. 5
13'-0"
SEE DETAIL THIS SHEET
DRAIN VALVE (TYP)
27'-4"
32'-9½"
℄ UNITS
17'-0"
18'-0"
36'-0"
31'-6"
3'-0"
13'-9"
15'-8"
4'-0"
11'-3"
℄ UNIT No 1
℄ UNIT No 2
℄ UNIT No 3
END OF DRAFT TUBES
PLAN AT EL 48.76
0 10 20 30 40
STOP LOG SLOT
GATE SLOT
TYPICAL DETAIL OF PIER (EXISTING)
SHOWING STOP LOG SLOT, RAKE GUIDE, RAILS AND GATE SLOTS
NTS
ORIGINAL CONSTRUCTION, FOR INFORMATION ONLY
WILLIAM M. BISHOP
CONSULTING ENGINEERS, INC.
CITY OF TALLAHASSEE
FLORIDA
JACKSON BLUFF HYDROELECTRIC
PLANT
FERC PROJECT NO 2821
DIVISION 3
POWERHOUSE SUBSTRUCTURE
GENERAL PLAN AND
SECTIONAL GATES



SECTION A-A'

NOTE:

1. The sand and silt elevations on the drawing are the elevations prior to construction of the spillway modifications. As part on the modifications, the sand was placed to elevation 34ft in each bay.

CITY OF TALLAHASSEE

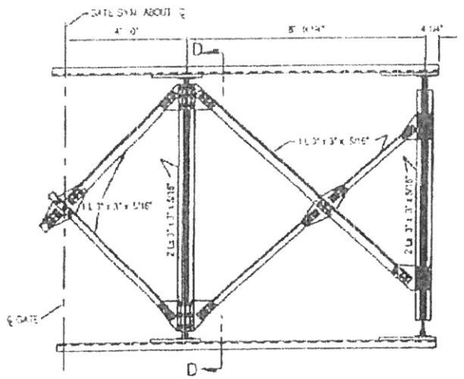
JACKSON BLUFF HYDROELECTRIC PROJECT

SPILLWAY AND POWERHOUSE
GENERALIZED SUBSURFACE PROFILE
SECTION A-A'

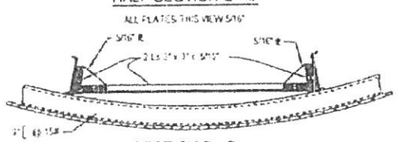
RAYTHEON INFRASTRUCTURE, INC. | EXHIBIT IV-2

REVISION	DESCRIPTION	REV. DATE	APRVD	DRAWN	REVISION	DESCRIPTION	REV. DATE	APRVD	DRAWN	REVISION	DESCRIPTION	REV. DATE	APRVD	DRAWN	REVISION	DESCRIPTION	REV. DATE	APRVD	DRAWN	REVISION	DESCRIPTION	REV. DATE	APRVD	DRAWN	REVISION	DESCRIPTION	REV. DATE	APRVD	DRAWN
A	AS-BUILT	09/24/28	-	-	D	-	-	-	-	G	-	-	-	-	J					H					P				
B	AS-BUILT	02/01/29	-	-	E	-	-	-	-	H					K					N					Q				
C	AS-BUILT	03/19/02	-	PRIDE	F	-	-	-	-	J					L					O					R				

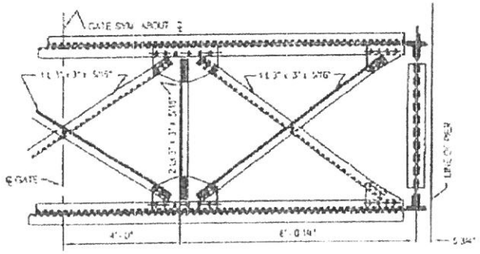
DO NOT SCALE DRAWING



HALF SECTION C - C



SECTION D - D



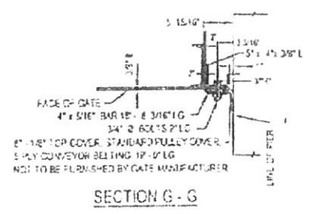
HALF SECTION H - H

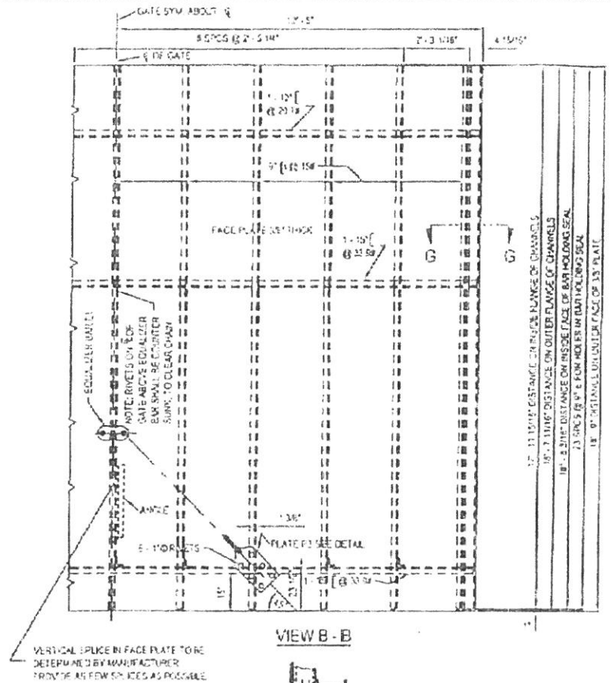
ALL PLATES THIS VIEW 5/16"

SECTION L - L

DETAIL A

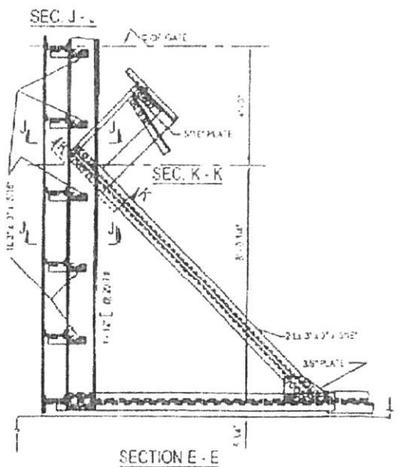
SCALE 1" = 1'-0"





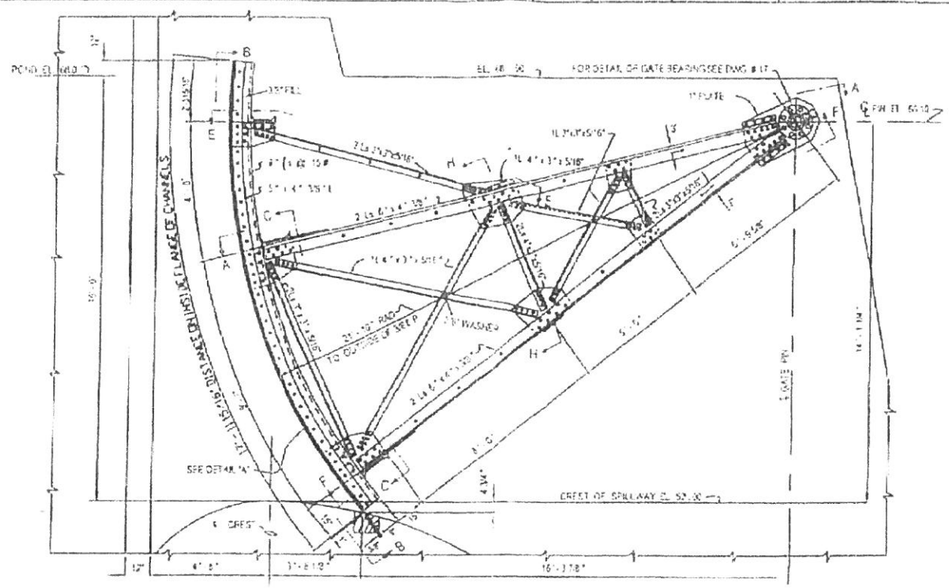
VIEW B - B

VERTICAL SPLICE IN FACE PLATE TO BE DETERMINED BY MANUFACTURER PROVIDE AS FEW SPLICES AS POSSIBLE



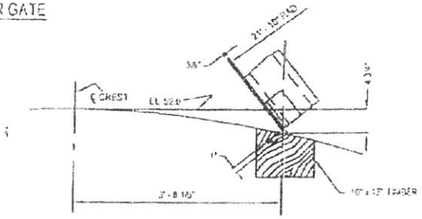
SECTION E - E

NOTES:
1. GATES REQUIRED (1) COMPLETE.
GATES DESIGNED FOR 57'-0" (+) DEPTH OF WATER.
ESTIMATED WEIGHT OF STEEL AND BOLTS FOR ONE GATE 25,000 LBS.
FURNISH EACH GATE 60 - 3/4" ø 2" MACHINE BOLTS WITH HEX NUTS.
FIELD RIVETS 5% IN EXCESS OF EACH SIZE AND LENGTH REQUIRED.
ERECTION BOLTS 5.00% OF NUMBER OF EACH KIND OF FIELD RIVETS REQUIRED.
ALL RIVETS 3/4" ø, (OPEN HOLES 13/16" ø), EXCEPT AS NOTED.
ALL PLATES 3/8" EXCEPT AS NOTED.
STRUCTURAL STEEL TO BE MFGRS. STD. CLASS "A", O.H.
PAINT ONE SHOP COAT OF RED LEAD IN BOILED LINSEED OIL.
ONE GATE SHIPS COMPLETELY SHOP ASSEMBLED.
MFGR. SHALL FURNISH SHOP DRAWINGS FOR APPROVAL BEFORE FABRICATING.



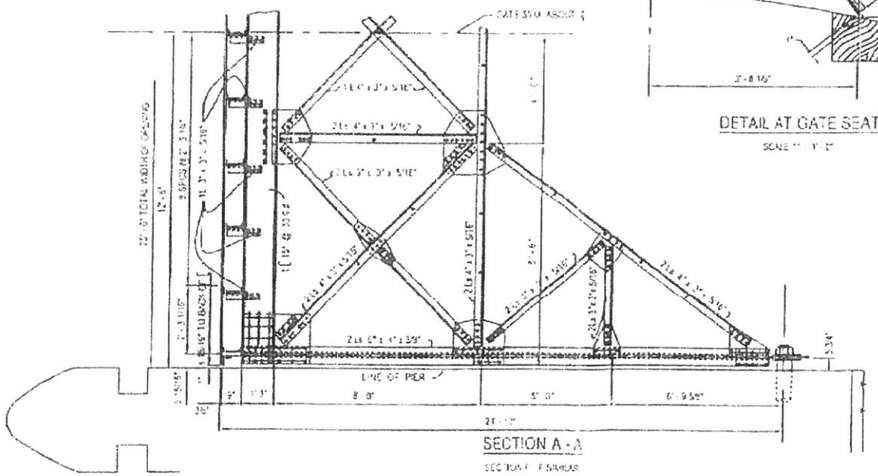
ELEVATION TAINTER GATE

REFERENCE DWGS.
FOR GATE ANCHORAGE, PINS AND BEARING DETAILS SEE DWG. # 17
FOR CONCRETE WORK SEE DWGS. # 15 & 16
FOR DETAIL OF PAVILION SEE DWG. # 12



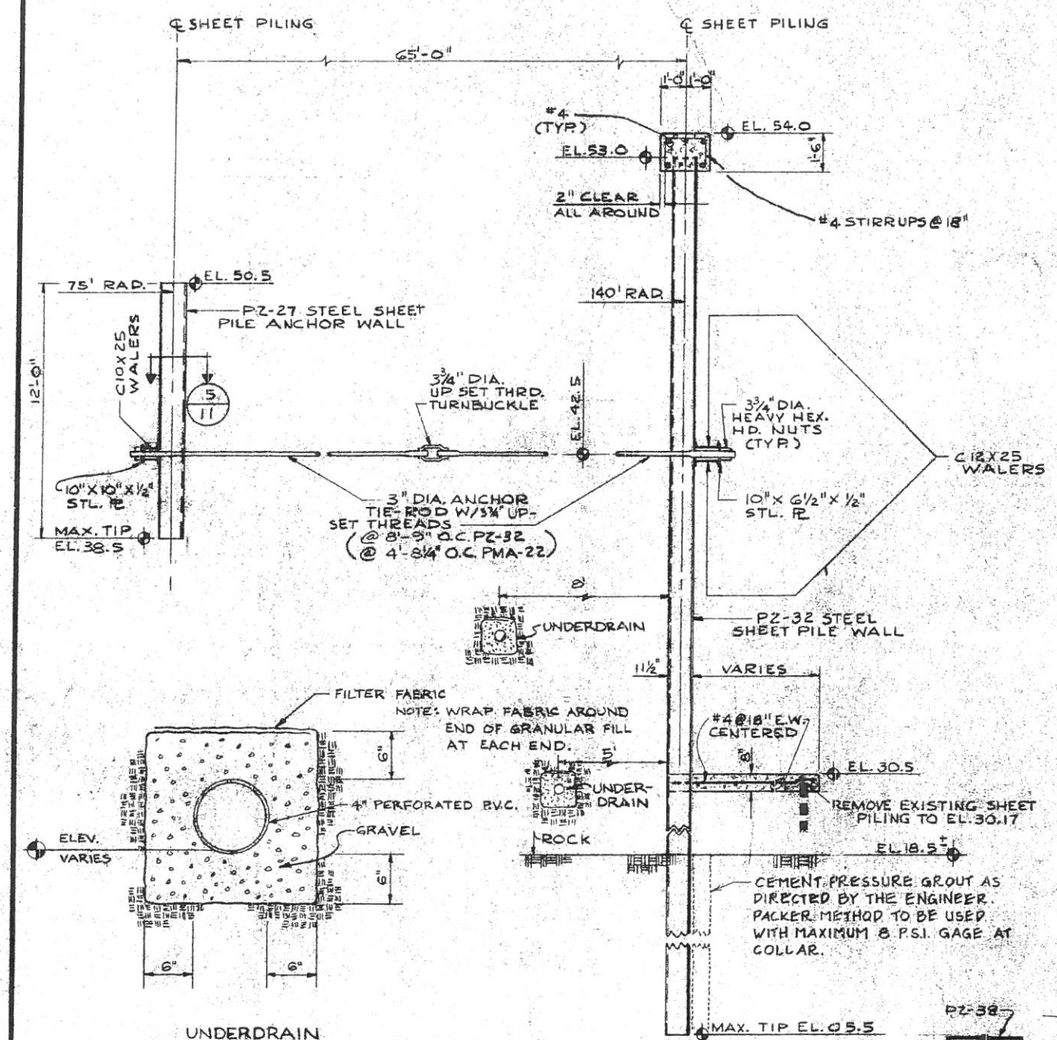
DETAIL AT GATE SEAT

SCALE 1" = 1'-0"



SECTION A - A IS SIMILAR

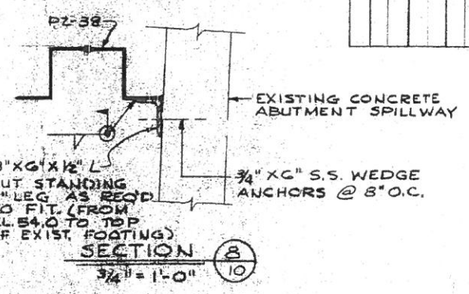
CITY OF TALLAHASSEE ELECTRIC RELIABILITY	MEES & MEES INC., ENGINEERS	DRAWN BY		City Electric TALLAHASSEE RELAY & COMMUNICATIONS GROUP	TAINTER GATES ASSEMBLY DETAILS
		ENGINEER			ST. MARKS GENERATING STATION
	CHARLOTTE, N. C.	PROJECT NO.			SHEET 1 OF 1
	05/14/27	SCALE	NTS		REVISION C



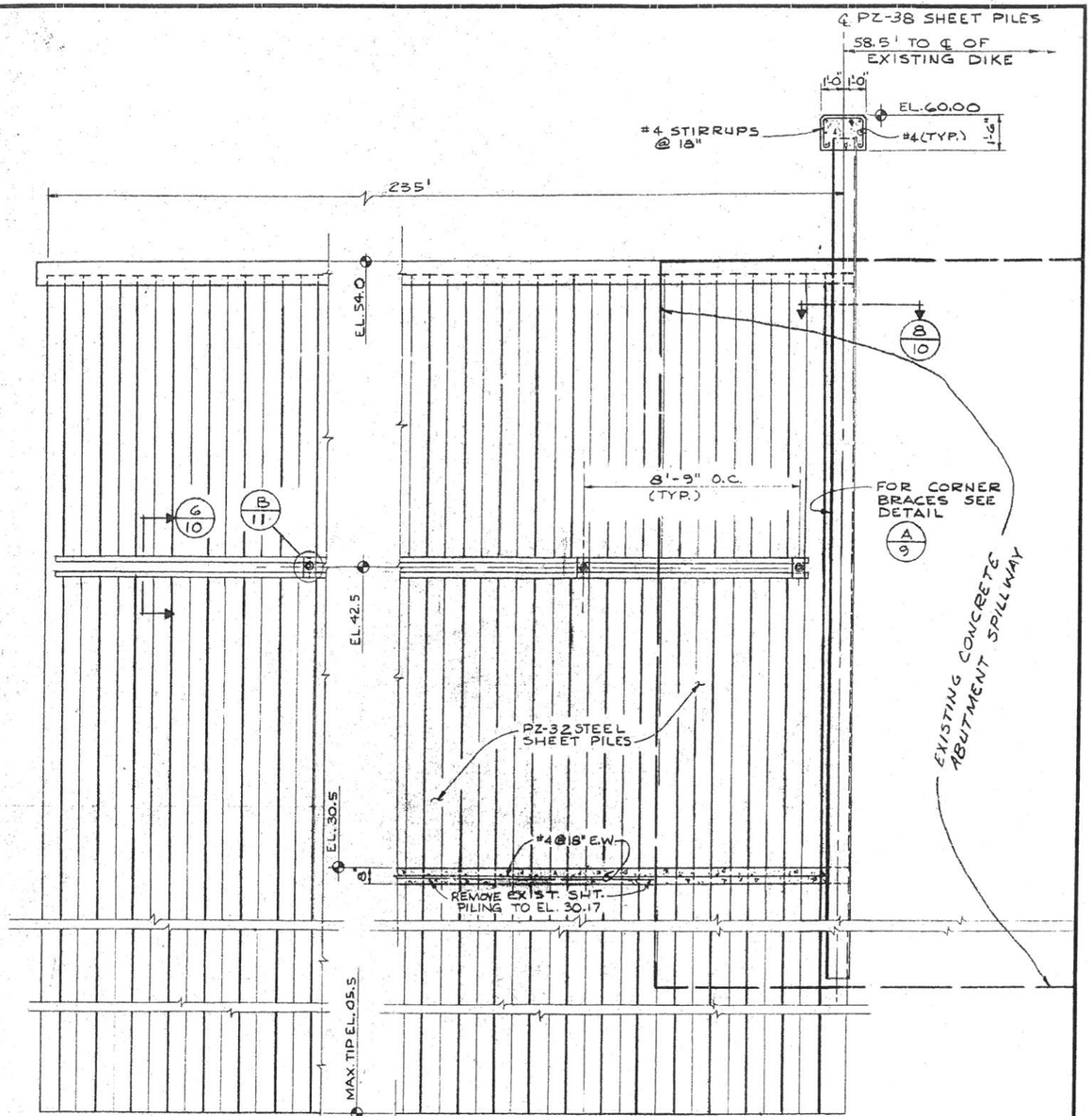
UNDERDRAIN TYPICAL SECTION
1½" = 1'-0"

NOTE: CONSTRUCT (2) UNDERDRAINS (250 L.F. EACH) BEHIND PROPOSED PZ 32 SHEET PILING AS SHOWN. BEGIN CONSTRUCTION AT INV. EL. 30.0 (LOWER) AND INV. EL. 36.0 (UPPER). SLOPE ON 0.50% GRADE PAST END OF EXISTING RIP-RAP EMBANKMENT. SEE SHEET 12 FOR APPROXIMATE LOCATION.

SECTION 1/9
3/8" = 1'-0"



SECTION 8/10
¾" = 1'-0"



ELEVATION - NORTH TRAINING WALL
3/8" = 1'-0"

DIVISION II

NO.	DATE	BY	REVISION

WILLIAM M. BISHOP
CONSULTING ENGINEERS, INC.
POST OFFICE BOX 1487 TALLAHASSEE, FLORIDA

TOMLINSON and DOUGHERTY
Consulting Engineers, Inc.
MECHANICAL ELECTRICAL TALLAHASSEE, FLORIDA

THE MAGUIRE GROUP

MAGUIRE/BEEBE
Architects • Engineers • Planners
Spartanburg, South Carolina

SCALE: AS SHOWN
DATE: JULY, 1982
DRAWN BY: M.P.K.
CHECKED BY: E.W.V.
APPROVED BY: E.W.V.

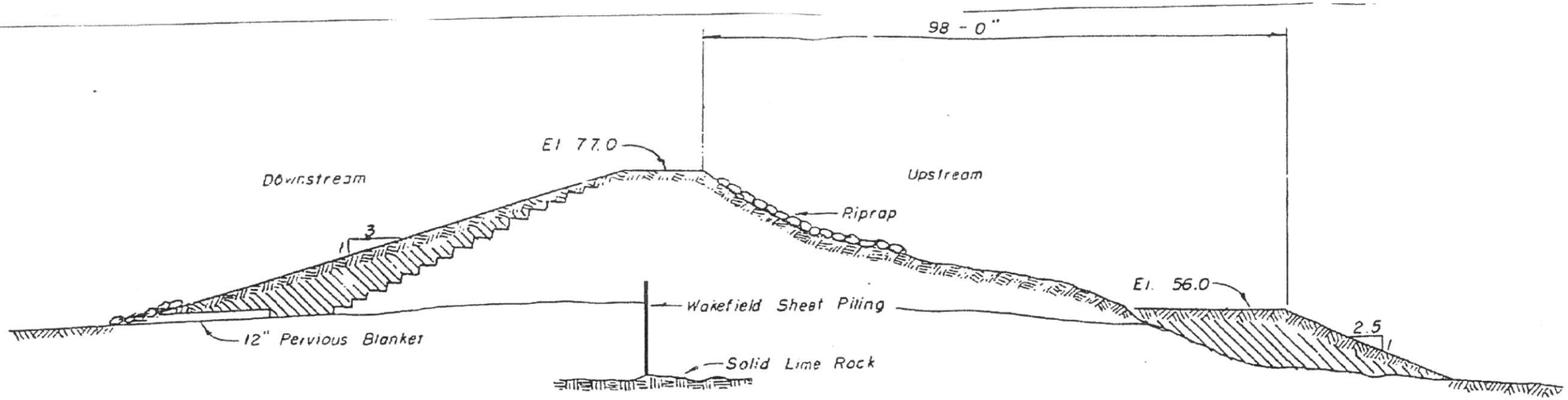
CITY OF TALLAHASSEE
FLORIDA
JACKSON BLUFF HYDROELECTRIC PLANT
F.E.R.C. PROJECT NO. 2891

NORTH TRAINING WALL

STRUCTURAL DETAILS

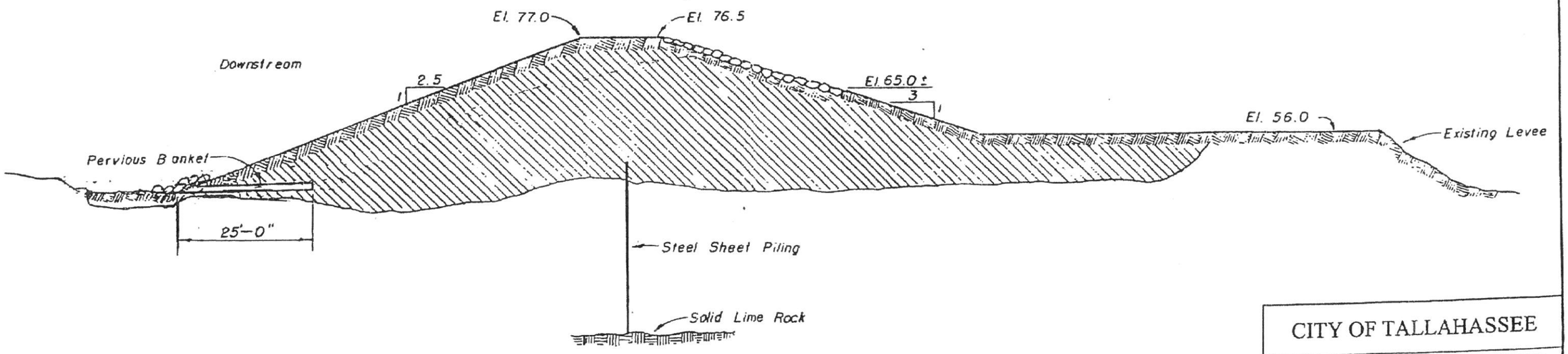
DRAWING NO. C 10 OF 19

SEQUENCE NO. 65 OF 86



STRENGTHENING OF STANDING SECTIONS

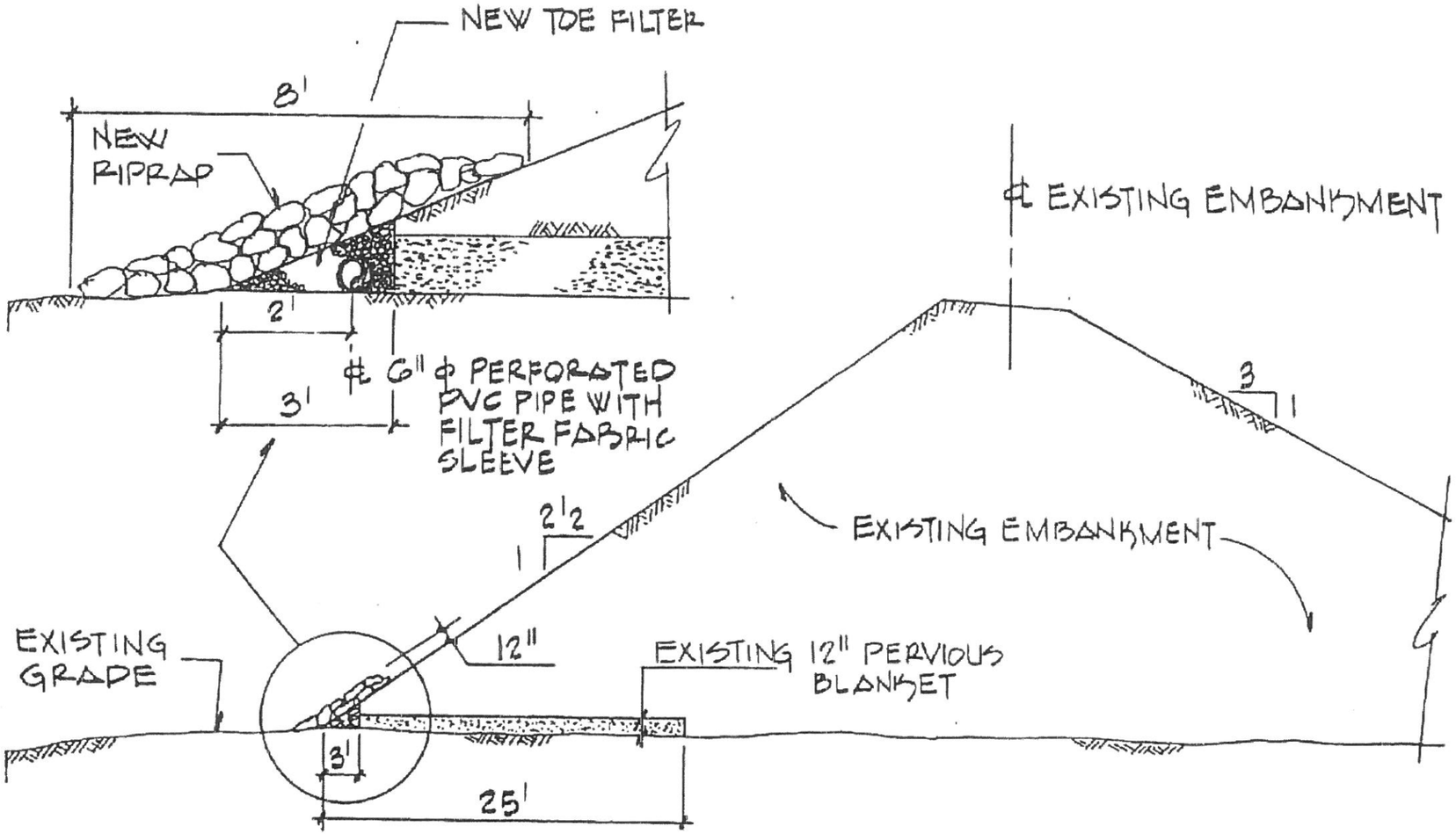
Scale: 1"= 20'



RECONSTRUCTION OF FAILED SECTIONS

TYPICAL SECTIONS OF EMBANKMENT RECONSTRUCTION
FROM BLACK & VEATCH EXHIBIT L DRAWING, SHEET 12
JANUARY, 1959

CITY OF TALLAHASSEE	
JACKSON BLUFF HYDROELECTRIC PROJECT	
RECONSTRUCTION ENBANKMENT SECTIONS	
RAYTHEON INFRASTRUCTURE, INC.	EXHIBIT VI-2



TOE DRAIN MODIFICATION
TYP SECT
NTS

CITY OF TALLAHASSEE	
JACKSON BLUFF HYDROELECTRIC PROJECT	
TOE DRAIN MODIFICATION PROFILE	
RAYTHEON INFRASTRUCTURE, INC.	EXHIBIT VI-4

BENCH MARK DATA

☆ EL. 74.84 CHS X'S TIRESHOLD & ENTRANCE TO POWER HOUSE

NOTES:

1. ✱ DENOTES APPROXIMATE LOCATION OF CONDUIT STUB-UPS, VERIFY EXACT LOCATIONS WITH GENERATOR EQUIPMENT SUPPLIED.
2. PROVIDE A 1½"C. W/ 3#8,(2-120V.CIRCS) 8#12,(CONTROL) 1#10G. TO EXIST.11G MANHOLE & THEN TO 120/240VOLT POWER PANEL.
3. PROVIDE A 2½"C. W/ 4#3/0, 1#6G. TO EXIST. MANHOLE & THEN TO AUTOMATIC TRANSFER SWITCH.

EL 73.76

EL 73.76

EL 74.59

EL 74.76

EL 74.92

EXISTING GENERATOR FOUNDATIONS (TYP.)

EXISTING FLOOR EL 58.26
(NEW EQUIPMENT NOT SHOWN)

UNIT #1

UNIT #2

UNIT #3

EXISTING TRENCH

EXIST. ROTOR SHAFT PIT

℄ RAIL

℄ RAIL

℄ RAIL

℄ DAM

℄ UNITS

EL. 74.76

NEW RETAINING WALL DIV. II

EXIST. 13'-0" ROLLING DOOR

NEW FLOOR BARRIERS SEE DETAIL DWG. NO. 17

EXIST. DOOR (1) (TO BE REPLACED)

EL. 74.76

NEW 6"-6" CURB

FUEL TANK

NOTE 2

GENERATOR

NEW 100 KW ENCLOSED GENERATOR (480V.)

RADIATOR

NOTE 3

EXIST. CHIEF LINES

3'-0" MIN.

EXISTING CONCRETE SLAB REPAIR ALL CRACKS AS SPECIFIED PREPARE SURFACE AS REQ'D APPLY SIKA FLEXIBLE SYSTEM OR EQUAL.

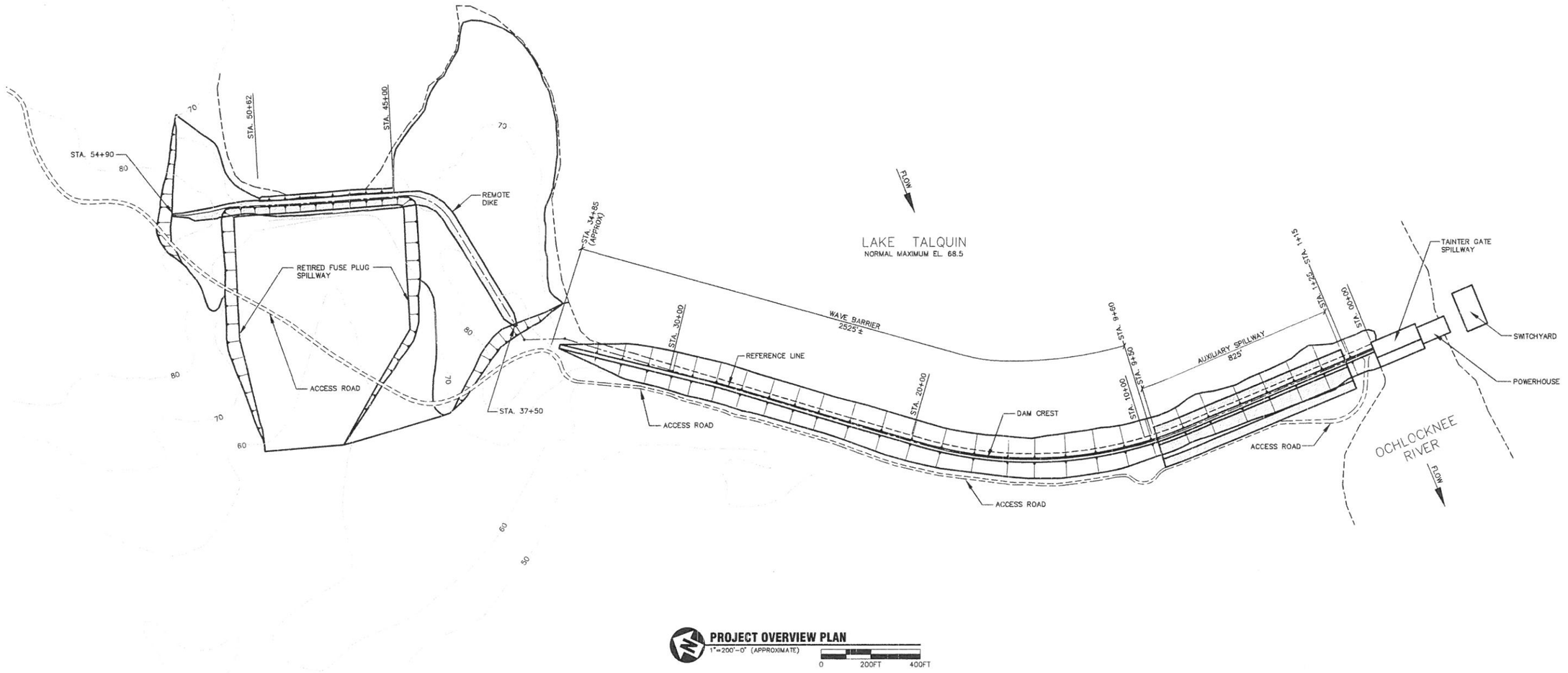
EL. 73.76

PLAN AT EL. 74.76 EXISTING U.O.N.
1/4"=1'-0"

ORIGINAL CONSTRUCTION, FOR INFORMATION ONLY

DIVISION I

NO. DATE BY REVISION	WILLIAM M. BISHOP CONSULTING ENGINEERS, INC. TALLAHASSEE, FLORIDA	TOMLINSON and DOUGHERTY Consulting Engineers, Inc. MECHANICAL ELECTRICAL TALLAHASSEE FLORIDA	THE MAGUIRE GROUP MAGUIRE/BEEBE Architects • Engineers • Planners Spartanburg South Carolina	SCALE AS NOTED DATE JULY, 1982 DRAWN BY LWW CHECKED BY MA APPROVED BY JDB	CITY OF TALLAHASSEE FLORIDA JACKSON BLUFF HYDROELECTRIC PLANT F.E.R.C PROJECT NO. 2891	POWERHOUSE SUBSTRUCTURE PLAN VIEW AT OPERATING GATE DECK, UNLOADING PLATFORM, AND GENERATOR FLOOR	DRAWING NO. C 3 OF 22 SEQUENCE NO. 4 OF 86



PROJECT OVERVIEW PLAN
1"=200'-0" (APPROXIMATE)
0 200FT 400FT

		DATE	NO.	BY	REVISIONS	DATE
FIELD BOOK -		-	-	-	-	-
DRAWN BY	DMS	-	-	-	-	-
SURVEYED BY	-	-	-	-	-	-
CHECKED BY	MHK	-	-	-	-	-
SCALE: -AS SHOWN		-	-	-	-	-

City of Tallahassee
Your Own Utilities℠

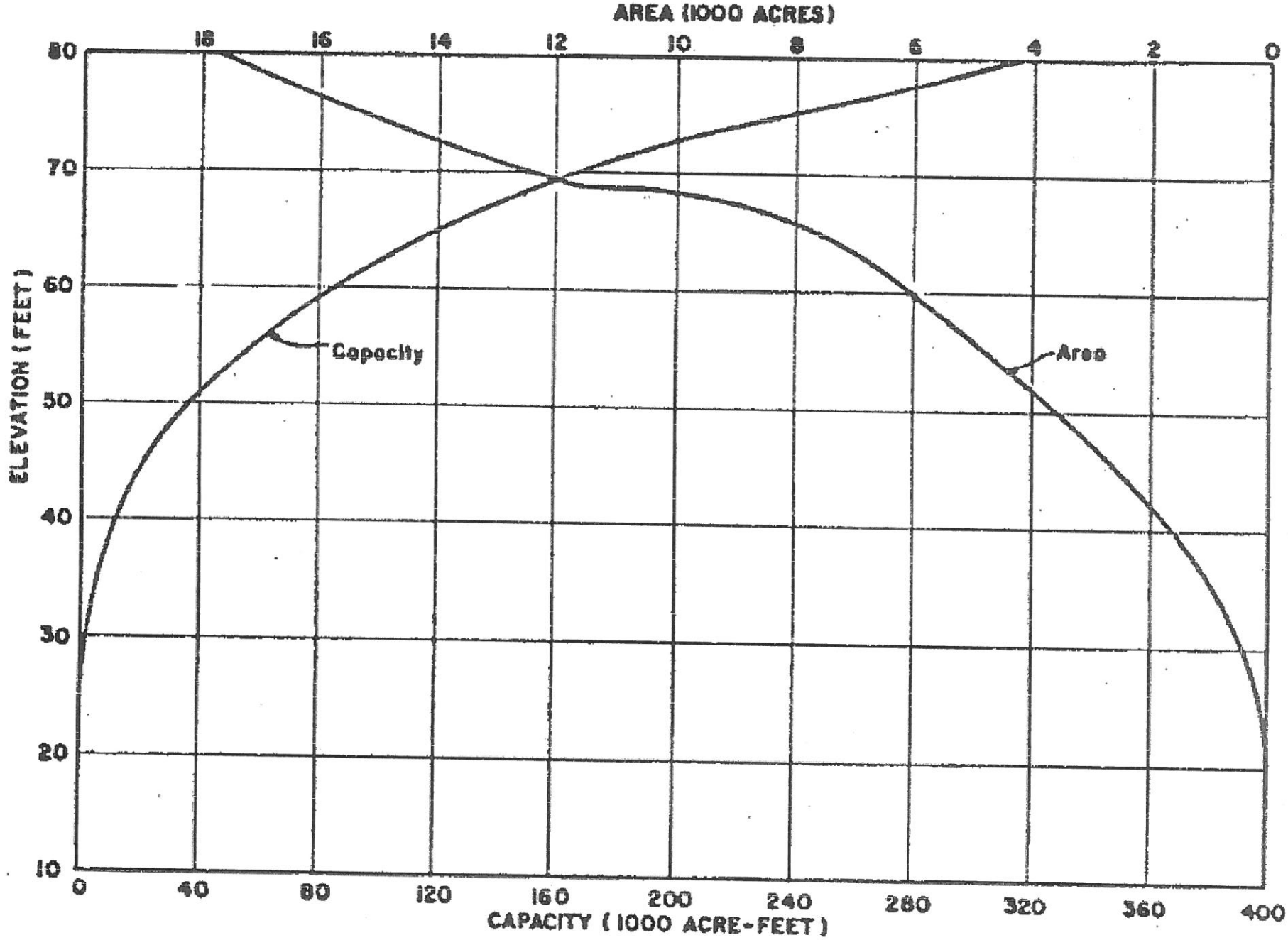
MEAD & HUNT ENGINEERS ARCHITECTS SCIENTISTS PLANNERS
6501 Watts Road, Madison, Wisconsin 53719-2700
608-273-6380 Fax: 608-273-6391
Certificate of Authorization No. 26730

PROJECT
JACKSON BLUFF HYDRO PROJECT
F.E.R.C. PROJECT No. 2891
AUXILIARY SPILLWAY

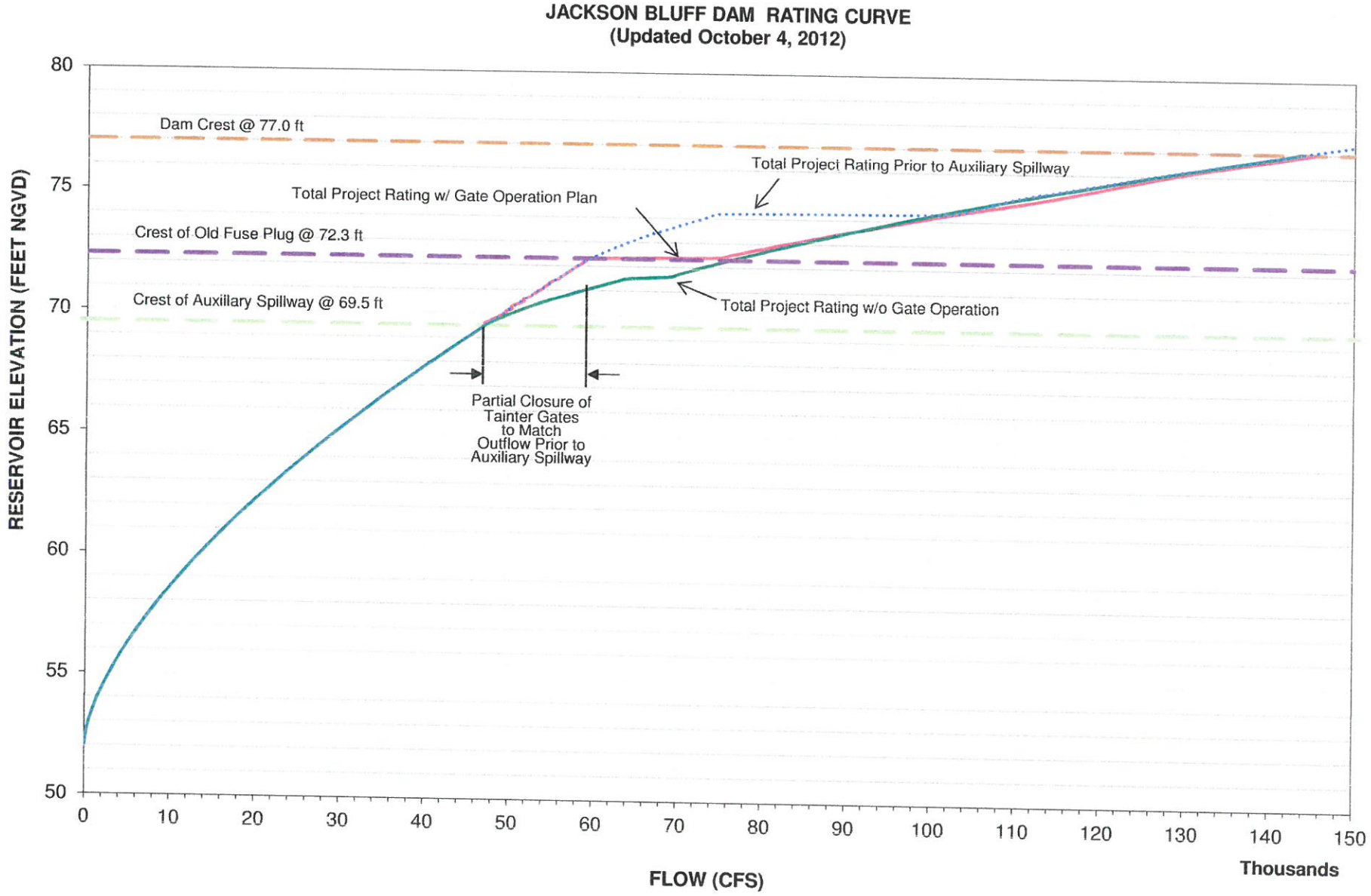
SHEET TITLE
PROJECT OVERVIEW

FILE PATH X:\20147-00\10002\TECH\CADD\
FILE NO. PROJECT NO. 20147-00-10002

SHEET 1 OF 1



AREA-CAPACITY CURVE


JACKSON BLUFF DAM RATING CURVE
(Updated October 4, 2012)
RESERVOIR ELEVATION (FEET NGVD)
80
75
70
65
60
55
50
Dam Crest @ 77.0 ft
Total Project Rating Prior to Auxiliary Spillway
Total Project Rating w/ Gate Operation Plan
Crest of Old Fuse Plug @ 72.3 ft
Crest of Auxillary Spillway @ 69.5 ft
Total Project Rating w/o Gate Operation
Partial Closure of
Tainter Gates
to Match
Outflow Prior to
Auxiliary Spillway
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
Thousands
FLOW (CFS)

JACKSON BLUFF DAM PROJECT NO. 2891

TAILWATER RATING CURVE

FLOW (CFS)	TAILWATER ELEV. (NGVD)
4,800	40
6,000	42
9,600	44
15,600	46
20,000	47
26,000	48
43,200	50
65,200	52
90,000	54
100,000	54.7
110,000	55.3
120,000	55.8
130,000	56.4
140,000	56.9
148,000	57.3

JACKSON BLUFF GATE OPERATION MODIFICATION

Revised October 4, 2012

GATE OPENING ADJUSTMENT IS NOT REQUIRED.
GATE OPENING ADJUSTMENT IS REQUIRED.

