TX08004 - AQUILLA DAM
BRAZOS RIVER BASIN
AQUILLA CREEK, TEXAS

Water Supply Re-Allocation Feasibility Study

Appendix J – Geotechnical Engineering

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GEOTECHNICAL EVALUATION OF SITE CONDITIONS
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I - Executive Summary

1.1 Purpose and Scope

To support Water Supply in the Brazos River Basin, USACE is coordinating with the Brazos River Authority (BRA) to study potential re-allocation alternatives for Aquilla Dam and Reservoir, per the Cost Share Agreement signed December 2005. Phase I of the study identified Aquilla Reservoir for producing the yield required for projected development of the basin, and the Feasibility Scoping Document, dated November 2008, directed the preparation of a feasibility study on the impacts of a pool re-allocation. This Appendix evaluates the dam safety and geotechnical impacts of re-allocation with respect to the performance and risks associated with project operations.

1.2 Background Information

Aquilla Dam is a high hazard potential dam located on Aquilla Creek, 23.3 river miles upstream from its confluence with the Brazos River in Hill County, Texas, about 7 miles southwest of the city of Hillsboro, Texas and approximately 7 miles southeast of the town of Whitney, Texas. Major structures at the project consist of a rolled fill earthen embankment, an Outlet Works (OW) gated conduit, and an uncontrolled trapezoidal broad crested weir Spillway (SW) as shown in Figure I-1.

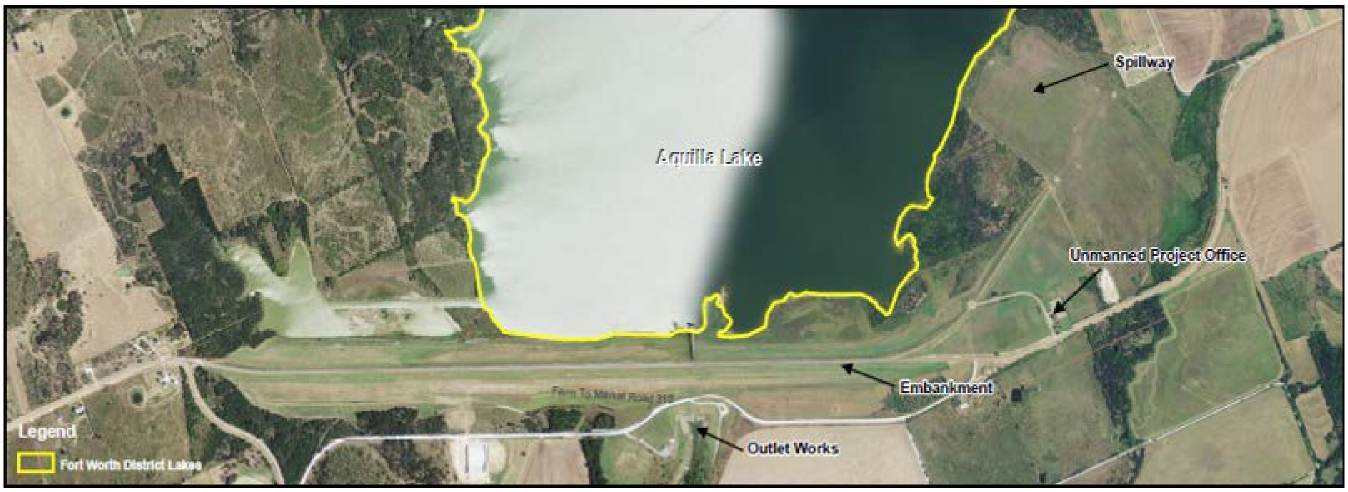

Figure I-1: Aquilla Dam Project Overview

1.3 Pertinent Data

<u>Dam (Embankment)</u>

Type: Earth fill

Length (exclusive of spillway): 11,890 feet

Maximum height above streambed: 104.5 feet

Crest width: 38 feet

Roadway width: 38 feet

Top of dam elevation: 582.5 feet NGVD

<u>Outlet Works</u>

Location: Embankment Station 65+35

Type: Gate controlled conduit

Size: 10-foot diameter

Control: Two 4.5-foot wide by 10-foot high sluice-type gates

Low flow control: One 12-inch-diameter pipe

Capacity: 3,000 cubic feet per second (cfs)

Spillway

Location: Left abutment

Type: Trapezoidal broad-crested weir

Width: 1200 feet

Crest elevation: 564.5 feet NGVD

Capacity: 126,800 cfs

Drainage Area

252 square miles

1.4 Risk Management Overview

Aquilla Dam was assessed by a National Risk Cadre for the Screening Portfolio Risk Assessment (SPRA) in July 2005 and was subsequently assigned a Dam Safety Action Classification (DSAC) of 3 (moderate urgency). The Fort Worth District implemented Interim Risk Reduction Measures (IRRMs) to improve project conditions and further evaluate the known Dam Safety concerns. These IRRMs included stockpiling flood-fighting materials, updating the Emergency Action Plan, and conducting emergency preparedness exercises with downstream emergency management agencies. In September 2012, the Fort Worth District requested a re-evaluation of the DSAC rating in the 2005 SPRA report. The request for reconsideration was based on additional studies of the identified potential failure modes (PFMs) and construction completed since the original DSAC assignment. Background information for Aquilla Dam was presented to the Dam Senior Oversight Group (DSOG) in April 2013. The Headquarter Dam Safety Officer (DSO) agreed with the recommendations based on the information presented, and the DSAC 3 was changed to a DSAC 4 (low urgency) in August 2013.

A facilitated Potential Failure Mode Analysis (PFMA) was conducted by the Fort Worth District in November 2014. This PFMA did not take into account incremental risks. Therefore, a DSAC rating was not evaluated.

Periodic Assessment (PA) #1 was conducted in June 2016. The PA consisted of a facilitated Potential Failure Mode Analysis (PFMA), a Periodic Inspection (PI), and a semi-quantitative risk assessment of potential failure modes. The incremental risks associated with Aquilla Dam are considered to be low. Risks from the PA are not driven by pools near conservation, so it is not anticipated the pool increase would change the results; however, the dam should be monitored closely once the pool raise is implemented for any changes in project performance. The PA for Aquilla Dam was presented to the DSOG in February 2016, and recommended maintaining the DSAC 4. Based on a detailed review of all project data, the Headquarter DSO approved the DSAC 4 rating in May 2017.

II - Site Conditions and Proposed Pool Raise Alternatives

2.1 General Geology and Topography of Area and Dam Site

The Aquilla Dam site is located on Aquilla Creek in the Eastern Cross Timber physiographic province. The Eastern Cross Timbers is a narrow north-south trending belt bounded on the west by the Grand Prairie province and on the east by Black Prairie province. Most of the watershed is located in the Eastern Cross Timbers; however, the extreme eastern and western portions include areas designated as the Black and Grand Prairies, respectively.

Generally the area topography reflects the eastward dipping outcrops of the Lower and Upper Cretaceous formations. Unlike the Brazos River, located 9 to 10 miles west of Aquilla Creek, Aquilla Creek has a weakly developed meander pattern which is not deeply incised into bedrock, and is only noticeable downstream with its confluence with Hackberry Creek.

