GEOTECHNICAL

Geotechnical Report

Arkansas River Low Water Dams

Prepared for
Tulsa County

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CH2MHILL®

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Introduction

1.1 Overview

This draft report summarizes the geotechnical analysis and recommendations prepared for the schematic design of three low water dams proposed for the Arkansas River Low Water Dams Project (Project). The Project is located within the Arkansas River Corridor near Tulsa, Oklahoma which stretches roughly 42 miles from Keystone Dam to the Tulsa County/Wagoner County border.

This report includes a review of available subsurface information and development of schematic design level geotechnical recommendations for new dams at Sand Springs, South Tulsa / Jenks, and Bixby. Additionally, schematic earth retention concepts for a conceptual floodwall structure supporting the proposed West Park near the right (west) abutment of South Tulsa / Jenks dam are described in this report. A discussion of geotechnical data gaps and recommended additional geotechnical data is also included. Other Project features, such as the proposed modifications to the Zink dam and riverbank stabilization design elsewhere along the river corridor are *not* addressed in this report.

The schematic design will be used to develop a *preliminary* cost opinion for the overall project which will be used by Tulsa County leaders to aid in project planning decisions and to seek funding for design and construction. Once funding is obtained, a formal design process for the dams and floodwall structures will commence. As recommended in this report, formal design will include a comprehensive site-specific geotechnical field and laboratory program to support detailed design development and confirmation of the assumptions made in this report.

1.2 Background

Tulsa County, as part of the Arkansas River Corridor Master Plan, is undertaking an improvement project within the Arkansas River Corridor. The Arkansas River Corridor follows the Arkansas River as it flows southeasterly across Tulsa County for nearly 42 miles from Keystone Dam to the Tulsa County/Wagoner County border. The primary goals of the overall Project are to improve least tern habitat, improve fish habitat and fish passage, improve the function of the river system itself, enhance economic development, increase recreational opportunities, and increase connectivity between the river and surrounding communities.

Key components of the Project include:

- Design of habitat improvements along the corridor
- Design of bank stabilization in select areas
- Design of a new Sand Springs low-head dam, pedestrian bridge, and amenities, including the proposed floodwall on the right abutment.
- Design of modifications to Zink Dam and lake with whitewater features
- Design of a new South Tulsa/Jenks low-head dam, pedestrian bridge, and amenities
- Design of a new Bixby low-head dam and amenities

Of these project components, the Sand Springs Dam, South Tulsa / Jenks Dam and floodwall, and the Bixby Dam are addressed by this report. The locations of these structures within the Arkansas River Corridor is shown in Figure 1-1. The proposed modifications to Zink Dam are not addressed in this report. The Zink Dam is discussed in the *Schematic Design and Cost Estimates Report*, preliminary design efforts were completed in 2012 and summarized elsewhere.

1.2.1 General Configuration of Low Water Dams

The proposed dams at Sand Springs, South Tulsa / Jenks, and Bixby will span the width of the Arkansas River; each dam will have a total crest width of nearly 1900 feet (Table 1). Each dam will consist of multiple cross-sections with fixed crest and adjustable Obermeyer weirs which will be configured and operated to manage instream flows, sediment, and fish passage within the river channel. All sections of the dam are hydraulically designed to function as overflow sections and generally include a fixed crest section containing no gate, a 3-foot crest gate section, and a full-height gate section. The full height gate represents the maximum operational projection of the dam structure above the riverbed at each dam site. The full height gate is 10 feet tall at Sand Springs, 7 feet tall at South Tulsa / Jenks, and 4 feet tall at Bixby. Bixby dam does not have a crest gate section. The maximum elevation of the gates will be set 6 inches lower than the fixed crest, to promote concentrated flows over the gate sections during low-flow periods. Additionally, the Sand Springs and South Tulsa / Jenks Dams will support an integral pedestrian bridge over the river.

The proposed dams are similar in cross-section, but differ at each location based on subsurface conditions, hydraulic constraints, and proposed park amenities as applicable. The dams are proposed to be constructed of mass concrete and founded on the shale bedrock which underlies each of the proposed locations. The overall cross-section of the dams will incorporate a stepped downstream face to prevent formation of a hydraulic roller for the range of design flows. The stepped faces may be constructed of mass concrete, anchored stone blocks, or other material with density similar to concrete, based on cost or aesthetic preferences.

Additional background on each dam is provided in the overall *Schematic Design and Cost Estimates Report*, including Figures and Drawings showing the locations and schematics of the planned work. Table 1 summarizes the required dam crest and gate elevations provided by others at each dam location.

TABLE 1
Dam Geometry Summary

Low Water Dam Location	Dam Length (ft)	Fixed Crest	Crest Gate			Full Height Gate		
		Fixed Crest Elevation (ft)	Top of Gate Elevation (ft)	Gate Height (ft)	Sill Elevation (ft)	Top of Gate Elevation (ft)	Gate Height (ft)	Sill Elevation (ft)
Sand Springs	1,900	638.5	638.0	3	635.0	638.0	10	628.0
South Tulsa/Jenks	1,850	597.5	597.0	3	594.0	697.0	7	590.0
Bixby	1,950	583.5	n/a	n/a	n/a	583.0	4	579.0

n/a = not applicable

1.3 Description of Proposed Project Components

1.3.1 Sand Springs Dam

The proposed Sand Springs Low Water Dam is located downstream of the Keystone Dam, approximately 1,500 feet downstream of the existing Highway 97 bridge. The dam will be 1,900 feet in length with a fixed crest elevation of 638.5 feet. The proposed dam will have three different cross-sections: a fixed crest section, a 3-foot crest gate section, and a full-height gate section. The full-height gates will be 10-feet-tall, and crest gates will be 3-feet-tall. The maximum elevation of the gates will be set at elevation 638.0, which is 6 inches lower than the fixed crest gates. In addition, the dam will support a pedestrian bridge across the river. The pedestrian bridge will be supported by concrete columns integrated with the dam structure. Table 1 presents the key elevations for the Sand Springs dam; Figure 1-2 shows the proposed section geometry.

The left (north) abutment of the dam will terminate into an existing riverbank slope below a forested bluff. The edge of the bluff is located approximately 300 feet south of the riverward toe of the existing Arkansas River Levee at elevation ±647 feet. The abutment termination at this location will require a combined retaining wall and sloping ground to match the existing grade at the bluff. The retaining wall will extend from the base of the dam to approximately elevation 640 feet (2 feet above the dam crest), then slope at 3:1 (horizontal:vertical) to tie into the bluff elevation above. Depending on flow velocities at this location during overtopping, the slope may need to be armored. A seepage cutoff at the abutment contact will be formed by extending a concrete diaphragm approximately 20 feet into the abutment and backfilling with compacted low permeability backfill. Appropriate measures should be incorporated to prevent migration of fines, including placement of a sand filter on the downstream side of the dam

The right (south) abutment of the dam will terminate in a large natural earthen berm with a top elevation of ±651 feet. This berm is upstream of a tributary stream that enters the Arkansas River roughly 900 feet downstream of the proposed dam alignment. The right abutment will require a retaining wall which extends to roughly elevation 640 feet and allows water to spill over the dam structure. Above the crest of the wall, the abutment should be backfilled and graded upwards at 3:1 (horizontal:vertical) or flatter to intersect existing grade at the top of the berm. Depending on anticipated velocities during overtopping events, this slope may require armoring. A seepage cutoff at the abutment contact will be formed by extending a concrete diaphragm approximately 20 feet into the abutment and backfilling with compacted low permeability backfill. The diaphragm should be constructed in open cut and backfilled with compacted low permeability soil backfill to resist seepage around the abutment. A sand filter should be installed if appropriate to limit fines migration of the low permeability backfill.

1.3.2 South Tulsa / Jenks Dam

The South Tulsa / Jenks dam location is located approximately 2,000 feet downstream of the Creek Turnpike bridges. The dam will be 1,850 feet in length with a crest elevation of 597.5 feet. Like Sand Springs, the proposed dam will have three different cross-sections: a fixed crest section, a 3-foot crest gate section, and a full-height gate section. A pedestrian bridge structure will also be included over this dam, and will be supported by columns structurally connected to the underlying gravity dam structure. The full-height gates will be 7-feet-tall and the crest gates will be 3-feet-tall. The maximum gate elevation will be set 6 inches below the fixed crest elevation, at elevation 597.0. Table 1 presents the key elevations for the South Tulsa/Jenks dam; Figure 1-3 shows the proposed section geometry.

The left abutment of the dam will be deeply embedded into the left bank, requiring construction of a large anchored retaining wall. This wall is estimated to be on the order of 30 feet tall as measured from the top of the underlying bedrock and extending upwards to meet the existing grade at elevation ±614 feet. The length of the wall is not precisely known, but estimated to be 400 to 500 feet in length depending on finished grading of the left abutment. A tied-back soldier pile and lagging wall with reinforced concrete facing is recommended for the convenience of top-down construction. Where the dam abuts the wall, a 20 foot concrete diaphragm is recommended to be installed into the bank to form a seepage cutoff. The diaphragm should be constructed in open cut and backfilled with low permeability backfill. A sand filter should be installed if appropriate to limit fines migration of the low permeability backfill.

The right abutment of the dam is discussed in the following section on the South Tulsa / Jenks Floodwall.

1.3.3 South Tulsa / Jenks Floodwall

The right abutment of the dam will terminate at the proposed West Park site which will be a new park development and include numerous park amenities and features. The principal feature of the West Park concept is retaining wall (floodwall) at the right abutment of the South Tulsa / Jenks dam which extends upwards from the river channel to elevation ±612 feet to overlook the river and facilitate pedestrian bridge access. The overall length of this wall is on the order of 500 feet, but will transition to additional retaining wall structures envisioned as part of a tiered retaining wall grading plan for the park. The park amenities and tiered retaining walls will extend upstream to the Creek Turnpike.

The retained soil behind the floodwall is slightly above the 100 year flood elevation, but is expected to be inundated during large floods, including the 500 year flood for which the floodway is designed. Depending on the flood velocities expected at the park during high water events which overtop the floodwall, erosion protection measures, up to placement of articulated concrete block mats may need to be incorporated into the park surfacing. Relatively low velocities can be reinforced with geosynthetic turf reinforcement, velocities less than 6 feet per second can typically resist erosion with maintained grass.

A tied-back soldier-pile-and-lagging retaining wall with reinforced concrete facing is recommended for the convenience of top-down construction. The wall facing should be embedded into the underlying bedrock to maintain continuous protection of the retained backfill. It is anticipated that fill placement behind the wall will be necessary to bring the park surface up to elevation 612. This material could be locally excavated silty sand material excavated from the floodway within the dam footprint or from the right abutment where material removal is necessary to form the approach channel at the right abutment of the dam. Where the dam abuts the wall, a seepage cutoff is necessary to limit seepage around the abutment of the dam. A concrete diaphragm is recommended to be installed 20 feet into the bank and tied to the structure and to the underlying bedrock to form a seepage cutoff. The diaphragm should be constructed in open cut and backfilled with low permeability backfill. A sand filter should be installed if appropriate to limit fines migration from the low permeability backfill.

Additional retaining structures upstream of the floodwall are beyond the scope of this report. It is anticipated that for these short walls, cantilever sheet pile wall, cantilevered concrete walls, mechanically stabilized earth walls, or gravity walls constructed of concrete or cut stone could be valid concepts. Any such wall should have adequate scour protection to prevent undermining of the wall during high flow events. At the north end of the West Park, where the walls approach the Creek Turnpike, the presence of existing foundation elements beneath the bridge should be considered, along with the potentially low headroom if walls will extend under the existing bridge superstructure.

1.3.4 Bixby Dam

The Bixby Dam is located approximately 4,000 feet downstream of the Highway 64 Bridge. The dam will be approximately 1,950 feet long and have a crest elevation of 583.5 feet. The Bixby dam will only have two sections – a fixed crest and a full-height gate. The full-height gate sections will be 4 feet tall, with the maximum gate height set at elevation 583.0—6 inches lower than the fixed crest. Two arching spillways will be placed near each riverbank with a radius of 88 feet and a crest elevation of 583.0 feet. Like the other dams, flows will be concentrated through the gates. No bridge across the dam structure is planned. Table 1 provides a summary of the key dam elevations.

1.4 Limitations

This report has been prepared for the exclusive use of Tulsa County and CH2M HILL, for specific application to the Arkansas River Low Water Dams project. This report is prepared for a concept-level design of limited scope using extremely limited subsurface information. This report does not present foundation recommendations suitable for final design. This work has been performed in accordance with generally accepted geotechnical engineering practice using geotechnical data, survey data, river stage data, dam locations, and geometries provided by others. No other warranty, express or implied, is made.

It is noted here, and elsewhere in this report, that the schematic design work summarized herein is based on very limited and fragmentary subsurface information available within the river corridor. In a very real sense, the nearest subsurface information available for many the proposed major project components is thousands of feet away. Much of this data is old, contains limited laboratory data, was collected with poor survey control, and was collected for purposes other than dam design. The existing body of geotechnical data does not meet the standard of care for geotechnical design of the proposed structures; and review of the recommendations of this report should consider the likely implications of variation in subsurface conditions. Actual conditions at the sites will vary from those shown herein. In every case, these interpretations of available data were made in good faith to address the primary geotechnical risks associated with the project and provide a cost effective solutions.

Exploration data indicate soil conditions and groundwater conditions only at specific locations and times, and only to the depths penetrated. Also, the passage of time may result in a change in conditions at these locations. Most of the geologic information and geotechnical analysis is based upon interpretation, and reflects only the opinion of the geologist or engineer with regard to the character and extent of geologic materials. Subsurface conditions at other locations will differ from conditions occurring at these explored locations.

If any changes in the nature, design, or location of the facilities are planned, the conclusions and recommendations contained within this report should not be considered valid unless the changes are reviewed and the conclusions of the report are modified or verified in writing by CH2M HILL and Tulsa County. This report has not been prepared to meet the technical requirements or needs of Contractors. Any such use of this report without guidance from CH2M HILL and Tulsa County consists of improper use that could lead to erroneous assumptions and faulty conclusions.

Geology

2.1 Regional Geology

The project location lies within Tulsa County, Oklahoma where native topography was shaped by erosion of bedrock of differing hardness. The predominant bedrock is shale, with thinner sandstone and limestone beds. The bedrock belongs to the Des Moines and Missouri series of the middle and upper Pennsylvanian age. The original bedrock deposits were horizontal, but have been subsequently uplifted and tilted such that most dip generally to slightly north of west at rates of 30 to 50 feet per mile. (Oakes, 1952)

The region is drained by the Arkansas River and its tributaries. West of the city of Tulsa, the river is fairly deeply incised, with the flood plain generally less than 2 miles wide. Surrounding hills rise to 250 to 300 feet above the flood plain. To the east of Tulsa, the flood plain widens and the adjacent topography is more moderate. (Oakes, 1952)

2.2 Site Geology

The Oklahoma Geological Society has published recent revisions to geologic mapping of the project vicinity. This mapping covers all three of the proposed dam sites within the Arkansas River Corridor, and published maps were reviewed for each of the dam sites. Several mapped geologic units were identified in the project vicinity which vary along the river corridor and include, from upstream to downstream, the Nellie Bly formation, the Coffeyville formation, the Memorial formation, the Seminole formation, the Nowata formation, the Wewoka formation, and the Senora formation—all of which are shale bedrock. The maps generally describe the shale as gray, thinly bedded, and highly weathered in zones (independent of depth below surface). Additional discussion for each location is presented below.

2.2.1 Sand Springs Dam

The relevant geologic maps from the Oklahoma Geologic Survey are Wekiwa (Stanley, 2010), Sand Springs (Chang and Stanley, 2010), and Saluda North (Chang and Stanley, 2011). Geologic mapping suggests that the shale bedrock which underlies the site is part of the Nellie Bly Formation on the northern end of the dam location (left abutment and the river bed) and part of the Coffeyville Formation on the southern end (southern portion of river bed and right abutment) of the project. The Nellie Bly Formation is described with interbedded sandstone, siltstone and shale which cannot easily be separately mapped. The shales are described as interbedded, light olive gray to olive gray and consist of silty claystones to mudstones. The Coffeyville Formation is described with four units, including the sandstones and shales. The shale portion (unit 3) is described as light olive gray to dusky yellow to medium gray, well-laminated to fissile and as a slightly silty clayshale.

The bedrock at the site is generally overlain by variable thicknesses of sandy recent alluvium within the Arkansas River Corridor.

2.2.2 South Tulsa / Jenks Dam and Floodwall

The relevant geologic map from the Oklahoma Geological Survey is for the Jenks quadrangle (Stanley and Chang, 2012). The geologic mapping suggests that the shale bedrock is part of the Memorial and/or the Nowata formations. The Memorial Formation is described as having four units including a coal unit (which is noted as not being observed in the mapped area), and two shale intervals separated by a sandstone unit. The shale units are described as being light olive brown to grayish yellow in the upper unit and light olive gray to greenish gray in the lower unit. The upper unit is described as being interbedded as a weakly calcareous mudstone and a friable fine-grained sandstone; the lower unit is described as a silty clayshale.

The Jenks sandstone, which separates the two, is described as varying degrees of gray to brown, friable to weakly indurated sandstone. The Nowata Formation is described as having two units in the mapped area – an upper unit called the Nowata flagstone and consists of an interbedded shale and limestone unit; this unit is underlain by a medium to light gray to light brown well laminated to locally fissile slightly silty clayshale. The bedrock at the site is generally overlain by variable thicknesses of sandy recent alluvium within the Arkansas River Corridor.

2.2.3 Bixby Dam

The relevant geologic maps for the proposed Bixby dam site from the Oklahoma Geological Survey are the Bixby quadrangle (Chang and Stanley, 2009) and the Leonard quadrangle (Stanley, 2008). Review of this mapping confirms that the shale layer is part of the upper layer of the Nowata and/or the Wewoka Formations. The Nowata Formation consists of an upper shale layer, interbedded limestone and shale layer, and a lower shale layer, and has an overall thickness of approximately 400 feet. The Wewoka Formation consists of interbedded sandstone and shale materials, depending on the location. The bedrock at the site is generally overlain by variable thicknesses of sandy recent alluvium within the Arkansas River Corridor.

2.3 Seismicity

The Tulsa area experiences relatively low seismicity. The U.S. Geological Survey's *Quaternary Fault and Fold Database of the United States* was reviewed as a check for the presence of active (seismogenic) faults in the project vicinity. This database contains information on faults and associated folds in the United States that are believed to be the sources of earthquakes with Moment Magnitude greater than 6 during the Quaternary Period (the last 1.6 million years). No such mapped faults were identified within a 100 kilometer radius of the project sites. Active faulting is not expected to impact the project site.

Seismic parameters for the project were estimated using the U.S. Geological Survey *Earthquake Hazards Program* website [http://earthquake.usgs.gov/hazards/]. The website includes an interactive tool which allows input of latitude and longitude of a project location, and outputs mapped peak ground acceleration values and spectral acceleration parameters on bedrock (Site Class B) at various recurrence intervals, and in accordance with most applicable building codes. The U.S. Army Corps of Engineers (USACE) publishes criteria for seismic design of concrete gravity dams. The USACE Engineering Manual 1110-2-2200 *Gravity Dam Design* identified two primary earthquake scenarios for consideration in the evaluation of concrete gravity dams. These earthquakes include are the Operating Basis Earthquake (OBE) and the Maximum Credible Earthquake (MCE).

The OBE is an earthquake which can reasonably be expected to occur (50-percent probability of exceedance) within the service life of the project. This corresponds to a return period of 144 years for a project with a service life of 100 years, as assumed for the low water dams. The OBE is determined using a Probabilistic Seismic Hazard Analysis (PSHA).

The MCE is defined as the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of a seismological and geological evidence. The USACE recommends that a Deterministic Seismic Hazard Analysis (DSHA) be performed for evaluation of the MCE. For schematic design level analyses, a formal DSHA was not performed. Past experience in dam design indicates that the MCE generally falls between a 2,475 year recurrence interval and a 4,975 year recurrence interval event. To be conservative, a PSHA value corresponding to a 4,975 year event (1 percent probability of exceedance in 100 years) was selected.

The recommended peak ground acceleration (pga) values on bedrock for each of the proposed dam sites are summarized in Table 2.

TABLE 2
Seismicity Parameters Summary

Dam	Dam Location		Peak Ground Acceleration (Site Class B)		Site Coefficient (Site Class C)	Peak Ground Acceleration (Site Class C)	
	Latitude (deg)	Longitude (deg)	OBE (g)	MCE (g)		OBE (g)	MCE (g)
Sand Springs	36.125°	-96.111°	0.009	0.088	1.2	0.011	0.106
South Tulsa/Jenks	36.013°	-95.952°	0.009	0.091	1.2	0.011	0.013
Bixby	35.952°	-95.874°	0.009	0.093	1.2	0.011	0.112

Shear wave velocity profile data is not available for any of the damsites within the Arkansas River Corridor. Although all of the dams are expected to bear on the native shale bedrock, the shale is anticipated to be soft and weathered; such shale will likely not correspond to the site class B for which the USGS publishes gridded acceleration values. The site class was determined based Table 3.10.3.1-1 (AASHTO, 2012) assuming a “Very dense soil and soft rock” with a shear wave velocity of 1,200 feet/sec—which equates to a Site Class C. Site Class C equates to a site coefficient of 1.2, which is used to modify the PGA value to estimate the ground surface acceleration values for design. In this case, the individual PGA values will be increased by 20 percent. The resulting parameters for pseudostatic design are then given as horizontal and vertical coefficients of k_h and k_v. Per the USACE *Engineering Manual for Stability Analysis of Concrete Structures* (EM 1110-2-2100), the recommended k_h value is 2/3 of the site-adjusted PGA value. The vertical component is taken as zero. The horizontal seismic coefficient is used to determine the seismic loads acting on the dam structures.

Subsurface Investigations

3.1 Overview

All subsurface information used was from previous exploration reports or data summaries conducted by others for purposes of these project elements or nearby structures within the Arkansas River Corridor. No subsurface exploration or laboratory testing was performed for this project.

3.2 Existing Information

CH2M HILL reviewed available geotechnical reports and published geologic literature to estimate geotechnical properties for schematic design of the proposed project features. Overall geologic context for the dam locations was obtained by reviewing available geologic mapping data prepared by the U.S. Geological Survey and the Oklahoma Geologic Survey. Key existing subsurface investigation reports are as follows:

- At the proposed Sand Springs and South Tulsa / Jenks locations, a report prepared by Stantec (2008) includes subsurface data for geotechnical explorations performed on the riverbanks and within the river channel. (Attachment 1)
- At the seawall feature planned for the west abutment of the South Tulsa / Jenks Dam, a report prepared by Kleinfelder (2008) defines subsurface conditions along the proposed alignment. (Attachment 2)
- At the proposed Bixby Dam, a 2009 report prepared by Terracon for a pipeline project (unrelated to the dam project) provides subsurface information at the riverbank approximately 300 downstream feet from the proposed alignment. (Attachment 3)

Numerous other historical reports and foundation data sheets for roadway bridges within the floodway were also reviewed. Although these historical reports contain useful data which generally confirms the subsurface conditions within the river corridor, the projects and facilities they describe are generally located too far (often more than 1,000 feet away) from the proposed dam locations to be of specific use the project. Additionally, the subsurface information published in these reports is generally not specific to design of dams. The available subsurface data provides a broad overview of subsurface conditions at the proposed project features; additional data will be needed to support actual designs in the future. A summary of these reports and data are as follows:

- Sand Springs: Highway 97 Bridge and South Bank Overpass Plan Sheets (ODOT, 1971 and 1975, respectively)
- South Tulsa/Jenks: Creekside Highway Widening (Terracon, 2011), which includes a copy of the original ODOT plan sheets (ODOT, 1994)
- Bixby: Highway 64 (Memorial Drive) Project (ODOT 1984)
- Zink Dam: Design details for the existing Zink Dam and an investigation report for modifications were reviewed (W.R. Holway and Associates, 1983 and Terracon, 2012, respectively).

The available subsurface data indicates relatively similar subsurface conditions at the Sand Springs, South Tulsa / Jenks, and Bixby proposed dam locations. Typical subsurface conditions on the riverbanks are interbedded alluvial silt, clay, and sand overburden overlying shale bedrock; in the river bed, the available data suggests that alluvial sands and gravels overlie the shale bedrock. In some locations residual clay soils are found overlying the shale on the riverbanks. The proposed seawall at South Tulsa / Jenks is also underlain by sandy alluvium overburden and shale bedrock. At all dams, the alluvium overburden was found to be very loose to medium dense. Thin discontinuous layers of very soft to soft lean to fat clay were also encountered within the alluvium in limited locations.

Typically, shale bedrock was encountered at depths ranging from 5 to 10 feet below the ground surface within the river bottom, to between 20 and 30 feet along the riverbanks, although the overburden thicknesses vary at each site. Many of the existing boreholes described in the available geotechnical reports were not surveyed at the time of drilling, complicating interpretation of the bedrock elevation at each site.

3.3 General Subsurface Conditions

Based on conditions reviewed from sources summarized above, the subsurface conditions are anticipated to be relatively consistent near the Arkansas River Corridor and in the project vicinity. The typical subsurface conditions documented in existing reports and bridge investigations include interbedded silt, clay, and sand overburden with little gravel overlying predominantly shale bedrock. The stick logs from borings advanced at the Zink Lake Dam indicate a layer of sandstone overlying the shale bedrock at this location. The sand-dominated overburden was observed to have a consistency of very loose to medium dense. Layers of clay were observed to range from lean clay to fat clay, with a consistency of very soft to soft. Groundwater elevations along the river banks are presumably similar to the adjacent river level.

The shale bedrock in the project vicinity is massive and was encountered in every boring location reviewed for this Project. Typically, it is encountered at depths ranging from 5 to 10 feet below ground surface (bgs) within the river bottom, to between 20 and 30 feet along the river banks. The shale is identified as part of several geologic formations, including the Nellie Bly formation, the Wewoka formation, and the Senora formation. The shale is described as gray, thinly bedded, and highly weathered in zones (independent from depth below surface). Based on limited Texas Cone Penetration (TCP) testing within the shale, it is estimated to have a typical allowable point bearing greater than 30 tons/square foot (±420 psi).

In recent borings advanced at the Sand Springs and South Tulsa/Jenks areas (Stantec, 2008), Rock Quality Designation (RQD) and unconfined compressive strength were evaluated at each location. Near the Sand Springs location, the average RQD of the shale was observed to be 48 percent, based on findings from 3 borings. The average unconfined compressive strength of tested specimens was 565 psi. At the South Tulsa/Jenks location, the average RQD of the shale was observed to be 72 percent. The average tested unconfined compressive strength of shale was ±737 psi.

3.4 Subsurface Conditions at Individual Dam Sites

3.4.1 Sand Springs

Stantec advanced three boreholes along the proposed Sand Springs dam alignment in 2008. Foundation data sheets from the Highway 97 structure (ODOT, 1974), located roughly 1,500 feet upstream, include stick logs showing subsurface materials encountered in 1974. The locations of the previous investigations are shown in Attachments 1, 2, and 3.

The 2008 Stantec report indicates that one borehole was advanced at each river bank near potential abutment locations, and one borehole was advanced near the center of the river channel. The borings were advanced between 50.5 and 75.0 feet below ground surface. The boring near the north (left) abutment consisted of silty sand overlying shale bedrock; the boring near the south (right) abutment consisted of sandy silt and silty sand overlying shale bedrock. The boring in the riverbed encountered sandy alluvial materials overlying a thin layer (less than 2 feet thick) of sandstone overlying shale bedrock to the maximum depth penetrated. The Highway 97 investigations indicate sandy overburden overlying shale bedrock beneath the existing bridge. Available laboratory testing available from the Stantec borings included index testing in the soil samples (Atterberg limits and grain size analyses) and unconfined compression tests in collected rock cores.

As previously mentioned, no ground survey was performed during the Stantec investigation. Due to the natural variation and migration of sandbars within the river channel, the ground surface elevation and the corresponding bedrock elevation, within the river channel at the time of drilling is difficult to ascertain. In the Stantec borings, the bedrock was encountered at an estimated elevation of 615 feet on both riverbanks, and at about 618 feet in the riverbed. As noted previously, the borings were not surveyed and elevations are based values from the surveyed topographic maps created for this project, using the handheld GPS coordinates in the Stantec report. The Highway 97 bridge borings for the line of 17 borings closest to the Sand Springs dam site (downstream side of the bridge), show the top of rock as ranging from and elevation of 619 on the south (right) riverbank to an elevation of 615 on the north (left) riverbank, ranging between 615 and 619.5 feet and with an average of 617.5 feet. In the Stantec report, the upper few feet of the bedrock was noted as being highly weathered or weak in the upper portions of the borings on the riverbanks (S-1 and S-4), and highly weathered throughout the borings in the river bed (S-2). The available data suggest that the top of rock is relatively level in the proposed dam location.

3.4.2 South Tulsa/Jenks

The existing information published by Stantec (2008) and Kleinfelder (2008) applies to the proposed South Tulsa / Jenks dam and seawall projects, respectively. The available 2008 Stantec boreholes were advanced roughly 1,000 feet downstream of the current dam alignment, so the subsurface conditions along the dam alignment are not precisely known. Some subsurface data is also available from an investigation performed by Terracon (2011) at the Creekside Highway, approximately 2,000 feet upstream of the dam location.

The Stantec investigation included five borings, one on each riverbank, and three in the river bed. The boring (J-5) on the left riverbank showed clean to silty sand overlying shale bedrock; the borings (J-1 and J-2) on the right riverbank showed poorly graded sand with some silt overlying shale bedrock. The two borings (J-3 and J-4) in the riverbed showed clean to silty sand with varying amounts of gravel overlying shale bedrock. Like the Sand Springs location, the borings were not surveyed elevations of the borings and the bedrock were based values from the surveyed topographic maps created for this project, using the handheld GPS coordinates in the Stantec report. Based on the Stantec logs, the top of bedrock was estimate at an elevation of 584 on the left abutment, at an elevation 584 to 587 in the riverbed, and at an elevation of 579 to 588 on the right riverbank.

The Terracon (2011) report for the Creekside widening included the foundation data sheet stick logs for the original Oklahoma Department of Transportation borings. It was noted that all of the investigations for the Creekside Bridge showed interbedded silt and sand alluvium with minor amounts of clay overlying shale at the abutments; in the riverbed, sand was reported to directly overlie the shale. The top of bedrock elevations in 20 borings conducted by ODOT (1994) for the original bridge design showed elevations of about 592 on the left abutment, elevations of 583 to 586 on the right abutment, and values between 583 and 588 in the riverbed. The average elevation of the top of the bedrock was 586 feet. The Terracon investigation in 2011 included 18 additional borings adjacent to the ODOT borings and showed very similar elevations and subsurface materials to the original ODOT investigation.

The Kleinfelder borings along the floodwall alignment included eight borings. Like the Stantec borings, the Kleinfelder borings were only located in the field with a handheld GPS. The elevations of the subsurface materials for these borings should only be considered estimates. The borings all generally showed sandy alluvium overlying shale along the alignment. Two borings (B-03 and B-07) showed a one to 2.5-foot thick layers of lean to fat clay overlying the shale bedrock. The top of the bedrock was reported to vary between about elevation 573 and 584 feet.

The Stantec report included laboratory index test results for the soil samples (Atterberg limits and grain size analyses) and unconfined compression tests in collected rock cores. The Kleinfelder and Terracon data provide grain size analysis results for select soil samples, and Texas Cone Penetration (TCP) testing in rock. The Kleinfelder report also included Atterberg limit testing on one of the clay layers. The ODOT data includes only TCP results. The TCP testing provides an indication of strength, but does not provide a direct strength measurement or interpretation of rock mass properties for seepage control or interface strength. The testing does provide a useful indication of the variation in density with depth.

The existing investigations, particularly at the Creekside Turnpike, support that the bedrock surface elevation is roughly uniform across the proposed dam site. The previous information indicates that the upper few feet of the bedrock may be weathered.

3.4.3 Bixby

Terracon (2009) performed a geotechnical exploration near the proposed Bixby Dam site for an unrelated pipeline project. As part of their investigation, Terracon advanced one borehole on the east bank (B-5) and one on the west bank (B-1) of the river. Boring B-1 is located approximately 2,000 feet downstream of the dam location, and Boring B-5 is located approximately 1,000 feet downstream of the proposed dam location. Boreholes were also planned in the riverbed, but were not conducted due to high water levels. As a result, no boreholes are available in the river bed near the proposed Bixby location.

The boreholes were advanced by standard penetration testing through the soil and rock in both borings. Boring B-1 also included NX diamond coring in the shale at deeper depths. Laboratory analyses of the soils was limited to moisture contents and sieve analyses. The Terracon report indicates that Borings B-1 and B-5 locations and elevations were determined based on the initial staked locations provided by a presumed surveying firm.

Borehole B-1 (west bank) encountered alluvial soils (silt and poorly graded sand with silt) overlying a layer of shaley lean clay at elevation 571 feet overlying shale bedrock at about elevation 566.5 feet. The shale bedrock was described as dark gray, soft to moderately hard rock. Boring B-5 (east bank) encountered a layer of topsoil and clay fill for the upper 6.5 feet (to elevation 591.5 feet) overlying clayey sand and well graded sand to elevation 570.5 feet where shale was encountered. The shale was described in boring B-5 as dark gray and moderately hard.

The Highway 64 Bridge plans (ODOT, 1984) showed stick logs of the nineteen borings conducted for the bridge piers. With the exception of one log, all of the bridge pier investigations showed alluvial materials (sand and gravel) overlying shale bedrock. The top of the shale was encountered at about elevation 571 feet on the west bank and generally incrementally increased in elevation to the east bank to reach an elevation of about 572.5 feet. One boring on the west bank (B-2) showed sandy clay overlying shale bedrock. The bridge is located over 5,000 feet upstream of the planned dam location, however, the available data shows very similar conditions to the other bridge locations upstream of Bixby – sand and gravel alluvium overlying shale bedrock with little elevation change in the bedrock.

The clayey layer found in the Terracon boring B-1 is assumed to be residual soil derived from the weathering of the underlying parent shale bedrock. Based on the data available for the Highway 64 bridge along with the data reviewed for the Sand Springs and South Tulsa/Jenks locations (discussed above), any residual clayey layers overlying the parent shale bedrock are assumed to have been removed in the river channel by active scouring forces.

While there are no nearby data in the river channel to the Bixby location, the Highway 64 bridge information suggests that the elevation of the top of shale is relatively uniform across the river channel, similar to the other dam locations, including Zink. The top of the shale was conservatively assumed to be at elevation 567 feet (the lower elevation encountered by Terracon), and based on the information at the other dam locations, the upper four feet was assumed to be highly weathered and would need to be removed. The bottom of the dam foundation would therefore be placed at about elevation 563 feet. The top of the sediment in the riverbed was assumed to vary between approximately elevation 572 and 580 in the riverbed, and was conservatively assumed to be at elevation 580 feet for design purposes.

3.4.4 Groundwater Conditions

Groundwater data or seasonal trends were not available for the project. For purposes of schematic design, groundwater elevations within the Arkansas River Corridor were assumed to correspond closely with river levels.

Geotechnical Analysis

4.1 Subsurface Profiles for Design

As noted previously, the shale bedrock is expected to provide suitable foundation conditions for the proposed dams. It is recommended at that the alluvial overburden materials, residual clay soils, and soft weathered shale be excavated to expose an adequate foundation surface at each new dam site. The amount of excavation is not precisely known, but estimated as 4 feet below the estimated top of rock elevation for purposes of preliminary design. The rock excavation also provides a buffer against the possible undulation of the bedrock surface, which may be possible according to available subsurface information. Excavation of this material also provides increased scour resistance if the soft shale erodes below the downstream toe. A summary of the recommended subsurface elevations for each location is provided in Table 2.

TABLE 2
Summary of Key Subsurface Elevations for Geotechnical Analysis

Dam	Top of Sediment (feet)	Top of Shale Bedrock (feet)	Dam Foundation Elevation (feet)	Total Dam Height (feet)
Sand Springs	628	615	611	27.5
South Tulsa/Jenks	592	584	580	17.5
Bixby	580	567	563	20.5

Notes: Total dam height is based on the fixed crest elevation provided in Table 1.

4.2 Structure Loading for Design

Once the dams are constructed, sediment will be deposited against the upstream face of the dams. The sediment loading will vary but gate operations will keep the sediment levels at or below the sill elevations for each section of the dam. It is noted that the failure mechanism of Obermeyer gates is in the “down” position, which, in addition to the redundancy provided by multiple gates prevents and extreme sediment load from accumulating above this elevation.

The operational maximum sediment levels are summarized in Table 3 below. The estimation of lateral earth pressure distributions from accumulated sediment is described later in this section. Elevations were estimated considering the sediment observations at the existing Zink Dam, for which sediment levels are naturally maintained at a depth of 2 feet below the crest of the dam. For the gated sections, sediment is assumed to accumulate to the sill of each gate.

In addition to sediment, the low water dam structures are assumed to be subjected to loadings from earthquakes, floods, ice, and hydrostatic uplift. The approach used to estimate these loads is described later in this section.

TABLE 3
Summary of Key Water and Sediment Elevations for Stability Analysis

Dam	Fixed Crest		Crest Gate		Full Height Gate	
	Top of Water (ft)	Top of Sediment (ft)	Top of Water (ft)	Top of Sediment (ft)	Top of Water (ft)	Top of Sediment (ft)
Sand Springs	638.5	636.5	638	633	638	628

South Tulsa/Jenks	597.5	595.5	597	592	597	590
Bixby	583.5	581.5	n/a	n/a	583	579

Notes:

1. Values for water and sediment were provided by the schematic design team.

4.3 Sliding and Overturning Stability Analysis

The proposed low water dam geometries were evaluated for overall sliding and overturning stability. Two-dimensional stability evaluations were performed for the cross sections at each dam (fixed crest, crest gate, and full-height gate as applicable for each dam). Stability evaluations were performed in general accordance with USACE criteria for evaluation of concrete structures as described in EM 1110-2-2100 *Gravity Dam Design*. In this engineering manual, the USACE recommends 7 fundamental loading conditions for analysis. For this project, every one of these analyses was not performed, since several could be discounted by inspection as not critical for design in this setting. The details of this are summarized later in this section.

The USACE groups the fundamental stability analysis cases into three categories and recommends minimum factors of safety for each. These cases are *usual*, *unusual*, and *extreme* with corresponding minimum sliding factors of safety or overturning stability criteria. The recommended design criteria are summarized in Table 5.

TABLE 5
Sliding and Overturning Stability Analysis Criteria (from USACE EM 1110-2-2100 *Gravity Dam Design*)

Load Condition	Overturning Resultant at Base	Minimum Sliding Factor of Safety
Usual	Middle 1/3	2.0
Unusual	Middle 1/2	1.7
Extreme	Within Base	1.3

Abutment configurations and dam end-walls have not been designed at this time and were not included in these evaluations. Future abutment designs will need to consider seepage, scour, and stability needs at these locations within the floodway.

Sliding and overturning stability analyses were performed for each section of each dam. Analyses were prepared for the critical loading conditions as described in the following subsections. The critical loading condition for each dam was the Normal Operating case, however, the Normal Operating Case with OBE and the Normal Operating Case with MCE loadings were also evaluated. The calculations for each dam section are attached to this report as follows:

- Attachment 4: Sand Springs Fixed Crest
- Attachment 5: Sand Springs Crest Gate
- Attachment 6: Sand Springs Full Height Gate
- Attachment 7: South Tulsa / Jenks Fixed Crest
- Attachment 8: South Tulsa / Jenks Crest Gate
- Attachment 9: South Tulsa / Jenks Full Height Gate
- Attachment 10: Bixby Fixed Crest
- Attachment 11: Bixby Full Height Gate

4.3.1 Load Condition 1: Construction Case

The construction case considers the risks associated with failure of the structure during construction phase loadings before the reservoir is filled. This case includes the dam structure completed with no headwater or tailwater. For this project this also means no sediment loading. This is an unusual loading condition for which the applicable overturning resultant must fall within the middle 1/2 of the structure base, and the sliding factor of safety must be greater than 1.7.

With no water or sediment loading, there are no net horizontal loads applied to the completed structure and no risk of sliding or overturning. On a properly prepared horizontal bedrock surface, there is no lateral loading. Considering the details of this condition, an analysis was not completed, since the criteria are satisfied by inspection.

4.3.2 Load Condition 2: Normal Operating Case

The normal operating case considers the risks associated with the normal operation of the dam structure. In this case the pool elevation is at the top of the closed spillway gates, the tailwater is at the minimum elevation (the foundation elevation of the dam), uplift is considered, ice pressure is neglected in this case, and sediment pressures are considered. Two scenarios were analyzed to check the normal operating case: a full reservoir with no sediment, and a full reservoir with the maximum sediment levels shown in Table 3. This is a usual loading condition, requiring a factor of safety of 2.0 against sliding and an overturning resultant acting in the middle 1/3 of the dam footprint.

Of all the load cases examined, the normal operating case, with a required factor of safety of 2.0 against sliding, was found to control over other critical cases. As considered, the full sediment case was found to control over the hydrostatic only case; thus the critical case was found to be the normal operating case with full sediment accumulation.

For each dam, it was found that one or more sections did not satisfy the USACE criteria for the normal operating case; the most critical case being the fixed crest section of the Sand Springs Dam. Permanent pre-stressed ground anchors were found to be necessary to stabilize the dams for this loading condition. Pre-stressed anchors can be installed in the structure and post tensioned to achieve the required minimum factor of safety. Preliminary anchor configurations were determined for each section of each dam to achieve the critical loading case (Load Condition 2).

4.3.3 Load Condition 3: Flood Discharge Case

The flood discharge case considers the risks associated with the operation of the dam under the standard project flood case, which is conservatively taken as the 100 year flood. In this case, the gates are assumed to be lowered and the flood waters are passing over the dam with nearly-equal elevations on both sides. In this case the only net static lateral load on the structure is the sediment loading, which, by inspection of the analysis of the normal operating case, is a less severe loading condition than that applied during the normal operating condition, and which requires a factor of safety of 2.0. This is an unusual loading case and requires a factor of safety of 1.7 against sliding, and an overturning resultant acting in the middle 1/2 of the dam footprint. Considering this, the flood discharge case was considered not to be a critical loading case, and a formal calculation was not prepared, so long as the dam satisfies the normal operating case. Because of this, if the normal operating case is satisfied, thereby so too is Load Condition 3.

4.3.4 Load Condition 4: Construction Case with OBE

The construction case with OBE considers the risks associated with the extreme case whereby the completed dam has not yet been filled and the operations basis earthquake is applied in the upstream direction. No headwater or tailwater is considered in this case, and similarly no sediment loading is considered. Although this is an extreme loading condition, requiring a sliding factor of safety of 1.3 and an overturning resultant acting anywhere in the base of the structure, the operations basis earthquake loading is low and there are no other lateral loadings applied to the structure. The loading is applied in an unconventional direction (upstream), however, for these structures it is noted by inspection, that the pseudostatic loading (which acts through the centroid of the structure) will not result in sliding or overturning of the structure. This loading condition is not a critical loading condition and a formal calculation was not prepared; Load Condition 4 is considered to be satisfied so long as the normal operating case and operating OBE case (discussed below) are also satisfied.

4.3.5 Load Condition 5: Normal Operating Case with OBE

Load Condition 5, the normal operating case with the OBE is identical to the normal operating case, but with the added operations basis earthquake loading in the downstream direction. This is an unusual loading condition, and as such the required factor of safety against sliding is 1.7 and the overturning resultant must fall in the middle 1/2 of the structure base. A formal calculation was prepared to analyze this loading condition which could not, by inspection, be discounted. It was found that the low acceleration imparted by the OBE (which was found to be identical for all proposed structures) does not represent a critical loading case. As noted with other loading conditions, it was found that if the structure satisfies the normal operating case (usual condition) then Load Condition 5 should also be satisfied.

4.3.6 Loading Condition 6: Normal Operating Case with MCE

Loading Condition 6, the normal operating case with the MCE is similar to loading condition 5 and identical to the normal operating case, except for the addition of the maximum credible earthquake loading acting in the downstream direction. This is an extreme loading condition, and as such the required factor of safety against sliding is 1.3 and the overturning resultant must fall anywhere within the base of the structure. A formal calculation was prepared to analyze this loading condition which could not, by inspection, be discounted. It was found that the relatively low acceleration imparted by the MCE does not represent a critical loading case for the proposed low water dam structures. As noted with several other loading conditions, it was found that if the structure satisfies the critical normal operating load case, then it will also satisfy Loading Condition 6.

4.3.7 Loading Condition 7: Probable Maximum Flood (PMF) Case

Loading Condition 7 is identical to loading condition 3 (the flood discharge case), with the exception that the probable maximum flood (PMF) flow is imparted on the structure. This is an extreme loading condition requiring a factor of safety against sliding of 1.3 and an overturning resultant falling anywhere within the base of the structure. The probable maximum flood was not explicitly considered for this load case, since at such a large flood event the dam is completely overtopped with headwater and tailwater elevations being approximately equal. By inspection, this loading condition involved only the lateral earth pressure loading from the impounded sediment, and no additional horizontal loading. The net horizontal loading is less than the normal operating case, and the criteria is also less—thus, by inspection, if the structure satisfies the normal operating load case, then it will also satisfy Loading Condition 7.

4.4 Structure Loads

This section summarizes the rationale and assumptions used in the estimation of the loads applied to the low water dam structures for evaluation of sliding and overturning stability.

4.4.1 Hydrostatic Loading

Hydrostatic loading on the low water dam structures was estimated using a triangular pressure distribution assuming the water surface coincides with the crest of the dam and extends downwards to the foundation elevation. The unit weight of water was assumed to be 62.4 pounds per cubic foot. The resultant of this pressure distribution acts horizontally at 1/3 the height of the dam.

The dam structures are designed to be overtopped during flood events at each section. It is assumed that during overtopping scenarios that the height of water flowing over the upstream face of the dam will be approximately equal to the height of water at the downstream face of the dam, and that the net hydrostatic loading will act in the downstream direction, and be approximately equal in magnitude to the full height reservoir with no tailwater.

4.4.2 Lateral Earth Pressures from Sediment

Sediment loading was applied to the upstream face of the dam assuming at-rest lateral earth pressures. The sediment was assumed to be fully submerged, and to contribute a triangular lateral earth pressure distribution which extended from the levels summarized in Table 3 down to the top of shale bedrock. The shale bedrock was assumed to be self-supporting with concrete cast directly against the rock. The rock was assumed to contribute no net lateral loading to the structure. The sediment is assumed to be consistent with the sandy material currently observed in the river corridor, and to have a total unit weight of 120 pounds per cubic foot, an internal friction angle, ϕ', of 28 degrees, and zero cohesion. This assumption is consistent with the USACE recommendations for silt loading as published in EM 1110-2-2200 *Gravity Dam Design.* The at-rest earth pressure coefficient was estimated as $1-\sin(\phi')$ or 0.53. This estimate was used consistently for all analyses for all dams to estimate the lateral earth pressure contribution of the buoyant sediment.

4.4.3 Scour

For preliminary design it was assumed that sediment would accumulate on the upstream side of the dam and would be scoured from the downstream side of the dam. Additionally, it was assumed that the soft weathered shale bedrock exposed at the downstream toe of the dam would scour. Scour was assumed to advance to the foundation elevation of the structure and the resistance provided by these materials was not used for stability evaluations of the dams. A seepage and scour cutoff is incorporated into the structure which extends 3 feet below the foundation elevation on both the upstream and downstream sides of the dam.

4.4.4 Ice Loading

Ice loading is considered to be possible in the floodway corridor, however, it was assumed that ice loading would not represent a critical loading case. Ice loading on the structure was not included in sliding and overturning calculations. Final design tasks, when authorized, should confirm this assumption.

4.4.5 Seismic (Pseudostatic) Loading

Seismic forces were applied to the dam cross sections for evaluation of the OBE and MCE loading cases. Seismic forces were estimated by estimating the horizontal pseudostatic coefficient, kh, as discussed in Section X, *Seismicity*. The pseudostatic coefficient was then multiplied by the mass of the dam and overlying pedestrian bridge, if applicable, to estimate the horizontal pseudostatic loading on the structure. The pseudostatic loading was assumed to act through the centroid of the low water dam structure. Pseudostatic loading could act in either direction, but was applied in the downstream direction for the sliding and overturning analysis.

4.4.6 Hydrodynamic Loading

Hydrodynamic loading is generated by the acceleration of the dam structure into the impounded free water on the upstream side of the dam. This loading could act in either direction during a seismic event, but was assumed to act in the downstream direction and quantified as recommended by the USACE EM 1110-2-2200 *Gravity Dam Design.* The hydrodynamic loading was estimated using the Westergaard method, as represented by the following equation:

$$P_E = (7/12)\, k_h \cdot \gamma_w \cdot h^2$$

Where P_E is the horizontal resultant of the overall hydrostatic pressure distribution, k_h is the horizontal pseudostatic coefficient, $\cdot\gamma_w$ is the unit weight of water, and h is the height of free water acting against the structure. The resultant of this pressure distribution acts at a height of 0.4*h above the reservoir. This force is generated only in free water, and is not applied in saturated materials below the sediment or bedrock.

4.4.7 Hydrostatic Uplift

The USACE recommends consideration of hydrostatic uplift pressure estimation considering the maximum upstream storage pool elevation with the minimum tailwater. In each case, for this Project, this condition was estimated assuming headwater elevations equal to the crest elevation or top of gate elevation on the upstream face, and tailwater elevations equal to the dam foundation elevation at the downstream toe. The full hydrostatic pressure was assumed to dissipate uniformly across the base of the dam in a triangular distribution. This dissipation is modelled as a hydrostatic uplift force acting at 1/3 of the base width downstream of the upstream heal of the dam, and reduces the effective bearing pressure of the structure.

Note that concrete turndowns are included on the upstream and downstream ends of the structure base. These turndowns are expected to reduce the under seepage and limit the uplift pressures beneath the dam. The exact reduction and long term performance of this cutoff is difficult to quantify, and is neglected for purposes of preliminary analysis.

4.4.8 Base Friction

The base friction along the foundation of the structure is the only resisting force considered in structure sliding calculations. The interface strength between mass concrete and the underlying shale bedrock is not precisely known, and laboratory testing to evaluate candidate interfaces strengths has not been performed. For purposes of this preliminary design, the interface strength was selected using typical values for mass concrete cast against weak rock or stiff clay, as summarized in NAVFAC DM7.2. The interface strength was selected from the middle of the range for these materials and a value of 24 degrees with zero cohesion was used. It was assumed that under long-term seepage conditions beneath the dam, water would soften the shale materials at the dam interface and reduce the cohesion.

The interface friction between shale and concrete is highly variable in the literature. The base friction value for the dams should be reviewed during future design efforts.

4.4.9 Anchorage

Sliding stability analysis results indicate that additional horizontal resistance is needed to provide an adequate factor of safety to satisfy USACE criteria. The required anchor force varies for each dam and for each section of each dam. Pre-stressed ground anchors are recommended to provide additional horizontal resistance. The use of shear keys was considered but found to require significant additional excavation in rock below the groundwater table to achieve the required resistance. The Anchors were assumed to be installed inclined at 45 degrees from horizontal, downward in the upstream direction. The anchors would be multiple corrosion protected anchors grouted into the underlying shale and locked off in tension against the low-head dam structure.

The Post Tensioning Institute (PTI) provides industry-standard recommendations for preliminary sizing of pre-stressed rock and soil anchors. For anchors installed in soft shale, a typical ultimate grout-to-ground bond strength of 50psi is recommended. The allowable bond strength of pre-stressed anchors was estimated using the ultimate bond strength, a factor of safety of 2.0, and an assumed bond zone diameter of 6 inches. Bond lengths were limited to 20 feet and anchor spacings were adjusted accordingly. The preliminarily –selected anchor sizes varied between 1.75 and 2.25 inch-diameter 150ksi steel anchors with multiple corrosion protection casing, similar to the All-Thread bars manufactured by the Williams Form Engineering Company. Anchor spacings were found to vary between 5.0 and 10.0 feet on-center. Global stability evaluations of the anchors was not considered for the preliminary analysis, but an unbonded length of 20 feet is preliminarily recommended for design. The anchor size, spacing, and bond zone configuration can be optimized during final design to economically meet stability requirements and to facilitate constructability. Note that some sections are not expected to require anchors. Depending on final design recommendations, particularly with respect to the formal evaluation of base friction strength and overall structure geometry, the required anchor force is expected to change from that estimated in this preliminary evaluation.

Preliminary anchor forces estimated for the various sections are summarized in Table 4 below.

Dam	**Preliminary Required Allowable Anchor Force, kips/linear foot**		
	Fixed Crest	**Crest Gate**	**Full Height Gate**
Sand Springs	10	13	28
South Tulsa / Jenks	0	2	7
Bixby	4	N/A	13

Design Recommendations

5.1 Low Water Dam Geometry

The overall geometry for the low water dams is shown in the drawings. The overall geometry of the structure and crest is controlled, primarily, but hydraulic considerations. Geotechnically, it is recommended that the structures be founded entirely on the underlying shale bedrock. Bedrock elevations for design were estimated based on review of the available subsurface information. Due to uncertainty in the bedrock elevation, its variation across the nearly 2,000 foot dam alignment, and its degree of weathering, it is recommended that the dam foundation be placed approximately 4 feet below the top of rock. This foundation elevation is a critical parameter and should be expected to be revised, upward or downward, based upon reconsideration of geotechnical data collected to fill the geotechnical data gaps described herein.

The prepared shale foundation will provide suitable bearing capacity and resistance to structure settlement. Although future site-specific geotechnical investigations conducted in support of dam design will confirm this, the literature suggests that the local shale units are generally massive and of low permeability. As such, under-seepage through the shale foundation or through rock joints is assumed to be acceptable so long as a tight contact is achieved between the structure and the foundation. This contact will be achieved in part by concrete turndowns, which are recommended below both the upstream and downstream ends of the structure foundation. These turndowns should extend 3 feet below the dam bottom elevation. The downstream turndown also to resist scour forces below the dam.

The schematic design team conducted preliminary evaluations for the proposed dam cross-sections under the critical design load cases as established by the U.S. Army Corps of Engineers (USACE). These evaluations indicate that the new dam structures shown in the drawings are stable under anticipated static, flood, and seismic loading cases. The sliding forces developed by large net hydrostatic and sediment loadings on the upstream face of the dam structures will require installation of pre-stressed ground anchors to meet required factors of safety for structure stability.

5.2 Low Water Dam Constructability

Construction of the low water dams will require a large earthwork and mass concrete construction effort in the active river corridor and floodway. A detailed constructability review has not been conducted, however, it is considered that each low water dam will be constructed in segments which are cofferdammed off from the rest of the river to protect against worksite inundation. Flood control during the construction period will be an important risk-management consideration during the construction process.

The overburden materials will need to be removed to expose the top of shale bedrock along the dam alignments. The overburden materials can be removed by large earth moving equipment, likely bulldozers. This overburden can potentially be arranged in berms for cofferdamming purposes. There is little vegetation in the river corridor, and little-to-no topsoil stripping is anticipated.

The shale bedrock for foundation preparation is expected to be performed with a large bulldozer, Caterpillar Model D8 or equivalent. Removal of soft and weathered shale is generally expected to be accomplished with a blade on the bulldozer. In places, if harder layers of share are identified overlying softer or undesirable shale materials, the use of a single-tooth ripper map be necessary. Careful foundation inspection by a geotechnical engineer or engineering geologist is necessary to confirm suitability of foundation materials prior to placement of dam materials. The foundation surface should be prepared and cleaned to form a tight bond between the dam structure and the underlying rock. The surface should be rough to promote bond strength, but should flat and with no loose material or sediment which could a seepage path beneath the dam. Concrete turndowns at the upstream and downstream face of the dam should extend a minimum of 3 feet below the foundation surface as shown in the drawings. The groundwater should be actively controlled during all phases of foundation preparation and initial structure construction. Diking or ditching for groundwater control should be performed outside the dam footprint. The overall groundwater in the area is not well understood at this time, but anticipated to correspond to river levels within the Arkansas River Corridor. Due to the low permeability of the shale bedrock, dewatering, if needed, is expected to be accomplished using rows of well-points.

As individual segments of the dam are completed, river diversion through the full height gate sections may be practical, and cofferdams be reconfigured to protect the active work area. All manipulation of river sediments within the corridor should comply with applicable regulations.

Abutment details will require particular detail to provide a tight connection between the dam and the riverbank materials which are abutted. The increased gradients provided by the impounded water behind the dams have the potential to induce piping of low-plasticity fine sands and silts. To guard against this, the dams are recommended to have concrete diaphragm walls which penetrate into the abutment to increase the seepage path length around the abutment. Such diaphragm walls should be, wherever possible, constructed in open cut. The walls should be tied into the underlying bedrock with similar foundation preparation efforts as the dam foundation and be structurally connected to the dam. The diaphragm wall should extend 2 feet above the fixed crest of the dam so that overflow conditions do not “overtop” the embedded diaphragm wall. The diaphragm wall should be backfilled with low permeability material within the excavation. If the local sandy alluvium is confirmed to be compatible with filtration criteria for the identified source material, then a sand filter may not be necessary. Otherwise, a filter zone of fine sand is recommended. ASTM C33 concrete sand is commonly found to be filter compatible with most low-permeability backfill, however, this should be confirmed in final geotechnical design.

Each of the dams are expected to require Careful consideration will need to be given to integration of abutment wall details with the abutments. Details of this connection are beyond the scope if this evaluation, however, general wall concepts and recommendations are provided. All

5.3 Floodwall

The Jenks Floodwall is a significant wall structure with a top elevation of 612 feet and a bedrock elevation of 584 feet—a total grade differential of approximately 28 feet. Note that the anticipated sediment level in the river extends above the bedrock elevation, however this sandy material has the potential to scour to the bedrock and cannot be relied upon for wall design. Although several wall types are feasible at this location, considering the available subsurface data and the conceptual level park design, it is preliminarily recommended that a tied-back soldier-pile-and-lagging wall with reinforced concrete facing will provide a robust solution for this location well suited for the nature of the project. Sheet piles are not recommended for the large floodwall due to the large quantity of steel required and anticipated difficulty achieving embedment into the shale bedrock. A mechanically stabilized earth (MSE) wall would be feasible at this location, but would require mass excavation of native soil down to bedrock, disposal of the soil material, prolonged dewatering efforts, and large quantities of imported backfill material. Reinforced concrete walls or concrete gravity walls are also feasible, but require large quantities of concrete and formwork labor to construct, in addition to the excavation required.

A soldier pile and lagging wall, can be constructed top-down from the existing grade by first drilling through the overburden and into the shale bedrock along the wall alignment. Preliminarily, rock socket embedment of 15-20 feet into shale should be considered until sufficient subsurface data can be collected to support advanced evaluation of lateral earth pressures and sizing of wall elements. Steel H-pile sections are then placed into the rock socket holes and the piles concreted in place; above the bedrock, the holes are backfilled around the pile with lean concrete. Once the soldier piles are installed, excavation can begin on the riverward side of the wall to form the approach channel upstream of the dam and the outlet channel on the downstream side of the dam. The top down excavation will require placement of timber or concrete lagging members between the soldier piles to retain the native soil. Waler beams and tieback anchors will be installed at prescribed intervals as the excavation proceeds. The sizing and number of anchors will depend on the soils present and the loads imparted by the park amenities, but for planning purposes it should be assumed that three rows of anchors are installed in the face of the wall. Note that, in locations where 8 to 10 feet of fill is necessary to bring the west park up to the desired elevation, deadman anchors could be installed in the backfill. Due to the displacement required to develop passive resistance behind deadman anchors, the tolerable movements of the wall at these locations should be carefully reviewed.

Excavation and tieback installation should proceed to the bedrock elevation. Once the wall is excavated and tied back down to the top of bedrock, permanent reinforced concrete facing can be applied to the wall to prevent backfill migration out of the wall and to provide desired architectural details. It is recommended that the lagging and concrete facing be placed down to the depth of potential scour of the shale bedrock—preliminarily estimated to be elevation 580 feet. Drainage provisions should be included in the wall facing to prevent building up hydrostatic pressures behind the wall.

Fill is anticipated behind the proposed retaining wall to bring the grade up to elevation 612. This fill is recommended to be placed after wall construction. The fill material has not been identified at this time, however, the use of excavated sandy alluvium from the foundation excavation for the South Tulsa / Jenks dam or for excavation of an approach channel upstream of the right abutment of the dam may be suitable. Clayey backfill should be avoided.

5.4 Geotechnical Data Gaps

The depth and properties of the bedrock, based on the limited information reviewed, indicate that the material would provide a suitable foundation for the proposed dams and improvements. However, there is still key information that needs to be obtained in order to minimize the potential geotechnical risk at each of the sites. The primary concerns at the three dam sites include identifying the erosion and seepage potential of the shale bedrock upon which the dams will be constructed, quantifying the interface strength between shale bedrock and concrete, determining the depth of weathering, and identifying the locations of possible ancient river channels in the bedrock surface.

Additional geotechnical information on consistency, strength, and grain size should be collected within the overburden at both proposed dam sites. Abutment conditions at each abutment should be inspected, documented, and evaluated upstream and downstream of the proposed dams to develop appropriate abutment termination details, as well as seepage and piping control measures. Standard Penetration Tests (SPTs) should be conducted at regular intervals in boreholes advanced within the overburden. The thickness of the overburden and depth to rock should also be more closely examined and surveyed along specific alignments of the proposed dams.

At the Sand Springs, South Tulsa / Jenks, and Bixby dam sites, additional strength data and characterization of the bedrock are necessary both in the river channel and along the banks in order to determine specific rock mass properties and guide final selection of dam foundation elevations. Borings should be advanced into rock using rock coring methods, in order to determine rock quality, degree of weathering, durability, and strength as a function of depth. CH2M HILL prepared a *Scope of Work* for a preliminary geotechnical exploration at the Bixby Dam site (for which there is no site-specific geotechnical data) in January 2015 to support the schematic design of the dam. This SOW was not executed due to project schedule and adverse winter weather conditions. It is recommended that this scope be executed and the data be reviewed as a preliminary check on the assumptions and recommendations presented herein. This Scope of Work is included in Attachment 12.

Final design will need to include data from additional borings and soil and rock samples to confirm several key items, including:

- Confirm elevation of bedrock along each dam alignment
- Confirm bedrock strength variation with depth to confirm selection of dam foundation elevation
- Complete detailed manual and video logging of the shale to identify bedding and potential seepage pathways
- Estimate shear strength of the shale bedrock
- Estimate interface strength between proposed dam structure and shale bedrock
- Investigate shale durability, including potential for slaking once exposed and potential to scour.
- Investigate soil strength, permeability, and compressibility at the abutments for design of the dam abutment details, retaining walls, scour protection, and other project amenities. An assessment of permeability, density/consistency, compressibility, and strength will be required.
- Additional detailed data will be necessary along the various proposed floodwalls at the South Tulsa / Jenks location to support conceptual park designs and amenities.

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Figures

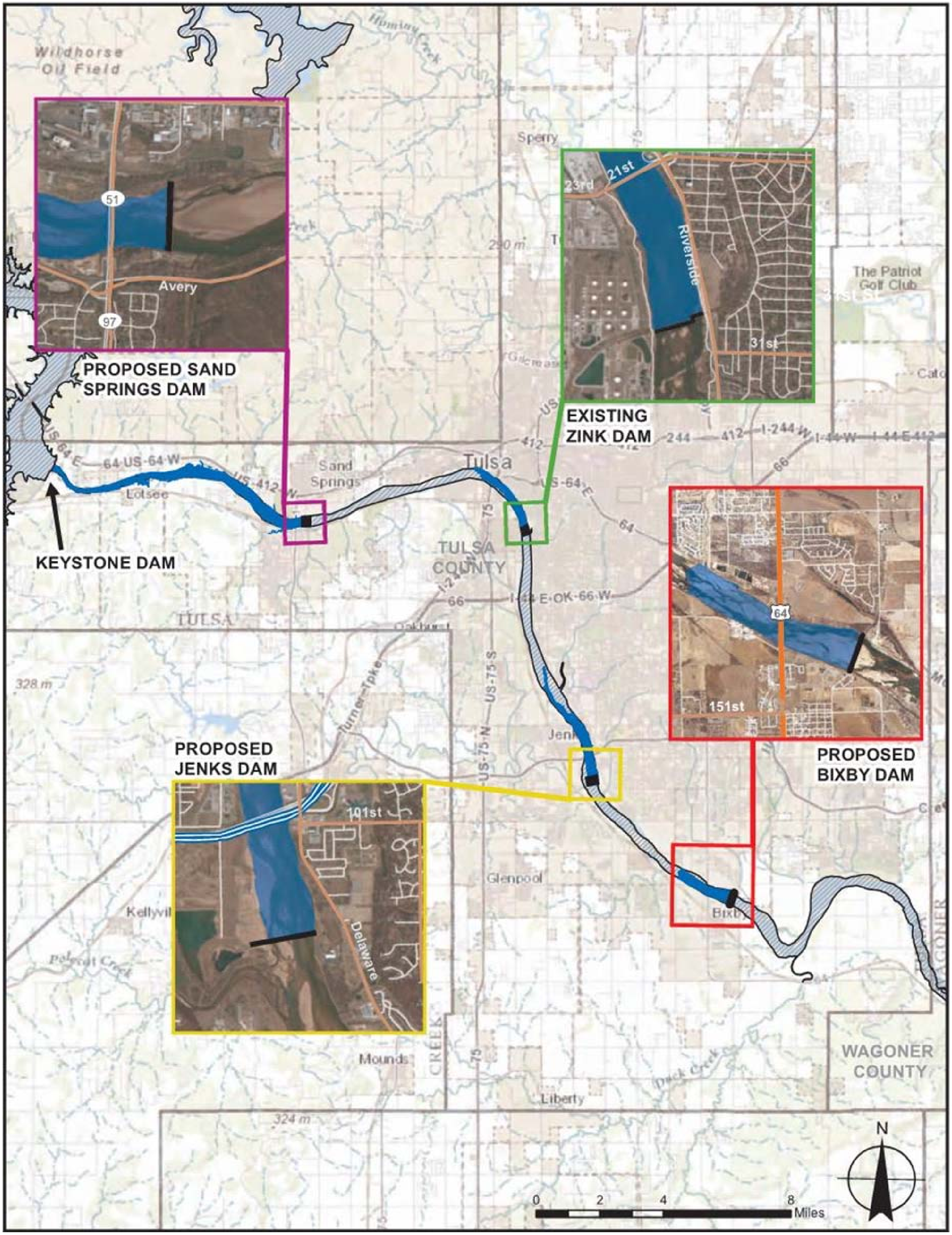
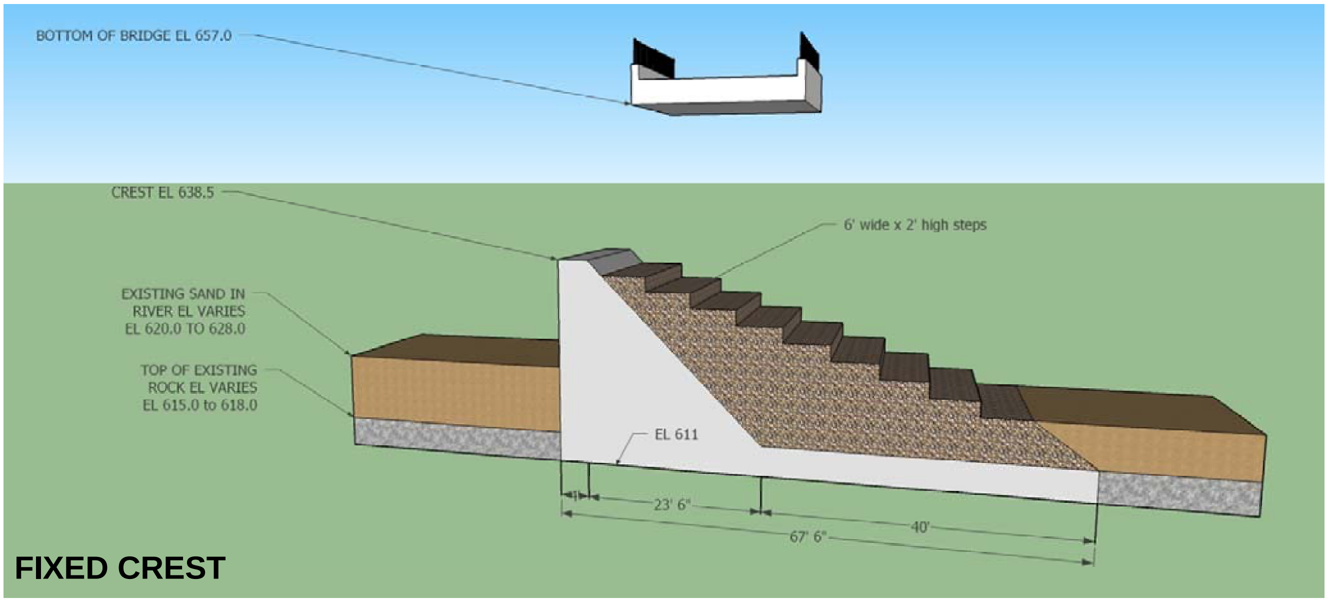
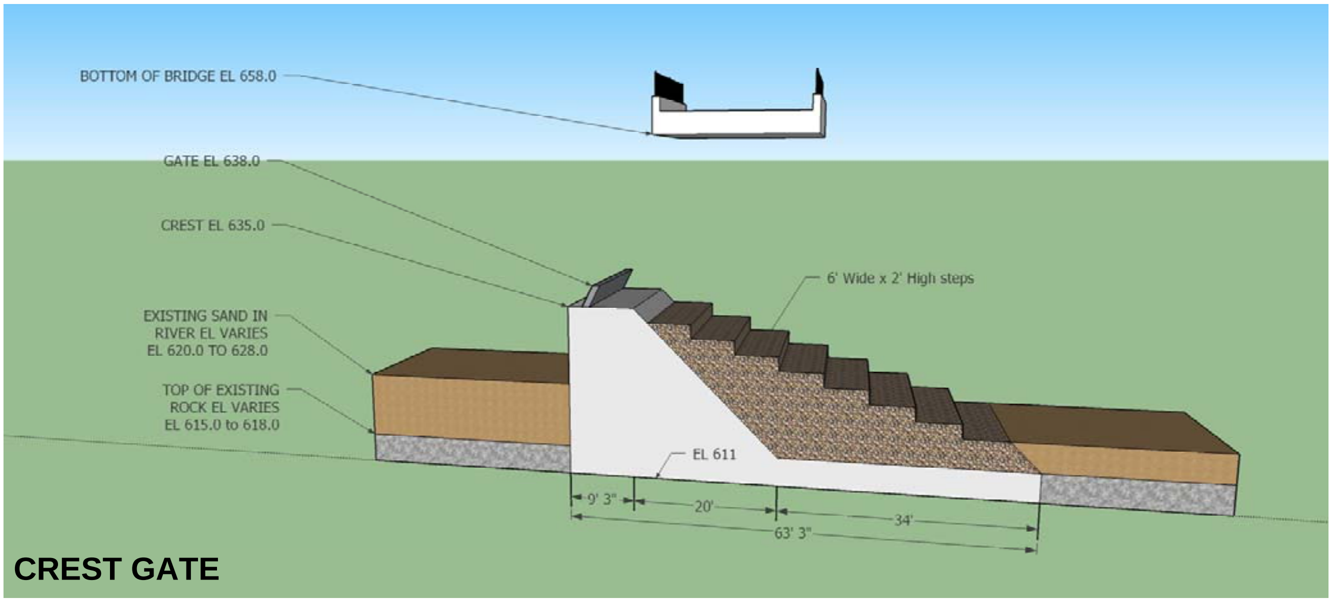


FIGURE 1-1
Locations of Low Water Dams
Arkansas River Low Water Dams – Geotechnical Report

CH2MHILL





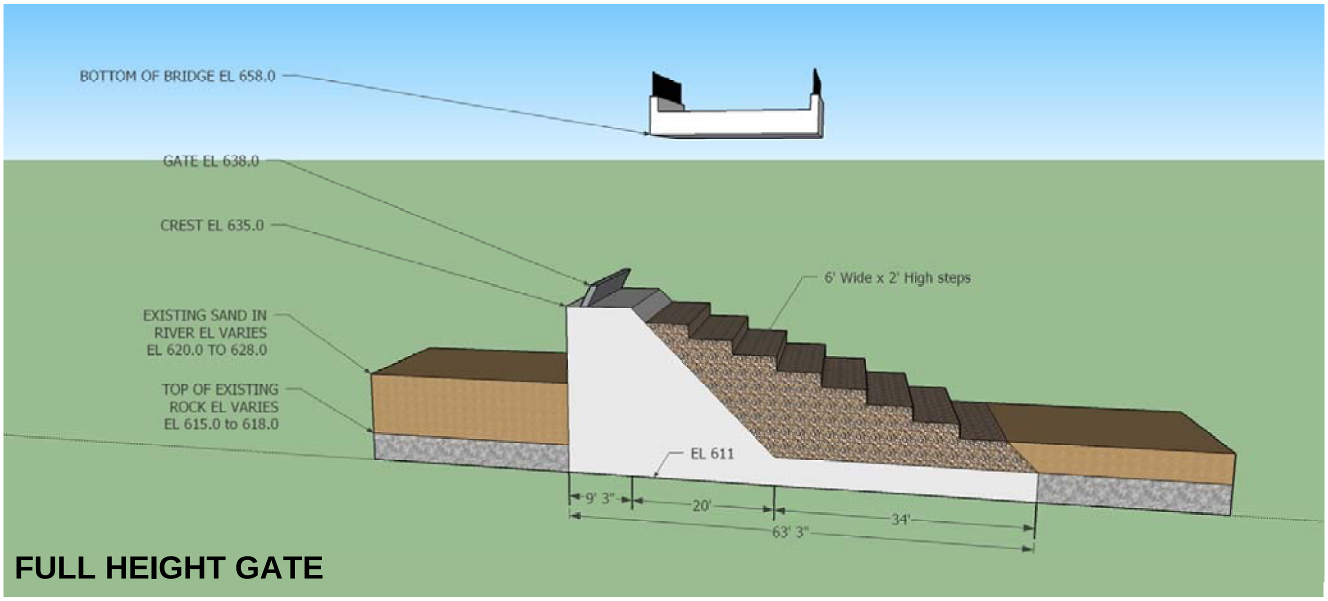


FIGURE 1-2
Sand Springs Dam Concept Sections
Arkansas River Low Water Dams – Geotechnical Report

CH2MHILL

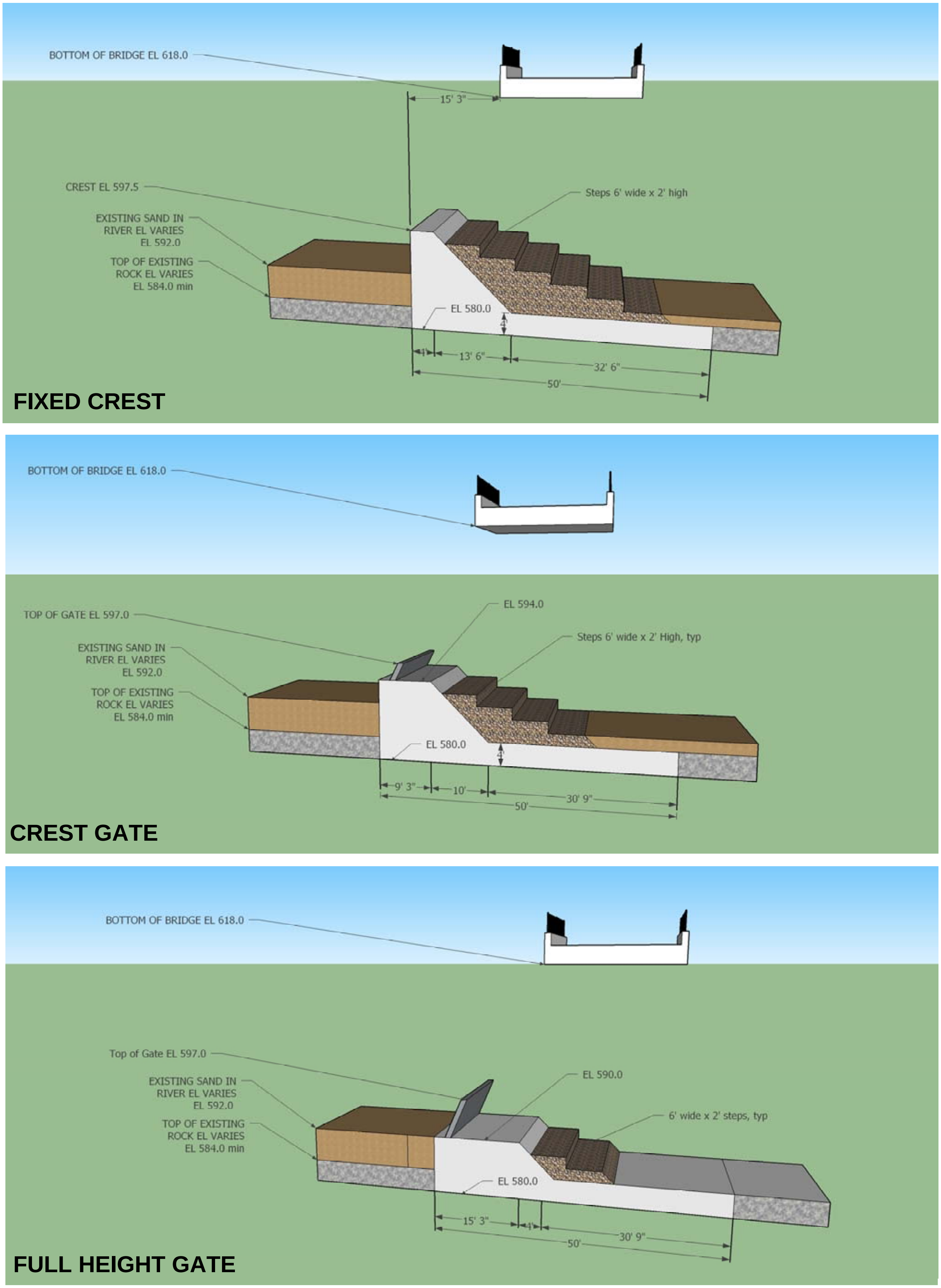


FIGURE 1-3
South Tulsa / Jenks Dam Concept Sections
Arkansas River Low Water Dams – Geotechnical Report

CH2MHILL

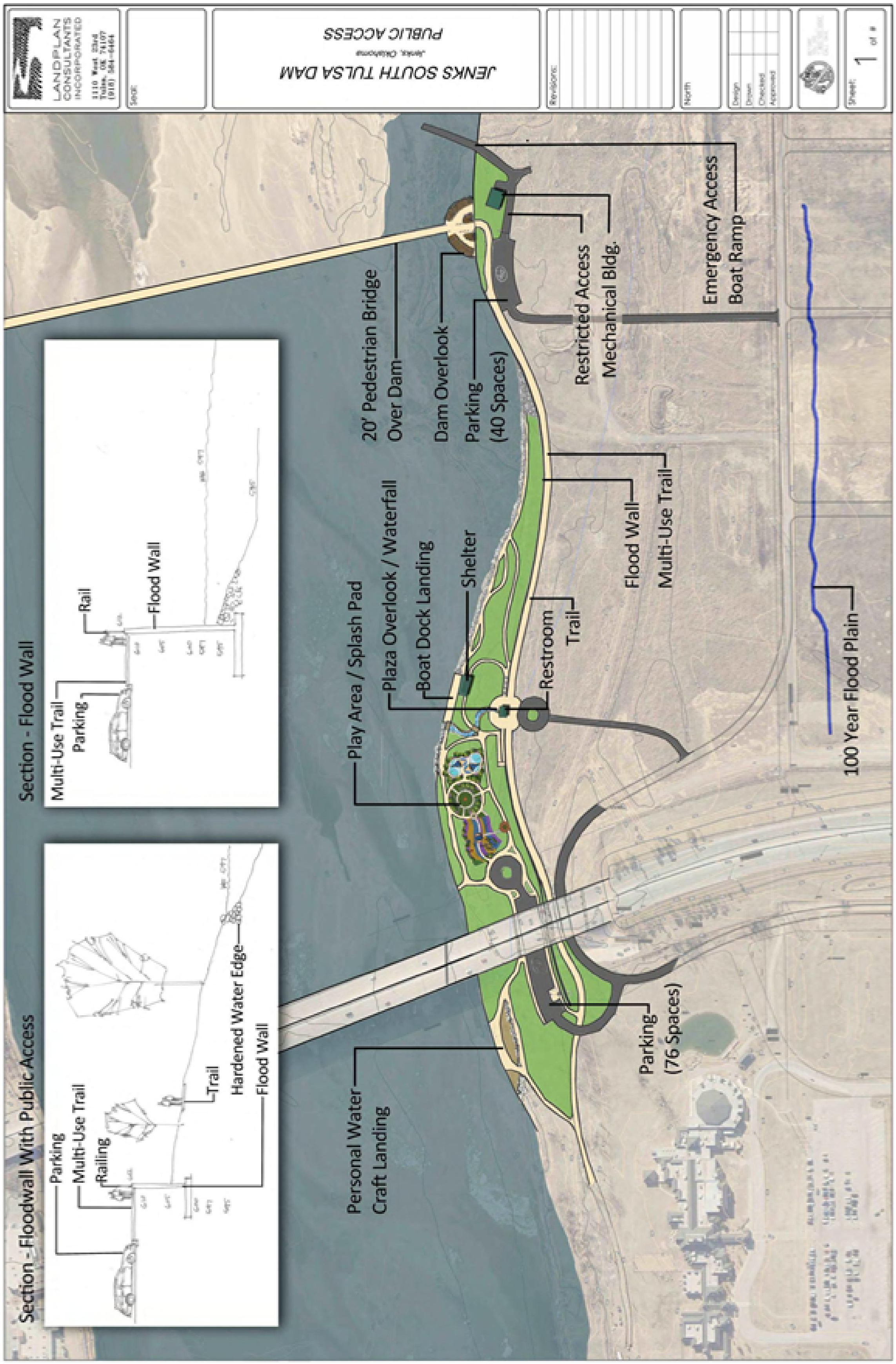


FIGURE 1-4A
South Tulsa / Jenks Floodwall Concept
Arkansas River Low Water Dams – Geotechnical Report

CH2MHILL

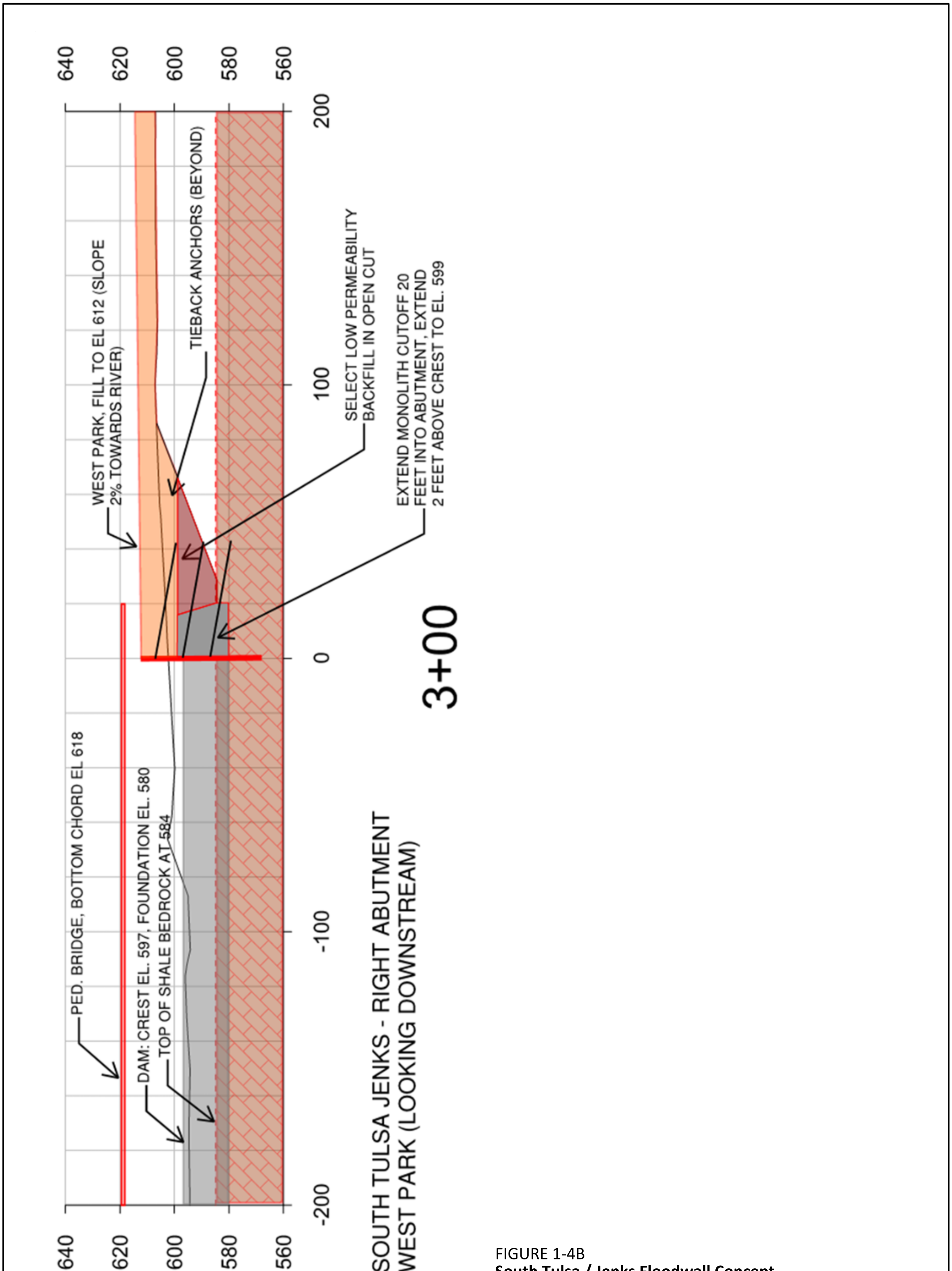
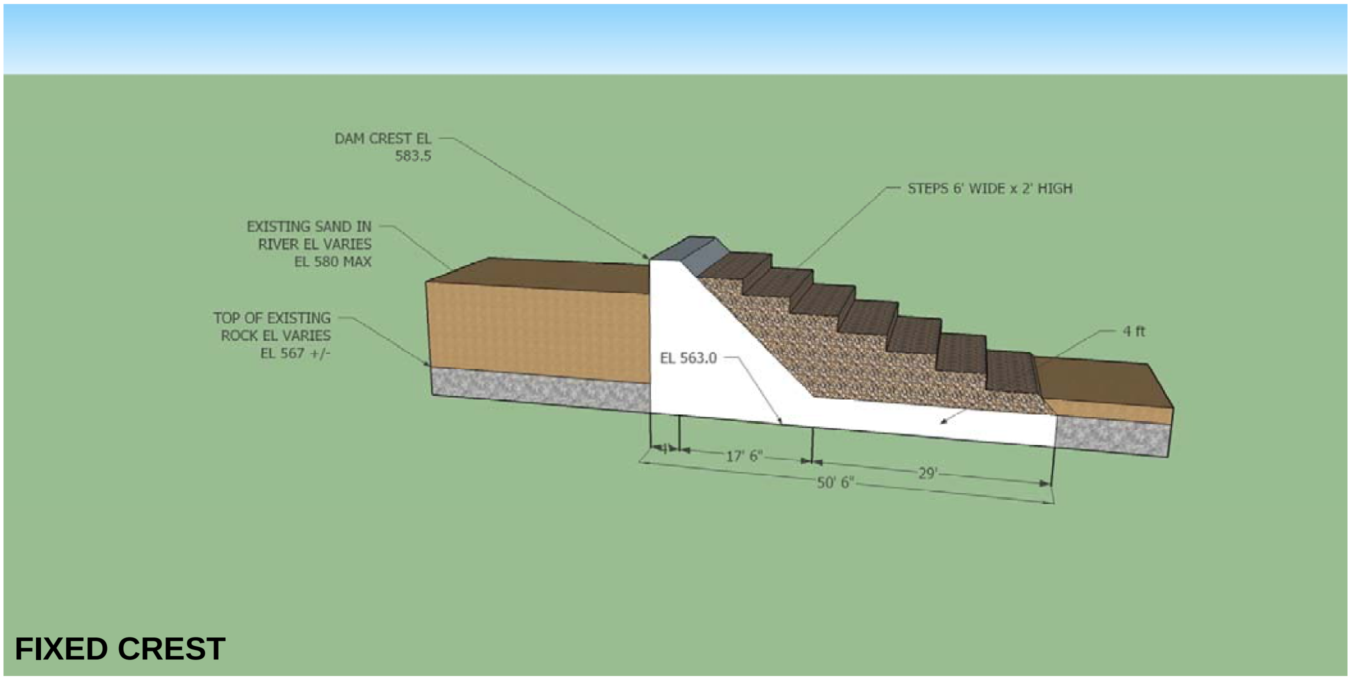


FIGURE 1-4B
South Tulsa / Jenks Floodwall Concept
Arkansas River Low Water Dams – Geotechnical Report

CH2MHILL



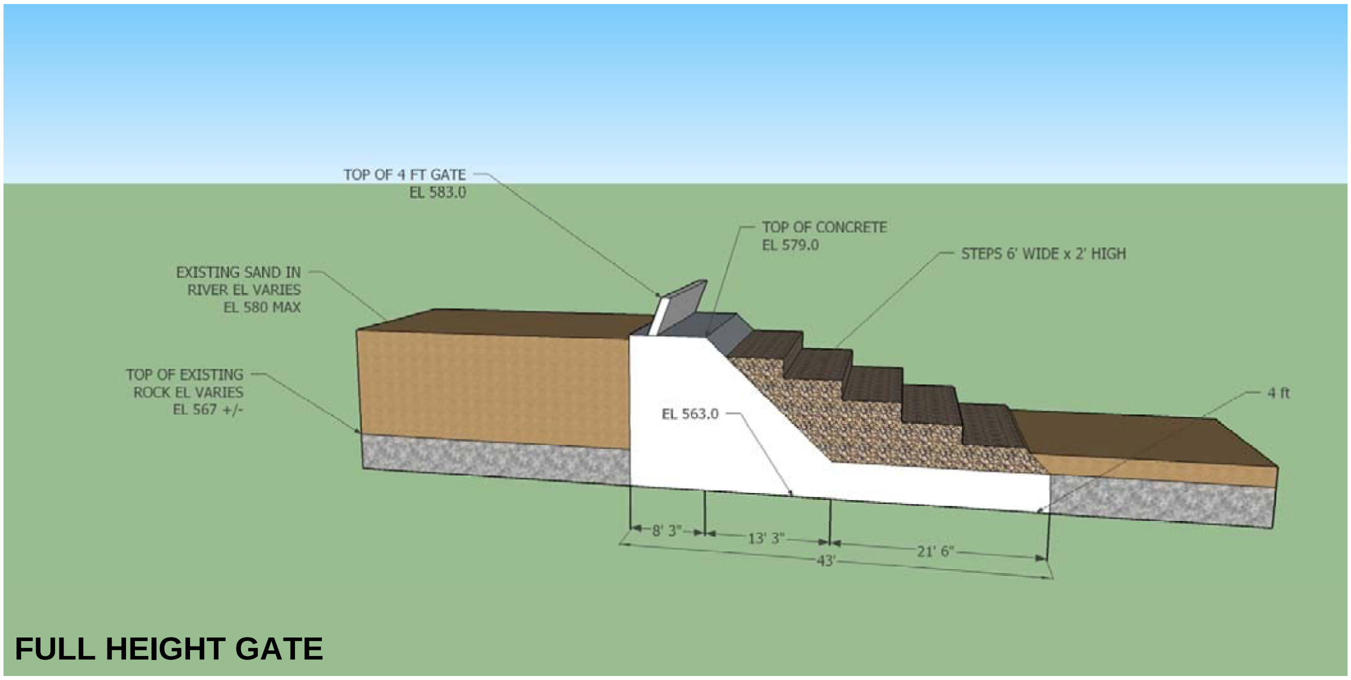


FIGURE 1-5
Bixby Dam Concept Sections
Arkansas River Low Water Dams – Geotechnical Report

CH2MHILL

Attachment 1

Geotechnical Investigation and Testing

Arkansas River Corridor Project
Arkansas River
Contract No. DACW912BV-07-D-1000
Sand Springs/Jenks, Oklahoma

Prepared for
USACE Tulsa District
Tulsa, Oklahoma

May 1, 2008

STANTEC CONSULTING SERVICES INC.