Controlled Spillway (Tainter Gates)						
ASSUMED DESIGN HEAD	19.5	feet	WEIR BOTTOM	16	feet(ngvd)
CREST ELEVATION	52	feet (ngvd	WEIR HEIGHT	36	feet
CREST LENGTH	175	feet	U/S FACE SLOPE	1.09	
GATE FULL OPENING	18	feet	P/H_0	1.85	
NUMBER OF BAYS	7		C_0 [2]	3.93	
CREST X COORDINATE [1]	5.11		U/S FACE SLOPE CORREC.[2]	0.99	
WEIR LENGTH EACH BAY	25	feet	PIER CONTRACTION COEFF.[3]	0.02	
			ABUTMENT CONTR. COEFF.[4]	0.1	

Auxillary Spillway	
CREST ELEVATION	69.5 feet (ngvd)
DESIGN HEAD, H_0	4.69 feet
APPROACH HEIGHT, P	2.35 feet
WEIR COEFFICIENT FOR DESIGN HEAD, C_o [2]	3.80
UPSTREAM SLOPE CORRECTION COEFFICIENT, $C_{inclined}/C_{vertical}$	1.015
SPILLWAY LENGTH, L	825 feet

Normal Reservoir Elevation = 68.5 feet

								NAPPE PROFILE				GATE #1 OPERATION		GATE # 2 OPERATION			GATE #3 OPERATION			GATE # 4 OPERATION		GATE # 5 OPERATION			GATE # 6 OPERATION			GATE # 7 OPERATION							
RESERVOIR ELEVATION feet (NGVD)	HEAD OVER CREST feet	He/H_0	C/C_0 [5]	Effective Weir Length	C_{WEIR}	NUMBER OF GATES FULLY OPEN	WEIR FLOW cfs	x POSITION OF GATE BOTTOM [6]	x/h [7]	y/h [7,8]	NAPPE DEPTH AT GATE BOTTOM [7]	GATE OPENING G_0 (ft)	ORIFICE [9,10] FLOW cfs	GATE OPENING G_0 (ft)	$C_{d\ Orifice}$ [11]	ORIFICE [9,10] FLOW cfs	GATE OPENING G_0 (ft)	$C_{d\ Orifice}$ [11]	ORIFICE [9,10] FLOW cfs	GATE OPENING G_0 (ft)	ORIFICE [9,10] FLOW cfs	GATE OPENING G_0 (ft)	$C_{d\ Orifice}$ [11]	ORIFICE [9,10] FLOW cfs	GATE OPENING G_0 (ft)	$C_{d\ Orifice}$ [11]	ORIFICE [9,10] FLOW cfs	GATE OPENING G_0 (ft)	ORIFICE [9,10] FLOW cfs	HEAD OVER AUXILIARY feet	H_e/H_0	$C_{Vertical}/C_0$ [5]	$C_{vertical}$	$C_{inclined}$	FLOW OVER AUXILIARY cfs	TOTAL FLOW W/ GATE OP c's
68.00	16.00	0.82	0.975	157.96	3.79	7	40775					18.0	0	18.0		0	18.0		0	18.0	0	18.0		0	18.0		0	18.0	0	0.00	-				0	40775
68.50	16.50	0.85	0.979	157.74	3.81	7	42844					18.0	0	18.0		0	18.0		0	18.0	0	18.0		0	18.0		0	18.0	0	0.00	-				0	42844
69.00	17.00	0.87	0.983	157.52	3.82	7	44903					18.0	0	18.0		0	18.0		0	18.0	0	18.0		0	18.0		0	18.0	0	0.00	-				0	44903
69.50	17.50	0.90	0.986	167.3	3.84	7	47000					18.0	0	18.0		0	18.0		0	18.0	0	18.0		0	18.0		0	18.0	0	0.00	-				0	47000
69.60	17.60	0.90	0.987	142.96	3.84	6	40535	3.420	0.194	0.739	13.01	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	13.0	0.738	6408	18.0	0	0.10	0.02	0.790	3.00	3.05	79	47023
69.70	17.70	0.91	0.988	142.92	3.84	6	40898	3.420	0.193	0.739	13.08	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	13.0	0.738	6437	18.0	0	0.20	0.04	0.800	3.04	3.09	228	47563
69.80	17.80	0.91	0.988	142.88	3.85	6	41262	3.420	0.192	0.739	13.16	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	13.0	0.738	6466	18.0	0	0.30	0.06	0.805	3.06	3.10	421	48149
69.90	17.90	0.92	0.989	142.84	3.85	6	41627	3.420	0.191	0.739	13.23	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	13.0	0.738	6494	18.0	0	0.40	0.09	0.815	3.10	3.14	656	48778
70.00	18.00	0.92	0.990	142.8	3.85	6	41994	3.420	0.190	0.741	13.33	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	13.0	0.738	6523	18.0	0	0.50	0.11	0.824	3.13	3.18	927	49444
70.10	18.10	0.93	0.990	118.464	3.85	5	35158	3.420	0.189	0.741	13.41	18.0	0	13.0	0.738	6551	18.0	0.745	0	18.0	0	18.0	0.745	0	13.0	0.738	6551	18.0	0	0.60	0.13	0.830	3.15	3.20	1227	49488
70.20	18.20	0.93	0.991	118.448	3.86	5	35464	3.420	0.188	0.741	13.48	18.0	0	13.0	0.738	6579	18.0	0.745	0	18.0	0	18.0	0.745	0	13.0	0.738	6579	18.0	0	0.70	0.15	0.835	3.17	3.22	1556	50179
70.30	18.30	0.94	0.992	94.144	3.86	4	28439	3.420	0.187	0.741	13.56	18.0	0	13.0	0.738	6607	18.0	0.745	0	18.0	0	13.0	0.738	6607	13.0	0.738	6607	18.0	0	0.80	0.17	0.842	3.20	3.25	1917	50179
70.40	18.40	0.94	0.992	94.112	3.86	4	28683	3.420	0.186	0.741	13.63	18.0	0	13.0	0.738	6635	18.0	0.745	0	18.0	0	13.0	0.738	6635	13.0	0.738	6635	18.0	0	0.90	0.19	0.846	3.21	3.26	2298	50887
70.50	18.50	0.95	0.993	94.08	3.86	4	28927	3.420	0.185	0.742	13.73	18.0	0	13.0	0.738	6663	18.0	0.745	0	18.0	0	13.0	0.738	6663	13.0	0.738	6663	18.0	0	1.00	0.21	0.850	3.23	3.28	2705	51621
70.60	18.60	0.95	0.994	69.792	3.87	3	21648	3.420	0.184	0.742	13.81	18.0	0	13.0	0.738	6691	13.0	0.738	6691	18.0	0	13.0	0.738	6691	13.0	0.738	6691	18.0	0	1.10	0.23	0.860	3.27	3.32	3157	51569
70.70	18.70	0.96	0.995	69.764	3.87	3	21829	3.420	0.183	0.742	13.88	18.0	0	13.0	0.738	6718	13.0	0.738	6718	18.0	0	13.0	0.738	6718	13.0	0.738	6718	18.0	0	1.20	0.26	0.865	3.29	3.34	3618	52321
70.80	18.80	0.96	0.995	69.736	3.87	3	22011	3.420	0.182	0.742	13.96	18.0	0	12.9	0.729	6628	12.9	0.729	6628	18.0	0	12.9	0.729	6628	12.9	0.729	6628	18.0	0	1.30	0.28	0.873	3.32	3.37	4117	52641
70.90	18.90	0.97	0.996	69.708	3.87	3	22193	3.420	0.181	0.742	14.03	18.0	0	12.7	0.729	6578	12.7	0.729	6578	18.0	0	12.7	0.729	6578	12.7	0.729	6578	18.0	0	1.40	0.30	0.878	3.34	3.39	4628	53133
71.00	19.00	0.97	0.997	69.68	3.88	3	22376	3.420	0.180	0.744	14.14	18.0	0	12.5	0.729	6526	12.5	0.729	6526	18.0	0	12.5	0.729	6526	12.5	0.729	6526	18.0	0	1.50	0.32	0.880	3.34	3.39	5144	53623
71.10	19.10	0.98	0.997	69.652	3.88	3	22559	3.420	0.179	0.744	14.21	18.0	0	12.4	0.720	6433	12.4	0.720	6433	18.0	0	12.4	0.720	6433	12.4	0.720	6433	18.0	0	1.60	0.34	0.888	3.37	3.43	5719	54011
71.20	19.20	0.98	0.998	69.624	3.88	3	22743	3.420	0.178	0.744	14.28	18.0	0	12.1	0.720	6338	12.1	0.720	6338	18.0	0	12.1	0.720	6338	12.1	0.720	6338	18.0	0	1.70	0.36	0.892	3.39	3.44	6291	54387
71.30	19.30	0.99	0.999	69.596	3.89	3	22927	3.420	0.177	0.744	14.36	18.0	0	11.9	0.714	6226	11.9	0.714	6226	18.0	0	11.9	0.714	6226	11.9	0.714	6226	18.0	0	1.80	0.38	0.897	3.41	3.45	6893	54725
71.40	19.40	0.99	0.999	69.568	3.89	3	23112	3.420	0.176	0.744	14.43	18.0	0	11.7	0.714	6167	11.7	0.714	6167	18.0	0	11.7	0.714	6167	11.7	0.714	6167	18.0	0	1.90	0.41	0.903	3.43	3.48	7525	55306
71.50	19.50	1.00	1.000	69.54	3.89	3	23298	3.420	0.175	0.744	14.51	18.0	0	11.5	0.714	6106	11.5	0.714	6106	18.0	0	11.5	0.714	6106	11.5	0.714	6106	18.0	0	2.00	0.43	0.910	3.46	3.51	8190	55913
71.60	19.60	1.01	1.001	69.512	3.89	3	23484	3.420	0.174	0.746	14.61	18.0	0	11.3	0.708	5991	11.3	0.708	5991	18.0	0	11.3	0.708	5991	11.3	0.703	5991	18.0	0	2.10	0.45	0.915	3.48	3.53	8860	56307
71.70	19.70	1.01	1.001	69.484	3.90	3	23670	3.420	0.174	0.746	14.69	18.0	0	11.0	0.708	5884	11.0	0.708	5884	18.0	0	11.0	0.708	5884	11.0	0.703	5884	18.0	0	2.20	0.47	0.918	3.49	3.54	9532	56737
71.80	19.80	1.02	1.002	69.456	3.90	3	23858	3.420	0.173	0.746	14.76	18.0	0	10.8	0.701	5766	10.8	0.701	5766	18.0	0	10.8	0.701	5766	10.8	0.701	5766	18.0	0	2.30	0.49	0.920	3.50	3.55	10211	57132
71.90	19.90	1.02	1.003	69.428	3.90	3	24045	3.420	0.172	0.746	14.84	18.0	0	10.5	0.701	5654	10.5	0.701	5654	18.0	0	10.5	0.701	5654	10.5	0.701	5654	18.0	0	2.40	0.51	0.923	3.51	3.56	10920	57582
72.00	20.00	1.03	1.003	59.4	3.90	3	24233	3.420	0.171	0.746	14.91	18.0	0	10.3	0.695	5534	10.3	0.695	5534	18.0	0	10.3	0.695	5534	10.3	0.693	5534	18.0	0	2.50	0.53	0.927	3.52	3.58	11660	58031
72.10	20.10	1.03	1.004	69.372	3.91	3	24422	3.420	0.1701	0.746	14.99	18.0	0	10.0	0.695	5418	10.0	0.695	5418	18.0	0	10.0	0.695	5418	10.0	0.693	5418	18.0	0	2.60	0.55	0.931	3.54	3.59	12420	58515
72.20	20.20	1.04	1.005	69.344	3.91	3	24611	3.420	0.169	0.747	15.09	18.0	0	9.7	0.694	5289	9.7	0.694	5289	18.0	0	9.7	0.694	5289	9.7	0.694	5289	18.0	0	2.70	0.58	0.935	3.55	3.61	13200	58969
72.30	20.30	1.04	1.005	69.316	3.91	3	24801	3.420	0.168	0.747	15.17	18.0	0	9.4	0.693	5158	9.4	0.693	5158	18.0	0	9.4	0.693	5158	9.4	0.693	5158	18.0	0	2.80	0.60	0.940	3.57	3.63	14014	59448
72.40	20.40	1.05	1.006	166.024	3.91	7	59884					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	2.90	0.62	0.945	3.59	3.64	14850	74734
72.50	20.50	1.05	1.007	155.98	3.92	7	60350					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	3.00	0.64	0.949	3.61	3.66	15691	76041
72.60	20.60	1.06	1.008	165.936	3.92	7	60817					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	3.10	0.66	0.952	3.62	3.67	16534	77351
72.70	20.70	1.06	1.008	165.892	3.92	7	61285					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	3.20	0.68	0.955	3.63	3.68	17395	78681
72.80	20.80	1.07	1.009	165.848	3.93	7	61756					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	3.30	0.70	0.958	3.64	3.70	18274	80030
72.90	20.90	1.07	1.010	165.804	3.93	7	62227					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	3.40	0.72	0.961	3.65	3.71	19171	81398
73.00	21.00	1.08	1.010	155.76	3.93	7	62700					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	3.50	0.75	0.965	3.67	3.72	20106	82806
73.10	21.10	1.08	1.011	165.716	3.93	7	63174					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	3.60	0.77	0.969	3.68	3.74	21061	84236
73.20	21.20	1.09	1.012	165.672	3.94	7	63650					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	3.70	0.79	0.972	3.69	3.75	22013	85663
73.30	21.30	1.09	1.012	165.628	3.94	7	64127					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	3.80	0.81	0.975	3.71	3.76	22982	87109
73.40	21.40	1.10	1.013	165.584	3.94	7	64606					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	3.90	0.83	0.978	3.72	3.77	23968	88574
73.50	21.50	1.10	1.014	155.54	3.94	7	65086					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	4.00	0.85	0.981	3.73	3.78	24973	90058
73.60	21.60	1.11	1.014	165.496	3.95	7	65567					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	4.10	0.87	0.985	3.74	3.80	26020	91588
73.70	21.70	1.11	1.015	165.452	3.95	7	66050					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	4.20	0.90	0.988	3.75	3.81	27060	93111
73.80	21.80	1.12	1.016	165.408	3.95	7	66534					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	4.30	0.92	0.991	3.77	3.82	28118	94652
74.00	22.00	1.13	1.018	155.32	3.96	7	67539					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	4.50	0.96	0.995	3.78	3.84	30224	97763
74.20	22.20	1.14	1.019	165.232	3.96	7	68502					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	4.70	1.00	1.000	3.80	3.86	32423	100925
74.30	22.30	1.14	1.019	165.188	3.97	7	68986					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	4.80	1.02	1.003	3.81	3.87	33563	102549
74.50	22.50	1.15	1.020	165.1	3.97	7	69957					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	5.00	1.07	1.009	3.83	3.89	35896	105853
75.00	23.00	1.18	1.023	154.88	3.98	7	72407					18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	18.0	0.745	0	18.0	0.745	0	18.0	0	5.50	1.17	1.021	3.88	3.94	41906	114313
76.00	24.00	1.23	1.029	0	4.00	0	0	3.420	0.142	0.755	18.12	18.0	10420	18.0	0.745	10420	18.0	0.745	10420	18.0	10420	18.0	0.745	10420	18.0	0.745	10420	18.0	10420	6.50	1.39	1.048	3.98	4.04	55263	128201
77.00	25.00	1.28	1.035	0	4.03	0	0	3.420	0.137	0.758	18.96	18.0	10761	18.0	0.745	10761	18.0	0.745	10761	18.0	10761	18.0	0.745	10761	18.0	0.745	10761	18.0	10761	7.50	1.60	1.070	4.07	4.13	69933	145263

NOTES:

"Gate opening" is the height from the bottom of the gate to the crest of the gate concrete sill.

[1] Handbook of Applied Hydraulics, p. 20-2, Fig.1

[2] Obtained from "Design of Small Dams", Bureau of Reclamation

[3] Obtained from "Hydraulic Design of Spillways", 1990, U.S. Army Corps of Engineers, Plate 3-6

[4] Obtained from "Hydraulic Design of Spillways", 1990, U.S. Army Corps of Engineers, Plate 3-11

[5] Design of Small Dams, p 371, Figure 9-24. Discharge coefficients for other than the design head. 288-D-2410.

[6] See lookup table on "Gate Position" sheet

[7] Handbook of Applied Hydraulics, p 20-6, Table 1

[8] See lookup table on "Upper Nappe Coord." sheet

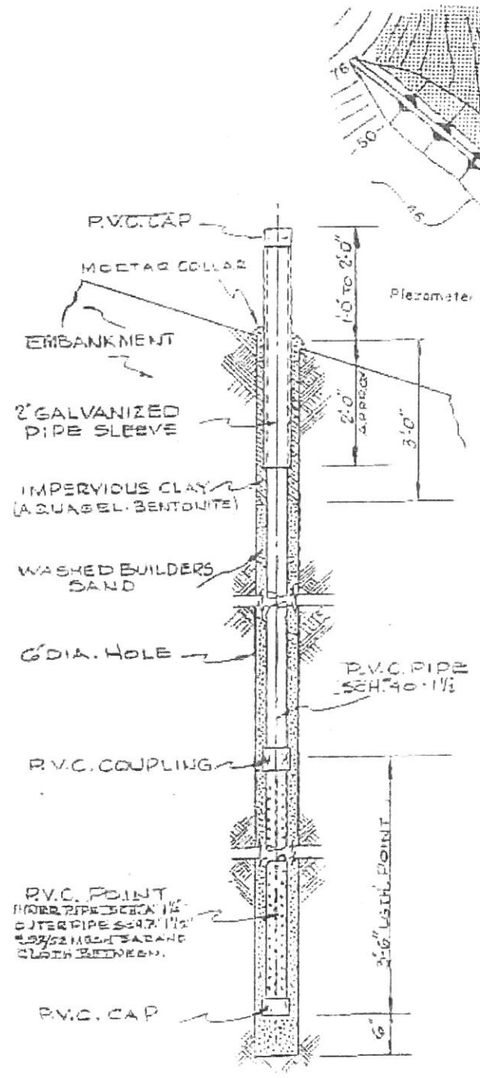
[9] Computed using Equation 6-1, "Hydraulic Design of Spillways", 1990, U.S. Army Corps of Engineers

[10] For transition between the weir flow to orifice flow see "Hydraulic Design of Spillway", 1965, U.S. Army Corps of Engineers

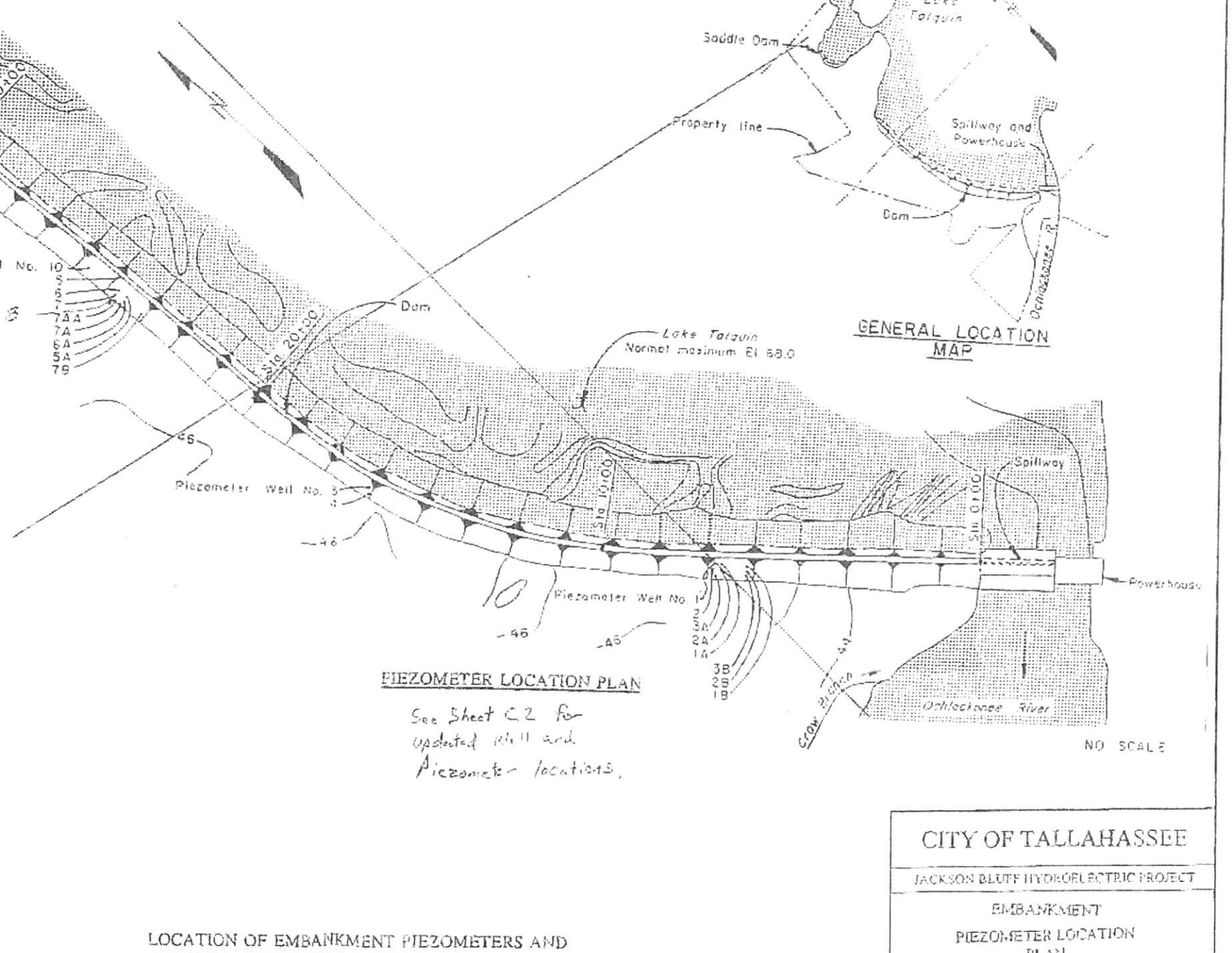
[11] Computed using Plate 6-1 of "Hydarulic Design of Spillways" bu USACE (see Orifice Cd Lookup tab)

APPENDIX D

Instrumentation Monitoring Data Plots



TYPICAL PIEZOMETER CONSTRUCTION DETAIL



PIEZOMETER LOCATION PLAN

See Sheet C2 for updated Well and Piezometer locations.

LOCATION OF EMBANKMENT PIEZOMETERS AND TYPICAL PIEZOMETER INSTALLATION DETAILS

CITY OF TALLAHASSEE

JACKSON BLUFF HYDROELECTRIC PROJECT

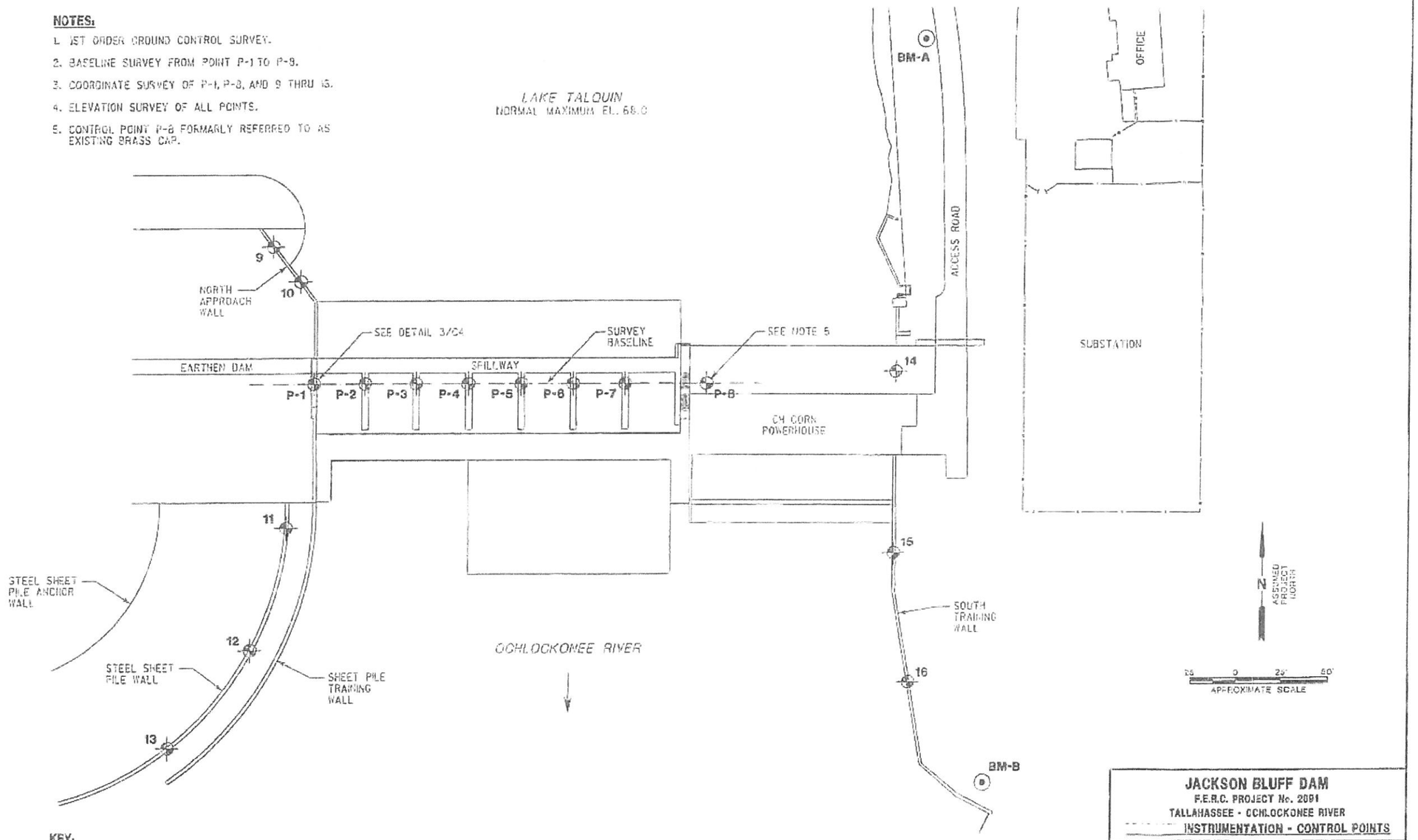
EMBANKMENT

PIEZOMETER LOCATION PLAN

RAYTHEON INFRASTRUCTURE, INC. | EXHIBIT V-1

NOTES:

1. 1ST ORDER GROUND CONTROL SURVEY.
2. BASELINE SURVEY FROM POINT P-1 TO P-8.
3. COORDINATE SURVEY OF P-1, P-8, AND 9 THRU 16.
4. ELEVATION SURVEY OF ALL POINTS.
5. CONTROL POINT P-8 FORMARLY REFERRED TO AS EXISTING BRASS CAP.



KEY:

- (B) BM = BENCHMARK (SEE DETAIL 1/C4)
- ⊕ CONTROL POINT (SEE DETAIL 2/C4)

JACKSON BLUFF DAM

F.E.R.C. PROJECT No. 2091

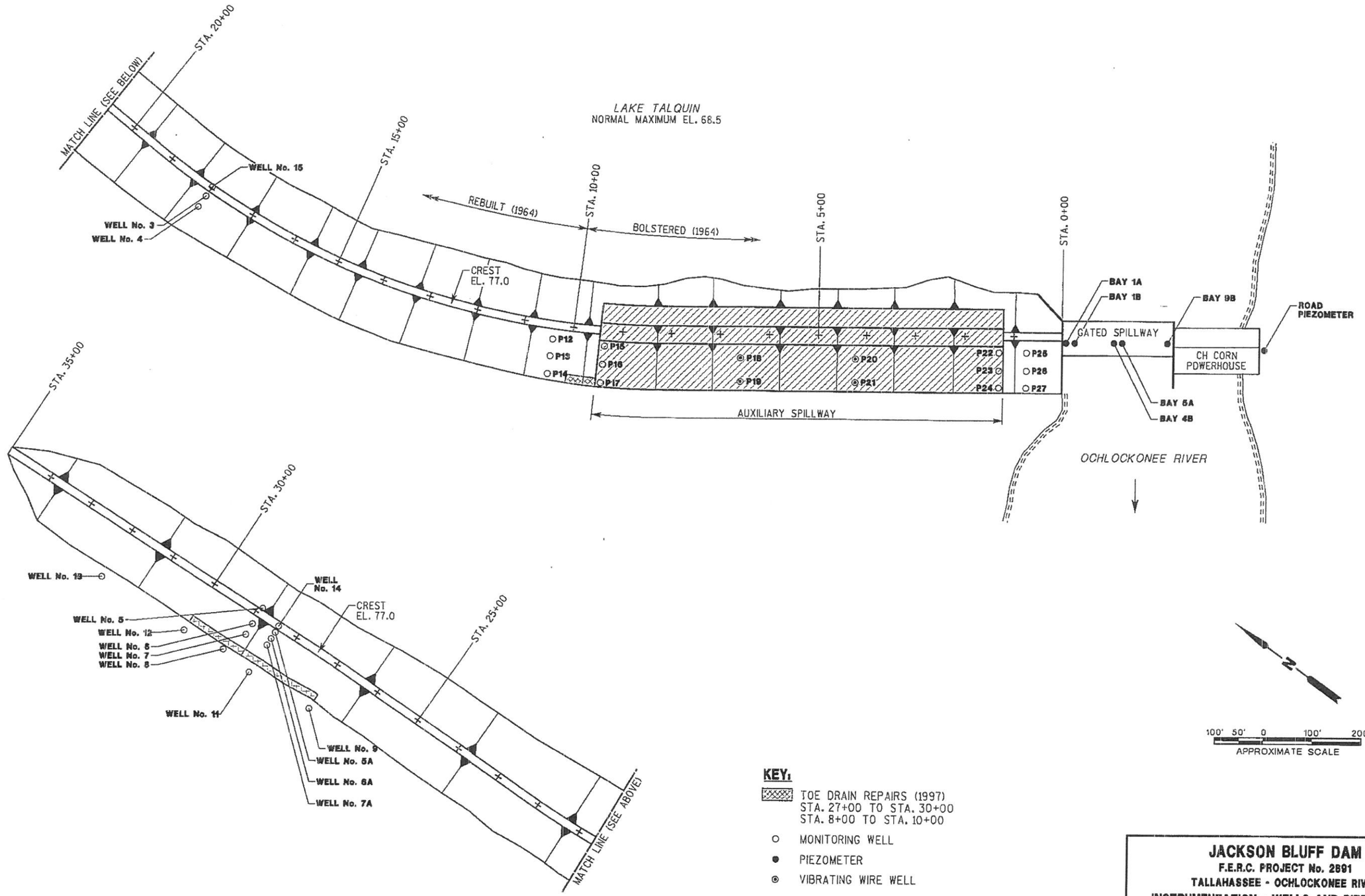
TALLAHASSEE - OCHLOCKONEE RIVER

INSTRUMENTATION - CONTROL POINTS

NO.	REVISION	DATE
-	-	-
-	-	-
-	-	-
-	-	-

MEAD & HUNT

DATE	SCALE	DRAWN/CHECKED	M&H JOB NO.	SHEET NO.
JANUARY 2024	AS SHOWN	DMS/RAH	5563-044	C1



KEY:

- TOE DRAIN REPAIRS (1997)
  STA. 27+00 TO STA. 30+00
  STA. 8+00 TO STA. 10+00
- ○ MONITORING WELL
- ● PIEZOMETER
- ◉ VIBRATING WIRE WELL

JACKSON BLUFF DAM

F.E.R.C. PROJECT No. 2891

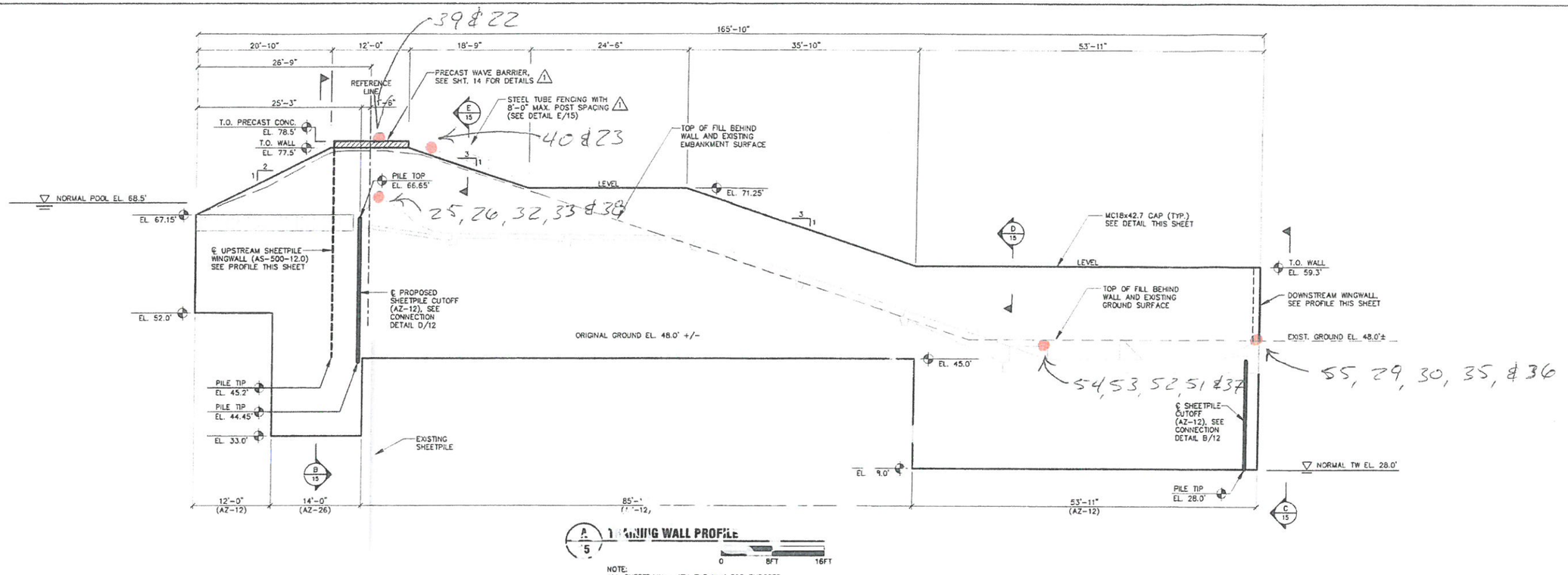
TALLAHASSEE - OCHLOCKONEE RIVER

INSTRUMENTATION - WELLS AND PIEZOMETERS

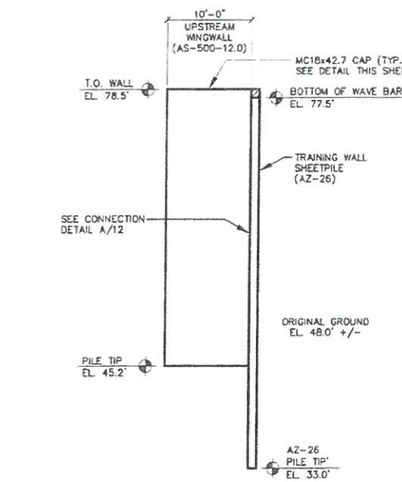
NO.	REVISION	DATE
-	-	-
-	-	-
-	-	-
-	-	-
-	-	-

Mead & Hunt, Inc.
6501 Watts Road
Madison, WI 53719
608.273.6380
fax: 608.273.6391
www.meadhunt.com

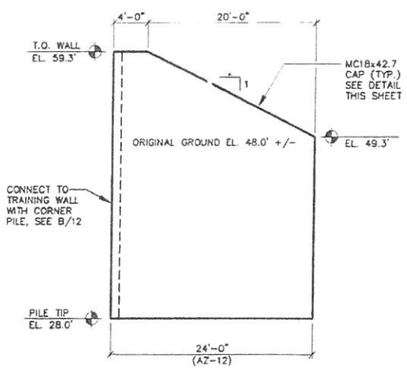
DATE	SCALE	DRAWN/CHECKED	M&H JOB NO.	SHEET NO.
JUNE 2012	AS SHOWN	DNS/MHK	20147-00-10002	C2



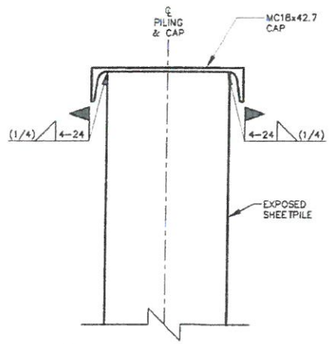
NOTE:
ALL SHEETPILING WITH THE PILE TOP EXPOSED ABOVE GRADE TO BE CAPPED WITH MC18x42.7 CHANNEL. SEE DETAIL THIS SHEET.



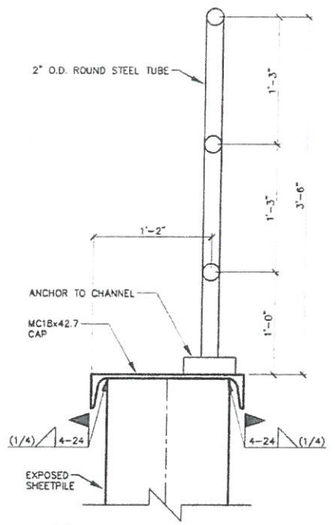
1 B/15 UPSTREAM WINGWALL PROFILE
TYP. BOTH ENDS
0 8FT 16FT



C/15 DOWNSTREAM WINGWALL PROFILE
TYP. BOTH ENDS
0 8FT 16FT

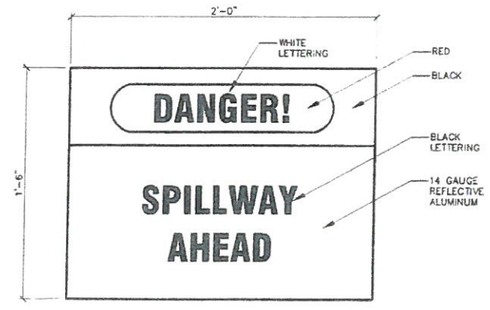


D/15 SHEETPILE CAP DETAIL
TYP. BOTH ENDS
NOT TO SCALE



1 E/15 STEEL TUBE FENCE DETAIL
TYP. BOTH ENDS
NOT TO SCALE

NOTE:
SUBMIT SHOP DRAWINGS SHOWING CONNECTIONS.



1 F/15 WARNING SIGN DETAIL
NOT TO SCALE

NOTE:
CONNECT TO HANDRAIL ALONG CENTER OF CREST AT EACH SIDE OF AUXILIARY SPILLWAY WITH STAINLESS STEEL HARDWARE.

