Presented To:

WESTERN FARMERS ELECTRIC COOPERATIVE

October 17, 2016

Combined Initial Hazard Potential Classification, Structural Stability, and Safety Factor Assessment Report

Project No.

OK00032129

guernsey

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REALIZE THE DIFFERENCE

ENGINEERING CERTIFICATION

Pursuant to 40 CFR 257.73(a)(2), (d), and (e) and by means of this certification I attest that:

(i) I am familiar with the requirements of 40 CFR Part 257 (CCR Rule);

(ii) I, or my agent, have visited and inspected the bottom ash impoundment at the Facility that is the subject of this Combined Initial Hazard Potential Classification, Structural Stability, and Safety Factor Assessment Report;

(iii) The aforementioned inspection(s) and this Combined Initial Hazard Potential Classification, Structural Stability, and Safety Factor Assessment Report, and the assessments described herein, have been conducted and prepared in accordance with good engineering practices, including consideration of applicable industry standards, and with the requirements of the CCR Rule; and

(iv) This Combined Initial Hazard Potential Classification, Structural Stability, and Safety Factor Assessment Report meets the requirements of 40 CFR 257.73(a)(2), (d), and (e).

(v) I am a "Qualified Professional Engineer" as required by 40 CFR 257.73(a)(2), (d), and (e) and as defined by 40 CFR 257.53 by the fact that I am a currently registered Civil Engineer in the State of Oklahoma and I have the technical knowledge and experience to make the specific technical certifications set forth herein.

C.H. GUERNSEY & COMPANY

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1 BACKGROUND

Western Farmers Electric Cooperative (WFEC) retained **Guernsey** to perform the Initial Hazard Potential Classification, Structural Stability, and Safety Factors Assessments (collectively, Assessment) of the Coal Combustion Residual (CCR) impoundment at its Hugo Power Plant (Hugo Plant) pursuant to the Environmental Protection Agency (EPA) final rule titled *Standards for the Disposal of Coal Combustion Residuals in Landfills and Surface Impoundments* in 40 CFR Part 257 Subpart D, published in the Federal Register on April 17, 2015 and pursuant to the Oklahoma Department of Environmental Quality's counterpart rule, OAC 252:517.

The Hugo Plant is located on U.S. Hwy 70, west of Fort Towson, Oklahoma in Choctaw County. Operation of the Hugo Plant began in April 1982. The Hugo Plant has one unit that burns Wyoming coal with a net output of 450 mega-watts (MW). The Hugo Plant generates three types of ash from burning coal – fly ash, economizer ash, and bottom ash. At the Hugo Plant, fly ash is stored in silos or managed in a CCR landfill (labeled CCR Unit 1), economizer ash is managed in CCR Unit 1, and bottom ash is sluiced to a CCR impoundment (Impoundment). Fly ash, economizer ash, and bottom ash are beneficially reused.

The Impoundment is divided into two cells at the Hugo Plant. The north cell is labeled CCR Unit 2 and the south cell is labeled CCR Unit 3. The combined storage capacity of CCR Unit 2 and CCR Unit 3 is 1,064,000 cyds. Bottom ash from the boiler is sluiced to either CCR Unit 2 or CCR Unit 3. There is an estimated 231,000 cyds of bottom ash in the Impoundment and a remaining capacity of 833,000 cyds.

WFEC personnel observe both cells of the Impoundment each day. A formal inspection of both cells of the Impoundment is conducted weekly and results in a written record of inspection. The cells are designed with a three-foot normal pool level freeboard. This normal pool level freeboard is maintained by two 24" diameter constant elevation vertical pipe spillways (one for each cell) that discharge into the Process Waste Pond located on the east side of the Impoundment. Water level below the three-foot normal pool level freeboard is controlled by operating a set of manual valves.

As part of this Assessment, Guernsey performed an inspection of the Impoundment. The inspection included four (4) site visits during which the Hugo Plant's Operating Record, and available data and drawings were collected and reviewed. During each visit, **Guernsey** walked the perimeter of the Impoundment to inspect for signs of distress or malfunction of each cell and appurtenant structures, and to obtain field measurements.

2 SCOPE

The purpose of this Assessment is to meet the requirements outlined in 40 CFR 257.73(a)(2) (Initial Hazard Potential Classification Assessment), 40 CFR 257.73(d) (Initial Structural Stability

Assessment), and 40 CFR 257.73(e) (Initial Safety Factor Assessment). These requirements only apply to existing CCR surface impoundments. See 40 CFR 257.73(a). The CCR Rule requires this Assessment include a certification by a “Qualified Professional Engineer” as defined in 40 CFR 257.53.

3 SITE INSPECTION

Guernsey made four (4) trips to the Hugo Plant in order to review documentation and gather all of the necessary field data and measurements for completion of the requirements of this Assessment. The first trip on November 5, 2015 consisted of data gathering and initial site reconnaissance. The second trip on December 3, 4, and 5, 2015, was for the purpose of obtaining field measurements of the Impoundment. The *Qualified Professional Engineer* made a visit to the Hugo Plant on December 22, 2015. The fourth trip on May 11, 2016, was for the purpose of overseeing the core drilling activities and to field verify area drainage into the Impoundment.

All four (4) site visits included visual inspections of CCR Unit 2 and CCR Unit 3. Any Impoundment integrity issues, vegetation growth, or other potential detrimental activity was noted during the visual inspections.

The field measurements for the Impoundment cells (CCR Unit 2 and CCR Unit 3) included the following:

- Verify the elevation of the vertical pipe spillways
- Verify the relation between the level gauges, 3 ft freeboard, and the overflow (vertical pipe spillways)
- Verify the dimensions of the Impoundment, its cells, and slopes of the dikes
- Determine the bottom topography of the Impoundment cells using bathometric survey techniques
- Obtain core samples from two test borings on the CCR Unit 3 east embankment
- Field verify drainage areas and structures on topo map

During the initial site visit on November 5, 2015, **Guernsey** gathered records of operation, operation manuals, and construction drawings, as well as made a cursory inspection of CCR Unit 2 and CCR Unit 3.

Guernsey staff members visually inspected both CCR Unit 2 and CCR Unit 3 on December 3, 4, and 5, 2015. The inspection included walking around both cells, taking photographs, taking notes, taking GPS positions, taking level measurements to determine water surface elevation, and taking water depth measurements. The visual inspection revealed several items of note including the slope of the embankments (2H:1V - horizontal distance to vertical distance), areas of slope sloughing and erosion rills and evidence of burrowing or rooting animals. CCR Unit 3 had a slope slough on the outside embankment of the east portion of the embankment between CCR Unit 3 and the Process Waste Pond. This slough area was estimated to be 50 ft long by 10 ft wide by 3 feet deep. All other noted slope sloughing issues were minor. There were two to three erosion rills noted on the same embankment further to the north. Other erosion rills were evident on the inside south embankment of CCR Unit 3. There was evidence of feral pigs along the south inside embankment of CCR Unit 3 and the outslope of the north embankment of CCR Unit 2. There was evidence of fire ants mid-way on the inside south embankment of CCR Unit 3. Vegetation was uneven on the north outside embankment of CCR Unit 2. On the inside slope of the north embankment of CCR Unit 2, there was additional evidence of erosion rills and minor sloughs. Vegetation was uneven with some dead vegetation throughout areas of the embankments with riprap. The center embankment between CCR Unit 2 and CCR Unit 3 had a couple of areas of uneven vegetation and a couple of areas of erosion rills.

On December 22, 2015, **Guernsey's** Qualified Professional Engineer conducted a third site inspection and obtained additional field measurements. **Guernsey** inspected the integrity of the hydraulic structures that passed through the cells to the extent possible.

On May 11, 2016, **Guernsey** field verified site drainage from previous reports, existing drawings, and recent area survey. During this time, Terracon drilled three (3) test bores on the CCR Unit 3 east embankment.

4 INITIAL HAZARD POTENTIAL CLASSIFICATION ASSESSMENT

40 CFR 257.73(a)(2) requires a classification of the Impoundment as one of the following: high hazard potential, significant hazard potential, or low hazard potential and documentation for the basis of the classification. The CCR Rule (40 CFR 257.53) defines these classes of hazard as follows:

High Hazard Potential CCR Surface Impoundment means a diked surface impoundment where failure or mis-operation will probably cause loss of human life.

Low Hazard Potential CCR Surface Impoundment means a diked surface impoundment where where failure or mis-operation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the surface impoundment owner's property.

Significant Hazard Potential CCR Surface Impoundment means a diked surface impoundment where failure or mis-operation results in no probable loss of human life, but can cause economic loss, environmental damage, disruption of lifeline facilities, or impact other concerns.

Guernsey has determined that the Impoundment is classified as a **Low Hazard Potential CCR Surface Impoundment** based on the following:

1. Failure or mis-operation of CCR Unit 2 and CCR Unit 3 will **NOT** result in loss of human life. There is no population immediately downstream of CCR Unit 2 and CCR Unit 3.
2. Failure or mis-operation of CCR Unit 2 and CCR Unit 3 will **NOT** create economic loss, or environmental damage, disruption of lifeline facilities, or impact other concerns.
3. Any economic and/or environmental losses due to failure or mis-operation will be principally limited to WFEC's property.

a. The Impoundment is partially incised.
b. A majority of the impounded bottom ash is on the west end of the Impoundment, over 1000 feet from the east embankment (the critical cross section), and is dry, therefore there is a low likelihood that any significant quantity of bottom ash will be released into the environment and cause an economic and/or environmental loss.

5 INITIAL STRUCTURAL STABILITY ASSESSMENT

The CCR Rule, in particular 40 CFR 257.73(d), requires an initial assessment of the structural stability of an existing CCR surface impoundment. The purpose of such assessment is to document whether the design, construction, operation, and maintenance of an existing CCR surface impoundment is consistent with recognized and generally accepted good engineering practices for the maximum volume of CCR and CCR wastewater that can be impounded therein. Such assessment must also identify any structural stability deficiencies and recommend corrective measures. The following Initial Structural Stability Assessment is provided to fulfill the requirements of 40 CFR 257.73(d), and is based on field measurements and verification, review of historical data provided by WFEC, and the Geotechnical Engineering Report prepared by Terracon. See Appendix A for Terracon's *Geotechnical Engineering Report*, *WFEC Hugo Plant Embankment Evaluation with Supplementary Analysis,* dated October 10, 2016..

5.1 Structural Stability Assessment

Structural stability of an embankment is affected by several factors including original construction specifications, construction techniques, type of soil used, slope of the embankment, the ongoing maintenance of the embankment, such as vegetation management, and finally, repair of sloughs, erosion rills, and damage from burrowing animals.

The maintenance of the Impoundment is critical due to the 2H:1V slope. Best engineering practice at the time of construction would have been to construct the eastern embankment of the Impoundment with a 3H:1V slope. Since the Impoundment embankments have a 2H:1V slope, slope protection maintenance becomes very important, including preventing intrusion and water seepage into embankments due to large vegetation roots, such as tree roots, and damaged caused by burrowing animals and erosion rills.

The high plasticity of the clay used for the construction also presents challenges. These clays are susceptible to shrink and swells caused by wet and dry weather cycles. Eventually, these cycles can cause the soil to loose shear strength and embankments become more susceptible to sloughs and erosion rills.

5.1.1 Regulation Citation §257.73(d)(1)(i) through (vii),

(d)(1) The assessment must document whether the design, construction, operation, and maintenance of the CCR unit is consistent with recognized and generally accepted good engineering practices for the maximum volume of CCR and CCR wastewater which can be

impounded therein. The assessment must at a minimum, document whether the CCR unit has been designed, constructed, operated, and maintained with:

(i) ***Stable foundations and abutments***

Based on Terracon's report, **Guernsey's** site inspections, and review of documentation, the foundation and abutments of CCR Unit 2 and CCR Unit 3 are stable. The foundation and abutments are a native fat clay and weathered shale, which form a stable foundation.

(ii) ***Adequate slope protection to protect against: erosion, wave action, and adverse effects of sudden drawdown***

CCR Unit 2 and CCR Unit 3 have been divided into three areas for the purpose of assessment:

- Area 1 - The north and west embankments of CCR Unit 2, and south and west embankments of CCR Unit 3 – all cut slopes
- Area 2 - The center embankment between CCR Unit 2 and CCR Unit 3 – combination of cut slope and compacted fill
- Area 3 -The east embankment of CCR Unit 2 and CCR Unit 3 – mostly compacted fill

5.1.1.1 Erosion

Erosion of an embankment is caused by water carrying away embankment material. Once erosion rills form, more embankment surface is exposed to stormwater runoff and it becomes more susceptible to erosion. The water finds a compromised area of an embankment and starts to cut deeper into the embankment. Other areas of erosion can occur when slope protection is compromised or inadequate. **Guernsey** observed erosion rills on the innerslope and outslope in Area 1 and 2, and on the outslope of Area 3. Observations of the embankments noted vegetation and riprap are not uniform on the embankments. Embankments maintained with proper vegetation and riprap are less susceptible to erosion. **Guernsey** recommends that WFEC identify and repair erosion rills at its earliest opportunity.

5.1.1.2 Wave Action

Wave action is caused as wind pushes water against an embankment and the intensity of the wave action is directly related to the "reach length" or "fetch", i.e. the longest distance the wind blows over a surface of water without interruption. The longest north-south reach length of the Impoundment is approximately 620 ft. The longest west-east reach length is approximately 1,200 ft. During the four (4) site investigations, **Guernsey** did not observe any damage in Area 1, 2 or 3 of the Impoundment due to wave action and concludes that the slope protection is adequate to protect against wave action.

5.1.1.3 Adverse effects of sudden drawdown

The Impoundment is designed, constructed, and operated such that there is no mechanism to cause a sudden drawdown except for a failure of one of the outer embankments of the

Impoundment. A failure of the Area 2 embankment would be of no consequence. **Guernsey** concludes that there is adequate slope protection to protect against the adverse effects of sudden drawdown.

(iii) ***Dikes mechanically compacted to a density sufficient to withstand the range of loading conditions in the CCR unit***

Based upon documentation provided by WFEC, the embankments of CCR Unit 2 and CCR Unit 3 required a specific compaction during construction. No test records of original compaction are available. Terracon states, on page 6 of the Terracon report, the following:

> "Although not shown…, it is likely that the central core of the embankment fill…still possess apparent overconsolidated shear strength…due to the original compaction process and lack of weathering exposure"

Based on visual inspection, the embankment has been compacted and has been in operation for 30 years, through a range of loading conditions, without failure, demonstrating a sufficient density of compaction.

