REPORT

Hazard Potential Classification, Structural Stability, and Safety Factor Assessments, Revision 1

Upstream Raise 91 CCR Surface Impoundment, Coal Creek Station

Submitted to:

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APPENDICES

1.0 INTRODUCTION

Golder Associates Inc. (Golder) has prepared this hazard potential classification, structural stability, and safety factor assessment for the Upstream Raise 91 CCR Surface Impoundment (Upstream Raise 91) at Great River Energy's (GRE's) Coal Creek Station (CCS). The United States Environmental Protection Agency's (USEPA's) Coal Combustion Residual (CCR) Rule, 40 Code of Federal Regulations (CFR) Part 257 (USEPA 2015) requires a hazard potential classification, structural stability, and safety factor assessment be completed as specified in 40 CFR 257.73(a)(2), 40 CFR 257.73(d), and 40 CFR 257.73(e) for all existing CCR surface impoundments. Per 40 CFR 257.73(f), the hazard potential classification, structural stability, and safety factor assessment must be revisited every five years. This document serves as the current version of the hazard potential classification, structural stability, and safety factor assessment.

Upstream Raise 91 is also regulated by the North Dakota Department of Environmental Quality (NDDEQ) under Permit 0033. The NDDEQ requires a hazard potential classification assessment, structural stability assessment, and safety factor assessment as part of the application for a permit as described in Section 33.1-20-08-04.3.f.1 of the North Dakota Administrative Code (NDAC 2020). This report satisfies the state-specific requirement.

1.1 Site Background

CCS is located in McLean County, approximately 10 miles northwest of Washburn, North Dakota. Upstream Raise 91 covers approximately 75 acres (lined area) and is used as a combined dewatering and storage facility for CCRs including fly ash, bottom ash, and flue-gas desulfurization (FGD) material.

Upstream Raise 91 was originally part of the South Ash Pond, a legacy facility. The South Ash Pond was divided into Ash Pond 91 and Ash Pond 92. Ash Pond 91 was deepened and a new composite liner consisting of a 2-foot thick clay and 40-mil high-density polyethylene (HDPE) liner was completed in 1993. The liner is overlain with 1 foot of sand, 1 foot of Pit Run gravel, and a drainage system with collection pipes that slope to the north side of the facility. The composite liners of the Upstream Raise 92 CCR Surface Impoundment and Upstream Raise 91 are connected.

1.1 Geological Conditions

Upstream Raise 91 is generally constructed over a glacial till layer consisting of sandy silty-clay soils. Glacial till varies in thickness from 20 feet to several hundred feet in the area of Coal Creek Station. Silty sand and sand lenses are present throughout the glacial till formation, which is underlain by poorly consolidated siltstone/sandstone bedrock (Barr 1982; CPA and UPA 1989).

1.2 Dam Oversight/Permits

The North Dakota State Engineer regulates, controls, and supervises the construction and operation of dams within the state of North Dakota. All dams and impoundments that contain more than 50 acre-feet of water require a construction permit (NDCC 2003).

The NDDEQ Division of Waste Management is the environmental regulatory body for the CCR facilities at Coal Creek Station. Upstream Raise 91 is currently permitted with the NDDEQ under Permit Number 0033.

1.3 Previous Evaluations

The following evaluations were previously performed for Upstream Raise 91 at CCS:

- Golder Associates Inc., Evaluation of Ash Pond 91 Berm Stability Report, Great River Energy Coal Creek Station, dated April 13, 2010
- Golder Associates Inc., Seismic Stability Evaluation Addendum to Evaluation of Ash Pond 92/SW Section 16 Stability and Evaluation of Ash Pond 91 Stability, Great River Energy Coal Creek Station, dated February 27, 2012
- Kleinfelder, Coal Ash Impoundment Site Assessment Final Report, Coal Creek Station, dated October 31, 2012
- Golder Associates Inc., Hazard Potential Classification, Structural Stability, and Safety Factor Assessments, Ash Pond 91 CCR Surface Impoundment, Great River Energy – Coal Creek Station, Underwood, North Dakota, dated October 13, 2016

We have reviewed the previous analyses, modified the analyses as deemed appropriate, and added suitable cases to evaluate whether the impoundment meets the required safety factors in 40 CFR 257.73(e)(1)(i)-(iv) and NDAC Section 33.1-20-08-04.3.e.1.

2.0 HAZARD POTENTIAL CLASSIFICATION ASSESSMENT – 40 CFR 257.73(A)(2) AND NDAC SECTION 33.1-20-08-04.3.A.2

Both the federal CCR rules and North Dakota specific rules require conducting initial and periodic hazard potential classification assessments with certification by a qualified professional engineer to document the hazard potential classification of CCR surface impoundments and the basis for the classifications. Hazard classifications for CCR surface impoundments are divided in 40 CFR 257.73(a)(2) and NDAC Section 33.1-20-08-04.3.a.2 as follows:

- high hazard potential CCR surface impoundment
- significant hazard potential CCR surface impoundment
- low hazard potential CCR surface impoundment

The hazard classifications are defined under 40 CFR 257.53 and NDAC Section 33.1-20-08-01 as:

1) *High hazard potential CCR surface impoundment* means a diked surface impoundment where failure or mis-operation will probably cause loss of human life.
2) *Low hazard potential CCR surface impoundment* means a diked surface impoundment 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.
3) *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.

2.1 Hazard Potential Classification Assessment

Based on the hazard classification definitions and review of the site and surroundings, Golder recommends Upstream Raise 91 be categorized as a Low Hazard Potential CCR Surface Impoundment. This recommended designation is based on the following:

- The volume of impounded water is relatively small.
- There are no residences or occupied structures directly adjacent to the facility and loss of human life is not probable.
- Discharge of water or other materials contained within the impoundment is unlikely to have a significant environmental impact and is not likely to leave the owner’s property. Samuelson Slough has significant volume to contain discharge from the impoundment and has a controlled discharge that can be closed in the event of a release.
- The economic impacts associated with a failure will primarily be to the owner’s property.
- A USEPA consultant performed a site assessment of Coal Creek Station in 2011 under USEPA supervision. Following the USEPA dam hazard classification system, Upstream Raise 91 was given a “Low Hazard” classification (Kleinfelder 2012).

2.2 Emergency Action Plan – 40 CFR 257.73(a)(3) and NDAC Section 33.1-20-08-04.3.a.3

The federal CCR rules and North Dakota-specific rules require the development of an Emergency Action Plan (EAP) for a CCR unit determined to be either a high hazard potential CCR surface impoundment or a significant hazard potential CCR surface impoundment. Upstream Raise 91 has been categorized as a low hazard potential CCR surface impoundment and no EAP is required.

3.0 STRUCTURAL STABILITY ASSESSMENT – 40 CFR 257.73(D)(1)(I)-(VII) AND NDAC SECTION 33.1-20-08-04.3.D.1.A-G

The federal CCR rules (40 CFR 257.73(d)(1)) and North Dakota-specific CCR rules (NDAC Section 33.1-20-08-04.3.d.1) require conducting initial and periodic structural stability assessments certified by a qualified professional engineer to “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 that can be impounded therein”.

To support this structural stability assessment, annual inspections by a qualified professional engineer have been performed since 2015. These inspections have evaluated the current condition of constructed elements, operations and maintenance activities, and a review of weekly inspection reports to assist in identifying any signs of structural instability at the site. The most recent evaluation was conducted on August 30, 2021 by the authors and was used to inform this assessment.