		DATE	NO.	BY	REVISIONS	DATE
FIELD BOOK	-	-	-	-	ISSUED FOR FERC REVIEW	8/30/07
DRAWN BY	DMS	-	-	-	ISSUED FOR BID	4/16/08
SURVEYED BY	-	-	1	DMS	REVISED FOR BID	10/6/08
CHECKED BY	KAH	-	-	DMS	FINAL FOR BID (FERC REVIEW - NOT FOR CONSTRUCTION)	3/27/09
SCALE:	--	-	-	-	-	-

City of Tallahassee
Your Own Utilities℠

MEAD & HUNT ENGINEERS ARCHITECTS SCIENTISTS PLANNERS
6501 Watts Road, Madison, Wisconsin 53719-2700
608-273-6380 Fax: 608-273-6391
Certificate of Authorization No. 2670

PROJECT
JACKSON BLUFF HYDRO PROJECT
F.E.R.C. PROJECT No. 2891
AUXILIARY SPILLWAY

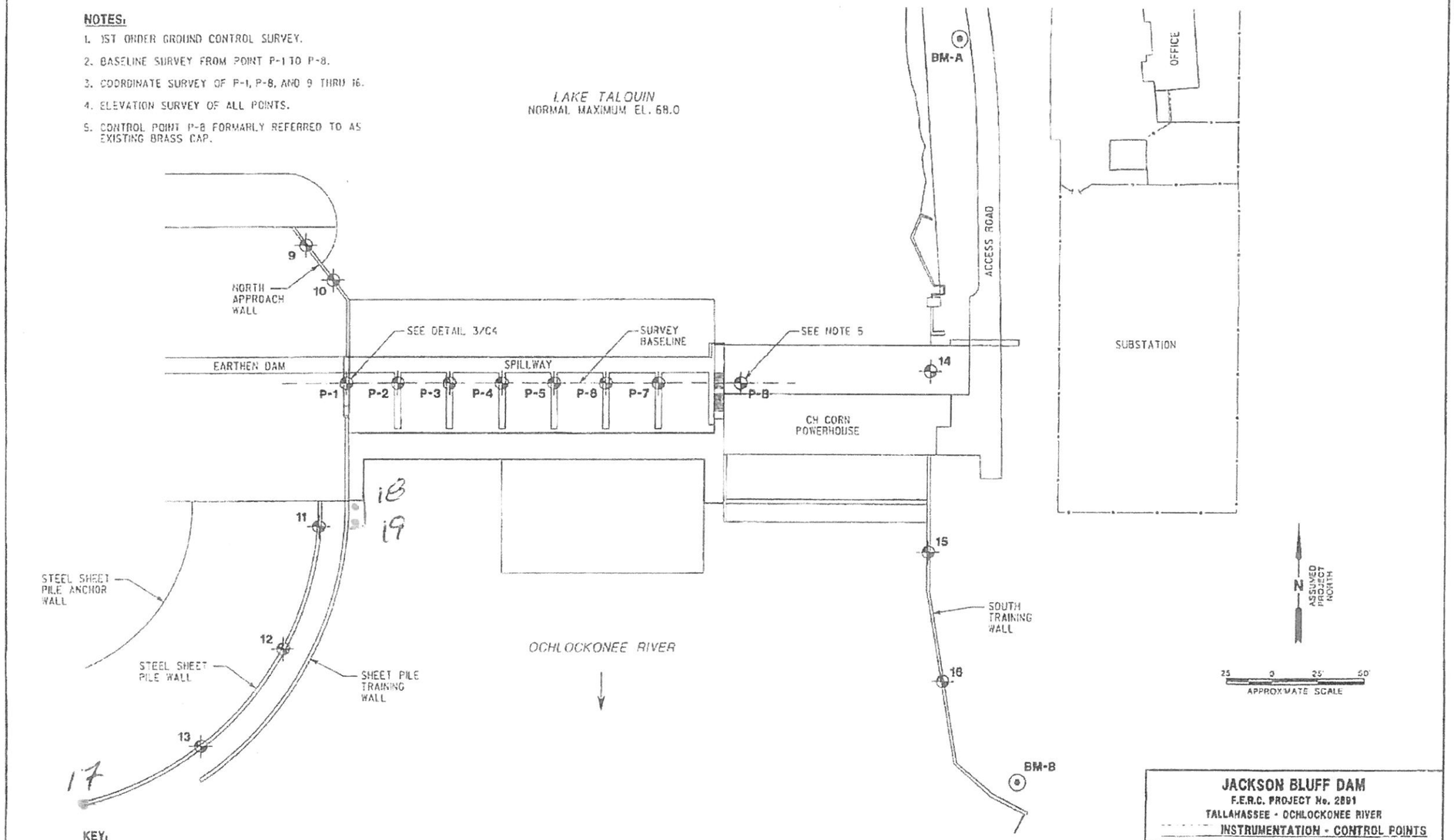
SHEET TITLE
SHEETPILE TRAINING WALL AND WINGWALL PROFILES

FILE PATH X:\20147-00\07001\TECH\CADD\16529
FILE NO.
PROJECT NO. 20147-00-07001

SHEET 15 OF 44

NOTES:

1. 1ST ORDER GROUND CONTROL SURVEY.
2. BASELINE SURVEY FROM POINT P-1 TO P-8.
3. COORDINATE SURVEY OF P-1, P-8, AND 9 THRU 16.
4. ELEVATION SURVEY OF ALL POINTS.
5. CONTROL POINT P-8 FORMARLY REFERRED TO AS EXISTING BRASS CAP.



KEY:

⊙ BM = BENCHMARK (SEE DETAIL 1/C4)

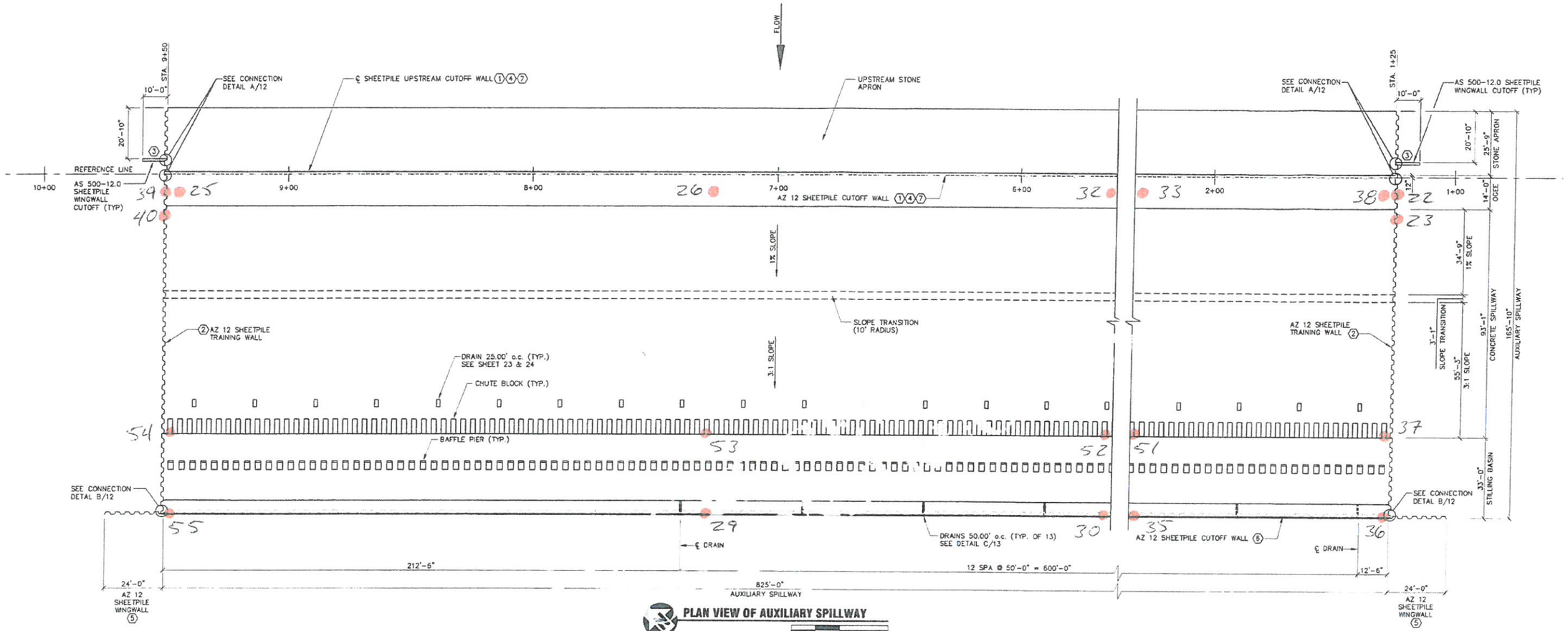
⊕ CONTROL POINT (SEE DETAIL 2/C4)

JACKSON BLUFF DAM
F.E.R.C. PROJECT No. 2891
TALLAHASSEE - OCHLOCKONEE RIVER
INSTRUMENTATION - CONTROL POINTS

NO.	REVISION	DATE
-	-	-
-	-	-
-	-	-
-	-	-

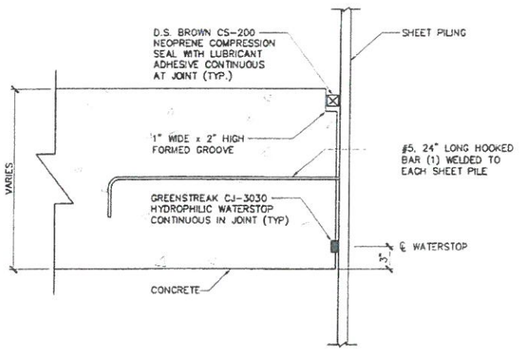
MEAD & HUNT ENGINEERS ARCHITECTS ENVIRONMENTAL SCIENTISTS PLANNERS

DATE	SCALE	DRAWN/CHECKED	M&H JOB NO.	SHEET NO.
JANUARY 2004	AS SHOWN	GMS/RAH	5383-04A	C1



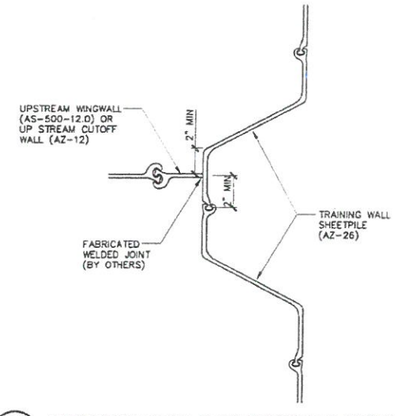
PLAN VIEW OF AUXILIARY SPILLWAY

0 20FT 40FT



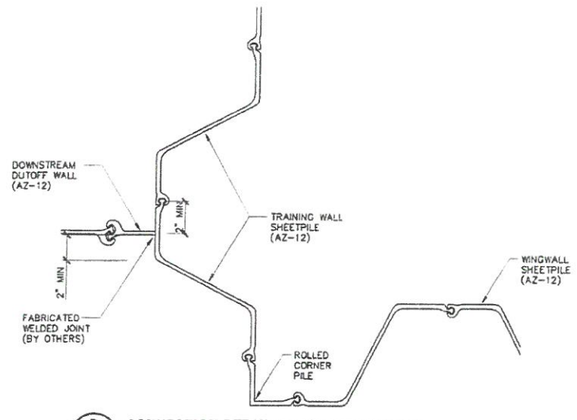
SHEETPILE WALL-CHUTE JOINT DETAIL

TYPICAL AT OGEE, SPILLWAY SLABS AND STILLING BASIN



A / 12 CONNECTION DETAIL @ U/S WINGWALL & CUTOFF WALL

NOT TO SCALE



B / 12 CONNECTION DETAIL @ D/S CUTOFF WALL

NOT TO SCALE

SHEETPILE SEQUENCE	CREST SEQUENCE TASK REFERENCE SEE SHEET 11
(1) UPSTREAM CUTOFF, INITIAL DRIVING	- 1A
(2) TRAINING WALLS	- 1B
(3) UPSTREAM WING CUTOFFS	- 1C
(4) UPSTREAM CUTOFF, SECONDARY DRIVING	- 2B
(5) BASIN TRAINING WINGWALLS	- 2D
(6) DOWNSTREAM CUTOFF	- 2D
(7) UPSTREAM CUTOFF, FINISH DRIVING	- 3A

NOTE:
SEE SPECIFICATIONS AND SHEET 11 FOR ADDITIONAL SEQUENCING INFORMATION.

SEE SHEET 14 & 15 FOR ADDITIONAL SHEET PILING INFORMATION.

		DATE	NO.	BY	REVISIONS	DATE
FIELD BOOK	-	-	-	-	ISSUED FOR FERC REVIEW	8/30/07
DRAWN BY	DMS	-	-	-	ISSUED FOR BID	4/16/08
SURVEYED BY	-	-	1	DMS	REVISED FOR BID	10/6/08
CHECKED BY	KAH	-	-	DMS	FINAL FOR BID (FERC REVIEW - NOT FOR CONSTRUCTION)	3/27/09
SCALE	--	-	-	-	-	-

City of Tallahassee
Your Own Utilities℠

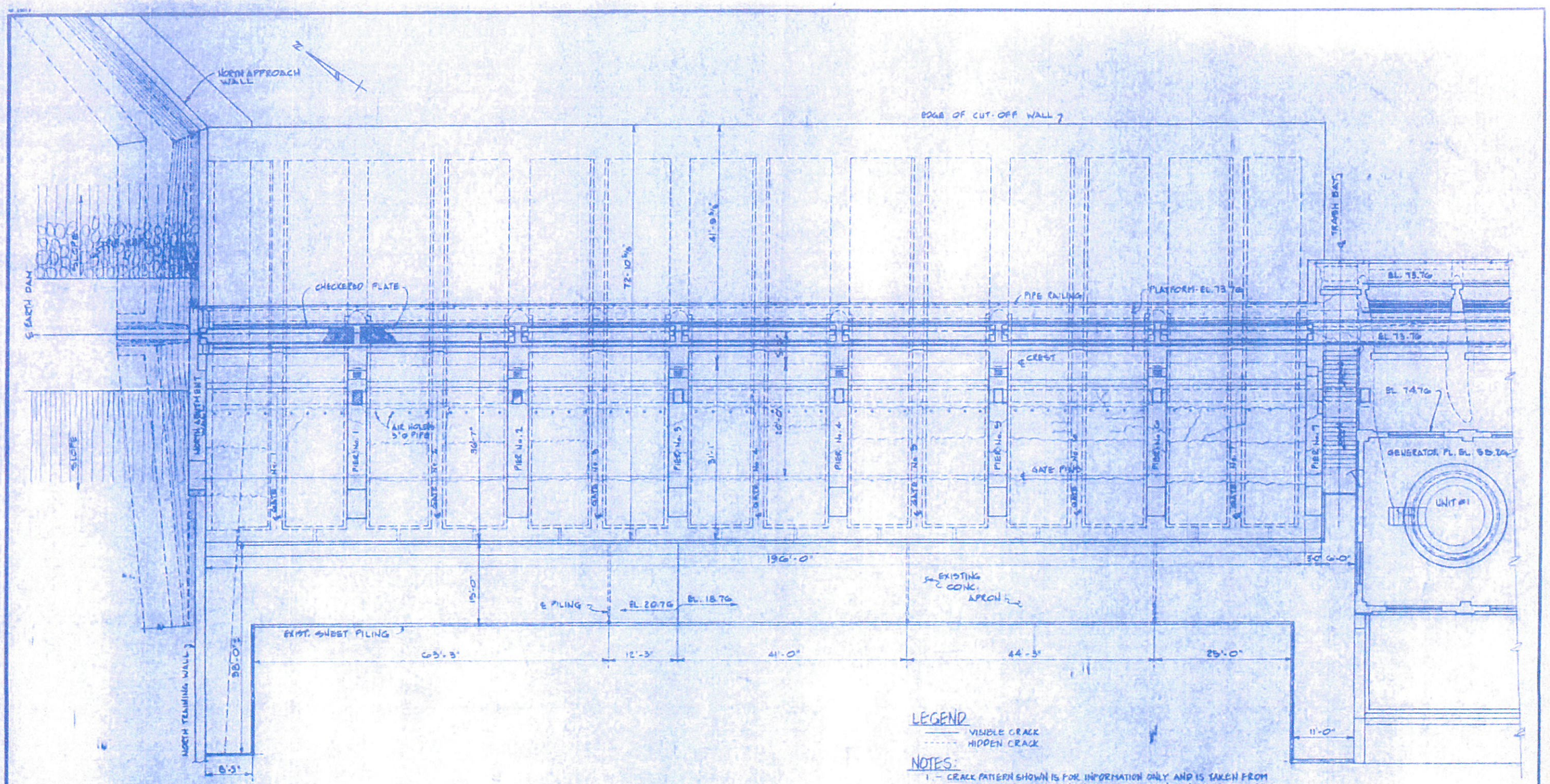
MEAD & HUNT
ENGINEERS ARCHITECTS SCIENTISTS PLANNERS
6501 Watts Road, Madison, Wisconsin 53719-2700
608-273-6380 Fax: 608-273-6391
Certificate of Authorization No. 26730

PROJECT
JACKSON BLUFF HYDRO PROJECT
F.E.R.C. PROJECT No. 2891
AUXILIARY SPILLWAY

SHEET TITLE
PLAN OF SPILLWAY & SHEET PILE SEQUENCE

FILE PATH X:\20147-00\07001\TECH\CADD\10529
FILE NO.
PROJECT NO. 20147-00-07001

SHEET 12 OF 44



LEGEND

——— VISIBLE CRACK
- - - - - - HIDDEN CRACK

NOTES:

1. – CRACK PATTERN SHOWN IS FOR INFORMATION ONLY AND IS TAKEN FROM PREVIOUS INSPECTIONS AND REPORTS, COVERING ONLY EXPOSED AREAS.
2. – PATCHING AREAS ARE NOT SHOWN.
3. – CONTRACTOR SHALL INSPECT ALL CRACKS AND SPALLED AREAS PREVIOUS TO ANY REPAIR WORK IS STARTED. ONLY QUALIFIED PERSONNEL AND EQUIPMENT FOR THIS TYPE OF WORK WILL BE ACCEPTED. SEE SPECS.

EXISTING SPILLWAY – PLAN

⅛" = 1'-0"

(TAINTER GATES NOT SHOWN FOR CLARITY)

DIVISION II

WILLIAM M. BISHOP CONSULTING ENGINEERS, INC.	TOMLINSON and DOUGHERTY Consulting Engineers, Inc.	MAGUIRE/BEEBE Architects • Engineers • Planners Spartanburg South Carolina	SCALE AS NOTED DATE JULY, 1982 DRAWN BY DRG CHECKED BY MA APPROVED BY JRB

CITY OF TALLAHASSEE
FLORIDA
JACKSON BLUFF HYDROELECTRIC PLANT
F.E.R.C. PROJECT NO. 2891

SPILLWAY DAM

PLAN VIEW

DRAWING NO. C 12 OF 19

SEQUENCE NO. 67 OF 86

Surveyor's Report
Jackson Bluff Dam Survey
Meridian Project No. 20838.07
Completed August 11, 2014

The intent of this report is to describe the methods utilized to assign horizontal and vertical values to the control points set in the main dam structure and the auxiliary dam structure at the Jackson Bluff Dam. The field notes for this project can be found in Meridian Surveying and Mapping Inc. fieldbook #14-05, pages 47, and 58 – 71, dated 7/14/2014 thru 7/31/2014.

Published Control Points –
Prior to beginning work on the project site, Meridian personnel used our gps equipment to verify published control points in the area of the project site. The following control points were checked :
Horizontal - FDOT D86, FDOT 55-01-C01G
Both points were found to be within 0.14 or less of published values in a Northeasterly direction. Adjustment was made to agree with published values.

Vertical Control – FDOT 55-09-D01V, FDOT 55-09-D02V
Both points found to be within 0.15 of published elevations. Adjustment made to agree with said published values.

Based upon the horizontal and vertical control points listed above and the adjustments made to agree with the published values, the following on-site control points were set or verified as depicted on page 62 of fieldbook 14-05:

GPS 800	N. 503723.97	E. 1920307.49	El. 60.265	Set Pk in Bridge
GPS 801	N. 508475.51	E. 1921130.61	El. 83.40	Set Iron Bar
GPS 802	N. 504867.06	E. 1922480.10	El. 92.50	COT BM D
GPS 804	N. 504979.14	E. 1922529.28	El. 81.75	COT BM A
GPS 805	N. 504719.74	E. 1922278.28	El. 85.185	COT BM B

Based upon the on-site control points listed above, a closed triangle was traversed with conventional survey equipment (non GPS) to verify the positions of the control points This traverse is depicted on page 47 of fieldbook 14-05. Once the control points were verified, the points installed on both the dam and the auxiliary dam were located from either 2 or three different control points depending on the options available with line-of-site observations. Each dam or aux. dam point was located with repetitions of four observations from 2 or 3 separate control points. The multiple values assigned to each dam / aux. dam point were found to be within acceptable tolerances, so these values were averaged to assign final values listed in the attached spreadsheet.

Comparison to Previous Data:
Upon completion of the survey effort described above, it was noted that the horizontal and vertical values assigned to the dam / aux. dam points did not agree with the earlier positions by approximately 0.9 feet horizontally and 0.5 feet vertically. The amount of this error was considered too high by Meridian Surveying and Mapping Inc. so a second survey effort was begun using non-gps, conventional surveying equipment to verify either the previous data or the newly collected gps data.

Meridian Surveying personnel ran a closed benchrun from the Jackson Bluff Dam on-site control points back to the FDOT vertical control points as recorded in fieldbook 14-05, pages 58 – 61, pages 63-65 and page 68. The most vertical error found between this benchrun and the Meridian vertical values assigned to the on-site control points and dam / aux. dam points was 0.02 feet. The vertical values collected by Meridian were held and are listed in the attached spreadsheet.

Meridian Surveying personnel also ran a closed traverse from the on-site horizontal control points back to the published FDOT control points. (See page 68, Fieldbook 14-05). This traverse closed within 0.11 feet, thereby verifying the horizontal values established by Meridian Surveying Inc. for the on-site control and the dam / aux. dam points. The horizontal values collected by Meridian were held and are listed in the attached spreadsheet and also on pages 70 – 71 of fieldbook 14-05.

This report is intended to provide a brief overview of the methods used to complete this survey project and should be considered a part of the final survey product. Any questions regarding the information contained herein should be directed to:

Steve Stinson, PSM
President
Meridian Surveying and Mapping, Inc.
3201 Shamrock St. S, Suite 101
Tallahassee, Fl. 32309
(850) 668-7641

_____ _____

Steve Stinson Date

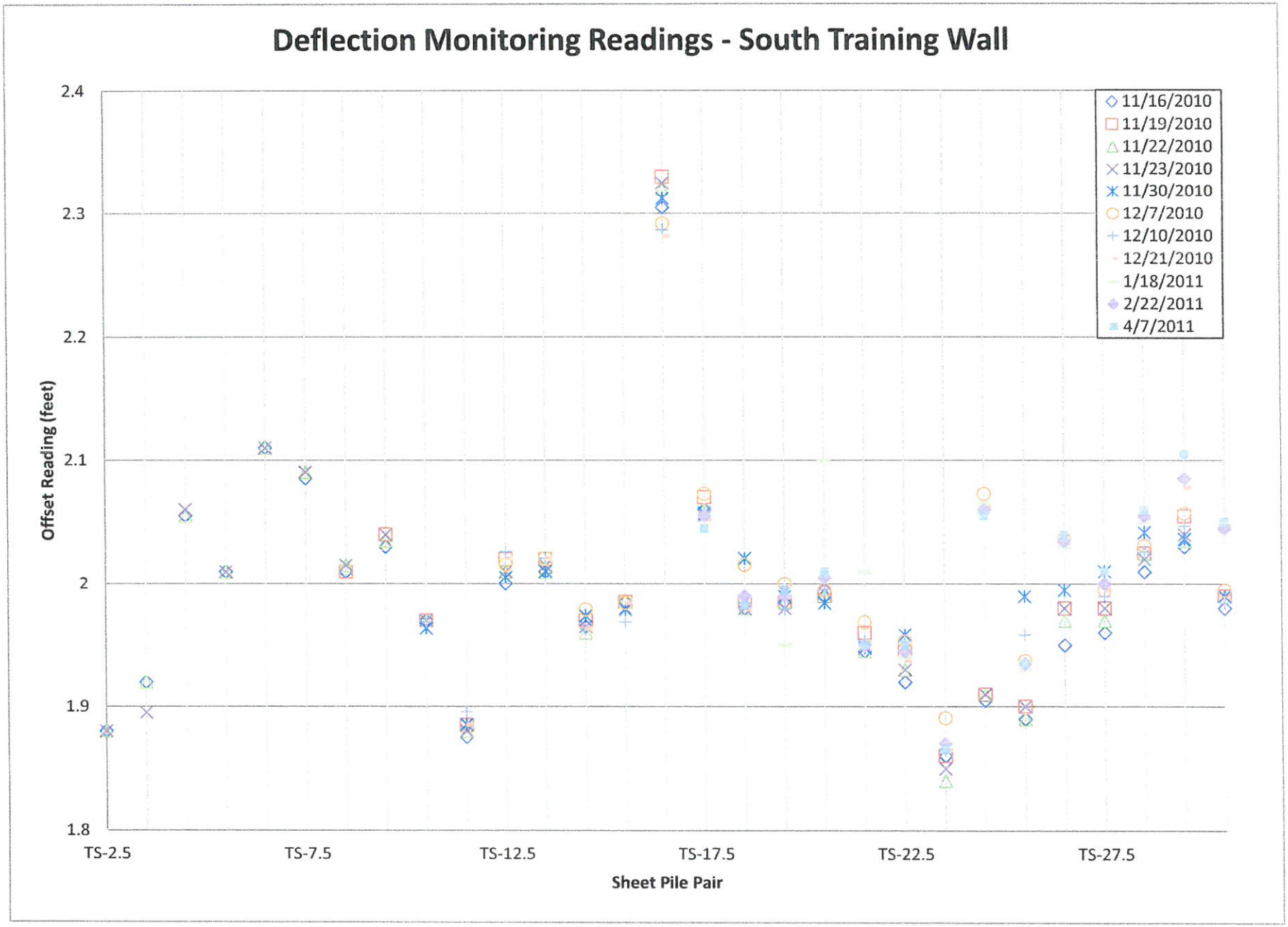
Jackson Bluff Dam Survey Point Values
Meridian Surveying Inc. / August 2014

505113.8600	1922178.3540	71.458	1
505093.2008	1922196.5510	67.190	2
505071.8618	1922215.3770	67.170	3
505050.4596	1922234.2540	67.190	4
505029.1019	1922253.0800	67.195	5
505007.7227	1922271.9410	67.175	6
504986.3266	1922290.8160	67.180	7
504957.5718	1922316.9950	74.160	8
505182.9176	1922218.5260	76.185	9
505153.1542	1922216.4700	76.195	10
505082.8572	1922128.0070	53.280	11
505048.0909	1922067.8070	53.280	12
505047.8744	1921992.6952	53.280	13
504876.1218	1922385.9502	74.190	14
504833.8093	1922338.3087	57.725	15
Point Missing			16
505090.4613	1921924.4970	53.230	17
505079.4306	1922140.1332	52.770	18
505072.7887	1922132.7215	52.740	19
505284.1814	1922050.4271	77.140	22
505275.3632	1922040.1139	75.370	23
505903.2676	1921506.3304	69.045	25
505747.5475	1921641.5032	68.935	26
505658.2186	1921536.8127	46.520	29
505501.7029	1921672.4613	46.580	30
505591.5079	1921776.4690	68.940	32
505435.9345	1921911.7853	68.960	33
505345.7549	1921807.5259	46.570	35
505190.2877	1921942.3218	46.540	36
505214.0943	1921965.4287	46.390	37
505281.6294	1922045.4605	68.930	38
505903.9564	1921505.9266	77.000	39
505899.6096	1921500.8617	75.780	40
505368.2932	1921831.6948	46.465	51
505522.6832	1921697.9506	46.420	52
505679.6025	1921561.8299	46.400	53
505833.8949	1921428.1344	46.425	54
505814.2487	1921401.6187	46.447	55

Jackson Bluff Dam Survey Point Values
Meridian Surveying Inc. / August 2014

On-Site Control

Northing	Easting	Elev.	Point
504979.1398	1922529.2820	81.750	BM A
504719.7391	1922278.2805	85.185	BM B
504867.0600	1922480.1000	92.500	BM D
505155.2249	1922119.6090	76.005	CHK TRAV PT 807
505217.9148	1921969.7240	46.400	CP 103
505372.1139	1921835.9250	46.490	CP 202
505526.5422	1921702.2480	46.395	CP 302
505683.5277	1921566.2860	46.450	CP 402
Point Missing			CP 503
508475.5100	1921130.6100	83.400	TRAV 1
503723.9700	1920307.4900	60.265	TRAV 2
505524.2353	1921800.8154	63.616	TRAV PT 806
505524.2235	1921800.8122	63.606	TRAV PT 806
505155.2168	1922119.6441	76.062	TRAV PT 807
505155.2179	1922119.6174	76.063	TRAV PT 807


Deflection Monitoring Readings - South Training Wall
2.4
2.3
2.2
2.1
2
1.9
1.8
Offset Reading (feet)
TS-2.5
TS-7.5
TS-12.5
TS-17.5
TS-22.5
TS-27.5
Sheet Pile Pair
11/16/2010
11/19/2010
11/22/2010
11/23/2010
11/30/2010
12/7/2010
12/10/2010
12/21/2010
1/18/2011
2/22/2011
4/7/2011

5. Instrumentation

A. Monitoring wells

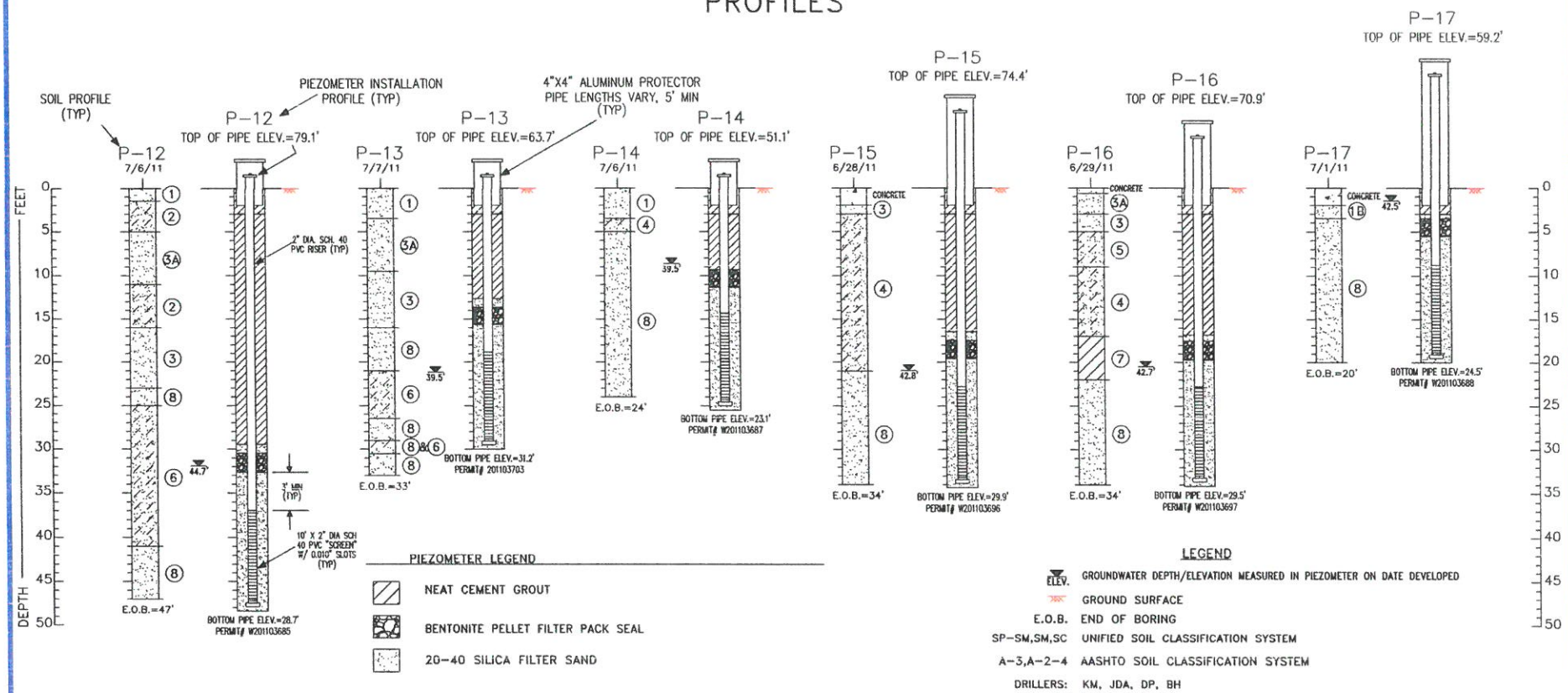
Piezometers within the vicinity of the auxiliary spillway were monitored periodically for changes. The average and standard deviation of each well's bi-monthly readings for the past 10-month period have been analyzed to compare changes in phreatic conditions against historical values. Appendix F shows the relative locations of piezometers along the auxiliary spillway as well as plot water elevations during construction. Table 2 summarizes piezometer readings during construction from July 1 through August 15, 2011.

Table 2. Piezometer Reading Comparisons

Piezometer Name	Pre-Construction 1-1-10 - 9/28/10 Bi-Monthly Readings	Pile Driving 9/28/11 - 6/3/11 Readings	Dewatering 2/1/11 - 4/15/11 Semi-Daily Readings	End of Construction 8/13/11 Readings
	Average Water Surface Elevation (ft)	*Maximum Water Surface Elevation (ft)*	*Average Water Surface Elevation (ft)*	*Water Surface Elevation (ft)*
0A	57.88	65.46	57.05	-
1A	51.82	51.68	50.27	-
1B	52.58	52.14	49.56	-
P2	63.38	67.29	63.09	63.21
P3	48.83	49.31	45.05	49.05
P4	41.12	43.63	35.02	39.96
P5	42.03	43.14	38.84	-
P6	38.81	39.14	32.46	-
P7	56.38	63.34	54.52	-
P8	42.94	42.45	(Dry)	-
P9	42.66	42.69	(Dry)	-
P10	43.77	44.82	39.88	-
P11	43.05	44.06	36.89	41.11
SP-UC3	60.50*		59.33	-
SP-UC179	46.61*		44.39	-

** Stabilized water elevation given.*

PROFILES



PIEZOMETER LEGEND

- NEAT CEMENT GROUT
- BENTONITE PELLET FILTER PACK SEAL
- 20-40 SILICA FILTER SAND

LEGEND

ELEV.	GROUNDWATER DEPTH/ELEVATION MEASURED IN PIEZOMETER ON DATE DEVELOPED
	GROUND SURFACE
E.O.B.	END OF BORING
SP-SM,SM,SC	UNIFIED SOIL CLASSIFICATION SYSTEM
A-3,A-2-4	AASHTO SOIL CLASSIFICATION SYSTEM
DRILLERS:	KM, JDA, DP, BH

SOIL LEGEND

- (1) BROWN MEDIUM TO FINE SAND W/ SILT (SP-SM; A-3)
- (1A) BROWN AND TAN MEDIUM TO FINE SAND W/ TR CLAYEY SAND (SP-SM; A-3)
- (1B) TAN GRAVEL AND LIMESTONE FRAGMENTS
- (2) BROWN AND LIGHT BROWN MEDIUM TO FINE SAND W/ CLAY (SP-SC; A-2-4)
- (2A) TAN MEDIUM TO FINE SAND W/ CLAY (SP-SC; A-2-4)
- (3) MOTTLED BROWN AND TAN SILTY FINE SAND (SM; A-2-4)
- (3A) ORANGISH-BROWN MEDIUM TO FINE SAND W/ CLAY (SP-SC; A-2-4)
- (4) GRAY AND LIGHT BROWN SILTY FINE SAND W/ CLAY (SM-SC; A-2-4)
- (5) LIGHT GRAY AND DARK GRAY MEDIUM TO FINE SAND W/ CLAY (SP-SC; A-2-4)
- (6) GRAY AND TAN CLAYEY, SILTY FINE SAND (SM-SC; A-2-4)
- (7) MOTTLED BROWN, GRAY, AND TAN SILTY, CLAYEY FINE SAND (SC; A-2-6)
- (8) TAN TO WHITE MEDIUM TO FINE SAND (SP; A-3)
- (8A) TAN TO WHITE CLAYEY MEDIUM TO FINE SAND (SC; A-2-6)

WHILE THE BORINGS ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THEIR RESPECTIVE LOCATIONS AND FOR THEIR RESPECTIVE VERTICAL REACHES, LOCAL VARIATIONS CHARACTERISTIC OF THE SUBSURFACE MATERIALS OF THE REGION ARE ANTICIPATED AND MAY BE ENCOUNTERED. THE BORING LOGS AND RELATED INFORMATION ARE BASED ON THE DRILLER'S LOGS AND VISUAL EXAMINATION OF SELECTED SAMPLES IN THE LABORATORY. THE DELINEATION BETWEEN SOIL TYPES SHOWN ON THE LOGS IS APPROXIMATE AND THE DESCRIPTION REPRESENTS OUR INTERPRETATION OF SUBSURFACE CONDITIONS AT THE DESIGNATED BORING LOCATIONS ON THE PARTICULAR DATE DRILLED.

GROUNDWATER ELEVATIONS SHOWN ON THE BORING LOGS REPRESENT GROUNDWATER SURFACES ENCOUNTERED ON THE DATES SHOWN. FLUCTUATIONS IN WATER TABLE LEVELS SHOULD BE ANTICIPATED THROUGHOUT THE YEAR. ABSENCE OF WATER SURFACE DATA ON CERTAIN BORINGS IMPLIES THAT NO GROUNDWATER DATA IS AVAILABLE, BUT DOES NOT NECESSARILY MEAN THAT GROUNDWATER WILL NOT BE ENCOUNTERED AT THESE LOCATIONS OR WITHIN THE VERTICAL REACHES OF THESE BORINGS IN THE FUTURE.

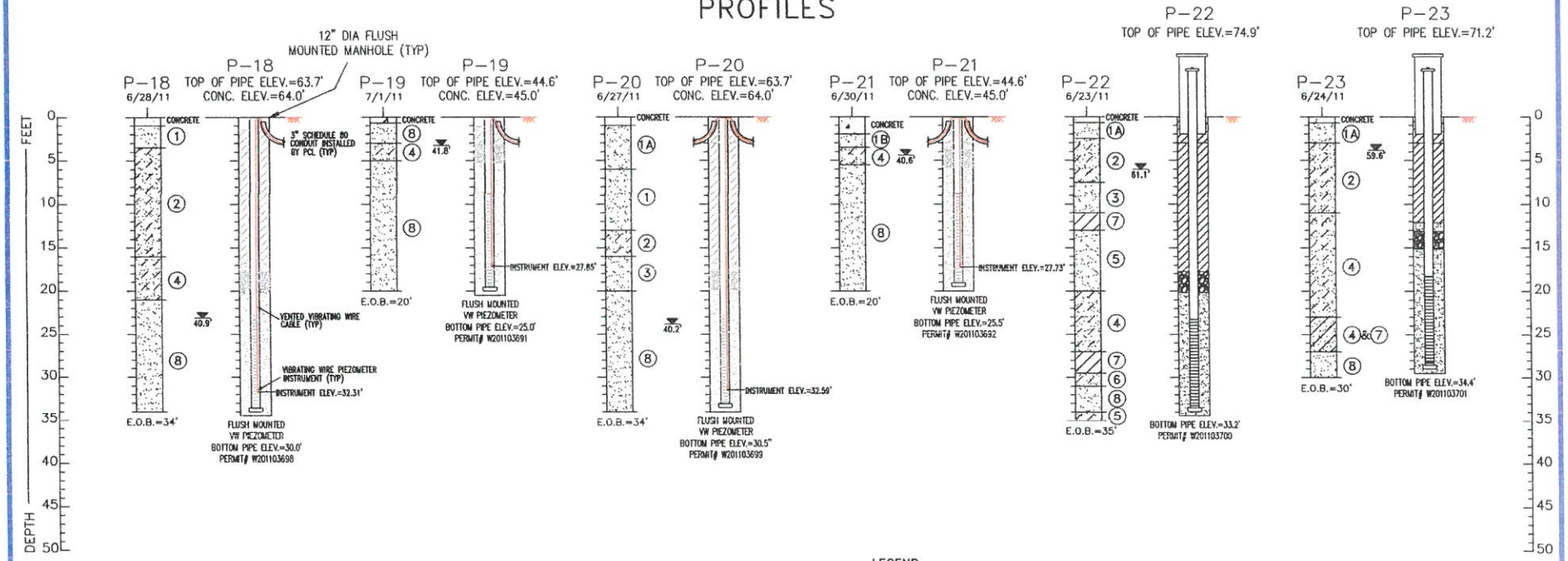
Ardaman & Associates, Inc.
3175 W. Tharpe Street
Tallahassee, Florida 32303
(850) 576-6131

SOIL BORING PROFILES AND PIEZOMETER INSTALLATION DETAILS
JACKSON BLUFF DAM
AUXILIARY SPILLWAY

8/24/11

113-10-40-1452 W.S. JORDAN, P.E.

PROFILES



PIEZOMETER LEGEND

- NEAT CEMENT GROUT
- BENTONITE PELLET FILTER PACK SEAL
- 20-40 SILICA FILTER SAND

LEGEND

- ELEV. GROUNDWATER DEPTH/ELEVATION MEASURED IN PIEZOMETER ON DATE DEVELOPED
- GROUND SURFACE
- E.O.B. END OF BORING
- SP-SM,SM,SC UNIFIED SOIL CLASSIFICATION SYSTEM
- A-3,A-2-4 AASHTO SOIL CLASSIFICATION SYSTEM
- DRILLERS: KM, JDA, DP, BH

SOIL LEGEND

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Ardaman & Associates, Inc.
3175 W. Thorpe Street
Tallahassee, Florida 32303
(850) 576-6131

SOIL BORING PROFILES AND PIEZOMETER INSTALLATION DETAILS
JACKSON BLUFF DAM
AUXILIARY SPILLWAY

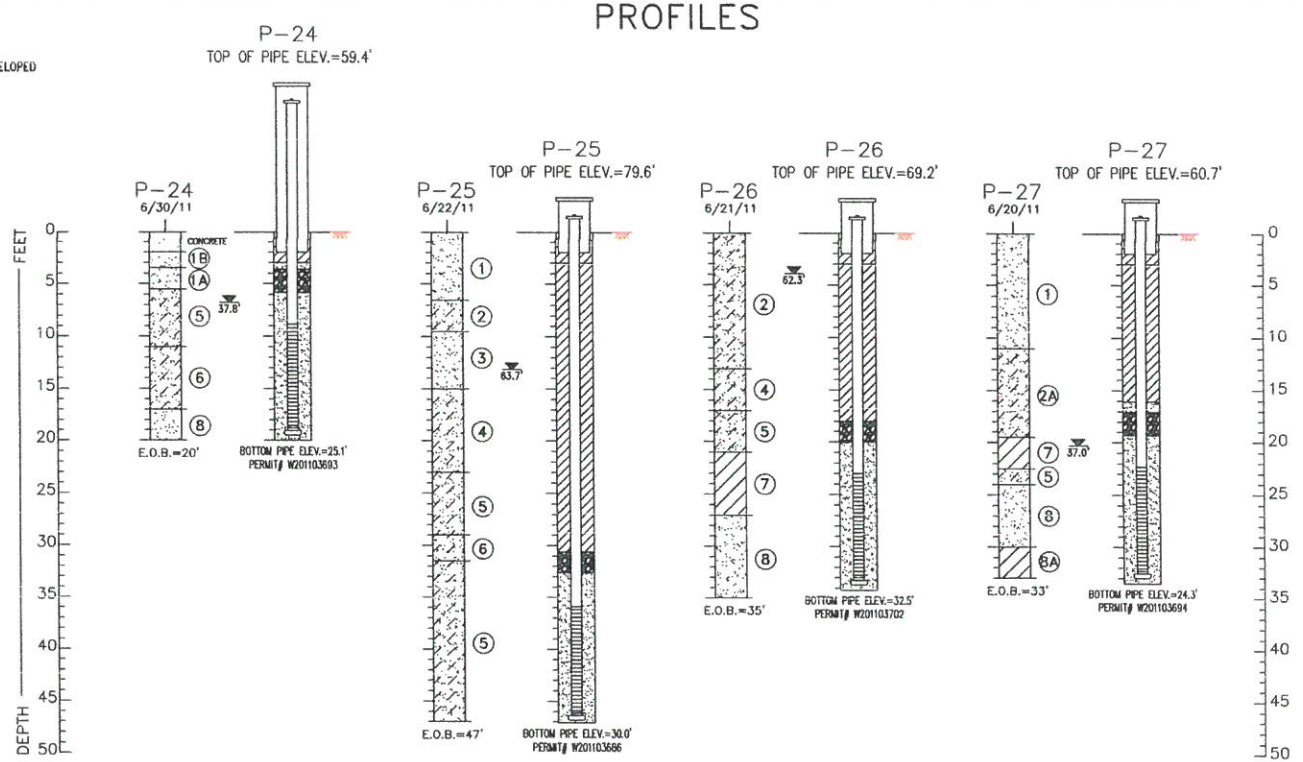
DRAWN BY: JDC | CHECKED BY: KS | DATE: 8/24/11

PROJECT: 113-10-40-1452 W.S. JORDAN, P.E. | DRAWING NO. 3

PROFILES

LEGEND

- ELEV. — GROUNDWATER DEPTH/ELEVATION MEASURED IN PIEZOMETER ON DATE DEVELOPED
- GROUND SURFACE
- E.O.B. — END OF BORING
- SP-SM,SM,SC — UNIFIED SOIL CLASSIFICATION SYSTEM
- A-3,A-2-4 — AASHTO SOIL CLASSIFICATION SYSTEM
- DRILLERS: KM, JDA, DP, BH



PIEZOMETER LEGEND

- NEAT CEMENT GROUT
- BENTONITE PELLET FILTER PACK SEAL
- 20-40 SILICA FILTER SAND

SOIL LEGEND

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- (8) TAN TO WHITE MEDIUM TO FINE SAND (SP; A-3)
- (8A) TAN TO WHITE CLAYEY MEDIUM TO FINE SAND (SC; A-2-6)

WHILE THE BORINGS ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THEIR RESPECTIVE LOCATIONS AND FOR THEIR RESPECTIVE VERTICAL REACHES, LOCAL VARIATIONS CHARACTERISTIC OF THE SUBSURFACE MATERIALS OF THE REGION ARE ANTICIPATED AND MAY BE ENCOUNTERED. THE BORING LOGS AND RELATED INFORMATION ARE BASED ON THE DRILLER'S LOGS AND VISUAL EXAMINATION OF SELECTED SAMPLES IN THE LABORATORY. THE DELINEATION BETWEEN SOIL TYPES SHOWN ON THE LOGS IS APPROXIMATE AND THE DESCRIPTION REPRESENTS OUR INTERPRETATION OF SUBSURFACE CONDITIONS AT THE DESIGNATED BORING LOCATIONS ON THE PARTICULAR DATE DRILLED.