Stantec

Stantec Consulting Services Inc.
1409 North Forbes Road
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Tel: (859) 422-3000
Fax: (859) 422-3100

May 1, 2008 O.1.1.LX2007282R01

Mr. James L. McHenry
Geotechnical Engineering and Dam Safety
USACE Tulsa District
1645 S. 101st E Avenue
Tulsa, Oklahoma 74128

Re: Geotechnical Investigation and Testing
Arkansas River Corridor Project
Arkansas River
DACW912BV-07-D-1000
Sand Springs/Jenks, Oklahoma

Dear Mr. McHenry:

Please find enclosed our project deliverables for our work on the Arkansas River corridor Project, accomplished under Task Order No. 0008, Contract No. DACW912BV-07-D-1000.

This report describes our engineering materials testing of the soils and rock in the vicinity of the proposed Sand Springs and Jenks low head dams. Included are a summary of the boring locations and rock strength testing, full soil classification and unconfined rock strength test results. Additionally, photographs are included of the tested rock core both before and after testing, photographs of the drilling sites and photographs of the rock core before it was preserved and placed in the Tulsa County garage.

Please contact us with any comments or questions you may have.

Sincerely,

FULLER, MOSSBARGER, SCOTT AND MAY
ENGINEERS, INC.

Daniel Rogers

Daniel B. Rogers, EIT
Project Engineer

Greg Yankey, PE
Practice Leader

/cmw

Geotechnical Investigation and Testing
Arkansas River Corridor Project
Arkansas River
Contract No. DACW912BV-07-D-1000
Sand Springs/Jenks, Oklahoma

Table of Contents

Summary of Results

1.0 Chapter 1

Summary of Results for Arkansas River Borings at Jenks and Sand Springs

Table 1.Boring Locations (+/- 20 feet accuracy)

Boring ID	Latitude	Longitude
J1	N 36.01026	W 95.75590
J2	N 33.01013	W 95.95394
J3	N 36.00994	W 95.95173
J4	N 36.01010	W 95.95016
J5	N 36.00962	W 95.94730
S1 *	N 36.12765	W 96.11081
S2	N 36.12585	W 96.11058
S4	N 36.12263	W 96.11125

* Coordinates for this location were estimated from available mapping (Staked by USACE Representative)

No elevations were reported due to the extreme inaccuracy of the handheld unit. Referring to the manuals and trained persons within FMSM, the vertical accuracy would be expected to be no better than 40 feet of probable error.

Table 2. Unconfined Rock Strength Test Results

Boring ID	Depth (ft)	Strength (psi)	Repaired Sample
J1	29.9-30.55	1360	X
J1	47.80-48.45	1380	
J2	15.6-16.3	250	X
J2	33.0-33.65	1000	X
J3	9.2-10.0	390	
J4	12.2-12.8	800	X
J4	29.0-29.65	390	X
J5	28.4-29.0	380	X
J5	59.0-59.65	430	X
S1	27.9-28.5	560	X
S1	45.2-45.8	1060	X
S2	21.4-22.0	740	X
S2	31.2-31.8	630	X
S4	40.0-40.6	70	
S4	71.1-71.75	580	X

All soils encountered in the overburden were determined, either visually or by lab testing, to be non-plastic.

In order to test the rock core samples in accordance to ASTM standards, it was necessary to epoxy some of the specimens. The epoxy is of low strength and was only used to bond clean horizontal fractures. The epoxy is not believed to affect the results of the strength testing and this was demonstrated by the failure surfaces penetrating across the epoxy layer on several of the specimens.

Boring Logs

Hole No. J1

DRILLING LOG	DIVISION Southwestern	INSTALLATION Tulsa	SHEET 1 OF 4 SHEETS

1. PROJECT Arkansas River Drilling	10. SIZE AND TYPE OF BIT 6.25 HSA + PQ Core
2. LOCATION (Coordinates or Station) N 36.01026 W 95.75590 (+/- 20ft)	11. DATUM FOR ELEVATION SHOWN (*TBM or MSL*) TBM
3. DRILLING AGENCY Thornburg Contract Drilling	12. MANUFACTURER'S DESIGNATION OF DRILL CME 55
4. HOLE NO. (*As shown on drawing title and file number*) J1	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 5; UNDISTURBED 0
5. NAME OF DRILLER Audie Thornburg	14. TOTAL NUMBER CORE BOXES 8
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED 3/19/2008; COMPLETED 3/19/2008
7. THICKNESS OF OVERBURDEN 21.0	17. ELEVATION TOP OF HOLE
8. DEPTH DRILLED INTO ROCK 39.0	18. TOTAL CORE RECOVERY FOR BORING %
9. TOTAL DEPTH OF HOLE 60.0	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
	0.0		Sand, medium to fine grained, tan, very loose to loose, damp to wet		1 0.0 1.5	1/1/1 1.4ft Recovery
					2 5.0 6.5	2/4/4 1.5ft Recovery
					3 10.0 11.5	2/3/4 1.5ft Recovery

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT Arkansas River Drilling HOLE NO. J1

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE | Hole No. J1

PROJECT: Arkansas River Drilling | INSTALLATION: Tulsa | SHEET 2 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Sand, medium to fine grained, tan, very loose to loose, damp to wet *(continued)*			
					4 15.0 16.5	2/3/1 1.5ft Recovery
					5 20.0 21.1	3/6/50+(0.1) 1.1ft Recovery
	21.0					
			Shale, gray to dark gray, moderately hard to hard, very thin bedded, fractured zones to 8' zones clayey and soft			
				58	RC-1 21.5 31.5	RQD = 58% Top 8 feet Highly Fractured UC Sample (29.9'-31.2') 1360 psi (Repaired Sample)
				89	RC-2 31.5	RQD = 48% Fractured into 0.3'-0.6' layers

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE	Hole No. J1	
PROJECT Arkansas River Drilling		INSTALLATION Tulsa	SHEET 3 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Shale, gray to dark gray, moderately hard to hard, very thin bedded, fractured zones to 8' zones clayey and soft *(continued)*		41.5	
				99	RC-3 41.5 51.5	RQD = 91% Cored Hard Angular Fractures at 49.4' Thicker Bedding UC Sample (47.8'-49.3') 1380 psi

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. J1
PROJECT Arkansas River Drilling	INSTALLATION Tulsa		SHEET 4 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
			Shale, gray to dark gray, moderately hard to hard, very thin bedded, fractured zones to 8' zones clayey and soft *(continued)*			
				100	RC-4 51.5 60.0	RQD = 94% Clayey, soft layer 52.2'-52.4' Harder zones are silty
	60.0					
			BOTTOM OF BORING			

Hole No. J2

DRILLING LOG	DIVISION Southwestern	INSTALLATION Tulsa	SHEET 1 OF 3 SHEETS

1. PROJECT: Arkansas River Drilling	10. SIZE AND TYPE OF BIT: 6.25 HSA + PQ Core
2. LOCATION (Coordinates or Station): N 36.01013 W 95.95394 (+/- 20ft)	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: TBM
3. DRILLING AGENCY: Thornburg Contract Drilling	12. MANUFACTURER'S DESIGNATION OF DRILL: Diedrich D50T
4. HOLE NO. *(As shown on drawing title and file number)*: J2	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 2; UNDISTURBED 0
5. NAME OF DRILLER: Audie Thornburg	14. TOTAL NUMBER CORE BOXES: 7
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN: 8.0	16. DATE HOLE: STARTED 3/16/2008; COMPLETED 3/17/2008
8. DEPTH DRILLED INTO ROCK: 32.0	17. ELEVATION TOP OF HOLE
9. TOTAL DEPTH OF HOLE: 40.0	18. TOTAL CORE RECOVERY FOR BORING %
	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
	0.0		Sand with trace gravel and silt, medium to fine grained, tan to gray, wet		1 0.0 1.5	0/1/1 1.4ft Recovery
					2 5.0 6.5	2/4/5 1.5ft Recovery
	8.0		Shale, gray, moderately hard, very thin bedded, clayey to silty, top 8' fractured and weathered	96	RC-1 8.0 18.0	RQD = 71% Top 8 feet highly fractured UC Sample Interval (15.6'-16.3') 250 psi (Repaired Sample)

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT Arkansas River Drilling HOLE NO. J2

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE | **Hole No. J2**

PROJECT	INSTALLATION	SHEET 2
Arkansas River Drilling	Tulsa	OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Shale, gray, moderately hard, very thin bedded, clayey to silty, top 8' fractured and weathered *(continued)*			
				100	RC-2 18.0 23.0	RQD = 52%
				100	RC-3 23.0 28.0	RQD = 96% Grading very hard Silty from 23.6'-27.1'
				94	RC-4 28.0 38.0	RQD = 94% UC Sample Interval (33.0'-34.2') 1000psi (Repaired Sample)

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE | Hole No. J2

PROJECT: Arkansas River Drilling | INSTALLATION: Tulsa | SHEET 3 OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Shale, gray, moderately hard, very thin bedded, clayey to silty, top 8' fractured and weathered *(continued)*			
				100	RC-5 38.0 40.0	RQD = 100%
	40.0					
			BOTTOM OF BORING			

Hole No. J3

DRILLING LOG	DIVISION Southwestern	INSTALLATION Tulsa	SHEET 1 OF 1 SHEETS

1. PROJECT Arkansas River Drilling	10. SIZE AND TYPE OF BIT 6.25 HSA + PQ Core
2. LOCATION (Coordinates or Station) N 36.00994 W 95.95173 (+/-20 ft)	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)* TBM
3. DRILLING AGENCY Thornburg Contract Drilling	12. MANUFACTURER'S DESIGNATION OF DRILL Diedrich D50T
4. HOLE NO. *(As shown on drawing title and file number)* J3	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 2; UNDISTURBED 0
5. NAME OF DRILLER Audie Thornburg	14. TOTAL NUMBER CORE BOXES 1
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN 7.5	16. DATE HOLE: STARTED 3/15/2008; COMPLETED 3/15/2008
8. DEPTH DRILLED INTO ROCK 3.0	17. ELEVATION TOP OF HOLE
9. TOTAL DEPTH OF HOLE 10.5	18. TOTAL CORE RECOVERY FOR BORING %
	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
	0.0		Sand with little gravel, tan to brown, well-graded, very loose to loose, damp to wet		1 0.0 1.5	0/1/1 1.4ft Recovery
					2 5.0 6.5	2/3/6 1.5ft Recovery
	7.5		Shale, gray, moderately hard, weathered, very thin bedded	100	RC-1 7.5 10.5	RQD = 46% UC Sample Interval (9.2'-10.0') 390 psi Clayey Lense 10.2'-10.4'
	10.5		BOTTOM OF BORING			

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. | PROJECT Arkansas River Drilling | HOLE NO. J3

DRILLING LOG	DIVISION Southwestern	INSTALLATION Tulsa	SHEET 1 OF 3 SHEETS

1. PROJECT Arkansas River Drilling	10. SIZE AND TYPE OF BIT 6.25 HSA + PQ Core
2. LOCATION (Coordinates or Station) N 36.01010 W 95.95016 (+/- 20ft)	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)* TBM
3. DRILLING AGENCY Thornburg Contract Drilling	12. MANUFACTURER'S DESIGNATION OF DRILL Diedrich D50T
4. HOLE NO. *(As shown on drawing title and file number)* J4	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 2; UNDISTURBED 0
5. NAME OF DRILLER Audie Thornburg	14. TOTAL NUMBER CORE BOXES 8
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN 3.8	16. DATE HOLE: STARTED 3/14/2008; COMPLETED 3/14/2008
8. DEPTH DRILLED INTO ROCK 36.2	17. ELEVATION TOP OF HOLE
9. TOTAL DEPTH OF HOLE 40.0	18. TOTAL CORE RECOVERY FOR BORING %
	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
	0.0		Gravelly Sand with clay seams, fine to coarse grained, brown with gray mottling, loose, dry to wet, cobble size stones intermixed		1 0.0 1.5	1/2/4 1.2' Recovery
	3.8		Shale, gray, sandy, moderately hard to hard, thin bedded, weathered to 7'		2 4.0 4.4	50+/0.4 0.3' Recovery
				86	RC-1 5.0 14.0	RQD = 56% UC Sample Interval (12.2'-13.0') 800 psi (Repaired Sample) Fractured zone 9.5'-9.8' Less weathered below 7.0'

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. J4
PROJECT Arkansas River Drilling	INSTALLATION Tulsa		SHEET 2 OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
			Shale, gray, sandy, moderately hard to hard, thin bedded, weathered to 7' *(continued)*	100	RC-2 14.0 24.0	RQD = 85% Zones Very Hard, Silty Mechanical Fractures @ 23.7'-24.0'
				100	RC-3 24.0 34.0	RQD = 97% UC Sample Interval (29.0'-30.1') 390 psi (Repaired Sample)

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE	Hole No. J4
PROJECT Arkansas River Drilling	INSTALLATION Tulsa	SHEET 3 OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
			Shale, gray, sandy, moderately hard to hard, thin bedded, weathered to 7' *(continued)*			
				80	RC-4 34.0 40.0	RQD = 30% Weaker, fractured, erodable
	40.0					
			BOTTOM OF BORING			

DRILLING LOG	DIVISION Southwestern	INSTALLATION Tulsa	SHEET 1 OF 5 SHEETS

1. PROJECT Arkansas River Drilling	10. SIZE AND TYPE OF BIT 6.25 HSA + PQ Core
2. LOCATION (Coordinates or Station) N 36.00962 W 95.94730 (+/- 20ft)	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)* TBM
3. DRILLING AGENCY Thornburg Contract Drilling	12. MANUFACTURER'S DESIGNATION OF DRILL CME 55
4. HOLE NO. *(As shown on drawing title and file number)* J5	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 6; UNDISTURBED 0
5. NAME OF DRILLER Audie Thornburg	14. TOTAL NUMBER CORE BOXES 10
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED 3/20/2008; COMPLETED 3/20/2008
7. THICKNESS OF OVERBURDEN 27.5	17. ELEVATION TOP OF HOLE
8. DEPTH DRILLED INTO ROCK 42.5	18. TOTAL CORE RECOVERY FOR BORING %
9. TOTAL DEPTH OF HOLE 75.0	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
	0.0		Silty Sand, fine grained, medium brown, very loose to loose, damp to wet		1 0.0 1.5	1/1/1 0.8' Recovery
					2 5.0 6.5	1/2/1 1.5' Recovery
	10.0		Sand, very fine grained, tan, loose, damp to wet		3 10.0 11.5	2/3/4 1.1' Recovery

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT Arkansas River Drilling HOLE NO. J5

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE | Hole No. J5

PROJECT	INSTALLATION	SHEET 2
Arkansas River Drilling	Tulsa	OF 5 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Sand, very fine grained, tan, loose, damp to wet *(continued)*			
					4 15.0 16.5	3/3/4 0.9' Recovery
					5 20.0 21.5	3/4/5 1.5' Recovery
	25.0					
			Sand, medium grained, gray loose, wet		6 25.0 26.5	3/4/4 0.8' Recovery
	27.5					
			Shale, gray, moderately hard to hard, very thin bedded, zones clayey			
				290	RC-1 28.0 31.0	RQD = 83% UC Sample Interval (28.4'-29.6') 380 psi (Repaired Sample)
				97	RC-2 31.0 41.0	RQD = 75%

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. J5
PROJECT Arkansas River Drilling	INSTALLATION Tulsa		SHEET 3 OF 5 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
			Shale, gray, moderately hard to hard, very thin bedded, zones clayey *(continued)*			
				100	RC-3 41.0 46.0	RQD = 96%
				92	RC-4 46.0 51.0	RQD = 82%

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE | **Hole No. J5**

PROJECT	INSTALLATION	SHEET 4 OF 5 SHEETS
Arkansas River Drilling	Tulsa	

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Shale, gray, moderately hard to hard, very thin bedded, zones clayey *(continued)*			
				100	RC-5 51.0 56.0	RQD = 88%
				99	RC-6 56.0 66.0	RQD = 92% UC Sample Interval (59.0'-59.9') 430 psi (Repaired Sample) Cored Hard Fractured Zones 56.0'-56.4' and 60.3-60.9 Angular Fractures between 56' and 61'
				98	RC-7 66.0 75.0	RQD = 92% Very hard below 67.0

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. J5	
PROJECT Arkansas River Drilling		INSTALLATION Tulsa		SHEET 5 OF 5 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Shale, gray, moderately hard to hard, very thin bedded, zones clayey *(continued)*			
	75.0					
			BOTTOM OF BORING			

DRILLING LOG	DIVISION Southwestern	INSTALLATION Tulsa	SHEET 1 OF 5 SHEETS

1. PROJECT Arkansas River Drilling	10. SIZE AND TYPE OF BIT 6.25 HSA + PQ Core
2. LOCATION (Coordinates or Station) N 36.12765 W 96.11081 (+/- 20ft)	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)* TBM
3. DRILLING AGENCY Thornburg Contract Drilling	12. MANUFACTURER'S DESIGNATION OF DRILL CME 55
4. HOLE NO. *(As shown on drawing title and file number)* S1	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 4; UNDISTURBED 0
5. NAME OF DRILLER Chris Mead / Audie Thornburg	14. TOTAL NUMBER CORE BOXES 8
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN 18.3	16. DATE HOLE: STARTED 3/9/2008; COMPLETED 3/12/2008
8. DEPTH DRILLED INTO ROCK 41.7	17. ELEVATION TOP OF HOLE
9. TOTAL DEPTH OF HOLE 75.0	18. TOTAL CORE RECOVERY FOR BORING %
	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
	0.0		Silty Sand with little gravel, fine to coarse grained, tan, poor graded, loose to medium dense, moist to wet, occasional clay lenses		1 0.0 1.5	2/3/5 1.1' Recovery GPS Coordinate taken from available mapping
					2 5.0 6.5	4/4/7 1.5' Recovery
					3 10.0 11.5	1/2/3 1.5' Recovery

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT Arkansas River Drilling HOLE NO. S1

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE	Hole No. S1
PROJECT Arkansas River Drilling	INSTALLATION Tulsa	SHEET 2 OF 5 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Silty Sand with little gravel, fine to coarse grained, tan, poor graded, loose to medium dense, moist to wet, occasional clay lenses *(continued)*			
					4 15.0 16.5	1/2/2 1.5' Recovery
	18.3		Shale, gray, soft to moderately hard, thin bedded, laminated, zones clayey and soft			
				30	RC-1 19.0 25.0	RQD = 18% Highly Weathered Very weak in top 3 feet 4.2' core loss
				93	RC-2 25.0 35.0	RQD = 77% 0.7' loss 25.0'-25.7' UC Sample Interval (27.8'-28.5') 560psi (Repaired Sample)

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE | Hole No. S1

PROJECT: Arkansas River Drilling | INSTALLATION: Tulsa | SHEET 3 OF 5 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Shale, gray, soft to moderately hard, thin bedded, laminated, zones clayey and soft *(continued)*			
				100	RC-3 35.0 45.0	RQD = 35% Highly fractured 35.0'-42.5' Sandy, weakly cemented 44.1'-44.8'
				95	RC-4 45.0 55.0	RQD = 61% Grading darker gray Solid piece (45.2'-47.8') UC Sample Interval (45.2'-45.8') 1060 psi (Repaired Sample)

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. S1	
PROJECT Arkansas River Drilling		INSTALLATION Tulsa		SHEET 4 OF 5 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
			Shale, gray, soft to moderately hard, thin bedded, laminated, zones clayey and soft *(continued)*			
				100	RC-5 55.0 60.0	RQD = 42% Fractured Easily broken
	60.0					

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. S1	
PROJECT Arkansas River Drilling		INSTALLATION Tulsa		SHEET 5 OF 5 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			BOTTOM OF BORING			

Hole No. S2

DRILLING LOG	DIVISION Southwestern	INSTALLATION Tulsa	SHEET 1 OF 4 SHEETS

1. PROJECT Arkansas River Drilling	10. SIZE AND TYPE OF BIT 6.25 HSA + PQ Core
2. LOCATION (Coordinates or Station) N 36.12585W 96.11058 (+/- 20ft)	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)* TBM
3. DRILLING AGENCY Thornburg Contract Drilling	12. MANUFACTURER'S DESIGNATION OF DRILL CME 55
4. HOLE NO. *(As shown on drawing title and file number)* S2	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 3; UNDISTURBED 0
5. NAME OF DRILLER Audie Thornburg	14. TOTAL NUMBER CORE BOXES 8
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN 10.0	16. DATE HOLE: STARTED 3/12/2008; COMPLETED 3/12/2008
8. DEPTH DRILLED INTO ROCK 40.5	17. ELEVATION TOP OF HOLE
9. TOTAL DEPTH OF HOLE 50.0	18. TOTAL CORE RECOVERY FOR BORING %
	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
	0.0		Sand, fine to coarse grained, tan to brown, very loose, dry to wet, zones with gravel and clay lenses		1 0.0 1.5	0/1/1 1.4' Recovery Bulk Bag taken for classification (0.0'-5.0')
					2 5.0 6.5	1/1/1 0.9' Recovery Slightly Clayey
	10.0		Sandstone, gray, hard, thin bedded		3 10.0 10.4	50+/0.4 0.4' Recovery
				78	RC-1 10.5 20.5	RQD = 23% Highly fractured top 10.5'
	11.7		Shale, gray, soft, clayey, thin bedded, fissile, highly weathered			

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE	Hole No. S2		
PROJECT Arkansas River Drilling		INSTALLATION Tulsa		SHEET 2 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Shale, gray, soft, clayey, thin bedded, fissile, highly weathered *(continued)*			
				91	RC-2 20.5 30.5	RQD = 62% Harder below 21.0' UC Sample Interval (21.1'-22.0') 740 psi (Repaired Sample) Sand on samples is from the riverbed 30+mph winds during sample preservation
				100	RC-3 30.5 40.5	RQD = 64% UC Sample Interval (31.0'-31.8') 630 psi (Repaired Sample)

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE | Hole No. S2

PROJECT: Arkansas River Drilling | INSTALLATION: Tulsa | SHEET 3 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Shale, gray, soft, clayey, thin bedded, fissile, highly weathered *(continued)*			
				85	RC-4 40.5 50.5	RQD = 0% Shale bedded in thin laminar beds Weak cementation between layers Some fine sand present

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. S2
PROJECT Arkansas River Drilling		INSTALLATION Tulsa	SHEET 4 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
	50.5		BOTTOM OF BORING			

DRILLING LOG	DIVISION Southwestern	INSTALLATION Tulsa	SHEET 1 OF 5 SHEETS

1. PROJECT Arkansas River Drilling	10. SIZE AND TYPE OF BIT 6.25 HSA + PQ Core
2. LOCATION (Coordinates or Station) N 36.12263 W 96.11125 (+/- 20ft)	11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)* TBM
3. DRILLING AGENCY Thornburg Contract Drilling	12. MANUFACTURER'S DESIGNATION OF DRILL CME 55
4. HOLE NO. *(As shown on drawing title and file number)* S4	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 7, UNDISTURBED 0
5. NAME OF DRILLER Audie Thornburg	14. TOTAL NUMBER CORE BOXES 8
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED 3/21/2008, COMPLETED 3/22/2008
	17. ELEVATION TOP OF HOLE
7. THICKNESS OF OVERBURDEN 34.8	18. TOTAL CORE RECOVERY FOR BORING %
8. DEPTH DRILLED INTO ROCK 40.2	19. SIGNATURE OF INSPECTOR
9. TOTAL DEPTH OF HOLE 60.0	

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
	0.0		Topsoil		1 0.0 1.5	1/2/2 1.5' Recovery
	0.8		Sandy silt with some clay lenses, reddish brown, damp, loose			
					2 5.0 6.5	1/2/3 1.5' Recovery
					3 10.0 11.5	2/4/5 1.5' Recovery

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT Arkansas River Drilling HOLE NO. S4

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. S4
PROJECT Arkansas River Drilling	INSTALLATION Tulsa		SHEET 2 OF 5 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Sandy silt with some clay lenses, reddish brown, damp, loose *(continued)*			
					4 15.0 16.5	2/2/3 1.5' Recovery Clean lense of sand (15.2'-15.4') 70 psi Clayey lense 17.0'-19.0'
					5 20.0 21.5	2/3/4 1.5' Recovery
	25.0					
			Silty sand, fine grained, tan to reddish brown, wet		6 25.0 26.5	6/12/13 1.5' Recovery
					7 30.0 31.5	12/13/18 1.5' Recovery

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. S4
PROJECT Arkansas River Drilling	INSTALLATION Tulsa		SHEET 3 OF 5 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
	34.8		Silty sand, fine grained, tan to reddish brown, wet *(continued)*			
			Shale, gray, thin bedded, soft, zones clayey, highly weathered (top 4')	94	RC-1 35.0 45.0	RQD = 63% UC Sample Interval (40.0'-41.0') 580 psi (Repaired Sample) moderately fractured top 4'
				85	RC-2 45.0 55.0	RQD = 41% zones silty and harder(glassy, smooth cut)

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE		Hole No. S4
PROJECT Arkansas River Drilling		INSTALLATION Tulsa	SHEET 4 OF 5 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Shale, gray, thin bedded, soft, zones clayey, highly weathered (top 4') *(continued)*			
				91	RC-3 55.0 65.0	RQD = 28% Broken along bedding planes
			BOTTOM OF BORING			
				100	RC-4 65.0 75.0	RQD = 96% Hard UC Sample Interval (71.1'-71.9')

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE			Hole No. S4		
PROJECT Arkansas River Drilling			INSTALLATION Tulsa		SHEET 5 OF 5 SHEETS	

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			Shale, gray, thin bedded, soft, zones clayey, highly weathered (top 4') *(continued)*			
	75.0					

Soils Test Results

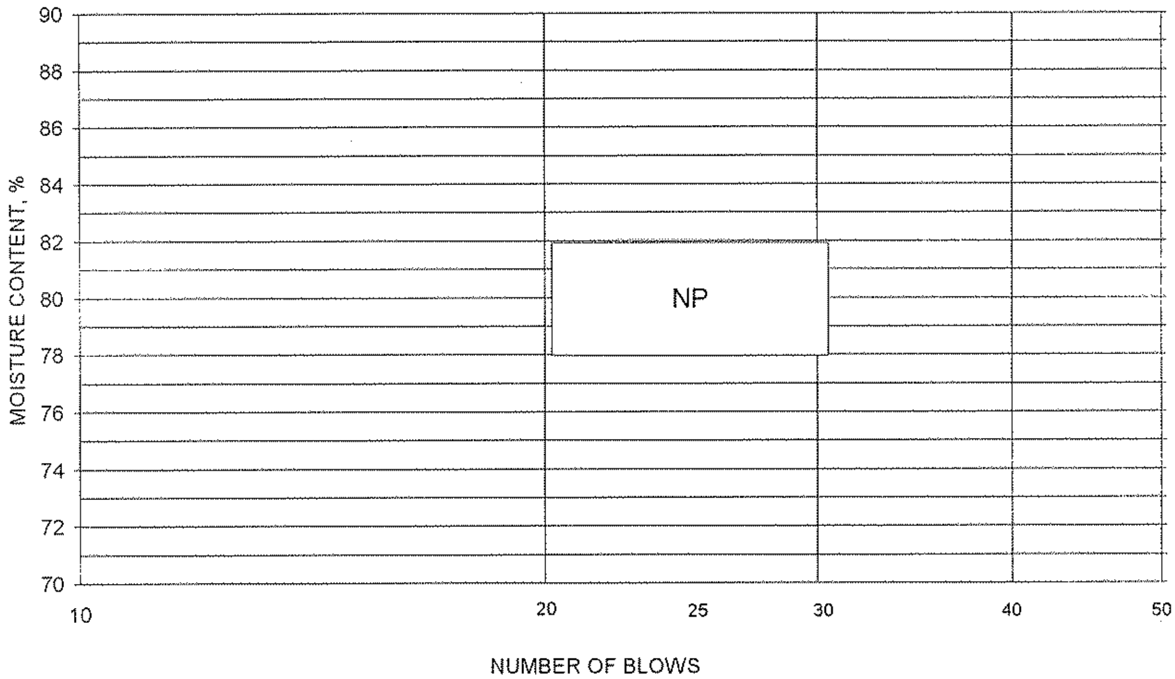
ATTERBERG LIMITS

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Project	Arkansas River Drilling	Project No.	LX2007282
Source	S4, 5.0'-6.5'	Lab ID	3
Tested By	KWS Test Method ASTM D 4318 Method A	% + No. 40	1
Test Date	04-14-2008 Prepared Dry	Date Received	03-26-2008

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Number of Blows	Water Content (%)	Liquid Limit

Liquid Limit



PLASTIC LIMIT AND PLASTICITY INDEX

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Water Content (%)	Plastic Limit	Plasticity Index

Remarks:

Reviewed By

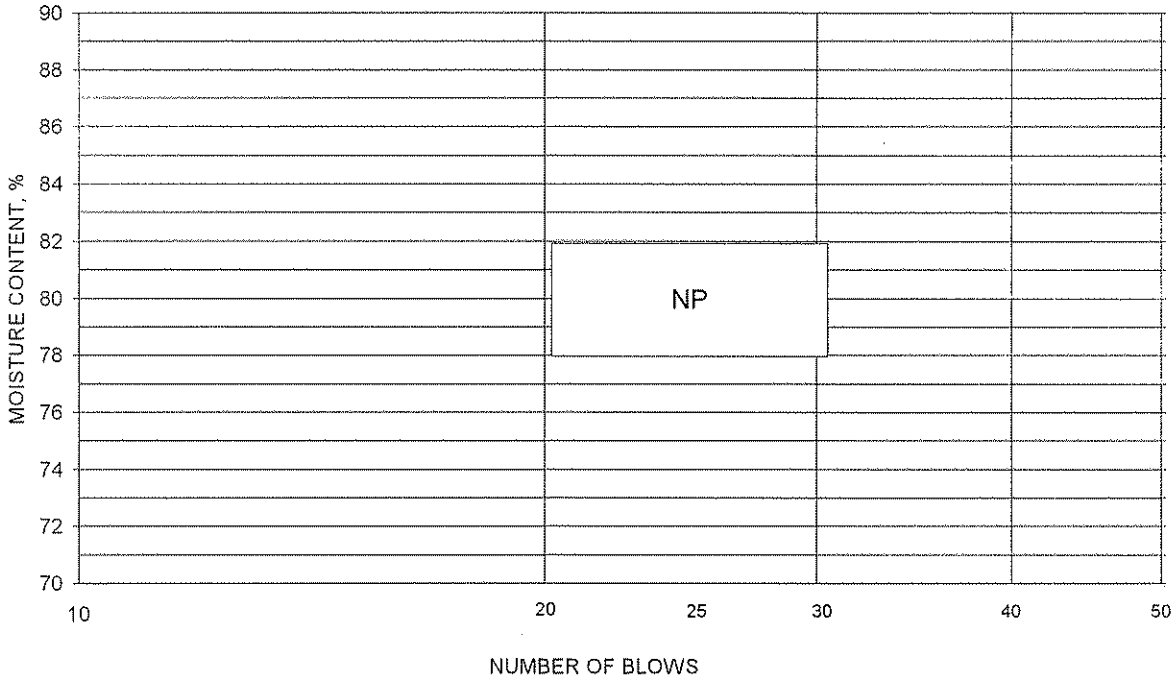
Fuller Mossbarger Scott & May
FMSM ENGINEERS

ATTERBERG LIMITS

Project	Arkansas River Drilling	Project No.	LX2007282
Source	S4, 30.0'-31.5'	Lab ID	4
Tested By	KWS Test Method ASTM D 4318 Method A	% + No. 40	4
Test Date	04-10-2008 Prepared Dry	Date Received	03-26-2008

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Number of Blows	Water Content (%)	Liquid Limit

Liquid Limit



PLASTIC LIMIT AND PLASTICITY INDEX

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Water Content (%)	Plastic Limit	Plasticity Index

Remarks:

Reviewed By

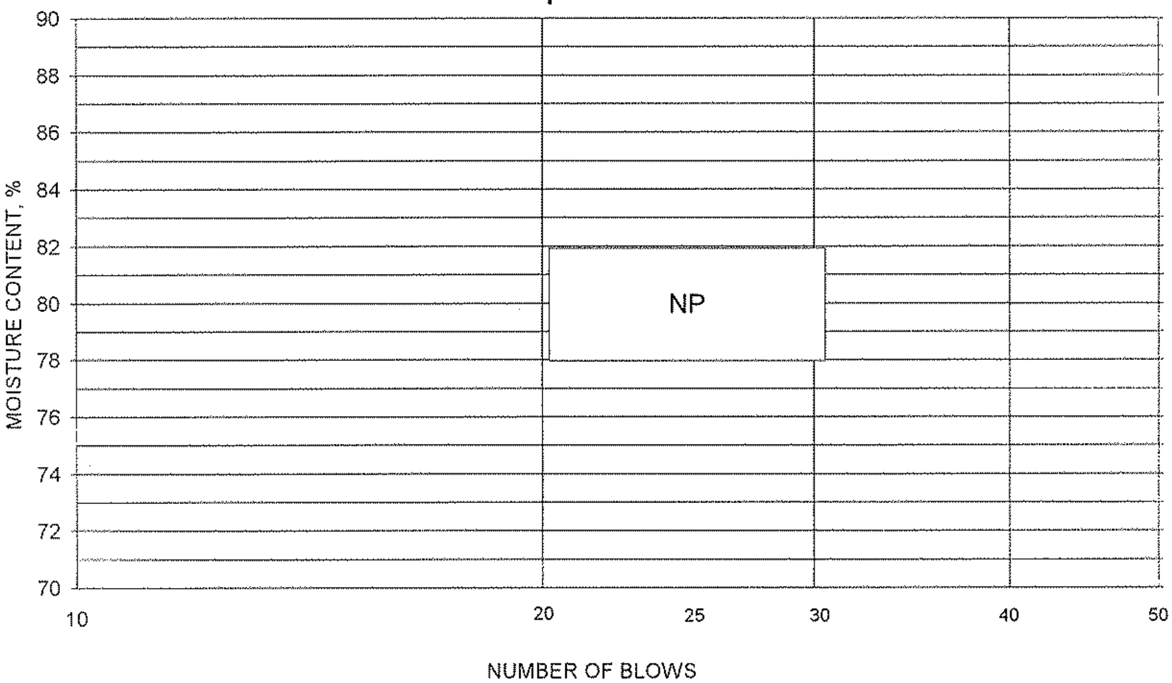
Fuller Mossbarger Scott & May
FMSM ENGINEERS

ATTERBERG LIMITS

Project	Arkansas River Drilling	Project No.	LX2007282
Source	J5, 5.0'-6.5'	Lab ID	9
Tested By	KWS Test Method ASTM D 4318 Method A	% + No. 40	3
Test Date	Need! Input Prepared Dry	Date Received	03-26-2008

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Number of Blows	Water Content (%)	Liquid Limit

Liquid Limit



PLASTIC LIMIT AND PLASTICITY INDEX

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Water Content (%)	Plastic Limit	Plasticity Index

Remarks: ____________________

Reviewed By ____________________

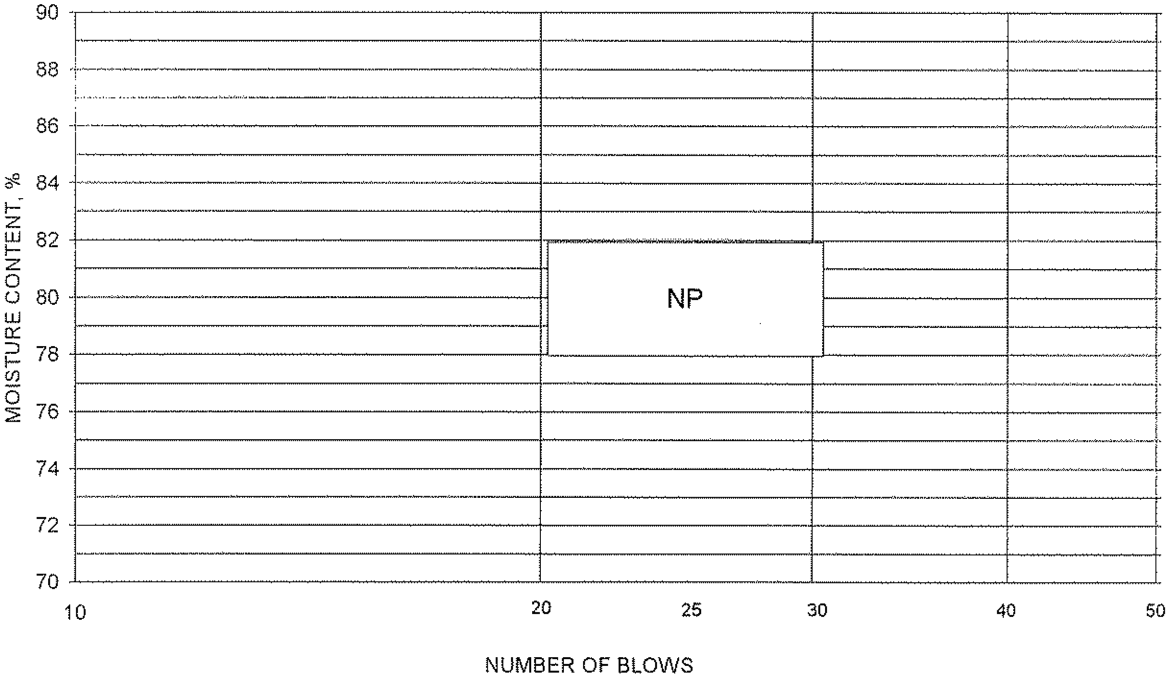
FMSM ENGINEERS

ATTERBERG LIMITS

Project	Arkansas River Drilling	Project No.	LX2007282
Source	J5, 20.0'-21.5'	Lab ID	10
Tested By	KWS Test Method ASTM D 4318 Method A	% + No. 40	32
Test Date	04-10-2008 Prepared Dry	Date Received	03-26-2008

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Number of Blows	Water Content (%)	Liquid Limit

Liquid Limit



PLASTIC LIMIT AND PLASTICITY INDEX

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Water Content (%)	Plastic Limit	Plasticity Index

Remarks: ______

Reviewed By ______

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Gradation Analysis

ASTM D 422

Project Name Arkansas River Drilling
Source S1, 10.0'-11.5'

Preparation Method ASTM D 1140 Method A
Particle Shape Rounded
Particle Hardness Hard and Durable
Sample Dry Mass (g) 509.02

Project Number LX2007282
Lab ID 1
Date Received 03-26-2008
Preparation Date 04-11-2008
Test Date 04-11-2008

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
1"	0.00	0.0	100.0
3/4"	17.74	3.5	96.5
3/8"	5.57	1.1	95.4
No. 4	38.55	7.6	87.8
No. 10	90.87	17.9	70.0
No. 20	93.18	18.3	51.7
No. 40	93.30	18.3	33.4
No. 100	101.72	20.0	13.4
No. 200	8.14	1.6	11.8
Pan	59.95	11.8	---

% Gravel 12.2
% Sand 76.1
% Fines 11.8
Fines Classification N/A

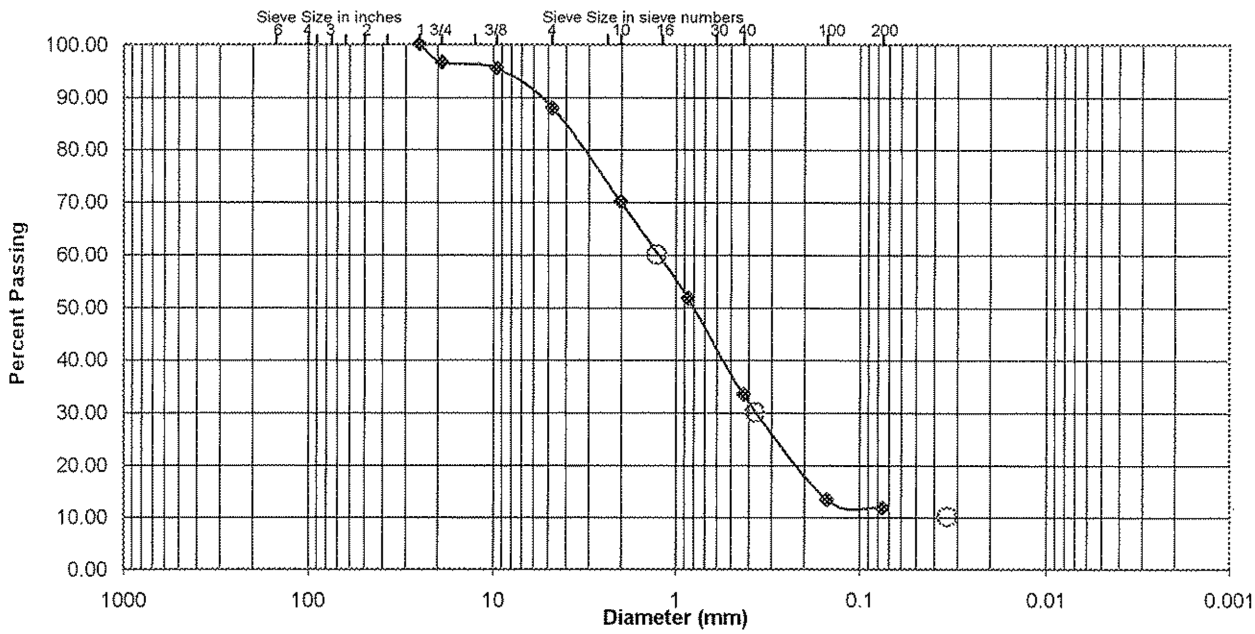
D_{10} (mm) 0.0337
D_{30} (mm) 0.3667
D_{60} (mm) 1.2635

Cu 37.48
Cc 3.16

Classification

N/A

Particle Size Distribution



Comments

Reviewed By

FMSM ENGINEERS

Gradation Analysis

ASTM D 422

Project Name Arkansas River Drilling

Source S2, 0.0'-5.0'

Project Number LX2007282

Lab ID 2

Date Received 03-26-2008

Preparation Date 04-08-2008

Test Date 04-11-2008

Preparation Method ASTM D 1140 Method A

Particle Shape Rounded

Particle Hardness Hard and Durable

Sample Dry Mass (g) 432.07

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/4"	0.00	0.0	100.0
3/8"	19.50	4.5	95.5
No. 4	48.73	11.3	84.2
No. 10	92.78	21.5	62.7
No. 20	96.28	22.3	40.5
No. 40	85.95	19.9	20.6
No. 100	64.27	14.9	5.7
No. 200	3.52	0.8	4.9
Pan	21.04	4.9	---

% Gravel 15.8

% Sand 79.3

% Fines 4.9

Fines Classification N/A

D_{10} (mm) 0.2229

D_{30} (mm) 0.6005

D_{60} (mm) 1.8006

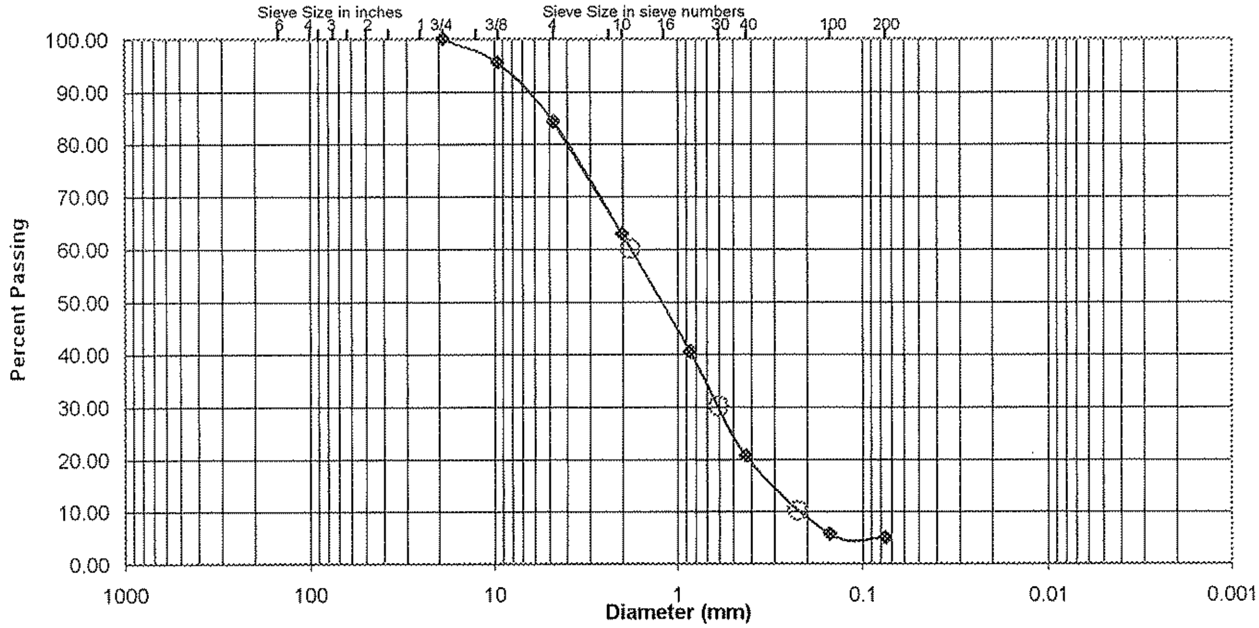
Cu 8.08

Cc 0.90

Classification

N/A

Particle Size Distribution



Comments

Reviewed By

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Gradation Analysis
ASTM D 422

Project Name Arkansas River Drilling
Source S4, 5.0'-6.5'

Project Number LX2007282
Lab ID 3
Date Received 03-26-2008
Preparation Date 04-08-2008
Test Date 04-08-2008

Preparation Method ASTM D 1140 Method A
Particle Shape Angular
Particle Hardness Hard and Durable
Sample Dry Mass (g) 182.91

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 4	0.00	0.0	100.0
No. 10	0.06	0.0	100.0
No. 20	0.47	0.3	99.7
No. 40	1.23	0.7	99.0
No. 100	2.87	1.6	97.5
No. 200	21.12	11.5	85.9
Pan	157.16	85.9	---

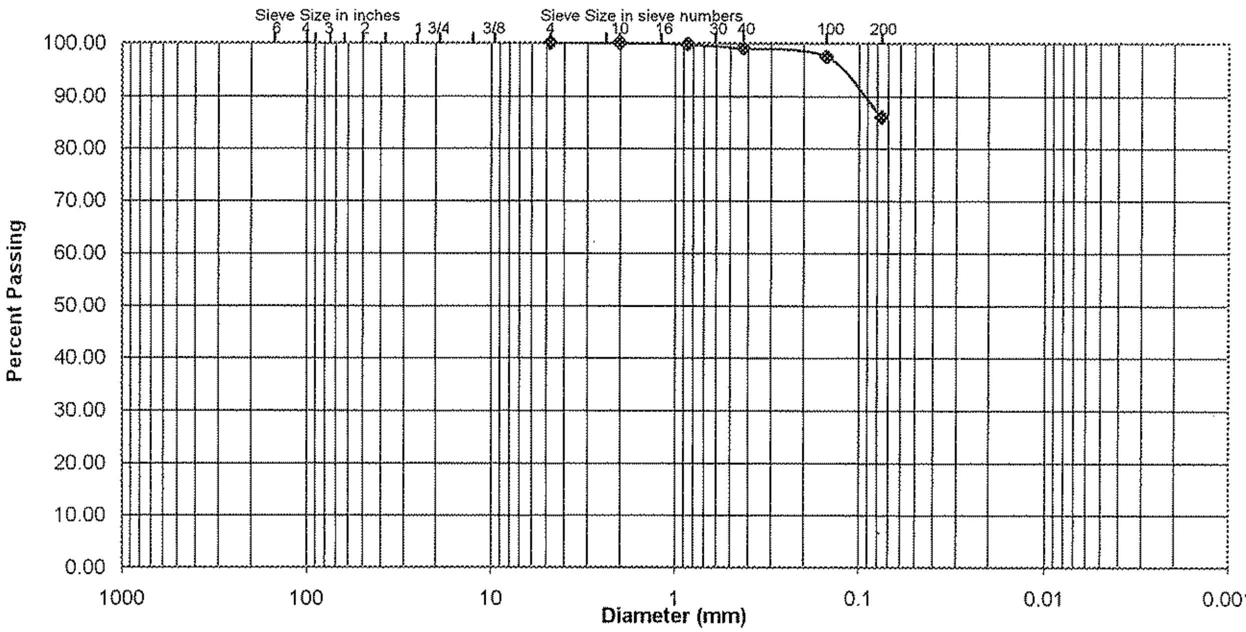
% Gravel 0.0
% Sand 14.1
% Fines 85.9
Fines Classification N/A

D_{10} (mm) N/A
D_{30} (mm) N/A
D_{60} (mm) N/A

Cu N/A
Cc N/A

Classification
N/A

Particle Size Distribution



Comments

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File: LX2007282_200-3.xls Sheet: Report
Preparation Date: 5-1998
Revision Date: 4-2008

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: JW
Approved By: TLK

FMSM ENGINEERS

Gradation Analysis
ASTM D 422

Project Name Arkansas River Drilling
Source S4, 30.0'-31.5'

Project Number LX2007282
Lab ID 4
Date Received 03-26-2008
Preparation Date 04-08-2008
Test Date 04-11-2008

Preparation Method ASTM D 1140 Method A
Particle Shape Rounded
Particle Hardness Hard and Durable
Sample Dry Mass (g) 286.03

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 4	0.00	0.0	100.0
No. 10	2.06	0.7	99.3
No. 20	2.95	1.0	98.2
No. 40	5.28	1.8	96.4
No. 100	159.73	55.8	40.6
No. 200	22.20	7.8	32.8
Pan	93.81	32.8	---

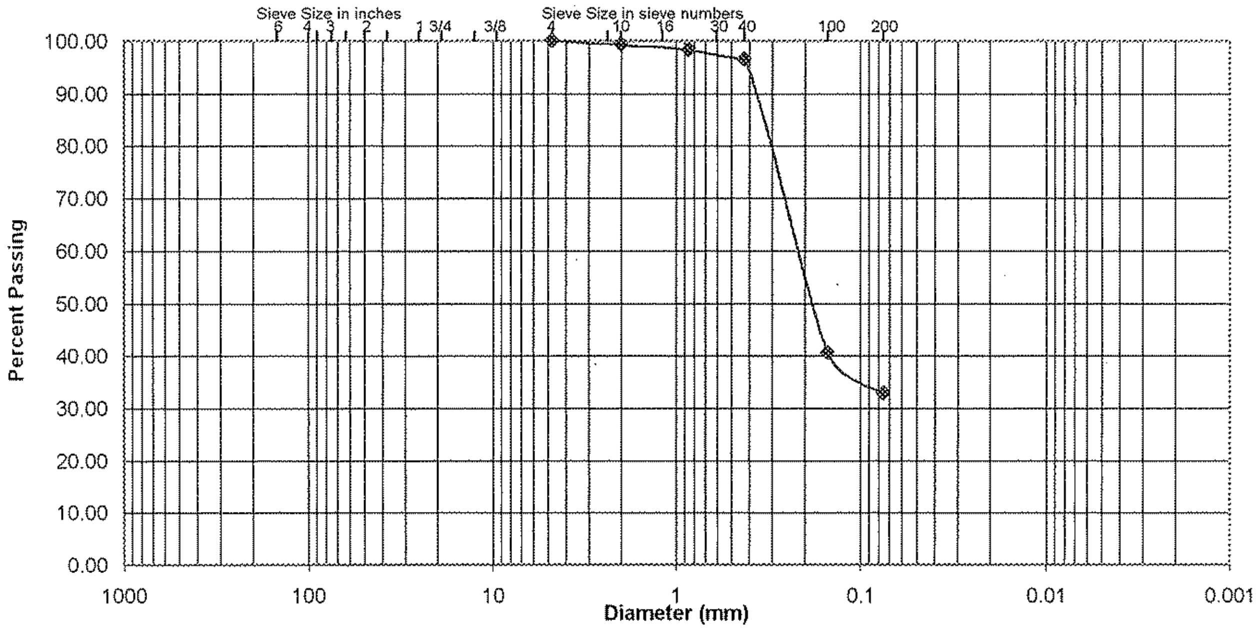
% Gravel 0.0
% Sand 67.2
% Fines 32.8
Fines Classification N/A

D_{10} (mm) N/A
D_{30} (mm) N/A
D_{60} (mm) N/A

Cu N/A
Cc N/A

Classification
N/A

Particle Size Distribution



Comments

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Fuller Mossbarger Scott & May FMSM ENGINEERS

Gradation Analysis
ASTM D 422

Project Name Arkansas River Drilling
Source J1, 5.0'-6.5'

Project Number LX2007282
Lab ID 5
Date Received 03-26-2008
Preparation Date 04-08-2008
Test Date 04-11-2008

Preparation Method ASTM D 1140 Method A
Particle Shape Rounded
Particle Hardness Hard and Durable
Sample Dry Mass (g) 361.74

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/8"	0.00	0.0	100.0
No. 4	3.78	1.0	99.0
No. 10	9.60	2.7	96.3
No. 20	19.33	5.3	91.0
No. 40	101.83	28.2	62.8
No. 100	179.42	49.6	13.2
No. 200	20.23	5.6	7.6
Pan	27.55	7.6	---

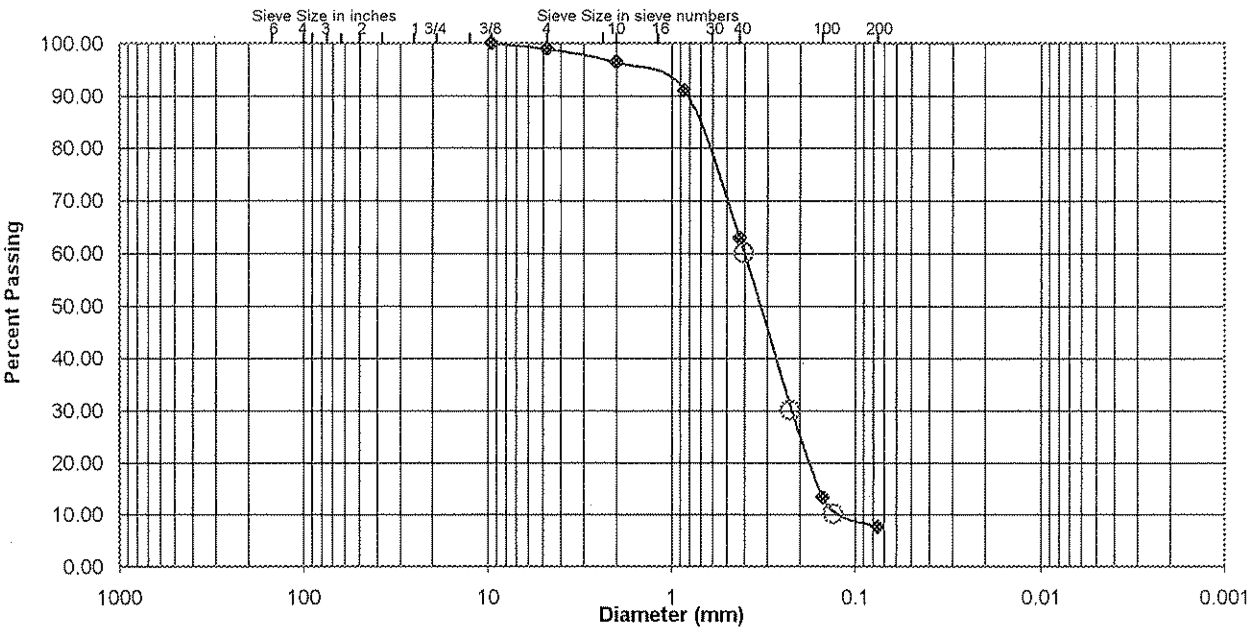
% Gravel 1.0
% Sand 91.3
% Fines 7.6
Fines Classification N/A

D_{10} (mm) 0.1308
D_{30} (mm) 0.2234
D_{60} (mm) 0.4007

Cu 3.06
Cc 0.95

Classification
N/A

Particle Size Distribution



Comments

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File: LX2007282_200-5.xls Sheet: Report
Preparation Date: 5-1998
Revision Date: 4-2008

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: JW
Approved By: TLK

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Gradation Analysis
ASTM D 422

Project Name Arkansas River Drilling
Source J2, 5.0'-6.5'

Preparation Method ASTM D 1140 Method A
Particle Shape Rounded
Particle Hardness Hard and Durable
Sample Dry Mass (g) 287.03

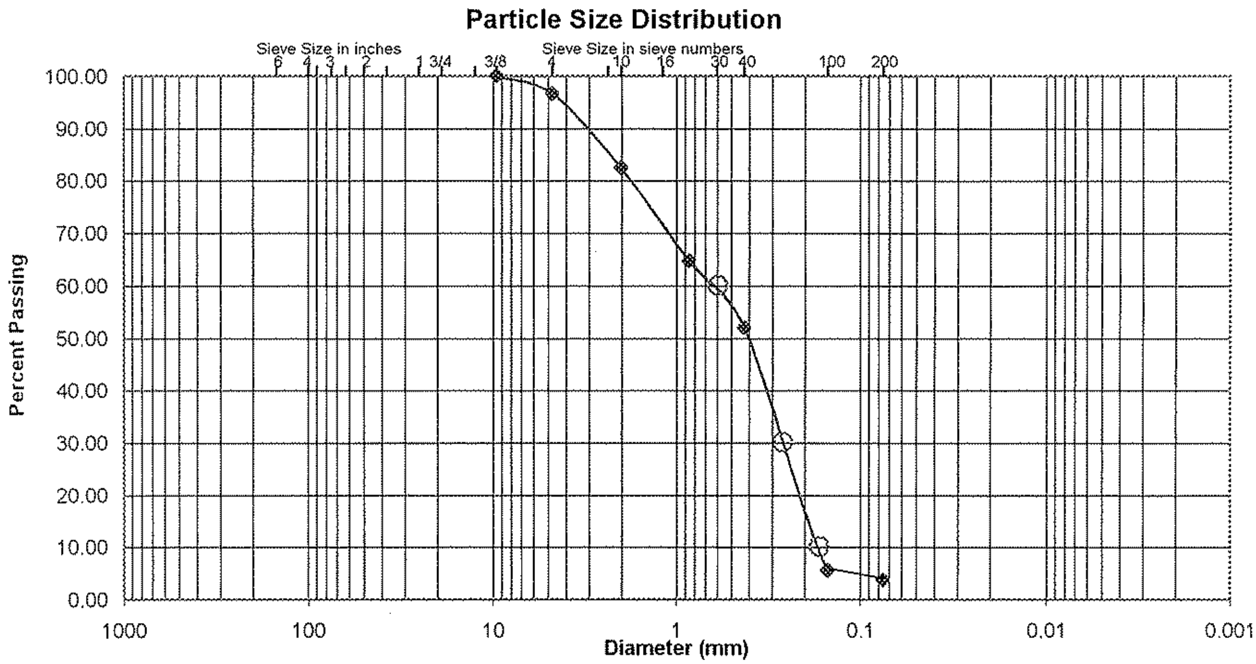
Project Number LX2007282
Lab ID 6
Date Received 03-26-2008
Preparation Date 04-08-2008
Test Date 04-11-2008

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/8"	0.00	0.0	100.0
No. 4	9.91	3.5	96.5
No. 10	40.84	14.2	82.3
No. 20	50.71	17.7	64.7
No. 40	36.44	12.7	52.0
No. 100	133.42	46.5	5.5
No. 200	5.14	1.8	3.7
Pan	10.57	3.7	---

% Gravel	3.5
% Sand	92.9
% Fines	3.7
Fines Classification	N/A
D_{10} (mm)	0.1660
D_{30} (mm)	0.2599
D_{60} (mm)	0.5894
Cu	3.55
Cc	0.69

Classification
N/A



Comments

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Gradation Analysis
ASTM D 422

Project Name Arkansas River Drilling
Source J3, 5.0'-6.5'

Preparation Method ASTM D 1140 Method A
Particle Shape Rounded
Particle Hardness Hard and Durable
Sample Dry Mass (g) 444.99

Project Number LX2007282
Lab ID 7
Date Received 03-26-2008
Preparation Date 04-08-2008
Test Date 04-11-2008

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
1"	0.00	0.0	100.0
3/4"	12.60	2.8	97.2
3/8"	4.17	0.9	96.2
No. 4	13.09	2.9	93.3
No. 10	65.02	14.6	78.7
No. 20	128.26	28.8	49.9
No. 40	142.43	32.0	17.8
No. 100	73.14	16.4	1.4
No. 200	2.58	0.6	0.8
Pan	3.70	0.8	---

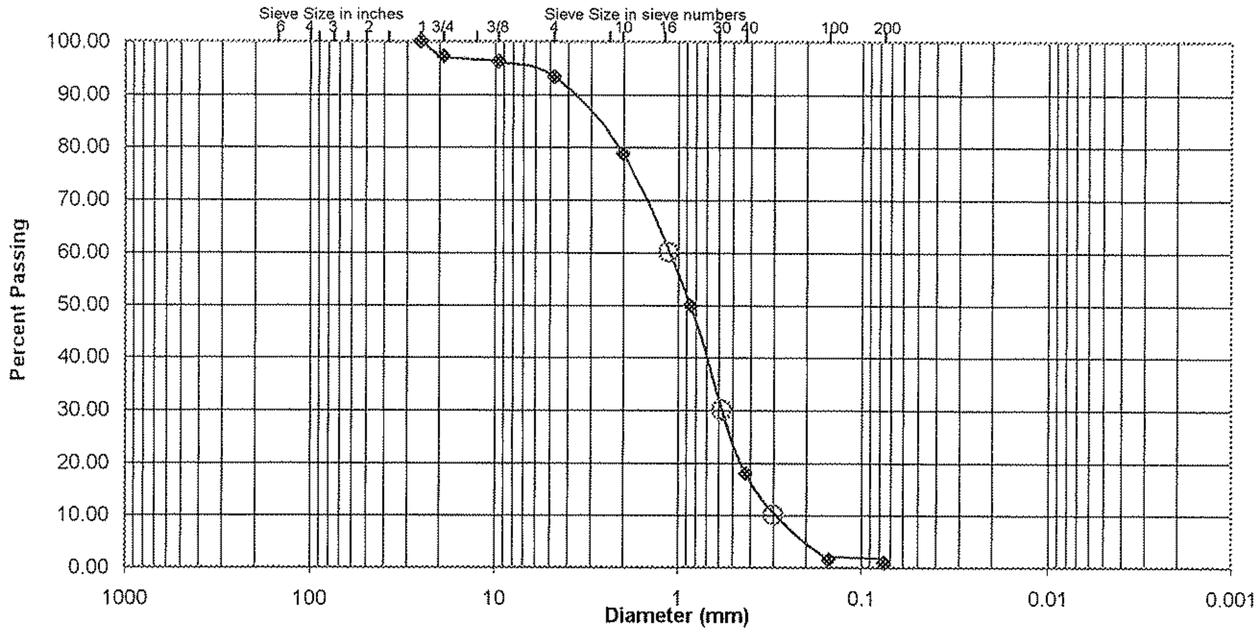
% Gravel 6.7
% Sand 92.5
% Fines 0.8
Fines Classification N/A

D_{10} (mm) 0.2985
D_{30} (mm) 0.5729
D_{60} (mm) 1.1187

Cu 3.75
Cc 0.98

Classification
N/A

Particle Size Distribution



Comments

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File: LX2007282_200-7 Sheet: Report
Preparation Date: 5-1998
Revision Date: 4-2008

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: JW
Approved By: TLK

ENGINEERS

Gradation Analysis
ASTM D 422

Project Name Arkansas River Drilling
Source J4, 0.0'-1.5'

Preparation Method ASTM D 1140 Method A
Particle Shape Rounded
Particle Hardness Hard and Durable
Sample Dry Mass (g) 330.32

Project Number LX2007282
Lab ID 8
Date Received 03-26-2008
Preparation Date 04-08-2008
Test Date 04-11-2008

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
1"	0.00	0.0	100.0
3/4"	20.01	6.1	93.9
3/8"	21.07	6.4	87.6
No. 4	20.05	6.1	81.5
No. 10	58.41	17.7	63.8
No. 20	65.91	20.0	43.9
No. 40	41.00	12.4	31.4
No. 100	18.98	5.7	25.7
No. 200	3.57	1.1	24.6
Pan	81.32	24.6	---

% Gravel 18.5
% Sand 56.9
% Fines 24.6
Fines Classification N/A

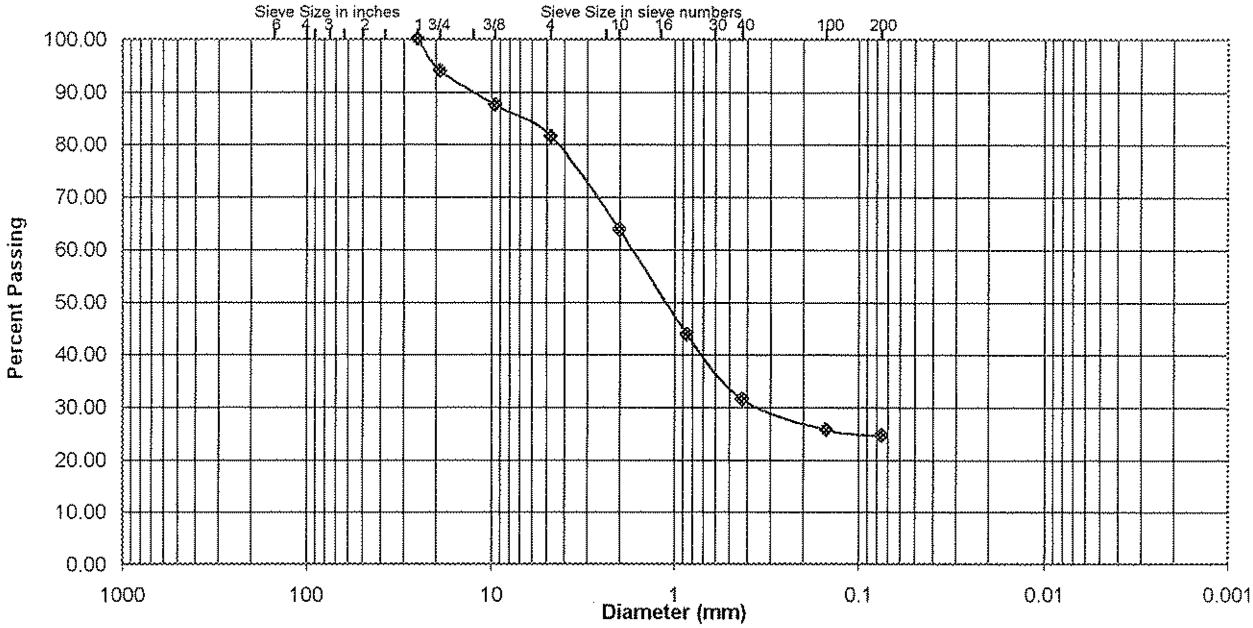
D_{10} (mm) N/A
D_{30} (mm) N/A
D_{60} (mm) N/A

Cu N/A
Cc N/A

Classification

N/A

Particle Size Distribution



Comments

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File: LX2007282_200-8.xls Sheet: Report
Preparation Date: 5-1998
Revision Date: 4-2008

Fuller, Mossbarger, Scott and May Engineers, Inc

Laboratory Document
Prepared By: JW
Approved By: TLK

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Gradation Analysis

ASTM D 422

Project Name Arkansas River Drilling
Source J5, 5.0'-6.5'

Project Number LX2007282
Lab ID 9
Date Received 03-26-2008
Preparation Date 04-09-2008
Test Date 04-11-2008

Preparation Method ASTM D 1140 Method A
Particle Shape Rounded
Particle Hardness Hard and Durable
Sample Dry Mass (g) 191.25

Analysis based on total sample.

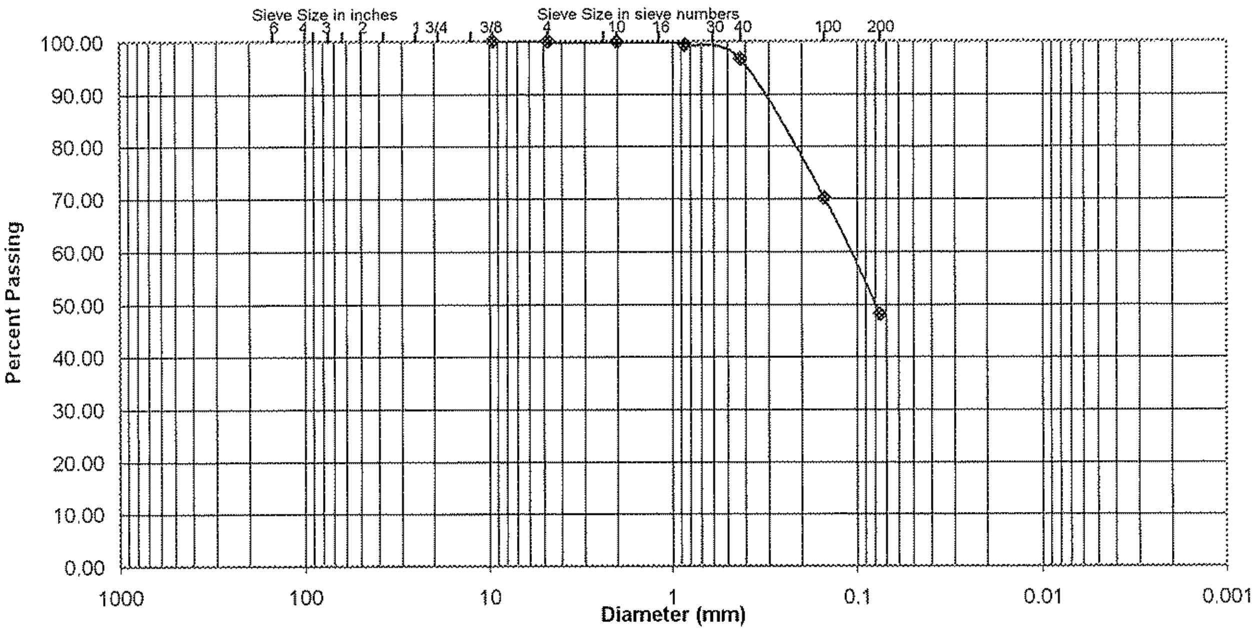
Sieve Size	Grams Retained	% Retained	% Passing
3/8"	0.00	0.0	100.0
No. 4	0.09	0.0	100.0
No. 10	0.15	0.1	99.9
No. 20	1.20	0.6	99.2
No. 40	5.03	2.6	96.6
No. 100	50.86	26.6	70.0
No. 200	42.09	22.0	48.0
Pan	91.83	48.0	---

% Gravel	0.0
% Sand	51.9
% Fines	48.0
Fines Classification	N/A
D_{10} (mm)	N/A
D_{30} (mm)	N/A
D_{60} (mm)	N/A
Cu	N/A
Cc	N/A

Classification

N/A

Particle Size Distribution



Comments

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Fuller Mossbarger Scott & May
FMSM ENGINEERS

Gradation Analysis
ASTM D 422

Project Name Arkansas River Drilling
Source J5, 20.0'-21.5'

Project Number LX2007282
Lab ID 10
Date Received 03-26-2008

Preparation Method ASTM D 1140 Method A
Particle Shape Rounded
Particle Hardness Hard and Durable
Sample Dry Mass (g) 181.29

Preparation Date 04-09-2008
Test Date 04-11-2008

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/8"	0.00	0.0	100.0
No. 4	0.42	0.2	99.8
No. 10	6.73	3.7	96.1
No. 20	19.60	10.8	85.2
No. 40	32.05	17.7	67.6
No. 100	112.27	61.9	5.6
No. 200	5.18	2.9	2.8
Pan	5.04	2.8	---

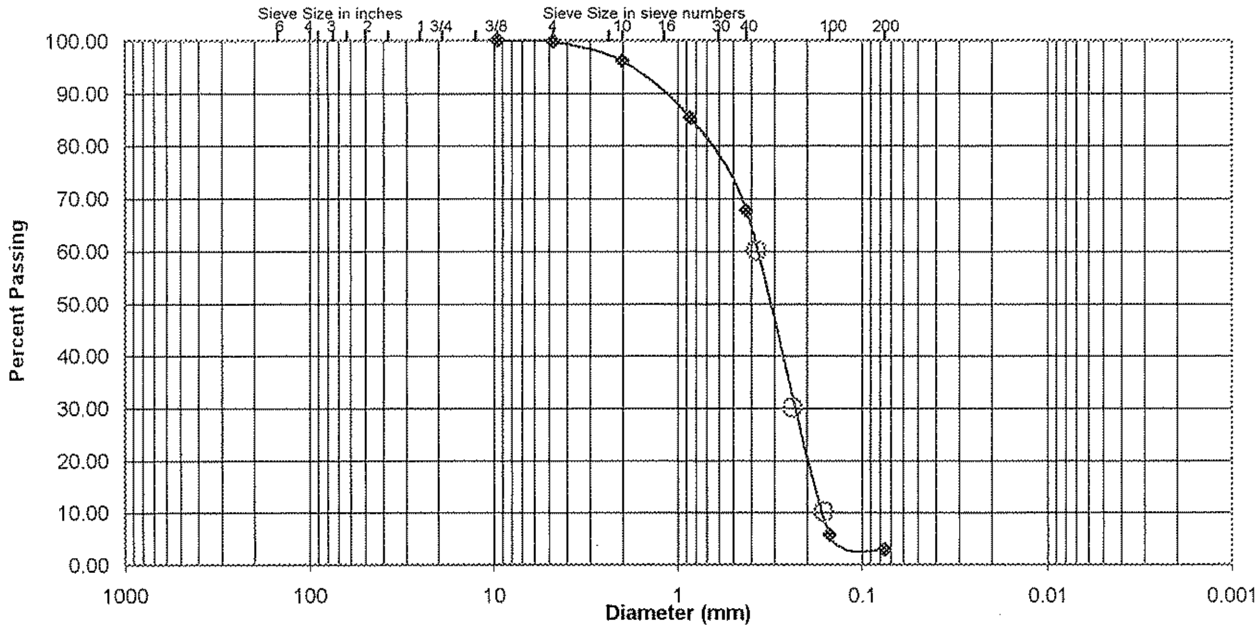
% Gravel 0.2
% Sand 97.0
% Fines 2.8
Fines Classification N/A

D_{10} (mm) 0.1614
D_{30} (mm) 0.2360
D_{60} (mm) 0.3742

Cu 2.32
Cc 0.92

Classification
N/A

Particle Size Distribution



Comments

Reviewed By

Rock Core Unconfined Test Results

Fuller Mossbarger Scott & May FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

ASTM D 2938

Project Name Arkansas River Drilling
Project Number LX2007282
Lithology Shale, dark gray, soft
Lab ID UCR-11
Hole Number S1
Depth (ft/elev) 27.9' - 28.5'
Date Received 03-26-2008

Temperature (°C) 19.8
Moisture Condition As received, moist
Date Tested 04-11-2008

Side Planeness N/A
Perpendicularity N/A
End Planeness N/A
Dimensions were not confirmed.

Height (in) 7.903
Diameter (in) 3.187
Area (in^2) 7.976

Wet Unit Weight (pcf) 153.7
Dry Unit Weight (pcf) N/A
Moisture Content (%) N/A

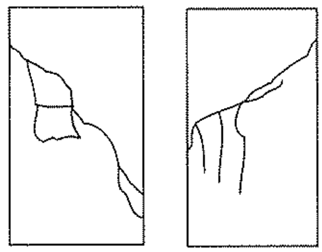
Loading Rate (lbf/sec) 20
Peak Load (lbf) 4460

Failure Type Shear

Compressive Strength (psi) 560

Compressive Strength (tsf) 40

Failure Sketches



Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

FMSM ENGINEERS

Fuller Mossbarger Scott & May

Unconfined Compressive Strength Of Intact Rock Core

ASTM D 2938

Project Name Arkansas River Drilling

Project Number LX2007282

Lithology Shale, dark gray, soft

Lab ID UCR-12

Hole Number S1

Depth (ft/elev) 45.2' - 45.8'

Date Received 03-26-2008

Temperature (°C) 19.8

Moisture Condition As received, moist

Date Tested 04-11-2008

Side Planeness N/A

Perpendicularity N/A

End Planeness N/A

Dimensions were not confirmed.

Height (in) 7.736

Diameter (in) 3.278

Area (in^2) 8.438

Wet Unit Weight (pcf) 155.1

Dry Unit Weight (pcf) N/A

Moisture Content (%) N/A

Loading Rate (lbf/sec) 20

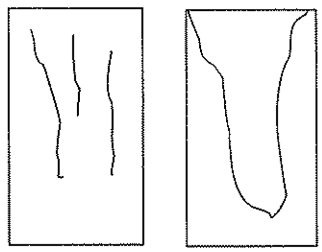
Peak Load (lbf) 8940

Failure Type Cone and Split

Compressive Strength (psi) 1060

Compressive Strength (tsf) 76

Failure Sketches



Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

Fuller Mossbarger Scott & May FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core
ASTM D 2938

Project Name Arkansas River Drilling
Project Number LX2007282
Lithology Shale, dark gray, soft
Lab ID UCR-13
Hole Number S2
Depth (ft/elev) 21.4' - 22.0'
Date Received 03-26-2008

Temperature (°C) 19.8
Moisture Condition As received, moist
Date Tested 04-11-2008

Side Planeness	N/A	Height (in)	6.703	Wet Unit Weight (pcf)	153.7
Perpendicularity	N/A	Diameter (in)	3.284	Dry Unit Weight (pcf)	N/A
End Planeness	N/A	Area (in^2)	8.472	Moisture Content (%)	N/A

Dimensions were not confirmed.

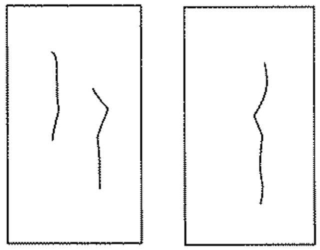
Loading Rate (lbf/sec) 20
Peak Load (lbf) 6280

Failure Type Undetermined

Compressive Strength (psi) 740

Compressive Strength (tsf) 53

Failure Sketches



Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

Fuller Mossbarger Scott & May
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Unconfined Compressive Strength Of Intact Rock Core

ASTM D 2938

Project Name: Arkansas River Drilling
Project Number: LX2007282
Lithology: Shale, dark gray, soft
Lab ID: UCR-14
Hole Number: S2
Depth (ft/elev): 31.2' - 31.8'
Date Received: 03-26-2008

Temperature (°C): 19.8
Moisture Condition: As received, moist
Date Tested: 04-11-2008

Side Planeness	N/A	Height (in)	8.248	Wet Unit Weight (pcf)	155.1
Perpendicularity	N/A	Diameter (in)	3.297	Dry Unit Weight (pcf)	N/A
End Planeness	N/A	Area (in^2)	8.536	Moisture Content (%)	N/A

Dimensions were not confirmed.

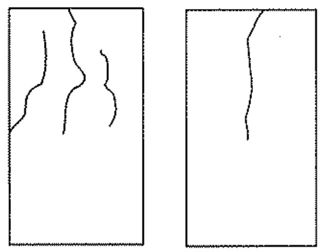
Loading Rate (lbf/sec): 20
Peak Load (lbf): 5360

Failure Type: Undetermined

Compressive Strength (psi): 630

Compressive Strength (tsf): 45

Failure Sketches



Comments: Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

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Unconfined Compressive Strength Of Intact Rock Core

ASTM D 2938

Project Name Arkansas River Drilling
Project Number LX2007282
Lithology Shale, dark gray, very soft
Lab ID UCR-15
Hole Number S4
Depth (ft/elev) 40.0' - 40.6'
Date Received 03-26-2008

Temperature (°C) 19.8
Moisture Condition As received, moist
Date Tested 04-11-2008

Side Planeness N/A
Perpendicularity N/A
End Planeness N/A
Dimensions were not confirmed.

Height (in) 6.809
Diameter (in) 3.376
Area (in^2) 8.953

Wet Unit Weight (pcf) 143.3
Dry Unit Weight (pcf) N/A
Moisture Content (%) N/A

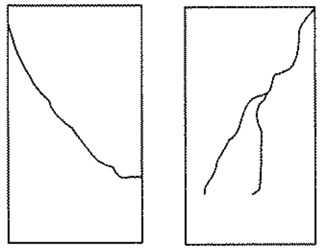
Loading Rate (lbf/sec) 20
Peak Load (lbf) 630

Failure Type Shear

Compressive Strength (psi) 70

Compressive Strength (tsf) 5

Failure Sketches



Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core
ASTM D 2938

Project Name Arkansas River Drilling
Project Number LX2007282
Lithology Shale, dark gray, soft
Lab ID UCR-16
Hole Number S4
Depth (ft/elev) 71.10' - 71.75'
Date Received 03-26-2008

Temperature (°C) 19.8
Moisture Condition As received, moist
Date Tested 04-11-2008

Side Planeness	N/A	Height (in)	8.063	Wet Unit Weight (pcf)	155.7
Perpendicularity	N/A	Diameter (in)	3.321	Dry Unit Weight (pcf)	N/A
End Planeness	N/A	Area (in^2)	8.662	Moisture Content (%)	N/A

Dimensions were not confirmed.

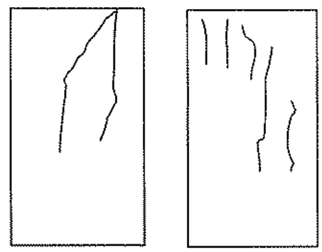
Loading Rate (lbf/sec) 20
Peak Load (lbf) 5010

Failure Type Undetermined

Compressive Strength (psi) 580

Compressive Strength (tsf) 42

Failure Sketches



Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

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Unconfined Compressive Strength Of Intact Rock Core

ASTM D 2938

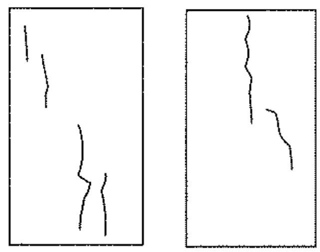
Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale , dark gray, soft	Lab ID	UCR-17
Hole Number	J1	Date Received	03-26-2008
Depth (ft/elev)	29.90' - 30.55		

Temperature (°C)	20.6	Moisture Condition	As received, moist	Date Tested	04-11-2008
Side Planeness	N/A	Height (in)	7.407	Wet Unit Weight (pcf)	161.4
Perpendicularity	N/A	Diameter (in)	3.312	Dry Unit Weight (pcf)	N/A
End Planeness	N/A	Area (in^2)	8.615	Moisture Content (%)	N/A

Dimensions were not confirmed.

Loading Rate (lbf/sec)	20
Peak Load (lbf)	11700
Failure Type	Undetermined
Compressive Strength (psi)	1360
Compressive Strength (tsf)	98

Failure Sketches



Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

Unconfined Compressive Strength Of Intact Rock Core

ASTM D 2938

Project Name: Arkansas River Drilling
Project Number: LX2007282
Lithology: Shale , dark gray, soft
Lab ID: UCR-18
Hole Number: J1
Depth (ft/elev): 47.80' - 48.45'
Date Received: 03-26-2008

Temperature (°C): 20.6
Moisture Condition: As received, moist
Date Tested: 04-11-2008

Side Planeness	N/A	Height (in)	7.436	Wet Unit Weight (pcf)	155.5
Perpendicularity	N/A	Diameter (in)	3.311	Dry Unit Weight (pcf)	N/A
End Planeness	N/A	Area (in^2)	8.610	Moisture Content (%)	N/A

Dimensions were not confirmed.

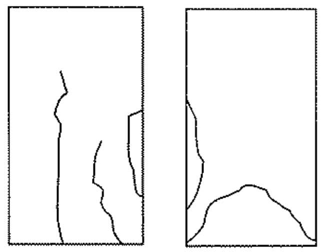
Loading Rate (lbf/sec): 20
Peak Load (lbf): 11880

Failure Type: Cone and Shear

Compressive Strength (psi): 1380

Compressive Strength (tsf): 99

Failure Sketches



Comments: Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

Fuller Mossbarger Scott & May

FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

ASTM D 2938

Project Name Arkansas River Drilling

Project Number LX2007282

Lithology Shale, dark gray, soft

Lab ID UCR-19

Hole Number J2

Depth (ft/elev) 15.6' - 16.3'

Date Received 03-26-2008

Temperature (°C) 20.5

Moisture Condition As received, moist

Date Tested 04-11-2008

Side Planeness N/A

Perpendicularity N/A

End Planeness N/A

Dimensions were not confirmed.

Height (in) 8.177

Diameter (in) 3.260

Area (in^2) 8.349

Wet Unit Weight (pcf) 149.2

Dry Unit Weight (pcf) N/A

Moisture Content (%) N/A

Loading Rate (lbf/sec) 20

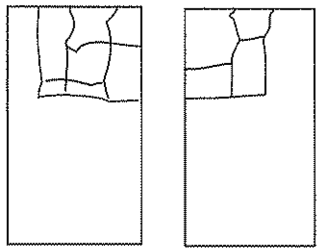
Peak Load (lbf) 2080

Failure Type Undetermined

Compressive Strength (psi) 250

Compressive Strength (tsf) 18

Failure Sketches



Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

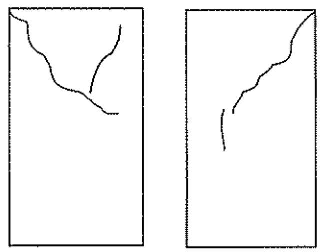
ASTM D 2938

Project Name	Arkansas River Drilling			Project Number	LX2007282
Lithology	Shale, dark gray, soft			Lab ID	UCR-20
Hole Number	J2	Depth (ft/elev)	33.00' - 33.65'	Date Received	03-26-2008
Temperature (°C)	20.6	Moisture Condition	As received, moist	Date Tested	04-11-2008
Side Planeness	N/A	Height (in)	7.620	Wet Unit Weight (pcf)	154.5
Perpendicularity	N/A	Diameter (in)	3.315	Dry Unit Weight (pcf)	N/A
End Planeness	N/A	Area (in^2)	8.629	Moisture Content (%)	N/A

Dimensions were not confirmed.

Loading Rate (lbf/sec)	20
Peak Load (lbf)	8620
Failure Type	Shear
Compressive Strength (psi)	1000
Compressive Strength (tsf)	72

Failure Sketches



Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

FMSM ENGINEERS
Fuller Mossbarger Scott & May

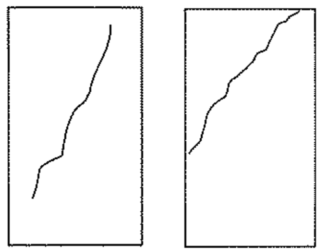
Unconfined Compressive Strength Of Intact Rock Core
ASTM D 2938

Project Name	Arkansas River Drilling			Project Number	LX2007282
Lithology	Shale, dark gray, very soft			Lab ID	UCR-21
Hole Number	J3	Depth (ft/elev)	9.2' - 10.0'	Date Received	03-26-2008
Temperature (°C)	20.5	Moisture Condition	As received, moist	Date Tested	04-11-2008
Side Planeness	N/A	Height (in)	7.618	Wet Unit Weight (pcf)	147.3
Perpendicularity	N/A	Diameter (in)	3.419	Dry Unit Weight (pcf)	N/A
End Planeness	N/A	Area (in^2)	9.181	Moisture Content (%)	N/A

Dimensions were not confirmed.

Loading Rate (lbf/sec)	20
Peak Load (lbf)	3550
Failure Type	Shear
Compressive Strength (psi)	390
Compressive Strength (tsf)	28

Failure Sketches



Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

Fuller Mossbarger Scott & May
FMSM
ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core
ASTM D 2938

Project Name: Arkansas River Drilling
Project Number: LX2007282
Lithology: Shale, dark gray, soft
Lab ID: UCR-22
Hole Number: J4
Depth (ft/elev): 12.2' - 12.8'
Date Received: 03-26-2008

Temperature (°C): 21.2
Moisture Condition: As received, moist
Date Tested: 04-11-2008

Side Planeness	N/A	Height (in)	7.211	Wet Unit Weight (pcf)	152.9
Perpendicularity	N/A	Diameter (in)	3.293	Dry Unit Weight (pcf)	N/A
End Planeness	N/A	Area (in^2)	8.517	Moisture Content (%)	N/A

Dimensions were not confirmed.

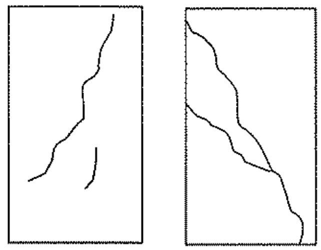
Loading Rate (lbf/sec): 20
Peak Load (lbf): 6790

Failure Type: Shear

Compressive Strength (psi): 800

Compressive Strength (tsf): 57

Failure Sketches



Comments: Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

Fuller Mossbarger Scott & May

ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

ASTM D 2938

Project Name Arkansas River Drilling

Project Number LX2007282

Lithology Shale, dark gray, soft

Lab ID UCR-23

Hole Number J4

Depth (ft/elev) 29.00' - 29.65'

Date Received 03-26-2008

Temperature (°C) 21.2

Moisture Condition As received, moist

Date Tested 04-11-2008

Side Planeness N/A

Perpendicularity N/A

End Planeness N/A

Dimensions were not confirmed.

Height (in) 8.016

Diameter (in) 3.326

Area (in^2) 8.690

Wet Unit Weight (pcf) 146.8

Dry Unit Weight (pcf) N/A

Moisture Content (%) N/A

Loading Rate (lbf/sec) 20

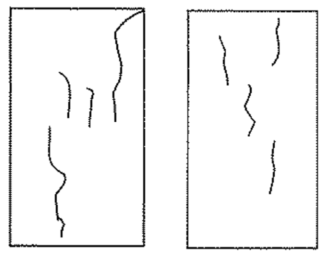
Peak Load (lbf) 3370

Failure Type Undetermined

Compressive Strength (psi) 390

Compressive Strength (tsf) 28

Failure Sketches



Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

FMSM ENGINEERS

Unconfined Compressive Strength Of Intact Rock Core

ASTM D 2938

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-24
Hole Number	J5	Date Received	03-26-2008
Depth (ft/elev)	28.4' - 29.0'		
Temperature (°C)	21.2	Date Tested	04-11-2008
Moisture Condition	As received, moist		

Side Planeness	N/A	Height (in)	8.042	Wet Unit Weight (pcf)	147.1
Perpendicularity	N/A	Diameter (in)	3.306	Dry Unit Weight (pcf)	N/A
End Planeness	N/A	Area (in^2)	8.582	Moisture Content (%)	N/A

Dimensions were not confirmed.

Loading Rate (lbf/sec) 20

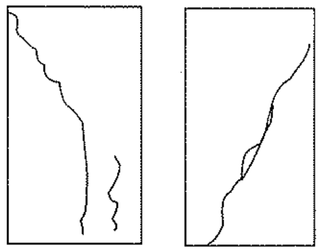
Peak Load (lbf) 3220

Failure Type Shear

Compressive Strength (psi) 380

Compressive Strength (tsf) 27

Failure Sketches



Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

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Unconfined Compressive Strength Of Intact Rock Core
ASTM D 2938

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-25
Hole Number	J5	Depth (ft/elev)	59.00' - 59.65'
Date Received	03-26-2008		

Temperature (°C) 21.2 Moisture Condition As received, moist Date Tested 04-11-2008

Side Planeness	N/A	Height (in)	8.093	Wet Unit Weight (pcf)	150.9
Perpendicularity	N/A	Diameter (in)	3.291	Dry Unit Weight (pcf)	N/A
End Planeness	N/A	Area (in^2)	8.505	Moisture Content (%)	N/A

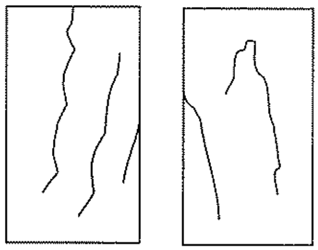
Dimensions were not confirmed.

