Stantec Consulting Services Inc.
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October 12, 2016
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Revision 0

Tennessee Valley Authority
1101 Market Street
Chattanooga, Tennessee 37402

RE: Initial Structural Stability Assessment
East Ash Disposal Area
EPA Final Coal Combustion Residuals (CCR) Rule
TVA Allen Fossil Plant
Memphis, Tennessee

1.0 PURPOSE

This letter documents Stantec's certification of the initial structural stability assessment for the TVA Allen Fossil Plant's (ALF) East Ash Disposal Area. Based on this assessment, the East Ash Disposal Area is in compliance with the structural stability requirements in the EPA Final CCR Rule at 40 CFR 257.73(d).

2.0 INITIAL STRUCTURAL STABILITY ASSESSMENT

As described in 40 CFR 257.73(d), documentation is required on how the East Ash Disposal Area has been designed, constructed, operated, and maintained according to the structural stability requirements listed in the section. The combined capacity of all spillways must also be designed, constructed, operated, and maintained to adequately manage flow from the 1000-year storm event based upon a hazard potential classification of "significant."

3.0 SUMMARY OF FINDINGS

The attached report presents the initial structural stability assessment of the East Ash Disposal Area. The results show that the impoundment meets the structural stability requirements set forth in 40 CFR 257.73(d)(1)-(2).

4.0 QUALIFIED PROFESSIONAL ENGINEER CERTIFICATION

I, Stephen H. Bickel, being a Professional Engineer in good standing in the State of Tennessee, do hereby certify, to the best of my knowledge, information, and belief:

1. that the information contained in this certification is prepared in accordance with the accepted practice of engineering;

2. that the information contained herein is accurate as of the date of my signature below; and

3. that the initial structural stability assessment for the TVA Allen Fossil Plant's East Ash Disposal Area meets the requirements specified in 40 CFR 257.73(d)(1)-(2).

Re: **Initial Structural Stability Assessment**
East Ash Disposal Area
EPA Final Coal Combustion Residuals (CCR) Rule
TVA Allen Fossil Plant
Memphis, Tennessee

SIGNATURE ______________________ DATE 10/12/2016

ADDRESS: Stantec Consulting Services Inc.
10509 Timberwood Circle, Suite 100
Louisville, Kentucky 40223-5308

TELEPHONE: (502) 212-5075

ATTACHMENTS: Initial Structural Stability Assessment Report

Initial Structural Stability Assessment

Allen Fossil Plant –
East Ash Disposal Area,
Tennessee

Prepared for:
Tennessee Valley Authority

Prepared by:
Stantec Consulting Services Inc.
Lexington, Kentucky

October 12, 2016
Revision 0

Table of Contents

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 PROJECT BACKGROUND

On April 17, 2015 the "Disposal of Coal Combustion Residuals (CCR) from Electric Utilities" (EPA Final CCR Rule) was published in the Federal Register (USEPA, 2015). Stantec Consulting Services, Inc. (Stantec) was contracted by the Tennessee Valley Authority (TVA) to analyze the Structural Stability of the East Ash Disposal Area for Allen Fossil Plant (ALF) CCR surface impoundments (SI) and evaluate compliance with section §257.73(d) of the CCR Rule.

As required by §257.73(d) of the EPA Final CCR Rule, an initial structural integrity evaluation is required by October 17, 2016. The evaluation must include an initial structural stability assessment for each existing CCR surface impoundment that meets the conditions of paragraph (b) as follows:

1. Has a height of five feet or more and a storage volume of 20 acre-feet or more or
2. Has a height of 20 feet or more.

2.0 UNIT DESCRIPTION

The ALF is a coal-fired, electric generating plant. The plant is located in Shelby County Tennessee, near McKellar Lake adjacent to the Mississippi River. The plant is located immediately west of Memphis, Tennessee.

The East Ash Disposal Area is located at the east side of the plant. It is bordered on the north by the Mississippi River and the south by Riverport Road. The East Ash Disposal Area encompasses approximately 70 acres and is formed by three dikes discussed in section 2.1 below. TVA has determined that the East Ash Pond and East Ash Stilling Pond ("referred to herein as the "East Ash Disposal Area") are CCR surface impoundments and therefore subject to the CCR rule.

Figure 1 provides an overview of the East Ash Disposal Area and appurtenances.

Elevations included in this document and appendices are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29).

2.1 EMBANKMENTS

2.1.1 USACE Levee

The United States Army Corps of Engineers (USACE) Levee is located along the northern boundary of the East Ash Disposal Area separating it from McKellar Lake. According to Dewberry (2013), the USACE Levee was originally constructed in the 1960s using soil from within the footprint of the East Ash Disposal Area. In the 1970s, TVA expanded the East Ash Disposal Area by constructing the East Stilling Pond and raising the height of the pond embankments (including the USACE Levee part of the pond structure).

2.1.2 Railroad Embankment

The Railroad Embankment follows an east-west alignment and forms the southern containment for the East Ash Disposal Area. An active rail line, plant access road, and East Ash Disposal Area access road are located along the embankment crest.

Limited information on the history of the railroad embankment was available for review. It is assumed that the embankment was built during the period when the Allen Fossil Plant was built. According to Dewberry (2013), this was during the 1950s. Significant modifications to the railroad embankment since operation of the East Ash Disposal Area began in the 1960s were not identified during historical document review.

Legend

- Sewer Line
- Spillway
- Facility Boundary

0 300 Feet

1:3,600 (At original document size of 11x17)

Stantec

Project Location
Memphis, Shelby County, Tennessee

172675014
Prepared by WSW on 2016-03-08
Technical Review by TG on 2016-03-08
Independent Review by TR on 2016-03-08

Client/Project
Tennessee Valley Authority
Allen Fossil Plant

Figure No.
1

DRAFT

Title
East Ash Disposal Area

Notes
1. Coordinate System: NAD 1983 StatePlane Tennessee FIPS 4100 Feet
2. Imagery Provided By Client (Dated 2015)

U:\172675014_Tim-Moore\gis\mxd\Fig1_EastAshDisposalArea.mxd Revised: 2016-03-08 By: swheatley

2.1.3 Divider Dike

The Divider Dike has a north-south alignment and divides the East Ash Pond from the East Stilling Pond. A gravel access road is located along the crest of the embankment.

According to Dewberry (2013), the Divider Dike was constructed in 1978 as part of the expansion of the East Ash Disposal Area. TVA record drawing 10N226 (1978) indicates bottom ash was the material specified for construction of the divider dike.

2.1.4 East Perimeter Dike

The East Perimeter Dike has a north-south alignment and forms the eastern boundary of the East Stilling Pond/East Ash Disposal Area. A gravel access road is located along the crest of the embankment. According to Dewberry (2013), the East Perimeter Dike was constructed in 1978 as part of the expansion of the East Ash Disposal Area.

2.2 SPILLWAYS

2.2.1 ALF West and ALF East

The ALF West and ALF East spillways are the primary spillways for the East Stilling Pond. The spillways consist of a 48-inch diameter riser with a 36-inch diameter outlet pipe that discharges to McKellar Lake. The riser is reinforced concrete with the lower portion being cast-in-place reinforced concrete and the upper portion being constructed by stacking 2-foot sections of 48-inch diameter reinforced concrete pipe (RCP). The outlet pipe is 36-diameter RCP that discharges through a reinforced concrete outlet structure at the toe of the USACE Levee to a concrete flume. The flume discharges at the top of bank into a rip rap-armored section of the McKellar Lake bank.

2.2.2 ALF-Overflow 1 and ALF-Overflow 2

The ALF Overflow 1 and ALF Overflow 2 spillways are the overflow spillways for the East Stilling Pond. The spillways consist of a 48-inch diameter riser with a 36-inch diameter barrel that discharges to Horn Lake Cutoff. The riser is reinforced concrete with the lower portion being cast-in-place reinforced concrete and the upper portion being constructed by stacking 2-foot sections of 48-inch diameter reinforced concrete pipe (RCP). The overflow spillways have one additional 2-foot section of 48-inch diameter pipe as compared to ALF West and ALF East Risers. This additional riser section prevents the Overflow Spillways from operating during normal flow conditions.

2.2.3 Overflow Weir

The overflow weir is located on the southern end of the Divider Dike, and it discharges directly to the East Stilling Pond. The overflow weir is a reinforced concrete drop spillway that controls the water surface elevation in the East Ash Pond. The crest elevation of the weir is 222.1 feet, and its length is 28 feet. Stop logs can be added to increase the water surface elevation in the East Ash Pond up to elevation 232 feet. A splitter wall is located in the center of the spillway and it is aligned with the direction of the flow. This allows each half of the weir to be configured at different elevations with stop logs. The inlet and outlet for the spillway has 45^{o} degree angle, sloping reinforced concrete wing walls.

2.3 HYDRAULIC STRUCTURES

2.3.1 60-inch Diameter Sanitary Sewer Line

The 60-inch diameter RCP sanitary sewer line was built during the 1950s, prior to the Allen Fossil Plant construction. The City of Memphis stopped using the line during the 1970s. This gravity sewer line was originally built approximately 20 feet below grade on an east-west alignment. It is buried beneath the southern portion of the East Ash Disposal Area, approximately 400 feet north from the East Ash Disposal Area south limit. The length of this pipe, within the East Ash Disposal Area limits, is approximately 3000 feet.

2.3.2 30/42-inch Diameter Sanitary Sewer Line

This pipeline runs through the west portion of the East Ash Disposal Area in a north-south direction. It enters the East Ash Disposal Area across the USACE Levee crest, and it was constructed in two segments.

The first segment is a 30-inch diameter force main which crosses beneath McKellar Lake and conveys wastewater flows from the Presidents Island Industrial Park. It was installed during the 1970s using Ductile Iron pipe (DIP). In 2011 approximately 990 feet of the DIP force main was replaced with 30-inch diameter butt-fused High Density Polyethylene (HDPE) pipe. The new HDPE pipe is built at a level above the original section of DIP pipe. Both pipes (active and original) terminate at a concrete junction structure approximately 1000 feet south from the USACE Levee. The original DIP was bulk- headed at the junction structure to prevent a release of CCR into the structure.

The second segment begins at the concrete junction structure where flows from the 30-inch force main continue south through a 42-inch diameter RCP sanitary sewer. The gravity sewer was constructed during the 1970s, and it was lined using cured-in-place pipe (CIPP) during the 1990s. The sewer discharges to a 96-inch sanitary sewer located south of the Railroad Embankment. The length of this pipe, within the East Ash Disposal Area limits, is approximately 900 feet.

3.0 FOUNDATIONS AND ABUTMENTS

Per §257.73(d)(1)(i), the initial structural stability assessment must document whether the unit has been designed, constructed, operated and maintained with stable foundations and abutments (USEPA, 2015). The East Ash Disposal Area has the following features that fall within this requirement:

- USACE Levee
- Railroad Embankment
- Divider Dike
- East Perimeter Dike

Assessment of the foundations and abutments associated with these features considering the following criteria related to the CCR rule:

- Review inspection reports of the facility, considering frequency of inspections, and if the inspections included review and/or assessment of features including cracking, settlement, deformation or erosion of the foundations/abutments. Inspections should indicate that there are no significant signs of tension cracking, settlement, depressions, erosion, and/or deformations at the crest, slope and toe of the structure.
- Confirm that an assessment of seepage conditions of the foundation, with considerations for heave and vertical exit gradient, has been performed. Verify that the seepage assessment follows appropriate methodologies such as USACE (1986) and that the foundations exhibit acceptable factors of safety (i.e. FS for piping greater than or equal to 3.0).

3.1 USACE LEVEE

3.1.1 Background

The USACE Levee, located along the northern portion of the East Ash Disposal Area, is part of a ring dike system; therefore, there are no natural abutments. Based on previous geotechnical work (TVA 1975/1978b, Stantec (2010a, 2010b, 2010c, and 2010d), and Geocomp as reported in Dewberry (2013)), the foundation of the levee generally consists of Quaternary-age alluvial deposits that vary in thickness from 45 to 90 feet. The alluvium is generally underlain by a thick layer of highly consolidated clays and dense sands.

3.1.2 Assessment

Annual site inspections for the East Ash Disposal Area, including the USACE Levee portion, were conducted and documented regularly from 1967 to 2013 (TVA, 1967 to 2009) and (Stantec, 2010, 2011, 2013). Other than some instances of erosion and sloughing of interior dike slopes and a seepage area along the toe of the levee near the Fuel Unloader Cell, no indications of foundation issues (i.e. cracking, settlement, depressions, and/or deformation) have been noted on historic inspection reports.

Recent inspections of the USACE Levee, performed by Stantec (2010, 2011, 2013) and Dewberry (2013) note no signs of tension cracking, settlement, deformations or similar instabilities.

Seepage analyses for the original USACE Levee (north dike) construction were not available for review. Recent seepage analyses conducted for the levee, however, were available for review. These analyses were performed by Stantec (2010c). A portion of the seepage analysis performed by Stantec evaluated factors of safety with respect to heave and exit gradient within the foundation materials. The analysis followed methodology presented in USACE (1986) and evaluated three different pool loading conditions (normal pool, maximum storage, and surcharge pool). Results from the analysis indicated that the levee had factors of safety for piping/heave greater than 5 (and up to 10.8).

3.1.3 Conclusion

Based on the assessment of the foundation and abutments for the USACE Levee, the CCR Rule-related criteria listed above have been met.

3.2 RAILROAD EMBANKMENT

3.2.1 Background

Since this embankment is part of the ring dike system for the East Ash Disposal Area, there are no natural abutments. Based on previous geotechnical work (TVA 1975/1978b, Stantec (2010a, 2010b, 2010c, and 2010d), and Geocomp as reported in Dewberry (2013)), the foundation of the embankment generally consists of Quaternary-age alluvial deposits that vary in thickness from 45 to 90 feet. The alluvium is generally underlain by a thick layer of highly consolidated clays and dense sands.

3.2.2 Assessment

Annual site inspections for the East Ash Disposal Area, including the Railroad Embankment, were conducted and documented regularly from 1967 to 2013 (TVA, 1967 to 2009) and (Stantec, 2010, 2011, and 2013). Documents indicate that wave erosion and sloughing of the interior Railroad Embankment slope was observed in 2000 and subsequently repaired by installing a rip rap blanket (Dewberry, 2013). Other than this issue, no indications of foundation issues (i.e. cracking, settlement, depressions, and/or deformation) have been noted on historic inspection reports. Recent inspections of the Railroad Embankment, performed by Stantec (2010, 2011, 2013) and Dewberry (2013) note no significant signs of tension cracking, settlement, deformations or similar instabilities.

Seepage analyses of the Railroad Embankment (south dike) were not available for review. However, previous slope stability and seepage analyses performed for the East Ash Disposal Area (Stantec (2010a, 2010b, 2010c, and 2010d) and Geocomp as reported in Dewberry (2013)) did not focus on the Railroad Embankment since the east and north dikes were considered to have the more critical embankments for analysis (Dewberry, 2013). The embankments are believed to consist of similar materials, and the geometry and configuration of the Railroad Embankment are wider and flatter than the other embankments. Dewberry's report concludes in part that the assessment of structural stability of the East Ash Disposal Area without specific analysis associated with the Railroad Embankment "….appears satisfactory...." due to the location of more critical stability and seepage cross sections that were analyzed in the north and east dikes, results of these analysis indicating satisfactory factors of safety against a piping failure, and no indications of "scarps, sloughs, major depressions…boils, sinks, or uncontrolled seepage…. significant vertical or horizontal alignment variations" in the dikes.

3.2.3 Conclusion

Based on the assessment of the foundation and abutments for the Railroad Embankment, the CCR Rule-related criteria listed above have been met.

3.3 DIVIDER DIKE

3.3.1 Background

Since the Divider Dike is part of a ring dike system around the East Ash Disposal Area, there are no natural abutments. Please refer to Sections 2.1 and 3.1.1 for a summary of historic foundation soils information associated with the embankment.

3.3.2 Assessment

Annual site inspections for the East Ash Disposal Area, including the Divider Dike, were conducted and documented regularly from 1976 to 2013. Other than some instances of erosion and sloughing of interior dike slopes, no indications of foundation issues (i.e. cracking, settlement, depressions, and/or deformation) have been noted on historic inspection reports. Recent inspections of the Divider Dike performed by Stantec (2013) and Dewberry (2013) note no significant signs of tension cracking, settlement, deformations or similar instabilities.

Seepage analyses of the Divider Dike were not available for review, and do not appear to be warranted based on the reported pervious condition of the dike (i.e. ash fill), and its operational condition (i.e. equal water levels on both sides since it was put into service).

3.3.3 Conclusion

Based on the assessment of the foundation and abutments for the Divider Dike, the CCR Rule-related criteria listed above have been met.

3.4 EAST PERIMETER DIKE

3.4.1 Background

Since the East Perimeter Dike is part of a ring dike system around the East Ash Disposal Area, there are no natural abutments. Please refer to Sections 2.1 and 3.1.1 for a summary of historic foundation soils information associated with the embankment.

3.4.2 Assessment

Annual site inspections for the East Ash Disposal Area, including the East Perimeter Dike, were conducted and documented regularly from 1976 to 2013. Other than some instances of erosion and sloughing of interior dike slopes, and installation of rip- rap blanket to address slope stability concerns (see paragraphs below), no indications of foundation issues (i.e. cracking, settlement, depressions, and/or deformation) have been noted on historic inspection reports. Recent inspections of the East Perimeter Dike performed by Stantec (2013) and Dewberry (2013) note no significant signs of tension cracking, settlement, deformations or similar instabilities.

Seepage analyses of the original East Perimeter Dike are not available for review. However, seepage analyses reflecting recent improvements of the dike were available for review. These analyses were performed by Stantec (2010a). A portion of the seepage analysis performed by Stantec evaluated factors of safety with respect to heave and exit gradient within the foundation materials. The analysis followed methodology presented in USACE (1986) and evaluated two different pool loading conditions (normal pool and maximum storage). Results from the analysis indicated that the east dike had factors of safety for piping/heave as low as 1.3, which did not meet the recommended factor of safety of 3.0. Stantec recommended remedial measures to include construction of an earth or rock buttress, flattening dike slopes and/or lowering water level in the East Stilling Pond.

TVA initiated remedial measures for the dike that included placement of rip rap blankets on the dike slopes and lowering the water level by approximately 4 feet. Construction of the remedial measures were substantially completed in September 2011, and documented in Stantec (2012a). A subsequent stability and seepage analysis of the remedial improvements to the East Perimeter Dike was performed by Stantec (2011). Results of the seepage analyses indicated that with the remedial measures in place, the east dike has satisfactory factors of safety for piping/heave from 4.7 up to 6.0.

3.4.3 Conclusion

Based on the assessment of the foundation and abutments for the East Perimeter Dike, the CCR Rule-related criteria listed above have been met.

4.0 SLOPE PROTECTION

Per §257.73(d)(1)(ii), the initial structural stability assessment must document whether the unit has been designed, constructed, operated and maintained with stable adequate slope protection to protect against surface erosion, wave action, and adverse effects of sudden drawdown. The East Ash Disposal Area has the following features that fall within this requirement:

- USACE Levee
- Railroad Embankment
- Divider Dike
- East Perimeter Dike

Assessment of the slope protection associated with these features was completed considering the following criteria related to the CCR rule:

1. *Regular (weekly) inspections for erosion. Inspections should show there are no significant signs of deterioration in the slope protection configuration of the Item.*

2. *Appropriate slope protection shall be provided based on anticipated flow velocities. [Hydrologic / hydraulic calculations of flow velocities on the slope of the Item for the appropriate erosive forces. Some common slope protection include: Rip rap, Gabions, Paving (concrete or asphalt), or appropriate vegetative cover.]*

3. *If slope protection is Rip rap, filter layer(s) under the rip rap shall be designed according to established filter criteria. However, existing Rip rap cover may be evaluated based on performance and observations during inspections.*

4.1 USACE LEVEE

4.1.1 Background

Slope protection for the USACE Levee generally consists of a combination of grass vegetation cover, rip rap, or concrete aprons. Rip rap or concrete aprons are used at spillway outlets and other structures.

4.1.2 Assessment

Annual inspection reports from 1967 to 2013 (TVA, 1967 to 2009) and (Stantec, 2010, 2011, 2013) generally indicate appropriate maintenance of slope protection features of the dike, in accordance with the procedures outlined in Allen Fossil Plant Procedures (TVA, 2011). As observed in the November 2015 site visit by Stantec personnel, the slope protection for the USACE Levee is a vegetated slope. See Section 6.0 for additional details about vegetated slopes.

The use of rip rap in certain areas of the dike appears appropriate to address anticipated flows. Rip rap is specified on TVA drawings to consist of at least 50% by weight stones weighing a minimum of 200 pounds each. TVA drawing 10N226 (1978b), which appears to be part of the design drawing set for the 1976 Divider Dike construction, indicates that a 4-inch layer of crushed stone or gravel be applied to disturbed sections of the levee. It is unclear from the drawing if any layers of crushed stone or gravel that were used remained after construction.

4.1.3 Conclusion

Based on the assessment of the slope protection for the USACE Levee, the CCR Rule-related criteria listed above have been met.

4.2 RAILROAD EMBANKMENT

4.2.1 Background

Slope protection for the Railroad Embankment consists of a combination of grass vegetation cover or rip rap.

4.2.2 Assessment

Annual inspection reports from 1967 to 2013 (TVA, 1967 to 2009) and (Stantec, 2010, 2011, 2013) generally indicate that vegetation maintenance for the embankments has existed over the past 40 years and that regular mowing and removal of woody brush is occurring in accordance with the procedures outlined in Allen Fossil Plant Procedures (TVA, 2011). See Section 6.0 for additional details about vegetated slopes.

As observed in the November 2015 site visit by Stantec personnel, the interior slope of the Railroad Embankment along the East Ash Pond is armored with rip rap and the Stilling Pond is a vegetated cover. The exterior slope is vegetated. The rip rap was in good condition with no observed deficiencies. See Section 6.0 for additional details about vegetated slopes.

4.2.3 Conclusion

Based on the assessment of the foundation and abutments for the Railroad Embankment, the CCR Rule-related criteria listed above have been met.

4.3 DIVIDER DIKE

4.3.1 Background

Slope protection for the Divider Dike consists of rip rap.

4.3.2 Assessment

Annual inspection reports from 1967 to 2013 (TVA, 1967 to 2009) and (Stantec, 2010, 2011, 2013) generally indicate that vegetation maintenance for the embankments has existed over the past 40 years and that regular mowing and removal of woody brush is occurring in accordance with the procedures outlined in *Allen Fossil Plant Procedures* (TVA, 2011).

As observed in the November 2015 site visit by Stantec personnel, the slopes of the Divider Dike are armored with rip rap. The rip rap was in good condition with no deficiencies identified.

4.3.3 Conclusion

Based on the assessment of the slope protection for the Divider Dike, the CCR Rule-related criteria listed above have been met.

4.4 EAST PERIMETER DIKE

4.4.1 Background

Slope protection for the East Perimeter Dike consists of a combination of grass vegetation cover or rip rap.

4.4.2 Assessment

Annual inspection reports from 1967 to 2013 (TVA, 1967 to 2009) and (Stantec, 2010, 2011, 2013) generally indicate that vegetation maintenance for the embankments has existed over the past 40 years and that regular mowing and removal of woody brush is occurring in accordance with the procedures outlined in *Allen Fossil Plant Procedures* (TVA, 2011). See Section 6.0 for additional details about vegetated slopes.

As observed in the November 2015 site visit by Stantec personnel, the interior slope of the East Perimeter Dike is armored with rip rap along the lower portion of the slope and vegetated up to the crest of the embankment. The exterior slope protection is a vegetative cover. The rip rap was in good condition with no observed deficiencies. See Section 6.0 for additional details about vegetated slopes.

4.4.3 Conclusion

Based on the assessment of the foundation and abutments for the East Perimeter Dike, the CCR Rule-related criteria listed above have been met.

5.0 EMBANKMENT DIKE COMPACTION

Per §257.73(d)(1)(iii), the initial structural stability assessment must document whether the unit has been designed, constructed, operated and maintained with dikes mechanically compacted to a density sufficient to withstand the range of loading conditions in the CCR unit. The East Ash Disposal Area has the following features that fall within this requirement:

- USACE Levee
- Railroad Embankment
- Divider Dike
- East Perimeter Dike

Assessment of the dike compaction associated with these features was completed considering the following criteria related to the CCR rule:

1. Documentation showing the dike was mechanically compacted. Acceptable documentation may include construction drawings, field notes, construction photographs, correspondences, or any evidence showing the dike was mechanically compacted during construction.
2. If no construction documentation is available specific data from geotechnical explorations of dike may be used. Geotechnical borings with continuous SPT may be used to assess compaction of the dike. Appropriate methodology correlating blow counts and compaction (Density) should be used.

5.1 USACE LEVEE

5.1.1 Background

One original construction drawing for the USACE Levee was available for review (USACE, 1960); however, this drawing did not provide information regarding compaction of the USACE Levee. Other documentation showing the dike was mechanically compacted during its original construction was not identified during historical document review. TVA design drawings, developed in the 1970s for construction of an impermeable layer on the interior slope of the USACE Levee to facilitate the impoundment of the East Ash Disposal Area, provide proposed dike construction and compaction methods and were referenced in the assessment discussed below. Results from a subsurface exploration of the dike were also available that provided SPT data used in the assessment.

5.1.2 Assessment

TVA Drawings 10N224 and 10N226 provide documentation of compaction requirements related to the construction of the improvements made to the dike in the 1970s. Construction criteria related to dike embankment materials and compaction as noted on these drawings include:

- The impervious blanket was to be constructed from "core type material" and references Note 1 on TVA drawing 10N224 (TVA, 1978) for placement and compaction specifications.
- Note 1 on drawing 10N224 states "All dike construction shall be in accordance with applicable portions of general construction specification no. G-9 for rolled earth fill for dams and power plants. For additional information and fill compaction requirements see TVA (1975). This memorandum listed the following specifications for the core material/impermeable blanket:
    - All core material is to have at least 35 percent fines (minus 200 sieve), and be compacted to at least 95 percent of the maximum standard dry density determined by the TVA central laboratory.
    - Core material moisture content is to be maintained within ±3 percent of the optimum moisture content determined by the central laboratory.

Stantec completed a subsurface exploration of the USACE Levee to evaluate the condition of the impermeable layer constructed on the interior slope of the USACE Levee (Stantec, 2010b). The subsurface exploration program included drilling and sampling eight (8) soil test borings on the USACE Levee extending about 40 to 60 feet below existing ground surface. An automatic standard penetration test (SPT) hammer was used with a track-mounted drill rig to advance a split-barrel sampler in the borings. The SPT data was used to estimate relative density of dike embankment materials, referencing NAVFAC DM-7.1 (1986) and USACE (2001).

The SPT data reviewed shows average N-values ranging from 6 to 29 blows per foot (bpf) for the dike embankment materials. Correlating these results using NAVFAC DM-7.1 and USACE indicate appropriate compaction exists within the embankment of the USACE Levee.

5.1.3 Conclusion

Based on the assessment of the embankment dike compaction for the USACE Levee, the CCR Rule-related criteria listed above have been met.

5.2 RAILROAD EMBANKMENT

5.2.1 Background

No design or construction documentation showing the Railroad Embankment was mechanically compacted during construction was identified during document review.

5.2.2 Assessment

Stantec performed a geotechnical exploration of the East Perimeter Dike and Divider Dike in 2010. The boring layout in Stantec (2010a) shows one boring (STN-14) was located on the crest of the East Perimeter Dike where it intersects the Railroad Embankment. The boring log for STN-14 indicates the fill material transitions from sandy silt to lean clay at a depth of 14 feet. Based on the boring log, the sandy silt material at STN-14 corresponds to the East Perimeter Dike and the lean clay fill material is the Railroad Embankment. The boring log for STN-14 indicates the average blow count for the lean clay fill material within the Railroad Embankment is 15. Correlating this result using NAVFAC DM-7.1 and USACE (2001) indicate appropriate compaction exists within the Railroad Embankment.

5.2.3 Conclusion

Based on the assessment of the embankment dike compaction for the Railroad Embankment, the CCR Rule-related criteria listed above have been met.

5.3 DIVIDER DIKE

5.3.1 Background

Construction records related to the dike material placement and compaction for the Divider Dike were not available during this review. Certain TVA design drawings provide proposed dike construction and compaction methods and were referenced in the assessment discussed below. Results from a subsurface exploration of the dike were also available that provided SPT data used in the assessment.

5.3.2 Assessment

TVA Drawing 10N226 (TVA, 1978) indicates the Divider Dike was constructed of bottom ash placed in not more than 9-inch thick layers and compacted with rubber-tired hauling equipment.

Stantec completed a geotechnical exploration of the Divider Dike for TVA in 2011. The subsurface exploration program completed consisted of drilling and sampling three soil test borings along the crest of the Divider Dike. Continuous SPT data was collected at each boring location.

The SP test data from this study shows average N-values ranging from 12 to 52 for the dike embankment materials. Correlating these results using NAVFAC DM-7.1 indicate that appropriate compaction exists within the embankment of the Perimeter Dike.

5.3.3 Conclusion

Based on the assessment of the embankment dike compaction for the Divider Dike, the CCR Rule-related criteria listed above have been met.

5.4 EAST PERIMETER DIKE

5.4.1 Background

Construction records related to the dike material placement and compaction for the initial perimeter dike were not available during this review. Certain TVA design drawings provide proposed dike construction and compaction methods and were referenced in the assessment discussed below. Results from a subsurface exploration of the dike were also available that provided SPT data used in the assessment.

5.4.2 Assessment

TVA drawings 10N224 and 10N226 (TVA, 1978) provide documentation of compaction requirements related to the construction of the East Perimeter Dike. Construction criteria related to dike embankment materials and dike compaction as noted on these drawings include:

- *All dike core material was specified to have at least 35 percent fines (minus 200 sieve).*
- *Core material is to be compacted to at least 95 percent of the maximum standard dry density determined by the TVA central laboratory.*
- *Core material moisture content is to be maintained within ±3 percent of the optimum moisture content determined by the TVA central laboratory.*

Stantec completed a geotechnical exploration of the East Perimeter Dike for TVA in 2010. The subsurface exploration program consisted of drilling and sampling five soil test borings along the crest and near the toe of the East Perimeter Dike (Stantec, 2010a). One additional boring was located on the crest of the East Perimeter Dike where it intersects the Railroad Embankment. Standard Penetration (SP) Testing was performed at each boring location. The SP data from this study was used to estimate relative density of dike embankment materials, referencing NAVFAC DM-7.1 (1986) and USACE (2001).

The SP test data reviewed shows average N-values ranging from 13 to 64 blows per foot (bpf) for the dike embankment materials. Correlating these results using NAVFAC DM-7.1 and USACE indicate appropriate compaction exists within the embankment of the East Perimeter Dike.

5.4.3 Conclusion

Based on the assessment of the embankment dike compaction for the East Perimeter Dike, the CCR Rule-related criteria listed above have been met.

6.0 VEGETATED SLOPES

Per §257.73(d)(1)(iv), the initial structural stability assessment must document whether the unit has been designed, constructed, operated and maintained with vegetated slopes of dikes and surrounding areas not to exceed a height of six inches above the slope of the dike, except for slopes which have an alternate form or forms of slope protection. The East Ash Disposal Area has the following features that fall within this requirement:

- USACE Levee
- Railroad Embankment
- East Perimeter Dike

Assessment of the vegetated slopes associated with these features was completed considering the following criteria related to the CCR rule:

1. Regular inspection records showing vegetative cover not exceeding 6 inches above slope of dike.

6.1 BACKGROUND

The grass on the embankments of the East Ash Disposal Area is maintained by mowing at least 3 times per growing season in accordance with the Allen Fossil Plant Procedures (TVA, 2011).

The primary slope protection specified for the East Perimeter Dike, USACE Levee, and Railroad Embankment is grass.

The three embankments were observed during a site visit by Stantec in November 2015.

6.2 ASSESSMENT

Annual site inspections were conducted and documented regularly from 1967 to 2013 (TVA, 1967 to 2009) and (Stantec, 2010, 2011, 2013). Annual inspection reports for over 40 years are available, and document the vegetative cover over the dike structures. These reports indicate that maintenance has been routinely performed.

In November 2015, Stantec personnel visited the site to observe existing conditions. The vegetation along the slopes of the various embankments was 6 inches or less in height, and there was good coverage.

6.3 CONCLUSION

Based on the assessment of the vegetated slopes for the USACE Levee, Railroad Embankment, and East Perimeter Dike, the CCR Rule-related criteria listed above have been met.

7.0 SPILLWAY CONDITION AND CAPACITY

Per §257.73(d)(1)(v), the initial structural stability assessment must document whether the unit has been designed, constructed, operated and maintained with a single spillway or combination of spillways that meet the condition and capacity requirements as outlined in this section of the CCR Rule. The combined capacity of all spillways are to be designed, constructed, operated, and maintained to adequately manage flow during and following the peak discharge from the event specified in this section. The East Ash Disposal Area has the following features that fall within this requirement:

- ALF East and ALF West
- ALF Overflow 1 and ALF Overflow 2
- Overflow Weir

Assessment of the spillway condition and capacity associated with these features was completed considering the following criteria related to the CCR Rule:

1. Outlet channel must be of non-erodible material designed to carry sustained flow velocities based on the required flood events. [Estimate flow velocities and select appropriate material using hydraulic analysis for the following flood events: PMF (high hazard potential unit), 1000-year flood (Significant hazard unit), 100-year flood (low hazard potential unit).]

2. Must adequately manage flow during and following the peak discharge. [Estimate size of outlet structure based of hydraulic analysis for the following flood events: PMF (High hazard potential unit), 1000-year flood (Significant hazard potential unit), and 100-year flood (low hazard potential unit).]

3. Must be structurally stable. [Assess stability of structure using stability and stress analyses according to an appropriate methodology. Some acceptable methodologies may include: EM 1110-2-2400, EM 1110-2-2100, ACI 350, etc.]

4. Must maintain structural integrity. [Structural integrity may be warranted by periodic inspections of existing conduits. Inspections must show no significant presence of deformation, distortions, cracks, joint separation, etc.]

5. Must be free from significant amounts of obstruction and anomaly which may affect the operation of the hydraulic structure [Perform periodic pipe inspections to detect deterioration, deformation, distortion, bedding deficiencies, and sediment, and debris accumulations.]

7.1 ALF EAST, ALF WEST, ALF OVERFLOW 1, ALF OVERFLOW 2

7.1.1 Background

The East Ash Disposal Area is classified as a significant hazard structure requiring the combined capacity of all spillways be adequate to manage the flow during and following the peak discharge from a 1000-year flood.

The ALF West spillway, ALF East spillway, ALF Overflow 1, and ALF Overflow 2 structures consist of 48-inch diameter concrete risers with 36-inch diameter Class III reinforced concrete outflow pipes. All four spillway structures have a cured in place pipe liner. Refer to section 2.2 for a detailed description of the ALF West and East spillways and ALF Overflow 1 and 2.

7.1.2 Assessment

7.1.2.1 Spillway Capacity

The Inflow Design Flood Control System Plan for the Stilling Pond demonstrates the Stilling Pond meets the capacity requirements outlined in §257.73(d)(1)(v) of the CCR Rule. During the November 2015 site visit by Stantec personnel, the barrels were freely discharging with no observed deficiencies or blockages.

7.1.2.2 Structural Stability

7.1.2.2.1 Material Properties

The following properties were used in this analysis for determining weight, strength, and other required parameters:

Table 1 Material Properties Used in Analysis

Material	Property	Reference
Concrete	f'_c = 4,000 psi, γ_c = 150 pcf, ε_c = 0.003	TVA 10N229-2 R0
Reinforcing Steel	f_y = 60 ksi, E_s (Elastic Modulus) = 29,000 ksi	Assumed values based on available reinforcing steel during era of construction
Soil	γ_{sat} = 105 pcf, Friction Angle = 25°, Bearing Pressure = 3,855 psf	Stantec (2015a) Appendix B

7.1.2.2.2 Geometry

In order to determine the geometry of the riser structures for analysis, the original as-built plans were used in conjunction with revised plans provided in Stantec (2012a) which document the change in riser heights.

The four evaluated risers have identical geometry with the exception of the top of spillway crest elevations. ALF-West and ALF-East Risers have top of structure elevations of 225.39 and 225.40 feet, respectively. ALF Overflow 1 and ALF Overflow 2 have elevations of 226.47 and 226.30 feet, respectively.

7.1.2.2.3 Analysis Methodology

The four risers are identical in configuration except the variation in total structural height. For this reason, Stantec performed structural stability analysis for the tallest riser of ALF West/East and ALF Overflow 1/2. The calculations and results for the riser is included in Appendix B.

To determine the structural stability of the riser, the following "worst case" loading conditions were assumed for each failure limit state.

The failure limit states explored for the riser were as follows:

Table 2 Loading Conditions for Different Failure Limit States

Failure Limit State	Load Condition
Floatation	Water level is at the crest of the riser. Structure is empty. Weight of the trash filter is excluded in the event that it is removed.
Foundation Bearing	Case 1: Water level is at the crest of the riser and the structure is filled. Produces maximum weight, but considerable uplift. Includes weight of the trash filter. Case 2: Water elevation in pond is below riser slab. Riser structure is empty. Produces minimum weight, but no uplift
Strength of Riser Walls	Structure is fully submerged so as to produce maximum lateral pressure on the walls.

When determining the force effects in the walls of the riser due to the maximum soil and hydrostatic pressures, the walls were assumed to behave as plates fixed on three sides. Engineering Monograph 27 was referenced in the determination of the maximum force effects in the walls. For the capacity of the concrete walls, all tension reinforcing steel was assumed to yield. A stress block approach was used to determine the flexural capacity. Shear and flexure capacities for both directions were evaluated in accordance with ACI 350-06.

7.1.2.2.4 Acceptance Criteria

To comply with the CCR Rule, the following factors of safety were obtained from the indicated references:

Table 3 Minimum Recommended Factors of Safety for Stability

Limit State	Minimum Recommended Factor of Safety	Reference
Floatation	FS = 1.3	USACE (2005)
Foundation Bearing	FS = 3.0	USACE (1992)

For the reinforced concrete strength limit states, the analysis was performed in accordance with ACI 350-06. Appendix C in ACI 350-06 was not used to modify the load factors and resistance reductions. A performance ratio greater than 1 indicates the limit state is satisfied.