3.1 Foundation – 40 CFR 257.73(d)(1)(i) and NDAC Section 33.1-20-08-04.3.d.1.a

The location of Upstream Raise 91 was originally characterized by Burns & McDonnell in 1973 and a hydrogeologic study was performed for Coal Creek Station by Barr Engineering in 1982. Site geology, soils, and hydrology, including drainage and surface water flow, were examined during these prior studies to determine site suitability for disposal of CCRs.

The foundation soils of Upstream Raise 91 consist of native soils (lean clay, fat clay, and sandy clay) and embankment fill materials sourced from nearby native soils (lean clay, fat clay, and sandy clay). Upstream Raise 91 was cleaned out, deepened, and re-lined in 1993 (CPA 1993). According to construction documents, the foundation materials were compacted to 93% standard Proctor density.

Based on historic site information of CCRs and native soils, observations, and geotechnical testing, Upstream Raise 91 is not built over wet ash or other unsuitable materials and foundation soils are stable.

3.2 Slope Protection – 40 CFR 257.73(d)(1)(ii) and NDAC Section 33.1-20-08-04.3.d.1.b

Soil Embankment

Upstream Raise 91 is surrounded by a soil embankment with downstream and upstream slopes. The soil embankment downstream slopes are protected from erosion and deterioration by the establishment of a vegetative cover consisting of native grasses. The vegetative cover is inspected weekly for erosion, signs of seepage, animal burrows, sloughing, and woody vegetation that could affect the performance of the embankments. The upstream slopes of the soil embankments (composite liner slopes) are protected by a 2-foot thick layer of hardened fly ash.

CCR Slopes above Soil Embankment

The downstream faces of the CCR slopes are protected from erosion and deterioration by a hardened fly ash surface. The interior crest and upstream slope into the pool area of Upstream Raise 91 are protected by a 6- to 15-foot high layer of trafficked bottom ash or other CCR material. The location of this layer of bottom ash changes as Upstream Raise 91 increases in height and is maintained by GRE personnel on a weekly basis if deterioration is noted. Based on site experience and observations, the fly ash and bottom ash provide sufficient protection from deterioration due to wave action.

3.3 Dikes (Embankment) – 40 CFR 257.73(d)(1)(iii) and NDAC Section 33.1-20-08-04.3.d.1.c

Embankments that are a part of Upstream Raise 91 were constructed in 1992 and were compacted to a minimum of 93% standard Proctor density (CPA 1993). Based on historic site information and geotechnical testing during construction, the embankment soils are stable.

3.4 Vegetated Slopes – 40 CFR 257.73(d)(1)(iv) and NDAC Section 33.1-20-08-04.3.d.1.d

Upstream Raise 91 is inspected weekly. As part of these inspections, unusual or woody vegetative growth on slopes with final cover and soil embankments is documented. Vegetated slopes of Upstream Raise 91 are re-seeded and mowed as required to maintain good vegetative growth and to limit woody vegetation from growing on the side slopes or near the toe.

3.5 Spillways – 40 CFR 257.73(d)(1)(v) and NDAC Section 33.1-20-08-04.3.d.1.e

Existing controls are in place to monitor water levels in Upstream Raise 91 and limit potential overtopping of the impoundment. Inflows to the facility (besides precipitation) include hydraulically conveyed FGD material. Additional inflows include passive drainage from the adjacent Upstream Raise 92 CCR facility. The design crest of the soil embankments surrounding Upstream Raise 91 are at an approximate elevation of 1922 feet, which is approximately 20 feet above surrounding topography, preventing stormwater run-on into Upstream Raise 91.

Existing controls in place to monitor the water levels in Upstream Raise 91 include weekly observations of water levels by Coal Creek Station personnel, and daily observations by Coal Creek Station operations personnel. Additional observations are noted by GRE employees familiar with site CCR units. After large storm events, Coal Creek Station personnel evaluate site conditions, including impoundment water levels, and are able to adjust operations to maintain water levels below design maximum elevations. Should water levels within Upstream Raise 91 reach above desired operating levels, GRE has operating procedures to lower gravity drain (decant) pipelines into the water to transfer water to the adjacent Drains Pond System CCR Surface Impoundment (Drains Pond System) facility. Contact water in perimeter ditches flows passively from Upstream Raise 91 to the Drains Pond System.

Upstream Raise 91 is designed to operate with a minimum freeboard of approximately 3 feet and a design freeboard of approximately 6 feet. A run-on analysis was performed as part of the inflow design flood control system plan (Golder 2021) indicating that Upstream Raise 91 is operated with adequate freeboard to contain the 24-hour, 100-year storm event.

3.6 Hydraulic Structures – 40 CFR 257.73(d)(1)(vi) and NDAC Section 33.1-20-08-04.3.d.1.f

Upstream Raise 91 has an inflow pipe for depositing FGD material. The on-grade HDPE pipe conveying FGD material is 8 inches in diameter and is periodically moved to different areas of the facility to achieve an even distribution of FGD material in the facility.

The outflows from Upstream Raise 91 consist of a series of gravity drainage pipes and culverts that transfer CCR conveyance water from the facility to the adjacent Drains Pond System.

No significant deterioration, deformation, distortion, bedding deficiencies, sedimentation, or debris have been observed that may negatively affect the operation of the facility.

3.7 Downstream Slopes Adjacent to Water Body – 40 CFR 257.73(d)(1)(vii) and NDAC Section 33.1-20-08-04.3.d.1.g

Samuelson Slough lies downstream of Upstream Raise 91, approximately 400 feet away. However, the outlet elevation of Samuelson Slough and the site surface water drainages are controlled, and water cannot inundate the downstream toe or slopes of Upstream Raise 91.

3.8 Structural Stability Deficiencies – 40 CFR 257.73(d)(2) and NDAC Section 33.1-20-08-04.3.d.2

No structural stability deficiencies were identified during this assessment.

4.0 SAFETY FACTOR ASSESSMENT – 40 CFR 257.73(E) AND NDAC SECTION 33.1-20-08-04.3.E

The federal CCR rules and North Dakota-specific rules require safety factors to be evaluated for the critical cross-section of each CCR facility under static and seismic loading for long-term maximum storage pool loading conditions and maximum surcharge pool loading conditions. Liquefaction potential analysis is only necessary when soil sampling, construction documentation, or anecdotal evidence from personnel with knowledge about the facility indicates that soils of the embankment are susceptible to liquefaction.

Slope stability analyses were performed using a limit-equilibrium-based commercial computer program, SLIDE v.7.0 (Rocscience 2016). Factors of safety were computed for non-circular failure surfaces using Spencer's method for force and moment equilibrium (Spencer 1967). Global stability was analyzed, which evaluates the overall stability of a cross section through the entire facility.

4.1 Model Scenarios

Two types of loading conditions for the stability analyses were performed: static and seismic (pseudostatic analyses). For each of the two loading conditions, the critical cross section was modeled. Two impoundment pool loading scenarios were considered to evaluate the slope: long-term maximum storage pool loading condition and maximum surcharge pool loading condition. Four stability scenarios were analyzed (two static loading scenarios and two pseudostatic loading scenarios).

4.2 Slope Geometries and Critical Slope

A critical cross section for Upstream Raise 91 was identified and used for the stability analyses. The critical cross section is anticipated to be the most susceptible to structural failure and was selected based on loading conditions, geometry of the slopes, and the soil profile.