The alluvial deposits comprising the floodplain are a maximum of 37 feet thick. These deposits consist of an impervious clay blanket (CL) with an average thickness of about 16 feet underlain by clayey sand (SC). In some areas the clayey sand is underlain by a basal gravel. The right abutment is mantled by residual and slope wash material on its upper and middle slopes and in its tributary drainages. This material consists of sandy clay (CL) from 3 to 10 feet thick that is occasionally underlain by clayey sand (SC) and sandy, clayey gravel (GC). In contrast with the right abutment, only the upper slopes of the left abutment are mantled by a residual slope wash overburden. Here the overburden consists predominantly of sandy clay (CL) varying from 2 feet to 7.5 feet in thickness. Between approximate embankment stations 57+00 and 93+00 the overburden comprises a stream terrace remnant. Overburden thickness on the left abutment reaches a maximum thickness of approximately 50 feet in the central part of the terrace. Materials comprising the terrace consist of sandy clay (CL) varying from approximately 4 feet thick to approximately 23 feet thick, followed by silty sand (SC and SM), sandy clay (CL), and clayey, sandy gravel (GC). Figures II-1 and II-2 show the geologic cross sections for Aquilla Dam.

2.1.1 Primary Geologic Formations

Bedrock formations affecting construction of Aquilla Dam and operation of Aquilla Lake are all of Cretaceous age. The Primary Formations are as follows:

a. Eagle Ford Shale – The Eagle Ford Shale is present only on the left abutment in the area of the spillway. It is composed of soft calcareous shale with a few persistent, thin limestone beds and a few calcareous, sandstone streaks scattered through the section. Its contact with the underlying Woodbine is at the base of a thin limestone bed.
b. Woodbine Clay Shale – The Woodbine constitutes the primary foundation of the dam and its appurtenant structures. The Woodbine is characteristically a soft, non-calcareous, dark gray to black, montmorillonite-type clay shale. The upper portion of the formation is characterized by a sandstone unit, while the middle and lower portion are clay shale containing a number of variably thick sandy and silty shale units, some of which grade laterally into sandstone and a few thin sandstone beds.
c. Del Rio Shale – The Del Rio Formation consists of soft to moderately hard, calcareous, gray to greenish gray, massively bedded clay shale ranging from 70 to 80 feet thick at the dam site. Scattered, thin stringers of very calcareous shale and argillaceous limestone occur through the entire formation, but they increase in abundance downward through the lower half of the formation.
d. Georgetown Limestone - The Georgetown is comprised of varying argillaceous limestone interbedded with thin beds of limey shale. There was no apparent interruption in the deposition between that of the Georgetown Limestone and the overlying Del Rio Shale, merely a change of materials deposited.