The following results were obtained by the calculation in Appendix B of this report:

Table 4 Results for Different Limit States

Limit State	Recommended	Results
Floatation	FS = 1.3	FS = 1.4
Foundation Bearing	FS = 3.0	FS = 10.3 for riser fully submerged FS = 5.9 for riser in drained pond
Strength of Riser Walls	Performance Ratio $\geq$ 1.0	P.R. = 3.0 for flexure at corner P.R. = 4.4 for shear at corner P.R.= 3.6 for flexure at footing P.R. = 6.9 for shear at footing

The structure complies with the stability and strength limits listed above.

7.1.3 Conclusion

Based on the assessment of the spillway condition and capacity for the ALF East, ALF West, ALF Overflow 1, and ALF Overflow 2 Risers, the CCR Rule-related criteria listed above have been met.

7.2 OVERFLOW WEIR

7.2.1 Background

The Overflow Weir is a concrete drop structure, located at the south end of the Divider Dike, which controls the elevation and discharge from the East Ash Pond to the East Stilling Pond. Refer to Section 2.2 for further details.

7.2.2 Assessment

7.2.2.1 Spillway Capacity

The Inflow Design Flood Control System Plan for the Stilling Pond demonstrates the Stilling Pond meets the capacity requirements outlined in §257.73(d)(1)(v) of the CCR Rule. During the November 2015 site visit by Stantec personnel, the spillway was freely discharging with no observed deficiencies or blockages.

7.2.2.2 Structural Stability

7.2.2.2.1 Material Properties

The following properties were used in this analysis for determining, weight, strength, and other important parameters:

Table 5 Material Properties Used in Analysis

Material	Property	References
Concrete	f'_c = 4,000 psi, γ_c = 150 pcf, ε_c = 0.003	10W258-1
Reinforcing Steel	f_y = 60 ksi, E_s (Elastic Modulus) = 29,000 ksi	10W258-1
Soil	γ_{sat} = 124 pcf, Friction Angle = 31°, Bearing Pressure = 15,900 psf	Stantec (2015a) Appendix B

7.2.2.2.2 Geometry

To determine the geometry of the Overflow Weir for analysis, drawings 10W258-1 and 10W258-2 from the original as-built plans were referenced.

7.2.2.2.3 Analysis Methodology

Stantec performed a stability analysis of the structure, considering the most severe loading conditions. The controlling load condition for this structure is the reservoir at maximum normal pool (EL 232.0) with all stoplogs installed and the downstream pool at its minimum operating pool (EL 222.0). This load condition is the most severe for overturning, sliding, and bearing pressure, and strength limit states. The flood condition governs for floatation which was not assessed due to the limited pond differential.

Using this loading condition, equilibrium of forces was used to check global stability limit states as follows: the summation of lateral forces (namely, the upstream hydrostatic force against the base friction) to check sliding; the summation of vertical forces to check max soil bearing; and the summation of moments to check for sufficient compression of the base against the foundation.

The weir, wing walls, and side walls were assessed for strength limit states assuming the load condition described above. For the wing wall and side wall, the controlling location for strength evaluation was just downstream of the weir with the largest head differential.

The walls were assumed to behave as a cantilever beam bending about the base under maximum moment and shear due to hydrostatic and lateral earth pressures.

7.2.2.2.4 Acceptance Criteria

To comply with the CCR Rule, the following factors of safety were obtained from the indicated references:

Table 6 Minimum Recommended Factors of Safety for Stability

Limit State	Minimum Recommended Factor of Safety	References
Sliding	FS = 2.0, including passive lateral earth pressure	USACE (2005)
Moment Equilibrium	100% of Base in Compression	USACE (2005)
Foundation Bearing	FS = 3.0	USACE (1992)

For the reinforced concrete strength limit states, the analysis was performed in accordance with ACI 350-06. Appendix C in ACI 350-06 was not used to modify the load factors and resistance reductions. A performance ratio greater than 1 indicates the limit state is satisfied.

7.2.2.2.5 Results

The following results were obtained by the calculation in Appendix B of this report:

Table 7 Results for Different Limit States

Limit State	Recommended	Results
Sliding	FS = 2.0, including passive lateral earth pressure	FS = 3.1
Moment Equilibrium	100% of Base in Compression	e_0 = 3.801ft < B/6 = 3.875ft
Foundation Bearing	FS = 3.0	FS = 12.5
Strength of Weir Stem	Performance Ratio > 1.0	P.R.= 1.2 for flexure at base P.R. = 1.8 for shear at base
Strength of Wall Stem	Performance Ratio >1.0	P.R. = 1.6 for flexure at base P.R. = 1.4 for shear at base

7.2.3 Conclusions

Based on the assessment of the condition and capacity for the Overflow Weir, the CCR Rule-related criteria listed above have been met.

8.0 HYDRAULIC STRUCTURES CONDITIONS

Per §257.73(d)(1)(vi), the initial structural stability assessment must document whether the unit has been designed, constructed, operated and maintained with hydraulic structures underlying the base of the CCR unit or passing through the dike of the CCR unit that maintain structural integrity and are free of significant deterioration, deformation, distortion, bedding deficiencies, sedimentation, and debris which may negatively affect the operation of the hydraulic structure. The East Ash Disposal Area has the following features that fall within this requirement:

- 60-inch Diameter Sanitary Sewer Pipeline
- 30/42-inch Diameter Sanitary Sewer Pipeline

Assessment of the hydraulic structures condition associated with these features was completed considering the following criteria related to the CCR rule:

1. Must be able to manage the required flows. [Estimate size of pipes based on hydraulic analysis for the following flood events: PMF (High hazard potential unit), 1000-year flood (Significant hazard potential unit), and 100-year flood (low hazard potential unit).]

2. Must maintain structural integrity. [Structural integrity may be warranted by periodic inspections of existing conduits. Inspections must show no significant presence of deformation, distortions, cracks, joint separation, etc.]

3. Must be free from significant amounts of obstruction and anomaly which may affect the operation of the hydraulic structure. [Perform periodic pipe inspections to detect deterioration, deformation, distortion, bedding deficiencies, and sediment, and debris accumulations.]

8.1 60-INCH DIAMETER SANITARY SEWER PIPELINE

8.1.1 Background

The 60-inch Diameter Sanitary Sewer Pipeline has an east-west alignment and approximately 3000 feet is located beneath the southern portion of the East Ash Disposal Area. The concrete pipeline, also referred to as the Nonconnah Interceptor, was a primary sewer line to the City of Memphis T.E. Maxson Treatment Plant until 1976. At that time a new 96-inch sewer was installed and the 60-inch concrete line was closed. It currently does not convey sewage.

8.1.2 Assessment

During the November 2015 site visit by Stantec personnel, no signs of embankment settlement, slope movement or seepage were observed in the area where the 60-inch pipe is buried below the East Perimeter Dike. The ground surface along the pipe alignment and visible structures (manholes) were normal in appearance.

In January 2016, an inspection of the pipe was completed as described in the Pipe Inspection Report (Stantec, 2016a). The pipe was accessed via a manhole approximately 100 feet west from the East Perimeter Dike. The pipe was found to be water-filled; therefore, the inspection utilized camera/sonar-equipped underwater Remote Operated Vehicle (ROV). The inspection was able to cover about 1700 feet (all of the East Stilling Pond and most of the East Ash Pond) before encountering sediments filling the pipe.

The inspection revealed water is ponded in the pipe (there was no discernable flow), which indicated the pipe to be blocked downstream. A pressure transducer on the ROV provided data that measured the water depth, and it showed the water filling the pipe is near the McKellar Lake level (not the water elevation of the East Stilling or Ash Ponds). Further west, the pipe became completely sediment-filled, and could not be inspected.

Exposed aggregate in the pipe was observed, and this was attributed to previous hydrogen sulfide damage. The inspection did not reveal exposed steel reinforcement, structural cracks, internal circumference irregularities, differential settlements, or other signs of impending or potential structural failure.

An attempt was also made to inspect the pipe from a manhole located downstream from the East Ash Disposal Area. The pipe at this location was found to be completely filled with material that was judged to be resistant to physical removal. A small section of the pipe crown was visible at this location and it appeared free from structural defects.

8.1.3 Conclusion

Based on the assessment of the hydraulic structure condition for the 60-Inch Diameter Sanitary Sewer Pipeline and in accordance with §257.73(a)(4), the CCR Rule-related criteria have been partially met. The pipe is closed and does not carry required flows. The structural integrity appears adequate where it was accessed; however, pipe sections beneath the East Ash Disposal Area were not accessible for inspection. For that reason, plans to permanently close the portion of the 60-Inch Diameter Sanitary Sewer Pipeline located under the East Ash Disposal Area are being prepared as part of the Written Closure Plan (Stantec 2016b).

8.1.4 Recommendation

Per §257.73(a)(4), the recommendations noted in the Written Closure Plan (Stantec 2016b) should be followed.

8.2 30/42-INCH DIAMETER SANITARY SEWER PIPELINE

8.2.1 Background

The 30/42-inch Diameter Sanitary Sewer pipeline runs through the west portion of the East Ash Disposal Area in a north-south direction. It enters the East Ash Disposal Area across the USACE Levee crest, and it leaves the Area by penetrating through the Railroad Embankment. It was constructed in two segments – a 30-inch diameter high-density polyethylene (HDPE) force main, and a 42-inch concrete gravity sewer. The original force main segment was by-passed when a new force main segment was constructed within the Dredge Cell in 2011 (Stantec, 2012b).

8.2.2 Assessment

During the November 2015 site visit by Stantec personnel, no signs of embankment settlement, slope movement, or seepage were observed in the areas where the pipes are buried below the USCAE Levee and the Railroad Embankment. The ground surface along the pipe alignment and visible structures (manholes) were normal in appearance.

The original 30-inch HDPE force main segment was bypassed during 2011. The new force main was designed for hydraulic and structural integrity. Construction was inspected and reports were developed that documents compliance with its design. During construction, a structural bulkhead was placed to isolate the original force main from the pipeline. Therefore, its structural integrity is not a factor necessary for the integrity of the East Ash Disposal Area.

Access into the 42-inch concrete pipe was attempted by opening a manhole on the south side of the East Ash Disposal Area. Sewage through the pipe appeared to flowing normally; however, due to the fact that the City of Memphis was unable to temporarily shut down flows from the Presidents Island Pump Station, a complete assessment of the 42-inch concrete pipe could not be conducted. These conditions are documented in the Pipe Inspection Report (Stantec, 2016a).

8.2.3 Conclusion

While the 30/42-Inch Diameter Sanitary Sewer pipeline appears to be functioning correctly, the 42-inch concrete pipe beneath the East Ash Disposal Area was not accessible for inspection. For that reason, plans to relocate the 30/42-Inch Diameter Sanitary Sewer Pipeline outside the East Ash Disposal Area are being prepared as part of the Written Closure Plan (Stantec 2016b).

8.2.4 Recommendation

Per §257.73(a)(4), the recommendations noted in the Written Closure Plan (Stantec 2016b) should be followed.

9.0 SUDDEN DRAWDOWN ASSESSMENT

Per §257.73(d)(1)(vii), the initial structural stability assessment must document whether the unit has been designed, constructed, operated, and maintained with downstream slopes that can be inundated by an adjacent water body (such as a river, stream, or lake) to determine if structural stability is maintained during low pool or sudden drawdown of the adjacent water body. The East Ash Disposal Area has the following feature that falls within this requirement:

- USACE Levee

Assessment of the sudden drawdown associated with these features was completed considering the following criteria related to the CCR rule:

1. Maintain slope stability during sudden drawdown of adjacent water body.

Guidance provided by USEPA (2015) described the basis of the CCR Rule's factor of safety criteria and methodology as EM 1110-2-1902 (USACE, 2003) or other appropriate methodologies. Table 3-1 of USACE (2003) recommends a required minimum factor of safety of 1.1 for maximum surcharge pool under rapid drawdown conditions.

9.1 BACKGROUND

The East Ash Disposal Area has a potential sudden drawdown loading from McKellar Lake along its northern perimeter dike (USACE Levee). The lake is connected to the Mississippi River and lies at the toe of the USACE Levee. A sudden drawdown slope stability analysis of the downstream USACE Levee is required under the CCR Rule §257.73(d)(1)(vii).

Though the eastern embankment abuts the Horn Lake Cutoff, the cutoff is a wetland area without a normal pool and is not considered subject to sudden drawdown conditions. The southern embankment is the Railroad Embankment and does not have an adjacent water body.

The sudden drawdown slope stability analysis was performed in conjunction with the static safety factor assessment discussed in Stantec (2015d).

9.2 ASSESSMENT

9.2.1 Detailed Task Analysis Criteria

9.2.1.1 Material Properties

An overview of the subsurface conditions at the East Ash Disposal Area's northern perimeter dike (USACE Levee) is summarized in Table 8. A more in-depth review is found in Stantec Consulting Services Inc. (2010c).

Table 8 Generalized Subsurface Conditions – Northern Perimeter Dike (USACE Levee)

Approximate Elevation	Materials	Consistency/Density
El. 237 to El. 215	Dike fill – consists of sandy silt, silty sand, silty clay, sandy clay, and lean clay	Stiff to very stiff/medium dense
El. 215 to El. 175 (termination depth)	Alluvium – Irregularly bedded sandy silt, silty sand, silt, lean clay, sand, and fat clay	Very soft to stiff/very loose to medium dense

During the 2009 geotechnical explorations, Stantec performed a laboratory testing program consisting of natural moisture content determinations, sieve and hydrometer analyses, Atterberg limits, specific gravity determinations, consolidated-undrained triaxial compression tests, and permeability tests. The strength parameters derived using the laboratory data and used in this sudden drawdown slope stability evaluation are presented in Table 9. The results of the laboratory testing and derivation of the strength parameters can be found in Stantec (2010a, 2010c, and 2011).

Table 9 Strength Parameters for Stability Analysis for the Northern Perimeter Dike (USACE Levee)

Soil Horizon	Saturated Unit Weight (pcf)	Wet Unit Weight (pcf)	Effective Stress Strength Parameters		Total Stress Strength Parameters	
			c' (psf)	ϕ' (degrees)	c' (psf)	ϕ' (degrees)
Dike Fill – Sandy Silt, Silty Sand	125	125	0	31	200	22
Hydraulically Placed Ash	105	95	0	25	0	10
Rip rap	140	140	0	38	0	38
Foundation – Silt and Sandy Silt	115	110	0	28	200	12
Foundation - Lean and Fat Clay	115	110	0	26	400	12

9.2.1.2 Critical Cross Section Selection

Slope stability analyses were available from Stantec Consulting Services Inc. (2010a and 2010c) and Stantec Consulting Services Inc. and URS Corporation (2014). Five primary cross sections were previously analyzed, labeled A-A' through E-E' shown below in Figure 2.

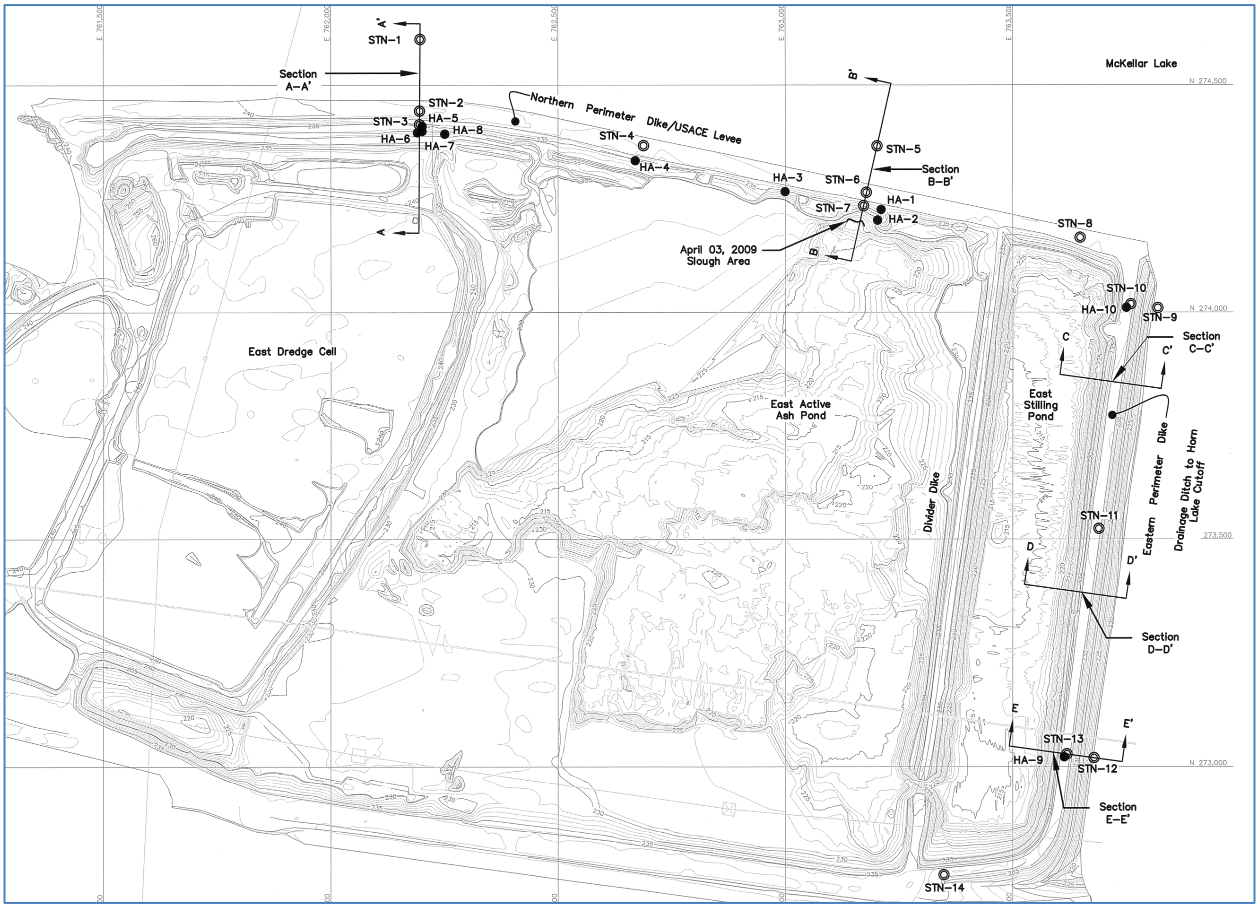

Figure 2 ALF East Ash Disposal Area – Plan View of Cross Sections

(Stantec Consulting Services Inc., 2010c)

Sections A-A' and B-B' reflect the geometry along the northern perimeter dike (USACE Levee). Sections C-C', D-D', and E-E' reflect the geometry along the eastern perimeter dike, separating the East Ash Stilling Pond and the drainage channel to the Horn Lake Cutoff. To determine if the two previous cross sections along the USACE Levee were still representative of field conditions, a review of recent construction activities, topographic, and bathymetric information was performed. The follow modifications were made to the East Ash Disposal Area since 2009:

- The water level was lowered in the East Stilling Pond.

- A rip rap buttress was constructed on the east slope of the Divider Dike between the East Ash Pond and the East Stilling Pond.
- A sanitary sewer bypass line was constructed through the Dredge Cell, and the sluice channel has been rerouted.

These structural modifications have not changed the exterior slope topography of the USACE Levee. Recent topographic data (TVA, 2015) also show that there has been no change to the exterior slope of the USACE Levee. Minor geometry modifications were made to the models based on recent bathymetric data (Tuck Mapping Solutions, Inc., 2014). The model surface geometry and material properties provided in the historical reports have been deemed appropriate for use in this slope stability assessment.

The summary of the historic slope stability results are listed in Table 10.

Table 10 Historic Slope Stability Results

Cross Section	Exterior Slope Global Failure	Pool Elevation	Reference
A-A'	2.0	Normal Pool	Stantec Consulting Services Inc., 2010c
	1.8	Max. Storage Pool	
	1.6	Max. Surcharge Pool	
B-B'	2.2	Normal Pool	Stantec Consulting Services Inc., 2010c
	1.9	Max. Storage Pool	
	1.6	Max. Surcharge Pool	
A-A'	1.9	Dredge Cell 227.1, McKellar Lake 212.9	Stantec Consulting Services Inc., 2014
B-B'	2.4	Dredge Cell 226.2, McKellar Lake 212.9	Stantec Consulting Services Inc., 2014

The subsurface geometry and surface topography for the two sections were similar. Section A-A' is located near the sanitary sewer bypass line discussed above and may have been improved by recent compaction and fill placement. Section B-B' is unmodified and considered critical based on previous stability analyses results and current field conditions. A new rapid drawdown stability analysis is required based on the proposed water levels discussed in Section 9.2.1.3.

9.2.1.3 Water Levels

The sudden drawdown slope stability analyses require assessment of changes in headwater and tailwater levels. In Stantec (2015c), the water elevations for East Ash Pond and East Stilling Pond were redefined to meet the requirements of the EPA CCR Rule inflow design flood cases. The long-term maximum storage pool elevation was selected as the high water level within the East Ash Disposal Area. Normal pool or the maximum storage pool elevation was selected as the low water level for the facility.

The long-term maximum storage pool elevations are based on the operating pool levels obtained from hydrographic survey dated February 2 and 3, 2015 (TVA, 2015). Under the CCR Rule, the inflow design flood for a significant hazard potential CCR surface impoundment is the 1,000-year flood (§257.82(a)(3)(ii)). The maximum surcharge pool elevations used in this slope stability analysis are representative of the "Late Storm Peak with Valves Closed" scenario for the 1,000-year, 6-hour storm in the hydraulic and hydrology (H&H) modeling (Stantec, 2015c). For this scenario, it is assumed that the two overflow spillways are active and discharging to the Horn Lake Cutoff. The East Ash Disposal Area is considered a significant hazard facility.

Tailwater for the model is the McKellar Lake elevation. The design flood elevation was obtained from USACE Memphis District (2015c) as 232.5 feet. The McKellar Lake drawdown level was established in Stantec (2015c) as 185 feet.

The pond elevations proposed for the sudden drawdown slope analyses are summarized in Table 11.

Table 11 ALF Water Elevations for Stability Modeling

CCR Rule Criteria	Headwater East Ash Pond Elevation (feet, NGVD29)	Tailwater McKellar Lake Elevation (feet, NGVD29)
Maximum surcharge pool loading condition	231.8	232.5
Long-term maximum storage pool loading condition	229.9	185.0

9.2.2 Analysis Methodology

Stantec performed the sudden drawdown slope stability analyses using the GeoStudio 2007, Version 7.23 software package developed by GEO-SLOPE International, Ltd. of Calgary, Alberta, Canada (GEO-SLOPE International, Ltd, 2007). This package includes the SLOPE/W module for slope stability analysis. The analyses were performed in accordance with the recommendations and criteria outlined in the USACE Design Manuals EM 1110-2-1902 "Slope Stability" (United States Army Corps of Engineers, 2003) and Stantec (2015a).

9.2.3 Acceptance Criteria

A minimum factor of safety is not explicitly specified within the EPA Final CCR Rule §257.73(d)(1)(vii). In the CCR Rule discussion, USACE (2003) is considered the basis for the slope stability analyses. Table 3-1, Minimum Required Factors of Safety: New Earth and Rock-Fill Dams, requires a factor of safety of 1.1 for a rapid drawdown condition from maximum surcharge pool (USACE, 2003).

9.2.4 Analysis Results

The slope stability assessments presented in this report are focused on the potential for slope failures of significant mass, which could directly impact potential release of water and CCR materials from the East Ash Disposal Area. The search for a critical slip surface in the slope stability assessments is thus restricted to consider only potential surfaces where the depth (measured at the base of at least one slice) is more than ten feet vertically below the ground surface. Table 12 summarizes the sudden drawdown safety factor evaluation results at the East Ash Disposal Area Section B-B'.

Table 12 Factor of Safety Assessment Results

Plant	Facility	Critical Cross Section	EPA Criteria	Recommended Factor of Safety Criteria	Calculated Factor of Safety
ALF	East Ash Disposal Area	B-B'	Sudden Drawdown	1.1	1.5

9.3 CONCLUSION

Based on the assessment of the sudden drawdown for the USACE Levee, the CCR Rule-related criteria listed above have been met.

10.0 REFERENCES

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APPENDIX A
SPILLWAY CONDITION AND CAPACITY

COMPUTATIONS

STATIC ANALYSIS OF RISER STRUCTURES
SQUARE RISER WITH CIRCULAR INLET PIPE
Structural Stability Assessment for Coal Combustion Residual
ALF - East Ash Disposal Area - ALF OVERFLOW 1 & 2
Tennessee Valley Authority

1 Objective

The purpose of this calculation is to demonstrate that under usual static loading conditions, a particular riser structure, the geometric and material properties of which have been entered by the user, will be stable considering floatation, soil bearing pressure, and strength of the reinforced concrete walls due to hydrostatic and earth pressures acting along the height of the structure.

Note: Blue text denotes an equation. Boxed text denotes a user input.

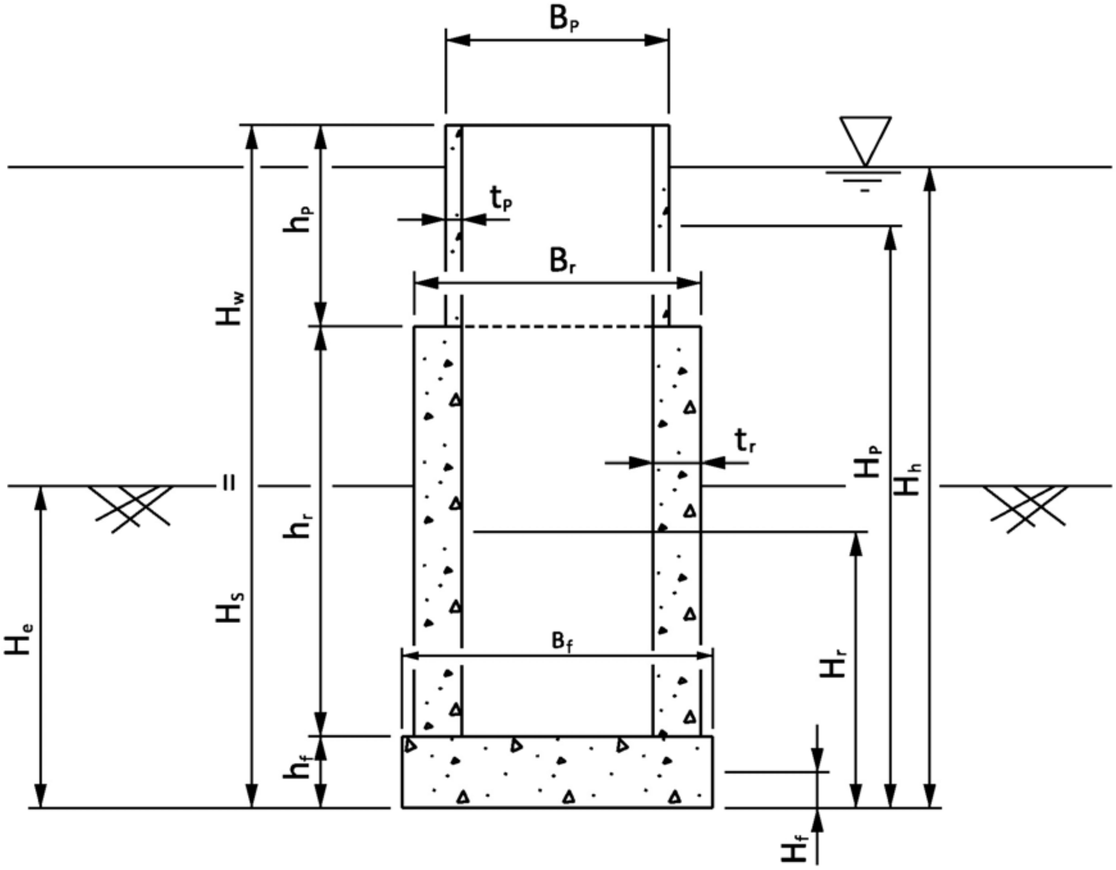

Figure 1 - Explanation of Geometric Parameters

2 Riser Geometry and Material Definitions

2.1 Riser Geometry *See original TVA Design Drawings for dimensions.*

Width of Footing (Square): $B_f := 6.5ft$

Height of Footing: $h_f := 1.5ft$

Top of Footing Elevation: $EL_{tf} := 216.0ft$ *Also reported as "Invert Elevation"*

Bottom of Footing Elevation: $EL_{bf} := EL_{tf} - h_f = 214.5\ ft$

Width of Square Riser: $B_r := 6ft$ *Out to out*

Height of Square Riser: $h_r := 4ft$

Thickness of Square Riser Walls: $t_r := 12in$

Top of Rectangular Section Elevation: $EL_{tr} := EL_{tf} + h_r = 220.0\ ft$

Diameter of Circular Inlet: $B_p := 4ft + 8in$ *Out to out*

Top of Structure Elevation: $EL_{ts} := 226.47ft$

Height of Circular Inlet: $h_p := EL_{ts} - EL_{tr} = 6.47\ ft$

Thickness of Circular Inlet Walls: $t_p := 4in$

Soil Embedment Elevation: $EL_e := 220.0ft$

Soil Embedment Height: $H_e := EL_e - EL_{bf} = 5.50\ ft$

Estimated Weight of Trash Filter on Top: $W_{trash} := 1.3kip$ *Approximated using original design plan bill of material. See TVA drawings. Weight per foot of angles taken from AISC Steel Construction Manual. Weight per foot of corrugated metal pipe taken from Contech (R) Corrugated Metal Pipe Design Guide (page 10). Extra 50 lbs added for bolts and other miscellaneous items.*

2.2 Material Properties

Unit Weight of Water: $\boxed{\gamma_w := 62.4\text{pcf}}$

2.2.1 Soil Materials

Saturated Unit Weight of Fill: $\boxed{\gamma_{sat} := 105\text{pcf}}$ *See TVA Static Slope Stability Analysis Report.*

Angle of Internal Friction: $\boxed{\phi_f := 25\text{deg}}$ *See TVA Static Slope Stability Analysis Report.*

Friction Angle: $\boxed{\delta := 27\text{deg}}$ *U.S. Department of the Navy, 1982a.*

Angle of face of wall to horizontal: $\theta := 90\text{deg}$ *Assumes vertical wall.*

Angle of fill to horizontal: $\beta := 0\text{deg}$ *Assumes level fill.*

Gamma: $\Gamma := \left(1 + \sqrt{\dfrac{\sin(\phi_f + \delta)\sin(\phi_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right)^2 = 2.596$ *U.S. Department of the Navy, 1982a.*

Active Lateral Earth Pressure Ratio: $K_a := \dfrac{\sin(\theta + \phi_f)^2}{\Gamma \cdot \sin(\theta)^2 \sin(\theta - \delta)} = 0.355$ *U.S. Department of the Navy, 1982a.*

Ultimate Bearing Pressure: $\boxed{P_b := 3855\text{psf}}$ *See geotechnical bearing pressure calculation.*

2.2.2 Reinforced Concrete Parameters

Unit Weight of Concrete: $\boxed{\gamma_c := 150\text{pcf}}$ *Normal weight concrete.*

Concrete Compressive Strength: $\boxed{f_c := 4\text{ksi}}$ *Class A concrete. See original TVA design plans.*

Assumed max. concrete compressive strain: $\boxed{\varepsilon_c := 0.003}$

Steel yield stress: $f_y := 60ksi$ *See TVA original design plans.*

Steel Modulus of Elasticity: $E_s := 29000ksi$ *Assumed*

Area of Horizontal Reinforcing Bar: $A_{horiz} := 0.2in^2$ *Area of one bar*

Horizontal Reinforcement Spacing: $s_{horiz} := 12in$ *#4 @ 12 - Single Layer*

Area of Vertical Reinforcing Bar: $A_{vert} := 0.2in^2$ *6-#4 Single Layer*

No. of Vertical Bars: $n_{vert} := 6$

Depth from concrete surface to steel: $d := 6in - \frac{9}{16}in = 5.437 \cdot in$ *Assume for both vertical and horizontal steel. This value is half of the wall thickness minus half the diameter of vertical steel minus half the diameter of horizontal steel.*

Stantec

COMPUTATIONS

3 Floatation Factor of Safety

3.1 Assumptions

(a) Riser structure is fully submerged.
(b) Buoyant unit weight of soil to be used for embedment fill.
(c) Structure is completely drained. That is to say, there is no water inside so that the uplift force from buoyancy is maximum.

3.2 Area and Weight Computations

Area of Footing: $A_f := B_f^2 = 42.25\,ft^2$

Weight of Footing: $W_f := A_f \cdot h_f \cdot \gamma_c = 9.51\,kip$

Area of Rectangular Section: $A_r := B_r^2 - (B_r - 2 \cdot t_r)^2 = 20\,ft^2$

Weight of Rectangular Section: $W_r := A_r \cdot \gamma_c \cdot h_r = 12.00\,kip$

Area of Circular Inlet: $A_p := \frac{\pi}{4} \cdot B_p^2 - \frac{\pi}{4} \cdot (B_p - 2 \cdot t_p)^2 = 4.54\,ft^2$

Weight of Circular Inlet: $W_p := A_p \cdot \gamma_c \cdot h_p = 4.40\,kip$

Total Dry Weight of Riser: $W_T := W_f + W_r + W_p = 25.91 \cdot kip$

Does not include trash filter in the event that it has fallen over (since the trash filter is not anchored).

3.3 Hydrostatic and Soil Fill Forces

Hydrostatic pressure at base: $P_h := \gamma_w \cdot (EL_{ts} - EL_{bf}) = 5.187 \cdot psi$

Hydrostatic uplift force: $U_h := P_h \cdot A_f = 31.56\,kip$

Weight of water around riser and above footing:

$$W_{wext} := \left[h_p \cdot \left(B_f^2 - \frac{\pi}{4} \cdot B_p^2\right) + (h_r - H_e) \cdot \left(B_f^2 - B_r^2\right)\right] \cdot \gamma_w = 9.567\,kip$$

Weight of Fill: $W_F := (H_e - h_f)\cdot(B_f^{\,2} - B_r^{\,2})\cdot\gamma_{sat} = 2.62\ \text{kip}$

3.4 Conclusion of Floatation Factor of Safety *Factor of Safety Limits Governed by TVA CCR Rule*

Factor of Safety: $FS_f := \dfrac{W_T + W_F}{U_h - W_{wext}} = 1.30$

Usual Load Combination: $U_f := \left|\begin{array}{ll} \text{"Satisfactory"} & \text{if } FS_f \geq 1.3 \\ \text{"Unsatisfactory"} & \text{otherwise} \end{array}\right. = \text{"Unsatisfactory"}$

4 Computation of Foundation Bearing Safety Factor

4.1 Assumptions

(a) Structure is submerged in water.
(b) Entire Structure is filled with water.

4.2 Weight of Water Inside Riser

Volume of Rectangular Section Void:

$$v_r := \left(B_r - 2t_r\right)^2 \cdot h_r = 64\,ft^3$$

Volume of Circular Inlet Pipe Void:

$$v_p := \frac{\pi}{4}\left(B_p - 2t_p\right)^2 \cdot h_p = 81.3\,ft^3$$

Total Volume of Void:

$$v_v := v_r + v_p = 145.3\,ft^3$$

Total Weight of Water Inside Riser:

$$W_{wint} := v_v \cdot \gamma_w = 9.07\,kip$$

4.3 Bearing Factor of Safety

Pressure on Foundation (Riser Fully Submerged and Filled):

$$P_{fsub} := \frac{W_T - U_h + W_F + W_{wext} + W_{wint} + W_{trash}}{B_f^{\,2}} = 400.27 \cdot psf$$

Pressure on Foundation (Riser in Air and Empty):

$$P_{fair} := \frac{W_T + W_F}{B_f^{\,2}} = 675.39 \cdot psf$$

Usual Load Factor of Safety:

$$FS_{UB} := \frac{P_b}{P_{fsub}} = 9.631$$

Unusual Load Factor of Safety:

$$FS_{UNB} := \frac{P_b}{P_{fair}} = 5.71$$

4.4 Conclusion of Foundation Bearing Capacity *Factor of Safety Limits Governed by TVA CCR Rule*

Usual Load Combination:

$$U_B := \begin{cases} \text{"Satisfactory"} & \text{if } FS_{UB} \ge 3.0 \\ \text{"Unsatisfactory"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$$

Unusual Load Combination:

$$UN_B := \begin{cases} \text{"Satisfactory"} & \text{if } FS_{UNB} \ge 2.6 \\ \text{"Unsatisfactory"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$$

5 Determination of Maximum Force Effects in Wall Using Plate Reactions and Moments Tables

5.1 Depths

Depth to Top of Rectangular Section: $z_1 := EL_{ts} - EL_{tr} = 6.47\,\mathrm{ft}$

Depth to Bottom of Rectangular Section: $z_2 := EL_{ts} - EL_{tf} = 10.47\,\mathrm{ft}$

5.2 Water Pressure

Factored Pressure at Top of Rectangular Section: $P_{w1} := \gamma_w \cdot z_1 = 403.73 \cdot \mathrm{psf}$

Factored Pressure at Bottom of Rectangular Section: $P_{w2} := \gamma_w \cdot z_2 = 653.328 \cdot \mathrm{psf}$

5.3 Soil Pressure

Factored Pressure at top of Rectangular Section:

$$P_{s1} := \left| \begin{array}{ll} K_a \cdot \left(EL_e - EL_{tr}\right) \cdot \gamma_{sat} & \text{if } EL_e > EL_{tr} \\ 0 \cdot \mathrm{psf} & \text{otherwise} \end{array} \right. = 0 \cdot \mathrm{psf}$$

Factored Pressure at Bottom of Rectangular Section: $P_{s2} := K_a \cdot \left(EL_e - EL_{tf}\right) \cdot \gamma_{sat} = 149.119 \cdot \mathrm{psf}$

5.4 Factored Line Loads

Load Factor: $F := 1.4$ *ACI 350-06, (9-1)*

Factored Line Load at top: $w_{top} := F \cdot \left(P_{w1} + P_{s1}\right) \cdot B_r = 3.391\,\mathrm{klf}$

Factored Line Load at bottom: $w_{bott} := F \cdot \left(P_{w2} + P_{s2}\right) \cdot B_r = 6.741\,\mathrm{klf}$

5.5 Moment and Shear Coefficients from Engineering Monograph 27 Table

5.5.1 Uniform Load

These values were taken from Engineering Monograph 27 figure 1 and figure 4 for a uniform load and a triangular load, respectively.