The critical cross section for Upstream Raise 91 is located across the north embankment (Figure 1) at the full height of the facility during operation as a surface impoundment. Subsequent to this phase of construction, Upstream Raise 91 is planned to be dewatered, covered, and closed with CCR in place. The design crest of the soil embankment at the critical section is at an approximate elevation of 1922 feet. The critical cross section has an approximately 100-foot-wide crest at elevation 2004 feet and downstream slopes between 15% and 20% from elevation 2004 to approximately elevation 1922 feet (Figure 2). Based on existing topography, soil embankment downstream slopes generally have 3H:1V slopes down to perimeter drainage ditches with an elevation of approximately 1898 feet. Original soil embankment upstream slopes have an approximate 3H:1V slope from the crest of the facility to the interior base elevation within the facility of 1900 feet.

4.3 Material Properties

Material properties (Table 1) were developed through Golder's site experience with onsite materials, laboratory tests completed on Coal Creek Station CCRs and onsite soils, review of technical literature, and Golder's professional judgment and experience with similar materials used for this slope stability analysis. Additional static materials properties information is included in Appendix A.

Table 1: Material Properties

Material	Wet Unit Weight (pcf)	Shear Strength Static Analysis		Shear Strength Pseudostatic Analysis		
		Effective Cohesion (psf)	Effective Friction Angle (degrees)	Undrained Cohesion (psf)	Undrained Friction Angle (degrees)	Undrained Shear Strength (psf)
Existing Natural Soil and Embankment Fill	127	500	19	165	14	NA*
Clay Liner	122	500	19	NA	NA	1600
Sand Layer	125	0	37	0	37	NA
Pit Run	125	420	34	420	34	NA
FGD Material	Varies	Appendix A	Appendix A	Appendix A	Appendix A	Appendix A
Bottom Ash	83	50	40	50	40	NA
Fly Ash	107	1610	32	1610	32	NA
Smooth Geomembrane – Clay Liner Interface	NA	200	7.5	200	7.5	NA
Smooth Geomembrane – Bottom Ash	NA	0	17	0	17	NA

* NA – Not applicable

4.3.1 Pseudostatic Material Properties

The material properties for each soil type included in the pseudostatic stability analysis of Upstream Raise 91 are provided in Table 1 and were developed following the recommendations contained within Section 6.1 of RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities (USEPA 1995). Fine-grained soils (existing natural soil, embankment fill, and clay liner) were assigned strength parameters corresponding to 80% of the total stress (undrained) strength parameters.

- The existing natural soil and embankment fill were assigned strength parameters based on laboratory testing of those materials. The reduction of the strength parameters to 80% of the total stress strength parameters resulted in cohesion intercept of 165 pounds per square foot (psf) and a friction angle of 14 degrees.
- The clay liner was assigned an undrained shear strength of 1,600 psf based on literature values for CH material (NAVFAC 1996).

- The geosynthetic interface strength parameters were not modified (from the static stability material properties) for the pseudostatic stability analyses based on recommendations contained within Section 6.1.2 of RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities (USEPA 1995).
- Fly ash is a cemented material and is modeled with static shear strengths with no seismic reduction.
- FGD material properties were unchanged since undrained strengths are used for this material in both the static and seismic conditions.

Coarse-grained materials (i.e., sand, pit run, and bottom ash) are modeled with static shear strengths with no seismic reduction because they are not prone to developing excess pore pressure from the cyclic loading caused during seismic conditions. These materials are well compacted within the facility and are not likely to be saturated in the critical section evaluated.

4.4 Impoundment Pool Loading Conditions

Two different impoundment pool loading conditions were considered in these stability analyses:

- long-term maximum storage pool loading condition
- maximum surcharge pool loading condition

The depth of impounded water in Upstream Raise 91 varies with time as more CCRs are deposited and as operational variables change (such as drainage pipe elevations). Based on facility operations of Upstream Raise 91 over the last 5 years and operations in the adjacent Upstream Raise 92 over the past 15 years, the maximum storage pool loading condition is anticipated to occur when the elevation of water within the impoundment is maintained at the approximate same elevation as FGD material, which equates to a freeboard to the surrounding embankments of approximately 9 feet. Slope stability was modeled at maximum facility height (when Upstream Raise 91 is operating as an impoundment) with an embankment elevation of 2004 feet and a FGD material/water elevation of 1995 feet.

The maximum surcharge pool loading condition is conservatively modeled with a freeboard of zero feet (embankment elevation of 2004 feet and a water elevation of 2004 feet). In this scenario, the FGD material elevation within the facility remains at approximately 1995 feet. The likelihood of the water within Upstream Raise 91 reaching an elevation of 2004 feet or overtopping the embankment is very low, based on the following reasoning:

- Primary design inflows to Upstream Raise 91 are via the FGD material pipeline from the plant. The FGD material inflow is actively managed per the operations plan and can be shut off at any time.
- Additional inflow may come from precipitation events; however, the only areas running into the impoundment pool include a small area on the upper surface of the facility (see Inflow Flood Control Plan, Golder 2021). Based on the Inflow Flood Control Plan, the maximum increase in elevation of the Upstream Raise 91 pool from a precipitation event is approximately 1 foot.

The operations plan discusses the operation and contingency plans associated with Upstream Raise 91 (Golder 2020).

4.5 Subsurface Water Conditions

Upstream Raise 91 is lined with a composite geomembrane/clay liner. Readings from the piezometers and monitoring wells installed near Upstream Raise 91 demonstrate that groundwater generally flows northeast under the facility toward Samuelson Slough and that groundwater elevations vary from approximately 1880 feet in the northeast corner of Upstream Raise 91 to 1900 feet in the southwest corner of Upstream Raise 91. Based on these data and the location of the critical section, the stability analyses were performed with a groundwater elevation of between approximately 1890 feet and 1895 feet.

4.6 Interior Impoundment Phreatic Surface

Seepage analyses were performed on Upstream Raise 91 to determine the phreatic surface within the facility at the maximum impoundment elevation (FGD material elevation 1995 feet, Golder 2015). The phreatic surface is shown in Figure 2 and was assumed to follow the bottom ash/FGD material interface for full height steady state conditions. Hydrostatic pore pressures were assumed below the phreatic surface as water is anticipated to drain relatively slowly through FGD materials toward perimeter drainage piping. After FGD material deposition ceases, the phreatic surface decreases in elevation as the FGD material consolidates and drains.

4.7 Seismic Loading Conditions

Coal Creek Station, located in central North Dakota, is in an area with low historic seismic activity. No earthquakes of Magnitude V (i.e., Moderate-Strong) or greater (Mercalli intensity scale) have occurred in North Dakota during historical times (USGS 2021).

4.7.1 Peak Ground Acceleration Determination

For the site location, the peak (bedrock) ground acceleration (PGA) with a 2% probability of exceedance in 50 years is between 0.02 gram (g) (1g equals 32.2 feet per second squared [ft/sec^2]) and 0.04g using the United States Seismic Hazard 2018 Map (Rukstales and Petersen 2019); for the purposes of this analysis, the bedrock PGA was estimated to be 0.03g (Appendix B). The peak ground acceleration at Coal Creek Station was estimated to be 0.05g using the simplified analysis guidelines presented in Section 4.1.1 of the RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities (USEPA 1995; see Appendix B).