Loading Rate (lbf/sec) 20
Peak Load (lbf) 3690

Failure Type Shear

Compressive Strength (psi) 430

Compressive Strength (tsf) 31

Failure Sketches


Comments Fragile nature of specimen inhibited preparation. Dimensional tolerances were not confirmed.

Rock Core Unconfined Test Photographs

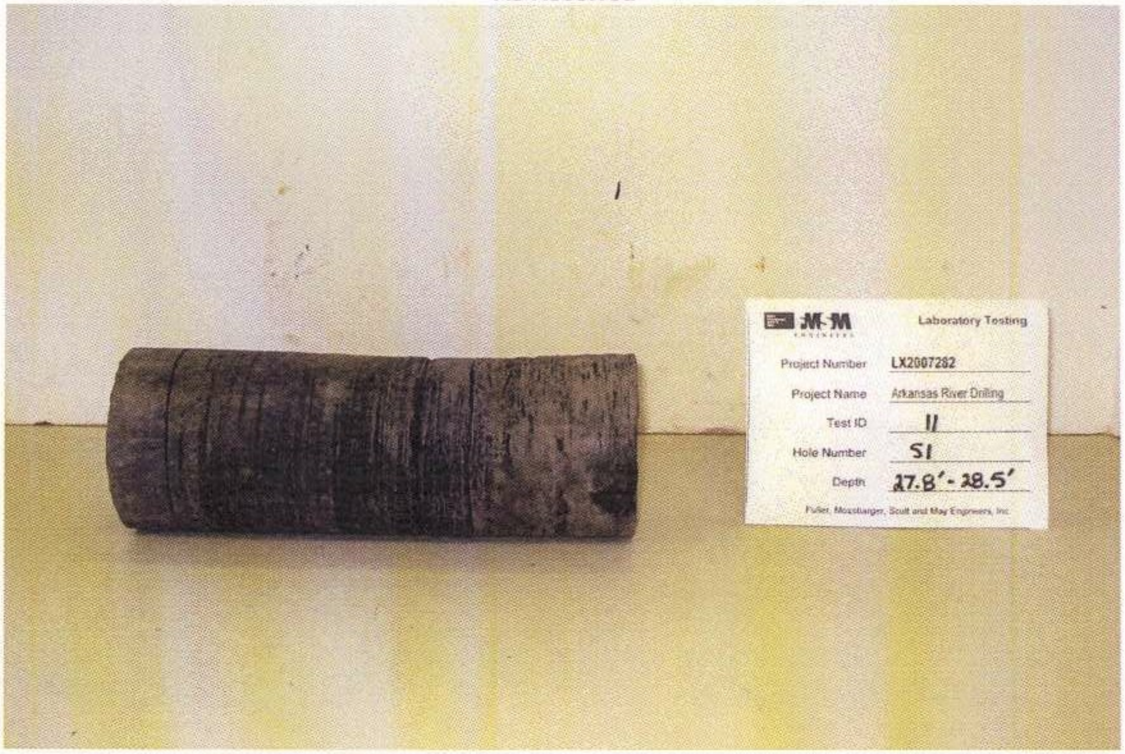
Fuller Mossbarger Scott & May
ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number S1 Depth (ft) 27.9' - 28.5'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-11

As Received



Core Preparation

Fuller Mossbarger Scott & May ENGINEERS

Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-11
Hole Number	S1 Depth (ft) 27.9' - 28.5'		
Test Type	Unconfined compressive strength		

Core Preparation

Post Test

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number S1 Depth (ft) 27.9' - 28.5'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-11

Post Test

Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-12
Hole Number	S1	Depth (ft)	45.2' - 45.8'
Test Type	Unconfined compressive strength		

As Received

Core Preparation

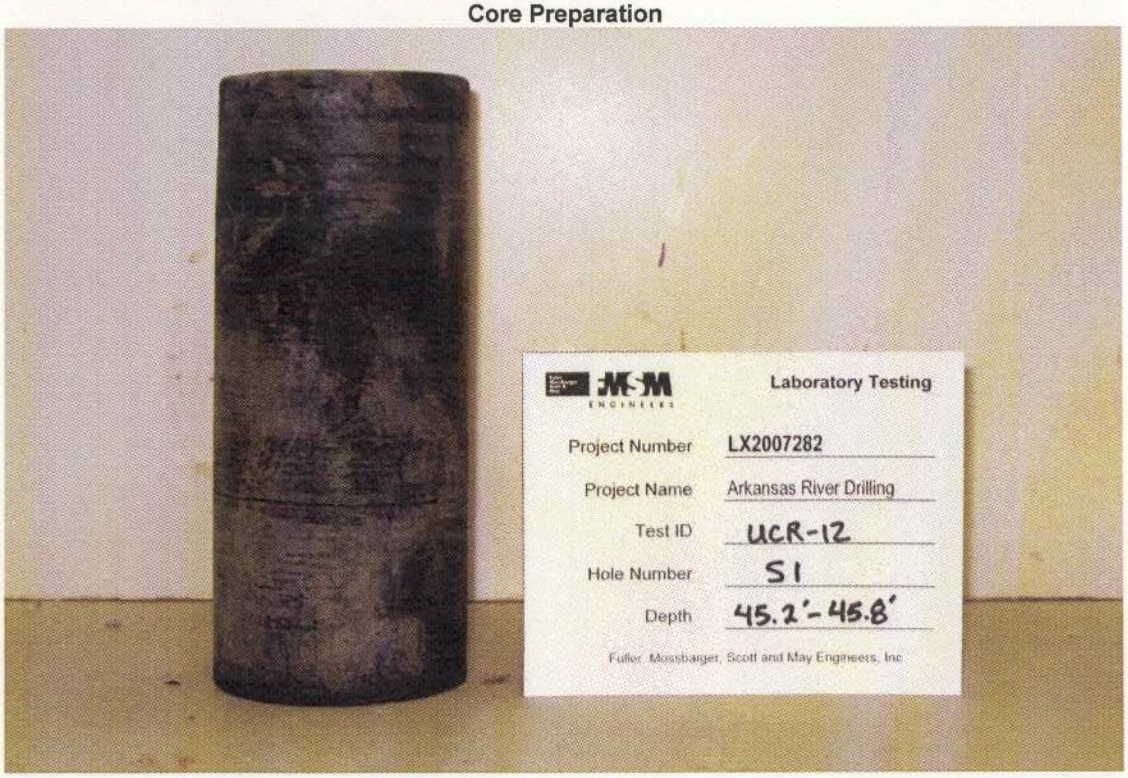


Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number S1 Depth (ft) 45.2' - 45.8'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-12

Core Preparation

Post Test

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-12
Hole Number	S1	Depth (ft)	45.2' - 45.8'
Test Type	Unconfined compressive strength		

Post Test

Fuller Mossbarger Scott & May
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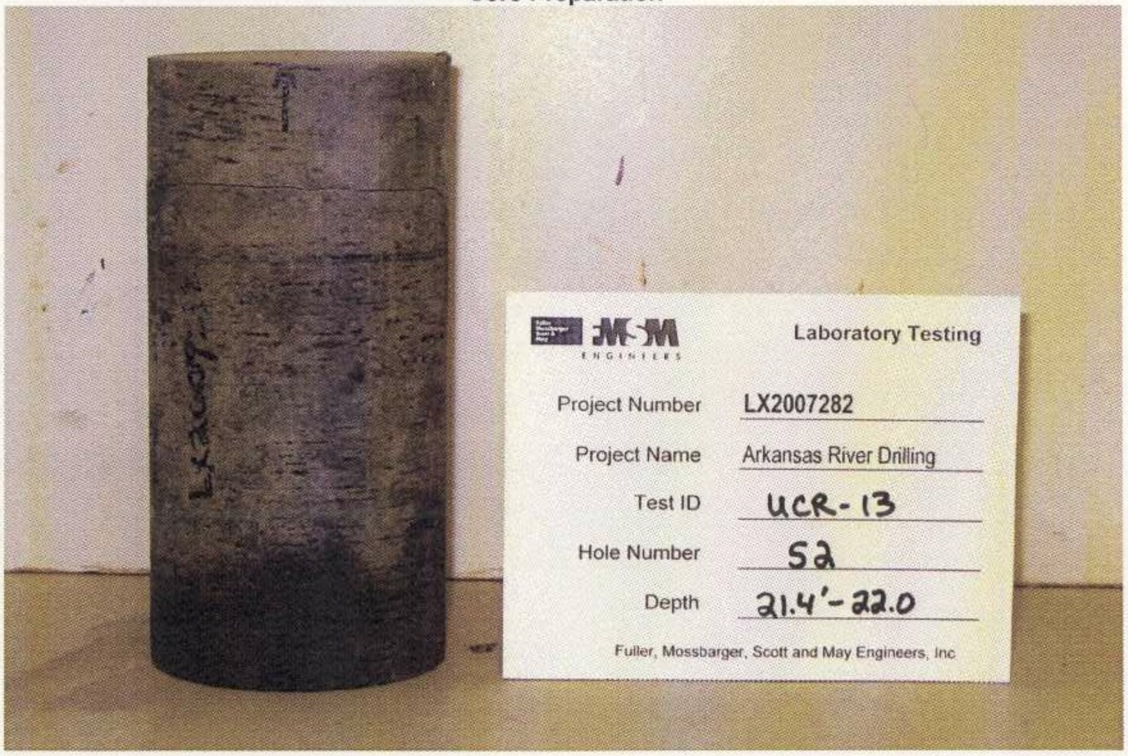
Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-13
Hole Number	S2	Depth (ft)	21.4' - 22.0'
Test Type	Unconfined compressive strength		

As Received

Core Preparation



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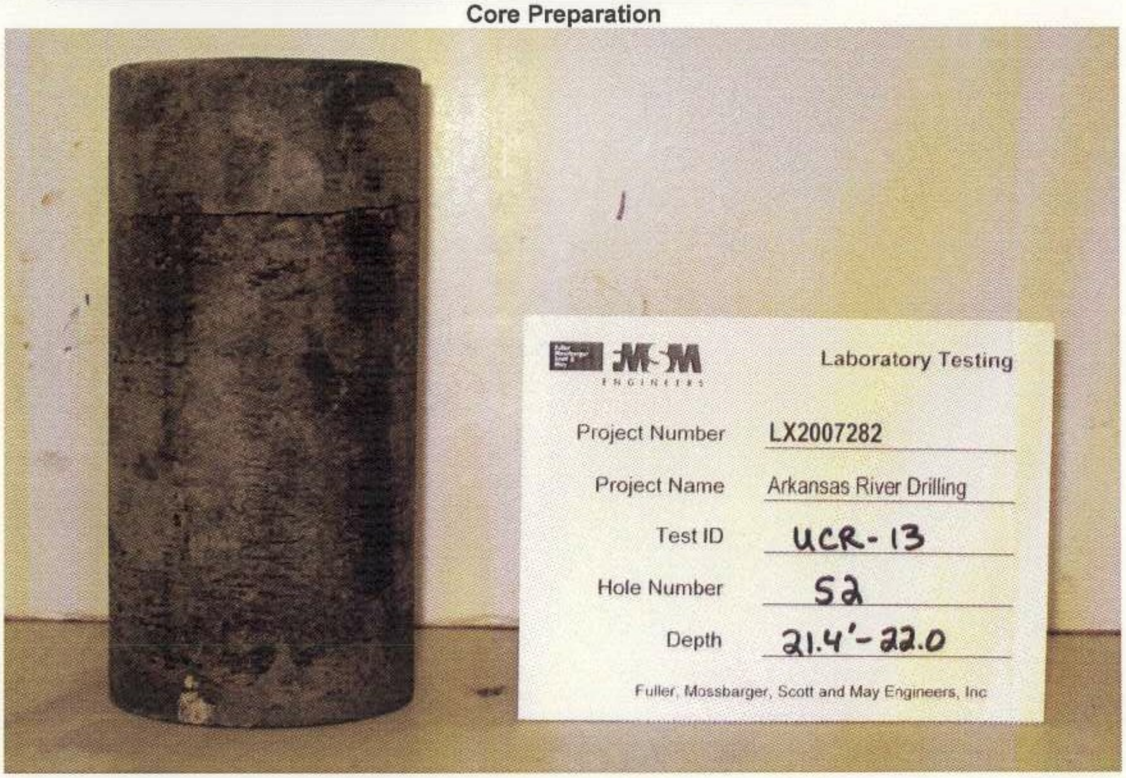
ENGINEERS

Photo Report

Project Name	Arkansas River Drilling
Lithology	Shale, dark gray, soft
Hole Number	S2
Depth (ft)	21.4' - 22.0'
Test Type	Unconfined compressive strength
Project Number	LX2007282
Lab ID	UCR-13

Core Preparation



Post Test

Photo Report

Project Name Arkansas River Drilling

Lithology Shale, dark gray, soft

Hole Number S2 Depth (ft) 21.4' - 22.0'

Test Type Unconfined compressive strength

Project Number LX2007282

Lab ID UCR-13

Post Test

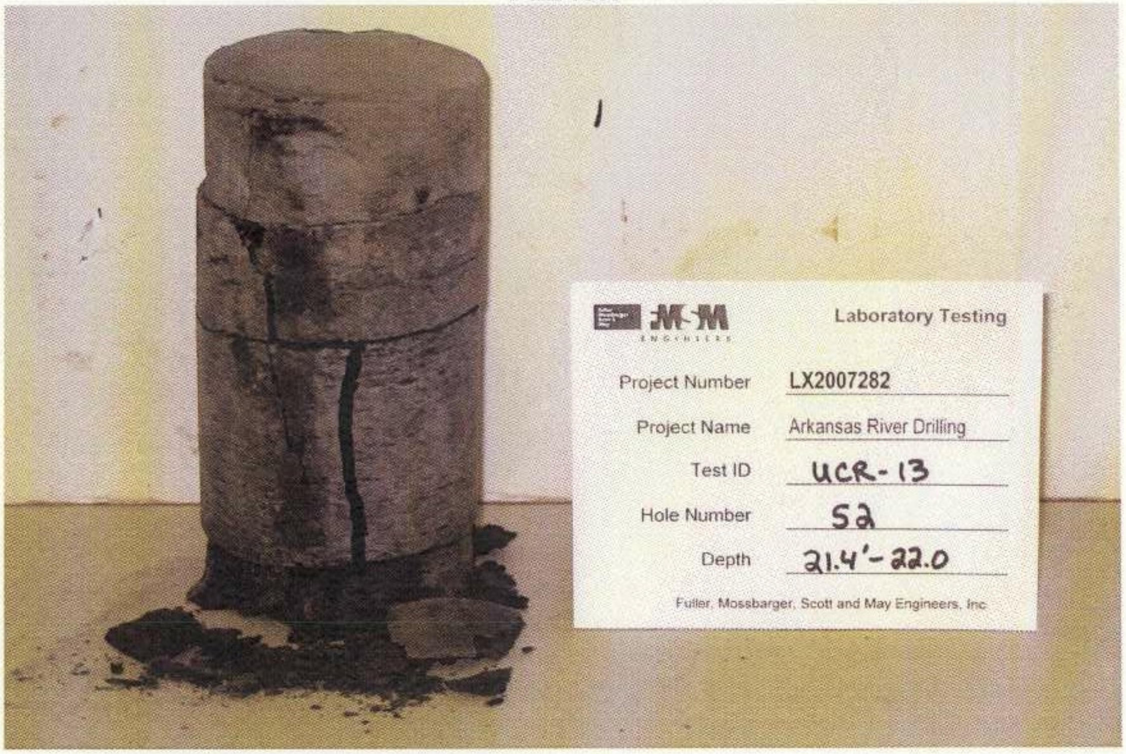


Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-14
Hole Number	S2	Depth (ft)	31.2' - 31.8'
Test Type	Unconfined compressive strength		

As Received

Core Preparation

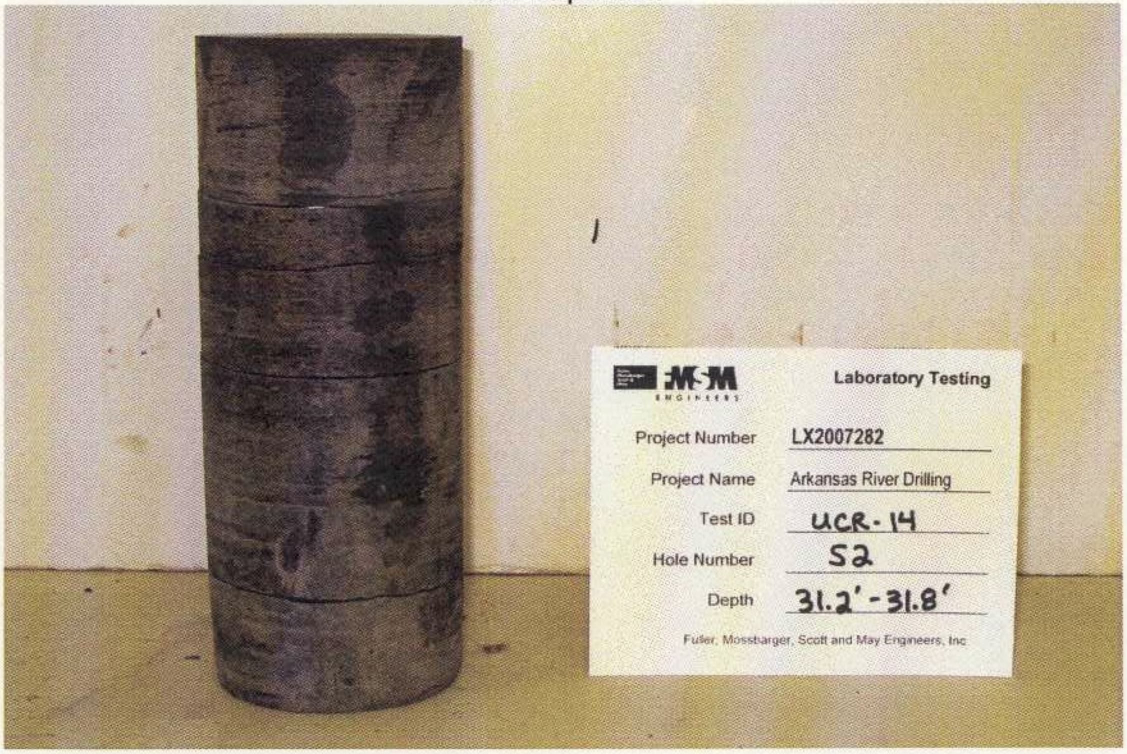


Photo Report

Project Name Arkansas River Drilling

Lithology Shale, dark gray, soft

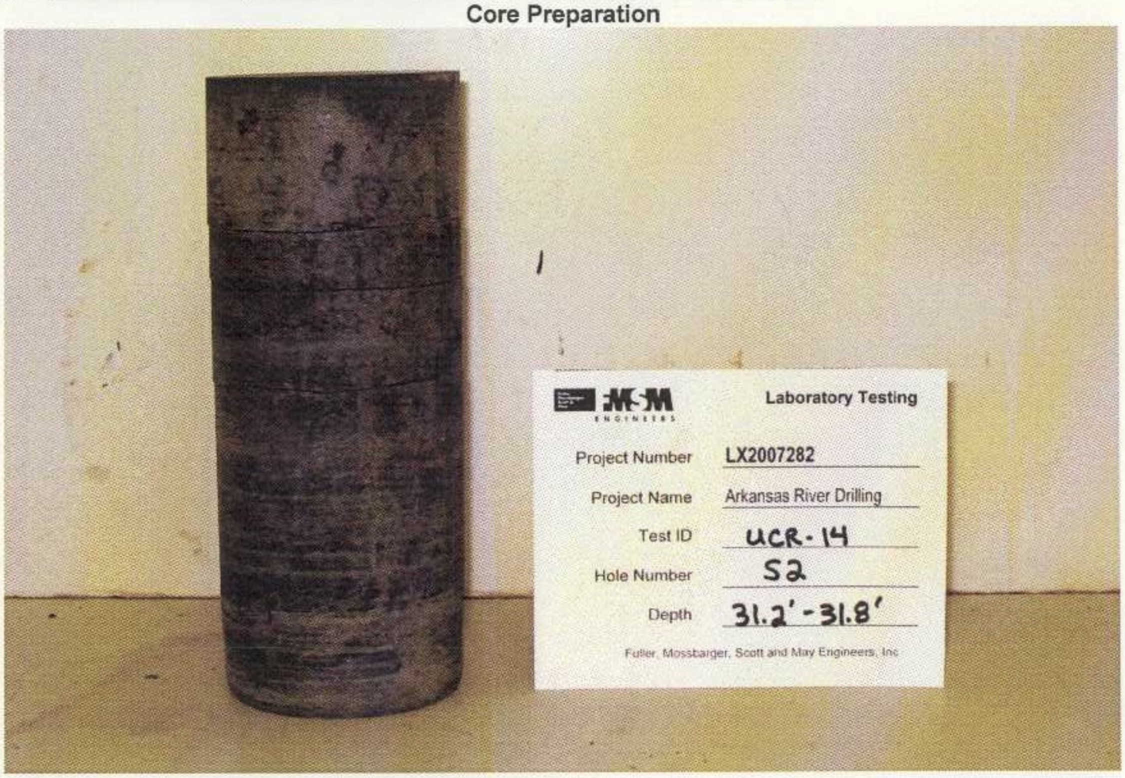
Hole Number S2 Depth (ft) 31.2' - 31.8'

Test Type Unconfined compressive strength

Project Number LX2007282

Lab ID UCR-14

Core Preparation



Post Test

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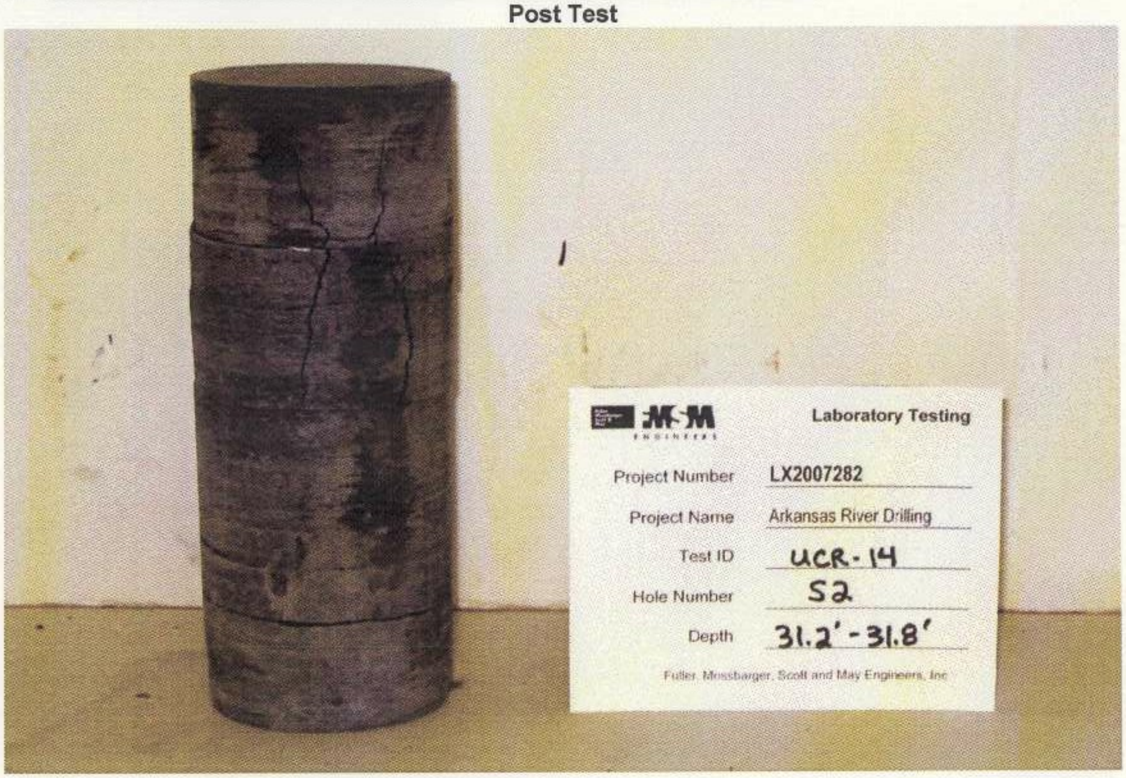
ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number S2 Depth (ft) 31.2' - 31.8'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-14

Post Test



Fuller Mossbarger Scott & May

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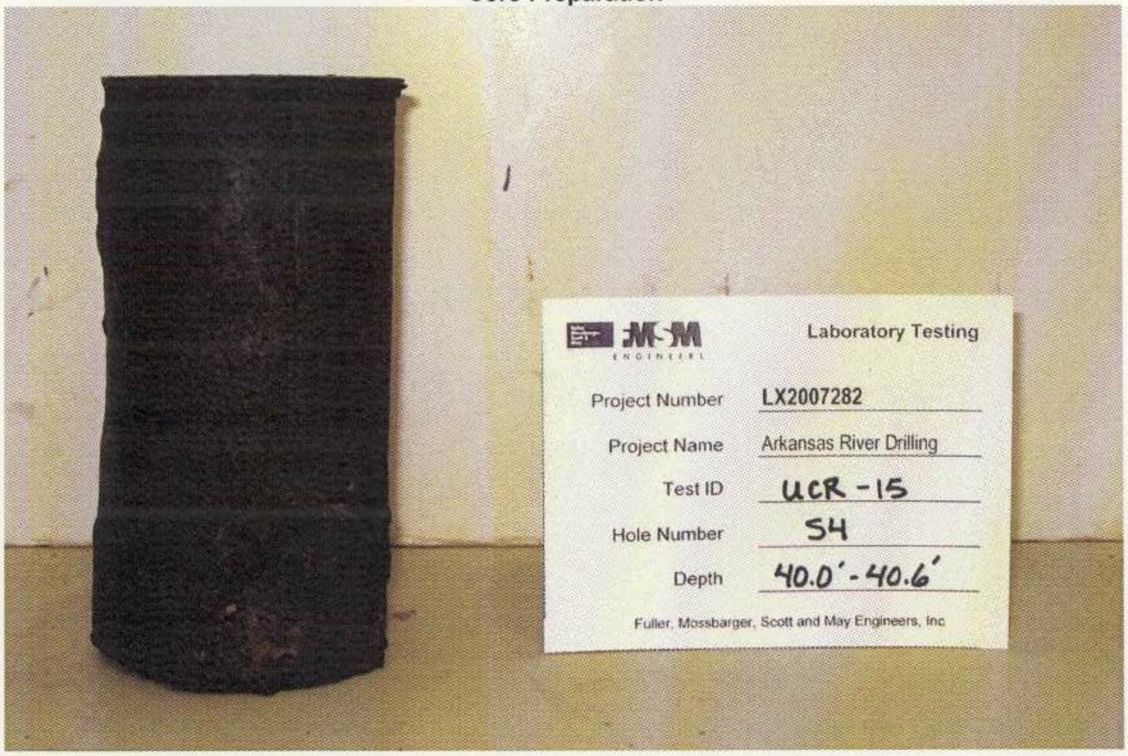
Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, very soft
Hole Number S4 Depth (ft) 40.0' - 40.6'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-15

As Received

Core Preparation



ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, very soft
Hole Number S4 Depth (ft) 40.0' - 40.6'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-15

Post Test

Post Test

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number S4 Depth (ft) 71.10' - 71.75'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-16

As Received

Core Preparation

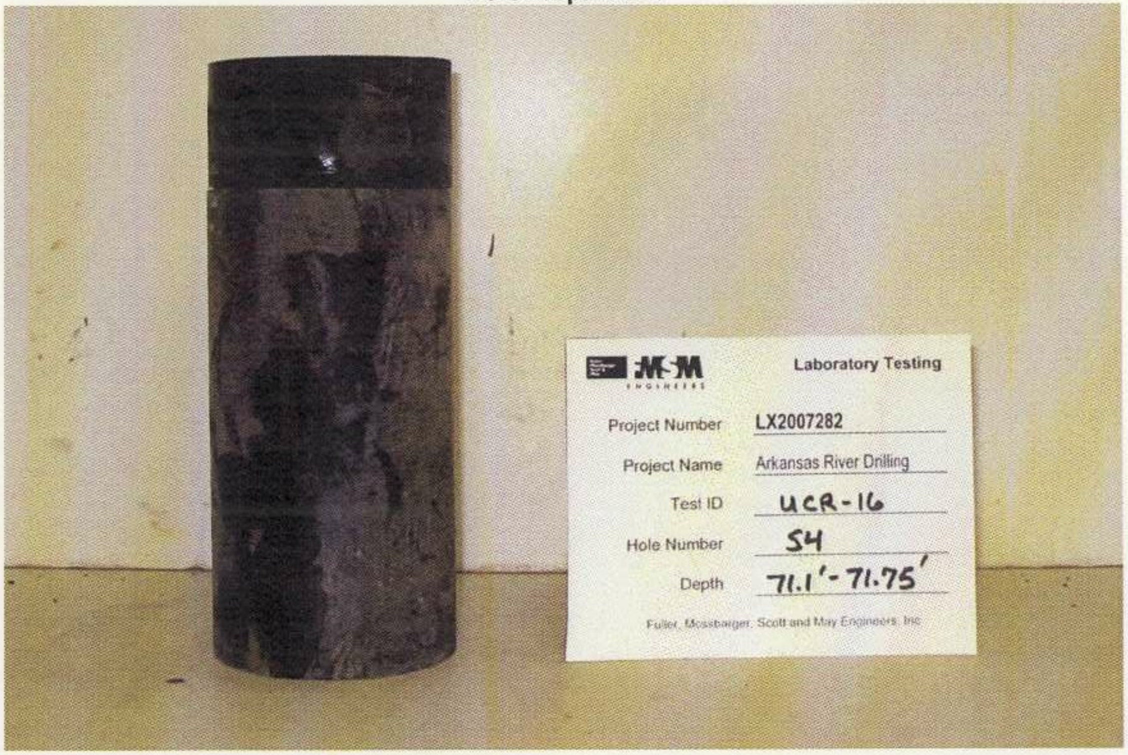


Photo Report

Project Name Arkansas River Drilling

Lithology Shale, dark gray, soft

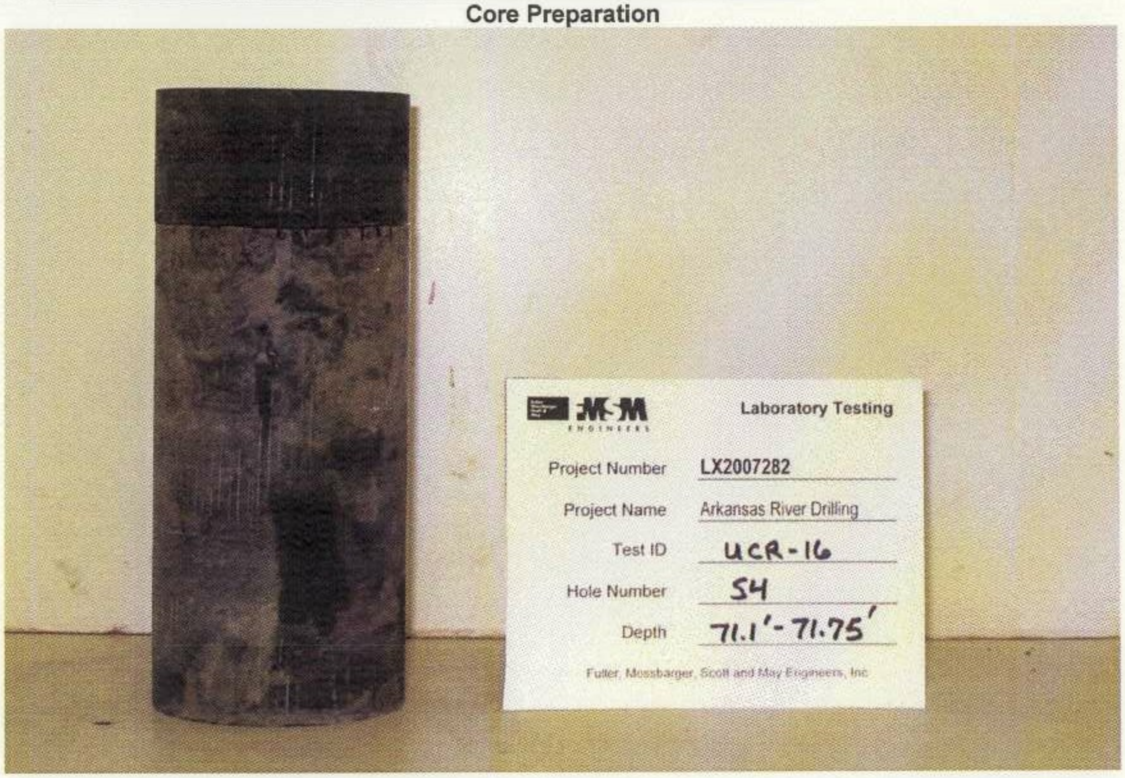
Hole Number S4 Depth (ft) 71.10' - 71.75'

Test Type Unconfined compressive strength

Project Number LX2007282

Lab ID UCR-16

Core Preparation



Post Test

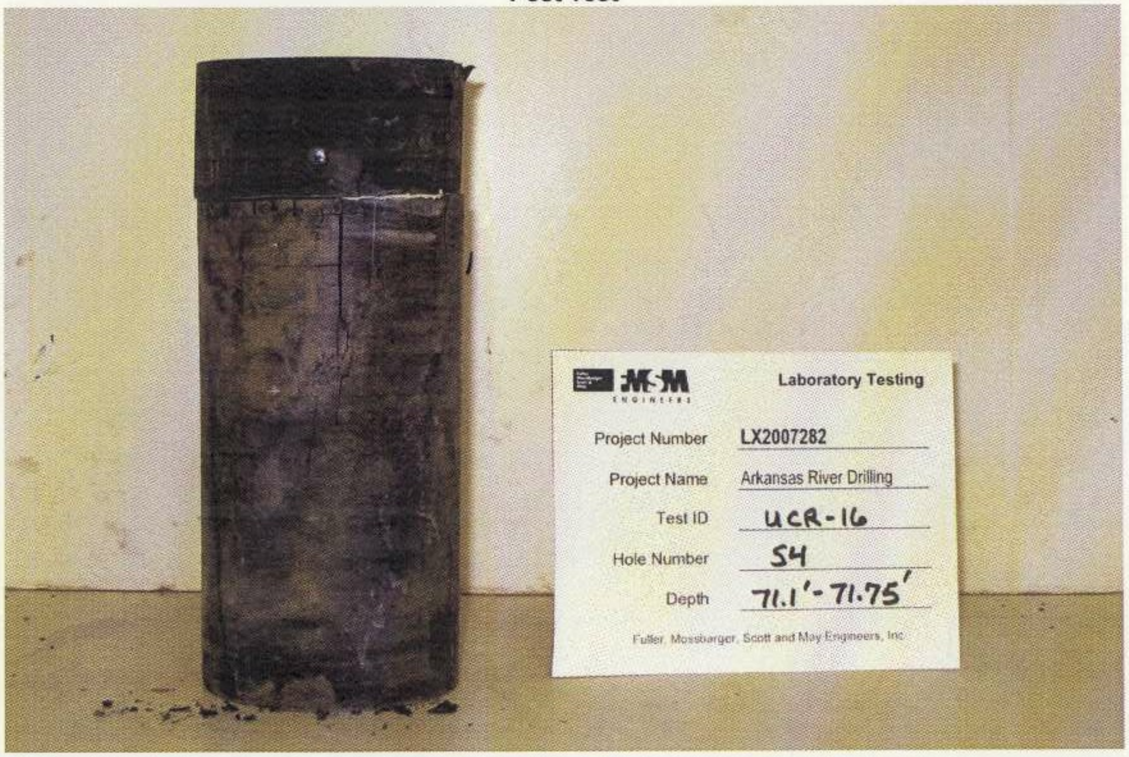


Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-16
Hole Number	S4	Depth (ft)	71.10' - 71.75'
Test Type	Unconfined compressive strength		

Post Test

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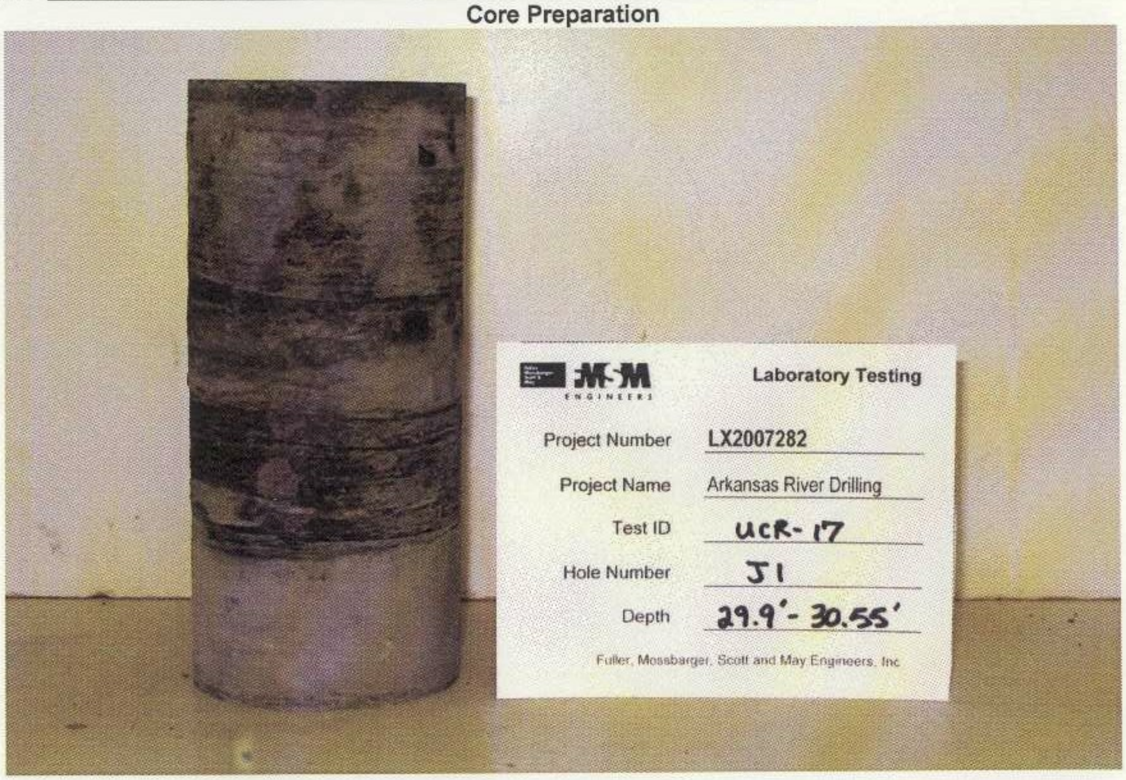
Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale , dark gray, soft	Lab ID	UCR-17
Hole Number	J1	Depth (ft)	29.90' - 30.55
Test Type	Unconfined compressive strength		

Core Preparation



Core Preparation

Fuller Mossbarger Scott & May ENGINEERS

Photo Report

Project Name Arkansas River Drilling

Lithology Shale , dark gray, soft

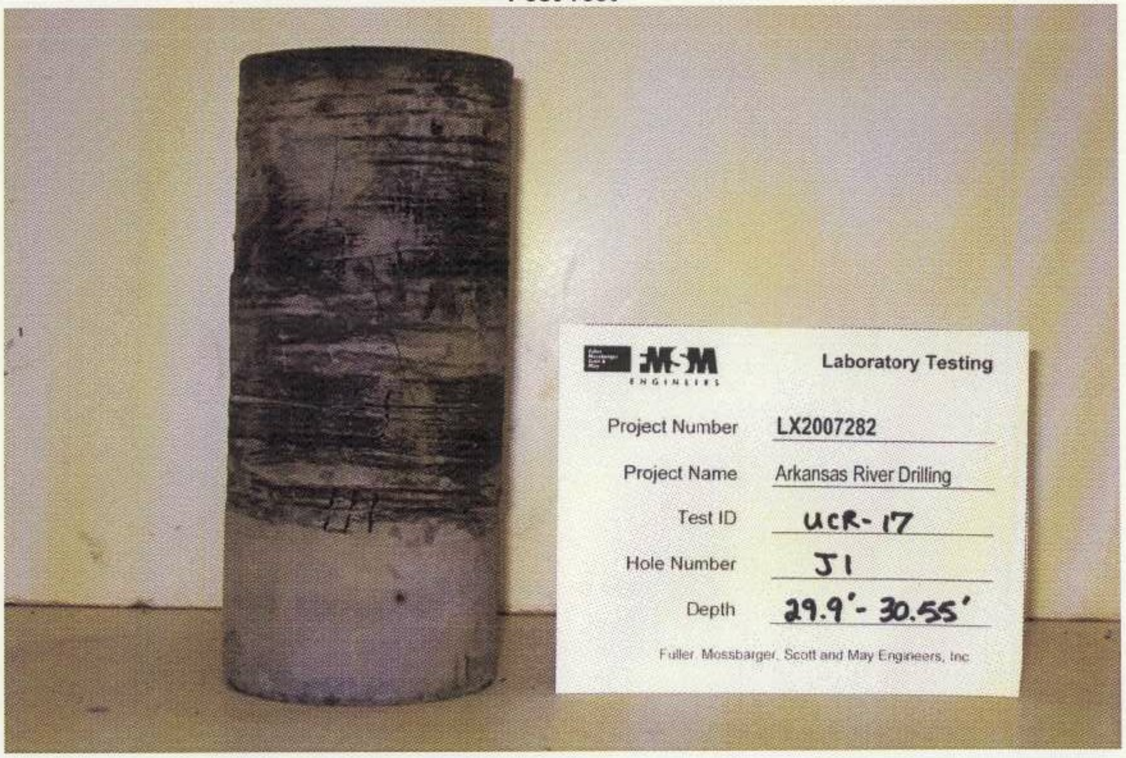
Hole Number J1 Depth (ft) 29.90' - 30.55

Test Type Unconfined compressive strength

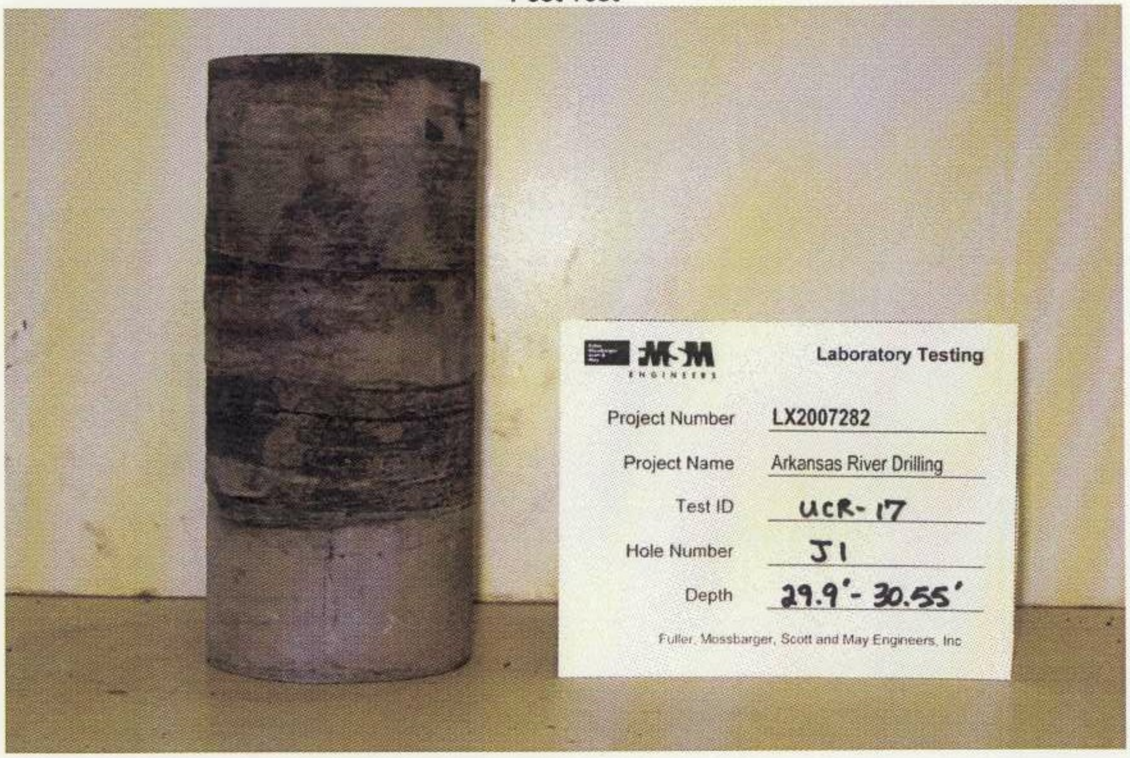
Project Number LX2007282

Lab ID UCR-17

Post Test



Post Test



ENGINEERS

Photo Report

Project Name Arkansas River Drilling

Project Number LX2007282

Lithology Shale , dark gray, soft

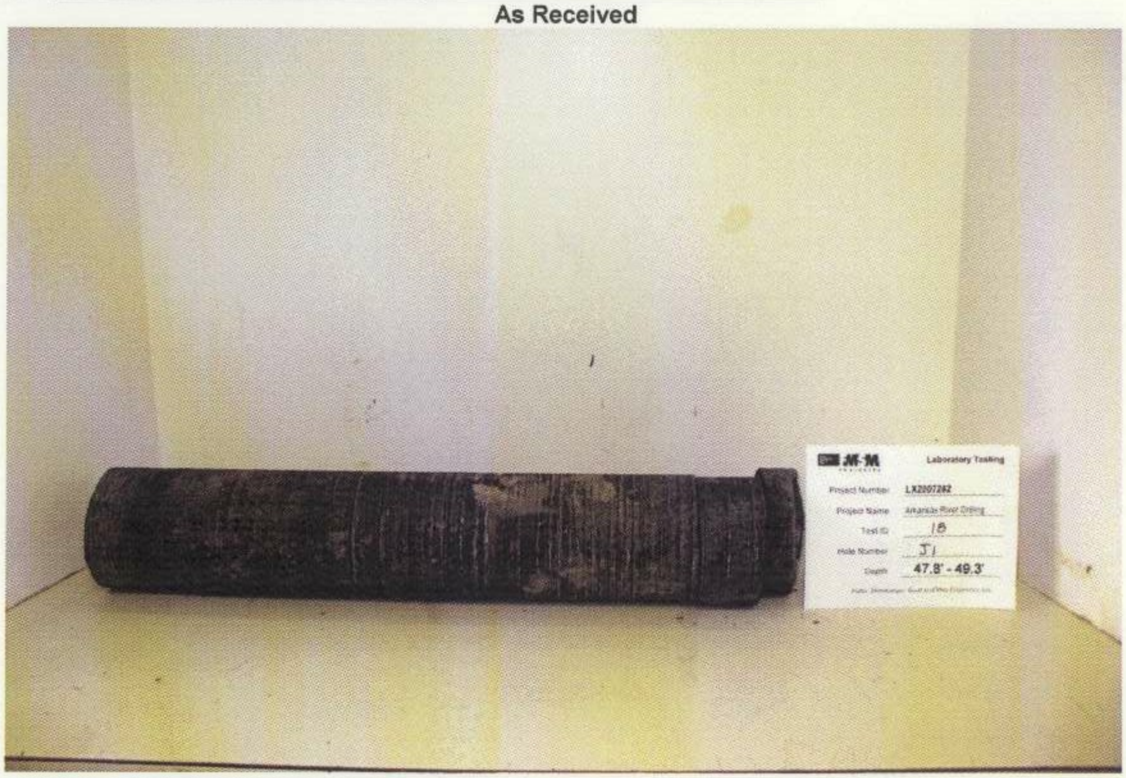
Lab ID UCR-18

Hole Number J1

Depth (ft) 47.80' - 48.45'

Test Type Unconfined compressive strength

As Received



Core Preparation

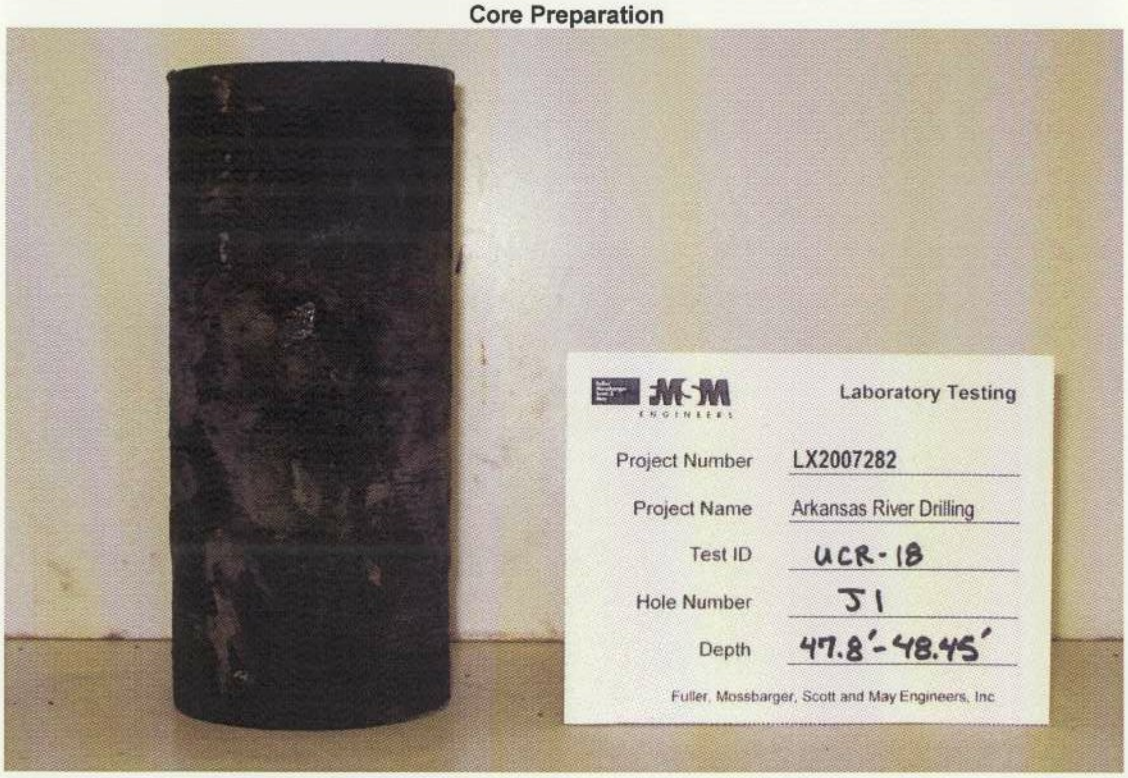


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Photo Report

Project Name Arkansas River Drilling

Lithology Shale , dark gray, soft

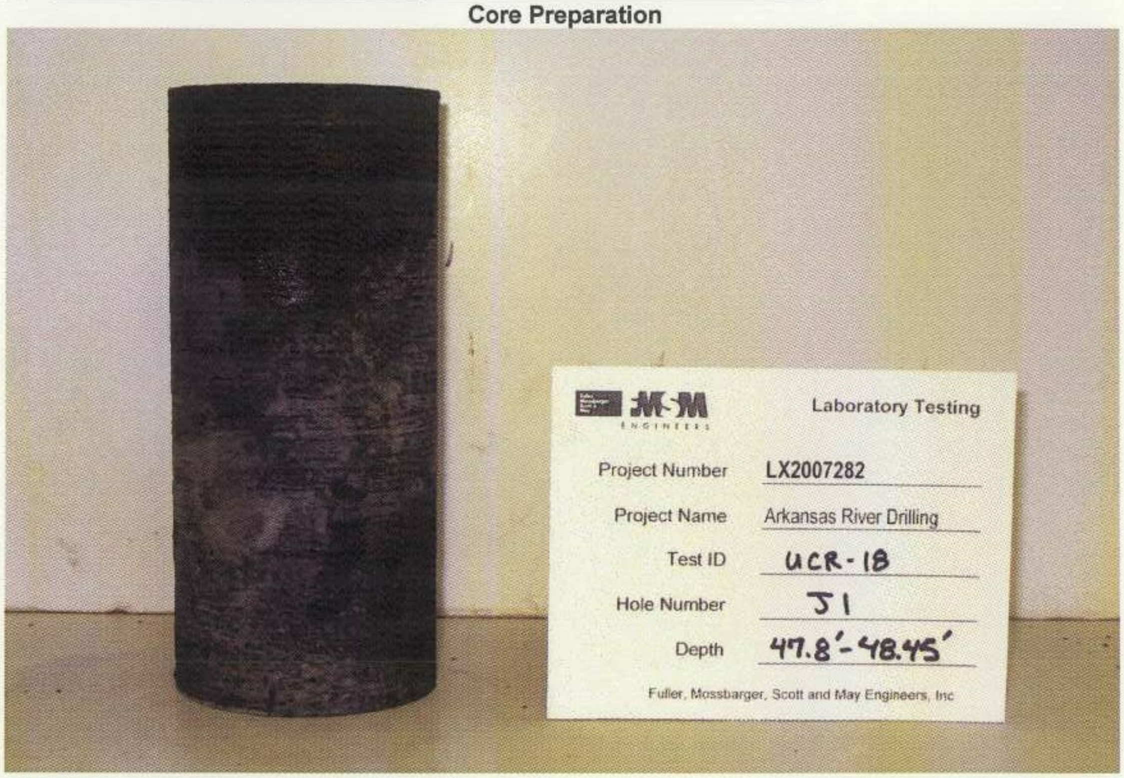
Hole Number J1 Depth (ft) 47.80' - 48.45'

Test Type Unconfined compressive strength

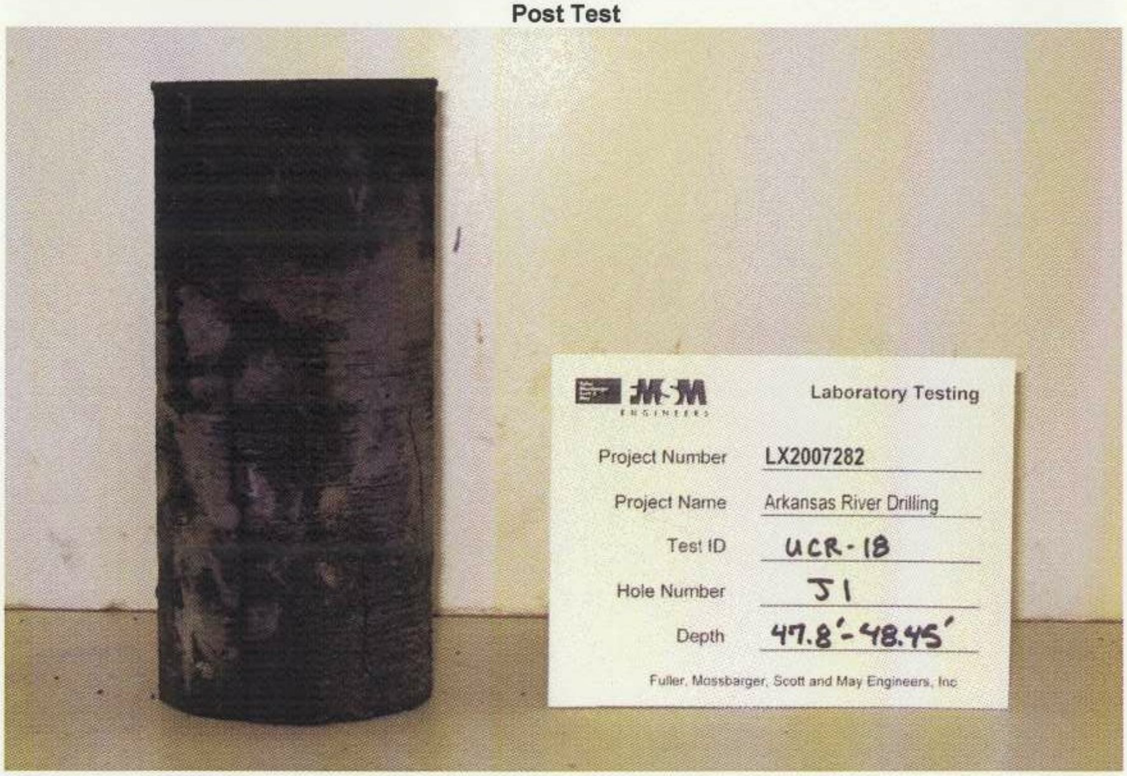
Project Number LX2007282

Lab ID UCR-18

Core Preparation



Post Test



Fuller Mossbarger Scott & May
FMSM ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale , dark gray, soft
Hole Number J1 Depth (ft) 47.80' - 48.45'
Test Type Unconfined compressive strength

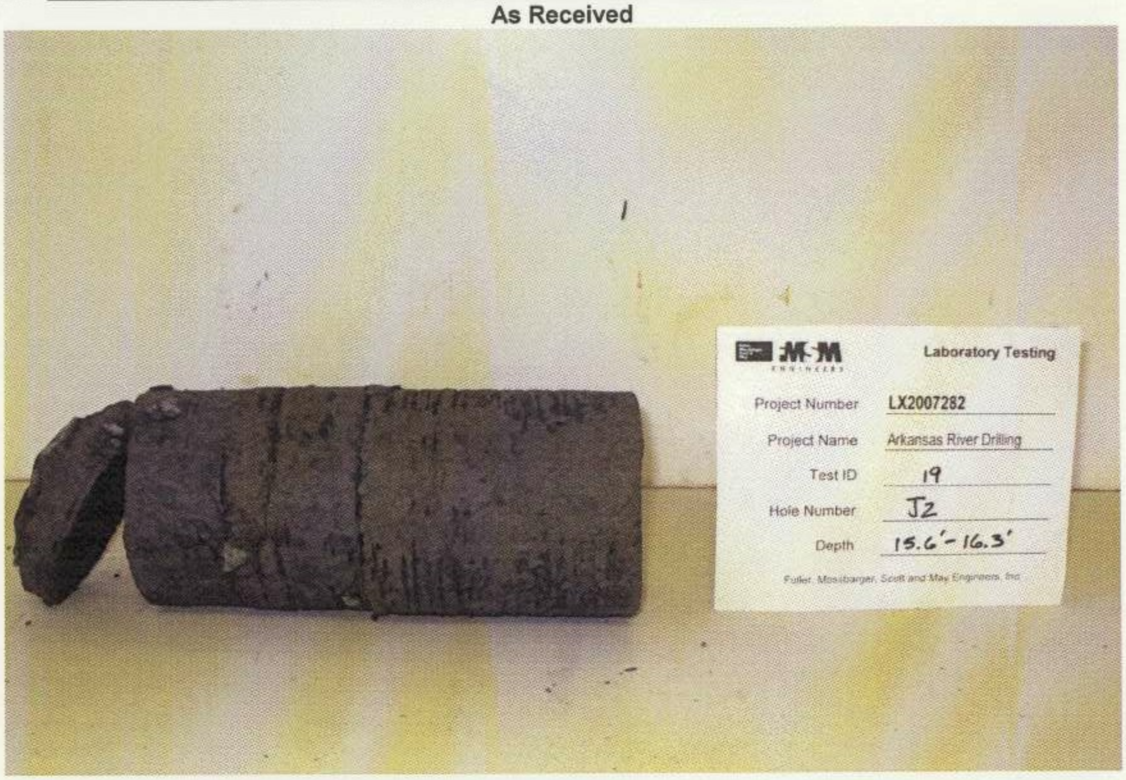
Project Number LX2007282
Lab ID UCR-18

Post Test

Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-19
Hole Number	J2	Depth (ft)	15.6' - 16.3'
Test Type	Unconfined compressive strength		

As Received



Core Preparation

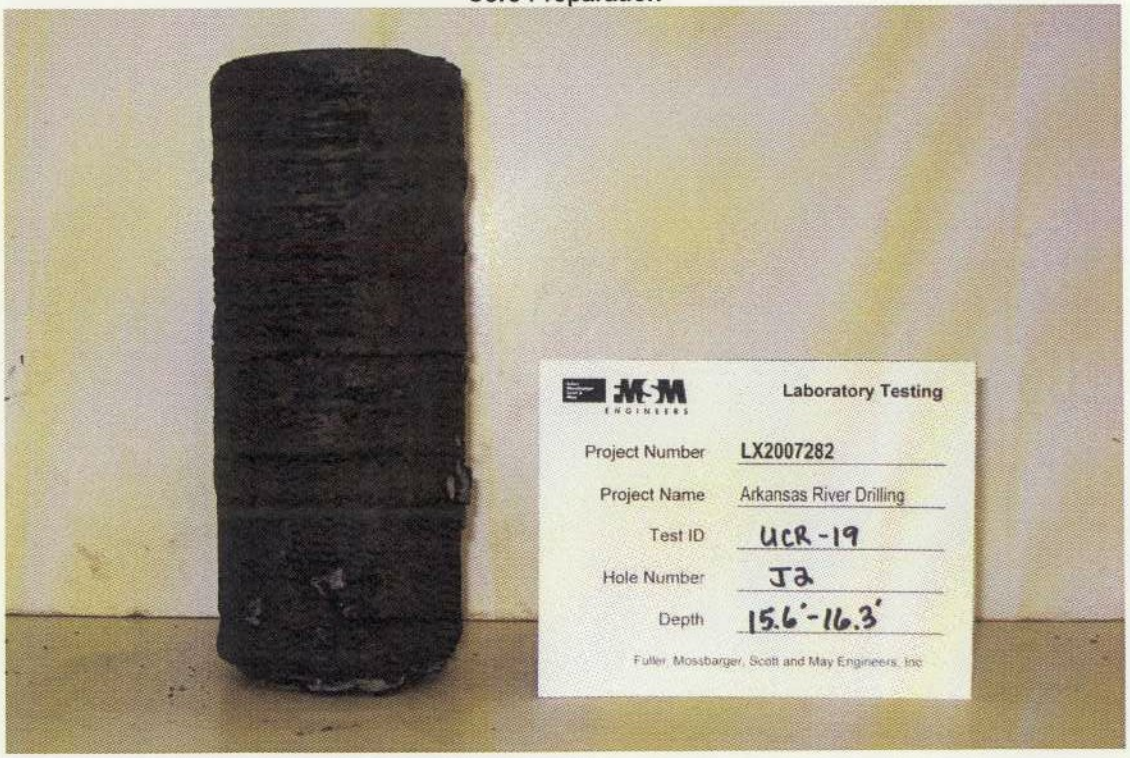


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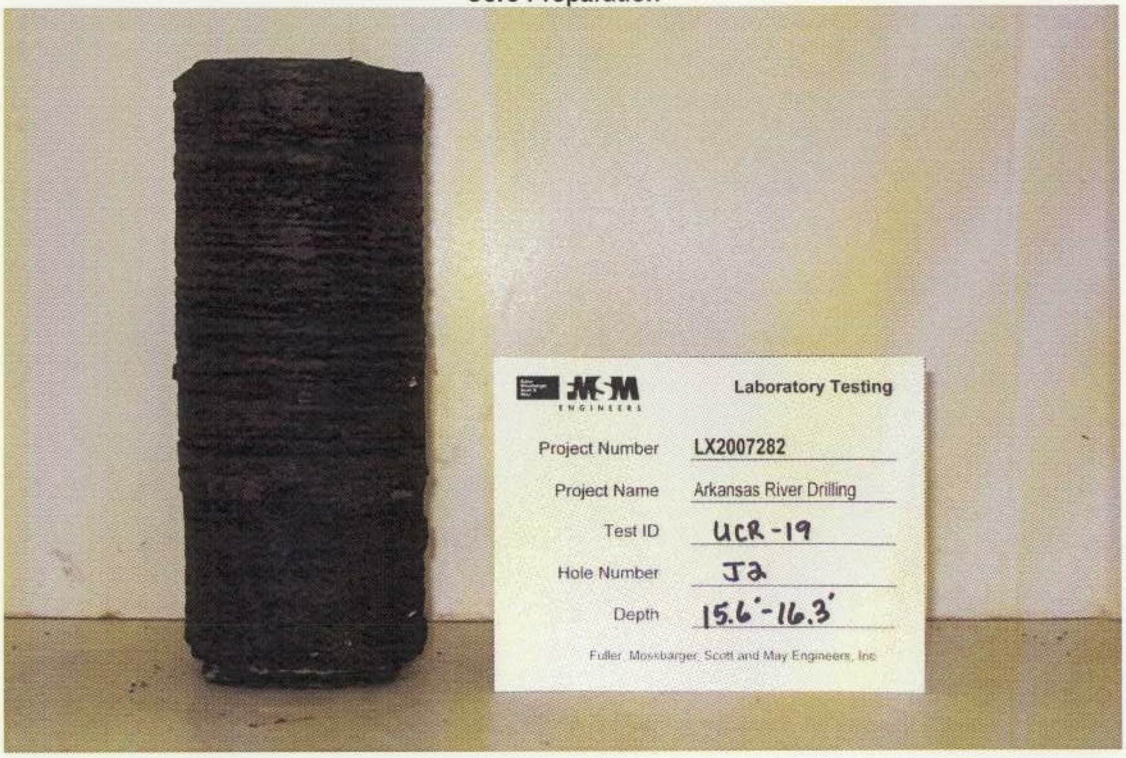
Photo Report

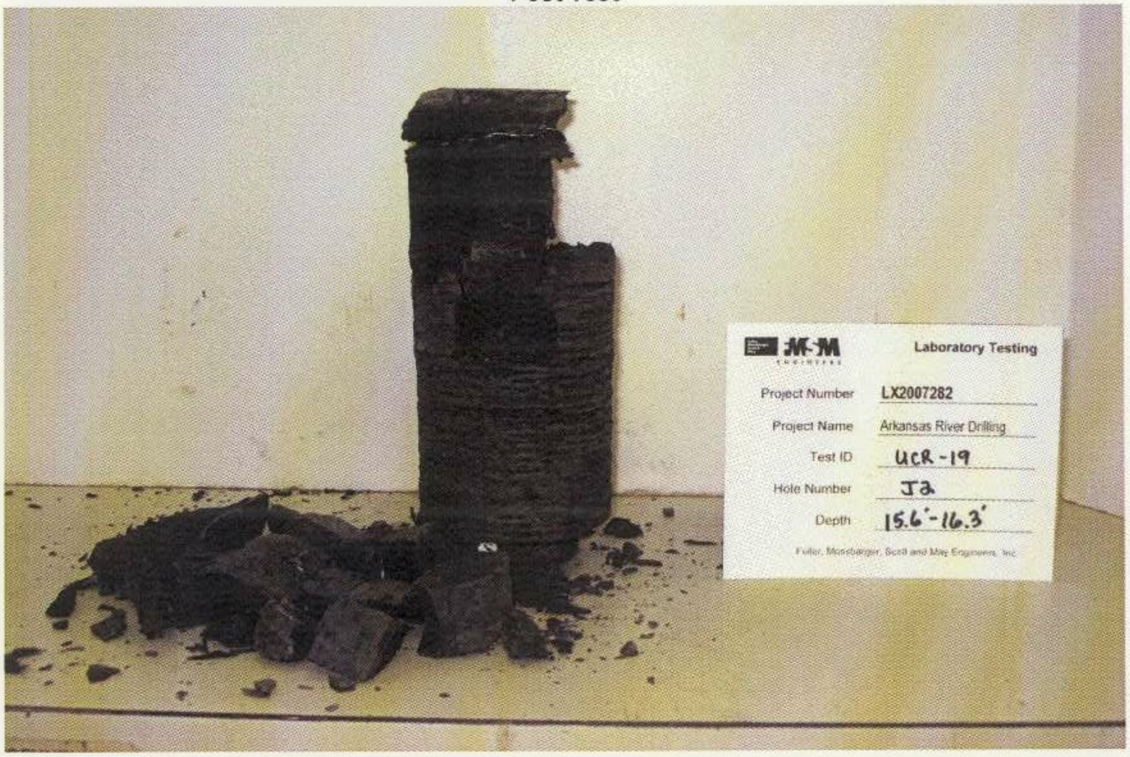
Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number J2   Depth (ft) 15.6' - 16.3'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-19

Core Preparation



Post Test



Fuller Mossbarger Scott & May ENGINEERS

Photo Report

Project Name Arkansas River Drilling

Lithology Shale, dark gray, soft

Hole Number J2 Depth (ft) 15.6' - 16.3'

Test Type Unconfined compressive strength

Project Number LX2007282

Lab ID UCR-19

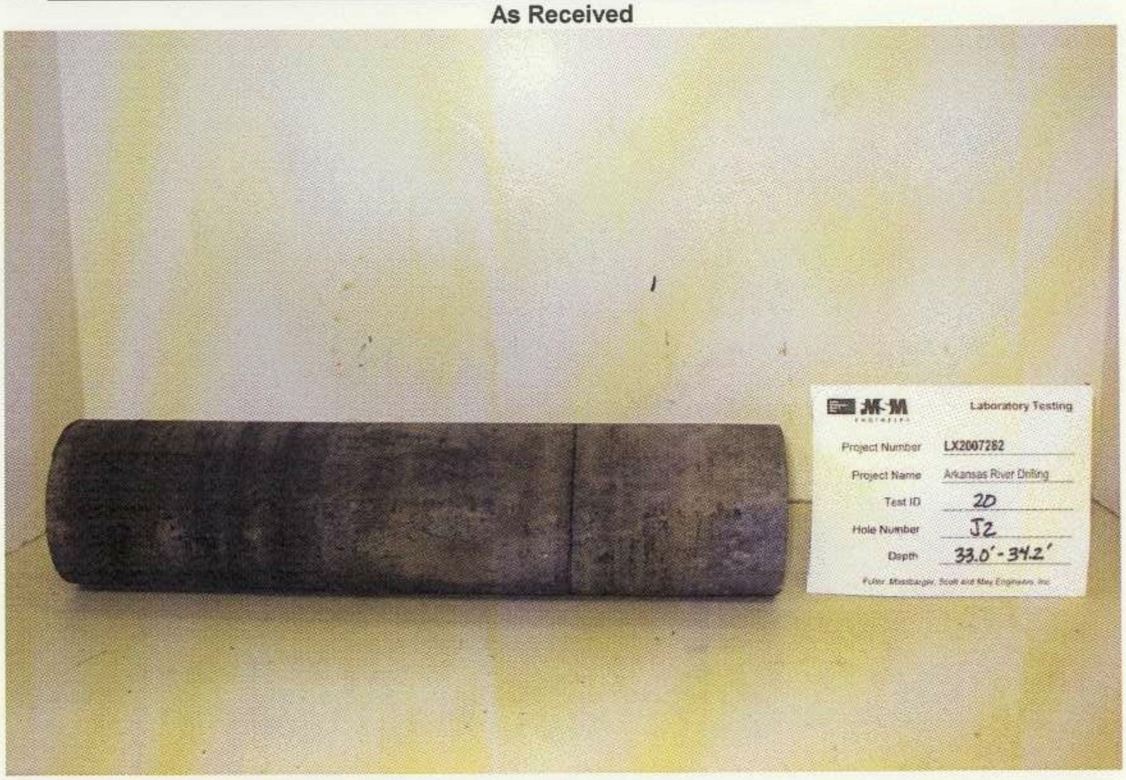
Post Test

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number J2 Depth (ft) 33.00' - 33.65'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-20

As Received



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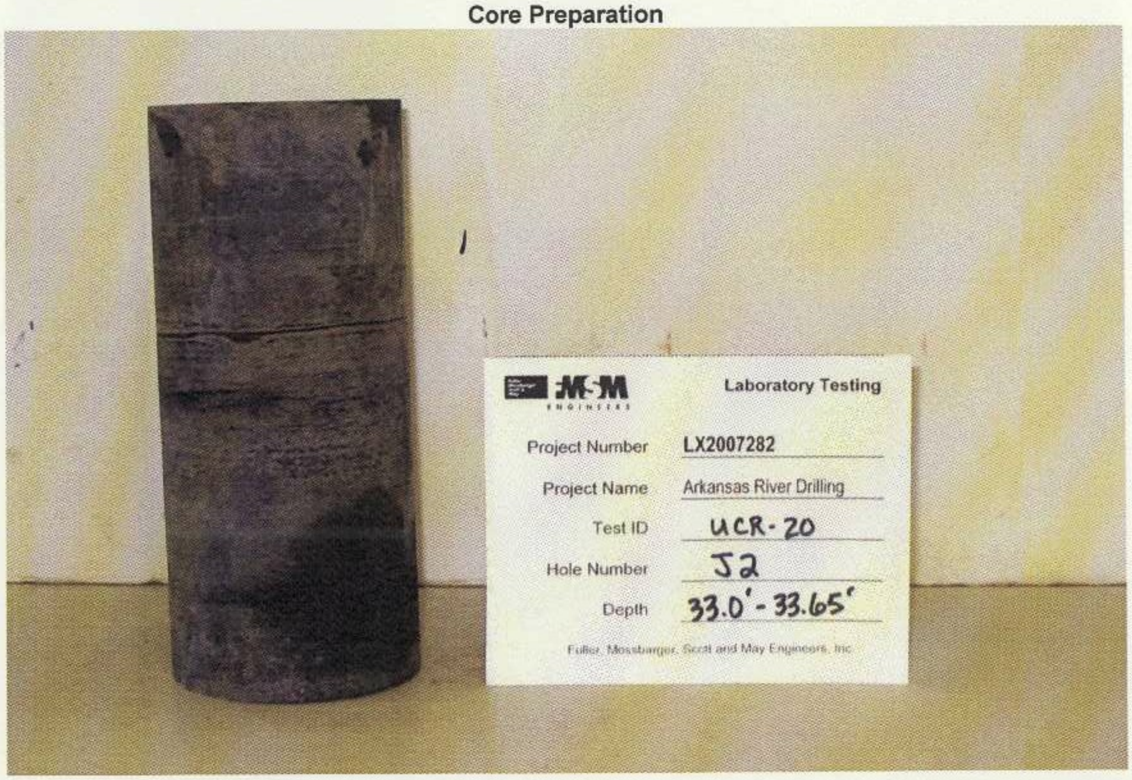


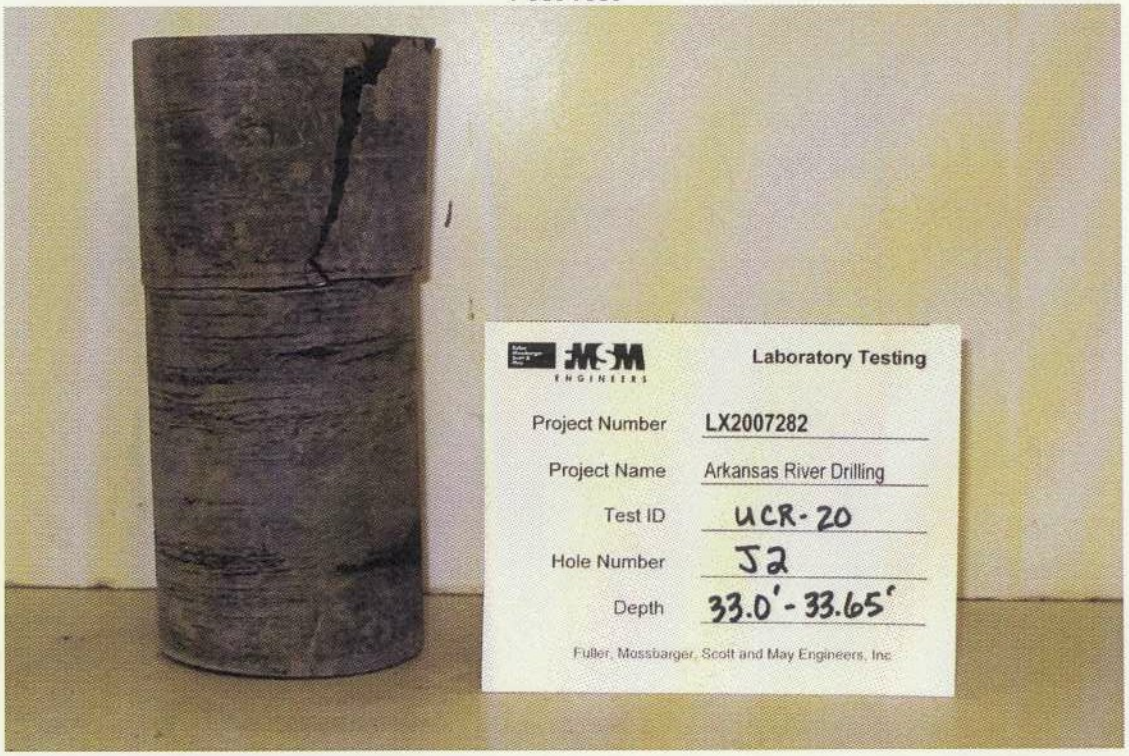
Fuller Mossbarger Scott & May
FMSM ENGINEERS

Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-20
Hole Number	J2	Depth (ft)	33.00' - 33.65'
Test Type	Unconfined compressive strength		

Core Preparation

Post Test



Fuller Mossbarger Scott & May
ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number J2 Depth (ft) 33.00' - 33.65'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-20

Post Test

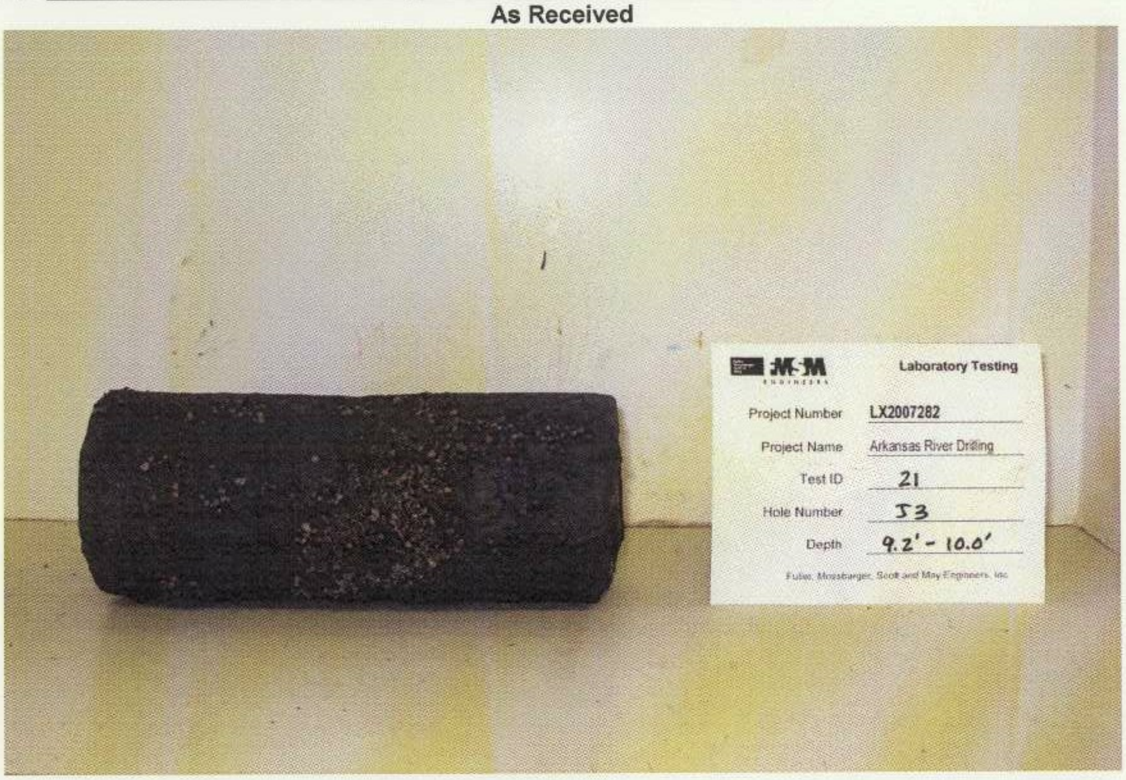
Fuller Mossbarger Scott & May
FMSM ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, very soft
Hole Number J3 Depth (ft) 9.2' - 10.0'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-21

As Received



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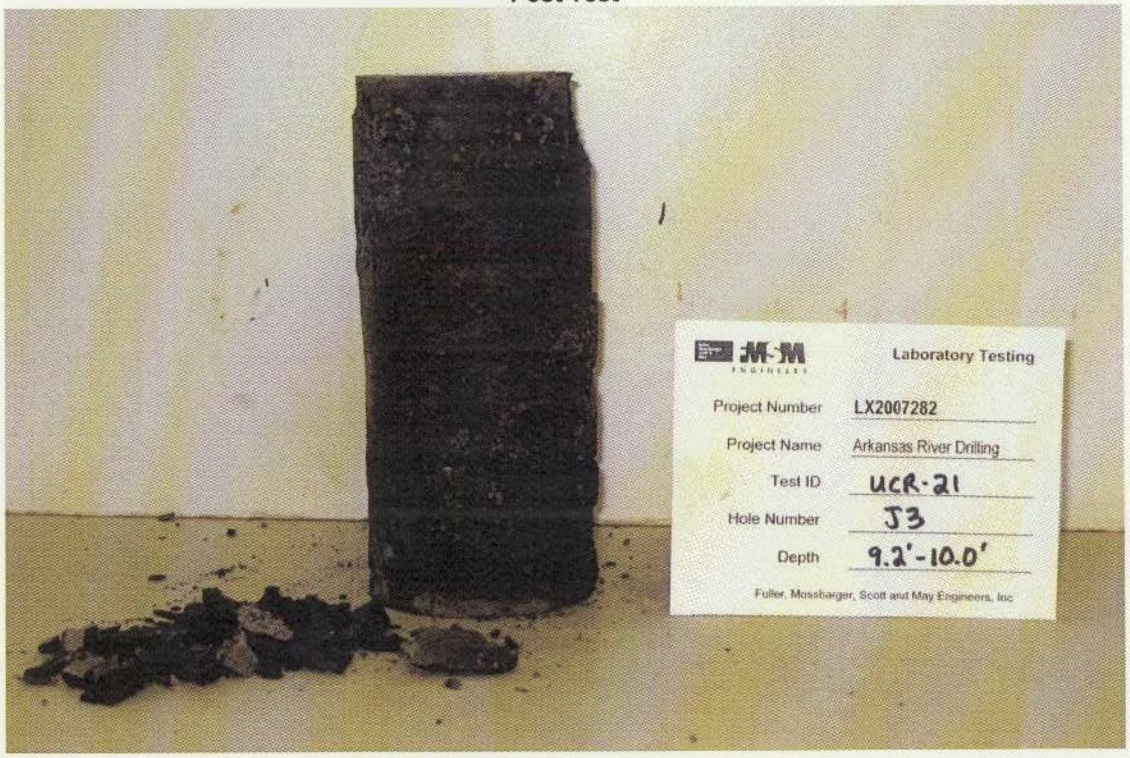


Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, very soft
Hole Number J3 Depth (ft) 9.2' - 10.0'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-21

Post Test

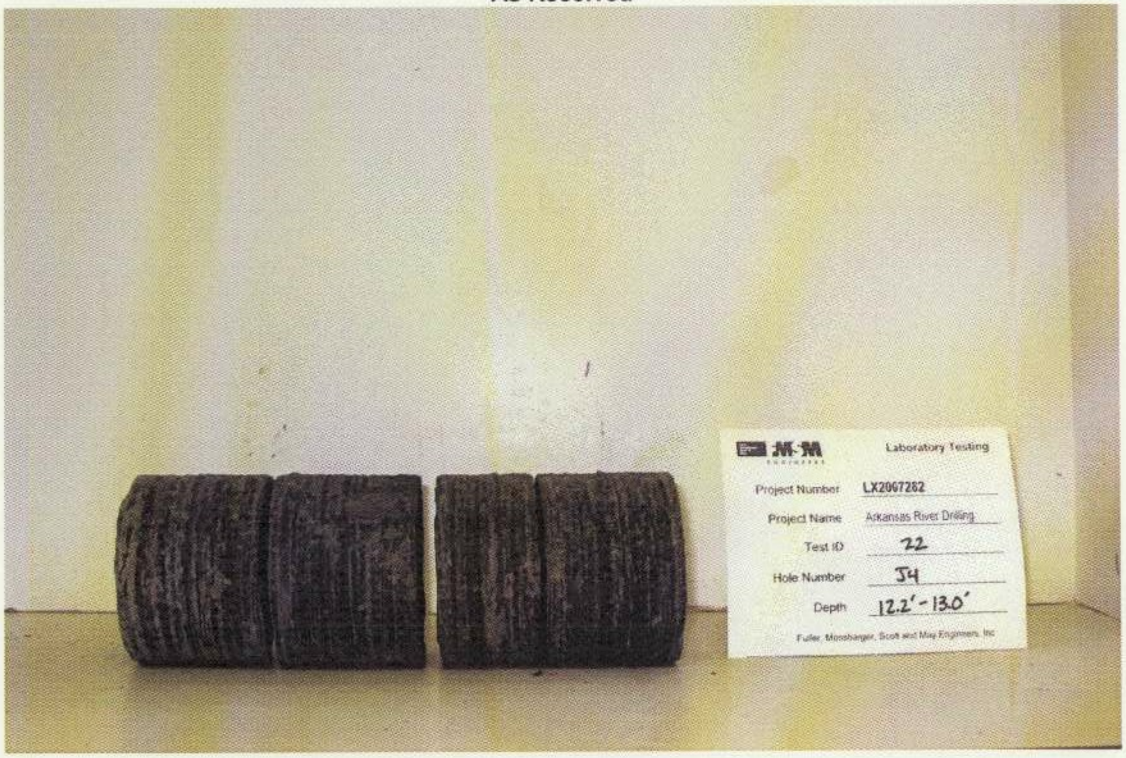
Fuller Mossbarger Scott & May
ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number J4 Depth (ft) 12.2' - 12.8'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-22

As Received



Core Preparation

Fuller Mossbarger Scott & May

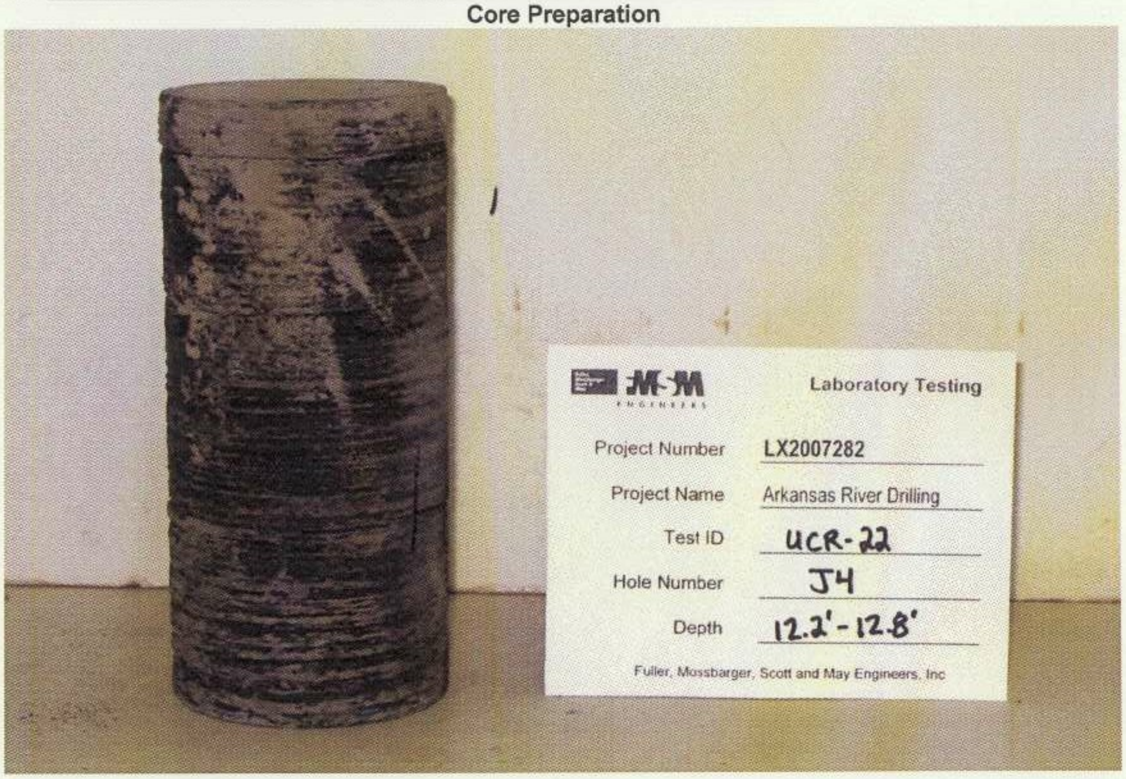
FMSM ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number J4 Depth (ft) 12.2' - 12.8'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-22

Core Preparation



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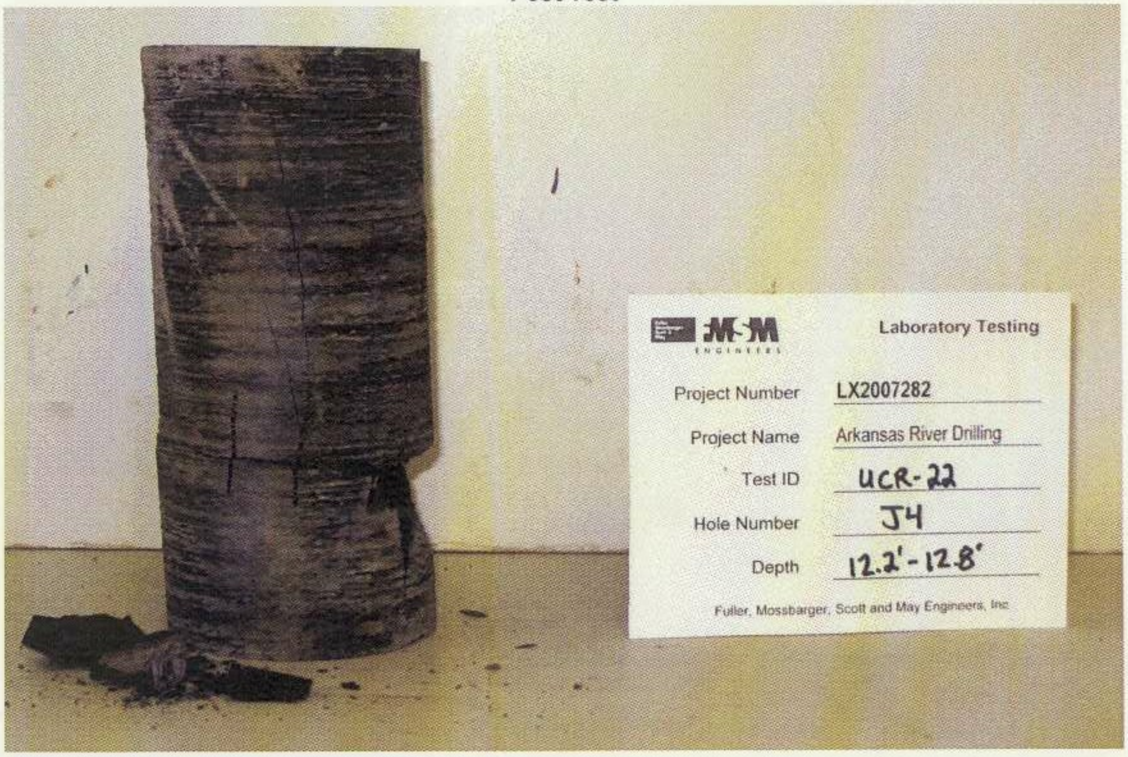

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number J4 Depth (ft) 12.2' - 12.8'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-22