(iv) ***Vegetation maintained (not growing more than 6") per CFR 257.73 or other forms of slope protection are provided***

At the time of inspection(s), the embankments did not have a consistent 6" or less vegetation. There were several small trees growing on the outslope of the south embankment of CCR Unit 3. Some areas of the embankments have small gravel, bottom ash cover, or riprap, which provide a form of slope protection.

(v) ***The combined capacity of all spillways must be designed, constructed, operated, and maintained to adequately manage flow during and following the peak discharge from the event specified in paragraph (d)(1)(v)(B) of §257.73***

(A) ***All spillways must be either***

(1) ***Of non-erodible construction and designed to carry sustained flows***

(2) ***or Earth or grass-lined and designed to carry short-term, infrequent flows at non-erosive velocities where sustained flows are not expected.***

The spillways for CCR Unit 2 and CCR Unit 3 are two 24" diameter constant elevation vertical pipe spillways of non-erodible construction and designed to carry sustained flows.

(B) ***The combined capacity of all spillways must adequately manage flow during and following the peak discharge from a:***

(1) ***PMF for a high hazard potential CCR impoundment; or***

(2) ***1000-yr flood for a significant hazard potential CCR surface impoundment; or***

(3) ***100-yr flood for a low hazard potential CCR surface impoundment***

Both CCR Unit 2 and CCR Unit 3 are cells in a low hazard potential CCR surface impoundment; therefore, the analysis is based on a 100-yr flood.

The approach to answer this question was to evaluate two scenarios. The worst-case (most conservative) scenario assumes that the most critical impoundment during a 100-year flood event is CCR Unit 2 because it has the largest drainage area. In order to determine the ability of CCR Unit 2 to manage flow from a 100-year flood, the following assumptions were used in the methodology:

1. A 100-yr storm with a rainfall of 9.6 inches in a 24 hour period and Type II distribution.
2. CCR Unit 2 receives all of the storm water flow from the coal pile runoff pond during the event.
3. CCR Unit 2 receives all of the process wastewater during the event.
4. The 24" valve at the discharge structure is closed.
5. The east embankment of CCR Unit 2 is the critical cross section.

Guernsey field verified storm drainage areas, and used topography data provided by Burns and McDonnell to calculate the storm water drainage basin.

The analysis used results from the HEC-HMS model. The majority of the storm water comes from the coal pile run-off. The run-off from the coal pile flows into the coal pile runoff pond. The level of the coal pile runoff pond is regulated by a horizontal, flat weir which overflows into a horizontal 24" corrugated metal pipe (CMP). This water gravity flows to the Impoundment inlet structure. The peak flow from the coal pile runoff pond during a 100 year storm is 24.1 cfs.

Flows from other sources enter this structure and account for 1.86 cfs of water. The sources including boiler blowdown, cooling tower blowdown, plant drains, and stormwater flow from the flyash landfill. Flow from this structure into CCR Unit 2 is regulated by a 24" sluice gate. As stated before, flow from CCR Unit 2 is regulated by the 24" diameter constant elevation vertical pipe spillway that discharges into the Process Waste Pond.

The maximum storm event does not exceed top of embankment. Without operational intervention, the level of CCR Unit 2 will rise to a level within twelve inches of the top of the embankment, elevation 445'. It will take 84 hours for the level to drop to a normal operating level of 443'. It is highly encouraged that the Hugo Plant institute (or continue) measures to minimize the amount of time that the Impoundment operates above the normal operating level.

A second scenario assumes both that CCR Unit 2 and CCR Unit 3 receive an equal distribution of the 100-yr storm water and the process water. Under this scenario, the level of both CCR Unit 2 and CCR Unit 3 will rise to a level within 1.8 feet of the top of the embankment, elevation 444.8'. Due to the length of time required to drop to the normal operating level, measures to increase the discharge should be taken after a major storm event.

Specifically, during a 100-year flood event, **Guernsey** recommends WFEC take measures to lower the water level in the Impoundment as quickly as possible to a normal operating level of 443'. These measures include opening of the 24" drain valves and reducing the process flow into the Impoundment such as the stormwater flow from CCR Unit 1 and the cooling tower blowdown. These actions should be taken to prevent overtopping of the embankments in case of successive flooding events.

For the reasons set forth above, the combined capacity of all spillways of the Impoundment adequately manages flow during and following peak discharges from a 100-yr flood.

(vi) ***Hydraulic structures underlying the base of the CCR unit or passing through the dike of the CCR unit that maintain structural integrity and are free of significant deterioration, deformation, distortion, bedding deficiencies, sedimentation, and debris which may negatively affect the operation of the hydraulic structure***

Concrete structures at each end of the 36” pipe passing through the bottom ash recycle structure are in good condition. The pipe itself was not visible due to complete submergence on both ends. However, this pipe is a recent HDPE replacement of the original corrugated metal pipe. HDPE pipe is less susceptible to corrosion versus corrugated pipe; therefore, it will not deteriorate as quickly and will not create a void for water to infiltrate and compromise the interior of the embankment.

(vii) ***For CCR units with downstream slopes which can be inundated by the pool of an adjacent water body, such as a river, stream or lake, downstream slopes that maintain structural stability during low pool of the adjacent water body or sudden drawdown of the adjacent water body***

CCR Unit 2 and CCR Unit 3 downstream slopes do not abut a river, stream, or a lake.

Subject to the limitation discussed herein (2H:1V slope of the eastern embankment of the Impoundment), the design, construction, operation, and maintenance of the Impoundment is consistent with recognized and generally accepted good engineering practices for the maximum volume of CCR and CCR wastewater that can be impounded therein.

5.1.2 Regulation Citation §257.73(d)(2)

(d)(2) The periodic assessment described in paragraph (d)(1) of this section must identify any structural stability deficiencies associated with the CCR unit in addition to recommending corrective measures.

The stability of an embankment is assessed in terms of safety factor. The safety factor is the ratio of the stabilizing forces to the destabilizing forces under various specified loads. The CCR regulations require the stability of the critical cross section to be rated at a minimum safety factor of 1.5 under normal loading conditions.

The embankments of the Impoundment can be grouped together as follows based on construction methods:

1. The embankments of CCR Unit 2 and CCR Unit 3, Area 1, were formed by excavation rather than fill. The soil is less likely to lose shear strength and therefore less likely to cause failure. There is no visual or documented evidence that the 2H:1V slope is posing structural or stability issues in this embankment area.
2. The center embankment between CCR Unit 2 and CCR Unit 3, Area 2, was formed by excavation and fill. The soil is less likely to lose shear strength and therefore less likely to cause failure. Additionally, failure of the center embankment is inconsequential and does not affect the environment or operation of the Impoundment. There is no visual or documented evidence that the 2H:1V slope is posing structural or stability issues in this embankment area.
3. The outslope of the Impoundment's east embankment is mostly constructed from fill and has experienced sloughing. The existing area of sloughing is an indication of a stability deficiency. The slough mechanism of the east embankment outslope is top down failure.

Terracon analyzed two east embankment core samples for the geotechnical parameters required to determine the safety factor of the east embankment for the current configuration and several additional configurations. The resulting stability (safety factor) of the current configuration and two additional configuration's resulting stability (safety factor) are listed below:

- The existing condition of the Impoundment east embankment with a slope of 2H:1V has a safety factor of 1.0
- Rebuilding the top eight feet of the Impoundment east embankment outslope with a geogrid material, maintaining the 2H:1V slope, results in a safety factor of 1.5 (Terracon Report Alternative 2)
- Modifying the outslope of the Impoundment east embankment to 3H:1V slope results in a safety factor of 1.5 (Terracon Alternative 3)

The options to improve the stability of the Impoundment east embankment to a safety factor of 1.5 are discussed in more detail in *Terracon's Geotechnical Engineering Report*, dated October 10, 2016, found in Appendix A of this Assessment. These two solutions evaluated by Terracon are only two of the many possible solutions. WFEC should consider other options and obtain design documents, construction specifications, construction documents, and use a qualified inspector, before attempting any modifications to the Impoundment east embankment.

Improved vegetation management, both removal of dead vegetation, removal of shrubs, trees and the addition of riprap, gravel, bottom ash, or grass, will improve the stability of all CCR embankments.

6 INITIAL SAFETY FACTOR ASSESSMENT

The CCR Rule, in particular 40 CFR 257.73 (e)(i) through (iv), addresses the requirement for the owner or operator of a CCR surface impoundment to perform a safety factor assessment. The purpose of such assessment is to document whether the calculated factors of safety achieve minimum safety factors for the critical cross section.

6.1 Critical Cross Section and Geotechnical evaluation

Based on field observation and review of construction documents, **Guernsey** determined the east embankment of CCR Unit 2 and CCR Unit 3 is the **Critical Cross Section** for purposes of this Assessment. The Critical Cross Section is the section of the impoundment that is most susceptible to structural failure based upon appropriate engineering considerations, including loading conditions. Unlike the other embankments of the Impoundment, the majority of the east embankment of CCR Unit 2 and CCR Unit 3 is constructed using fill and has experienced the largest slough area.

Guernsey subcontracted with Terracon to perform a geotechnical assessment of the east embankment. The scope included:

- Coring and obtaining samples of materials in the east embankment of CCR Unit 2 and CCRUnit 3
- Evaluating the stability of the east embankment of CCR Unit 2 and CCR Unit 3
- Providing geotechnical recommendations to achieve safety factors cited in the CCR Rule based on the following:
  - The water level at normal pool elevation (443')
  - The water level at maximum storage pool loading (445')
  - Seismic activity safety factor
- Evaluating the susceptibility of the materials used in the east embankment of CCR Unit 2 and CCR Unit 3 to liquefaction

Terracon completed two cores of the east embankment of CCR Unit 3 and an additional 10 ft Shelby tube sample. Since the east embankment for both CCR Unit 2 and CCR Unit 3 is essentially a single embankment (i.e., it was constructed at the same time in a similar fashion with similar material), it is **Guernsey**'s opinion that the core samples taken on the east embankment of CCR Unit 3 are representative of the entire east embankment of the Impoundment.

6.2 Safety Factor Evaluation

Terracon calculated the safety factors using the software Slope/W and the Morgenstern-Price method of analysis. Terracon used the data listed below in Table 1 and Table 2 as inputs to the Slope/W software.
Table 1 (page 4 of the Terracon *Geotechnical Engineering Report*)

ITEM	BOTTOM ASH IMPOUNDMENT EXISTING CONFIGURATION
Embankment crest width (ft)	25
Elevation of embankment crest	446
Elevation of upstream toe (ft)	426.5
Elevation of downstream toe (ft)	422.5
Inner slope	2H:1V
Outerslope	2H:1V
Water depth below existing crest surface (ft)	15

Table 2 (page 5 of the Terracon *Geotechnical Engineering Report*)

Material	Total Unit Weight (pcf)	Effective Stress Parameters	
		C' (psf)	Φ' (degrees)
Embankment Fill	122	40	19.5
Native Fat Clay	122	50	20
Weathered Shale	136	0	30

(e)(1) The owner operator must conduct an initial and periodic safety factor assessment for each CCR unit and document whether the calculated safety factors of safety for each CCR unit achieve the minimum safety factors specified in paragraphs (e)(1)(i) through (iv) of this section for the critical cross section.…The safety factor assessments must be supported by appropriate engineering calculations.

(i) ***The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.50***

The water elevation of the Impoundment under long term, maximum storage pool loading condition is 443'. The critical cross section has a safety factor of 1.0 under the long-term, maximum storage pool loading condition. This safety factor is below the safety factor required by the CCR Rule of 1.5 for this loading. Any additional loading of the Impoundment above the long-term, maximum storage pool loading, with the existing east embankment configuration, will render a safety factor below 1.0.

Terracon identified two alternatives that would increase the safety factor of the east embankment to 1.5 under maximum pool loading conditions. These alternatives include reconstructing the top eight feet of the embankment using geogrid reinforcement and riprap (Terracon Alternative 2) or changing the configuration of the outslope of embankment from 2H:1V to 3H:1V (Terracon Alternative 3).. Other configuration options are available to increase the safety factor to 1.5 during long term, maximum storage pool loading conditions but were not evaluated by Terracon.

(ii) ***The calculated static factor of safety for maximum surcharge pool loading condition must equal or exceed 1.40***

Guernsey determined that the elevation of the water within the Impoundment, during maximum surcharge pool loading, is 445’ (see Section 5.1). Terracon used this elevation to determine the safety factor of the critical cross section during maximum surcharge pool loading.

Under these conditions, the current configuration of the critical cross section, 2H:1V, will result in a safety factor below 1.0, which does not meet the safety factor required by the CCR Rule of 1.4 for this loading.

If the outslope of the critical cross section is reinforced with a geogrid (Terracon Alternative 2), the safety factor for the embankment would be 1.4 which meets the safety factor required by the CCR Rule of 1.4.

If the outslope of the critical cross section had a slope of 3H:1V (Terracon Alternative 3), the safety factor of the embankment would be 1.38, which does not meet the safety factor required by the CCR Rule of 1.4.

(iii) ***The calculated seismic factor of safety must equal or exceed 1.00***

The safety factor for the existing slope is 1.0, therefore, the seismic safety factor will be less than 1.0 which does not meet the safety factor required by the CCR Rule of 1.0.

If the outslope of the critical cross section is reinforced with a geogrid (Terracon Alternative 2), the seismic factor of safety factor would be 1.1 which meets the safety factor required by the CCR Rule of 1.1.

If the outslope of the critical cross section had a slope of 3H:1V (Terracon Alternative 3), the calculated seismic factor of safety would be 1.03, which meets the safety factor required by the CCR Rule of 1.0

(iv) ***For dikes constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20***

It is Terracon’s opinion that the soil used for the embankment, predominantly high plasticity clays, is not susceptible to liquefaction; therefore, the requirement of this regulation is not applicable.

7 CONCLUSION

The embankments of CCR Unit 2 and CCR Unit 3 at the Hugo Plant are in stable condition with the exception of the outslope of the east embankment, which has had noted areas of sloughing.

This embankment was determined to be the critical cross section for purposes of the Initial Safety Factor Assessment.

Based on all available data, **Guernsey** determined that the Initial Hazard Potential Assessment of the bottom ash impoundment is a “Low Hazard Potential”. This determination is based on the fact that mis-operation or failure of the Impoundment will not result in a loss of life, or have an economic or environmental impact.

Guernsey determined, based on a worst-case scenario (only CCR Unit 2 in operation) that during a 100-year flood event, the water level in CCR Unit 2 will rise to the elevation of 445’, and will not overtop the embankment. In the event of a 100-year flood, WFEC must take measures to lower the water level in the Impoundment, as quickly as possible, to a normal operating level of 443’. These measures include opening of the 24” drain valves and reducing the process flow into the Impoundment such as the stormwater flow from the flyash landfill and the cooling tower blowdown. These measures should be taken to guard against overtopping the embankment if successive 100-year events occur.