4.7.2 Horizontal Seismic Coefficient Determination

The horizontal seismic load coefficient (k_s), for use in pseudostatic slope stability analysis, was determined using the procedures recommended in Section 6.2 of the RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities (USEPA 1995). Based on Step 2 of the Seismic Stability and Deformation Analysis section of the guidance (Section 6.2), the maximum value of the horizontal seismic load coefficient may be safely determined as one-half of the peak ground acceleration (determined in Section 4.7.1). As a result, a horizontal seismic load coefficient of 0.025g (0.5 * 0.05g = 0.025g) was used in the pseudostatic analysis.

4.8 Liquefaction Potential

Soil liquefaction describes a phenomenon in which a saturated or partially saturated soil substantially loses strength and stiffness in response to an applied stress, usually earthquake shaking or other sudden change in stress condition, causing the soil to behave like a liquid. The phenomenon is most often observed in saturated, loose (low density or uncompacted), sandy soils. The foundation and embankment materials (existing natural soils, embankment fill, and clay liner) of Upstream Raise 91 are classified as either low or high plasticity clay.

In addition, an assessment of liquefaction potential was evaluated given the limited information about the site and in-situ testing of FGD material. The analysis of liquefaction potential is based partially on USEPA's Seismic Design Guidance for Municipal Solid Waste Landfill Facilities published in 1995 (USEPA 1995) and the evaluation of liquefaction resistance based on SPT values described by Youd and Idriss (2001). This method uses material SPT values, material unit weight, and expected ground motion for North Dakota (acceleration) to evaluate liquefaction potential. Based on a range of cyclic stress ratios (CSR) between 0.04 and 0.11 and corrected SPT blow counts between 3 and 14 for FGD material, Upstream Raise 91 is not anticipated to be susceptible to liquefaction due to low seismic stresses.

4.9 Stability Analysis Results

Slope stability analyses were performed under static and pseudostatic conditions for the critical section (Figure 3 and Figure 4). The results of the slope stability analyses are presented in Table 2. The results indicate that the Upstream Raise 91 slopes comply with the required safety factors per 40 CFR 257.73(e) and NDAC Section 33.1-20-08-04.3.e.

Table 2: Slope Stability Analyses Results

Loading Condition	Water Level	Required Factor of Safety	Calculated Factor of Safety	Figure for Stability Analysis Results
Static	Max Storage (el. 1995 ft)	1.50	1.59	3
	Max Surcharge (el. 2004 ft)	1.40	1.59	3
Pseudostatic	Max Storage (el. 1995 ft)	1.00	1.34	4
	Max Surcharge (el. 2004 ft)	1.00	1.34	4

5.0 REVISION HISTORY

A history of revisions to this document:

Revision 0 – Published October 13, 2016.

Revision 1 – 5-Year Update: Published October 13, 2021.

1) New CCR unit naming convention (Ash Pond 91 to Upstream Raise 91)

6.0 CERTIFICATION

Based on the review of the information provided by GRE and on-site observations, we have classified the Coal Creek Station Upstream Raise 91 CCR Surface Impoundment as a Low Hazard Potential CCR Surface Impoundment. Additionally, no structural stability deficiencies were identified during this assessment. Calculated factors of safety through the critical cross sections in the surface impoundment embankments exceed the values listed in 40 CFR 257.73(e)(1)(i)-(iv) and NDAC Section 33.1-20-08-04.3.e.1.

The undersigned attest to the completeness and accuracy of this hazard potential classification, structural stability assessment, and safety factor assessment, and certify that the assessments meet the requirements of 40 CFR 257.73(a)(2), 257.73(a)(3), 257.73(d), and 257.73(e) and NDAC Sections 33.1-20-08-04.3.a.2, 33.1-20-08-04.3.a.3, 33.1-20-08-04.3.d, and 33.1-20-08-04.3.e.

Signature Page

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https://golderassociates.sharepoint.com/sites/140044/project files/6 deliverables/21451024/reports/11-r-ur91_stability_rev1/11-r-0/21451024-11-r-0-ur91_stability rev1_13oct21.docx

7.0 REFERENCES

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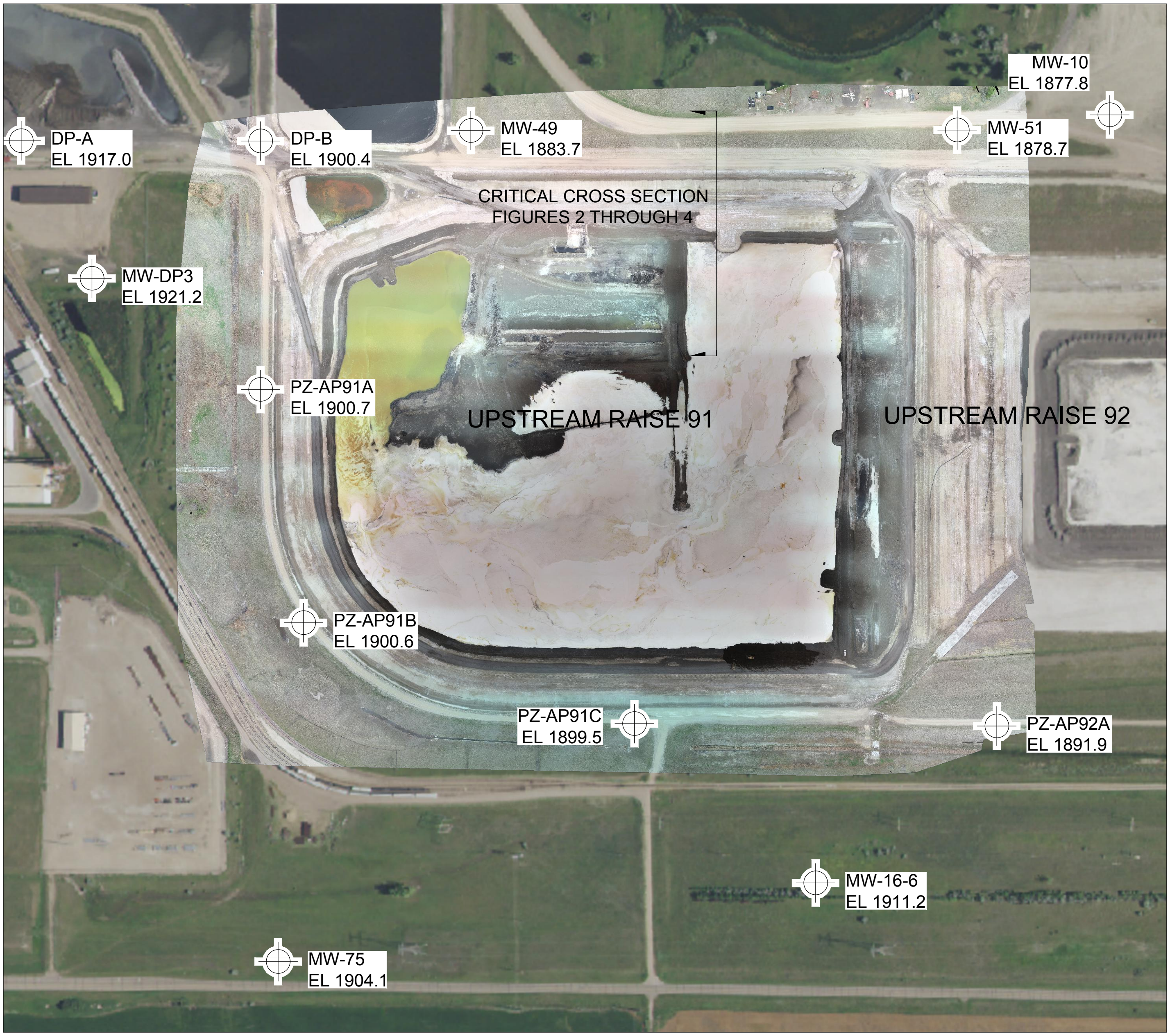
Figures



NOTE(S)

1. AERIAL IMAGERY OBTAINED FROM UNITED STATES DEPARTMENT OF AGRICULTURE NATIONAL AGRICULTURE IMAGERY PROGRAM 2020 AND DRONE PHOTOGRAPH PROVIDED BY GREAT RIVER ENERGY IN 2021.
2. MONITORING WELL GROUNDWATER ELEVATIONS FROM MAY 2021 SAMPLING EVENT.