Moment about vertical axis: $C_{mxu} := 0.1788$

Moment about horizontal axis: $C_{myu} := 0.1212$

Shear at corner of wall: $C_{rxu} := 0.8592$

Shear at base of wall: $C_{ryu} := 0.6725$

5.5.2 Triangular Load

Moment about vertical axis: $C_{mxt} := 0.0433$

Moment about horizontal axis: $C_{myt} := 0.0584$

Shear at corner of wall: $C_{rxt} := 0.2542$

Shear at base of wall: $C_{ryt} := 0.4055$

5.6 Maximum Applied Moment and Shear

Maximum Moment about vertical axis:

$$M_{u1} := C_{mxu} \cdot w_{top} \cdot h_r^2 + C_{mxt} \cdot \left(w_{bott} - w_{top}\right) \cdot h_r^2 = 12.022 \text{ ft} \cdot \text{kip}$$

Maximum Moment about horizontal axis:

$$M_{u2} := C_{myu} \cdot w_{top} \cdot h_r^2 + C_{myt} \cdot \left(w_{bott} - w_{top}\right) \cdot h_r^2 = 9.706 \text{ ft} \cdot \text{kip}$$

Maximum Shear at corner:

$$V_{u1} := C_{rxu} \cdot w_{top} \cdot h_r + C_{rxt} \cdot \left(w_{bott} - w_{top}\right) \cdot h_r = 15.061 \text{ kip}$$

Maximum Shear at base:

$$V_{u2} := C_{ryu} \cdot w_{top} \cdot h_r + C_{ryt} \cdot \left(w_{bott} - w_{top}\right) \cdot h_r = 14.555 \text{ kip}$$

6 Capacity of Wall for Flexure About Vertical Axis

6.1 Material Parameters

Total area of steel: $A_s := A_{horiz}\left(\frac{h_r}{s_{horiz}}\right) = 0.8 \cdot in^2$

6.2 Nominal Moment Capacity Using the Equivalent Stress Block Approach

See ACI 350-06, 10.2

Concrete stress block: $a := \frac{A_s f_y + 2 \cdot V_{u1}}{0.85 \cdot f_c \cdot h_r} = 0.479 \cdot in$

Concrete stress block reduction factor: $\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \left[0.85 - 0.05\left(\frac{f_c - 4000psi}{1000psi}\right)\right] & \text{otherwise} \end{cases} = 0.85$ *ACI 350-06, R10.2.7*

Concrete Compression Zone: $c := \frac{a}{\beta_1} = 0.563 \cdot in$

Strain in steel: $\varepsilon_s := \varepsilon_c \cdot \left(\frac{d - c}{c}\right) = 0.026$

Yield strain: $\varepsilon_y := \frac{f_y}{E_s} = 0.0021$

Reduction Factor: $\phi_{ff} := \begin{cases} 0.9 & \text{if } \varepsilon_s \ge 0.005 \\ \left[0.65 + \left(\varepsilon_s - \varepsilon_y\right) \cdot \frac{0.25}{0.005 - \varepsilon_y}\right] & \text{otherwise} \end{cases} = 0.9$ *ACI 350-06, 9.3.2.1, 9.3.2.2*

Steel Yielding?: $yield := \begin{cases} \text{"TRUE"} & \text{if } \varepsilon_s \ge \varepsilon_y \\ \text{"FALSE"} & \text{otherwise} \end{cases} = \text{"TRUE"}$

Nominal moment capacity:
$$M_c := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) + 2 \cdot V_{u1}\left(\frac{t_r}{2} - \frac{a}{2}\right) = 35.253 \cdot kip \cdot ft$$

Moment Capacity:
$$\phi M_n := \phi_{ff} \cdot M_c = 31.727\, ft \cdot kip$$

6.3 Shear Capacity

6.3.1 Material Parameters

Concrete shear stress:
$$f_v := 2 \cdot \sqrt{f_c \cdot \frac{1000}{ksi}} \cdot \frac{ksi}{1000} = 0.1265\, ksi$$
ACI 350-06, (11-3)

6.3.2 Computation of Shear Strength

Concrete Shear Strength:
$$V_c := f_v \cdot h_r \cdot t_r = 72.859\, kip$$

Reduction Factor:
$$\phi_{vv} := 0.75$$
ACI 350-06, 9.3.2.3

Total Shear Strength:
$$\phi V_n := \phi_{vv} \cdot V_c = 54.644\, kip$$

6.4 Conclusion on Strength of Rectangular Section for Flexure About Vertical Axis

Factor of Safety for Flexure at corner:
$$FS_f := \frac{\phi M_n}{M_{u1}} = 2.639$$

Factor of Safety for Shear:
$$FS_v := \frac{\phi V_n}{V_{u1}} = 3.628$$

7 Capacity of Wall For Flexure About Horizontal Axis

7.1 Material Parameters

Total area of steel: $A_s := A_{vert} n_{vert} = 1.2 \cdot in^2$

7.2 Nominal Moment Capacity Using the Equivalent Stress Block Approach

See ACI 350-06, 10.2

Concrete stress block: $a := \dfrac{A_s f_y}{0.85 \cdot f_c \cdot B_r} = 0.294 \cdot in$

Concrete stress block reduction factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \left[0.85 - 0.05\left(\dfrac{f_c - 4000psi}{1000psi}\right)\right] & \text{otherwise} \end{cases} = 0.85$$

ACI 350-06, R10.2.7

Concrete Compression Zone: $c := \dfrac{a}{\beta_1} = 0.346 \cdot in$

Strain in steel: $\varepsilon_s := \varepsilon_c \cdot \left(\dfrac{d - c}{c}\right) = 0.044$

Yield strain: $\varepsilon_y := \dfrac{f_y}{E_s} = 0.0021$

Reduction Factor:

$$\phi_{ff} := \begin{cases} 0.9 & \text{if } \varepsilon_s \ge 0.005 \\ \left[0.65 + \left(\varepsilon_s - \varepsilon_y\right) \cdot \dfrac{0.25}{0.005 - \varepsilon_y}\right] & \text{otherwise} \end{cases} = 0.9$$

ACI 350-06, 9.3.2.1, 9.3.2.2

Steel Yielding?:

$$yield := \begin{cases} \text{"TRUE"} & \text{if } \varepsilon_s \ge \varepsilon_y \\ \text{"FALSE"} & \text{otherwise} \end{cases} = \text{"TRUE"}$$

Nominal moment capacity: $M_c := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 31.743\,ft \cdot kip$

Moment Capacity: $\phi M_n := \phi_{ff} \cdot M_c = 28.568\,ft \cdot kip$

7.3 Shear Capacity

7.3.1 Material Parameters

Concrete shear stress: $f_v := 2 \cdot \sqrt{f_c \cdot \frac{1000}{ksi}} \cdot \frac{ksi}{1000} = 0.1265\,ksi$ *ACI 350-06, (11-3)*

7.3.2 Computation of Shear Strength

Concrete Shear Strength: $V_c := f_v \cdot B_r \cdot t_r = 109.288\,kip$

Reduction Factor: $\phi_{vv} := 0.75$ *ACI 350-06, 9.3.2.3*

Total Shear Strength: $\phi V_n := \phi_{vv} \cdot V_c = 81.966\,kip$

7.4 Conclusion on Strength of Rectangular Section for Flexure About Horizontal Axis

Factor of Safety for Flexure at footing: $FS_f := \frac{\phi M_n}{M_{u2}} = 2.943$

Factor of Safety for Shear: $FS_v := \frac{\phi V_n}{V_{u2}} = 5.631$

8 References

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STATIC ANALYSIS OF RISER STRUCTURES
SQUARE RISER WITH CIRCULAR INLET PIPE
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ALF - East Ash Disposal Area - ALF WEST & ALF EAST
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1 Objective

The purpose of this calculation is to demonstrate that under usual static loading conditions, a particular riser structure, the geometric and material properties of which have been entered by the user, will be stable considering floatation, soil bearing pressure, and strength of the reinforced concrete walls due to hydrostatic and earth pressures acting along the height of the structure.

Note: Blue text denotes an equation. Boxed text denotes a user input.

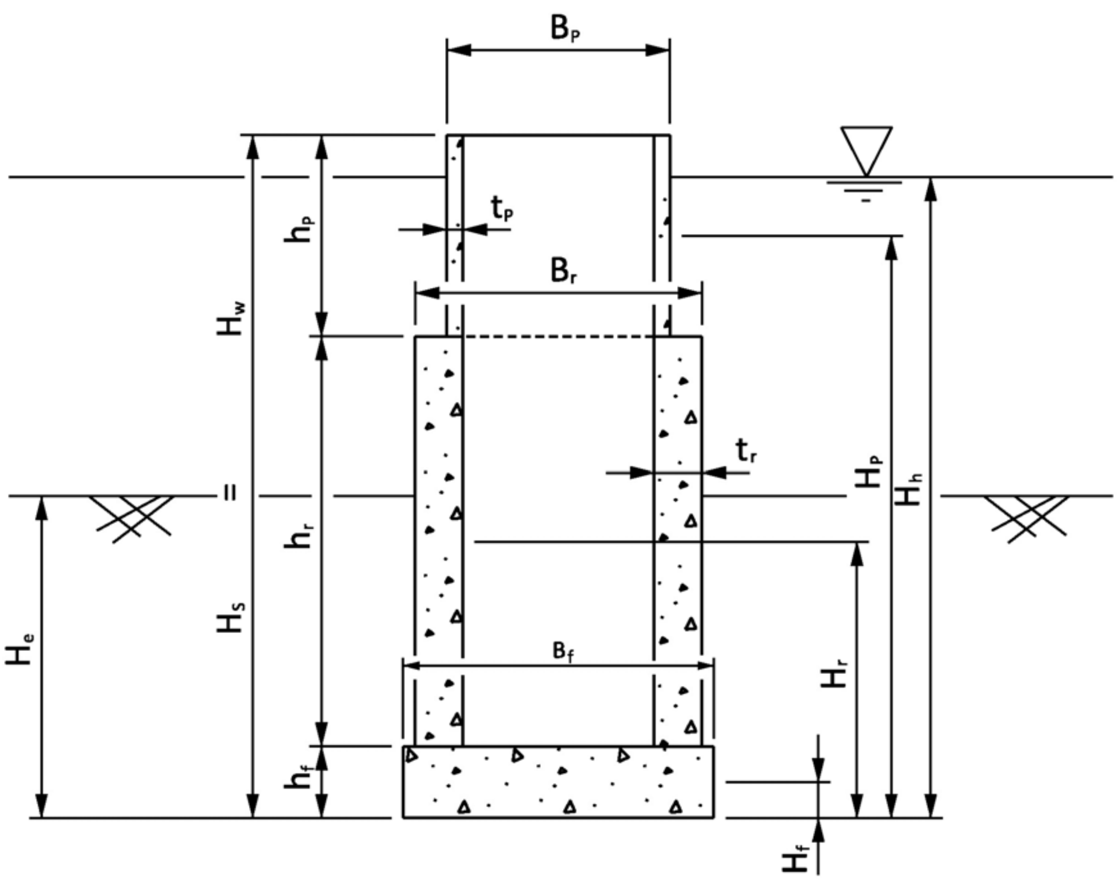

Figure 1 - Explanation of Geometric Parameters

2 Riser Geometry and Material Definitions

2.1 Riser Geometry *See original TVA Design Drawings for dimensions.*

Width of Footing (Square):	$B_f := 6.5ft$	
Height of Footing:	$h_f := 1.5ft$	
Top of Footing Elevation:	$EL_{tf} := 216.0ft$	*Also reported as "Invert Elevation"*
Bottom of Footing Elevation:	$EL_{bf} := EL_{tf} - h_f = 214.5\ ft$	
Width of Square Riser:	$B_r := 6ft$	*Out to out*
Height of Square Riser:	$h_r := 4ft$	
Thickness of Square Riser Walls:	$t_r := 12in$	
Top of Rectangular Section Elevation:	$EL_{tr} := EL_{tf} + h_r = 220.0\ ft$	
Diameter of Circular Inlet:	$B_p := 4ft + 8in$	*Out to out*
Top of Structure Elevation:	$EL_{ts} := 225.40ft$	
Height of Circular Inlet:	$h_p := EL_{ts} - EL_{tr} = 5.4\ ft$	
Thickness of Circular Inlet Walls:	$t_p := 4in$	
Soil Embedment Elevation:	$EL_e := 220.0ft$	
Soil Embedment Height:	$H_e := EL_e - EL_{bf} = 5.50\ ft$	
Estimated Weight of Trash Filter on Top:	$W_{trash} := 1.3kip$	*Approximated using original design plan bill of material. See TVA drawings. Weight per foot of angles taken from AISC Steel Construction Manual. Weight per foot of corrugated metal pipe taken from Contech (R) Corrugated Metal Pipe Design Guide (page 10). Extra 50 lbs added for bolts and other miscellaneous items.*

2.2 Material Properties

Unit Weight of Water: $\boxed{\gamma_w := 62.4\text{pcf}}$

2.2.1 Soil Materials

Saturated Unit Weight of Fill: $\boxed{\gamma_{sat} := 105\text{pcf}}$ *See TVA Static Slope Stability Analysis Report.*

Angle of Internal Friction: $\boxed{\phi_f := 25\text{deg}}$ *See TVA Static Slope Stability Analysis Report.*

Friction Angle: $\boxed{\delta := 27\text{deg}}$ *U.S. Department of the Navy, 1982a.*

Angle of face of wall to horizontal: $\theta := 90\text{deg}$ *Assumes vertical wall.*

Angle of fill to horizontal: $\beta := 0\text{deg}$ *Assumes level fill.*

Gamma: $\Gamma := \left(1 + \sqrt{\dfrac{\sin(\phi_f + \delta)\sin(\phi_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right)^2 = 2.596$ *U.S. Department of the Navy, 1982a.*

Active Lateral Earth Pressure Ratio: $K_a := \dfrac{\sin(\theta + \phi_f)^2}{\Gamma \cdot \sin(\theta)^2 \sin(\theta - \delta)} = 0.355$ *U.S. Department of the Navy, 1982a.*

Ultimate Bearing Pressure: $\boxed{P_b := 3855\text{psf}}$ *See geotechnical bearing pressure calculation.*

2.2.2 Reinforced Concrete Parameters

Unit Weight of Concrete: $\boxed{\gamma_c := 150\text{pcf}}$ *Normal weight concrete.*

Concrete Compressive Strength: $\boxed{f_c := 4\text{ksi}}$ *Class A concrete. See original TVA design plans.*

Assumed max. concrete compressive strain: $\boxed{\varepsilon_c := 0.003}$

Steel yield stress: $\boxed{f_y := 60ksi}$ *See TVA original design plans.*

Steel Modulus of Elasticity: $E_s := 29000ksi$ *Assumed*

Area of Horizontal Reinforcing Bar: $\boxed{A_{horiz} := 0.2in^2}$ *Area of one bar*

Horizontal Reinforcement Spacing: $\boxed{s_{horiz} := 12in}$ *#4 @ 12 - Single Layer*

Area of Vertical Reinforcing Bar: $\boxed{A_{vert} := 0.2in^2}$ *6-#4 Single Layer*

No. of Vertical Bars: $\boxed{n_{vert} := 6}$

Depth from concrete surface to steel: $\boxed{d := 6in - \frac{9}{16}in = 5.437 \cdot in}$ *Assume for both vertical and horizontal steel. This value is half of the wall thickness minus half the diameter of vertical steel minus half the diameter of horizontal steel.*

3 Floatation Factor of Safety

3.1 Assumptions

(a) Riser structure is fully submerged.
(b) Buoyant unit weight of soil to be used for embedment fill.
(c) Structure is completely drained. That is to say, there is no water inside so that the uplift force from buoyancy is maximum.

3.2 Area and Weight Computations

Area of Footing: $A_f := B_f^2 = 42.25\,ft^2$

Weight of Footing: $W_f := A_f \cdot h_f \cdot \gamma_c = 9.51\,kip$

Area of Rectangular Section: $A_r := B_r^2 - (B_r - 2 \cdot t_r)^2 = 20\,ft^2$

Weight of Rectangular Section: $W_r := A_r \cdot \gamma_c \cdot h_r = 12.00\,kip$

Area of Circular Inlet: $A_p := \frac{\pi}{4} \cdot B_p^2 - \frac{\pi}{4} \cdot (B_p - 2 \cdot t_p)^2 = 4.54\,ft^2$

Weight of Circular Inlet: $W_p := A_p \cdot \gamma_c \cdot h_p = 3.68\,kip$

Total Dry Weight of Riser: $W_T := W_f + W_r + W_p = 25.18 \cdot kip$

Does not include trash filter in the event that it has fallen over (since the trash filter is not anchored).

3.3 Hydrostatic and Soil Fill Forces

Hydrostatic pressure at base: $P_h := \gamma_w \cdot (EL_{ts} - EL_{bf}) = 4.723 \cdot psi$

Hydrostatic uplift force: $U_h := P_h \cdot A_f = 28.74\,kip$

Weight of water around riser and above footing: $W_{wext} := \left[h_p \cdot \left(B_f^2 - \frac{\pi}{4} \cdot B_p^2\right) + (h_r - H_e) \cdot \left(B_f^2 - B_r^2\right)\right] \cdot \gamma_w = 7.888\,kip$

Weight of Fill: $W_F := (H_e - h_f) \cdot (B_f^{\,2} - B_r^{\,2}) \cdot \gamma_{sat} = 2.62\,\text{kip}$

3.4 Conclusion of Floatation Factor of Safety *Factor of Safety Limits Governed by TVA CCR Rule*

Factor of Safety: $FS_f := \dfrac{W_T + W_F}{U_h - W_{wext}} = 1.33$

Usual Load Combination: $U_f := \begin{cases} \text{"Satisfactory"} & \text{if } FS_f \geq 1.3 \\ \text{"Unsatisfactory"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$

4 Computation of Foundation Bearing Safety Factor

4.1 Assumptions

(a) Structure is submerged in water.
(b) Entire Structure is filled with water.

4.2 Weight of Water Inside Riser

Volume of Rectangular Section Void:

$$v_r := (B_r - 2t_r)^2 \cdot h_r = 64\,ft^3$$

Volume of Circular Inlet Pipe Void:

$$v_p := \frac{\pi}{4}(B_p - 2t_p)^2 \cdot h_p = 67.86\,ft^3$$

Total Volume of Void:

$$v_v := v_r + v_p = 131.86\,ft^3$$

Total Weight of Water Inside Riser:

$$W_{wint} := v_v \cdot \gamma_w = 8.23\,kip$$

4.3 Bearing Factor of Safety

Pressure on Foundation (Riser Fully Submerged and Filled):

$$P_{fsub} := \frac{W_T - U_h + W_F + W_{wext} + W_{wint} + W_{trash}}{B_f^{\,2}} = 390.21 \cdot psf$$

Pressure on Foundation (Riser in Air and Empty):

$$P_{fair} := \frac{W_T + W_F}{B_f^{\,2}} = 658.15 \cdot psf$$

Usual Load Factor of Safety:

$$FS_{UB} := \frac{P_b}{P_{fsub}} = 9.879$$

Unusual Load Factor of Safety:

$$FS_{UNB} := \frac{P_b}{P_{fair}} = 5.86$$

4.4 Conclusion of Foundation Bearing Capacity *Factor of Safety Limits Governed by TVA CCR Rule*

Usual Load Combination:

$$U_B := \begin{vmatrix} \text{"Satisfactory"} \quad \text{if } FS_{UB} \geq 3.0 \\ \text{"Unsatisfactory"} \quad \text{otherwise} \end{vmatrix} = \text{"Satisfactory"}$$

Unusual Load Combination:

$$UN_B := \begin{vmatrix} \text{"Satisfactory"} \quad \text{if } FS_{UNB} \geq 2.6 \\ \text{"Unsatisfactory"} \quad \text{otherwise} \end{vmatrix} = \text{"Satisfactory"}$$

5 Determination of Maximum Force Effects in Wall Using Plate Reactions and Moments Tables

5.1 Depths

Depth to Top of Rectangular Section: $z_1 := EL_{ts} - EL_{tr} = 5.4\,ft$

Depth to Bottom of Rectangular Section: $z_2 := EL_{ts} - EL_{tf} = 9.4\,ft$

5.2 Water Pressure

Factored Pressure at Top of Rectangular Section: $P_{w1} := \gamma_w \cdot z_1 = 336.96 \cdot psf$

Factored Pressure at Bottom of Rectangular Section: $P_{w2} := \gamma_w \cdot z_2 = 586.56 \cdot psf$

5.3 Soil Pressure

Factored Pressure at top of Rectangular Section:

$$P_{s1} := \begin{vmatrix} K_a \cdot (EL_e - EL_{tr}) \cdot \gamma_{sat} \ \text{ if } \ EL_e > EL_{tr} \\ 0 \cdot psf \quad \text{otherwise} \end{vmatrix} = 0 \cdot psf$$

Factored Pressure at Bottom of Rectangular Section: $P_{s2} := K_a \cdot (EL_e - EL_{tf}) \cdot \gamma_{sat} = 149.119 \cdot psf$

5.4 Factored Line Loads

Load Factor: $F := 1.4$ *ACI 350-06, (9-1)*

Factored Line Load at top: $w_{top} := F \cdot (P_{w1} + P_{s1}) \cdot B_r = 2.83\,klf$

Factored Line Load at bottom: $w_{bott} := F \cdot (P_{w2} + P_{s2}) \cdot B_r = 6.18\,klf$

5.5 Moment and Shear Coefficients from Engineering Monograph 27 Table

5.5.1 Uniform Load

These values were taken from Engineering Monograph 27 figure 1 and figure 4 for a uniform load and a triangular load, respectively.

Moment about vertical axis: $C_{mxu} := 0.1788$

Moment about horizontal axis: $C_{myu} := 0.1212$

Shear at corner of wall: $C_{rxu} := 0.8592$

Shear at base of wall: $C_{ryu} := 0.6725$

5.5.2 Triangular Load

Moment about vertical axis: $C_{mxt} := 0.0433$

Moment about horizontal axis: $C_{myt} := 0.0584$

Shear at corner of wall: $C_{rxt} := 0.2542$

Shear at base of wall: $C_{ryt} := 0.4055$

5.6 Maximum Applied Moment and Shear

Maximum Moment about vertical axis:

$$M_{u1} := C_{mxu} \cdot w_{top} \cdot h_r^2 + C_{mxt} \cdot \left(w_{bott} - w_{top}\right) \cdot h_r^2 = 10.418\ ft \cdot kip$$

Maximum Moment about horizontal axis:

$$M_{u2} := C_{myu} \cdot w_{top} \cdot h_r^2 + C_{myt} \cdot \left(w_{bott} - w_{top}\right) \cdot h_r^2 = 8.618\ ft \cdot kip$$

Maximum Shear at corner:

$$V_{u1} := C_{rxu} \cdot w_{top} \cdot h_r + C_{rxt} \cdot \left(w_{bott} - w_{top}\right) \cdot h_r = 13.133\ kip$$

Maximum Shear at base:

$$V_{u2} := C_{ryu} \cdot w_{top} \cdot h_r + C_{ryt} \cdot \left(w_{bott} - w_{top}\right) \cdot h_r = 13.046\ kip$$

6 Capacity of Wall for Flexure About Vertical Axis

6.1 Material Parameters

Total area of steel: $A_s := A_{horiz}\left(\frac{h_r}{s_{horiz}}\right) = 0.8 \cdot in^2$

6.2 Nominal Moment Capacity Using the Equivalent Stress Block Approach

See ACI 350-06, 10.2

Concrete stress block: $a := \frac{A_s f_y + 2 \cdot V_{u1}}{0.85 \cdot f_c \cdot h_r} = 0.455 \cdot in$

Concrete stress block reduction factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \left[0.85 - 0.05\left(\frac{f_c - 4000psi}{1000psi}\right)\right] & \text{otherwise} \end{cases} = 0.85$$

ACI 350-06, R10.2.7

Concrete Compression Zone: $c := \frac{a}{\beta_1} = 0.535 \cdot in$

Strain in steel: $\varepsilon_s := \varepsilon_c \cdot \left(\frac{d - c}{c}\right) = 0.027$

Yield strain: $\varepsilon_y := \frac{f_y}{E_s} = 0.0021$

Reduction Factor:

$$\phi_{ff} := \begin{cases} 0.9 & \text{if } \varepsilon_s \ge 0.005 \\ \left[0.65 + \left(\varepsilon_s - \varepsilon_y\right) \cdot \frac{0.25}{0.005 - \varepsilon_y}\right] & \text{otherwise} \end{cases} = 0.9$$

ACI 350-06, 9.3.2.1, 9.3.2.2

Steel Yielding?:

$$yield := \begin{cases} \text{"TRUE"} & \text{if } \varepsilon_s \ge \varepsilon_y \\ \text{"FALSE"} & \text{otherwise} \end{cases} = \text{"TRUE"}$$

Nominal moment capacity: $M_c := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) + 2 \cdot V_{u1}\left(\frac{t_r}{2} - \frac{a}{2}\right) = 33.475 \cdot kip \cdot ft$

Moment Capacity: $\phi M_n := \phi_{ff} \cdot M_c = 30.128\, ft \cdot kip$

6.3 Shear Capacity

6.3.1 Material Parameters

Concrete shear stress: $f_v := 2 \cdot \sqrt{f_c \cdot \frac{1000}{ksi}} \cdot \frac{ksi}{1000} = 0.1265\, ksi$ *ACI 350-06, (11-3)*

6.3.2 Computation of Shear Strength

Concrete Shear Strength: $V_c := f_v \cdot h_r \cdot t_r = 72.859\, kip$

Reduction Factor: $\phi_{vv} := 0.75$ *ACI 350-06, 9.3.2.3*

Total Shear Strength: $\phi V_n := \phi_{vv} \cdot V_c = 54.644\, kip$

6.4 Conclusion on Strength of Rectangular Section for Flexure About Vertical Axis

Factor of Safety for Flexure at corner: $FS_f := \frac{\phi M_n}{M_{u1}} = 2.892$

Factor of Safety for Shear: $FS_v := \frac{\phi V_n}{V_{u1}} = 4.161$

7 Capacity of Wall For Flexure About Horizontal Axis

7.1 Material Parameters

Total area of steel: $A_s := A_{vert} n_{vert} = 1.2 \cdot in^2$

7.2 Nominal Moment Capacity Using the Equivalent Stress Block Approach

See ACI 350-06, 10.2

Concrete stress block: $a := \dfrac{A_s f_y}{0.85 \cdot f_c \cdot B_r} = 0.294 \cdot in$

Concrete stress block reduction factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \left[0.85 - 0.05\left(\dfrac{f_c - 4000psi}{1000psi}\right)\right] & \text{otherwise} \end{cases} = 0.85$$

ACI 350-06, R10.2.7

Concrete Compression Zone: $c := \dfrac{a}{\beta_1} = 0.346 \cdot in$

Strain in steel: $\varepsilon_s := \varepsilon_c \cdot \left(\dfrac{d - c}{c}\right) = 0.044$

Yield strain: $\varepsilon_y := \dfrac{f_y}{E_s} = 0.0021$

Reduction Factor:

$$\phi_{ff} := \begin{cases} 0.9 & \text{if } \varepsilon_s \ge 0.005 \\ \left[0.65 + \left(\varepsilon_s - \varepsilon_y\right) \cdot \dfrac{0.25}{0.005 - \varepsilon_y}\right] & \text{otherwise} \end{cases} = 0.9$$

ACI 350-06, 9.3.2.1, 9.3.2.2

Steel Yielding?:

$$yield := \begin{cases} \text{"TRUE"} & \text{if } \varepsilon_s \ge \varepsilon_y \\ \text{"FALSE"} & \text{otherwise} \end{cases} = \text{"TRUE"}$$

Nominal moment capacity: $M_c := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 31.743\,ft \cdot kip$

Moment Capacity: $\phi M_n := \phi_{ff} \cdot M_c = 28.568\,ft \cdot kip$

7.3 Shear Capacity

7.3.1 Material Parameters

Concrete shear stress: $f_v := 2 \cdot \sqrt{f_c \cdot \frac{1000}{ksi}} \cdot \frac{ksi}{1000} = 0.1265\,ksi$ *ACI 350-06, (11-3)*

7.3.2 Computation of Shear Strength

Concrete Shear Strength: $V_c := f_v \cdot B_r \cdot t_r = 109.288\,kip$

Reduction Factor: $\phi_{vv} := 0.75$ *ACI 350-06, 9.3.2.3*

Total Shear Strength: $\phi V_n := \phi_{vv} \cdot V_c = 81.966\,kip$

7.4 Conclusion on Strength of Rectangular Section for Flexure About Horizontal Axis

Factor of Safety for Flexure at footing: $FS_f := \frac{\phi M_n}{M_{u2}} = 3.315$

Factor of Safety for Shear: $FS_v := \frac{\phi V_n}{V_{u2}} = 6.283$

8 References

Engineer Manual 1110-2-2400: Structural Design and Evaluation of Outlet Works. 2003. U.S. Army Corps of Engineers.

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Engineering Monograph No. 27. *Moments and Reactions for Rectangular Plates*. United States Department of the Interior, Bureau of Reclamation. pp.7-10.

Corrugated Metal Pipe Design Guide. *Approximate Weight (Pounds/Foot) Contech Corrugated Steel Pipe*. p.10. CONTECH.

COMPUTATIONS

STATIC ANALYSIS OF HYDRAULIC DROP STRUCTURE

Structural Stability Assessment for Coal Combustion Residual
ALF - East Ash Disposal Area - OVERFLOW WEIR
Tennessee Valley Authority

1 Objective

The purpose of this calculation is to demonstrate that under static loading conditions, this particular structure, the geometric and material properties of which have been entered by the user, will be stable considering floatation, sliding, overturning, soil bearing, and strength under various loading conditions described throughout the document.

Note: Blue text denotes an equation. Boxed text denotes a user input.

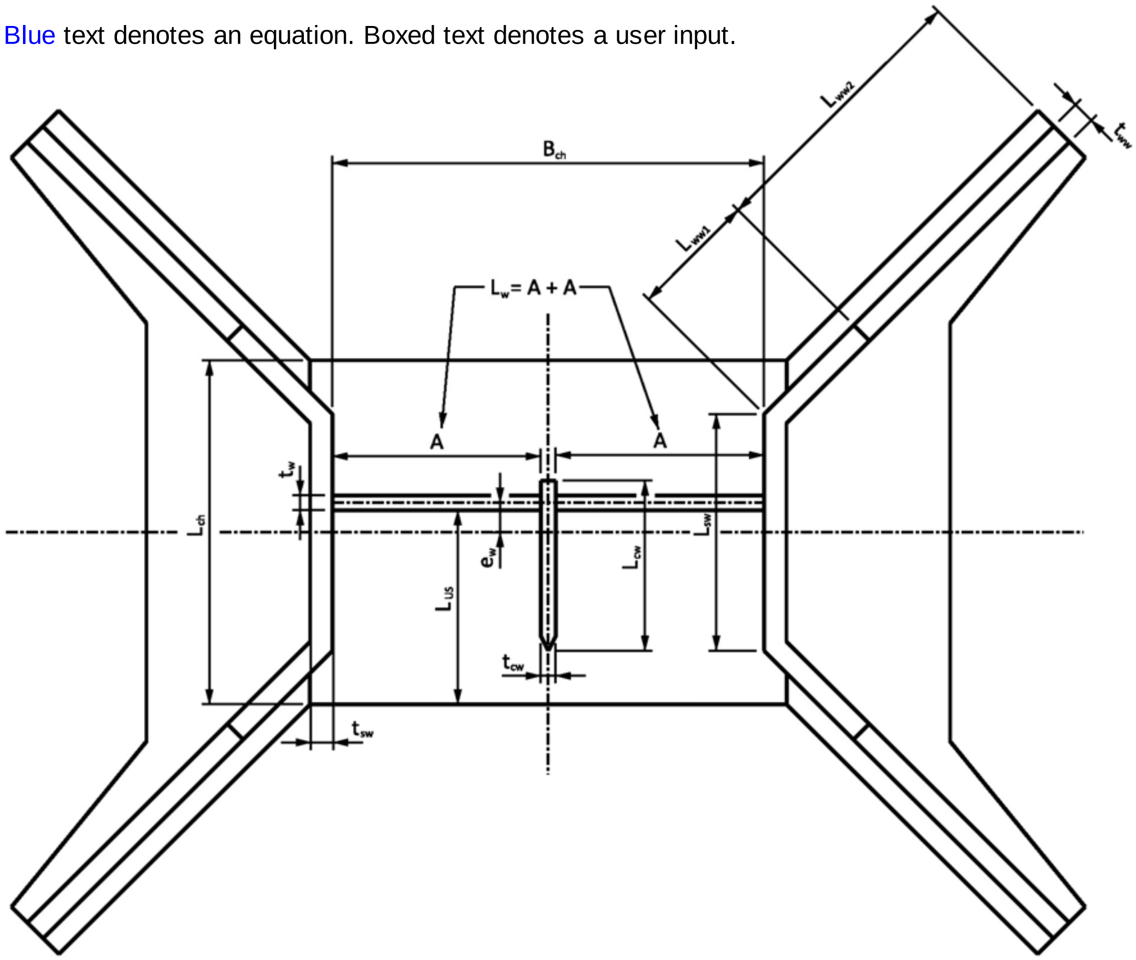

Figure 1 - Structure Plan View

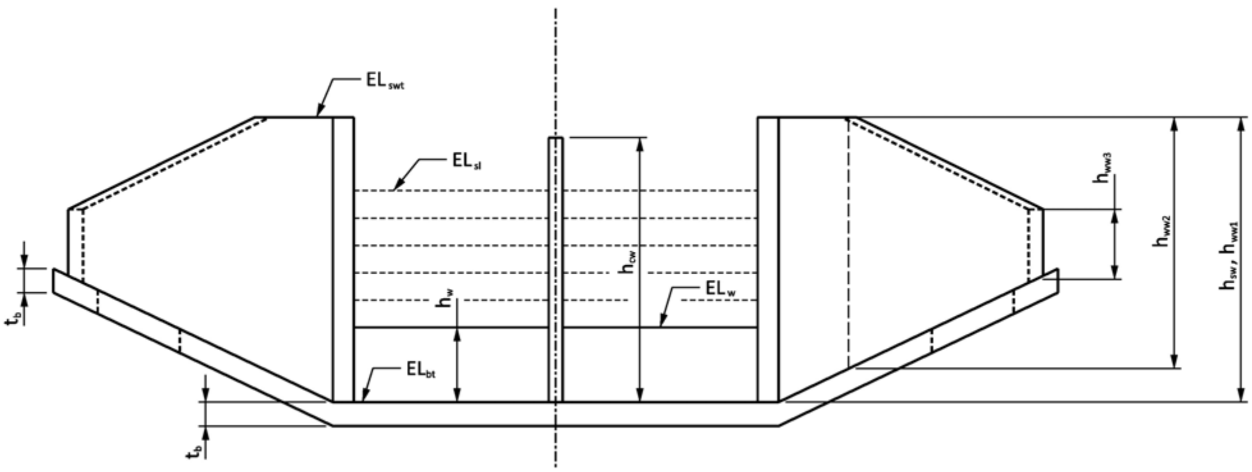

Figure 2 - Structure Elevation

2 Geometry and Material Definitions

2.1 Geometry *See TVA Drawing 10W258-1 R1*

2.1.1 Measured Elevations

Elevation - top of weir: $EL_{w} := 222.12ft$

Elevation - top of stop log beams (All installed): $EL_{sl} := 232.0ft$

Elevation - top of side wall: $EL_{swt} := 236.7ft$

Elevation - top of center wall: $EL_{cw} := 236.0ft$

Elevation - top of base: $EL_{bt} := 216.65ft$

Elevation - bottom of base: $EL_{bb} := 214.9ft$

Elevation - construction joint: $EL_{CJ} := 227.5ft$

2.1.2 Other Drawing Elevations

Elevation - top end of wing wall: $EL_{swe} := 230.75ft$

Elevation - top of wing wall footing at exposed edge: $EL_{swfe} := 225.0ft$

Elevation - top of wing wall footing at buried edge: $EL_{swfb} := 226.41ft$

Elevation - top of interior footing between wing walls: $EL_{swfi} := 222.39ft$

Elevation - top of wing wall footing at base of stem on exposed face:

$$EL_{swfw} := EL_{swfe} + \left(\frac{EL_{swfb} - EL_{swfe}}{3}\right) = 225.47\,ft$$

2.1.3 Dimensions of Structural Elements

Projected area of base: $A_b := 1725ft^2$ *Computed using Microstation*

Thickness of slab base and footings (measured vertically): $t_b := 1.75ft$

Thickness of weir stem: $t_w := 1.0ft$

Thickness of wing wall stem: $t_{ww} := 1.5ft$

Thickness of side wall stem: $t_{sw} := 1.5 \cdot ft$

Thickness of center wall stem: $t_{cw} := 1.0ft$

Length of channel along line of flow: $L_{ch} := 23.25ft$

Length of side wall at exposed face: $L_{sw} := 16.0ft$

Length of center wall: $L_{cw} := 11.5ft$

Length of flat portion of wing wall at exposed face: $\boxed{L_{ww1} := 8.5ft}$

Length of sloped portion of wing wall at exposed face: $\boxed{L_{ww2} := 19.0ft}$

Width of one weir bay: $\boxed{B_w := 14.0ft}$

Width of channel across weir: $B_{ch} := 2 \cdot B_w + t_{cw} = 29\,ft$

Width of wing wall interior footing (horizontal distance): $\boxed{B_{ww} := 11.0 \cdot ft}$

Width of base slab: $B_{sl} := 2 \cdot t_{sw} + B_{ch} = 32\,ft$

Height of weir: $h_w := EL_w - EL_{bt} = 5.47\,ft$

Height of side wall: $h_{sw} := EL_{swt} - EL_{bt} = 20.05\,ft$

Height of center wall: $h_{cw} := EL_{cw} - EL_{bt} = 19.35\,ft$

Height of wing wall at corner: $h_{ww1} := h_{sw} = 20.05\,ft$

Height of wing wall at beginning of slope on exposed face side:

$$h_{ww2} := EL_{swt} - EL_{swfw} + \left[\frac{L_{ww2}}{\left(L_{ww1} + L_{ww2}\right)}\right] \cdot \left(EL_{swfw} - EL_{bt}\right) = 17.32\,ft$$

Height of wing wall at end at exposed base of stem: $h_{ww3} := EL_{swe} - EL_{swfw} = 5.28\,ft$

Eccentricity of weir from centerline: $\boxed{e_w := 2ft}$

Eccentricity of center wall: $\boxed{e_{cw} := 2.25ft}$

Length of upstream portion of channel along line of flow:

$$L_{US} := \frac{L_{ch}}{2} + e_w - \frac{t_w}{2} = 13.125\,ft$$

2.2 Water Elevations

The controlling load condition for this structure is the reservoir at max normal pool elevation and the downstream stilling pool at minimum elevation. At flood conditions, the stop logs are removed and the tailwater rises inundating the downstream slab. Thus the tailwater provides a resisting force against the side and wing walls.