GROUNDWATER ELEVATIONS SHOWN ON THE BORING LOGS REPRESENT GROUNDWATER SURFACES ENCOUNTERED ON THE DATES SHOWN. FLUCTUATIONS IN WATER TABLE LEVELS SHOULD BE ANTICIPATED THROUGHOUT THE YEAR. ABSENCE OF WATER SURFACE DATA ON CERTAIN BORINGS IMPLIES THAT NO GROUNDWATER DATA IS AVAILABLE, BUT DOES NOT NECESSARILY MEAN THAT GROUNDWATER WILL NOT BE ENCOUNTERED AT THESE LOCATIONS OR WITHIN THE VERTICAL REACHES OF THESE BORINGS IN THE FUTURE.

Ardaman & Associates, Inc.
3175 W. Thorpe Street
Tallahassee, Florida 32303
(850) 576-6131

SOIL BORING PROFILES AND PIEZOMETER INSTALLATION DETAILS
JACKSON BLUFF DAM
AUXILIARY SPILLWAY

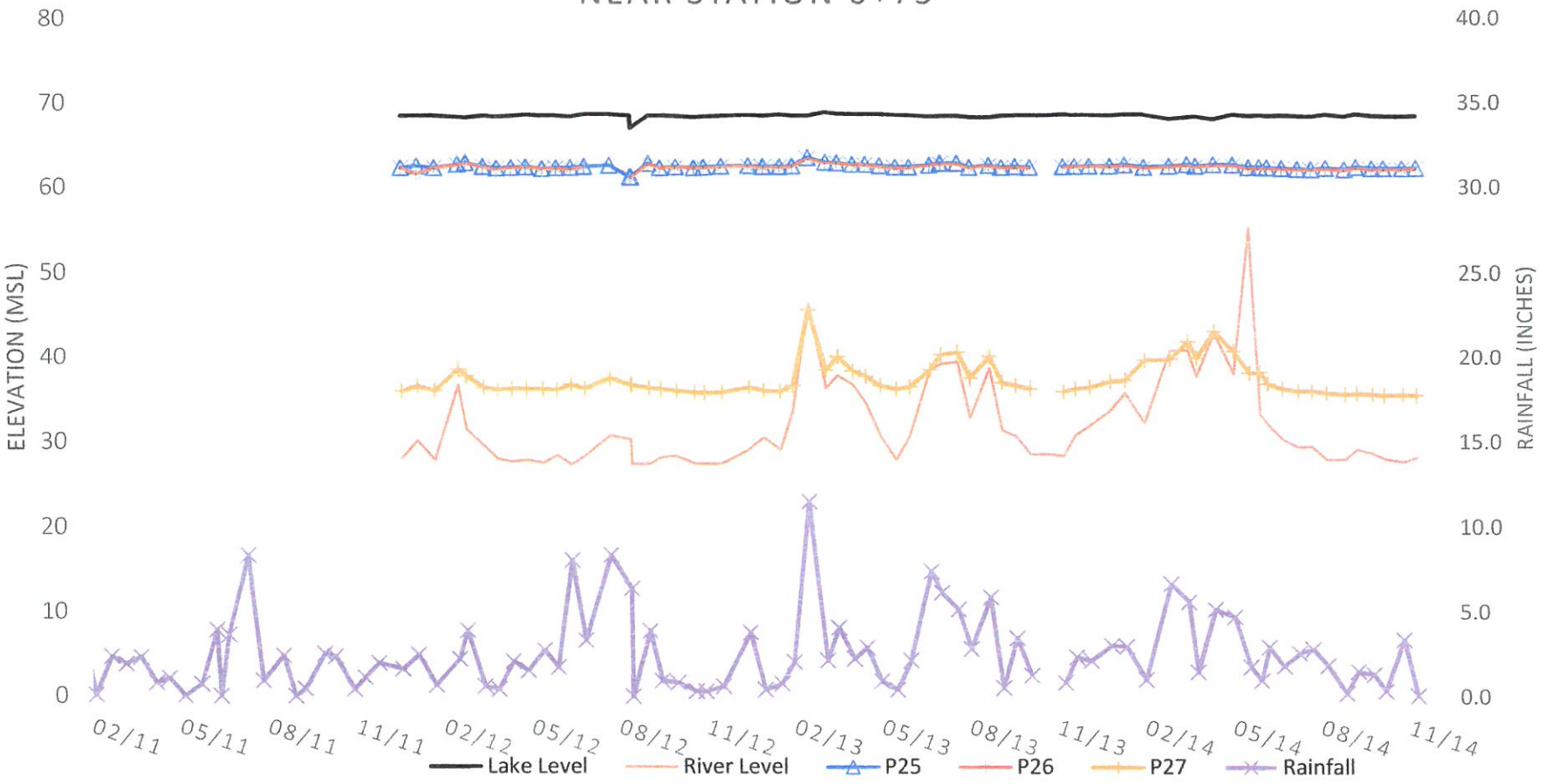
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FILE NO.: 113-10-40-1452 W.S. JORDAN, P.E. | 4

Appendix C. Monitoring Well Data

EMBANKMENT MONITORING WELL DATA

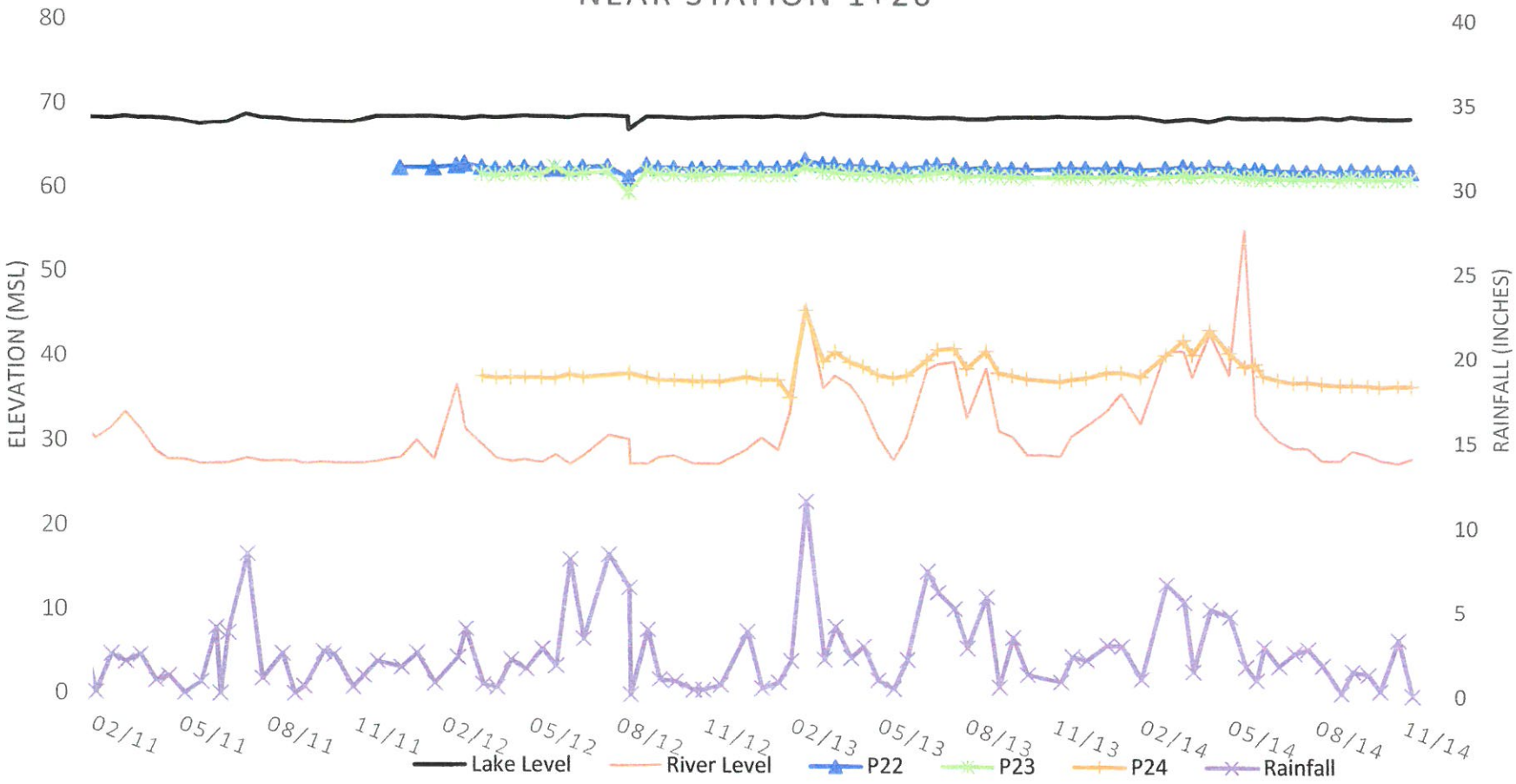
NEAR STATION 0+75



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

AUXILIARY SPILLWAY MONITORING WELL DATA

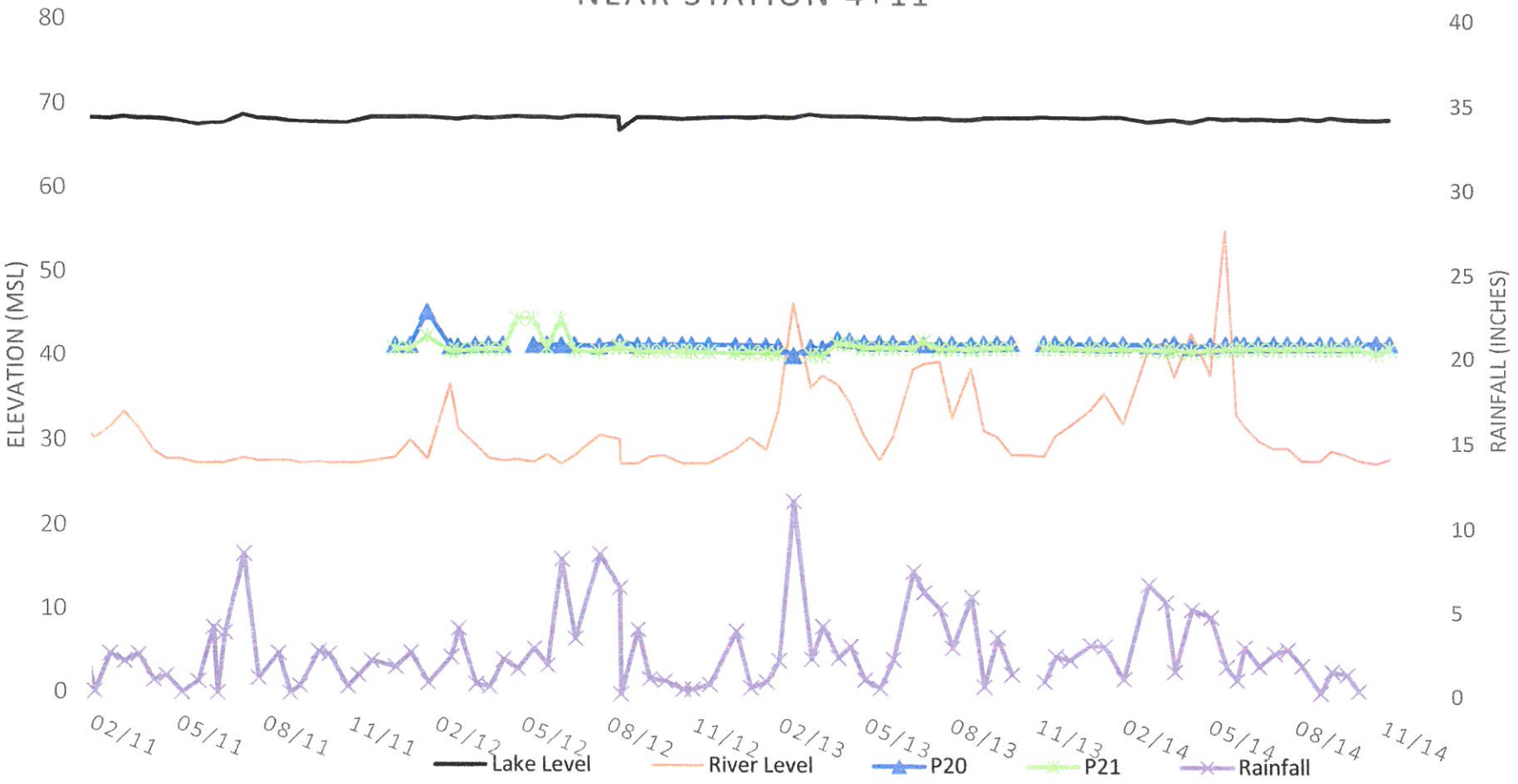
NEAR STATION 1+26



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

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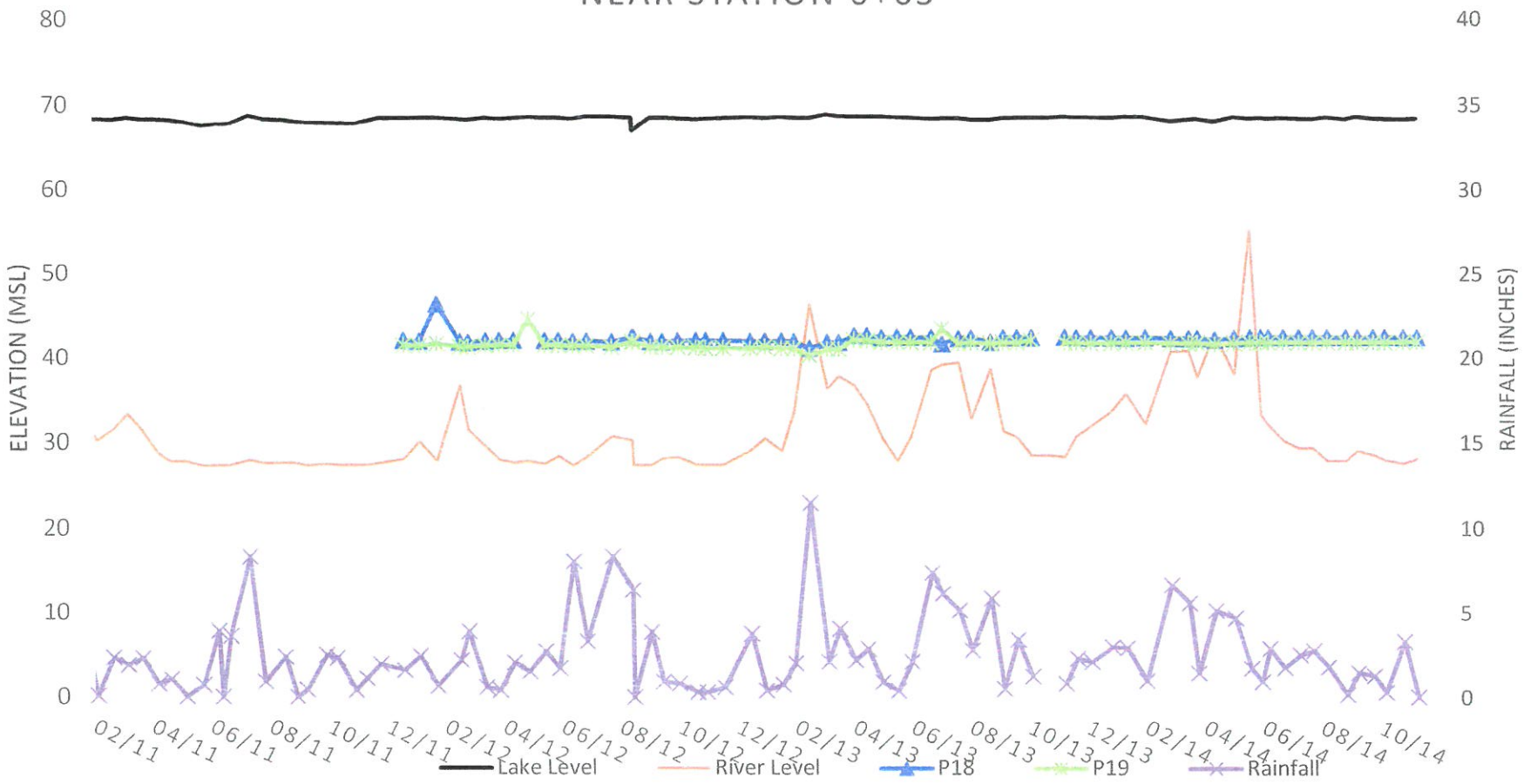
NEAR STATION 4+11



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

AUXILIARY SPILLWAY MONITORING WELL DATA

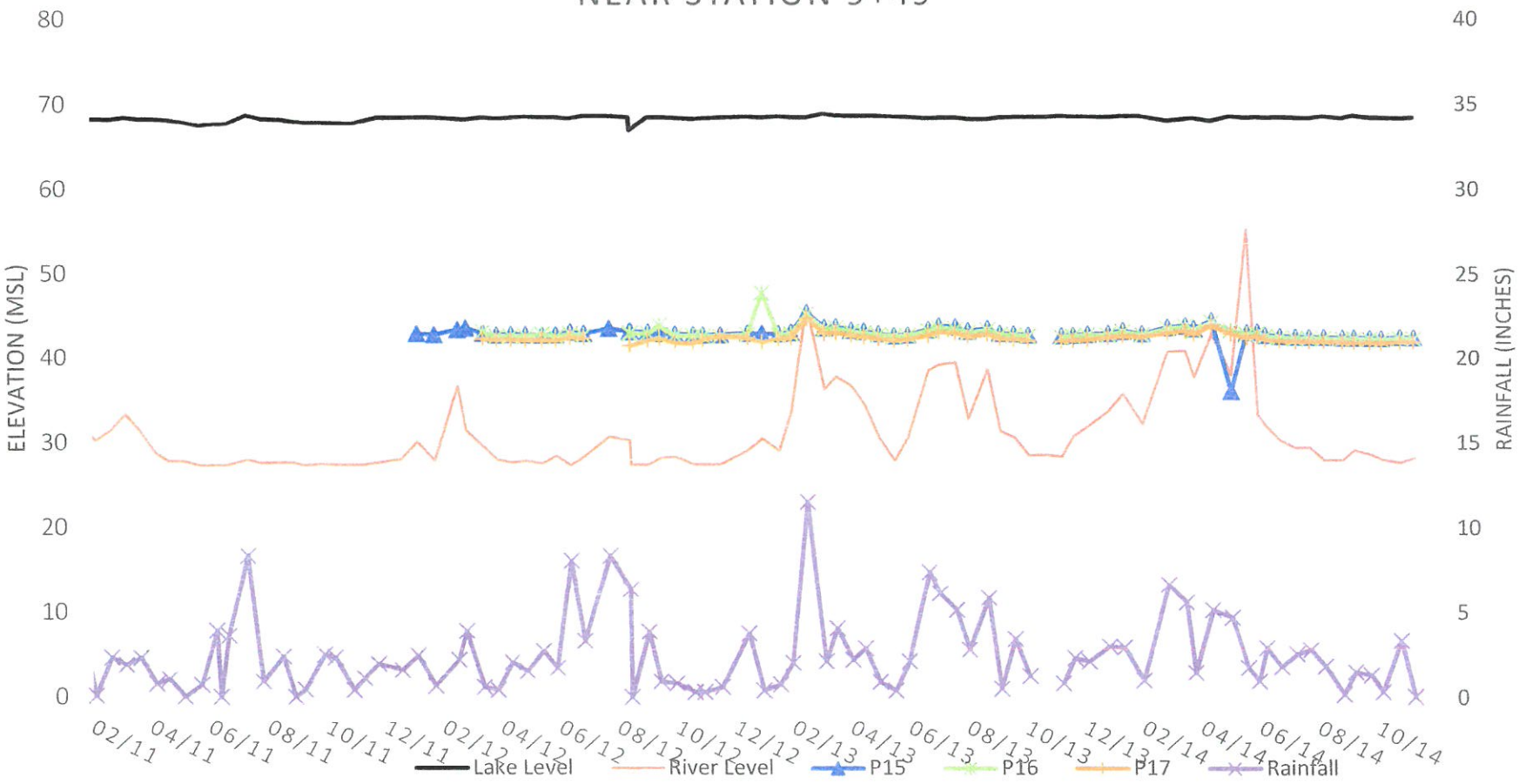
NEAR STATION 6+63



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

AUXILIARY SPILLWAY MONITORING WELL DATA

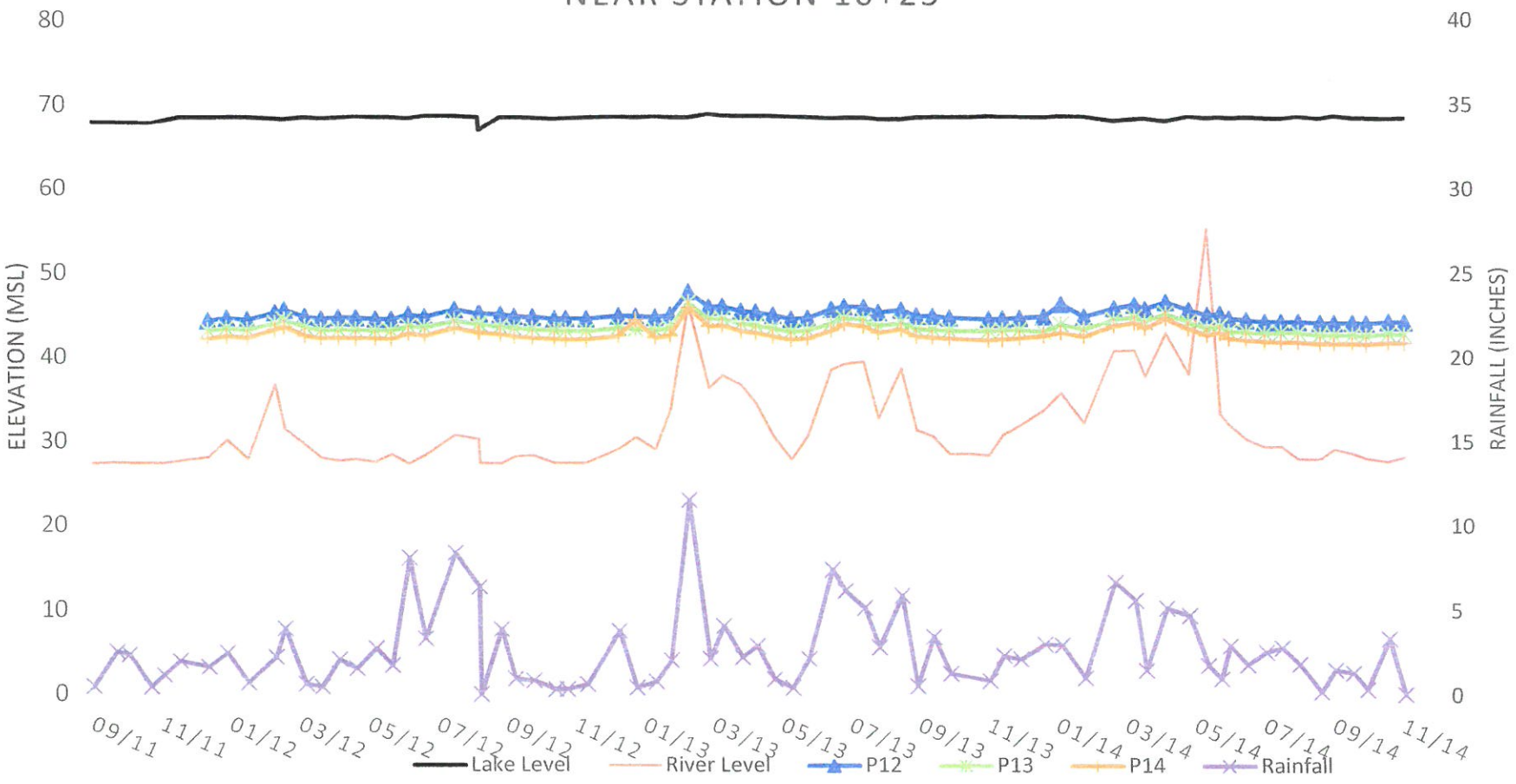
NEAR STATION 9+49



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

EMBANKMENT MONITORING WELL DATA

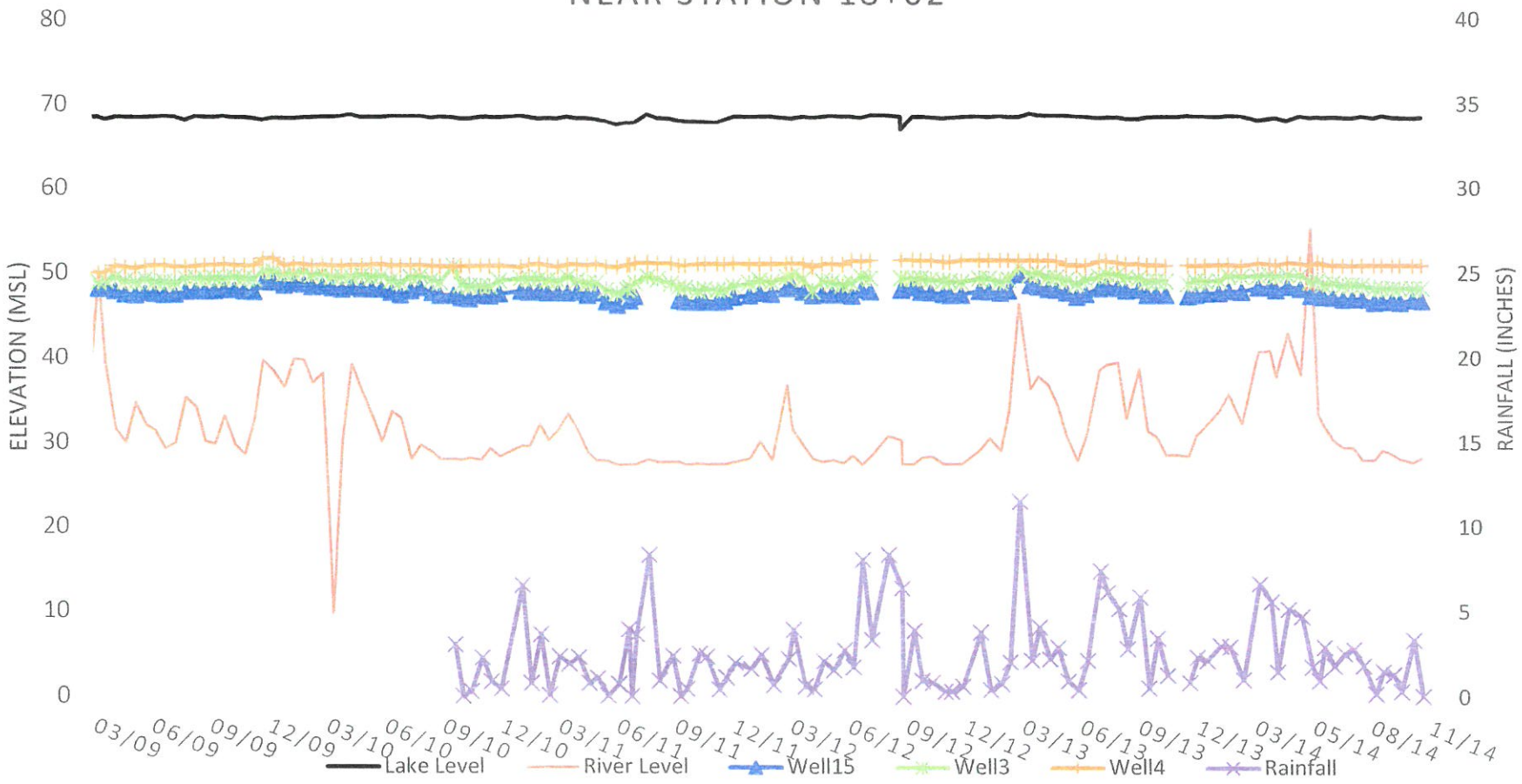
NEAR STATION 10+25



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

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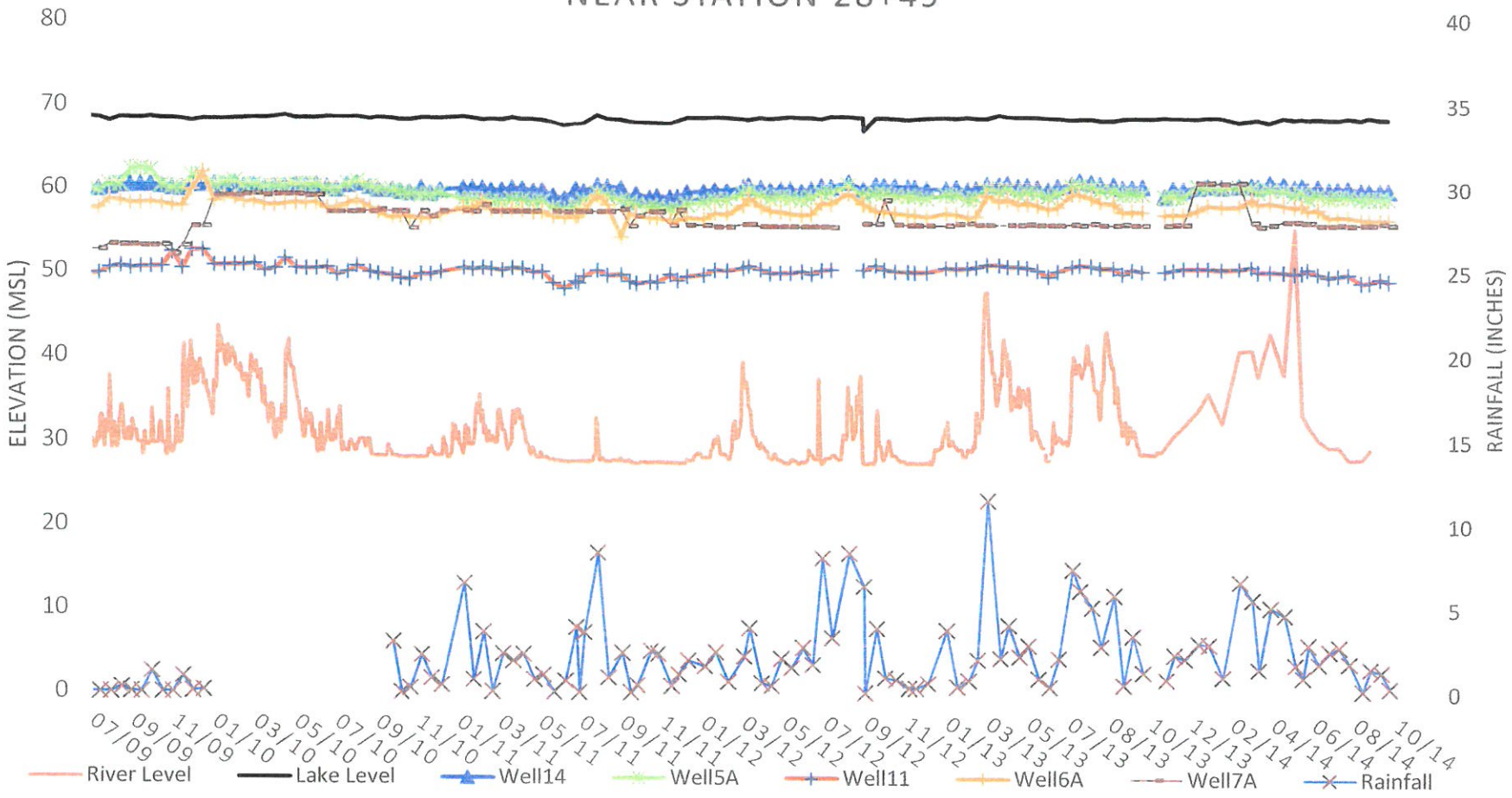
NEAR STATION 18+02



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

EMBANKMENT MONITORING WELL DATA

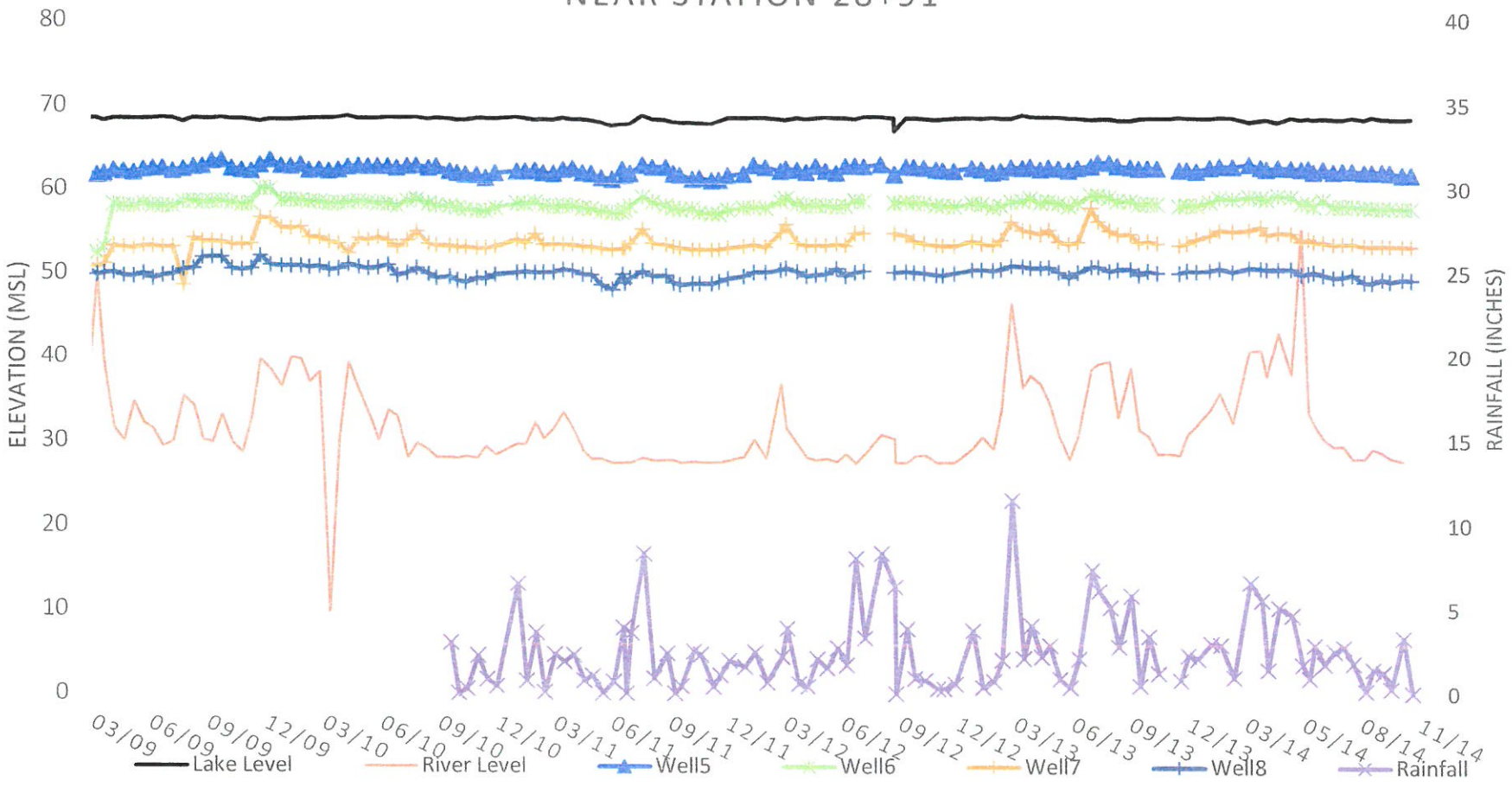
NEAR STATION 28+49



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

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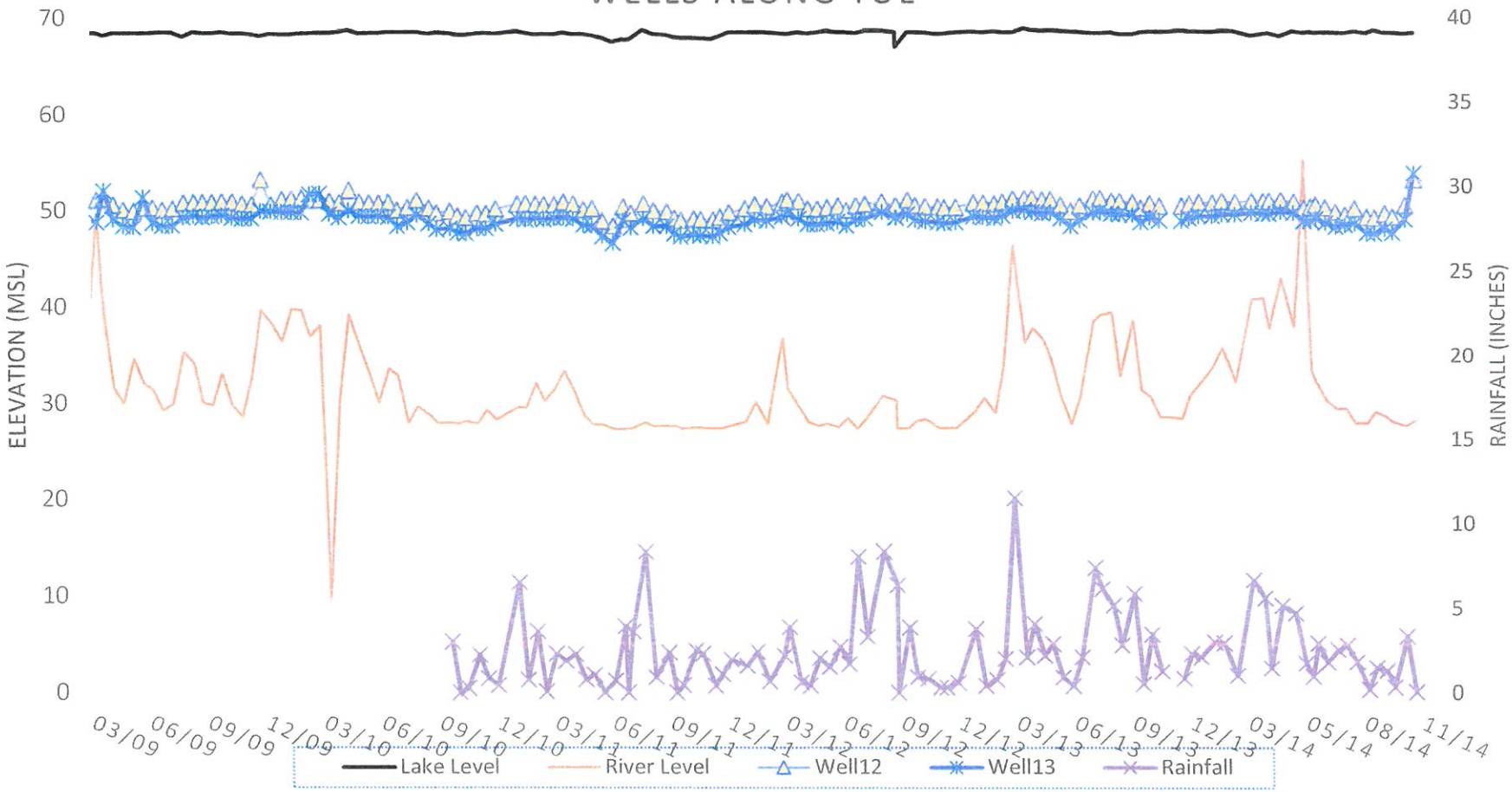
NEAR STATION 28+91