0 250 500
1" = 500' FEET

GOLDER
MEMBER OF WSP

GREAT RIVER ENERGY
COAL CREEK STATION - UPSTREAM RAISE 91
SITE OVERVIEW

FIGURE 1

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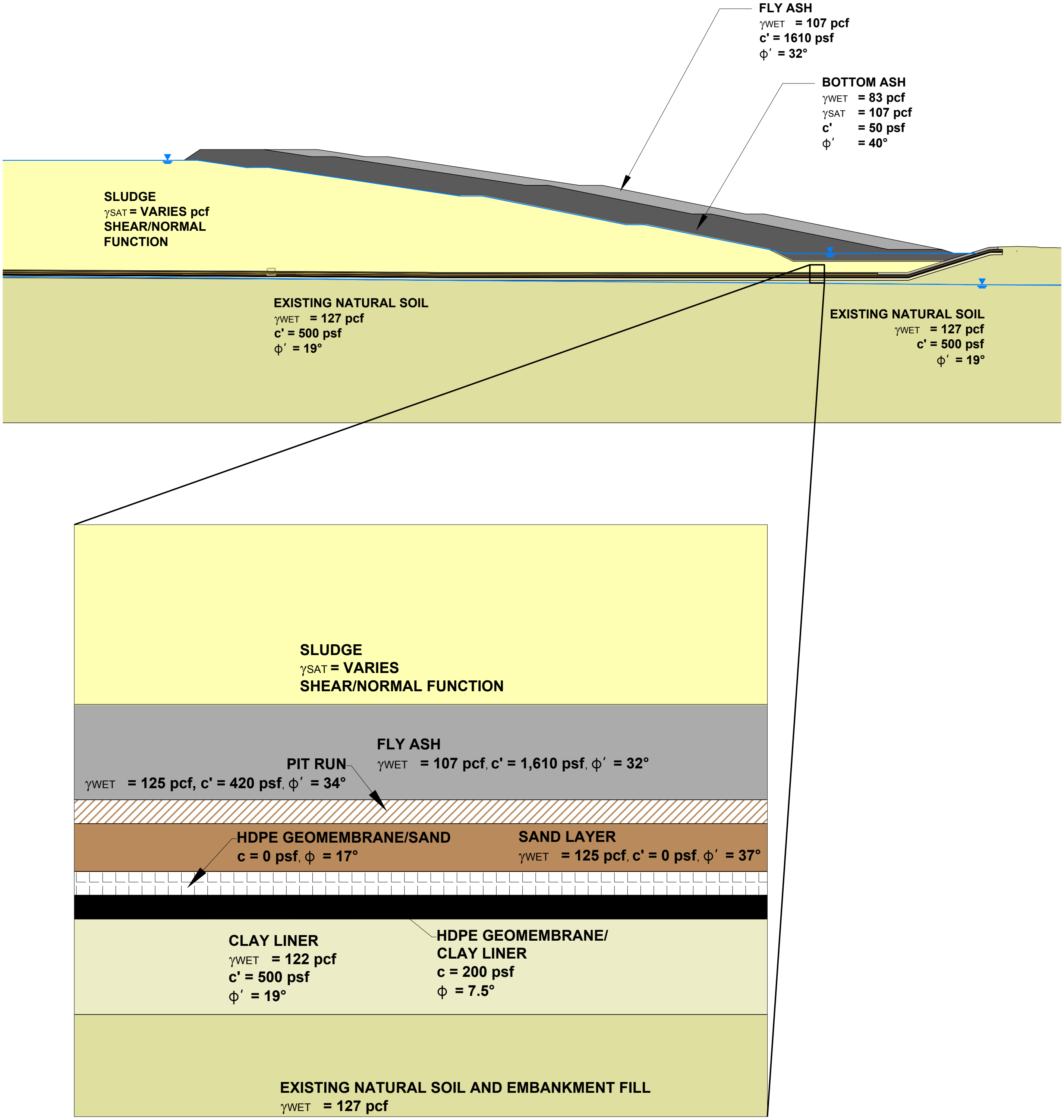

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GREAT RIVER ENERGY
COAL CREEK STATION - UPSTREAM RAISE 91
CRITICAL SECTION