The safety factor of the east embankment, a direct measure of stability, is 1.0. The stability, i.e. safety factor, of the embankment can be improved by implementing one of many solutions. Two solutions recommending by Terracon and presented in this report include:

1. Reduce the embankment slope angle from 2V:1H to 3V:1H. This modification improves the safety factor to 1.5.
2. Rebuild the 2H:1V slope using a geogrid material. By rebuilding the top eight feet of the embankment using a geogrid material with a long-term design strength (LTDS) of 3,000 per lineal foot, the safety factor of the embankment would increase to 1.5.

Other viable solutions certainly exist as well. Improved vegetation management, both removal of dead vegetation, removal of shrubs, trees and addition of riprap, gravel, bottom ash, or grass, will improve the stability of all CCR embankments. **Guernsey** recommends that WFEC identify and repair erosion rills at its earliest opportunity.

Subject to the limitation set forth below, **Guernsey** finds that at the time of this Assessment of the Hugo Plant, CCR Unit 2 and CCR Unit 3 are designed, constructed, operated, and maintained consistent with recognized and generally accepted good engineering standards. Limitation: best engineering practice at the time of construction of the eastern embankment of CCR Unit 2 and CCR Unit 3 would have been construction with a 3H:1V slope. Design and construction of modifications to this eastern embankment to increase the safety factors using the solutions described herein (or other potential solutions) would result in the Impoundment being designed, constructed, operated, and maintained consistent with recognized and generally accepted good engineering standards.

APPENDIX A
TERRACON GEOTHECHNICAL ENGINEERING REPORT

Geotechnical Engineering Report

WFEC Hugo Power Plant Embankment Evaluation
with Supplementary Analysis
Hugo Power Plant
Approximately 11.0 Miles East of Hugo
Hugo, Oklahoma

October 10, 2016
Terracon Project No. 03165346

Prepared for:
Guernsey
Oklahoma City, Oklahoma

Prepared by:
Terracon Consultants, Inc.
Oklahoma City, Oklahoma

October 10, 2016

Terracon

Guernsey
5555 North Grand Boulevard
Oklahoma City, Oklahoma 73112

Attn: Ms. Summer Goebel, P.E.
P: [405] 416 8117
F: [405] 743 9676
E: summer.goebel@guernsey.us

Re: Geotechnical Engineering Report
WFEC Hugo Power Plant Embankment Evaluation
with Supplementary Analysis
Hugo Power Plant
Approximately 11.0 Miles East of Hugo
Hugo, Oklahoma
Terracon Project No. 03165346

Dear Ms. Goebel:

Terracon Consultants, Inc. has completed additional embankment analysis, as requested, the subsurface exploration and geotechnical engineering evaluation for the detention pond embankment. This report presents our geotechnical site characterization, geotechnical observations, analysis, and modification recommendations for the embankment.

We appreciate the opportunity to work with you on this project and are prepared to provide the recommended construction testing services. If you have any questions regarding this report, or if we may be of further service to you, please contact us.

Sincerely,
Terracon Consultants, Inc.
Cert. Of Auth. #CA-4531 exp. 6/30/17

Jeff Dean

Jeff Dean, P.E.
Senior Engineer

LICENSED PROFESSIONAL ENGINEER
GERALD W. FINN
12463
OKLAHOMA

Gerald W. Finn

10-10-2016

Gerald W. Finn, P.E.
Oklahoma No. 12463

JD:GWF\srs\n:\projects\2016\03165139\project documents\oct2016

Copies to: Addressee (1 via email)

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WITH SUPPLEMENTARY ANALYSIS
HUGO POWER PLANT
APPROXIMATELY 11.0 MILE EAST OF HUGO
HUGO, OKLAHOMA

TERRACON Project No. 03165139
October 10, 2016

1.0 INTRODUCTION

The subsurface exploration has been completed for the bottom ash storage pond embankment at the WFEC Hugo Power Plant. Three (3) borings were drilled along the southeast embankment of the storage pond. The presence of underground utilities prevented drilling the two (2) planned borings on the northeast section of the storage pond embankment. Borings 1 and 2 were drilled to depths of 37 and 35 feet respectively below the surface of the existing embankment. A visual inspection of the embankment was also performed. This report describes the overall project, the exploration and laboratory testing programs and the subsurface conditions encountered at the boring locations. The report provides our opinions on performance deficiencies and includes the recommendations for correction based on our geotechnical observations that were in the original Terracon report, 03165139, dated July 7, 2016 as well as the recommendations based upon the supplementary analysis requested by email from Guernsey dated September 29, 2016. Recommendations regarding site preparation, earthwork, and placement and compaction of embankment fill are also provided in the report.

2.0 PROJECT INFORMATION

2.1 Project Description

ITEM	DESCRIPTION
Site layout	See Appendix A, Exhibit A-1 Boring Location Plan
Structures	The existing embankment of the bottom ash storage pond has an estimated height of 22.5 feet and extends a length of approximately 800 feet along the southeast edge of the bottom ash detention ponds.

ITEM	DESCRIPTION
Existing conditions	■ A small landslide approximately 30 to 40 feet long has appeared on the downstream portion of the southeast detention pond embankment. The embankment has dropped approximately 4 to 5 feet along the edge of the road atop the crest. ■ There are several small scarps or slides along the slopes of the detention ponds. ■ A portion of the embankment is covered with vegetation consisting of small trees, shrubs, grasses and weeds. ■ The rip rap on the embankment is not uniform and does not provide adequate protection of the slopes.

2.2 Site Location and Description

ITEM	DESCRIPTION
Location	The WFEC Power Plant is located approximately 11.0 miles east of Hugo, Oklahoma.
Current Ground Cover	Grass, small shrubs, and bare soil.

3.0 SUBSURFACE CONDITIONS

3.1 Typical Profile

Based on the results of the borings drilled through the embankment, subsurface conditions encountered throughout the embankment can be generalized as follows:

TYPICAL SUBSURFACE PROFILE

Description	Approximate Depth to Bottom of Stratum (feet)	Material Encountered	Comments
Stratum 1A	13.5 to 21	Fat Clay	Medium stiff to stiff
Stratum 2	32.5 to 33.5	Shaley Fat Clay	Very stiff to hard
Stratum 3	Below the boring termination depths	Weathered Shale[2]	Hard

Conditions encountered at each boring location are indicated on the individual boring logs presented in Appendix A. Stratification boundaries on the boring logs represent the approximate location of changes in soil and rock types; in-situ, the transition between materials may be gradual.

3.2 Groundwater

The boreholes through the embankment were observed for water while auger drilling and immediately after completion of auger drilling. Water level observations are summarized in the table below.

BORING NO.	DEPTH TO GROUNDWATER WHILE DRILLING, FT.	DEPTH TO GROUNDWATER AFTER DRILLING, FT.
B-1	Dry	Dry
B-2	13.2	Dry
B-2A	Dry	Dry

Groundwater level fluctuations occur primarily due to the seasonal level of water in the detention pond, but other factors not evident at the time the borings were performed could also affect the water level. Therefore, groundwater levels at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the rehabilitation plans for the project.

3.3 Geology

The site geology consists of the Fredricksburg Unit (Kf) and the Washita Unit (Kw). These units consist of shaley clays with varying amounts of limestone and sandstone. The clay shales are mostly gray to black while the limestones are highly fossiliferous, gray to yellowish, usually with interbedded clay beds. The topography is generally gently rolling hills with tree covered limestone scarps.

4.0 GEOTECHNICAL SITE CHARACTERIZATION

4.1 Embankment Centerline

Three borings, B-1, B-2 and B-2A, were drilled across the top of the embankment. Detailed locations are shown on Exhibit A1 in Appendix A. The embankment materials consisted of fat clays with varying amounts of shale fragments. The embankment fill was underlain by native fat clays and shaley fat clays overlying weathered shale.

5.0 STABILITY ANALYSES OF THE EMBANKMENT SLOPES

5.1 Embankment Geometries

The following configuration data were used in the analyses of the existing embankment. The configurations were based on the plans and cross sections, provided by Guernsey, of the existing detention pond embankment. If the configuration changes from those outlined below, we will need to be contacted to check the stability with the new geometry.

ITEM	BOTTOM ASH STORAGE POND
	Existing
Embankment crest width (ft.)	25
Elevation of embankment crest (ft.)	446
Elevation of upstream toe (ft.)	428.5
Elevation of downstream toe (ft.)	422.5
Upstream slope	2H:1V
Downstream slope	2H:1V
Water depth below existing crest surface (ft.)	15

5.2 Subsurface Profile

The typical profile, used in the analyses, consisted of the embankment soils underlain by the native overburden and bedrock. The native soils are composed of fat clays with varying amounts of shale fragments. Bedrock is composed of weathered shale. Depth to bedrock varies from 32.5 feet to 33.5 feet.

5.3 Soil Properties and Shear Strength Parameters

The soil shear strength parameters and properties, necessary for limit equilibrium analysis using the SLOPE/W program, are presented in Table 1. The strength parameters in Table 1 are based on the laboratory test results, field exploration data, established correlations of shear strength and index tests and our experience.

Foundation Material - Weathered shale

The bedrock strata encountered is generally comprised of weathered shale. The estimated values of the strength parameters in Table 1 are based on our experience with the local weathered shale properties.

<u>Embankment Materials</u>

Based on the subsurface exploration and laboratory tests performed, the on-site soils consist primarily of fat clays with varying amounts of sand. Select soil samples from borings B-1 and B-2 were tested for dispersivity or the potential for piping. There are three primary tests used to identify dispersive soils: the Pinhole Test, the SCS Double Hydrometer Test, and the Emerson Crumb test. Our investigation included the SCS Double Hydrometer test and the Emerson Crumb test. Laboratory tests conducted on soil samples obtained from Borings 1 and 2 indicate a slight potential for the soils to be dispersive. No single test is definitive but a combination of tests provide a more reliable indication of a soil's dispersion characteristics. We recommend all borrow soils, used for the embankment modification, be tested for dispersive characteristics.

The parameters in Table 1 below are based on limited laboratory tests on the embankment materials, the index properties of the on-site soils, and our experience with similar materials.

TABLE 1 - SHEAR STRENGTH PARAMETERS FOR SOIL AND BEDROCK STRATA

Material	Total Unit Weight (pcf)	Effective Stress Parameters	
		C' (psf)	f' (degrees)
Embankment Fill	122	40	19.5
Native Fat Clay	122	50	20
Weathered Shale	136	0	30

Note: Modification of the existing fill materials is not required beyond removing the existing vegetation and soft soils.

5.4 Static Analysis

Embankment slopes constructed of high plasticity clays, such as this embankment, although stable when originally constructed, experience shrink and swell movements during alternating wet / dry weather cycles. This shrink / swell activity tends to form cracks along the slope face which allow more water into the near surface zone along the slope causing it to swell and soften to a greater depth. This swelling/softening activity forms a weaker zone of soil along the surface of the embankment. The soils in this zone of the embankment become progressively more normally consolidated and can approach what is termed a "fully softened" condition. Creep and ongoing weathering can further reduce the shear strength of the soil in this zone, causing progressive strain-softening, leading to a residual strength condition in portions of the slope. Slopes of this nature can begin to fail progressively, either in a bottom-up, or top-down fashion, leading to full mobilization of the entire slope. In the case of this embankment, it appears that the failure is a top-down progression. Any tension crack that formed at the crest of the slope likely served to introduce additional water seepage into the shear surface causing further shear strength reduction and downslope progression. During the site visit to the facility, discussions with the Plant Engineer indicated the slide occurred after a series of heavy rain events in the area. Given the long dry periods that have been occurring in recent years, it is probable that shrinkage cracks developed in the embankment that allowed water to infiltrate into the embankment slope thus accelerating the shrink swell cycle.

Estimated ranges of fully softened and residual shear strength parameters for use in our analyses were developed for the embankment fill and native soils based on the index test results using the correlations developed by Dr. Timothy Stark of the University of Illinois. Using the furnished cross sections, the as-built/failed condition was modeled using the slope stability software, Slope/W and the Morgenstern-Price method of analysis. Extensive failure surface searches were performed in order to back-calculate the likely existing failure surface location.

The parameters for each material and zone were estimated based the soil boring data, the estimated shear strength parameter ranges discussed previously, previous experience with soil slopes in this region, and observations at the site.

For the back-calculation analysis of the failed slope, a target safety factor of 1 was used to analyze the existing slope. Soil parameters for each material and zone were varied within the anticipated ranges of fully softened and/or residual strength conditions until the factor of safety for the analyzed model reached approximately 1. The results of this back-calculation analysis are shown in Figure 1. The results of this analysis indicate a residual strength condition along the failure surface and normally consolidated (fully softened) conditions in the underlying native foundation soils. Although not shown in Figure 1, it is likely that the central core of the embankment fill (away from the likely failure surface zone) still possesses apparent overconsolidated shear strength (higher cohesion and friction) due to the original compaction process and lack of weathering exposure.

The back-calculation results displayed in Figure 1 provide our best estimate of the most likely location of the failure surface within the embankment based upon the embankment soil parameters used and the slope geometry. The back-calculated failure surface location is approximately 6 to 8 feet vertically below the existing embankment slope surface (about 8 to 10 feet normal to the slope surface). The actual location (depth and downslope extent) of the failure surface likely varies from the locus shown in Figure 1 due to material property, weathering and other variables not accounted for in the analysis. A test trench into the embankment would be required to accurately identify the depth, extent and actual location of the failure surface.