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

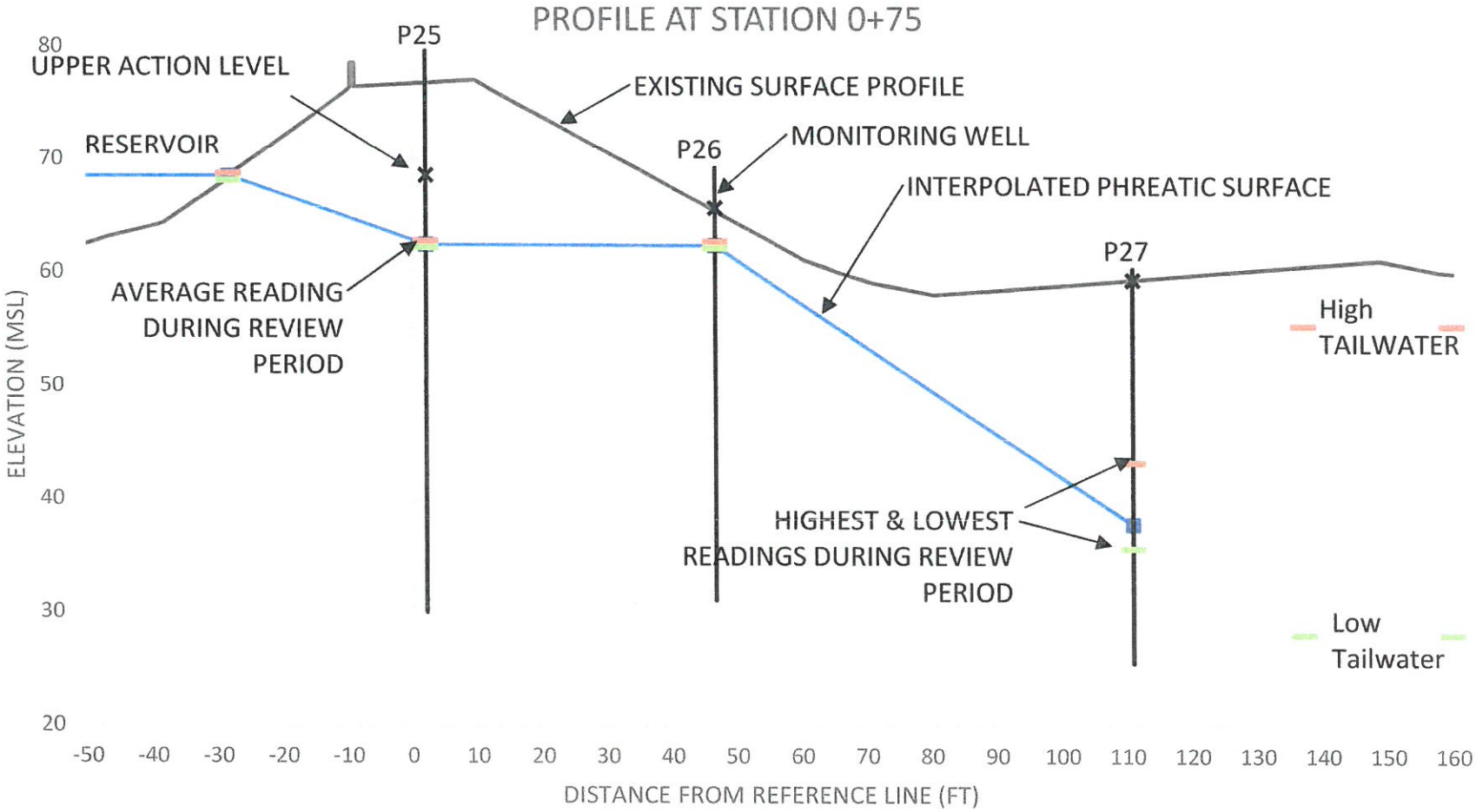
EMBANKMENT MONITORING WELL DATA

WELLS ALONG TOE



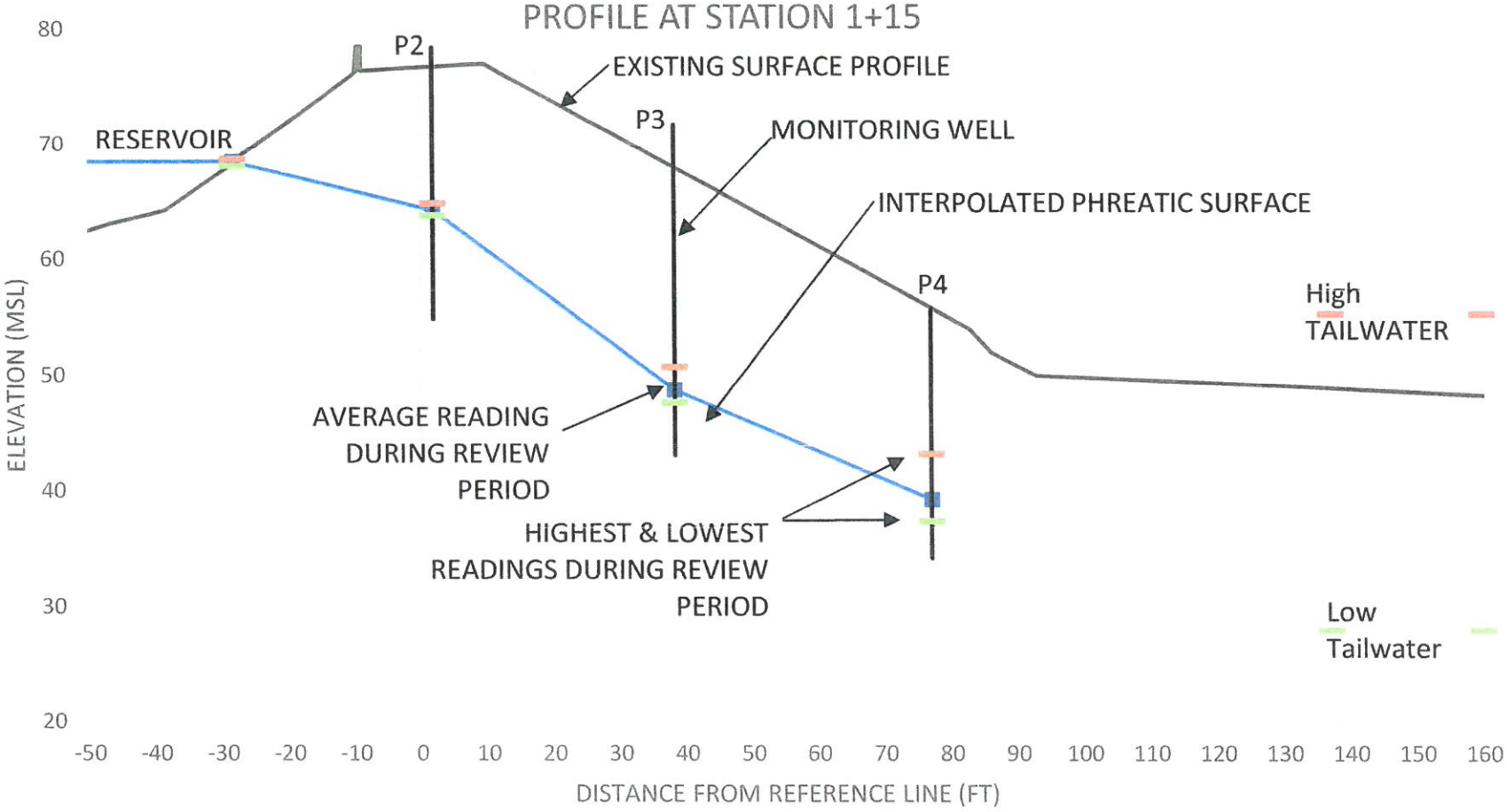
JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

EMBANKMENT MONITORING WELL DATA



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

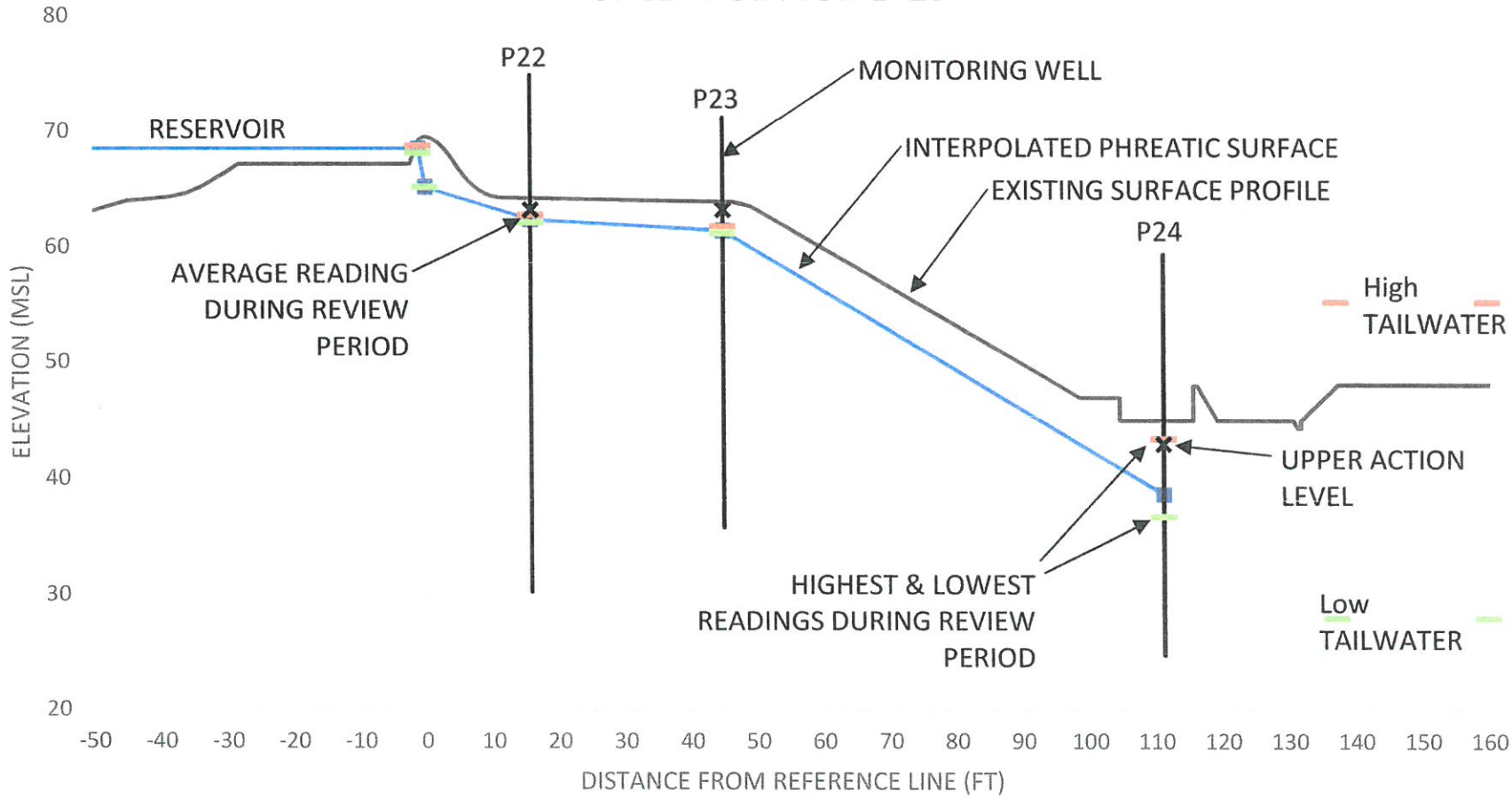
EMBANKMENT MONITORING WELL DATA



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

EMBANKMENT MONITORING WELL DATA

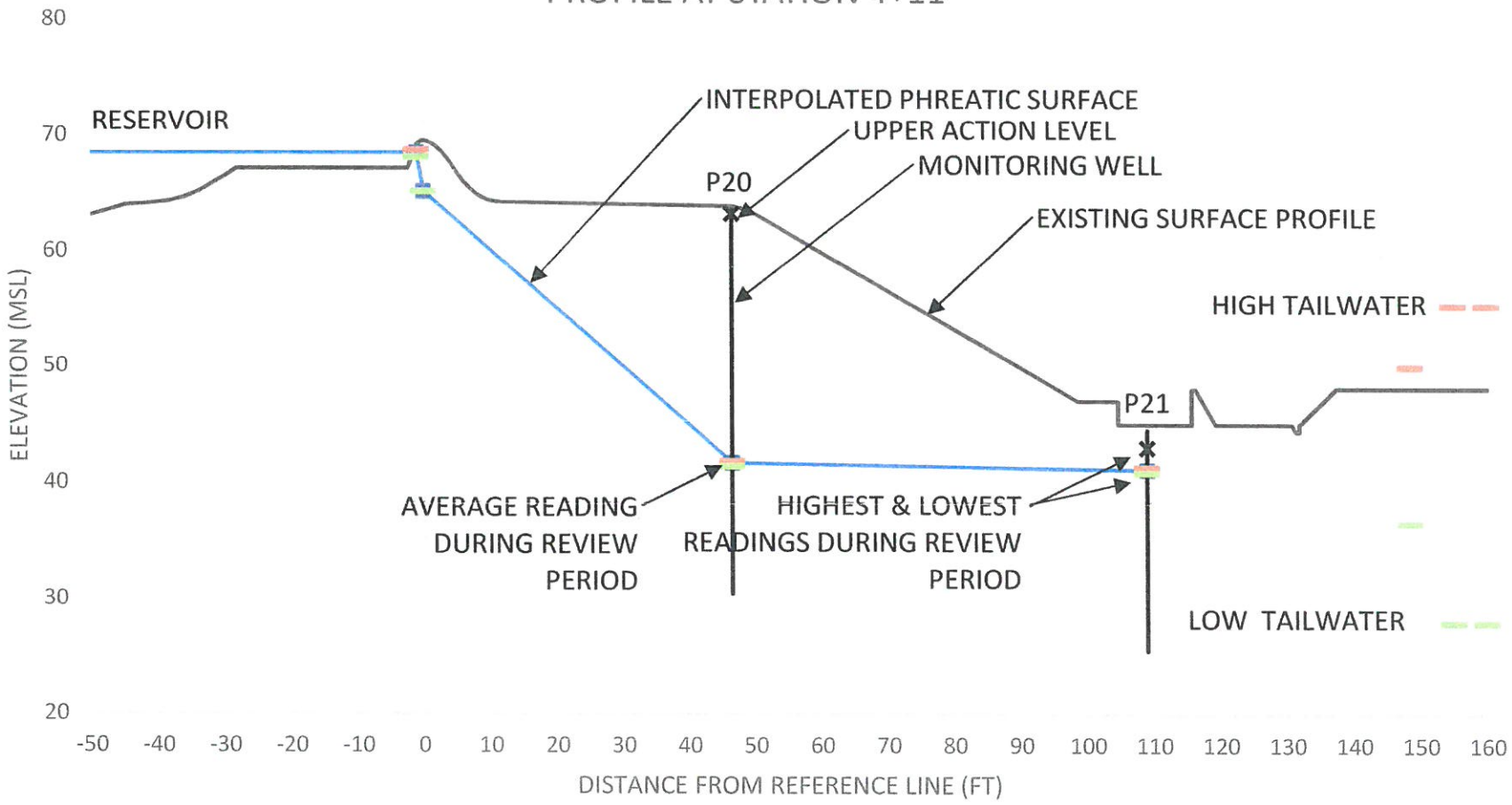
PROFILE AT STATION 1+26



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

EMBANKMENT MONITORING WELL DATA

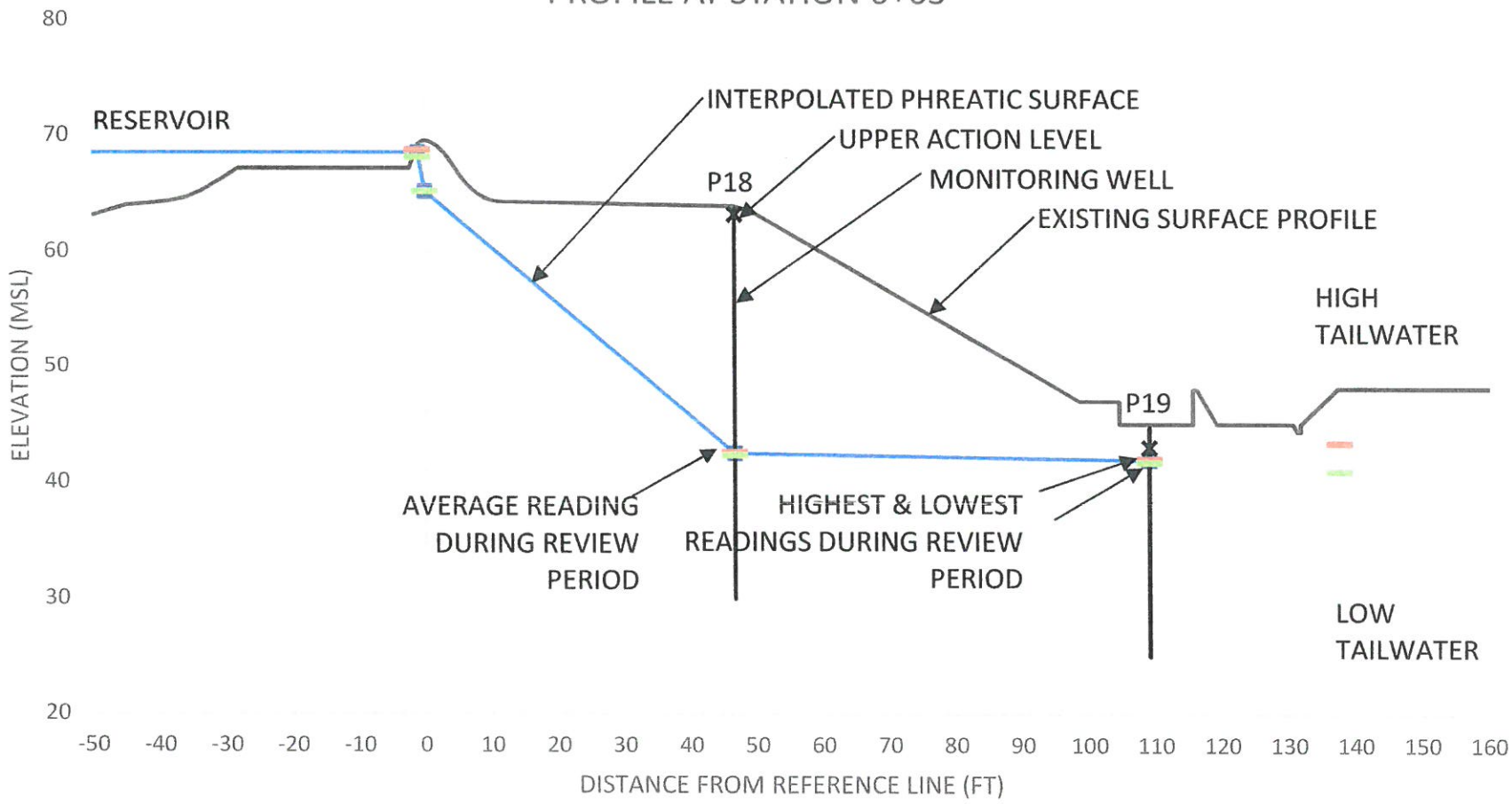
PROFILE AT STATION 4+11



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

EMBANKMENT MONITORING WELL DATA

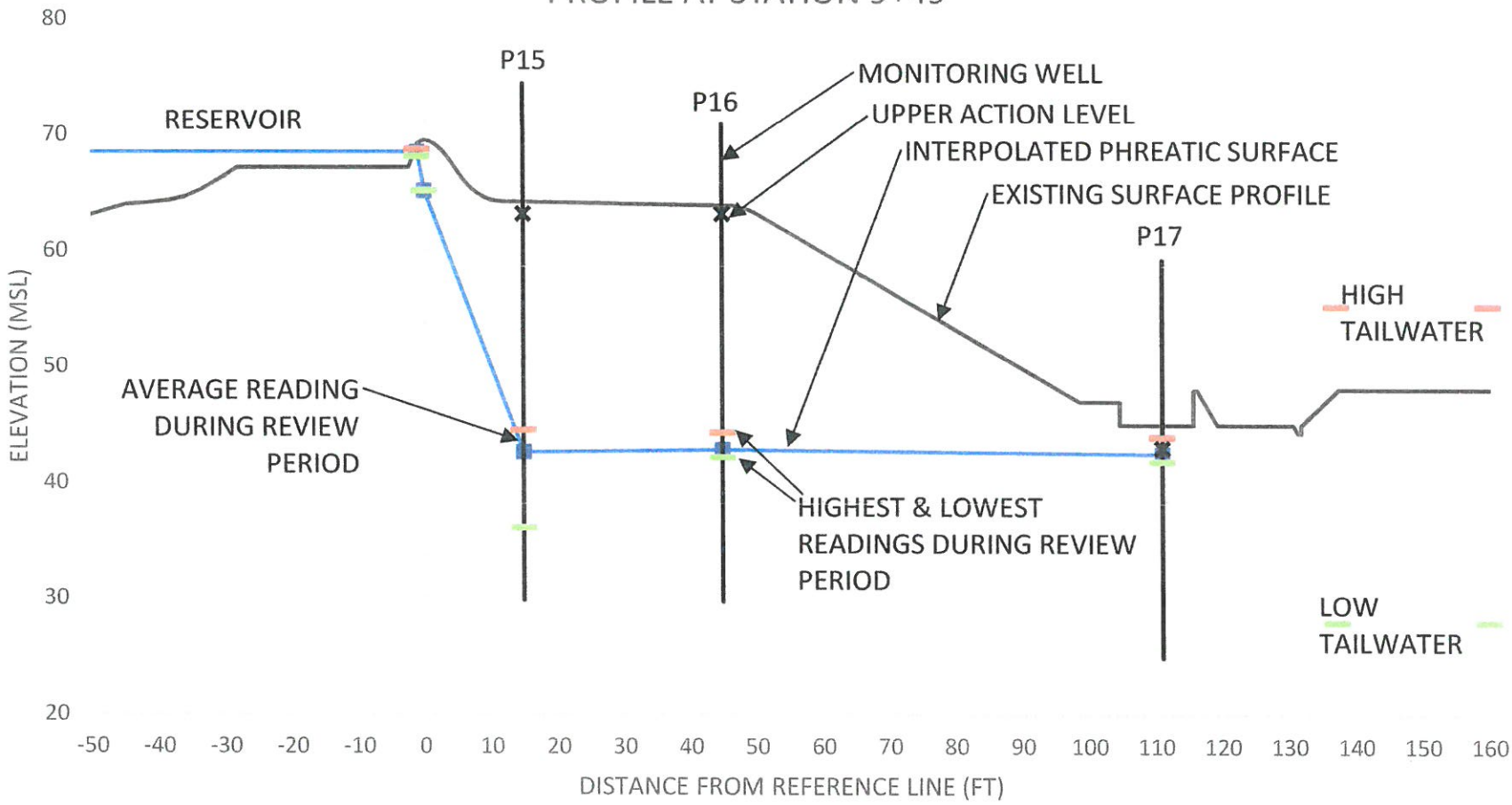
PROFILE AT STATION 6+63



JACKSON BLUFF HYDROELECTRIC PROJECT
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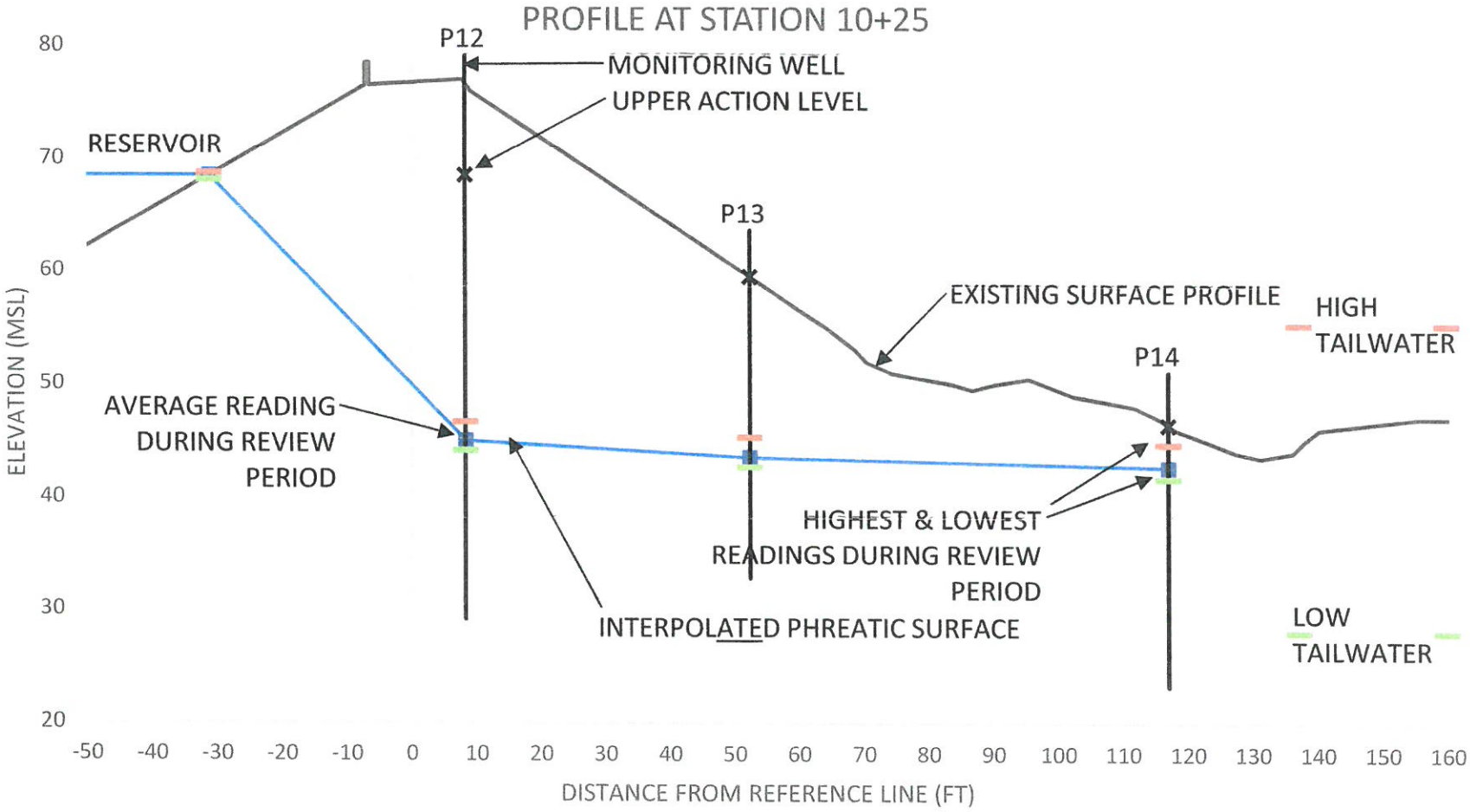
EMBANKMENT MONITORING WELL DATA