FIGURE 2

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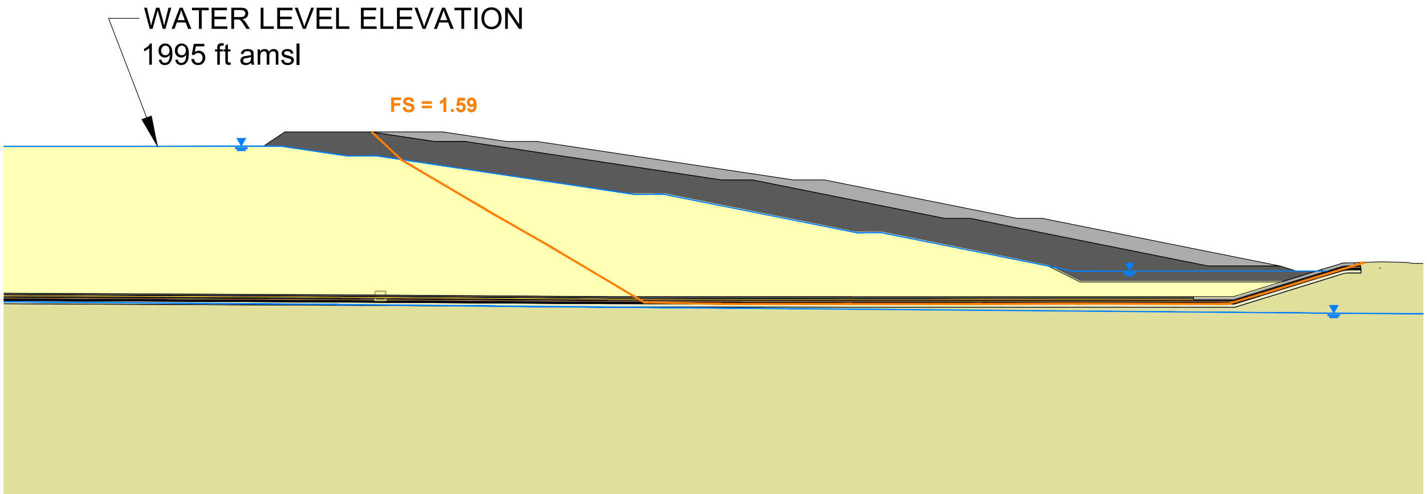
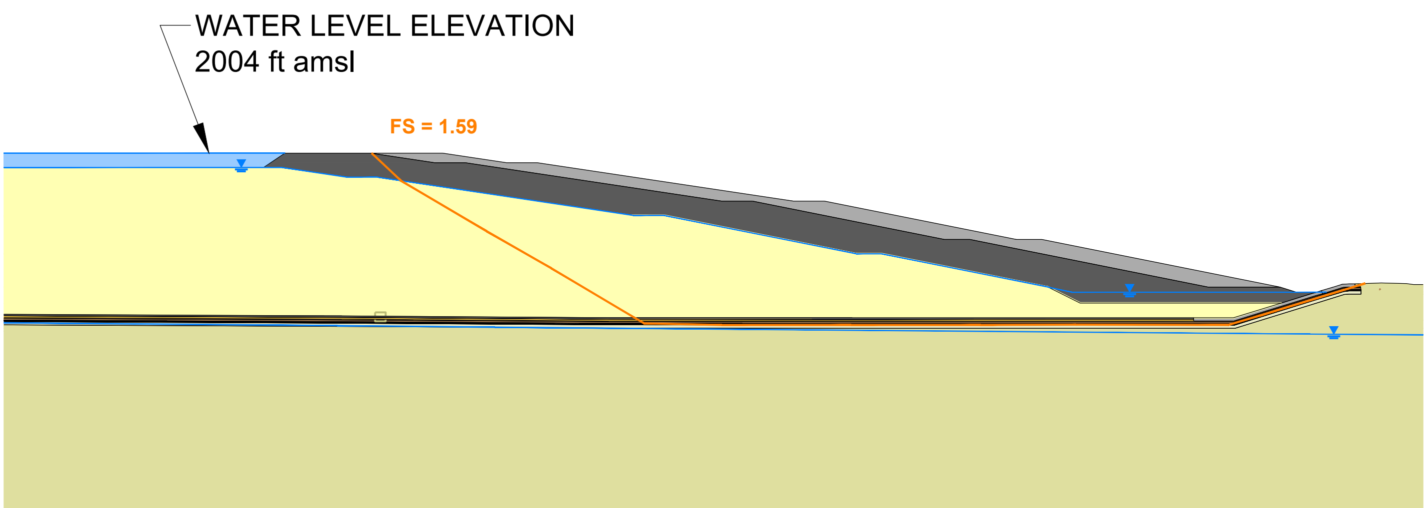


STATIC ANALYSIS
MAXIMUM STORAGE POOL



STATIC ANALYSIS
MAXIMUM SURCHARGE POOL

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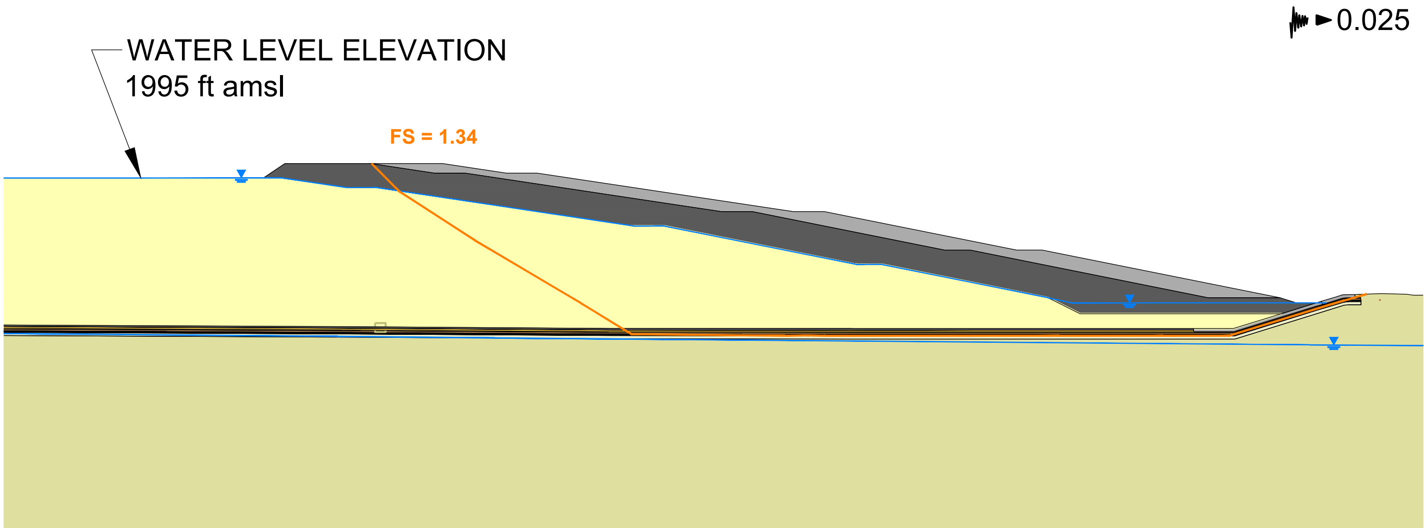
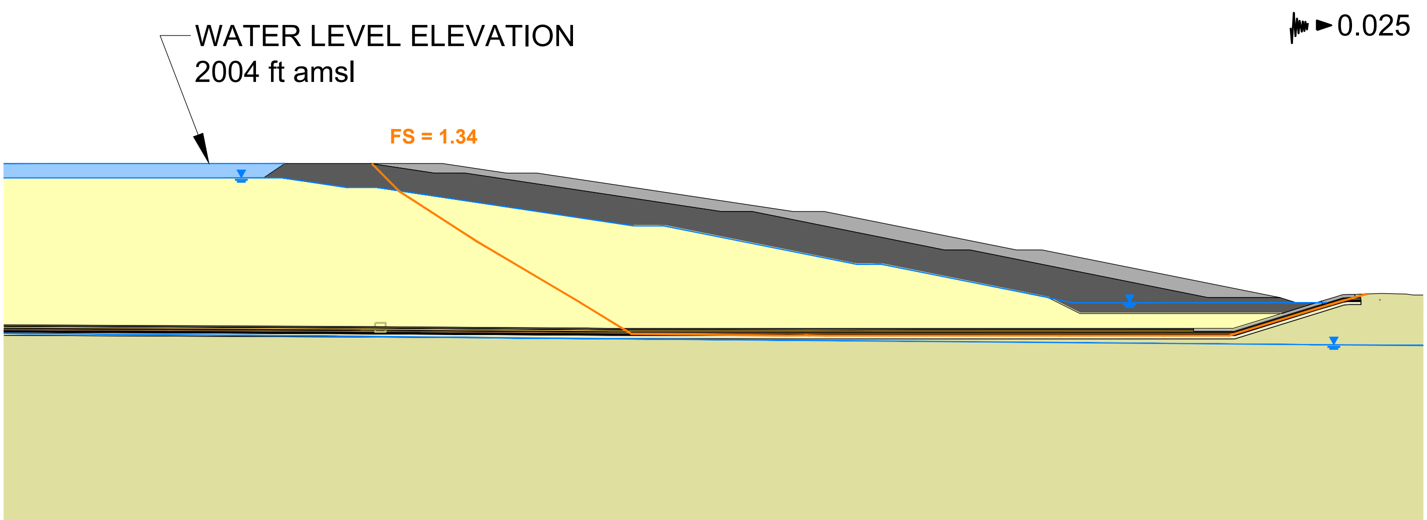
GREAT RIVER ENERGY
COAL CREEK STATION - UPSTREAM RAISE 91
STATIC STABILITY RESULTS

FIGURE 3



PSEUDOSTATIC ANALYSIS
MAXIMUM STORAGE POOL



PSEUDOSTATIC ANALYSIS
MAXIMUM SURCHARGE POOL

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GREAT RIVER ENERGY
COAL CREEK STATION - UPSTREAM RAISE 91
SEISMIC (PSUEDOSTATIC) STABILITY RESULTS

FIGURE 4

APPENDIX A

Material Properties

1.0 OBJECTIVE

Compile a list of material properties used in the engineering evaluation and indicate sources for all inputs.