Post Test

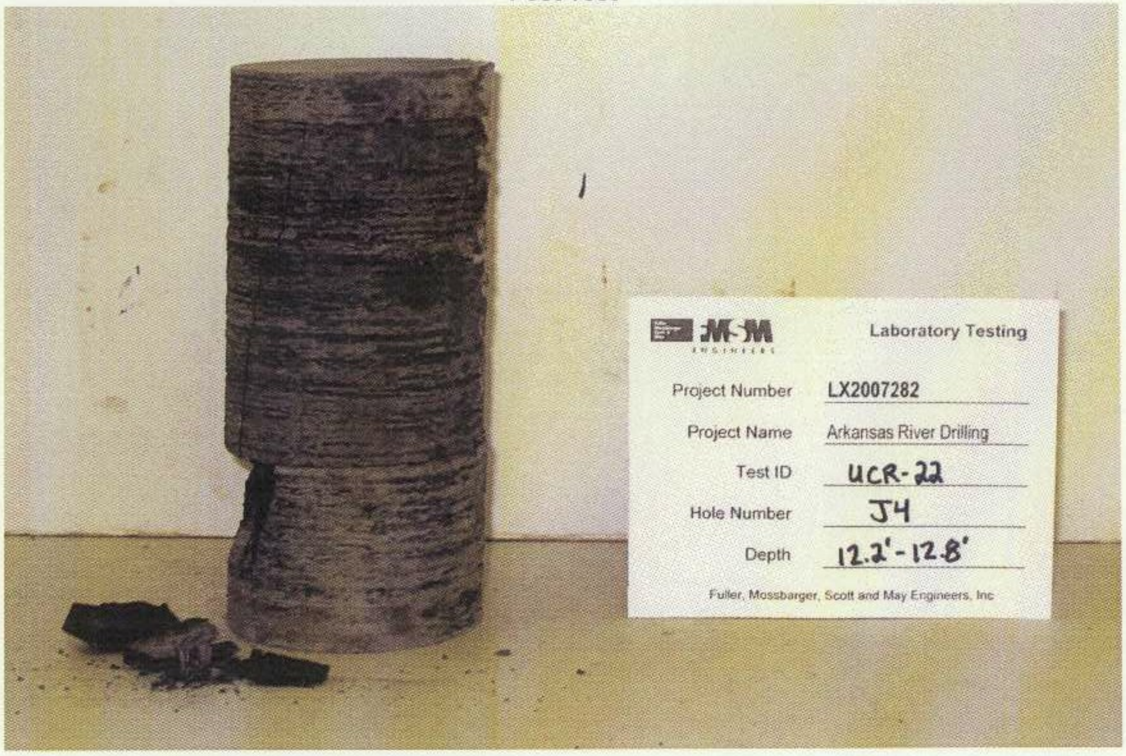

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number J4 Depth (ft) 29.00' - 29.65'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-23

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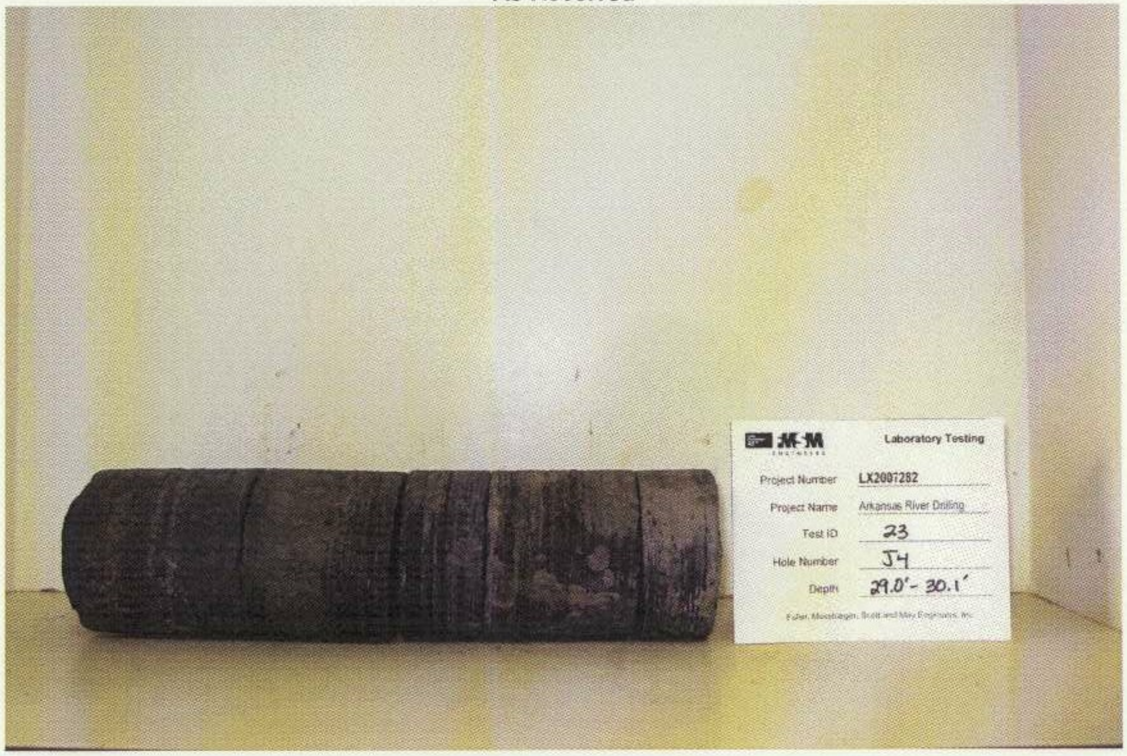

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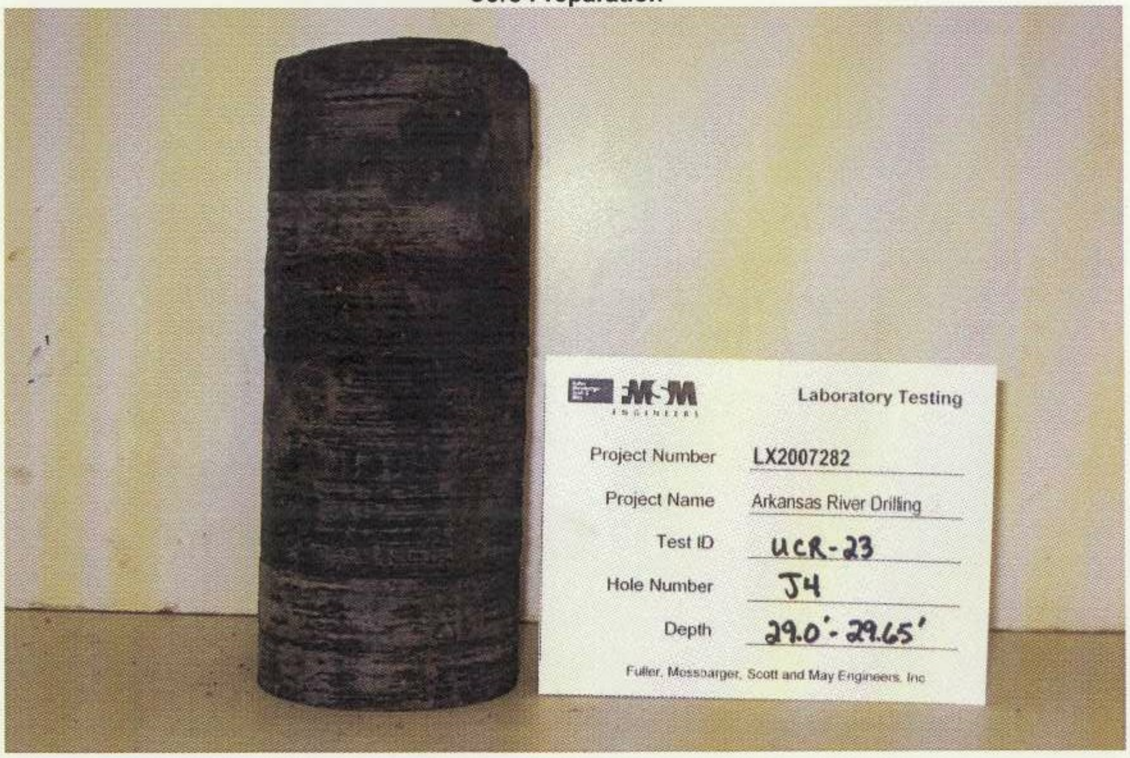


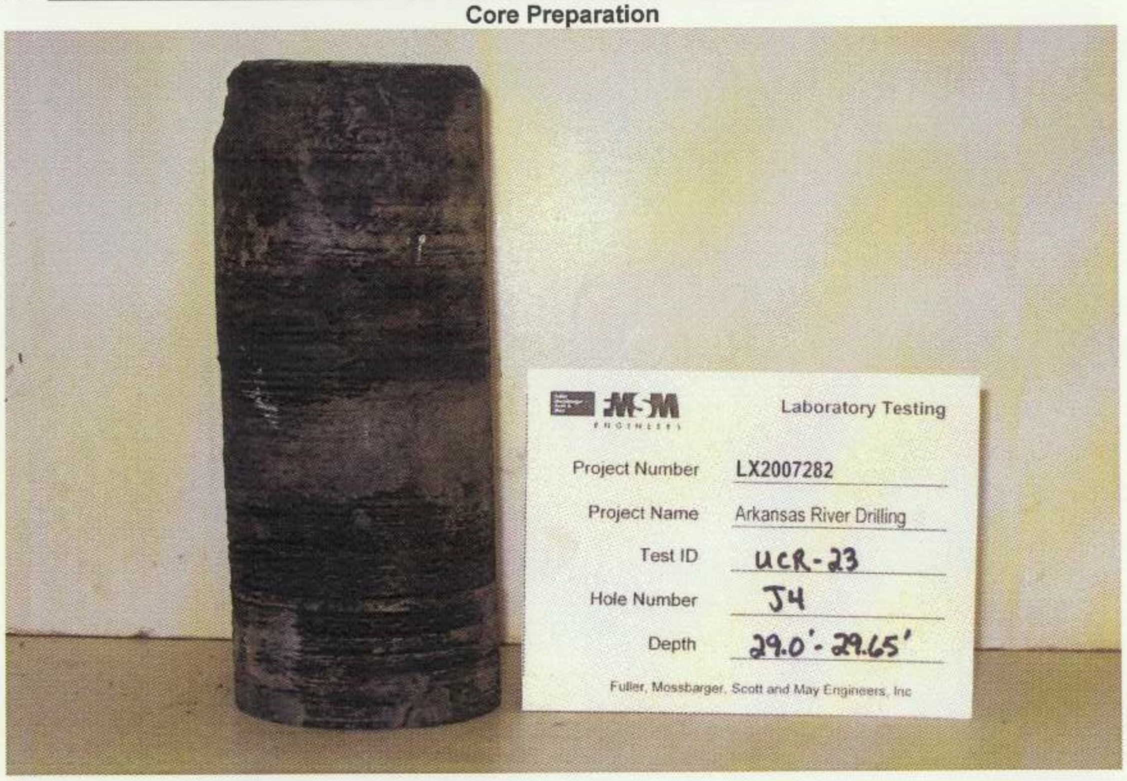
Fuller Mossbarger Scott & May
FMSM ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number J4 Depth (ft) 29.00' - 29.65'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-23

Core Preparation



Post Test

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-23
Hole Number	J4	Depth (ft)	29.00' - 29.65'
Test Type	Unconfined compressive strength		

Post Test

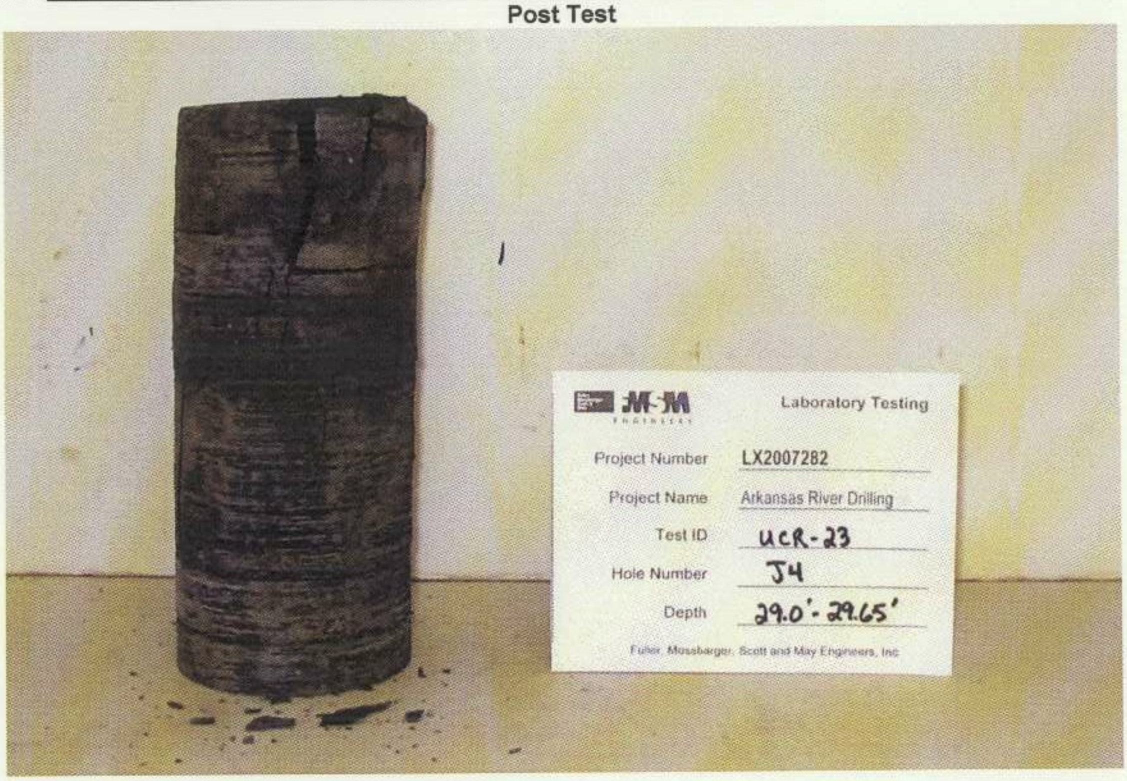


Photo Report

Project Name Arkansas River Drilling

Lithology Shale, dark gray, soft

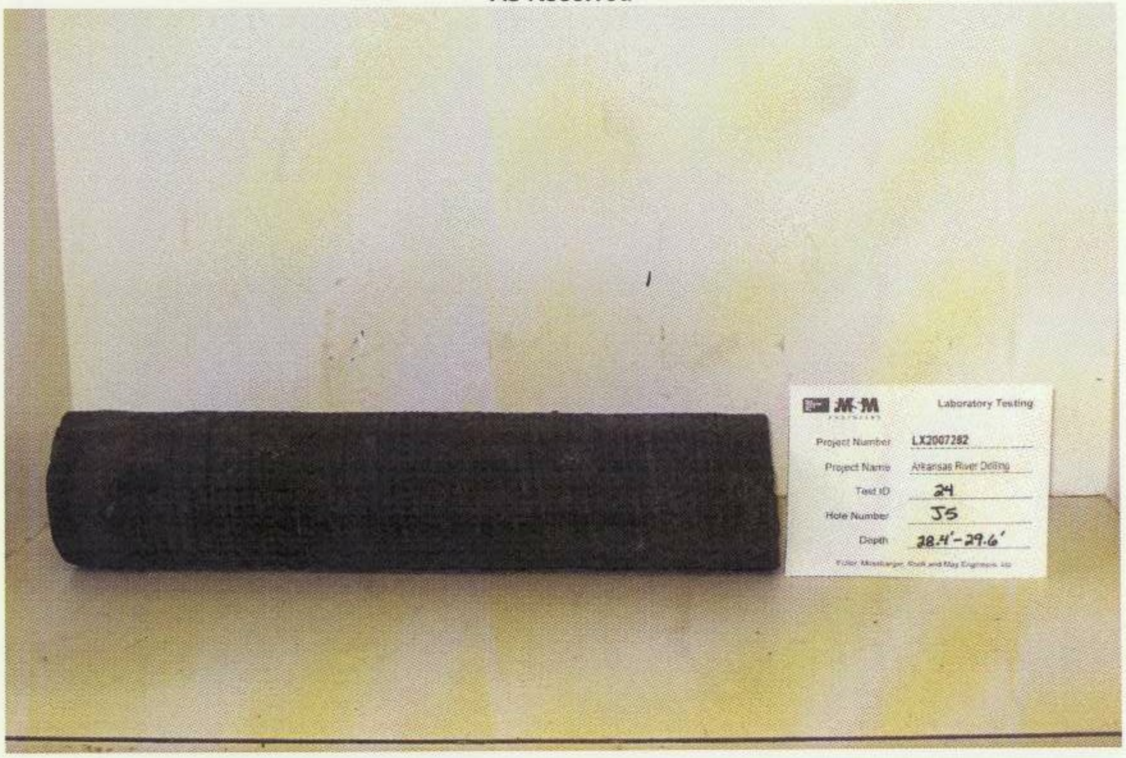
Hole Number J5 Depth (ft) 28.4' - 29.0'

Test Type Unconfined compressive strength

Project Number LX2007282

Lab ID UCR-24

As Received



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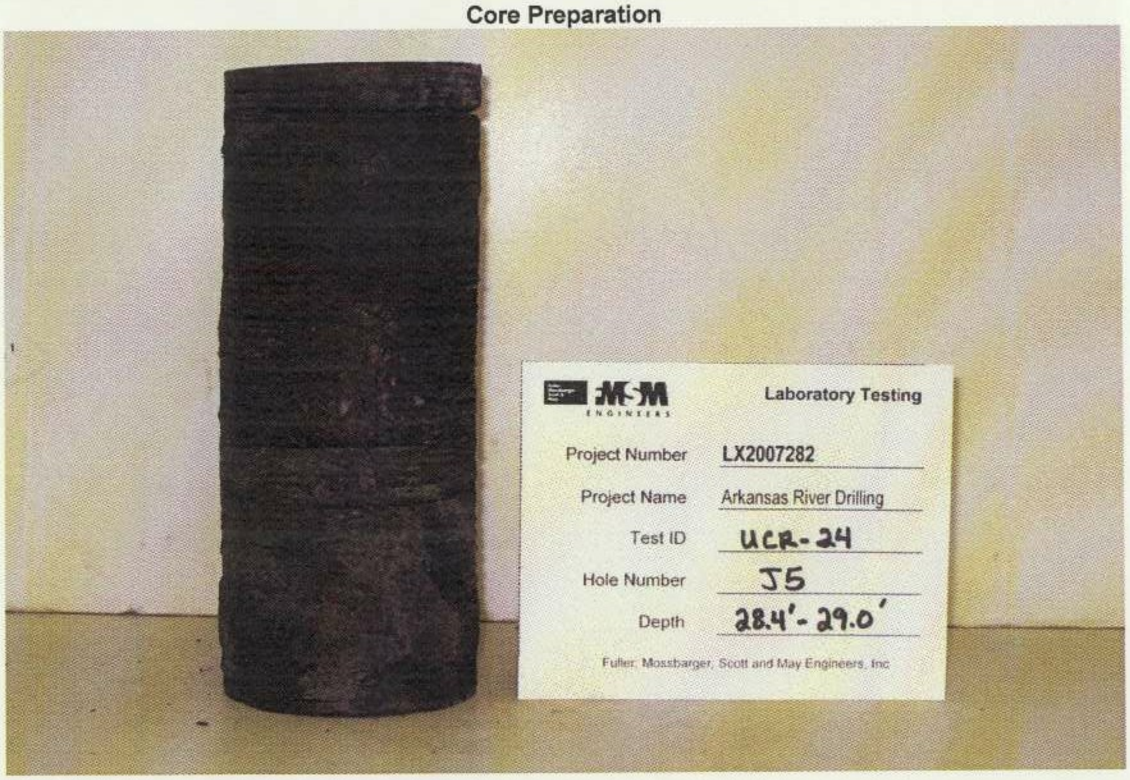
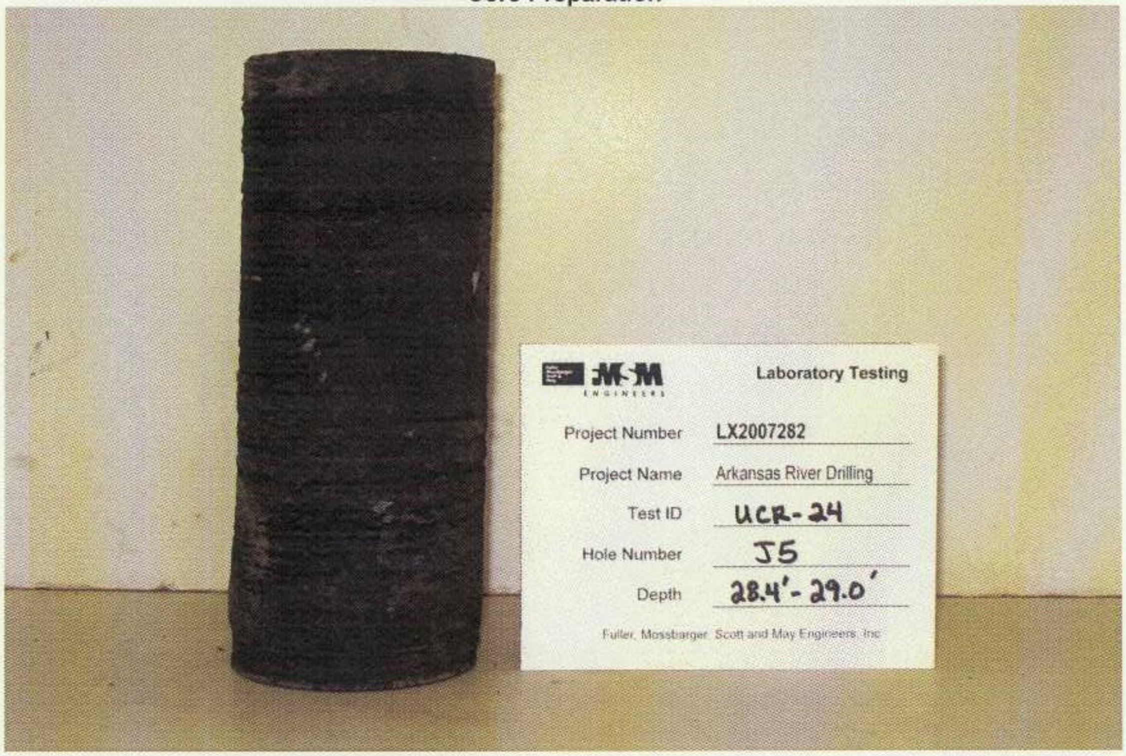


Fuller Mossbarger Scott & May
FMSM ENGINEERS

Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-24
Hole Number	J5	Depth (ft)	28.4' - 29.0'
Test Type	Unconfined compressive strength		

Core Preparation



Post Test

ENGINEERS

Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-24
Hole Number	J5	Depth (ft)	28.4' - 29.0'
Test Type	Unconfined compressive strength		

Post Test

Fuller Mossbarger Scott & May

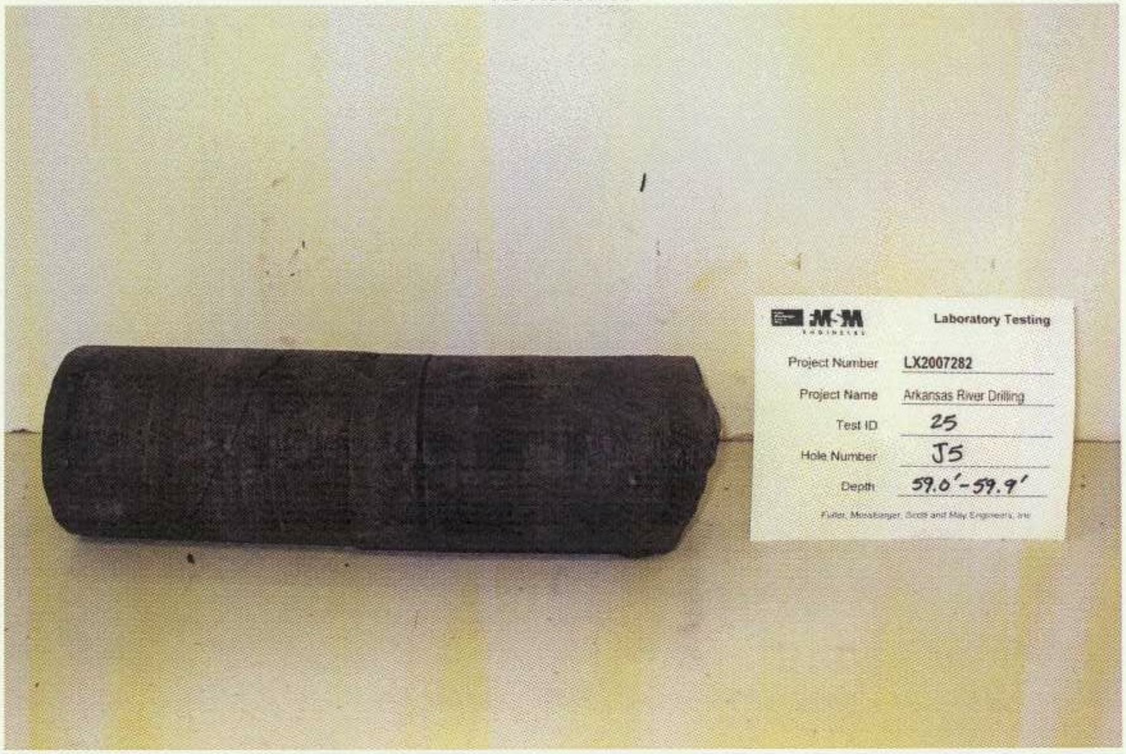
ENGINEERS

Photo Report

Project Name Arkansas River Drilling
Lithology Shale, dark gray, soft
Hole Number J5 Depth (ft) 59.00' - 59.65'
Test Type Unconfined compressive strength

Project Number LX2007282
Lab ID UCR-25

As Received



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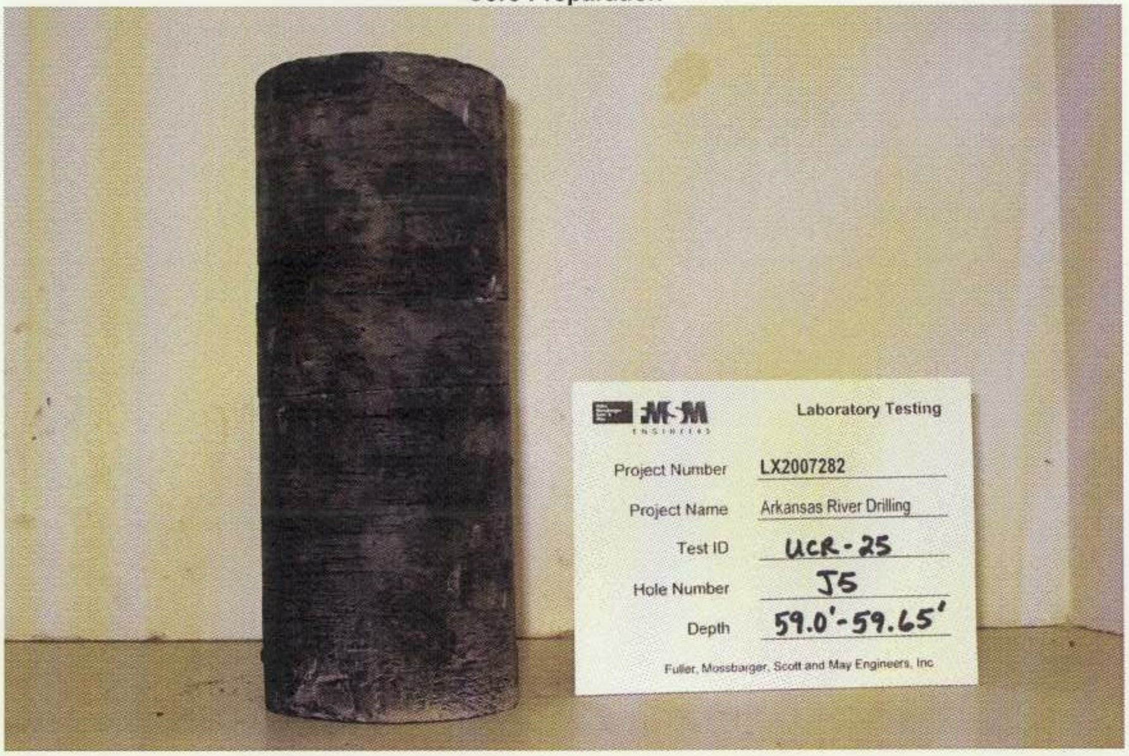

Fuller Mossbarger Scott & May ENGINEERS

Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-25
Hole Number	J5	Depth (ft)	59.00' - 59.65'
Test Type	Unconfined compressive strength		

Core Preparation

Post Test

Fuller Mossbarger Scott & May
FMSM ENGINEERS

Photo Report

Project Name	Arkansas River Drilling	Project Number	LX2007282
Lithology	Shale, dark gray, soft	Lab ID	UCR-25
Hole Number	J5	Depth (ft)	59.00' - 59.65'
Test Type	Unconfined compressive strength		

Post Test

Photographs of Rock Core

Boring ID

J-1

Figure 1 – 21.5-26.5 feet

Figure 2 – 26.5-31.5 feet

Figure 3 – 31.5-36.5 feet

Figure 4 – 36.5-41.5 feet

Figure 5 – 41.5-46.5 feet

Figure 6 – 46.5-51.5 feet

Figure 7 – 51.5-56.5 feet

Figure 8 – 56.5-60.0 feet

Boring ID

J-2

Figure 1 8.0-13.0 feet

Figure 2 13.0-18.0 feet

Figure 3 18.0-23.0 feet

Figure 4 23.0-28.0 feet

Figure 5 28.0-33.0 feet

Figure 6 33.0-38.0 feet

Figure 7 38.0-40.0 feet

Boring ID

J-3

Figure 1 7.5-10.5 feet

Boring ID

J-4

Figure 1 5.0-10.0 feet

Figure 2 10.0-14.0 feet

Figure 3 14.0-19.0 feet

Figure 4 19.0-24.0 feet

Figure 5 24.0-29.0 feet

Figure 6 29.0-34.0 feet

No photograph could be made of this interval due to inclement weather. It was deemed more important to preserve the samples.
Figure 7 34.0-40.0 feet

Boring ID

J-5

Figure 1 28.0-31.0 feet

Figure 2 31.0-36.0 feet

Figure 3 36.0-38.0 feet

Figure 4 38.0-41.0 feet

Figure 5 41.0-46.0 feet

Figure 6 46.0-51.0 feet

Figure 7 51.0-56.0 feet

Figure 8 56.0-61.0 feet

Figure 9 61.0-66.0 feet

Figure 10 66.0-71.0 feet

Figure 11 71.0-75.0 feet

Boring ID

S-1

Figure 1 -- 25-30 feet

Figure 2 – 30-35 feet

Figure 3 – 35-40 feet

Figure 4 – 40-45 feet

Figure 5 – 45-50 feet

Figure 6 – 50-55 feet

Figure 7 – 55-60 feet

Boring ID

S-2

Figure 1 – 10.5 - 15.5 feet

Figure 2 – 15.5 - 20.5 feet

Figure 3 – 20.5 - 25.5 feet

Figure 4 – 25.5 - 30.5 feet

Figure 5 – 30.5 - 35.5 feet

Figure 6 – 35.5 - 40.5 feet

Figure 7 – 40.5 - 45.5 feet

Figure 8 – 45.5 - 50.5 feet

Boring ID

S-4

Figure 1 – 35-40 feet

Figure 2 – 40-45 feet

Figure 3 – 45-50 feet

Figure 4 – 50-55 feet

Figure 5 – 55-60 feet

Figure 6 – 60-65 feet

Figure 7 – 65-70 feet

Figure 8 – 70-75 feet

CD with Electronic Files

Attachment 2

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SUBSURFACE EXPLORATION AND GEOTECHNICAL REPORT PROPOSED RIVER DISTRICT DEVELOPMENT RETAINING WALLS CREEK TURNPIKE AND LEWIS AVENUE JENKS, OKLAHOMA

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Precast Products

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November 21, 2008
Project No. 95463

KLEINFELDER
Bright People. Right Solutions.

Interchange Business Park
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Tulsa, OK
74116-5680

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f| 918.627.6262
kleinfelder.com

November 21, 2008

River District Development Group, LLC
c/o Tulsa Engineering and Planning Associates
Attn: Mr. Brent Cox, P.E.
6737 South 85th East Avenue
Tulsa, OK 74133

**Subject: Subsurface Exploration and Geotechnical Report
Proposed River District Development
Retaining Walls
Creek Turnpike and Lewis Avenue
Jenks, Oklahoma
Kleinfelder Project No.: 95463**

Dear Mr. Cox:

Kleinfelder has completed the authorized subsurface exploration and laboratory testing for the proposed Sea Wall to be constructed in conjunction with the River District Development in Jenks, Oklahoma. The purpose of the geotechnical study was to explore and evaluate the subsurface conditions at various locations on the site, and develop geotechnical design and construction recommendations for the proposed project. The attached Kleinfelder report contains a description of the findings of our field exploration and laboratory testing program, our engineering interpretation of the results with respect to the project characteristics, our geotechnical recommendations as well as construction guidelines for the planned project.

Recommendations provided herein are contingent on the provisions outlined in the ADDITIONAL SERVICES and LIMITATIONS sections of this report. The project Owner should become familiar with these provisions in order to assess further involvement by Kleinfelder and other potential impacts to the proposed project.

We appreciate the opportunity to be of service to you on this project and are prepared to provide the recommended additional services. Please call us if you have any questions concerning this report.

Sincerely,
KLEINFELDER CENTRAL, Inc.
Certificate of Authorization #3036, Expires 6/30/09

Brett Cowan, PhD. P.E.
Senior Project Engineer

Brian K. Marick, P.E.
Oklahoma: 21240

REGISTERED PROFESSIONAL ENGINEER
BRIAN K. MARICK
21240
11/24/08
OKLAHOMA

BC/BKM:hm
Attachments:

KLEINFELDER
Bright People. Right Solutions.

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FIGURES

APPENDICES

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SUBSURFACE EXPLORATION AND GEOTECHNICAL REPORT PROPOSED RIVER DISTRICT – RETAINING WALLS CREEK TURNPIKE AND LEWIS AVENUE JENKS, OKLAHOMA

1. INTRODUCTION

1.1 GENERAL

Kleinfelder has completed the authorized subsurface exploration and geotechnical engineering evaluation for the proposed River District Development - Sea Wall, located west of the Arkansas River, south of the intersection of the Creek Turnpike and South Lewis Avenue in Jenks, Oklahoma. These services were provided in general accordance with our proposal/contract (No.TUL8P221) dated June 13, 2008.

This report includes our recommendations related to the geotechnical aspects of the project design and construction. Conclusions and recommendations presented in the report are based on the subsurface information encountered at the location of our exploration and the provision and requirements outlined in the ADDITIONAL SERVICES and LIMITATIONS sections of this report. In addition, an article prepared by The Association of Engineering Firms Practicing in the Geosciences (ASFE), *Important Information about Your Geotechnical Engineering Report*, has been included in APPENDIX C. We recommend that all individuals read the report limitations along with the included ASFE document.

1.2 PROPOSED CONSTRUCTION

As we currently understand, the project will include construction of a series of retaining walls along the planned development on the west side of the Arkansas River. The retaining walls will be approximately 4,250 feet in length beginning just south of the Creek Turnpike and extending south along the west side of the Arkansas River. In general, the retaining wall(s) will be providing a maximum grade transition of 22 feet from the development to the Arkansas River floodplain.

A conceptual wall alignment (Figure 3) and a conceptual cross section (Figure 4) showing the wall alignment and existing grades has been provided by Tulsa Engineering and Planning. The plan indicates that three retaining wall tiers will be used to provide the grade transition and that existing grades along the proposed wall alignment range from approximately 615 to 593 feet. The lowest retaining wall tier is planned to be constructed using a proprietary T-WALL retaining wall system founded in the shale bedrock. The second and third tiers are anticipated to be cantilever concrete retaining walls supported in backfill of the lower retaining walls. Additional wall systems were also considered for the report with a discussion of their application. The following table presents the anticipated elevations at the top and toe of each retaining wall.

Table 1: Proposed Retaining Walls

Retaining Wall Tier	Top of Wall Elevation (feet)	Toe of Wall Elevation (feet)
1	600.0	593.0
2	604.0	≈600.0
3	615.5	607.0

The wall system chosen has two different stability analysis requirements (Internal and Global). Typically the wall designer only evaluates the internal stability analysis of the wall system. A global stability analysis will need to be performed after the wall system has been designed. Kleinfelder is capable of completing these services if requested.

The scope of the exploration and engineering evaluation for this study, as well as the conclusions and recommendations in this report, were based on our understanding of the project as described above. If pertinent details of the project have changed or otherwise differ from our descriptions, we must be notified and engaged to review the changes and modify our recommendations, if needed. *Recommendations with respect to pavements or structures other than the proposed retaining walls were beyond our authorized scope of work.*

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2. SITE CONDITIONS

2.1 SITE DESCRIPTION

The site is located south of the intersection of the Creek Turnpike and Lewis Avenue in Jenks, Oklahoma. The general location of the site is shown in Figure 1, Site Vicinity Map, included in APPENDIX A. The site is bounded by the Arkansas River to the east, the Creek Turnpike to the north, South Lewis Avenue to the west, and a wooded area to the south.

At the time of the subsurface exploration, the majority of the site was covered with grass or exposed sand. Trees were scattered along the western interior of the site and clusters of trees were present in the southern portion of the site. Along portions of the riverfront, rip-rap had been placed to control erosion. Standing pools of water are located along the river in the channels within the braided river bed. A grade differential of over 15 feet from the west to the east was estimated based upon the provided topographic plan. Existing utilities at the site included, but most likely are not limited to, overhead electric lines, overhead phone lines, and sewer lines. Additional utilities are likely to be present.

2.2 SUBSURFACE CONDITIONS

Kleinfelder explored the subsurface conditions at the site by drilling and sampling eight (8) borings on August 6 and 7, 2008. Approximate locations of the borings (labeled B-01 through B-08) are shown on Figure 2, Site Plan and Boring Locations, included in APPENDIX A. The field exploration and laboratory testing programs are presented in APPENDIX A and APPENDIX B of this report, respectively.

A 2 to 4 inch thick layer of topsoil was encountered at the ground surface in Borings B-01 and B-02. Boring B-08 encountered approximately 2 feet of rip-rap at the ground surface. In the remaining borings that were drilled on sand bars within the river, the surface materials have been revomed by erosion. The borings drilled on the sand bars encountered very loose poorly graded sand and silty sand to an approximate depth of 2 feet. Sand and silty sand were encountered beneath the topsoil, rip-rap and loose silty sand and continued to depths of 7.5 to 19 feet. The gradation and relative density of the upper 8 to 12 feet of the alluvial soils varied significantly along the retaining wall alignment. Clay was encountered below the sands in Borings B-03and B-07 between depths of 7.7 to 10.1 feet and 7.5 to 8.5 feet, respectively. The alluvial soil continued to approximate depths ranging from 5.5 to 19 feet (elevations 573.4 to 585.1 feet).

Shale bedrock was encountered below the alluvial soil and continued to the bottom of the borings at approximate depths ranging from 18.6 to 28.8 feet. The upper portion (about 1 foot) of the shale bedrock was weathered with the degree of weathering becoming less with depth. The dark gray shale bedrock encountered below the weathered portion was moderately hard to hard and continued to the bottom of our 18.6 to 28.8-foot borings.

2.3 GROUNDWATER OBSERVATIONS

Groundwater observations were made both during and after completion of drilling operations. The groundwater observations are indicated in the following table.

Table 2: Groundwater Observations

Boring No.	Groundwater Depth Measured During Drilling, (feet)	Groundwater Depth Measured Prior to Backfilling, (feet)
B-01	11.5	9.5
B-02	11.5	8.3
B-03	1.5	Dry (Caved)
B-04	3.0	Dry (Caved)
B-05	3.0	Dry (Caved)
B-06	0.5	Dry (Caved)
B-07	3.0	Dry (Caved)
B-08	2.0	2.0

The materials encountered in the test borings have a wide range of permeabilities and observations over an extended period of time through use of piezometers or cased borings would be required to better define current groundwater conditions. Fluctuations of groundwater levels can occur due to seasonal variations in the amount of rainfall, runoff, and other factors not evident at the time the borings were performed. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

The groundwater at the project site is hydraulically connected to the Arkansas River. The depth and quantity of water will generally be controlled by the water level in the Arkansas River. The potential fluctuations of the water levels in the river should be taken into consideration when developing design and construction plans as well as during constructions.

3. CONCLUSIONS AND RECOMMENDATIONS

3.1 GENERAL

The scope of services included provided recommendations for the proposed conceptual cross section of retaining walls as presented in Figure 3, and to discuss alternative retaining walls systems that could be utilized at the project site. Based on the results of our evaluation, it is our professional opinion that the proposed project site can be developed for the proposed retaining walls using conventional excavation and construction techniques. The primary geotechnical concerns for this project are the erodibility of the sand layer protecting the toe and toe protection required to reduce potential scour. Recommendations addressing the primary geotechnical concerns as well as general recommendations regarding geotechnical aspects of the project design and construction are presented below.

The recommendations submitted herein are based, in part, upon data obtained from our subsurface exploration and are for the conceptual retaining wall cross section provided by TEP. The alternative retaining wall systems discussed in this report may require other approaches for site development. If an alternative retaining wall system is to be utilized at the project site, Klienfelder should be provided the opportunity to review the recommendations presented in this report to determine if modifications to the recommendations would be warranted. The nature and extent of subsurface variations that may exist at the proposed project site will not become evident until construction. If variations appear evident, then the recommendations presented in this report should be evaluated. In the event that any changes in the nature, design, or location of the proposed project are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and our recommendations modified in writing.

3.2 ALTERNATIVE ETAINING WALL SYSTEMS

3.2.1 General

Based upon review of the subsurface conditions and proposed construction activities, several different retention systems would be suitable for site development. The retention systems that appear feasible include: 1) T-Wall System, 2) sheet piling, 3) segmented block retaining walls, 4) cast-in-place concrete, and 4) mechanically stabilized earth (MSE). Each method is unique and has different advantages and disadvantages, which are summarized below. We have provided the advantages and disadvantages for the different wall systems to aid in the planning and budget estimating for the project. The influence of the construction procedures should be taken in account in the final plans and specifications.

3.2.2 T-Wall Retaining Wall System – Proprietary System

This wall system is able to use the backfills unit weight to anchor the structure against global movement. This system is a Mechanically Stabilized Earth (MSE) Wall System, that utilizes the backfill material's unit weight and the friction coefficient between the backfill material and the precast concrete panels for the internal stability of the system. The system appears to be suitable for use at the project site, but the cost effectiveness of the system along with the architectural features will need to be reviewed by the designer.

Advantages:

- Excavation can be performed with conventional excavation equipment.
- No specialty contractor required.
- T-Wall panels are a manufactured product.
- These systems are common in the commercial industry and many aesthetic options are available to the owner.

Disadvantages:

- Excavation and removal of existing in-place material beyond the limits of the proposed T-Wall footprint area will be required.
- The retaining wall system will need to be founded on bedrock (minimum 2 feet embedment and a concrete footing.
- Deep excavations 7.5 to 19.5 feet below existing grades
- Bottom-up construction.
- Dewatering of the wall footprint area will be required.
- Proprietary System.

3.2.3 Sheet Piling

A sheet pile system appears to be suitable for the upper two tiers of retaining walls. The system is a cantilever system that requires a substantial amount of the sheet pile to be below grade to resist overturning forces. It may not feasible to use sheet piling for the lowest tier of the retaining wall due to the relative shallow depth to bedrock.

Advantages:

- Rough final grades can be established prior to installation of the sheet piling.
- No excavation is required with the exception of final grading.
- No specialty contractor required.
- Dewatering is not required for the lowest retaining wall.
- System can be designed as either a rigid or flexible system.

Disadvantages:

- Limited aesthetic options. Could consider connecting aesthetic panels to face of walls.
- Due to the shallow depth to bedrock for the lowest tier, this system may not be feasible without a secondary method of support. These secondary methods are discussed below.
- System would require scour protection for the lowest tier.

3.2.4 Sheet Pile with MSE Wall System

This system utilizes sheet piling in conjunction with uniaxial grids connected to the back of the sheet piles. This system appears to be suitable at the project site. The uniaxial grids connect to the back of the sheet piling would reduce the required length of penetration required to resist overturning forces. By reducing the overturning forces on the sheet piling, it may feasible to use sheet piling for the lowest tier of the retaining wall even with a relatively shallow depth to bedrock. Alternatively, some type of dead-man anchor could be considered in lieu of the reinforcing grids.

Advantages:

- No specialty contractor required.
- Dewatering is not required for the lowest retaining wall.
- System can be designed as either a rigid or flexible system.

Disadvantages:

- Limited aesthetic options. Could consider connecting aesthetic panels to face of walls.
- This system would require that rough grades be established below the lowest grid elevation.
- Installation of grids and compaction of backfill material would be required to establish final grades.
- System would require scour protection for the lowest tier.

3.2.5 Sheet Piles with Piles

Due to the relative shallow depth to bedrock, the lowest tier of the retaining wall could be constructed with H-pile or pipe piles reinforced sheet piles. This system utilizes sheet piling in conjunction with H-piles spaced periodically along the alignment. The H-piles or pipe piles stiffen the entire system, thus reducing the required embedment length of the sheet piling needed to resist overturning forces.

Advantages:

- Rough final grades can be established prior to installation of the sheet piling.
- No excavation is required with the exception of final grading.
- No specialty contractor required.
- Dewatering is not required for the lowest retaining wall.
- System can be designed as either a rigid or flexible system.

Disadvantages:

- Limited aesthetic options. Could consider connecting aesthetic panels to face of walls.
- Requires both sheet piling and H-piling or pipe pile installation equipment.
- May require pre-drilling at the H-piling or pipe pile locations to achieve required embedment depths.
- System would require scour protection for the lowest tier.

3.2.6 Sheet Piles with Piers

Alternatively to H-pile or pipe pile reinforced sheet piling, consideration could be given to utilizing drill piers to reinforce the sheet piling for the lowest tier of the retaining walls. This system utilizes sheet piling in conjunction with drilled piers spaced periodically along the alignment. The drilled piers stiffen the system, thus reducing the required embedment length of the sheet piling needed to resist overturning forces.

Advantages:

- Rough final grades can be established prior to installation of the sheet piling.
- No excavation is required with the exception of final grading.
- No specialty contractor required.
- Dewatering is not required for the lowest retaining wall.
- System can be designed as either a rigid or flexible system.

Disadvantages:

- Limited aesthetic options. Could consider connecting aesthetic panels to face of walls.
- Requires both sheet piling and drilled pier excavation equipment.
- Drilled piers will likely require temporary casing during installation, or slurry-method excavation.
- System would require scour protection for the lowest tier.

3.2.7 Cast-In-Place Wall Systems

Consideration could be given to utilizing cast-in-place retaining walls at the project site. The project site is suitable for the use of this type of retaining walls.

Advantages:

- Excavation can be performed with conventional excavation equipment.
- No specialty contractor required.

Disadvantages:

- Excavation will require the excavation and removal of existing in-place material beyond the limits of the proposed wall footprint area.
- Bottom-up construction.
- Dewatering of the lowest tier wall footprint area will be required.

3.2.8 MSE Wall System

The project site is suitable for use of MSE type retaining walls. The system consists of reinforced soil to act as a gravity type retaining wall. This wall system is able to use the backfills unit weight to anchor the structure against global movement.

Advantages:

- Excavation can be performed with conventional excavation equipment.
- No specialty contractor required.

Disadvantages:

- Excavation will require the excavation and removal of existing in-place material beyond the limits of the proposed wall footprint area.
- Bottom-up construction.
- Dewatering of the wall footprint area will be required.

3.3 PRIMARY GEOTECHNICAL CONCERNS

3.3.1 Erodibility of Sand

The sand varies in gradation, particle size, and relative density along the soil profile, however, the design of the Sea Wall will need to consider the highly erosive nature of non-cohesive sands. A basic understanding of the potential flows and the interaction of the wall and the river will allow the designer to either ignore the passive resistance gained by the river sediments on the east side of the structure or design for toe protection. In addition to the toe protection, the sandy soils will need to be protected from sheet flow.

3.3.2 Toe Protection

The toe has the potential to be undermined during high flows along the Arkansas River. The designer could place a graded rip rap along the toe of the sea wall to help reduce the potential of scour. A self-launching of 36-inch stone would provide added passive resistance for global stability but would also protect against the scour during the higher flows over the lifetime of the project. Another option is to use an articulating block wall system to protect the toe.

3.4 SITE DEVELOPMENT

3.4.1 Stripping and Grubbing

Initial site preparation should also include removal of debris associated with past site activity as well as stripping of all vegetation and topsoil from the construction areas. Stripping depths required will likely vary and should be adjusted to remove all vegetation and root systems. A Kleinfelder representative should monitor the stripping operations to observe that all unsuitable materials have been removed. Soils removed during site stripping operations could be used for final site grading outside the proposed wall alignment. Care should be exercised to separate these materials to avoid incorporation of the organic matter or cohesive soils in structural fill sections within the walls zone of influence.

Any required tree removal should also be accomplished at this time. Care should be taken to thoroughly remove all root systems from the construction areas. Materials disturbed during removal of stumps should be undercut and replaced with structural fill.

3.4.2 Scarification, Moisture Conditioning and Compaction

Prior to placement of structural fill, the moisture content of the exposed materials should be evaluated. Depending on the in-situ moisture content of the exposed materials, moisture conditioning of the exposed grade may be required prior to proofrolling and/or fill placement. The moisture content of the exposed materials in these fill areas should be adjusted to within the range recommended for structural fill, to allow the exposed material to be compacted to a minimum of 98 percent of the standard Proctor density. Extremely wet or unstable areas that hamper compaction of the subgrade may require undercutting and replacement with structural fill or other stabilization techniques. Suitable structural fill should be placed to design grade as soon as practical after reworking the subgrade to avoid moisture changes in the underlying soils. Any material used for backfill that is cohesive will need to be discussed with a Kleinfelder representative to review the changes and modify our recommendations, if needed.

It should be noted that low relative density soils were encountered at the project site. Undercutting of these materials may be required to establish a subgrade which is suitable to compaction of structural fill material. The depth of undercutting where unstable subgrade conditions or unsuitable materials are encountered should be anticipated to vary across the project site and will need to be adjusted at the time of construction. Suitable soils removed during the undercutting operations can be stockpiled for use as structural fill.

3.4.3 Proofrolling

Following any required undercutting and moisture conditioning, and prior to placement of structural fill, it is recommended that the exposed soil grade be proofrolled, where accessible. Proofrolling of the subgrade aids in identifying soft or disturbed areas. Unsuitable areas identified by the proofrolling operation should be undercut and replaced with structural fill. Proofrolling can be accomplished through use of a fully-loaded, tandem-axle dump truck or similar equipment providing an equivalent subgrade loading.

3.4.4 Construction Considerations

Sandy soils were encountered at the project site. These materials, in particular if they contain silt, will likely to become unstable with minor changes in moisture contents and/or repeated construction traffic. Close moisture control during compaction operations will be required to reduce the possibility that the soils will pump or become unstable. Stabilization of these soils may be required depending upon the conditions encountered at the site at the time of construction. Stabilization options are provided in Section 3.7.4 of this report.

3.4.5 Areas of Standing Water Within River Channel

In areas where standing water is located within the river channel, it is acceptable to place sand fill in bulk to raise the grade to a maximum level of 2 feet above the current water level within the low lying areas. The fill can be placed by using a dozer to push material into the low lying areas. Prior to filling the low areas, all reasonable efforts should be made to remove any deleterious materials present on the sand bars or below the water level in the low areas. Once the grade is raised to a maximum height of two feet above the standing water elevation, the fill material should be compacted with a smooth drum vibratory compactor to a minimum of 90 percent of the Standard Proctor value of the fill material. All fill placed following the initial fill to raise the low lying areas that are currently underwater, and all fill on dry land should be placed to the requirements of structural fill as outlined in Section 3.7 of this report. This recommendation is based upon our understanding that the once final grades are achieved, the areas located along the proposed sea wall alignment will be surcharged.

3.5 CLIMATIC CONDITIONS

Weather conditions will influence the site preparation required. In spring and late fall, following periods of rainfall, the moisture content of the near surface soils may be significantly above the optimum moisture content. These conditions could seriously impede grading by causing unstable subgrade conditions. Typical remedial measures include aerating the wet subgrade, removal of the wet materials and replacing them with dry materials, or treating the material with Class "C" fly ash, cement kiln dust, or Portland cement.

If site grading commences during summer months, moisture contents may be low. Typically, discing and moisture conditioning of the exposed subgrade materials to the moisture content criteria outlined in the STRUCTURAL FILL section will allow the materials to be properly compacted. As an alternative, the dry materials could be undercut and replaced with structural fill.

3.6 TEMPORARY EXCAVATIONS

3.6.1 Excavations

It is anticipated that excavations for the proposed structures and utilities will be in native soil, controlled structural fill and the shale bedrock. Excavation of the native soils and controlled structural fill should be possible with appropriately sized conventional equipment such as backhoes, loaders, etc. Typical temporary dewatering techniques are anticipated to be sufficient to remove any water seepage that may be encountered in shallow excavations above the water table. Excavations extending deeper into the bedrock or below the river level will likely require more sophisticated dewatering methods/equipment. The excavation contractor should review the boring logs and other available information to determine the project dewatering requirements.

Excavation of the shale bedrock may be required to achieve the design grades for support of the lower T-wall system in portions of the site and during utility installation. Highly weathered to weathered shale with a Standard Penetration Resistance Value of less than 25 blows per foot can generally be excavated with conventional soil equipment such as scrapers, loaders, etc. Excavation of harder shale will most likely be difficult and may require the use of single-tooth rippers mounted on large tractors such as a Caterpillar D-8 or larger, pneumatic breakers or other rock excavating techniques to complete the excavations. Excavation of the competent bedrock in confined excavations is anticipated to be more difficult.

3.6.2 Slopes

Excavations should be cut to a stable slope or be temporarily braced, depending on the excavation depths and the subsurface conditions encountered. ***Temporary construction slopes should be designed in strict compliance with the most recent governing regulations.*** The contractor should also be aware that slope height, slope inclination or excavation depths (including utility trench excavations) should in no case exceed those specified in local, state and/or federal safety regulations, such as OSHA Health and Safety Standard for Excavations, 29 CFR Part 1926, or successor regulations.

Construction slopes should be closely observed for signs of mass movement: tension cracks at the crest, bulging at the toe, etc. If potential stability problems are observed, a geotechnical engineer should be contacted immediately. ***The responsibility for excavation safety and stability of temporary construction slopes lie solely with the contractor.*** Shoring or bracing may be required to provide structural stability and to protect personnel working within the excavation.

3.6.3 Construction Considerations

Stockpiles should be placed well away from the edge of the excavation and their height should be controlled so they do not surcharge the sides of the excavation. Surface drainage should be carefully controlled to prevent flow of water into the excavations.

The soils encountered at the site were predominantly sand soils. The sand soils are highly susceptible to erosion. Excavations in materials of this type that are left open for even short durations may experience some form of failure such as sloughing of the sides of the excavation. Measures should be taken to stabilize the sloping face of the excavations. Such measures may include interception and diversion of surface water, placing a fabric over the material and/or bracing the sides of the excavation. If side slopes are not stabilized, reworking of the excavation should be anticipated.

3.7 CONSTRUCTION DEWATERING

Groundwater was encountered at approximate elevations ranging from 1.5 to 11.5 feet in the borings. Based on the current plan, we understand that the T-wall system used for the lower tier will extend to the underlying shale encountered at depths of 7.5 to 19.5 feet below existing grades. Thus, we anticipate that a dewatering system comprised of wellpoints will be necessary to keep the water table below the bottom of the planned excavation. Groundwater should be maintained a minimum of 2 feet below the excavation bottom throughout construction to maintain bottom stability. Attempts to dewater the excavation by pumping directly from the excavation should not be performed due to the potential of water flowing from the soils which could erode the soils and potentially create voids behind any retention systems and/or destabilize slopes.

Long term monitoring of groundwater conditions was not performed by Kleinfelder and variations in groundwater levels may be observed during construction. Kleinfelder recommends that long term groundwater monitoring be performed by Kleinfelder or other qualified personnel to determine fluctuations of groundwater levels.

3.8 STRUCTURAL FILL

3.8.1 Materials

It is our understanding that the project anticipates mining sand form the Arkansas River, and utilizing the mined sand to construction the embankment where the retaining walls are to be located and as retaining wall back fill material. Based upon our understanding, all structural fill and backfill required to achieve design grades should consist of SP or SW material, free of organic matter and debris. The structural fill should consist of a non plastic soil (i.e., sands), as determined by the Atterberg limits test ASTM D 4318, wet preparation procedure. Kleinfelder should be consulted prior to use of clay type materials. Approval for use of other materials will be given on a case by case basis depending on the material and planned location of placement.

3.8.2 Existing Soils

Based on the conditions encountered in the borings, it appears that the majority of the soils encountered at the site would be suitable for use as non plastic structural fill material within the zone of influence of the sea wall. However, additional testing at the time of construction is recommended to further evaluate the use of these soils as non plastic structural fill within the zone of influence of the wall system.

3.8.3 Compaction Criteria

Fill should be placed in lifts having a maximum loose lift thickness of 9 inches. All fill should be compacted to a minimum of 98 percent of the material's maximum dry density as determined by ASTM D 698 (standard Proctor compaction). The moisture content of the fill at time of compaction should be within a range of 2 percent below to 2 percent above optimum moisture content as defined by the standard Proctor compaction procedure. Moisture contents should be maintained within this range until completion of the wall system.

3.8.4 Chemical Stabilization/Modification

Depending upon site conditions at the time of construction and the construction schedule, unstable areas may require chemical stabilization. Unstable areas could be stabilized with Portland Cement, Cement Kiln Dust, or Class "C" fly ash. The producer of the proposed stabilizing/modifying agent should submit chemical analysis sheets to Kleinfelder for review and approval prior to beginning construction.

If Portland cement is used as the stabilizing agent, a Portland Cement content of 3 to 5 percent on a dry weight basis is generally sufficient to achieve the desired stable subgrade, less subject to disturbance during construction. Laboratory tests, including a standard Proctor test on a representative soil sample mixed with the proposed Portland Cement, will be necessary to determine the actual amount required and to determine the moisture content to achieve maximum potential strength. Laboratory tests should be completed with the specific Portland Cement source that will be used for construction. The Portland Cement should be placed, mixed, and compacted in general accordance with ODOT "Standard Specifications for Highway Construction, Section 312".

If Cement Kiln Dust (CKD) is used as the stabilizing agent, a CKD content of 8 to 10 percent on a dry weight basis is generally sufficient to achieve the desired stabilization. Laboratory tests will be necessary to determine the actual amount required. The CKD should be placed, mixed, and compacted in general accordance with ODOT "Standard Specifications for Highway Construction, Section 317" (1999).

If Class "C" fly ash is used as the stabilizing agent, a Class "C" fly ash content of 14 to 17 percent on a dry weight basis is generally sufficient to achieve the desired stabilization. Laboratory tests will be necessary to determine the actual amount required and to determine the moisture content to achieve maximum potential strength. Class "C" fly ash stabilization of the subgrade will provide a more stable subgrade, less subject to disturbance during construction. The Class "C" fly ash should be placed, mixed, and compacted in accordance with ODOT "Standard Specifications for Highway Construction, Section 317" (1999).

3.8.5 Organic Soils

The more highly organic soils removed during site preparation could be utilized during final grading in landscaped areas of the site. Depth of organic fill and degree of compaction should be established to provide a stable surface that will be conducive to growth of grass cover.

3.9 RETAINING WALL DESIGN PARAMETERS

3.9.1 General

The recommendations submitted herein are based, in part, upon data obtained from our subsurface exploration. The nature and extent of subsurface variations that may exist at the proposed project site will not become evident until construction. If variations appear evident, then the recommendations presented in this report should be evaluated. In the event that any changes in the nature, design, or location of the proposed project are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and our recommendations modified in writing.

Segmented blocks, cast in-place concrete, sheet piling, among other systems are being considered. Once the final retaining wall type has been determined, Kleinfelder should be provided this information to determine if additional recommendations, or modifications to the recommendations presented in this report would be warranted.

3.9.2 Segmented Retaining Walls Design

We understand the consideration is being given to constructing the upper two tiers of the retaining walls as a modular block faced - grid reinforced backfill system. These walls are typically subcontracted as design-build structures, since design details are often manufacturer specific. Established design methods for modular block walls address facing, internal, and external stability, but do not specifically address the global stability of the wall system. Therefore, we recommend the following general and specific considerations be included in the project specifications for the wall design.

Facing, internal, and external stability analyses should conform to the latest design methodology accepted for use by the National Concrete Masonry Association (NCMA). Global stability of the wall system should be analyzed using both drained and undrained strength parameters to evaluate long term (drained) and end-of-construction (undrained) conditions. Parameter values used in the analysis should not exceed those values given in the table presented in Section 3.8.5 for the materials at the project site. The wall contractor should be required to provide these analyses based on the planned final cross section, including the topography above and below the wall, using the generalized subsurface stratigraphy discussed in this report. Kleinfelder should be provided the opportunity to review and comment on the wall system design and analysis.

The wall designer should perform both internal stability analysis of the modular block faced - grid reinforced backfill system, as well as the **global stability** of any cuts, and global stability analysis of the modular block faced - grid reinforced backfill system and subsurface conditions. Global stability analysis is beyond Kleinfelder's scope of work and should be performed by the wall designer.

It has been estimated that the foundation soils within the footprint of the walls could experience 1 inch of settlement depending upon the location in reference to the proposed walls. A thorough settlement analysis should be performed by the wall designer following design of the SRW's. Specific recommendations to limit settlement of SRW's, are dependent upon the type of system and limiting factors that are determined by the wall designer, are beyond Kleinfelder's scope of work and should be addressed by the wall designer.

3.9.3 Cast In-Place Concrete Retaining Walls

Walls should be designed to resist lateral earth pressures equivalent to those induced by the surcharge of adjacent structures and the appropriate soil material. In most cases, lateral earth pressure can be assumed to increase linearly with depth and may be represented as the effective unit weight of the soil times the appropriate coefficient of lateral earth pressure times the thickness of the overlying soil. The design parameter values in the table presented in Section 3.8.5 may be used for determining lateral earth pressures at this site.

Where foundations are earth formed, the allowable passive earth pressure acting on the vertical edge of the base of the footing may be calculated using the values presented in the previous table. An ultimate coefficient of friction of 0.35 could be used for foundations placed directly on the sand that has been placed as structural fill that meet the criteria presented in Section 3.7. Passive earth pressure should be ignored within 2 feet of finished grade for design, due to possible disturbance of the adjacent materials during construction activities.

A factor of safety of at least 1.5 should be used with stability calculations involving lateral earth pressures. The safety factor should be computed as the sum of resisting forces or moments divided by the sum of driving forces or moments.

To prevent hydrostatic loading on the walls, it is recommended that a perforated drain line be installed at the base of the wall. The drain line should be sloped to provide positive gravity drainage from behind the wall area. The drain line should be wrapped with filter fabric to prevent intrusion of fines. The drain line should be backfilled with free draining granular material extending vertically above the drain line to within 2 feet of final grade. The remaining portion of the excavation should be backfilled with lean clay soils to minimize the infiltration of surface water. The granular section behind the wall should have a minimum width of 2 feet and should be encapsulated in the suitable filter fabric to minimize intrusion of fines. The use of a prefabricated drainage blanket on the retaining/below grade wall could also be considered to prevent hydrostatic loading. Drainage blankets should be installed in accordance with the manufacturer's recommendations.

The compactive effort used on backfill is limited to that required to achieve 95 percent of the standard Proctor maximum dry density. Lift thickness should be reduced and light compaction equipment should be used to limit the forces on the wall while achieving the recommended degree of compaction.

3.9.4 Sheet Piling Retaining Walls

The subsurface conditions appear to be suitable to utilize sheet piling for the proposed retaining wall system. Depending upon the final retaining wall tier configuration, dead-man anchors, tie-backs, or additional sheet pile stiffener elements may be required. The design parameter values in the table presented in Section 3.8.5 may be utilized for the design of sheet piling retaining walls.

3.9.5 Retaining Wall Design Parameters

RECOMMENDED STRENGTH PARAMETERS				
	Total Stress (Undrained) Parameters(1)		Effective Stress (Drained) Parameters(1)	
Material Type	c_u, psf	ϕ, degrees	c', psf	ϕ', degrees
Foundation and Retained Materials				
Structural Fill (On-Site Sand Soils)	0	28	0	30
Native Sand Soils	0	28	0	28
Shale Bedrock	5,000	0	250	18
Backfill Materials				
On-Site Sand Soils (Structural Fill)	0	28	0	30
ODOT TYPE A	0	28	0	36
Clean, Crushed Stone (3/8" to 3/4")	0	38	0	38
(1) = Based on previous experience with materials of this classification				

It is recommended that laboratory testing be performed on all materials to be utilized within the foundation, backfill zones and/or reinforced zones of the retaining walls to confirm the engineering design property values presented in the table.

3.10 SCOUR PROTECTION

3.10.1 Articulating Block

Articulating blocks could be used in conjunction with any of the systems described above to facilitate passive resistance along with toe protection for scour to maintain global stability during excessive flow events on the Arkansas River. This system can be placed in both wet and dry conditions. These systems are common in the commercial industry and many aesthetic options are available to the owner. These systems are generally considered structurally flexible.

3.10.2 Rip-Rap

Rip-rap could be used in conjunction with any of the systems described above to facilitate passive resistance along with toe protection for scour to maintain global stability during excessive flow events on the Arkansas River. This system would not require substantial cost for de-watering the toe and the cost of maintaining the excavation. Rip-rap can be designed as a self-launching (self-healing) system and protection can stabilize even after an extreme hydraulic event.

3.11 SEISMIC HAZARDS DETERMINATION

Based on the subsurface information, the project site would be characterized as a Site Class C per the 2003 International Building Code (IBC). Site class C is defined as a soil profile consisting of very dense soils and soft rock where the upper 100 feet of the soil profile has an average shear wave velocity between 1,200 ft/s and 2,500 ft/s, and an SPT N-Value greater than 50 or Su greater than 2,000 psf.

A seismic refraction survey should be performed to determine whether the project site could be characterized as Site Classes A or B. In addition, there is no risk of liquefaction or mass movement of the on-site soils due to a seismic event.

3.12 LANDSCAPING AND SITE GRADING CONSIDERATIONS

Wall performance depends greatly on how well surface water drains from the site. This drainage should be maintained both during construction and over the entire life of the project. The ground surface around structures should be graded such that water drains rapidly through or over the structure without ponding. The surface gradient needed to do this depends on the landscaping type.

Planters should be built such that water exiting from them will not seep into the backfill. Should excessive irrigation, waterline breaks or unusually high rainfall occur, saturated zones and "perched" groundwater may develop. Consequently, the site should be graded so that water drains away readily without saturating the wall's backfill, unless it is designed for this condition. Potential sources of water such as water pipes, drains, garden ponds and the like should be frequently examined for signs of leakage or damage. Any such leakage or damage should be promptly repaired.

Consideration should also be given to limit landscaping and irrigation adjacent to the sea wall. Trees and large bushes can develop an intricate root system that can draw moisture from the subgrade soils, causing them to shrink during dry periods of the year. Desiccation of soils below foundations can result in settlement of foundations or the backfill material.

4. ADDITIONAL SERVICES

4.1 PLANS AND SPECIFICATIONS REVIEW

We recommend that Kleinfelder conduct a general review of the final plans and specifications to evaluate that our earthwork and foundation recommendations have been properly interpreted and implemented during design. In the event Kleinfelder is not retained to perform this recommended review, we will assume no responsibility for misinterpretation of our recommendations.

4.2 DESIGN OF WALL SYSTEMS

Kleinfelder can design the retaining wall systems, and provided plans and specifications if requested. The system should be designed to handle all the geotechnical concerns addressed above but also be designed to ensure that the structures above the sea wall do not settle or migrate with time. Kleinfelder can design the retaining wall system that is chosen by the owners that will meet the geotechnical requirements described in this report and will meet the requirements set forth by the owners for the project's design life.

4.3 CONSTRUCTION OBSERVATION AND TESTING

We recommend that all earthwork during construction be monitored by a representative of Kleinfelder. These observations should include site preparation, placement of all engineered fill and trench backfill, construction of wall system and all foundation excavations concerning the sea wall system. The purpose of these services would be to provide Kleinfelder the opportunity to observe the soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

The following section outlines geotechnical engineering and construction testing services necessary to implement the recommendations presented in this report. To effectively achieve the intent of these recommendations and maintain continuity from design through construction, Kleinfelder should be retained to provide these services:

1. An experienced engineering technician should observe the subgrade throughout the proposed construction area immediately following debris removal, stripping, grubbing, fill evaluation, and undercutting to identify areas requiring additional undercutting and to evaluate the suitability of the exposed surface for fill placement.
2. An experienced engineering technician should monitor and test all fill placed within the wall areas to determine whether the type of material, moisture content and degree of compaction are within recommended limits.
3. An experienced engineering technician should observe the moisture conditioning and proofrolling of the subgrade prior to placement of structural fill to evaluate the suitability of the exposed surface for fill placement.
4. An experienced technician or engineer should observe and test all foundation excavations. Where unsuitable bearing conditions are observed, remedial procedures can be established in the field to avoid construction delays.
5. The condition of the subgrade should be evaluated immediately prior to construction of the foundation of a cast-in-place or MSE wall to determine whether the moisture content of subgrade soils and condition of soils are as recommended.

5. LIMITATIONS

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil and rock conditions could vary between or beyond the points explored. If soil or rock conditions are encountered during construction that differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction changes from that described in this report, our recommendations should also be reviewed.

We have prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty is expressed or implied. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by Kleinfelder during the construction phase in order to evaluate compliance with our recommendations. The scope of our services did not include any environmental assessment or exploration for the presence of hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than three years from the date of report. Land use, site conditions (both on-site and off-site), regulations, or other factors may change over time, and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party and client agrees to defend, indemnify and hold harmless Kleinfelder from any claim or liability associated with such unauthorized or non-compliance.

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APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A
FIELD EXPLORATION PROGRAM

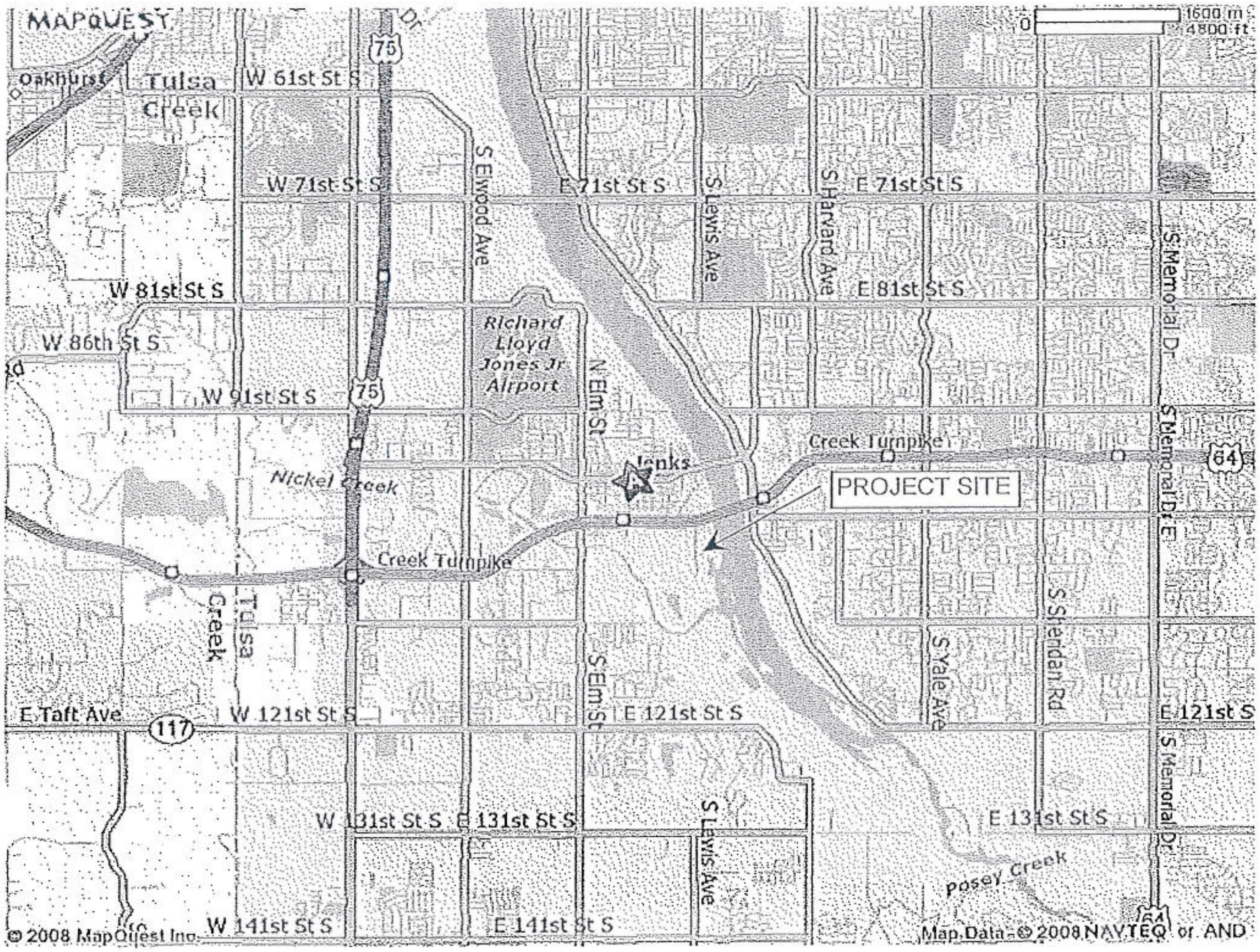
DRILLING & SAMPLING PROCEDURES

Kleinfelder conducted the field work for this study on August 6 and 7, 2008. The exploration consisted of eight borings extending to approximate depths ranging from 18.6 to 28.8 feet below existing ground surface levels.

Representatives of Kleinfelder established the boring locations in the field using the latitude and longitude coordinates for the boring locations provided by Tulsa Engineering and Planning Associates. Kleinfelder representatives located the proposed boring locations and determined ground surface elevations in the field using a handheld Trimble GeoHX. The locations and elevations of the borings should be considered accurate only to the degree implied by the methods used to obtain them.

The borings were performed with a rubber tired buggy-mounted (CME 550), rotary drill rig using hollow stem augers to advance the boreholes. Representative samples were obtained by the split-barrel sampling procedure in accordance with ASTM Specification D 1586. The split-barrel sampling procedure utilizes a standard 2-inch O.D. split-barrel sampler that is driven into the bottom of the boring with a 140-pound hammer falling a distance of 30 inches. The number of blows required to advance the sampler the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Resistance Value (N). These "N" values are indicated on the boring logs at their depth of occurrence and provide an indication of the consistency, relative density, and relative hardness of the subsurface materials. The samples were sealed and returned to our laboratory for further examination, classification and testing.

Boring logs included in this APPENDIX of this report, present such data as soil and bedrock descriptions, consistency, relative density, and relative hardness evaluations, approximate ground surface elevations, depths, sampling intervals, and observed groundwater conditions. Conditions encountered in each of the borings were monitored and recorded by the drill crew. Field logs included visual classification of the materials encountered during drilling, as well as drilling characteristics. Our final boring logs represent the engineer's interpretation of the field logs combined with laboratory observation and testing of the samples. Stratification boundaries indicated on the boring logs were based on observations during our field work, an extrapolation of information obtained by examining samples from the borings and comparisons of soils with similar engineering characteristics. Locations of these boundaries are approximate, and the transitions between material types may be gradual rather than clearly defined.

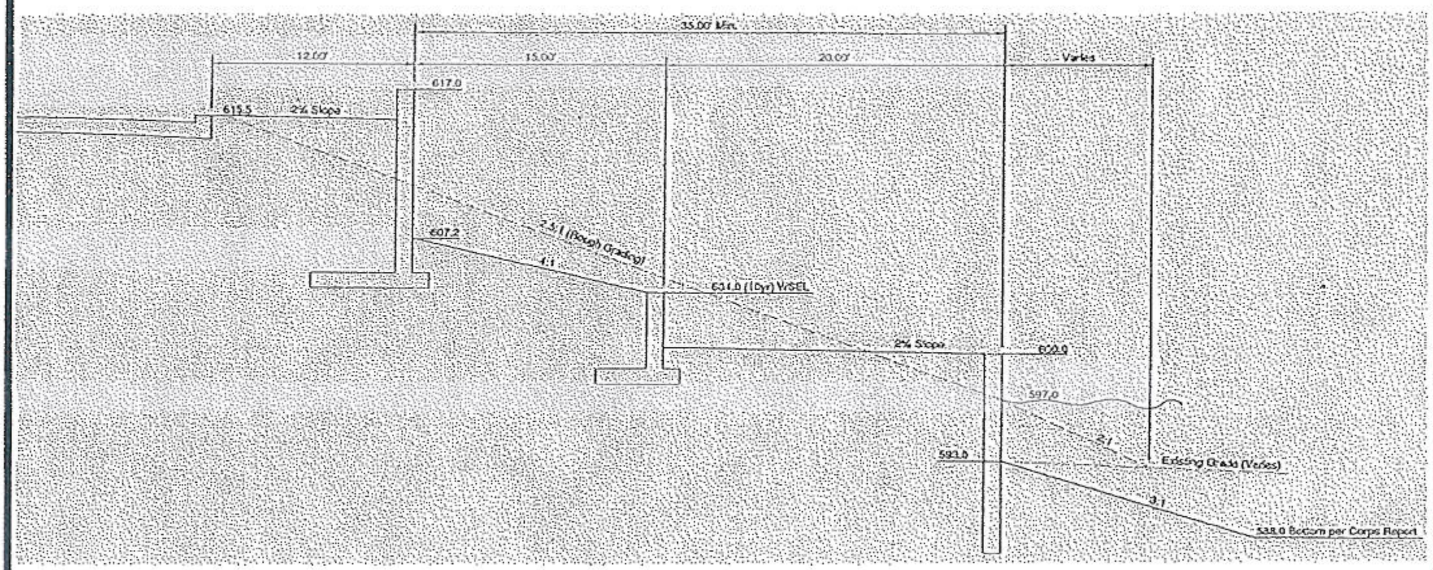


NORTH

KLEINFELDER Bright People. Right Solutions.		Site Vicinity Map	No.	REVISION	BY	DATE	DATE: 8/20/2008
Project No.: 95463	FILE NAME:	Proposed River District Development Creek Turnpike and Lewis Avenue Jenks, OK					SCALE: NONE
Drawn By: DLK	Checked By: BKM						FIGURE: 1

NORTH

KLEINFELDER Bright People. Right Solutions.	Site Plan and Boring Locations	No.	REVISION	BY	DATE	DATE: 8/20/2008
Project No: 95463 / FILE NAME:	Proposed River District Development Creek Turnpike and Lewis Avenue Jenks, OK					SCALE: NONE
Drawn By: DLK / Checked By:						FIGURE: 2

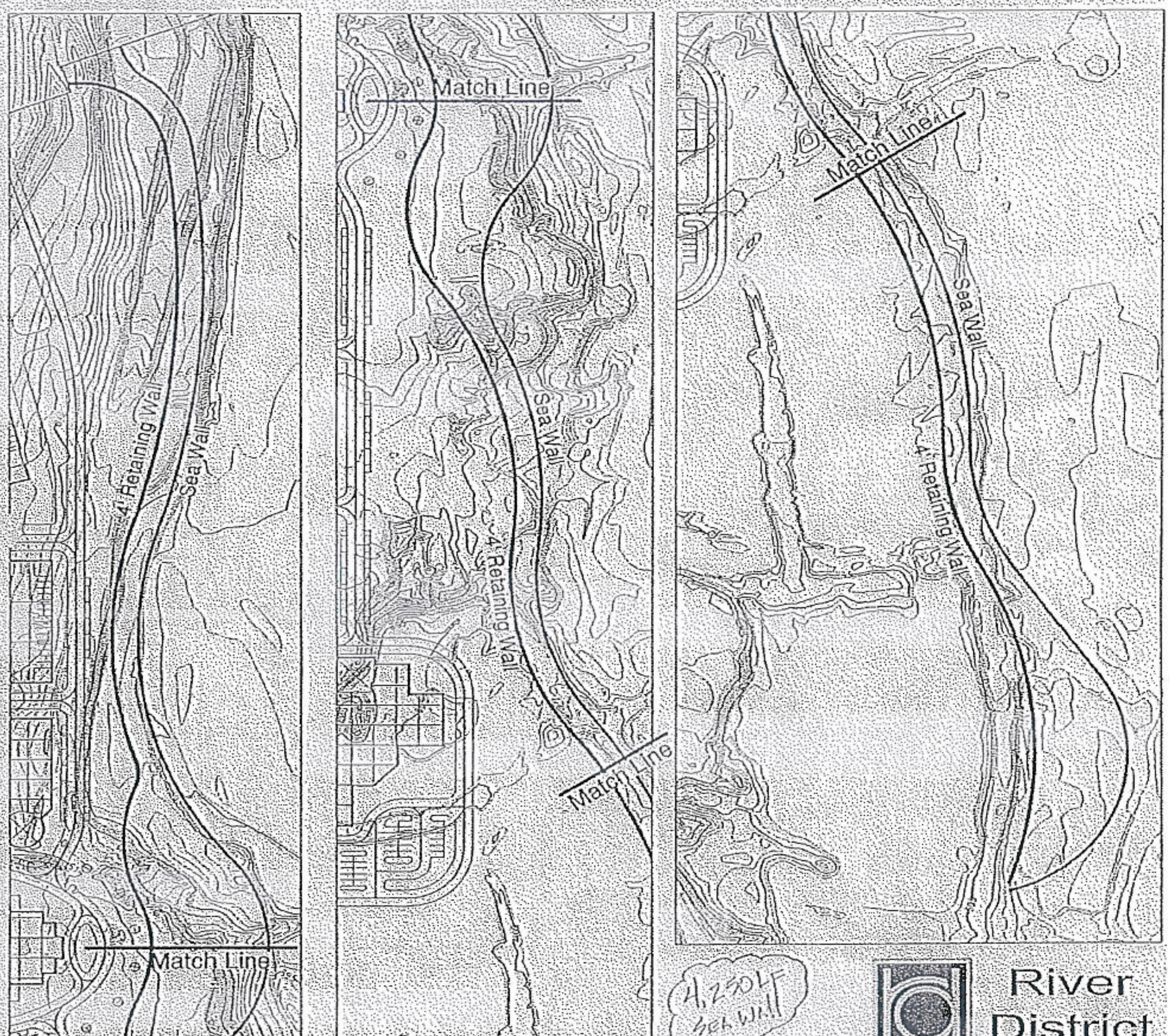


NORTH

KLEINFELDER *Bright People. Right Solutions.*	Conceptual Wall Cross Section	No.	REVISION	BY	DATE
Project No.: 95463 / FILE NAME:	Proposed River District Development Creek Turnpike and Lewis Avenue Jenks, OK				
Drawn By: DLK / Checked By:					

DATE:	11/21/2008
SCALE:	NONE
FIGURE:	3

Exhibit "A"



River District

Tulsa Engineering & Planning Associates, Inc.
6737 South 85th East Avenue · Tulsa, Oklahoma 74133
Phone: 918-252-9621 · Fax 918-250-4566
Civil Engineering · Land Surveying · Land Planning
Certificate of Authorization No. 331 Renewal Date June 30, 2009

N

Job No: 06-076
Scale: 1" = 200'
Date: 03-25-08

NORTH

KLEINFELDER
Bright People. Right Solutions.

Project No: 95463
File Name:
Drawn By: DLK
Checked By:

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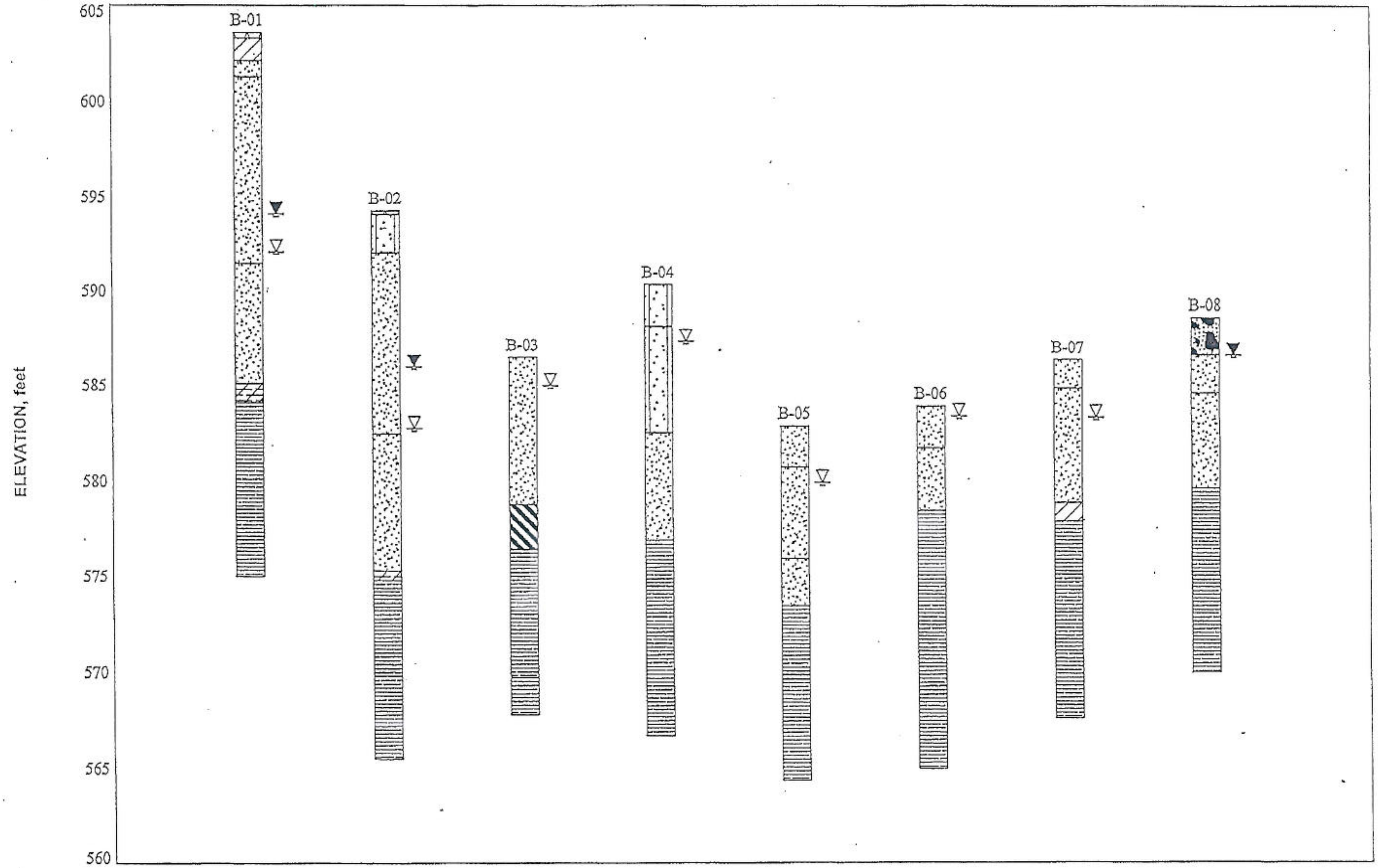
Proposed Retaining Wall Alignment

Proposed River District Development
Creek Turnpike and Lewis Avenue
Jenks, OK

No.	REVISION	BY	DATE

DATE: 11/21/2008
SCALE: NONE
FIGURE: 4



GENERALIZED SUBSURFACE PROFILE

SECTION 1

Proposed River District Development

Creek Turnpike and Lewis Avenue

Jenks, Oklahoma

Approved By: DLK

Job No.: 95463

NOTES

1. See attached legend sheet.
2. Data concerning the various strata have been obtained at boring locations only. The stratigraphy between borings may vary from that shown.
3. For strata details in full, see boring logs appended to this report.

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LOG OF BORING NO. B-01

OWNER/CLIENT: Tulsa Engineering and Planning Associates

PROJECT NAME: Proposed River District Development

ARCHITECT/ENGINEER: Tulsa Engineering and Planning Associates

LOCATION: Creek Turnpike and Lewis Avenue, Jenks, Oklahoma

LAT 36.01700477; LONG -95.95515194

SAMPLE NO.	SAMPLE TYPE	RECOVERY	***STANDARD PENETRATION BLOWS/FT.	UNCONFINED STRENGTH PSF	DRY DENSITY PCF	MOISTURE CONTENT, %	UNIFIED SOIL SYMBOL	DEPTH, FT.	DESCRIPTION
									Approximate Surface Elevation: 603.6
	PA							0.3	TOPSOIL — 603.3
1	SS	6	10			3.2	CL	1.5	LEAN CLAY, stiff, dry, brown — 602.1
	PA							2.3	SAND, poorly graded, medium dense, moist, light brown — 601.3
2	SS	12	14			1.9	SP		SAND, poorly graded, medium dense, moist, tan
	PA								
3	SS	12	11			8.0	SP	10	
	PA							12.1	591.5
									SAND, coarse grained, loose, moist, brown
4	SS	9	7			11.3	SP		
	PA							18.5	585.1
5	SS	6	10/6" 20/6" 50/6"			24.0		19.5	**WEATHERED SHALE, soft, wet, olive gray — 584.1
	PA							20	**SHALE, moderately hard to hard, wet, dark gray
6	SS	3	50/3.5"			14.5			
	PA								
7	SS	1	50/1.5"			19.6		28.6	575.0

BOTTOM OF BORING

**Rock classification is based on drilling characteristics and visual observation of disturbed samples. Core samples would be required for exact classification.

SIEVE ANALYSIS
Sample 2, Depth 3.5-5 feet
Percent Fines = 3.3

***CME Automatic Hammer

The stratification lines represent the approximate boundary lines between soil and rock types. In-situ the transition may be gradual.

WATER LEVEL OBSERVATIONS			
∇ 11.5 W.D.	BORING STARTED 8-7-08		
▼ 9.5 A.B.	BORING COMPLETED 8-7-08		
Backfilled @ Completion	DRILL RIG CME 550	DRILLER AT	
	APPROVED DLK	JOB NO. 95463	

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LOG OF BORING NO. B-02

OWNER/CLIENT: Tulsa Engineering and Planning Associates

ARCHITECT/ENGINEER: Tulsa Engineering and Planning Associates

PROJECT NAME: Proposed River District Development

LOCATION: Creek Turnpike and Lewis Avenue, Jenks, Oklahoma

LAT 36.01585389; LONG -95.95485746

SAMPLE NO.	SAMPLE TYPE	RECOVERY	***STANDARD PENETRATION BLOWS/FT.	UNCONFINED STRENGTH PSF	DRY DENSITY PCF	MOISTURE CONTENT, %	UNIFIED SOIL SYMBOL
	PA						
1	SS	12	9			2.9	SM
	PA						
2	SS	9	12			1.4	SP
	PA						
3	SS	12	17			3.6	SP
	PA						
4	SS	15	3			18.9	SP
	PA						
5	SS	15	4/6" 13/6" 50/5.5"			19.0	
	PA						
6	SS	3	50/3"			15.5	
	PA						
7	SS	3	50/3"			16.6	

***CMB Automatic Hammer

DESCRIPTION

Approximate Surface Elevation: 594.3

Depth, ft.	Description	
0.2	TOPSOIL	594.1
2.2	SILTY SAND with gravel, fine grained, loose, dry, light brown	592.1
	SAND, poorly graded, medium dense, moist, tan	
11.8	SAND, coarse grained, very loose, wet, brown	582.5
19.0	**WEATHERED SHALE, soft, olive gray	575.3
19.5	**SHALE, moderately hard, dark gray	574.8
28.8		565.5

BOTTOM OF BORING

**Rock classification is based on drilling characteristics and visual observation of disturbed samples. Core samples would be required for exact classification.