PROFILE AT STATION 9+49

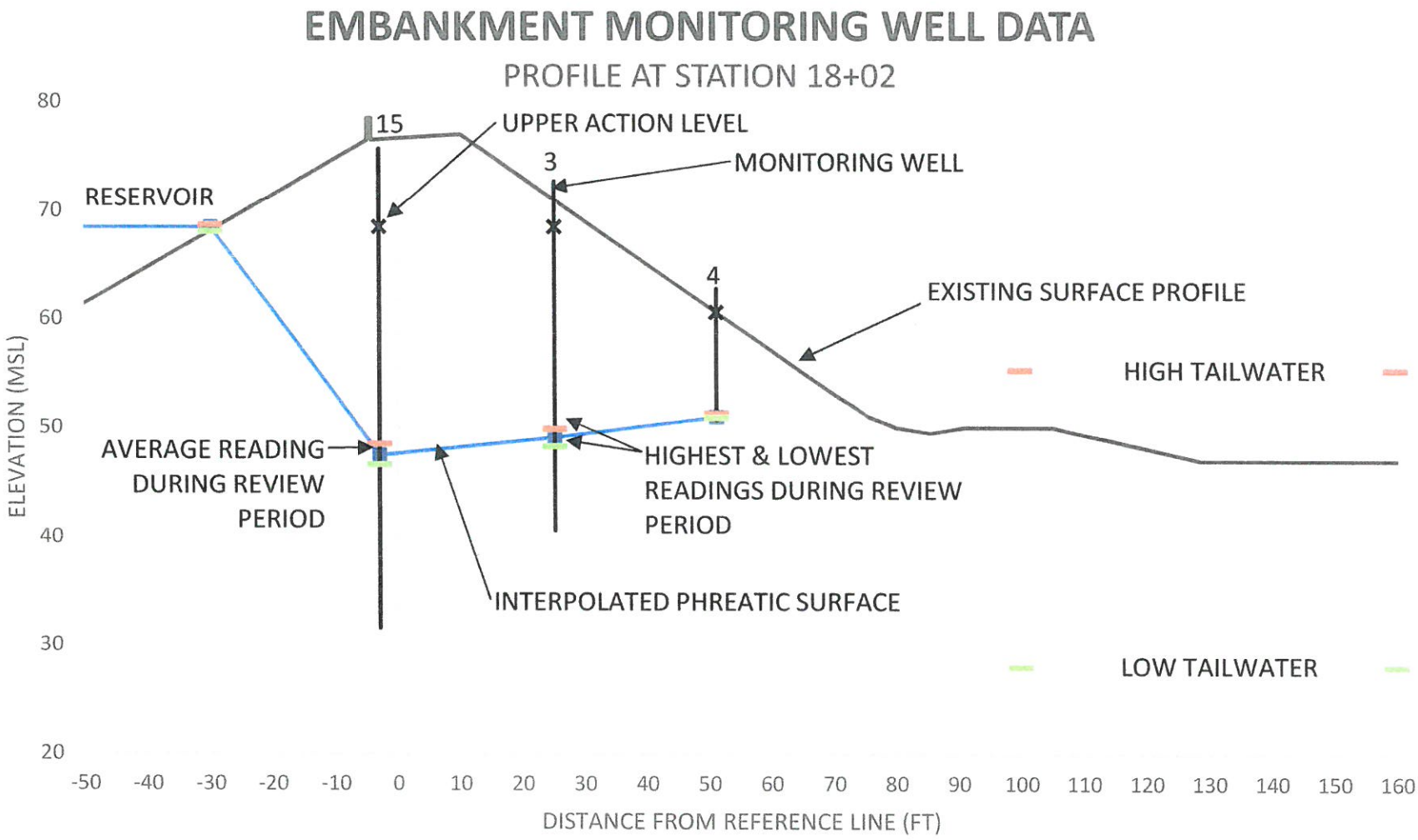


JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

EMBANKMENT MONITORING WELL DATA

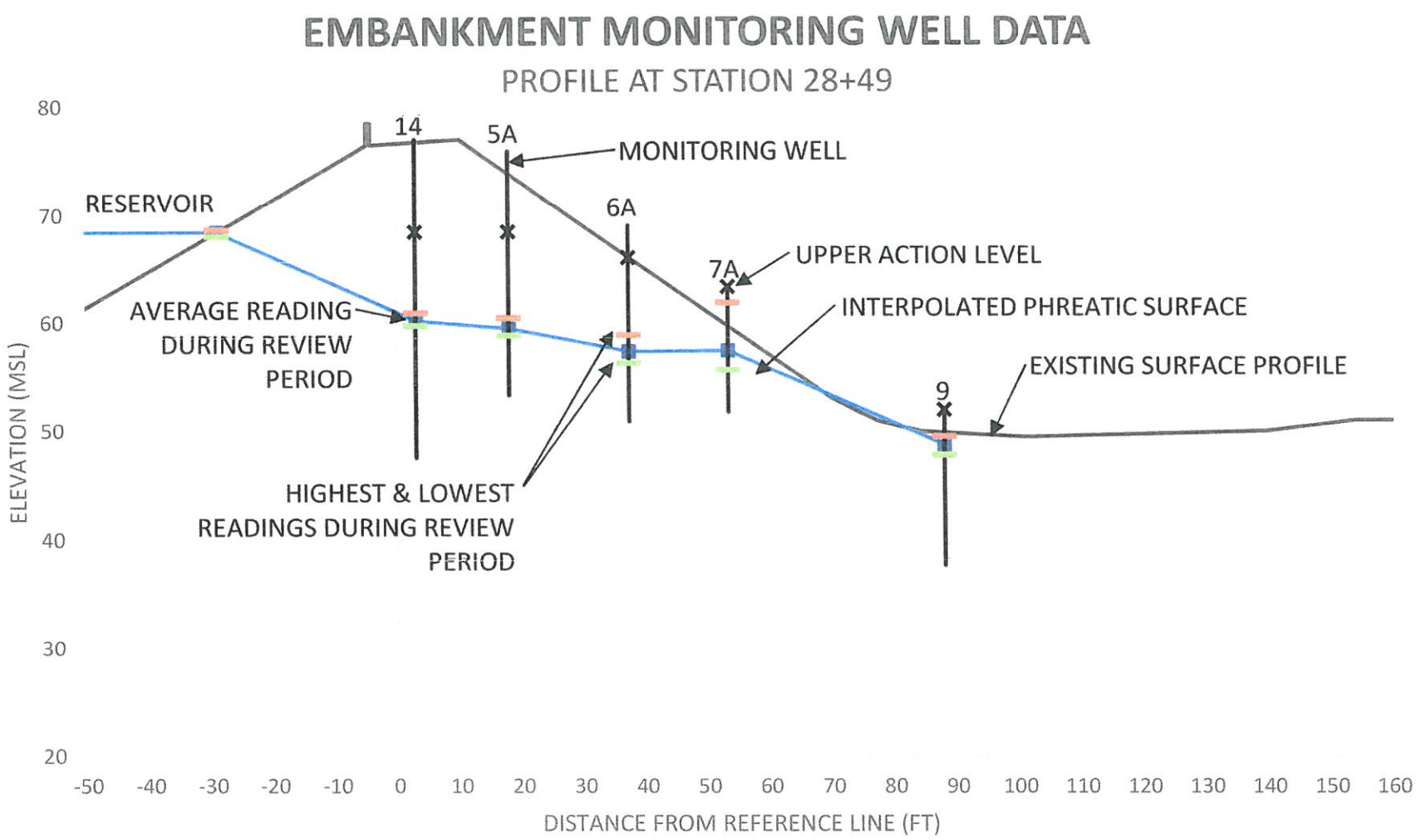


JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

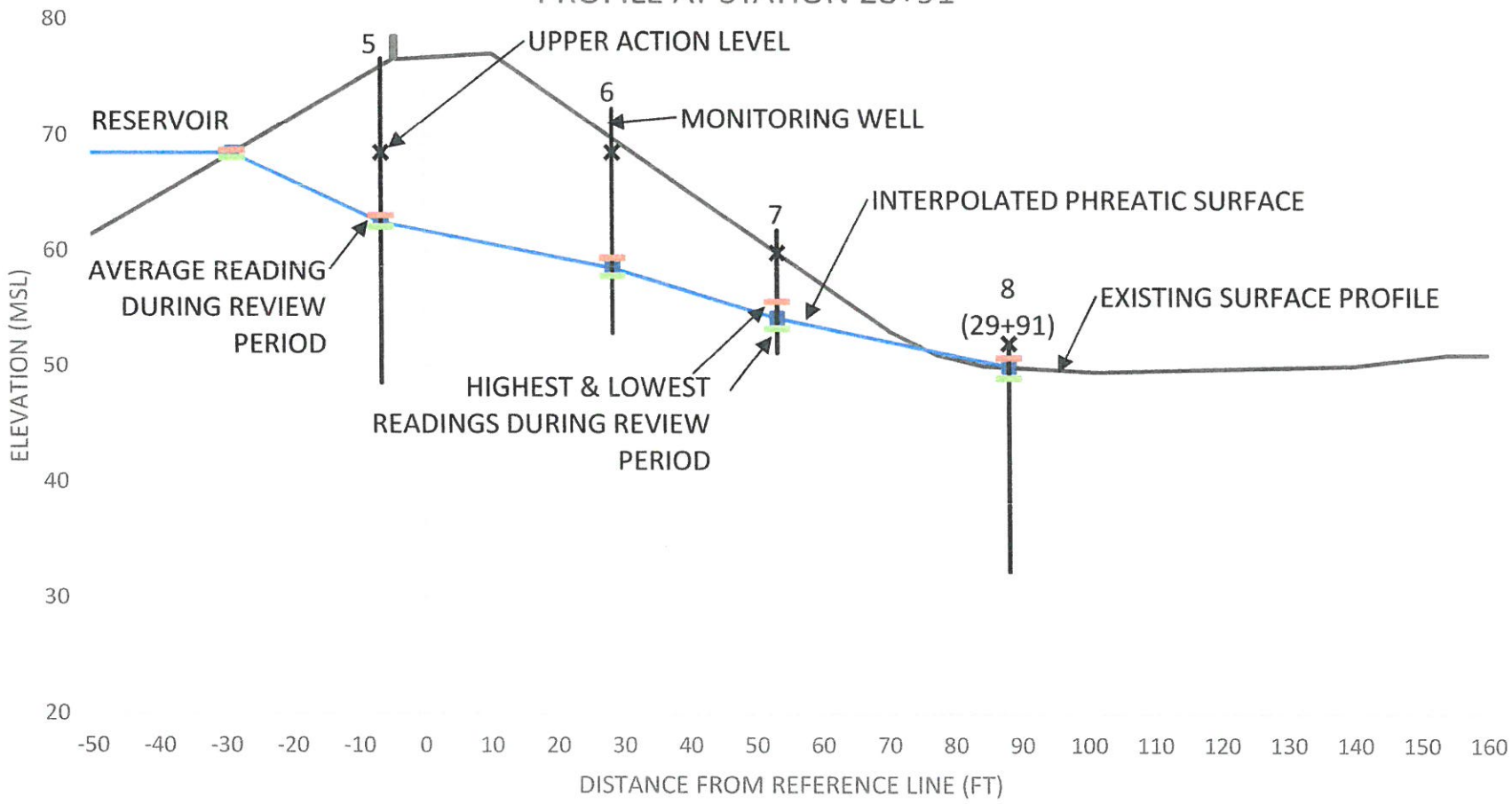
EMBANKMENT MONITORING WELL DATA



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

EMBANKMENT MONITORING WELL DATA

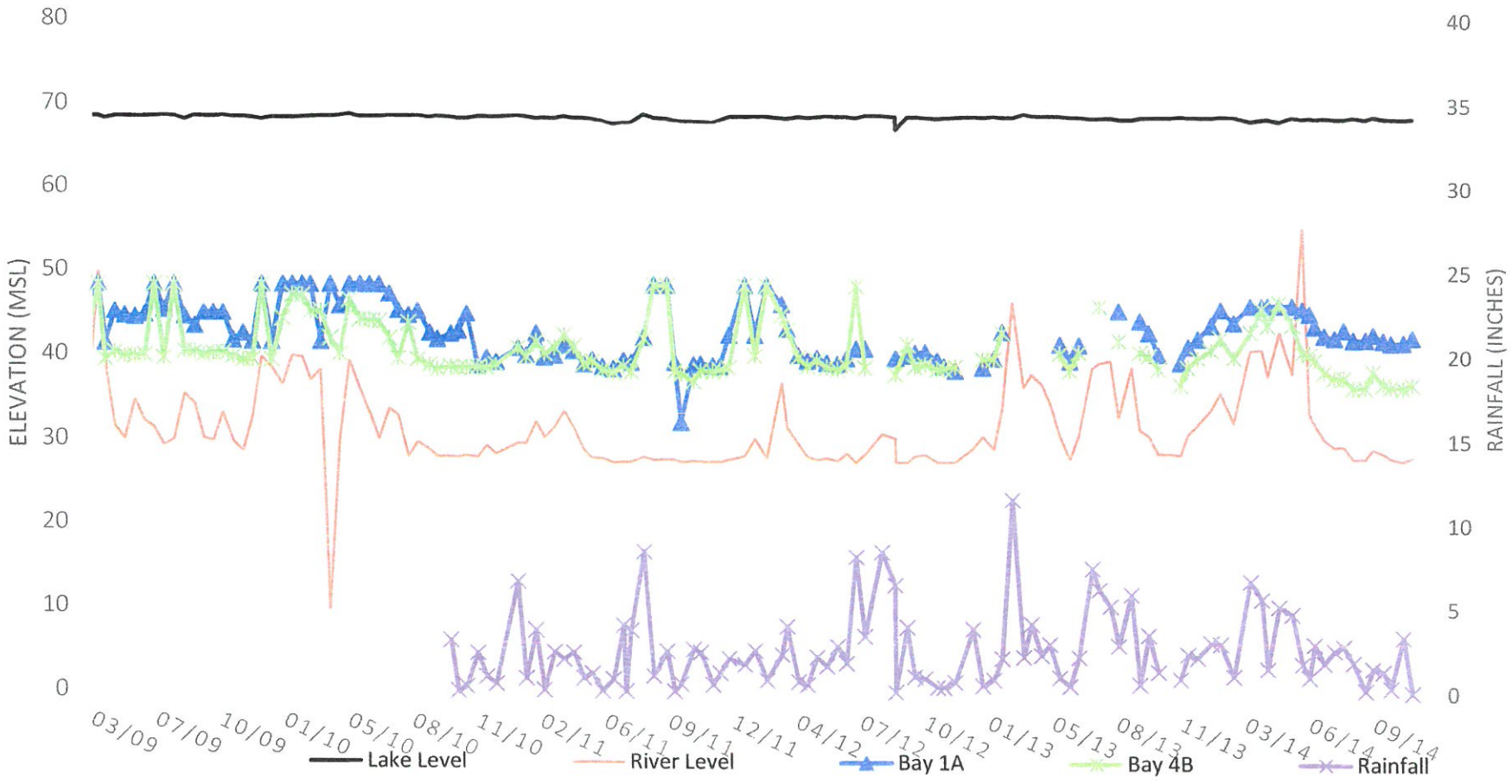
PROFILE AT STATION 28+91



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

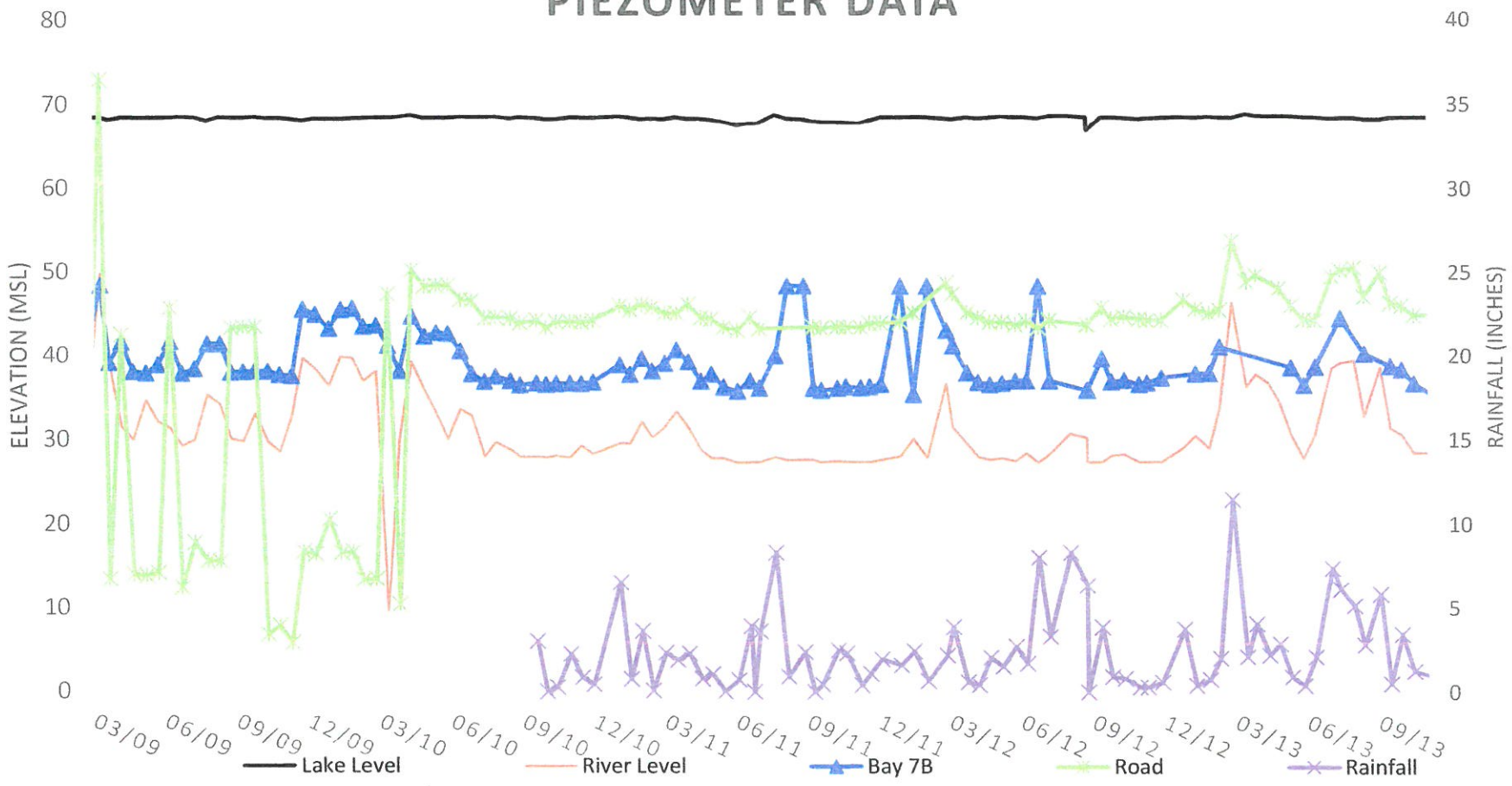
Appendix D. Piezometer Data

GATED SPILLWAY PIEZOMETER DATA



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

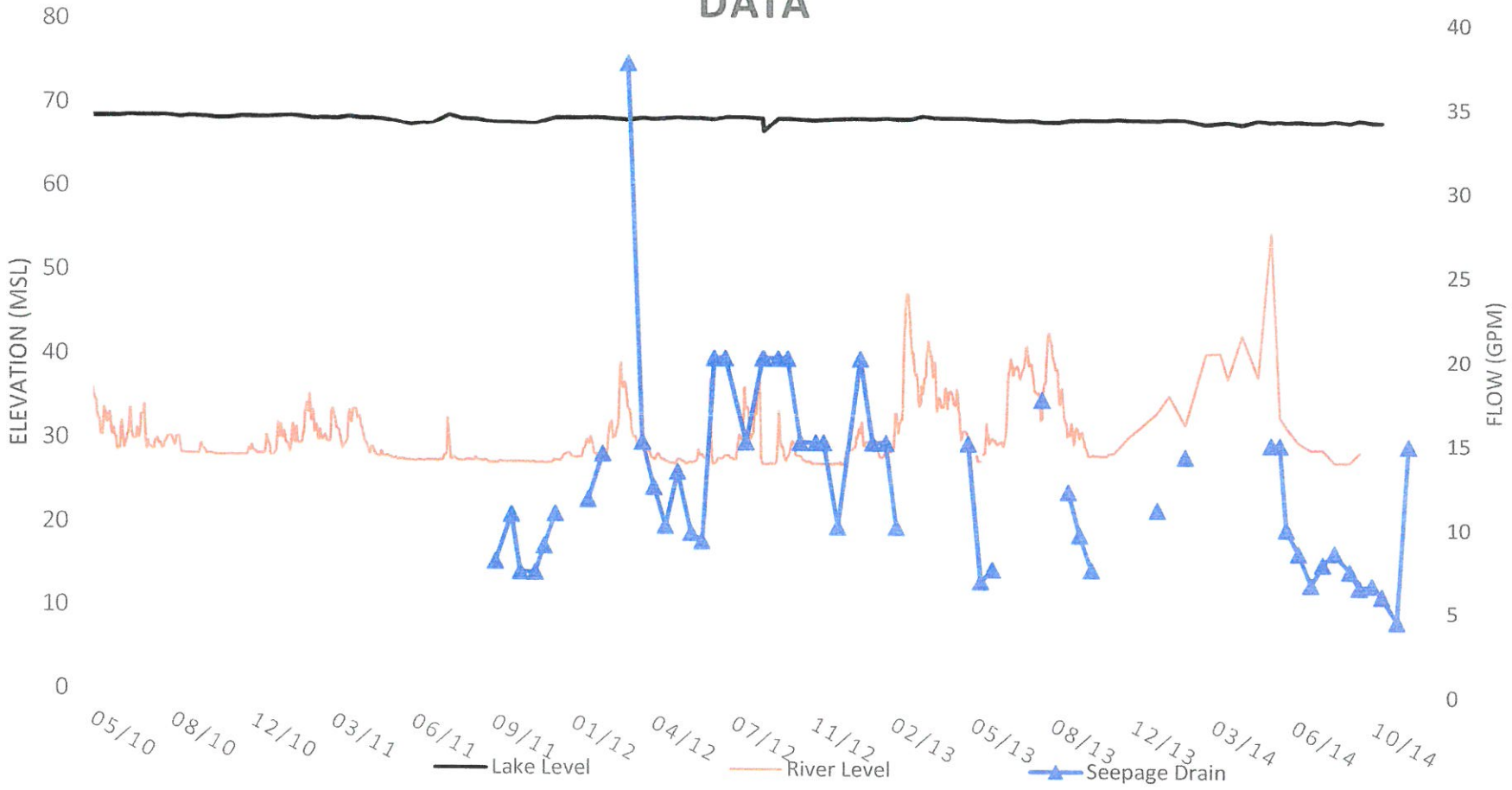
GATED SPILLWAY / POWERHOUSE (ADJACENT)
PIEZOMETER DATA



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

Appendix E. Seepage Data

AUXILIARY SPILLWAY SEEPAGE DRAIN MONITORING DATA



JACKSON BLUFF HYDROELECTRIC PROJECT
FERC PROJECT NO. 2891

Jackson Bluff Hydroelectric Project
FERC Project No. P-2891

Powerhouse Stability - Factors of Safety - Sliding (Ebasco 1992)

Condition	Case	Loading	Load Combination	Headwater	Tailwater	Sliding Plane Elev	FS	FERC FS	Comments
Dewatered	I	Usual	Normal Operating	68.5	28.0		2.59	1.5	
	II	Unusual	Flood	77.0	60.0		3.34	1.5	
	II-A	Unusual	Normal with Ice	68.5	28.0		2.59	1.5	Ice is not an expected condition
	III	Extreme	Normal with Seismic	68.5	28.0		2.59	1.3	
Not Dewatered	I	Usual	Normal Operating	68.5	28.0		2.73	1.5	
	II	Unusual	Flood	77.0	60.0		3.53	1.5	
	II-A	Unusual	Normal with Ice	68.5	28.0		2.73	1.5	Ice is not an expected condition
	III	Extreme	Normal with Seismic	68.5	28.0		2.73	1.3	
One Water Passage Dewatered	I	Usual	Normal Operating	68.5	28.0	10.0	2.55	1.5	
	II	Unusual	Flood	77.0	60.0	10.0	2.67	1.5	
	II-A	Unusual	Normal with Ice	68.5	28.0	10.0	1.2	1.5	Ice is not an expected condition
	III	Extreme	Normal with Seismic	68.5	28.0	10.0	2.02	1.3	
Two Water Passages Dewatered	I	Usual	Normal Operating	68.5	28.0	10.0	2.48	1.5	
	II	Unusual	Flood	77.0	60.0	10.0	2.57	1.5	
	II-A	Unusual	Normal with Ice	68.5	28.0	10.0	1.18	1.5	Ice is not an expected condition
	III	Extreme	Normal with Seismic	68.5	28.0	10.0	1.95	1.3	
One Water Passage Dewatered	I	Usual	Normal Operating	68.5	28.0	7.0	2.56	1.5	
	II	Unusual	Flood	77.0	60.0	7.0	2.46	1.5	
	II-A	Unusual	Normal with Ice	68.5	28.0	7.0	0.97	1.5	Ice is not an expected condition
	III	Extreme	Normal with Seismic	68.5	28.0	7.0	1.81	1.3	
Two Water Passages Dewatered	I	Usual	Normal Operating	68.5	28.0	7.0	2.49	1.5	
	II	Unusual	Flood	77.0	60.0	7.0	2.38	1.5	
	II-A	Unusual	Normal with Ice	68.5	28.0	7.0	0.93	1.5	Ice is not an expected condition
	III	Extreme	Normal with Seismic	68.5	28.0	7.0	1.75	1.3	
One Water Passage Dewatered	I	Usual	Normal Operating	68.5	28.0	0.0	2.6	1.5	
	II	Unusual	Flood	77.0	60.0	0.0	2.03	1.5	
	II-A	Unusual	Normal with Ice	68.5	28.0	0.0	0.83	1.5	Ice is not an expected condition
	III	Extreme	Normal with Seismic	68.5	28.0	0.0	1.43	1.3	
Two Water Passages Dewatered	I	Usual	Normal Operating	68.5	28.0	0.0	2.55	1.5	
	II	Unusual	Flood	77.0	60.0	0.0	1.94	1.5	
	II-A	Unusual	Normal with Ice	68.5	28.0	0.0	0.8	1.5	Ice is not an expected condition
	III	Extreme	Normal with Seismic	68.5	28.0	0.0	1.38	1.3	

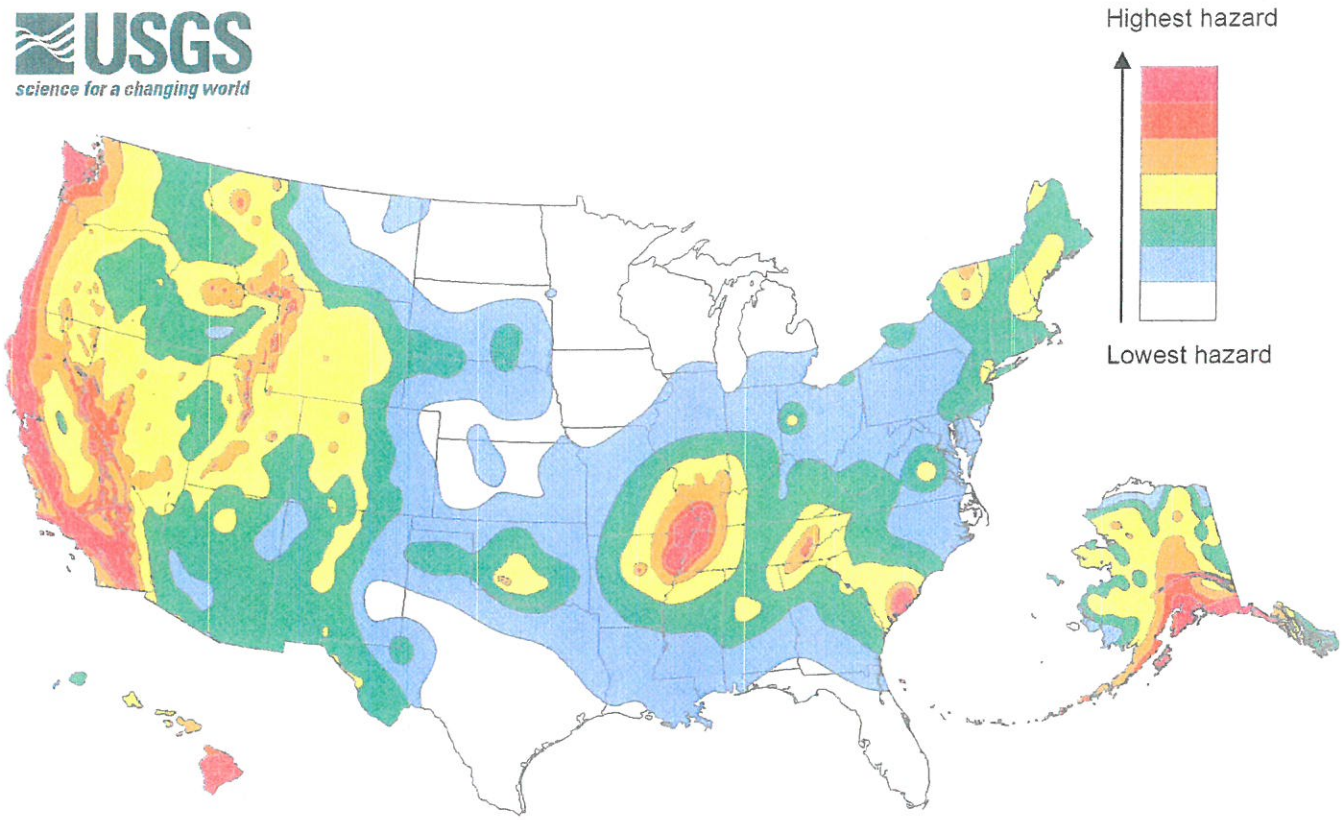
Summary of Embankment Stability Analysis Results
(Initial Safety Inspection Report)

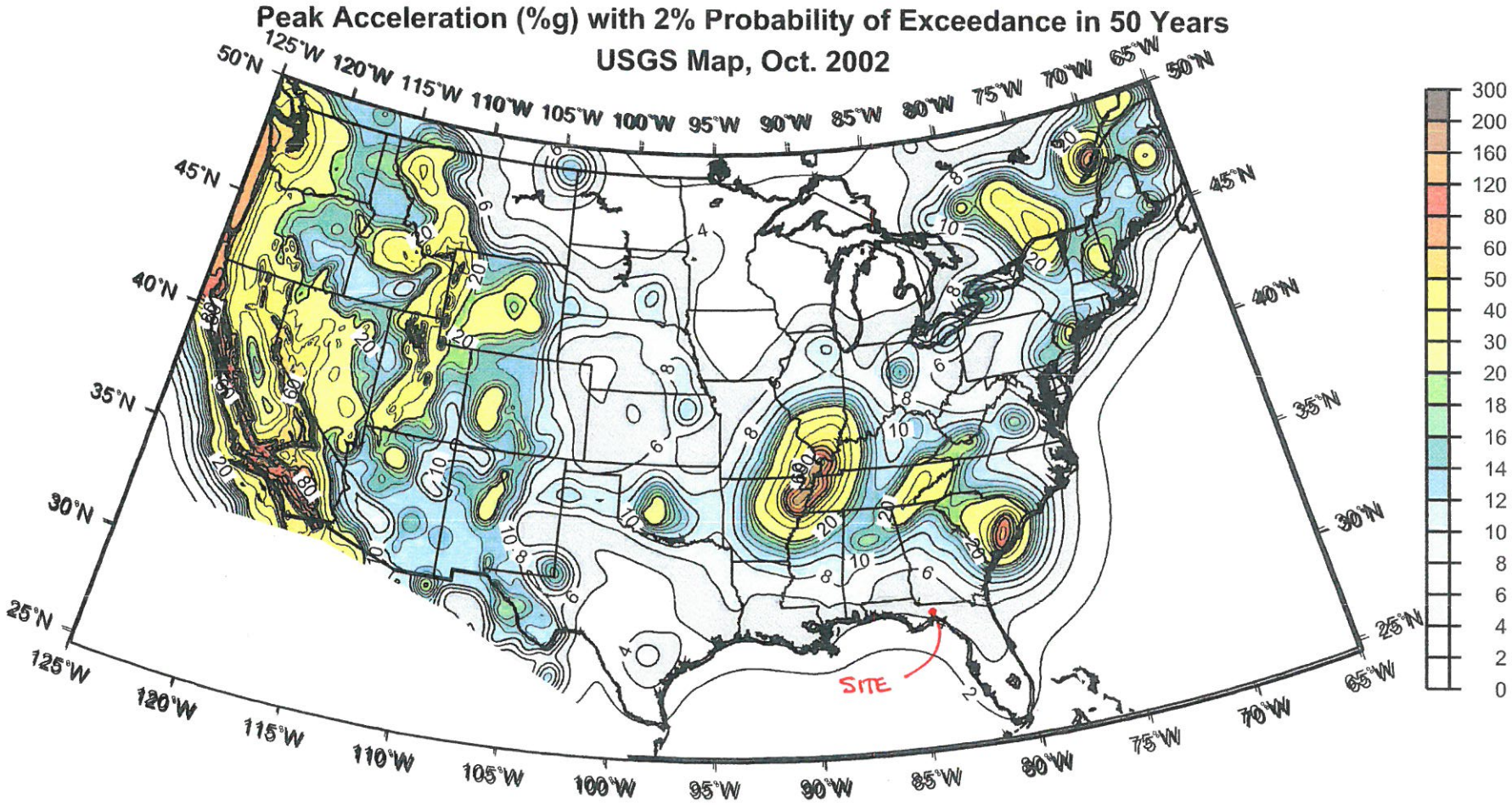
Load Case	Loading Condition	Slope	FERC-Recommended Safety Factors	Factor of Safety: With Cohesion	Factor of Safety: Without Cohesion
Rebuilt Section					
Case I	Steady seepage	U/S		3.4	1.7
		D/S	1.5	2.2	2.0
Case II	Rapid drawdown	U/S	1.2	2.4	1.5
Case III	Flood surcharge	U/S		4.0	1.7
		D/S	1.4	2.2	2.6
Case IV	Seismic	D/S	1.0	1.9	1.5
Bolstered Section					
Case I	Steady seepage	U/S		2.9	1.7
		D/S	1.5	2.4	1.5
Case II	Rapid drawdown	U/S	1.2	2.5	1.2
Case III	Flood surcharge	U/S		4.2	1.7
		D/S	1.4	2.4	1.8
Case IV	Seismic	D/S	1.0	2.1	1.5

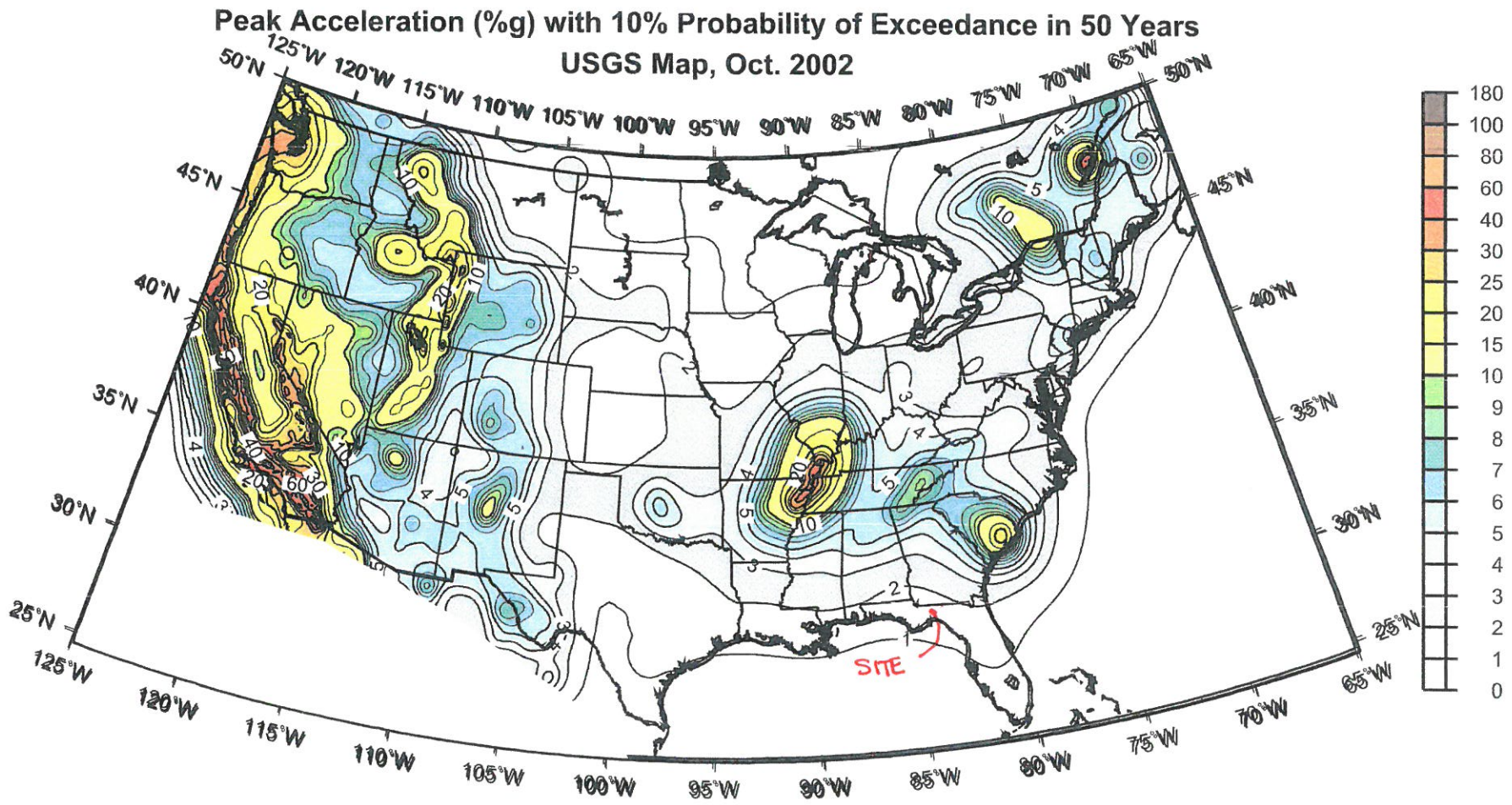
The following table is a summary of the embankment analyses for the loading conditions considered in the *Supplement to the Initial Safety Inspection Report*. Cross sections of the rebuilt and bolstered earth embankments for these loading conditions are included in this section.

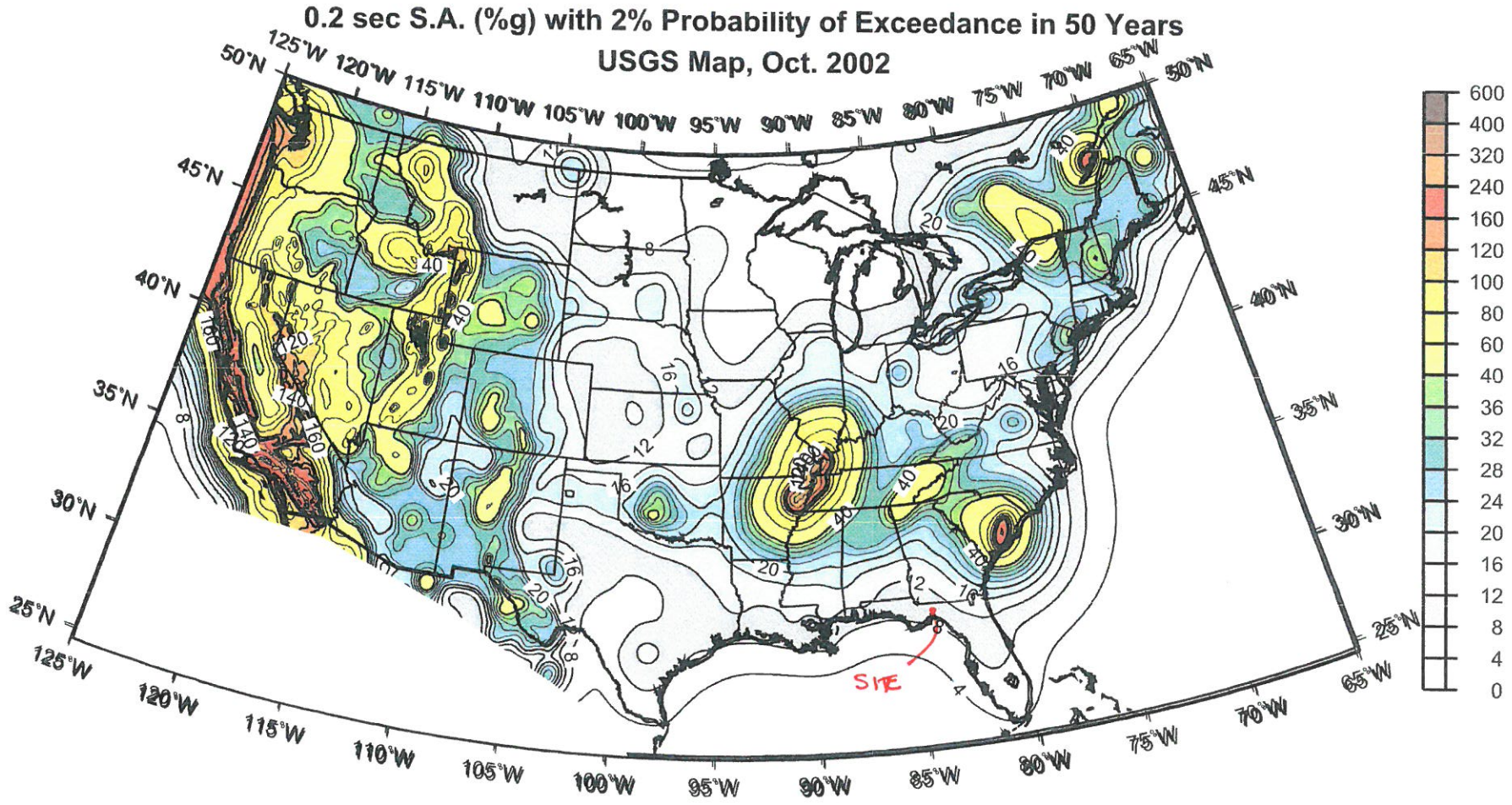
Summary of Embankment Stability Analysis Results
(Supplement to the Initial Safety Inspection Report)

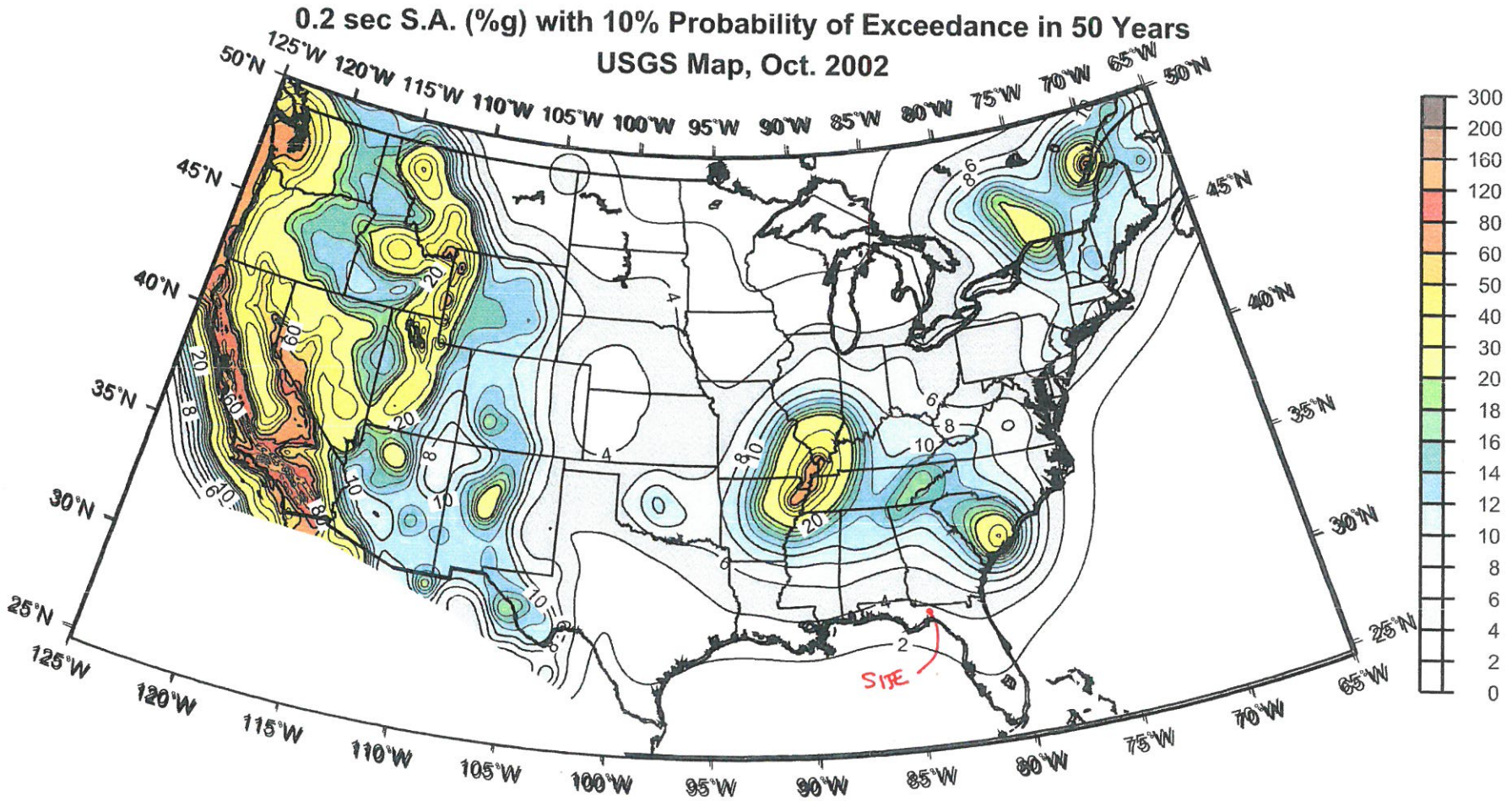
Load Case	Loading Condition	Slope	FERC-Recommended Safety Factors	Factor of Safety
Rebuilt Section				
Case I	Rainwater Infiltration	D/S	1.5	1.6
Case II	50% PMF	D/S	1.2	1.4
Bolstered Section				
Case I	Rainwater Infiltration	D/S	1.5	1.7
Case II	50% PMF	D/S	1.2	1.4

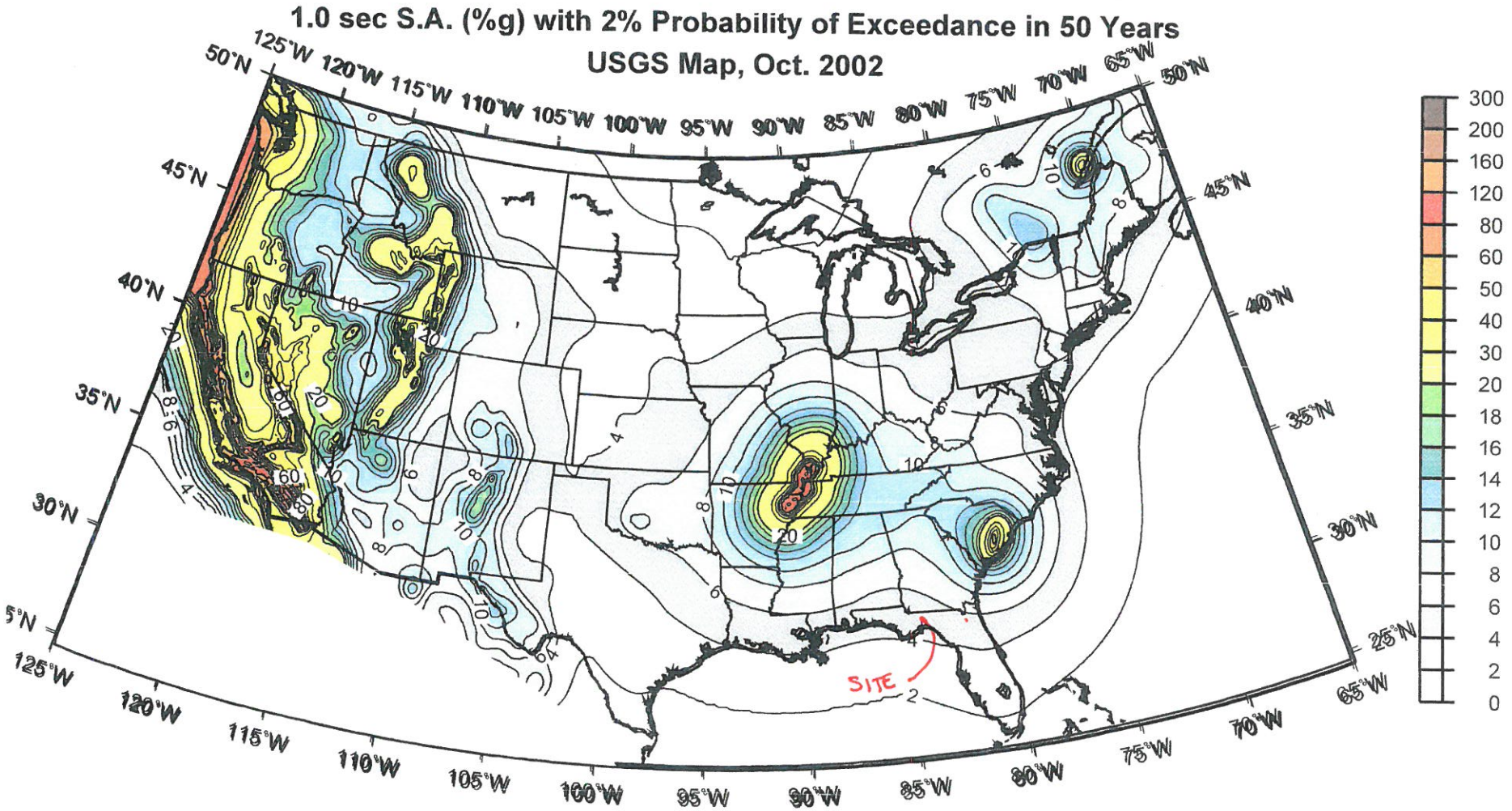

USGS
science for a changing world
Highest hazard
Lowest hazard

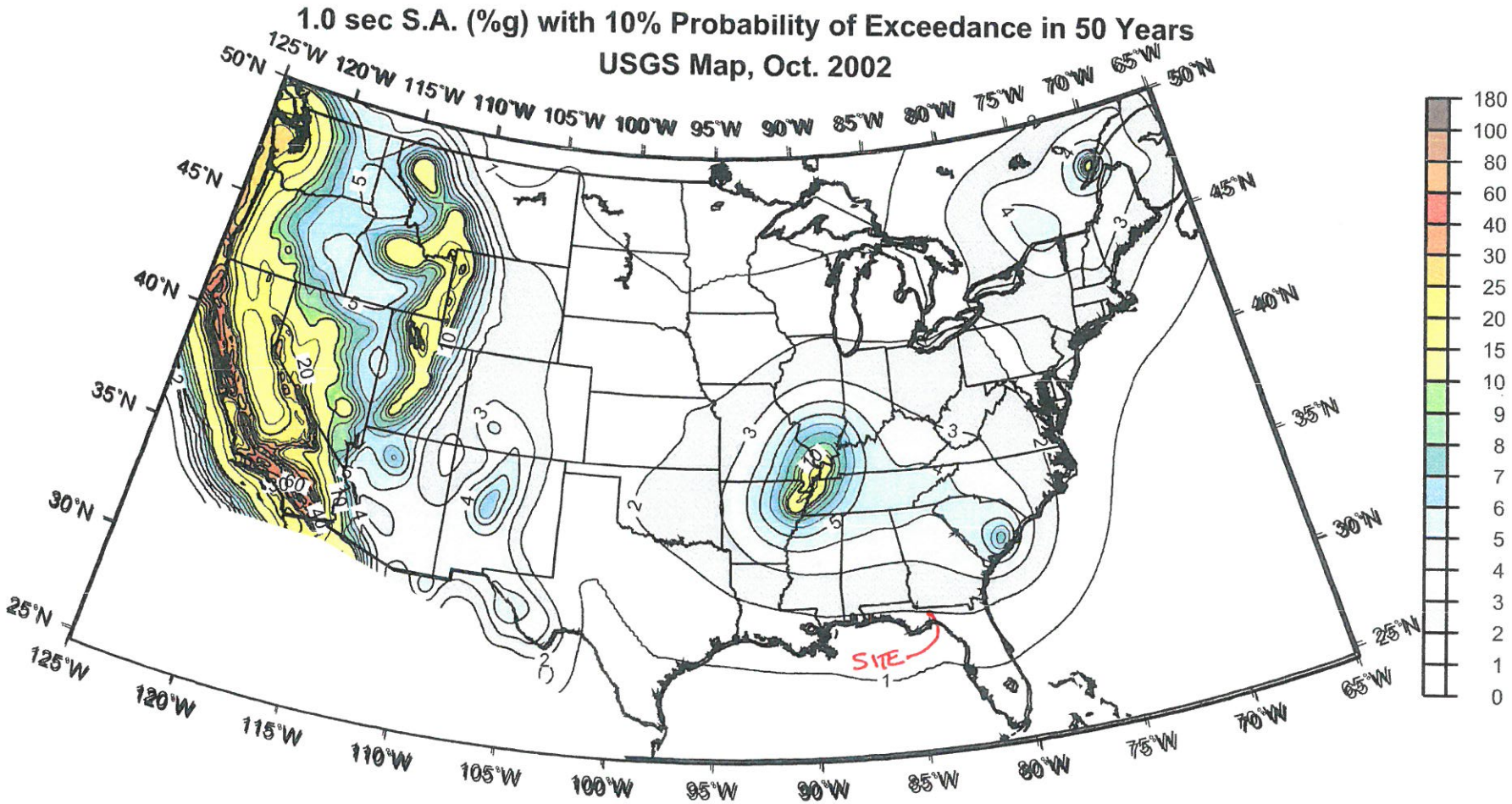

Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years
USGS Map, Oct. 2002
125°W 120°W 115°W 110°W 105°W 100°W 95°W 90°W 85°W 80°W 75°W 70°W 65°W
50°N
45°N
40°N
35°N
30°N
25°N
SITE
300
200
160
120
80
60
50
40
30
20
18
16
14
12
10
8
6
4
2
0


Peak Acceleration (%g) with 10% Probability of Exceedance in 50 Years
USGS Map, Oct. 2002
SITE
180
100
80
60
40
30
25
20
15
10
9
8
7
6
5
4
3
2
1
0


0.2 sec S.A. (%g) with 2% Probability of Exceedance in 50 Years
USGS Map, Oct. 2002
SITE
600
400
320
240
160
120
100
80
60
40
36
32
28
24
20
16
12
8
4
0


0.2 sec S.A. (%g) with 10% Probability of Exceedance in 50 Years
USGS Map, Oct. 2002
SITE
300
200
160
120
80
60
50
40
30
20
18
16
14
12
10
8
6
4
2
0


1.0 sec S.A. (%g) with 2% Probability of Exceedance in 50 Years
USGS Map, Oct. 2002
125°W 120°W 115°W 110°W 105°W 100°W 95°W 90°W 85°W 80°W 75°W 70°W 65°W
50°N
45°N
40°N
35°N
30°N
25°N
SITE
300
200
160
120
80
60
50
40
30
20
18
16
14
12
10
8
6
4
2
0


1.0 sec S.A. (%g) with 10% Probability of Exceedance in 50 Years
USGS Map, Oct. 2002
SITE
180
100
80
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APPENDIX E

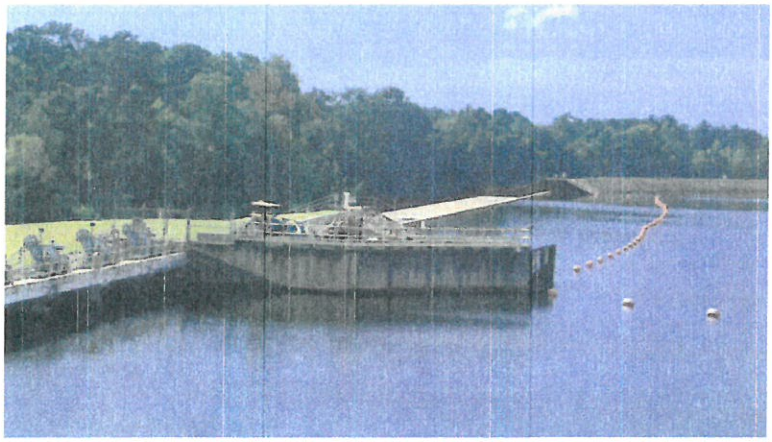
Inspection Photographs



Overall View of Powerhouse



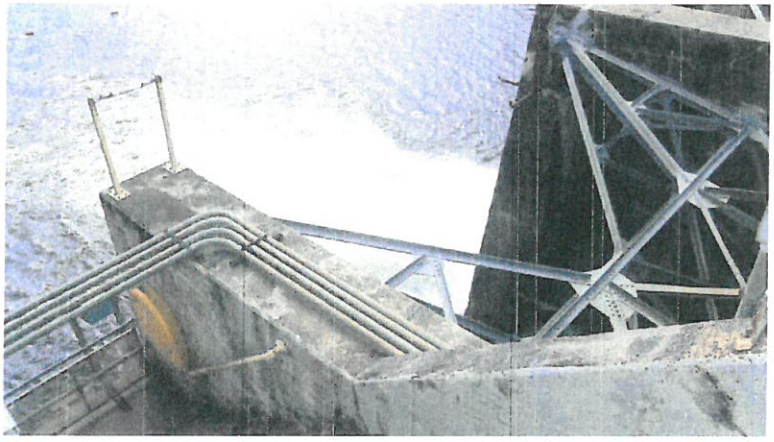
Overall View of Remote Dike, Auxiliary Spillway and Embankment



View of Auxiliary Spillway and Embankment



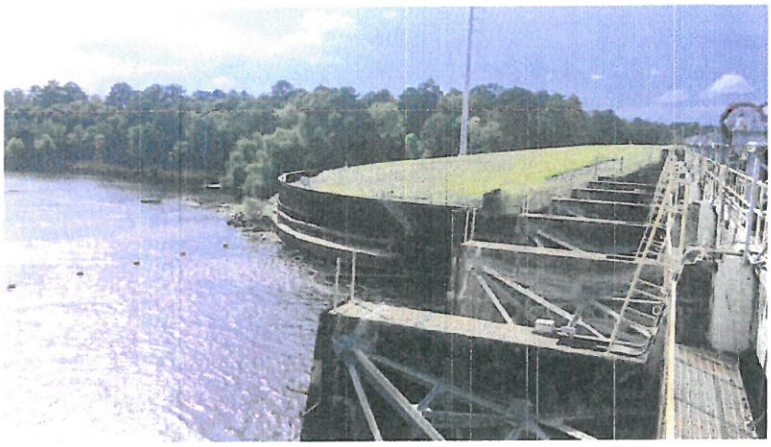
View of Upstream Buoys and Lake Area, Embankment in Background