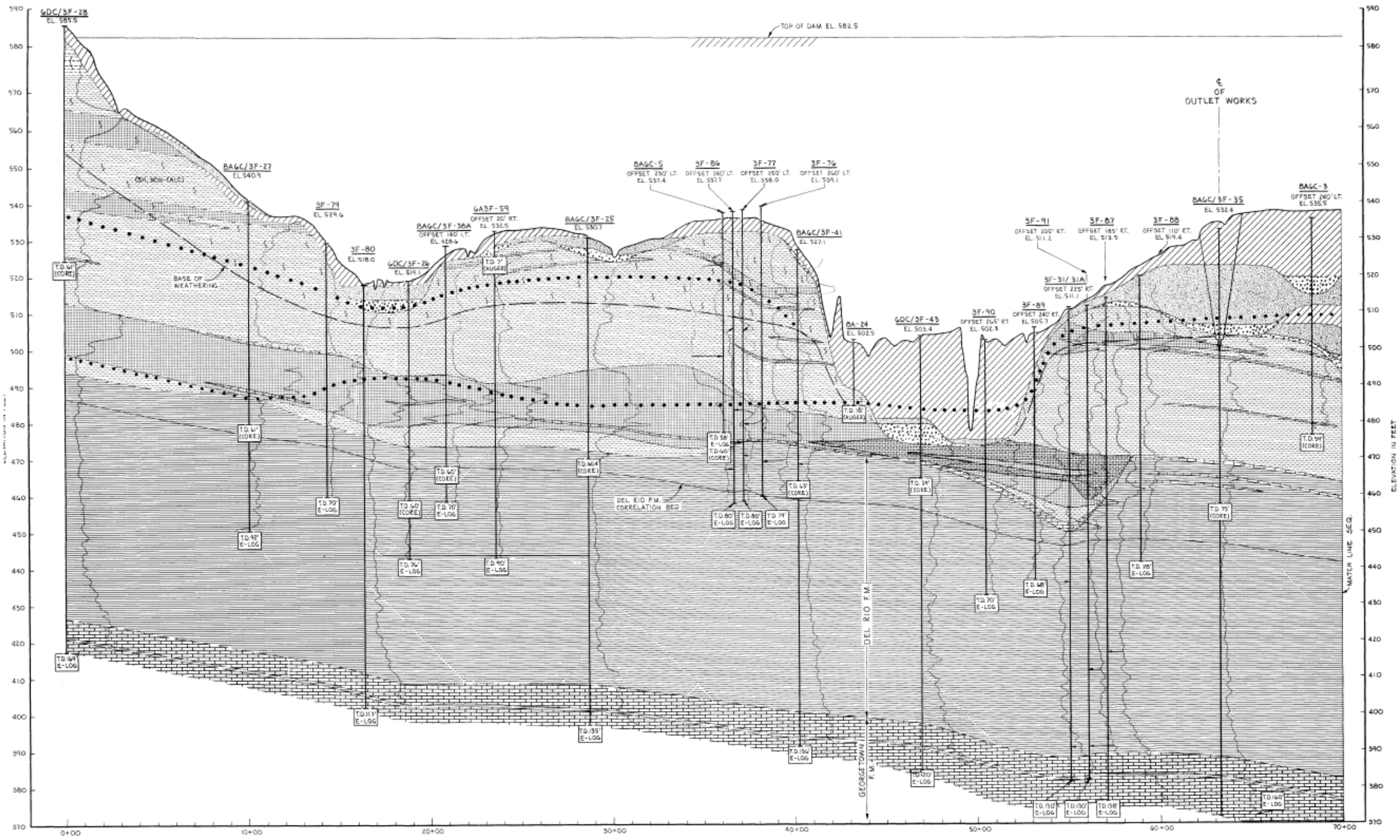

Figure II-1: Embankment Geologic Cross Section Station 0+00 to 70+00

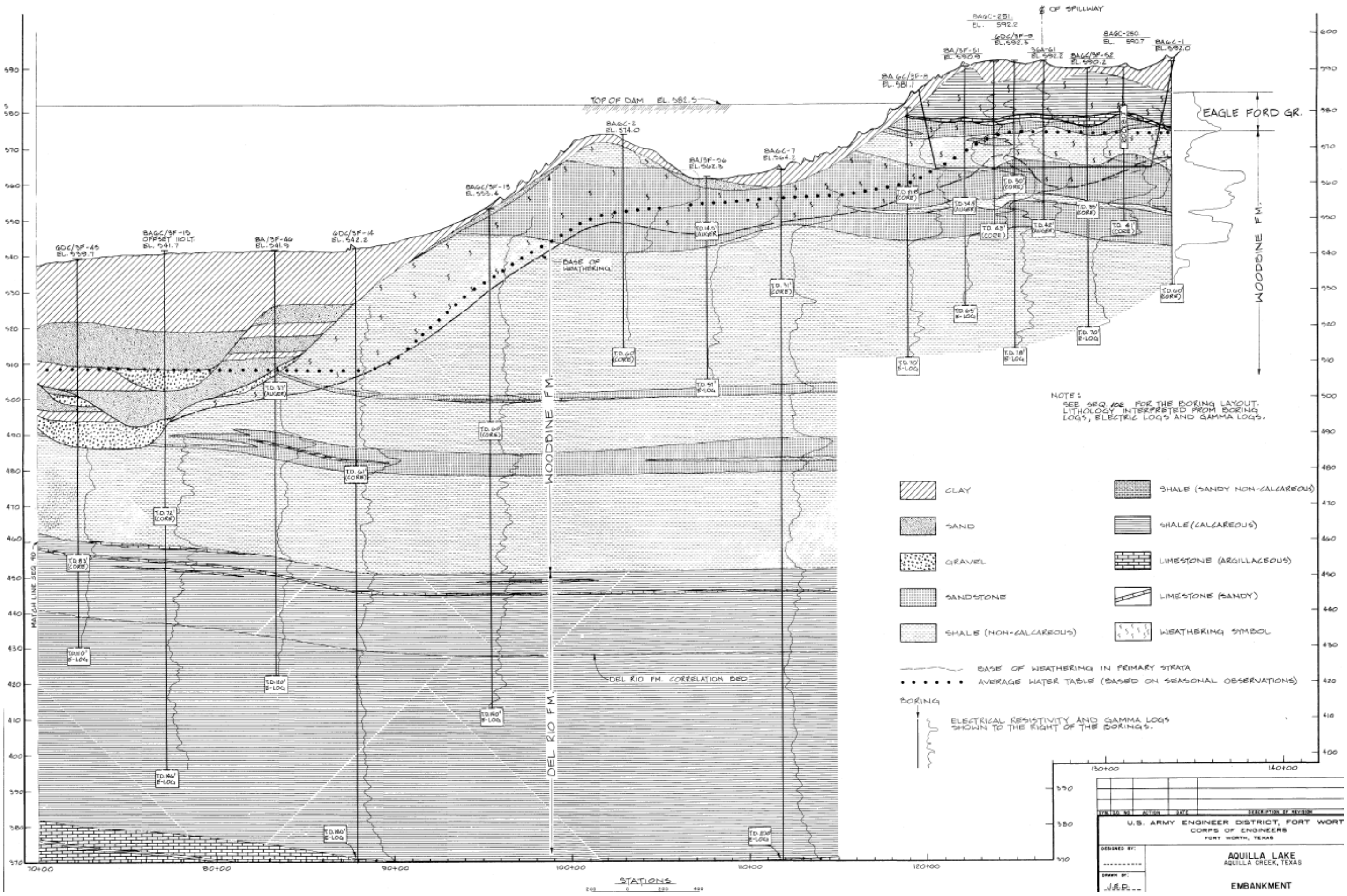

Figure II-2: Embankment Geologic Cross Section Station 70+00 to 134+36

2.2 Dam Embankment Conditions

The earthfill embankment is essentially symmetrical about its centerline and consists of a compacted, central impervious core with compacted random zones adjacent to the core, and semi-compacted berms contiguous to the random zones. A select impervious zone or "cap" was designed at the crest to retard future problems with shallow sliding. The embankment is founded on residual and alluvial overburden overlying the clay shales of the Woodbine and Del Rio formations. Figure II-3 shows a typical cross-section and is discussed below.

a. Impervious Core - Clay material from on-site borrow sources.
b. Compacted Random Fill - Clays and clayey sands from on-site borrow, except highly pervious materials were not acceptable.
c. Semi-Compacted Fill – Excavated, processed materials from required excavations.
d. Clay Cap – constructed of lean clay materials obtained from on-site borrow.

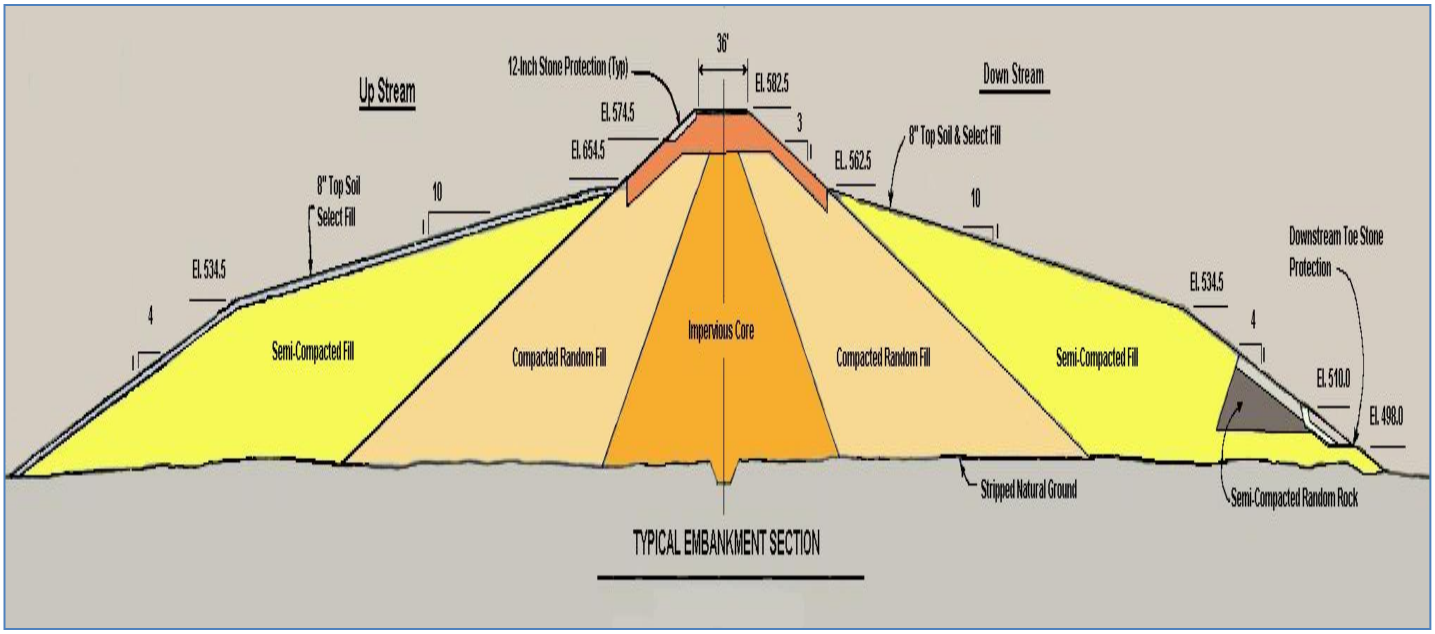

Figure II-3: Typical Zoned Embankment Section from CESWF Design

2.3 Outlet Works Conditions

The outlet works is located 3,134 feet west of the spillway portion of the dam and is founded on unweathered Woodbine clay shale. It crosses the embankment from north to south at embankment Station 65+35. The outlet works consists of an intake tower, a 13-foot wide service bridge, one 10-foot diameter conduit with invert elevation of 503.0 feet NGVD controlled by two 4.5 x 10-foot service gates, and a reinforced concrete stilling basin for release of floodwaters. There is one 12-inch diameter low flow pipe with invert elevation 505.0 feet placed within the central gate pier between the two service gate passages.

2.4 Spillway Conditions

The spillway is located near the left abutment with centerline at embankment Station 126+00. It consists of an uncontrolled trapezoidal broadcrested weir 1,200-foot wide crest width and 3,000 feet long with 4.5H:1V side slopes. The spillway bottom elevation of 564.5 ft NGVD is 18 feet below the crest of the earth dam. A reinforced concrete sill (20 feet long in the direction of flow) is on the weir crest along the embankment centerline and extends up the channel side slopes to elevation 577.5. The upstream and downstream edges of this sill are protected with 25 and 50 feet of 24-inch riprap blanket, respectively. The approach and discharge channels slope downward from the weir on a grade of 0.3 percent to natural ground. The discharge channel empties into a draw which conveys the spillway discharge about 1.6 miles to Cobb Creek. The approach channel is founded on weathered sandstone of the Woodbine. The sill is constructed in Woodbine sandstone, the upper part of which is weathered. The discharge channel is founded on weathered sandstone for a distance of approximately 100 feet downstream from the sill, beyond which it is founded on weathered clay shale of the Woodbine for a distance of 800 to 900 feet and on clay for a distance of 400 to 500 feet.