SIEVE ANALYSIS
Sample 1, Depth 0.5-2 feet
Percent Fines = 14.8

The stratification lines represent the approximate boundary lines between soil and rock types. In-situ the transition may be gradual.

WATER LEVEL OBSERVATIONS			
▽ 11.5 W.D.	BORING STARTED 8-7-08		
▼ 8.3 A.B.	BORING COMPLETED 8-7-08		
	DRILL RIG CME 550	DRILLER AT	
Backfilled @ Completion	APPROVED DLK	JOB NO. 95463	

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AUTO HAMMER 95463.GPJ GEOSYSTM.GDT 8/20/08

LOG OF BORING NO. B-03

OWNER/CLIENT: Tulsa Engineering and Planning Associates

PROJECT NAME: Proposed River District Development

ARCHITECT/ENGINEER: Tulsa Engineering and Planning Associates

LOCATION: Creek Turnpike and Lewis Avenue
Jenks, Oklahoma

LAT 36.01487681; LONG -95.95443530

SAMPLE NO.	SAMPLE TYPE	RECOVERY	***STANDARD PENETRATION BLOWS/FT.	UNCONFINED STRENGTH PSF	DRY DENSITY PCF	MOISTURE CONTENT, %	UNIFIED SOIL SYMBOL	GRAPHIC LOG	DEPTH, FT.	DESCRIPTION
										Approximate Surface Elevation: 586.5
	PA									SAND with gravel, coarse grained, very loose to loose, wet, brown ▽
1	SS	12	2			9.9	SP			
	PA									
2	SS	9	5			13.6	SP			
	PA									7.7 — 578.8
										LEAN TO FAT CLAY, stiff, wet, mottled olive, gray, and brown
3	SS	12	13				CL CH		10	10.1 — 576.4
	PA									**SHALE, soft, wet, gray
4	SS	5	50/5"			22.4				
	PA									16.8 — 569.7
										**SHALE, hard, wet, gray
~~5~~	~~SS~~	~~2~~	~~50/2"~~			~~32.5~~				18.7 — 567.8

BOTTOM OF BORING

**Rock classification is based on drilling characteristics and visual observation of disturbed samples. Core samples would be required for exact classification.

ATTERBERG LIMITS
Sample 3, Depth 8.5-10 feet

LL	PL	PI
47	26	21

***CME Automatic Hammer

The stratification lines represent the approximate boundary lines between soil and rock types. In-situ the transition may be gradual.

WATER LEVEL OBSERVATIONS		
▽ 1.5 W.S.	BORING STARTED	8-6-08
▼ Dry A.B.	BORING COMPLETED	8-6-08
Backfilled @ Completion	DRILL RIG CME 550	DRILLER AT
	APPROVED DLK	JOB NO. 95463

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AUTO HAMMER 95463.GPJ GEOSYSTM.GDT 8/20/08

LOG OF BORING NO. B-04

OWNER/CLIENT: Tulsa Engineering and Planning Associates

PROJECT NAME: Proposed River District Development

ARCHITECT/ENGINEER: Tulsa Engineering and Planning Associates

LOCATION: Creek Turnpike and Lewis Avenue
Jenks, Oklahoma

LAT 36.01299535; LONG -95.95443607

SAMPLE NO.	SAMPLE TYPE	RECOVERY	***STANDARD PENETRATION BLOWS/FT.	UNCONFINED STRENGTH PSF	DRY DENSITY PCF	MOISTURE CONTENT, %	UNIFIED SOIL SYMBOL	DEPTH, FT.	DESCRIPTION (Approximate Surface Elevation: 590.4)
	~~PA~~								SILTY SAND, fine grained, loose, dry, light brown
1	SS	9	4			4.2	SM	2.2	588.2
	PA								SILTY SAND, poorly graded, loose, wet, brown ▽
2	SS	12	5			19.8	SM		
	PA							7.8	582.6
3	SS	14	11			10.0	SP	10	SAND with gravel, coarse grained, medium dense, wet, brown
	PA							13.5	576.9
4	SS	8	40/6" 50/2.5"			17.5			**SHALE, moderately hard, wet, dark gray
	PA								
5	SS	3	50/3"			14.3		20	
	PA							23.7	566.7
~~6~~	~~SS~~	~~2~~	~~50/2.5"~~			~~13.2~~			

BOTTOM OF BORING

**Rock classification is based on drilling characteristics and visual observation of disturbed samples. Core samples would be required for exact classification.

SIEVE ANALYSIS
Sample 2, Depth 2.5-4 feet
Percent Fines = 29.8

***CME Automatic Hammer

The stratification lines represent the approximate boundary lines between soil and rock types. In-situ the transition may be gradual.

WATER LEVEL OBSERVATIONS			
▽ 3.0 W.D.	BORING STARTED 8-6-08		
▼ Dry A.B.	BORING COMPLETED 8-6-08		
	DRILL RIG CME 550	DRILLER AT	
Backfilled @ Completion	APPROVED DLK	JOB NO. 95463	

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LOG OF BORING NO. B-05

OWNER/CLIENT: Tulsa Engineering and Planning Associates

PROJECT NAME: Proposed River District Development

ARCHITECT/ENGINEER: Tulsa Engineering and Planning Associates

LOCATION: Creek Turnpike and Lewis Avenue, Jenks, Oklahoma

LAT 36.01164870; LONG -95.95453299

SAMPLE NO.	SAMPLE TYPE	RECOVERY	***STANDARD PENETRATION BLOWS/FT.	UNCONFINED STRENGTH PSF	DRY DENSITY PCF	MOISTURE CONTENT, %	UNIFIED SOIL SYMBOL	DEPTH, FT.	DESCRIPTION (Approximate Surface Elevation: 582.9)
	PA								SAND, poorly graded, very loose, wet, brown
1	SS	9	1			22.2	SP	2.2	580.7
	PA								SAND, poorly graded, loose, wet, brown ▽
2	SS	12	8			20.8	SP		
	PA							7.0	575.9
									SAND with gravel, coarse grained, medium dense, wet, brown
3	SS	18	37			11.3	SP	9.5	573.4
								10	**SHALE, moderately hard to hard, dark gray
	PA								
4	SS	2	50/2"			13.9			
	PA								
5	SS	1	50/0.5'			11.7		18.6	564.3

BOTTOM OF BORING

**Rock classification is based on drilling characteristics and visual observation of disturbed samples. Core samples would be required for exact classification.

SIEVE ANALYSIS
Sample 1, Depth 0.5-2 feet
Percent Fines = 1.9

***CME Automatic Hammer

The stratification lines represent the approximate boundary lines between soil and rock types. In-situ the transition may be gradual.

WATER LEVEL OBSERVATIONS		
▽ 3.0 W.S.	BORING STARTED	8-6-08
▼ Dry A.B.	BORING COMPLETED	8-6-08
	DRILL RIG	CME 550 — DRILLER AT
Backfilled @ Completion	APPROVED	DLK — JOB NO. 95463

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AUTO HAMMER 95463.GPJ GEOSYSTM.GDT 8/20/08

LOG OF BORING NO. B-06

OWNER/CLIENT: Tulsa Engineering and Planning Associates

PROJECT NAME: Proposed River District Development

ARCHITECT/ENGINEER: Tulsa Engineering and Planning Associates

LOCATION: Creek Turnpike and Lewis Avenue, Jenks, Oklahoma

LAT 36.01008197; LONG -95.95386019

SAMPLE NO.	SAMPLE TYPE	RECOVERY	***STANDARD PENETRATION BLOWS/FT.	UNCONFINED STRENGTH PSF	DRY DENSITY PCF	MOISTURE CONTENT, %	UNIFIED SOIL SYMBOL	GRAPHIC LOG	DEPTH, FT.	DESCRIPTION (Approximate Surface Elevation: 584.0)
	PA									SAND with trace gravel, poorly graded, loose, wet, brown ▽
1	SS	12	5			19.1	SP		2.2	581.8
	PA									SAND, poorly graded, loose, wet, dark brown
2	SS	12	4			21.2	SP		5.5	578.5
	PA									**SHALE, hard, dark gray
3	SS	2	50/1"			7.8			10	
	PA									
4	SS	2	50/2"			15.2				
	PA								17.7	566.3
5	SS	0	50/6"						19.0	**SHALE, soft, dark gray — 565.0

BOTTOM OF BORING

**Rock classification is based on drilling characteristics and visual observation of disturbed samples. Core samples would be required for exact classification.

<u>SIEVE ANALYSIS</u>
Sample 2, Depth 3.5-5 feet
Percent Fines = 5.0

***CME Automatic Hammer

The stratification lines represent the approximate boundary lines between soil and rock types. In-situ the transition may be gradual.

WATER LEVEL OBSERVATIONS			
▽ 0.5 W.S.	BORING STARTED 8-6-08		
▼ Dry A.B.	BORING COMPLETED 8-6-08		
Backfilled @ Completion	DRILL RIG CME 550	DRILLER AT	
	APPROVED DLK	JOB NO. 95463	

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AUTO HAMMER 95463.GPJ GEOSYSTM.GDT 8/20/08

LOG OF BORING NO. B-07

OWNER/CLIENT	PROJECT NAME
Tulsa Engineering and Planning Associates	Proposed River District Development
ARCHITECT/ENGINEER: Tulsa Engineering and Planning Associates	LOCATION: Creek Turnpike and Lewis Avenue, Jenks, Oklahoma

LAT 36.00799814; LONG -95.95317337

Approximate Surface Elevation: 586.4

SAMPLE NO.	SAMPLE TYPE	RECOVERY	***STANDARD PENETRATION BLOWS/FT.	UNCONFINED STRENGTH PSF	DRY DENSITY PCF	MOISTURE CONTENT, %	UNIFIED SOIL SYMBOL	GRAPHIC LOG	DEPTH, FT.	DESCRIPTION	ELEVATION
	PA										
1	SS	9	2			8.7	SP		1.5	SAND, poorly graded, very loose, wet, light brown	584.9
	PA									SAND, poorly graded, loose, wet, brown ▽	
2	SS	9	5			13.8	SP				
	PA								7.5		578.9
									8.5	LEAN CLAY, medium stiff, dark gray	577.9
3	SS	4	50/4"			13.1			10	**SHALE, moderately hard, gray to light gray	
	PA										
4	SS	2	50/2.5"			29.1					
	PA										
5	SS	0	50/4"						18.8		567.6

BOTTOM OF BORING

**Rock classification is based on drilling characteristics and visual observation of disturbed samples. Core samples would be required for exact classification.

SIEVE ANALYSIS
Sample 1, Depth 0.5-2 feet
Percent Fines = 0.3

***CME Automatic Hammer

The stratification lines represent the approximate boundary lines between soil and rock types. In-situ the transition may be gradual.

WATER LEVEL OBSERVATIONS			
▽ 3.0 W.S.	BORING STARTED	8-6-08	
▼ Dry A.B.	BORING COMPLETED	8-6-08	
	DRILL RIG CME 550	DRILLER AT	
Backfilled @ Completion	APPROVED DLK	JOB NO. 95463	

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AUTO HAMMER 95463.GPJ GEOSYSTM.GDT 8/20/08

LOG OF BORING NO. B-08

OWNER/CLIENT	PROJECT NAME
Tulsa Engineering and Planning Associates	Proposed River District Development
ARCHITECT/ENGINEER	LOCATION
Tulsa Engineering and Planning Associates	Creek Turnpike and Lewis Avenue Jenks, Oklahoma

LAT 36.00635344; LONG -95.95290423

SAMPLE NO.	SAMPLE TYPE	RECOVERY	***STANDARD PENETRATION BLOWS/FT.	UNCONFINED STRENGTH PSF	DRY DENSITY PCF	MOISTURE CONTENT, %	UNIFIED SOIL SYMBOL	GRAPHIC LOG	DEPTH, FT.	DESCRIPTION Approximate Surface Elevation: 588.7
	PA									RIP RAP
									2.0	586.7
										SAND with silt, poorly graded, medium dense, wet, brown
									4.0	584.7
2	SS	12	22			8.0	SP			SAND with gravel, coarse grained, medium dense, wet, brown
	PA									
									9.0	579.7
3	SS	3	4/6" 50/3"			18.2	.		10	**SHALE, moderately hard, wet, dark gray
	PA									
4	SS	3	50/3"			14.0				
	PA									
									18.7	570.0
~~5~~	~~SS~~	~~2~~	~~50/2"~~			~~13.0~~				

BOTTOM OF BORING

**Rock classification is based on drilling characteristics and visual observation of disturbed samples. Core samples would be required for exact classification.

SIEVE ANALYSIS
Sample 2, Depth 3.5-5 feet
Percent Fines = 6.5

***CME Automatic Hammer

The stratification lines represent the approximate boundary lines between soil and rock types. In-situ the transition may be gradual.

WATER LEVEL OBSERVATIONS	BORING STARTED 8-7-08	
2 W.S.	BORING COMPLETED 8-7-08	
2 A.B.	DRILL RIG CME 550	DRILLER AT
Backfilled @ Completion	APPROVED DLK	JOB NO. 95463

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AUTO HAMMER 95463.GPJ GEOSYSTM.GDT 8/20/08

General Notes

DRILLING NOTES

WATER LEVEL MEASUREMENTS

Water levels indicated on the boring logs are levels measured in the borings at the times indicated. In permeable materials, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of groundwater levels is not possible with only short-tewm observations.

WATER LEVEL OBSERVATION DESIGNATION

W.D.	While Drilling
A.B.	After Boring
B.C.R.	Before Casing Removal
A.C.R.	After Casing Removal
24 hr.	Water level taken approximately 24 hrs. after boring completion

DRILLING AND SAMPLING SYMBOLS

AS	Auger Sample
CS	Continuous Sampler
DB	Diamond Bit -NX unless otherwise noted
HA	Hand Auger
HS	Hollow Stem Auger
PA	Power Auger
RB	Rock Bit
SS*	Spit-Barrel
ST	Shelby Tube - 2" (51mm) unless otherwise noted
WB	Wash Bore

*The Standard Penetration Test is conducted in conjunction with the split-barrel sampling procedure. The 'N' value corresponds to the number of blows required to drive the last 1 foot (0.3m) of an 18 in. (0.46m) long, 2 in. (51 mm O.D. split-barrel ampler with a 140 lb. (63.5 kg) hammer falling a distance of 30 in. (0.76m). The Standard Penetration Test is carries out according to ASTM D-1586. (See'N' Value below.)

SOIL PROPERTIES & DESCRIPTIONS

TEXTURE

PARTICLE	SIZE	
Clay	<0.002mm	(<0.002 mm)
Silt	<#200 Sieve	(0.075 mm)
Sand	#4 to #200 Sieve	(4.75 to 0.075 mm)
Gravel	3 in. To #4 Sieve	(75 mm to 4.75 mm)
Cobbles	12 in. to 3 in.	(300 mm to 75 mm)
Boulders	>12 in.	(300 mm)

COMPOSITION

SAND & GRAVEL

Description	% by Dry Weight
trace	< 15
with	15 - 29
modifier	> 30

FINES

Decription	% by Dry Weight
trace	< 5
with	5 - 12
modifier	> 12

Soil descriptions are based on the United Soil Classification System (USCS) as outlined in ASTM Designations D-2487 and D-2488. The USCS group symbol shown on the boring logs correspond to the group names listed below. The description includes soil constituents, consistency, relative density, color and other appropriate descriptive terms. Geologic description of bedrock, when encountered, also is shown in the description column.

GROUP SYMBOL	GROUP NAME	GROUP SYMBOL	GROUP NAME
GW	Well Graded Gravel	CL	Lean Clay
GP	Poorly Graded Gravel	ML	Silt
GM	Silty Gravel	OL	Organic Clay or Silt
GC	Clayey Gravel	CH	Fat Clay
SW	Well Graded Sand	MH	Elastic Silt
SP	Poorly Graded Sand	OH	Organic Clay or Silt
SM	Silty Sand	PT	Peat
SC	Clayey Sand	CL-CH	Lean to Fat Clay

COHESIVE SOILS

CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (Qu) (psf)	(kPa)
Very Soft	< 500	(<24)
Soft	500 - 1000	(24 - 48)
Medium	1001 - 2000	(48 - 96)
Stiff	2001 - 4000	(96 - 192)
Very Stiff	4001 - 8000	(192 - 383)
Hard	> 8001	(>383)

PLASTICITY

Description	Liquid Limit (%)
Lean	< 45%
Lean to Fat	45 to 49%
Fat	≥ 50%

COHESIONLESS SOILS

RELATIVE DENSITY	N' VALUE*
Very Loose	0 - 3
Loose	4 - 9
Medium Dense	10 - 29
Dense	30 - 49
Very Dense	≥ 50

BEDROCK PROPERTIES & DESCRIPTIONS

ROCK QUALITY DESIGNATION (RQD**)

DESCRIPTION OF ROCK QUALITY	RQD (%)
Very Poor	0 - 25
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100

**RQD is defined as the total length of sound core pieces, 4 inches (102mm) or greater in length, expressed as a percentage of the total length cored. RQD provides an indication of the integrity of the rock mass and relative extent of seams and bedding planes.

DEGREE OF WEATHERING

Slightly Weathered	Slight decomposistion of parent material in joints and seams
Weathered	Well-developed and decomposed joints and seams.
Highly Weathered	Rock highly decomposed, may be extremely broken.

SOLUTION AND VOID CONDITIONS

Solid	Contains no voids.
Vuggy	Containing small pits or cavities < 1/2" (13mm).
Porous	Containing numerous voids which may be interconnected.
Cavernous	Containing cavities, sometimes quite large.

When classification of rock materials has been estimated from disturbed samples, core samples and petrographic analysis may reveal other rock types.

HARDNESS & DEGREE OF CEMENTATION

LIMESTONE

HARD	Difficult to scratch with knife.
Moderately Hard	Can scratch with knife but not with fingernail.
Soft	Can be scratched with fingernail.

SHALE

Hard	Can scratch with knife but not with fingernail.
Moderately Hard	Can be scratched with fingernail.
Soft	Can be molded easily with fingers.

SANDSTONE

Well Cemented	Capable of scratching a knife blade.
Cemented	Can be scratched with knife.
Poorly Cemented	Can be broken apart easily with fingers.

BEDDING CHARACTERISTICS

TERM	THICKNESS (inches)	THICKNESS (mm)
Very Thick Bedded	> 36	>915
Thick Bedded	12 - 36	305 - 915
Medium Bedded	4 - 12	102 - 305
Thin Bedded	1 - 4	25 - 102
Very Thin Bedded	0.4 - 1	10 - 25
Laminated	0.1 - 0.4	2.5 - 10
Thinly Laminated	< 0.1	<2.5

Bedding Planes	Planes dividing the individual layers, beds, or strata of rocks.
Joint	Fracture in rock, generally more or less vertical or traverse to the bedding.
Seam	Applies to bedding plane with an unspecified degree of weathering.

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Boring Log Symbols

SURFACE MATERIALS

- Topsoil
- Fill Material
- Asphaltic Concrete
- Concrete
- Granular Base
- Rubble Fill
- Wood Fill
- Water

WEATHERED BEDROCK

- Joint or Void
- Weathered Shale
- Weathered Sandstone
- Weathered Limestone
- Weathered Dolomite

FINE GRAINED SOILS

- Fat Clay
- Lean Fat Clay
- Lean Clay
- Clayey Silt
- Silt
- Elastic Silt
- Sandy Fat Clay
- Sandy Lean to Fat Clay
- Sandy Lean Clay
- Low Plasticity Organic
- High Plasticity Organic
- Peat

BEDROCK UNITS

- Shale
- Fissile Shale
- Sandstone
- Chalk
- Limestone
- Dolomite
- Siltstone
- Claystone
- Coal
- Gypsum
- Interbedded Limestone & Shale
- Interbedded Sandstone & Shale
- Cherty Bedrock

COARSE-GRAINED SOILS

- Cobbles and Boulders
- Well Graded Gravel
- Poorly Graded Gravel
- Silty Gravel
- Clayey Gravel
- Gravelly Sand
- Well Graded Sand
- Poorly Graded Sand
- Silty Sand
- Interbedded Sand & Silt
- Sandy Silt
- Clayey Sand

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APPENDIX B

LABORATORY TESTING PROGRAM

Attachment 3

W48-0605

Terracon

SUBSURFACE EXPLORATION REPORT

PROPOSED MINGO ROAD WATER LINE
BIXBY, OKLAHOMA

Terracon Project No. 04095060
May 5, 2009

Prepared for:

City of Bixby
Bixby, Oklahoma

Prepared by:

Terracon
Tulsa, Oklahoma

Terracon

May 5, 2009

Consulting Engineers & Scientists

Terracon Consultants, Inc.
10930 E. 56th St.
Tulsa, OK 74146
Phone (918) 250-0461
Fax (918) 250-4570
www.terracon.com

City of Bixby
c/o Holloway, Updike, and Bellen, Inc.
818 East Side Boulevard
Muskogee, Oklahoma 74402

Attn: Mr. Jay Updike

Re: Subsurface Exploration Report
Proposed Mingo Road Water Line
Bixby, Oklahoma
Terracon Project No. 04095060

Dear Mr. Updike:

We are submitting, herewith, the results of the subsurface exploration for the proposed water line to be constructed in Bixby, Oklahoma. This report presents the results of the field and laboratory testing, a description of the subsurface conditions encountered at the borings, and geotechnical recommendations regarding waterline construction.

If you have any questions regarding the contents of this report or if we can be of further service, please do not hesitate to contact us.

Sincerely,
Terracon
Cert. of Auth. #CA.-4531 exp. 6/30/09

REGISTERED PROFESSIONAL ENGINEER
5/5/09
CONRAD S. KOEHLER
20784
OKLAHOMA

James W. Davis, E.I.
Staff Professional

Conrad S. Koehler, P.E.
Oklahoma No. 20784

Michael H. Homan, P.E.
Senior Principal

JWD:CSK:MHH:jb
Enclosure

Copies to: Addressee (2)

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SUBSURFACE EXPLORATION REPORT

PROPOSED MINGO ROAD WATER LINE
BIXBY, OKLAHOMA

Terracon Project No. 04095060
May 5, 2009

INTRODUCTION

This report presents the results of the subsurface exploration for the proposed water line to be constructed in Bixby, Oklahoma. As part of our exploration, nine borings were advanced to depths of approximately 15 to 63.5 feet. The results of the borings and a diagram indicating their approximate location are attached. Three additional borings were planned as part of the field operations, but we did not drill them at this time due to the water level of the Arkansas River.

We understand the project involves constructing a 12 to 24-inch diameter water line along Mingo Road in Bixby, Oklahoma.

The approximately 2.8 mile long water line will start on the south side of the Arkansas River north of East 151st Street, cross underneath the river, and extend north along South Mingo Road to its intersection with East 121st Street South. The water line will begin as a 24-inch diameter line to the intersection with East 141st Street South where it will reduce to a 16-inch diameter line. The 16-inch line will continue along Mingo Road to its intersection with East 131st Street, where it will reduce to a 12-inch diameter line for the remainder of the extension. The line is expected to extend approximately 5 feet below the top of bedrock as it crosses underneath the Arkansas River. The line is expected to bear at a maximum depth of approximately 15 feet below the ground surface for the remainder of the extension.

SUBSURFACE EXPLORATION PROCEDURES

Terracon drilled and sampled nine borings for the project at the approximate locations shown on the attached boring location diagram. The boring locations and elevations of borings B-1 and B-5 Holloway, Updike, and Bellen provided staked. Terracon established the boring locations of Borings B-6 through B-12 in the field by pacing or taping from the available reference features. The boring locations should be considered accurate only to the degree implied by the methods used to define them.

The borings were drilled with an ATV track-mounted, rotary drill rig using continuous flight augers to advance the boreholes. Representative samples were obtained using the split-barrel sampling procedure.

Disturbed samples are obtained in the split-barrel sampling procedure by driving a 2-inch O.D. split-barrel sampling spoon into the ground using a 140-pound, automatic hammer falling 30 inches. The number of blows required to advance the sampling spoon were recorded in the field and are shown on the boring logs as the standard penetration resistance (N) value. The number of blows required to advance the sampling spoon the final 12 inches or less of a standard 18-inch sampling interval indicate the in-place relative density of granular soils and, to a lesser degree of accuracy, the consistency of cohesive soils and the hardness of weathered bedrock.

A greater efficiency is achieved with the automatic hammer, compared to a conventional safety hammer operated by a cathead and rope. The effect of the increased efficiency was considered in interpreting the standard penetration resistance values.

The rock in boring B-1 was cored using an NX-size diamond bit core barrel. The percent recovery (REC) was determined for each core run and is shown on the attached boring log.

Samples obtained in the field were tagged for identification, sealed to reduce moisture loss and returned to our laboratory for further examination, testing, and classification.

During the drilling operation, field logs were prepared by the drill crew. These logs report drilling and sampling methods, sampling intervals, soil, rock and groundwater conditions, and the driller's visual evaluation of the conditions encountered between samples. The final boring logs, included in this report, have been prepared based on the driller's field logs and have been modified, where appropriate, based on the results of the laboratory observation and testing.

LABORATORY TESTING PROGRAM

Selected soil samples were tested for the following:

- Moisture content
- Sieve analysis

Laboratory test results are shown on the boring logs and attached appendices.

The soil samples were examined in our laboratory by a geotechnical engineer and classified based on the soil's texture and plasticity in accordance with the attached General Notes and Unified Soil Classification System. The Unified System group symbols are shown on the borings logs. A brief description of the Unified Soil Classification System is attached. Bedrock materials were classified in accordance with the General Notes and described using commonly accepted geotechnical terminology.

SUBSURFACE CONDITIONS

Soil and Rock Conditions

The subsurface conditions encountered in the borings are shown on the boring logs and are briefly described below. The stratification lines shown on the boring logs represent the approximate boundary between soil and rock types; in-situ, the transition between materials may be gradual and indistinct. Classification of bedrock materials was made from disturbed and core samples. Petrographic analysis may reveal other rock types.

The generalized profile of the subsurface conditions encountered at the boring locations can be described as follows:

- We encountered approximately 1 to 3 inches of topsoil at the surface.
- The topsoil in boring B-5 was underlain by sand and fat clay fill to a depth of approximately 6.5 feet.
- The topsoil and fill material was underlain by very loose to medium dense sand, silty sand, clayey sand, and silt, and medium stiff to very stiff, lean clay, fat clay, and shaley lean clay to the boring termination depths of 15 feet in borings B-6 through B-12 and to depths of 27.5 to 33 feet in borings B-1 and B-5.
- The native sands and clays in borings B-1 and B-5 were underlain by dark gray, soft to moderately hard shale bedrock to the boring termination depths of 58.5 to 63.5 feet.

Groundwater Conditions

Groundwater level observations made while drilling and immediately after completion of the borings are shown in the lower left corner of the boring logs. As shown in the lower left corner of the boring logs, groundwater was encountered in all the borings except B-10 and B-11 at depths ranging from 4 to 17 feet during drilling operations. Groundwater was not encountered in borings B-10 and B-11 during drilling operations.

The groundwater level observations made during our exploration provide an indication of the groundwater conditions at the time the borings were drilled. Longer monitoring using piezometers or cased holes, sealed from the influence of surface water, would be required to evaluate longer-term groundwater conditions. During some periods of the year, perched water could develop at various depths. Fluctuations in the amount of perched water, if any, and long-term groundwater levels should be expected throughout the years depending upon variations in the amount of rainfall, runoff, evaporation, water level in the Arkansas River, and other hydrological conditions not apparent at the time the borings were drilled.

ANALYSIS AND RECOMMENDATIONS

Excavations

Existing fill material and native sands, silts, and clays were encountered the boring termination depths of 15 feet in borings B-6 through B-12 and to depths of 27.5 to 33 feet in borings B-1 and B-5. Soft and moderately hard shale bedrock materials were encountered below the sands and clays in borings B-1 and B-5 to the boring termination depths of 58.5 to 63.5 feet.

Excavations in the existing fill and native sand, silt, and clay soils can be accomplished using normal excavation equipment.

Groundwater Considerations

Groundwater was observed at all borings except B-10 and B-11 at depths of about 4 to 17 feet. However, fluctuations in the presence and level of groundwater can occur over time. Temporary dewatering using sumps and pumps would be required if groundwater is encountered in excavations to perform construction "in the dry."

Temporary Excavation Slopes

Short-term excavations in the native medium stiff to very stiff, clay soils above the groundwater level should remain stable at temporary slopes of 2 horizontal to 1 vertical (2H to 1V) or flatter. The existing fill and native soft, clay soils, and the native very loose to medium dense silt and sand soils above the groundwater level should remain stable at slopes of 3H to 1V or flatter. These temporary slope inclinations consider that groundwater will be effectively lowered below the bottom of the excavations. Where sloped temporary excavations are not feasible, bracing or shoring will be required.

No materials should be stockpiled within a lateral distance from the slope crest equal to the slope height. All excavations should comply with applicable local, state, and federal safety regulations. Construction site safety is the responsibility of the contractor, who shall also be responsible for the means, methods, and sequencing of construction operations. Under no circumstances should the information Terracon is providing be interpreted to mean that Terracon is assuming any responsibility for construction site safety or the contractor's activities.

GENERAL COMMENTS

The information presented in this report is based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

APPROXIMATE LOCATION
OF PROPOSED MINGO ROAD
WATER LINE

2000 0 1000 2000

APPROXIMATE SCALE IN FEET

N

Legend

- Boring Location
- Not drilled as part of this exploration

DIAGRAM IS FOR GENERAL LOCATION ONLY,
AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Mngr: JWD	Project No. 04095060	Terracon	BORING LOCATION DIAGRAM
Drawn By: DC	Scale: See Bar Scale	Consulting Engineers and Scientists	GEOTECHNICAL EXPLORATION
Checked By: JWD	File No. 04095060-1	10930 EAST 56th STREET TULSA, OKLAHOMA 74146	Proposed Mingo Road Water Line
Approved By: CSK	Date: May 2009	PH. (918) 250-0461 FAX. (918) 250-4570	Bixby Oklahoma

FIG. No. 1

LOG OF BORING NO. B-1

CLIENT	
City of Bixby	
SITE: Bixby, Oklahoma	PROJECT: Proposed Mingo Road Water Line

DESCRIPTION (Approx. Surface Elev.: 599.5 ft)	Elev.	DEPTH, ft.	USCS SYMBOL	NUMBER	TYPE	RECOVERY, in.	SPT-N BLOWS / ft.	WATER CONTENT, %	DRY UNIT WT pcf	UNCONFINED COMPRESSION, psf	TESTS
3" Topsoil					PA						
SILT reddish-brown, very loose to loose			ML	1	SS	18	5	13			
			ML	2	SS	18	3	21			S-2 P200=94%
					PA						
		5	ML	3	SS	18	4	22			
					PA						
			ML	4	SS	18	5	10			
		10			PA						
14	585.5										
POORLY GRADE SAND fine grained, with silt, brown, loose to medium dense			SP SM	5	SS	18	17	9			
		15			WB						
			SP SM	6	SS	18	6	16			S-6 P200=6%
		20			WB						
			SP SM	7	SS	18	17	23			
		25			WB						
28.5	571										
SHALEY LEAN CLAY with sand, dark gray and brown, very stiff			CL	8	SS	18	30	14			
		30			WB						

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	None	WD	17 AB
WL			
WL			

Terracon

BORING STARTED	4-21-09		
BORING COMPLETED	4-21-09		
RIG	ATV	FOREMAN	TS
APPROVED	CSK	JOB #	04095060

LOG OF BORING NO. B-1

CLIENT: City of Bixby

SITE: Bixby, Oklahoma

PROJECT: Proposed Mingo Road Water Line

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WT pcf	TESTS: UNCONFINED COMPRESSION, psf	
	33 — 566.5										
	SHALE+ dark gray, soft to moderately hard			9	SS WB	4	50/4"	11			
		35									
				10	SS WB	4	50/4"	18			
		40									
				11	SS	3	50/3"	20			
				12	DB	60					S-12 REC=100% RQD=48%
		45									
				13	DB	60					S-13 REC=100% RQD=23%
		50									
				14	DB	60					S-14 REC=100% RQD=60%
		55									
				15	DB	53					S-15 REC=88% RQD=42%
	+Classification estimated from disturbed and/or core samples. Petrographic analysis may reveal other rock types.	60									
	63.5 — 536										
	BOTTOM OF BORING										

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ None	WD ▼ 17	AB
WL	▼	▽	
WL			

Terracon

BORING STARTED			4-21-09
BORING COMPLETED			4-21-09
RIG	ATV	FOREMAN	TS
APPROVED	CSK	JOB #	04095060

LOG OF BORING NO. B-2

CLIENT	
City of Bixby	
SITE **Bixby, Oklahoma**	PROJECT **Proposed Mingo Road Water Line**

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WT pcf	TESTS: UNCONFINED COMPRESSION, psf	
	Not drilled as part of this exploration due to high water level in Arkansas River										

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			Terracon	BORING STARTED	
WL	▽	▼		BORING COMPLETED	
WL	▼	▽		RIG	FOREMAN
WL				APPROVED CSK	JOB # 04095060

LOG OF BORING NO. B-3

CLIENT **City of Bixby**	
SITE **Bixby, Oklahoma**	PROJECT **Proposed Mingo Road Water Line**

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WT pcf	TESTS: UNCONFINED COMPRESSION, psf	
	Not drilled as part of this exploration due to high water level in Arkansas River										

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL		
WL		
WL		

Terracon

BORING STARTED	
BORING COMPLETED	
RIG	FOREMAN
APPROVED CSK	JOB # 04095060

LOG OF BORING NO. B-4

CLIENT	
City of Bixby	
SITE Bixby, Oklahoma	PROJECT Proposed Mingo Road Water Line

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WT pcf	TESTS: UNCONFINED COMPRESSION, psf	
	Not drilled as part of this exploration due to high water level in Arkansas River										

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			BORING STARTED	
WL			BORING COMPLETED	
WL			RIG	FOREMAN
WL			APPROVED CSK	JOB # 04095060

LOG OF BORING NO. B-5

CLIENT
City of Bixby

SITE
Bixby, Oklahoma

PROJECT
Proposed Mingo Road Water Line

DESCRIPTION (Approx. Surface Elev.: 598.0 ft)	Depth / Elev.	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WT pcf	UNCONFINED COMPRESSION, psf	
3" Topsoil Fill: **SAND** with gravel and brick fragments, brown	2 / 596				PA						
				1	SS	12	9/6"	7			
					PA		50/4"				
Fill: **FAT CLAY** with asphalt fragments, brown and gray	6.5 / 591.5			2	SS	18	2	17			
					PA						
		5		3	SS	18	5	19			
					PA						
CLAYEY SAND fine to medium grained, brown, loose to medium dense	18.5 / 579.5		SC	4	SS	18	4	18			S-4 P200=31%
		10			PA						
			SC	5	SS	18	16	14			
		15			WB						
WELL GRADED SAND medium to coarse grained, brown, loose to medium dense	27.5 / 570.5		SW	6	SS	18	10	15			
		20			WB						
			SW	7	SS	18	7	18			S-7 P200=2%
		25			WB						
SHALE+ dark gray, moderately hard				8	SS	4	50/4"	8			
		30			PA						

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ None	WD	▼ 16 AB
WL	▼		∇
WL			

Terracon

BORING STARTED			4-22-09
BORING COMPLETED			4-22-09
RIG	ATV	FOREMAN	TS
APPROVED	CSK	JOB #	04095060

LOG OF BORING NO. B-5

CLIENT: **City of Bixby**

SITE: **Bixby, Oklahoma**

PROJECT: **Proposed Mingo Road Water Line**

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WT pcf	TESTS: UNCONFINED COMPRESSION, psf
	SHALE+ dark gray, moderately hard			9	SS WB	3	50/4"	16		
		35								
				10	SS WB	3	50/3"	15		
		40								
				11	SS WB	3	50/3"	15		
		45								
				12	SS WB	2	50/2"	15		
		50								
				13	SS WB	2	50/2"			
		55								
	58.7 … 539.5			14	SS	1	50/2"	16		

BOTTOM OF BORING

+Classification estimated from disturbed samples. Core samples and petrographic analysis may reveal other rock types.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ None	WD ▼ 16	AB
WL	▼	∇	
WL			

Terracon

BORING STARTED			4-22-09
BORING COMPLETED			4-22-09
RIG	ATV	FOREMAN	TS
APPROVED	CSK	JOB #	04095060

LOG OF BORING NO. B-6

CLIENT City of Bixby	
SITE Bixby, Oklahoma	PROJECT Proposed Mingo Road Water Line

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WT pcf	TESTS: UNCONFINED COMPRESSION, psf
	3" Topsoil				PA					
	SILTY SAND reddish-brown, loose		SM	1	SS	18	7	21		
			SM	2	SS	18	9	22		
					PA					
5	**SAND** fine grained, tan, medium dense	5	SP	3	SS	18	16	7		
					PA					
			SP	4	SS	18	10	10		
		10			PA					
13.5	**SAND** fine to medium grained, brown, loose		SP	5	SS	18	5	16		
15	BOTTOM OF BORING	15								

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ 12	WD ▼ 12	AB
WL	▼	∇	
WL			

BORING STARTED			4-23-09
BORING COMPLETED			4-23-09
RIG	ATV	FOREMAN	TS
APPROVED	CSK	JOB #	04095060

Terracon

LOG OF BORING NO. B-7

CLIENT
City of Bixby

SITE
Bixby, Oklahoma

PROJECT
Proposed Mingo Road Water Line

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WT pcf	TESTS: UNCONFINED COMPRESSION, psf
	3" Topsoil **SILTY SAND** fine grained, reddish-brown, loose to medium dense				PA					
			SM	1	SS	18	4	25		
			SM	2	SS	18	6	23		
					PA					
		5	SM	3	SS	18	7	22		
					PA					
	▽ ▼		SM	4	SS	18	10	20		
		10			PA					
13.5	**SAND** fine to medium grained, brown, very loose		SP	5	SS	18	3	16		
15	BOTTOM OF BORING	15								

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 8	WD ▼ 9	AB
WL	▽	▽	
WL			

Terracon

BORING STARTED			4-23-09
BORING COMPLETED			4-23-09
RIG	ATV	FOREMAN	TS
APPROVED	CSK	JOB #	04095060

LOG OF BORING NO. B-8

CLIENT	
City of Bixby	
SITE	PROJECT
Bixby, Oklahoma	Proposed Mingo Road Water Line

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WT pcf	TESTS: UNCONFINED COMPRESSION, psf
	3" Topsoil **SILTY LEAN CLAY** brown, very soft to medium stiff				PA					
			CL ML	1	SS	18	5	20		
			CL ML	2	SS	18	4	20		
					PA					
		5	CL ML	3	SS	18	2	20		
					PA					
8.5	**FAT CLAY** with sand, brown and gray, medium stiff		CH	4	SS	18	5	21		
		10			PA					
	▼									
			CH	5	SS	18	5	21		
15	BOTTOM OF BORING	15								

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 12	WD ▼ 12	AB
WL	▼	▽	
WL			

Terracon

BORING STARTED			4-23-09
BORING COMPLETED			4-23-09
RIG	ATV	FOREMAN	TS
APPROVED	CSK	JOB #	04095060

LOG OF BORING NO. B-9

CLIENT
City of Bixby

SITE
Bixby, Oklahoma

PROJECT
Proposed Mingo Road Water Line

Depth (graphic log)	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WT pcf	UNCONFINED COMPRESSION, psf
	1" Topsoil				PA					
2	**SILTY LEAN CLAY** brown, medium stiff		CL ML	1	SS	18	4	21		
5	**FAT CLAY** dark brown, medium stiff		CH	2	SS	18	6	20		
					PA					
8.5	**LEAN CLAY** reddish-brown, medium stiff	5	CL	3	SS	12	5	21		
					PA					
13.5	**FAT CLAY** reddish-brown and grayish-brown, stiff		CH	4	SS	18	7	21		
		10			PA					
15	**CLAYEY SAND** fine grained, reddish-brown, loose		SC	5	SS	18	5	26		
	BOTTOM OF BORING	15								

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ 13.5	WD	▼ None AB
WL	▼		∇
WL			

Terracon

BORING STARTED			4-24-09
BORING COMPLETED			4-24-09
RIG	ATV	FOREMAN	TS
APPROVED	CSK	JOB #	04095060

LOG OF BORING NO. B-10

CLIENT	
City of Bixby	
SITE: Bixby, Oklahoma	PROJECT: Proposed Mingo Road Water Line

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES RECOVERY, in.	SAMPLES SPT-N BLOWS / ft.	TESTS WATER CONTENT, %	TESTS DRY UNIT WT pcf	TESTS UNCONFINED COMPRESSION, psf
	3" Topsoil Fill: **FAT CLAY** with gravel, dark brown				PA					
				1	SS	12	6	20		
2	**FAT CLAY** dark brown and reddish-brown, medium stiff		CH	2	SS	18	5	24		
					PA					
		5	CH	3	SS	18	6	23		
					PA					
8.5	**FAT CLAY** brown, reddish-brown and gray, stiff		CH	4	SS	18	10	20		
		10			PA					
			CH	5	SS	18	9	27		
15	BOTTOM OF BORING	15								

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ None	WD ▼ None	AB
WL	▽	▽	
WL			

Terracon

BORING STARTED			4-24-09
BORING COMPLETED			4-24-09
RIG	ATV	FOREMAN	TS
APPROVED	CSK	JOB #	04095060

BOREHOLE 04095060.GPJ TERRACON.GDT 5/5/09

LOG OF BORING NO. B-11

CLIENT
City of Bixby

SITE
Bixby, Oklahoma

PROJECT
Proposed Mingo Road Water Line

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WT pcf	TESTS: UNCONFINED COMPRESSION, psf
	3" Topsoil				PA					
	FAT CLAY dark brown, medium stiff to stiff		CH	1	SS	12	4	29		
			CH	2	SS	18	5	27		
					PA					
		5	CH	3	SS	18	7	28		
					PA					
			CH	4	SS	18	10	24		
		10			PA					
13.5	**FAT CLAY** dark brown, reddish-brown, and grayish-brown, stiff		CH	5	SS	12	7	22		
15	BOTTOM OF BORING	15								

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ None	WD ▼ None	AB
WL			
WL			

Terracon

BORING STARTED			4-23-09
BORING COMPLETED			4-23-09
RIG	ATV	FOREMAN	TS
APPROVED	CSK	JOB #	04095060

LOG OF BORING NO. B-12

CLIENT: **City of Bixby**

SITE: **Bixby, Oklahoma**

PROJECT: **Proposed Mingo Road Water Line**

DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	SPT-N BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WT pcf	UNCONFINED COMPRESSION, psf
3" Topsoil **SILTY LEAN CLAY** dark brown and brown, very soft				PA					
		CL ML	1	SS	18	2	23		
		CL ML	2	SS	6	2	26		
5				PA					
FAT CLAY dark grayish-brown, medium stiff	5	CH	3	SS	18	6	23		
				PA					
		CH	4	SS	18	5	26		
13.5	10			PA					
FAT CLAY brown, reddish-brown, and gray, medium stiff		CH	5	SS	18	4	20		
15	15								
BOTTOM OF BORING									

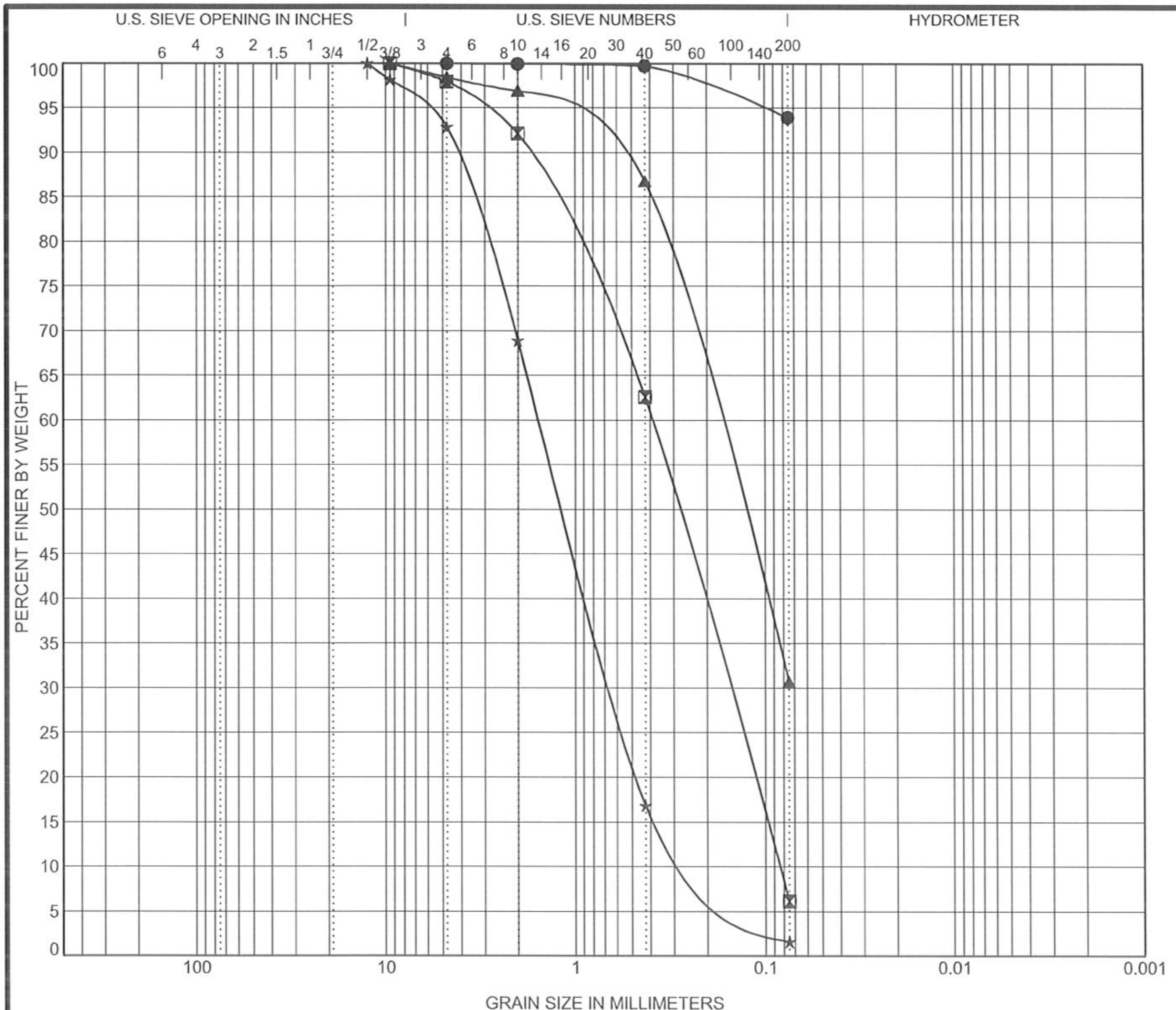
The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 8	WD ▼ 4	AB
WL	▼	▽	
WL			

Terracon

BORING STARTED	4-23-09
BORING COMPLETED	4-23-09
RIG ATV	FOREMAN TS
APPROVED CSK	JOB # 04095060

BOREHOLE 04095060.GPJ TERRACON.GDT 5/5/09



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	B-1	2.0ft	SILT (SM)					
⊠	B-1	18.5ft	POORLY-GRADED SAND WITH SILT (SP-SM)				0.74	4.64
▲	B-5	8.5ft	CLAYEY SAND (SC)					
★	B-5	23.5ft	WELL-GRADED SAND(SW)				1.32	7.85

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	2.0ft	4.75				0	6	94	
⊠	B-1	18.5ft	9.5	0.393	0.156	0.085	2	92	6	
▲	B-5	8.5ft	9.5	0.186			2	68	31	
★	B-5	23.5ft	12.5	1.534	0.629	0.195	7	91	2	

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GRAIN SIZE DISTRIBUTION

Project: Proposed Mingo Road Water Line
Site: Bixby, Oklahoma
Job #: 04095060
Date: 5-5-09

MODIFIED TC GRAIN SIZE 04095060.GPJ TERRACON.GDT 5/5/09

GENERAL NOTES

DRILLING & SAMPLING SYMBOLS:

SS:	Split Spoon - 1-3/8" I.D., 2" O.D., unless otherwise noted	HS:	Hollow Stem Auger
ST:	Thin-Walled Tube - 2" O.D., unless otherwise noted	PA:	Power Auger
RS:	Ring Sampler - 2.42" I.D., 3" O.D., unless otherwise noted	HA:	Hand Auger
DB:	Diamond Bit Coring - 4", N, B	RB:	Rock Bit
BS:	Bulk Sample or Auger Sample	WB:	Wash Boring or Mud Rotary

The number of blows required to advance a standard 2-inch O.D. split-spoon sampler (SS) the last 12 inches of the total 18-inch penetration with a 140-pound hammer falling 30 inches is considered the "Standard Penetration" or "N-value".

WATER LEVEL MEASUREMENT SYMBOLS:

WL:	Water Level	WS:	While Sampling
WCI:	Wet Cave in	WD:	While Drilling
DCI:	Dry Cave in	BCR:	Before Casing Removal
AB:	After Boring	ACR:	After Casing Removal

Water levels indicated on the boring logs are the levels measured in the borings at the times indicated. Groundwater levels at other times and other locations across the site could vary. In pervious soils, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of groundwater levels may not be possible with only short-term observations.

DESCRIPTIVE SOIL CLASSIFICATION: Soil classification is based on the Unified Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

CONSISTENCY OF FINE-GRAINED SOILS

Unconfined Compressive Strength, Qu, psf	Standard Penetration or N-value (SS) Blows/Ft.	Consistency
< 500	<2	Very Soft
500 – 1,000	2-3	Soft
1,001 – 2,000	4-6	Medium Stiff
2,001 – 4,000	7-12	Stiff
4,001 – 8,000	13-26	Very Stiff
8,000+	26+	Hard

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Standard Penetration or N-value (SS) Blows/Ft.	Relative Density
0 – 3	Very Loose
4 – 9	Loose
10 – 29	Medium Dense
30 – 49	Dense
50+	Very Dense

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 – 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300mm)
Cobbles	12 in. to 3 in. (300mm to 75 mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm)
Silt or Clay	Passing #200 Sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 – 12
Modifiers	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1-10
Medium	11-30
High	30+

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GENERAL NOTES

Sedimentary Rock Classification

DESCRIPTIVE ROCK CLASSIFICATION:

Sedimentary rocks are composed of cemented clay, silt and sand sized particles. The most common minerals are clay, quartz and calcite. Rock composed primarily of calcite is called limestone; rock of sand size grains is called sandstone, and rock of clay and silt size grains is called mudstone or claystone, siltstone, or shale. Modifiers such as shaly, sandy, dolomitic, calcareous, carbonaceous, etc. are used to describe various constituents. Examples: sandy shale; calcareous sandstone.

LIMESTONE	Light to dark colored, crystalline to fine-grained texture, composed of $CaCo_3$, reacts readily with HCl.
DOLOMITE	Light to dark colored, crystalline to fine-grained texture, composed of $CaMg(CO_3)_2$, harder than limestone, reacts with HCl when powdered.
CHERT	Light to dark colored, very fine-grained texture, composed of micro-crystalline quartz ($Si0_2$), brittle, breaks into angular fragments, will scratch glass.
SHALE	Very fine-grained texture, composed of consolidated silt or clay, bedded in thin layers. The unlaminated equivalent is frequently referred to as siltstone, claystone or mudstone.
SANDSTONE	Usually light colored, coarse to fine texture, composed of cemented sand size grains of quartz, feldspar, etc. Cement usually is silica but may be such minerals as calcite, iron-oxide, or some other carbonate.
CONGLOMERATE	Rounded rock fragments of variable mineralogy varying in size from near sand to boulder size but usually pebble to cobble size (½ inch to 6 inches). Cemented together with various cementing agents. Breccia is similar but composed of angular, fractured rock particles cemented together.

PHYSICAL PROPERTIES:

DEGREE OF WEATHERING

Slight	Slight decomposition of parent material on joints. May be color change.
Moderate	Some decomposition and color change throughout.
High	Rock highly decomposed, may be extremely broken.

HARDNESS AND DEGREE OF CEMENTATION

Limestone and Dolomite:

Hard	Difficult to scratch with knife.
Moderately Hard	Can be scratched easily with knife, cannot be scratched with fingernail.
Soft	Can be scratched with fingernail.

Shale, Siltstone and Claystone

Hard	Can be scratched easily with knife, cannot be scratched with fingernail.
Moderately Hard	Can be scratched with fingernail.
Soft	Can be easily dented but not molded with fingers.

Sandstone and Conglomerate

Well Cemented	Capable of scratching a knife blade.
Cemented	Can be scratched with knife.
Poorly Cemented	Can be broken apart easily with fingers.

BEDDING AND JOINT CHARACTERISTICS

Bed Thickness	Joint Spacing	Dimensions
Very Thick	Very Wide	>10′
Thick	Wide	3′ - 10′
Medium	Moderately Close	1′ - 3′
Thin	Close	2″ - 1′
Very Thin	Very Close	.4″ - 2″
Laminated	—	.1″ - .4″

Bedding Plane	A plane dividing sedimentary rocks of the same or different lithology.
Joint	Fracture in rock, generally more or less vertical or transverse to bedding, along which no appreciable movement has occurred.
Seam	Generally applies to bedding plane with an unspecified degree of weathering.

SOLUTION AND VOID CONDITIONS

Solid	Contains no voids.
Vuggy (Pitted)	Rock having small solution pits or cavities up to ½ inch diameter, frequently with a mineral lining.
Porous	Containing numerous voids, pores, or other openings, which may or may not interconnect.
Cavernous	Containing cavities or caverns, sometimes quite large.

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UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification	
				Group Symbol	Group Name[B]
Coarse Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]	GW	Well-graded gravel[F]
			$Cu < 4$ and/or $1 > Cc > 3$[E]	GP	Poorly graded gravel[F]
		Gravels with Fines More than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel[F,G,H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]	SW	Well-graded sand[I]
			$Cu < 6$ and/or $1 > Cc > 3$[E]	SP	Poorly graded sand[I]
		Sands with Fines More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G,H,I]
			Fines Classify as CL or CH	SC	Clayey sand[G,H,I]
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K,L,M]
		organic	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay[K,L,M,N]
					Organic silt[K,L,M,O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	Fat clay[K,L,M]
			PI plots below "A" line	MH	Elastic Silt[K,L,M]
		organic	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay[K,L,M,P]
					Organic silt[K,L,M,Q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10} \quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains $\geq$ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains $\geq$ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains $\geq$ 30% plus No. 200 predominantly sand, add "sandy" to group name.

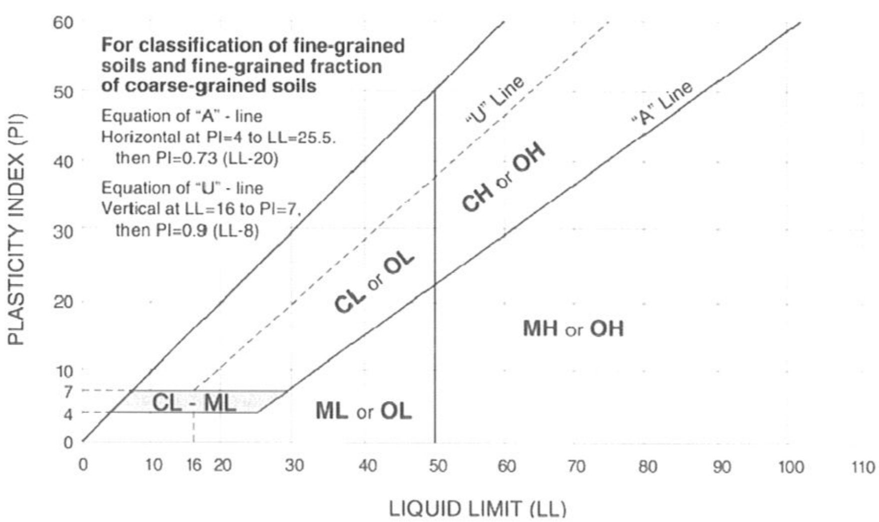
[M] If soil contains $\geq$ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] $PI \geq 4$ and plots on or above "A" line.

[O] $PI < 4$ or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



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Attachment 4

Evaluate Sliding and Overturning
Fixed Crest

CH2MHILL

PROJECT : Arkansas River Corridor Project - Sand Springs Dam
PROJECT #: 657971.04.02.01
CREATED BY: Jen Schaeffer/SEA **DATE:** 04/13/2015
REVIEWED BY: Mark Kacmarcik/CVO **DATE:** 04/14/2015

Given: Simplified gravity dam geometry shown and generalized subsurface profile. See sketch.

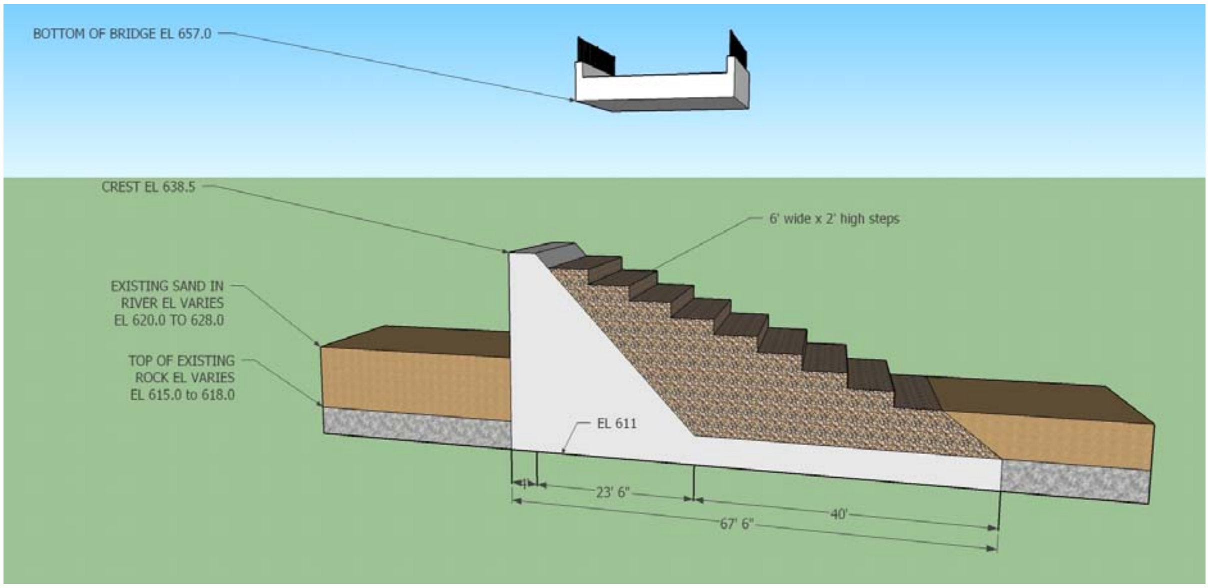
Find: Check sliding and overturning against USACE criteria for Static and Seismic cases as noted in the title. Anchor forces are included as needed to meet stability criteria. Note that this is not attempt to be a complete comprehensive check of all possible analysis cases, but rather the loading cases which are assumed to control overall dam design for preliminary sizing and concept evaluation.

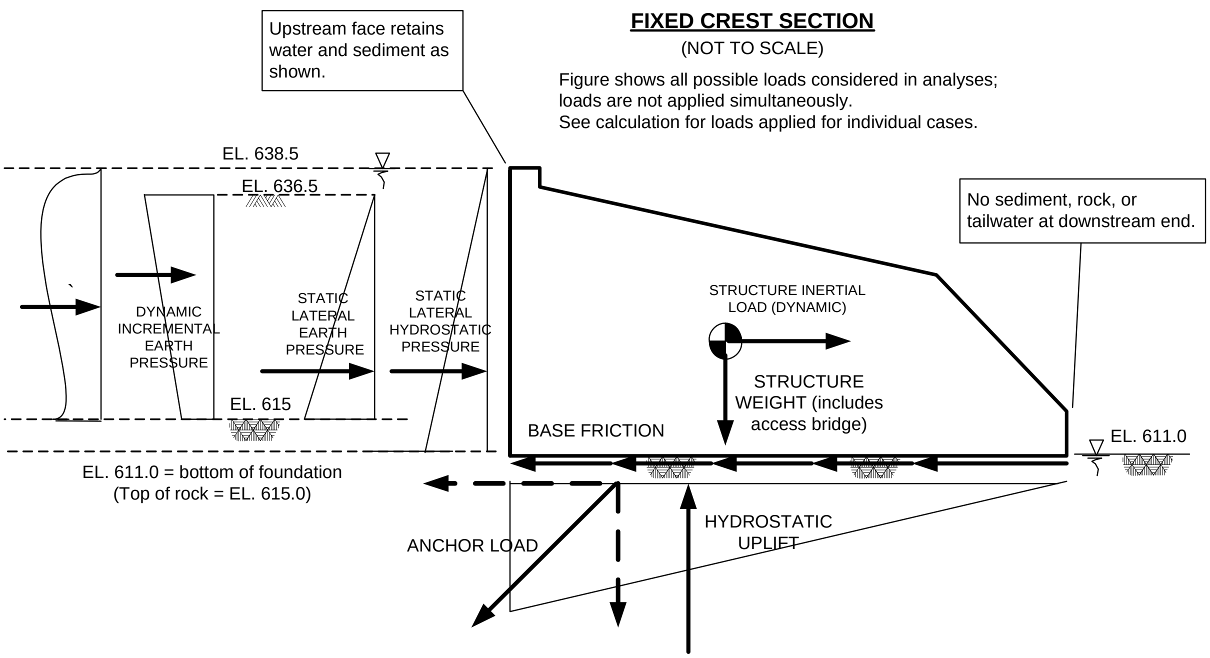
Assumptions:
Ignore resistance from sediment or rock on downstream toe.
Ice loading is not considered.
Structure is not undermined by scour
Upstream and downstream turndowns (not shown) are not relied upon for shear resistance.
All soil and rock layers are assumed to be horizontal.
Use single conservative frictional interface strength, as shown in the calculation.
Disregard cohesion for long term analysis.
Mass or contributions of pedestrian bridge ignored (conservative)
2 dimensional analysis considering dam geometry on a per-foot basis, 3Dimensional end effects not considered.
Steps shown in geometry are concrete or cut stone with similar unit weight to mass concrete.
Other assumptions as noted in the calculation

Inputs:
Approximate top of rock elevation for main dam, estimated at **El 615 ft**.
Dam foundation elevation assumed 4 feet below top of rock **(El 611 ft.)**
Water present to top of fixed crest at **El 638.5 ft**.
Sediment elevation varies from top of rock **(El 615 ft)** to 2 feet below top of crest **(El 636.5 ft)** as directed by Murry Fleming.
Tailwater elevation is coincident with dam foundation, **El 611.0 ft**.
2008 boreholes by Stantec used to estimate subsurface conditions and properties.
Other inputs as noted in the calculation.

References:
USACE EM 1110-2-2200 Gravity Dam Manual
USACE EM 1110-2-2100 Stability Analysis of Concrete Structures

Fixed Crest Section Geometry:





Define Geometry:

$El_{crest} := 638.5ft$ — Elevation at top of gravity dam

$El_{rock} := 615ft$ — Elevation of top of rock (shale)

$d_{excav} := 4ft$ — Excavate below top of rock to remove weathered shale

$El_{foundation} := El_{rock} - d_{excav} = 611 \cdot ft$ — Elevation of bottom of dam

$H_{dam} := El_{crest} - El_{foundation} = 27.5 \cdot ft$ — Total height of dam

$w_{foundation} := 67.5ft$ — Given width of dam base

$El_{sed.top} := El_{crest} - 2ft = 636.5 \cdot ft$

Dam collects sediment to within 2 feet of crest elevation.

$El_{water.US} := El_{crest} = 638.5 \cdot ft$

Elevation of water upstream of dam.

$El_{water.DS} := El_{foundation} = 611 \cdot ft$

Elevation of water downstream of dam (assume no water as recommended by USACE)

Material Properties:

Unit Weight:

$\gamma_{conc} := 150pcf$

Unit weight of concrete (assumed)

$\gamma_{sed} := 120pcf$

Unit weight of sediment against upstream face of dam (recommended by USACE EM 1110-2-2100)

$\gamma_{shale} := 152pcf$

Unit weight of Shale from Stantec, 2008 laboratory test results.

$\gamma_{w} := 62.4pcf$

Unit weight of water (assumed)

Shear Strength:

$\phi_{sed} := 28deg$ $c_{sed} := 0psf$

Effective stress shear strength of sediment.

Interface Strength (sliding):

$\delta_{base} := 24deg$

Consider only one sliding interface, mass concrete cast against shale bedrock. Assume no cohesion/adhesion along this interface, only base friciton. Typical value from NAVFAC DM7.2 for "Mass concrete cast against...very stiff and hard residual or preconsolidated clay"

Seismic:

$PGA_{OBE} := 0.009$ Peak ground acceleration on rock for Operations Basis Earthquake (OBE). 50% probability of exceedance in 100 years.

$PGA_{MCE} := 0.088$ Peak ground acceleration on rock for Maximum Credible Earthquake (MCE). 10% probability of exceedance in 50 years

$F_{PGA.scC} := 1.2$ Site coefficient for Site Class C, "Very Dense Soil and Soft Rock" (assumed).

$k_{h.MCE} := \frac{2}{3} \cdot PGA_{MCE} \cdot F_{PGA.scC} = 0.07$ Seismic coeff for MCE case (per EM 1110-2-2100 = 2/3 effective peak ground accel). Conservatively estimated using PGA for site class C.

$k_{h.OBE} := \frac{2}{3} \cdot PGA_{OBE} \cdot F_{PGA.scC} = 0.007$ Seismic coeff for OBE case.

$k_v := 0$ Neglect vertical component of earthquake accelleration (assumed).

Estimate Weight of Concrete Gravity Dam:

Estimate total (non buoyant) weight of concrete gravity dam by estimating area of the gravity dam polygon, and then multiplying it by the unit weight of the material. Use centroid function to for irregular dam geometry.

Centroid of polygon [edit] from Wikipedia (http://en.wikipedia.org/wiki/Polygon, February 27, 2014)

The centroid of a non-self-intersecting closed polygon defined by n vertices (x_0,y_0), (x_1,y_1), ..., (x_{n-1},y_{n-1}) is the point (C_x, C_y), where

$$C_x = \frac{1}{6A}\sum_{i=0}^{n-1}(x_i + x_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

$$C_y = \frac{1}{6A}\sum_{i=0}^{n-1}(y_i + y_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

and where A is the polygon's signed area,

$$A = \frac{1}{2}\sum_{i=0}^{n-1}(x_i\ y_{i+1} - x_{i+1}\ y_i).$$ [9]

In these formulas, the vertices are assumed to be numbered in order of their occurrence along the polygon's perimeter, and the vertex (x_n, y_n) is assumed to be the same as (x_0, y_0). Note that if the points are numbered in clockwise order the area A, computed as above, will have a negative sign; but the centroid coordinates will be correct even in this case.

Define function to calculate area of polygon whose plane coordinates are contained in matrix XY

$$\text{Area}(XY) := \left| \begin{array}{l} XY \leftarrow \text{stack}\left[XY, \left(XY^T\right)^{\langle 0 \rangle T}\right] \\ M \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left|\text{submatrix}(XY, i, i+1, 0, 1)\right| \\ 0.5 \cdot M \end{array} \right.$$

Define function to calculate coordinates of centroid of non-intersecting closed polygon

$$\text{Centroid}(XY) := \left| \begin{array}{l} XY \leftarrow \text{stack}\left[XY, \left(XY^{T}\right)^{\langle 0 \rangle^{T}}\right] \\ x \leftarrow XY^{\langle 0 \rangle} \\ y \leftarrow XY^{\langle 1 \rangle} \\ C_x \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left[\left(x_i + x_{i+1}\right)\cdot\left(x_i\cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ C_y \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left[\left(y_i + y_{i+1}\right)\cdot\left(x_i\cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ \left(C_x \ \ C_y\right)\cdot\dfrac{1}{6\cdot\text{Area}(XY)} \end{array} \right.$$

Area and Centroid of Concrete Gravity Dam

$$XY_{dam} := \begin{pmatrix} 0 & 611.0 \\ 0 & 638.5 \\ 4 & 638.5 \\ 6 & 636.5 \\ 60 & 620.5 \\ 67.5 & 615 \\ 67.5 & 611.0 \end{pmatrix}$$

- Values define cross-sectional geometry of dam, points are clockwise around cross section, starting at upstream heel.
- Left column is X coordinates, "0" is the upstream heel of the dam, sign convention is positive to the right (downstream).
- Right column is elevation.

$-\text{Area}\left(XY_{dam}\right) = 1158.625$ cross sectional area of dam section

$\text{Centroid}\left(XY_{dam}\right) = (26.741 \quad 620.701)$ coordinates of center of gravity of concrete gravity dam, ft

$x_{dam\_CG} := \text{Centroid}\left(XY_{dam}\right)_{0,0} = 26.741$

$x_{dam} := x_{dam\_CG} \cdot 1\text{ft} = 26.741 \cdot \text{ft}$ X-coordinate fo centroid, in feet

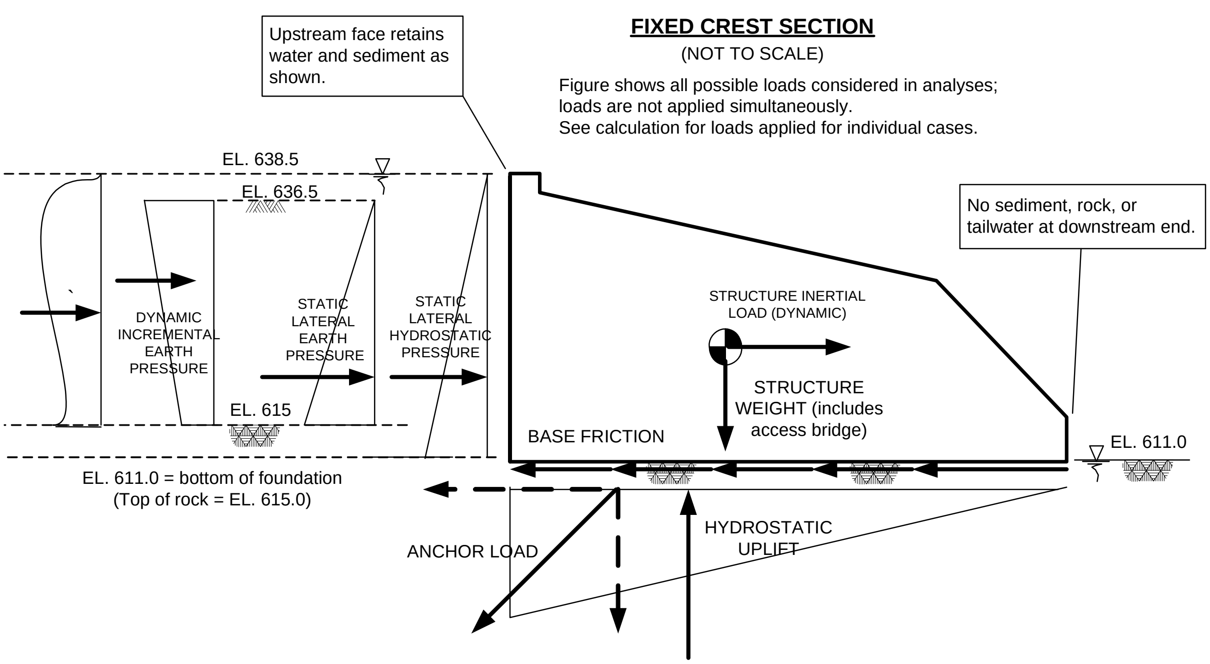
$y_{dam} := \text{Centroid}\left(XY_{dam}\right)_{0,1} = 620.701$

$El_{centroid} := y_{dam} \cdot 1\text{ft} = 620.701 \cdot \text{ft}$ Elevation of centroid

$$WT_{dam} := -\text{Area}\left(XY_{dam}\right) \cdot \text{ft}^2 \cdot \gamma_{conc} + 5\text{klf} = 178.8 \cdot \frac{\text{kip}}{\text{ft}}$$

Multiply cross-sectional area by unit weight of concrete to estimate total weight of concrete gravity dam, per lineal foot. Include weight of access bridge, estimated as 5klf per Kevin Whittier.

Estimate Lateral Driving Forces Acting on Concrete Gravity Dam



Lateral Hydrostatic Water Load on Upstream Face:

$H_w := El_{water.US} - El_{foundation} = 27.5 \cdot ft$ Height of water

$F_{h2o} := \frac{1}{2} \cdot \gamma_w \cdot H_w^{\ 2} = 23.6 \cdot \frac{kip}{ft}$ Magnitude of resultant of hydrostatic load on upstream face of dam

$El_{h2o} := El_{water.US} - \frac{2}{3} \cdot H_w = 620.2 \cdot ft$ Elevation of resultant

Static At-Rest Lateral Earth Pressure on Upstream Face:

Assume sediment contributesr at-rest soil pressure on upstream face of dam (active pressures are not developed).

$H_{ko} := El_{sed.top} - El_{rock} = 21.5 \cdot ft$ Maximum sediment accumulation extends from top of rock to 2 feet below fixed crest. Assume no lateral earth pressure from silt below top of rock.

$K_0 := 1 - \sin(\phi_{sed}) = 0.531$ At-rest lateral earth pressure coefficient.

$F_{ko} := \frac{1}{2} \cdot K_0 \cdot (\gamma_{sed} - \gamma_w) \cdot H_{ko}^{\ 2} = 7.06 \cdot \frac{kip}{ft}$ Magnitude of resultant of at-rest soil pressure on upstream face of dam (use buoyant unit weight)

$El_{ko} := El_{sed.top} - \frac{2}{3} \cdot H_{ko} = 622.2 \cdot ft$ Elevation of resultant.

Lateral Hydrodynamic Water Load on Upstream Face:

This load is applied assuming the dam has been flushed of sediment, and full height of water applies hydrodynamic loading to dam structure during a seismic event. Note that, when sediment levels accumulate, hydrodynamic loading is not considered to b a valid case.

$P_{hydro.MCE} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 1.415 \cdot klf$ Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$P_{hydro.OBE} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 0.145 \cdot klf$ Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$El_{hydro.MCE} := El_{rock} + 0.4 \cdot (El_{crest} - El_{rock}) = 624.4 \cdot ft$ Elevation of resultant of hydrodynamic load.

$El_{hydro.OBE} := El_{rock} + 0.4 \cdot (El_{crest} - El_{rock}) = 624.4 \cdot ft$ Elevation of resultant of hydrodynamic load.

SEISMIC: Lateral Earth Pressures Upstream Face:

$\theta_{wall} := 0deg$ Slope of upstream face of dam, 0 indicates vertical face

$\delta_{sed} := 0deg$ Interface friction angle between sediment and dam, assume zero degrees.

$\beta_{US} := 0deg$ Slope of top of sediment against upstream face of dam. 0 degrees is horizontal.

Define function to calculate Coulomb active lateral earth pressure coefficient:

$$f\_K_{A.c}(\phi,\delta,\beta,\theta) := \frac{\cos(\phi-\theta)^2}{\cos(\theta)^2\cdot\cos(\delta+\theta)\cdot\left(1+\sqrt{\frac{\sin(\delta+\phi)\cdot\sin(\phi-\beta)}{\cos(\delta+\theta)\cdot\cos(\beta-\theta)}}\right)^2}$$

$K_A := f\_K_{A.c}\left(\phi_{sed},\delta_{sed},\beta_{US},\theta_{wall}\right) = 0.361$ Coulomb active lateral earth pressure coefficient.

$P_A := \frac{1}{2}\cdot K_A\cdot\left(\gamma_{sed}-\gamma_w\right)\left(El_{sed.top}-El_{rock}\right)^2 = 4.806\cdot klf$ Coulomb active lateral earth pressure.

$$f\_\psi\left(k_h,k_v\right) := \mathrm{atan}\left(\frac{k_h}{1-k_v}\right)$$

Define function to calculate dynamic lateral earth pressure coefficient (KAE)

$$f\_K_{AE}(\phi,\delta,\beta,\theta,\psi) := \frac{\cos(\phi-\psi-\theta)^2}{\cos(\psi)\cdot\cos(\theta)^2\cdot\cos(\psi+\theta+\delta)\cdot\left(1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\psi-\beta)}{\cos(\delta+\psi+\theta)\cdot\cos(\beta-\theta)}}\right)^2}$$

Estimate Dynamic Lateral Earth Pressure for Maximum Credible Earthquake (MCE):

$\psi_{MCE} := f\_\psi\left(k_{h.MCE},k_v\right) = 0.07$

$K_{AE.MCE} := f\_K_{AE}\left(\phi_{sed},\delta_{sed},\beta_{US},\theta_{wall},\psi_{MCE}\right) = 0.406$

$P_{AE.MCE} := \frac{1}{2}K_{AE.MCE}\cdot\left(\gamma_{sed}-\gamma_w\right)\left(El_{sed.top}-El_{rock}\right)^2 = 5.407\cdot klf$ Total dynamic active earth pressure (static-active plus dynamic)

$\Delta P_{AE.MCE} := P_{AE.MCE}-P_A = 0.601\cdot klf$ Dynamic increment in MCE case.

Estimate Dynamic Lateral Earth Pressure for Operations Basis Earthquake (OBE):

$\psi_{OBE} := f\_\psi\left(k_{h.OBE},k_v\right) = 0.007$

$K_{AE.OBE} := f\_K_{AE}\left(\phi_{sed},\delta_{sed},\beta_{US},\theta_{wall},\psi_{OBE}\right) = 0.365$

$P_{AE.OBE} := \frac{1}{2}K_{AE.OBE}\cdot\left(\gamma_{sed}-\gamma_w\right)\left(El_{sed.top}-El_{rock}\right)^2 = 4.864\cdot klf$ Total dynamic active earth pressure (static-active plus dynamic)

$\Delta P_{AE.OBE} := P_{AE.OBE}-P_A = 0.058\cdot klf$ Additional applied earth pressure in seismic OBE case.

Compare Dynamic Lateral Earth Pressures to At-Rest Lateral Earth Pressures:

$P_{AE.OBE} = 4.864 \cdot klf$ Dynamic Active OBE

$P_{AE.MCE} = 5.407 \cdot klf$ Dynamic Active MCE

$F_{ko} = 7.063 \cdot klf$ Static At-Rest

Note that static at-rest loading is greater than dynamic active loading for both MCE and OBE cases. Use greater of static at-rest dynamic active lateral earth pressures. In this case, static at-rest pressure controls and should be used as the lateral earth pressure for the dynamic analysis cases..

Determine controlling load case for upstream loading on structure:

Structure couuld be free water (no sediment accumulation), or filled with sediment. For seismic stability evaluations, estimate controlling case: either hydrodynamic loading of silt-free dam or dynamic lateral earth pressure of silted-in dam.

$F_{ko} = 7.063 \cdot klf$ Lateral earth pressure loading (note that static at-rest is controlling case for seismic evaluation)

$F_{h2o} = 23.595 \cdot klf$ Hydrostatic pressure

$P_{hydro.MCE} = 1.415 \cdot klf$ Hydrodynamic pressure, MCE event

$P_{hydro.OBE} = 0.145 \cdot klf$ Hydrodynamic pressure, OBE event

$$check_{hydro.MCE} := \begin{cases} \text{"Hydrodynamic"} & \text{if } \left(P_{hydro.MCE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko}\right) \\ \text{"Soil"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{hydro.MCE} = \text{"Soil"}}$$

$$check_{hydro.OBE} := \begin{cases} \text{"Hydrodynamic"} & \text{if } \left(P_{hydro.OBE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko}\right) \\ \text{"Soil"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{hydro.OBE} = \text{"Soil"}}$$

SEISMIC: Inertial Load of Structure:

$F_{inertia.MCE} := k_{h.MCE} \cdot WT_{dam} = 12.587 \cdot klf$ Seismic inertia load of the dam for MCE, acts in downstream direction.

$F_{inertia.OBE} := k_{h.OBE} \cdot WT_{dam} = 1.287 \cdot klf$ Seismic inertia load of the dam for OBE acts in downstream direction.

$El_{inertia.MCE} := El_{centroid} = 620.701 \cdot ft$ Seismic inertial load acts through centroid of dam

$El_{inertia.OBE} := El_{centroid} = 620.701 \cdot ft$ Seismic inertial load acts through centroid of dam

Estimate Uplift Hydrostatic Forces Acting on Concrete Gravity Dam

Hydrostatic Uplift on Dam Base:

Magnitude of hydrostatic uplift is estimated as straightline interpolation between headwater and tailwater. Figure above shows uplift distribution below bottom of dam.

Use centroid equation to define uplift pressure.

$$XY_{uplift} := \begin{pmatrix} w_{foundation} & 0 \\ w_{foundation} & El_{water.DS} - El_{foundation} \\ 0 & El_{water.US} - El_{foundation} \\ 0 & 0 \end{pmatrix} \cdot ft^{-1}$$

$$Area\left(XY_{uplift}\right) = 928.125$$

$$Centroid\left(XY_{uplift}\right) = (22.5 \quad 9.167)$$

$$X_{uplift} := \left(1ft\,Centroid\left(XY_{uplift}\right)\right)_{0,0} = 22.5 \cdot ft$$

$$F_{uplift} := Area\left(XY_{uplift}\right) \cdot ft^2 \cdot \gamma_w = 57.915 \cdot \frac{kip}{ft}$$

Estimate Resisting Forces:

Estimate base sliding resistance for concrete gravity dam sliding on rock. Account for hydrostatic overburden above upstream face dam (if present):

Hydrostatic Overburden Volume above front slope of Dam:

$$F_{h2o.vert} := 0 \frac{kip}{ft}$$

This geometry has vertical face with no hydrostatic overburden.

$$x_{h2o.vert} := 0ft$$

Interface friction between concrete gravity dam and shale bedrock:

$$\delta_{base} = 24 \cdot deg$$

Base friction angle between dam and foundation.

$$F_{base} := \left(WT_{dam} + F_{h2o.vert} - F_{uplift}\right) \cdot \tan\left(\delta_{base}\right) = 53.8 \cdot \frac{kip}{ft}$$

Base friction, sum of vertical forces multiplied by tangent of interface friction times tangent of interface friction (delta).

Estimate Factor of Safety Against Sliding:

The recommended global stability design criteria is summarized in the USACE Gravity Dam Design EM 1110-2-2200. Stability criteria is summarized in Table 4-1 below.

EM 1110-2-2200
30 Jun 95

Table 4-1
Stability and stress criteria

Load Condition	Resultant Location at Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress	
				Compressive	Tensile
Usual	Middle 1/3	2.0	≤ allowable	$0.3\ f'_c$	0
Unusual	Middle 1/2	1.7	≤ allowable	$0.5\ f'_c$	$0.6\ f'^{2/3}_c$
Extreme	Within base	1.3	≤ 1.33 × allowable	$0.9\ f'_c$	$1.5\ f'^{2/3}_c$

Note: f'_c is 1-year unconfined compressive strength of concrete. The sliding factors of safety (*FS*) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions.

Static Sliding:

$FS_{min.static} := 2.0$ Usual loading. Minimum sliding factor of safety recommended by USACE (from table above)

$\Sigma F_{h.drive} := F_{h2o} + F_{ko} = 30.7 \cdot \frac{kip}{ft}$ Sum of driving forces (hydrostatic pressure + at rest lateral earth pressure)

$\Sigma F_{h.resist} := F_{base} = 53.8 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$FS_{slide.shale} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive}} = 1.76$ Factor of safety against sliding:

$$check_{slide.shale} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale} > FS_{min.static} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$check_{slide.shale} = \text{"NOT OK-anchors required"}$

MCE Sliding:

$FS_{min.MCE} := 1.3$ MCE, extreme loading. Minimum sliding factor of safety recommended by USACE (from table above).

$\Sigma F_{h.drive.MCE} := F_{h2o} + F_{ko} + F_{inertia.MCE} = 43.2 \cdot \frac{kip}{ft}$ Sum of lateral driving forces during MCE event

$\Sigma F_{h.resist} := F_{base} = 53.8 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$FS_{slide.shale.MCE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.MCE}} = 1.24$ Factor of safety against sliding

$$check_{slide.shale.MCE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.MCE} > FS_{min.MCE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.MCE} = \text{"NOT OK-anchors required"}$$

OBE Sliding:

$FS_{min.OBE} := 1.7$

OBE, Unusual loading. Minimum sliding factor of safety recommended by USACE (from table above).

$$\Sigma F_{h.drive.OBE} := F_{h2o} + F_{ko} + F_{inertia.OBE} = 31.9 \cdot \frac{kip}{ft}$$

Sum of lateral driving forces during MCE event

$$\Sigma F_{h.resist} := F_{base} = 53.8 \cdot \frac{kip}{ft}$$

Sum of resisting forces (base friction)

$$FS_{slide.shale.OBE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.OBE}} = 1.68$$

Factor of safety against sliding - seismic OBE:

$$check_{slide.shale.OBE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.OBE} > FS_{min.OBE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.OBE} = \text{"NOT OK-anchors required"}$$

Estimate Required Anchor Forces Based on FS against Sliding:

Static Case:

$$F_{anchor} := FS_{min.static} \cdot \Sigma F_{h.drive} - \Sigma F_{h.resist} = 7.497 \cdot klf$$

Use minimum FS against sliding to determine anchor forc This is the horizontal component of the anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed)

$$T_{anchor.static} := \frac{F_{anchor}}{\cos(\alpha_{anchor})} = 10.602 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam for static loading.

Seismic MCE:

$$F_{anchor.MCE} := FS_{min.MCE} \cdot \Sigma F_{h.drive.MCE} - \Sigma F_{h.resist} = 2.4 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of the anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.MCE} := \frac{F_{anchor.MCE}}{\cos(\alpha_{anchor})} = 3.394 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Seismic OBE:

$$F_{anchor.OBE} := FS_{min.OBE} \cdot \Sigma F_{h.drive.OBE} - \Sigma F_{h.resist} = 0.488 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of the anchor.

$$\alpha_{anchor} := 45deg$$

Angle of anchor installation measured from horizontal.

$$T_{anchor.OBE} := \frac{F_{anchor.OBE}}{\cos(\alpha_{anchor})} = 0.69 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Determine Critical Anchor Force for Design:

$$\boxed{T_{anchor.critical} := \max(T_{anchor.static}, T_{anchor.MCE}, T_{anchor.OBE}) = 10.602 \cdot klf}$$

Estimate Factor of Safety Against Overturning:

Sum moments around downstream toe. Note this is not directly comparable to USACE overturning criteria but useful as a quick che of stability, see estimation of overturning resultant and % base compression below.

Because static controls sliding stability, only examine static case.

$$\Sigma M_{toe.drive.static} := F_{ko} \cdot (El_{ko} - El_{foundation}) + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \dots = 2901.331 \cdot \frac{kip \cdot ft}{ft}$$
$$+ F_{uplift} \cdot (w_{foundation} - X_{uplift})$$

$$\Sigma M_{toe.resist} := WT_{dam} \cdot (w_{foundation} - x_{dam}) + F_{h2o.vert} \cdot x_{h2o.vert} = 7287.438 \cdot \frac{kip \cdot ft}{ft}$$

$$\boxed{FS_{overturning.static} := \frac{\Sigma M_{toe.resist}}{\Sigma M_{toe.drive.static}} = 2.51}$$

Factor of safety against overturning, static case.

There is no specified factor of safety provided by USACE against overturning. The USACE does recommend that for the Normal/Usual loading scenario, the overturning resultant should be located within the middle 1/3 of the base of the dam, and for the unusual loading scenario, the middle 1/2 of the dam.

Check Overturning Criteria:

Static Case:

Check that location of overturning resultant falls in middle 1/3 of base of concrete gravity dam (usual case)

$$\Sigma M_{toe.total} := \Sigma M_{toe.drive.static} - \Sigma M_{toe.resist} = -4386.108 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.total} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 120.879 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant} := \frac{-\Sigma M_{toe.total}}{\Sigma F_{vertical.total}} = 36.3 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the wall

$$\frac{1}{3} \cdot w_{foundation} = 22.5 \cdot ft$$

defines middle third of base

$$\frac{2}{3} \cdot w_{foundation} = 45 \cdot ft$$

defines middle third of base

$$check_{OT} := \left| \begin{array}{ll} \text{"OK"} & \text{if } \frac{1}{3} w_{foundation} \le X_{Resultant} \le \frac{2}{3} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right.$$

$$\boxed{check_{OT} = \text{"OK"}}$$

OBE Case:

Check that location of overturning resultant falls in middle 1/2 of base of concrete gravity dam (unusual case)

$$\begin{aligned} \Sigma M_{toe.drive.OBE} := {} & F_{ko} \cdot \left(El_{ko} - El_{foundation}\right) \ldots \\ & + F_{h2o} \cdot \left(El_{h2o} - El_{foundation}\right) \ldots \\ & + F_{uplift} \cdot \left(w_{foundation} - X_{uplift}\right) \ldots \\ & + F_{inertia.OBE} \cdot \left(El_{inertia.OBE} - El_{foundation}\right) \end{aligned} = 2913.819 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.OBE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 120.879 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant.OBE} := \frac{-\Sigma M_{toe.drive.OBE}}{\Sigma F_{vertical.total}} = -24.1 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the dam

$$\frac{1}{4} \cdot w_{foundation} = 16.9 \cdot ft$$

defines middle half of base

$$\frac{3}{4} \cdot w_{foundation} = 50.6 \cdot ft$$

defines middle half of base

$$check_{OT.OBE} := \left| \begin{array}{ll} \text{"OK"} & \text{if } \frac{1}{4} w_{foundation} \le X_{Resultant} \le \frac{3}{4} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right.$$

$$\boxed{check_{OT.OBE} = \text{"OK"}}$$

MCE Case:

Check that location of overturning resultant falls within base of concrete gravity dam (extreme case)

$$\begin{aligned}\Sigma M_{toe.drive.MCE} := {} & F_{ko} \cdot \left(El_{ko} - El_{foundation}\right) \dots \\ & + F_{h2o} \cdot \left(El_{h2o} - El_{foundation}\right) \dots \\ & + F_{uplift} \cdot \left(w_{foundation} - X_{uplift}\right) \dots \\ & + F_{inertia.MCE} \cdot \left(El_{inertia.MCE} - El_{foundation}\right)\end{aligned} \qquad = 3023.441 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.MCE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 120.879 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant.MCE} := \frac{-\Sigma M_{toe.drive.MCE}}{\Sigma F_{vertical.total}} = -25 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the dam

$0 \cdot w_{foundation} = 0 \cdot ft$ defines upstream edge of base

$1 \cdot w_{foundation} = 67.5 \cdot ft$ defines downstream edge of base

$$check_{OT.MCE} := \begin{vmatrix} \text{"OK"} \quad \text{if } 0 w_{foundation} \le X_{Resultant} \le 1 w_{foundation} \\ \text{"NOT OK"} \quad \text{otherwise} \end{vmatrix}$$

$$\boxed{check_{OT.MCE} = \text{"OK"}}$$

Remarks and Recapitulation:

- Calculation addresses sliding and overturning of the fixed crest section of Sand Springs Dam under anticipated static operating conditions, OBE seismic case, and MCE seismic case noted.
- For all cases, it is identified that permanent ground anchors are necessary for slidign stability.
- Anchors are not necessary for overturning stability
- The static case (usual loading) was found to control

Attachment 5

Evaluate Sliding and Overturning Crest Gate:

CH2MHILL

PROJECT : Arkansas River Corridor Project - Sand Springs Dam

PROJECT #: 657971.04.02.01

CREATED BY: **Mark Kacmarcik** **DATE:** 04/16/2015

REVIEWED BY: Jen Schaeffer **DATE:** 04/17/2015

Given: Simplified gravity dam geometry shown and generalized subsurface profile. See sketch.