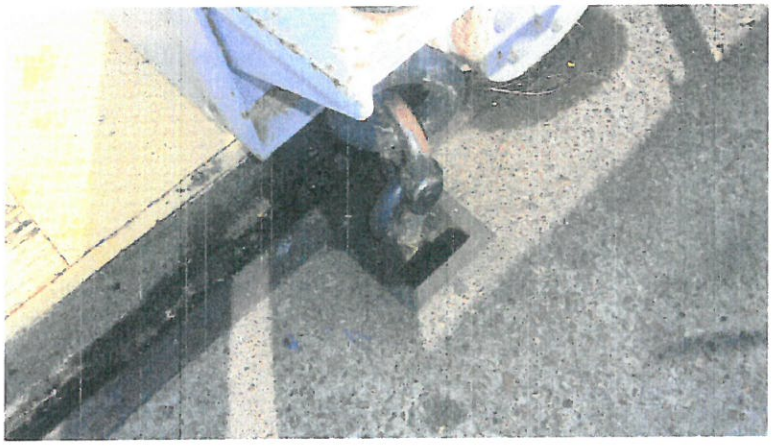
View of Tainter Gate Structure and Trunnions



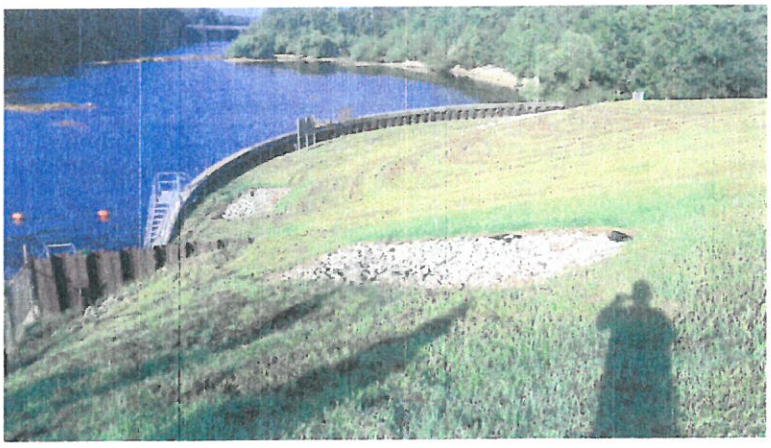
View of Tailrace and Hwy 20



View of Gated Spillway and Area between Gated Spillway and Auxiliary Spillway



Anchoring of Tainter Gate Operator



Downstream Ochlockonee River Channel (US 20 in background)



Embankment-Gated Spillway Area



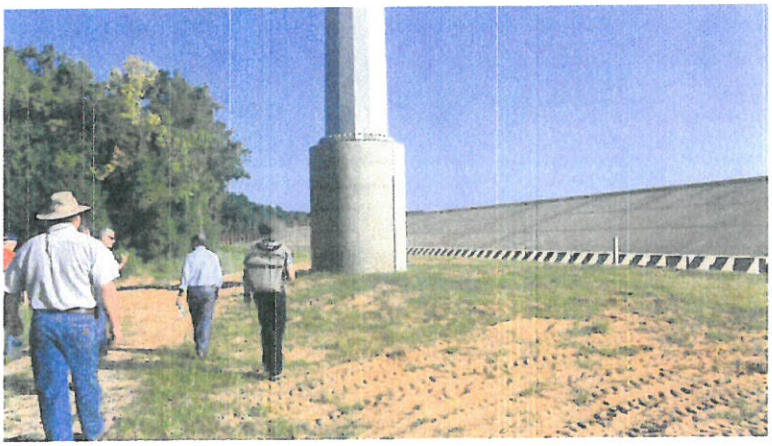
Wave Barrier - Constructed 2011



Auxiliary Spillway Looking Northerly



Looking Easterly at Auxiliary Spillway



Toe of Auxiliary Spillway



Survey Monuments on Auxiliary Spillway Chute Slab



Survey Monuments on Auxiliary Spillway Top Slab



Embankment Area with Piezometers



Access Road along Toe of Embankment, Culvert Drains from Toe Drain to DS River



Culvert Drains from Toe Drain to DS River



Area of Embankment Repair



Toe Drain Seepage to Culvert



Toe Drain Seepage



Toe Drain Seepage



Culvert under Access Road



Culvert Pipe from Toe Drain



Embankment with Piezometers, Toe Drain Discharge Pipe



Embankment and Wave Barrier - Looking Southerly



North Embankment Abutment Riprap at interface with natural ground



Rip Rap along north toe of abutment of Embankment



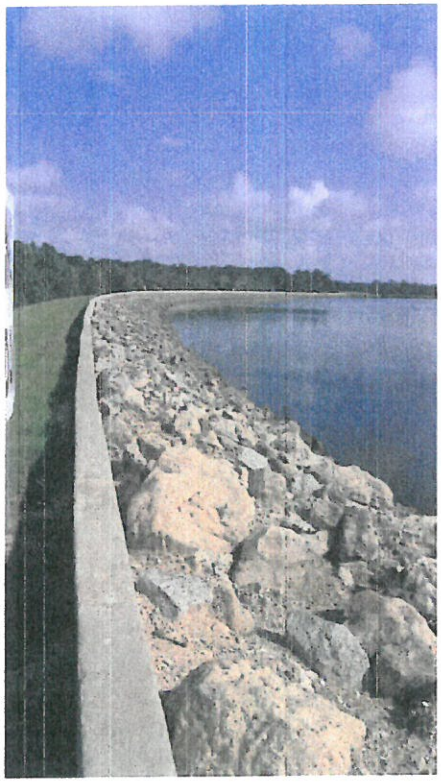
Embankment Looking South toward Powerhouse



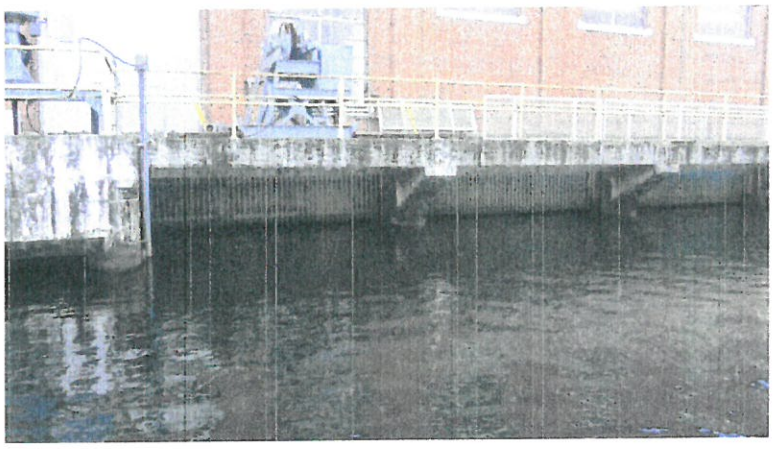
Auxiliary Spillway looking southerly at Powerhouse



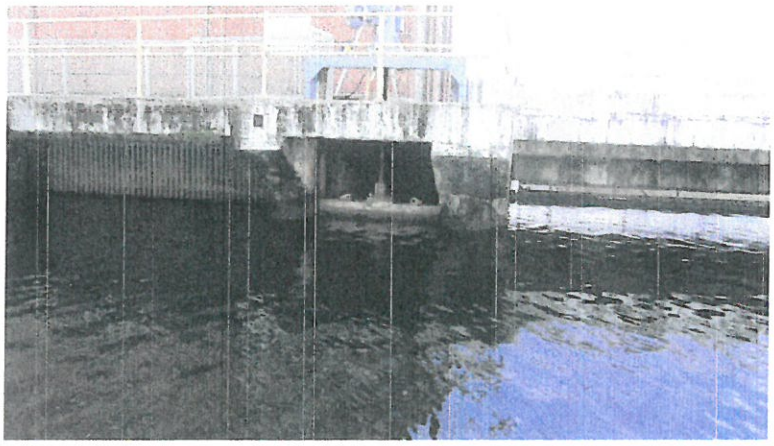
Riprap against upstream face of Auxiliary Spillway



Concrete Wave Barrier - Looking northerly from north end of Auxiliary Spillway



Boat Tour - Powerhouse Intake and Trashracks



Boat Tour - Powerhouse Intake and Trashracks



Boat Tour - Gated Spillway Section



Boat Tour - Gated Spillway Section at Interface with Raised Embankment



Boat Tour - Raised Embankment at Interface with Auxiliary Spillway



Boat Tour - Auxiliary Spillway at Interface with Embankment with Wave Barrier Wall



Boat Tour - Embankment with Wave Barrier Wall



Boat Tour - Embankment with Wave Barrier Wall



Boat Tour - Embankment with Rip-Rap at Water's Edge



Boat Tour - Embankment with Rip-Rap at Water's Edge

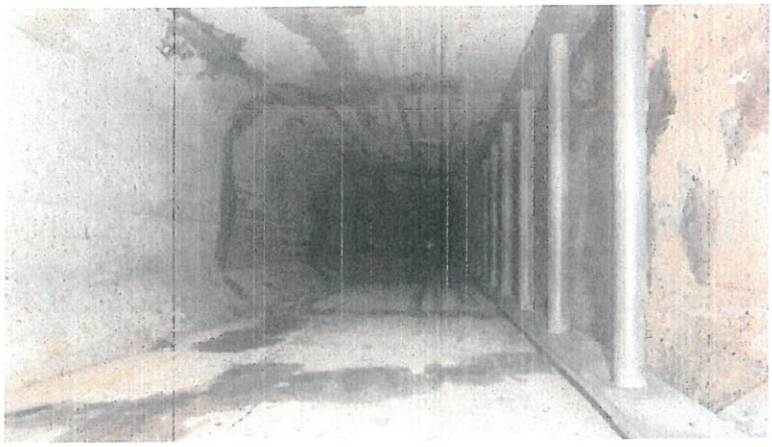


Boat Tour - Remote Dike Area

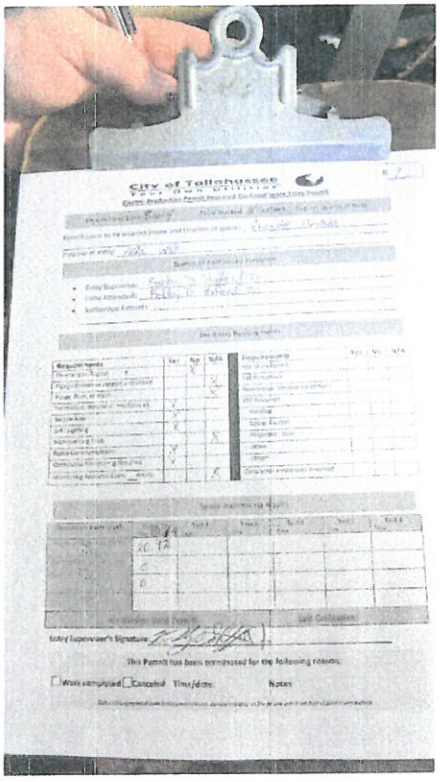




Safety Cable System for Entering Powerhouse Gallery



Ladder to Powerhouse Gallery

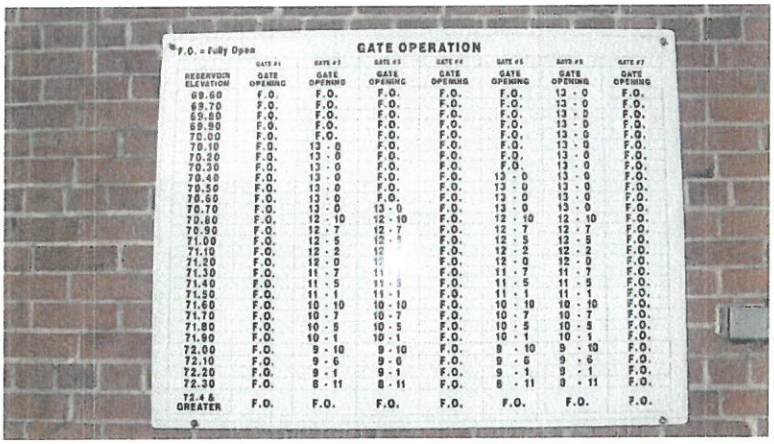




Confined Entry Permit to Powerhouse Gallery/County Boundary Monument



Access to Powerhouse Gallery



GATE OPERATION

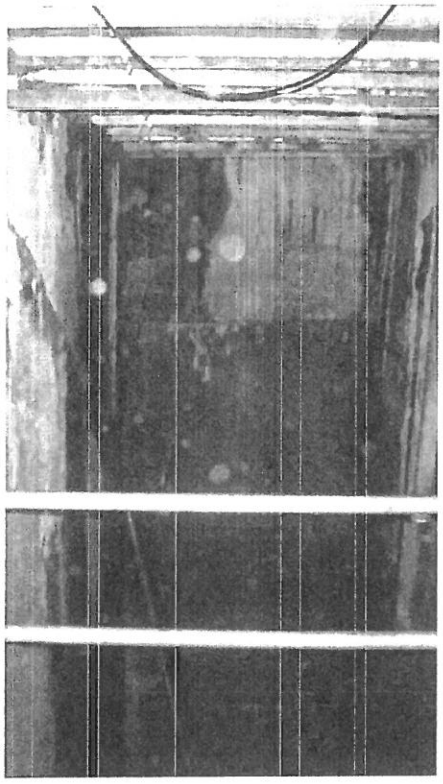
* F.O. = Fully Open

RESERVOIR ELEVATION	GATE #1 GATE OPENING	GATE #2 GATE OPENING	GATE #3 GATE OPENING	GATE #4 GATE OPENING	GATE #5 GATE OPENING	GATE #6 GATE OPENING	GATE #7 GATE OPENING
69.60	F.O.	F.O.	F.O.	F.O.	F.O.	13 - 0	F.O.
69.70	F.O.	F.O.	F.O.	F.O.	F.O.	13 - 0	F.O.
69.80	F.O.	F.O.	F.O.	F.O.	F.O.	13 - 0	F.O.
69.90	F.O.	F.O.	F.O.	F.O.	F.O.	13 - 0	F.O.
70.00	F.O.	F.O.	F.O.	F.O.	F.O.	13 - 0	F.O.
70.10	F.O.	13 - 0	F.O.	F.O.	F.O.	13 - 0	F.O.
70.20	F.O.	13 - 0	F.O.	F.O.	F.O.	13 - 0	F.O.
70.30	F.O.	13 - 0	F.O.	F.O.	F.O.	13 - 0	F.O.
70.40	F.O.	13 - 0	F.O.	F.O.	13 - 0	13 - 0	F.O.
70.50	F.O.	13 - 0	F.O.	F.O.	13 - 0	13 - 0	F.O.
70.60	F.O.	13 - 0	F.O.	F.O.	13 - 0	13 - 0	F.O.
70.70	F.O.	13 - 0	13 - 0	F.O.	13 - 0	13 - 0	F.O.
70.80	F.O.	12 - 10	12 - 10	F.O.	12 - 10	12 - 10	F.O.
70.90	F.O.	12 - 7	12 - 7	F.O.	12 - 7	12 - 7	F.O.
71.00	F.O.	12 - 5	12 - 5	F.O.	12 - 5	12 - 5	F.O.
71.10	F.O.	12 - 2	12	F.O.	12 - 2	12 - 2	F.O.
71.20	F.O.	12 - 0	12	F.O.	12 - 0	12 - 0	F.O.
71.30	F.O.	11 - 7	11	F.O.	11 - 7	11 - 7	F.O.
71.40	F.O.	11 - 5	11 - 5	F.O.	11 - 5	11 - 5	F.O.
71.50	F.O.	11 - 1	11 - 1	F.O.	11 - 1	11 - 1	F.O.
71.60	F.O.	10 - 10	10 - 10	F.O.	10 - 10	10 - 10	F.O.
71.70	F.O.	10 - 7	10 - 7	F.O.	10 - 7	10 - 7	F.O.
71.80	F.O.	10 - 5	10 - 5	F.O.	10 - 5	10 - 5	F.O.
71.90	F.O.	10 - 1	10 - 1	F.O.	10 - 1	10 - 1	F.O.
72.00	F.O.	9 - 10	9 - 10	F.O.	9 - 10	9 - 10	F.O.
72.10	F.O.	9 - 6	9 - 6	F.O.	9 - 6	9 - 6	F.O.
72.20	F.O.	9 - 1	9 - 1	F.O.	9 - 1	9 - 1	F.O.
72.30	F.O.	8 - 11	8 - 11	F.O.	8 - 11	8 - 11	F.O.
72.4 & GREATER	F.O.	F.O.	F.O.	F.O.	F.O.	F.O.	F.O.