2.5 Hydrologic & Hydraulic Loading Conditions

The reservoir data was obtained from the Pertinent Data table in the May 1998 Water Control Manual for Aquilla Lake, as shown in Table II-1 below. Deliberate impoundment of the reservoir began in April 1983 and conservation pool elevation of 537.5 feet was first attained on 21 March 1985. The pool of record of elevation 551.9 feet occurred on 23 December 1991.

Table II-1: Summary of H&H Design Conditions

Feature	Elevation (ft, NGVD)	Area (acres)	Capacity (acre-ft)
Top of Dam	582.5		
Maximum Design Water Surface	577.5	14,495	359,900
Spillway Crest	564.5	8,980	213,800
Top of Flood Control Pool	556.0	7,000	146,000
Top of Conservation Pool	537.5	3,280	52,400
Top of Inactive Pool	503.0		25,700
Streambed	478.0		

2.5.1 Hydrologic & Hydraulic Analysis and Evaluation

As part of the reallocation study for Aquilla Lake, the Hydrology and Hydraulics appendix documents a 1939-2009 period of record reservoir system simulation analysis based on current Water Control Plans of Regulation for the Brazos River Basin reservoirs. The objective of the analysis was to provide updated Aquilla Lake pool elevation-frequency, pool elevation-duration, total discharge-frequency, and total discharge-duration curves for existing conditions and each of the three alternatives. The flood frequency analysis developed for this feasibility study for water supply used effective criteria and approved methods for watershed modeling and storm routing, including a joint probability analysis.

For the PA, the objective of the analysis was to determine if the dam meets the essential guidelines of being hydraulically adequate as well as provide hydrologic loading data such as stage-frequency curves, stage-duration curves, and tailwater rating curves to estimate failure likelihoods for potential failure modes.

III - Geotechnical Evaluation of Dam Safety Conditions

3.1 Background

Aquilla Dam was screened by a national risk cadre in July 2005 and subsequently assigned a DSAC of 3 in December 2008.

Aquilla Dam was re-evaluated by an SPRA cadre member and the DSOG in April 2013. This request for re-evaluation was based on additional studies and construction completed since the 2005 SPRA. The request was approved by HQ DSO in August 2013, and the DSAC 3 was changed to a DSAC 4.

Aquilla Dam underwent a facilitated potential failure mode analysis in November 2014 to better define the risks associated with operation of the Federal Project. This was a crucial step to confirming the 4.5-foot change to the conservation pool will not substantially increase the risks for the project. A full risk assessment was not included as part of the PFMA. Since incremental consequences were not evaluated as part of the PFMA, the dam remains classified as a DSAC 4.

A Periodic Assessment of Aquilla Dam was conducted in July 2016. The PA consisted of a facilitated Potential Failure Mode Analysis (PFMA), a Periodic Inspection (PI), and a risk assessment of potential failure modes judged to be risk-drivers. The incremental risks associated with Aquilla Dam were determined to be low. A DSAC of 4 (no change) was recommended by the PA team and DSOG, and approved by HQ DSO in May 2017.

3.2 Seepage Conditions

Seepage was identified during PI #2 in 1985 at both the left and right abutments of the OW stilling basin, as shown in the following figures. Flow was estimated as 20-30 GPM and was clear for a pool elevation of 537.5 feet. The seepage on the right abutment appears to have been successfully controlled with the seepage collection system and filter berm installed in 1987. Seepage is monitored and measured through a 6-inch pipe (weir 1). In 1994-1995, twelve relief wells were installed on the left cut slope of the outlet works discharge channel. This system provides protection against uplift during the maximum pool, but does not control normal pool-related seepage exiting on the cut slope. A seepage collection system and filter berm was installed on the left abutment in April 2010 as part of the Risk Management Plan to control and collect the seepage, prevent the loss of material, dry up the slope, and improve maintainability. A measuring weir was constructed near the downstream limit of the riprap slope protection.

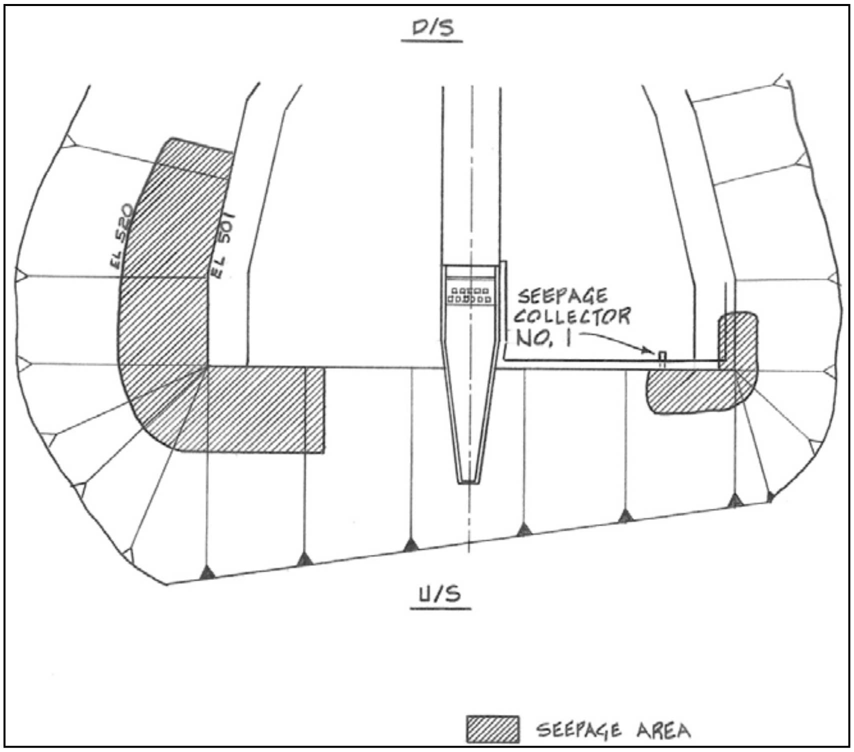

Figure III-1: Seepage Areas at Outlet Works (CESWF, 1985)