2.0 MATERIALS

2.1 Existing Natural Soil and Embankment Fill

Existing natural soil and embankment fill properties were based on lab work performed by Golder on samples taken from the SW Section 16 area in 2001, 2008, and 2011. Soils on site are generally low plasticity clay (CL), but high plasticity clays (CH) are also present. Samples yielded an average dry unit weight of 105.1 pcf and an average moisture content of 20.7 %. Values of 105 pcf for the dry unit weight and 21 % for the moisture content were chosen resulting in a moist unit weight of approximately 127 pcf.

Two triaxial shear strength tests were performed from the SW Section 16 Shelby tube samples in 2001 (BH3 and BH6). Three additional triaxial tests were performed on existing natural soil near the River Water Holding Basin in 1981 (boreholes BR-1 and BR-2) and two triaxial tests were performed on existing natural soil near the cooling towers in 2008 (Tower 91 and Tower 92). Based on triaxial test information, a conservative strength envelope was developed for existing natural soil. The strength envelope is shown in Figure 1 and is defined by an effective cohesion of 500 psf and an effective friction angle of 19 degrees.

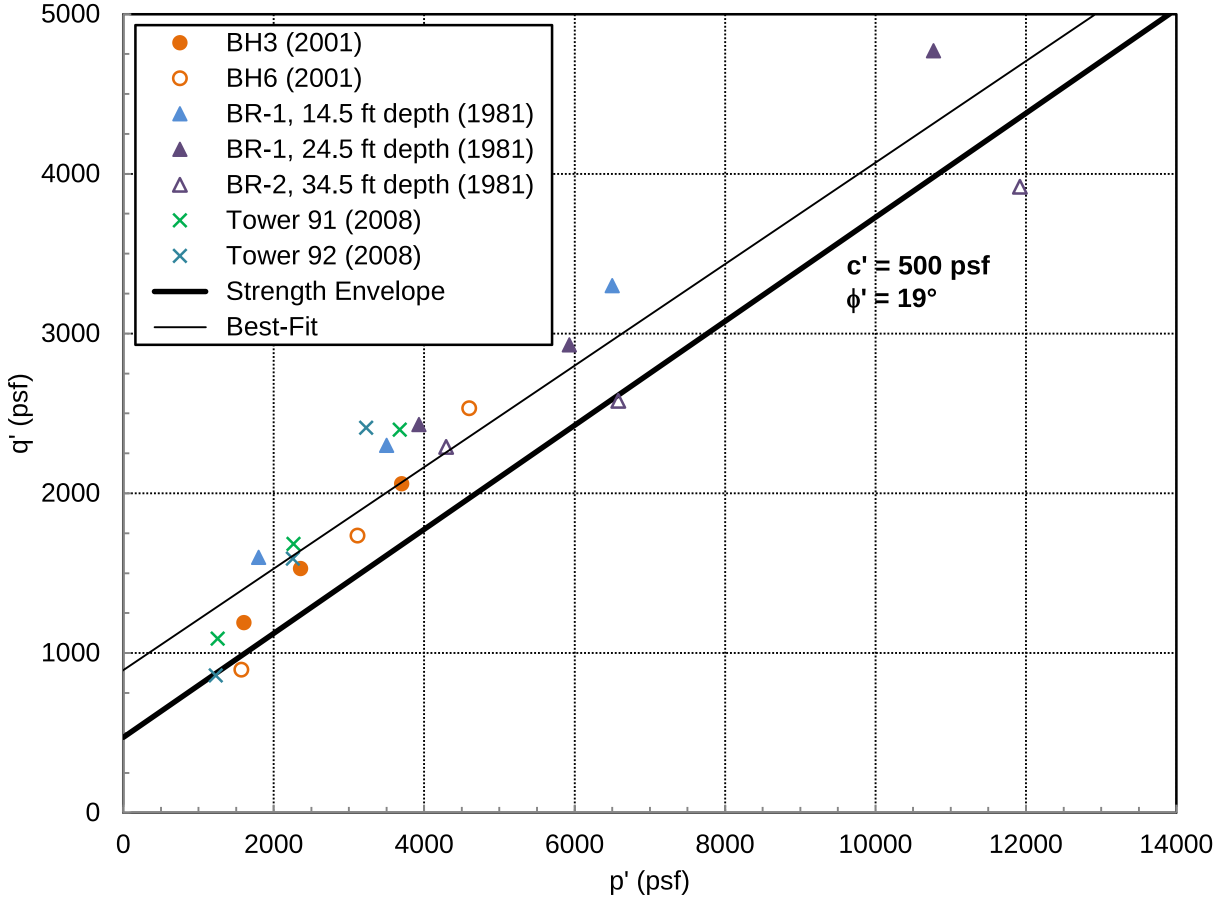


Figure 1: Strength Envelope for Existing Natural Soil Based on Triaxial Test Information

2.2 Clay Liner

Clay liner inputs are based on field experience and laboratory testing of materials at CCS. Dry unit weight and moisture content have been evaluated based on Shelby tube samples of other clay liners at CCS. Results yielded a dry unit weight ranging between 91.9 and 103.8 pcf (99.5 pcf average) and moisture content ranging between 18.6 % and 27. 7% (22.8 % average). Using a dry unit weight of 99.5 pcf and a moisture content of 23 %, a moist unit weight of 122 pcf was calculated and used for these stability analyses. Clay liner materials used at CCS come from existing natural clays onsite. Therefore, shear strength properties of the clay liner are the same as the existing natural soils: the effective cohesion is 500 psf and the effective friction angle is 19 degrees.

2.3 Sand Layer

Sand layer inputs were based on published values for SW and SP type material (NAVFAC 7.02).

Published maximum dry unit weight values range between 100 and 130 pcf (115 pcf average) with optimum moisture contents between 9 % and 21 % (15 % average). Assuming a construction specification of 95 % maximum dry density and optimum moisture, the dry unit weight chosen is 109 pcf with a moisture content of 15 %. This results in a moist unit weight value of approximately 125 pcf.

Published values for effective cohesion of SW and SP material suggest a value of 0 psf. Published values for effective friction angle of SW material suggest a value of 38 degrees. Published values for effective friction angle of SP material suggest a value of 37 degrees. For conservatism, the lower effective friction angle of the SP material was chosen for analyses.

2.4 Pit Run

Pit Run is described as a silty sand and inputs were based on published values for SM type material (NAVFAC 7.02).

Published maximum dry unit weight values range between 110 and 125 pcf (117.5 pcf average), with optimum moisture contents between 11 % and 16 % (13.5 % average). Assuming a construction specification of 95 % maximum dry density and optimum moisture, the dry unit weight chosen is 112 pcf with a moisture content of 13.5 %. This results in a moist unit weight value of approximately 125 pcf.

Strength parameters were based on the published values of 420 psf for effective cohesion and 34 degrees for effective friction angle (NAVFAC 7.02).