Upstream pool elevation: $EL_{US} := 232.0ft$ *All stop logs installed*

Downstream pool elevation: $EL_{DS} := 222.0ft$

2.3 Material Properties

Unit weight of water: $\gamma_w := 62.4pcf$

2.3.1 Soil Materials

Dry unit weight of fill: $\gamma_d := 105pcf$ *See TVA Static Slope Stability Analysis Report.*

Saturated unit weight of fill: $\gamma_{sat} := 124pcf$ *See TVA Static Slope Stability Analysis Report.*

Buoyant unit weight of fill: $\gamma_B := \gamma_{sat} - \gamma_w = 61.60\ pcf$

Angle of internal friction: $\phi_f := 31deg$ *See TVA Static Slope Stability Analysis Report.*

Friction angle: $\delta := 0deg$ *Conservative per USACE EM 1110-2-2502*

Angle of face of wall to horizontal: $\theta := 90deg$ *Vertical wall.*

Angle of fill to horizontal: $\beta := 0deg$ *Level fill.*

Gamma: $\Gamma := \left(1 + \sqrt{\frac{\sin(\phi_f + \delta)\sin(\phi_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right)^2 = 2.295$ *Part of Columb's Equation.*

Active lateral earth pressure ratio: $K_a := \frac{\sin(\theta + \phi_f)^2}{\Gamma \cdot \sin(\theta)^2 \sin(\theta - \delta)} = 0.32$ *Coulomb's Equation; USACE EM 1110-2-2502, pg 3-17*

Ultimate bearing pressure: $P_b := 15900psf$ *See geotechnical bearing pressure calculation.*

2.3.2 Reinforced Concrete Parameters

Unit weight of concrete:	$\gamma_c := 150pcf$	*Assumed*	
Concrete compressive strength:	$f_c := 4ksi$	*Drawing 10W258-1 R1*	
Assumed max. concrete compressive strain:	$\varepsilon_c := 0.003$	*ACI 350-06, (11-3)*	
Steel yield stress:	$f_y := 60ksi$	*Drawing 10W258-1 R1*	
Steel modulus of elasticity:	$E_s := 29000ksi$		
Load factor - water:	$LF_f := 1.4$	Static fluids	*ACI 350-06, (Eq'n 9-1)*
Load factor - soil:	$LF_h := 1.6$	Soils and water saturated soils	*ACI 350-06, (Eq'n 9-2)*
#6 Bar diameter:	$d_{b6} := 0.75 \cdot in$		
#9 Bar diameter:	$d_{b9} := 1.128 \cdot in$		
Clear cover for formed and finished slab surfaces:	$Clr := 2in$	*Drawing 10W258-1 R1*	

Area of steel is computed for 1foot wide strip of concrete.

Area of vertical steel in weir:	$A_1 := 0.44 \dfrac{in^2}{ft}$	*#6 @ 12" EF*
Area of vertical steel in wing wall and side wall below CJ:	$A_2 := 2.00 \dfrac{in^2}{ft}$	*#9 @ 6" FF (flexural) - Below C.J.*
Area of vertical steel in wing wall and side wall above CJ:	$A_3 := 0.44 \dfrac{in^2}{ft}$	*#6 @ 12" EF - Above C.J.*

3 Force Equilibrium

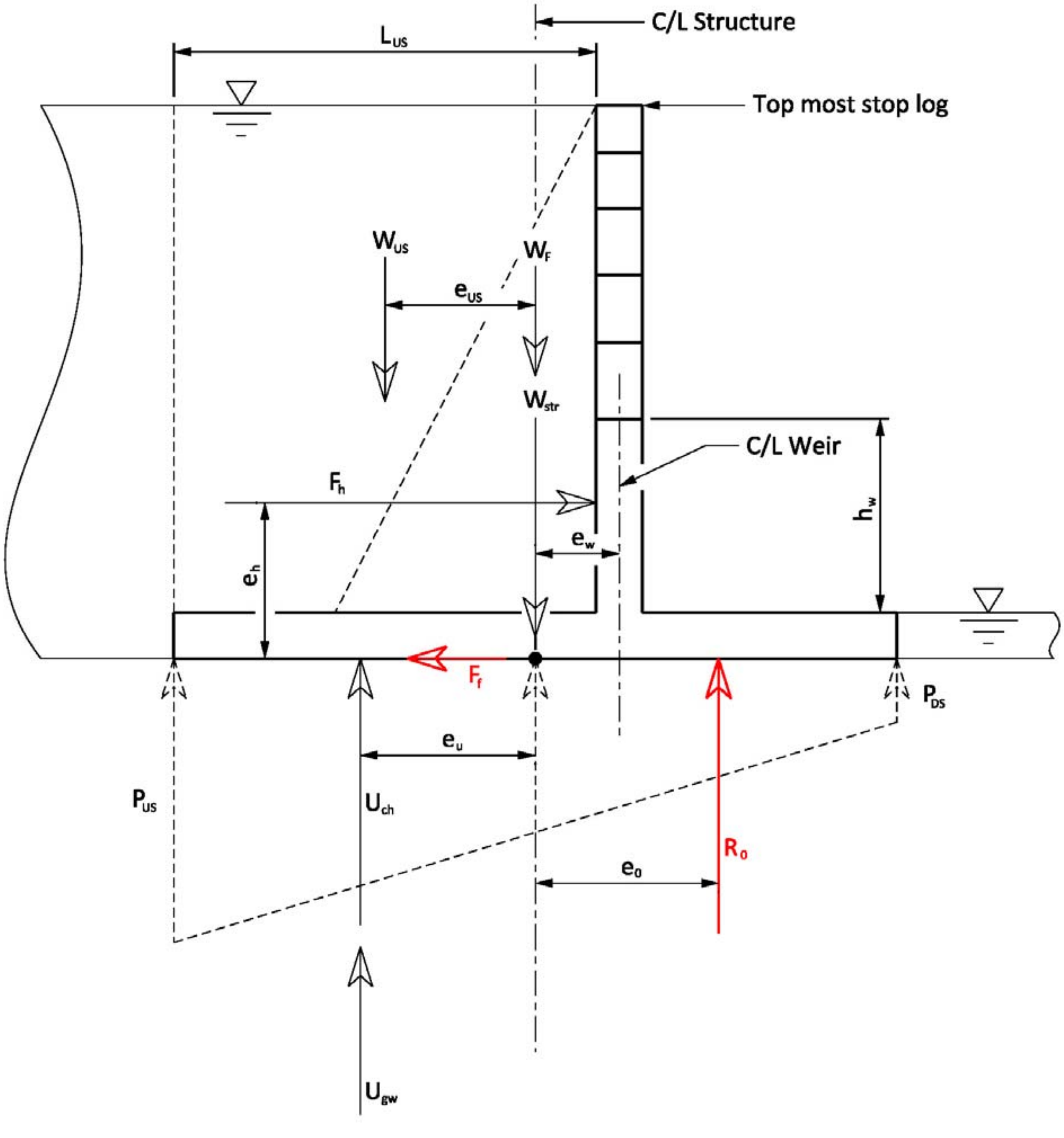

Figure 3: Force Equilibrium

3.1 Weight of Structure

Assess weir without wing walls for stability - Assume wing walls act like cutoff walls providing passive pressure resistance.

Area of base (channel portion): $A_{bch} := L_{ch} \cdot (B_{ch} + 2 \cdot t_{sw}) = 744\ ft^2$

Weight of base (channel portion): $W_{bch} := (t_b \cdot A_{bch}) \cdot \gamma_c = 195\ kip$

Weight of weir: $W_w := [(2 \cdot B_w) \cdot h_w \cdot t_w] \cdot \gamma_c = 22.97\ kip$

Weight of center wall stem: $W_{cw} := (L_{cw} \cdot h_{cw} \cdot t_{cw}) \cdot \gamma_c = 33.4\ kip$

Weight of sidewall and wing retainment over channel portion: $W_{ret} := \gamma_c \cdot t_{sw} \cdot h_{sw} \cdot L_{ch} = 104.9\ kip$

Weight of stop log beam: $W_{sl} := (1ft)(2ft) \cdot (B_w - 4in) \cdot \gamma_c = 4.1\ kip$

See TVA Plan sheet 10W258-2 for stop log beam dimensions.

Total weight of structure: $W_{chstr} := W_{bch} + W_w + 2 \cdot W_{ret} + W_{cw} + 5 \cdot 2 \cdot W_{sl} = 502\ kip$

Moment about CL of base: (resisting is positive)

weir *center wall* *stop logs*

$M_{chstr} := -e_w \cdot W_w + e_{cw} \cdot W_{cw} - e_w \cdot (5 \cdot 2 \cdot W_{sl}) = -52.8\ ft \cdot kip$

3.2 Weight of Water Over Channel

Weight of upstream water: $W_{US1} := (EL_{US} - EL_{bb}) \cdot (2 \cdot B_w) \cdot L_{US} \cdot \gamma_w = 392.1\ kip$

Eccentricity of weight of upstream water over channel: $e_{US1} := \frac{L_{US}}{2} - e_w + \frac{t_w}{2} = 5.06\ ft$

Weight of upstream water in front of center wall: $W_{US2} := (EL_{US} - EL_{bb}) \cdot (t_{cw}) \cdot \left(\frac{L_{ch}}{2} - e_{cw} - \frac{L_{cw}}{2}\right) \cdot \gamma_w = 3.9\ kip$

Eccentricity of water weight in front of center pier:

$$e_{US2} := \frac{L_{ch}}{2} - \frac{\frac{L_{ch}}{2} - e_{cw} - \frac{L_{cw}}{2}}{2} = 9.81\ ft$$

Weight of upstream water over channel: $W_{US} := W_{US1} + W_{US2} = 396.0\ kip$

Moment about CL of base:
(resisting is positive)

$M_{USWv} := W_{US1} \cdot e_{US1} + W_{US2} \cdot e_{US2} = 2023.1\ ft \cdot kip$

Weight of downstream water:

$W_{DS1} := (EL_{DS} - EL_{bb}) \cdot (2 \cdot B_w) \cdot (L_{ch} - L_{US} - t_w) \cdot \gamma_w = 113.2\ kip$

Eccentricity of weight of
upstream water over channel:

$$e_{DS1} := \frac{(L_{ch} - L_{US} - t_w)}{2} - \frac{L_{ch}}{2} = -7.06\ ft$$

Weight of upstream water in front
of center wall:

$W_{DS2} := (EL_{DS} - EL_{bb}) \cdot (t_{cw}) \cdot (L_{ch} - L_{US} - 2 \cdot t_w) \cdot \gamma_w = 3.6\ kip$

Eccentricity of water weight in front
of center pier:

$$e_{DS2} := \frac{(L_{ch} - L_{US} - 2 \cdot t_w)}{2} - \frac{L_{ch}}{2} = -7.56\ ft$$

Weight of upstream water over
channel:

$W_{DS} := W_{DS1} + W_{DS2} = 116.8\ kip$

Moment about CL of base:
(resisting is positive)

$M_{DSWv} := W_{DS1} \cdot e_{DS1} + W_{DS2} \cdot e_{DS2} = -826.7\ ft \cdot kip$

3.3 Uplift Under Channel

Upstream depth:

$h_{US} := EL_{US} - EL_{bb} = 17.10\ ft$

Upstream pressure:

$P_{US} := h_{US} \cdot \gamma_w = 1067.0 \cdot psf$

Downstream depth:

$h_{DS} := EL_{DS} - EL_{bb} = 7.1\ ft$

Downstream pressure:

$P_{DS} := h_{DS} \cdot \gamma_w = 443.0 \cdot psf$

Uplift force:

channel width

$$U_{ch} := \frac{(P_{US} + P_{DS})}{2} \cdot L_{ch} \cdot (B_{ch} + 2 \cdot t_{sw}) = 562\ kip$$

Eccentricity of channel uplift force:

$$e_u := \frac{L_{ch}}{2} - \left[\frac{L_{ch} \cdot (2h_{DS} + h_{US})}{3 \cdot (h_{DS} + h_{US})}\right] = 1.60\ ft$$

Moment about CL of base:
(resisting is positive)

$M_U := -U_{ch} \cdot e_u = -899.5\ ft \cdot kip$

3.4 Water Lateral Force

Upstream hydrostatic pressure: $P_{hUS} := h_{US} \cdot \gamma_w = 1067 \cdot psf$

channel width

Upstream lateral hydrostatic force: $F_{hUS} := \frac{1}{2} \cdot P_{hUS} \cdot h_{US} \cdot (B_{ch} + 2 \cdot t_{sw}) = 291.9\ kip$

Eccentricity of upstream hydrostatic force: $e_{hUS} := \frac{1}{3} \cdot (h_{US}) + t_b = 7.45\ ft$

Downstream hydrostatic pressure: $P_{hDS} := h_{DS} \cdot \gamma_w = 443 \cdot psf$

channel width

Downstream lateral hydrostatic force: $F_{hDS} := \frac{1}{2} \cdot P_{hDS} \cdot h_{DS} \cdot (B_{ch} + 2 \cdot t_{sw}) = 50.3\ kip$

Eccentricity of downstream hydrostatic force: $e_{hDS} := \frac{1}{3} \cdot (h_{DS}) + t_b = 4.12\ ft$

Moment about CL of base: (resisting is positive) $M_{Wh} := -F_{hUS} \cdot e_{hUS} + F_{hDS} \cdot e_{hDS} = -1967.8\ ft \cdot kip$

3.5 Moment Equilibrium (Overturning)

Vertical loads resultant: $R_{bch} := W_{chstr} + W_{US} + W_{DS} - U_{ch} = 453.478\ kip$

Eccentricity of resultant load: $e_0 := \frac{M_{chstr} + M_{USWv} + M_{DSWv} + M_U + M_{Wh}}{R_{bch}} = -3.801 \cdot ft$

Usual Load Combination: $U_{ME} := \begin{cases} \text{"Sufficient"} & \text{if } |e_0| \le \frac{L_{ch}}{6} \\ \text{"Insufficient"} & \text{otherwise} \end{cases} = \text{"Sufficient"} \quad \frac{L_{ch}}{6} = 3.875\ ft$

This ensures that 100% of the base is in compression. The limit of Base/6 corresponds to the distance of the resultant to the center of the base.

3.6 Sliding Assessment at Base

Evaluate channel section assuming wing walls act as passive restraint, (i.e., horizontal loads only), with no cohesion at base, c = 0 psf

Coefficient of friction between soil and concrete base: $\mu := \tan(\phi_f) = 0.601$

Friction Force: $F_f := \mu \cdot R_{bch} = 272\ \text{kip}$

Sliding factor of safety: $FS_s := \dfrac{F_f}{F_{hUS} - F_{hDS}} = 1.128$ Friction only

Usual load combination: $U_s := \left| \begin{array}{ll} \text{"Satisfactory"} & \text{if } FS_s \geq 1.5 \\ \text{"Unsatisfactory"} & \text{otherwise} \end{array} \right. = \text{"Unsatisfactory"}$ FS with friction only

Assessment - Passive resistance of upstream wing walls is needed for sliding stability; the downstream wing walls do not provide any passive resistance. Assume passive pressure is exerted on basis of linear variation from maximum height of upstream wing wall at side wall connection to end of upstream wing wall. Also assume that the soil behind the upstream wing wall is saturated and that hydrostatic forces on the upstream and downstream of the wing wall are balanced due to headwater saturation. All of these assumptions are conservative.

Passive pressure coefficient $K_p := \dfrac{1}{K_a} = 3.12$

Saturated passive soil pressure at base of wing wall: $P_{wwb} := K_p \cdot (\gamma_{sat} - \gamma_w) \cdot (h_{sw}) = 3858.4 \cdot \text{psf}$

Passive lateral soil force at base of wing wall: $F_{wwb} := P_{wwb} \cdot \dfrac{h_{sw}}{2} = 38.68\ \text{klf}$

Saturated passive soil pressure at end of wing wall: $P_{wwe} := K_p \cdot (\gamma_{sat} - \gamma_w) \cdot (h_{ww3}) = 1016.1 \cdot \text{psf}$

Passive lateral soil force at end of wing wall: $F_{wwe} := P_{wwb} \cdot \dfrac{h_{ww3}}{2} = 10.19\ \text{klf}$

Length of wing wall: $L_{ww} := L_{ww1} + L_{ww2} = 27.5\ \text{ft}$

Projected length of wing wall parallel to dam axis: $L_{wwp} := \sin\left(45 \cdot \dfrac{\pi}{180}\right) \cdot L_{ww} = 19.45\ \text{ft}$

Passive soil resistance force $F_{sp} := 2 \cdot 0.5 \cdot L_{wwp} \cdot \left(\dfrac{F_{wwb} + F_{wwe}}{2}\right) = 475.1\ \text{kip}$ Reduce passive load by 50% according to EM 1110-2-2502

Resisting force fpr sliding at base: $F_{resist} := F_f + F_{sp} = 747.6\ \text{kip}$

Sliding factor of safety: $FS_s := \dfrac{F_{resist}}{F_{hUS} - F_{hDS}} = 3.09$

Usual load combination: $U_s := \left| \begin{array}{ll} \text{"Satisfactory"} & \text{if } FS_s \geq 2.0 \\ \text{"Unsatisfactory"} & \text{otherwise} \end{array} \right. = \text{"Satisfactory"}$

FS using friction and passive resistance

3.7 Bearing Pressure Assessment at Base

Base section modulus: $S_{bch} := \dfrac{B_{ch} \cdot L_{ch}^{\ 2}}{6} = 2613\ \text{ft}^3$

Moment over base: $M_{bch} := R_{bch} \cdot \left| e_0 \right| = 1723.6\ \text{ft} \cdot \text{kip}$

Maximum Pressure: $P_{max} := \dfrac{R_{bch}}{A_{bch}} + \dfrac{M_{bch}}{S_{bch}} = 1269 \cdot \text{psf}$

Factor of safety: $FS_b := \dfrac{P_b}{P_{max}} = 12.5$

Usual load combination: $U_b := \left| \begin{array}{ll} \text{"Satisfactory"} & \text{if } FS_b \geq 3.0 \\ \text{"Unsatisfactory"} & \text{otherwise} \end{array} \right. = \text{"Satisfactory"}$

4 Strength Assessment of Side Wall Stem and Wing Walls

Conservatively assume that the side wall stem acts as a cantilever wall when actually it is supported on three edges. Determine the applied loads at the base of stem just downstream of weir as this is the most extreme stress condition and at the horizontal construction joint. For wing walls, the stem thickness and reinforcement are the same as the side wall stem except that the wing wall stem height decreases as the footing rises and the top of wall falls. Thus, the strength of the wing walls is acceptable if the strength of the side wall stem is acceptable.

4.1 External Loads on 1 foot Wide Vertical Strip of Side Wall Stem

4.1.1 Water Behind Wall

Assuming homogeneous fill and linear variation of seepage upstream to downstream.

Groundwater elevation behind wall at weir: $EL_{GW} := EL_{US} - \left(EL_{US} - EL_{DS}\right)\cdot\frac{\left(L_{US} + t_w\right)}{L_{ch}} = 225.92\ ft$

Groundwater height behind wall at weir: $h_{GW} := EL_{GW} - EL_{bt} = 9.275\ ft$

4.1.2 Soil Pressures

Moist soil pressure at CJ: $P_{CJ} := K_a\cdot\gamma_d\cdot\left(EL_{swt} - EL_{CJ}\right) = 309.2\cdot psf$

Soil pressure at water elevation: $P_1 := K_a\cdot\gamma_d\cdot\left(h_{sw} - h_{GW}\right) = 362.2\cdot psf$

Soil Load above water elevation: $V_{moist} := \frac{1}{2}\cdot P_1\cdot\left(h_{sw} - h_{GW}\right)\cdot(1\cdot ft) = 1.95\ kip$

Incremental saturated soil pressure at base of side wall: $P_2 := K_a\cdot\left(\gamma_{sat} - \gamma_w\right)\cdot h_{GW} = 182.9\cdot psf$

Incremental saturated soil load above base of side wall: $V_{sat} := \frac{1}{2}\cdot P_2\cdot h_{GW}\cdot(1\cdot ft) = 0.85\ kip$

Saturated water pressure $P_3 := \gamma_w\cdot\left(h_{GW}\right) = 578.7\cdot psf$

Water load behind wall: $V_w := \frac{1}{2}\cdot P_3\cdot h_{GW}\cdot(1\cdot ft) = 2.68\ kip$

4.1.3 Lateral Hydrostatic Pressure on Channel Side of Wall from Downstream Water

Lateral force from downstream channel water: $V_{hDS} := \frac{1}{2}P_{hDS}\cdot h_{DS}\cdot(1ft) = 1.573\ kip$

4.1.4 Maximum Factored Force Effects at CJ

Maximum shear at CJ: $V_{CJ} := LF_h \cdot \frac{1}{2} \cdot P_{CJ} \cdot \left(EL_{swt} - EL_{CJ}\right) \cdot (1 \cdot ft) = 2.28\,kip$

Maximum moment at CJ: $M_{CJ} := V_{CJ} \cdot \frac{1}{3} \cdot \left(EL_{swt} - EL_{CJ}\right) = 6.98\,ft \cdot kip$

4.1.5 Maximum Factored Force Effects at Base of Side Wall

Maximum shear at base: $V_{walls} := LF_h \cdot \left[V_{moist} + P_1 \cdot h_{GW} \cdot (1 \cdot ft) + V_{sat} + V_w\right] - V_{hDS} = 12.57\,kip$

Maximum moment at base:

$$M_{walls} := LF_h \cdot \left[V_{moist} \cdot \left(\frac{h_{sw} - h_{GW}}{3} + h_{GW}\right) + P_1 \cdot \frac{h_{GW}^{\,2}}{2} \cdot (1 \cdot ft) + \left(V_{sat} + V_w\right) \cdot \frac{h_{GW}}{3}\right] - V_{hDS} \cdot \frac{\left(EL_{DS} - EL_{bt}\right)}{3} = 79.8\,f$$

4.2 Shear Capacity of Side Wall Stem

Depth to Tension Steel: $d := t_{sw} - \left(Clr + \frac{d_{b9}}{2}\right) = 15.436 \cdot in$

Reduction Factor: $\phi_v := 0.75$ *ACI 350-06, 9.3.2.3*

Available Concrete Shear Strength: $\phi V_n := \phi_v \cdot d \cdot (1 \cdot ft) \cdot 2 \cdot \sqrt{f_c \cdot \frac{1000}{ksi}} \cdot psi = 17.573\,kip$ *ACI 350-06, (11-3)*

Shear Performance Ratio at base: $FS_v := \frac{\phi V_n}{V_{walls}} = 1.40$ > 1 **OK**

4.3 Flexural Capacity of Side Wall at CJ

Concrete stress block: $a := \frac{\left(A_3 \cdot 1ft\right) f_y}{0.85 \cdot f_c \cdot (1ft)} = 0.647 \cdot in$

Concrete stress block reduction factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \left[0.85 - 0.05\left(\frac{f_c - 4000psi}{1000psi}\right)\right] & \text{otherwise} \end{cases} = 0.85$$

ACI 350-06, R10.2.7

Concrete Compression Zone: $c := \frac{a}{\beta_1} = 0.761 \cdot in$

Strain in steel: $\varepsilon_s := \varepsilon_c \cdot \left(\frac{d - c}{c}\right) = 0.058$

Yield strain: $\varepsilon_y := \frac{f_y}{E_s} = 0.0021$

Steel Yielding?: $yield := \begin{cases} \text{"TRUE"} & \text{if } \varepsilon_s \geq \varepsilon_y \\ \text{"FALSE"} & \text{otherwise} \end{cases} = \text{"TRUE"}$

Reduction Factor: $\phi_{ff} := \begin{cases} 0.9 & \text{if } \varepsilon_s \geq 0.005 \\ \left[0.65 + (\varepsilon_s - \varepsilon_y) \cdot \frac{0.25}{0.005 - \varepsilon_y}\right] & \text{otherwise} \end{cases} = 0.9$

ACI 350-06, 9.3.2.1, 9.3.2.2

Available Moment Capacity: $\phi M_n := \phi_{ff} \cdot (A_3 \cdot 1ft) \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 29.9\, ft \cdot kip$

Moment Performance Ratio: $FS_f := \frac{\phi M_n}{M_{CJ}} = 4.287$ **> 1 OK**

4.4 Flexural Capacity of Side Wall at Base

Concrete stress block: $a := \frac{(A_2 \cdot 1ft) f_y}{0.85 \cdot f_c \cdot (1ft)} = 2.941 \cdot in$

Concrete stress block reduction factor: $\beta_1 := \begin{cases} 0.85 & \text{if } f_c \leq 4000psi \\ \left[0.85 - 0.05\left(\frac{f_c - 4000psi}{1000psi}\right)\right] & \text{otherwise} \end{cases} = 0.85$

ACI 350-06, R10.2.7

Concrete Compression Zone: $c := \frac{a}{\beta_1} = 3.46 \cdot in$

Strain in steel: $\varepsilon_s := \varepsilon_c \cdot \left(\frac{d - c}{c}\right) = 0.01$

Yield strain: $\varepsilon_y := \frac{f_y}{E_s} = 0.0021$

Steel Yielding?: $yield := \begin{cases} \text{"TRUE"} & \text{if } \varepsilon_s \geq \varepsilon_y \\ \text{"FALSE"} & \text{otherwise} \end{cases} = \text{"TRUE"}$

Reduction Factor: $\phi_{ff} := \begin{cases} 0.9 & \text{if } \varepsilon_s \geq 0.005 \\ \left[0.65 + \left(\varepsilon_s - \varepsilon_y\right) \cdot \frac{0.25}{0.005 - \varepsilon_y}\right] & \text{otherwise} \end{cases} = 0.9$

ACI 350-06, 9.3.2.1, 9.3.2.2

Available Moment Capacity: $\phi M_n := \phi_{ff} \cdot \left(A_2 \cdot 1ft\right) \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 125.7\ ft \cdot kip$

Moment Performance Ratio: $FS_f := \frac{\phi M_n}{M_{walls}} = 1.576$ **> 1 OK**

5 Strength Assessment of Weir Stem

Conservatively assume that weir stem acts as a cantilever wall when actually it is supported on three edges. Determine the applied loads at the base of weir stem in middle of bay - most extreme stress condition and without consideration of tailwater.

5.1 External Loads on 1 foot Wide Vertical Strip of Weir Stem

5.1.1 Water Depths at Upstream Pool

Depth to top of weir: $z_1 := EL_{US} - EL_w = 9.88\,ft$

Depth to bottom of weir: $z_2 := EL_{US} - EL_{bt} = 15.35\,ft$

5.1.2 Water Pressure

Pressure at top of weir: $P_{w1} := \gamma_w \cdot z_1 = 617 \cdot psf$

Pressure at bottom of weir: $P_{w2} := \gamma_w \cdot z_2 = 958 \cdot psf$

5.1.3 Factored Line Loads

Factored line load at top: $w_{top} := LF_f \cdot P_{w1} \cdot 1 \cdot ft = 0.863\,klf$

Factored line load at bottom: $w_{bott} := LF_f \cdot P_{w2} \cdot 1ft = 1.341\,klf$

5.1.4 Maximum Factored Force Effects at Base of Side Wall

Maximum shear at base: $V_{we} := \left(w_{bott} - w_{top}\right) \cdot \dfrac{h_w}{2} + w_{top} \cdot h_w = 6.03\,kip$

Maximum moment at base: $M_{we} := \left(w_{bott} - w_{top}\right) \cdot \dfrac{{h_w}^2}{6} + w_{top} \cdot \dfrac{{h_w}^2}{2} = 15.30\,ft \cdot kip$

5.2 Shear Capacity of Weir

Depth to Tension Steel: $d := t_w - \left(Clr + \dfrac{d_{b6}}{2}\right) = 9.625 \cdot in$

Reduction Factor: $\phi_{vv} := 0.75$ *ACI 350-06, 9.3.2.3*

Total Shear Strength at base: $\phi V_n := \phi_{vv} \cdot d \cdot (1 \cdot ft) \cdot 2 \cdot \sqrt{f_c \cdot \dfrac{1000}{ksi}} \cdot psi = 10.96\,kip$ *ACI 350-06, (11-3)*

Shear Performance Ratio at base: $FS_v := \frac{\phi V_n}{V_{we}} = 1.818$ > 1 **OK**

5.3 Flexural Capacity of Weir at Base

Concrete stress block: $a := \frac{(A_1 \cdot 1ft) f_y}{0.85 \cdot f_c \cdot (1ft)} = 0.647 \cdot in$

Concrete stress block reduction factor:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \left[0.85 - 0.05\left(\frac{f_c - 4000psi}{1000psi}\right)\right] & \text{otherwise} \end{cases} = 0.85$$

ACI 350-06, R10.2.7

Concrete Compression Zone: $c := \frac{a}{\beta_1} = 0.761 \cdot in$

Strain in steel: $\varepsilon_s := \varepsilon_c \cdot \left(\frac{d - c}{c}\right) = 0.035$

Yield strain: $\varepsilon_y := \frac{f_y}{E_s} = 0.0021$

Steel Yielding?:

$$yield := \begin{cases} \text{"TRUE"} & \text{if } \varepsilon_s \ge \varepsilon_y \\ \text{"FALSE"} & \text{otherwise} \end{cases} = \text{"TRUE"}$$

Reduction Factor:

$$\phi_{ff} := \begin{cases} 0.9 & \text{if } \varepsilon_s \ge 0.005 \\ \left[0.65 + (\varepsilon_s - \varepsilon_y) \cdot \frac{0.25}{0.005 - \varepsilon_y}\right] & \text{otherwise} \end{cases} = 0.9$$

ACI 350-06, 9.3.2.1, 9.3.2.2

Available Moment Capacity: $\phi M_n := \phi_{ff} \cdot \left[(A_1 \cdot 1ft) \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right] = 18.417 \, ft \cdot kip$

Moment Performance Ratio: $FS_f := \frac{\phi M_n}{M_{we}} = 1.204$ > 1 **OK**

6 References

TVA-CCR Rule. 2015. TVA.

ACI 350-06: *Environmental Structures Code/Commentary*. 2006. American Concrete Institute.

EM 1110-2-2502. *Retaining and Flood Walls*. 1989. US Army Corps of Engineers.

EM 1110-2-2100. *Stability Analysis of Concrete Structures*. Dec 2005 US Army Corps of Engineers.

Stantec

BEARING CAPACITY CALCULATION
FOR EAST STILLING POND SKIMMER FOUNDATIONS

N_{80} VALUES IN THE FOUNDATION SANDY SILT:

NEAREST BORING – STN-10 – N_{80}-VALUES → 4, 1, WOH, WOH, WOH, 6, 11
STN-9 – 9, 6, 2, 2, WOR, 9, 2
STN-8 11, 4, 2, 2, 1, 3, 7

AVERAGE N_{80} VALUE @ STN-8, 9 & 10 (WOH WAS IGNORED)
= 4

EQUIVALENT N_{60} VALUE = 5

ESTIMATED UNDRAINED STRENGTH (PECK 1974) S_u = 600 psf –

COHESION $c = S_u$ = 600 psf

Square footing: 6.5 ft × 6.5 ft

ULTIMATE BEARING CAPACITY,

$$q_{ult} = \frac{1}{2}\gamma B N_\gamma + cN_c + (P_q + \gamma D_f) N_q$$

FOR CLAY IN UNDRAINED CONDITION, $\phi = 0$

USING MEYERHOF'S TABLE FOR $\phi = 0$, $N_c = 5.14$, $N_q = 1$ & $N_\gamma = 0.0$

$$q_{ult} = cN_c + P_q + \gamma D_f$$

NET BEARING CAPACITY,

$q_{net} = q_{ult}$ – OVERBURDEN
$= cN_c$
$= 600 \times 5.14 \times 1.25$ (1.25 - CORR. FACTOR FOR SQ FTG.)
= 3,855 psf

ALLOWABLE BEARING CAPACITY,

$q_a = q_{net} / FS$
$= 3855 / 3$
= 1,285 psf

Designed by: S. RAHMAN
Checked by: S. RAHMAN

Stantec

Current Load Conditions for East Stilling Pond Skimmer Foundations

Current Dead Loads

(1) current footing (1.5' x 6.5' x 6.5') assuming 150 pcf concrete = $\boxed{9{,}506 \text{ lbf}}$
Top of footing elevation at 216.0'

(2) 6 foot OD connector concrete pipe from footing to risers, 4 ft in height w/ 4 foot ID (from ele. 216.0' to 220.0')

$\Rightarrow [\pi(3^2) - \pi(2^2)] \times 4' \times 150 \text{ pcf} \Rightarrow 9{,}425 \text{ lbf}$

$\Rightarrow$ Remove volume for 36" horizontal spillway pipe (1.5' in thickness)

$[\pi(1.5^2) \cdot 1.5] \cdot 150 \text{ pcf} = 1{,}590 \text{ lbf}$

$\Rightarrow$ Total Load for concrete connector pipe: $9{,}425 - 1590 = \boxed{7{,}835 \text{ lbf}}$

(3) Concrete 60 inch OD (48 inch ID) Riser Sections connecting concrete connector pipe to Skimmer Frame According to TVA Drawing 10W507-04 elevation 220.0 to 226.0 (6 feet in height)

$[\pi(2.5^2) - \pi(2^2)] \times 6.0' \times 150 \text{ pcf} = \boxed{6{,}362 \text{ lbf}}$

(4) Skimmer Structure and Frame: According TVA CCR Calculation Package, each skimmer and frame weighs = $\boxed{1{,}800 \text{ lbf}}$

Current Pressure on Footing

(1) Total Load = $9{,}506 + 7{,}835 + 6{,}362 + 1{,}800 = 25{,}503 \text{ lbf}$

$B = L = 6.5$, footing Area = 42.25 ft^2

$\Rightarrow$ Actual Pressure $(p_a) = \text{Load}/\text{Area} = 25{,}503 \text{ lbf} / 42.25 \text{ ft}^2 = \boxed{604 \text{ psf} = p_a}$

Designed by: M. McDONALD Checked by: S. RAHMAN

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TVA ALF Bearing Capacity for Spillway Foundation

From SPT Correction Spreadsheet, Average N_{60} value of Embankment Shell using STN-18 = 20

Estimated undrained strength (Peck 1974) S_u = 2667

Cohesion $c = S_u$ = 2,667 psf

Ultimate Bearing Capacity, (Meyerhof, 1963) Ref: Civil Engineering Reference Manual for the PE Exam 9th

Eq 36.1 (B) $q_{ult} = \frac{1}{2}\gamma B N_\gamma + cN_c + (p_q + \gamma D_f) N_q$

For Clay in undrained condition, $\phi = 0$

From table 36.3 Bearing Capacity factors Meyerhof, Vesic

for $\phi = 0$, $N_c = 5.14$

$N_q = 1.0$

$N_\gamma = 0.0$

Therefor $q_{ult} = cN_c + p_q + \gamma D_f$

Net Bearing Capacity

$q_{net} = q_{ult}$ − Overburden

$= cN_c$

$= 2{,}667 \times 5.14 \times 1.16$

$= 15{,}902$ psf

(Debeer, 1970)

N_c multiplier for shape correction

$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$

$= 1 + \frac{15}{18}\left(\frac{1}{5.14}\right)$

≈ 1.16

Allowable Bearing Capacity

$q_{all} = q_{net} / FS$

$= 15{,}902 / 3$

$= 5{,}300$ psf

Designed by: T. GUNTER

Checked by: S. RAHMAN

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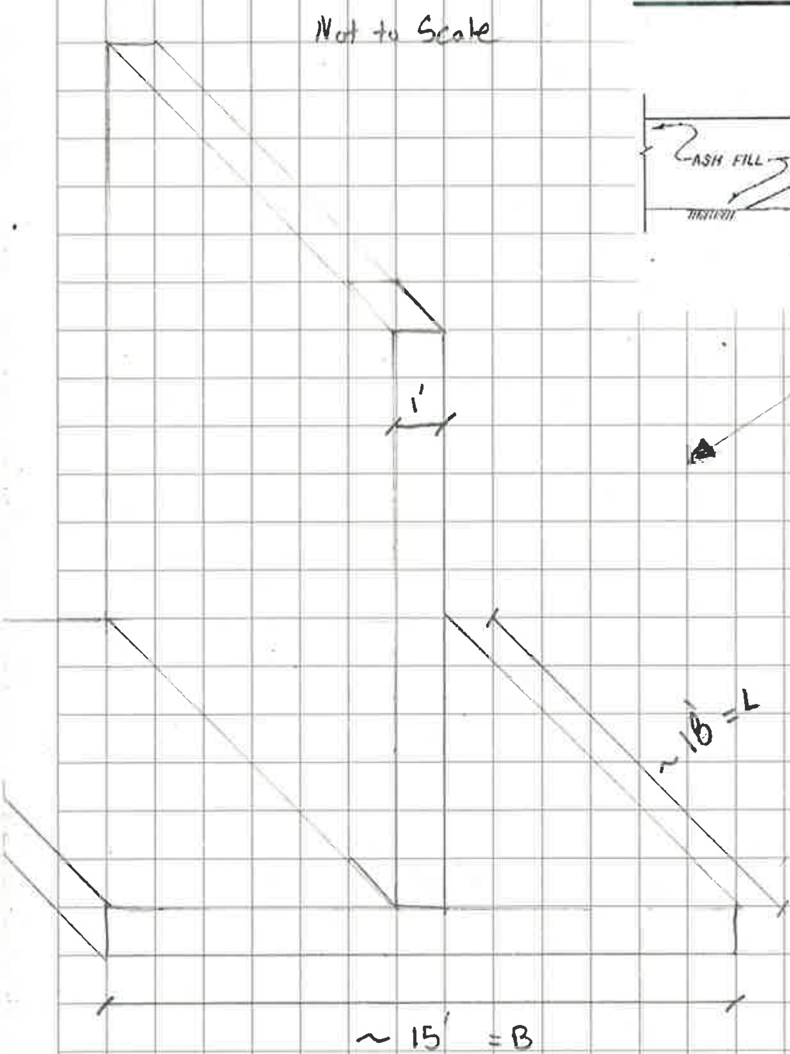
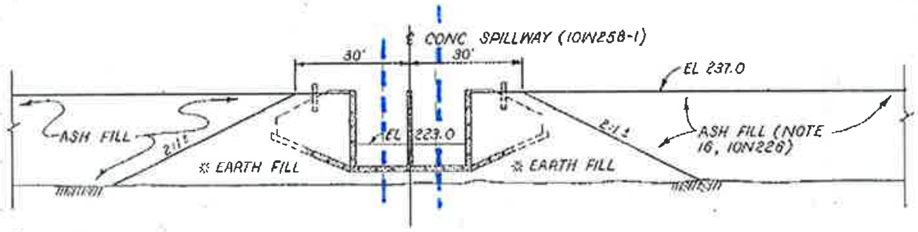




AA-AA

To simplify analysis, the centerwall of the spillway was chosen and analyzed to obtain the dimensions of B and L

Designed by: T. GUNTER Checked by: S. RAHMAN

Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, tan δ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
○ dressed soft rock on dressed soft rock	35	0.70
○ dressed hard rock on dressed soft rock	33	0.65
○ dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f.