Figure III-2: Left Abutment of Outlet Works Showing Relief Wells and Weir

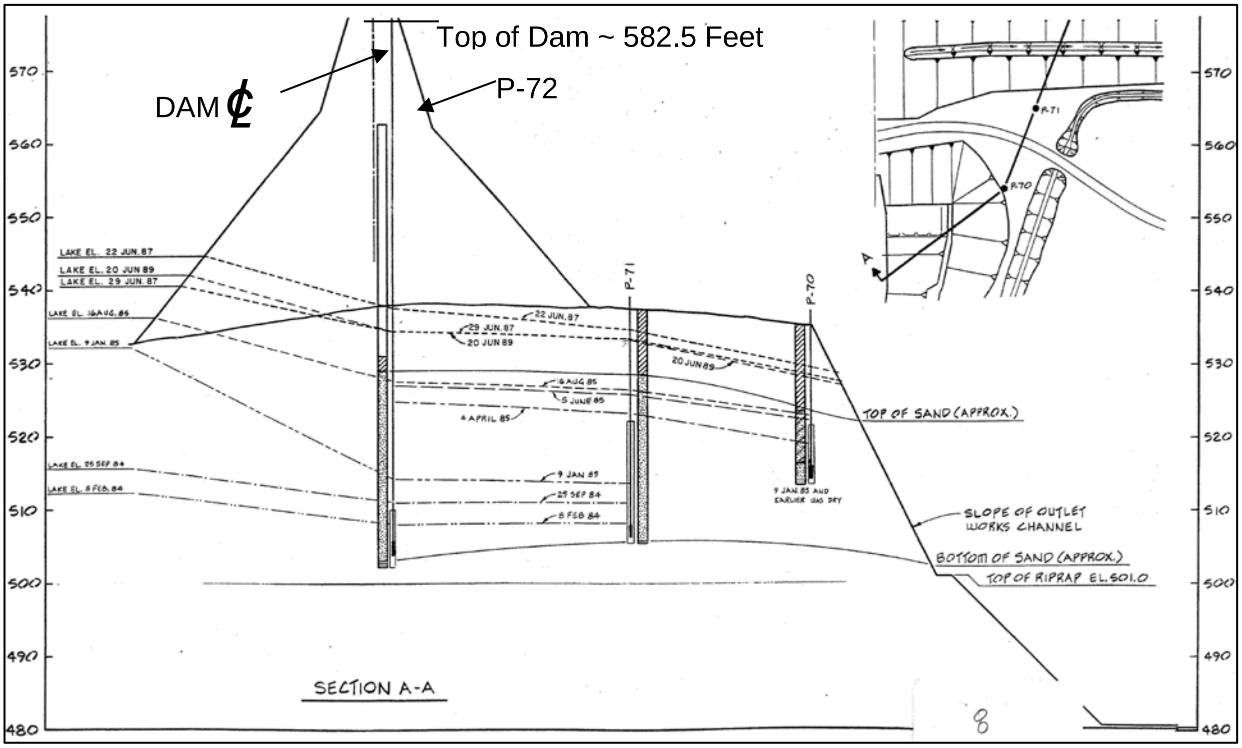

Figure III-3: Profile for a Line of Instrumentation at Left Abutment

3.2.1 Instrumentation for Monitoring Seepage

The project consists of the following instruments installed during construction and with the aforementioned improvements: piezometers, inclinometers, stilling basin reference marks, settlement plates, service bridge reference marks, embankment crest reference pins, outlet works reference conduit reference marks, relief wells, and seepage weirs. The number and monitoring frequency of the instruments are summarized in Table III-1 below. Figure III-4 shows the plan view of instrumentation. Instrumentation evaluations indicate the project is performing as intended.

Table III-1: Summary of Instruments

Instrument	Total	Active	Monitoring Frequency
Piezometers	53	48	Quarterly
Inclinometers	9	9	Annually
Stilling Basin Reference Marks	20	20	Annually
Settlement Plates	6	6	Annually
Service Bridge Reference	11	11	Annually
Embankment Crest Reference	123	123	Every 5 Years
Outlet Works Conduit	100	100	Annually
Relief Wells	12	12	Monthly
Weirs	2	2	Monthly

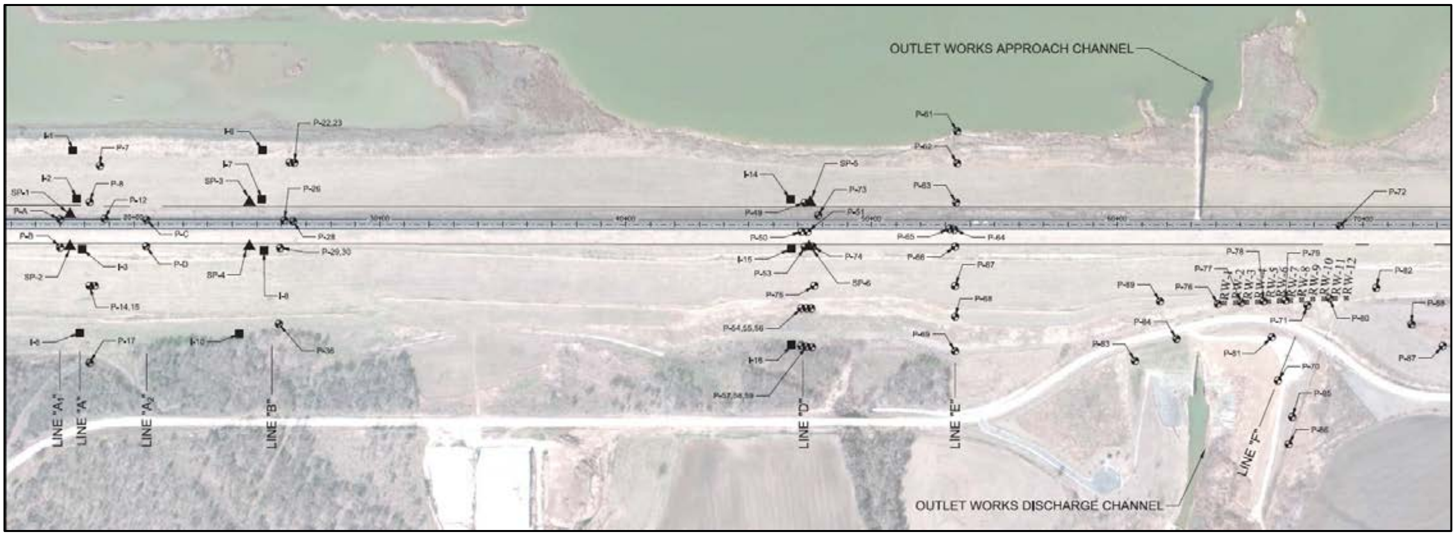

Figure III-4: Plan View of Embankment Instrumentation

3.3 Embankment Stability

Embankment Stability has not been much of an issue for the Aquilla Dam historically, but the Clay embankment is susceptible to shallow slides of steep slopes. The embankment for Aquilla has a robust section with the majority of the slopes greater than 4V:1H.

3.3.1 Seismic Conditions

There were no seismic considerations in the original design, and no subsequent seismic evaluations have been performed. A site-specific seismic study was performed for this project as part of the 2016 Periodic Assessment. The 2014 update of the National Seismic Hazard Map Program (NSHMP) by the U.S. Geological Survey (USGS) (Peterson et al. 2014) provides consensus-based analyses of seismic hazard, using Central and Eastern United States (CEUS) seismic sources, Western United States (WUS) seismic sources, combined geologic/geodetic information, and revised ground-motion models. This report represents the most current assessment of seismic hazard for large regions of North America. The dam lies in the “North American Craton” seismotectonic zone but is less than 35 miles west of the “Gulf Coast” seismotectonic zone and could be affected by strong ground shaking resulting from earthquakes in either of these zones. The dam has not been subjected to large seismic loads since construction. The most recent recorded earthquakes since construction occurred between 2009 and 2012 with varying magnitudes ranging from moment magnitude (M_w) of 2.3 to 3.3. These occurred approximately 25 miles north-northeast of the project. These events were most likely the result of induced seismicity from injection wells.

The New Madrid seismic zone, located approximately 450 miles northeast of the site, is the primary source for strong ground motion at the site and has a high rate of active seismicity and can produce large earthquakes. The Meers fault, located approximately 230 miles northwest of the site, is an area where multiple faults have been mapped with this fault being active in the last 150 years. It would also be a source of ground motion at the site.

The seismic hazard may be higher in areas of potentially induced seismicity than the hazard depicted on the 2014 national seismic hazard maps. At least one injection well is located within 12.5 miles of Aquilla Dam. The record high earthquake of M_w 4.0 occurred on 7 May 2015 and was approximately 30 miles southwest of Dallas and 40 miles north of Aquilla Dam, as shown in Figure III-5.