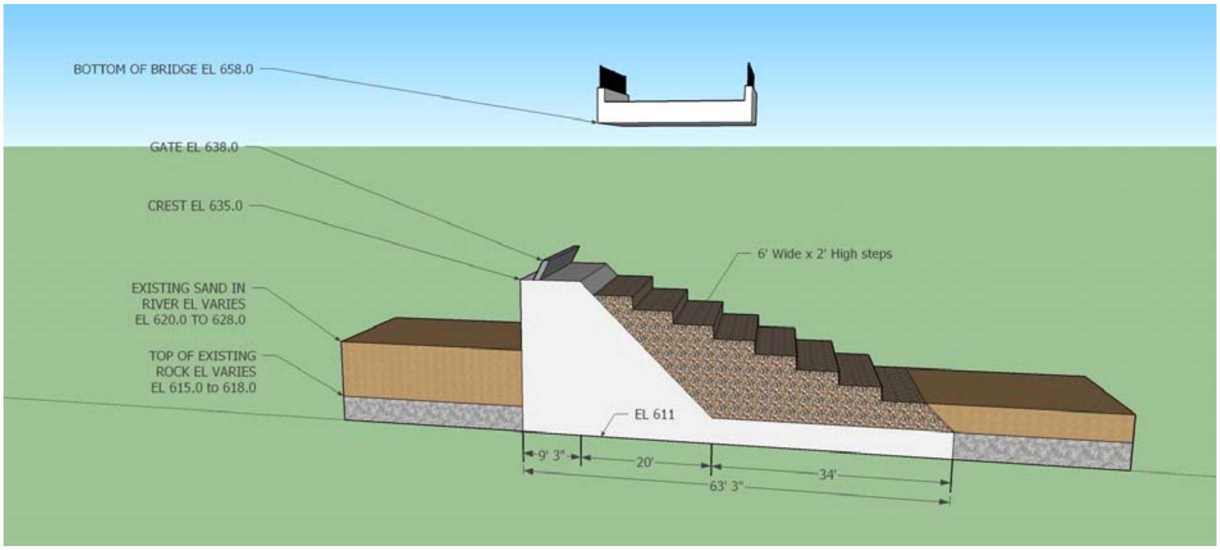
Find: Check sliding and overturning against USACE criteria for Static and Seismic cases as noted in the title. Anchor forces are included as needed to meet stability criteria. Note that this is not attempt to be a complete comprehensive check of all possible analysis cases, but rather the loading cases which are assumed to control overall dam design for preliminary sizing and concept evaluation.

Assumptions:
Ignore resistance from sediment or rock on downstream toe.
Ice loading is not considered.
Structure is not undermined by scour.
Upstream and downstream turndowns (not shown) are not relied upon for shear resistance.
All soil and rock layers are assumed to be horizontal.
Use single conservative frictional interface strength, as shown in the calculation.
Disregard cohesion for long term analysis.
Mass or contributions of pedestrian bridge ignored (conservative)
2 dimensional analysis considering dam geometry on a per-foot basis, 3Dimensional end effects not considered.
Steps shown in geometry are concrete or cut stone with similar unit weight to mass concrete.
Other assumptions as noted in the calculation

Inputs:
Approximate top of rock elevation for main dam, estimated at **El 615 ft**.
Dam foundation elevation assumed 4 feet below top of rock **(El 611 ft.)**
Water present to top of gate at **El 638.0 ft**.
3 foot tall crest gate with sill at **EL 635.0 ft.**
Sediment elevation present to 2 feet below sill at **El 633.0 ft** as directed by Murry Fleming.
Tailwater elevation is coincident with dam foundation, **El 611.0 ft**.
2008 boreholes by Stantec used to estimate subsurface conditions and properties.
Other inputs as noted in the calculation.

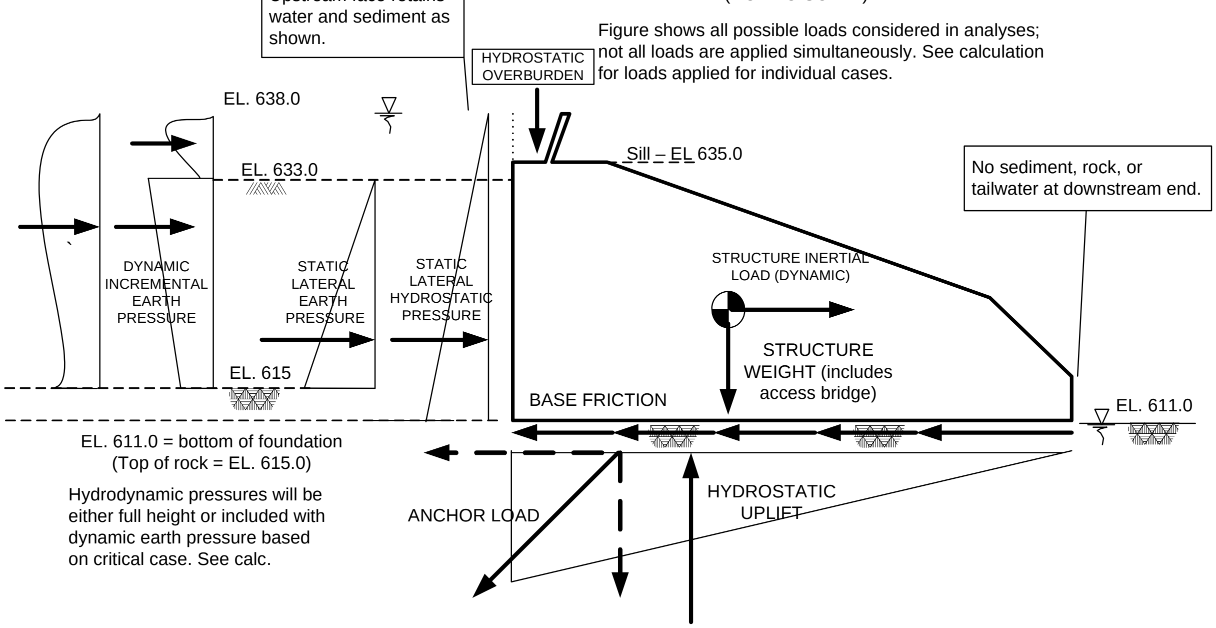
References:
USACE EM 1110-2-2200 Gravity Dam Manual
USACE EM 1110-2-2100 Stability Analysis of Concrete Structures

Crest Gate Section Geometry:



CREST GATE SECTION

(NOT TO SCALE)



Define Geometry:

$El_{crest} := 638.0ft$ Elevation at top of dam crest

$El_{sill} := 635ft$ Elevation at the top of the sill (top of concrete)

$El_{rock} := 615ft$ Elevation of top of rock (shale)

$d_{excav} := 4ft$ Excavte below top of rock to remove weathered shale.

$El_{foundation} := El_{rock} - d_{excav} = 611 \cdot ft$ Elevation of bottom of dam

$H_{dam} := El_{crest} - El_{foundation} = 27 \cdot ft$ Total height of dam

$w_{foundation} := 63.25ft$ Given width of dam base

$El_{sed.top} := El_{sill} - 2ft = 633 \cdot ft$

Assume that dam impounds sediment to top of concrete (sill).

$El_{water.US} := El_{crest} = 638 \cdot ft$

Elevation of water upstream of dam.

$El_{water.DS} := El_{foundation} = 611 \cdot ft$

Elevation of water downstream of dam (assume no water as recommended by USACE).

Material Properties:

Unit Weight:

$\gamma_{conc} := 150pcf$

Unit weight of concrete (assumed).

$\gamma_{sed} := 120pcf$

Unit weight of sediment against upstream face of dam (recommended by USACE EM 1110-2-2100)

$\gamma_{shale} := 152pcf$

Unit weight of Shale from Stantec, 2008 laboratory test results.

$\gamma_{w} := 62.4pcf$

Unit weight of water (assumed).

Shear Strength:

$\phi_{sed} := 28deg$ $\quad c_{sed} := 0psf$

Effective stress shear strength of sediment.

Interface Strength (sliding):

$\delta_{base} := 24deg$

Consider only one sliding interface, mass concrete cast against shale bedrock. Assume no cohesion/adhesion along this interface, only base friciton. Typical value from NAVFAC DM7.2 for "Mass concrete cast against...very stiff and hard residual or preconsolidated clay".

Seismic:

$PGA_{OBE} := 0.009$

Peak ground acceleration on rock for Operations Basis Earthquake (OBE). 50% probability of exceedance in 100 years.

$PGA_{MCE} := 0.088$

Peak ground acceleration on rock for Maximum Credible Earthquake (MCE). 10% probability of exceedance in 50 years

$F_{PGA.scC} := 1.2$

Site coefficient for Site Class C, "Very Dense Soil and Soft Rock" (assumed).

$k_{h.MCE} := \frac{2}{3} \cdot PGA_{MCE} \cdot F_{PGA.scC} = 0.07$

Seismic coeff for MCE case (per EM 1110-2-2100 = 2/3 effective peak ground accel). Conservatively estimated using PGA for site class C.

$k_{h.OBE} := \frac{2}{3} \cdot PGA_{OBE} \cdot F_{PGA.scC} = 0.007$

Seismic coeff for OBE case.

$k_{v} := 0$

Neglect vertical component of earthquake acceleration (assumed).

Estimate Weight of Concrete Gravity Dam:

Estimate total stress (non buoyant) weight of concrete gravity dam by estimating area of the gravity dam polygon, and then multiplying it by the unit weight of the material

Centroid of polygon [edit] from Wikipedia (http://en.wikipedia.org/wiki/Polygon, February 27, 2014)

The centroid of a non-self-intersecting closed polygon defined by n vertices (x_0,y_0), (x_1,y_1), ..., (x_{n-1},y_{n-1}) is the point (C_x, C_y), where

$$C_x = \frac{1}{6A}\sum_{i=0}^{n-1}(x_i + x_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

$$C_y = \frac{1}{6A}\sum_{i=0}^{n-1}(y_i + y_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

and where A is the polygon's signed area,

$$A = \frac{1}{2}\sum_{i=0}^{n-1}(x_i\ y_{i+1} - x_{i+1}\ y_i).$$ [9]

In these formulas, the vertices are assumed to be numbered in order of their occurrence along the polygon's perimeter, and the vertex (x_n, y_n) is assumed to be the same as (x_0, y_0). Note that if the points are numbered in clockwise order the area A, computed as above, will have a negative sign; but the centroid coordinates will be correct even in this case.

Define function to calculate area of polygon whose plane coordinates are contained in matrix XY

$$\mathrm{Area}(XY) := \left|\begin{array}{l} XY \leftarrow \mathrm{stack}\left[XY, \left(XY^T\right)^{\langle 0\rangle T}\right] \\ M \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left|\mathrm{submatrix}(XY, i, i+1, 0, 1)\right| \\ 0.5\cdot M \end{array}\right.$$

Define function to calculate coordinates of centroid of non-intersecting closed polygon

$$\mathrm{Centroid}(XY) := \left|\begin{array}{l} XY \leftarrow \mathrm{stack}\left[XY, \left(XY^T\right)^{\langle 0\rangle T}\right] \\ x \leftarrow XY^{\langle 0\rangle} \\ y \leftarrow XY^{\langle 1\rangle} \\ C_x \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left[\left(x_i + x_{i+1}\right)\cdot\left(x_i\cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ C_y \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left[\left(y_i + y_{i+1}\right)\cdot\left(x_i\cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ \left(C_x \ \ C_y\right)\cdot\dfrac{1}{6\cdot \mathrm{Area}(XY)} \end{array}\right.$$

Area and Centroid of Concrete Gravity Dam

$$XY_{dam} := \begin{pmatrix} 0 & 611 \\ 0 & 635 \\ 1.5 & 635 \\ 3 & 638 \\ 3.5 & 638 \\ 2 & 635 \\ 9.25 & 635 \\ 11.25 & 633 \\ 59.25 & 619 \\ 63.25 & 615 \\ 63.25 & 611 \end{pmatrix}$$

- Values define cross-sectional geometry of dam, points are clockwise around cross section, starting at upstream heel.
- Left column is X coordinates, "0" is the upstream heel of the dam, sign convention is positive to the right (downstream).
- Right column is elevation.

$$-\mathrm{Area}(XY_{dam}) = 1013.5$$

$$\mathrm{Centroid}(XY_{dam}) = (25.316 \quad 619.977)$$ center of gravity for concrete gravity dam, ft

$$x_{dam\_CG} := \mathrm{Centroid}(XY_{dam})_{0,0} = 25.316$$

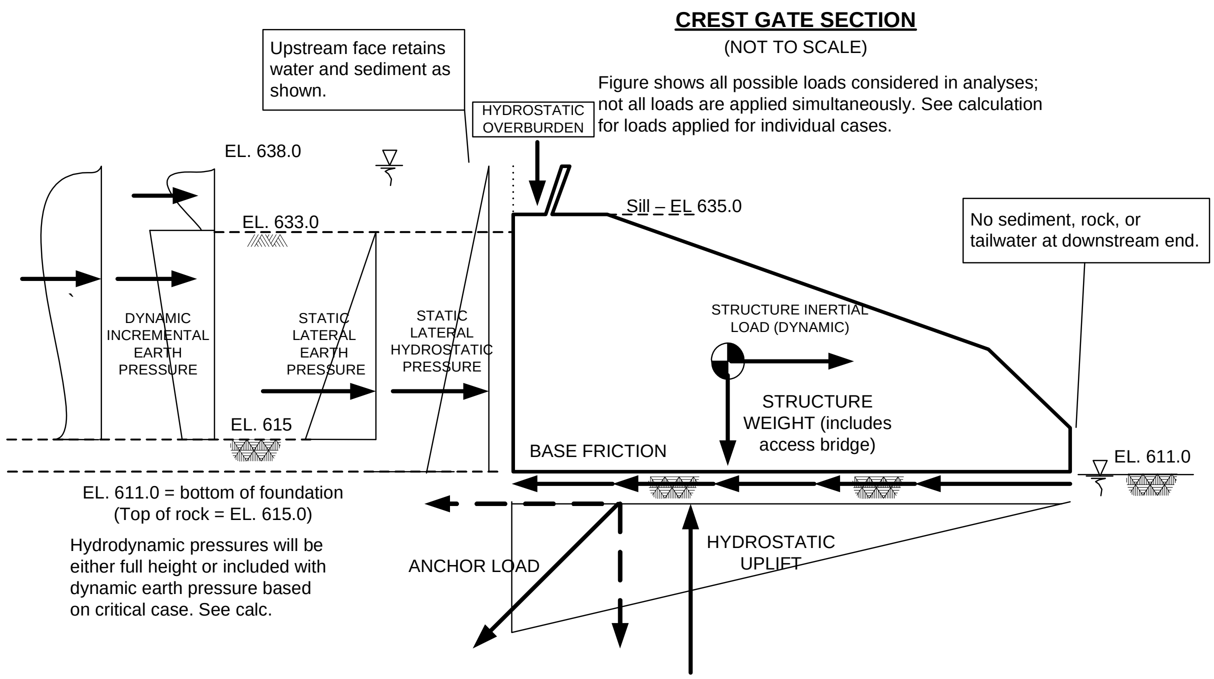
$$x_{dam} := x_{dam\_CG} \cdot 1ft = 25.316 \cdot ft$$ X-coordinate fo centroid, in feet

$$y_{dam} := \mathrm{Centroid}(XY_{dam})_{0,1} = 619.977$$

$$El_{centroid} := y_{dam} \cdot 1ft = 619.977 \cdot ft$$ Elevation of centroid

$$WT_{dam} := -\mathrm{Area}(XY_{dam}) \cdot ft^2 \cdot \gamma_{conc} + 5klf = 157 \cdot \frac{kip}{ft}$$ total weight of concrete gravity dam, per foot. include weight of pedestrian bridge, estimated as 5 klf per Kevin Whittier

Estimate Lateral Driving Forces Acting on Concrete Gravity Dam



Lateral Hydrostatic Water Load on Upstream Face:

$H_w := El_{water.US} - El_{foundation} = 27 \cdot ft$ — Height of water

$F_{h2o} := \frac{1}{2} \cdot \gamma_w \cdot H_w^{\,2} = 22.7 \cdot \frac{kip}{ft}$ — Magnitude of resultant of hydrostatic load on upstream face of dam

$El_{h2o} := El_{water.US} - \frac{2}{3} \cdot H_w = 620 \cdot ft$ — Elevation of resultant

Static At-Rest Lateral Earth Pressure on Upstream Face:

Assume sediment contributes at-rest soil pressure on upstream face of dam (active pressures are not developed).

$H_{ko} := El_{sed.top} - El_{rock} = 18 \cdot ft$ — Height of sediment acting against structure

$K_0 := 1 - \sin(\phi_{sed}) = 0.531$ — At-rest soil pressure coefficient.

$F_{ko} := \frac{1}{2} \cdot K_0 \cdot (\gamma_{sed} - \gamma_w) \cdot H_{ko}^{\,2} = 4.95 \cdot \frac{kip}{ft}$ — Magnitude of resultant of at-rest soil pressure on upstream face of dam

$El_{ko} := El_{sed.top} - \frac{2}{3} \cdot H_{ko} = 621 \cdot ft$ — Elevation of at rest earth pressure resultant.

SEISMIC: Lateral Hydrodynamic Water Load on Upstream Face:

This load is applied assuming the dam has been flushed of sediment, and full height of water applies hydrodynamic loading to dam structure during a seismic event. Note that, when sediment levels accumulate, hydrodynamic loading is not considered to be a valid case.

$P_{hydro.MCE} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 1.356 \cdot klf$ — Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$P_{hydro.OBE} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 0.139 \cdot klf$ — Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$$P_{hydro.MCE.partial} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot \left(El_{crest} - El_{sed.top}\right)^2 = 0.064 \cdot klf$$

Magnitude of hydrodynamic loading over accumulated sediment.

$$P_{hydro.OBE.partial} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot \left(El_{crest} - El_{sed.top}\right)^2 = 0.007 \cdot klf$$

Magnitude of hydrodynamic loadign over accumulated sediment.

$$El_{hydro.MCE} := El_{rock} + 0.4 \cdot \left(El_{crest} - El_{rock}\right) = 624.2 \cdot ft$$

Elevation of resultant of hydrodynamic load for full height water case.

$$El_{hydro.OBE} := El_{rock} + 0.4 \cdot \left(El_{crest} - El_{rock}\right) = 624.2 \cdot ft$$

Elevation of resultant of hydrodynamic load for full height water case.

$$El_{hydro.MCE.partial} := El_{sed.top} + 0.4 \cdot \left(El_{crest} - El_{sed.top}\right) = 635 \cdot ft$$

Elevation of resultant of hydrodynamic load for full sediment case.

$$El_{hydro.OBE.partial} := El_{sed.top} + 0.4 \cdot \left(El_{crest} - El_{sed.top}\right) = 635 \cdot ft$$

Elevation of resultant of hydrodynamic load for full sediment case.

SEISMIC: Lateral Earth Pressures Upstream Face:

$\theta_{wall} := 0deg$ Slope of upstream face of dam, 0 indicates vertical face

$\delta_{sed} := 0deg$ Interface friction angle between sediment and dam, assume zero degrees.

$\beta_{US} := 0deg$ Slope of top of sediment against upstream face of dam. 0 degrees is horizontal.

$$f\_K_{A.c}(\phi, \delta, \beta, \theta) := \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\delta + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\delta + \phi) \cdot \sin(\phi - \beta)}{\cos(\delta + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

Function to calculate Coulomb active earth pressure coefficient

$$K_A := f\_K_{A.c}\left(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}\right) = 0.361$$

Coulomb active earth perssure coefficient

$$P_A := \frac{1}{2} \cdot K_A \cdot \left(\gamma_{sed} - \gamma_w\right) \left(El_{sed.top} - El_{rock}\right)^2 = 3.369 \cdot klf$$

Active earth pressure force.

$$f\_\psi\left(k_h, k_v\right) := atan\left(\frac{k_h}{1 - k_v}\right)$$

$$f\_K_{AE}(\phi, \delta, \beta, \theta, \psi) := \frac{\cos(\phi - \psi - \theta)^2}{\cos(\psi) \cdot \cos(\theta)^2 \cdot \cos(\psi + \theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \psi - \beta)}{\cos(\delta + \psi + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

Check MCE:

$$\psi_{MCE} := f\_\psi\left(k_{h.MCE}, k_v\right) = 0.07$$

$K_{AE.MCE} := f\_K_{AE}(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{MCE}) = 0.406$

Examine active case only for upstream sediment. Neglect any downstream passive resistance.

$P_{AE.MCE} := (0.5 \cdot K_{AE.MCE}) \cdot (\gamma_{sed} - \gamma_w)(El_{sed.top} - El_{rock})^2 = 3.79 \cdot klf$

Seismic active earth pressure.

$\Delta P_{AE.MCE} := P_{AE.MCE} - P_A = 0.421 \cdot klf$

Dynamic incremental earth pressure in seismic MCE case.

$F_{MCE.partial} := \Delta P_{AE.MCE} + P_{hydro.MCE.partial} = 0.485 \cdot klf$

Soil + hydrodynamic water load above soil, total horiztonal applied force.

Check OBE:

$\psi_{OBE} := f\_\psi(k_{h.OBE}, k_v) = 0.007$

$K_{AE.OBE} := f\_K_{AE}(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{OBE}) = 0.365$

Examine active case only for upstream sediment. Neglect any downstream passive resistance.

$P_{AE.OBE} := (0.5 \cdot K_{AE.OBE}) \cdot (\gamma_{sed} - \gamma_w)(El_{sed.top} - El_{rock})^2 = 3.41 \cdot klf$

Active seismic earth pressure between top of sediment and top of rock.

$\Delta P_{AE.OBE} := P_{AE.OBE} - P_A = 0.041 \cdot klf$

Additional applied earth pressure in seismic OBE case.

$F_{OBE.partial} := \Delta P_{AE.OBE} + P_{hydro.OBE.partial} = 0.047 \cdot klf$

Soil + hydrodynamic water load above soil, total horiztonal applied force.

Compare Dynamic Lateral Earth Pressures to At-Rest Lateral Earth Pressures:

$P_{AE.OBE} = 3.41 \cdot klf$ Dynamic Active OBE

$P_{AE.MCE} = 3.79 \cdot klf$ Dynamic Active MCE

$F_{ko} = 4.95 \cdot klf$ Static At-Rest

Note that static at-rest loading is greater than dynamic active loading for both MCE and OBE cases. Use greater of static at-rest or dynamic active lateral earth pressures. In this case, static at-rest pressure controls and should be used as the lateral earth pressure for the dynamic analysis cases.

Determine controlling load case for upstream loading on structure:

Structure could experience free water (no sediment accumulation), or filled with sediment. For seismic stability evaluations, estimate controlling case: either hydrodynamic loading of silt-free dam or dynamic lateral earth pressure + water over top of sediment.

$F_{ko} = 4.95 \cdot klf$ At-rest lateral earth pressure loading (note that static at-rest is controlling case for seismic evaluation)

$F_{h2o} = 22.745 \cdot klf$ Hydrostatic pressure

$P_{hydro.MCE} = 1.356 \cdot klf$ Hydrodynamic pressure over full height of structure, MCE event

$P_{hydro.OBE} = 0.139 \cdot klf$ Hydrodynamic pressure over full height of structure, OBE event

$P_{hydro.MCE.partial} = 0.064 \cdot klf$ Hydrodynamic pressure above top of sediment, MCE event. Include with soil loading case.

$P_{hydro.OBE.partial} = 0.007 \cdot klf$

Hydrodynamic pressure above top of sediment, OBE event. Include with soil loading case.

$P_{hydro.MCE} + F_{h2o} = 24.1 \cdot klf$

$F_{h2o} + F_{ko} + P_{hydro.MCE.partial} = 27.759 \cdot klf$

$$check_{hydro.MCE} := \begin{array}{|l} \text{"Hydrodynamic"} \quad \text{if } \left(P_{hydro.MCE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko} + P_{hydro.MCE.partial}\right) \\ \text{"Soil"} \quad \text{otherwise} \end{array}$$

$check_{hydro.MCE} = \text{"Soil"}$

$P_{hydro.OBE} + F_{h2o} = 22.883 \cdot klf$

$F_{h2o} + F_{ko} + P_{hydro.OBE.partial} = 27.702 \cdot klf$

$$check_{hydro.OBE} := \begin{array}{|l} \text{"Hydrodynamic"} \quad \text{if } \left(P_{hydro.OBE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko} + P_{hydro.OBE.partial}\right) \\ \text{"Soil"} \quad \text{otherwise} \end{array}$$

$check_{hydro.OBE} = \text{"Soil"}$

SEISMIC: Inertial Load of Structure:

$F_{inertia.MCE} := k_{h.MCE} \cdot WT_{dam} = 11.055 \cdot klf$

Seismic inertia load of the dam for MCE, acts in downstream direction.

$F_{inertia.OBE} := k_{h.OBE} \cdot WT_{dam} = 1.131 \cdot klf$

Seismic inertia load of the dam for OBE acts in downstream direction.

$El_{inertia.MCE} := El_{centroid} = 619.977 \cdot ft$

Seismic inertial load acts through centroid of dam

$El_{inertia.OBE} := El_{centroid} = 619.977 \cdot ft$

Seismic inertial load acts through centroid of dam

Estimate Uplift Hydrostatic Forces Acting on Concrete Gravity Dam

Hydrostatic Uplift on Dam Base:

Magnitude of hydrostatic uplift is estimated as straightline interpolation between headwater and tailwater across width of structure. Figure above shows assumed uplift distribution below bottom of dam.

Use centroid equation to define uplift pressure.

$$XY_{uplift} := \begin{pmatrix} w_{foundation} & 0 \\ w_{foundation} & El_{water.DS} - El_{foundation} \\ 0 & El_{water.US} - El_{foundation} \\ 0 & 0 \end{pmatrix} \cdot ft^{-1}$$

$$w_{foundation} = 63.25 \cdot ft$$

$$Area(XY_{uplift}) = 853.875$$

$$Centroid(XY_{uplift}) = (21.083 \quad 9)$$

$$X_{uplift} := (1ft\,Centroid(XY_{uplift}))_{0,0} = 21.083 \cdot ft$$

$$F_{uplift} := Area(XY_{uplift}) \cdot ft^2 \cdot \gamma_w = 53.282 \cdot \frac{kip}{ft}$$

Estimate Resisting Forces:

Estimate base sliding resistance for concrete gravity dam sliding on rock. Account for hydrostatic overburden above upstream face dam:

Hydrostatic Overburden Volume above upstream face of Dam:

$$XY_{hydroOB} := \begin{pmatrix} 0 & 635 \\ 1.5 & 635 \\ 3 & 638 \\ 0 & 638 \end{pmatrix}$$

$$Area(XY_{hydroOB}) = 6.75$$

$$Centroid(XY_{hydroOB}) = (1.167 \quad 636.667)$$

$$x_{h2o.vert} := (1ft\,Centroid(XY_{hydroOB}))_{0,0} = 1.167 \cdot ft$$

$$F_{h2o.vert} := Area(XY_{hydroOB}) \cdot ft^2 \cdot \gamma_w = 0.421 \cdot \frac{kip}{ft}$$

Interface friction between concrete gravity dam and shale bedrock:

$\delta_{base} = 24 \cdot deg$ Base friction angle between dam and foundation.

$F_{base} := (WT_{dam} + F_{h2o.vert} - F_{uplift}) \cdot \tan(\delta_{base}) = 46.4 \cdot \frac{kip}{ft}$ Base friction, sum of vertical forces multiplied by tangent of interface friction times tangent of interface friction (delta).

Estimate Factor of Safety Against Sliding:

The recommended global stability design criteria is summarized in the USACE Gravity Dam Design EM 1110-2-2200. Stability criteria is summarized in Table 4-1 below.

EM 1110-2-2200
30 Jun 95

Table 4-1
Stability and stress criteria

Load Condition	Resultant Location at Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress	
				Compressive	Tensile
Usual	Middle 1/3	2.0	≤ allowable	$0.3\ f'_c$	0
Unusual	Middle 1/2	1.7	≤ allowable	$0.5\ f'_c$	$0.6\ f'^{2/3}_c$
Extreme	Within base	1.3	≤ 1.33 × allowable	$0.9\ f'_c$	$1.5\ f'^{2/3}_c$

Note: f'_c is 1-year unconfined compressive strength of concrete. The sliding factors of safety (*FS*) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions.

$FS_{min} := 2.0$ Usual loading. Minimum sliding factor of safety recommended by USACE (from table above)

$\Sigma F_{h.drive} := F_{h2o} + F_{ko} = 27.7 \cdot \frac{kip}{ft}$ Sum of driving forces (hydrostatic pressure + at rest lateral earth pressure)

$\Sigma F_{h.resist} := F_{base} = 46.4 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$$\boxed{FS_{slide.shale} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive}} = 1.67}$$

$$check_{slide.shale} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale} > FS_{min} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{slide.shale} = \text{"NOT OK-anchors required"}}$$

MCE Seismic Sliding:

$FS_{min.MCE} := 1.3$ MCE, extreme loading. Minimum sliding factor of safety recommended by USACE (from table above).

$\Sigma F_{h.drive.MCE} := F_{h2o} + F_{ko} + P_{hydro.MCE.partial} + F_{inertia.MCE} = 38.8 \cdot \frac{kip}{ft}$

Sum of lateral driving forces during MCE event.

$\Sigma F_{h.resist} := F_{base} = 46.4 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$$\boxed{FS_{slide.shale.MCE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.MCE}} = 1.19}$$

$$check_{slide.shale.MCE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.MCE} > FS_{min.MCE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.MCE} = \text{"NOT OK-anchors required"}$$

OBE Sliding:

$FS_{min.OBE} := 1.7$

OBE, Unusual loading. Minimum sliding factor of safety recommended by USACE (from table above).

$$\Sigma F_{h.drive.OBE} := F_{h2o} + F_{ko} + P_{hydro.OBE.partial} + F_{inertia.OBE} = 28.8 \cdot \frac{kip}{ft}$$

Sum of lateral driving forces during OBE event.

$$\Sigma F_{h.resist} := F_{base} = 46.4 \cdot \frac{kip}{ft}$$

Sum of resisting forces (base friction)

$$FS_{slide.shale.OBE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.OBE}} = 1.61$$

$$check_{slide.shale.OBE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.OBE} > FS_{min.OBE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.OBE} = \text{"NOT OK-anchors required"}$$

Estimate Required Anchor Force to Acheive Minimum Sliding Factor of Safety:

$$F_{anchor} := FS_{min} \cdot \Sigma F_{h.drive} - \Sigma F_{h.resist} = 9.014 \cdot klf$$

Use minimum FS against sliding to determine anchor forc This is the horizontal component of the anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.static} := \frac{F_{anchor}}{\cos(\alpha_{anchor})} = 12.747 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Check Seismic:

MCE case:

$$F_{anchor.MCE} := FS_{min.MCE} \cdot \Sigma F_{h.drive.MCE} - \Sigma F_{h.resist} = 4.081 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of tl anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.MCE} := \frac{F_{anchor.MCE}}{\cos(\alpha_{anchor})} = 5.772 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

OBE case:

$$F_{anchor.OBE} := FS_{min.OBE} \cdot \Sigma F_{h.drive.OBE} - \Sigma F_{h.resist} = 2.638 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of tl anchor.

$$\alpha_{anchor} := 45deg$$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.OBE} := \frac{F_{anchor.OBE}}{\cos(\alpha_{anchor})} = 3.731 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Determine Critical Anchor Force for Design:

$$\boxed{T_{anchor.critical} := \max(T_{anchor.static}, T_{anchor.MCE}, T_{anchor.OBE}) = 12.747 \cdot klf}$$

Estimate Factor of Safety Against Overturning:

Sum moments around downstream toe. Note this is not directly comparable to USACE overturning criteria but useful as a quick che of stability, see estimation of overturning resultant and % base compression below.

$$\Sigma M_{toe.drive} := F_{ko} \cdot (El_{ko} - El_{foundation}) + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \ldots = 2500.924 \cdot \frac{kip \cdot ft}{ft}$$
$$+ F_{uplift} \cdot (w_{foundation} - X_{uplift})$$

$$\Sigma M_{toe.resist} := WT_{dam} \cdot (w_{foundation} - x_{dam}) + F_{h2o.vert} \cdot (w_{foundation} - x_{h2o.vert}) = 5982.7 \cdot \frac{kip \cdot ft}{ft}$$

$$\boxed{FS_{overturning} := \frac{\Sigma M_{toe.resist}}{\Sigma M_{toe.drive}} = 2.39}$$

Factor of safety against overturning, static case.

There is no specified factor of safety provided by USACE against overturning. The USACE does recommend that for the Normal/Usual loading scenario, the overturning resultant should be located within the middle 1/3 of the base of the dam, and for the unusual loading scenario, the middle 1/2 of the dam.

Estimate Location of Overturning Resultant:

Static Case:

Check that location of overturning resultant falls in middle 1/3 of base of concrete gravity dam (usual case)

$$\Sigma M_{toe.total} := \Sigma M_{toe.drive} - \Sigma M_{toe.resist} = -3481.776 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.total} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 104.164 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant} := \frac{-\Sigma M_{toe.total}}{\Sigma F_{vertical.total}} = 33.4 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the wall

$$\frac{1}{3} \cdot w_{foundation} = 21.1 \cdot ft$$

defines middle third of base

$$\frac{2}{3} \cdot w_{foundation} = 42.2 \cdot ft$$

defines middle third of base

$$check_{OT} := \begin{cases} \text{"OK"} & \text{if } \frac{1}{3} w_{foundation} \le X_{Resultant} \le \frac{2}{3} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{OT} = \text{"OK"}}$$

OBE Case:

Check that location of overturning resultant falls in middle 1/2 of base of concrete gravity dam (unusual case)

$$\begin{aligned} \Sigma M_{toe.drive.OBE} := \; & F_{ko} \cdot (El_{ko} - El_{foundation}) \ldots \\ & + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \ldots \\ & + F_{uplift} \cdot (w_{foundation} - X_{uplift}) \ldots \\ & + F_{inertia.OBE} \cdot (El_{inertia.OBE} - El_{foundation}) \ldots \\ & + P_{hydro.OBE.partial} \cdot (El_{hydro.OBE.partial} - El_{foundation}) \end{aligned} = 2511.23 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.OBE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 104.164 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant.OBE} := \frac{-\Sigma M_{toe.drive.OBE}}{\Sigma F_{vertical.total}} = -24.1 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the dam

$$\frac{1}{4} \cdot w_{foundation} = 15.8 \cdot ft$$

defines middle half of base

$$\frac{3}{4} \cdot w_{foundation} = 47.4 \cdot ft$$

defines middle half of base

$$check_{OT.OBE} := \begin{cases} \text{"OK"} & \text{if } \frac{1}{4} w_{foundation} \le X_{Resultant} \le \frac{3}{4} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{OT.OBE} = \text{"OK"}}$$

MCE Case:

Check that location of overturning resultant falls within base of concrete gravity dam (extreme case)

$$\Sigma M_{toe.drive.MCE} := F_{ko} \cdot (El_{ko} - El_{foundation}) \cdots + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \cdots + F_{uplift} \cdot (w_{foundation} - X_{uplift}) \cdots + F_{inertia.MCE} \cdot (El_{inertia.MCE} - El_{foundation}) \cdots + P_{hydro.MCE.partial} \cdot (El_{hydro.OBE.partial} - El_{foundation}) = 2601.699 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.MCE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 104.164 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant.MCE} := \frac{-\Sigma M_{toe.drive.MCE}}{\Sigma F_{vertical.total}} = -25 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the dam

$0 \cdot w_{foundation} = 0 \cdot ft$ defines upstream edge of base

$1 \cdot w_{foundation} = 63.3 \cdot ft$ defines downstream edge of base

$$check_{OT.MCE} := \begin{cases} \text{"OK"} & \text{if } 0 w_{foundation} \le X_{Resultant} \le 1 w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{OT.MCE} = \text{"OK"}}$$

Remarks and Recapitulation:

- Calculation addresses sliding and overturning of the crest gate section of Sand Springs Dam under anticipated static operating conditions, OBE seismic case, and MCE seismic case noted.
- For all cases, it is identified that permanent ground anchors are necessary for sliding stability.
- Anchors are not necessary for overturning stability.
- The static case (usual loading) was found to control.

Attachment 6

Evaluate Sliding and Overturning Full Height Gate: Normal Operating

PROJECT : Arkansas River Corridor Project - Sand Springs Dam
PROJECT #: 657971.04.02.01
CREATED BY: Jen Schaeffer/SEA **DATE:** 04/14/2015
REVIEWED BY: Mark Kacmarcik **DATE:** 04/16/2015

Given: Simplified gravity dam geometry shown and generalized subsurface profile. See sketch.

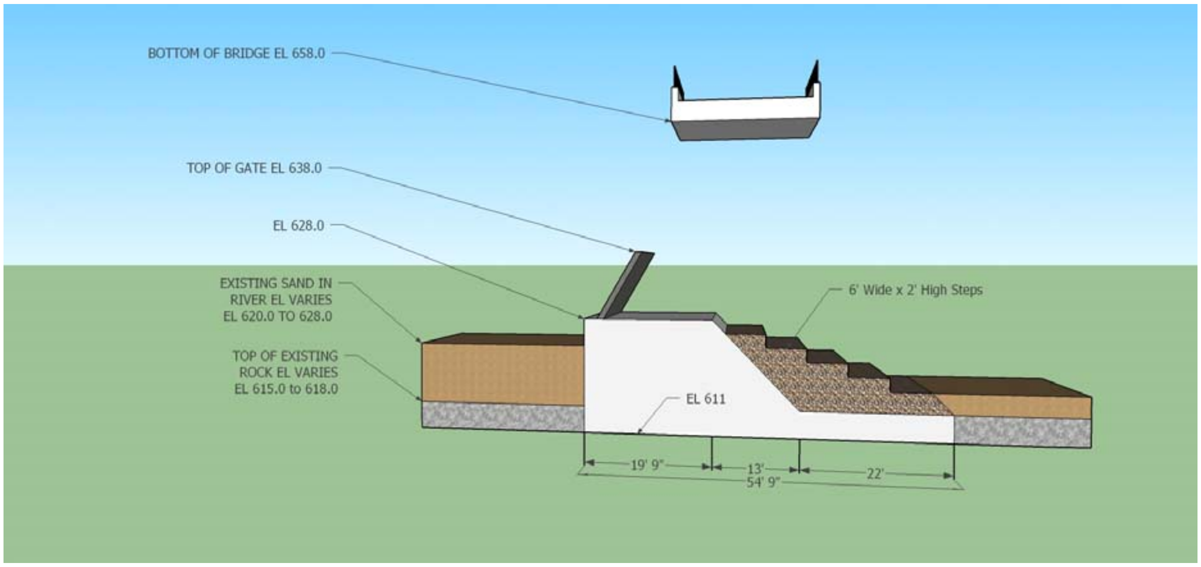
Find: Check sliding and overturning against USACE criteria for Static and Seismic cases as noted in the title. Anchor forces are included as needed to meet stability criteria. Note that this is not attempt to be a complete comprehensive check of all possible analysis cases, but rather the loading cases which are assumed to control overall dam design for preliminary sizing and concept evaluation.

Assumptions: Ignore resistance from sediment or rock on downstream toe.
Ice loading is not considered.
Structure is not undermined by scour.
Upstream and downstream turndowns (not shown) are not relied upon for shear resistance.
All soil and rock layers are assumed to be horizontal.
Use single conservative frictional interface strength, as shown in the calculation.
Disregard cohesion for long term analysis.
Mass or contributions of pedestrian bridge ignored (conservative)
2 dimensional analysis considering dam geometry on a per-foot basis, 3Dimensional end effects not considered.
Steps shown in geometry are concrete or cut stone with similar unit weight to mass concrete.
Other assumptions as noted in the calculation

Inputs: Approximate top of rock elevation for main dam, estimated at **El 615 ft**.
Dam foundation elevation assumed 4 feet below top of rock **(El 611 ft.)**
Water present to top of gate at **El 638.0 ft**.
Sediment elevation present to top of sill at **El 628.0 ft** as directed by Murry Fleming.
Tailwater elevation is coincident with dam foundation, **El 611.0 ft**.
2008 boreholes by Stantec used to estimate subsurface conditions and properties.
Other inputs as noted in the calculation.

References: USACE EM 1110-2-2200 Gravity Dam Manual
USACE EM 1110-2-2100 Stability Analysis of Concrete Structures

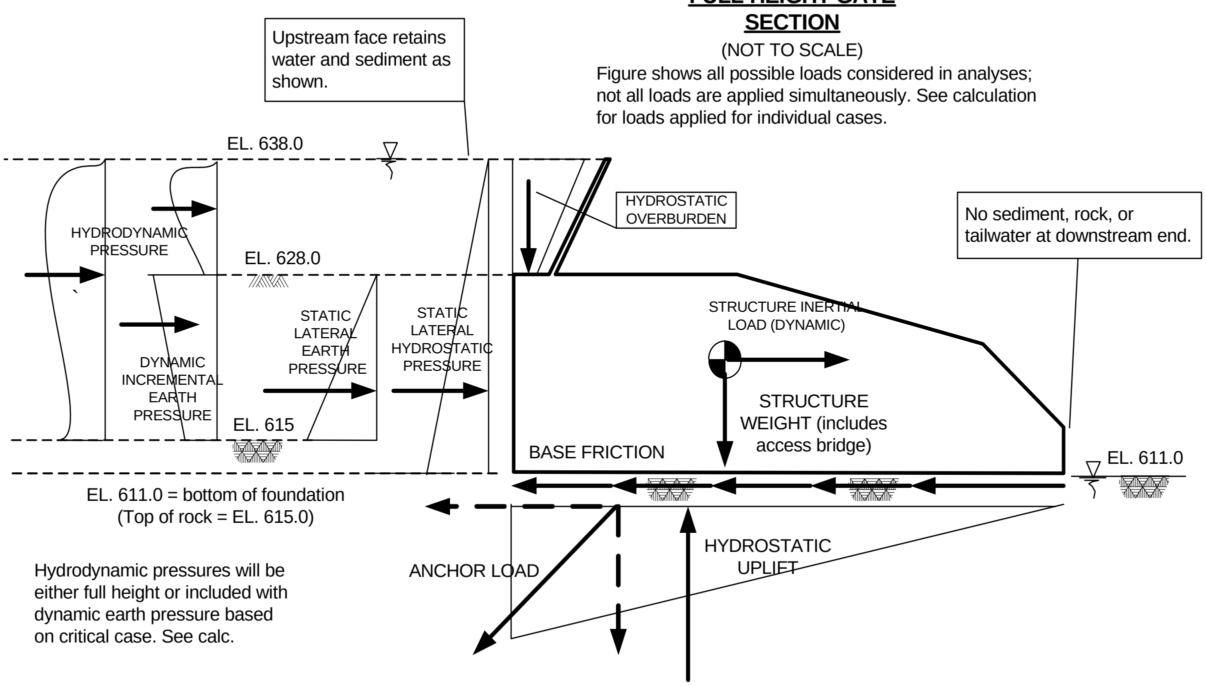
Full Height Gate Section Geometry:



FULL HEIGHT GATE SECTION

(NOT TO SCALE)

Figure shows all possible loads considered in analyses; not all loads are applied simultaneously. See calculation for loads applied for individual cases.



Define Geometry:

$El_{crest} := 638.0ft$ — Elevation at top of dam crest

$El_{sill} := 628ft$ — Elevation at the top of the sill (top of concrete)

$El_{rock} := 615ft$ — Elevation of top of rock (shale)

$d_{excav} := 4ft$ — Excavte below top of rock to remove weathered shale.

$El_{foundation} := El_{rock} - d_{excav} = 611 \cdot ft$ — Elevation of bottom of dam

$H_{dam} := El_{crest} - El_{foundation} = 27 \cdot ft$ — Total height of dam

$w_{foundation} := 54.75ft$ Given width of dam base

$El_{sed.top} := El_{sill} = 628 \cdot ft$ Assume that dam impounds sediment to top of concrete (sill).

$El_{water.US} := El_{crest} = 638 \cdot ft$ Elevation of water upstream of dam.

$El_{water.DS} := El_{foundation} = 611 \cdot ft$ Elevation of water downstream of dam (assume no water as recommended by USACE).

Material Properties:

Unit Weight:

$\gamma_{conc} := 150pcf$ Unit weight of concrete (assumed).

$\gamma_{sed} := 120pcf$ Unit weight of sediment against upstream face of dam (recommended by USACE EM 1110-2-2100)

$\gamma_{shale} := 152pcf$ Unit weight of Shale from Stantec, 2008 laboratory test results.

$\gamma_{w} := 62.4pcf$ Unit weight of water (assumed).

Shear Strength:

$\phi_{sed} := 28deg$ $c_{sed} := 0psf$ Effective stress shear strength of sediment.

Interface Strength (sliding):

$\delta_{base} := 24deg$ Consider only one sliding interface, mass concrete cast against shale bedrock. Assume no cohesion/adhesion along this interface, only base friciton. Typical value from NAVFAC DM7.2 for "Mass concrete cast against...very stiff and hard residual or preconsolidated clay".

Seismic:

$PGA_{OBE} := 0.009$ Peak ground acceleration on rock for Operations Basis Earthquake (OBE). 50% probability of exceedance in 100 years.

$PGA_{MCE} := 0.088$ Peak ground acceleration on rock for Maximum Credible Earthquake (MCE). 10% probability of exceedance in 50 years

$F_{PGA.scC} := 1.2$ Site coefficient for Site Class C, "Very Dense Soil and Soft Rock" (assumed).

$k_{h.MCE} := \frac{2}{3} \cdot PGA_{MCE} \cdot F_{PGA.scC} = 0.07$ Seismic coeff for MCE case (per EM 1110-2-2100 = 2/3 effective peak ground accel). Conservatively estimated using PGA for site class C.

$k_{h.OBE} := \frac{2}{3} \cdot PGA_{OBE} \cdot F_{PGA.scC} = 0.007$ Seismic coeff for OBE case.

$k_{v} := 0$ Neglect vertical component of earthquake acceleration (assumed).

Estimate Weight of Concrete Gravity Dam:

Estimate total stress (non buoyant) weight of concrete gravity dam by estimating area of the gravity dam polygon, and then multiplying it by the unit weight of the material

Centroid of polygon [edit]from Wikipedia (http://en.wikipedia.org/wiki/Polygon, February 27, 2014)

The centroid of a non-self-intersecting closed polygon defined by n vertices (x_0,y_0), (x_1,y_1), ..., (x_{n-1},y_{n-1}) is the point (C_x, C_y), where

$$C_x = \frac{1}{6A}\sum_{i=0}^{n-1}(x_i + x_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

$$C_y = \frac{1}{6A}\sum_{i=0}^{n-1}(y_i + y_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

and where A is the polygon's signed area,

$$A = \frac{1}{2}\sum_{i=0}^{n-1}(x_i\ y_{i+1} - x_{i+1}\ y_i).$$ [9]

In these formulas, the vertices are assumed to be numbered in order of their occurrence along the polygon's perimeter, and the vertex (x_n, y_n) is assumed to be the same as (x_0, y_0). Note that if the points are numbered in clockwise order the area A, computed as above, will have a negative sign; but the centroid coordinates will be correct even in this case.

Define function to calculate area of polygon whose plane coordinates are contained in matrix XY

$$\text{Area}(XY) := \left|\begin{array}{l} XY \leftarrow \text{stack}\left[XY, \left(\left(XY^T\right)^{\langle 0\rangle}\right)^T\right] \\ M \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left|\text{submatrix}(XY, i, i+1, 0, 1)\right| \\ 0.5\cdot M \end{array}\right.$$

Define function to calculate coordinates of centroid of non-intersecting closed polygon

$$\text{Centroid}(XY) := \left|\begin{array}{l} XY \leftarrow \text{stack}\left[XY, \left(\left(XY^T\right)^{\langle 0\rangle}\right)^T\right] \\ x \leftarrow XY^{\langle 0\rangle} \\ y \leftarrow XY^{\langle 1\rangle} \\ C_x \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left[\left(x_i + x_{i+1}\right)\cdot\left(x_i\cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ C_y \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left[\left(y_i + y_{i+1}\right)\cdot\left(x_i\cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ \left(C_x\ \ C_y\right)\cdot\dfrac{1}{6\cdot\text{Area}(XY)} \end{array}\right.$$

Area and Centroid of Concrete Gravity Dam

$$XY_{dam} := \begin{pmatrix} 0 & 611 \\ 0 & 628 \\ 2 & 628 \\ 7 & 638 \\ 7.5 & 638 \\ 2.5 & 628 \\ 19.75 & 628 \\ 21.75 & 626 \\ 51.75 & 618 \\ 54.75 & 615 \\ 54.75 & 611 \end{pmatrix}$$

- Values define cross-sectional geometry of dam, points are clockwise around cross section, starting at upstream heel.
- Left column is X coordinates, "0" is the upstream heel of the dam, sign convention is positive to the right (downstream).
- Right column is elevation.

$$-\mathrm{Area}\left(XY_{dam}\right) = 719.25$$

$$\mathrm{Centroid}\left(XY_{dam}\right) = (22.811 \quad 618.177)$$ center of gravity for concrete gravity dam, ft

$$x_{dam\_CG} := \mathrm{Centroid}\left(XY_{dam}\right)_{0,0} = 22.811$$

$$x_{dam} := x_{dam\_CG} \cdot 1ft = 22.811 \cdot ft$$ X-coordinate fo centroid, in feet

$$y_{dam} := \mathrm{Centroid}\left(XY_{dam}\right)_{0,1} = 618.177$$

$$El_{centroid} := y_{dam} \cdot 1ft = 618.177 \cdot ft$$ Elevation of centroid

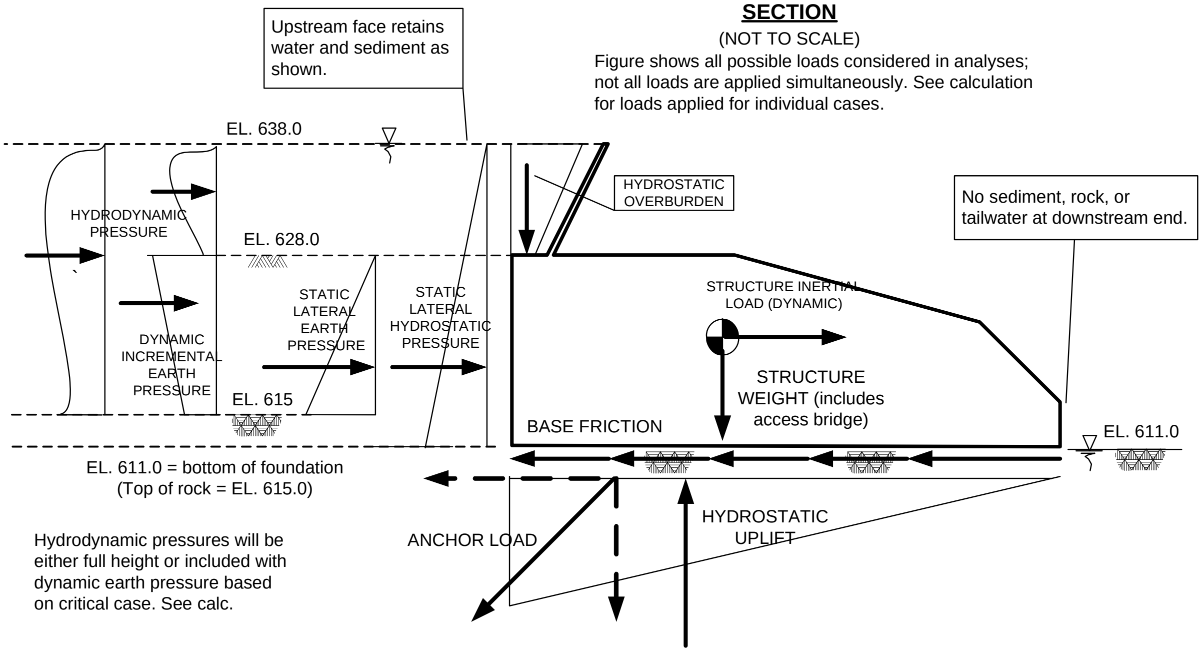
$$WT_{dam} := -\mathrm{Area}\left(XY_{dam}\right) \cdot ft^2 \cdot \gamma_{conc} + 5klf = 112.9 \cdot \frac{kip}{ft}$$

Total weight of concrete gravity dam, per foot. include weight of pedestrian bridge, estimated as 5 klf per Kevin Whittier.

Estimate Lateral Driving Forces Acting on Concrete Gravity Dam

FULL HEIGHT GATE SECTION

(NOT TO SCALE)



Lateral Hydrostatic Water Load on Upstream Face:

$H_w := El_{water.US} - El_{foundation} = 27 \cdot ft$ — Height of water

$F_{h2o} := \frac{1}{2} \cdot \gamma_w \cdot H_w^{\,2} = 22.7 \cdot \frac{kip}{ft}$ — Magnitude of resultant of hydrostatic load on upstream face of dam

$El_{h2o} := El_{water.US} - \frac{2}{3} \cdot H_w = 620 \cdot ft$ — Elevation of resultant

Static At-Rest Lateral Earth Pressure on Upstream Face:

Assume sediment contributesr at-rest soil pressure on upstream face of dam (active pressures are not developed).

$H_{ko} := El_{sed.top} - El_{rock} = 13 \cdot ft$

$K_0 := 1 - \sin(\phi_{sed}) = 0.531$ — At-rest soil pressure coefficient.

$F_{ko} := \frac{1}{2} \cdot K_0 \cdot (\gamma_{sed} - \gamma_w) \cdot H_{ko}^{\,2} = 2.58 \cdot \frac{kip}{ft}$ — Magnitude of resultant of at-rest soil pressure on upstream face of dam

$El_{ko} := El_{sed.top} - \frac{2}{3} \cdot H_{ko} = 619.3 \cdot ft$ — Elevation of resultant.

SEISMIC: Lateral Hydrodynamic Water Load on Upstream Face:

This load is applied assuming the dam has been flushed of sediment, and full height of water applies hydrodynamic loading to dam structure during a seismic event. Note that, when sediment levels accumulate, hydrodynamic loading is not considered to be a valid case.

$P_{hydro.MCE} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 1.356 \cdot klf$ — Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$P_{hydro.OBE} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 0.139 \cdot klf$ — Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$$P_{hydro.MCE.partial} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot \left(El_{crest} - El_{sill}\right)^2 = 0.256 \cdot klf$$

Magnitude of hydrodynamic loading over accumulated sediment.

$$P_{hydro.OBE.partial} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot \left(El_{crest} - El_{sill}\right)^2 = 0.026 \cdot klf$$

Magnitude of hydrodynamic loadign over accumulated sediment.

$$El_{hydro.MCE} := El_{rock} + 0.4 \cdot \left(El_{crest} - El_{rock}\right) = 624.2 \cdot ft$$

Elevation of resultant of hydrodynamic load for full height water case.

$$El_{hydro.OBE} := El_{rock} + 0.4 \cdot \left(El_{crest} - El_{rock}\right) = 624.2 \cdot ft$$

Elevation of resultant of hydrodynamic load for full height water case.

$$El_{hydro.MCE.partial} := El_{sed.top} + 0.4 \cdot \left(El_{crest} - El_{sed.top}\right) = 632 \cdot ft$$

Elevation of resultant of hydrodynamic load for full sediment case.

$$El_{hydro.OBE.partial} := El_{sed.top} + 0.4 \cdot \left(El_{crest} - El_{sed.top}\right) = 632 \cdot ft$$

Elevation of resultant of hydrodynamic load for full sediment case.

SEISMIC: Lateral Earth Pressures Upstream Face:

$\theta_{wall} := 0deg$ Slope of upstream face of dam, 0 indicates vertical face

$\delta_{sed} := 0deg$ Interface friction angle between sediment and dam, assume zero degrees.

$\beta_{US} := 0deg$ Slope of top of sediment against upstream face of dam. 0 degrees is horizontal.

$$f\_K_{A.c}(\phi, \delta, \beta, \theta) := \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\delta + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\delta + \phi) \cdot \sin(\phi - \beta)}{\cos(\delta + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

Function to calculate Coulomb active earth pressure coefficient

$$K_A := f\_K_{A.c}\left(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}\right) = 0.361$$

Coulomb active earth perssure coefficient

$$P_A := \frac{1}{2} \cdot K_A \cdot \left(\gamma_{sed} - \gamma_w\right)\left(El_{sed.top} - El_{rock}\right)^2 = 1.757 \cdot klf$$

Active earth pressure force.

$$f\_\psi\left(k_h, k_v\right) := atan\left(\frac{k_h}{1 - k_v}\right)$$

$$f\_K_{AE}(\phi, \delta, \beta, \theta, \psi) := \frac{\cos(\phi - \psi - \theta)^2}{\cos(\psi) \cdot \cos(\theta)^2 \cdot \cos(\psi + \theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \psi - \beta)}{\cos(\delta + \psi + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

Check MCE:

$$\psi_{MCE} := f\_\psi\left(k_{h.MCE}, k_v\right) = 0.07$$

$K_{AE.MCE} := f\_K_{AE}\left(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{MCE}\right) = 0.406$

Examine active case only for upstream sediment. Neglect any downstream passive resistance.

$P_{AE.MCE} := \left(0.5 \cdot K_{AE.MCE}\right) \cdot \left(\gamma_{sed} - \gamma_w\right)\left(El_{sed.top} - El_{rock}\right)^2 = 1.977 \cdot klf$

Seismic active earth pressure.

$\Delta P_{AE.MCE} := P_{AE.MCE} - P_A = 0.22 \cdot klf$

Dynamic incremental earth pressure in seismic MCE case.

$F_{MCE.partial} := \Delta P_{AE.MCE} + P_{hydro.MCE.partial} = 0.476 \cdot klf$

Soil + hydrodynamic water load above soil, total horiztonal applied force.

Check OBE:

$\psi_{OBE} := f\_\psi\left(k_{h.OBE}, k_v\right) = 0.007$

$K_{AE.OBE} := f\_K_{AE}\left(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{OBE}\right) = 0.365$

Examine active case only for upstream sediment. Neglect any downstream passive resistance.

$P_{AE.OBE} := \left(0.5 \cdot K_{AE.OBE}\right) \cdot \left(\gamma_{sed} - \gamma_w\right)\left(El_{sed.top} - El_{rock}\right)^2 = 1.778 \cdot klf$

Active seismic earth pressure between top of sediment and top of rock.

$\Delta P_{AE.OBE} := P_{AE.OBE} - P_A = 0.021 \cdot klf$

Additional applied earth pressure in seismic OBE case.

$F_{OBE.partial} := \Delta P_{AE.OBE} + P_{hydro.OBE.partial} = 0.047 \cdot klf$

Soil + hydrodynamic water load above soil, total horiztonal applied force.

Compare Dynamic Lateral Earth Pressures to At-Rest Lateral Earth Pressures:

$P_{AE.OBE} = 1.778 \cdot klf$ Dynamic Active OBE

$P_{AE.MCE} = 1.977 \cdot klf$ Dynamic Active MCE

$F_{ko} = 2.582 \cdot klf$ Static At-Rest

Note that static at-rest loading is greater than dynamic active loading for both MCE and OBE cases. Use greater of static at-rest or dynamic active lateral earth pressures. In this case, static at-rest pressure controls and should be used as the lateral earth pressure for the dynamic analysis cases..

Determine controlling load case for upstream loading on structure:

Structure could be free water (no sediment accumulation), or filled with sediment. For seismic stability evaluations, estimate controlling case: either hydrodynamic loading of silt-free dam or dynamic lateral earth pressure + water over top of sediment.

$F_{ko} = 2.582 \cdot klf$ At-rest lateral earth pressure loading (note that static at-rest is controlling case for seismic evaluation)

$F_{h2o} = 22.745 \cdot klf$ Hydrostatic pressure

$P_{hydro.MCE} = 1.356 \cdot klf$ Hydrodynamic pressure over full height of structure, MCE event

$P_{hydro.OBE} = 0.139 \cdot klf$ Hydrodynamic pressure over full height of structure, OBE event

$P_{hydro.MCE.partial} = 0.256 \cdot klf$ Hydrodynamic pressure above top of sediment, MCE event. Include with soil loading case.

$P_{hydro.OBE.partial} = 0.026 \cdot klf$ Hydrodynamic pressure above top of sediment, OBE event. Include with soil loading case.

$P_{hydro.MCE} + F_{h2o} = 24.1 \cdot klf$

$F_{h2o} + F_{ko} + P_{hydro.MCE.partial} = 25.583 \cdot klf$

$$check_{hydro.MCE} := \begin{cases} \text{"Hydrodynamic"} & \text{if } \left(P_{hydro.MCE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko} + P_{hydro.MCE.partial}\right) \\ \text{"Soil"} & \text{otherwise} \end{cases}$$

$check_{hydro.MCE} = \text{"Soil"}$

$P_{hydro.OBE} + F_{h2o} = 22.883 \cdot klf$

$F_{h2o} + F_{ko} + P_{hydro.OBE.partial} = 25.353 \cdot klf$

$$check_{hydro.OBE} := \begin{cases} \text{"Hydrodynamic"} & \text{if } \left(P_{hydro.OBE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko} + P_{hydro.OBE.partial}\right) \\ \text{"Soil"} & \text{otherwise} \end{cases}$$

$check_{hydro.OBE} = \text{"Soil"}$

SEISMIC: Inertial Load of Structure:

$F_{inertia.MCE} := k_{h.MCE} \cdot WT_{dam} = 7.947 \cdot klf$ Seismic inertia load of the dam for MCE, acts in downstream direction.

$F_{inertia.OBE} := k_{h.OBE} \cdot WT_{dam} = 0.813 \cdot klf$ Seismic inertia load of the dam for OBE acts in downstream direction.

$El_{inertia.MCE} := El_{centroid} = 618.177 \cdot ft$ Seismic inertial load acts through centroid of dam

$El_{inertia.OBE} := El_{centroid} = 618.177 \cdot ft$ Seismic inertial load acts through centroid of dam

Estimate Uplift Hydrostatic Forces Acting on Concrete Gravity Dam

Hydrostatic Uplift on Dam Base:

Magnitude of hydrostatic uplift is estimated as straightline interpolation between headwater and tailwater across width of structure. Figure above shows assumed uplift distribution below bottom of dam.

Use centroid equation to define uplift pressure.

$$XY_{uplift} := \begin{pmatrix} w_{foundation} & 0 \\ w_{foundation} & El_{water.DS} - El_{foundation} \\ 0 & El_{water.US} - El_{foundation} \\ 0 & 0 \end{pmatrix} \cdot ft^{-1}$$

$$w_{foundation} = 54.75 \cdot ft$$

$$Area(XY_{uplift}) = 739.125$$

$$Centroid(XY_{uplift}) = (18.25 \quad 9)$$

$$X_{uplift} := (1ft\,Centroid(XY_{uplift}))_{0,0} = 18.25 \cdot ft$$

$$F_{uplift} := Area(XY_{uplift}) \cdot ft^2 \cdot \gamma_w = 46.121 \cdot \frac{kip}{ft}$$

Estimate Resisting Forces:

Estimate base sliding resistance for concrete gravity dam sliding on rock. Account for hydrostatic overburden above upstream face dam:

Hydrostatic Overburden Volume above upstream face of Dam:

$$XY_{hydroOB} := \begin{pmatrix} 0 & 628 \\ 2 & 628 \\ 7 & 638 \\ 0 & 638 \end{pmatrix}$$

$$Area(XY_{hydroOB}) = 45$$

$$Centroid(XY_{hydroOB}) = (2.481 \quad 633.926)$$

$$x_{h2o.vert} := (1ft\,Centroid(XY_{hydroOB}))_{0,0} = 2.481 \cdot ft$$

$$F_{h2o.vert} := Area(XY_{hydroOB}) \cdot ft^2 \cdot \gamma_w = 2.808 \cdot \frac{kip}{ft}$$

Interface friction between concrete gravity dam and shale bedrock:

$\delta_{base} = 24 \cdot deg$ Base friction angle between dam and foundation.

$F_{base} := (WT_{dam} + F_{h2o.vert} - F_{uplift}) \cdot \tan(\delta_{base}) = 31 \cdot \frac{kip}{ft}$ Base friction, sum of vertical forces multiplied by tangent of interface friction times tangent of interface friction (delta).

Estimate Factor of Safety Against Sliding:

The recommended global stability design criteria is summarized in the USACE Gravity Dam Design EM 1110-2-2200. Stability criteria is summarized in Table 4-1 below.

EM 1110-2-2200
30 Jun 95

Table 4-1
Stability and stress criteria

Load Condition	Resultant Location at Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress	
				Compressive	Tensile
Usual	Middle 1/3	2.0	≤ allowable	$0.3\,f'_c$	0
Unusual	Middle 1/2	1.7	≤ allowable	$0.5\,f'_c$	$0.6\,f_c'^{2/3}$
Extreme	Within base	1.3	≤ 1.33 × allowable	$0.9\,f'_c$	$1.5\,f_c'^{2/3}$

Note: f'_c is 1-year unconfined compressive strength of concrete. The sliding factors of safety (*FS*) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions.

$FS_{min} := 2.0$ Usual loading. Minimum sliding factor of safety recommended by USACE (from table above)

$\Sigma F_{h.drive} := F_{h2o} + F_{ko} = 25.3 \cdot \frac{kip}{ft}$ Sum of driving forces (hydrostatic pressure + at rest lateral earth pressure)

$\Sigma F_{h.resist} := F_{base} = 31 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$$FS_{slide.shale} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive}} = 1.22$$

$$check_{slide.shale} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale} > FS_{min} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$check_{slide.shale} = \text{"NOT OK-anchors required"}$

MCE Seismic Sliding:

$FS_{min.MCE} := 1.3$ MCE, extreme loading. Minimum sliding factor of safety recommended by USACE (from table above).

$$\Sigma F_{h.drive.MCE} := F_{h2o} + F_{ko} + P_{hydro.MCE.partial} + F_{inertia.MCE} = 33.5 \cdot \frac{kip}{ft}$$

Sum of lateral driving forces during MCE event.

$\Sigma F_{h.resist} := F_{base} = 31 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$$FS_{slide.shale.MCE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.MCE}} = 0.92$$

$$check_{slide.shale.MCE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.MCE} > FS_{min.MCE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.MCE} = \text{"NOT OK-anchors required"}$$

OBE Sliding:

$FS_{min.OBE} := 1.7$

OBE, Unusual loading. Minimum sliding factor of safety recommended by USACE (from table above).

$$\Sigma F_{h.drive.OBE} := F_{h2o} + F_{ko} + P_{hydro.OBE.partial} + F_{inertia.OBE} = 26.2 \cdot \frac{kip}{ft}$$

Sum of lateral driving forces during OBE event.

$$\Sigma F_{h.resist} := F_{base} = 31 \cdot \frac{kip}{ft}$$

Sum of resisting forces (base friction)

$$FS_{slide.shale.OBE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.OBE}} = 1.18$$

$$check_{slide.shale.OBE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.OBE} > FS_{min.OBE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.OBE} = \text{"NOT OK-anchors required"}$$

Estimate Required Anchor Force to Acheive Minimum Sliding Factor of Safety:

$$F_{anchor} := FS_{min} \cdot \Sigma F_{h.drive} - \Sigma F_{h.resist} = 19.678 \cdot klf$$

Use minimum FS against sliding to determine anchor forc This is the horizontal component of the anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.static} := \frac{F_{anchor}}{\cos(\alpha_{anchor})} = 27.828 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Check Seismic:

MCE case:

$$F_{anchor.MCE} := FS_{min.MCE} \cdot \Sigma F_{h.drive.MCE} - \Sigma F_{h.resist} = 12.613 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of tl anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.MCE} := \frac{F_{anchor.MCE}}{\cos(\alpha_{anchor})} = 17.838 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

OBE case:

$$F_{anchor.OBE} := FS_{min.OBE} \cdot \Sigma F_{h.drive.OBE} - \Sigma F_{h.resist} = 13.506 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of tl anchor.

$$\alpha_{anchor} := 45deg$$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.OBE} := \frac{F_{anchor.OBE}}{\cos(\alpha_{anchor})} = 19.1 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Determine Critical Anchor Force for Design:

$$\boxed{T_{anchor.critical} := \max(T_{anchor.static}, T_{anchor.MCE}, T_{anchor.OBE}) = 27.828 \cdot klf}$$

Estimate Factor of Safety Against Overturning:

Sum moments around downstream toe. Note this is not directly comparable to USACE overturning criteria but useful as a quick che of stability, see estimation of overturning resultant and % base compression below.

$$\Sigma M_{toe.drive} := F_{ko} \cdot (El_{ko} - El_{foundation}) + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \ldots + F_{uplift} \cdot (w_{foundation} - X_{uplift}) = 1909.653 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma M_{toe.resist} := WT_{dam} \cdot (w_{foundation} - x_{dam}) + F_{h2o.vert} \cdot (w_{foundation} - x_{h2o.vert}) = 3752.334 \cdot \frac{kip \cdot ft}{ft}$$

$$\boxed{FS_{overturning} := \frac{\Sigma M_{toe.resist}}{\Sigma M_{toe.drive}} = 1.96}$$

Factor of safety against overturning, static case.

There is no specified factor of safety provided by USACE against overturning. The USACE does recommend that for the Normal/Usual loading scenario, the overturning resultant should be located within the middle 1/3 of the base of the dam, and for the unusual loading scenario, the middle 1/2 of the dam.

Estimate Location of Overturning Resultant:

Static Case:

Check that location of overturning resultant falls in middle 1/3 of base of concrete gravity dam (usual case)

$$\Sigma M_{toe.total} := \Sigma M_{toe.drive} - \Sigma M_{toe.resist} = -1842.682 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.total} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 69.574 \cdot \frac{kip}{ft}$$

$$X_{Resultant} := \frac{-\Sigma M_{toe.total}}{\Sigma F_{vertical.total}} = 26.5 \cdot ft$$

horizontal distance to resultant of overturning moment relative to face of the wall

$$\frac{1}{3} \cdot w_{foundation} = 18.3 \cdot ft$$

defines middle third of base

$$\frac{2}{3} \cdot w_{foundation} = 36.5 \cdot ft$$

defines middle third of base

$$check_{OT} := \begin{cases} \text{"OK"} & \text{if } \frac{1}{3} w_{foundation} \le X_{Resultant} \le \frac{2}{3} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$check_{OT} = \text{"OK"}$$

OBE Case:

Check that location of overturning resultant falls in middle 1/2 of base of concrete gravity dam (unusual case)

$$\begin{aligned} \Sigma M_{toe.drive.OBE} := {} & F_{ko} \cdot \left(El_{ko} - El_{foundation}\right) \ldots \\ & + F_{h2o} \cdot \left(El_{h2o} - El_{foundation}\right) \ldots \\ & + F_{uplift} \cdot \left(w_{foundation} - X_{uplift}\right) \ldots \\ & + F_{inertia.OBE} \cdot \left(El_{inertia.OBE} - El_{foundation}\right) \ldots \\ & + P_{hydro.OBE.partial} \cdot \left(El_{hydro.OBE.partial} - El_{foundation}\right) \end{aligned} = 1916.036 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.OBE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 69.574 \cdot \frac{kip}{ft}$$

$$X_{Resultant.OBE} := \frac{-\Sigma M_{toe.drive.OBE}}{\Sigma F_{vertical.total}} = -27.5 \cdot ft$$

horizontal distance to resultant of overturning moment relative to face of the dam

$$\frac{1}{4} \cdot w_{foundation} = 13.7 \cdot ft$$

defines middle half of base

$$\frac{3}{4} \cdot w_{foundation} = 41.1 \cdot ft$$

defines middle half of base

$$check_{OT.OBE} := \begin{cases} \text{"OK"} & \text{if } \frac{1}{4} w_{foundation} \le X_{Resultant} \le \frac{3}{4} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{OT.OBE} = \text{"OK"}}$$

MCE Case:

Check that location of overturning resultant falls within base of concrete gravity dam (extreme case)

$$\begin{aligned} \Sigma M_{toe.drive.MCE} := {} & F_{ko} \cdot (El_{ko} - El_{foundation}) \ldots \\ & + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \ldots \\ & + F_{uplift} \cdot (w_{foundation} - X_{uplift}) \ldots \\ & + F_{inertia.MCE} \cdot (El_{inertia.MCE} - El_{foundation}) \ldots \\ & + P_{hydro.MCE.partial} \cdot (El_{hydro.OBE.partial} - El_{foundation}) \end{aligned} = 1972.068 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.MCE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 69.574 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant.MCE} := \frac{-\Sigma M_{toe.drive.MCE}}{\Sigma F_{vertical.total}} = -28.3 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the dam

$0 \cdot w_{foundation} = 0 \cdot ft$ defines upstream edge of base

$1 \cdot w_{foundation} = 54.7 \cdot ft$ defines downstream edge of base

$$check_{OT.MCE} := \begin{cases} \text{"OK"} & \text{if } 0 w_{foundation} \le X_{Resultant} \le 1 w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{OT.MCE} = \text{"OK"}}$$

Remarks and Recapitulation:

- Calculation addresses sliding and overturning of the fixed crest section of Sand Springs Dam under anticipated static operating conditions, OBE seismic case, and MCE seismic case noted.
- For all cases, it is identified that permanent ground anchors are necessary for sliding stability.
- Anchors are not necessary for overturning stability.
- The static case (usual loading) was found to control.

Attachment 7

Evaluate Sliding and Overturning
Fixed Crest Case

CH2MHILL

PROJECT : Arkansas River Corridor Project - South Tulsa Jenks Dam
PROJECT #: 657971.04.02.01
CREATED BY: Jen Schaeffer/SEA **DATE:** 04/16/2015
REVIEWED BY: Mark Kacmarcik/CVO **DATE:** 04/16/2015

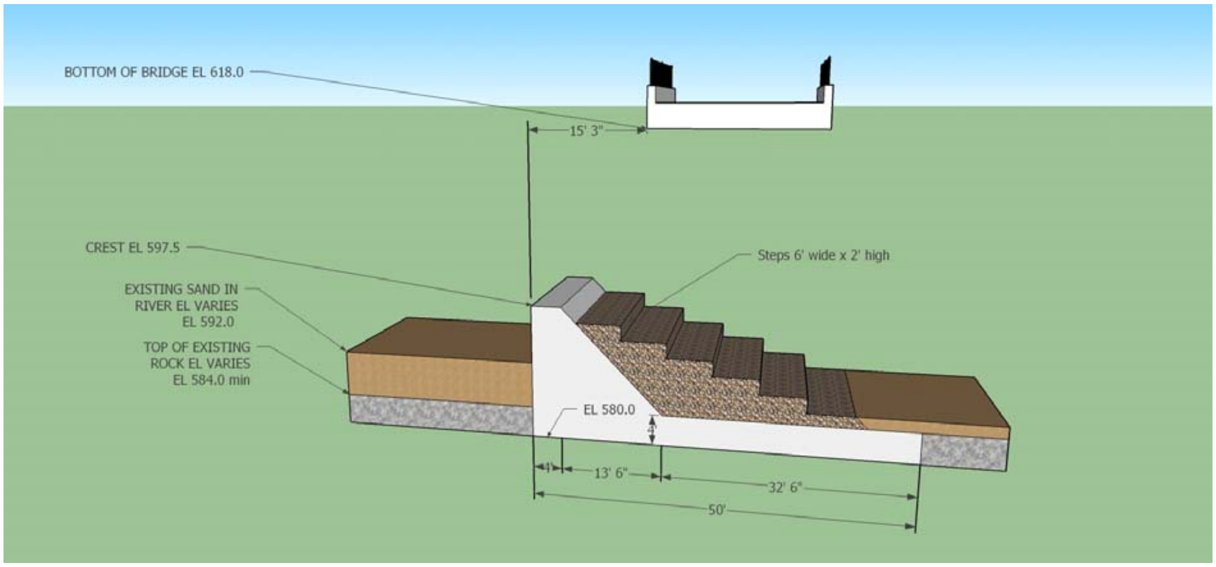
Given: Simplified gravity dam geometry shown and generalized subsurface profile. See sketch.
Find: Check sliding and overturning against USACE criteria for Static and Seismic cases as noted in the title. Anchor forces are included as needed to meet stability criteria. Note that this is not attempt to be a complete comprehensive check of all possible analysis cases, but rather the loading cases which are assumed to control overall dam design for preliminary sizing and concept evaluation.

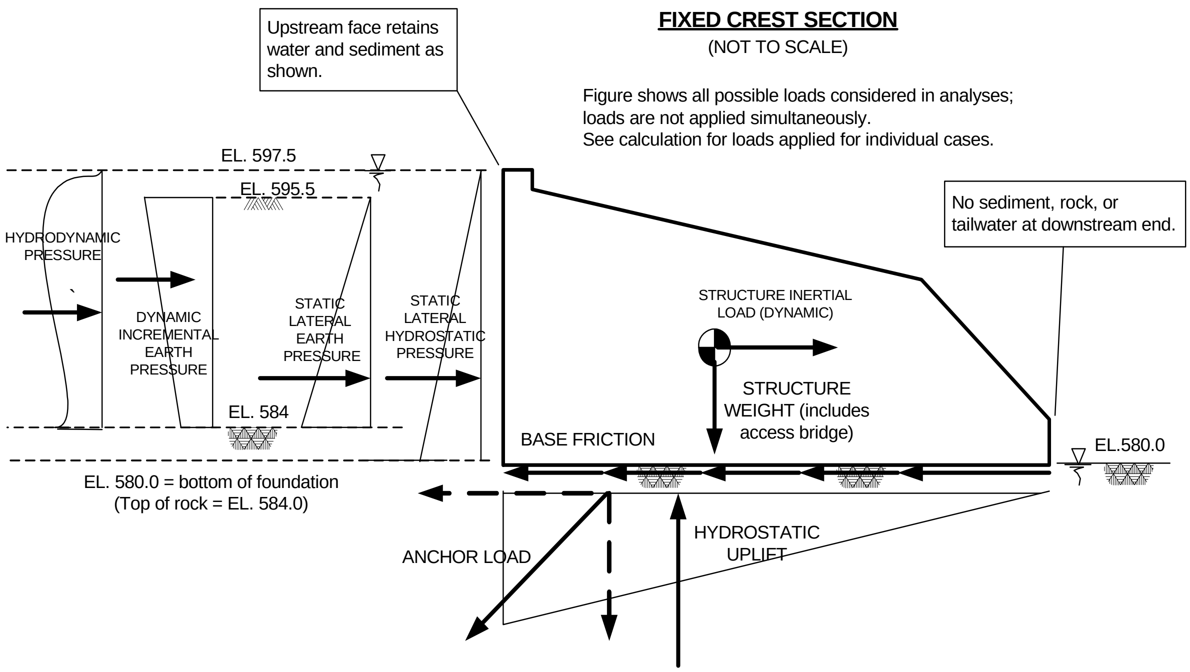
Assumptions: Ignore resistance from sediment or rock on downstream toe.
Ice loading is not considered.
Structure is not undermined by scour
Upstream and downstream turndowns (not shown) are not relied upon for shear resistance.
All soil and rock layers are assumed to be horizontal.
Use single conservative frictional interface strength, as shown in the calculation.
Disregard cohesion for long term analysis.
Mass or contributions of pedestrian bridge ignored (conservative)
2 dimensional analysis considering dam geometry on a per-foot basis, 3Dimensional end effects not considered.
Steps shown in geometry are concrete or cut stone with similar unit weight to mass concrete.
Other assumptions as noted in the calculation

Inputs: Approximate top of rock elevation for main dam, estimated at **El 584 ft**.
Dam foundation elevation assumed 4 feet below top of rock **(El 580 ft.)**
Water present to top of fixed crest at **El 597.5 ft**.
Sediment elevation varies from top of rock **(El 584 ft)** to 2 feet below top of crest **(El 595.5 ft)** as directed by Murry Fleming.
Tailwater elevation is coincident with dam foundation, **El 580.0 ft**.
2008 boreholes by Stantec used to estimate subsurface conditions and properties.
Other inputs as noted in the calculation.

References: USACE EM 1110-2-2200 Gravity Dam Manual
USACE EM 1110-2-2100 Stability Analysis of Concrete Structures

Fixed Crest Section Geometry:





Define Geometry:

$El_{crest} := 597.5ft$ — Elevation at top of gravity dam

$El_{rock} := 584ft$ — Elevation of top of rock (shale)

$d_{excav} := 4ft$ — Excavate below top of rock to remove weathered shale

$El_{foundation} := El_{rock} - d_{excav} = 580 \cdot ft$ — Elevation of bottom of dam

$H_{dam} := El_{crest} - El_{foundation} = 17.5 \cdot ft$ — Total height of dam

$w_{foundation} := 50.0ft$ — Given width of dam base

$El_{sed.top} := El_{crest} - 2ft = 595.5 \cdot ft$

Dam collects sediment to within 2 feet of crest elevation.

$El_{water.US} := El_{crest} = 597.5 \cdot ft$

Elevation of water upstream of dam.

$El_{water.DS} := El_{foundation} = 580 \cdot ft$

Elevation of water downstream of dam (assume no water as recommended by USACE)

Material Properties:

Unit Weight:

$\gamma_{conc} := 150pcf$

Unit weight of concrete (assumed)

$\gamma_{sed} := 120pcf$

Unit weight of sediment against upstream face of dam (recommended by USACE EM 1110-2-2100)

$\gamma_{shale} := 152pcf$

Unit weight of Shale from Stantec, 2008 laboratory test results.

$\gamma_w := 62.4pcf$

Unit weight of water (assumed)

Shear Strength:

$\phi_{sed} := 28deg$ $c_{sed} := 0psf$

Effective stress shear strength of sediment.

Interface Strength (sliding):

$\delta_{base} := 24deg$

Consider only one sliding interface, mass concrete cast against shale bedrock. Assume no cohesion/adhesion along this interface, only base friciton. Typical value from NAVFAC DM7.2 for "Mass concrete cast against...very stiff and hard residual or preconsolidated clay"

Seismic:

$PGA_{OBE} := 0.009$

Peak ground acceleration on rock for Operations Basis Earthquake (OBE). 50% probability of exceedance in 100 years.

$PGA_{MCE} := 0.091$

Peak ground acceleration on rock for Maximum Credible Earthquake (MCE). 10% probability of exceedance in 50 years

$F_{PGA.scC} := 1.2$

Site coefficient for Site Class C, "Very Dense Soil and Soft Rock" (assumed).

$k_{h.MCE} := \frac{2}{3} \cdot PGA_{MCE} \cdot F_{PGA.scC} = 0.073$

Seismic coeff for MCE case (per EM 1110-2-2100 = 2/3 effective peak ground accel). Conservatively estimated using PGA for site class C.

$k_{h.OBE} := \frac{2}{3} \cdot PGA_{OBE} \cdot F_{PGA.scC} = 0.007$

Seismic coeff for OBE case.

$k_v := 0$

Neglect vertical component of earthquake accelleration (assumed).

Estimate Weight of Concrete Gravity Dam:

Estimate total (non buoyant) weight of concrete gravity dam by estimating area of the gravity dam polygon, and then multiplying it by the unit weight of the material. Use centroid function to for irregular dam geometry.

Centroid of polygon [edit] from Wikipedia (http://en.wikipedia.org/wiki/Polygon, February 27, 2014)

The centroid of a non-self-intersecting closed polygon defined by n vertices (x_0,y_0), (x_1,y_1), ..., (x_{n-1},y_{n-1}) is the point (C_x, C_y), where

$$C_{\mathrm{x}} = \frac{1}{6A}\sum_{i=0}^{n-1}(x_i + x_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

$$C_{\mathrm{y}} = \frac{1}{6A}\sum_{i=0}^{n-1}(y_i + y_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

and where A is the polygon's signed area,

$$A = \frac{1}{2}\sum_{i=0}^{n-1}(x_i\ y_{i+1} - x_{i+1}\ y_i).$$ [9]

In these formulas, the vertices are assumed to be numbered in order of their occurrence along the polygon's perimeter, and the vertex (x_n, y_n) is assumed to be the same as (x_0, y_0). Note that if the points are numbered in clockwise order the area A, computed as above, will have a negative sign; but the centroid coordinates will be correct even in this case.

Define function to calculate area of polygon whose plane coordinates are contained in matrix XY

$$\mathrm{Area}(XY) := \left| \begin{array}{l} XY \leftarrow \mathrm{stack}\left[XY, \left(\left(XY^T\right)^{\langle 0 \rangle}\right)^T\right] \\ M \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left|\mathrm{submatrix}(XY, i, i+1, 0, 1)\right| \\ 0.5 \cdot M \end{array} \right.$$

Define function to calculate coordinates of centroid of non-intersecting closed polygon

$$\text{Centroid}(XY) := \left| \begin{array}{l} XY \leftarrow \text{stack}\left[XY, \left(XY^{T}\right)^{\langle 0 \rangle^{T}}\right] \\ x \leftarrow XY^{\langle 0 \rangle} \\ y \leftarrow XY^{\langle 1 \rangle} \\ C_x \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left[\left(x_i + x_{i+1}\right)\cdot\left(x_i \cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ C_y \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left[\left(y_i + y_{i+1}\right)\cdot\left(x_i \cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ \left(C_x \;\; C_y\right)\cdot\dfrac{1}{6\cdot\text{Area}(XY)} \end{array} \right.$$

Area and Centroid of Concrete Gravity Dam

$$XY_{dam} := \begin{pmatrix} 0 & 580.0 \\ 0 & 597.5 \\ 4 & 597.5 \\ 6 & 595.5 \\ 42 & 585.5 \\ 43.5 & 584 \\ 50 & 584 \\ 50 & 580.0 \end{pmatrix}$$

- Values define cross-sectional geometry of dam, points are clockwise around cross section, starting at upstream heel.
- Left column is X coordinates, "0" is the upstream heel of the dam, sign convention is positive to the right (downstream).
- Right column is elevation.

$-\mathrm{Area}(XY_{dam}) = 514.125$ cross sectional area of dam section

$\mathrm{Centroid}(XY_{dam}) = (19.093 \quad 586.008)$ coordinates of center of gravity of concrete gravity dam, ft

$x_{dam\_CG} := \mathrm{Centroid}(XY_{dam})_{0,0} = 19.093$

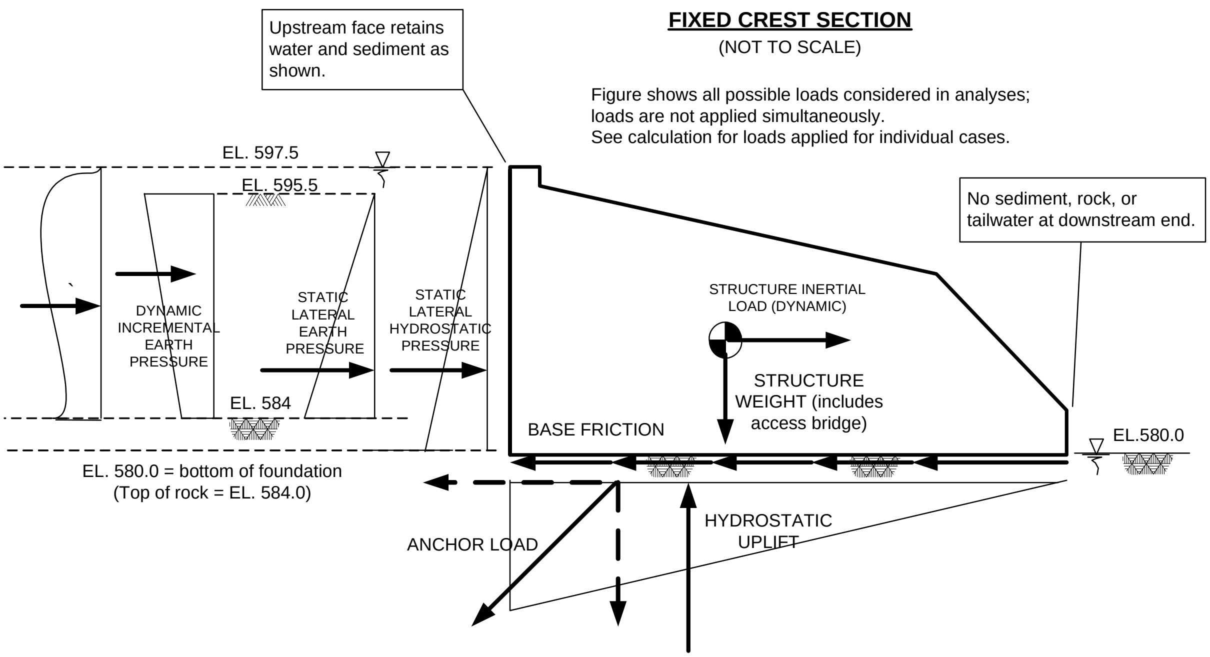
$x_{dam} := x_{dam\_CG} \cdot 1ft = 19.093 \cdot ft$ X-coordinate fo centroid, in feet

$y_{dam} := \mathrm{Centroid}(XY_{dam})_{0,1} = 586.008$

$El_{centroid} := y_{dam} \cdot 1ft = 586.008 \cdot ft$ Elevation of centroid

$WT_{dam} := -\mathrm{Area}(XY_{dam}) \cdot ft^2 \cdot \gamma_{conc} + 5klf = 82.1 \cdot \frac{kip}{ft}$ Multiply cross-sectional area by unit weight of concrete to estimate total weight of concrete gravity dam, per lineal foot. Include 5 kips/lf for pedestrian bridge per Kevin Whittier's estimate.

Estimate Lateral Driving Forces Acting on Concrete Gravity Dam



Lateral Hydrostatic Water Load on Upstream Face:

$$H_w := El_{water.US} - El_{foundation} = 17.5 \cdot ft$$ Height of water

$$F_{h2o} := \frac{1}{2} \cdot \gamma_w \cdot H_w^{\ 2} = 9.6 \cdot \frac{kip}{ft}$$ Magnitude of resultant of hydrostatic load on upstream face of dam

$$El_{h2o} := El_{water.US} - \frac{2}{3} \cdot H_w = 585.8 \cdot ft$$ Elevation of resultant

Static At-Rest Lateral Earth Pressure on Upstream Face:

Assume sediment contributesr at-rest soil pressure on upstream face of dam (active pressures are not developed).

$$H_{ko} := El_{sed.top} - El_{rock} = 11.5 \cdot ft$$ Maximum sediment accumulation extends from top of rock to 2 feet below fixed crest. Assume no lateral earth pressure from silt below top of rock.

$$K_0 := 1 - \sin(\phi_{sed}) = 0.531$$ At-rest lateral earth pressure coefficient.

$$F_{ko} := \frac{1}{2} \cdot K_0 \cdot (\gamma_{sed} - \gamma_w) \cdot H_{ko}^{\ 2} = 2.02 \cdot \frac{kip}{ft}$$ Magnitude of resultant of at-rest soil pressure on upstream face of dam (use buoyant unit weight)

$$El_{ko} := El_{sed.top} - \frac{2}{3} \cdot H_{ko} = 587.8 \cdot ft$$ Elevation of resultant.

Lateral Hydrodynamic Water Load on Upstream Face:

This load is applied assuming the dam has been flushed of sediment, and full height of water applies hydrodynamic loading to dam structure during a seismic event. Note that, when sediment levels accumulate, hydrodynamic loading is not considered to b a valid case.

$$P_{hydro.MCE} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 0.483 \cdot klf$$ Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$$P_{hydro.OBE} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot \left(El_{crest} - El_{rock}\right)^2 = 0.048 \cdot klf$$

Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$$El_{hydro.MCE} := El_{rock} + 0.4 \cdot \left(El_{crest} - El_{rock}\right) = 589.4 \cdot ft$$

Elevation of resultant of hydrodynamic load.

$$El_{hydro.OBE} := El_{rock} + 0.4 \cdot \left(El_{crest} - El_{rock}\right) = 589.4 \cdot ft$$

Elevation of resultant of hydrodynamic load.