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

Static Slope Stability Analysis

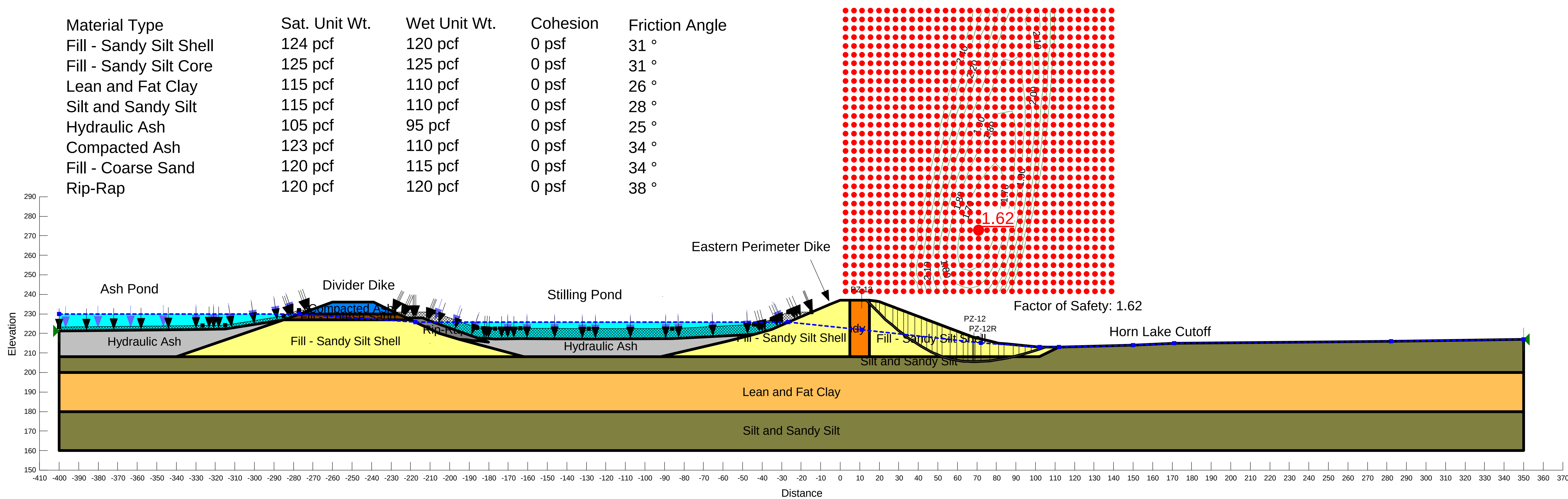
Tennessee Valley Authority
Allen Fossil Plant East Ash Disposal Area
Memphis, Tennessee
Section E-E'

Existing Geometry;
Maximum Surcharge Pool Loading;
Effective Stress Analysis;
Drained Strengths

Global Failure

Note: The results of the analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. The drawing depicts approximate subsurface conditions based on historical drawings or specific borings at the time of drilling. No warranties can be made regarding the continuity of subsurface conditions.

Material Type	Sat. Unit Wt.	Wet Unit Wt.	Cohesion	Friction Angle
Fill - Sandy Silt Shell	124 pcf	120 pcf	0 psf	31 °
Fill - Sandy Silt Core	125 pcf	125 pcf	0 psf	31 °
Lean and Fat Clay	115 pcf	110 pcf	0 psf	26 °
Silt and Sandy Silt	115 pcf	110 pcf	0 psf	28 °
Hydraulic Ash	105 pcf	95 pcf	0 psf	25 °
Compacted Ash	123 pcf	110 pcf	0 psf	34 °
Fill - Coarse Sand	120 pcf	115 pcf	0 psf	34 °
Rip-Rap	120 pcf	120 pcf	0 psf	38 °



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APPENDIX B
HYDRAULIC STRUCTURES CONDITION

TENNESSEE VALLEY AUTHORITY (TVA)
ALLEN FOSSIL PLANT – EAST ASH DISPOSAL AREA
PIPE INSPECTION REPORT

Site: Allen Fossil Plant, Shelby County, Tennessee

Inspection Date(s): January 26 – January 28, 2016 **Weather** Varied. Temperature in low 30's to mid 40's

1. Company and Inspection Team Information

Stantec Consulting Services Inc. (Stantec) led the closed circuit television (CCTV) inspection and condition assessment of two sanitary sewer pipes at the Allen Fossil Plant. The inspection team also included Hydromax USA, LLC and Glenn Underwater Services, Inc.

Stantec Team

The following team members scouted the site, reviewed historic documents, interviewed representatives from the City of Memphis, assessed field conditions, conducted the inspection, oversaw field explorations, and reviewed the field data and inspection videos. This team has provided detailed observations and findings based upon those activities:

Jason C. Maxwell, PE, Project Lead and Technical Review

Over 10 years of experience in stormwater and wastewater design, planning, and infrastructure condition assessment and remediation. His experience includes hydraulic infrastructure evaluation through dams, levees, and power facilities. Mr. Maxwell managed the project team performing the field and office work and provided technical review of collected data.

Stephen L. Bickel, PE, Quality Control/Quality Assurance

Over 35 years of experience in the engineering design, investigation and evaluation of dams, roadways, landfills, and other large civil engineering projects, specifically projects that include coal combustion product (CCP) management and infrastructure assessments. Mr. Bickel provided quality control/quality assurance of inspection observations and findings.

Robert L. Bernard, PE, Site Scouting

Over 17 years of experience in water, wastewater and stormwater design, planning, assessment, and management. He performed the site scouting and overall work plan development to facilitate the field inspection effort.

Bradley T. Allgeier, PE, Field Oversight

Over 5 years of experience in stormwater and wastewater design, planning, and condition assessments. He provided on-site engineering oversight during field inspection activities and technical review of collected data.

Hydromax USA, LLC.

Hydromax USA, LLC (Hydromax) performed the closed circuit television (CCTV) inspection of the unsubmerged portions of the 60-inch diameter reinforced concrete pipe (RCP) interior. Hydromax provided inspection videos, thumbnail images, and prepared pipe inspection logs for review by Stantec.

Glenn Underwater Services, Inc.

Glenn Underwater Services, Inc. (Glenn Diving) was responsible for providing personnel and equipment necessary to perform the remote operated vehicle (ROV) CCTV inspection for the submerged 60-inch diameter RCP interior.

2. Engineering Documents, Interviews and Data Reviewed

Listed below is a summary of relevant documents (record drawings, inspection reports, etc.) reviewed and interviews performed prior to the field inspections and the completion of this report.

Previous CCTV Inspections:

- A CCTV Inspection was reportedly performed on the 42-inch diameter RCP by the City of Memphis in the 1990's, but was not provided for review.

Drawings:

- Mississippi River, Memphis Harbor Project, Nonconnah Basin Sewer Extension Plan and Profile, 60" Diameter Sewer, STA. 0+00 to STA. 160+00; USACE, Memphis District, September 1951.
- Mississippi River, Memphis Harbor Project, Nonconnah Basin Sewer Extension Plan and Profile, 60" Diameter Sewer, STA. 160+00 to STA. 283+00; USACE, Memphis District, September 1951.
- Mississippi River, Memphis Harbor Project, Levee Work, Item No. L-725 Ensley, Tenn.; Drawing No. 1, Revision No. 1; USACE, Memphis District; February 2, 1960.
- Diversion Sewer for Abandonment of 60" Nonconnah Interceptor Sewer, Revision No. 1, W.H. Porter/Consulting Engineers, September 21, 1976.
- DWG. No. 10W208-4, President's Island Interceptor: TVA Ash Pond Crossing Plan Memphis, TN, October 2004.
- DWG. No. 10W208-6, President's Island Interceptor: TVA Ash Pond Crossing Details and Sections, Memphis, TN, November 2004.
- DWG. No. 10W208-1, East Ash Pond Dredge Cell Plan, October 2005.

Documents:

- Inspection of the Abandoned Nonconnah Interceptor; City of Memphis; March 17, 1986.
- City of Memphis, City Contract #N10672, Contract Change Order for 42-inch at South Plant, Ash Pond to Interceptor; Wednesday, August 9, 1995.

Interview/Conference Call with the City of Memphis:

- An interview with the City of Memphis occurred on Wednesday, January 6, 2016 to discuss historic and current matters pertaining to the 60-inch and 42-inch diameter RCPs. Representatives from the City of Memphis, Stantec and TVA were present.

3. Description of Sanitary Sewer Structures and Piping

An aerial overview of the Allen Plant is attached as **Figure 1**. The location of known pipes that are the subject of this evaluation report are illustrated in the figure. The pipe notation for the 60-inch and 42-inch diameter RCPs are listed as ALLEN-1 and ALLEN-2, respectively.

A brief description of pipe infrastructure at the Allen Plant is summarized below. This description is based on reviewing available drawings and historic documentation. Locations dimensions, distances, and elevations referenced in the following paragraphs were taken from historical information provided for review. Where no historical information was available, dimensions noted are from scouting and/or field inspection activities. Copies of the relevant historic drawings are provided in **Attachment C**.

<u>ALLEN-1: 60-inch diameter Reinforced Concrete Pipe (RCP)</u>

Reported to be inactive and abandoned by the City of Memphis, a 60-inch diameter sanitary sewer RCP crosses the Allen Plant's property boundary in an east to west direction. The sewer line extends approximately 4,200 linear feet (LF) across TVA's boundary, beneath the East Ash and Stilling Ponds, East Perimeter Dike, and the Ash Divider Dike. The first manhole along the site property boundary is noted as Manhole No. 17 *(Mississippi River, Memphis Harbor Project, Levee Work, Item No. L-725 Ensley, Tenn.; Drawing No. 1, Revision No. 1; USACE, Memphis District; February 2, 1960)* and extends to Manhole No. 22 at the Allen Plant's west property boundary.

The 60-inch diameter RCP was constructed in 1951 *(Mississippi River, Memphis Harbor Project, Nonconnah Basin Sewer Extension Plan and Profile, 60" Diameter Sewer; USACE, Memphis District, September 1951)* with an average invert depth of approximately 25 feet below the grade at locations extending under the East Ash Disposal Area footprint. The length of pipe extending under the East Ash Disposal Area is approximately 3,000 LF.

The 60-inch diameter RCP terminates at a junction manhole located outside of the Allen Plant's west property boundary. The junction manhole contains an active 96-inch diameter sanitary sewer line which ultimately flows west, to the T.E. Maxson Wastewater Treatment Facility.

During a 1986 inspection, *(Inspection of the Abandoned Nonconnah Interceptor; City of Memphis; March 17, 1986.)* the 60-inch diameter RCP was inspected by walking and visually observing the pipe segments, if accessible. Access to the 60-inch diameter RCP segments within the Allen Plant Boundary were by way of Manhole Nos. 17 and 22. Issues and observations noted are listed in the paragraphs below:

- Manhole No. 22 was accessed and approximately 200 LF of the upstream (toward Manhole No. 21) and downstream (toward Manhole No. 23) pipe segments were walked and visually inspected. In both pipe segments, it was noted that signs of H_2S damage was present and loose aggregate would crumble when touched by hand. Additionally, no leaking joints were observed in either segment.

- Manhole No. 17 was accessed and the downstream pipe segment (toward Manhole No. 18) was walked an unspecified length, but it was stated that the inspection continued a distance under the ash pond. During the inspection, it was noted that 3 inches of sludge was observed along the bottom of the 60-inch diameter RCP. Additionally, as the inspection continued, it was observed under the ash pond, water flow and sludge increased. It was noted "water from the ash pond pouring in through the joints of the pipe," when the downstream pipe segment was "shined" or lamped. Additionally, the upstream pipe segment (toward Manhole No. 16) was visually inspected approximately 100 LF. No issues or observations were noted.
- It was noted the inspection team contacted Mr. Frank Yetter (TVA personnel) to discuss how the 60-inch diameter RCP had been plugged/abandoned. It was stated that Mr. Yetter recollected Manhole No. 20 had been plugged using sandbags.

It was also noted during the January 6th, 2016 conference call that no CCTV inspections were performed on the 60-inch diameter RCP after the 1986 visual inspection.

ALLEN-2: 42-inch diameter Reinforced Concrete Pipe (RCP)

The 42-inch diameter gravity sewer RCP was constructed in the 1970's and receives its flow from a 30-inch diameter force main carrying sewage pumped from the President's Island Industrial Park. The 42-inch diameter RCP flows in a north to south direction into a short section (approx. 15 LF) of a 48-inch diameter pipe which ties into a 96-inch diameter sanitary sewer trunk line near the south boundary of the Allen Plant, south of the railroad embankment. The entire length of the 42-inch diameter RCP is approximately 875 LF with pipe invert depths varying between 10 and 25 feet from grade.

Pipe material length, diameter, depth and general alignment and structures associated with the 30-inch diameter and 42-inch diameter sanitary sewer RCP segments were derived from three drawings (*DWG. No. 10W208-6, President's Island Interceptor: TVA Ash Pond Crossing Details and Sections, Memphis, TN, November 2004; DWG. No. 10W208-4, President's Island Interceptor: TVA Ash Pond Crossing Plan Memphis, TN, October 2004*; and *Diversion Sewer for Abandonment of 60" Nonconnah Interceptor Sewer, Revision No. 1, W.H. Porter/Consulting Engineers, September 21, 1976*.)

Documentation *(City of Memphis, City Contract #N10672, Contract Change Order for 42-inch at South Plant, Ash Pond to Interceptor; Wednesday, August 9, 1995)* provided by the City of Memphis, identifies portions of the 42-inch diameter RCP was rehabilitated in the 1990's using cured-in-place pipe (CIPP) methods. According to this document, the 42-inch diameter RCP had 21 mm (approximately 4/5 of an inch) of Insituform lining (Insitutube) installed rather than the original contract thickness of 18 mm.

It was noted during the January 6th, 2016 conference call that a CCTV inspection was performed on the 42-inch diameter RCP after the 21mm Insitutube CIPP liner was installed. However, the City of Memphis no longer knew the location of that inspection video and was unable to provide for Stantec's review. It was also stated by the City of Memphis the CIPP liner was installed satisfactorily because the project was approved for payment.

It was also noted during the January 6th, 2016 conference call, the City of Memphis would be unable to temporarily shut off or reduce flows from the President's Island Pump Station to facilitate an upstream CCTV inspection for the 42-inch diameter RCP. This was due to the limited time window between pump cycles (less than one hour).

4. Visual Site Observations

Notable observations from visual inspection of the abandoned 60-inch diameter RCP and active 42-inch diameter RCP are summarized below.

ALLEN-1: 60-inch Reinforced Concrete Pipe (RCP)

Manhole No. 22:

Manhole No. 22 is in a stormwater runoff basin area located on the west boundary of the Allen Plant. Portions of the manhole chimney and frame are approximately 4 feet above grade. The incoming and outgoing pipe alignment and general location was not observed when viewing the manhole's interior from the surface due to sediment observed within the manhole. It appeared the bottom of the manhole and approximate incoming and outgoing pipe locations were full of sediment (this was later confirmed when the CCTV camera was lowered into the manhole). Sediment was measured at a depth of 28.5 feet from the manhole's rim. The depth of Manhole No. 22 is estimated at 33.5 feet indicating sediment within the manhole is filled to the crown of the incoming and outgoing pipe segments.

A ¾-inch diameter rigid wall electrical conduit was used to probe the sediment to locate the manhole bottom. The sediment became resistant to probing approximately 4-6 inches below its surface. From the surface, the material appeared to be fine with dark grey color. A small amount of flow was observed coming from the general location of the upstream pipe.

Manhole No. 17:

Manhole No. 17 is located near the east boundary of the Allen Plant. Portions of the manhole chimney and frame are approximately 4 feet above grade. During the site visit, the manhole was surcharged above the pipe crown. Using ¾ -inch diameter rigid wall electrical conduit, the depth of water from the bottom of the manhole was measured at 8 feet thus approximating the manhole's surcharge depth at 3 feet. Additionally, the water did not appear to be flowing and contained a film on the water's surface indicative of stagnant water.

The manholes between Manhole Nos. 17 and 22 (Manhole Nos. 18, 19, 20 and 21) could not be located and therefore, were not visually observed from the surface. Based on historic drawings *(Mississippi River, Memphis Harbor Project, Levee Work, Item No. L-725 Ensley, Tenn.; Drawing No. 1, Revision No. 1, February 2, 1960; USACE, Memphis District)*, Manhole Nos. 18, 19 and 20 are located within the East Ash Pond and East Dredge Cell.

Field personnel attempted to locate Manhole Nos. 16 and 21 but were unsuccessful. An attempt was made to locate Manhole No. 16 to the east of the East Ash Pond, but the general location of the manhole appeared to be in a low-lying area with approximately 1-2 feet of standing water. Alternatively, Manhole No. 21 appears to be below grade within or near a storage tank containment area.

ALLEN-2: 42-inch diameter Reinforced Concrete Pipe (RCP)

An access manhole was identified along the 42-inch diameter sanitary sewer RCP, hazardous gas monitoring devices were worn by personnel working around the manhole. As personnel prepared for inspection activities, the monitoring devices detected quantities of Hydrogen Sulfide (H_2S) above safe working limits. No further visual observations were made.

5. Piping CCTV Internal Inspection Observations

CCTV inspections at the Allen Plant were performed using closed-circuit television (CCTV) camera equipment. Pre-cleaning activities were not performed as part of the inspections.

A memorandum summarizing the field observations and findings of the completed pipe inspections was issued on February 4, 2016.

Station locations referenced are based upon those identified in the plan and profile drawing shown in **Attachment A**. Pipe depths and grade elevations in Attachment A are estimated and intended solely for informational purposes.

ALLEN-1: 60-inch Reinforced Concrete Pipe (RCP)

Submerged Inspection Activities

A submerged inspection of the abandoned 60-inch diameter RCP was attempted at Manhole No. 17. The Remote Operated Vehicle (ROV) unit utilizes video and sonar equipment to collect data during submerged pipe conditions. During the submerged inspection, the total length measured using a tether counter on the ROV's reel was 2,980 linear feet (approximately 1,730 linear feet in the downstream (west) direction and 1,250 linear feet in the upstream (east) direction). Because portions of this manhole are above grade, temporary scaffolding was erected prior to CCTV inspection activities.

The Submerged Pipe Inspection Report for the 60-inch diameter RCP at the Allen Plant is provided in **Attachment B**.

Downstream (West) direction from Manhole No. 17:

- Sediment was observed along the pipe's inspected length and was estimated to be 6-inches in depth.
- The horizontal and vertical alignment of the pipe is constant (no deviations or abrupt changes).

- Manhole No. 18 was located by the ROV approximately 820 linear feet downstream of Manhole No. 17. The ROV estimated Manhole No. 18 was surcharged more than 3 feet.
- The manhole steps in Manhole No. 18 were still intact and clearly visible.
- At three locations in the downstream direction from Manhole No. 17 (Station Nos. 22+95, 23+05 and 28+10), fine sediment/silt deposits formed in a "cone-shape" were observed at the pipe's invert. Also, the same "cone-shaped" sediment formation was observed at various joints along the pipe's length.
- Exposed aggregate was identified continuously with increased severity along the pipe's crown (10 o'clock to 2 o'clock pipe position). However, the pipe's reinforcement material was not observed.
- No cracks or fractures were observed.
- The inspection was terminated at approximately 1,730 linear feet downstream of Manhole No. 17 due to buildup of sediment not allowing the ROV to traverse further downstream.

Upstream (East) direction from Manhole No. 17:

- Sediment was observed along the pipe's inspected length and was estimated to be 6-inches in depth.
- The horizontal and vertical alignment of the pipe is constant (no deviations or abrupt changes).
- Manhole No. 16 was located by the ROV approximately 1,050 linear feet upstream of Manhole No. 17. The ROV estimated Manhole No. 16 was surcharged more than 1 foot. Daylight was observed coming from the lid in Manhole No. 16. However, the manhole was not located from the surface.
- Exposed aggregate was identified continuously through the pipe with increased severity along the pipe's crown (10 o'clock to 2 o'clock pipe position). However, the pipe's reinforcement material was not observed.
- No cracks or fractures were observed.
- The inspection was terminated approximately 1,250 linear feet upstream from Manhole No. 17.

Unsubmerged Inspection Activities

An unsubmerged CCTV inspection of the abandoned 60-inch diameter RCP was attempted at Manhole No. 22. Because portions of this manhole are above grade, temporary scaffolding was erected prior to CCTV inspection activities.

Manhole No. 22

- Once the camera was lowered into the manhole, it was observed that sediment blocked approximately 95% of the upstream and downstream pipe's cross-sectional area. Due to the blockage, entry to the incoming and outgoing pipes was not permissible and the camera survey was abandoned.

- A small amount of flow was observed entering the manhole form the upstream pipe segment. Due to the sediment blockage in the manhole, the origin of the flow is unknown.

- No bulkhead was observed.

- No cracks or fractures were observed in the manhole.

<u>ALLEN-2: 42-inch diameter Reinforced Concrete Pipe (RCP)</u>

No CCTV inspection was completed on the 42-inch diameter RCP.

ATTACHMENTS

Attachment A	Plan/Profile Drawing of 60-inch Diameter RCP
Attachment B	Submerged Pipe Inspection Report for 60-inch Diameter RCP
Attachment C	Historic Drawings

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Revised: 2016-02-24 By: swheatley

Stantec

Legend

- Manhole
- Pipe Segment (Inspected)
- Pipe Segment (Not Inspected)
- Facility Boundary

Notes

1. Coordinate System: NAD 1983 StatePlane Tennessee FIPS 4100 Feet
2. Aerial Source: Provided By Client (Dated 3/12/2015)
3. Pipe Location and Length Shown are Approximate

Client/Project

Tennessee Valley Authority
Allen Fossil Plant
Initial Structural Stability Assessment

Figure No.

1 - **Aerial Overview**

Title

East Ash Pond
Disposal Area
Pipe Inspection

Attachment A

Plan/Profile Drawing of
60-inch Diameter RCP

GRAPHIC SCALE: 1" = 100'
CONTOUR INTERVAL = 2 FEET

STA. 21+55 TYPICAL VIEW OF EXPOSED AGGREGATE

STA. 22+13 VIEW OF JOINT WITH CALCITE MATERIAL

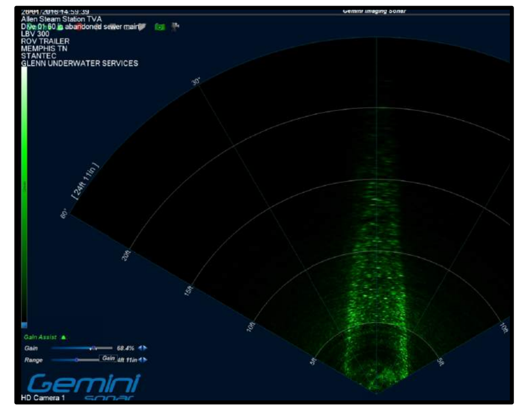
STA. 22+95 SEDIMENT DEPOSIT IN A CONE SHAPE AT INVERT OF RADIAL JOINT

STA. 23+05 SEDIMENT DEPOSIT IN A CONE SHAPE AT INVERT OF RADIAL JOINT

STA. 28+10 SEDIMENT DEPOSIT IN A CONE SHAPE AT INVERT OF RADIAL JOINT

STA. 33+90 TYPICAL VIEW OF EXPOSED AGGREGATE

STA. 37+30 DEBRIS IN PIPE UNPASSABLE; INSPECTION ENDED



DRAFT COPY
FOR INFORMATION PURPOSES ONLY
NOT FOR CONSTRUCTION

ABANDONED 60-INCH SEWER

PLAN AND PROFILE

DRAWING-01

Stantec
10509 Timberwood Circle, Suite 100
Louisville, Kentucky 40223-5301
www.stantec.com

ALLEN FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

FEBRUARY 12, 2016

Attachment B

Submerged Pipe Inspection Report for 60-inch Diameter RCP

Inspection Report

Underwater ROV Inspection of 60" Dia. Sanitary Sewer Line – East Ash Disposal Area

FACILITY: TVA Allen Fossil Plant

Location: Manhole 17 Access

Client: Stantec

Inspection Date: January 26-28, 2016

Prepared by: Richard L. Glenn

Signature: R. L. G

GLENN
Underwater Services

TABLE OF CONTENTS

1.0 INTRODUCTION

Glenn Underwater Services, Inc. (GUS) was contracted by Stantec Consultants to perform a Level 1[1] underwater remotely operated vehicle (ROV)[2] CCTV inspection of the submerged portion of the 60-inch dia. sewer line that runs underneath the East Ash Disposal Area at the Allen Fossil Plant located in Memphis, Shelby County, Tennessee. The plant is owned and operated by the Tennessee Valley Authority. The inspection began on January 26 and was completed on January 28, 2016.

2.0 INSPECTION MEDIUM

Glenn Underwater Services used an ROV specifically designed to inspect pipelines up to distances of 3,000 feet. To aid in the inspection, real-time multi-beam sonar was incorporated into the ROV along with the SeaTec tracking software. The sonar utilizes a multi-beam sound frequency in order to illustrate images of observations from the ROV. With 256 transmitters, the sonar projects extremely fast sound frequencies 127-degrees outwards. These sound frequencies then bounce back and portray an image. This processor allows for sound frequencies to be received and deciphered at a fast speed, making the images appear real-time. This allows data to be obtained in low visibility environments.

3.0 INSPECTION PROCEDURES

GUS began the inspection of the 60" diameter abandon sewer main by entering manhole No. 17 and then traveling first downstream towards the Ash Basin for a distance of 1,729'. At that point, debris that was present prevented further inspection of the pipeline. On the third day, the Remotely Operated Vehicle (ROV) was flown upstream from manhole No. 17 approximately 1,055' to the Manhole No. 16. Location of pipe, manhole and direction of flow can be found in Appendix A.

GUS used an ROV with 3,000' of tether, HD camera, navigation and onboard multi-bean imaging sonar to perform the inspection. Water visibility in the pipe was good, to approximately 4 to 5'. This allowed the ROV to fly along the springline of the pipe to video the crown while the sonar was directed at the invert of the pipe to obtain data. To measure

[1] Level I - A simple visual or tactile inspection, without the aid of tools or measuring devices. It is usually employed to gain an overview of the structure and will precede or verify the need for a more detailed Level II or III inspection.

[2] ROV, is a tethered underwater vehicle

the distance into the pipeline, the tether was fed through an electronic counter that measures in feet. Once the ROV was lowered into the manhole and positioned in front of the pipe opening, the tether counter was zeroed out to allow for maximum measurement accuracy. All incoming information was monitored real-time in GUS' operations trailer. The operations trailer with an ROV pilot and Sonar Tech was parked on the roadway on top of the ash basin dike with GUS personnel positioned at the manhole to manage the ROV and tether.

GUS's computer inspection platform recorded the video and sonar images side by side for better comparison during post processing. The sonar also allowed for measurement of debris in the pipe and forward navigation. For the purpose of this inspection, Manhole No. 17 was labeled Sta. 0+00. The downstream pipeline, which is the primary focus since this section is on TVA's property and under the ash pond, was inspected to Sta. 17+29 and the upstream side Sta. -10+60.

4.0 INSPECTION FINDINGS

4.1 DOWNSTREAM (MANHOLE NO. 17)

a) DEBRIS IN PIPELINE

The debris at the invert of the pipe is approximately 6" in depth from Manhole No. 17 to Sta. 17+29'. At Sta. 17+29' the debris in the invert increased rapidly and prevented the ROV from performing the pipeline inspection. At this location, the debris was approximately 42" in depth. The sonar image could see further into the pipe. The images can be found in Appendix B.

Figure 1 – 810' from manhole No. 17. Fine sand deposit at invert of pipe.

The debris consisted of a fine silt, waste, aggregate and cement from the concrete surface of the pipe. At three locations, Sta. 2+95', 3+05' and Sta. 8+10', a fine sand was found deposited in a cone shape along the invert of the pipe (See Figure 1). At each location the sand was distributed at a radial joint. The sand appears to have passed through the crown of the pipe at an open joint which appears to be an access for water and material to flow in and out of the pipeline.

b) CONCRETE CONDITION

Uniform exposed aggregate with weak cement paste was identified throughout the pipeline, with the heaviest concentration being at the crown of the pipeline between the 10 o'clock to 2 o'clock positions. The ROV, while traveling along the crown of the pipeline, was able to easily dislodge aggregate and delaminated concrete. In fact, the ROV struggled because of the amount of dislodged aggregate that was trapped into the vehicle's thrusters and other components (See Figures 2 & 3). It appears that the material transferred in this pipe may have contributed to the concrete deterioration.

Figure 2 – 155' downstream from manhole No. 17. Example of exposed aggregate.

Silt layers on the pipe's surface did mask some of the concrete delamination but could be visually confirmed by the ROV hitting the concrete surface of the pipe. At Sta. 1+85' and 2+13' there were radial joints that exhibited a calcite leaching from the joint opening. This appears to be similar to what is found in autogenous (self) healing of concrete cracking (See Figure 4).

No major concrete cracking of the pipeline was noted. However, the debris and fouling on the concrete surfaces may have masked small cracks that could not be seen by the ROV video or sonar.

Figure 3 – 800' downstream from manhole No. 17. Example of exposed aggregate with loose cement paste.

c) MANHOLE 18

The inspection located Manhole No. 18 at Sta. 8+20'. The ladder rungs are still intact and clearly visible. The ROV attempted to fly further into the manhole, however the water level appeared to be below the top of the manhole bulkhead.

Figure 4 – 213'' downstream from manhole No. 17. View of joint with calcite material.

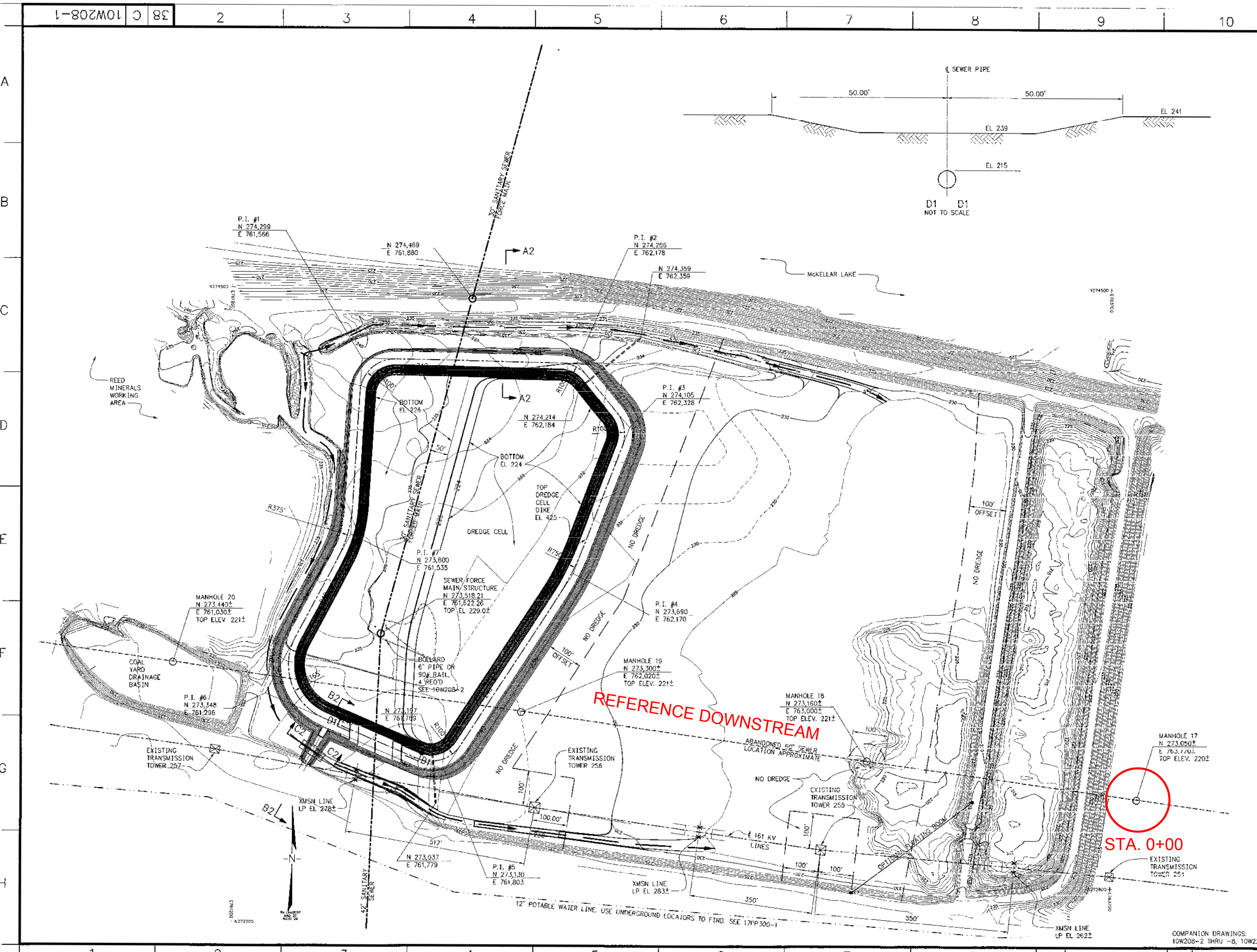
4.2 UPSTREAM OF MANHOLE NO. 17 (Manhole 16)

The ROV traveled 1,055' and terminated the inspection at Manhole No. 16. The ROV could have traveled further, but this was the extent of the inspection requested in this contract. The debris level was consistent with the 6" depth documented in the downstream section of the pipeline. Exposed aggregate with weak cement paste similar to the downstream section of the pipeline was also noted. No other major defects were noted in this section of the pipeline.

- End -

Attachment A

Site Drawings
Pipe Location and Access



NOTES:

1. ALL WORK SHALL BE DONE IN ACCORDANCE WITH THE T-1 GENERAL CONSTRUCTION SPECIFICATIONS, UNLESS OTHERWISE NOTED. SECTION NUMBERS REFER DIRECTLY TO THE T-1 SPECIFICATION.
2. THE AREA FOR THE BASE OF THE DIKE IS TO BE CLEARED OF VEGETATION BY SCRAPING THE SURFACE ACCORDING TO SECTION 101.
3. BASE REINFORCEMENT SHALL CONSIST OF 18" TO 24" OF BOTTOM ASH.
4. TYPE A RIPRAP SHALL BE 9" THICK. A MINIMUM OF 50%, BY WEIGHT, OF THE STONES SHALL BE 25 LB. EACH AND IN ACCORDANCE WITH SECTION 575.
5. TYPE B RIPRAP SHALL BE 18" THICK. A MINIMUM OF 50%, BY WEIGHT, OF THE STONES SHALL BE 100 LB. EACH AND IN ACCORDANCE WITH SECTION 575.
6. FACTORS OF SAFETY FOR THE 3.5:1 OUTSIDE SLOPE OF DREDGE CELL USING PCSTABLE 5M
   FOR Q VALUES F.S. = 2.00
   FOR RBAR VALUES F.S. = 1.61

SOIL	UNIT WEIGHT (pcf)		Q		RBAR	
	MOIST	SATURATED	c (psf)	ø (deg.)	c (psf)	ø (deg.)
EXIST. DIKE & INSITU	125	125	1000	35	1000	35
EXIST. ASH	100	100	150	25	150	25
DREDGED ASH	100	100	150	25	150	25
ASH DIKE FILL	102.8	102.8	250	28	0	30
EXIST. ASH UNDER DIKE	101	101	165	27	165	27

7. COMPACTED ASH IN DREDGE CELL DIKE TO 95% OF STANDARD PROCTOR USING COMPACTION EQUIPMENT. SPREAD ASH IN 6" TO 9" LAYERS BEFORE COMPACTION. TEST FOR COMPACTION AT LEAST ONCE PER WEEK OR EVERY 5000 CUBIC YARDS OF ASH PLACED, WHICHEVER IS MORE FREQUENT, USING NUCLEAR DENSIMETER OR EQUAL TECHNOLOGY. COMPACTION TESTING OF THE FIRST 3 FEET OF THE BASE OF THE DIKE IS OPTIONAL. RECORDS OF COMPACTION TESTING MUST BE KEPT OF THE JOB UNTIL COMPLETION AND THEN FORWARDED TO JOHN ALBRIGHT, LP 2P-C.
8. LOCATION OF DREDGE POND SPILLWAYS SUGGESTED. ANY OTHER LOCATION IN THE DREDGE POND IS ACCEPTABLE.
9. FLOATING BOOM OPTIONAL FOR CONTROL OF CENOSPHERES.

ADDITIONAL NOTES

1. FOR ADDITIONAL NOTES SEE SHEET 2
2. EXISTING CONTOURS FROM MAY 20, 2004 POND SURVEY.
3. MINIMUM CLEARANCE FROM EQUIPMENT TO 161KV TRANSMISSION LINE CONDUCTORS IS 14 FEET. MINIMUM CLEARANCE FROM 161KV CONDUCTORS TO GROUND IS 26 FEET.