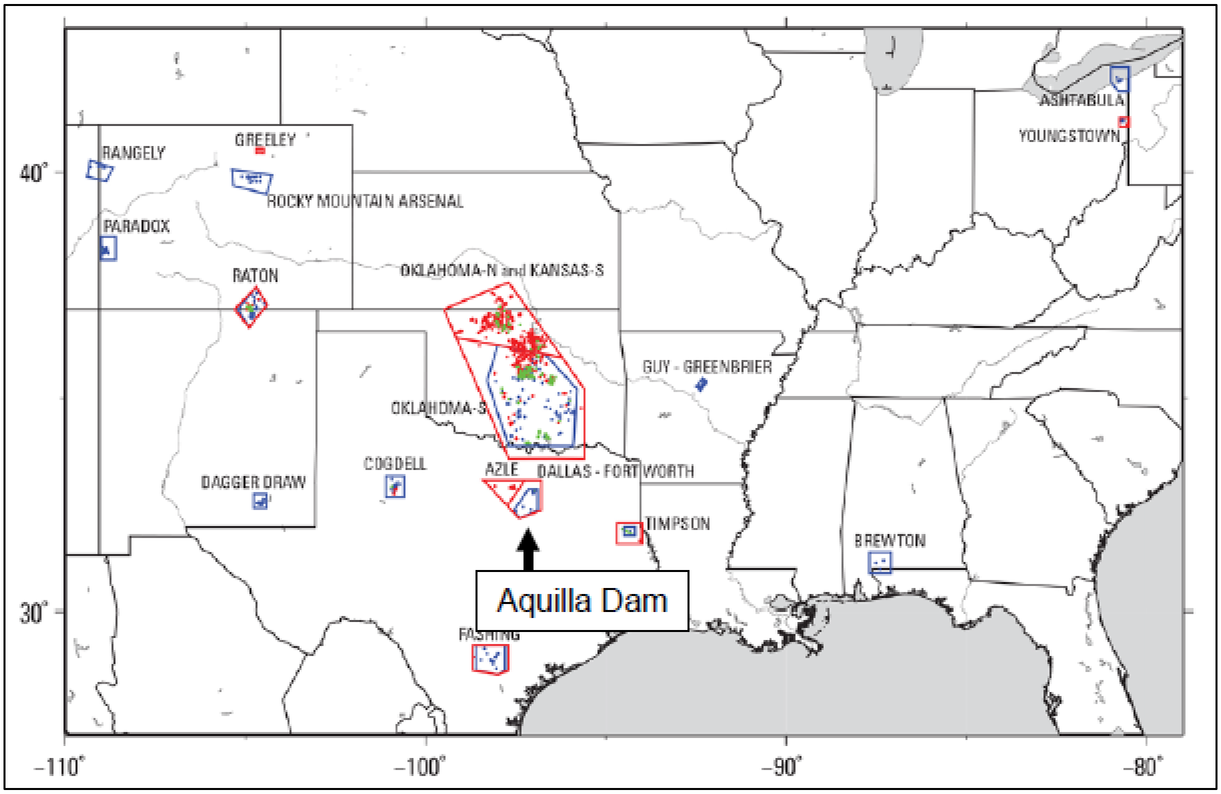

Figure III-5: Potentially Induced Earthquakes (Peterson et al. 2015)

USACE design guidelines utilize an operating basis earthquake (OBE) and a maximum design earthquake (MDE). The probabilistically determined OBE is considered to be an earthquake that has a 50-percent probability of exceedance (PE) in 100 years (i.e., 144-year return period) and is estimated from a probabilistic seismic hazard analysis (PSHA). The MDE is the maximum level of ground motion for which a structure is designed or evaluated. A local (site-specific) PSHA has not been performed for this project. The mean seismic hazard curve for the peak horizontal ground acceleration (PGA) was generated using the regional (USGS 2014) PSHA. Based on the USGS's 2014 data, a PGA of 0.01g was estimated for the OBE and 0.02g for the MDE.

In June 2016, the USGS published new guidance on the seismicity of the north central Texas area. The USGS utilized natural and induced earthquake information collected through the end of 2015 to produce a one-year seismic hazard forecast for 2016 for the CEUS. It provides some useful insight on the increased seismicity not captured in the USGS 2014 NSHM. The mean seismic hazard curve (one-year forecast) for the peak horizontal ground acceleration (PGA) was generated using the regional (USGS 2016) PSHA. Hazard estimates from induced earthquakes are not compatible with estimates of long-term seismic hazard caused by tectonic processes. Therefore, there is significant uncertainty beyond the 1/144 return period. Based on the USGS's 2016 data, a PGA of 0.03g was estimated for the OBE.

For sites in the vicinity of the dam, seismic hazard is considered quite low. The proposed pool raises should have negligible effect on the seismic loading conditions and/or evaluation.

3.4 Erosion Conditions

While localized surface erosion of the embankment/overburden materials have been observed at areas of concentrated flow, most of the subsurface materials are minimally erodible. High plasticity clays are prominent, which are fairly resistant to erosion. Significant flow velocities and durations would be required to sufficiently deplete the embankment section to cause a breach. Current loading conditions indicate that the dam embankment may be overtopped for a short period during peaked PMF inflow scenarios. Such flows may also be encountered during a spillway release and the underlying Eagle Ford Shale and Woodbine Sandstone may be susceptible to erosion during extreme events. Any increase of conservation pool will also increase the frequency and duration of spillway releases.

3.4.1 **Bank Erosion**

Erosion of the upstream embankment is a known issue for the Aquilla Dam. Historical survey data indicates that erosion along the dam has continued at the pace of approximately 1 foot per year towards the crest of the dam. The scarps along the unprotected shoreline at about the conservation pool elevation of 537.5 feet are up to 4 feet high. At this time, the erosion has not affected the operation of the dam.

3.5 Consequences of Failure

USACE has established a national standard of modeling procedures to support the estimation of consequences for breach and non-breach flood inundation scenarios over the full range of loading conditions. Inundation models extend from the dam downstream to a point of no significant consequences. The Modeling, Mapping and Consequence (MMC) Production Center is charged with producing scalable hydrologic and hydraulic dam break models, flood inundation maps, and consequence estimations to support risk assessments. Life loss due to flooding and economic losses in the form of property damage and foregone project benefits were estimated in the consequence evaluation. Consequences associated with the estimated performance of the project with breach, component malfunction, or misoperation and consequences associated with the estimated performance of the project without breach, component malfunction, or misoperation were evaluated. The differences between these two sets of consequences for a particular loading condition are the incremental consequences (i.e., those directly attributable to the dam failure for that loading condition).

The largest portion of the Aquilla Creek watershed is located in Hill County, Texas, with small portions in McLennan and Johnson Counties. The Aquilla Creek watershed is located approximately 25 miles north of Waco, Texas and just southwest of Hillsboro, Texas. The city of Waco (2010 population of 124,805) is located approximately 25 river miles below the dam in McLennan County, TX. The city of Sugarland (2010 population of 78,817) is located approximately 368 river miles below the dam in Fort Bend County, TX. The city of Freeport (2010 population of 12,049) is located approximately 449 river miles below the dam in Brazoria County, TX. Population at risk (PAR) is defined as the number of people downstream of a dam that would be subject to inundation risk. PAR estimates were generated using HEC's Flood Impact Analysis (FIA) software for breach and non-breach inundation scenarios. The estimated consequences with a rare flood event with and without breach are summarized in Table III-2 below.

Economic considerations help inform risk management decisions. Remediation costs include repair or replacement of downstream property directly damaged by the inundation such as residential, commercial and industrial property, and critical infrastructure in general. Total damage from dam failure was evaluated approximately 458 miles downstream of the dam to the Gulf of Mexico, impacting approximately 67,700 structures. Development in this area is rural in nature, with cities located throughout the inundation area. Structures are mostly single story residential with some commercial and industrial. The estimated direct property damage is summarized in Table III-2 below.

Table III-2: Estimated Consequences from 2016 PA

Aquilla Dam Facts	
Estimated consequences with rare flood event and breach:	Estimated consequences with rare flood event and no breach:
• Population at risk: ~235,000 • Structures at risk: 67,500 • Land and property at risk: $9.7 billion	• Population at risk: ~210,000 • Structures at risk: No data available • Land and Property at risk: $7.6 billion
Damages prevented to date: $48.4 million (1983-2015)	

IV - Geotechnical Analysis of Pool Raise Impacts

4.1 Seepage & Stability Analyses of the Dam Embankment

Seepage and Stability conditions were evaluated with GeoStudio® 2007 (v7.16) at the Left Abutment of the Outlet Works, which has been identified as the most critical section with respect to seepage, for the three alternative pool raise elevations. Dam profile and material properties were maintained from the SPRA Re-Evaluation Report previously conducted.