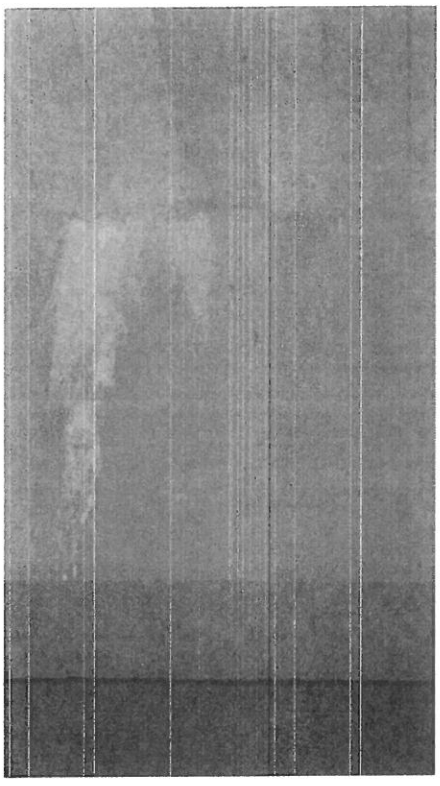
Tainter Gate Operation Guide

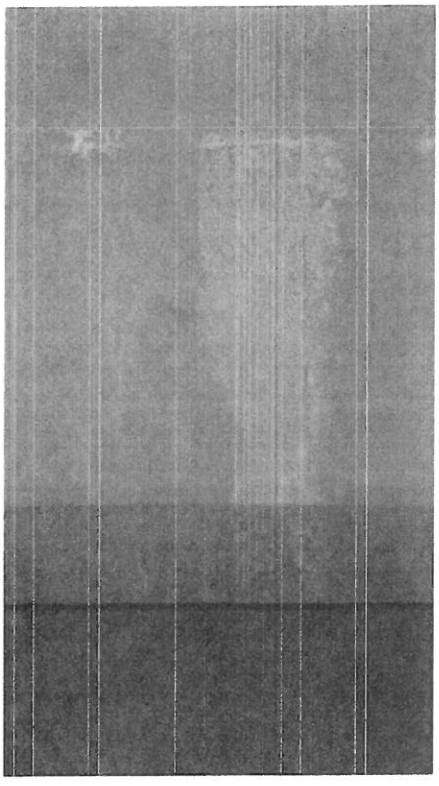


Gated Spillway Gallery Entrance



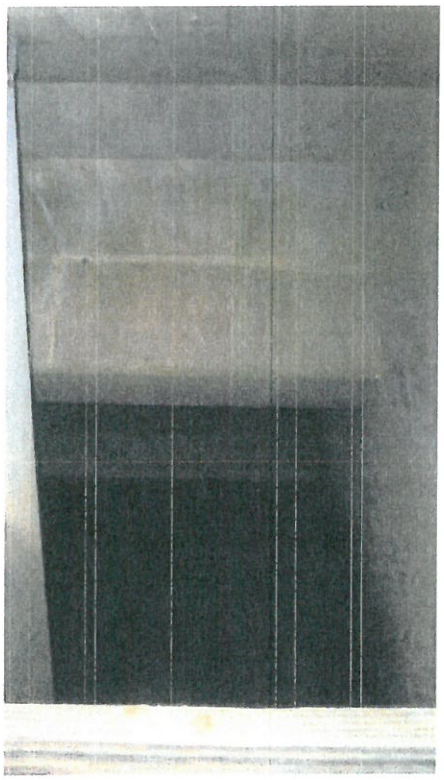
Gated Spillway Gallery Entrance

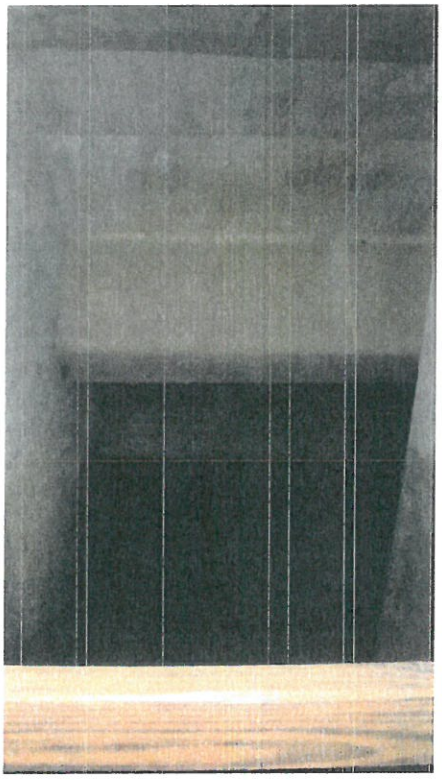


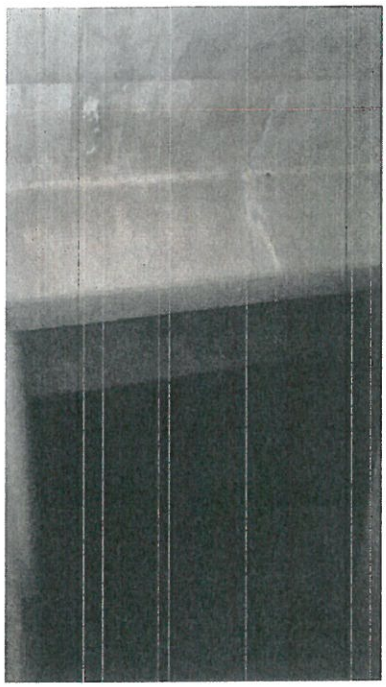


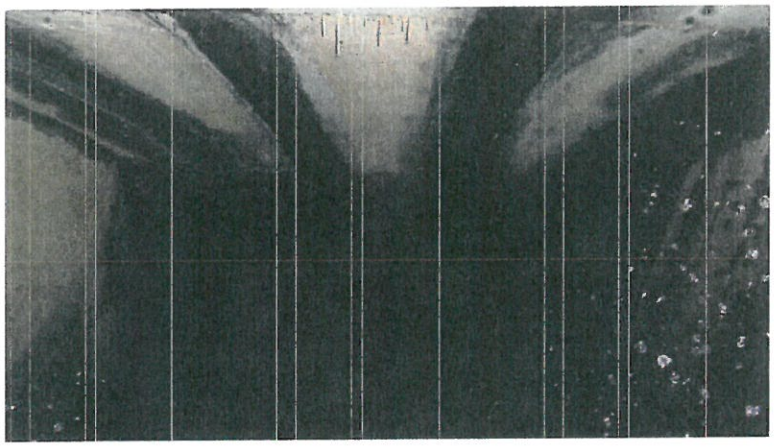


Gated Spillway Gallery Area 5B



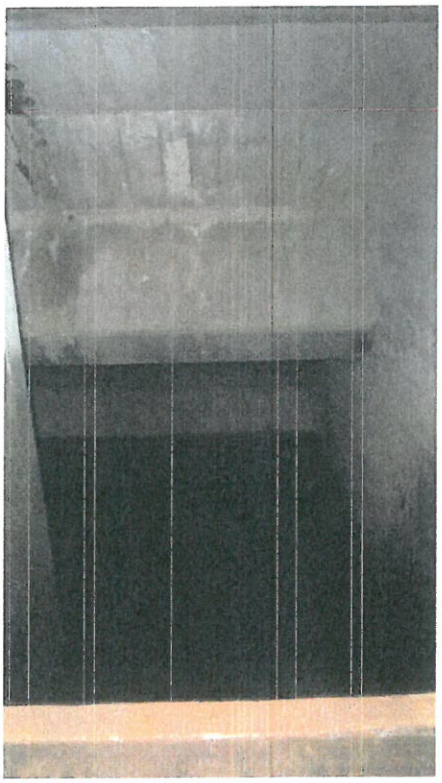




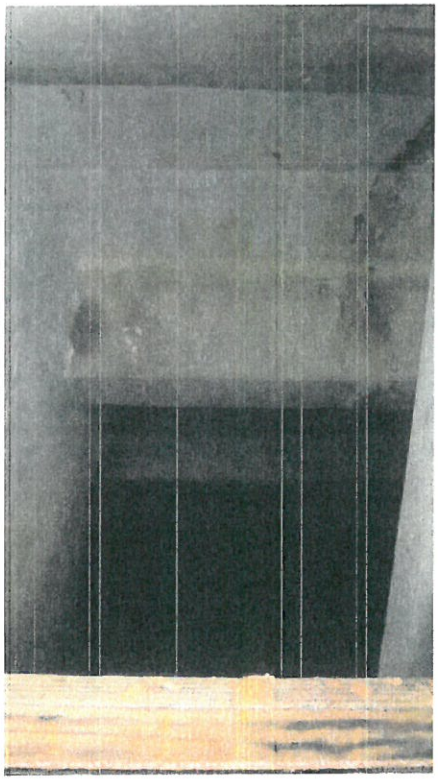


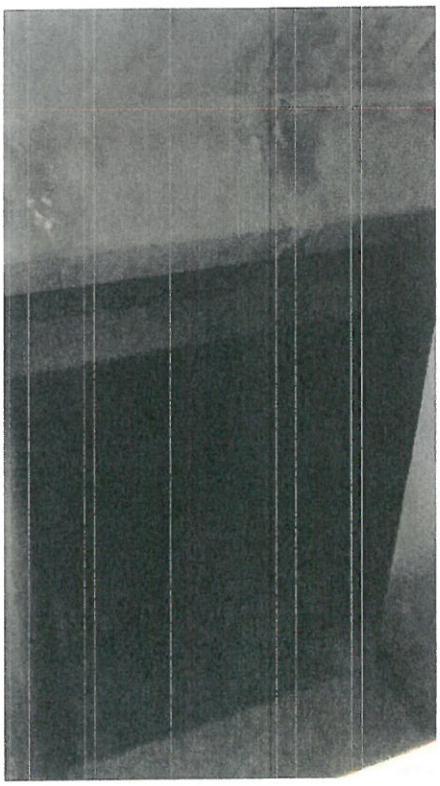


Gated Spillway Gallery Area 6A





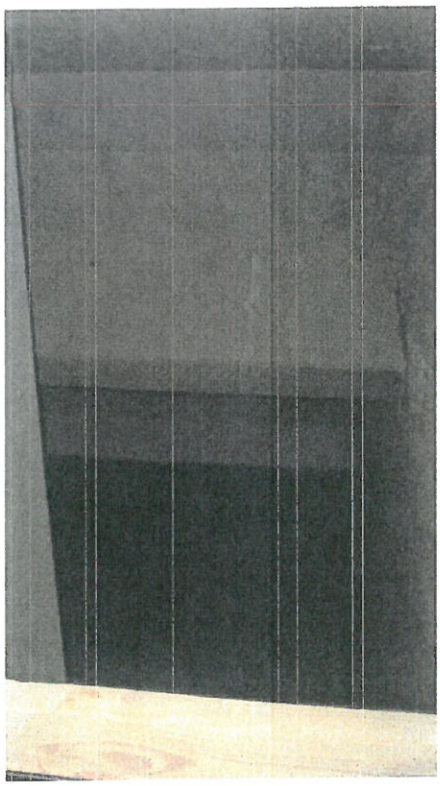


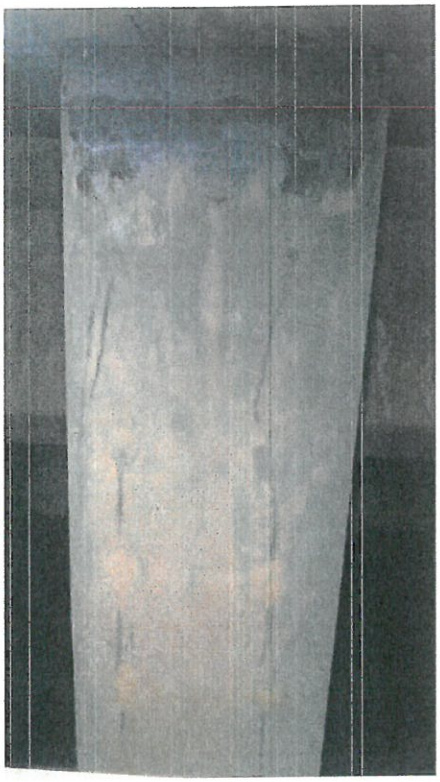


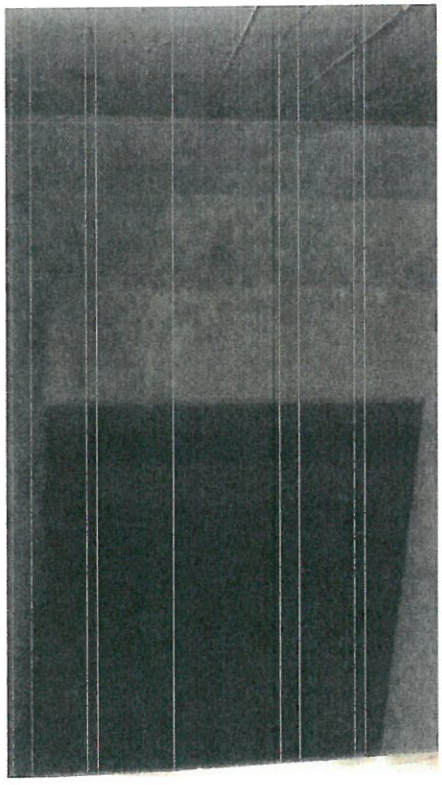




Gated Spillway Gallery Area 5B

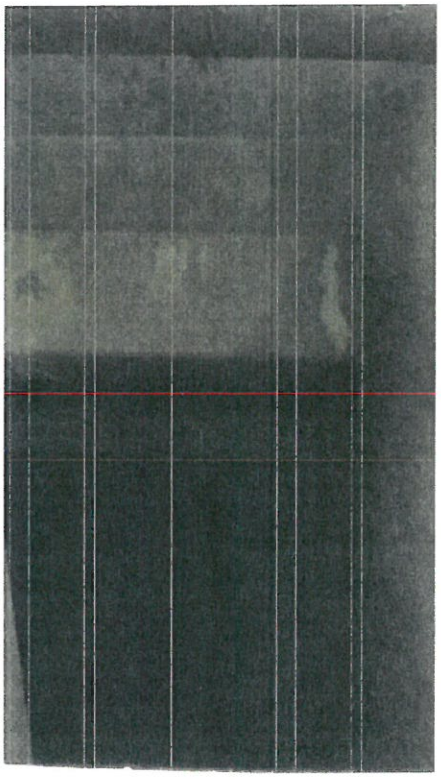


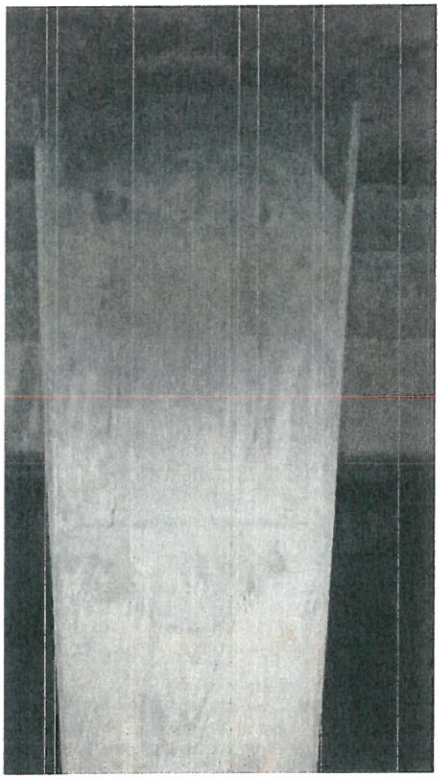


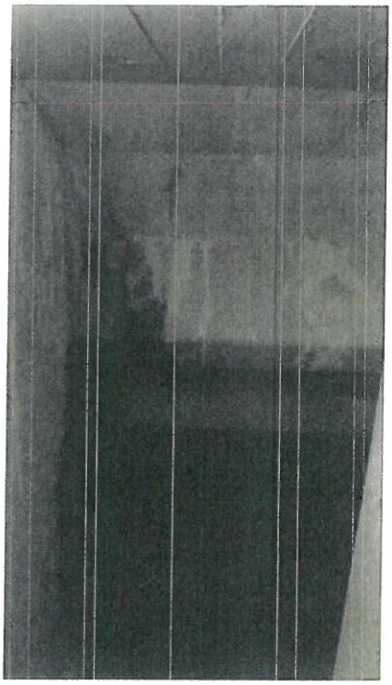


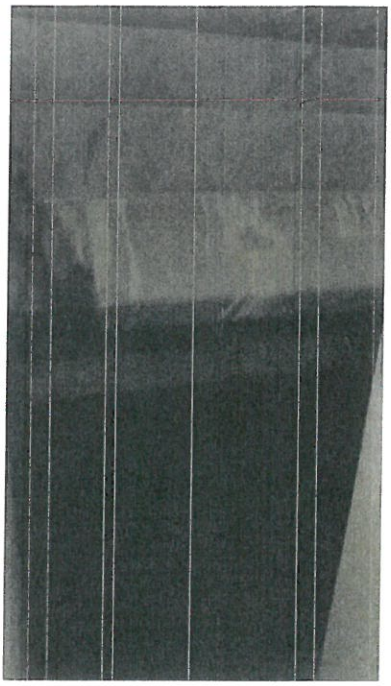


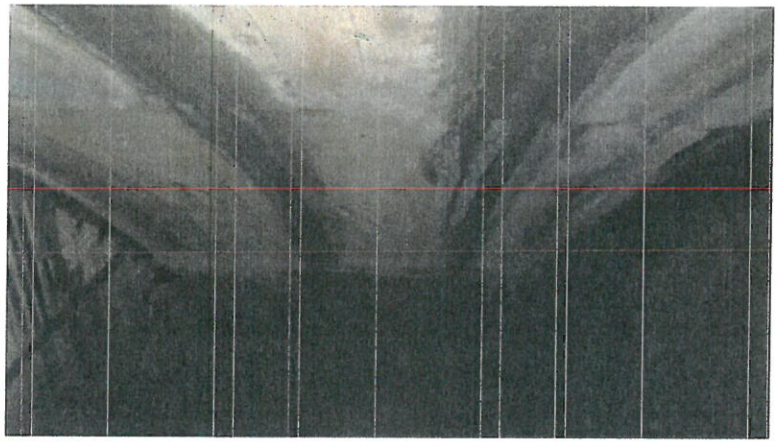
Gated Spillway Gallery Area 5A





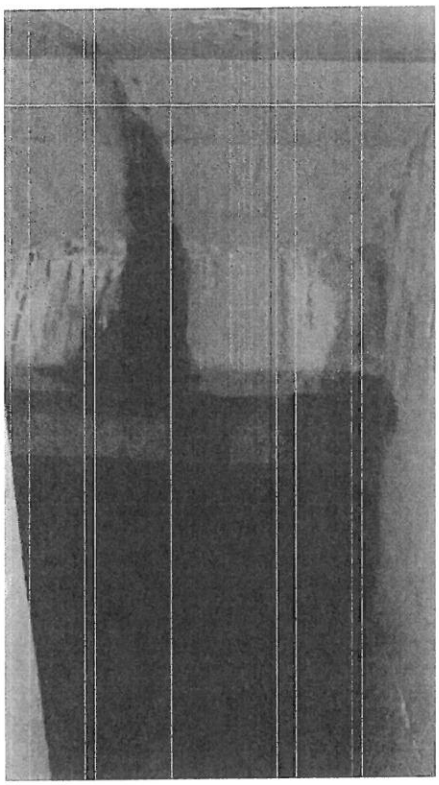


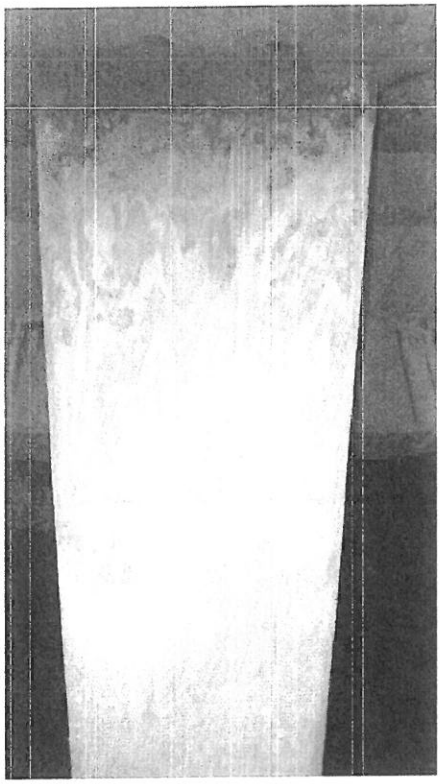


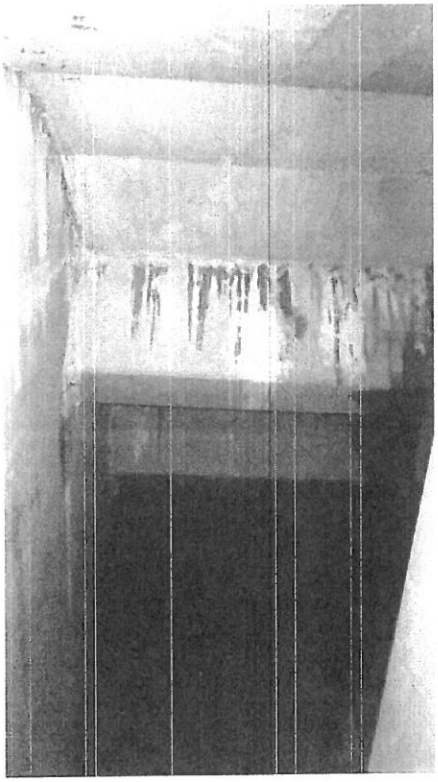


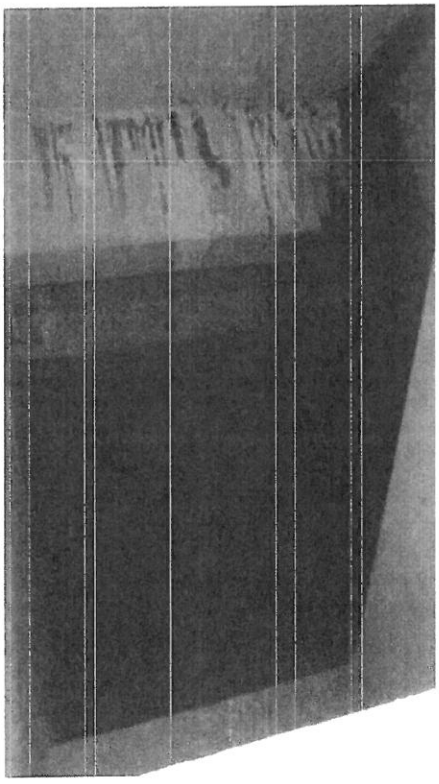


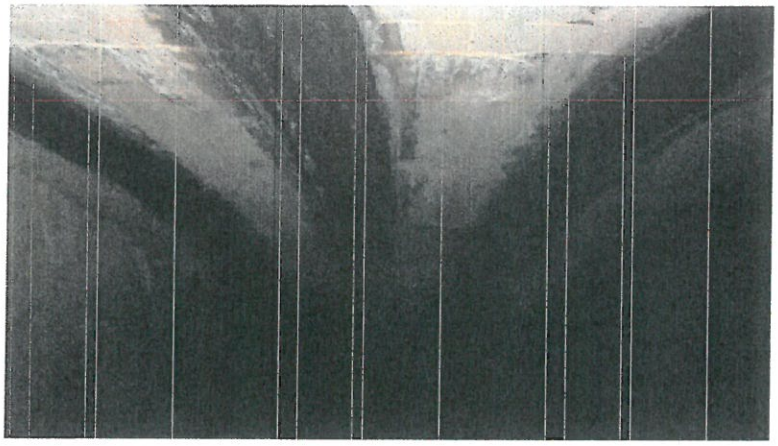
Gated Spillway Gallery Area 4B

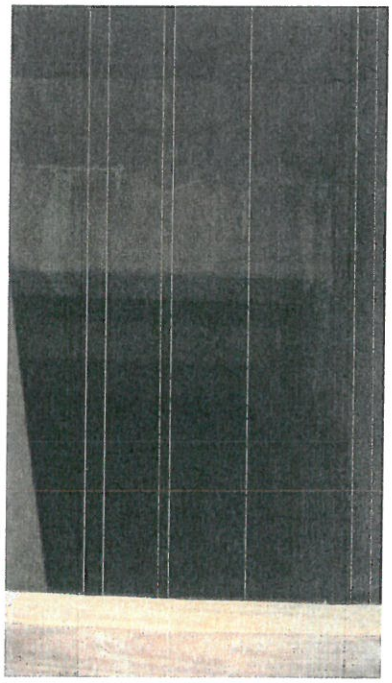






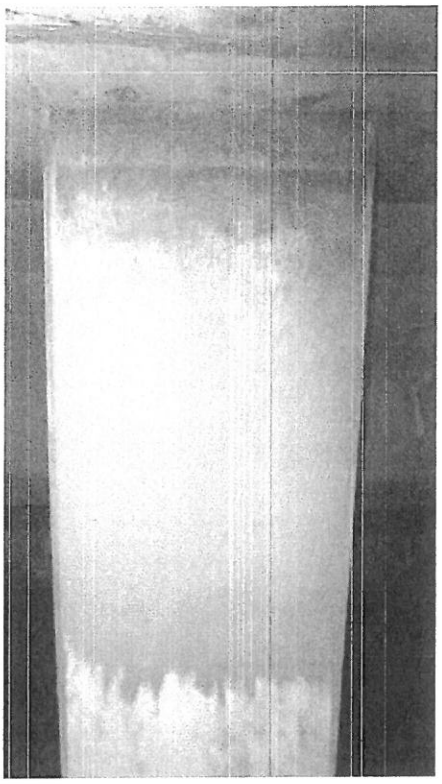


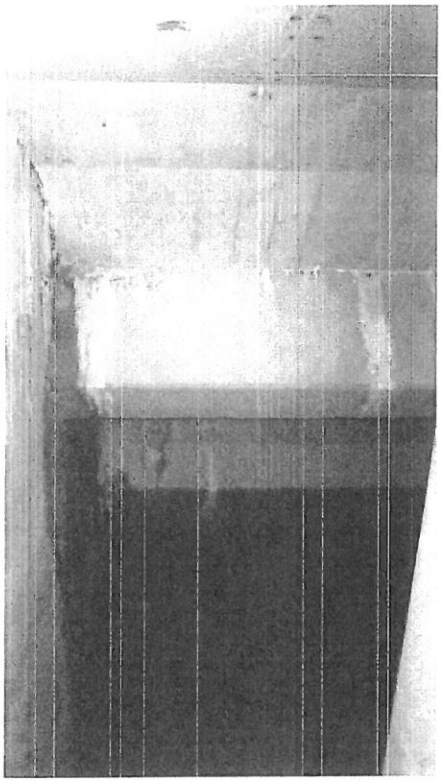


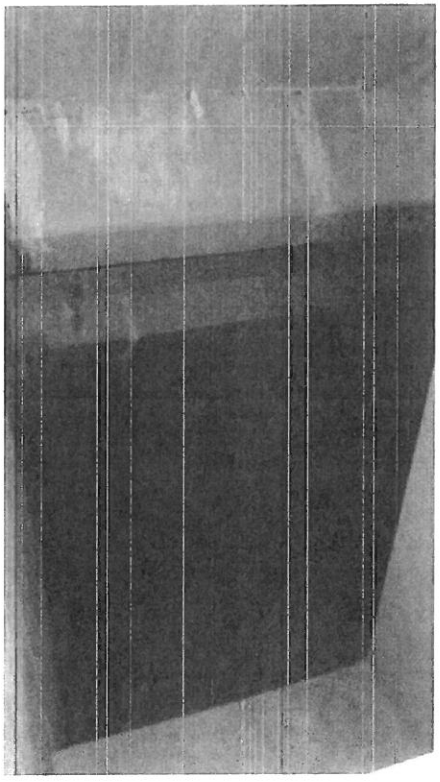


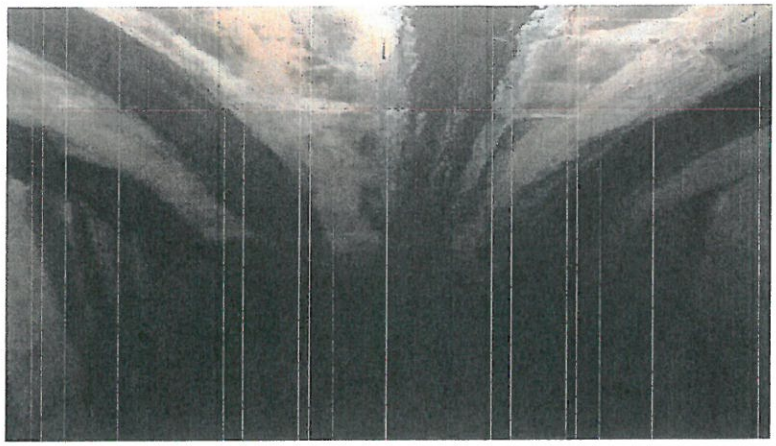


Gated Spillway Gallery Area 4A



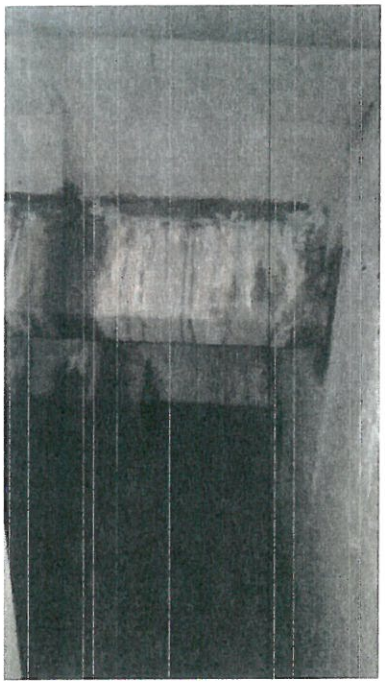


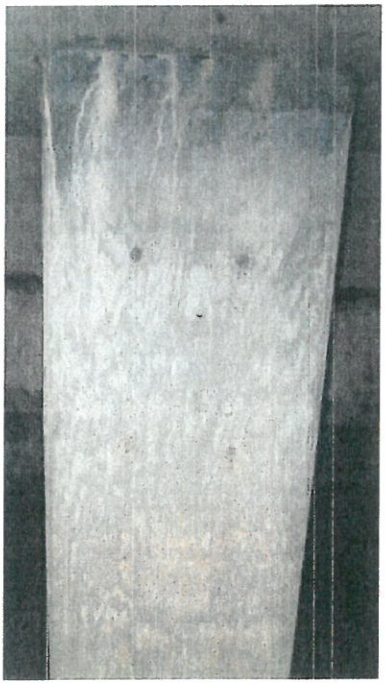


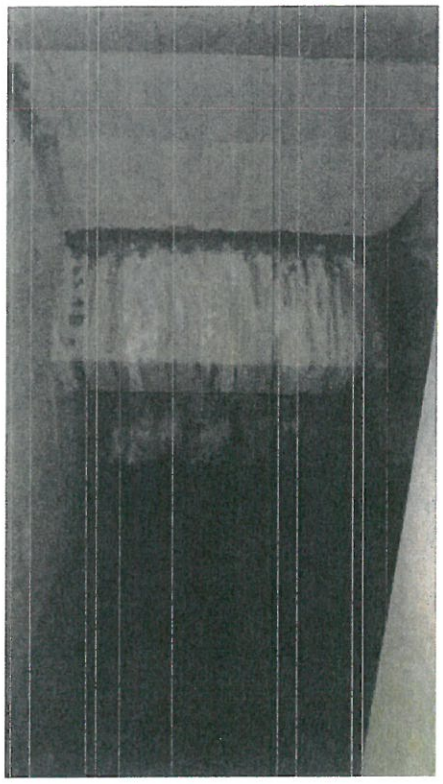


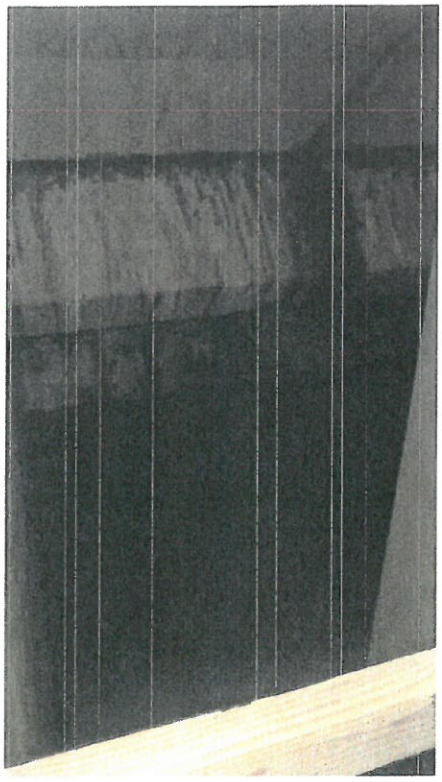


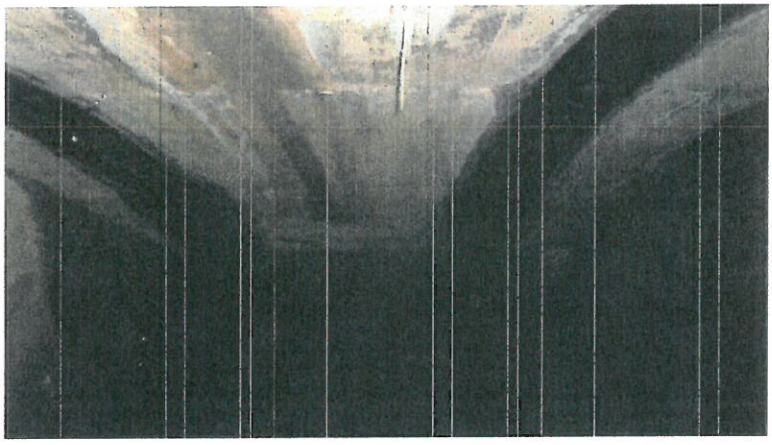
Gated Spillway Gallery Area 3B









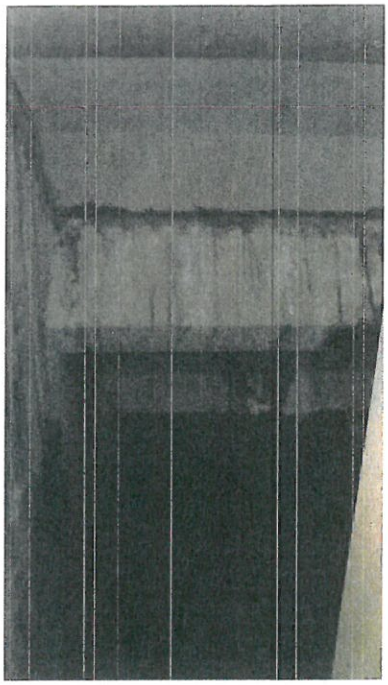


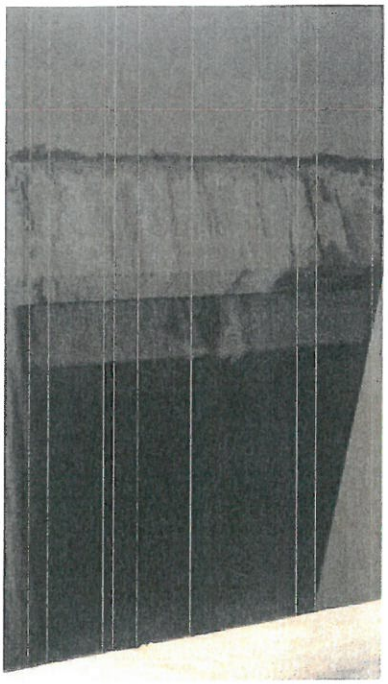


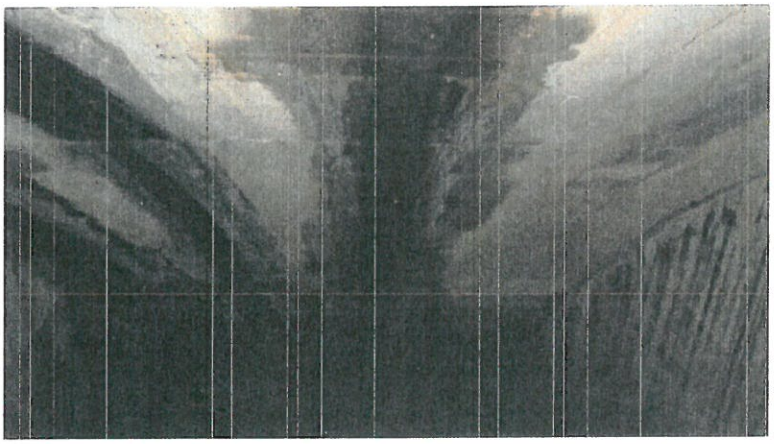
Gated Spillway Gallery Area 3A







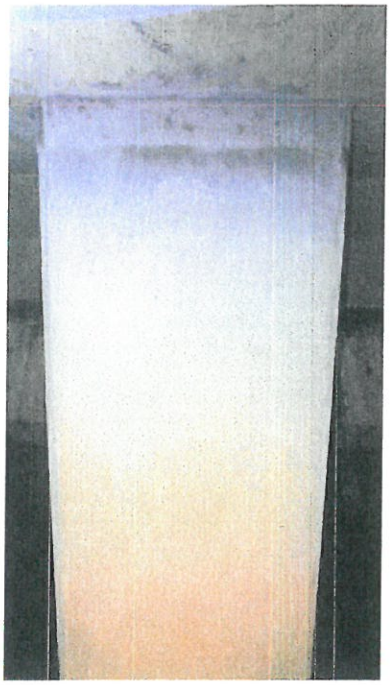


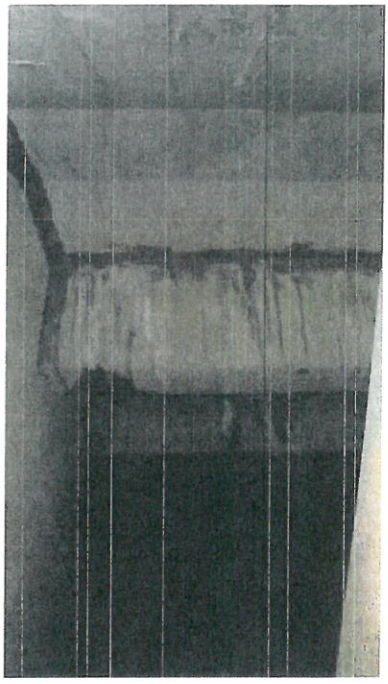


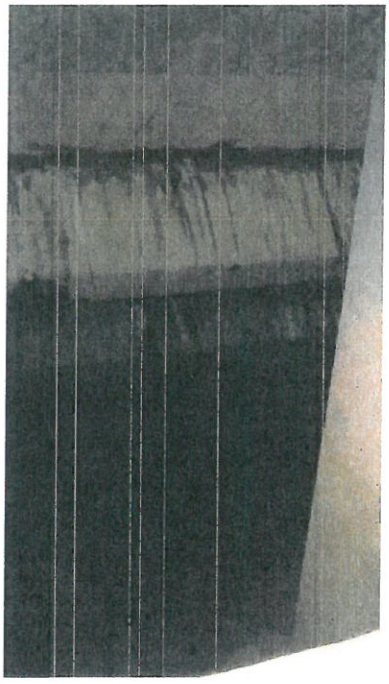


Gated Spillway Gallery Area 2B





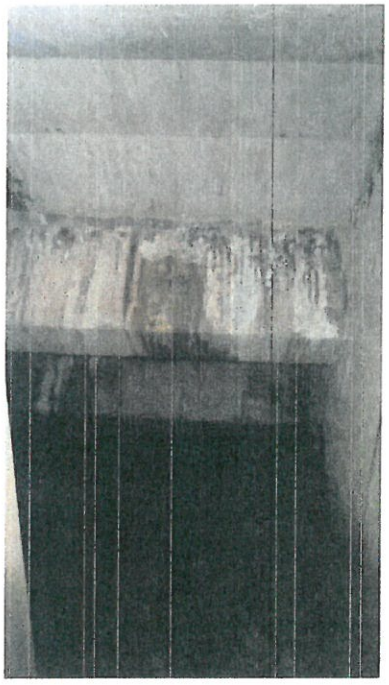


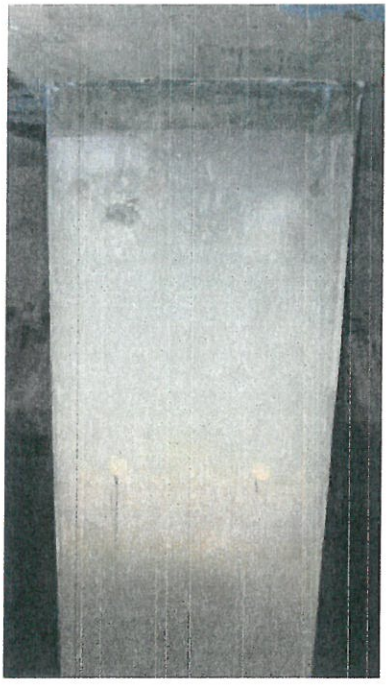


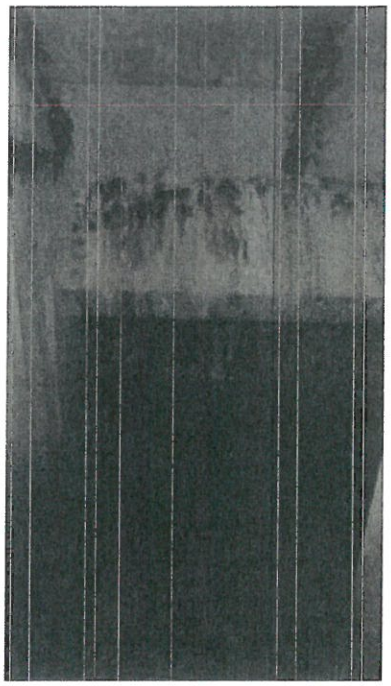




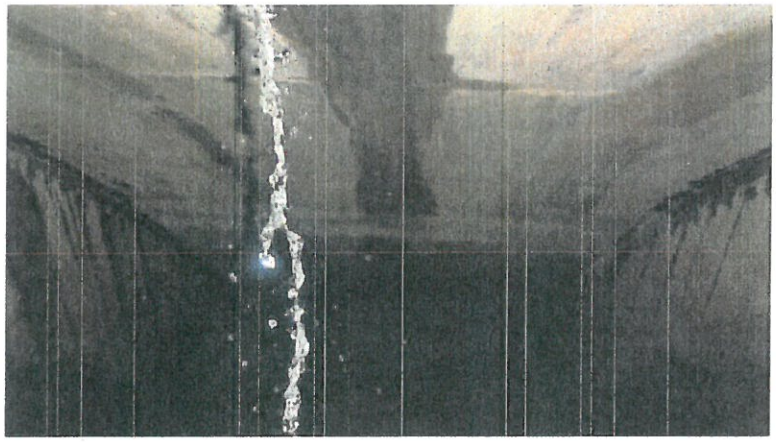
Gated Spillway Gallery Area 2A





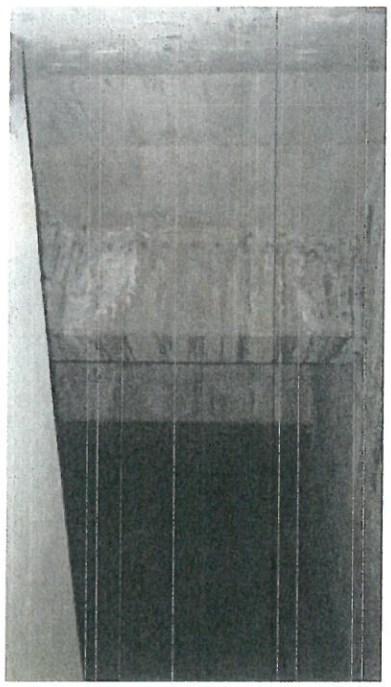


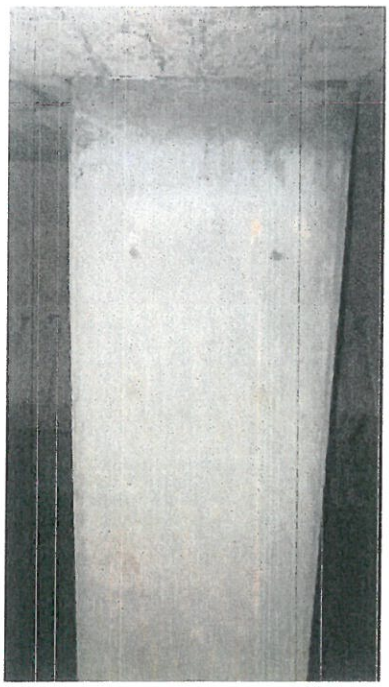




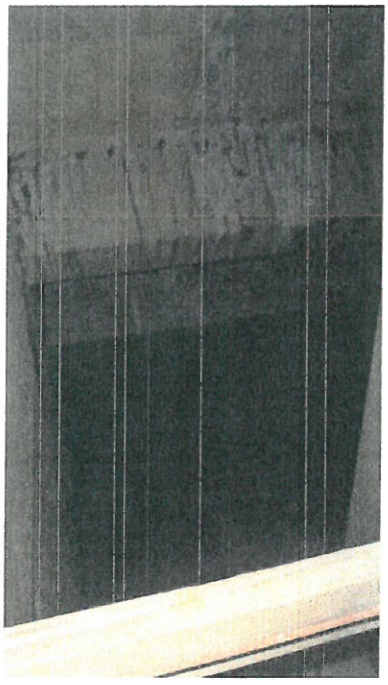


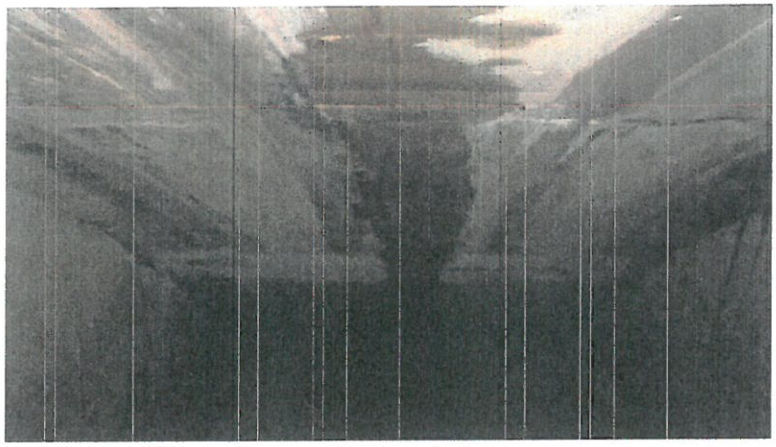
Gated Spillway Gallery Area 1B





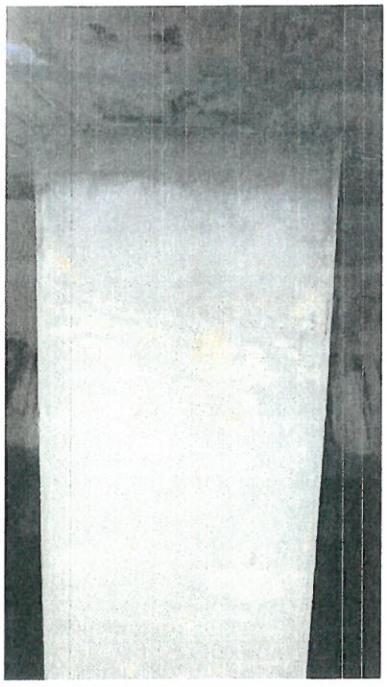


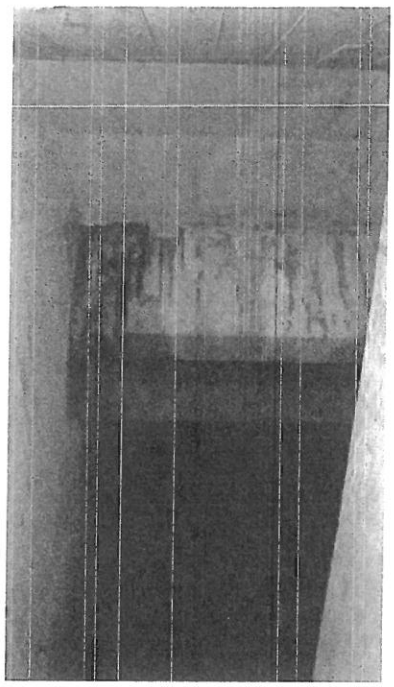


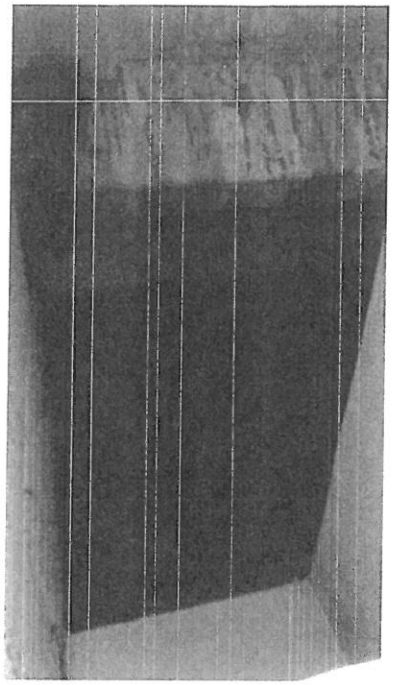


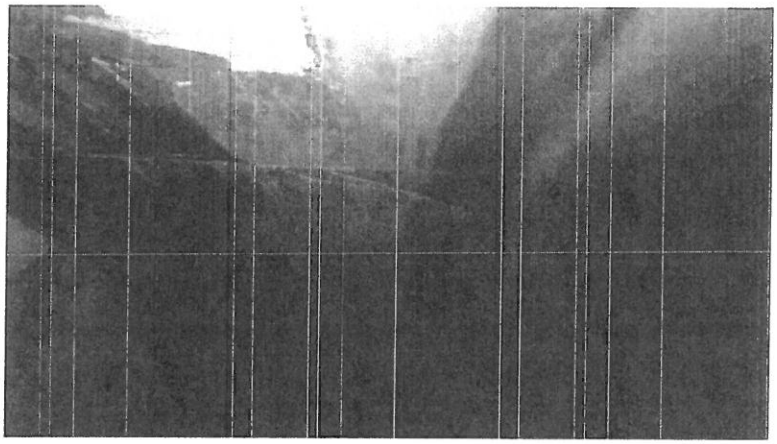


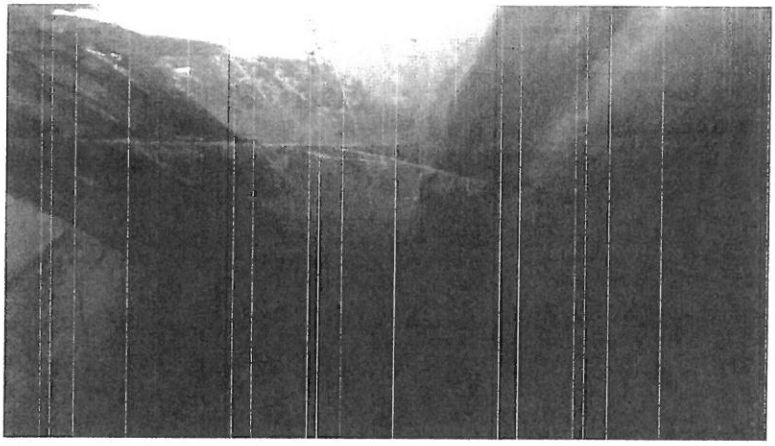


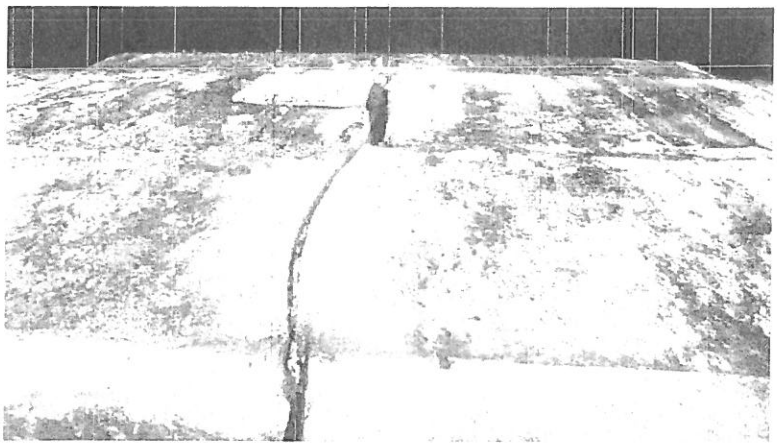










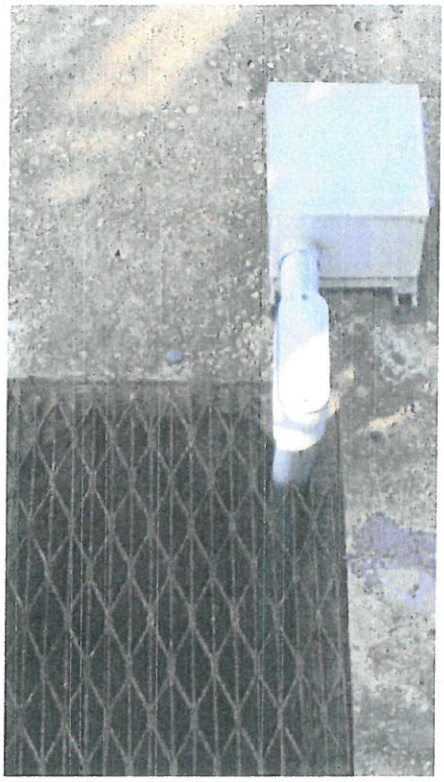


Tailrace Landside Retaining Wall DS of Powerhouse



Tailrace Landside Retaining Wall DS of Powerhouse





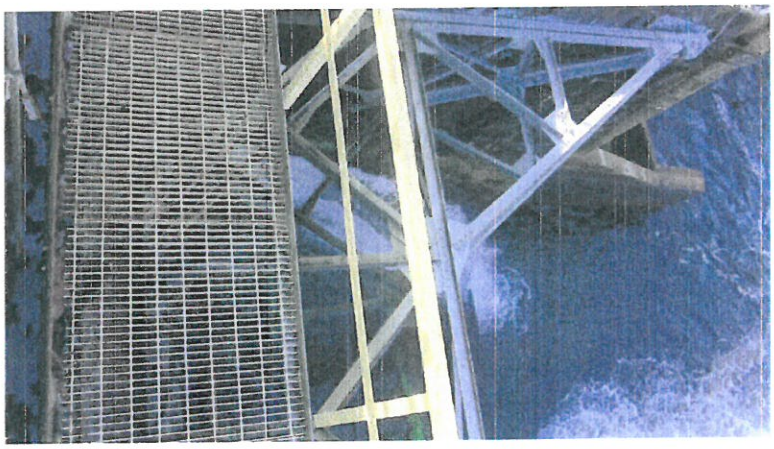
Tainter Gate Trunnion Wall/Electrical Supply



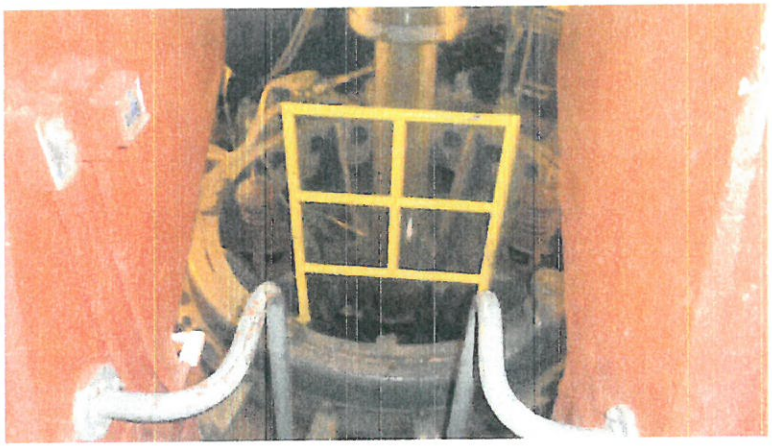
Tainter Gate Trunnion Wall



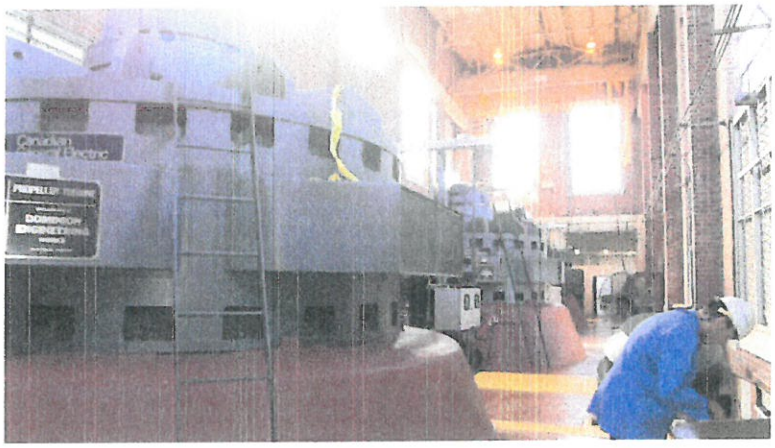
Tainter Gate Structural Members (After 2005-2006 Remediation)



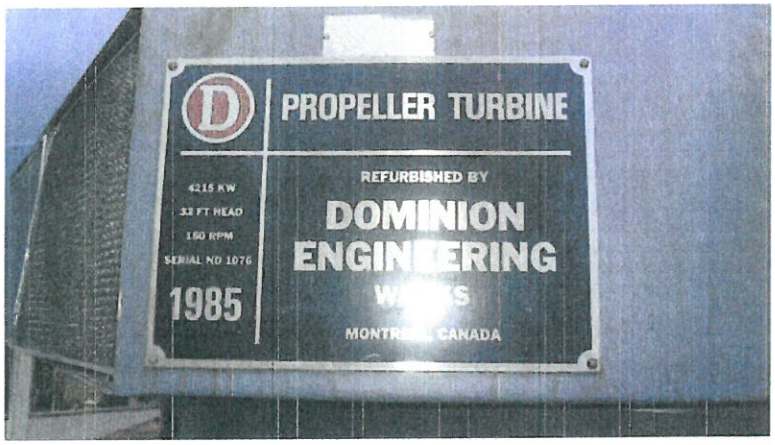
Tainter Gate Structural Members (After 2005-2006 Remediation)



Generator Access



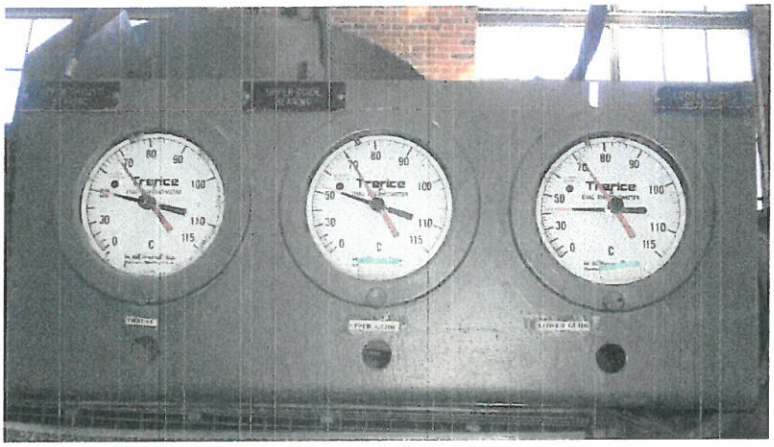
Inside Powerhouse - Generator Lineup



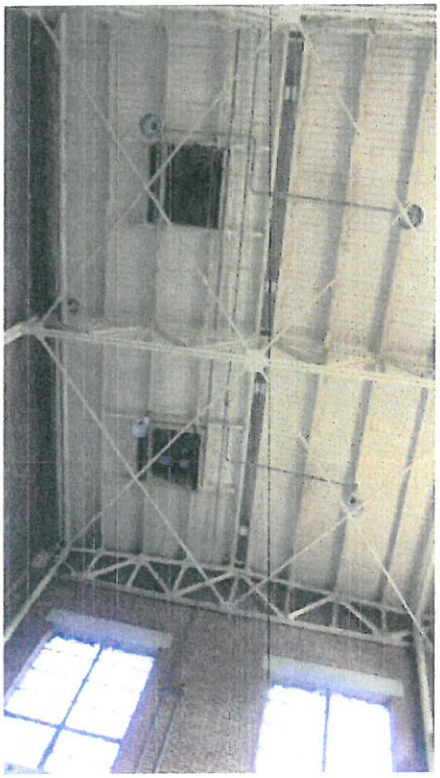
Nameplate for Turbine Unit

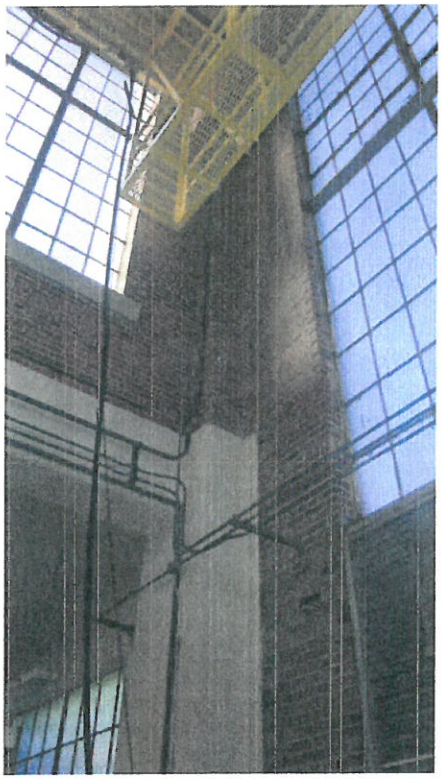


Plant Control System Touchscreen

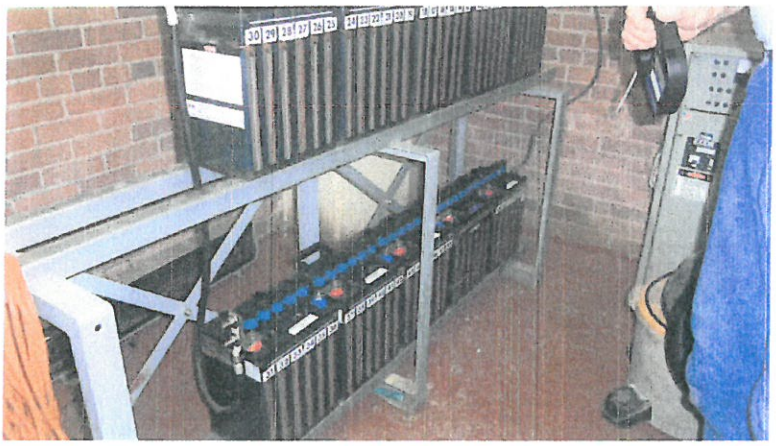


Thrust and Guide Bearing Temperature Gauges

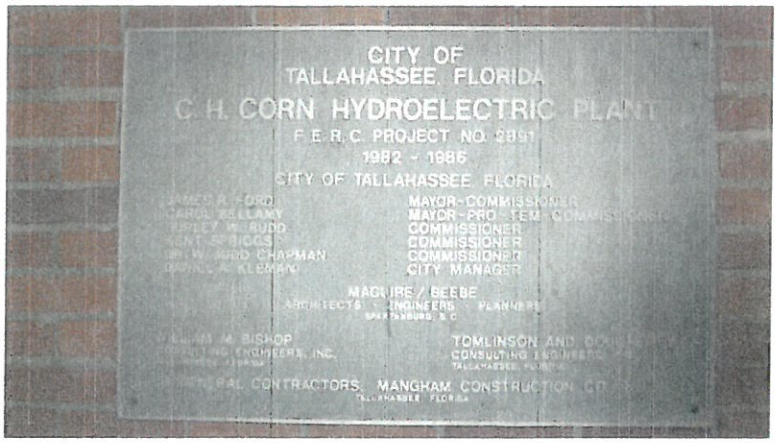




Structural Review of Roof Hatches and Powerhouse Columns



Battery Rack



City of Tallahassee Dedication Plaque



Overview of Generator Gallery



Switchyard



Tie-Back Retaining Wall for Switchyard Fill

APPENDIX F

Inspection Checklists and/or Field Notes

APPENDIX G

Operation and Maintenance Documentation

EXISTING

CITY OF TALLAHASSEE
JACKSON BLUFF HYDROELECTRIC PROJECT
PIEZOMETER WELL AND CULVERT READINGS

DATE: 5-1-2010
TIME: 5:00 P.M.

WELL NUMBER	TOC to WATER (ft)	W or D	TOC ELEVATION	WATER ELEVATION	TOC to SURFACE	DRY COVERAGE	MIN DRY COVERAGE
1B	24.3	W	76.44	76.44	1.54	-1.5	12.2
2B	14.65	D	65.60	65.60	1.70	-1.7	5.1
3B	2.95	D	60.30	60.30	1.80	-1.8	2.7
0A	20.83	W		0.00		.0	N/A
1A	25.57	W	76.98	76.98	1.88	-1.7	12.6
2A	15.21	D	68.47	68.47	2.07	-2.1	5.6
3A	9.82	D	60.92	60.82	1.57	-1.8	3.4
15	27.47			0.00		.0	
3	23.30	W	72.73	72.73	1.18	-1.2	11.0
4	11.02	D	62.81	62.81	1.81	-1.6	6.1
9	3.62	W		0.00		.0	
14	16.85			0.00		.0	
5A	14.73	W	75.96	75.96	2.08	-2.1	12.7
6A	10.92	W	69.11	69.11	1.81	-1.8	9.1
7A	6.11	W	63.95	63.95	2.03	-2.0	6.7
11	1.10	W		0.00		.0	
5	13.89	W	76.60	76.80	1.80	-1.8	11.2
6	13.93	W	72.27	72.27	2.27	-2.3	10.0
7	9.41	W	61.77	61.77	1.07	-1.1	5.9
8	1.10	W		0.00		.0	N/A
12	3.10			0.00		.0	
13	3.85			0.00		.0	
P2	12.05		75.80	75.83	-0.32	.3	
P3	22.45		71.71	71.71	3.27	-3.3	
P4	16.32		57.55	57.55	3.21	-3.2	
P5	3.20		50.66	50.66	3.21	-3.2	
P6	11.55		49.94	49.94	2.70	-2.7	
P7	18.14		76.06	76.06	-0.32	.3	
P8	26.92		71.73	71.73	3.19	-3.2	
P9	17.73		60.09	60.09	3.19	-3.2	
P10	5.52		50.26	50.26	2.88	-2.9	
P11	6.57		49.78	49.78	2.80	-2.8	

D=DRY M=MUD W=WATER

CULVERTS BEHIND DAM		W or D
NO.1		D
NO.2		W semi wet
NO.3		W
NO.4	1 GPM	W
NO.5	2 GPM	W
NO.6	3 GPM	W
NO.7	5 GPM	W
CROW	10 GPM	W

Rivers-up to Crows

SPILLWAY PIEZOMETERS			
SPILLWAY BAY#	TOC to WATER	TOC ELEV.	WATER ELEVATION
1A	7.50	48.35	48.35
1B	16.695	22.26	22.26
1B Temperature	54.5		
4B	19.72	48.28	48.28
5A	17.682	20.76	20.76
5A Temperature	52.43		
7B	31.61	48.35	48.36
ROAD	23.17	72.98	72.98

* At top of Riser

RAINFALL

DATE	AMOUNT	DATE	AMOUNT
4-8	1.15		
4-19	1.40		
4-20	2.04		
4-27	.10		
4-30	2.99		

TAIL WATER ELEV. 39.27
LAKE ELEVATION 68.80
TOTAL RAINFALL 6.08

OPERATOR

COMMENTS:
#1B Reading is below bottom of well. #5A Well needs to be looked at.
#8 14 from top of Riser.
#12 14 1/2 from top of Riser.

ANNUAL SPILLWAY GATE OPERATION CERTIFICATE

FERC PROJECT NO. 2891, NAME JACKSON BLUFF DAM

DEVELOPMENT NAME C.H CORN HYDRO DAM ELECTRIC PROJECT

LICENSEE/EXEMPTEE/APPLICANT CITY OF TALLAHASSEE,FLORIDA

Gate No. [1]	Date of Opening		Purpose [3]	Opening (ft)	Voltage [4] (line to line)	Phase [4] Current	Rated [4] Horsepower
	Current	Last Fall [2]					
1	12/23/09	3/1/08	TESTING	1	480	3	6.0
2	12/23/09	3/1/08	TESTING	1	480	3	6.0
3	12/23/09	3/1/08	TESTING	1	480	3	6.0
4	12/23/09	3/1/08	TESTING	1	480	3	6.0
5	12/23/09	3/1/08	TESTING	1	480	3	6.0
6	12/23/09	3/1/08	TESTING	1	480	3	5.0
7	12/23/09	3/1/08	TESTING	1	480	3	5.0

PROBLEMS NOTED/MAINTENANCE REQUIRED: NONE

The above gate(s) was (were) operated as indicated by: DAVID MCCRANIE
(Signature of Operator)

TYPE OF EMERGENCY OPERATION

_____Manual

__X__Standby power source – Describe 100 KW DIESEL GENERATOR

Date of last test: 12/21/09
Gate operator: WILLIAM POWELL
Problems noted/Maintenance required: NONE

The standby power source was load-tested as indicated by: WILLIAM POWELL
(Signature of Operator)