R 0 INITIAL ISSUE PER DCA ALF-04-1001-001-R0

SCALE: 1" =100' EXCEPT AS NOTED

YARD

EAST ASH POND
DREDGE CELL
PLAN

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.C. ALBRIGHT	D.R. DODSON	H.L. PETTY	S.R. CATHEY	L.J. PETERSON	B.L. LUNDY	

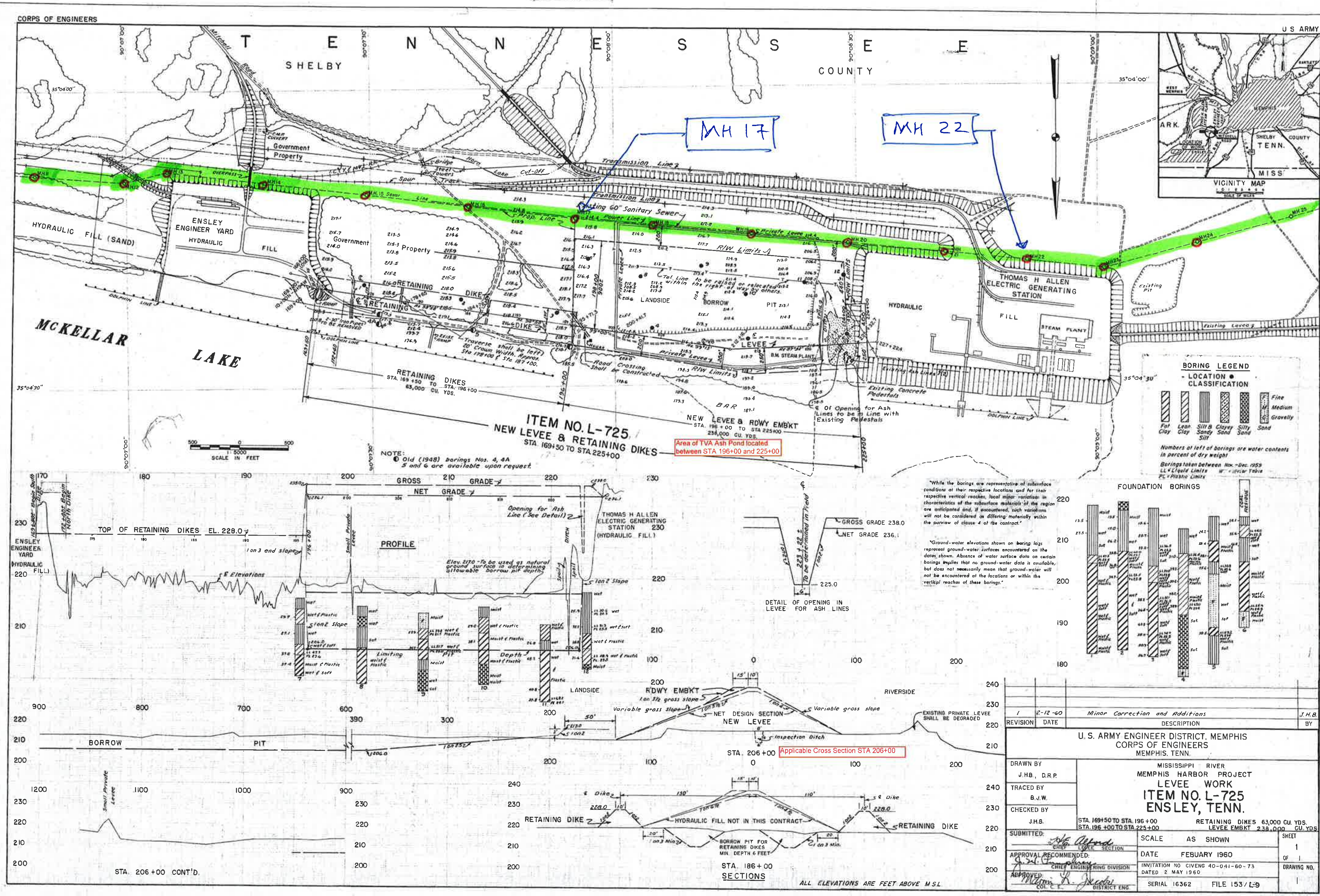
ALLEN FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R14 | 38 | C | 10W208-1 | R 0

PLOT FACTOR:100
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

TASK COMPLETED BY: REV NO.


CORPS OF ENGINEERS
U S ARMY
TENNESSEE
SHELBY COUNTY
MH 17
MH 22
VICINITY MAP
ARK.
SHELBY COUNTY TENN.
MISS.
MEMPHIS
Government Property
Transmission Line
Existing 60" Sanitary Sewer
Prop. Line
Power Line
Private Levee
R/W Limits
ENSLEY ENGINEER YARD
HYDRAULIC FILL (SAND)
HYDRAULIC FILL
Government Property
RETAINING DIKE
LANDSIDE BORROW PIT
THOMAS H ALLEN ELECTRIC GENERATING STATION
HYDRAULIC FILL
STEAM PLANT
Existing Levee
Existing Pit
MCKELLAR LAKE
BAR
Road Crossing Shall be Constructed
Traverse shall be left 20' Crown Width Approx. Sta 178+00 & Sta 187+00.
Existing Concrete Pedestals
Of Opening for Ash Lines to be in Line with Existing Pedestals
RETAINING DIKES STA. 169+50 TO STA. 196+00 63,000 CU. YDS.
NEW LEVEE & RDWY EMBKT STA. 196+00 TO STA 225+00 238,000 CU. YDS.
ITEM NO. L-725
NEW LEVEE & RETAINING DIKES
STA. 169+50 TO STA 225+00
Area of TVA Ash Pond located between STA 196+00 and 225+00
SCALE IN FEET
NOTE: Old (1948) borings Nos. 4, 4A 5 and 6 are available upon request.
BORING LEGEND
LOCATION
CLASSIFICATION
Fat Clay
Lean Clay
Silt & Sandy Silt
Clayey Sand
Silty Sand
Sand
Fine
Medium
Gravelly
Numbers at left of borings are water contents in percent of dry weight
Borings taken between Nov.-Dec. 1959
LL = Liquid Limits
PL = Plastic Limits
FOUNDATION BORINGS
"While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local minor variations in characteristics of the subsurface materials of the region are anticipated and, if encountered, such variations will not be considered as differing materially within the purview of clause 4 of the contract."
"Ground-water elevations shown on boring logs represent ground-water surfaces encountered on the dates shown. Absence of water surface data on certain borings implies that no ground-water data is available, but does not necessarily mean that ground-water will not be encountered at the locations or within the vertical reaches of these borings."
GROSS GRADE
NET GRADE
TOP OF RETAINING DIKES EL. 228.0
PROFILE
Opening for Ash Line (See Detail)
THOMAS H ALLEN ELECTRIC GENERATING STATION (HYDRAULIC FILL)
ENSLEY ENGINEER YARD (HYDRAULIC FILL)
Elev. 217.0 - To be used as natural ground surface in determining allowable borrow pit depth.
Elevations
GROSS GRADE 238.0
NET GRADE 236.1
To be determined in Field
225.0
DETAIL OF OPENING IN LEVEE FOR ASH LINES
LANDSIDE
RDWY EMBKT
NET DESIGN SECTION NEW LEVEE
Variable grass slope
RIVERSIDE
EXISTING PRIVATE LEVEE SHALL BE DEGRADED
Inspection Ditch
BORROW PIT
STA. 206+00
Applicable Cross Section STA 206+00
STA. 206+00 CONT'D.
Small Private Levee
RETAINING DIKE
HYDRAULIC FILL NOT IN THIS CONTRACT
BORROW PIT FOR RETAINING DIKES MIN. DEPTH 6 FEET
STA. 186+00
SECTIONS
ALL ELEVATIONS ARE FEET ABOVE M.S.L.
1 | 2-12-60 | Minor Correction and Additions | J.H.B.
REVISION | DATE | DESCRIPTION | BY
U. S. ARMY ENGINEER DISTRICT, MEMPHIS
CORPS OF ENGINEERS
MEMPHIS, TENN.
DRAWN BY J.H.B., D.R.P.
TRACED BY B.J.W.
CHECKED BY J.H.B.
SUBMITTED: CHIEF LEVEE SECTION
APPROVAL RECOMMENDED: CHIEF ENGINEERING DIVISION
APPROVED: COL. C. E. DISTRICT ENG.
MISSISSIPPI RIVER
MEMPHIS HARBOR PROJECT
LEVEE WORK
ITEM NO. L-725
ENSLEY, TENN.
STA. 169+50 TO STA. 196+00 RETAINING DIKES 63,000 CU. YDS.
STA. 196+00 TO STA. 225+00 LEVEE EMBKT 238,000 CU. YDS.
SCALE AS SHOWN
DATE FEBUARY 1960
INVITATION NO CIVENG 40-041-60-73 DATED 2 MAY 1960
SERIAL 16362
FILE 153/L-9
SHEET 1 OF 1
DRAWING NO. 1

Map ID	Pipe ID	Description
001	ALF-EAP-FB-001-A	Primary Spillway (Concrete gated box structure)
002	ALF-ESP-MG-002-A	East Stilling Pond Auxiliary Spillway
003	ALF-ESP-MG-003-A	East Stilling Pond Auxiliary Spillway
004	ALF-ESP-MG-004-A	East Stilling Pond Primary Spillway
005	ALF-ESP-MG-005-A	East Stilling Pond Primary Spillway

STANTEC CONSULTING SERVICES INC.
1409 N. Forbes Rd.
Lexington, Kentucky
40511-2050
859-422-3000

Stantec

Spillway/Conduit Inventory

Tennessee Valley Authority
Allen Fossil Plant
Memphis, Shelby County, Tennessee

PROJECT NO.	175609016
DATE	MARCH 2010
DRAWN BY	LJW
CHECKED BY	MAH
CHECKED BY	
SCALE	AS SHOWN
REVISED	
1.	
2.	
3.	
4.	
5.	
6.	
7.	
8.	

SHEET

1 OF 1

Attachment B

Photographic Log

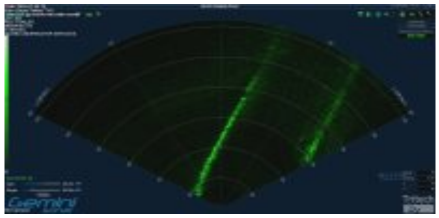
Photographic Log

Project:	Allen Steam Station TVA	**Date:**	26/01/2016
Dive:	Dive 01 60 in abandoned sewer main	**System:**	LBV 300
Location:	MEMPHIS TN	**Vessel:**	ROV TRAILER
Client:	STANTEC		

Key: Photo | Anomaly

120-feet from manway 17 buildup at crown of pipe

120-feet from manway 17 buildup at crown of pipe_C1.jpg

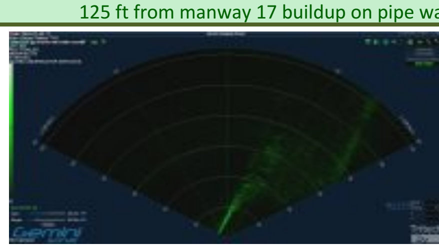


120-feet from manway 17 buildup at crown of pipe_C2.jpg

Time:	2016-01-26 12:58:21.3	**Video Time:**	00:22:57
Log Event:	Photo [C1,C2]: 120-feet from manway 17 buildup at crown of pipe		
Log Session:	120-feet from manway 17 buildup at crown of pipe_C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

125 ft from manway 17 buildup on pipe wall left springline

125 ft from manway 17 buildup on pipe wall left springline_C1.jpg

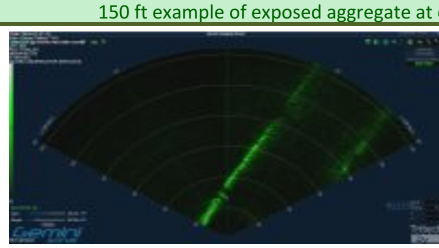


125 ft from manway 17 buildup on pipe wall left springline_C2.jpg

Time:	2016-01-26 13:00:17.4	**Video Time:**	00:24:54
Log Event:	Photo [C1,C2]: 125 ft from manway 17 buildup on pipe wall left springline		
Log Session:	125 ft from manway 17 buildup on pipe wall left springline_C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

150 ft example of exposed aggregate at crown of pipe

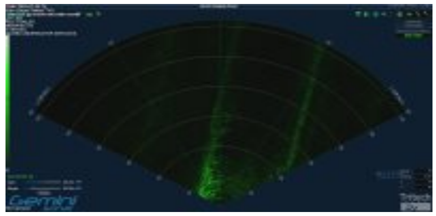
150 ft example of exposed aggregate at crown of pipe_C1.jpg



150 ft example of exposed aggregate at crown of pipe_C2.jpg

Time:	2016-01-26 13:01:42.2	**Video Time:**	00:26:18
Log Event:	Photo [C1,C2]: 150 ft example of exposed aggregate at crown of pipe		
Log Session:	150 ft example of exposed aggregate at crown of pipe_C1.jpg		
Structure:			

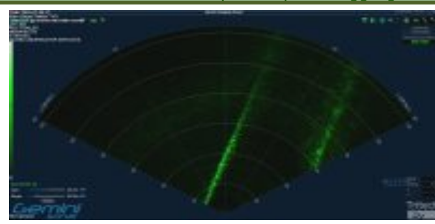
145 ft from manway 17 view of silt at invert of pipe



[145 ft from manway 17 view of silt at invert of pipe_C1.jpg] [145 ft from manway 17 view of silt at invert of pipe_C2.jpg]

Time:	2016-01-26 13:05:24.8	**Video Time:**	00:00:01
Log Event:	Photo [C1,C2]: 145 ft from manway 17 view of silt at invert of pipe		
Log Session:	145 ft from manway 17 view of silt at invert of pipe_C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

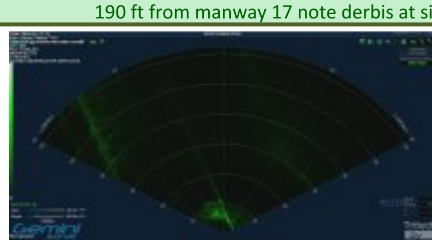
155 ft from manway 17 exposed aggregate at 10 0clock postion



[155 ft from manway 17 exposed aggregate at 10 0clock postion_C1.jpg] [155 ft from manway 17 exposed aggregate at 10 0clock postion_C2.jpg]

Time:	2016-01-26 13:08:47.6	**Video Time:**	00:03:24
Log Event:	Photo [C1,C2]: 155 ft from manway 17 exposed aggregate at 10 0clock postion		
Log Session:	155 ft from manway 17 exposed aggregate at 10 0clock postion_C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

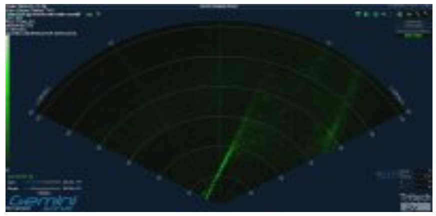
190 ft from manway 17 note derbis at side wall of pipe



[190 ft from manway 17 note derbis at side wall of pipe_C1.jpg] [190 ft from manway 17 note derbis at side wall of pipe_C2.jpg]

Time:	2016-01-26 13:12:22.3	**Video Time:**	00:06:58
Log Event:	Photo [C1,C2]: 190 ft from manway 17 note derbis at side wall of pipe		
Log Session:	190 ft from manway 17 note derbis at side wall of pipe_C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

185 ft from manway 17 note debris along wall on oppostie side

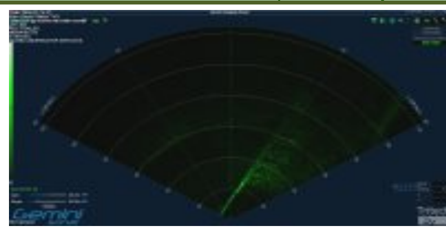


185 ft from manway 17 note debris along wall on oppostie side_C1.jpg

185 ft from manway 17 note debris along wall on oppostie side_C2.jpg

Time:	2016-01-26 13:13:07.4	Video Time:	00:07:43
Log Event:	Photo [C1,C2]: 185 ft from manway 17 note debris along wall on oppostie side		
Log Session:	185 ft from manway 17 note debris along wall on oppostie side_C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

213 ft from manway 17 view of joint with material



213 ft from manway 17 view of joint with material_C1.jpg

213 ft from manway 17 view of joint with material_C2.jpg

Time:	2016-01-26 13:14:57.8	Video Time:	00:09:34
Log Event:	Photo [C1,C2]: 213 ft from manway 17 view of joint with material		
Log Session:	213 ft from manway 17 view of joint with material_C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

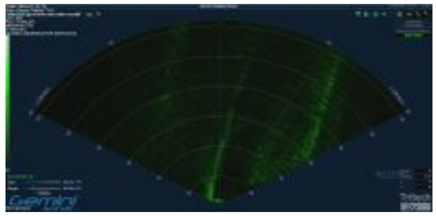
295 ft from manway 17. Sand deposit at invert of pipe. below joint.

295 ft from manway 17. Sand deposit at invert of pipe. below joint._C1.jpg

295 ft from manway 17. Sand deposit at invert of pipe. below joint._C2.jpg

Time:	2016-01-26 13:22:03.2	Video Time:	00:16:39
Log Event:	Photo [C1,C2]: 295 ft from manway 17. Sand deposit at invert of pipe. below joint.		
Log Session:	295 ft from manway 17. Sand deposit at invert of pipe. below joint._C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

305 ft from manway 17 sand depoist ontop of silt.

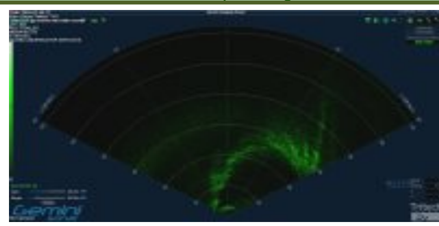


[305 ft from manway 17 sand depoist ontop of silt._C1.jpg]

[305 ft from manway 17 sand depoist ontop of silt._C2.jpg]

Time:	2016-01-26 13:25:24.9	**Video Time:**	00:20:01
Log Event:	Photo [C1,C2]: 305 ft from manway 17 sand depoist ontop of silt.		
Log Session:	305 ft from manway 17 sand depoist ontop of silt._C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

810 ft from manway 17 large amount of debris at invert of pipe

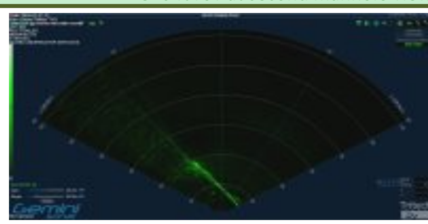


[810 ft from manway 17 large amount of debris at invert of pipe_C1.jpg]

[810 ft from manway 17 large amount of debris at invert of pipe_C2.jpg]

Time:	2016-01-26 13:48:37.5	**Video Time:**	00:13:13
Log Event:	Photo [C1,C2]: 810 ft from manway 17 large amount of debris at invert of pipe		
Log Session:	810 ft from manway 17 large amount of debris at invert of pipe_C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

820 ft rov at second manhole view of ladder



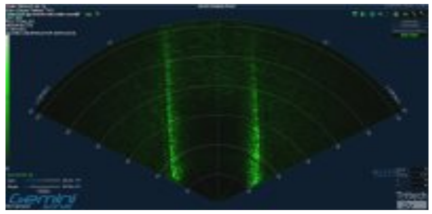
[820 ft rov at second manhole view of ladder_C1.jpg]

[820 ft rov at second manhole view of ladder_C2.jpg]

Time:	2016-01-26 13:51:51.8	**Video Time:**	00:16:28
Log Event:	Photo [C1,C2]: 820 ft rov at second manhole view of ladder		
Log Session:	820 ft rov at second manhole view of ladder_C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

825 ft from manway 17 debris hanging from crown of pipe

825 ft from manway 17 debris hanging from crown of pipe_C1.jpg

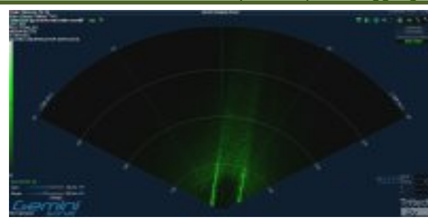


825 ft from manway 17 debris hanging from crown of pipe_C2.jpg

Time:	2016-01-26 13:54:15.4	Video Time:	00:18:51
Log Event:	Photo [C1,C2]: 825 ft from manway 17 debris hanging from crown of pipe		
Log Session:	825 ft from manway 17 debris hanging from crown of pipe_C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

1390 ft from manway 17 exsposed aggregate at crown of pipe

1390 ft from manway 17 exsposed aggregate at crown of pipe_C1.jpg



1390 ft from manway 17 exsposed aggregate at crown of pipe_C2.jpg

Time:	2016-01-26 14:25:30.7	Video Time:	00:20:07
Log Event:	Photo [C1,C2]: 1390 ft from manway 17 exsposed aggregate at crown of pipe		
Log Session:	1390 ft from manway 17 exsposed aggregate at crown of pipe_C1.jpg		
Structure:			
Sub-Structure:			
Anomaly ID:			

Attachment C

Sonar Images Termination of Inspection Downstream

SONAR COMPARSION

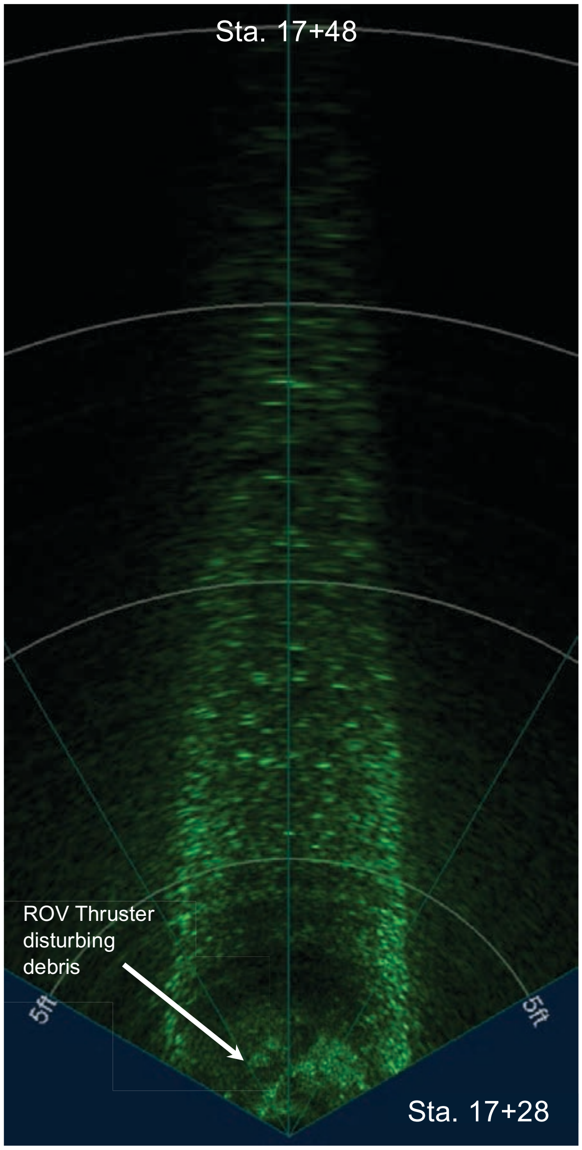


Image 1 - This sonar image shows the approximate location of the closure in the pipe from debris or other materials. It appears the blockage is at approximately Sta. 17+48'. The top of the ROV was positioned against the crown of the pipe and the sonar which is connect to the bottom of the ROV was in the debris. The vehicle could not move any further downstream in the pipeline.

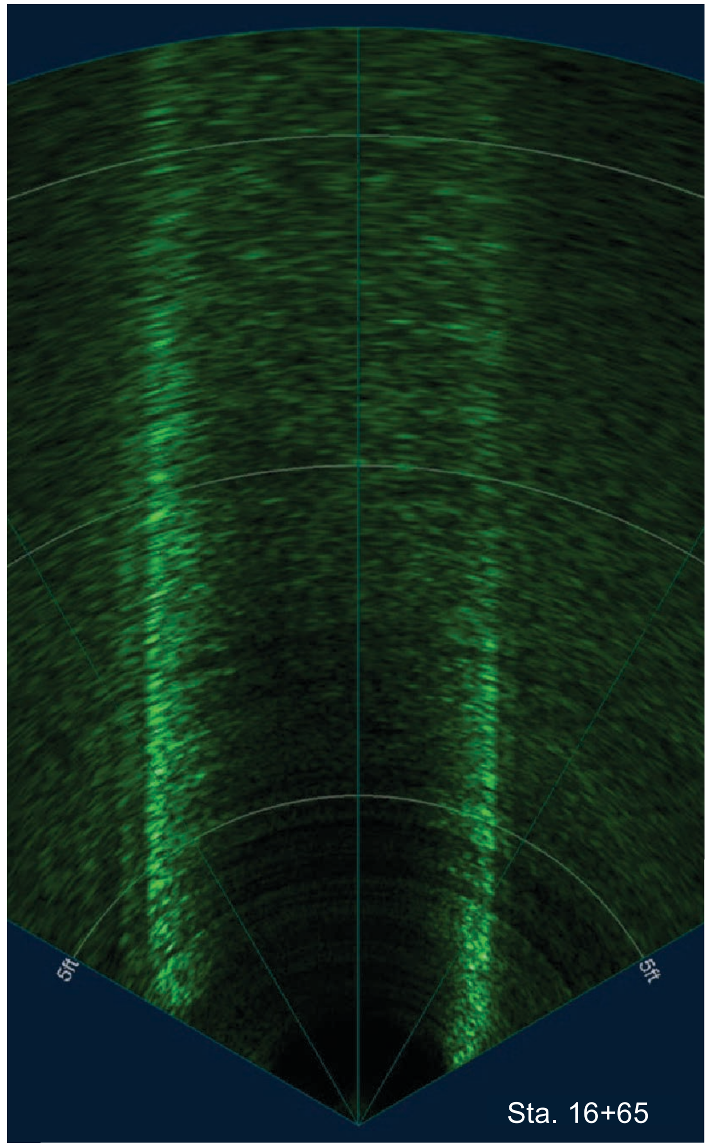
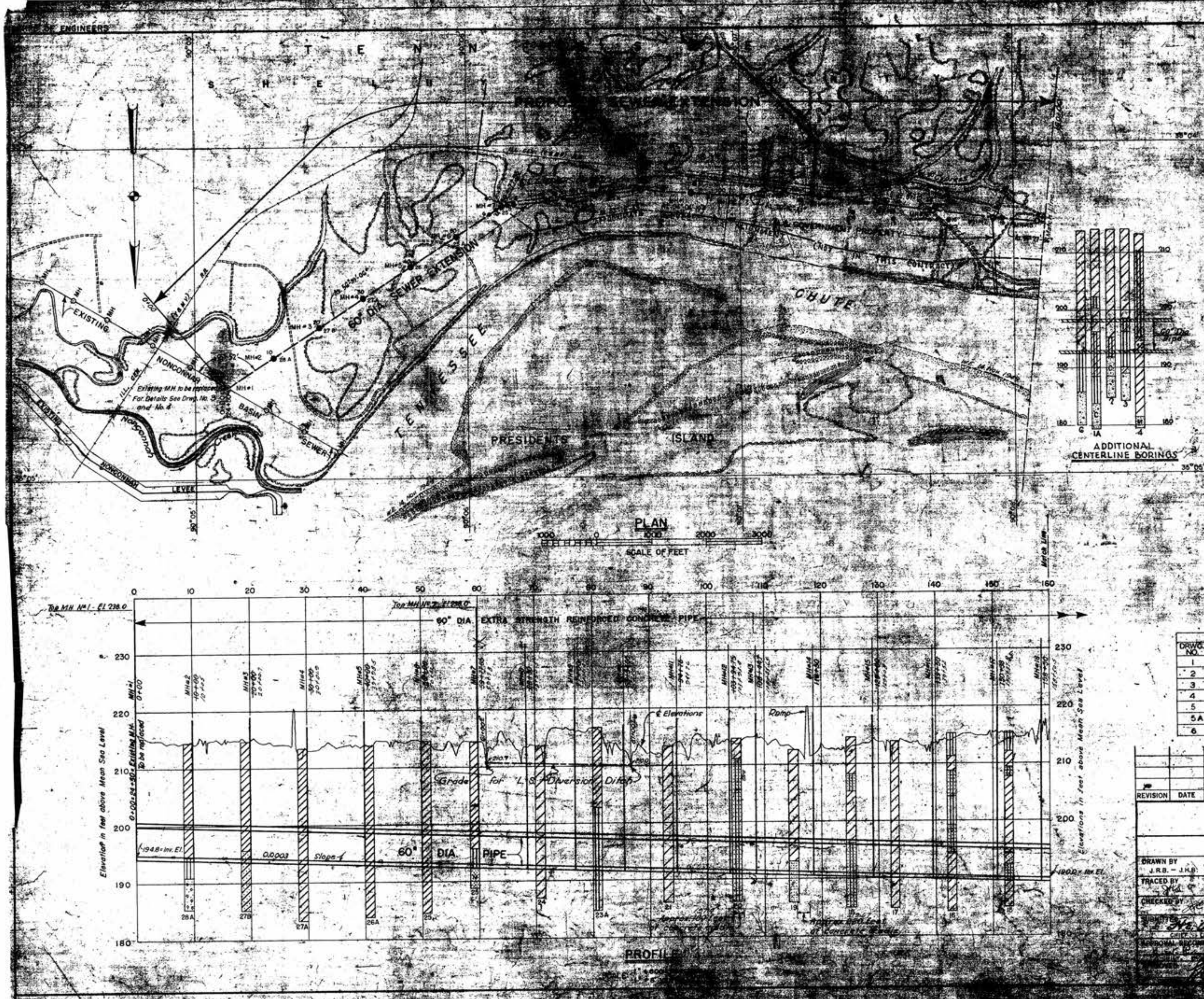
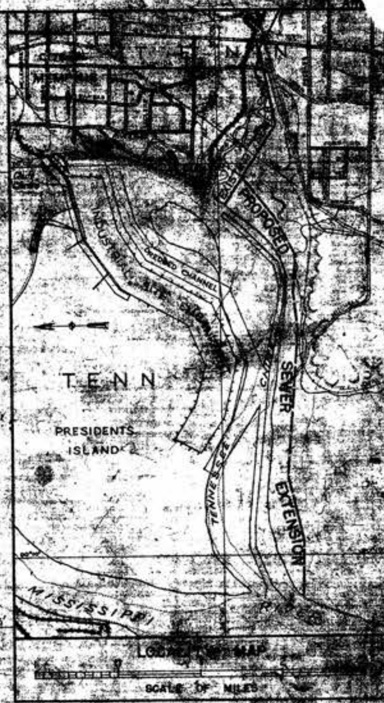


Image 2 – Shows a typical sonar image in a clear pipe. The sonar is positioned at the springline of the pipeline. (-6" debris at invert).

Attachment C

Historic Drawings





General Notes:
For Manhole Schedule, See Drawing No. 2.
For Manhole Details, See Drawing No. 3, 4 & 6.
For Section of L.S. Diversion Ditch, See Drwg. No.

COPY

ORIGINAL IN ARCHIVE DR

INDEX OF DRAWINGS		
DRWG. NO.	FILE NO.	TITLE OF DRAWING
1	153/S-57	PLAN AND PROFILE-60" DIAMETER SEWER
2	153/S-58	PLAN AND PROFILE-60" DIAMETER SEWER
3	153/S-59	PLANS AND SECTIONS-STD. M.H. & MANHOLES
4	153/S-60	MISCELLANEOUS DETAILS-MANHOLES
5	153/S-61	DETAILS-M.H. Nº 7, CONC. PIPE ENCASEMENT
5A	153/S-62	PROFILE AND DETAILS-SEWER OUTFALL
6	153/S-56	RIGHT-OF-WAY MAP

REVISION	DATE	DESCRIPTION

CORPS OF ENGINEERS, U.S. ARMY
OFFICE OF THE DISTRICT ENGINEER
MEMPHIS, TENN.

3AA 031

MISSISSIPPI RIVER
MEMPHIS HARBOR PROJECT
NONCONNAH BASIN SEWER EXTENSION
PLAN AND PROFILE
60" DIAMETER SEWER
STA. 0+00 TO STA. 160+00±

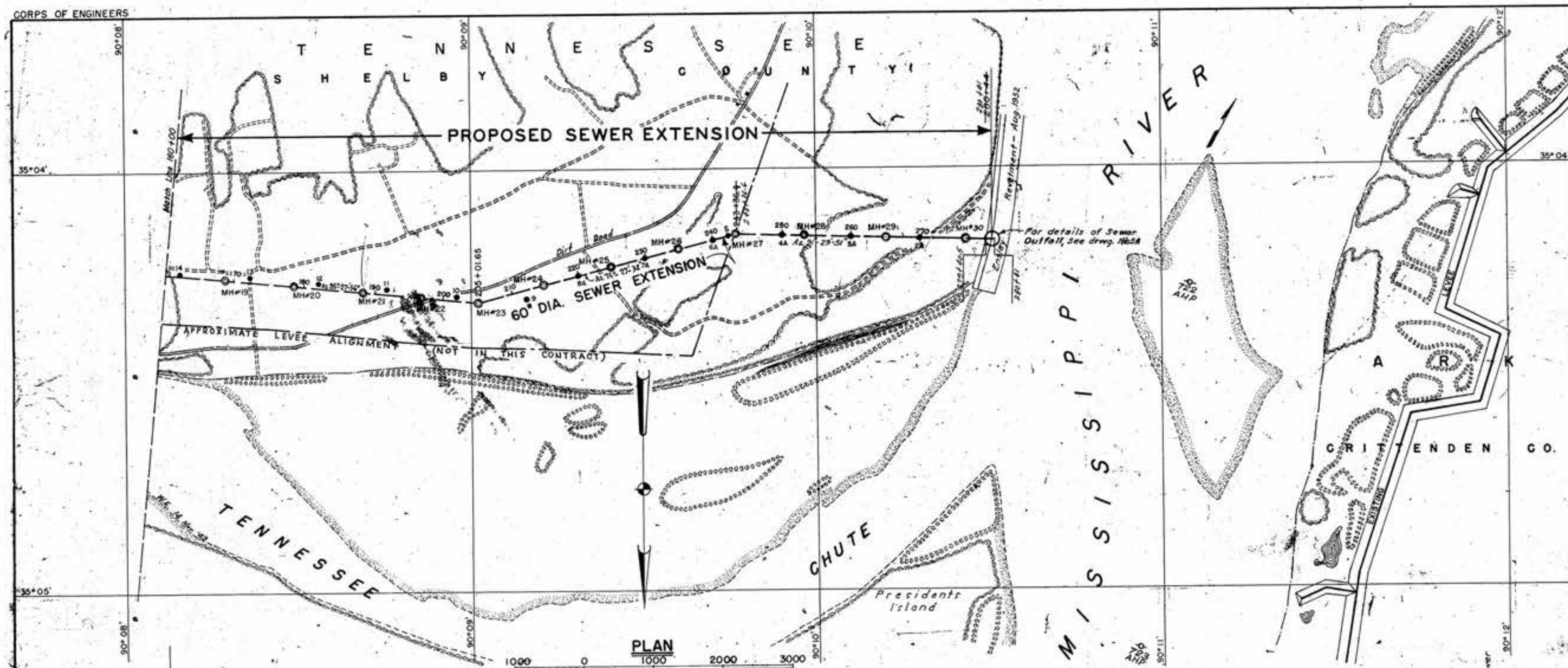
DRAWN BY J.R.B. – J.H.B.
TRACED BY
CHECKED BY
SUBMITTED
CHIEF
APPROVAL RECOMMENDED
APPROVED

SCALE AS SHOWN
DATE SEPTEMBER 1951
FILE 153/S-57

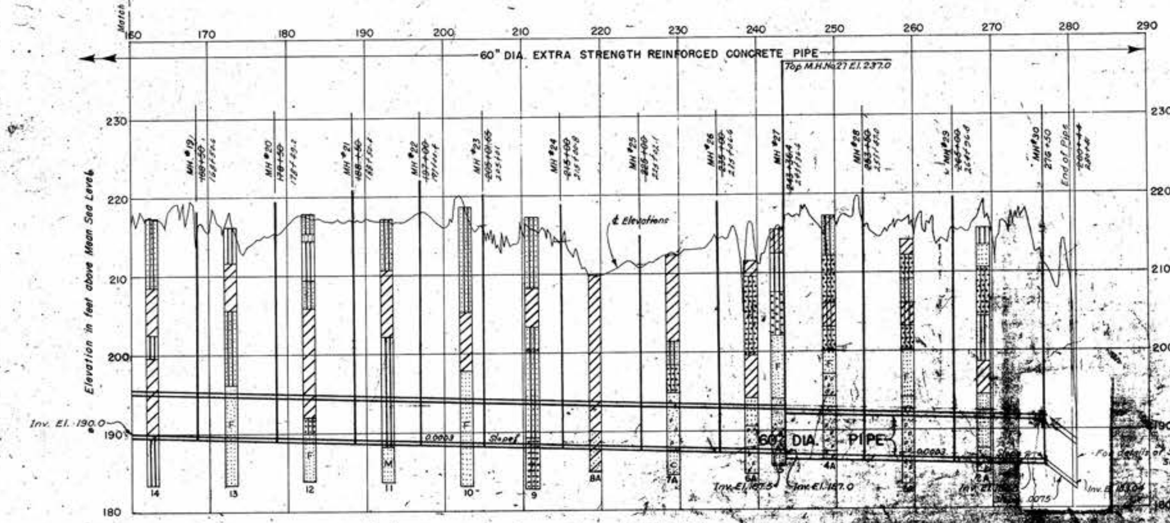
NO-1

CORPS OF ENGINEERS

U.S. ARMY

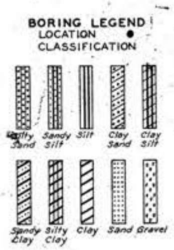


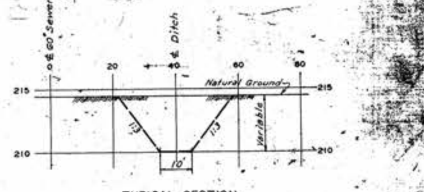
PLAN



PROFILE

SCALE: 1" = 600' HORIZONTAL, 1" = 6' VERTICAL





TYPICAL SECTION
L. S. DIVERSION DITCH

For location and grade of ditch, See Plan & Profile, Drwg. No. 1

MANHOLE SCHEDULE

M.H. NO.	TYPE	STATION	* TOP ELEV.	* INVERT ELEV.	REMARKS
1	Detailed	0+00	23[illegible]	194.80	
2	Std.	10+00	218.5	194.50	
3	Std.	20+00	219.2	194.20	
4	Std.	30+00	216.9	193.90	
5	Std.	40+00	218.6	193.60	
6	Std.	50+00	219.8	193.30	
7	Detailed	∠ 59+78	238.0	193.01	
8	Std.	68+50	217.8	192.75	
9	Std.	77+00	220.5	192.50	
10	Std.	∠ 85+54	217.3	192.24	
11	Std.	94+75	218.0	191.96	
12	Std.	∠ 103+95	218.7	191.69	
13	Std.	∠ 108+45	219.5	191.55	
14	Std.	118+50	219.2	191.25	
15	Std.	129+00	218.9	190.93	
16	Std.	139+50	217.6	190.62	
17	Std.	∠ 150+38	219.3	190.30	
18	Std.	158+50	221.1	190.06	
19	Std.	168+50	218.8	189.76	
20	Std.	178+50	219.5	189.46	
21	Std.	188+50	221.2	189.16	
22	Std.	197+00	221.9	188.90	
23	Std.	∠ 205+02	220.7	188.66	
24	Std.	215+00	218.4	188.36	
25	Std.	225+00	215.1	188.06	
26	Std.	235+00	218.8	187.76	
27	Detailed	∠ 243+36	237.0	187.50	
28	Std.	253+50	220.7	186.70	
29	Std.	265+00	220.8	186.35	
30	Std.	276+50	216.0	186.00	
End Pipe	Concrete	280+44	------	185.0	

* *Elevation in feet above Mean Sea Level*

For Details of Manholes, See Drwgs. No. 3, 4 & 5

∠ Angle in Centerline

COPY

ORIGINAL IN ARCHIVE DRAWER

REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U.S. ARMY
OFFICE OF THE DISTRICT ENGINEER
MEMPHIS, TENN.