<u>SEISMIC: Lateral Earth Pressures Upstream Face:</u>

$\theta_{wall} := 0deg$ — Slope of upstream face of dam, 0 indicates vertical face

$\delta_{sed} := 0deg$ — Interface friction angle between sediment and dam, assume zero degrees.

$\beta_{US} := 0deg$ — Slope of top of sediment against upstream face of dam. 0 degrees is horizontal.

Define function to calculate Coulomb active lateral earth pressure coefficient:

$$f\_K_{A.c}(\phi, \delta, \beta, \theta) := \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\delta + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\delta + \phi) \cdot \sin(\phi - \beta)}{\cos(\delta + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

$$K_A := f\_K_{A.c}\left(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}\right) = 0.361$$

Coulomb active lateral earth pressure coefficient.

$$P_A := \frac{1}{2} \cdot K_A \cdot \left(\gamma_{sed} - \gamma_w\right)\left(El_{sed.top} - El_{rock}\right)^2 = 1.375 \cdot klf$$

Coulomb active lateral earth pressure.

$$f\_\psi\left(k_h, k_v\right) := atan\left(\frac{k_h}{1 - k_v}\right)$$

Define function to calculate dynamic lateral earth pressure coefficient (KAE)

$$f\_K_{AE}(\phi, \delta, \beta, \theta, \psi) := \frac{\cos(\phi - \psi - \theta)^2}{\cos(\psi) \cdot \cos(\theta)^2 \cdot \cos(\psi + \theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \psi - \beta)}{\cos(\delta + \psi + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

<u>Estimate Dynamic Lateral Earth Pressure for Maximum Credible Earthquake (MCE):</u>

$$\psi_{MCE} := f\_\psi\left(k_{h.MCE}, k_v\right) = 0.073$$

$$K_{AE.MCE} := f\_K_{AE}\left(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{MCE}\right) = 0.408$$

$P_{AE.MCE} := \frac{1}{2} K_{AE.MCE} \cdot (\gamma_{sed} - \gamma_w)(El_{sed.top} - El_{rock})^2 = 1.553 \cdot klf$

Total dynamic active earth pressure (static-active plus dynamic)

$\Delta P_{AE.MCE} := P_{AE.MCE} - P_A = 0.178 \cdot klf$

Dynamic increment in MCE case.

Estimate Dynamic Lateral Earth Pressure for Operations Basis Earthquake (OBE):

$\psi_{OBE} := f\_\psi(k_{h.OBE}, k_v) = 0.007$

$K_{AE.OBE} := f\_K_{AE}(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{OBE}) = 0.365$

$P_{AE.OBE} := \frac{1}{2} K_{AE.OBE} \cdot (\gamma_{sed} - \gamma_w)(El_{sed.top} - El_{rock})^2 = 1.392 \cdot klf$

Total dynamic active earth pressure (static-active plus dynamic)

$\Delta P_{AE.OBE} := P_{AE.OBE} - P_A = 0.017 \cdot klf$

Additional applied earth pressure in seismic OBE case.

Compare Dynamic Lateral Earth Pressures to At-Rest Lateral Earth Pressures:

$P_{AE.OBE} = 1.392 \cdot klf$ Dynamic Active OBE

$P_{AE.MCE} = 1.553 \cdot klf$ Dynamic Active MCE

$F_{ko} = 2.021 \cdot klf$ Static At-Rest

Note that static at-rest loading is greater than dynamic active loading for both MCE and OBE cases. Use greater of static at-rest dynamic active lateral earth pressures. In this case, static at-rest pressure controls and should be used as the lateral earth pressure for the dynamic analysis cases.

Determine controlling load case for upstream loading on structure:

Structure couuld be free water (no sediment accumulation), or filled with sediment. For seismic stability evaluations, estimate controlling case: either hydrodynamic loading of silt-free dam or dynamic lateral earth pressure of silted-in dam.

$F_{ko} = 2.021 \cdot klf$ Lateral earth pressure loading (note that static at-rest is controlling case for seismic evaluation)

$F_{h2o} = 9.555 \cdot klf$ Hydrostatic pressure

$P_{hydro.MCE} = 0.483 \cdot klf$ Hydrodynamic pressure, MCE event

$P_{hydro.OBE} = 0.048 \cdot klf$ Hydrodynamic pressure, OBE event

$P_{hydro.MCE} + F_{h2o} = 10.038 \cdot klf$

$F_{h2o} + F_{ko} = 11.576 \cdot klf$

$$check_{hydro.MCE} := \begin{cases} \text{"Hydrodynamic"} & \text{if } \left(P_{hydro.MCE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko}\right) \\ \text{"Soil"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{hydro.MCE} = \text{"Soil"}}$$

$$P_{hydro.OBE} + F_{h2o} = 9.603 \cdot klf$$

$$F_{h2o} + F_{ko} = 11.576 \cdot klf$$

$$check_{hydro.OBE} := \begin{cases} \text{"Hydrodynamic"} & \text{if } \left(P_{hydro.OBE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko}\right) \\ \text{"Soil"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{hydro.OBE} = \text{"Soil"}}$$

SEISMIC: Inertial Load of Structure:

$$F_{inertia.MCE} := k_{h.MCE} \cdot WT_{dam} = 5.978 \cdot klf$$

Seismic inertia load of the dam for MCE, acts in downstream direction.

$$F_{inertia.OBE} := k_{h.OBE} \cdot WT_{dam} = 0.591 \cdot klf$$

Seismic inertia load of the dam for OBE acts in downstream direction.

$$El_{inertia.MCE} := El_{centroid} = 586.008 \cdot ft$$

Seismic inertial load acts through centroid of dam

$$El_{inertia.OBE} := El_{centroid} = 586.008 \cdot ft$$

Seismic inertial load acts through centroid of dam

Estimate Uplift Hydrostatic Forces Acting on Concrete Gravity Dam

Hydrostatic Uplift on Dam Base:

Magnitude of hydrostatic uplift is estimated as straightline interpolation between headwater and tailwater. Figure above shows uplift distribution below bottom of dam.

Use centroid equation to define uplift pressure.

$$XY_{uplift} := \begin{pmatrix} w_{foundation} & 0 \\ w_{foundation} & El_{water.DS} - El_{foundation} \\ 0 & El_{water.US} - El_{foundation} \\ 0 & 0 \end{pmatrix} \cdot ft^{-1}$$

$$Area\left(XY_{uplift}\right) = 437.5$$

$$Centroid\left(XY_{uplift}\right) = (16.667 \quad 5.833)$$

$$X_{uplift} := \left(1ft\, Centroid\left(XY_{uplift}\right)\right)_{0,0} = 16.667 \cdot ft$$

$$F_{uplift} := Area\left(XY_{uplift}\right) \cdot ft^2 \cdot \gamma_w = 27.3 \cdot \frac{kip}{ft}$$

Estimate Resisting Forces:

Estimate base sliding resistance for concrete gravity dam sliding on rock. Account for hydrostatic overburden above upstream face dam (if present):

Hydrostatic Overburden Volume above front slope of Dam:

$F_{h2o.vert} := 0\,\frac{kip}{ft}$ This geometry has vertical face with no hydrostatic overburden.

$x_{h2o.vert} := 0ft$

Interface friction between concrete gravity dam and shale bedrock:

$\delta_{base} = 24 \cdot deg$ Base friction angle between dam and foundation.

$F_{base} := \left(WT_{dam} + F_{h2o.vert} - F_{uplift}\right) \cdot \tan\left(\delta_{base}\right) = 24.4 \cdot \frac{kip}{ft}$

Base friction, sum of vertical forces multiplied by tangent of interface friction times tangent of interface friction (delta).

Estimate Factor of Safety Against Sliding:

The recommended global stability design criteria is summarized in the USACE Gravity Dam Design EM 1110-2-2200. Stability criteria is summarized in Table 4-1 below.

EM 1110-2-2200
30 Jun 95

Table 4-1
Stability and stress criteria

Load Condition	Resultant Location at Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress	
				Compressive	Tensile
Usual	Middle 1/3	2.0	≤ allowable	$0.3\ f'_c$	0
Unusual	Middle 1/2	1.7	≤ allowable	$0.5\ f'_c$	$0.6\ f'^{2/3}_c$
Extreme	Within base	1.3	≤ 1.33 × allowable	$0.9\ f'_c$	$1.5\ f'^{2/3}_c$

Note: f'_c is 1-year unconfined compressive strength of concrete. The sliding factors of safety (*FS*) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions.

Static Sliding:

$FS_{min.static} := 2.0$ Usual loading. Minimum sliding factor of safety recommended by USACE (from table above)

$\Sigma F_{h.drive} := F_{h2o} + F_{ko} = 11.6 \cdot \frac{kip}{ft}$ Sum of driving forces (hydrostatic pressure + at rest lateral earth pressure)

$\Sigma F_{h.resist} := F_{base} = 24.4 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$FS_{slide.shale} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive}} = 2.11$ Factor of safety against sliding:

$$check_{slide.shale} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale} > FS_{min.static} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$check_{slide.shale} = \text{"OK"}$

MCE Sliding:

$FS_{min.MCE} := 1.3$ MCE, extreme loading. Minimum sliding factor of safety recommended by USACE (from table above).

$\Sigma F_{h.drive.MCE} := F_{h2o} + F_{ko} + F_{inertia.MCE} = 17.6 \cdot \frac{kip}{ft}$ Sum of lateral driving forces during MCE event

$\Sigma F_{h.resist} := F_{base} = 24.4 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$FS_{slide.shale.MCE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.MCE}} = 1.39$ Factor of safety against sliding

$$check_{slide.shale.MCE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.MCE} > FS_{min.MCE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.MCE} = \text{"OK"}$$

OBE Sliding:

$FS_{min.OBE} := 1.7$

OBE, Unusual loading. Minimum sliding factor of safety recommended by USACE (from table above).

$$\Sigma F_{h.drive.OBE} := F_{h2o} + F_{ko} + F_{inertia.OBE} = 12.2 \cdot \frac{kip}{ft}$$

Sum of lateral driving forces during MCE event

$$\Sigma F_{h.resist} := F_{base} = 24.4 \cdot \frac{kip}{ft}$$

Sum of resisting forces (base friction)

$$FS_{slide.shale.OBE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.OBE}} = 2.01$$

Factor of safety against sliding - seismic OBE:

$$check_{slide.shale.OBE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.OBE} > FS_{min.OBE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.OBE} = \text{"OK"}$$

Estimate Required Anchor Forces Based on FS against Sliding:

Static Case:

$$F_{anchor} := FS_{min.static} \cdot \Sigma F_{h.drive} - \Sigma F_{h.resist} = -1.256 \cdot klf$$

Use minimum FS against sliding to determine anchor forc This is the horizontal component of the anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed)

$$T_{anchor.static} := \frac{F_{anchor}}{\cos(\alpha_{anchor})} = -1.776 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam for static loading.

Seismic MCE:

$$F_{anchor.MCE} := FS_{min.MCE} \cdot \Sigma F_{h.drive.MCE} - \Sigma F_{h.resist} = -1.587 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of the anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.MCE} := \frac{F_{anchor.MCE}}{\cos(\alpha_{anchor})} = -2.244 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Seismic OBE:

$$F_{anchor.OBE} := FS_{min.OBE} \cdot \Sigma F_{h.drive.OBE} - \Sigma F_{h.resist} = -3.723 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of the anchor.

$$\alpha_{anchor} := 45deg$$

Angle of anchor installation measured from horizontal.

$$T_{anchor.OBE} := \frac{F_{anchor.OBE}}{\cos(\alpha_{anchor})} = -5.265 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Determine Critical Anchor Force for Design:

$$\boxed{T_{anchor.critical} := \max(T_{anchor.static}, T_{anchor.MCE}, T_{anchor.OBE}) = -1.776 \cdot klf}$$

Estimate Factor of Safety Against Overturning:

Sum moments around downstream toe. Note this is not directly comparable to USACE overturning criteria but useful as a quick che of stability, see estimation of overturning resultant and % base compression below.

Because static controls sliding stability, only examine static case.

$$\Sigma M_{toe.drive.static} := F_{ko} \cdot (El_{ko} - El_{foundation}) + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \ldots + F_{uplift} \cdot (w_{foundation} - X_{uplift}) = 981.566 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma M_{toe.resist} := WT_{dam} \cdot (w_{foundation} - x_{dam}) + F_{h2o.vert} \cdot x_{h2o.vert} = 2538.051 \cdot \frac{kip \cdot ft}{ft}$$

$$\boxed{FS_{overturning.static} := \frac{\Sigma M_{toe.resist}}{\Sigma M_{toe.drive.static}} = 2.59}$$

Factor of safety against overturning, static case.

There is no specified factor of safety provided by USACE against overturning. The USACE does recommend that for the Normal/Usual loading scenario, the overturning resultant should be located within the middle 1/3 of the base of the dam, and for the unusual loading scenario, the middle 1/2 of the dam.

Check Overturning Criteria:

Static Case:

Check that location of overturning resultant falls in middle 1/3 of base of concrete gravity dam (usual case)

$$\Sigma M_{toe.total} := \Sigma M_{toe.drive.static} - \Sigma M_{toe.resist} = -1556.485 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.total} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 54.819 \cdot \frac{kip}{ft}$$

$$X_{Resultant} := \frac{-\Sigma M_{toe.total}}{\Sigma F_{vertical.total}} = 28.4 \cdot ft$$

horizontal distance to resultant of overturning moment relative to face of the wall

$$\frac{1}{3} \cdot w_{foundation} = 16.7 \cdot ft$$

defines middle third of base

$$\frac{2}{3} \cdot w_{foundation} = 33.3 \cdot ft$$

defines middle third of base

$$check_{OT} := \begin{cases} \text{"OK"} & \text{if } \frac{1}{3} w_{foundation} \le X_{Resultant} \le \frac{2}{3} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$check_{OT} = \text{"OK"}$$

OBE Case:

Check that location of overturning resultant falls in middle 1/2 of base of concrete gravity dam (unusual case)

$$\begin{aligned} \Sigma M_{toe.drive.OBE} := & F_{ko} \cdot (El_{ko} - El_{foundation}) \ldots \\ & + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \ldots \\ & + F_{uplift} \cdot (w_{foundation} - X_{uplift}) \ldots \\ & + F_{inertia.OBE} \cdot (El_{inertia.OBE} - El_{foundation}) \end{aligned} = 985.118 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.OBE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 54.819 \cdot \frac{kip}{ft}$$

$$X_{Resultant.OBE} := \frac{-\Sigma M_{toe.drive.OBE}}{\Sigma F_{vertical.total}} = -18 \cdot ft$$

horizontal distance to resultant of overturning moment relative to face of the dam

$$\frac{1}{4} \cdot w_{foundation} = 12.5 \cdot ft$$

defines middle half of base

$$\frac{3}{4} \cdot w_{foundation} = 37.5 \cdot ft$$

defines middle half of base

$$check_{OT.OBE} := \begin{cases} \text{"OK"} & \text{if } \frac{1}{4} w_{foundation} \le X_{Resultant} \le \frac{3}{4} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$check_{OT.OBE} = \text{"OK"}$$

MCE Case:

Check that location of overturning resultant falls within base of concrete gravity dam (extreme case)

$$\Sigma M_{toe.drive.MCE} := F_{ko} \cdot (El_{ko} - El_{foundation}) \ldots + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \ldots + F_{uplift} \cdot (w_{foundation} - X_{uplift}) \ldots + F_{inertia.MCE} \cdot (El_{inertia.MCE} - El_{foundation}) = 1017.481 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.MCE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 54.819 \cdot \frac{kip}{ft}$$

$$X_{Resultant.MCE} := \frac{-\Sigma M_{toe.drive.MCE}}{\Sigma F_{vertical.total}} = -18.6 \cdot ft$$

horizontal distance to resultant of overturning moment relative to face of the dam

$$0 \cdot w_{foundation} = 0 \cdot ft$$

defines upstream edge of base

$$1 \cdot w_{foundation} = 50 \cdot ft$$

defines downstream edge of base

$$check_{OT.MCE} := \begin{cases} \text{"OK"} & \text{if } 0 w_{foundation} \le X_{Resultant} \le 1 w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$check_{OT.MCE} = \text{"OK"}$$

Remarks and Recapitulation:

- Calculation addresses sliding and overturning of the fixed crest section of the South Tulsa/Jenks Dam under anticipated static operating conditions, OBE seismic case, and MCE seismic case noted.
- No anchors required for this case.
- Anchors are not necessary for overturning stability in any case.
- The static case (usual loading) was found to control.

Attachment 8

Evaluate Sliding and Overturning Crest Gate

CH2MHILL

PROJECT : Arkansas River Corridor Project - South Tulsa / Jenks Dam
PROJECT #: 657971.04.02.01
CREATED BY: **Mark Kacmarcik** **DATE:** 04/16/2015
REVIEWED BY: Jen Schaeffer **DATE:** 04/17/2015

Given: Simplified gravity dam geometry shown and generalized subsurface profile. See sketch.

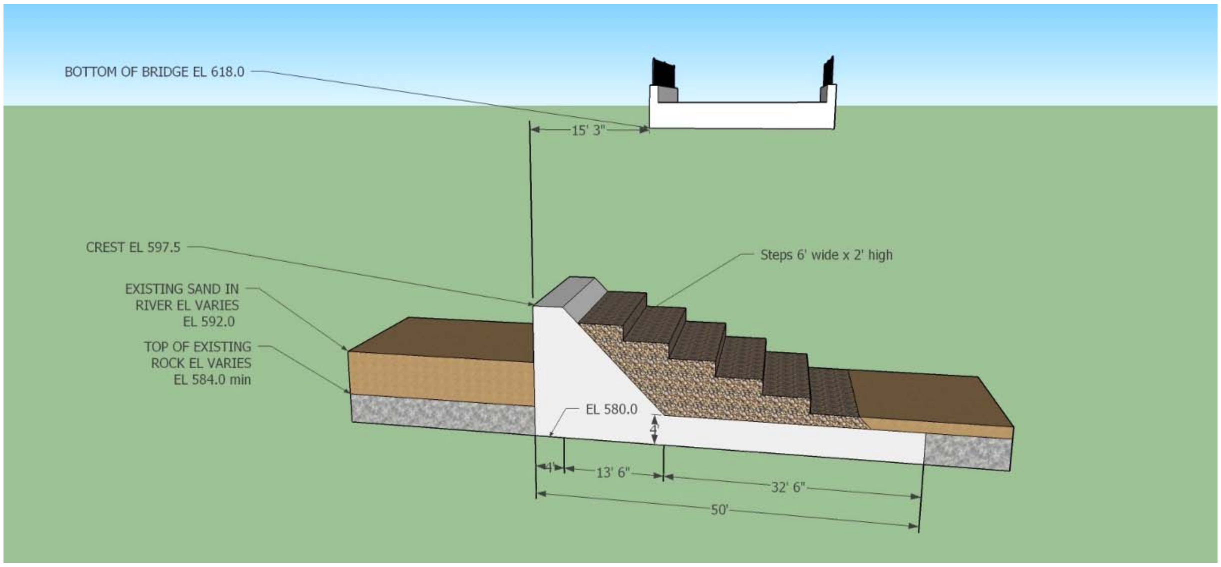
Find: Check sliding and overturning against USACE criteria for Static and Seismic cases as noted in the title. Anchor forces are included as needed to meet stability criteria. Note that this is not attempt to be a complete comprehensive check of all possible analysis cases, but rather the loading cases which are assumed to control overall dam design for preliminary sizing and concept evaluation.

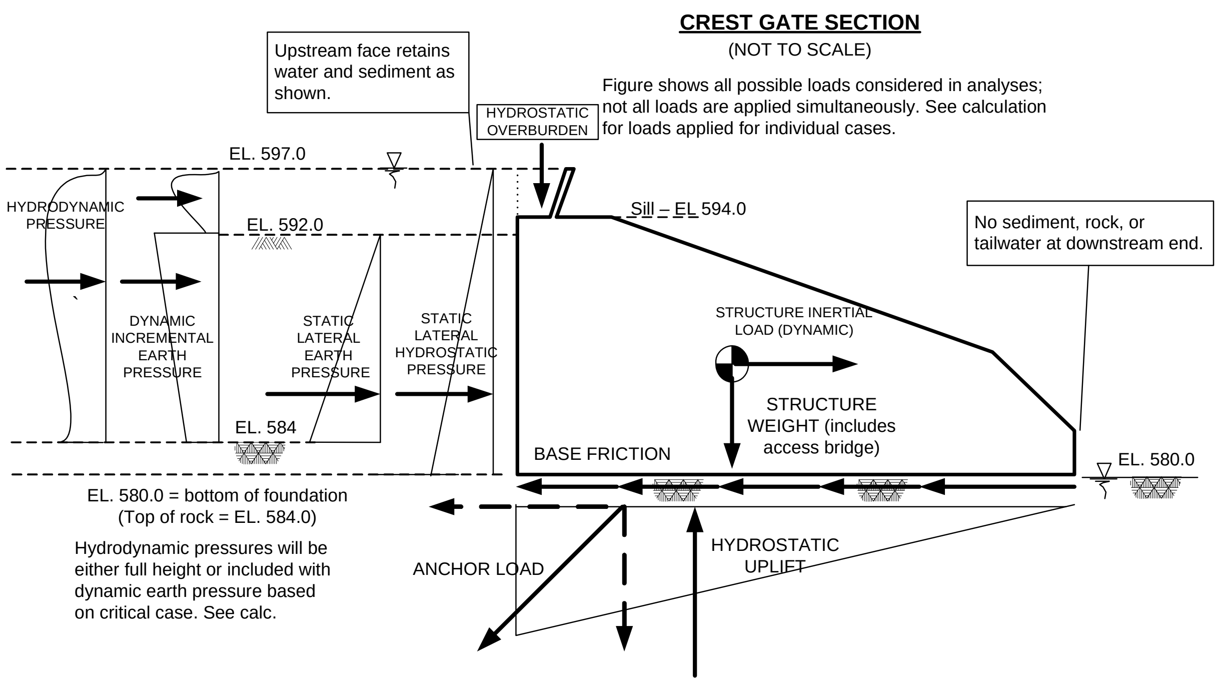
Assumptions:
Ignore resistance from sediment or rock on downstream toe.
Ice loading is not considered.
Structure is not undermined by scour.
Upstream and downstream turndowns (not shown) are not relied upon for shear resistance.
All soil and rock layers are assumed to be horizontal.
Use single conservative frictional interface strength, as shown in the calculation.
Disregard cohesion for long term analysis.
Mass or contributions of pedestrian bridge ignored (conservative)
2 dimensional analysis considering dam geometry on a per-foot basis, 3Dimensional end effects not considered.
Steps shown in geometry are concrete or cut stone with similar unit weight to mass concrete.
Other assumptions as noted in the calculation

Inputs:
Approximate top of rock elevation for main dam, estimated at **El 584 ft**.
Dam foundation elevation assumed 4 feet below top of rock **(El 580 ft.)**
Water present to top of gate at **El 597.0 ft**.
Sill elevation **594.0 ft**
Sediment elevation present to 2 feet below sill at **El 592.0 ft** as directed by Murry Fleming.
Tailwater elevation is coincident with dam foundation, **El 580.0 ft**.
2008 boreholes by Stantec used to estimate subsurface conditions and properties.
Other inputs as noted in the calculation.

References:
USACE EM 1110-2-2200 Gravity Dam Manual
USACE EM 1110-2-2100 Stability Analysis of Concrete Structures

Full Height Gate Section Geometry:





Define Geometry:

$El_{crest} := 597.0ft$ — Elevation at top of dam crest

$El_{sill} := 594ft$ — Elevation at the top of the sill (top of concrete)

$El_{rock} := 584ft$ — Elevation of top of rock (shale)

$d_{excav} := 4ft$ — Excavte below top of rock to remove weathered shale.

$El_{foundation} := El_{rock} - d_{excav} = 580 \cdot ft$ — Elevation of bottom of dam

$H_{dam} := El_{crest} - El_{foundation} = 17 \cdot ft$ — Total height of dam

$w_{foundation} := 50ft$

Given width of dam base

$El_{sed.top} := El_{sill} = 594 \cdot ft$

Assume that dam impounds sediment to top of concrete (sill).

$El_{water.US} := El_{crest} = 597 \cdot ft$

Elevation of water upstream of dam.

$El_{water.DS} := El_{foundation} = 580 \cdot ft$

Elevation of water downstream of dam (assume no water as recommended by USACE).

Material Properties:

Unit Weight:

$\gamma_{conc} := 150pcf$

Unit weight of concrete (assumed).

$\gamma_{sed} := 120pcf$

Unit weight of sediment against upstream face of dam (recommended by USACE EM 1110-2-2100)

$\gamma_{shale} := 152pcf$

Unit weight of Shale from Stantec, 2008 laboratory test results.

$\gamma_{w} := 62.4pcf$

Unit weight of water (assumed).

Shear Strength:

$\phi_{sed} := 28deg$ $c_{sed} := 0psf$

Effective stress shear strength of sediment.

Interface Strength (sliding):

$\delta_{base} := 24deg$

Consider only one sliding interface, mass concrete cast against shale bedrock. Assume no cohesion/adhesion along this interface, only base friciton. Typical value from NAVFAC DM7.2 for "Mass concrete cast against...very stiff and hard residual or preconsolidated clay".

Seismic:

$PGA_{OBE} := 0.009$

Peak ground acceleration on rock for Operations Basis Earthquake (OBE). 50% probability of exceedance in 100 years.

$PGA_{MCE} := 0.091$

Peak ground acceleration on rock for Maximum Credible Earthquake (MCE). 10% probability of exceedance in 50 years

$F_{PGA.scC} := 1.2$

Site coefficient for Site Class C, "Very Dense Soil and Soft Rock" (assumed).

$k_{h.MCE} := \frac{2}{3} \cdot PGA_{MCE} \cdot F_{PGA.scC} = 0.073$

Seismic coeff for MCE case (per EM 1110-2-2100 = 2/3 effective peak ground accel). Conservatively estimated using PGA for site class C.

$k_{h.OBE} := \frac{2}{3} \cdot PGA_{OBE} \cdot F_{PGA.scC} = 0.007$

Seismic coeff for OBE case.

$k_{v} := 0$

Neglect vertical component of earthquake acceleration (assumed).

Estimate Weight of Concrete Gravity Dam:

Estimate total stress (non buoyant) weight of concrete gravity dam by estimating area of the gravity dam polygon, and then multiplying it by the unit weight of the material

Centroid of polygon [edit] from Wikipedia (http://en.wikipedia.org/wiki/Polygon, February 27, 2014)

The centroid of a non-self-intersecting closed polygon defined by n vertices (x_0,y_0), (x_1,y_1), ..., (x_{n-1},y_{n-1}) is the point (C_x, C_y), where

$$C_x = \frac{1}{6A}\sum_{i=0}^{n-1}(x_i + x_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

$$C_y = \frac{1}{6A}\sum_{i=0}^{n-1}(y_i + y_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

and where A is the polygon's signed area,

$$A = \frac{1}{2}\sum_{i=0}^{n-1}(x_i\ y_{i+1} - x_{i+1}\ y_i).$$ [9]

In these formulas, the vertices are assumed to be numbered in order of their occurrence along the polygon's perimeter, and the vertex (x_n, y_n) is assumed to be the same as (x_0, y_0). Note that if the points are numbered in clockwise order the area A, computed as above, will have a negative sign; but the centroid coordinates will be correct even in this case.

Define function to calculate area of polygon whose plane coordinates are contained in matrix XY

$$\mathrm{Area}(XY) := \left| \begin{array}{l} XY \leftarrow \mathrm{stack}\left[XY, \left(\left(XY^{T}\right)^{\langle 0\rangle}\right)^{T}\right] \\ M \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left|\mathrm{submatrix}(XY, i, i+1, 0, 1)\right| \\ 0.5\cdot M \end{array} \right.$$

Define function to calculate coordinates of centroid of non-intersecting closed polygon

$$\mathrm{Centroid}(XY) := \left| \begin{array}{l} XY \leftarrow \mathrm{stack}\left[XY, \left(\left(XY^{T}\right)^{\langle 0\rangle}\right)^{T}\right] \\ x \leftarrow XY^{\langle 0\rangle} \\ y \leftarrow XY^{\langle 1\rangle} \\ C_x \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left[\left(x_i + x_{i+1}\right)\cdot\left(x_i\cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ C_y \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left[\left(y_i + y_{i+1}\right)\cdot\left(x_i\cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ \left(C_x\ \ C_y\right)\cdot\dfrac{1}{6\cdot\mathrm{Area}(XY)} \end{array} \right.$$

Area and Centroid of Concrete Gravity Dam

$$XY_{dam} := \begin{pmatrix} 0 & 580 \\ 0 & 594 \\ 1.5 & 594 \\ 3 & 597 \\ 3.5 & 597 \\ 2 & 594 \\ 9.25 & 594 \\ 11.25 & 592 \\ 35.25 & 586 \\ 37.25 & 584 \\ 50 & 584 \\ 50 & 580 \end{pmatrix}$$

- Values define cross-sectional geometry of dam, points are clockwise around cross section, starting at upstream heel.
- Left column is X coordinates, "0" is the upstream heel of the dam, sign convention is positive to the right (downstream).
- Right column is elevation.

$-\text{Area}(XY_{dam}) = 434$

$\text{Centroid}(XY_{dam}) = (18.869 \quad 585.148)$ center of gravity for concrete gravity dam, ft

$x_{dam\_CG} := \text{Centroid}(XY_{dam})_{0,0} = 18.869$

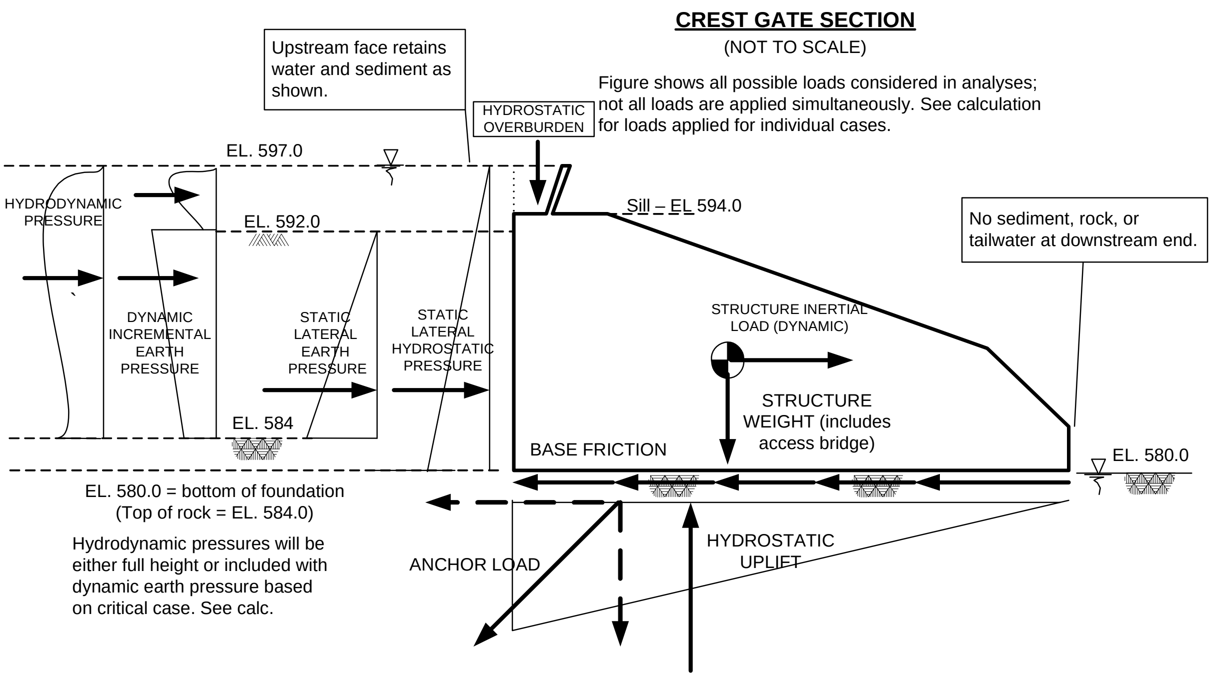
$x_{dam} := x_{dam\_CG} \cdot 1\text{ft} = 18.869 \cdot \text{ft}$ X-coordinate fo centroid, in feet

$y_{dam} := \text{Centroid}(XY_{dam})_{0,1} = 585.148$

$El_{centroid} := y_{dam} \cdot 1\text{ft} = 585.148 \cdot \text{ft}$ Elevation of centroid

$WT_{dam} := -\text{Area}(XY_{dam}) \cdot \text{ft}^2 \cdot \gamma_{conc} + 5\text{klf} = 70.1 \cdot \frac{\text{kip}}{\text{ft}}$ Total weight of concrete gravity dam, per foot. Includes 5 kips per LF for pedestrian bridge per Kevin Whittier's estimate.

Estimate Lateral Driving Forces Acting on Concrete Gravity Dam



Lateral Hydrostatic Water Load on Upstream Face:

$H_w := El_{water.US} - El_{foundation} = 17 \cdot ft$ — Height of water

$F_{h2o} := \frac{1}{2} \cdot \gamma_w \cdot H_w^{\ 2} = 9 \cdot \frac{kip}{ft}$ — Magnitude of resultant of hydrostatic load on upstream face of dam

$El_{h2o} := El_{water.US} - \frac{2}{3} \cdot H_w = 585.7 \cdot ft$ — Elevation of resultant

Static At-Rest Lateral Earth Pressure on Upstream Face:

Assume sediment contributesr at-rest soil pressure on upstream face of dam (active pressures are not developed).

$H_{ko} := El_{sed.top} - El_{rock} = 10 \cdot ft$

$K_0 := 1 - \sin(\phi_{sed}) = 0.531$ — At-rest soil pressure coefficient.

$F_{ko} := \frac{1}{2} \cdot K_0 \cdot (\gamma_{sed} - \gamma_w) \cdot H_{ko}^{\ 2} = 1.53 \cdot \frac{kip}{ft}$ — Magnitude of resultant of at-rest soil pressure on upstream face of dam

$El_{ko} := El_{sed.top} - \frac{2}{3} \cdot H_{ko} = 587.3 \cdot ft$ — Elevation of resultant.

SEISMIC: Lateral Hydrodynamic Water Load on Upstream Face:

This load is applied assuming the dam has been flushed of sediment, and full height of water applies hydrodynamic loading to dam structure during a seismic event. Note that, when sediment levels accumulate, hydrodynamic loading is not considered to be a valid case.

$P_{hydro.MCE} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 0.448 \cdot klf$ — Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$$P_{hydro.OBE} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot \left(El_{crest} - El_{rock}\right)^2 = 0.044 \cdot klf$$

Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$$P_{hydro.MCE.partial} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot \left(El_{crest} - El_{sed.top}\right)^2 = 0.024 \cdot klf$$

Magnitude of hydrodynamic loading over accumulated sediment.

$$P_{hydro.OBE.partial} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot \left(El_{crest} - El_{sed.top}\right)^2 = 0.002 \cdot klf$$

Magnitude of hydrodynamic loadign over accumulated sediment.

$$El_{hydro.MCE} := El_{rock} + 0.4 \cdot \left(El_{crest} - El_{rock}\right) = 589.2 \cdot ft$$

Elevation of resultant of hydrodynamic load for full height water case.

$$El_{hydro.OBE} := El_{rock} + 0.4 \cdot \left(El_{crest} - El_{rock}\right) = 589.2 \cdot ft$$

Elevation of resultant of hydrodynamic load for full height water case.

$$El_{hydro.MCE.partial} := El_{sed.top} + 0.4 \cdot \left(El_{crest} - El_{sed.top}\right) = 595.2 \cdot ft$$

Elevation of resultant of hydrodynamic load for full sediment case.

$$El_{hydro.OBE.partial} := El_{sed.top} + 0.4 \cdot \left(El_{crest} - El_{sed.top}\right) = 595.2 \cdot ft$$

Elevation of resultant of hydrodynamic load for full sediment case.

SEISMIC: Lateral Earth Pressures Upstream Face:

$\theta_{wall} := 0deg$ — Slope of upstream face of dam, 0 indicates vertical face

$\delta_{sed} := 0deg$ — Interface friction angle between sediment and dam, assume zero degrees.

$\beta_{US} := 0deg$ — Slope of top of sediment against upstream face of dam. 0 degrees is horizontal.

$$f\_K_{A.c}(\phi, \delta, \beta, \theta) := \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\delta + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\delta + \phi) \cdot \sin(\phi - \beta)}{\cos(\delta + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

Function to calculate Coulomb active earth pressure coefficient

$$K_A := f\_K_{A.c}\left(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}\right) = 0.361$$

Coulomb active earth perssure coefficient

$$P_A := \frac{1}{2} \cdot K_A \cdot \left(\gamma_{sed} - \gamma_w\right) \left(El_{sed.top} - El_{rock}\right)^2 = 1.04 \cdot klf$$

Active earth pressure force.

$$f\_\psi\left(k_h, k_v\right) := \operatorname{atan}\left(\frac{k_h}{1 - k_v}\right)$$

$$f\_K_{AE}(\phi, \delta, \beta, \theta, \psi) := \frac{\cos(\phi - \psi - \theta)^2}{\cos(\psi) \cdot \cos(\theta)^2 \cdot \cos(\psi + \theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \psi - \beta)}{\cos(\delta + \psi + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

Check MCE:

$\psi_{MCE} := f\_\psi(k_{h.MCE}, k_v) = 0.073$

$K_{AE.MCE} := f\_K_{AE}(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{MCE}) = 0.408$ Examine active case only for upstream sediment. Neglect any downstream passive resistance.

$P_{AE.MCE} := (0.5 \cdot K_{AE.MCE}) \cdot (\gamma_{sed} - \gamma_w)(El_{sed.top} - El_{rock})^2 = 1.174 \cdot klf$ Seismic active earth pressure.

$\Delta P_{AE.MCE} := P_{AE.MCE} - P_A = 0.135 \cdot klf$ Dynamic incremental earth pressure in seismic MCE case.

$F_{MCE.partial} := \Delta P_{AE.MCE} + P_{hydro.MCE.partial} = 0.159 \cdot klf$ Soil + hydrodynamic water load above soil, total horiztonal applied force.

Check OBE:

$\psi_{OBE} := f\_\psi(k_{h.OBE}, k_v) = 0.007$

$K_{AE.OBE} := f\_K_{AE}(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{OBE}) = 0.365$ Examine active case only for upstream sediment. Neglect any downstream passive resistance.

$P_{AE.OBE} := (0.5 \cdot K_{AE.OBE}) \cdot (\gamma_{sed} - \gamma_w)(El_{sed.top} - El_{rock})^2 = 1.052 \cdot klf$ Active seismic earth pressure between top of sediment and top of rock.

$\Delta P_{AE.OBE} := P_{AE.OBE} - P_A = 0.013 \cdot klf$ Additional applied earth pressure in seismic OBE case.

$F_{OBE.partial} := \Delta P_{AE.OBE} + P_{hydro.OBE.partial} = 0.015 \cdot klf$ Soil + hydrodynamic water load above soil, total horiztonal applied force.

Compare Dynamic Lateral Earth Pressures to At-Rest Lateral Earth Pressures:

$P_{AE.OBE} = 1.052 \cdot klf$ Dynamic Active OBE

$P_{AE.MCE} = 1.174 \cdot klf$ Dynamic Active MCE

$F_{ko} = 1.528 \cdot klf$ Static At-Rest

Note that static at-rest loading is greater than dynamic active loading for both MCE and OBE cases. Use greater of static at-rest or dynamic active lateral earth pressures. In this case, static at-rest pressure controls and should be used as the lateral earth pressure for the dynamic analysis cases..

Determine controlling load case for upstream loading on structure:

Structure could be free water (no sediment accumulation), or filled with sediment. For seismic stability evaluations, estimate controlling case: either hydrodynamic loading of silt-free dam or dynamic lateral earth pressure + water over top of sediment.

$F_{ko} = 1.528 \cdot klf$ At-rest lateral earth pressure loading (note that static at-rest is controlling case for seismic evaluation)

$F_{h2o} = 9.017 \cdot klf$ Hydrostatic pressure

$P_{hydro.MCE} = 0.448 \cdot klf$ Hydrodynamic pressure over full height of structure, MCE event

$P_{hydro.OBE} = 0.044 \cdot klf$ Hydrodynamic pressure over full height of structure, OBE event

$P_{hydro.MCE.partial} = 0.024 \cdot klf$ Hydrodynamic pressure above top of sediment, MCE event. Include with soil loading case.

$P_{hydro.OBE.partial} = 0.002 \cdot klf$ Hydrodynamic pressure above top of sediment, OBE event. Include with soil loading case.

$$P_{hydro.MCE} + F_{h2o} = 9.465 \cdot klf$$

$$F_{h2o} + F_{ko} + P_{hydro.MCE.partial} = 10.569 \cdot klf$$

$$check_{hydro.MCE} := \begin{cases} \text{"Hydrodynamic"} & \text{if } \left(P_{hydro.MCE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko} + P_{hydro.MCE.partial}\right) \\ \text{"Soil"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{hydro.MCE} = \text{"Soil"}}$$

$$P_{hydro.OBE} + F_{h2o} = 9.061 \cdot klf$$

$$F_{h2o} + F_{ko} + P_{hydro.OBE.partial} = 10.547 \cdot klf$$

$$check_{hydro.OBE} := \begin{cases} \text{"Hydrodynamic"} & \text{if } \left(P_{hydro.OBE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko} + P_{hydro.OBE.partial}\right) \\ \text{"Soil"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{hydro.OBE} = \text{"Soil"}}$$

SEISMIC: Inertial Load of Structure:

$F_{inertia.MCE} := k_{h.MCE} \cdot WT_{dam} = 5.103 \cdot klf$ Seismic inertia load of the dam for MCE, acts in downstream direction.

$F_{inertia.OBE} := k_{h.OBE} \cdot WT_{dam} = 0.505 \cdot klf$ Seismic inertia load of the dam for OBE acts in downstream direction.

$El_{inertia.MCE} := El_{centroid} = 585.148 \cdot ft$ Seismic inertial load acts through centroid of dam

$El_{inertia.OBE} := El_{centroid} = 585.148 \cdot ft$ Seismic inertial load acts through centroid of dam

Estimate Uplift Hydrostatic Forces Acting on Concrete Gravity Dam

Hydrostatic Uplift on Dam Base:

Magnitude of hydrostatic uplift is estimated as straightline interpolation between headwater and tailwater across width of structure. Figure above shows assumed uplift distribution below bottom of dam.

Use centroid equation to define uplift pressure.

$$XY_{uplift} := \begin{pmatrix} w_{foundation} & 0 \\ w_{foundation} & El_{water.DS} - El_{foundation} \\ 0 & El_{water.US} - El_{foundation} \\ 0 & 0 \end{pmatrix} \cdot ft^{-1}$$

$w_{foundation} = 50 \cdot ft$

$Area(XY_{uplift}) = 425$

$Centroid(XY_{uplift}) = (16.667 \quad 5.667)$

$X_{uplift} := (1ft\,Centroid(XY_{uplift}))_{0,0} = 16.667 \cdot ft$

$F_{uplift} := Area(XY_{uplift}) \cdot ft^2 \cdot \gamma_w = 26.52 \cdot \frac{kip}{ft}$

Estimate Resisting Forces:

Estimate base sliding resistance for concrete gravity dam sliding on rock. Account for hydrostatic overburden above upstream face dam:

Hydrostatic Overburden Volume above upstream face of Dam:

$$XY_{hydroOB} := \begin{pmatrix} 0 & 594 \\ 1.5 & 594 \\ 3 & 597 \\ 0 & 597 \end{pmatrix}$$

$Area(XY_{hydroOB}) = 6.75$

$Centroid(XY_{hydroOB}) = (1.167 \quad 595.667)$

$x_{h2o.vert} := (1ft\,Centroid(XY_{hydroOB}))_{0,0} = 1.167 \cdot ft$

$F_{h2o.vert} := Area(XY_{hydroOB}) \cdot ft^2 \cdot \gamma_w = 0.421 \cdot \frac{kip}{ft}$

Interface friction between concrete gravity dam and shale bedrock:

$\delta_{base} = 24 \cdot deg$ Base friction angle between dam and foundation.

$F_{base} := (WT_{dam} + F_{h2o.vert} - F_{uplift}) \cdot \tan(\delta_{base}) = 19.6 \cdot \frac{kip}{ft}$ Base friction, sum of vertical forces multiplied by tangent of interface friction times tangent of interface friction (delta).

Estimate Factor of Safety Against Sliding:

The recommended global stability design criteria is summarized in the USACE Gravity Dam Design EM 1110-2-2200. Stability criteria is summarized in Table 4-1 below.

EM 1110-2-2200
30 Jun 95

Table 4-1
Stability and stress criteria

Load Condition	Resultant Location at Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress Compressive	Concrete Stress Tensile
Usual	Middle 1/3	2.0	≤ allowable	$0.3\ f'_c$	0
Unusual	Middle 1/2	1.7	≤ allowable	$0.5\ f'_c$	$0.6\ f_c'^{2/3}$
Extreme	Within base	1.3	≤ 1.33 × allowable	$0.9\ f'_c$	$1.5\ f_c'^{2/3}$

Note: f'_c is 1-year unconfined compressive strength of concrete. The sliding factors of safety (*FS*) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions.

$FS_{min} := 2.0$ Usual loading. Minimum sliding factor of safety recommended by USACE (from table above)

$\Sigma F_{h.drive} := F_{h2o} + F_{ko} = 10.5 \cdot \frac{kip}{ft}$ Sum of driving forces (hydrostatic pressure + at rest lateral earth pressure)

$\Sigma F_{h.resist} := F_{base} = 19.6 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$$FS_{slide.shale} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive}} = 1.86$$

$$check_{slide.shale} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale} > FS_{min} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$check_{slide.shale} = \text{"NOT OK-anchors required"}$

MCE Seismic Sliding:

$FS_{min.MCE} := 1.3$ MCE, extreme loading. Minimum sliding factor of safety recommended by USACE (from table above).

$\Sigma F_{h.drive.MCE} := F_{h2o} + F_{ko} + P_{hydro.MCE.partial} + F_{inertia.MCE} = 15.7 \cdot \frac{kip}{ft}$

Sum of lateral driving forces during MCE event.

$\Sigma F_{h.resist} := F_{base} = 19.6 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$$FS_{slide.shale.MCE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.MCE}} = 1.25$$

$$check_{slide.shale.MCE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.MCE} > FS_{min.MCE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.MCE} = \text{"NOT OK-anchors required"}$$

OBE Sliding:

$FS_{min.OBE} := 1.7$

OBE, Unusual loading. Minimum sliding factor of safety recommended by USACE (from table above).

$$\Sigma F_{h.drive.OBE} := F_{h2o} + F_{ko} + P_{hydro.OBE.partial} + F_{inertia.OBE} = 11.1 \cdot \frac{kip}{ft}$$

Sum of lateral driving forces during OBE event.

$$\Sigma F_{h.resist} := F_{base} = 19.6 \cdot \frac{kip}{ft}$$

Sum of resisting forces (base friction)

$$FS_{slide.shale.OBE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.OBE}} = 1.77$$

$$check_{slide.shale.OBE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.OBE} > FS_{min.OBE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.OBE} = \text{"OK"}$$

Estimate Required Anchor Force to Acheive Minimum Sliding Factor of Safety:

$$F_{anchor} := FS_{min} \cdot \Sigma F_{h.drive} - \Sigma F_{h.resist} = 1.499 \cdot klf$$

Use minimum FS against sliding to determine anchor forc
This is the horizontal component of the anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.static} := \frac{F_{anchor}}{\cos(\alpha_{anchor})} = 2.12 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Check Seismic:

MCE case:

$$F_{anchor.MCE} := FS_{min.MCE} \cdot \Sigma F_{h.drive.MCE} - \Sigma F_{h.resist} = 0.783 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of tl anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.MCE} := \frac{F_{anchor.MCE}}{\cos(\alpha_{anchor})} = 1.107 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

OBE case:

$$F_{anchor.OBE} := FS_{min.OBE} \cdot \Sigma F_{h.drive.OBE} - \Sigma F_{h.resist} = -0.803 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of tl anchor.

$$\alpha_{anchor} := 45deg$$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.OBE} := \frac{F_{anchor.OBE}}{\cos(\alpha_{anchor})} = -1.135 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Determine Critical Anchor Force for Design:

$$\boxed{T_{anchor.critical} := \max(T_{anchor.static}, T_{anchor.MCE}, T_{anchor.OBE}) = 2.12 \cdot klf}$$

Estimate Factor of Safety Against Overturning:

Sum moments around downstream toe. Note this is not directly comparable to USACE overturning criteria but useful as a quick che of stability, see estimation of overturning resultant and % base compression below.

$$\Sigma M_{toe.drive} := F_{ko} \cdot (El_{ko} - El_{foundation}) + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \dots + F_{uplift} \cdot (w_{foundation} - X_{uplift}) = 946.3 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma M_{toe.resist} := WT_{dam} \cdot (w_{foundation} - x_{dam}) + F_{h2o.vert} \cdot (w_{foundation} - x_{h2o.vert}) = 2202.838 \cdot \frac{kip \cdot ft}{ft}$$

$$\boxed{FS_{overturning} := \frac{\Sigma M_{toe.resist}}{\Sigma M_{toe.drive}} = 2.33}$$

Factor of safety against overturning, static case.

There is no specified factor of safety provided by USACE against overturning. The USACE does recommend that for the Normal/Usual loading scenario, the overturning resultant should be located within the middle 1/3 of the base of the dam, and for the unusual loading scenario, the middle 1/2 of the dam.

Estimate Location of Overturning Resultant:

Static Case:

Check that location of overturning resultant falls in middle 1/3 of base of concrete gravity dam (usual case)

$$\Sigma M_{toe.total} := \Sigma M_{toe.drive} - \Sigma M_{toe.resist} = -1256.538 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.total} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 44.001 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant} := \frac{-\Sigma M_{toe.total}}{\Sigma F_{vertical.total}} = 28.6 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the wall

$$\frac{1}{3} \cdot w_{foundation} = 16.7 \cdot ft$$

defines middle third of base

$$\frac{2}{3} \cdot w_{foundation} = 33.3 \cdot ft$$

defines middle third of base

$$check_{OT} := \begin{cases} \text{"OK"} & \text{if } \frac{1}{3} w_{foundation} \le X_{Resultant} \le \frac{2}{3} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{OT} = \text{"OK"}}$$

OBE Case:

Check that location of overturning resultant falls in middle 1/2 of base of concrete gravity dam (unusual case)

$$\begin{aligned}\Sigma M_{toe.drive.OBE} := & \ F_{ko} \cdot \left(El_{ko} - El_{foundation}\right) \ldots \\ & + F_{h2o} \cdot \left(El_{h2o} - El_{foundation}\right) \ldots \\ & + F_{uplift} \cdot \left(w_{foundation} - X_{uplift}\right) \ldots \\ & + F_{inertia.OBE} \cdot \left(El_{inertia.OBE} - El_{foundation}\right) \ldots \\ & + P_{hydro.OBE.partial} \cdot \left(El_{hydro.OBE.partial} - El_{foundation}\right) \end{aligned} = 948.934 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.OBE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 44.001 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant.OBE} := \frac{-\Sigma M_{toe.drive.OBE}}{\Sigma F_{vertical.total}} = -21.6 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the dam

$$\frac{1}{4} \cdot w_{foundation} = 12.5 \cdot ft$$

defines middle half of base

$$\frac{3}{4} \cdot w_{foundation} = 37.5 \cdot ft$$

defines middle half of base

$$check_{OT.OBE} := \begin{cases} \text{"OK"} & \text{if } \frac{1}{4} w_{foundation} \le X_{Resultant} \le \frac{3}{4} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{OT.OBE} = \text{"OK"}}$$

MCE Case:

Check that location of overturning resultant falls within base of concrete gravity dam (extreme case)

$$\begin{aligned} \Sigma M_{toe.drive.MCE} := {} & F_{ko} \cdot (El_{ko} - El_{foundation}) \ldots \\ & + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \ldots \\ & + F_{uplift} \cdot (w_{foundation} - X_{uplift}) \ldots \\ & + F_{inertia.MCE} \cdot (El_{inertia.MCE} - El_{foundation}) \ldots \\ & + P_{hydro.MCE.partial} \cdot (El_{hydro.OBE.partial} - El_{foundation}) \end{aligned} = 972.936 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.MCE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 44.001 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant.MCE} := \frac{-\Sigma M_{toe.drive.MCE}}{\Sigma F_{vertical.total}} = -22.1 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the dam

$0 \cdot w_{foundation} = 0 \cdot ft$ defines upstream edge of base

$1 \cdot w_{foundation} = 50 \cdot ft$ defines downstream edge of base

$$check_{OT.MCE} := \begin{cases} \text{"OK"} & \text{if } 0 w_{foundation} \le X_{Resultant} \le 1 w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{OT.MCE} = \text{"OK"}}$$

Remarks and Recapitulation:

- Calculation addresses sliding and overturning of the fixed crest section of South Tulsa Jenks Dam under anticipated static operating conditions, OBE seismic case, and MCE seismic case noted.
- For the static and MCE cases, it is identified that permanent ground anchors are necessary for sliding stability.
- Anchors are not necessary for overturning stability for any case.
- The static case (usual loading) was found to control.

Attachment 9

Evaluate Sliding and Overturning Full Height Gate

CH2MHILL

PROJECT : Arkansas River Corridor Project - South Tulsa / Jenks Dam
PROJECT #: 657971.04.02.01
CREATED BY: Jen Schaeffer/SEA **DATE:** 04/14/2015
REVIEWED BY: Mark Kacmarcik **DATE:** 04/17/2015

Given: Simplified gravity dam geometry shown and generalized subsurface profile. See sketch.

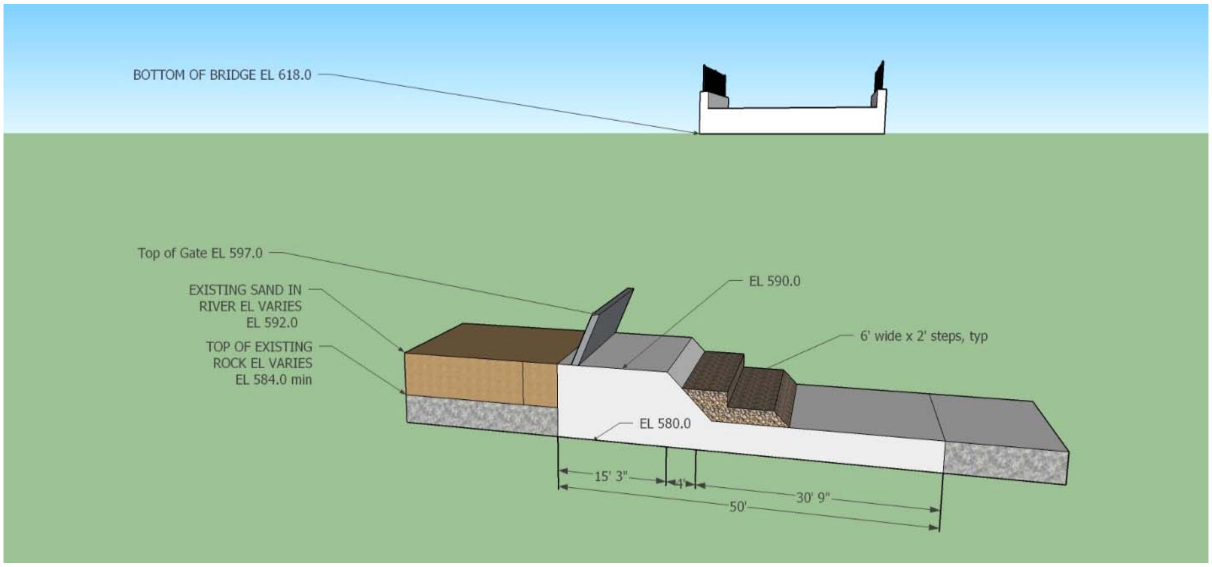
Find: Check sliding and overturning against USACE criteria for Static and Seismic cases as noted in the title. Anchor forces are included as needed to meet stability criteria. Note that this is not attempt to be a complete comprehensive check of all possible analysis cases, but rather the loading cases which are assumed to control overall dam design for preliminary sizing and concept evaluation.

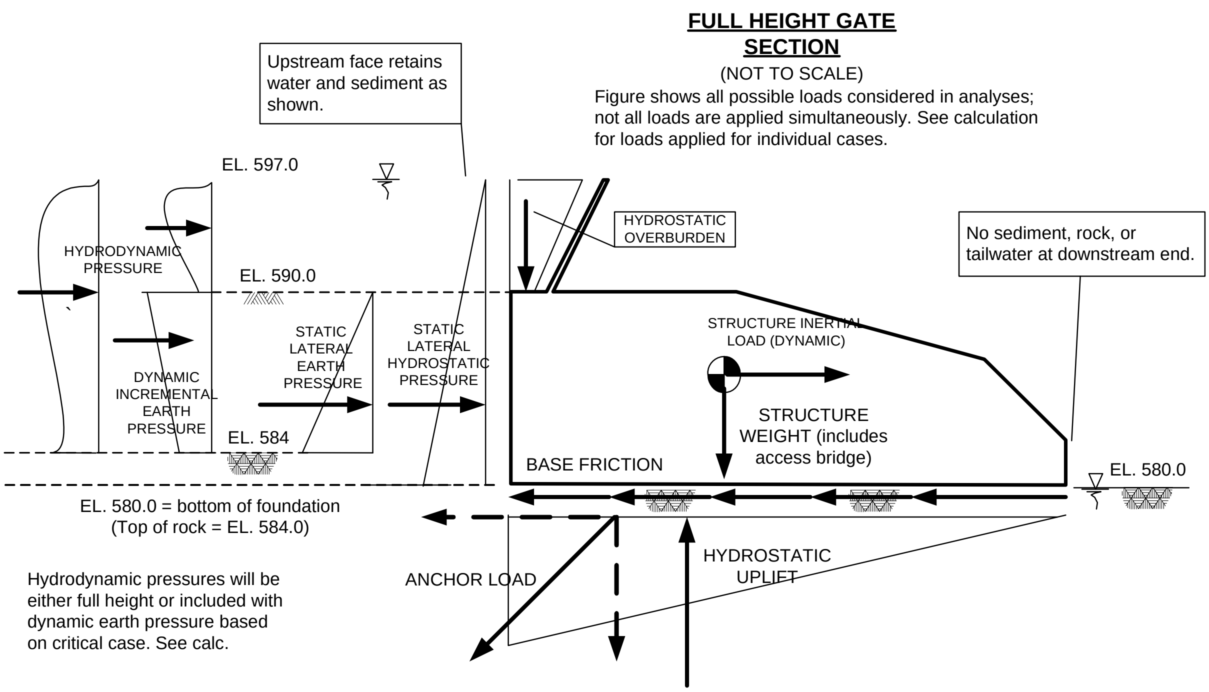
Assumptions: Ignore resistance from sediment or rock on downstream toe.
Ice loading is not considered.
Structure is not undermined by scour.
Upstream and downstream turndowns (not shown) are not relied upon for shear resistance.
All soil and rock layers are assumed to be horizontal.
Use single conservative frictional interface strength, as shown in the calculation.
Disregard cohesion for long term analysis.
Mass or contributions of pedestrian bridge ignored (conservative)
2 dimensional analysis considering dam geometry on a per-foot basis, 3Dimensional end effects not considered.
Steps shown in geometry are concrete or cut stone with similar unit weight to mass concrete.
Other assumptions as noted in the calculation

Inputs: Approximate top of rock elevation for main dam, estimated at **El 584 ft**.
Dam foundation elevation assumed 4 feet below top of rock **(El 580 ft.)**
Water present to top of gate at **El 597.0 ft**.
Sediment elevation present to top of sill at **El 590.0 ft** as directed by Murry Fleming.
Tailwater elevation is coincident with dam foundation, **El 580.0 ft**.
2008 boreholes by Stantec used to estimate subsurface conditions and properties.
Other inputs as noted in the calculation.

References: USACE EM 1110-2-2200 Gravity Dam Manual
USACE EM 1110-2-2100 Stability Analysis of Concrete Structures

Full Height Gate Section Geometry:





Define Geometry:

$El_{crest} := 597.0ft$ Elevation at top of dam crest

$El_{sill} := 590ft$ Elevation at the top of the sill (top of concrete)

$El_{rock} := 584ft$ Elevation of top of rock (shale)

$d_{excav} := 4ft$ Excavte below top of rock to remove weathered shale.

$El_{foundation} := El_{rock} - d_{excav} = 580 \cdot ft$ Elevation of bottom of dam

$H_{dam} := El_{crest} - El_{foundation} = 17 \cdot ft$ Total height of dam

$w_{foundation} := 50ft$ — Given width of dam base

$El_{sed.top} := El_{sill} = 590 \cdot ft$ — Assume that dam impounds sediment to top of concrete (sill).

$El_{water.US} := El_{crest} = 597 \cdot ft$ — Elevation of water upstream of dam.

$El_{water.DS} := El_{foundation} = 580 \cdot ft$ — Elevation of water downstream of dam (assume no water as recommended by USACE).

Material Properties:

Unit Weight:

$\gamma_{conc} := 150pcf$ — Unit weight of concrete (assumed).

$\gamma_{sed} := 120pcf$ — Unit weight of sediment against upstream face of dam (recommended by USACE EM 1110-2-2100)

$\gamma_{shale} := 152pcf$ — Unit weight of Shale from Stantec, 2008 laboratory test results.

$\gamma_{w} := 62.4pcf$ — Unit weight of water (assumed).

Shear Strength:

$\phi_{sed} := 28deg$ $\quad c_{sed} := 0psf$ — Effective stress shear strength of sediment.

Interface Strength (sliding):

$\delta_{base} := 24deg$ — Consider only one sliding interface, mass concrete cast against shale bedrock. Assume no cohesion/adhesion along this interface, only base friciton. Typical value from NAVFAC DM7.2 for "Mass concrete cast against...very stiff and hard residual or preconsolidated clay".

Seismic:

$PGA_{OBE} := 0.009$ — Peak ground acceleration on rock for Operations Basis Earthquake (OBE). 50% probability of exceedance in 100 years.

$PGA_{MCE} := 0.091$ — Peak ground acceleration on rock for Maximum Credible Earthquake (MCE). 10% probability of exceedance in 50 years

$F_{PGA.scC} := 1.2$ — Site coefficient for Site Class C, "Very Dense Soil and Soft Rock" (assumed).

$k_{h.MCE} := \frac{2}{3} \cdot PGA_{MCE} \cdot F_{PGA.scC} = 0.073$ — Seismic coeff for MCE case (per EM 1110-2-2100 = 2/3 effective peak ground accel). Conservatively estimated using PGA for site class C.

$k_{h.OBE} := \frac{2}{3} \cdot PGA_{OBE} \cdot F_{PGA.scC} = 0.007$ — Seismic coeff for OBE case.

$k_{v} := 0$ — Neglect vertical component of earthquake acceleration (assumed).

Estimate Weight of Concrete Gravity Dam:

Estimate total stress (non buoyant) weight of concrete gravity dam by estimating area of the gravity dam polygon, and then multiplying it by the unit weight of the material

Centroid of polygon [edit] from Wikipedia (http://en.wikipedia.org/wiki/Polygon, February 27, 2014)

The centroid of a non-self-intersecting closed polygon defined by n vertices (x_0,y_0), (x_1,y_1), ..., (x_{n-1},y_{n-1}) is the point (C_x, C_y), where

$$C_x = \frac{1}{6A}\sum_{i=0}^{n-1}(x_i + x_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

$$C_y = \frac{1}{6A}\sum_{i=0}^{n-1}(y_i + y_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

and where A is the polygon's signed area,

$$A = \frac{1}{2}\sum_{i=0}^{n-1}(x_i\ y_{i+1} - x_{i+1}\ y_i).$$ [9]

In these formulas, the vertices are assumed to be numbered in order of their occurrence along the polygon's perimeter, and the vertex (x_n, y_n) is assumed to be the same as (x_0, y_0). Note that if the points are numbered in clockwise order the area A, computed as above, will have a negative sign; but the centroid coordinates will be correct even in this case.

Define function to calculate area of polygon whose plane coordinates are contained in matrix XY

$$\mathrm{Area}(XY) := \left| \begin{array}{l} XY \leftarrow \mathrm{stack}\left[XY, \left(\left(XY^{T}\right)^{\langle 0\rangle}\right)^{T}\right] \\ M \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left|\mathrm{submatrix}(XY, i, i+1, 0, 1)\right| \\ 0.5 \cdot M \end{array} \right.$$

Define function to calculate coordinates of centroid of non-intersecting closed polygon

$$\mathrm{Centroid}(XY) := \left| \begin{array}{l} XY \leftarrow \mathrm{stack}\left[XY, \left(\left(XY^{T}\right)^{\langle 0\rangle}\right)^{T}\right] \\ x \leftarrow XY^{\langle 0\rangle} \\ y \leftarrow XY^{\langle 1\rangle} \\ C_x \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left[\left(x_i + x_{i+1}\right)\cdot\left(x_i \cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ C_y \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left[\left(y_i + y_{i+1}\right)\cdot\left(x_i \cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ \left(C_x \ \ C_y\right)\cdot\dfrac{1}{6\cdot \mathrm{Area}(XY)} \end{array} \right.$$

Area and Centroid of Concrete Gravity Dam

$$XY_{dam} := \begin{pmatrix} 0 & 580 \\ 0 & 590 \\ 2 & 590 \\ 5.5 & 597 \\ 6 & 597 \\ 2.5 & 590 \\ 15.25 & 590 \\ 17.25 & 588 \\ 29.25 & 586 \\ 31.25 & 584 \\ 50 & 584 \\ 50 & 580 \end{pmatrix}$$

- Values define cross-sectional geometry of dam, points are clockwise around cross section, starting at upstream heel.
- Left column is X coordinates, "0" is the upstream heel of the dam, sign convention is positive to the right (downstream).
- Right column is elevation.

$$-\mathrm{Area}(XY_{dam}) = 343$$

$$\mathrm{Centroid}(XY_{dam}) = (19.669 \quad 583.972)$$

center of gravity for concrete gravity dam, ft

$$x_{dam\_CG} := \mathrm{Centroid}(XY_{dam})_{0,0} = 19.669$$

$$x_{dam} := x_{dam\_CG} \cdot 1\mathrm{ft} = 19.669 \cdot \mathrm{ft}$$

X-coordinate fo centroid, in feet

$$y_{dam} := \mathrm{Centroid}(XY_{dam})_{0,1} = 583.972$$

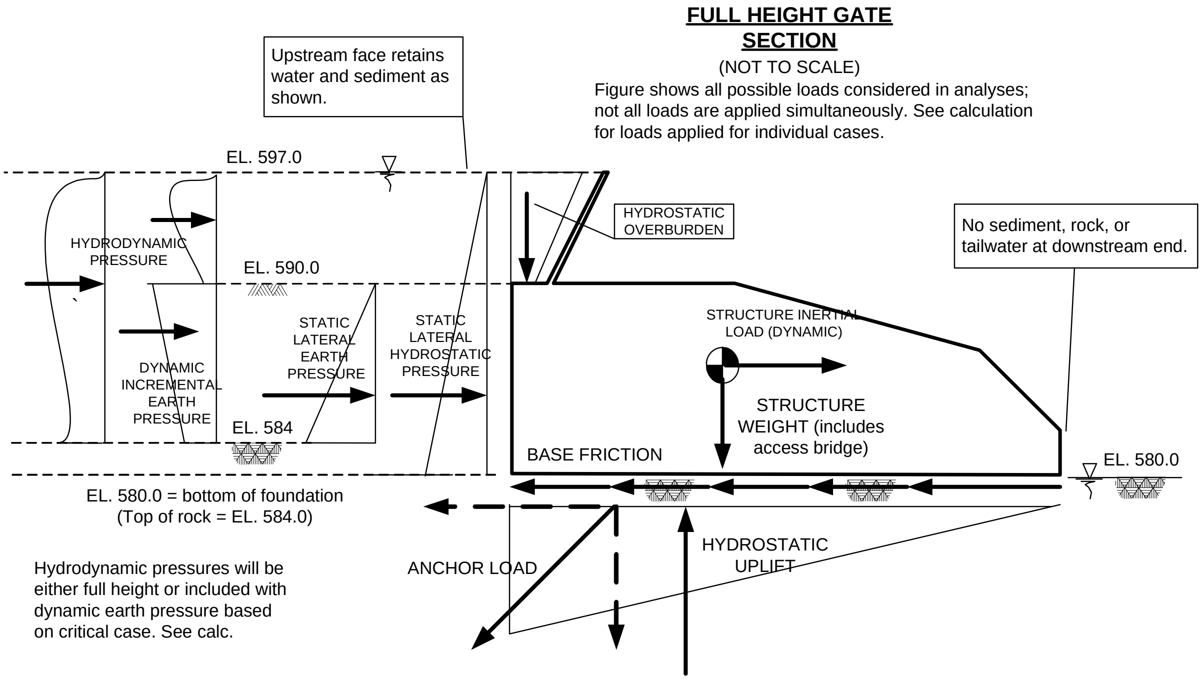
$$El_{centroid} := y_{dam} \cdot 1\mathrm{ft} = 583.972 \cdot \mathrm{ft}$$

Elevation of centroid

$$WT_{dam} := -\mathrm{Area}(XY_{dam}) \cdot \mathrm{ft}^2 \cdot \gamma_{conc} + 5\mathrm{klf} = 56.5 \cdot \frac{\mathrm{kip}}{\mathrm{ft}}$$

Total weight of concrete gravity dam, per foot. Includes 5 kips per LF for pedestrian bridge per Kevin Whittier's estimate

Estimate Lateral Driving Forces Acting on Concrete Gravity Dam



Lateral Hydrostatic Water Load on Upstream Face:

$H_w := El_{water.US} - El_{foundation} = 17 \cdot ft$ — Height of water

$F_{h2o} := \frac{1}{2} \cdot \gamma_w \cdot H_w^{\ 2} = 9 \cdot \frac{kip}{ft}$ — Magnitude of resultant of hydrostatic load on upstream face of dam

$El_{h2o} := El_{water.US} - \frac{2}{3} \cdot H_w = 585.7 \cdot ft$ — Elevation of resultant

Static At-Rest Lateral Earth Pressure on Upstream Face:

Assume sediment contributesr at-rest soil pressure on upstream face of dam (active pressures are not developed).

$H_{ko} := El_{sed.top} - El_{rock} = 6 \cdot ft$

$K_0 := 1 - \sin(\phi_{sed}) = 0.531$ — At-rest soil pressure coefficient.

$F_{ko} := \frac{1}{2} \cdot K_0 \cdot (\gamma_{sed} - \gamma_w) \cdot H_{ko}^{\ 2} = 0.55 \cdot \frac{kip}{ft}$ — Magnitude of resultant of at-rest soil pressure on upstream face of dam

$El_{ko} := El_{sed.top} - \frac{2}{3} \cdot H_{ko} = 586 \cdot ft$ — Elevation of resultant.

SEISMIC: Lateral Hydrodynamic Water Load on Upstream Face:

This load is applied assuming the dam has been flushed of sediment, and full height of water applies hydrodynamic loading to dam structure during a seismic event. Note that, when sediment levels accumulate, hydrodynamic loading is not considered to be a valid case.

$P_{hydro.MCE} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 0.448 \cdot klf$ — Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$P_{hydro.OBE} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 0.044 \cdot klf$ — Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$$P_{hydro.MCE.partial} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot (El_{crest} - El_{sed.top})^2 = 0.13 \cdot klf$$

Magnitude of hydrodynamic loading over accumulated sediment.

$$P_{hydro.OBE.partial} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot (El_{crest} - El_{sed.top})^2 = 0.013 \cdot klf$$

Magnitude of hydrodynamic loadign over accumulated sediment.

$$El_{hydro.MCE} := El_{rock} + 0.4 \cdot (El_{crest} - El_{rock}) = 589.2 \cdot ft$$

Elevation of resultant of hydrodynamic load for full height water case.

$$El_{hydro.OBE} := El_{rock} + 0.4 \cdot (El_{crest} - El_{rock}) = 589.2 \cdot ft$$

Elevation of resultant of hydrodynamic load for full height water case.

$$El_{hydro.MCE.partial} := El_{sed.top} + 0.4 \cdot (El_{crest} - El_{sed.top}) = 592.8 \cdot ft$$

Elevation of resultant of hydrodynamic load for full sediment case.

$$El_{hydro.OBE.partial} := El_{sed.top} + 0.4 \cdot (El_{crest} - El_{sed.top}) = 592.8 \cdot ft$$

Elevation of resultant of hydrodynamic load for full sediment case.

SEISMIC: Lateral Earth Pressures Upstream Face:

$\theta_{wall} := 0deg$ Slope of upstream face of dam, 0 indicates vertical face

$\delta_{sed} := 0deg$ Interface friction angle between sediment and dam, assume zero degrees.

$\beta_{US} := 0deg$ Slope of top of sediment against upstream face of dam. 0 degrees is horizontal.

$$f\_K_{A.c}(\phi, \delta, \beta, \theta) := \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\delta + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\delta + \phi) \cdot \sin(\phi - \beta)}{\cos(\delta + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

Function to calculate Coulomb active earth pressure coefficient

$$K_A := f\_K_{A.c}(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}) = 0.361$$

Coulomb active earth perssure coefficient

$$P_A := \frac{1}{2} \cdot K_A \cdot (\gamma_{sed} - \gamma_w)(El_{sed.top} - El_{rock})^2 = 0.374 \cdot klf$$

Active earth pressure force.

$$f\_\psi(k_h, k_v) := \operatorname{atan}\left(\frac{k_h}{1 - k_v}\right)$$

$$f\_K_{AE}(\phi, \delta, \beta, \theta, \psi) := \frac{\cos(\phi - \psi - \theta)^2}{\cos(\psi) \cdot \cos(\theta)^2 \cdot \cos(\psi + \theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \psi - \beta)}{\cos(\delta + \psi + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

Check MCE:

$$\psi_{MCE} := f\_\psi(k_{h.MCE}, k_v) = 0.073$$

$K_{AE.MCE} := f\_K_{AE}\left(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{MCE}\right) = 0.408$ Examine active case only for upstream sediment. Neglect any downstream passive resistance.

$P_{AE.MCE} := \left(0.5 \cdot K_{AE.MCE}\right) \cdot \left(\gamma_{sed} - \gamma_w\right)\left(El_{sed.top} - El_{rock}\right)^2 = 0.423 \cdot klf$ Seismic active earth pressure.

$\Delta P_{AE.MCE} := P_{AE.MCE} - P_A = 0.048 \cdot klf$ Dynamic incremental earth pressure in seismic MCE case.

$F_{MCE.partial} := \Delta P_{AE.MCE} + P_{hydro.MCE.partial} = 0.178 \cdot klf$ Soil + hydrodynamic water load above soil, total horiztonal applied force.

Check OBE:

$\psi_{OBE} := f\_\psi\left(k_{h.OBE}, k_v\right) = 0.007$

$K_{AE.OBE} := f\_K_{AE}\left(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{OBE}\right) = 0.365$ Examine active case only for upstream sediment. Neglect any downstream passive resistance.

$P_{AE.OBE} := \left(0.5 \cdot K_{AE.OBE}\right) \cdot \left(\gamma_{sed} - \gamma_w\right)\left(El_{sed.top} - El_{rock}\right)^2 = 0.379 \cdot klf$ Active seismic earth pressure between top of sediment and top of rock.

$\Delta P_{AE.OBE} := P_{AE.OBE} - P_A = 0.005 \cdot klf$ Additional applied earth pressure in seismic OBE case.

$F_{OBE.partial} := \Delta P_{AE.OBE} + P_{hydro.OBE.partial} = 0.017 \cdot klf$ Soil + hydrodynamic water load above soil, total horiztonal applied force.

Compare Dynamic Lateral Earth Pressures to At-Rest Lateral Earth Pressures:

$P_{AE.OBE} = 0.379 \cdot klf$ Dynamic Active OBE

$P_{AE.MCE} = 0.423 \cdot klf$ Dynamic Active MCE

$F_{ko} = 0.55 \cdot klf$ Static At-Rest

Note that static at-rest loading is greater than dynamic active loading for both MCE and OBE cases. Use greater of static at-rest or dynamic active lateral earth pressures. In this case, static at-rest pressure controls and should be used as the lateral earth pressure for the dynamic analysis cases..

Determine controlling load case for upstream loading on structure:

Structure could be free water (no sediment accumulation), or filled with sediment. For seismic stability evaluations, estimate controlling case: either hydrodynamic loading of silt-free dam or dynamic lateral earth pressure + water over top of sediment.

$F_{ko} = 0.55 \cdot klf$ At-rest lateral earth pressure loading (note that static at-rest is controlling case for seismic evaluation)

$F_{h2o} = 9.017 \cdot klf$ Hydrostatic pressure

$P_{hydro.MCE} = 0.448 \cdot klf$ Hydrodynamic pressure over full height of structure, MCE event

$P_{hydro.OBE} = 0.044 \cdot klf$ Hydrodynamic pressure over full height of structure, OBE event

$P_{hydro.MCE.partial} = 0.13 \cdot klf$ Hydrodynamic pressure above top of sediment, MCE event. Include with soil loading case.

$P_{hydro.OBE.partial} = 0.013 \cdot klf$

Hydrodynamic pressure above top of sediment, OBE event. Include with soil loading case.

$P_{hydro.MCE} + F_{h2o} = 9.465 \cdot klf$

$F_{h2o} + F_{ko} + P_{hydro.MCE.partial} = 9.697 \cdot klf$

$$check_{hydro.MCE} := \begin{cases} \text{"Hydrodynamic"} & \text{if } \left(P_{hydro.MCE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko} + P_{hydro.MCE.partial}\right) \\ \text{"Soil"} & \text{otherwise} \end{cases}$$

$check_{hydro.MCE} = \text{"Soil"}$

$P_{hydro.OBE} + F_{h2o} = 9.061 \cdot klf$

$F_{h2o} + F_{ko} + P_{hydro.OBE.partial} = 9.58 \cdot klf$

$$check_{hydro.OBE} := \begin{cases} \text{"Hydrodynamic"} & \text{if } \left(P_{hydro.OBE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko} + P_{hydro.OBE.partial}\right) \\ \text{"Soil"} & \text{otherwise} \end{cases}$$

$check_{hydro.OBE} = \text{"Soil"}$

SEISMIC: Inertial Load of Structure:

$F_{inertia.MCE} := k_{h.MCE} \cdot WT_{dam} = 4.11 \cdot klf$

Seismic inertia load of the dam for MCE, acts in downstream direction.

$F_{inertia.OBE} := k_{h.OBE} \cdot WT_{dam} = 0.406 \cdot klf$

Seismic inertia load of the dam for OBE acts in downstream direction.

$El_{inertia.MCE} := El_{centroid} = 583.972 \cdot ft$

Seismic inertial load acts through centroid of dam

$El_{inertia.OBE} := El_{centroid} = 583.972 \cdot ft$

Seismic inertial load acts through centroid of dam

Estimate Uplift Hydrostatic Forces Acting on Concrete Gravity Dam

Hydrostatic Uplift on Dam Base:

Magnitude of hydrostatic uplift is estimated as straightline interpolation between headwater and tailwater across width of structure. Figure above shows assumed uplift distribution below bottom of dam.

Use centroid equation to define uplift pressure.

$$XY_{uplift} := \begin{pmatrix} w_{foundation} & 0 \\ w_{foundation} & El_{water.DS} - El_{foundation} \\ 0 & El_{water.US} - El_{foundation} \\ 0 & 0 \end{pmatrix} \cdot ft^{-1}$$

$w_{foundation} = 50 \cdot ft$

$Area(XY_{uplift}) = 425$

$Centroid(XY_{uplift}) = (16.667 \quad 5.667)$

$X_{uplift} := (1ft\,Centroid(XY_{uplift}))_{0,0} = 16.667 \cdot ft$

$F_{uplift} := Area(XY_{uplift}) \cdot ft^2 \cdot \gamma_w = 26.52 \cdot \frac{kip}{ft}$

Estimate Resisting Forces:

Estimate base sliding resistance for concrete gravity dam sliding on rock. Account for hydrostatic overburden above upstream face dam:

Hydrostatic Overburden Volume above upstream face of Dam:

$$XY_{hydroOB} := \begin{pmatrix} 0 & 590 \\ 2 & 590 \\ 5.5 & 597 \\ 0 & 597 \end{pmatrix}$$

$Area(XY_{hydroOB}) = 26.25$

$Centroid(XY_{hydroOB}) = (2.011 \quad 594.044)$

$x_{h2o.vert} := (1ft\,Centroid(XY_{hydroOB}))_{0,0} = 2.011 \cdot ft$

$F_{h2o.vert} := Area(XY_{hydroOB}) \cdot ft^2 \cdot \gamma_w = 1.638 \cdot \frac{kip}{ft}$

Interface friction between concrete gravity dam and shale bedrock:

$\delta_{base} = 24 \cdot deg$ Base friction angle between dam and foundation.

$F_{base} := (WT_{dam} + F_{h2o.vert} - F_{uplift}) \cdot \tan(\delta_{base}) = 14.1 \cdot \frac{kip}{ft}$ Base friction, sum of vertical forces multiplied by tangent of interface friction times tangent of interface friction (delta).

Estimate Factor of Safety Against Sliding:

The recommended global stability design criteria is summarized in the USACE Gravity Dam Design EM 1110-2-2200. Stability criteria is summarized in Table 4-1 below.

EM 1110-2-2200
30 Jun 95

Table 4-1
Stability and stress criteria

Load Condition	Resultant Location at Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress	
				Compressive	Tensile
Usual	Middle 1/3	2.0	≤ allowable	$0.3\ f'_c$	0
Unusual	Middle 1/2	1.7	≤ allowable	$0.5\ f'_c$	$0.6\ f_c'^{2/3}$
Extreme	Within base	1.3	≤ 1.33 × allowable	$0.9\ f'_c$	$1.5\ f_c'^{2/3}$

Note: f'_c is 1-year unconfined compressive strength of concrete. The sliding factors of safety (*FS*) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions.

$FS_{min} := 2.0$ Usual loading. Minimum sliding factor of safety recommended by USACE (from table above)

$\Sigma F_{h.drive} := F_{h2o} + F_{ko} = 9.6 \cdot \frac{kip}{ft}$ Sum of driving forces (hydrostatic pressure + at rest lateral earth pressure)

$\Sigma F_{h.resist} := F_{base} = 14.1 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$$FS_{slide.shale} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive}} = 1.47$$

$$check_{slide.shale} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale} > FS_{min} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$check_{slide.shale} = \text{"NOT OK-anchors required"}$

MCE Seismic Sliding:

$FS_{min.MCE} := 1.3$ MCE, extreme loading. Minimum sliding factor of safety recommended by USACE (from table above).

$\Sigma F_{h.drive.MCE} := F_{h2o} + F_{ko} + P_{hydro.MCE.partial} + F_{inertia.MCE} = 13.8 \cdot \frac{kip}{ft}$

Sum of lateral driving forces during MCE event.

$\Sigma F_{h.resist} := F_{base} = 14.1 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$$FS_{slide.shale.MCE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.MCE}} = 1.02$$

$$check_{slide.shale.MCE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.MCE} > FS_{min.MCE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.MCE} = \text{"NOT OK-anchors required"}$$

OBE Sliding:

$FS_{min.OBE} := 1.7$

OBE, Unusual loading. Minimum sliding factor of safety recommended by USACE (from table above).

$$\Sigma F_{h.drive.OBE} := F_{h2o} + F_{ko} + P_{hydro.OBE.partial} + F_{inertia.OBE} = 10 \cdot \frac{kip}{ft}$$

Sum of lateral driving forces during OBE event.

$$\Sigma F_{h.resist} := F_{base} = 14.1 \cdot \frac{kip}{ft}$$

Sum of resisting forces (base friction)

$$FS_{slide.shale.OBE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.OBE}} = 1.41$$

$$check_{slide.shale.OBE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.OBE} > FS_{min.OBE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.OBE} = \text{"NOT OK-anchors required"}$$

Estimate Required Anchor Force to Acheive Minimum Sliding Factor of Safety:

$$F_{anchor} := FS_{min} \cdot \Sigma F_{h.drive} - \Sigma F_{h.resist} = 5.079 \cdot klf$$

Use minimum FS against sliding to determine anchor forc This is the horizontal component of the anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.static} := \frac{F_{anchor}}{\cos(\alpha_{anchor})} = 7.182 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Check Seismic:

MCE case:

$$F_{anchor.MCE} := FS_{min.MCE} \cdot \Sigma F_{h.drive.MCE} - \Sigma F_{h.resist} = 3.893 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of tl anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.MCE} := \frac{F_{anchor.MCE}}{\cos(\alpha_{anchor})} = 5.506 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

OBE case:

$$F_{anchor.OBE} := FS_{min.OBE} \cdot \Sigma F_{h.drive.OBE} - \Sigma F_{h.resist} = 2.921 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of tl anchor.

$$\alpha_{anchor} := 45deg$$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.OBE} := \frac{F_{anchor.OBE}}{\cos(\alpha_{anchor})} = 4.132 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Determine Critical Anchor Force for Design:

$$\boxed{T_{anchor.critical} := \max(T_{anchor.static}, T_{anchor.MCE}, T_{anchor.OBE}) = 7.182 \cdot klf}$$

Estimate Factor of Safety Against Overturning:

Sum moments around downstream toe. Note this is not directly comparable to USACE overturning criteria but useful as a quick che of stability, see estimation of overturning resultant and % base compression below.

$$\Sigma M_{toe.drive} := F_{ko} \cdot (El_{ko} - El_{foundation}) + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \ldots = 938.396 \cdot \frac{kip \cdot ft}{ft}$$
$$+ F_{uplift} \cdot (w_{foundation} - X_{uplift})$$

$$\Sigma M_{toe.resist} := WT_{dam} \cdot (w_{foundation} - x_{dam}) + F_{h2o.vert} \cdot (w_{foundation} - x_{h2o.vert}) = 1790.809 \cdot \frac{kip \cdot ft}{ft}$$

$$\boxed{FS_{overturning} := \frac{\Sigma M_{toe.resist}}{\Sigma M_{toe.drive}} = 1.91}$$

Factor of safety against overturning, static case.

There is no specified factor of safety provided by USACE against overturning. The USACE does recommend that for the Normal/Usual loading scenario, the overturning resultant should be located within the middle 1/3 of the base of the dam, and for the unusual loading scenario, the middle 1/2 of the dam.

Estimate Location of Overturning Resultant:

Static Case:

Check that location of overturning resultant falls in middle 1/3 of base of concrete gravity dam (usual case)

$$\Sigma M_{toe.total} := \Sigma M_{toe.drive} - \Sigma M_{toe.resist} = -852.414 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.total} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 31.568 \cdot \frac{kip}{ft}$$

$$X_{Resultant} := \frac{-\Sigma M_{toe.total}}{\Sigma F_{vertical.total}} = 27 \cdot ft$$

horizontal distance to resultant of overturning moment relative to face of the wall

$$\frac{1}{3} \cdot w_{foundation} = 16.7 \cdot ft$$

defines middle third of base

$$\frac{2}{3} \cdot w_{foundation} = 33.3 \cdot ft$$

defines middle third of base

$$check_{OT} := \begin{cases} \text{"OK"} & \text{if } \frac{1}{3} w_{foundation} \le X_{Resultant} \le \frac{2}{3} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$check_{OT} = \text{"OK"}$$

OBE Case:

Check that location of overturning resultant falls in middle 1/2 of base of concrete gravity dam (unusual case)

$$\begin{aligned} \Sigma M_{toe.drive.OBE} := {} & F_{ko} \cdot (El_{ko} - El_{foundation}) \dots \\ & + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \dots \\ & + F_{uplift} \cdot (w_{foundation} - X_{uplift}) \dots \\ & + F_{inertia.OBE} \cdot (El_{inertia.OBE} - El_{foundation}) \dots \\ & + P_{hydro.OBE.partial} \cdot (El_{hydro.OBE.partial} - El_{foundation}) \end{aligned} = 940.174 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.OBE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 31.568 \cdot \frac{kip}{ft}$$

$$X_{Resultant.OBE} := \frac{-\Sigma M_{toe.drive.OBE}}{\Sigma F_{vertical.total}} = -29.8 \cdot ft$$

horizontal distance to resultant of overturning moment relative to face of the dam

$$\frac{1}{4} \cdot w_{foundation} = 12.5 \cdot ft$$

defines middle half of base

$$\frac{3}{4} \cdot w_{foundation} = 37.5 \cdot ft$$

defines middle half of base

$$check_{OT.OBE} := \begin{cases} "OK" & \text{if } \frac{1}{4} w_{foundation} \le X_{Resultant} \le \frac{3}{4} w_{foundation} \\ "NOT\ OK" & \text{otherwise} \end{cases}$$

$$check_{OT.OBE} = "OK"$$

MCE Case:

Check that location of overturning resultant falls within base of concrete gravity dam (extreme case)

$$\Sigma M_{toe.drive.MCE} := F_{ko} \cdot (El_{ko} - El_{foundation}) \dots + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \dots + F_{uplift} \cdot (w_{foundation} - X_{uplift}) \dots + F_{inertia.MCE} \cdot (El_{inertia.MCE} - El_{foundation}) \dots + P_{hydro.MCE.partial} \cdot (El_{hydro.OBE.partial} - El_{foundation}) = 956.381 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.MCE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 31.568 \cdot \frac{kip}{ft}$$

$$X_{Resultant.MCE} := \frac{-\Sigma M_{toe.drive.MCE}}{\Sigma F_{vertical.total}} = -30.3 \cdot ft$$

horizontal distance to resultant of overturning moment relative to face of the dam

$0 \cdot w_{foundation} = 0 \cdot ft$ defines upstream edge of base

$1 \cdot w_{foundation} = 50 \cdot ft$ defines downstream edge of base

$$check_{OT.MCE} := \begin{cases} "OK" & \text{if } 0 w_{foundation} \le X_{Resultant} \le 1 w_{foundation} \\ "NOT\ OK" & \text{otherwise} \end{cases}$$

$$check_{OT.MCE} = "OK"$$

Remarks and Recapitulation:

- Calculation addresses sliding and overturning of the fixed crest section of South Tulsa Jenks Dam under anticipated static operating conditions, OBE seismic case, and MCE seismic case noted.
- For all cases, it is identified that permanent ground anchors are necessary for sliding stability.
- Anchors are not necessary for overturning stability.
- The static case (usual loading) was found to control.

Attachment 10

Evaluate Sliding and Overturning
Fixed Crest Case

CH2MHILL

PROJECT : Arkansas River Corridor Project - Bixby Dam
PROJECT #: 657971.04.02.01
CREATED BY: Jen Schaeffer/SEA **DATE:** 04/16/2015
REVIEWED BY: Mark Kacmarcik/CVO **DATE:** 04/16/2015

Given: Simplified gravity dam geometry shown and generalized subsurface profile. See sketch.