The cross section for seepage and stability analyses was cut as shown in the figure below. The section follows a potential shortest path around the cutoff zone of the outlet works excavation, passes through piezometers P-72, P-71, and P-70 before exiting in the outlet channel slopes.

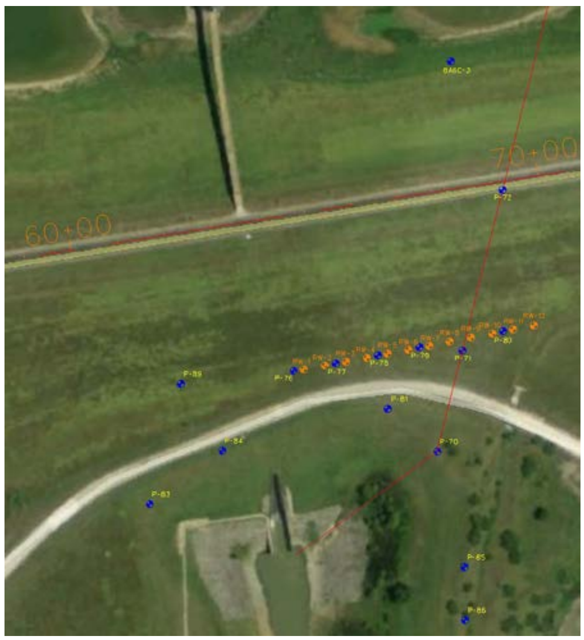

Figure IV-1: Embankment Section used for Seepage/Stability Analysis

The design embankment profile was used to define the geometry of the analysis section, assuming settlement and/or erosion would be negligible. The foundation stratigraphy was defined using available subsurface data, as shown in Figure IV-2, with the following assumptions.

a. Clayey Sand (pervious) stratum extends from elevation 503.0 feet to 517.0 feet at U/S limit, with a gradual rise to 520.0 feet at D/S stilling basin.
b. Thin strata of Gravel and Weathered Shale were approximated from boring data.
c. Seepage path through the pervious alluvium ~ 1100 feet from entry to exit.

The loading conditions evaluated for this analysis include the following.

a. Conservation Pool – Steady State Analysis at elevation 537.5 feet
b. 2.5-ft pool raise – Steady State Analysis at elevation 540.0 feet
c. 4.5-ft pool raise – Steady State Analysis at elevation 542.0 feet
d. 6.5-ft pool raise – Steady State Analysis at elevation 544.0 feet

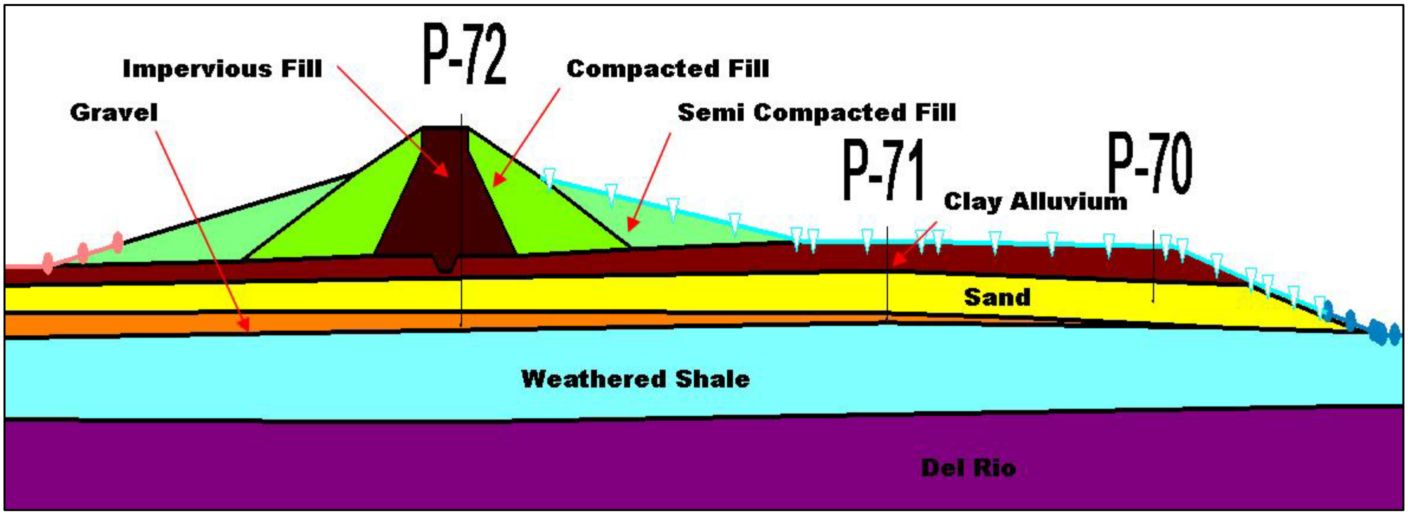

Figure IV-2: Section at Outlet Works used for Seepage/Stability Analyses

4.1.1 Seepage/Stability Analysis Results

Seepage and Stability were evaluated in accordance with effective USACE criteria. Seepage is evaluated based on the exit gradient, or force at which the water exits the surface. The critical gradient, or gradient sufficient to displace soil particles, for the site was determined to be around 1.0 ft/ft. Stability is evaluated based on the Factor of Safety to resist sliding of the embankment, this is typically a minimum of 1.3 for Life Safety Structures. The Entry and Exit method in GeoStudio specifies the location of where the slip surfaces will enter the ground surface and where they will exit. The Block Specified method specifies two grids of points for where the slip surface will enter the ground surface and where it will exit.

The results of the stability analysis indicate that embankment is stable for steady state conditions at the conservation pool elevations evaluated. A summary of the results is provided below.

Table IV-1: Summary of Seepage/Stability Analysis Results

Loading Condition (feet)	Exit Gradient	Factor of Safety	
		Entry & Exit	Block Search
537.5	0.256	2.03	2.56
540	0.257	2.03	2.57
542	0.257	2.03	2.46
544	0.258	2.03	2.23

4.2 Erosion Analysis and Evaluation

4.2.1 Embankment Erosion for PMF Overtopping

The revised hydrologic loading conditions result in a peak inflow that exceeds the dam crest by up to 0.83 feet, overtopping the embankment for a short period of around 5 hours. However, the robust earthfill embankment should be somewhat resistant to the short duration flow, sustaining only minor damage that will require repair. The erosion potential for the earthen embankment was previously evaluated, using the Overtopping Toolbox, as part of the SPRA Re-Evaluation Report. Results indicated that overtopping erosion is not considered a primary risk driver.

4.2.2 Spillway Erosion Analysis Method

Potential impact of erodibility of the spillway during spillway release was evaluated as part of the SPRA Re-Evaluation Report previously conducted, and is based on USACE EM 1110-2-1603 Hydraulic Design of Spillways. The Aquilla uncontrolled spillway is classified as a limited service spillway, and is designed to operate very infrequently with the knowledge that some degree of damage or erosion will occur during operation. Limited service spillways are designed to meet the following criteria:

a. The spillway flow and/or resulting erosion will not endanger the dam or dam foundation.
b. The control of the discharge will remain at the predetermined control section and will not be lost due to erosion.