3AA 032

MISSISSIPPI RIVER
MEMPHIS HARBOR PROJECT
NONCONNAH BASIN SEWER EXTENSION
PLAN AND PROFILE
60" DIAMETER SEWER
STA. 160+00 TO STA. 283+00±

DRAWN BY J.R.B.–J.H.B.

TRACED BY

CHECKED BY

SUBMITTED: CHIEF, [illegible] SECTION

APPROVAL RECOMMENDED: CHIEF, ENGINEERING DIVISION

APPROVED:

SCALE AS SHOWN

DATE SEPTEMBER 1951

SPEC. SERIAL NO. CIVENG-40-041-53-27 DATED 2 SEPT 1952

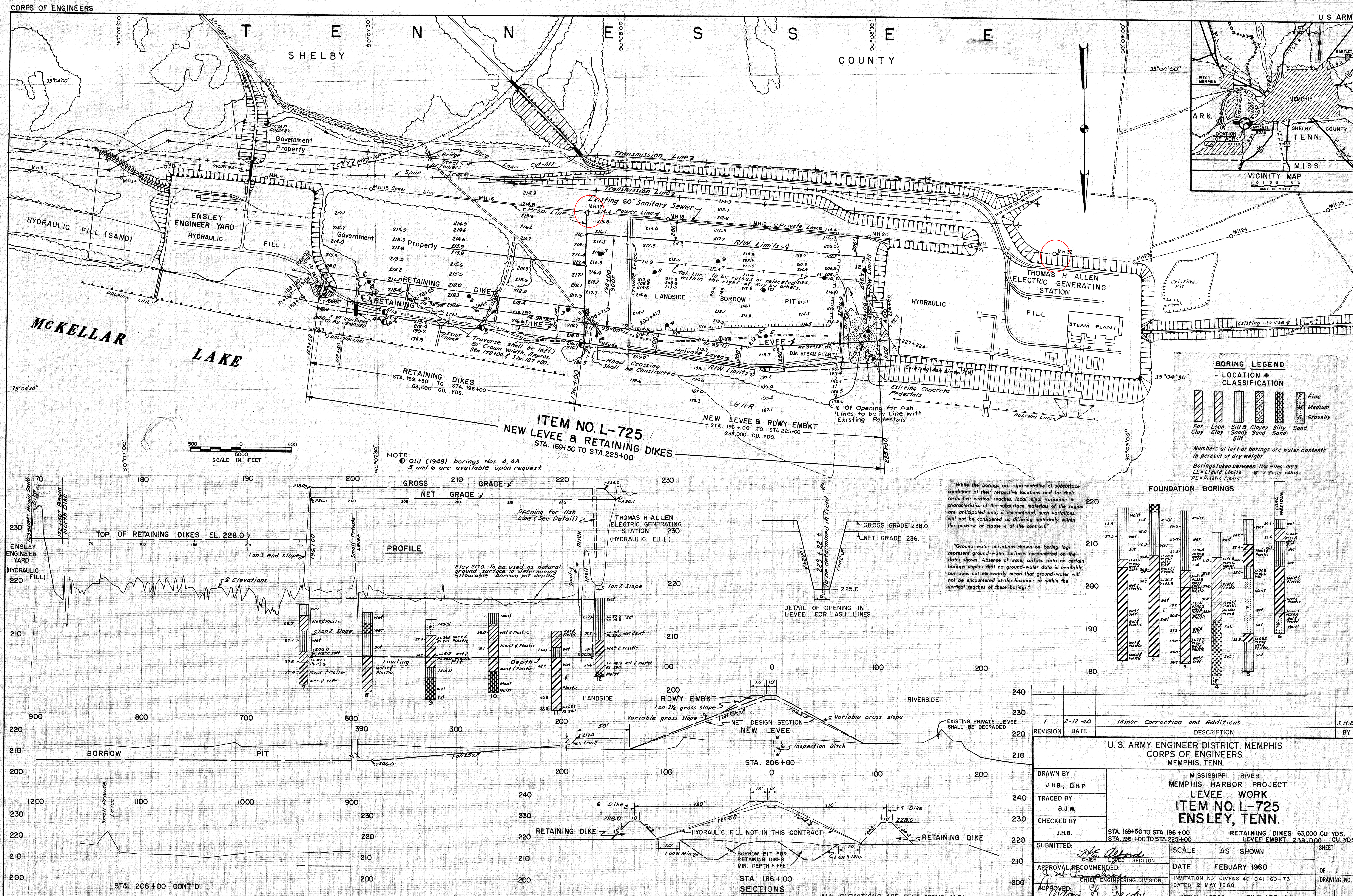
FILE 153/5-58

SHEET 2 OF [illegible]

NO-1

CORPS OF ENGINEERS

U S ARMY



"While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local minor variations in characteristics of the subsurface materials of the region are anticipated and, if encountered, such variations will not be considered as differing materially within the purview of clause 4 of the contract."

"Ground-water elevations shown on boring logs represent ground-water surfaces encountered on the dates shown. Absence of water surface data on certain borings implies that no ground-water data is available, but does not necessarily mean that ground-water will not be encountered at the locations or within the vertical reaches of these borings."

REVISION	DATE	DESCRIPTION	BY
1	2-12-60	Minor Correction and Additions	J.H.B.

U. S. ARMY ENGINEER DISTRICT, MEMPHIS
CORPS OF ENGINEERS
MEMPHIS, TENN.

DRAWN BY J.H.B., D.R.P.

TRACED BY B.J.W.

CHECKED BY J.H.B.

MISSISSIPPI RIVER
MEMPHIS HARBOR PROJECT
LEVEE WORK
ITEM NO. L-725
ENSLEY, TENN.

STA. 169+50 TO STA. 196+00 RETAINING DIKES 63,000 CU. YDS.
STA. 196+00 TO STA. 225+00 LEVEE EMBKT 238,000 CU. YDS.

SUBMITTED: H. E. Alford, CHIEF LEVEE SECTION

APPROVAL RECOMMENDED: J. W. Humphrey, CHIEF ENGINEERING DIVISION

APPROVED: Marvin L. Jacobs, COL. C. E., DISTRICT ENG.

SCALE AS SHOWN

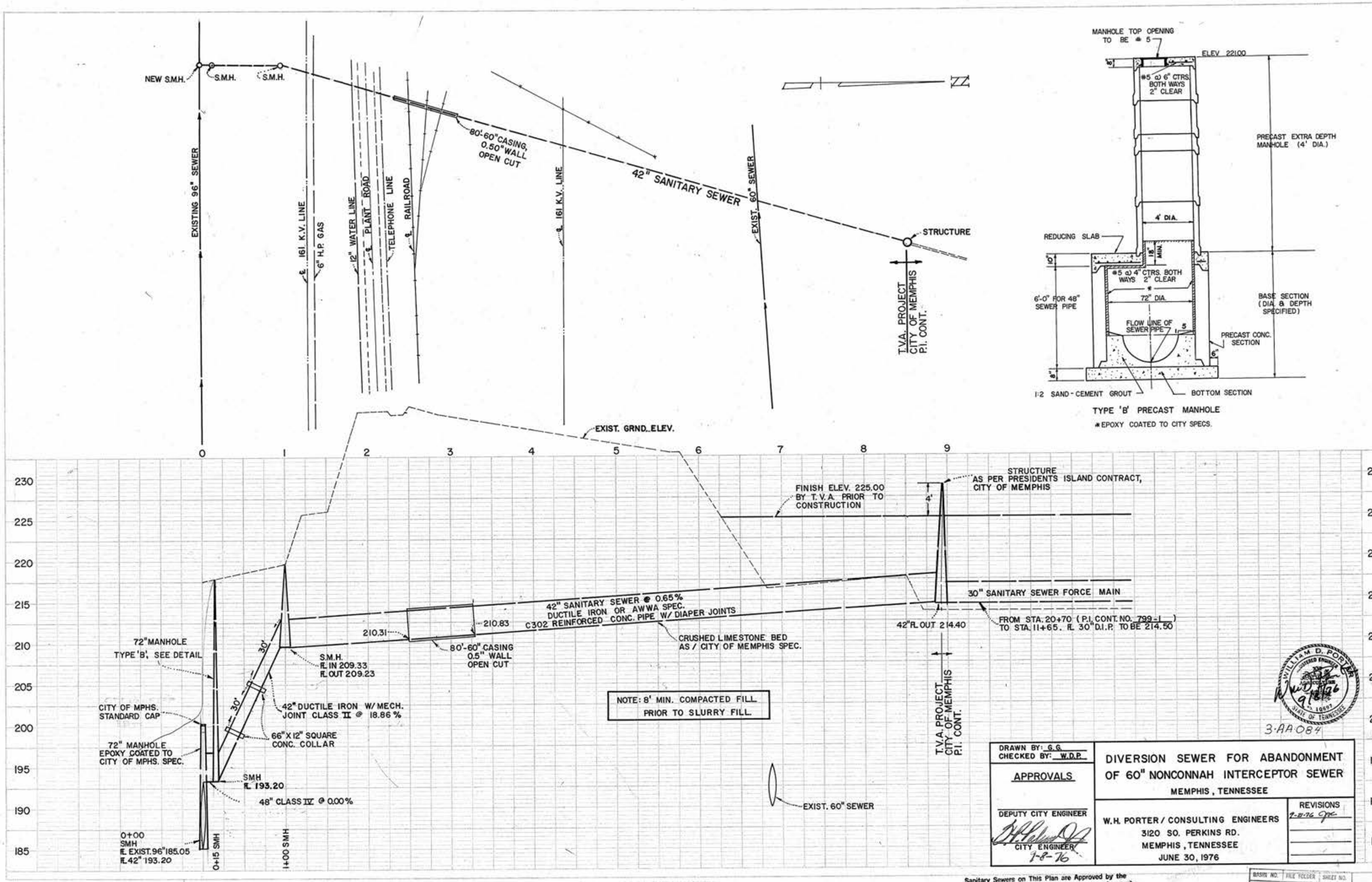
DATE FEBUARY 1960

INVITATION NO CIVENG 40-041-60-73
DATED 2 MAY 1960

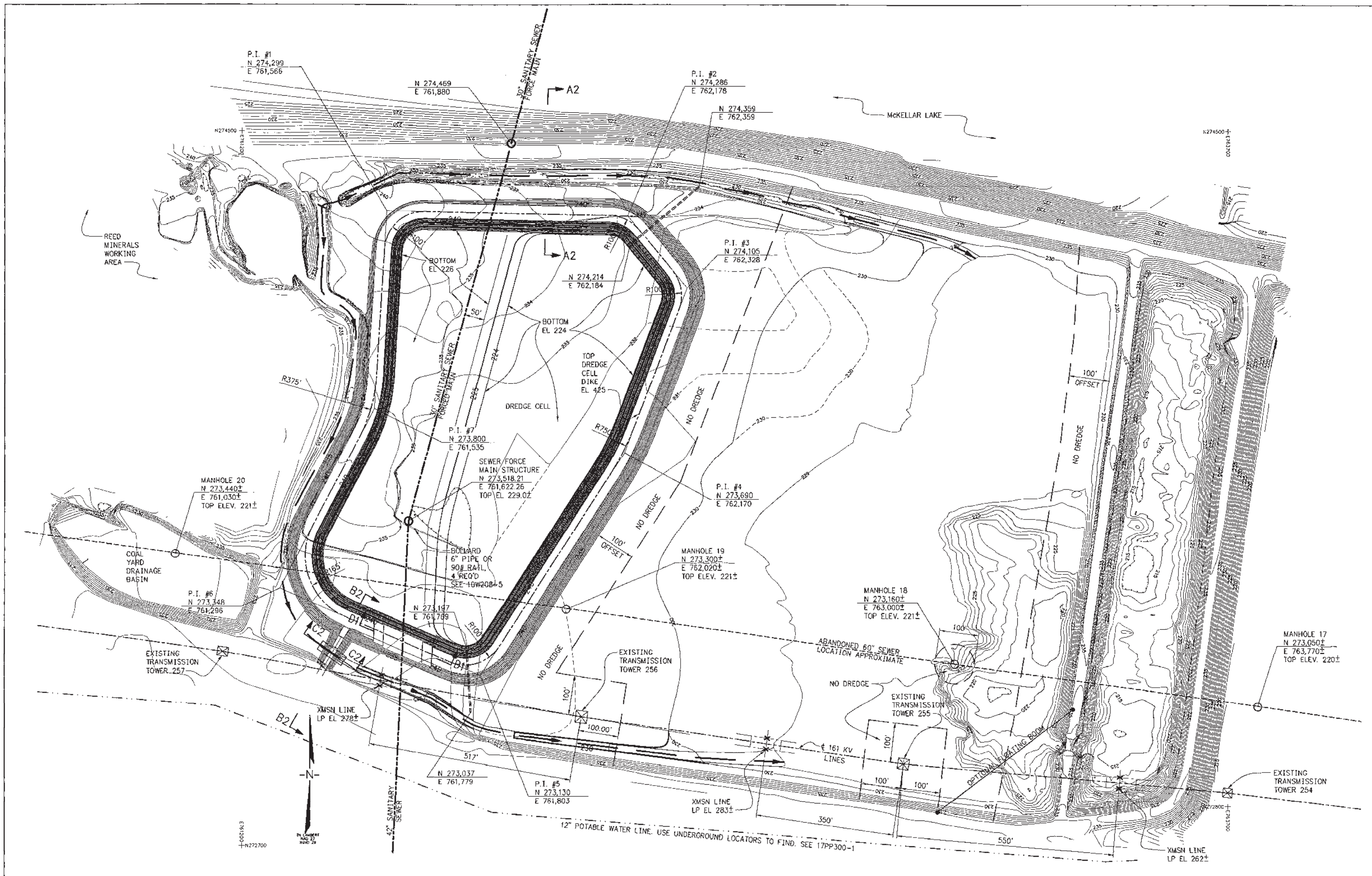
SERIAL 16362 FILE 153/L-9

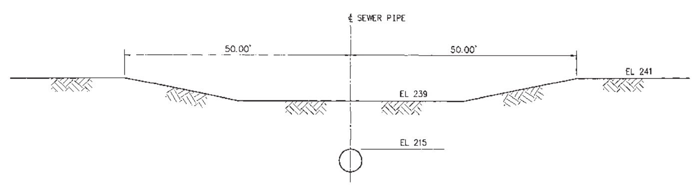
SHEET 1 OF 1

DRAWING NO. 1



Sanitary Sewers on This Plan are Approved by the City Engineer, of Memphis, Contingent upon Approval by State of Tennessee Department of Public Health as Required under Tennessee code annotated 53-2002





D1 – D1
NOT TO SCALE

NOTES:

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4. TYPE A RIPRAP SHALL BE 9" THICK. A MINIMUM OF 50%, BY WEIGHT, OF THE STONES SHALL BE 25 LB. EACH AND IN ACCORDANCE WITH SECTION 575.
5. TYPE B RIPRAP SHALL BE 18" THICK. A MINIMUM OF 50%, BY WEIGHT, OF THE STONES SHALL BE 100 LB. EACH AND IN ACCORDANCE WITH SECTION 575.
6. FACTORS OF SAFETY FOR THE 3.5:1 OUTSIDE SLOPE OF DREDGE CELL USING PCSTABLE 5M
   FOR Q VALUES F.S. = 2.00
   FOR RBAR VALUES F.S. = 1.51

SOIL	UNIT WEIGHT (pcf)		Q		RBAR	
	MOIST	SATURATED	c (psf)	φ (deg.)	c (psf)	φ (deg.)
EXIST. DIKE & INSITU	125	125	1000	35	1000	35
EXIST. ASH	100	100	150	25	150	25
DREDGED ASH	100	100	150	25	150	25
ASH DIKE FILL	102.8	102.8	250	28	0	30
EXIST. ASH UNDER DIKE	101	101	165	27	165	27

7. COMPACTED ASH IN DREDGE CELL DIKE TO 95% OF STANDARD PROCTOR USING COMPACTION EQUIPMENT. SPREAD ASH IN 6" TO 9" LAYERS BEFORE COMPACTION. TEST FOR COMPACTION AT LEAST ONCE PER WEEK OR EVERY 5000 CUBIC YARDS OF ASH PLACED, WHICHEVER IS MORE FREQUENT, USING NUCLEAR DENSIMETER OR EQUAL TECHNOLOGY. COMPACTION TESTING OF THE FIRST 3 FEET OF THE BASE OF THE DIKE IS OPTIONAL. RECORDS OF COMPACTION TESTING MUST BE KEPT OF THE JOB UNTIL COMPLETION AND THEN FORWARDED TO JOHN ALBRIGHT, LP 2P-C.
8. LOCATION OF DREDGE POND SPILLWAYS SUGGESTED. ANY OTHER LOCATION IN THE DREDGE POND IS ACCEPTABLE.
9. FLOATING BOOM OPTIONAL FOR CONTROL OF CENOSPHERES.

ADDITIONAL NOTES

1. FOR ADDITIONAL NOTES SEE SHEET 2
2. EXISTING CONTOURS FROM MAY 20, 2004 POND SURVEY.
3. MINIMUM CLEARANCE FROM EQUIPMENT TO 161KV TRANSMISSION LINE CONDUCTORS IS 14 FEET. MINIMUM CLEARANCE FROM 161KV CONDUCTORS TO GROUND IS 26 FEET.

APPROVED FOR CONSTRUCTION

THE DOCUMENT BEARING THIS STAMP HAS BEEN RECEIVED AND REVIEWED BY THE CITY OF MEMPHIS DIVISION OF ENGINEERING UNDER AUTHORITY DELEGATED BY THE TENNESSEE DEPARTMENT OF ENVIRONMENT AND CONSERVATION DIVISION OF WATER POLLUTION CONTROL AND IS HEREBY APPROVED FOR CONSTRUCTION BY THE CITY ENGINEER.

APPROVAL EXPIRES 1 YEAR FROM APPROVAL DATE BELOW. THIS APPROVAL SHALL NOT BE CONSTRUED AS CREATING A PRESUMPTION OF CORRECT OPERATION OR AS WARRANTING BY CITY ENGINEER THAT THE APPROVED FACILITIES WILL REACH THE DESIRED GOALS.

REVISION		
ITEM NO	DESCRIPTION of CHANGE	APPROVAL DATE

TENNESSEE VALLEY AUTHORITY
FOSSIL POWER GROUP
ENGINEERING DESIGN SERVICES

MGH JGA HLP DLL

SEWER BASIN PI–2

SHEET 1 OF 4 – TVA DWG. NO. 10W208-4

DIVISION OF ENGINEERING

PRESIDENT'S ISLAND INTERCEPTOR

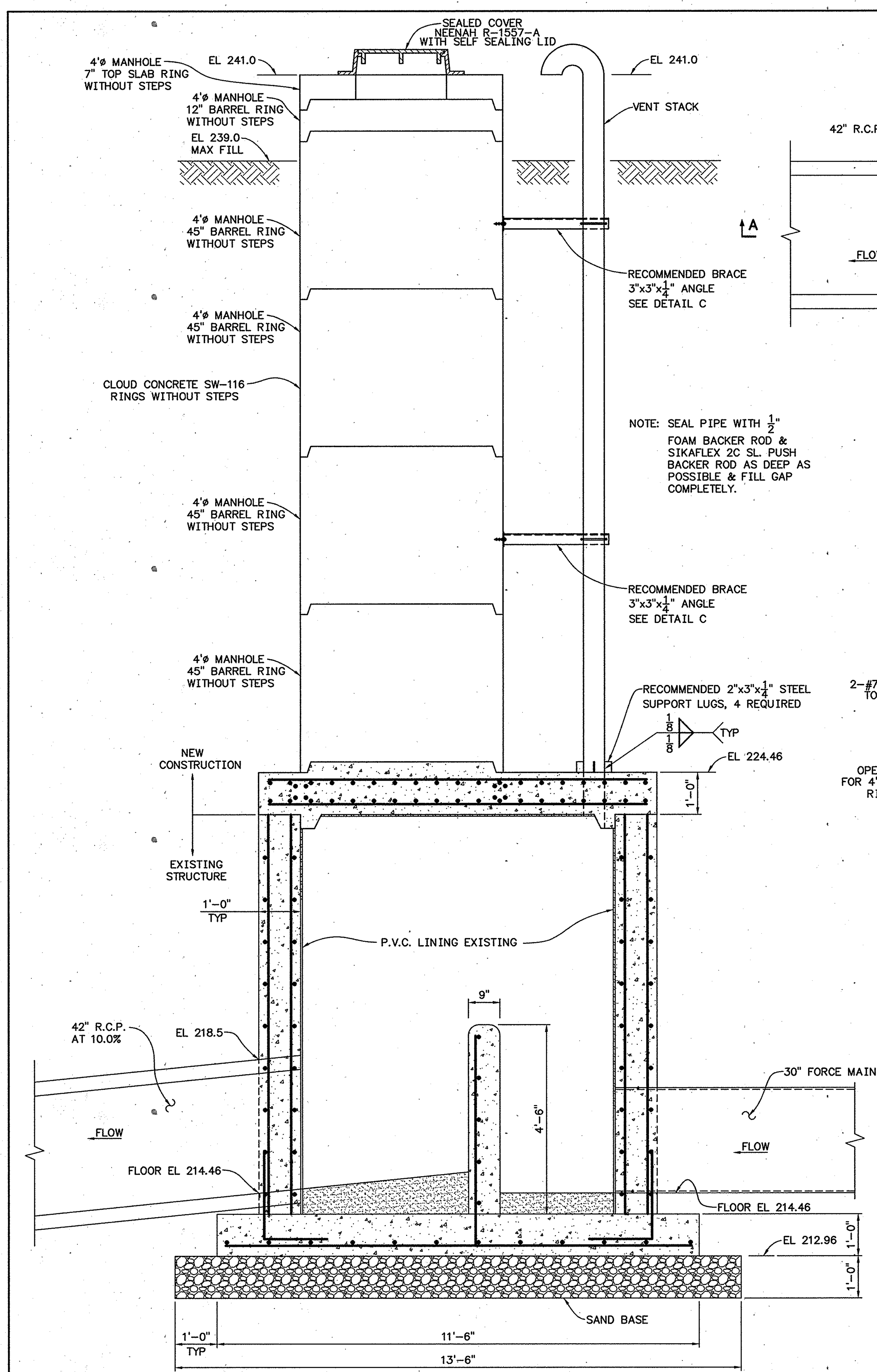
LOCATION: 1381' NORTH & 413' WEST OF INTERSECTION OF PLANT ROAD & RIVERPORT ROAD

TVA ASH POND CROSSING
PLAN
MEMPHIS, TN

SURVEY ____ DATE ____ BOOK ____
DESIGN BY TVA DATE 10/04 SCALE NOT TO SCALE
REVIEWED

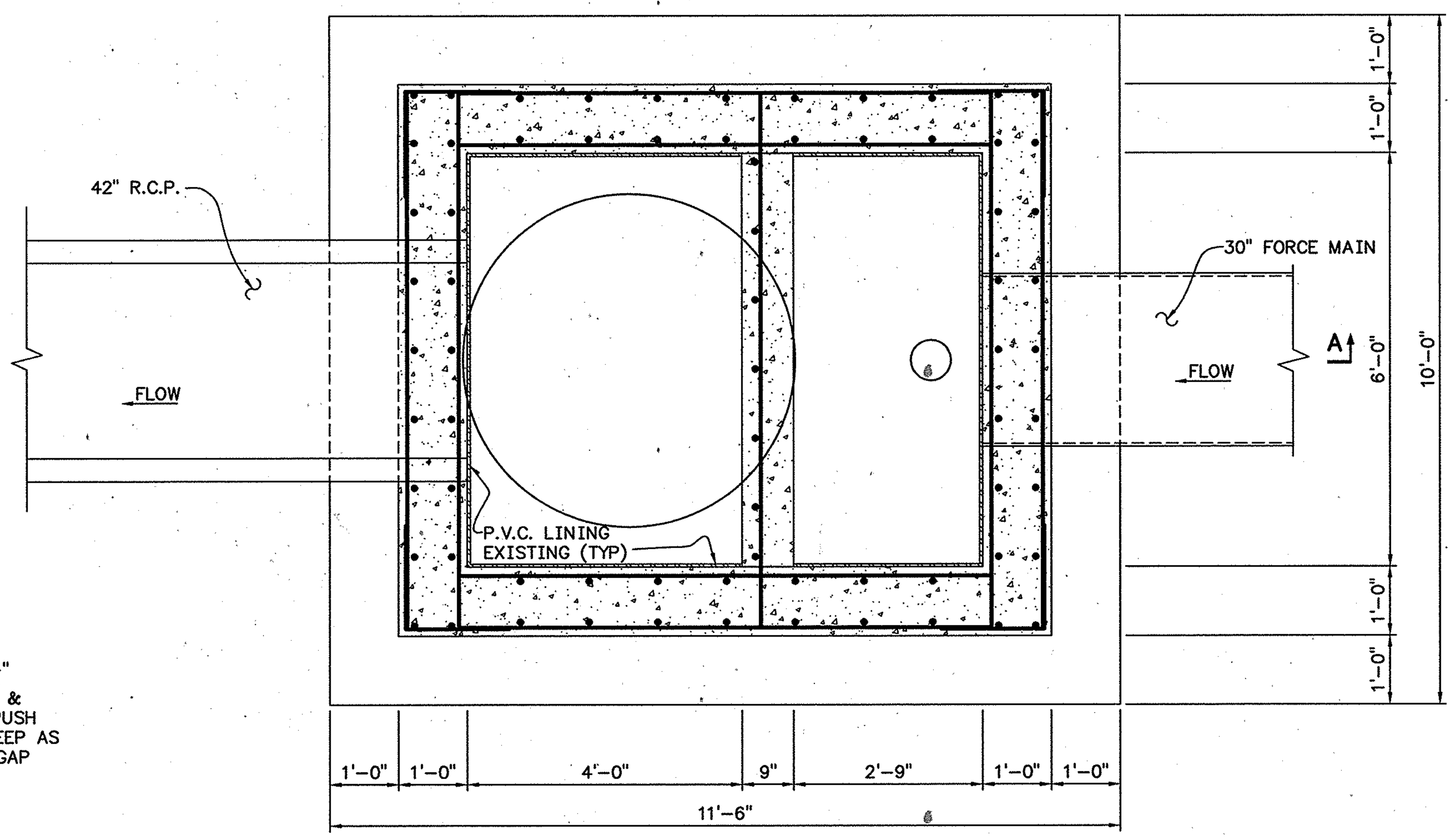
DEPUTY CITY ENGINEER DATE CITY ENGINEER DATE

RESERVED FOR G I S INFORMATION



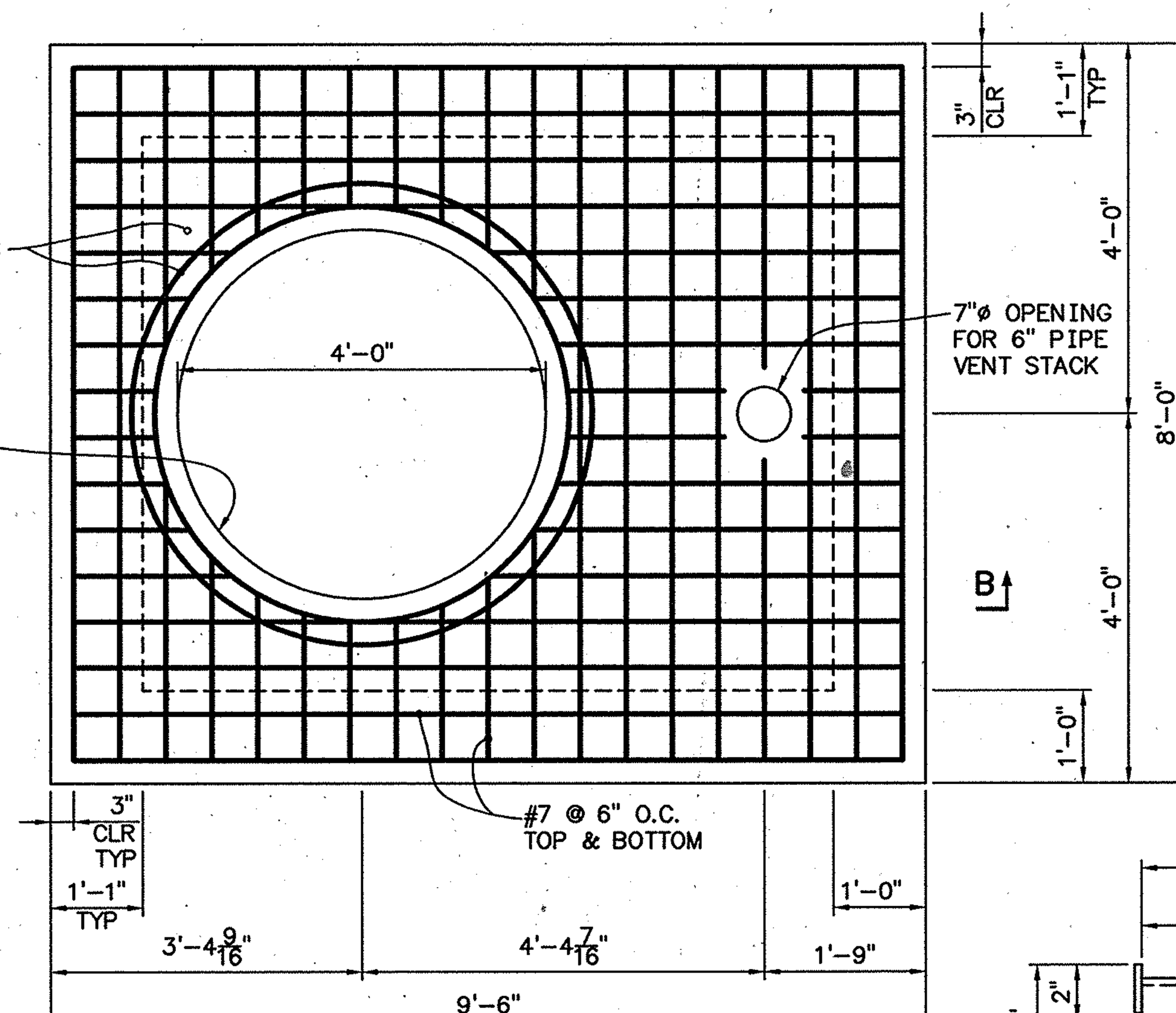
SECTION A-A

NOTE: SEAL PIPE WITH 1/2" FOAM BACKER ROD & SIKAFLEX 2C SL. PUSH BACKER ROD AS DEEP AS POSSIBLE & FILL GAP COMPLETELY.

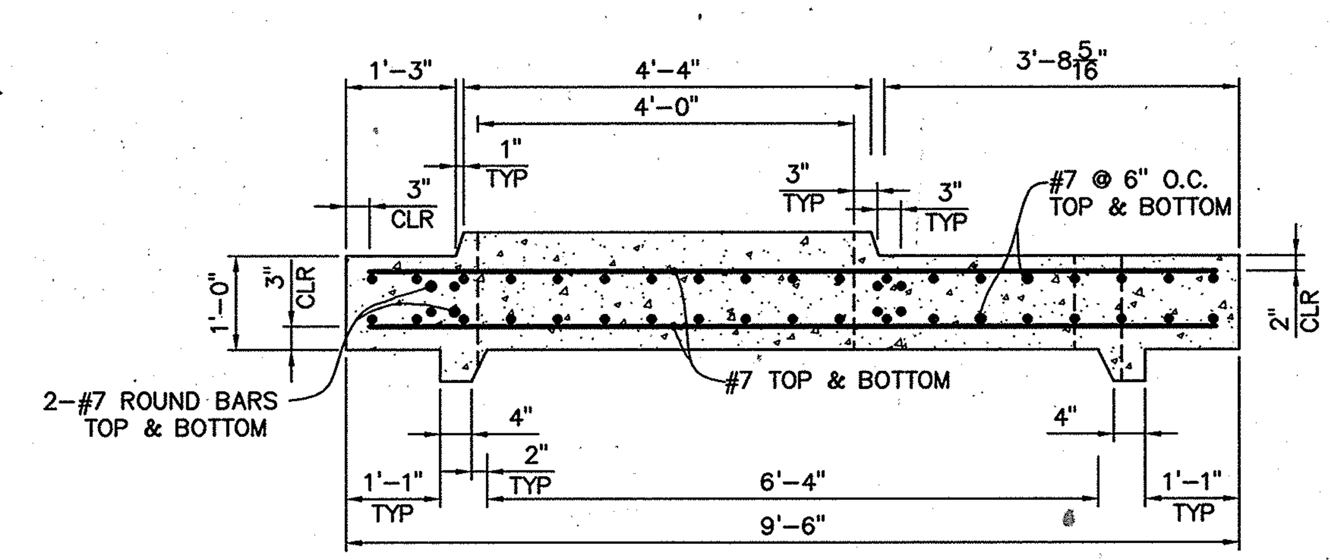


PLAN

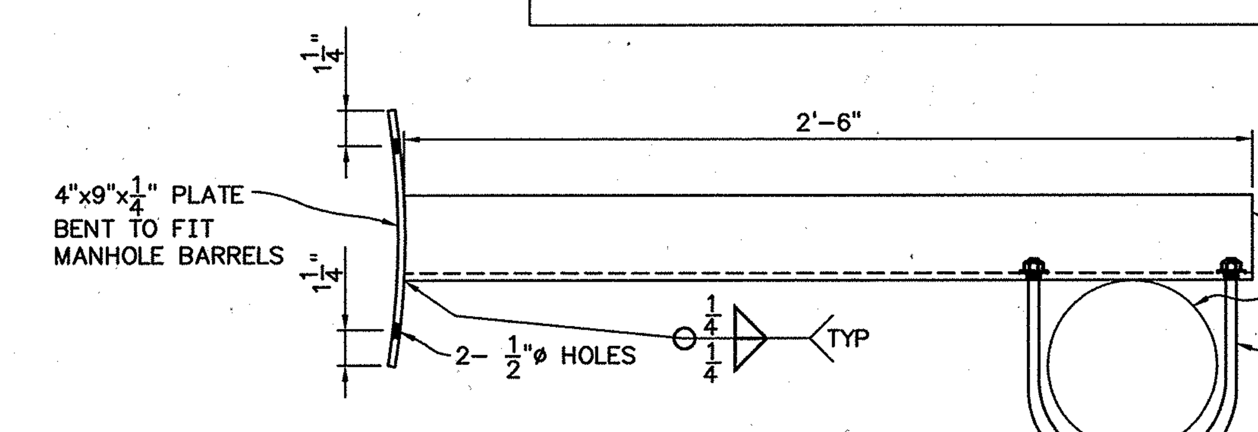
ITEM NO.	QTY	MATERIAL DESCRIPTION REBAR LIST	TOTAL BAR LENGTH
1	20	#7 STRAIGHT BARS – 7'-6" LG	150'-0"
2	4	#7 STRAIGHT BARS – 3'-2" LG	12'-8"
4	4	#7 STRAIGHT BARS – 2'-4" LG	9'-4"
5	4	#7 STRAIGHT BARS – 1'-10" LG	7'-4"
6	4	#7 STRAIGHT BARS – 1'-7" LG	6'-4"
7	8	#7 STRAIGHT BARS – 1'-6" LG	12'-0"
8	4	#7 STRAIGHT BARS – 1'-8" LG	6'-8"
9	4	#7 STRAIGHT BARS – 2'-0" LG	8'-0"
10	4	#7 STRAIGHT BARS – 2'-9" LG	11'-0"
11	16	#7 STRAIGHT BARS – 9'-0" LG	144'-0"
12	4	#7 STRAIGHT BARS – 1'-11" LG	7'-8"
13	4	#7 STRAIGHT BARS – 1'-4" LG	5'-4"
14	4	#7 STRAIGHT BARS – 1'-0" LG	4'-0"
15	4	#7 STRAIGHT BARS – 0'-11" LG	3'-8"
16	4	#7 STRAIGHT BARS – 4'-8" LG	18'-8"
17	4	#7 STRAIGHT BARS – 4'-1" LG	16'-4"
18	4	#7 STRAIGHT BARS – 3'-9" LG	15'-0"
19	4	#7 STRAIGHT BARS – 1'-9" LG	7'-0"
20	4	#7 STRAIGHT BARS – 1'-1" LG	4'-4"
21	2	#7 ROUND BARS – 4'-6"ø + 2'-6" = 16'-8" LG	33'-4"
22	2	#7 ROUND BARS – 5'-0"ø + 2'-6" = 18'-4" LG	36'-4"
		NOTE: ITEMS 21 & 22 MUST HAVE A 2'-6" OVERLAP	



TOP COVER FOR BOX



SECTION B-B



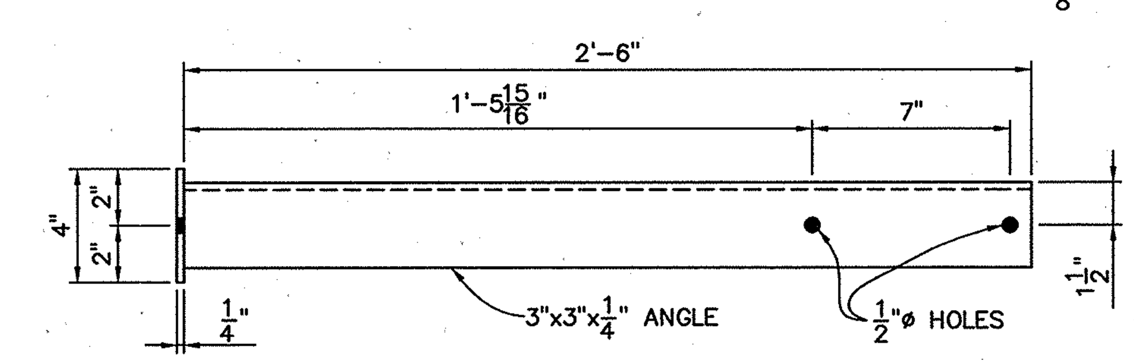
DETAIL C
NTS
2 RECOMMENDED

FASTEN TO MANHOLE BARREL WITH 3/8" HILTI QUIK BOLT II WITH 2 1/2" EMBEDMENT

FASTEN TO VENT STACK WITH 3/8"x7"ø U-BOLT

HAROLD L. PETTY
REGISTERED ENGINEER
No. 015528
STATE OF TENNESSEE
11/19/04

NOTES:
1. CONCRETE AND STEEL DESIGN AND CONSTRUCTION SHALL MEET ACI CODE.
2. CONCRETE SHALL MEET A 28 DAY COMPRESSION STRENGTH OF AT LEAST 4500 PSI.