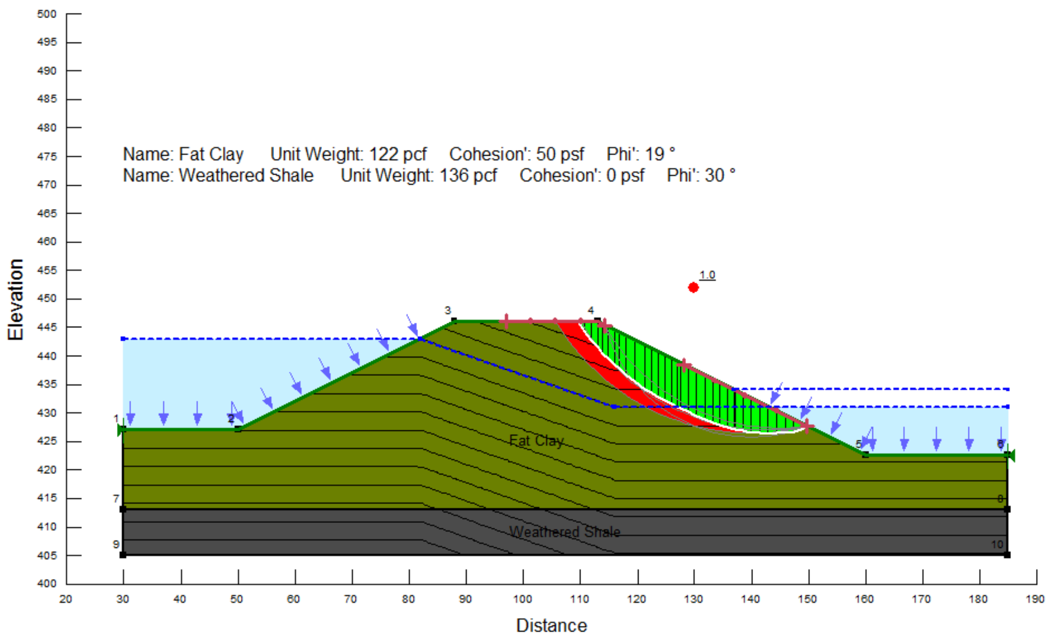


FIGURE 1: EXISTING 2H:1V SLOPE

5.5 Repair Alternatives

Three alternative slope repair options have been analyzed under various pool elevation conditions at the request of Guernsey. Alternative 1 consisted of excavating the materials in the failure zone, benching into the unfailed, existing embankment fill and reconstructing the embankment slope to match the original 2H:1V slope. A layer of riprap is added to flatten the slope to approximately 2.5H:1V and provide additional protection to the slope and add passive resistance against sliding. Alternative 2 consisted of more extensive excavation into the existing slope to allow placement of geotextile reinforcement layers, then reconstructing the slope to the original 2H:1V slope angle using the excavated slope material. A layer of riprap provides additional protection to the slope and passive resistance against sliding. Alternative 3 consisted of reconstructing the existing embankment slope to 3H:1V.

The new 2.5H:1V slope configuration of Alternative 1 is shown in Figure 2. This new slope requires a series of benches two feet in height by at least 15 feet in width to be cut into the existing slope to allow placement of new fill similar to the existing mixture of fat clays in the existing slope. The actual width of the benches should be adjusted to extend at least 5 feet beyond the actual extent of the failure surface to ensure that all failed-sheared-softened materials are removed and that materials impacted by the failure are not left in place which would leave a weak zone in the new embankment slope. This will require test trenches prior to construction to define the actual extent of the failure surface. Fill should be placed in lift thicknesses of no more than 8 inches, within 2 percent of its optimum moisture content, and be compacted to at least 95 percent of the material's maximum dry density as determined by test method AASHTO T-99 (standard Proctor).

The outer layer of riprap should extend laterally 12 feet from the toe and 3 feet from the crest of the embankment. The riprap should be hard, sound and durable, crushed stone meeting ODOT Specifications 713.02. The stone should be well graded with an average stone size (D50) of 24 inches, placed and compacted to maximize density and aggregate interlock between stones.

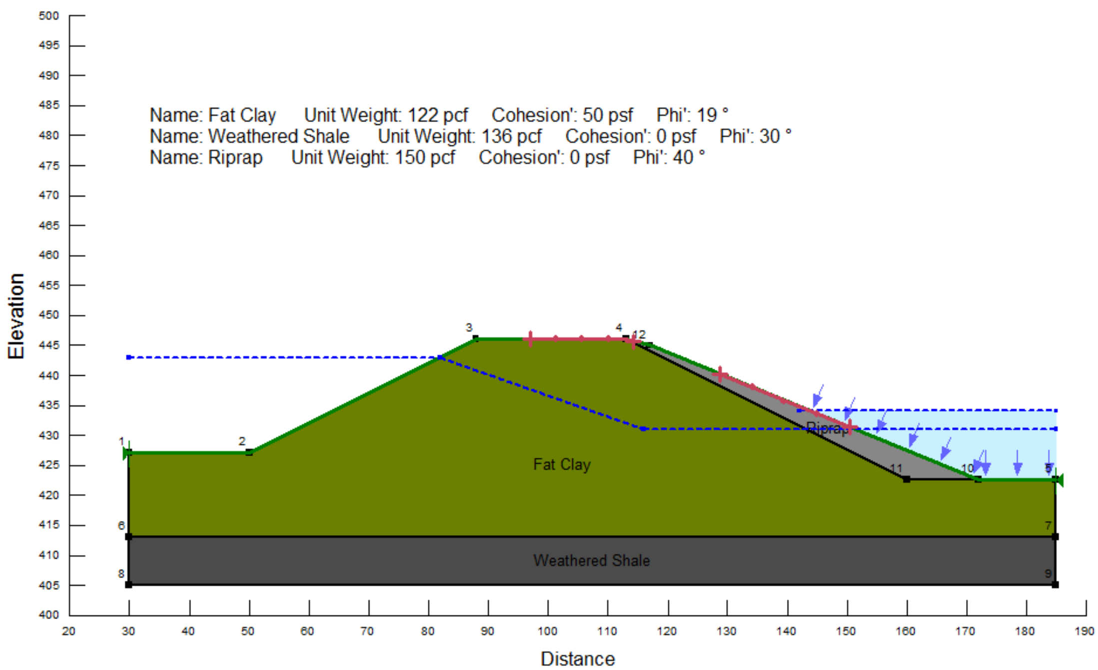


FIGURE 2: ALTERNATIVE 1 - 2H:1V SLOPE - WITH RIP RAP

Analysis of this new reconstructed slope indicated a safety factor of around 1.3 as shown in Figure 3.

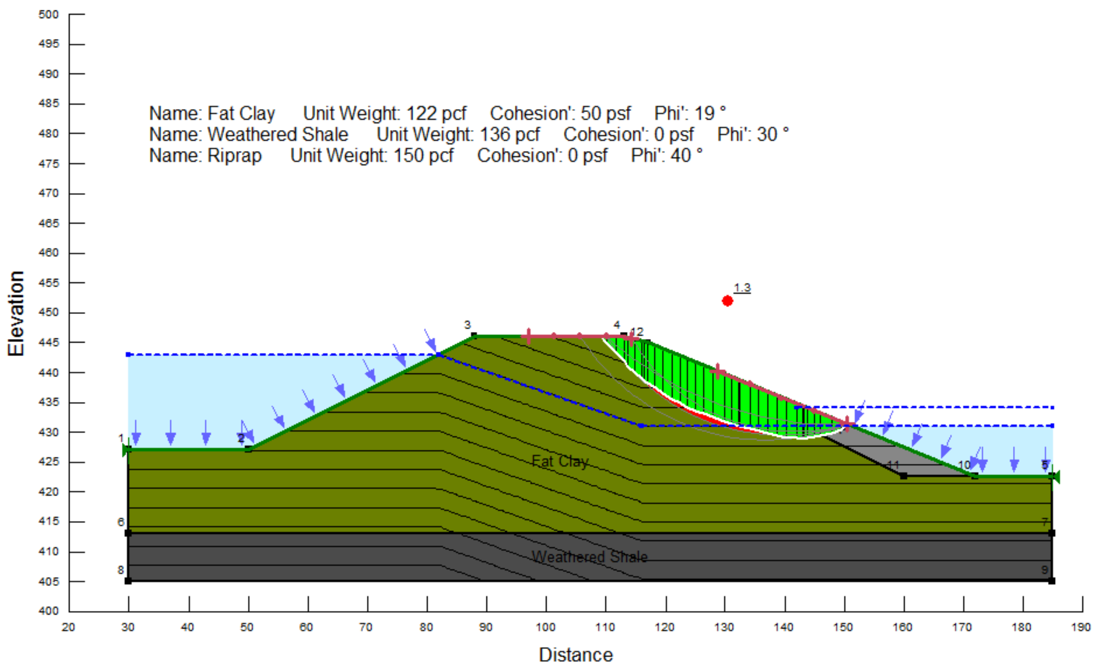


FIGURE 3: ALTERNATIVE 1 - FACTOR OF SAFETY IS 1.3

Alternative 2 consists of reinforcing the existing 2H:1V slope using a geogrid with a minimum long-term design strength (LTDS) of 3,000 pounds per lineal foot, similar to Miragrid 7XT. The top 8 feet of the embankment would be reconstructed and the geogrid reinforcement layers would be placed every two feet vertically up the slope. as shown in Figure 4. Four layers of reinforcement would be required to provide a factor of safety of approximately 1.5 as shown in Figure 5. Each layer of reinforcement would be extended the width of the embankment. Fill should be placed in lift thicknesses of no more than 8 inches, within 2 percent of its optimum moisture content, and be compacted to at least 95 percent of the material's maximum dry density as determined by test method AASHTO T-99 (standard Proctor).

The layer of riprap should extend laterally 7 feet from the toe and 3 feet from the crest of the embankment. The riprap should be hard, sound and durable, crushed stone meeting ODOT Specifications 713.02. The stone should be well graded with an average stone size (D50) of 24 inches, placed and compacted to maximize density and aggregate interlock between stones.

As requested, alternative 2 was also analyzed to reflect a maximum pool elevation of 445 feet and also to reflect possible seismic activity for the project site. Analysis of the increased pool elevation of 445 feet produced a factor of safety of 1.5 as shown in Figure 6. According to the soil properties obtained from the boring logs, the project site is classified as Site Class D. A review of the USGS seismic maps provided horizontal and vertical pseudo static acceleration coefficients of 0.15 and 0.1 respectively for analysis of the embankment under seismic loads using SlopeW. Based upon further literature review, these coefficients provide results that are conservative for this region with regards to the analysis of seismic induced loads on the embankment slope. Using this information, a safety factor of 1.1 was achieved as shown in Figure 7.