The Spillway erosion analysis suggests less than 0.5 foot of erosion is expected for up to the 500 year flood event, but less frequent events with higher/longer flows have potential to erode the sandstone to expose the clay shale to weathering. However, because of the geometry and construction of the spillway it would be very difficult to erode sufficiently upstream of the sill to cause a breach of pool.

Results for the worst case and the practical case scenarios are shown in Figure IV-3, and are as follows.

- **Worst Case -** Poorly maintained grass cover showed the first half foot of topsoil would be lost and the spillway would erode 300 feet upstream of the sill, leaving 1250 feet before potential breach of pool. Headcutting would extend 13 feet below the design elevation.
- **Practical Case –** Good grass cover showed some loss of topsoil and head cutting to within several hundred feet to the sill. Headcutting is 13 feet deep, but the sill is not compromised.

Figure IV-3: Limits of Spillway Erosion from Spillway Erosion Analyses

4.2.3 Bank Erosion Analysis and Remedial Measures

Minor erosion from wave action of the upstream shoreline has been reported in areas not protected by rip rap revetment, which is currently being addressed with the O&M program. Any change in conservation pool will likely result in significant impacts on embankment slopes not protected with riprap. The details of slope protection depend on the historic performance, expected wind velocities and duration, the size and configuration of the reservoir, the permanent water-surface elevation, and the frequency of the pool elevation. Slope protection has been evaluated in accordance with ER 1110-2-2300 for the current conservation pool to repair existing erosion. The proposed water surface for potential pool raise alternatives have also been evaluated for further consideration of necessary improvements.

In order to reduce the potential for erosion along the upstream bank, stone riprap has been designed to repair existing conditions and to provide additional protection relative to potential pool raise elevations. EM 1110-2-2300 does not recommend less than 18 inches for riprap thickness on dams. The distance to be covered would be approximately 3500 to 6000 feet long between stations 30+00 to 90+00. The extent of the protection is dependent on the fetch length, area of inundation, and the existing topography at the toe of the dam. Figure IV-4 shows the proposed extent of the riprap for the current condition and Alternative 3.

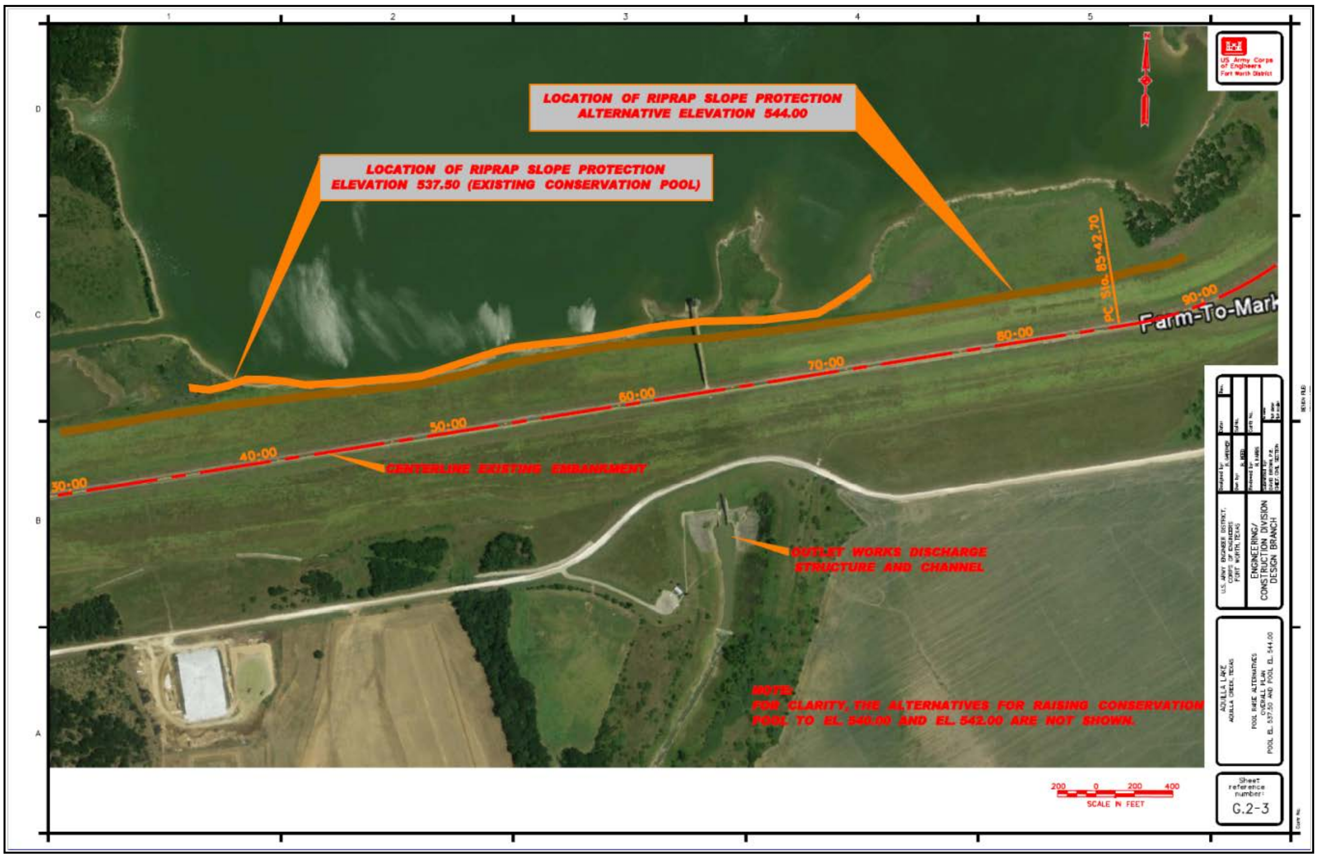

Figure IV-4: Rip Rap Slope Protection for current & potential pool elevations (SWF, 2011)

V - Conclusions and Recommendations

USACE is coordinating with the regional water supply sponsor to evaluate potential re-allocation scenarios for the Aquilla Dam and Reservoir to provide the anticipated yield to support development of the Brazos River Basin. This requires that all known Dam Safety conditions are evaluated with respect to the proposed pool elevations to ensure that the increased pool loading does not have a negative impact on the Project's performance. Initial conditions were evaluated as part of the Risk Management Plan to confirm the Dam Safety Action Classification (DSAC). Potential pool raises were evaluated for this study using methods consistent with the Risk Management Plan.

5.1 Risk Management Considerations

In September 2012, a SPRA re-evaluation was submitted for consideration by the Dam Safety Oversight Group (DSOG) recommending Aquilla Lake Dam be changed from a DSAC 3 rating to a DSAC 4 based on implementation of the risk management measures detailed in this report. In August 2013, the DSAC change was approved by the HQUSACE Dam Safety Officer (DSO). In June 2016, Periodic Assessment (PA) #1 was conducted for Aquilla Dam. The PA consisted of a facilitated Potential Failure Mode Analysis (PFMA), a Periodic Inspection (PI), and a risk assessment of potential failure modes judged to be risk-drivers. Risks from the PA are not driven by pools near conservation, so it is not anticipated the pool increase would change the results; however, the dam should be monitored closely once the pool raise is implemented for any changes in project performance. No change in the DSAC was recommended by the PA team, District, and DSOG, and approved by the HQUSACE DSO in May 2017.

5.2 Pool Re-Allocation Considerations

Each of the potential pool raise alternatives were evaluated with respect to the known Dam Safety concerns as it relates to seepage, stability, and erosion impacts. Initial results indicate that all alternatives are technically viable. However, further evaluation of the selected alternative will be required to confirm that the integrity of the dam embankment will not be impacted by increasing the conservation pool.

There is erosion damage to the upstream embankment that needs to be repaired, and additional rip rap stone protection will be required for any change in the conservation pool. Preliminary designs have been prepared to support the cost analysis for the pool raise alternatives.