ELEVATION VIEW
NTS

APPROVED FOR CONSTRUCTION

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REVISION		
ITEM NO.	DESCRIPTION of CHANGE	APPROVAL DATE

SEWER BASIN PI-2
SHEET 3 OF 4 TVA DWG # 10W208-6
DIVISION OF ENGINEERING

PRESIDENT'S ISLAND INTERCEPTOR
LOCATION: 1381' NORTH & 413' WEST OF INTERSECTION OF PLANT ROAD & RIVERPORT ROAD
TVA ASH POND CROSSING
DETAILS & SECTIONS
MEMPHIS, TN

TENNESSEE VALLEY AUTHORITY
FOSSIL POWER GROUP
ENGINEERING DESIGN SERVICES
CWH JGA HLP DLL

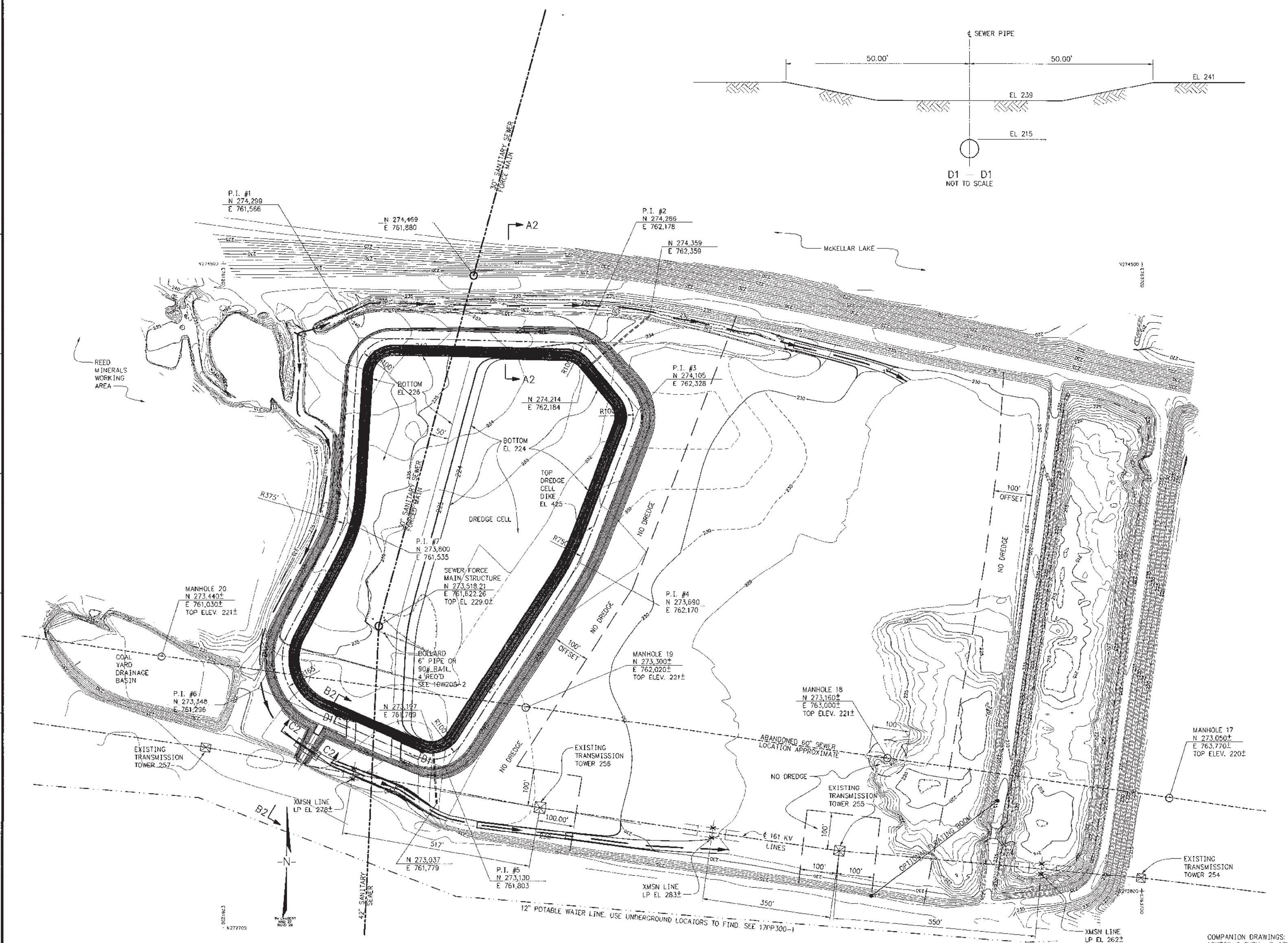
SURVEY ____ DATE ____ BOOK ____
DESIGN BY TVA DATE 11/04 SCALE ~~3/4"=1'-0"~~ N.T.S.
REVIEWED
11/23/04
DEPUTY CITY ENGINEER DATE CITY ENGINEER DATE

RESERVED FOR G I S INFORMATION

0-(-4) 868

PI-2
S/D
2387 B

| PI-2 | S/D | 2387B |



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   FOR Q VALUES F.S. = 2.00
   FOR RBAR VALUES F.S. = 1.51

SOIL	UNIT WEIGHT (pcf)		Q		RBAR	
	MOIST	SATURATED	c (psf)	ϕ (deg.)	c (psf)	ϕ (deg.)
EXIST. DIKE & INSITU	125	125	1000	35	1000	35
EXIST. ASH	100	100	150	25	150	25
DREDGED ASH	100	100	150	25	150	25
ASH DIKE FILL	102.8	102.8	250	28	0	30
EXIST. ASH UNDER DIKE	101	101	165	27	165	27

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ADDITIONAL NOTES

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3. MINIMUM CLEARANCE FROM EQUIPMENT TO 161KV TRANSMISSION LINE CONDUCTORS IS 14 FEET. MINIMUM CLEARANCE FROM 161KV CONDUCTORS TO GROUND IS 26 FEET.

R 0 | OCT 07 2005 | JCA | DRD | HLP | SRC | LJP | DLL | ALFSHORT

INITIAL ISSUE PER DCA ALF-04-1001-001-R0

SCALE: 1" =100' EXCEPT AS NOTED

YARD

EAST ASH POND DREDGE CELL PLAN

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.G. ALBRIGHT	D.R. DODSON	H.L. PETTY	S.R. CATHEY	L.J. PETERSON	D.L. LUNDY	

ALLEN FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

COMPANION DRAWINGS: 10W208-2 THRU -8, 10W225

AUTOCAD R14 | 38 | C | 10W208-1 | R 0

PLOT FACTOR:100
W_TVA

C.A.D. DRAWING DO NOT ALTER MANUALLY

TASK COMPLETED BY: REV NO.

38 C 10W208-1

CITY OF MEMPHIS INTER-OFFICE MEMORANDUM

TO: Mr. John Leonard

DATE: March 17, 1986

FROM: Don Tennyson

SUBJECT: Inspection of the Abandoned Nonconnah Interceptor

The attached is our findings of the inspection made to the abandoned Nonconnah Interceptor. Our survey will start in the box structure at the T. E. Maxon Treatment Plant and work upstream to the siphon crossing Nonconnah Creek west of Highway 51.

There is a total of 16,173 feet of 60" concrete sewer line still in the ground, and 6,610 feet having been removed. The line has sludge deposited in the bottom and will need extensive cleaning all the way through the pipe. Quite a few Sections have the joints leaking with ground water and will need to have joints wiped. Each Section that is located beneath a lake or close to a slough has this problem and would need to be addressed before usage.

There are two (2) box Structures and 17 manholes found in this Section, a number of manholes could not be physically inspected due to water surrounding some of the manholes or not being able to remove the covers. There are a number of manholes on the 6,610 foot Section that Engineering would have to look at installing.

The next Section is 48" concrete sewer from the turning manhole at the 60" west of Riverport Road to the west side of Horn Lake Road. In this Section of 48" pipe there is approximately 4,140 feet of pipe in the ground and 2,470 feet out of the ground. All of the areas inspected a build up of sludge was found in the manholes and would require cleaning of the lines. The plans show a total of ten (10) manholes in this Section we were able to locate and inspect two (2) of them. The plans also show one (1) siphon crossing a small creek, again we could not locate the manholes or siphon.

The next Section inspected consists of 42" concrete pipe and starts on the west side of Old Horn Lake Road east to the siphon crossing Nonconnah Creek on the west side of Highway 51. There is approximately 8,465 feet of pipe that was inspected of this approximately 450 feet of pipe has been washed out by Nonconnah Creek or contributor creeks into Nonconnah Creek. There is approximately 2,250 feet of 42" concrete pipe believed to have been removed. Six (6) manholes also are believed to have been removed. Of the remaining 24 manholes a majority of them are damaged and would require total rebuilding. See attached manhole sheets for specific manholes in need of repairs.

Sincerely,

Don Tennyson

Don Tennyson,
Supervisor

DT/v

SECTION I

Gate Structure at South Treatment Plant and Manhole Number 23.

We were unable to physically inspect this Section. The entrance of this line into the box structure is in the N.E. corner of the structure and Manhole Number 23 is under the R.R. Spur. This Section is made up of 60" concrete pipe and is approximately 700' in length. We also found that T.V.A. enters the Sewer System in this manhole.

SECTION II

Manhole Number 23 east to Manhole Number 22.

This Section was found to be approximately 800' in length. The diameter of the pipe was found to be 60", we physically walked this Section approximately 200' from Manhole Number 22 downstream. There was no apparent leakage of the joints of pipe. However there was some H_2S damage to the crown of the pipe. The aggregate was showing and loose when touched by the hand, aggregate would fall off of the pipe.

SECTION III

Manhole Number 22 east to Manhole Number 21.

This Section is approximately 850' long and consists of 60" concrete pipe. This Section was physically inspected from Manhole Number 22 upstream approximately 200'. This Section also showed no sign of the joints leaking, however there were signs of H_2S damage to the crown of the pipe. The aggregate was showing and would fall off at the touch of your hand. Manhole Number 21 was not located during inspections.

SECTION IV

Manhole Number 21 east to Manhole Number 20.

This Section is approximately 1000' in length. We were unable to visually inspect this Section. Manhole Number 21 could not be located and Manhole Number 20 is under T.V.A's ash pond.

SECTION V

Manhole Number 20 east to Manhole Number 19.

This Section is approximately 1000' in length and is located under T.V.A's ash pond. This Section was not accessible for inspection.

SECTION VI

Manhole Number 19 east to Manhole Number 18.

This Section is approximately 1000' in length and is also located under T.V.A's ash pond and was not accessible for inspection.

SECTION VII

Manhole Number 18 east to Manhole Number 17.

This Section is approximately 812' in length. Manhole Number 18 is located under T.V.A's ash pond and Manhole Number 17 is located east of the levee at T.V.A. We inspected inside Manhole Number 17 downstream and found there was approximately 3" of sludge in the bottom of the 60" concrete pipe. As we walked downstream under T.V.A's ash pond the depth of flow increased along with the depth of sludge. We then shined this Section and noticed water from the ash pond pouring in through the joints of pipe.

We contacted Mr. Frank Yetter of T.V.A and talked to him about Manhole Number 20 having been plugged when the line was abandoned. Mr. Yetter informed me that he had talked to a man that had retired from T.V.A, but was there when the Manhole was plugged. His recollection was that the Manhole was plugged off using only sandbags. Our logic is if this Manhole was completely sealed and the ash pond is steadily leaking into the Sanitary Sewer then this 60" line should be completely full of water. We have been in further contact with T.V.A about lowering the level of the ash pond, but they are unable to do so.

SECTION VIII

Manhole Number 17 east to Manhole Number 16.

This Section is approximately 1088' of 60" concrete sewer pipe. We inspected this Section from Manhole Number 17 upstream approximately 100' and found no problem with the concrete pipe. We were unable to inspect Manhole Number 16. The cover was sealed and we were unable to break the cover loose.

SECTION IX

Manhole Number 16 east to Manhole Number 15.

This Section includes Manhole Number 16 east to Manhole Number 15. Both of these manholes were sealed and we were unable to inspect this Section of concrete pipe. The length of this Section is approximately 1050'.

SECTION X

Manhole Number 15 east to Manhole Number 14.

This Section is approximately 1050' in length and includes Manhole Number 15 east to Manhole Number 14. We were able to inspect Manhole Number 14 downstream approximately 75'. This Section of 60' concrete pipe was found to be in fairly good shape. There was minor H_2S damage to the crown of the pipe along with 3" of sludge in the bottom and approximately 2" of flow. The joints appeared to be in good shape. There was no Apparent leakage.

SECTION XI

Manhole Number 14 east to Manhole Number 13.

This Section is approximately 1000' in length and includes Manhole Number 14 east to Manhole Number 13. Our first inspection was from Manhole Number 14

SECTION XVI

Manhole Number 9 east to Manhole Number 8.

This Section is approximately 850' in length and includes Manholes Numbered 9 east to Number 8. This Section consists of 60" concrete pipe and the manholes were also surrounded by water making physical inspection impossible.

SECTION XVII

Manhole Numberd 8 east to Manhole Number 7.

This Section is approximately 873' in length and includes Manholes Numbered 8 east to Number 7. Section XVII was made up of 60" concrete pipe and the manholes were surrounded by water again making inspection impossible.

SECTION XVIII

Manhole Number 7 east to Box Structure Number 2.

This Section is approximately 950' in length and includes Manhole Number 7 east to the Box Structure Number 2. This Box is located on top of the levee in the Corps of Engineers' Property. The box structure was inspected and found to have two (2) gates inside the structure. One being on the downstream side and having a skew for raising the gate. The upstream side had a gate mounted to the wall. Manhole Number 7 was located in the slough surrounded by water and not accessible for inspection.

There was found to have been 18 Sections of 60" concrete sewer pipe from the levee in the Corp of Engineers to the structure at the South Treatment Plant. There is approximately 16,173 linear feet of 60" concrete pipe. Two (2) concrete box structures and 17 manholes. Of the 17 manholes, four (4) were entered and physically inspected on the downstream and upstream Sections of pipe in the Manholes.

Through discussion with T.V.A, we found that they discharge into manhole Number 23 and enter into the South Plant's influent at the box structure outside the Plant.

There were two (2) Manholes that could not be located, and nine (9) Manholes that are under or surrounded by water. Two (2) Manholes were sealed and not accessible. We know of three (3) Sections that had the joints leaking however we feel that there may be as many as eleven Sections that may have water entering through the joints of pipe.

City of Memphis
TENNESSEE

DR. W. W. HERENTON - Mayor
DAVID F. HANSEN - Chief Administrative Officer
DIVISION OF FINANCE & ADMINISTRATION
RICK MASSON - Director
Purchasing Agent

Wednesday, August 9, 1995

City Contract #N10672

RECEIVED
AUG 17 1995

Insituform Midsouth
3343 Northwest Drive
Knoxville, TN 37921

Gentlemen:

We are enclosing, herewith, an executed copy of a Change Order for Emergency Repair on Sewer Line Change Order #2 shoows an increase of $14,304.00 for the division of Public Works/Environmental Engineering.

This copy is for your files.

Sincerely,

Linzie Thomas

Mr. Linzie Thomas, C.P.M.
Purchasing Agent

cc: City Comptroller
PUBLIC WORKS
City Contract #N10672

Room 354 - 125 North Main Street - Memphis, Tennessee 38103-2083 - (901) 576-6683

Original
File Copy/Copies

CITY OF MEMPHIS

CONTRACT CHANGE ORDER

CONTRACT NO. N10672 CHANGE ORDER NO. #2 DATE OF CONTRACT 12/15/94

Division Public Works Department Environmental Engineering

Contractor Insituform Midsouth

Address 3343 Northwest Drive, Knoxville, TN Zip Code 37921

Project 42 inch at South Plant, Ash Pond to Interceptor

CIP Project No. 602-179316-59547 Other

DESCRIPTION OF CHANGE: (x) Over-run ( ) Under-run

3 additional millimeters of tube thickness required on 894 ft of insitutube for the TVA ash pond to 96 inch interceptor at 16.00 per linear foot

REASON FOR CHANGE:

Emergency repair of 42 inch sewer requiring extra thick insitutube.

Original Contract Price	Latest Revised Contract Price	Addition to Contract	Deduction to Contract	Revised Contract Price
1,000,000.00	1,013,910.00	14,304.00		1,028,214.00

APPROVED BY:

Jerry R. Collins Jr. 7/5/95
BUREAU HEAD DATE

DIVISION DIRECTOR DATE

7/20/95
City Engineer DATE

7/14/95
CITY ATTORNEY DATE

6/30/95
CONTRACTOR D. Thomas DATE

7/25/95
MAYOR DATE

1000.102

F-3040.9 Comptroller Bureau - White & Yellow Copy - Pink Copy

CHECK REQUEST

R-534117
D-534049

PURCHASE ORDER OR CONTRACT NUMBER: N 1 1 0 6 7 2 (2-6)

VENDOR NUMBER: (7-12)

PAYEE NAME: INSITUFORM MIDSOUTH (99-128)

ADDRESS: 3343 NORTHWEST PARK DRIVE (11-40)

KNOXVILLE, TN 37921
CITY (41-70) STATE ZIP

PURPOSE OF EXPENDITURE: Payment on Contract (13-42)

REQUESTOR:
PUBLIC WORKS/ENVIRONMENTAL ENGINEERING

DATE: July 06/ 1995

Invoice No. (7-12)	Invoice Date (13-18)	Due Date (19-24)	Fund (25-27)	Organ/CIP (28-33)	Work Ord (34-38)	Account (39-43)	SER (44)	Gross Amount (45-56)	Discount (57-68)	Net Amount (69-80)
923	06/27/95		602	179316	00000	59547		227,942.00		227,942.00
924	06/27/95		602	179316	00000	59547		12,752.00		12,752.00
					00000					
					00000					
					00000					
					00000					
					00000					
					00000					
					00000					
					00000					
						TOTALS		240,694.00 (71-82)	(83-94)	240,694.00 (95-106)

TO BE COMPLETED BY COMPTROLLER

MANUAL CHECK PREPARED (108)	Yes	No	IF MANUAL CHECK NOT PREPARED, OVERRIDE WHEN	Yes	No	
NUMBER (109-114)	DATE (115-120)		INV. AMT. GREATER THAN CONTRACT/P.O. AMT.	(121)		AUTHORIZED ______
			INSUFF. FUNDS FOR EXCESS OVER CONTRACT/P.O. AMT.	(122)		DATE ______

INSITUFORM MIDSOUTH
3343 Northwest Park Drive
Knoxville, Tennessee 37921
(615) 947-4211

I N V O I C E

No.	924
Date:	June 27, 1995
Contract No.	N10672
Job No.	274 506
Terms:	Net 30

Sold To: City of Memphis
ATTN: Jerry Collins
125 N. Mid America Mall, Room 620
Memphis, Tennessee 38103

DATE	DESCRIPTION	MANHOLE #	PRICE	UNIT	AMOUNT
	Steam Plant Rd.				
06/20/95	Insituform (P.E.) 797 LF x 42 x 21mm	2-3	$16.00	LF	$12,752.00 *
			TOTAL		$12,752.00

* Difference between 42 x 18mm (contract) and 42 x 21mm (installed)

CONTRACT BALANCE: $315,423.75

PLEASE REMIT TO: INSITUFORM MIDSOUTH
3343 Northwest Park Drive
Knoxville, Tennessee 37921

THANK YOU ! ! !

INSITUFORM MIDSOUTH
3343 Northwest Park Drive
Knoxville, Tennessee 37921
(615) 947-4211

I N V O I C E

No. 923

Date: June 27, 1995

Sold To: City of Memphis
ATTN: Jerry Collins
125 N. Mid America Mall, Room 620
Memphis, Tennessee 38103

Contract No. N10672

Job No. 274 506

Terms: Net 30

DATE	DESCRIPTION		MANHOLE #	PRICE	UNIT	AMOUNT
	Steam Plant Rd.					
06/20/95	Insituform (P.E.)	797 LF x 42 x 18mm	2-3	$286.00	LF	$227,942.00
				TOTAL		$227,942.00

CONTRACT BALANCE: $328,175.75

PLEASE REMIT TO: INSITUFORM MIDSOUTH
3343 Northwest Park Drive
Knoxville, Tennessee 37921

THANK YOU ! ! !

F-3040.9 Comptroller Bureau - White & Yellow Copy - Pink Copy

CHECK REQUEST

R-534117
D-534049

PURCHASE ORDER OR CONTRACT NUMBER: N 1 0 6 7 2 (2-6)

VENDOR NUMBER: (7-12)

PAYEE NAME: INSITUFORM MIDSOUTH (99-128)

ADDRESS: 3343 NORTHWEST PARK DRIVE (11-40)

KNOXVILLE, TN 37921

CITY (41-70) STATE ZIP

PURPOSE OF EXPENDITURE: PAYMENT ON CONTRACT (13-42)

REQUESTOR: [signature]

DATE: JULY 18, 1995

PUBLIC WORKS/ENVIRONMENTAL ENGINEERING

Invoice No. (7-12)	Invoice Date (13-18)	Due Date (19-24)	Fund (25-27)	Organ/CIP (28-33)	Work Ord (34-38)	Account (39-43)	SER (44)	Gross Amount (45-56)	Discount (57-68)	Net Amount (69-80)
930	07/10/95		602	179316	00000	59547		1,552.00		1,552.00
					00000					
					00000					
					00000					
					00000					
					00000					
					00000					
					00000					
					00000					
					00000					
						TOTALS		1,552.00 (71-82)	(83-94)	1,552.00 (95-106)

TO BE COMPLETED BY COMPTROLLER

MANUAL CHECK PREPARED (108)	Yes	No	IF MANUAL CHECK NOT PREPARED, OVERRIDE WHEN	Yes	No	AUTHORIZED
NUMBER (109-114)	DATE (115-120)		INV. AMT. GREATER THAN CONTRACT/P.O. AMT.	(121)		DATE
			INSUFF. FUNDS FOR EXCESS OVER CONTRACT/P.O. AMT.	(122)		

INSITUFORM MIDSOUTH
3343 Northwest Park Drive
Knoxville, Tennessee 37921
(615) 947-4211

I N V O I C E

No. 930

Date: July 10, 1995

Contract No. N10672

Job No. 274 506

Terms: Net 30

Sold To: City of Memphis
ATTN: Jerry Collins
125 N. Mid America Mall, Room 620
Memphis, Tennessee 38103

DATE	DESCRIPTION		MANHOLE #	PRICE	UNIT	AMOUNT
	Treatment Plant					
06/28/95	Insituform (P.E.)	97 LF x 42 x 21mm	5-6	$16.00	LF	$1,552.00 *
				TOTAL		$1,552.00

* Difference between 42 x 18mm (contract) and 42 x 21mm (installed)

CONTRACT BALANCE: $247,604.75

PLEASE REMIT TO: INSITUFORM MIDSOUTH
3343 Northwest Park Drive
Knoxville, Tennessee 37921

THANK YOU ! ! !

F-3040.9 Comptroller Bureau - White & Yellow Copy - Pink Copy

CHECK REQUEST

R-534117
D-534049

PURCHASE ORDER OR CONTRACT NUMBER	N 1 0 6 7 2	VENDOR NUMBER	
	(2-6)		(7-12)

PAYEE NAME: INSITUFORM MIDSOUTH
(99-128)

ADDRESS: 3343 NORTHWEST PARK DRIVE
(11-40)

KNOXVILLE, TN 37921
CITY (41-70) STATE ZIP

PURPOSE OF EXPENDITURE: Payment on Contract
(13-42)

REQUESTOR: [signature] DATE: July 13, 1995

Public Works/Environmental Engineering

Invoice No. (7-12)	Invoice Date (13-18)	Due Date (19-24)	Fund (25-27)	Organ/CIP (28-33)	Work Ord (34-38)	Account (39-43)	SER (44)	Gross Amount (45-56)	Discount (57-68)	Net Amount (69-80)
929	07/10/95		602	179316 ~~178216~~	00000	59547 ~~595476~~		27,742.00		27,742.00
					00000					
					00000					
					00000					
					00000					
					00000					
					00000					
					00000					
					00000					
					00000					
						TOTALS		27,742.00		27,742.00
								(71-82)	(83-94)	(95-106)

TO BE COMPLETED BY COMPTROLLER

MANUAL CHECK PREPARED	(108) Yes	No	IF MANUAL CHECK NOT PREPARED, OVERRIDE WHEN	Yes	No	
NUMBER	DATE		INV. AMT. GREATER THAN CONTRACT/P.O. AMT.	(121)		AUTHORIZED
(109-114)	(115-120)		INSUFF. FUNDS FOR EXCESS OVER CONTRACT/P.O. AMT.	(122)		DATE

INSITUFORM MIDSOUTH
3343 Northwest Park Drive
Knoxville, Tennessee 37921
(615) 947-4211

I N V O I C E

No. 929

Date: July 10, 1995

Contract No. N10672

Job No. 274 506

Terms: Net 30

Sold To: City of Memphis
ATTN: Jerry Collins
125 N. Mid America Mall, Room 620
Memphis, Tennessee 38103

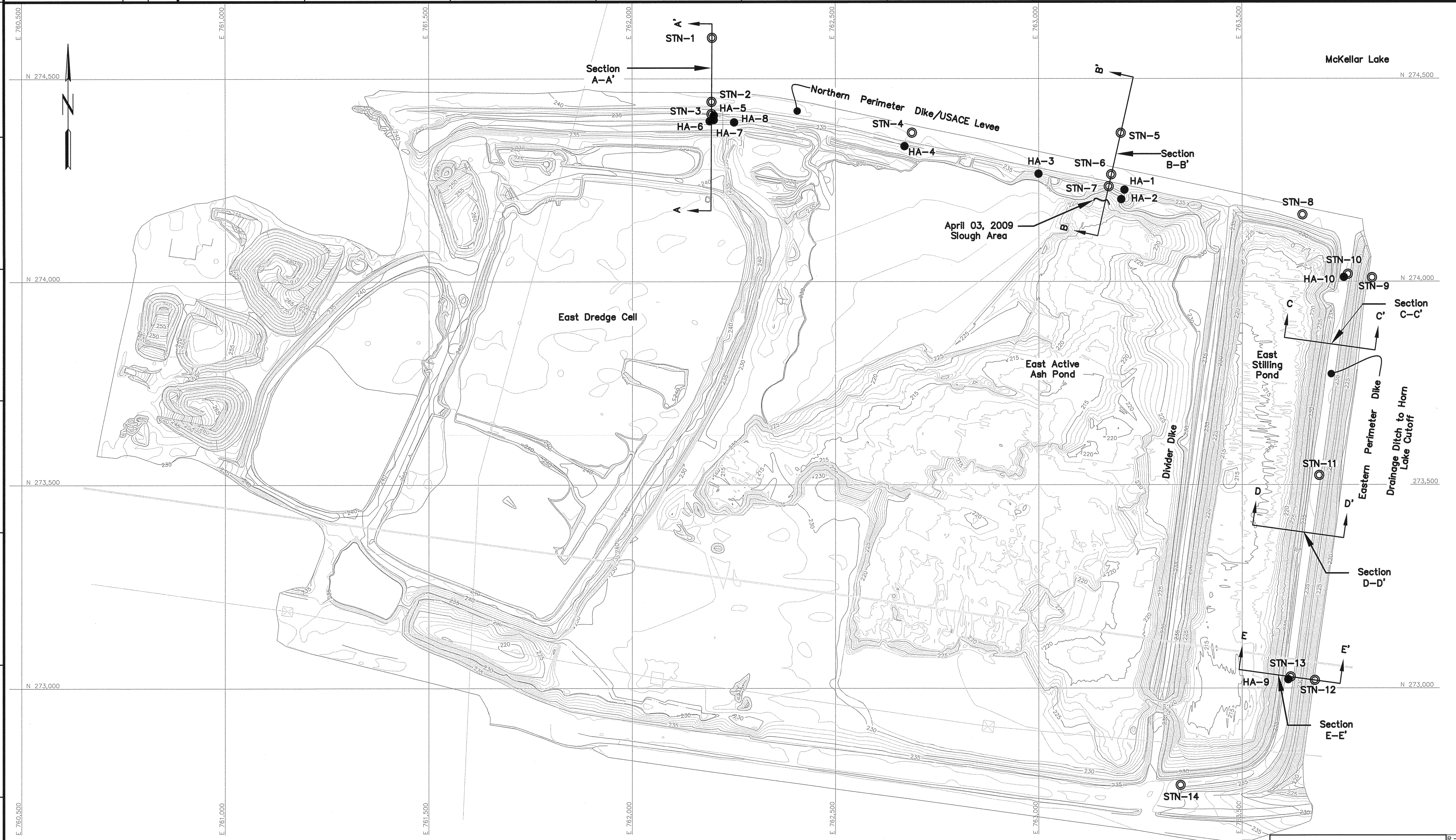
DATE	DESCRIPTION		MANHOLE #	PRICE	UNIT	AMOUNT
	Treatment Plant					
06/28/95	Insituform (P.E.)	97 LF x 42 x 18mm	5-6	$286.00	LF	$27,742.00
					TOTAL	$27,742.00

CONTRACT BALANCE: $249,156.75

PLEASE REMIT TO: INSITUFORM MIDSOUTH
3343 Northwest Park Drive
Knoxville, Tennessee 37921

THANK YOU ! ! !

APPENDIX C
SUDDEN DRAWDOWN ASSESSMENT



LEGEND

◎ Soil Boring with Undisturbed (Shelby) Tube Samples and/or Standard Penetration Tests (Piezometers Installed)

● Hand Auger Boring

NOTES:

1. The topographic mapping presented on this drawing was provided to Stantec by TVA Surveying and Project Services. This plan view was prepared to support development of the geotechnical exploration program and should not be used for construction.
2. The geotechnical information and data furnished herein are not intended as representation or warranties but are furnished for information only. It shall be distinctly understood that the Owner or Engineer will not be responsible for any deduction, interpretation or conclusion drawn therefrom. The information is made available in order that the Contractor may have ready access to the same information available to the Owner and the Engineer and is not part of this contract.

BORING LOCATION TABLE

BORING	NORTHING	EASTING	ELEV. (FT.)
STN-1	274,600.73	762,196.97	215.3
STN-2	274,443.30	762,195.73	238.8
STN-3	274,413.31	762,195.54	234.6
STN-4	274,366.97	762,687.88	237.5
STN-5	274,366.34	763,200.60	217.5
STN-6	274,264.03	763,177.15	238.3
STN-7	274,234.70	763,171.06	235.6
STN-8	274,165.16	763,646.95	237.5
STN-9	274,009.16	763,820.47	221.2
STN-10	274,018.24	763,758.37	236.9
STN-11	273,523.29	763,688.16	237.8
STN-12	273,018.83	763,676.83	216.7
STN-13	273,020.21	763,618.05	236.9
STN-14	272,761.34	763,347.91	236.5

HAND AUGER BORING LOCATION TABLE

BORING	NORTHING	EASTING	ELEV. (FT.)
HA-1	274,226.34	763,209.32	235.7
HA-2	274,202.98	763,201.48	230.5
HA-3	274,265.91	762,999.17	234.2
HA-4	274,333.93	762,669.73	233.6
HA-5	274,409.74	762,201.44	233.7
HA-6	274,395.67	762,191.04	232.5
HA-7	274,398.58	762,201.29	232.4
HA-8	274,392.70	762,251.17	232.1
HA-9	273,021.67	763,612.37	237.0
HA-10	274,010.59	763,748.55	237.2

RECORD DRAWING

For Supporting Design Calculations See FPGALFFESCDX00000020100002

PATRICK V. KISER REGISTERED ENGINEER AGRICULTURE COMMERCE No. 111395 STATE OF TENNESSEE
3/25/2010

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV CD
R –	–	–	–	–	–	–	–	–	–	–	–
R 0	03/25/10	PVK	PS	SF	HRA	HRA	PVK	TJ	–	–	–

RECORD DRAWING

SCALE: 1"=100' EXCEPT AS NOTED

DISCIPLINE INTERFACE 1 2 3 4

YARD
–

GEOTECHNICAL EXPLORATION USACE LEVEE – EAST ASH POND BORING LAYOUT

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
P. KISER	P. SILPACHARN	S. FIELD	H. APARICIO	H. APARICIO	P. KISER	T. JOHNSON

Stantec Consulting Services Inc.
100 Westwood Place Suite 420
Brentwood, Tennessee
37027-5044
Tel. 615.885.1144
Fax 615.885.1102
www.stantec.com

ALLEN FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | DATE 03/25/10 | 38 | C | 10W504-01 | R 0

STANTEC | 0
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:XX
TJ

C.A.D. DRAWING
DO NOT ALTER MANUALLY

PLOT DATE: 03/25/2010

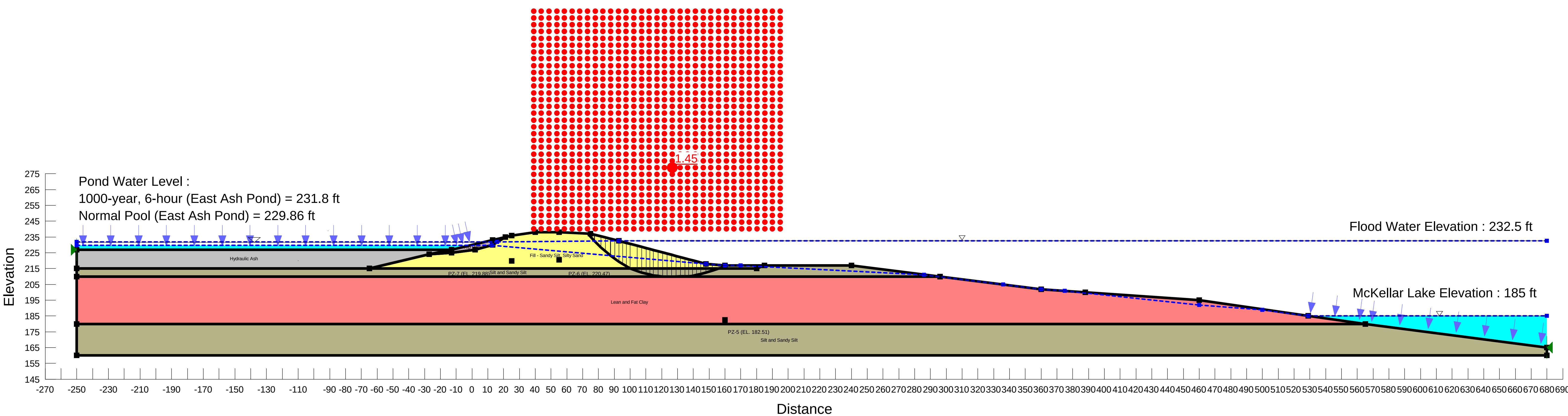
Tennessee Valley Authority
Allen Fossil Plant East Ash Disposal Area
Memphis, Tennessee
Section B-B'

Sudden Drawdown

Existing Geometry;
Rapid Drawdown
Short-Term;
Effective Stress Analysis;
Drained Strengths

Note:
The results of this analysis are based on available subsurface information, field and laboratory test results and approximate soil properties. The drawing depicts approximate subsurface conditions based on historical drawings or specific borings at the time of drilling. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Material Type	Sat. Unit Wt.	Effective - c'	Effective - phi'	Total - c	Total - phi
Fill - Sandy Silt, Silty Sand	125 pcf	0 psf	31 °	200 psf	22 °
Rip Rap	140 pcf	0 psf	38 °	0 psf	38 °
Silt and Sandy Silt	115 pcf	0 psf	28 °	200 psf	12 °
Hydraulic Ash	105 pcf	0 psf	25 °	0 psf	10 °
Lean and Fat Clay	115 pcf	0 psf	26 °	400 psf	12 °
Sandy Silt	125 pcf	0 psf	30 °	200 psf	22 °



Project No. 172675014

12/10/2015 9:35:28 AM