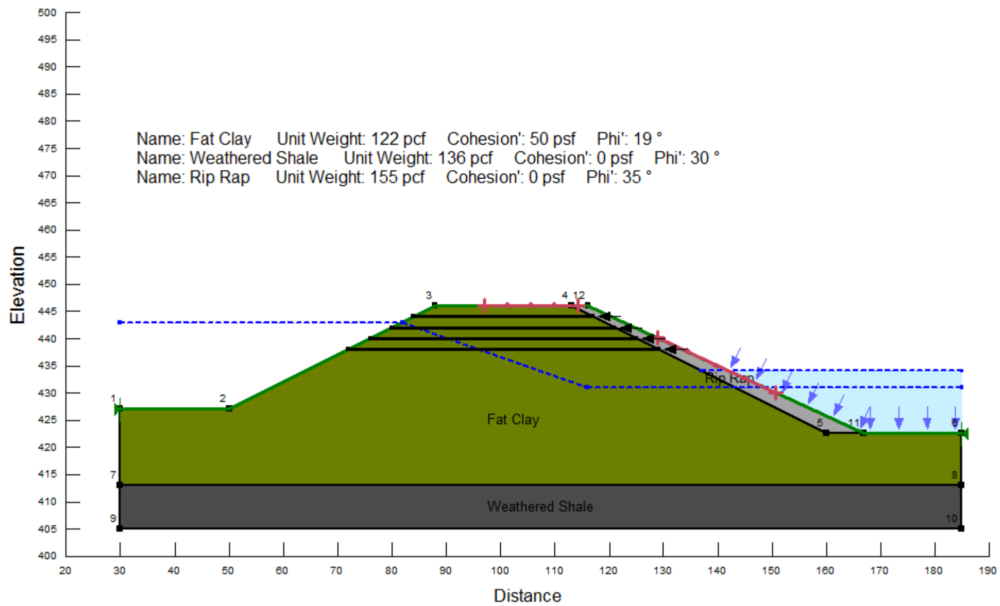


FIGURE 4: ALTERNATIVE 2 - 2H:1V REINFORCED SLOPE – WITH RIPRAP

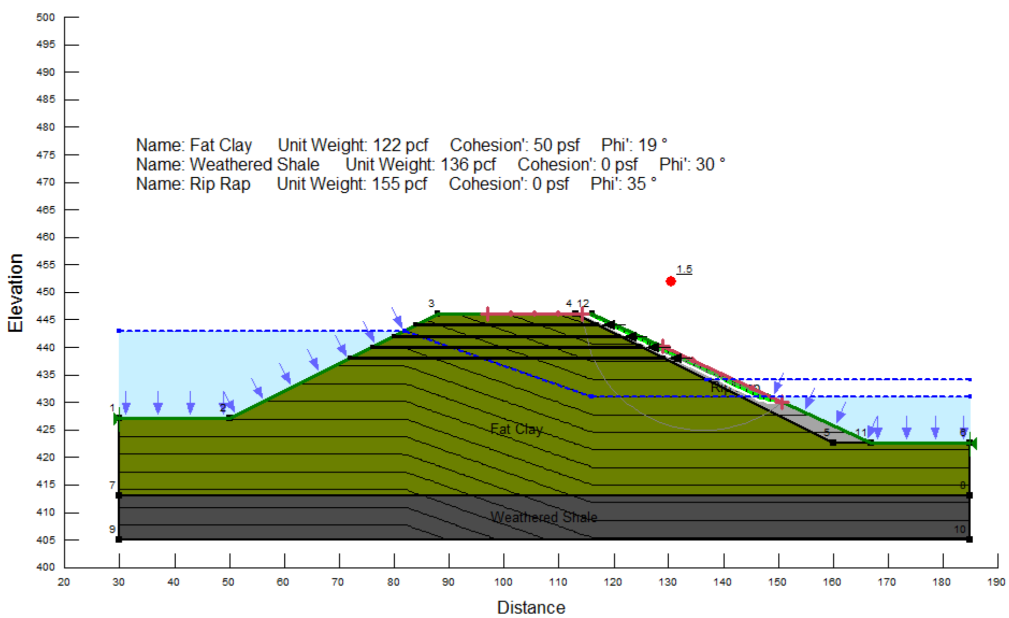


FIGURE 5: ALTERNATIVE 2 - FACTOR OF SAFETY IS 1.5

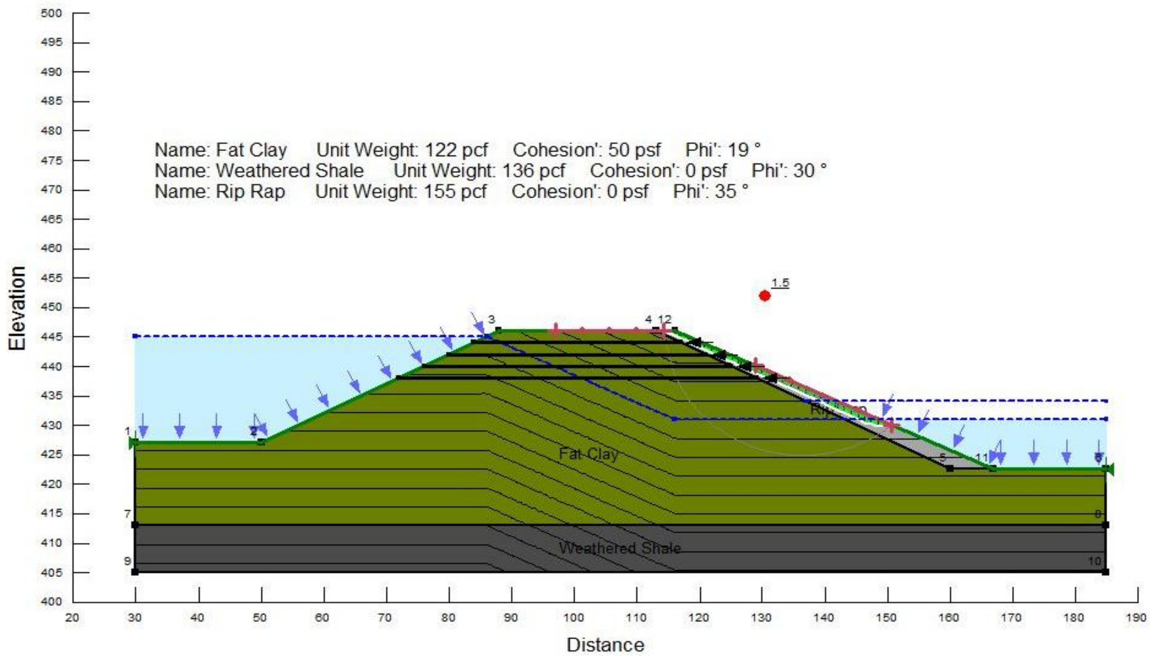


FIGURE 6: ALTERNATIVE 2 AT POOL 445 FT. - FACTOR OF SAFETY IS 1.5

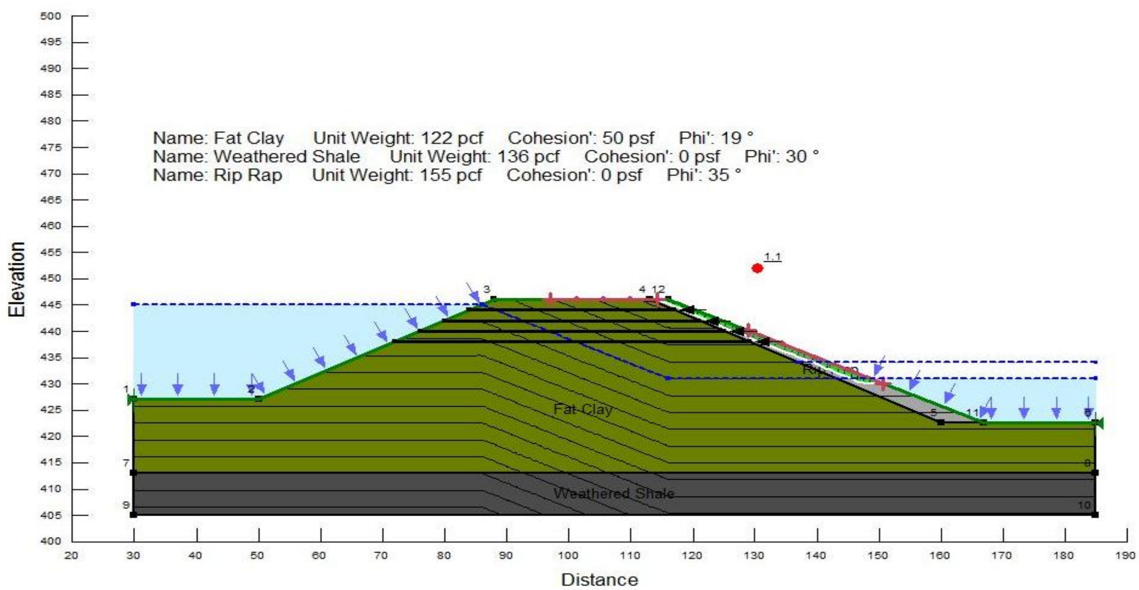


FIGURE 7: ALTERNATIVE 2 AT POOL 445 FT. WITH SEISMIC LOADS - FACTOR OF SAFETY IS 1.1

A third alternative was requested to achieve an unreinforced slope with a safety factor of 1.5. This analysis produced a slope of 3H:1V as shown in Figure 8. This alternative was also analyzed under three different situations:

- a maximum pool elevation at the top of the embankment
- a pool elevation of 445
- under seismic loads

If the water surface elevation was able to reach the top of the embankment, this would place the water elevation at 446.0 feet. Additional analysis using SlopeW indicates that increasing the water elevation to 446.0 feet for the 3H:1V slope achieved a safety factor of 1.38 as shown in Figure 9 below.

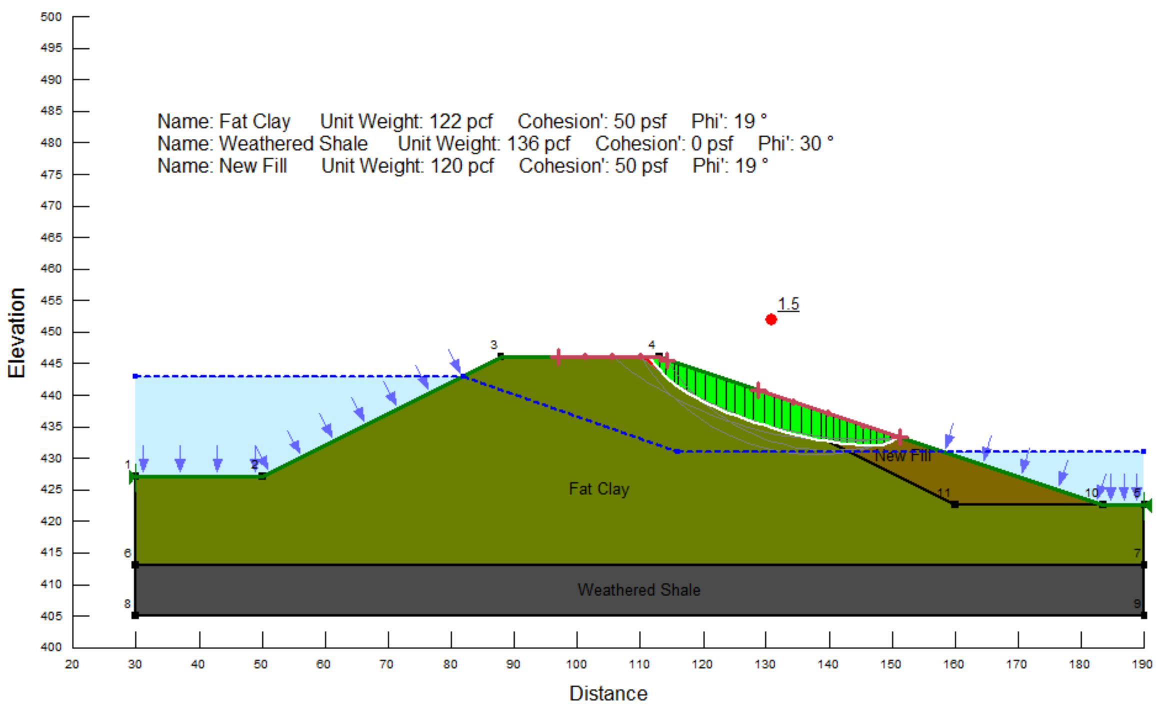


FIGURE 8: SLOPE MODIFIED TO 3H:1V TO ACHIEVE SAFETY FACTOR OF 1.5

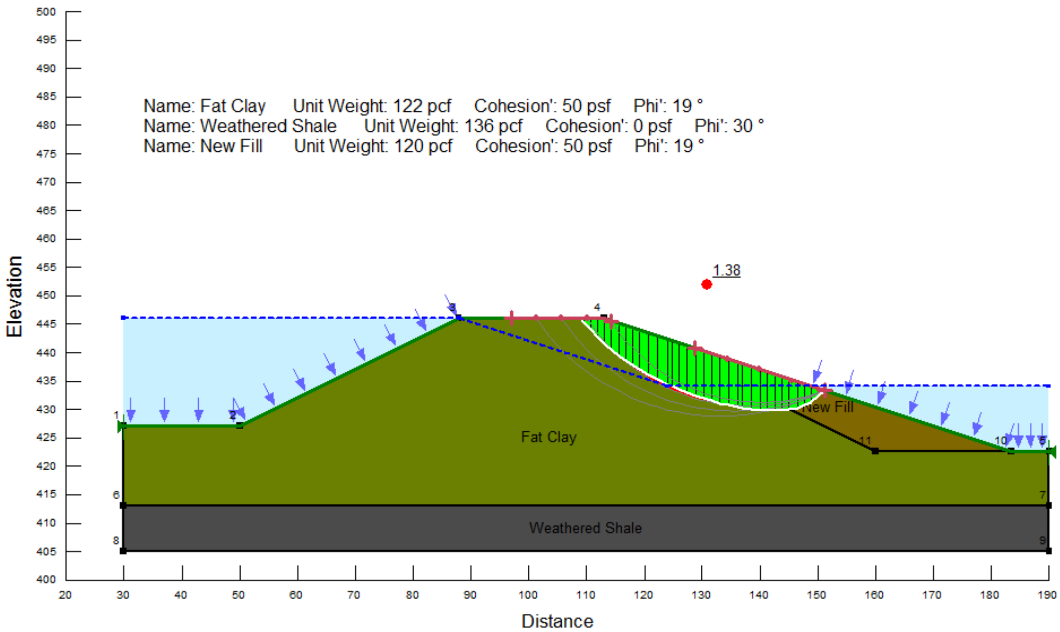


FIGURE 9: ALTERNATIVE 3 WITH MAXIMUM WATER ELEVATION - FACTOR OF SAFETY IS 1.38

Analysis of the of the 3H:1V slope with a reduced pool elevation of 445 produced little difference in factor of safety still achieving a safety factor of 1.38 as shown in Figure 10 below.

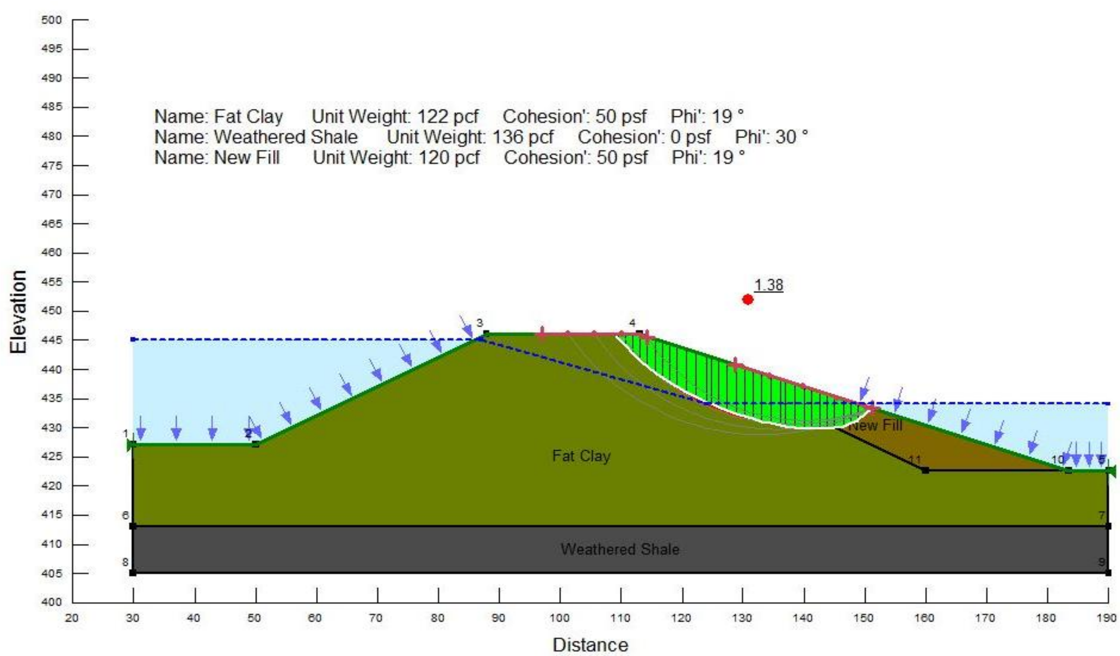


FIGURE 10: ALTERNATIVE 3 AT POOL 445 FT. - FACTOR OF SAFETY IS 1.38

A final analysis of alternative 3 at a pool elevation of 445 feet with seismic loads produced a factor of safety of 1.03 as shown in Figure 11.

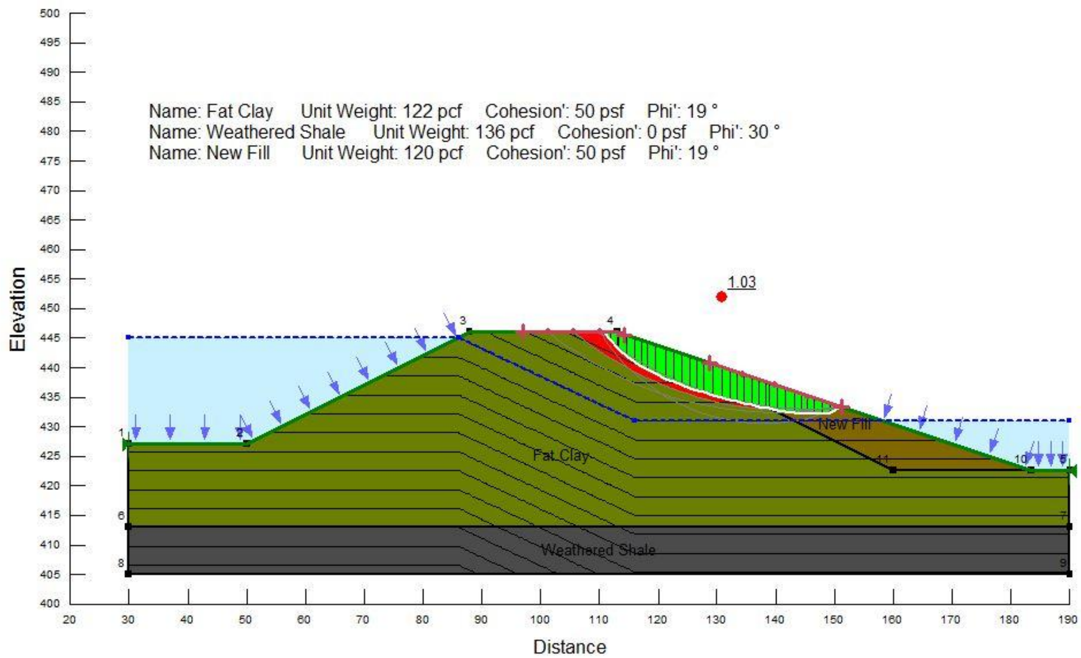


Figure 11: Alternative 3 at pool 445 ft. with seismic loads - Factor of Safety is 1.03

A compilation of stability analyses for the existing embankment and the three alternatives is presented in Table 2

TABLE 2 - SLOPE STABILITY ANALYSES RESULTS

WFEC Hugo Power Plant Bottom Ash Storage Embankment	REF FIGURE	FACTOR OF SAFETY
Existing - 2H:1V Slope	1	1.0
2.5H:1V Slope w/ Riprap	3	1.3
2H:1V Reinforced Slope	5	1.5
2H:1V Reinforced Slope - pool elevation at 445 ft.	6	1.5
2H:1V Reinforced Slope - pool elevation at 445 ft. & seismic load	7	1.1
3H:1V Slope	8	1.5

WFEC Hugo Power Plant Bottom Ash Storage Embankment	REF FIGURE	FACTOR OF SAFETY
3H:1V Slope - maximum pool elevation 446 ft.	9	1.38
3H:1V Slope – pool elevation 445 ft.	10	1.38
3H:1V Slope – pool elevation 445 ft and seismic loads	11	1.03

6.0 CONSTRUCTION RECOMMENDATIONS

6.1 General Site Preparation

Before any construction modification is made to the embankment, we recommend the detention pond level be lowered to maintain structural stability and to allow for construction access. We should be contacted once it is lowered to evaluate the existing conditions and stability of the embankment.