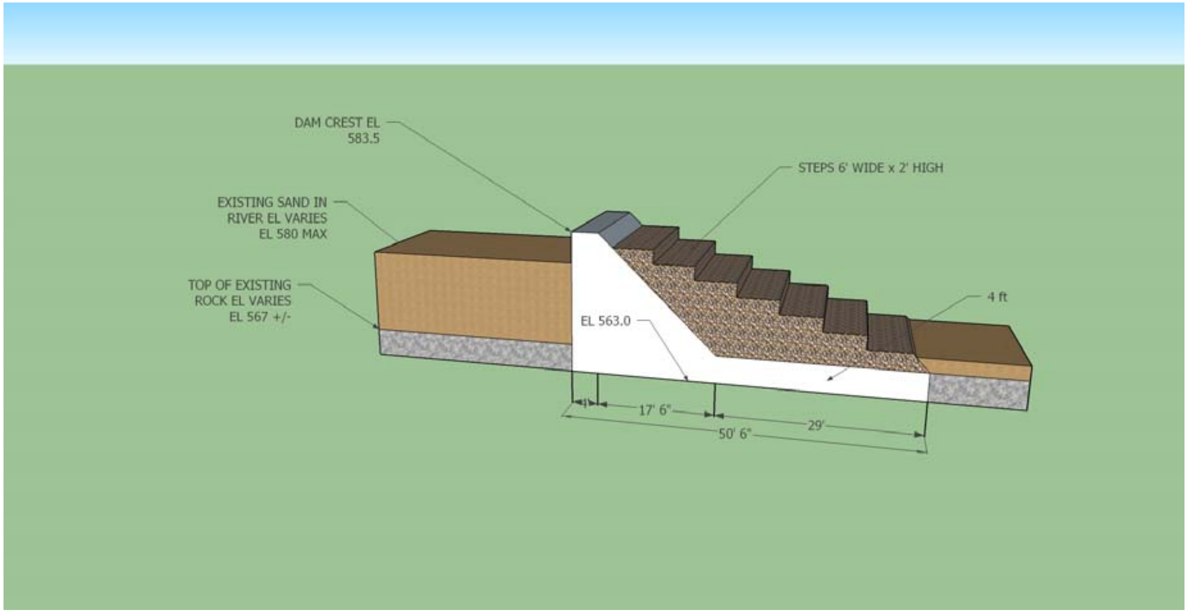
Find: Check sliding and overturning against USACE criteria for Static and Seismic cases as noted in the title. Anchor forces are included as needed to meet stability criteria. Note that this is not attempt to be a complete comprehensive check of all possible analysis cases, but rather the loading cases which are assumed to control overall dam design for preliminary sizing and concept evaluation.

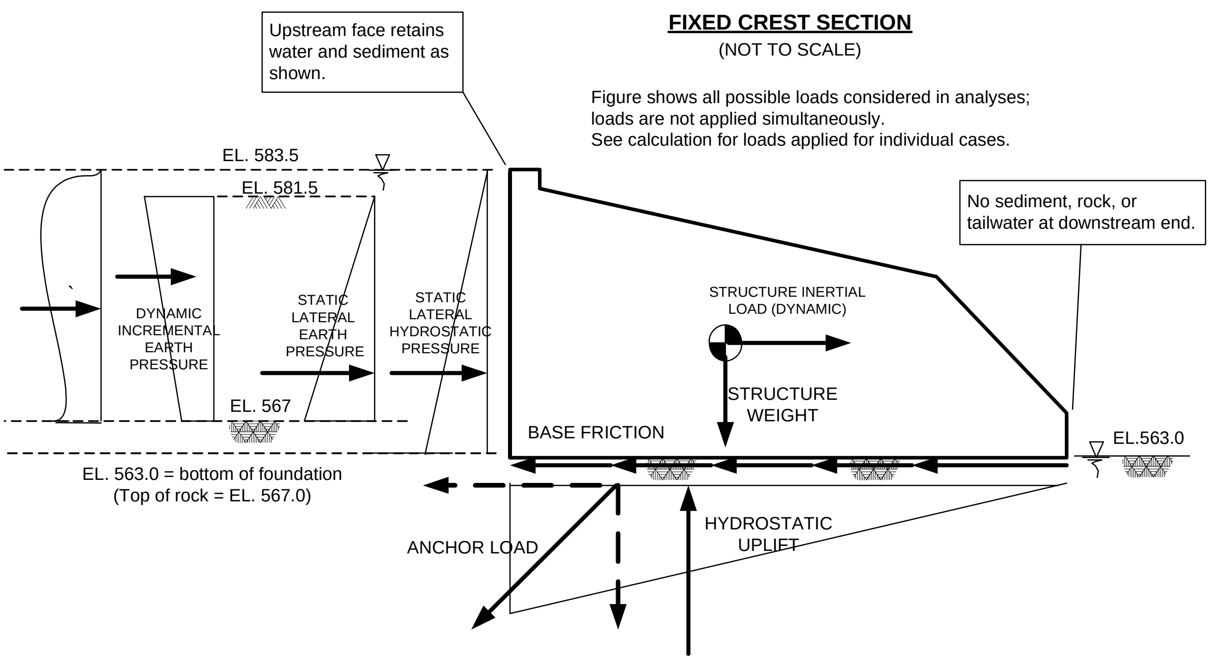
Assumptions: Ignore resistance from sediment or rock on downstream toe.
Ice loading is not considered.
Structure is not undermined by scour
Upstream and downstream turndowns (not shown) are not relied upon for shear resistance.
All soil and rock layers are assumed to be horizontal.
Use single conservative frictional interface strength, as shown in the calculation.
Disregard cohesion for long term analysis.
Mass or contributions of pedestrian bridge ignored (conservative)
2 dimensional analysis considering dam geometry on a per-foot basis, 3Dimensional end effects not considered.
Steps shown in geometry are concrete or cut stone with similar unit weight to mass concrete.
Other assumptions as noted in the calculation

Inputs: Approximate top of rock elevation for main dam, estimated at **El 567 ft**.
Dam foundation elevation assumed 4 feet below top of rock **(El 563 ft.)**
Water present to top of fixed crest at **El 583.5 ft**.
Sediment elevation varies from top of rock **(El 567 ft)** to 2 feet below top of crest **(El 581.5 ft)** as directed by Murry Fleming.
Tailwater elevation is coincident with dam foundation, **El 563.0 ft**.
2008 boreholes by Stantec used to estimate subsurface conditions and properties.
Other inputs as noted in the calculation.

References: USACE EM 1110-2-2200 Gravity Dam Manual
USACE EM 1110-2-2100 Stability Analysis of Concrete Structures

Fixed Crest Section Geometry:





Define Geometry:

$El_{crest} := 583.5ft$ Elevation at top of gravity dam

$El_{rock} := 567ft$ Elevation of top of rock (shale)

$d_{excav} := 4ft$ Excavate below top of rock to remove weathered shale

$El_{foundation} := El_{rock} - d_{excav} = 563 \cdot ft$ Elevation of bottom of dam

$H_{dam} := El_{crest} - El_{foundation} = 20.5 \cdot ft$ Total height of dam

$w_{foundation} := 50.5ft$ Given width of dam base

$El_{sed.top} := El_{crest} - 2ft = 581.5 \cdot ft$

Dam collects sediment to within 2 feet of crest elevation.

$El_{water.US} := El_{crest} = 583.5 \cdot ft$

Elevation of water upstream of dam.

$El_{water.DS} := El_{foundation} = 563 \cdot ft$

Elevation of water downstream of dam (assume no water as recommended by USACE)

Material Properties:

Unit Weight:

$\gamma_{conc} := 150pcf$

Unit weight of concrete (assumed)

$\gamma_{sed} := 120pcf$

Unit weight of sediment against upstream face of dam (recommended by USACE EM 1110-2-2100)

$\gamma_{shale} := 152pcf$

Unit weight of Shale from Stantec, 2008 laboratory test results.

$\gamma_{w} := 62.4pcf$

Unit weight of water (assumed)

Shear Strength:

$\phi_{sed} := 28deg$ $c_{sed} := 0psf$

Effective stress shear strength of sediment.

Interface Strength (sliding):

$\delta_{base} := 24deg$

Consider only one sliding interface, mass concrete cast against shale bedrock. Assume no cohesion/adhesion along this interface, only base friciton. Typical value from NAVFAC DM7.2 for "Mass concrete cast against...very stiff and hard residual or preconsolidated clay"

Seismic:

$PGA_{OBE} := 0.009$ Peak ground acceleration on rock for Operations Basis Earthquake (OBE). 50% probability of exceedance in 100 years.

$PGA_{MCE} := 0.093$ Peak ground acceleration on rock for Maximum Credible Earthquake (MCE). 10% probability of exceedance in 50 years

$F_{PGA.scC} := 1.2$ Site coefficient for Site Class C, "Very Dense Soil and Soft Rock" (assumed).

$k_{h.MCE} := \frac{2}{3} \cdot PGA_{MCE} \cdot F_{PGA.scC} = 0.074$ Seismic coeff for MCE case (per EM 1110-2-2100 = 2/3 effective peak ground accel). Conservatively estimated using PGA for site class C.

$k_{h.OBE} := \frac{2}{3} \cdot PGA_{OBE} \cdot F_{PGA.scC} = 0.007$ Seismic coeff for OBE case.

$k_v := 0$ Neglect vertical component of earthquake accelleration (assumed).

Estimate Weight of Concrete Gravity Dam:

Estimate total (non buoyant) weight of concrete gravity dam by estimating area of the gravity dam polygon, and then multiplying it by the unit weight of the material. Use centroid function to for irregular dam geometry.

Centroid of polygon [edit]from Wikipedia (http://en.wikipedia.org/wiki/Polygon, February 27, 2014)

The centroid of a non-self-intersecting closed polygon defined by n vertices (x_0,y_0), (x_1,y_1), ..., (x_{n-1},y_{n-1}) is the point (C_x, C_y), where

$$C_x = \frac{1}{6A}\sum_{i=0}^{n-1}(x_i + x_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

$$C_y = \frac{1}{6A}\sum_{i=0}^{n-1}(y_i + y_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

and where A is the polygon's signed area,

$$A = \frac{1}{2}\sum_{i=0}^{n-1}(x_i\ y_{i+1} - x_{i+1}\ y_i).$$ [9]

In these formulas, the vertices are assumed to be numbered in order of their occurrence along the polygon's perimeter, and the vertex (x_n, y_n) is assumed to be the same as (x_0, y_0). Note that if the points are numbered in clockwise order the area A, computed as above, will have a negative sign; but the centroid coordinates will be correct even in this case.

Define function to calculate area of polygon whose plane coordinates are contained in matrix XY

$$\text{Area}(XY) := \left| \begin{array}{l} XY \leftarrow \text{stack}\left[XY, \left(XY^T\right)^{\langle 0 \rangle T}\right] \\ M \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left|\text{submatrix}(XY, i, i+1, 0, 1)\right| \\ 0.5 \cdot M \end{array} \right.$$

Define function to calculate coordinates of centroid of non-intersecting closed polygon

$$\mathrm{Centroid}(XY) := \left| \begin{array}{l} XY \leftarrow \mathrm{stack}\left[XY, \left(XY^{T}\right)^{\langle 0 \rangle^{T}}\right] \\ x \leftarrow XY^{\langle 0 \rangle} \\ y \leftarrow XY^{\langle 1 \rangle} \\ C_x \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left[\left(x_i + x_{i+1}\right)\cdot\left(x_i \cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ C_y \leftarrow \displaystyle\sum_{i=0}^{\mathrm{rows}(XY)-2} \left[\left(y_i + y_{i+1}\right)\cdot\left(x_i \cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ \left(C_x \;\; C_y\right)\cdot\dfrac{1}{6\cdot \mathrm{Area}(XY)} \end{array} \right.$$

Area and Centroid of Concrete Gravity Dam

$$XY_{dam} := \begin{pmatrix} 0 & 563.0 \\ 0 & 583.5 \\ 4 & 583.5 \\ 6 & 581.5 \\ 48 & 569.5 \\ 50.5 & 567 \\ 50.5 & 563.0 \end{pmatrix}$$

- Values define cross-sectional geometry of dam, points are clockwise around cross section, starting at upstream heel.
- Left column is X coordinates, "0" is the upstream heel of the dam, sign convention is positive to the right (downstream).
- Right column is elevation.

$-\mathrm{Area}(XY_{dam}) = 659.125$ cross sectional area of dam section

$\mathrm{Centroid}(XY_{dam}) = (20.352 \quad 570.266)$ coordinates of center of gravity of concrete gravity dam, ft

$x_{dam\_CG} := \mathrm{Centroid}(XY_{dam})_{0,0} = 20.352$

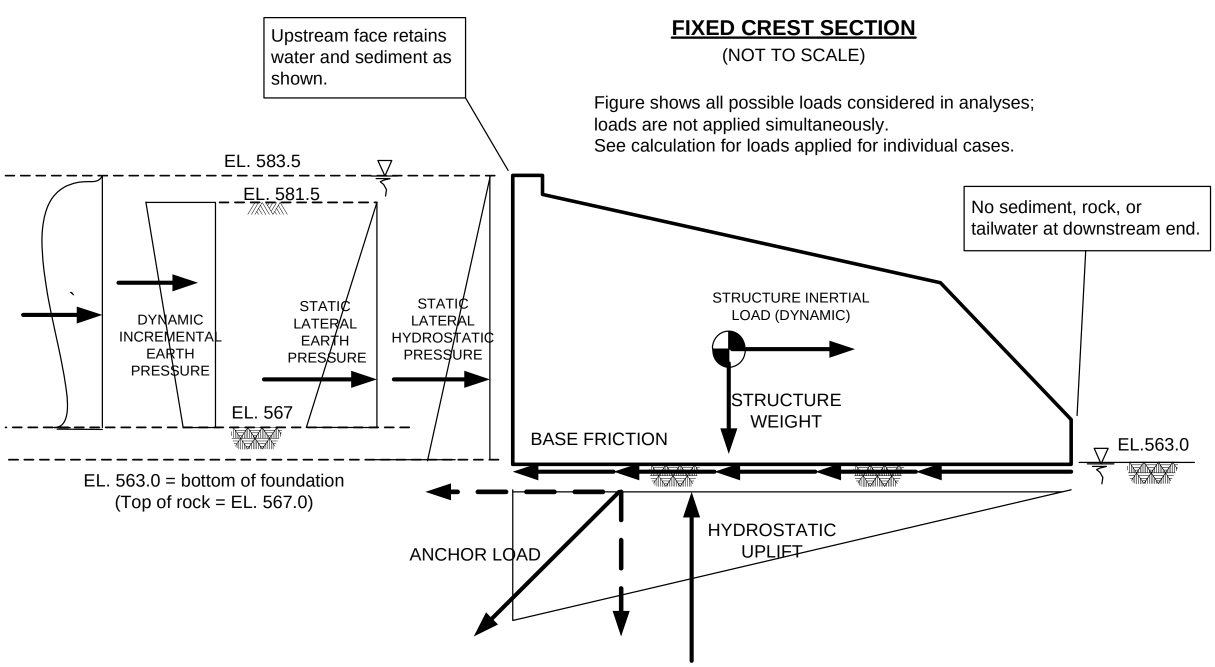
$x_{dam} := x_{dam\_CG} \cdot 1\mathrm{ft} = 20.352 \cdot \mathrm{ft}$ X-coordinate fo centroid, in feet

$y_{dam} := \mathrm{Centroid}(XY_{dam})_{0,1} = 570.266$

$El_{centroid} := y_{dam} \cdot 1\mathrm{ft} = 570.266 \cdot \mathrm{ft}$ Elevation of centroid

$WT_{dam} := -\mathrm{Area}(XY_{dam}) \cdot \mathrm{ft}^2 \cdot \gamma_{conc} = 98.9 \cdot \frac{\mathrm{kip}}{\mathrm{ft}}$ Multiply cross-sectional area by unit weight of concrete to estimate total weight of concrete gravity dam, per lineal foot. No access bridge at Bixby.

Estimate Lateral Driving Forces Acting on Concrete Gravity Dam



Lateral Hydrostatic Water Load on Upstream Face:

$H_w := El_{water.US} - El_{foundation} = 20.5 \cdot ft$ Height of water

$F_{h2o} := \frac{1}{2} \cdot \gamma_w \cdot H_w^{\,2} = 13.1 \cdot \frac{kip}{ft}$ Magnitude of resultant of hydrostatic load on upstream face of dam

$El_{h2o} := El_{water.US} - \frac{2}{3} \cdot H_w = 569.8 \cdot ft$ Elevation of resultant

Static At-Rest Lateral Earth Pressure on Upstream Face:

Assume sediment contributesr at-rest soil pressure on upstream face of dam (active pressures are not developed).

$H_{ko} := El_{sed.top} - El_{rock} = 14.5 \cdot ft$ Maximum sediment accumulation extends from top of rock to 2 feet below fixed crest. Assume no lateral earth pressure from silt below top of rock.

$K_0 := 1 - \sin(\phi_{sed}) = 0.531$ At-rest lateral earth pressure coefficient.

$F_{ko} := \frac{1}{2} \cdot K_0 \cdot (\gamma_{sed} - \gamma_w) \cdot H_{ko}^{\,2} = 3.21 \cdot \frac{kip}{ft}$ Magnitude of resultant of at-rest soil pressure on upstream face of dam (use buoyant unit weight)

$El_{ko} := El_{sed.top} - \frac{2}{3} \cdot H_{ko} = 571.8 \cdot ft$ Elevation of resultant.

Lateral Hydrodynamic Water Load on Upstream Face:

This load is applied assuming the dam has been flushed of sediment, and full height of water applies hydrodynamic loading to dam structure during a seismic event. Note that, when sediment levels accumulate, hydrodynamic loading is not considered to b a valid case.

$P_{hydro.MCE} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 0.737 \cdot klf$ Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$P_{hydro.OBE} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 0.071 \cdot klf$ Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$El_{hydro.MCE} := El_{rock} + 0.4 \cdot (El_{crest} - El_{rock}) = 573.6 \cdot ft$ Elevation of resultant of hydrodynamic load.

$El_{hydro.OBE} := El_{rock} + 0.4 \cdot (El_{crest} - El_{rock}) = 573.6 \cdot ft$ Elevation of resultant of hydrodynamic load.

SEISMIC: Lateral Earth Pressures Upstream Face:

$\theta_{wall} := 0deg$ Slope of upstream face of dam, 0 indicates vertical face

$\delta_{sed} := 0deg$ Interface friction angle between sediment and dam, assume zero degrees.

$\beta_{US} := 0deg$ Slope of top of sediment against upstream face of dam. 0 degrees is horizontal.

Define function to calculate Coulomb active lateral earth pressure coefficient:

$$f\_K_{A.c}(\phi,\delta,\beta,\theta) := \frac{\cos(\phi-\theta)^2}{\cos(\theta)^2\cdot\cos(\delta+\theta)\cdot\left(1+\sqrt{\frac{\sin(\delta+\phi)\cdot\sin(\phi-\beta)}{\cos(\delta+\theta)\cdot\cos(\beta-\theta)}}\right)^2}$$

$K_A := f\_K_{A.c}\left(\phi_{sed},\delta_{sed},\beta_{US},\theta_{wall}\right) = 0.361$ Coulomb active lateral earth pressure coefficient.

$P_A := \frac{1}{2}\cdot K_A\cdot\left(\gamma_{sed}-\gamma_w\right)\left(El_{sed.top}-El_{rock}\right)^2 = 2.186\cdot klf$ Coulomb active lateral earth pressure.

$$f\_\psi\left(k_h,k_v\right) := \operatorname{atan}\left(\frac{k_h}{1-k_v}\right)$$

Define function to calculate dynamic lateral earth pressure coefficient (KAE)

$$f\_K_{AE}(\phi,\delta,\beta,\theta,\psi) := \frac{\cos(\phi-\psi-\theta)^2}{\cos(\psi)\cdot\cos(\theta)^2\cdot\cos(\psi+\theta+\delta)\cdot\left(1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\psi-\beta)}{\cos(\delta+\psi+\theta)\cdot\cos(\beta-\theta)}}\right)^2}$$

Estimate Dynamic Lateral Earth Pressure for Maximum Credible Earthquake (MCE):

$\psi_{MCE} := f\_\psi\left(k_{h.MCE},k_v\right) = 0.074$

$K_{AE.MCE} := f\_K_{AE}\left(\phi_{sed},\delta_{sed},\beta_{US},\theta_{wall},\psi_{MCE}\right) = 0.409$

$P_{AE.MCE} := \frac{1}{2}K_{AE.MCE}\cdot\left(\gamma_{sed}-\gamma_w\right)\left(El_{sed.top}-El_{rock}\right)^2 = 2.476\cdot klf$ Total dynamic active earth pressure (static-active plus dynamic)

$\Delta P_{AE.MCE} := P_{AE.MCE} - P_A = 0.29\cdot klf$ Dynamic increment in MCE case.

Estimate Dynamic Lateral Earth Pressure for Operations Basis Earthquake (OBE):

$\psi_{OBE} := f\_\psi\left(k_{h.OBE},k_v\right) = 0.007$

$K_{AE.OBE} := f\_K_{AE}\left(\phi_{sed},\delta_{sed},\beta_{US},\theta_{wall},\psi_{OBE}\right) = 0.365$

$P_{AE.OBE} := \frac{1}{2}K_{AE.OBE}\cdot\left(\gamma_{sed}-\gamma_w\right)\left(El_{sed.top}-El_{rock}\right)^2 = 2.212\cdot klf$ Total dynamic active earth pressure (static-active plus dynamic)

$\Delta P_{AE.OBE} := P_{AE.OBE} - P_A = 0.026\cdot klf$ Additional applied earth pressure in seismic OBE case.

Compare Dynamic Lateral Earth Pressures to At-Rest Lateral Earth Pressures:

$P_{AE.OBE} = 2.212 \cdot klf$ Dynamic Active OBE

$P_{AE.MCE} = 2.476 \cdot klf$ Dynamic Active MCE

$F_{ko} = 3.212 \cdot klf$ Static At-Rest

Note that static at-rest loading is greater than dynamic active loading for both MCE and OBE cases. Use greater of static at-rest dynamic active lateral earth pressures. In this case, static at-rest pressure controls and should be used as the lateral earth pressure for the dynamic analysis cases.

Determine controlling load case for upstream loading on structure:

Structure couuld be free water (no sediment accumulation), or filled with sediment. For seismic stability evaluations, estimate controlling case: either hydrodynamic loading of silt-free dam or dynamic lateral earth pressure of silted-in dam.

$F_{ko} = 3.212 \cdot klf$ Lateral earth pressure loading (note that static at-rest is controlling case for seismic evaluation)

$F_{h2o} = 13.112 \cdot klf$ Hydrostatic pressure

$P_{hydro.MCE} = 0.737 \cdot klf$ Hydrodynamic pressure, MCE event

$P_{hydro.OBE} = 0.071 \cdot klf$ Hydrodynamic pressure, OBE event

$$P_{hydro.MCE} + F_{h2o} = 13.849 \cdot klf$$

$$F_{h2o} + F_{ko} = 16.324 \cdot klf$$

$$check_{hydro.MCE} := \begin{vmatrix} \text{"Hydrodynamic"} \quad \text{if } \left(P_{hydro.MCE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko}\right) \\ \text{"Soil"} \quad \text{otherwise} \end{vmatrix}$$

$$\boxed{check_{hydro.MCE} = \text{"Soil"}}$$

$$P_{hydro.OBE} + F_{h2o} = 13.183 \cdot klf$$

$$F_{h2o} + F_{ko} = 16.324 \cdot klf$$

$$check_{hydro.OBE} := \begin{vmatrix} \text{"Hydrodynamic"} \quad \text{if } \left(P_{hydro.OBE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko}\right) \\ \text{"Soil"} \quad \text{otherwise} \end{vmatrix}$$

$$\boxed{check_{hydro.OBE} = \text{"Soil"}}$$

SEISMIC: Inertial Load of Structure:

$F_{inertia.MCE} := k_{h.MCE} \cdot WT_{dam} = 7.356 \cdot klf$ Seismic inertia load of the dam for MCE, acts in downstream direction.

$F_{inertia.OBE} := k_{h.OBE} \cdot WT_{dam} = 0.712 \cdot klf$ Seismic inertia load of the dam for OBE acts in downstream direction.

$El_{inertia.MCE} := El_{centroid} = 570.266 \cdot ft$ Seismic inertial load acts through centroid of dam

$El_{inertia.OBE} := El_{centroid} = 570.266 \cdot ft$ Seismic inertial load acts through centroid of dam

Estimate Uplift Hydrostatic Forces Acting on Concrete Gravity Dam

Hydrostatic Uplift on Dam Base:

Magnitude of hydrostatic uplift is estimated as straightline interpolation between headwater and tailwater. Figure above shows uplift distribution below bottom of dam.

Use centroid equation to define uplift pressure.

$$XY_{uplift} := \begin{pmatrix} w_{foundation} & 0 \\ w_{foundation} & El_{water.DS} - El_{foundation} \\ 0 & El_{water.US} - El_{foundation} \\ 0 & 0 \end{pmatrix} \cdot ft^{-1}$$

$Area(XY_{uplift}) = 517.625$

$Centroid(XY_{uplift}) = (16.833 \quad 6.833)$

$X_{uplift} := (1ft\,Centroid(XY_{uplift}))_{0,0} = 16.833 \cdot ft$

$F_{uplift} := Area(XY_{uplift}) \cdot ft^2 \cdot \gamma_w = 32.3 \cdot \frac{kip}{ft}$

Estimate Resisting Forces:

Estimate base sliding resistance for concrete gravity dam sliding on rock. Account for hydrostatic overburden above upstream face dam (if present):

Hydrostatic Overburden Volume above front slope of Dam:

$F_{h2o.vert} := 0 \frac{kip}{ft}$ This geometry has vertical face with no hydrostatic overburden.

$x_{h2o.vert} := 0ft$

Interface friction between concrete gravity dam and shale bedrock:

$\delta_{base} = 24 \cdot deg$ Base friction angle between dam and foundation.

$F_{base} := (WT_{dam} + F_{h2o.vert} - F_{uplift}) \cdot \tan(\delta_{base}) = 29.6 \cdot \frac{kip}{ft}$ Base friction, sum of vertical forces multiplied by tangent of interface friction times tangent of interface friction (delta).

Estimate Factor of Safety Against Sliding:

The recommended global stability design criteria is summarized in the USACE Gravity Dam Design EM 1110-2-2200. Stability criteria is summarized in Table 4-1 below.

EM 1110-2-2200
30 Jun 95

Table 4-1
Stability and stress criteria

Load Condition	Resultant Location at Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress	
				Compressive	Tensile
Usual	Middle 1/3	2.0	≤ allowable	$0.3\ f'_c$	0
Unusual	Middle 1/2	1.7	≤ allowable	$0.5\ f'_c$	$0.6\ f_c'^{2/3}$
Extreme	Within base	1.3	≤ 1.33 × allowable	$0.9\ f'_c$	$1.5\ f_c'^{2/3}$

Note: f'_c is 1-year unconfined compressive strength of concrete. The sliding factors of safety (*FS*) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions.

Static Sliding:

$FS_{min.static} := 2.0$ Usual loading. Minimum sliding factor of safety recommended by USACE (from table above)

$\Sigma F_{h.drive} := F_{h2o} + F_{ko} = 16.3 \cdot \frac{kip}{ft}$ Sum of driving forces (hydrostatic pressure + at rest lateral earth pressure)

$\Sigma F_{h.resist} := F_{base} = 29.6 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$FS_{slide.shale} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive}} = 1.82$ Factor of safety against sliding:

$$check_{slide.shale} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale} > FS_{min.static} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$check_{slide.shale} = \text{"NOT OK-anchors required"}$

MCE Sliding:

$FS_{min.MCE} := 1.3$ MCE, extreme loading. Minimum sliding factor of safety recommended by USACE (from table above).

$\Sigma F_{h.drive.MCE} := F_{h2o} + F_{ko} + F_{inertia.MCE} = 23.7 \cdot \frac{kip}{ft}$ Sum of lateral driving forces during MCE event

$\Sigma F_{h.resist} := F_{base} = 29.6 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$FS_{slide.shale.MCE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.MCE}} = 1.25$ Factor of safety against sliding

$$check_{slide.shale.MCE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.MCE} > FS_{min.MCE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.MCE} = \text{"NOT OK-anchors required"}$$

OBE Sliding:

$FS_{min.OBE} := 1.7$

OBE, Unusual loading. Minimum sliding factor of safety recommended by USACE (from table above).

$$\Sigma F_{h.drive.OBE} := F_{h2o} + F_{ko} + F_{inertia.OBE} = 17 \cdot \frac{kip}{ft}$$

Sum of lateral driving forces during MCE event

$$\Sigma F_{h.resist} := F_{base} = 29.6 \cdot \frac{kip}{ft}$$

Sum of resisting forces (base friction)

$$FS_{slide.shale.OBE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.OBE}} = 1.74$$

Factor of safety against sliding - seismic OBE:

$$check_{slide.shale.OBE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.OBE} > FS_{min.OBE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.OBE} = \text{"OK"}$$

Estimate Required Anchor Forces Based on FS against Sliding:

Static Case:

$$F_{anchor} := FS_{min.static} \cdot \Sigma F_{h.drive} - \Sigma F_{h.resist} = 3.01 \cdot klf$$

Use minimum FS against sliding to determine anchor forc This is the horizontal component of the anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed)

$$T_{anchor.static} := \frac{F_{anchor}}{\cos(\alpha_{anchor})} = 4.257 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam for static loading.

Seismic MCE:

$$F_{anchor.MCE} := FS_{min.MCE} \cdot \Sigma F_{h.drive.MCE} - \Sigma F_{h.resist} = 1.146 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of the anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.MCE} := \frac{F_{anchor.MCE}}{\cos(\alpha_{anchor})} = 1.62 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Seismic OBE:

$$F_{anchor.OBE} := FS_{min.OBE} \cdot \Sigma F_{h.drive.OBE} - \Sigma F_{h.resist} = -0.677 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of the anchor.

$$\alpha_{anchor} := 45deg$$

Angle of anchor installation measured from horizontal.

$$T_{anchor.OBE} := \frac{F_{anchor.OBE}}{\cos(\alpha_{anchor})} = -0.957 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Determine Critical Anchor Force for Design:

$$T_{anchor.critical} := \max(T_{anchor.static}, T_{anchor.MCE}, T_{anchor.OBE}) = 4.257 \cdot klf$$

Estimate Factor of Safety Against Overturning:

Sum moments around downstream toe. Note this is not directly comparable to USACE overturning criteria but useful as a quick che of stability, see estimation of overturning resultant and % base compression below.

Because static controls sliding stability, only examine static case.

$$\Sigma M_{toe.drive.static} := F_{ko} \cdot (El_{ko} - El_{foundation}) + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \ldots + F_{uplift} \cdot (w_{foundation} - X_{uplift}) = 1205.401 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma M_{toe.resist} := WT_{dam} \cdot (w_{foundation} - x_{dam}) + F_{h2o.vert} \cdot x_{h2o.vert} = 2980.706 \cdot \frac{kip \cdot ft}{ft}$$

$$FS_{overturning.static} := \frac{\Sigma M_{toe.resist}}{\Sigma M_{toe.drive.static}} = 2.47$$

Factor of safety against overturning, static case.

There is no specified factor of safety provided by USACE against overturning. The USACE does recommend that for the Normal/Usual loading scenario, the overturning resultant should be located within the middle 1/3 of the base of the dam, and for the unusual loading scenario, the middle 1/2 of the dam.

Check Overturning Criteria:

Static Case:

Check that location of overturning resultant falls in middle 1/3 of base of concrete gravity dam (usual case)

$$\Sigma M_{toe.total} := \Sigma M_{toe.drive.static} - \Sigma M_{toe.resist} = -1775.306 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.total} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 66.569 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant} := \frac{-\Sigma M_{toe.total}}{\Sigma F_{vertical.total}} = 26.7 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the wall

$$\frac{1}{3} \cdot w_{foundation} = 16.8 \cdot ft$$

defines middle third of base

$$\frac{2}{3} \cdot w_{foundation} = 33.7 \cdot ft$$

defines middle third of base

$$check_{OT} := \begin{vmatrix} \text{"OK"} \quad \text{if } \frac{1}{3} w_{foundation} \le X_{Resultant} \le \frac{2}{3} w_{foundation} \\ \text{"NOT OK"} \quad \text{otherwise} \end{vmatrix}$$

$$\boxed{check_{OT} = \text{"OK"}}$$

OBE Case:

Check that location of overturning resultant falls in middle 1/2 of base of concrete gravity dam (unusual case)

$$\begin{aligned} \Sigma M_{toe.drive.OBE} := {} & F_{ko} \cdot (El_{ko} - El_{foundation}) \ldots \\ & + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \ldots \\ & + F_{uplift} \cdot (w_{foundation} - X_{uplift}) \ldots \\ & + F_{inertia.OBE} \cdot (El_{inertia.OBE} - El_{foundation}) \end{aligned} = 1210.573 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.OBE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 66.569 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant.OBE} := \frac{-\Sigma M_{toe.drive.OBE}}{\Sigma F_{vertical.total}} = -18.2 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the dam

$$\frac{1}{4} \cdot w_{foundation} = 12.6 \cdot ft$$

defines middle half of base

$$\frac{3}{4} \cdot w_{foundation} = 37.9 \cdot ft$$

defines middle half of base

$$check_{OT.OBE} := \begin{vmatrix} \text{"OK"} \quad \text{if } \frac{1}{4} w_{foundation} \le X_{Resultant} \le \frac{3}{4} w_{foundation} \\ \text{"NOT OK"} \quad \text{otherwise} \end{vmatrix}$$

$check_{OT.OBE} = \text{"OK"}$

MCE Case:

Check that location of overturning resultant falls within base of concrete gravity dam (extreme case)

$$\Sigma M_{toe.drive.MCE} := F_{ko}\cdot\left(El_{ko} - El_{foundation}\right) + F_{h2o}\cdot\left(El_{h2o} - El_{foundation}\right) + F_{uplift}\cdot\left(w_{foundation} - X_{uplift}\right) + F_{inertia.MCE}\cdot\left(El_{inertia.MCE} - El_{foundation}\right) = 1258.851\cdot\frac{kip\cdot ft}{ft}$$

$$\Sigma F_{vertical.MCE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 66.569\cdot\frac{kip}{ft}$$

$$X_{Resultant.MCE} := \frac{-\Sigma M_{toe.drive.MCE}}{\Sigma F_{vertical.total}} = -18.9\cdot ft$$

horizontal distance to resultant of overturning moment relative to face of the dam

$0\cdot w_{foundation} = 0\cdot ft$ defines upstream edge of base

$1\cdot w_{foundation} = 50.5\cdot ft$ defines downstream edge of base

$$check_{OT.MCE} := \begin{cases} \text{"OK"} & \text{if } 0w_{foundation} \le X_{Resultant} \le 1w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$check_{OT.MCE} = \text{"OK"}$

Remarks and Recapitulation:

- Calculation addresses sliding and overturning of the fixed crest section of Bixby Dam under anticipated static operating conditions, OBE seismic case, and MCE seismic case noted.
- For the static case and the MCE case, it is identified that permanent ground anchors would be necessary to achieve sliding stability.
- Anchors are not necessary for overturning stability for any cases.
- The static case (usual loading) was found to control.

Attachment 11

Evaluate Sliding and Overturning Full Height Gate

CH2MHILL

PROJECT : Arkansas River Corridor Project - Bixby Dam
PROJECT #: 657971.04.02.01
CREATED BY: **Mark Kacmarcik** **DATE:** 04/16/2015
REVIEWED BY: Jen Schaeffer **DATE:** 04/17/2015

Given: Simplified gravity dam geometry shown and generalized subsurface profile. See sketch.

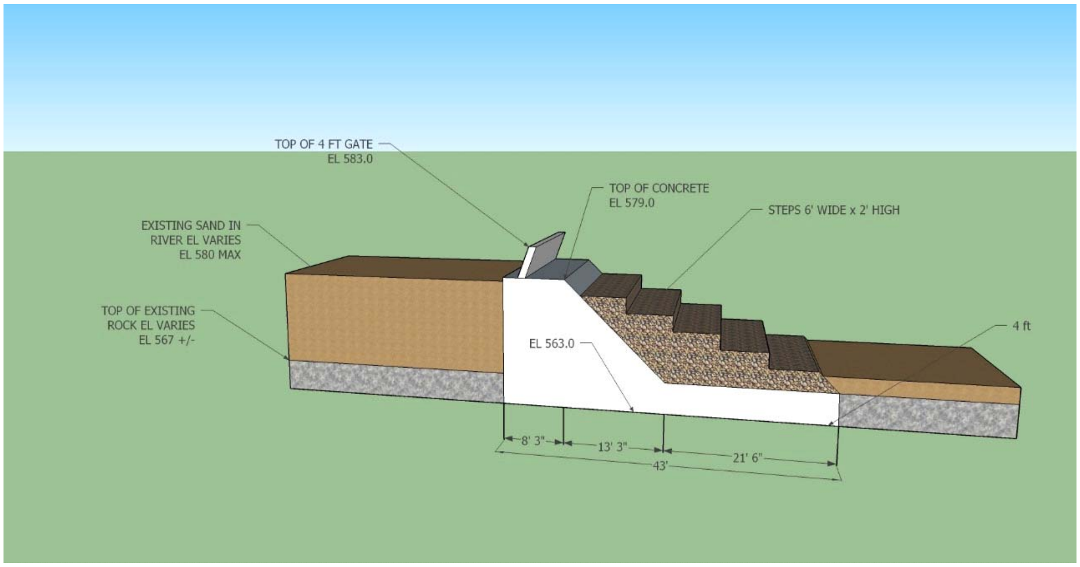
Find: Check sliding and overturning against USACE criteria for Static and Seismic cases as noted in the title. Anchor forces are included as needed to meet stability criteria. Note that this is not attempt to be a complete comprehensive check of all possible analysis cases, but rather the loading cases which are assumed to control overall dam design for preliminary sizing and concept evaluation.

Assumptions:
Ignore resistance from sediment or rock on downstream toe.
Ice loading is not considered.
Structure is not undermined by scour.
Upstream and downstream turndowns (not shown) are not relied upon for shear resistance.
All soil and rock layers are assumed to be horizontal.
Use single conservative frictional interface strength, as shown in the calculation.
Disregard cohesion for long term analysis.
Mass or contributions of pedestrian bridge ignored (conservative)
2 dimensional analysis considering dam geometry on a per-foot basis, 3Dimensional end effects not considered.
Steps shown in geometry are concrete or cut stone with similar unit weight to mass concrete.
Other assumptions as noted in the calculation

Inputs:
Approximate top of rock elevation for main dam, estimated at **El 567 ft**.
Dam foundation elevation assumed 4 feet below top of rock **(El 563 ft.)**
Water present to top of gate at **El 583.0 ft**.
Sediment elevation present to top of sill at **El 579.0 ft** as directed by Murry Fleming.
Tailwater elevation is coincident with dam foundation, **El 563.0 ft**.
2008 boreholes by Stantec used to estimate subsurface conditions and properties.
Other inputs as noted in the calculation.

References:
USACE EM 1110-2-2200 Gravity Dam Manual
USACE EM 1110-2-2100 Stability Analysis of Concrete Structures

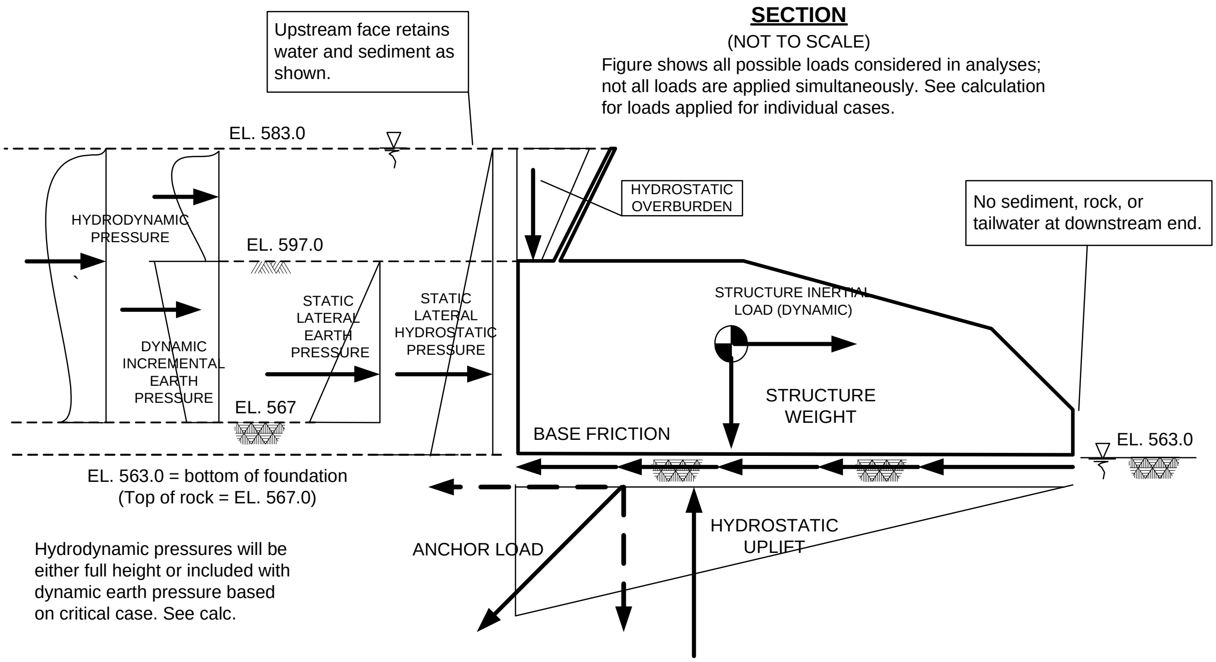
Full Height Gate Section Geometry:



FULL HEIGHT GATE SECTION

(NOT TO SCALE)

Figure shows all possible loads considered in analyses; not all loads are applied simultaneously. See calculation for loads applied for individual cases.



Define Geometry:

$El_{crest} := 583.0ft$ Elevation at top of dam crest

$El_{sill} := 579ft$ Elevation at the top of the sill (top of concrete)

$El_{rock} := 567ft$ Elevation of top of rock (shale)

$d_{excav} := 4ft$ Excavte below top of rock to remove weathered shale.

$El_{foundation} := El_{rock} - d_{excav} = 563 \cdot ft$ Elevation of bottom of dam

$H_{dam} := El_{crest} - El_{foundation} = 20 \cdot ft$ Total height of dam

$w_{foundation} := 43ft$ Given width of dam base

$El_{sed.top} := El_{sill} = 579 \cdot ft$ Assume that dam impounds sediment to top of concrete (sill).

$El_{water.US} := El_{crest} = 583 \cdot ft$ Elevation of water upstream of dam.

$El_{water.DS} := El_{foundation} = 563 \cdot ft$ Elevation of water downstream of dam (assume no water as recommended by USACE).

Material Properties:

Unit Weight:

$\gamma_{conc} := 150pcf$ Unit weight of concrete (assumed).

$\gamma_{sed} := 120pcf$ Unit weight of sediment against upstream face of dam (recommended by USACE EM 1110-2-2100)

$\gamma_{shale} := 152pcf$ Unit weight of Shale from Stantec, 2008 laboratory test results.

$\gamma_{w} := 62.4pcf$ Unit weight of water (assumed).

Shear Strength:

$\phi_{sed} := 28deg$ $c_{sed} := 0psf$ Effective stress shear strength of sediment.

Interface Strength (sliding):

$\delta_{base} := 24deg$ Consider only one sliding interface, mass concrete cast against shale bedrock. Assume no cohesion/adhesion along this interface, only base friciton. Typical value from NAVFAC DM7.2 for "Mass concrete cast against...very stiff and hard residual or preconsolidated clay".

Seismic:

$PGA_{OBE} := 0.009$ Peak ground acceleration on rock for Operations Basis Earthquake (OBE). 50% probability of exceedance in 100 years.

$PGA_{MCE} := 0.093$ Peak ground acceleration on rock for Maximum Credible Earthquake (MCE). 10% probability of exceedance in 50 years

$F_{PGA.scC} := 1.2$ Site coefficient for Site Class C, "Very Dense Soil and Soft Rock" (assumed).

$k_{h.MCE} := \frac{2}{3} \cdot PGA_{MCE} \cdot F_{PGA.scC} = 0.074$ Seismic coeff for MCE case (per EM 1110-2-2100 = 2/3 effective peak ground accel). Conservatively estimated using PGA for site class C.

$k_{h.OBE} := \frac{2}{3} \cdot PGA_{OBE} \cdot F_{PGA.scC} = 0.007$ Seismic coeff for OBE case.

$k_{v} := 0$ Neglect vertical component of earthquake acceleration (assumed).

Estimate Weight of Concrete Gravity Dam:

Estimate total stress (non buoyant) weight of concrete gravity dam by estimating area of the gravity dam polygon, and then multiplying it by the unit weight of the material

Centroid of polygon [edit] from Wikipedia (http://en.wikipedia.org/wiki/Polygon, February 27, 2014)

The centroid of a non-self-intersecting closed polygon defined by n vertices (x_0,y_0), (x_1,y_1), ..., (x_{n-1},y_{n-1}) is the point (C_x, C_y), where

$$C_x = \frac{1}{6A}\sum_{i=0}^{n-1}(x_i + x_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

$$C_y = \frac{1}{6A}\sum_{i=0}^{n-1}(y_i + y_{i+1})(x_i\ y_{i+1} - x_{i+1}\ y_i)$$

and where A is the polygon's signed area,

$$A = \frac{1}{2}\sum_{i=0}^{n-1}(x_i\ y_{i+1} - x_{i+1}\ y_i).$$ [9]

In these formulas, the vertices are assumed to be numbered in order of their occurrence along the polygon's perimeter, and the vertex (x_n, y_n) is assumed to be the same as (x_0, y_0). Note that if the points are numbered in clockwise order the area A, computed as above, will have a negative sign; but the centroid coordinates will be correct even in this case.

Define function to calculate area of polygon whose plane coordinates are contained in matrix XY

$$\text{Area}(XY) := \left|\begin{array}{l} XY \leftarrow \text{stack}\left[XY, \left(XY^T\right)^{\langle 0\rangle T}\right] \\ M \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left|\text{submatrix}(XY, i, i+1, 0, 1)\right| \\ 0.5\cdot M \end{array}\right.$$

Define function to calculate coordinates of centroid of non-intersecting closed polygon

$$\text{Centroid}(XY) := \left|\begin{array}{l} XY \leftarrow \text{stack}\left[XY, \left(XY^T\right)^{\langle 0\rangle T}\right] \\ x \leftarrow XY^{\langle 0\rangle} \\ y \leftarrow XY^{\langle 1\rangle} \\ C_x \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left[\left(x_i + x_{i+1}\right)\cdot\left(x_i\cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ C_y \leftarrow \displaystyle\sum_{i=0}^{\text{rows}(XY)-2} \left[\left(y_i + y_{i+1}\right)\cdot\left(x_i\cdot y_{i+1} - x_{i+1}\cdot y_i\right)\right] \\ \left(C_x \;\; C_y\right)\cdot\dfrac{1}{6\cdot\text{Area}(XY)} \end{array}\right.$$

Area and Centroid of Concrete Gravity Dam

$$XY_{dam} := \begin{pmatrix} 0 & 563 \\ 0 & 579 \\ 2 & 579 \\ 4 & 583 \\ 4.5 & 583 \\ 2.5 & 579 \\ 8.25 & 579 \\ 10.25 & 577 \\ 40.25 & 569 \\ 43 & 567 \\ 43 & 563 \\ 0 & 563 \end{pmatrix}$$

- Values define cross-sectional geometry of dam, points are clockwise around cross section, starting at upstream heel.
- Left column is X coordinates, "0" is the upstream heel of the dam, sign convention is positive to the right (downstream).
- Right column is elevation.

$$-\mathrm{Area}(XY_{dam}) = 477.75$$

$$\mathrm{Centroid}(XY_{dam}) = (17.528 \quad 569.137)$$ center of gravity for concrete gravity dam, ft

$$x_{dam\_CG} := \mathrm{Centroid}(XY_{dam})_{0,0} = 17.528$$

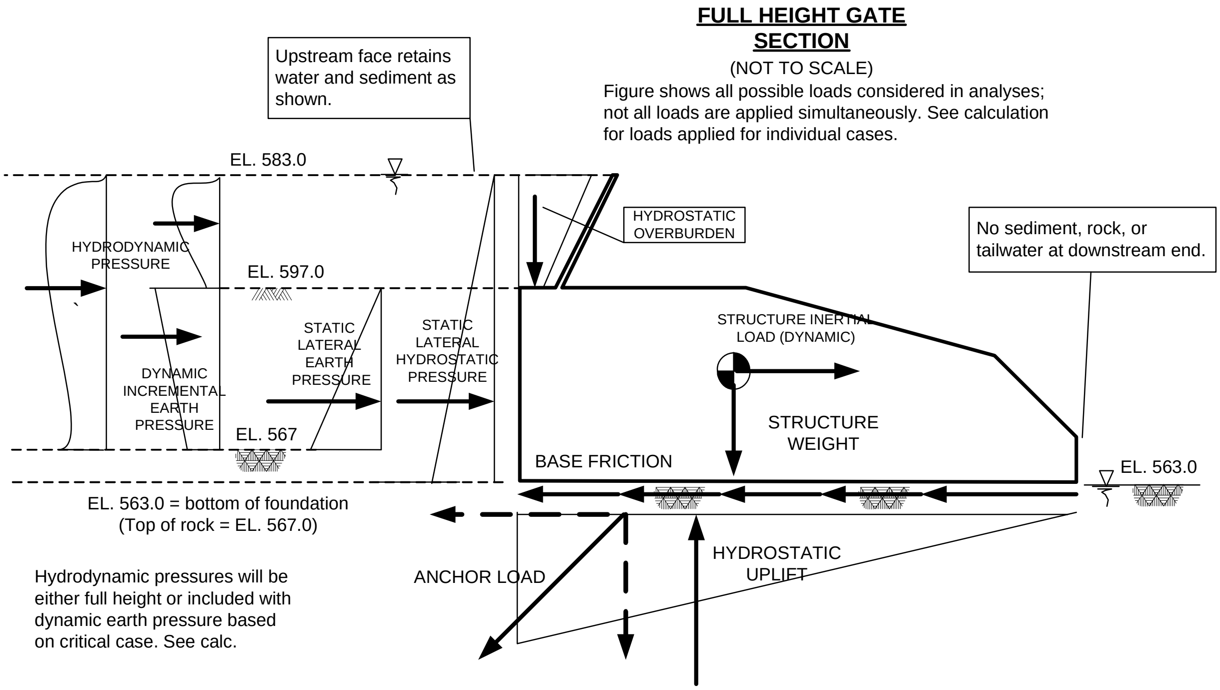
$$x_{dam} := x_{dam\_CG} \cdot 1ft = 17.528 \cdot ft$$ X-coordinate fo centroid, in feet

$$y_{dam} := \mathrm{Centroid}(XY_{dam})_{0,1} = 569.137$$

$$El_{centroid} := y_{dam} \cdot 1ft = 569.137 \cdot ft$$ Elevation of centroid

$$WT_{dam} := -\mathrm{Area}(XY_{dam}) \cdot ft^2 \cdot \gamma_{conc} = 71.7 \cdot \frac{kip}{ft}$$ Total weight of concrete gravity dam, per foot. No pedestrian bridge at Bixby.

Estimate Lateral Driving Forces Acting on Concrete Gravity Dam



Lateral Hydrostatic Water Load on Upstream Face:

$$H_w := El_{water.US} - El_{foundation} = 20 \cdot ft$$ Height of water

$$F_{h2o} := \frac{1}{2} \cdot \gamma_w \cdot H_w^{\ 2} = 12.5 \cdot \frac{kip}{ft}$$ Magnitude of resultant of hydrostatic load on upstream face of dam

$$El_{h2o} := El_{water.US} - \frac{2}{3} \cdot H_w = 569.7 \cdot ft$$ Elevation of resultant

Static At-Rest Lateral Earth Pressure on Upstream Face:

Assume sediment contributesr at-rest soil pressure on upstream face of dam (active pressures are not developed).

$$H_{ko} := El_{sed.top} - El_{rock} = 12 \cdot ft$$

$$K_0 := 1 - \sin(\phi_{sed}) = 0.531$$ At-rest soil pressure coefficient.

$$F_{ko} := \frac{1}{2} \cdot K_0 \cdot (\gamma_{sed} - \gamma_w) \cdot H_{ko}^{\ 2} = 2.2 \cdot \frac{kip}{ft}$$ Magnitude of resultant of at-rest soil pressure on upstream face of dam

$$El_{ko} := El_{sed.top} - \frac{2}{3} \cdot H_{ko} = 571 \cdot ft$$ Elevation of resultant.

SEISMIC: Lateral Hydrodynamic Water Load on Upstream Face:

This load is applied assuming the dam has been flushed of sediment, and full height of water applies hydrodynamic loading to dam structure during a seismic event. Note that, when sediment levels accumulate, hydrodynamic loading is not considered to be a valid case.

$$P_{hydro.MCE} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 0.693 \cdot klf$$ Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$$P_{hydro.OBE} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot (El_{crest} - El_{rock})^2 = 0.067 \cdot klf$$ Magnitude of hydrodynamic loading from free water from crest of dam to top of rock.

$$P_{hydro.MCE.partial} := \frac{7}{12} \cdot k_{h.MCE} \cdot \gamma_w \cdot \left(El_{crest} - El_{sed.top}\right)^2 = 0.043 \cdot klf$$

Magnitude of hydrodynamic loading over accumulated sediment.

$$P_{hydro.OBE.partial} := \frac{7}{12} \cdot k_{h.OBE} \cdot \gamma_w \cdot \left(El_{crest} - El_{sed.top}\right)^2 = 0.004 \cdot klf$$

Magnitude of hydrodynamic loadign over accumulated sediment.

$$El_{hydro.MCE} := El_{rock} + 0.4 \cdot \left(El_{crest} - El_{rock}\right) = 573.4 \cdot ft$$

Elevation of resultant of hydrodynamic load for full height water case.

$$El_{hydro.OBE} := El_{rock} + 0.4 \cdot \left(El_{crest} - El_{rock}\right) = 573.4 \cdot ft$$

Elevation of resultant of hydrodynamic load for full height water case.

$$El_{hydro.MCE.partial} := El_{sed.top} + 0.4 \cdot \left(El_{crest} - El_{sed.top}\right) = 580.6 \cdot ft$$

Elevation of resultant of hydrodynamic load for full sediment case.

$$El_{hydro.OBE.partial} := El_{sed.top} + 0.4 \cdot \left(El_{crest} - El_{sed.top}\right) = 580.6 \cdot ft$$

Elevation of resultant of hydrodynamic load for full sediment case.

SEISMIC: Lateral Earth Pressures Upstream Face:

$\theta_{wall} := 0deg$ — Slope of upstream face of dam, 0 indicates vertical face

$\delta_{sed} := 0deg$ — Interface friction angle between sediment and dam, assume zero degrees.

$\beta_{US} := 0deg$ — Slope of top of sediment against upstream face of dam. 0 degrees is horizontal.

$$f\_K_{A.c}(\phi, \delta, \beta, \theta) := \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\delta + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\delta + \phi) \cdot \sin(\phi - \beta)}{\cos(\delta + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

Function to calculate Coulomb active earth pressure coefficient

$$K_A := f\_K_{A.c}\left(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}\right) = 0.361$$

Coulomb active earth perssure coefficient

$$P_A := \frac{1}{2} \cdot K_A \cdot \left(\gamma_{sed} - \gamma_w\right)\left(El_{sed.top} - El_{rock}\right)^2 = 1.497 \cdot klf$$

Active earth pressure force.

$$f\_\psi\left(k_h, k_v\right) := atan\left(\frac{k_h}{1 - k_v}\right)$$

$$f\_K_{AE}(\phi, \delta, \beta, \theta, \psi) := \frac{\cos(\phi - \psi - \theta)^2}{\cos(\psi) \cdot \cos(\theta)^2 \cdot \cos(\psi + \theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \psi - \beta)}{\cos(\delta + \psi + \theta) \cdot \cos(\beta - \theta)}}\right)^2}$$

Check MCE:

$$\psi_{MCE} := f\_\psi\left(k_{h.MCE}, k_v\right) = 0.074$$

$K_{AE.MCE} := f\_K_{AE}(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{MCE}) = 0.409$

Examine active case only for upstream sediment. Neglect any downstream passive resistance.

$P_{AE.MCE} := (0.5 \cdot K_{AE.MCE}) \cdot (\gamma_{sed} - \gamma_w)(El_{sed.top} - El_{rock})^2 = 1.696 \cdot klf$

Seismic active earth pressure.

$\Delta P_{AE.MCE} := P_{AE.MCE} - P_A = 0.199 \cdot klf$

Dynamic incremental earth pressure in seismic MCE case.

$F_{MCE.partial} := \Delta P_{AE.MCE} + P_{hydro.MCE.partial} = 0.242 \cdot klf$

Soil + hydrodynamic water load above soil, total horiztonal applied force.

Check OBE:

$\psi_{OBE} := f\_\psi(k_{h.OBE}, k_v) = 0.007$

$K_{AE.OBE} := f\_K_{AE}(\phi_{sed}, \delta_{sed}, \beta_{US}, \theta_{wall}, \psi_{OBE}) = 0.365$

Examine active case only for upstream sediment. Neglect any downstream passive resistance.

$P_{AE.OBE} := (0.5 \cdot K_{AE.OBE}) \cdot (\gamma_{sed} - \gamma_w)(El_{sed.top} - El_{rock})^2 = 1.515 \cdot klf$

Active seismic earth pressure between top of sediment and top of rock.

$\Delta P_{AE.OBE} := P_{AE.OBE} - P_A = 0.018 \cdot klf$

Additional applied earth pressure in seismic OBE case.

$F_{OBE.partial} := \Delta P_{AE.OBE} + P_{hydro.OBE.partial} = 0.022 \cdot klf$

Soil + hydrodynamic water load above soil, total horiztonal applied force.

Compare Dynamic Lateral Earth Pressures to At-Rest Lateral Earth Pressures:

$P_{AE.OBE} = 1.515 \cdot klf$ Dynamic Active OBE

$P_{AE.MCE} = 1.696 \cdot klf$ Dynamic Active MCE

$F_{ko} = 2.2 \cdot klf$ Static At-Rest

Note that static at-rest loading is greater than dynamic active loading for both MCE and OBE cases. Use greater of static at-rest or dynamic active lateral earth pressures. In this case, static at-rest pressure controls and should be used as the lateral earth pressure for the dynamic analysis cases..

Determine controlling load case for upstream loading on structure:

Structure could be free water (no sediment accumulation), or filled with sediment. For seismic stability evaluations, estimate controlling case: either hydrodynamic loading of silt-free dam or dynamic lateral earth pressure + water over top of sediment.

$F_{ko} = 2.2 \cdot klf$ At-rest lateral earth pressure loading (note that static at-rest is controlling case for seismic evaluation)

$F_{h2o} = 12.48 \cdot klf$ Hydrostatic pressure

$P_{hydro.MCE} = 0.693 \cdot klf$ Hydrodynamic pressure over full height of structure, MCE event

$P_{hydro.OBE} = 0.067 \cdot klf$ Hydrodynamic pressure over full height of structure, OBE event

$P_{hydro.MCE.partial} = 0.043 \cdot klf$ Hydrodynamic pressure above top of sediment, MCE event. Include with soil loading case.

$P_{hydro.OBE.partial} = 0.004 \cdot klf$ Hydrodynamic pressure above top of sediment, OBE event. Include with soil loading case.

$P_{hydro.MCE} + F_{h2o} = 13.173 \cdot klf$

$F_{h2o} + F_{ko} + P_{hydro.MCE.partial} = 14.724 \cdot klf$

$$check_{hydro.MCE} := \begin{vmatrix} \text{"Hydrodynamic"} \quad \text{if } \left(P_{hydro.MCE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko} + P_{hydro.MCE.partial}\right) \\ \text{"Soil"} \quad \text{otherwise} \end{vmatrix}$$

$check_{hydro.MCE} = \text{"Soil"}$

$P_{hydro.OBE} + F_{h2o} = 12.547 \cdot klf$

$F_{h2o} + F_{ko} + P_{hydro.OBE.partial} = 14.684 \cdot klf$

$$check_{hydro.OBE} := \begin{vmatrix} \text{"Hydrodynamic"} \quad \text{if } \left(P_{hydro.OBE} + F_{h2o}\right) > \left(F_{h2o} + F_{ko} + P_{hydro.OBE.partial}\right) \\ \text{"Soil"} \quad \text{otherwise} \end{vmatrix}$$

$check_{hydro.OBE} = \text{"Soil"}$

SEISMIC: Inertial Load of Structure:

$F_{inertia.MCE} := k_{h.MCE} \cdot WT_{dam} = 5.332 \cdot klf$ Seismic inertia load of the dam for MCE, acts in downstream direction.

$F_{inertia.OBE} := k_{h.OBE} \cdot WT_{dam} = 0.516 \cdot klf$ Seismic inertia load of the dam for OBE acts in downstream direction.

$El_{inertia.MCE} := El_{centroid} = 569.137 \cdot ft$ Seismic inertial load acts through centroid of dam

$El_{inertia.OBE} := El_{centroid} = 569.137 \cdot ft$ Seismic inertial load acts through centroid of dam

Estimate Uplift Hydrostatic Forces Acting on Concrete Gravity Dam

Hydrostatic Uplift on Dam Base:

Magnitude of hydrostatic uplift is estimated as straightline interpolation between headwater and tailwater across width of structure. Figure above shows assumed uplift distribution below bottom of dam.

Use centroid equation to define uplift pressure.

$$XY_{uplift} := \begin{pmatrix} w_{foundation} & 0 \\ w_{foundation} & El_{water.DS} - El_{foundation} \\ 0 & El_{water.US} - El_{foundation} \\ 0 & 0 \end{pmatrix} \cdot ft^{-1}$$

$w_{foundation} = 43 \cdot ft$

$Area(XY_{uplift}) = 430$

$Centroid(XY_{uplift}) = (14.333 \quad 6.667)$

$X_{uplift} := (1ft\,Centroid(XY_{uplift}))_{0,0} = 14.333 \cdot ft$

$F_{uplift} := Area(XY_{uplift}) \cdot ft^2 \cdot \gamma_w = 26.832 \cdot \frac{kip}{ft}$

Estimate Resisting Forces:

Estimate base sliding resistance for concrete gravity dam sliding on rock. Account for hydrostatic overburden above upstream face dam:

Hydrostatic Overburden Volume above upstream face of Dam:

$$XY_{hydroOB} := \begin{pmatrix} 0 & 579 \\ 2 & 579 \\ 4 & 583 \\ 0 & 583 \end{pmatrix}$$

$Area(XY_{hydroOB}) = 12$

$Centroid(XY_{hydroOB}) = (1.556 \quad 581.222)$

$x_{h2o.vert} := (1ft\,Centroid(XY_{hydroOB}))_{0,0} = 1.556 \cdot ft$

$F_{h2o.vert} := Area(XY_{hydroOB}) \cdot ft^2 \cdot \gamma_w = 0.749 \cdot \frac{kip}{ft}$

Interface friction between concrete gravity dam and shale bedrock:

$\delta_{base} = 24 \cdot deg$ Base friction angle between dam and foundation.

$F_{base} := (WT_{dam} + F_{h2o.vert} - F_{uplift}) \cdot tan(\delta_{base}) = 20.3 \cdot \frac{kip}{ft}$ Base friction, sum of vertical forces multiplied by tangent of interface friction times tangent of interface friction (delta).

Estimate Factor of Safety Against Sliding:

The recommended global stability design criteria is summarized in the USACE Gravity Dam Design EM 1110-2-2200. Stability criteria is summarized in Table 4-1 below.

EM 1110-2-2200
30 Jun 95

Table 4-1
Stability and stress criteria

Load Condition	Resultant Location at Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress: Compressive	Concrete Stress: Tensile
Usual	Middle 1/3	2.0	≤ allowable	$0.3\ f'_c$	0
Unusual	Middle 1/2	1.7	≤ allowable	$0.5\ f'_c$	$0.6\ f_c'^{2/3}$
Extreme	Within base	1.3	≤ 1.33 × allowable	$0.9\ f'_c$	$1.5\ f_c'^{2/3}$

Note: f'_c is 1-year unconfined compressive strength of concrete. The sliding factors of safety (*FS*) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions.

$FS_{min} := 2.0$ Usual loading. Minimum sliding factor of safety recommended by USACE (from table above)

$\Sigma F_{h.drive} := F_{h2o} + F_{ko} = 14.7 \cdot \frac{kip}{ft}$ Sum of driving forces (hydrostatic pressure + at rest lateral earth pressure)

$\Sigma F_{h.resist} := F_{base} = 20.3 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$$\boxed{FS_{slide.shale} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive}} = 1.38}$$

$$check_{slide.shale} := \begin{array}{|l} \text{"OK"} \quad \text{if } FS_{slide.shale} > FS_{min} \\ \text{"NOT OK-anchors required"} \quad \text{otherwise} \end{array}$$

$$\boxed{check_{slide.shale} = \text{"NOT OK-anchors required"}}$$

MCE Seismic Sliding:

$FS_{min.MCE} := 1.3$ MCE, extreme loading. Minimum sliding factor of safety recommended by USACE (from table above).

$\Sigma F_{h.drive.MCE} := F_{h2o} + F_{ko} + P_{hydro.MCE.partial} + F_{inertia.MCE} = 20.1 \cdot \frac{kip}{ft}$

Sum of lateral driving forces during MCE event.

$\Sigma F_{h.resist} := F_{base} = 20.3 \cdot \frac{kip}{ft}$ Sum of resisting forces (base friction)

$$\boxed{FS_{slide.shale.MCE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.MCE}} = 1.01}$$

$$check_{slide.shale.MCE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.MCE} > FS_{min.MCE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.MCE} = \text{"NOT OK-anchors required"}$$

OBE Sliding:

$FS_{min.OBE} := 1.7$

OBE, Unusual loading. Minimum sliding factor of safety recommended by USACE (from table above).

$$\Sigma F_{h.drive.OBE} := F_{h2o} + F_{ko} + P_{hydro.OBE.partial} + F_{inertia.OBE} = 15.2 \cdot \frac{kip}{ft}$$

Sum of lateral driving forces during OBE event.

$$\Sigma F_{h.resist} := F_{base} = 20.3 \cdot \frac{kip}{ft}$$

Sum of resisting forces (base friction)

$$FS_{slide.shale.OBE} := \frac{\Sigma F_{h.resist}}{\Sigma F_{h.drive.OBE}} = 1.34$$

$$check_{slide.shale.OBE} := \begin{cases} \text{"OK"} & \text{if } FS_{slide.shale.OBE} > FS_{min.OBE} \\ \text{"NOT OK-anchors required"} & \text{otherwise} \end{cases}$$

$$check_{slide.shale.OBE} = \text{"NOT OK-anchors required"}$$

Estimate Required Anchor Force to Acheive Minimum Sliding Factor of Safety:

$$F_{anchor} := FS_{min} \cdot \Sigma F_{h.drive} - \Sigma F_{h.resist} = 9.067 \cdot klf$$

Use minimum FS against sliding to determine anchor forc
This is the horizontal component of the anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.static} := \frac{F_{anchor}}{\cos(\alpha_{anchor})} = 12.823 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Check Seismic:

MCE case:

$$F_{anchor.MCE} := FS_{min.MCE} \cdot \Sigma F_{h.drive.MCE} - \Sigma F_{h.resist} = 5.779 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of tl anchor.

$\alpha_{anchor} := 45deg$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.MCE} := \frac{F_{anchor.MCE}}{\cos(\alpha_{anchor})} = 8.172 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

OBE case:

$$F_{anchor.OBE} := FS_{min.OBE} \cdot \Sigma F_{h.drive.OBE} - \Sigma F_{h.resist} = 5.547 \cdot klf$$

Use minimum FS against sliding to determine anchor force. This is the horizontal component of tl anchor.

$$\alpha_{anchor} := 45deg$$

Angle of anchor installation measured from horizontal (assumed).

$$T_{anchor.OBE} := \frac{F_{anchor.OBE}}{\cos(\alpha_{anchor})} = 7.845 \cdot klf$$

Total **allowable** anchor force required per linear foot of dam.

Determine Critical Anchor Force for Design:

$$\boxed{T_{anchor.critical} := \max(T_{anchor.static}, T_{anchor.MCE}, T_{anchor.OBE}) = 12.823 \cdot klf}$$

Estimate Factor of Safety Against Overturning:

Sum moments around downstream toe. Note this is not directly comparable to USACE overturning criteria but useful as a quick che of stability, see estimation of overturning resultant and % base compression below.

$$\Sigma M_{toe.drive} := F_{ko} \cdot (El_{ko} - El_{foundation}) + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \dots + F_{uplift} \cdot (w_{foundation} - X_{uplift}) = 869.986 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma M_{toe.resist} := WT_{dam} \cdot (w_{foundation} - x_{dam}) + F_{h2o.vert} \cdot (w_{foundation} - x_{h2o.vert}) = 1856.434 \cdot \frac{kip \cdot ft}{ft}$$

$$\boxed{FS_{overturning} := \frac{\Sigma M_{toe.resist}}{\Sigma M_{toe.drive}} = 2.13}$$

Factor of safety against overturning, static case.

There is no specified factor of safety provided by USACE against overturning. The USACE does recommend that for the Normal/Usual loading scenario, the overturning resultant should be located within the middle 1/3 of the base of the dam, and for the unusual loading scenario, the middle 1/2 of the dam.

Estimate Location of Overturning Resultant:

Static Case:

Check that location of overturning resultant falls in middle 1/3 of base of concrete gravity dam (usual case)

$$\Sigma M_{toe.total} := \Sigma M_{toe.drive} - \Sigma M_{toe.resist} = -986.448 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.total} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 45.579 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant} := \frac{-\Sigma M_{toe.total}}{\Sigma F_{vertical.total}} = 21.6 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the wall

$$\frac{1}{3} \cdot w_{foundation} = 14.3 \cdot ft$$

defines middle third of base

$$\frac{2}{3} \cdot w_{foundation} = 28.7 \cdot ft$$

defines middle third of base

$$check_{OT} := \left| \begin{array}{ll} \text{"OK"} & \text{if } \frac{1}{3} w_{foundation} \le X_{Resultant} \le \frac{2}{3} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right.$$

$$\boxed{check_{OT} = \text{"OK"}}$$

OBE Case:

Check that location of overturning resultant falls in middle 1/2 of base of concrete gravity dam (unusual case)

$$\begin{aligned} \Sigma M_{toe.drive.OBE} := & F_{ko} \cdot \left(El_{ko} - El_{foundation}\right) \ldots \\ & + F_{h2o} \cdot \left(El_{h2o} - El_{foundation}\right) \ldots \\ & + F_{uplift} \cdot \left(w_{foundation} - X_{uplift}\right) \ldots \\ & + F_{inertia.OBE} \cdot \left(El_{inertia.OBE} - El_{foundation}\right) \ldots \\ & + P_{hydro.OBE.partial} \cdot \left(El_{hydro.OBE.partial} - El_{foundation}\right) \end{aligned} = 873.226 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.OBE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 45.579 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant.OBE} := \frac{-\Sigma M_{toe.drive.OBE}}{\Sigma F_{vertical.total}} = -19.2 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the dam

$$\frac{1}{4} \cdot w_{foundation} = 10.8 \cdot ft$$

defines middle half of base

$$\frac{3}{4} \cdot w_{foundation} = 32.3 \cdot ft$$

defines middle half of base

$$check_{OT.OBE} := \begin{cases} \text{"OK"} & \text{if } \frac{1}{4} w_{foundation} \le X_{Resultant} \le \frac{3}{4} w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{OT.OBE} = \text{"OK"}}$$

MCE Case:

Check that location of overturning resultant falls within base of concrete gravity dam (extreme case)

$$\begin{aligned} \Sigma M_{toe.drive.MCE} := {} & F_{ko} \cdot (El_{ko} - El_{foundation}) \cdots \\ & + F_{h2o} \cdot (El_{h2o} - El_{foundation}) \cdots \\ & + F_{uplift} \cdot (w_{foundation} - X_{uplift}) \cdots \\ & + F_{inertia.MCE} \cdot (El_{inertia.MCE} - El_{foundation}) \cdots \\ & + P_{hydro.MCE.partial} \cdot (El_{hydro.OBE.partial} - El_{foundation}) \end{aligned} \quad = 903.471 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma F_{vertical.MCE} := WT_{dam} + F_{h2o.vert} - F_{uplift} = 45.579 \cdot \frac{kip}{ft}$$

$$\boxed{X_{Resultant.MCE} := \frac{-\Sigma M_{toe.drive.MCE}}{\Sigma F_{vertical.total}} = -19.8 \cdot ft}$$

horizontal distance to resultant of overturning moment relative to face of the dam

$0 \cdot w_{foundation} = 0 \cdot ft$ defines upstream edge of base

$1 \cdot w_{foundation} = 43 \cdot ft$ defines downstream edge of base

$$check_{OT.MCE} := \begin{cases} \text{"OK"} & \text{if } 0 w_{foundation} \le X_{Resultant} \le 1 w_{foundation} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\boxed{check_{OT.MCE} = \text{"OK"}}$$

Remarks and Recapitulation:

- Calculation addresses sliding and overturning of the fixed crest section of the Bixby Dam under anticipated static operating conditions, OBE seismic case, and MCE seismic case noted.
- For all cases, it is identified that permanent ground anchors are necessary for sliding stability.
- Anchors are not necessary for overturning stability.
- The static case (usual loading) was found to control.

Attachment 12

Scope of Work - Geotechnical Exploration Program for Bixby Low Water Dam

PREPARED BY:	CH2M HILL
DATE:	1/26/2015
PROJECT NUMBER:	386594.02.2D.03

Background

Tulsa County, as part of the Arkansas River Corridor Master Plan is undertaking an improvement project on the Arkansas River. The primary goals of the overall project are to improve safety, improve fish habitat and fish passage, improve the function of the river system itself, enhance economic development, increase recreational opportunities, and increase connectivity between the river and surrounding communities. The conceptual project components include design of a new low water dam on the Arkansas River at Bixby, Oklahoma.

Site-specific geotechnical information is needed to support preliminary evaluations of the dam for concept development and budgeting. This document provides requirements for geotechnical data collection, laboratory testing, and reporting to support evaluations at Bixby Dam.

Previous Geotechnical Data

At this time, no project specific geotechnical data is available for the proposed dam alignment. A previous study was conducted by Terracon (2009) to support a water pipeline crossing. The nearest boreholes conducted for this project were more than 300 feet from the proposed Bixby Dam alignment, and no boreholes were advanced in the river channel. The Terracon report is included for reference as Attachment 1.

General Requirements

Characterization of the bedrock and overburden materials at the dam site is the primary geotechnical data needed. Data is needed to support preliminary feasibility evaluations and structure sizing including global stability (bearing or sliding), underseepage, excavation design, cofferdamming, etc. Identification of subsurface features such as bedding planes, depths of weathering, scour channels, coal seams, organic materials, or fill materials, are of particular interest.

Drilling and Sampling Requirements

The drilling contractor will mobilize sufficient drilling equipment and experienced drill crew personnel to the site to advance boreholes through soil and rock overburden and into bedrock, and then advance and collect the required size rock cores as specified in the following sections to the depths required. Wire-line rock coring methodology is preferred and rock core recovery in excess of 95% in sandstone formations and 90% in other lithologies should be achieved. Drilling will commence within two weeks of notice to proceed and continue generally uninterrupted until all borings are completed. Work schedule will generally be Monday through Friday, five days per week unless prior arrangements are made. For safety, all drilling work shall be performed during daylight hours and will stop during inclement weather. Contractor will mobilize sufficient equipment and experienced personnel to complete the field work described within 4 weeks of the Notice to Proceed. Notice to proceed will be issued following driller selection and contracting completion.

Whole, continuous, HQ size rock core will be drilled and collected from the top of sound, unweathered bedrock downward to a depth specified. Rock core will be immediately placed in wooden core boxes constructed such that each box contains and maintains the alignment and orientation of 10 to 15 feet of rock core. Core boxes will be labeled with the project site name, the borehole number, and the depth of the rock core contained within each box. The driller's personnel on site will document the depths drilled, rock core recovered, and provide a general description of the rock encountered.

Prior to commencing work, the Contractor should visit the site and become familiar with the overall site conditions, including the road network (or lack thereof) leading to each borehole site and plan on mobilizing equipment needed to locate the drilling equipment at the planned borehole sites. Borehole locations may not be moved from the proposed locations marked and located by the Engineer's staff prior to drill rig mobilization unless specific approval in writing is provided by the Engineer.

The Driller is responsible for monitoring weather and water level conditions and removing any equipment that may be in danger of flood damage. Neither the Owner nor the Engineer are responsible for any losses due to flooding, etc. the Contractor may experience while performing this work. Upon completion of each borehole, the hole will be tremie grouted with neat cement from the bottom to the ground surface. Grout will be mixed to no less than 13 pounds per gallon and pumped to the bottom of each borehole using a grout tremie pipe. Grout will be pumped until grout returns at the surface are within 5% of the weight of the grout being pumped into the borehole. Any grout overfill or grout water mixtures that leave the boreholes will be collected or contained and not allowed to flow into the adjacent streams.

Upon completion of each boring, including grouting, all Contractor materials, including any waste or supplies, will be collected and removed as the drilling equipment is moved to the next borehole. The Contractor shall be responsible for disposal of all waste. Contractor shall repair any damage to turf, landscaping, pavements, curbs, sidewalks, or other infrastructure which are incurred during the investigation at his sole expense. Contractor shall provide erosion control and/or stabilization measures at all drill sites or access routes to comply with all applicable regulations. Under no circumstances shall drilling fluids, oils, lubricants, hydraulic fluids, fuels, or other substances be allowed to enter water or soil. For each borehole, the drill rig shall be underlain by plastic sheeting to contain spills and leaks. The Contractor shall have appropriate prevention, containment, and cleanup measures on hand at all times. Contractor shall be responsible for all permits which may be required to complete the Work.

Driller will provide copies of drill logs, core descriptions and daily footage and bid item tallies at the end of each work day to the Engineer's representative on site. Contractor is responsible for any required business or professional licenses needed to perform this work. All equipment provided for completion of this work shall be in good working condition and manned by personnel having the knowledge needed to perform the work described. Should delays in work due to equipment failure or breakdown occur, the Owner may at their discretion, terminate the work and secure the services of another Contractor to complete the work.

The geotechnical subconsultant shall be responsible to coordinate with the client for identification and location of all surface and underground utilities in the area of the work. If any of the proposed boreholes will not be at a safe distance from a utility, the geotechnical subconsultant shall notify CH2M HILL, who will relocate the proposed borehole(s) as needed.

Work Plan, Schedule, and Qualifications

The geotechnical subconsultant shall prepare a Statement of Qualifications (SOQ), Work Plan, Schedule, and Price for executing the geotechnical exploration and laboratory testing required by this Scope of Work (SOW). Include the number of drill rigs or other major pieces of equipment to be employed, and the completion date for the proposed scope of work. Include the anticipated completion from Notice to Proceed (NTP) for each component of the work (drilling, laboratory testing and geotechnical report).

Geotechnical subconsultant shall submit the qualifications of all relevant subcontractors (if applicable) that will be used to assist in the geotechnical investigation; including the geotechnical drilling contractor, the laboratory performing materials testing, and any other subcontractor(s).

Geotechnical subconsultant shall describe the equipment proposed for use by the drilling subcontractor. Since access to drilling sites may be relatively difficult, drilling shall be performed using a track-mounted drill rig, if necessary. Drill rigs shall he capable of drilling and sampling to depths of 100 feet in the project area. The geotechnical subconsultant shall require its drilling subcontractor to provide drilling equipment, personnel, and drilling method that accomplishes the objectives of the scope of work including; ability to recover continuous rock core of adequate quality to accomplish the proposed classifications and testing; ability to achieve the drilling depths required for the project; and ability to properly abandon borings in accordance with all applicable local, state, and federal regulations. The geotechnical subconsultant shall coordinate with its drilling subcontractor to determine the equipment and personnel required to perform the work.

Geotechnical subconsultant shall coordinate with the Owner to have all boreholes surveyed. Northing, Easting, and Elevation values shall be reported to the nearest 0.1 feet.

Rock core shall be logged and data regarding number of joints, condition of joints, rock type, weathering and hardness, RQD, core recovery, etc. shall be recorded in a rock core log. Rock core shall be stored in wooden core boxes with hinged lids. Photos shall be taken of each rock core immediately following removal from the core barrel and be included in the geotechnical report. Photos shall also be taken of the final core boxes, with all appropriate labeling, and also included in the report. Samples of rock core will be used for Unconfined Compressive Strength and other testing as defined in the specific investigation requirements, and these samples should be wrapped or otherwise preserved to as best as possible maintain its in-situ character until testing. About three times as many samples should be taken as will be eventually tested. The samples that are selected for wrapping shall be coordinated with the CH2M HILL geotechnical engineer.

The client in conjunction with the CH2M HILL geotechnical engineer will determine which of the firms is best qualified to perform the work based on the consultant's qualifications, work plan, schedule and price to complete the required work. At the conclusion of all required geotechnical field exploration and laboratory testing work, the geotechnical consultant shall prepare and submit a written Geotechnical Data Report (GDR) describing the results of the field investigation and laboratory testing.

A draft copy of the GDR shall be submitted to the CH2M HILL geotechnical engineer for review. The CH2M HILL geotechnical engineer will provide comments that shall be addressed by the geotechnical consultant to the CH2M HILL geotechnical engineer's satisfaction prior to preparing and submitting the final GDR.

Specific Investigation Requirements

Drill five HQ-size borings at the proposed dam location to the approximate depths indicated on the boring location plan attached (Figure 1). One 100-ft boring will be advanced at each river bank, and three 60-ft borings will be advanced in the river channel. All borings will be advanced in a vertical orientation. Casing, either driven or drilled, shall be used where necessary to prevent caving. Casing shall not be advanced to a depth greater than that of the next sample or test interval. When bedrock is encountered, the casing shall be so seated that core drilling can proceed efficiently. This will typically require casing being drilled into rock. Casing shall be removed after completion of drilling, sampling, and grouting.

Standard Penetration Testing shall be performed at 2.5 foot intervals in all overburden materials, or more frequently if poor recovery is obtained or additional information is needed to characterize the overburden. All overburden drilling below the groundwater table shall be conducted with mud rotary drilling techniques to control heave. Hollow stem auger techniques may be used above the groundwater table.

If soft cohesive materials are encountered, relatively undisturbed thin-walled Shelby tube samples shall be collected for laboratory testing. Tube samples shall be sealed, labelled, stored upright and protected from vibrations and extreme fluctuations in temperature. Where possible, conduct Torvane testing on relatively fresh faces of the tube samples before sealing and note the materials exposed. All Shelby tube samples shall be immediately followed-up with Standard Penetration Test samples before drilling is resumed.

Laboratory Testing Requirements

The Laboratory testing program will be tailored to characterize subsurface materials and develop index and engineering properties to be used in the geotechnical design. All laboratory tests will be conducted in accordance with the appropriate ASTM standards. Samples for testing will be selected as coordinated with CH2M HILL. The following tests are anticipated, however, the exact number of tests will vary from that shown depending on the actual conditions encountered and sampled from the boreholes:

Test Type	No. of Tests required
Soil Samples	
Grain Size Distribution (no hydrometer) [ASTM D422]	10
Grain size Distribution (with hydrometer) [ASTM D422]	5
Percent Fines Determination [ASTM D1140]	10
Water Content [ASTM D2216]	10
Atterberg Limits [ASTM D4318]	5
Direct Shear Test on Undisturbed Samples [ASTM D3080]	2
One-Dimensional Consolidation [ASTM D2435]	2
Rock Core	
Unconfined compression Strength Tests Dry (rock) [ASTM D7012]	10
Slake Durability [ASTM D4644]	10

Deliverables

The firm responsible for performing the geotechnical investigation program will be directly contracted to the client, but will closely coordinate with the responsible CH2M HILL geotechnical engineer for (1) confirmation of the scope of work, (2) submittal of results of ongoing field investigations at a frequency determined by the CH2M HILL geotechnical engineer, and (3) any changes in site exploration or laboratory testing that need to be made during the investigation as a result of findings during the investigation. The reason for this oversight is to ensure that CH2M HILL as the responsible dam designer obtains the applicable information necessary for the various analyses and evaluations.

CH2M HILL will communicate early design concepts to the geotechnical subconsultant for planning any needed adjustments to the investigation. The geotechnical subconsultant's field geotechnical engineer shall be in daily contact with the CH2M HILL geotechnical engineer to report on the status of the work and to communicate results of investigations, and shall submit field boring and test pit logs to the CH2M HILL engineer within 48 hours of data collection.

Proposal

The following items shall be submitted by the geotechnical subconsultant as part of their proposal:

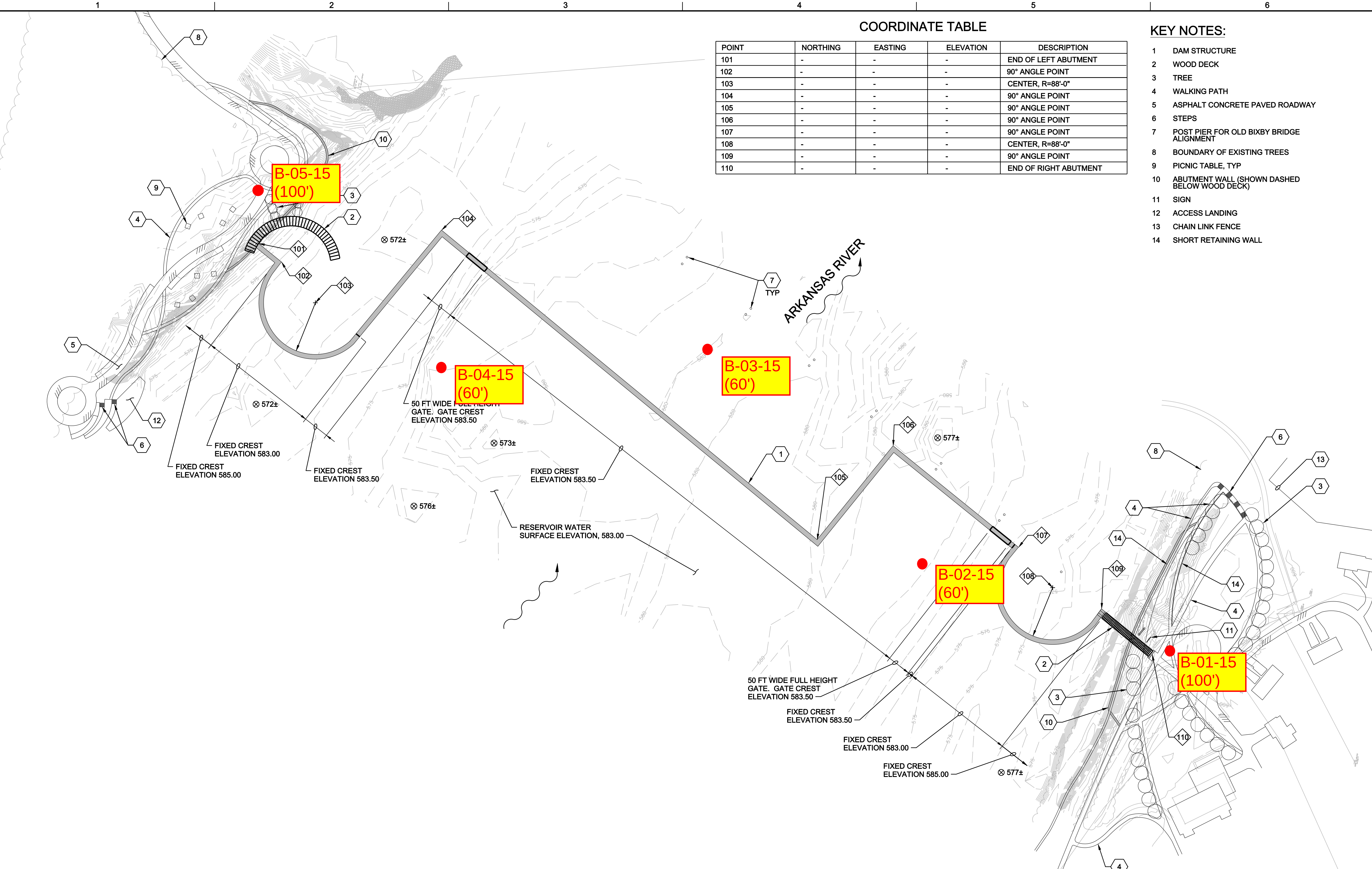
- Cost to complete the Scope of Work, and a breakdown of unit price items;
- Price per foot for borings in addition to those required per the scope of work described above;
- Definition of the complete proposal of work including equipment, tools, materials, and labor required to perform field investigation, laboratory tests, and Geotechnical Data Report (see requirements below); and
- Schedule including site investigation, laboratory analyses, and preparation of written report.
- An example of the rock core log template proposed for recording the boring logs. This template shall be revised based on the preferences of CH2M HILL.

Geotechnical Data Report (GDR)

Following completion of the field investigation and laboratory testing, a draft and final Geotechnical Data Report shall be prepared by the geotechnical subconsultant. The draft report will be reviewed by the CH2M HILL geotechnical engineer responsible for the geotechnical design. The Geotechnical Data Report shall provide the following information:

- Map showing the locations of the soil/rock core borings in the context of the proposed dam layout and topography of the terrain (base map with the site topography and the conceptual layout of the dam footprint will be made available to the geotechnical consultant);
- Logs of the borings showing subsurface lithology, measured ground water levels, notations describing drilling conditions, and other pertinent information;
- Results of all in-situ field testing;
- Results of the laboratory test results (raw data shall be included in an Appendix);
- Summary of the site investigation including drilling methods and equipment, sampling methods, and in situ testing methods.
- Summary table of borehole locations, including northing, easting, and elevation, depth penetrated, and depth to bedrock.
- Summary of drilling logistics, weather conditions, river stage, access logistics, and difficulties encountered.
- Photos of each rock core run immediately after sample retrieval. Photos shall clearly indicate borehole number, run number, depth interval, top and bottom of sample, date, and a graphical scale. (photos shall be included in an Appendix)
- Photos of each completed core box. Boxes shall be clearly labeled to indicate borehole number, run number, depth intervals, depths, recovery, RQD, and a graphical scale. (photos shall be included in an appendix)
- Photos of each borehole site and other general photos of the exploration activities or access which are pertinent to the project.
- Summary of the laboratory testing program including testing methods and equipment; and test results;

Three printed and one electronic (Adobe® Acrobat® PDF formatted to allow commenting) copies of a draft report shall be submitted to the client and CH2M HILL for review and comment. The geotechnical consultant shall respond to all comments and issue three final printed and one Adobe® Acrobat® PDF report copies to the client and CH2M HILL. Electronic copies of final figures shall be submitted in Adobe® Acrobat® PDF as well as in native CAD format (AutoCAD, MicroStation, etc.) if CAD format was used for figure preparation.



COORDINATE TABLE

POINT	NORTHING	EASTING	ELEVATION	DESCRIPTION
101	-	-	-	END OF LEFT ABUTMENT
102	-	-	-	90° ANGLE POINT
103	-	-	-	CENTER, R=88'-0"
104	-	-	-	90° ANGLE POINT
105	-	-	-	90° ANGLE POINT
106	-	-	-	90° ANGLE POINT
107	-	-	-	90° ANGLE POINT
108	-	-	-	CENTER, R=88'-0"
109	-	-	-	90° ANGLE POINT
110	-	-	-	END OF RIGHT ABUTMENT

KEY NOTES:

1 DAM STRUCTURE
2 WOOD DECK
3 TREE
4 WALKING PATH
5 ASPHALT CONCRETE PAVED ROADWAY
6 STEPS
7 POST PIER FOR OLD BIXBY BRIDGE ALIGNMENT
8 BOUNDARY OF EXISTING TREES
9 PICNIC TABLE, TYP
10 ABUTMENT WALL (SHOWN DASHED BELOW WOOD DECK)
11 SIGN
12 ACCESS LANDING
13 CHAIN LINK FENCE
14 SHORT RETAINING WALL

FIGURE 1: BOREHOLE LOCATIONS MAP
BIXBY LOW WATER DAM: SOW FOR PRELIMINARY GEOTECHNICAL INVESTIGATION

M. KACMARCIK
26 JANUARY 2015

NO.	DATE	REVISION	BY	APVD

DSGN M LITTLE · DR M RICKABY · CHK K WHITTIER · APVD L OSTERVOLD

CH2MHILL

BIXBY LOW WATER DAM
BIXBY, OKLAHOMA

CIVIL
OVERALL SITE PLAN

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

DATE
PROJ 386594
DWG 05-C-2000
SHEET

ISSUED FOR CONSTRUCTION