The existing riprap stone should be removed and stockpiled for reuse. There are unsuitable materials such as small trees and shrubs located within the embankment area. These will compromise stability of the embankment and can lead to seepage and instability; therefore, we recommend this vegetation be removed. Tree root systems should be thoroughly grubbed to remove all roots larger than 1/2 inch; however, the excavation to remove tree roots should be limited to a maximum depth of 5 feet. If this is not done, future decomposition of the roots will shorten the effective service life of the dam. Areas disturbed during the grubbing operation should be repaired before any new fill is placed. Any soft materials encountered should be completely removed. We recommend a qualified geotechnical engineer observe the exposed surface before new fill is placed for the embankment. We should be contacted immediately if any of these conditions are encountered. Organic material, topsoil and any deleterious and unsuitable material recovered from site stripping should not be incorporated in the new engineered fill section.

Any animal burrows and any existing voids should be backfilled. The extermination of the rodent population should be performed before backfilling the voids. The use of cement-bentonite-grout with at least 150 psi strength may be necessary to effectively fill the voids.

Following stripping but before placement of new fill on the crest or along the slopes of the embankment, the exposed subgrade should be scarified, moisture conditioned and compacted to at least 95% of the material's standard Proctor density, ASTM D-698. The moisture content should be within 2 percent of the material's optimum value during compaction.

6.2 Embankment Fill

We recommend all borrow soils used to repair and reshape the embankment to be a lean clay soil with a plasticity index within the range of 8 to 25 and free of organic matter and debris. We also recommend these soils to be tested for dispersion. These tests should be conducted by a qualified lab and consist of at least two of the three primary tests for dispersion; the Pinhole Test, the SCS Double Hydrometer Test, and/or the Emerson crumb test. All engineered fill should be approved by the engineer-of-record. All fill materials must be placed in controlled lifts that are 8 inches or less in loose thickness and should be compacted to at least 95 percent of the material's maximum standard Proctor dry density. The moisture content of the fill should be within 2 percentage points of its optimum value at the time of compaction.

The embankment fill should be benched horizontally into the slope to improve structural stability. Fill should not be frozen when placed or placed on frozen surfaces.

6.3 Slope Protection

To protect against erosion, any exposed slope should be vegetated with solid slab sod immediately following completion of remedial construction.

The crest surface of the finished dam should slope down slightly toward the reservoir so that precipitation falling on the crest will not accumulate or flow over the downstream slope.

7.0 GENERAL COMMENTS

Terracon Consultants, Inc. should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. A qualified geotechnical engineering and testing firm should be retained to provide observation and testing services during grading, excavation and other earth-related construction phases of the project.

The analysis and recommendations presented in this report are 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 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 of this project does not include either specifically or by implication any environmental assessment of the site or identification of contaminated or hazardous materials or conditions. If the owner is concerned about the potential of such contamination, 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 any 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 Consultants, Inc. reviews the changes, and either verifies or modifies the conclusions of this report in writing.

APPENDIX A
FIELD EXPLORATION

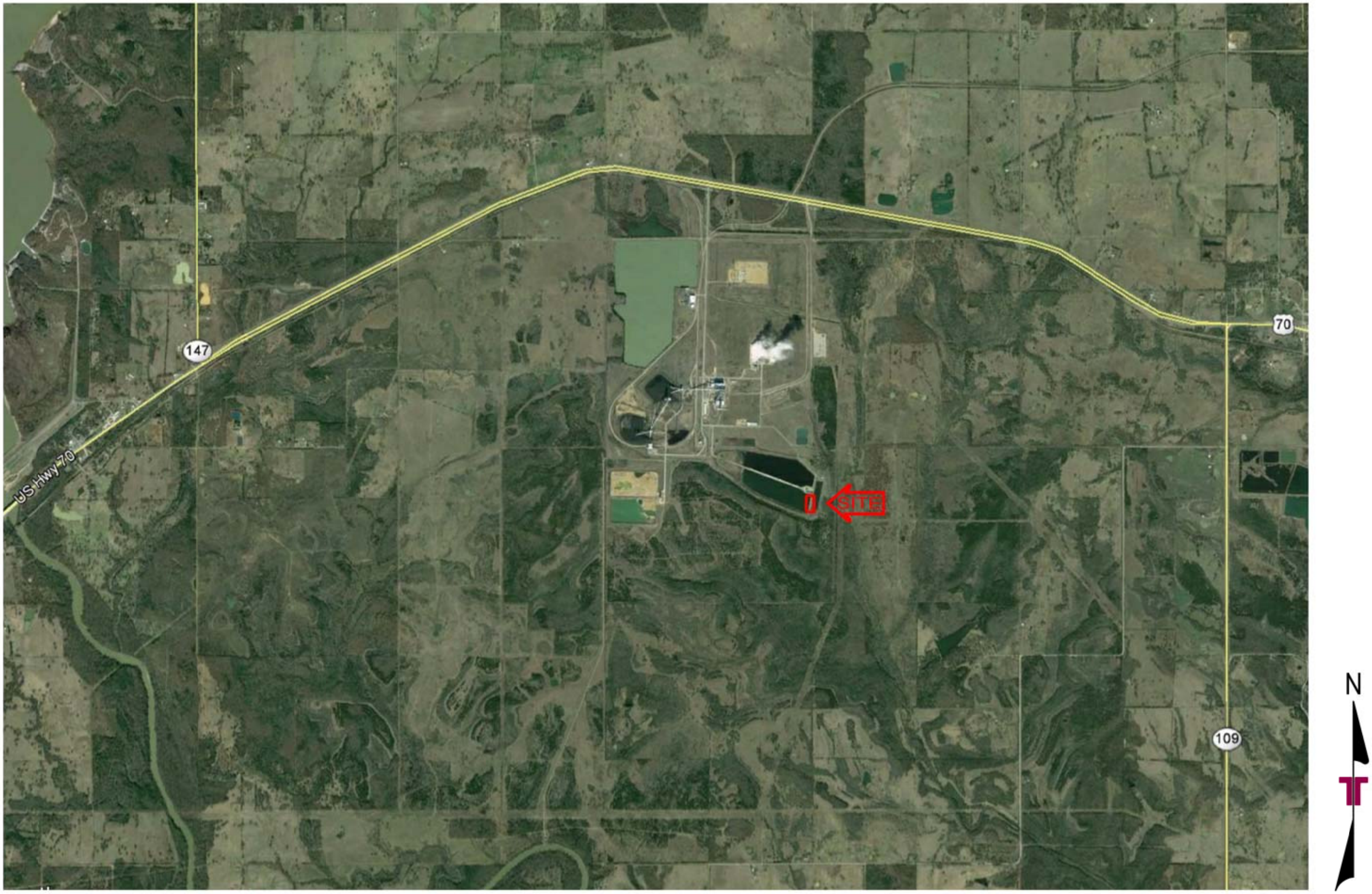


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES.

Project Mngr: JLD	Project No. 03165346 (03165139)	**Terracon** Consulting Engineers and Scientists	SITE LOCATION PLAN
Drawn By: CAN	Scale: NTS	4701 N STILES AVE OKLAHOMA CITY, OKLAHOMA 73105	WFEC HUGO POWER PLANT EMBANKMENT EVALUATION HUGO POWER PLANT APPROXIMATELY 11.0 MILES EAST OF HUGO HUGO, OKLAHOMA
Checked By: JLD	File No. 03165346 A1-A2 (03165139)	PH. (405) 525-0453 FAX. (405) 557-0549	EXHIBIT
Approved By: JWB	Date: OCTOBER 2016		A1

LEGEND

⊙ BORING LOCATION

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

Project Mngr: JLD	Project No. 03165346 (03165139)	Terracon Consulting Engineers and Scientists	BORING LOCATION PLAN	EXHIBIT
Drawn By: CAN	Scale: NTS		WFEC HUGO POWER PLANT EMBANKMENT EVALUATION HUGO POWER PLANT APPROXIMATELY 11.0 MILES EAST OF HUGO HUGO, OKLAHOMA	A2
Checked By: JLD	File No. 03165346 A1-A2 (03165139)	4701 N STILES AVE OKLAHOMA CITY, OKLAHOMA 73105		
Approved By: JWB	Date: OCTOBER 2016	PH. (405) 525-0453 FAX. (405) 557-0549		

FIELD EXPLORATION PROCEDURES

A total of two (2) test borings were drilled at the dam location on May 11, 2016. The boring depths ranged from approximately 35 to 37 feet below the ground surface at the approximate locations shown on the attached Boring Location Diagram, Exhibit A-1.

Terracon personnel located the borings in the field by taping distances from the references shown on the attached boring location diagrams. The surface elevations at the boring locations were determined using the plans provided by the client. The elevations on the boring logs have been rounded to the nearest 1/2 foot. The locations and elevations of the borings should be considered accurate only to the degree implied by the methods used to define them.

The borings were drilled with a truck mounted rotary drill rig using continuous flight augers to advance the boreholes. Representative samples were obtained by the split-barrel and thin-walled tube sampling procedures.

The split-barrel sampling procedure uses a standard 2-inch O.D. split-barrel sampling spoon that is driven into the bottom of the boring with a 140-pound drive hammer falling 30 inches. The number of blows required to advance the sampling spoon the last 12 inches, or less, of a typical 18-inch sampling interval or portion thereof, is recorded as the standard penetration resistance value, N. The N value is used to estimate the in-situ relative density of cohesionless soils and, to a lesser degree of accuracy, the consistency of cohesive soils and the hardness of sedimentary bedrock. In the thin-walled tube sampling procedure, a seamless steel tube with a sharpened cutting end is hydraulically pushed into the bottom of the boring to obtain a relatively undisturbed cohesive soil sample. The sampling depths, penetration distances, and the N values are reported on the boring logs. The samples were tagged for identification, sealed to reduce moisture loss and returned to the laboratory for further examination, testing and classification.

An automatic Standard Penetration Test (SPT) drive hammer was used to advance the split-barrel sampler. The automatic drive hammer achieves a greater mechanical efficiency when compared to a conventional safety drive hammer operated with a cathead and rope. We considered this higher efficiency in our interpretation and analysis of the subsurface information provided with this report.

Field logs were prepared as part of the drilling operations. These boring logs included visual classifications of the materials encountered during drilling and the field personnel's interpretation of the subsurface conditions between samples. The final boring logs included with this report may include modifications based on observations and tests of the samples in the laboratory.

As required by the Oklahoma Water Resources Board, any borings deeper than 20 feet, or borings that encounter groundwater or contaminated materials must be grouted or plugged in accordance with Oklahoma State statutes. One boring log must also be submitted to the Oklahoma Water Resources Board for each 10 acres of project site area. Terracon grouted the borings and submitted a log in order to comply with the Oklahoma Water Resources Board requirements.

BORING LOG NO. B-1 (03165139)

PROJECT: WFEC Hugo Power Plant Embankment Eval. with Supplementary Analysis

CLIENT: Guernsey Oklahoma City, Oklahoma

SITE: Approx. 11.0 Miles East of Hugo Hugo, Oklahoma

LOCATION See Exhibit A-2

Latitude: 34.00833° Longitude: -95.31319°

Approximate Surface Elev: 446.0 (Ft.) +/-

DEPTH	Description	ELEVATION (Ft.)
1.5	**FILL - GRAVELLY SAND**, light brownish-gray (2.5Y 6/2)	444.5+/-
21.0	**FAT CLAY WITH SAND (CH)**, trace gravel, olive (5Y 5/3) and dark olive (5Y 3/2), medium stiff -olive-gray (5Y 5/2), stiff below 4.5' -olive (5Y 5/3) and dark olive-gray (5Y 3/2), medium stiff below 7.5' -stiff below 9' -dark olive-gray (5Y 3/2) below 12' -trace sand, olive-gray (5Y 4/2) below 13.5'	425+/-
32.5	**SHALEY FAT CLAY**, very dark gray (5Y 3/1), very stiff -hard below 27.5'	413.5+/-
37.0	**+WEATHERED SHALE**, trace white (2.5Y 8/1) calcareous seams, dark gray (2.5Y 4/1), hard -light gray (2.5Y 7/1) below 35'	409+/-
	Auger Refusal at 37 Feet	

RECOVERY (In.)	FIELD TEST RESULTS	LABORATORY TORVANE/HP (psf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
2	9-2-2 N=4			28			
16	2-3-4 N=7			32			
18	3-3-5 N=8			32		76-25-51	78
18	3-4-5 N=9			30			
18	2-4-4 N=8			31		68-23-45	73
18	3-3-4 N=7			29		73-25-48	78
18	3-5-9 N=14			32			
6	6-4-5 N=9			22			
10	4-4-6 N=10			25		68-22-46	
18	4-6-7 N=13			28		56-21-35	73
6	4-4-7 N=11			29			
14	4-6-7 N=13			27			
18	5-4-5 N=9			33			
18	4-5-7 N=12			28			
18	5-9-11 N=20			23		67-22-45	91
18	8-10-12 N=22			23			
18	10-23-26 N=49			21			
3	50/1"			14			
1	50/1"			8			

Stratification lines are approximate. In-situ, the transition may be gradual.

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

Hammer Type: Automatic

Advancement Method:
Power Auger

Abandonment Method:
Backfilled with cuttings above 4'; grouted 4' to 14'; backfilled with cuttings from 14' to termination depth.

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

Dry While Drilling

Terracon
4701 N Stiles Ave
Oklahoma City, OK

Boring Started: 5/11/2016	Boring Completed: 5/11/2016
Drill Rig: 747	Driller: R. Peters
Project No.: 03165346 (03165139)	Exhibit: A-4

BORING LOG NO. B-2 (03165139)

PROJECT: WFEC Hugo Power Plant Embankment Eval. with Supplementary Analysis

CLIENT: Guernsey Oklahoma City, Oklahoma

SITE: Approx. 11.0 Miles East of Hugo Hugo, Oklahoma

LOCATION See Exhibit A-2

Latitude: 34.00798° Longitude: -95.31325°

Approximate Surface Elev: 446.0 (Ft.) +/-

DEPTH	Description	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	LABORATORY TORVANE/HP (psf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	FAT CLAY WITH SAND (CH), olive (5Y 5/3) and dark olive-gray (5Y 3/2), stiff					18	3-4-5 N=9			30		76-26-50	76
			5										
	-dark olive-gray (5YR 3/2) below 6'					18	3-4-4 N=8			31		71-29-42	76
	-olive (5Y 4/3) below 8.5'												
			10										
13.5		432.5+/-		▽									
	SHALEY FAT CLAY (CH), olive (5Y 4/3), trace dark olive-gray (5Y 3/2), very stiff					18	3-6-10 N=16			29		65-25-40	87
			15										
	-very dark gray (5Y 3/1), hard below 18.5'					18	13-21-25 N=46			20			
			20										
						18	14-25-40 N=65			19			
			25										
						18	18-26-33 N=59			20		61-30-31	87
			30										
33.5		412.5+/-											
35.0	**+WEATHERED SHALE**, trace white (2.5Y 3/1) calcareous seams, very dark gray (2.5Y 4/1), hard	411+/-	35			3	50/3"			13			

Boring Terminated at 35 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

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

Hammer Type: Automatic

Advancement Method:
Power Auger

Abandonment Method:
Backfilled with cuttings above 4'; grouted 4' to 14'; backfilled with cuttings from 14' to termination depth.

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

▽ *13.5 ft While Drilling*

Dry After Boring

Terracon
4701 N Stiles Ave
Oklahoma City, OK

Boring Started: 5/11/2016	Boring Completed: 5/11/2016
Drill Rig: 747	Driller: R. Peters
Project No.: 03165346 (03165139)	Exhibit: A-5

BORING LOG NO. B-2A (03165139)

PROJECT: WFEC Hugo Power Plant Embankment Eval. with Supplementary Analysis

CLIENT: Guernsey Oklahoma City, Oklahoma

SITE: Approx. 11.0 Miles East of Hugo Hugo, Oklahoma

LOCATION See Exhibit A-2

Latitude: 35.00798° Longitude: -95.31325°

Approximate Surface Elev: 446.0 (Ft.) +/-

DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	LABORATORY TORVANE/HP (psf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	FAT CLAY WITH SAND (CH), olive (5Y 5/3) and dark olive-gray (5Y 3/2), stiff				SPT	18	3-4-5 N=9						
					Shelby tube								
	-dark olive-gray (5YR 3/2) below 6'		5		SPT	18	3-4-4 N=8						
10.0	-olive (5YR 4/3) below 8.5'	436+/-	10		SPT		3-4-5 N=9						
	Boring Terminated at 10 Feet												

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
Power Auger

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

Dry After Boring

Terracon
4701 N Stiles Ave
Oklahoma City, OK

Boring Started: 5/11/2016	Boring Completed: 5/11/2016
Drill Rig: 747	Driller: R. Peters
Project No.: 03165346 (03165139)	Exhibit: A-6

APPENDIX B
LABORATORY TESTING

Laboratory Testing Program

Samples retrieved during the field exploration were taken to the laboratory for further observation by the project geotechnical engineer and were classified in accordance with the Unified Soil Classification System (USCS) described in Appendix C. Samples of bedrock were classified in accordance with the general notes for Sedimentary Rock Classification. At that time, the field descriptions were confirmed or modified as necessary and an applicable laboratory testing program was formulated to determine engineering properties of the subsurface materials.

The laboratory test results were used for the geotechnical engineering analyses, and the development of earthwork recommendations. Laboratory tests were performed in general accordance with the applicable ASTM, local or other accepted standards. The test results are presented in this appendix and in the boring logs.

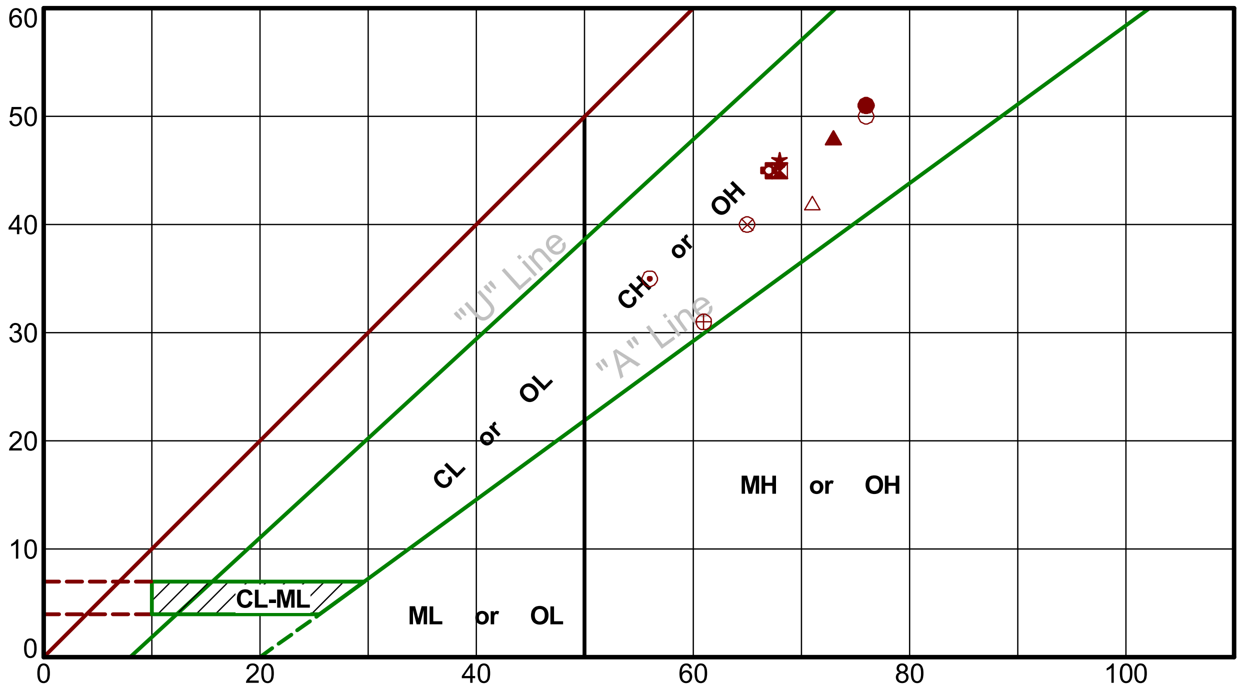
Selected soil and bedrock samples obtained from the site were tested for the following engineering properties:

- Atterberg Limits (ASTM D4318)
- Sieve Analysis (ASTM D422)
- Consolidated-Undrained Triaxial Compression (ASTM D4767)
- In-Situ Water Content (ASTM D2216)
- In-Situ Dry Density (ASTM D7263)
- Crumb Test (ASTM D6572)
- Double Hydrometer (ASTM D4221

Procedural standards noted above are for reference to methodology in general. In some cases variations to methods are applied as a result of local practice or professional judgment.

ATTERBERG LIMITS RESULTS

ASTM D4318

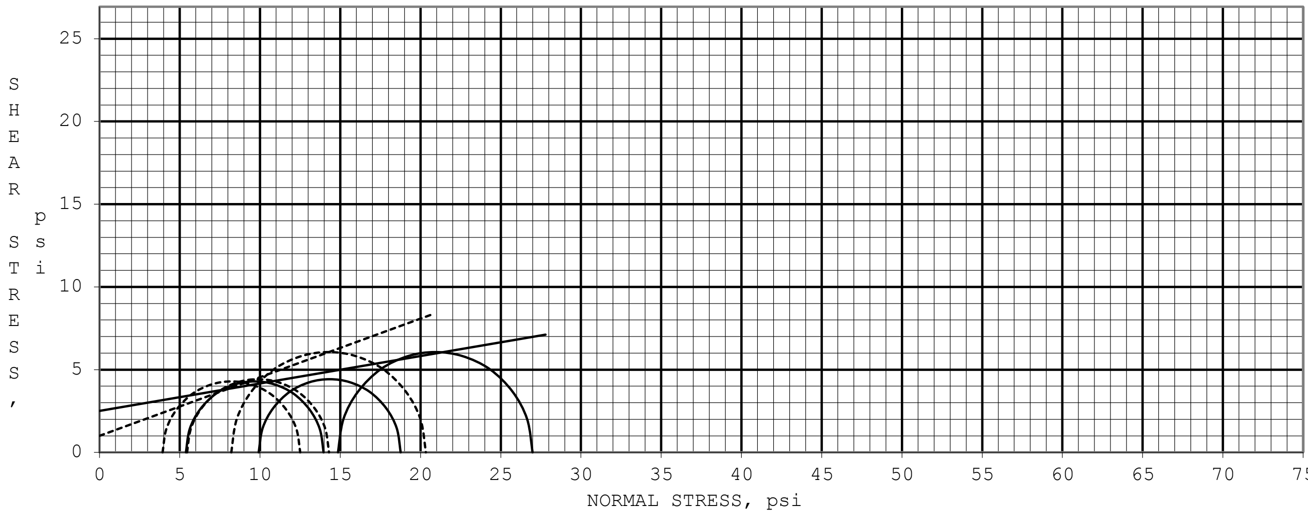


	Boring ID	Depth	LL	PL	PI	Fines	USCS	Description
●	**B-1 (03165139)**	**3 - 4.5**	**76**	**25**	**51**	**78**	**CH**	**FAT CLAY with SAND**
⊠	**B-1 (03165139)**	**6 - 7.5**	**68**	**23**	**45**	**73**	**CH**	**FAT CLAY with SAND**
▲	**B-1 (03165139)**	**7.5 - 9**	**73**	**25**	**48**	**78**	**CH**	**FAT CLAY with SAND**
★	**B-1 (03165139)**	**12 - 13.5**	**68**	**22**	**46**			**FAT CLAY with SAND**
⊙	**B-1 (03165139)**	**13.5 - 15**	**56**	**21**	**35**	**73**	**CH**	**FAT CLAY with SAND**
✿	**B-1 (03165139)**	**21 - 22.5**	**67**	**22**	**45**	**91**	**CH**	**FAT CLAY**
○	**B-2 (03165139)**	**1 - 2.5**	**76**	**26**	**50**	**76**	**CH**	**FAT CLAY with SAND**
△	**B-2 (03165139)**	**6 - 7.5**	**71**	**29**	**42**	**76**	**CH**	**FAT CLAY with SAND**
⊗	**B-2 (03165139)**	**13.5 - 15**	**65**	**25**	**40**	**87**	**CH**	**FAT CLAY**
⊕	**B-2 (03165139)**	**28.5 - 30**	**61**	**30**	**31**	**87**	**CH**	**FAT CLAY**

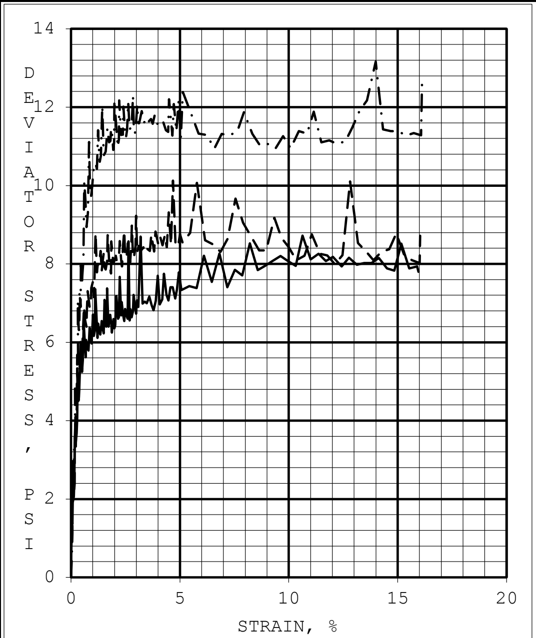
PROJECT: WFEC Hugo Power Plant Embankment Eval. with Supplementary Analysis
SITE: Approx. 11.0 Miles East of Hugo
Hugo, Oklahoma

Terracon
4701 N Stiles Ave
Oklahoma City, OK

PROJECT NUMBER: 03165346 (03165139)
CLIENT: Guernsey
Oklahoma City, Oklahoma
EXHIBIT: B-2



EFFECTIVE STRESS ---	ANGLE OF INTERNAL FRICTION, deg	**19.5**	COHESION, psi	**1.0**
TOTAL STRESS ——	ANGLE OF INTERNAL FRICTION, deg	**9.4**	COHESION, psi	**2.5**



SPECIMEN ID:		A	B	C
INITIAL	WATER CONTENT, %	27.8	31.6	27.3
	DRY DENSITY, pcf	95.6	92.1	96.9
	SATURATION, %	98	103	100
	VOID RATIO	0.76	0.83	0.74
BEFORE SHEAR	WATER CONTENT, %	28.1	29.7	26.1
	DRY DENSITY, pcf	95.8	93.5	98.8
	SATURATION (B PARAMETER)	0.97	0.96	0.97
	VOID RATIO	0.76	0.80	0.70
	FINAL BACK PRESSURE, psi	99.0	99.4	99.1
MINOR PRINCIPAL STRESS, psi		5.4	9.9	14.9
EFFECTIVE STRESS PEAK AT % STRAIN		2.6	2.4	2.6
EFF. DEVIATOR STRESS AT PEAK STRAIN, psi		8.6	8.8	12.1
TOTAL STRESS PEAK AT % STRAIN		2.6	2.4	2.6
TOTAL DEVIATOR STRESS AT PEAK STRAIN, psi		8.6	8.8	12.1
ULTIMATE DEVIATOR STRESS (15% STR), psi		8.2	8.7	11.3
TIME TO 50% PRIMARY CONSOLIDATION, min		8.19	10.00	13.00
STRAIN RATE, % / hour		1.09	1.10	1.11
INITIAL DIAMETER, inch		1.362	1.367	1.362
INITIAL HEIGHT, inch		2.864	2.907	2.877
AREA AFTER CONSOLIDATION, inch2		1.447	1.447	1.433

CONTROLLED - STRAIN TEST

SAMPLE TYPE: 3" SHELBY TUBE

DESCRIPTION OF SPECIMENS:

FAT CLAY (WEATHERED SHALE), YELLOWSIH BROWN WITH BROWN & GRAYISH BROWN

LL 65	PL 25	PI 40	Gs 2.7 EST.

PROJECT NO. 03165346 (03165139)

PROJECT: WFEC HUGO POWER PLANT EMBANKMENT INVESTIGATION

BORING #: B-2

LABORATORY: TERRACON - LENEXA

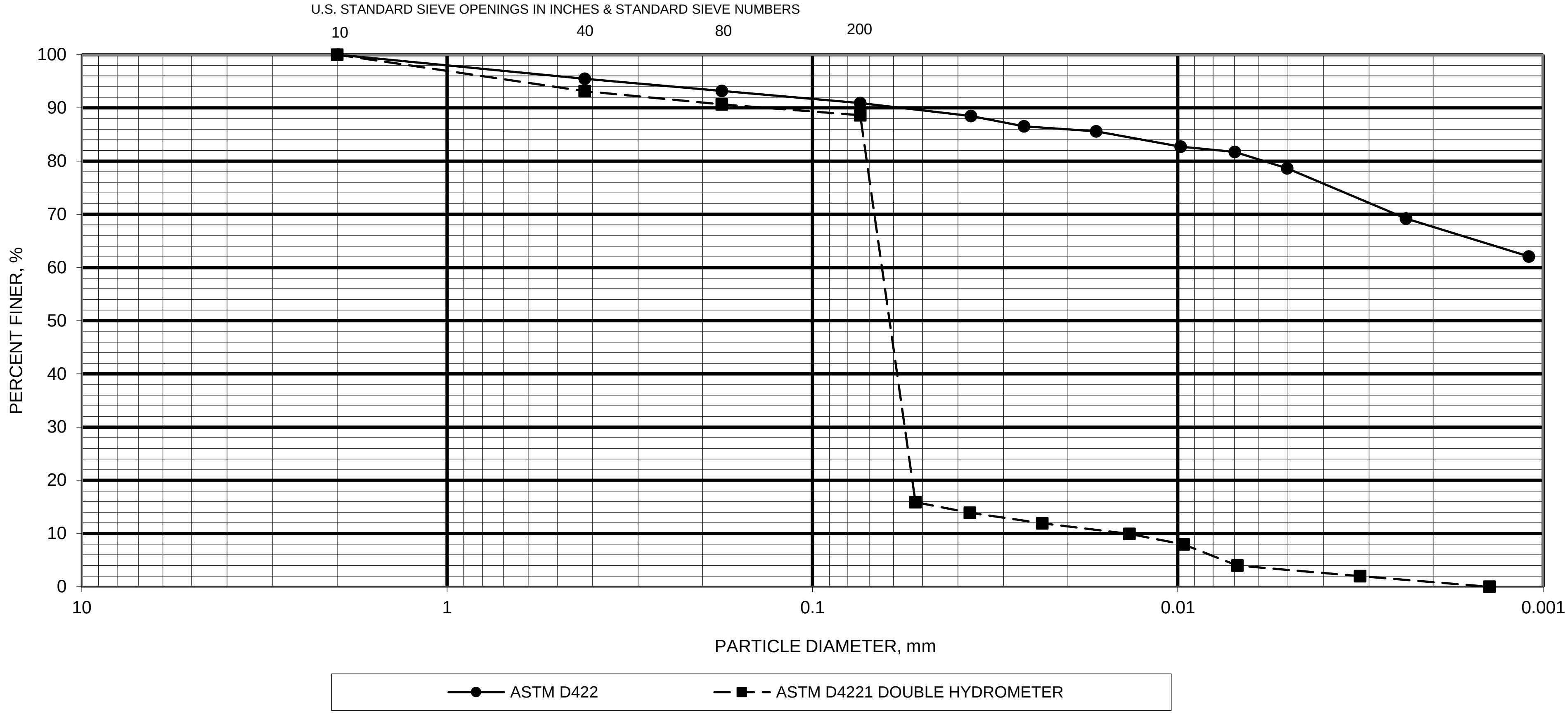
SAMPLE #: 4

DATE: 6/2/2016

DEPTH, feet: 8.0 - 10.0

PROCEDURE: ASTM D4767, CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS

Terracon



ASTM D4221 GRAIN SIZE DISTRIBUTION CURVES

ASTM D442, % 0.005mm	79	ASTM D4221 DOUBLE HYDROMETER, % 0.005 mm	3	DISPERSION, %	**4**

BORING NO.	SAMPLE NO.	DEPTH, feet	ASTM DESCRIPTION	UNIFIED SYMBOL	NAT M%	ATTERBERG LIMITS LL	PL	PI
B-2	2	3.5 - 5.5	FAT CLAY, YELLOWISH BROWN WITH GRAYISH BROWN & VERY DARK GRAY	CH	28.7	79	27	52

PROJECT WFEC HUGO POWER PLANT EMBANKMENT INVESTIGATION

APPROXIMATELY 11.0 MILES EAST OF HUGO, HUGO, OKLAHOMA

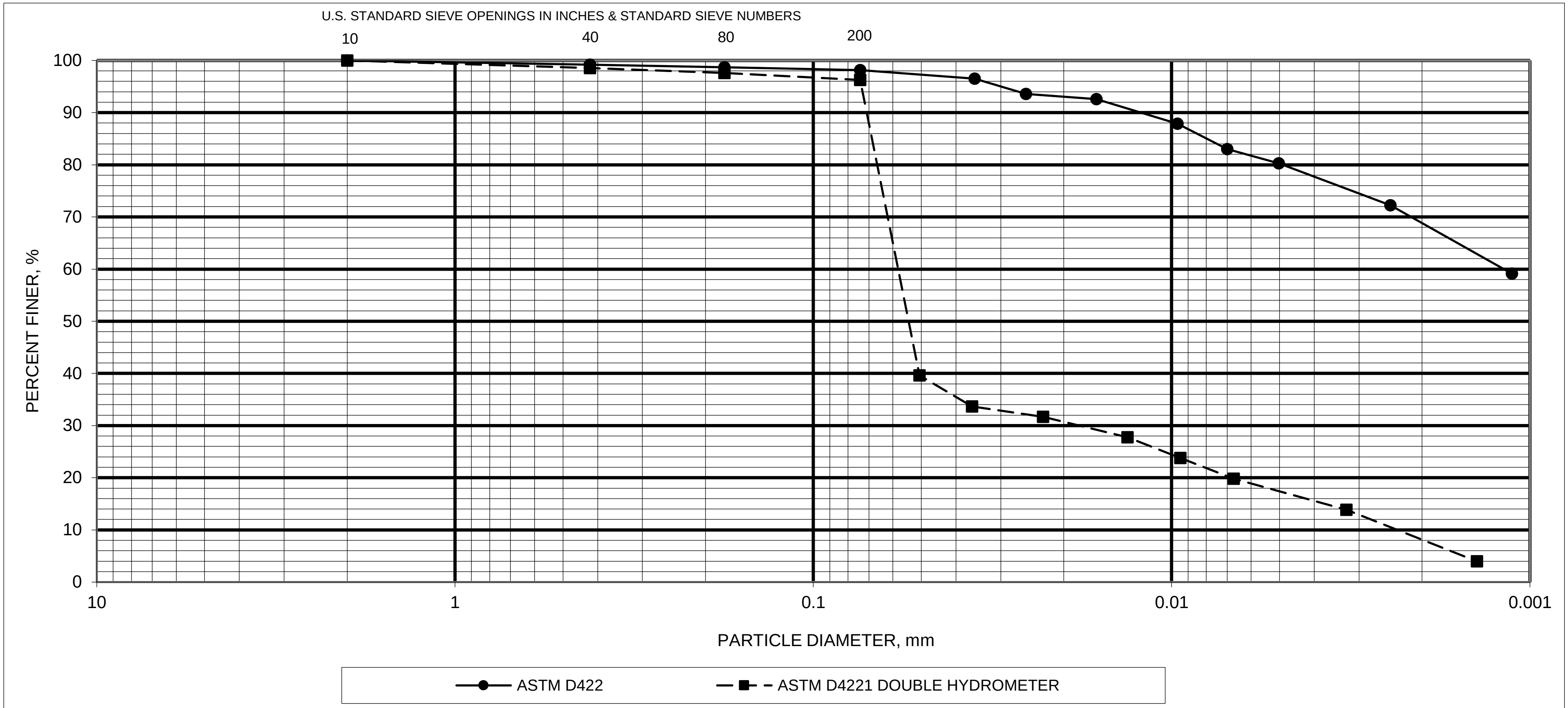
JOB NO. 03165346 (03165139)

DATE 6/3/2016

Terracon

N:\CM\LAB_DATA\00 Projects in Progress\2016 Projects in Progress\03165139 lab data\[03165139 HydrometerDoublePlot B2-2-3.5.xlsx]Report

Exhibit B-4



ASTM D4221 GRAIN SIZE DISTRIBUTION CURVES

ASTM D442, % 0.005mm	80	ASTM D4221 DOUBLE HYDROMETER, % 0.005 mm	18	DISPERSION, %	**23**

BORING NO.	SAMPLE NO.	DEPTH, feet	ASTM DESCRIPTION	UNIFIED SYMBOL	NAT M%	ATTERBERG LIMITS LL	PL	PI
B-2	4	8.0 - 10.0	FAT CLAY, YELLOWISH BROWN WITH BROWN & GRAYISH BROWN	CH	28.9	65	25	40

PROJECT WFEC HUGO POWER PLANT EMBANKMENT INVESTIGATION

APPROXIMATELY 11.0 MILES EAST OF HUGO, HUGO, OKLAHOMA

JOB NO. 03165346 (03165139)

DATE 6/3/2016

Terracon

N:\CM\LAB_DATA\00 Projects in Progress\2016 Projects in Progress\03165139 lab data\[03165139 HydrometerDoublePlot B2-4-8.0.xlsx]Double Hydrometer Data

APPENDIX C
PHOTOGRAPHS

No.1 - Slide on Downslope of Embankment

No. 2 - Scarp of the Slide Near the Crest of the Embankment

APPENDIX D
SUPPORTING DOCUMENTS

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

SAMPLING

Auger	Split Spoon
Shelby Tube	Pressure Meter
Texas Cone	Rock Core
Grab Sample	No Recovery

WATER LEVEL

- Water Initially Encountered
- Water Level After a Specified Period of Time
- Water Level After a Specified Period of Time

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

FIELD TESTS

(HP)	Hand Penetrometer
(T)	Torvane
(b/f)	Standard Penetration Test (blows per foot)
(PID)	Photo-Ionization Detector
(OVA)	Organic Vapor Analyzer
(TCP)	Texas Cone Penetrometer

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil 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.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance Includes gravels, sands and silts.			**CONSISTENCY OF FINE-GRAINED SOILS** (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance			
Descriptive Term (Density)	**Standard Penetration or N-Value Blows/Ft.**	**Ring Sampler Blows/Ft.**	**Descriptive Term (Consistency)**	**Unconfined Compressive Strength, Qu, psf**	**Standard Penetration or N-Value Blows/Ft.**	**Ring Sampler Blows/Ft.**
Very Loose	0 - 3	0 - 6	Very Soft	less than 500	0 - 1	< 3
Loose	4 - 9	7 - 18	Soft	500 to 1,000	2 - 4	3 - 4
Medium Dense	10 - 29	19 - 58	Medium-Stiff	1,000 to 2,000	4 - 8	5 - 9
Dense	30 - 50	59 - 98	Stiff	2,000 to 4,000	8 - 15	10 - 18
Very Dense	> 50	$\geq$ 99	Very Stiff	4,000 to 8,000	15 - 30	19 - 42
			Hard	> 8,000	> 30	> 42

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. (300 mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
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
Modifier	> 12

PLASTICITY DESCRIPTION

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

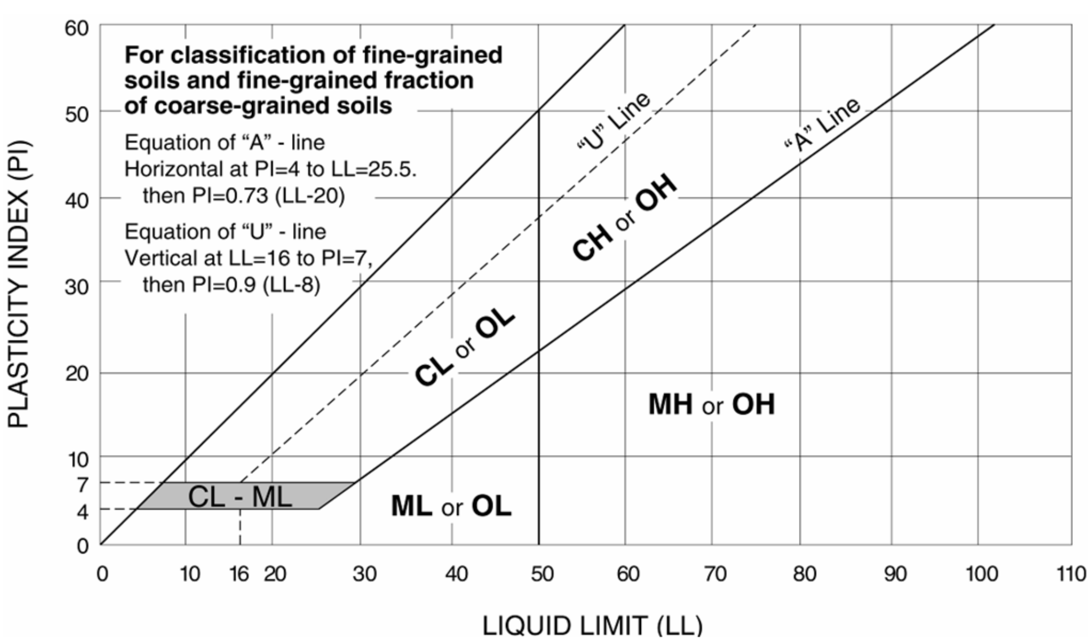
UNIFIED SOIL CLASSIFICATION SYSTEM

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-inch (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 ≥ 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 ≥ 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 ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



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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 (SiO_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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