2.5 FGD Sludge Waste

FGD sludge waste input parameters are based on published data, field testing, design calculations, and lab work performed by Golder between 2001 and 2014.

Nine laboratory tests between 2002 and 2012 indicate an average specific gravity (G_s) of 2.7. Consolidation analyses indicate an average dry unit weight of about 45 pcf at the end of FGD sludge deposition, and an average dry unit weight of approximately 53 pcf after closure. Field sampling of saturated FGD sludge in test pits and from Shelby tubes indicate dry unit weights between 27 and 60 pcf. Assuming the FGD sludge is fully saturated during the active life of the facility, the saturated unit weight ranges between approximately 80 pcf at initial deposition and during intermediate deposition and 100 pcf at final deposition heights. Saturated unit weights are based on moisture contents as low as 65 % after closure to more than 100 % during deposition. Saturated unit weights used in analyses were based on consolidation analyses and generally vary between 89 pcf and 98 pcf depending on material depth. Saturated unit weight as a function of depth is shown in the following table for the end of sludge deposition and after final cover has been placed:

Saturated Unit Weight vs. Height of Sludge

	Saturated Unit Weight	
Average Depth (ft)	**End of Sludge Deposition (pcf)**	**Final Cover Placed (pcf)**
>75	95	98
54-75	94	97
30-54	92	96
0-30	89	96

Consolidated-undrained triaxial lab testing was used to evaluate the strength of FGD sludge. Triaxial tests were performed on seventeen samples of FGD material between 2010 and 2014 at normal stresses between 1,000 and 10,000 psf on both remolded samples (2010 and 2012 samples) and intact Shelby tube samples (2014 samples). Golder conservatively developed the shear strength envelope based on triaxial tests to approximate the lower bound of shear strength test results (Figure 3). The strength envelope chosen for use in stability analyses is given in the following table:

Normal Stress (psf)	**Shear Strength (psf)**
0	270
5,500	1,500
10,000	2,500

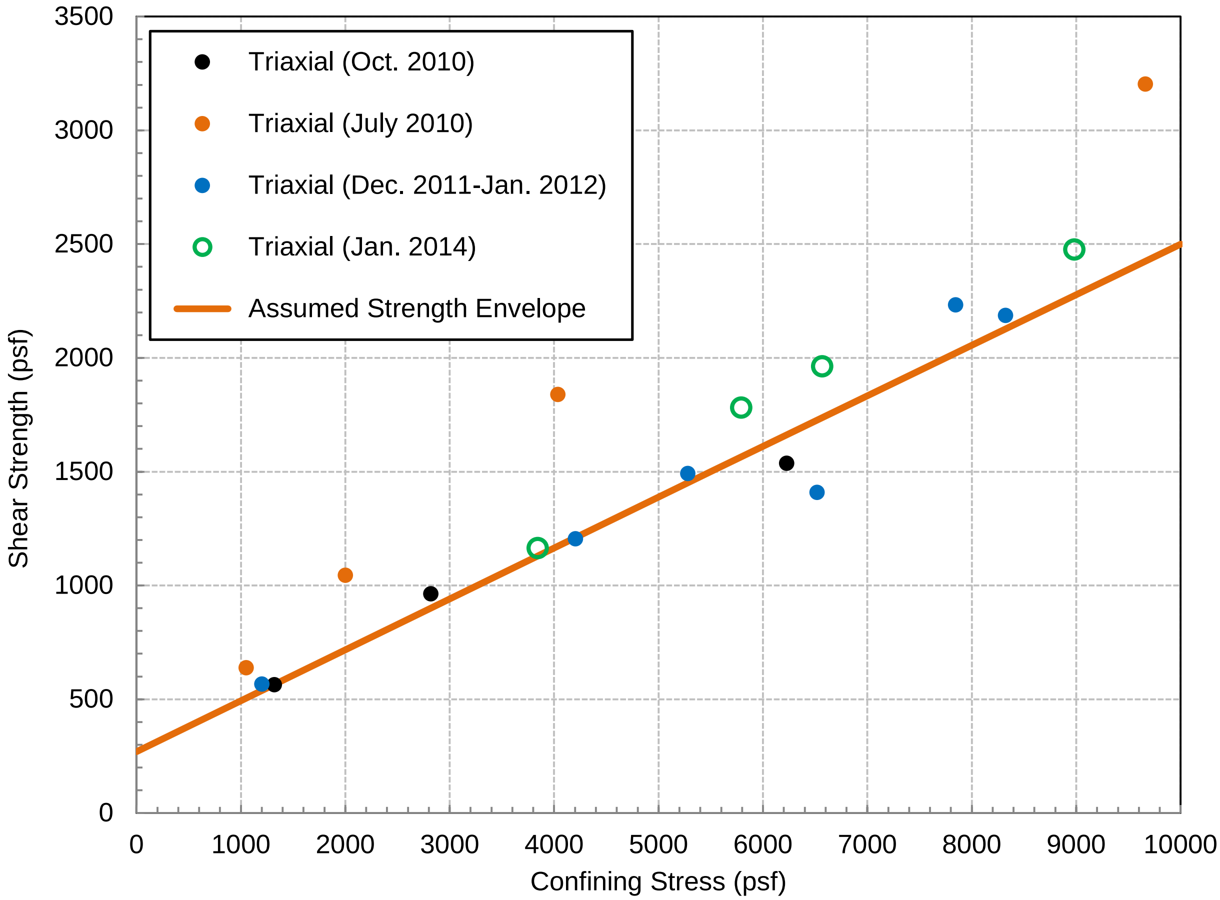


Figure 2: Shear Strength Tests Performed on FGD Sludge

2.6 Bottom Ash

Bottom Ash input parameters are based on lab and field work performed by Golder.

The dry unit weight for compacted bottom ash is based on 95 % standard Proctor densities from lab testing which gives a value of approximately 81 pcf. The dry unit weight of sluiced bottom ash is 60 pcf. An average value of 70 pcf was chosen for analysis based on a combination of compacted material and loosely placed material. The moisture content from field sampling of drained and saturated bottom ash ranged between 12 % and 61 %. For unsaturated conditions, a moisture content of 19 % was assumed. Using the lab measured specific gravity of bottom ash (2.60); the moisture content of bottom ash for saturated conditions was determined to be between 40 % and 65 % (average 53 %). Bottom ash was assigned an average moist unit weight of 83 pcf and an average saturated unit weight of 107 pcf.

Lab direct shear strength testing of bottom ash indicated a residual effective cohesion of 463 psf and a residual friction angle of 40.3 degrees. Visual observations of the bottom ash material indicates little cohesion, therefore the effective cohesion was chosen as 50 psf and an effective friction angle of 40 degrees was chosen for analysis.

2.7 Fly Ash

Fly Ash input parameters are based on triaxial tests performed by Golder for a 75 % solids paste mix and unconfined compression tests performed on fly ash samples prepared using standard Proctor compaction techniques.

Dry unit weights from triaxial tests ranged between 87.8 pcf and 94.5 pcf with an average value of 91.9 pcf; a value of 92 pcf was chosen. Moisture contents from the same testing ranged between 6.3 % and 27.7 % with an average value of 16 %; a value of 16% was chosen. These values result in a moist unit weight of 107 pcf.

Triaxial tests were performed on the fly ash to evaluate the shear strength. Samples were allowed to “cure” for between 28 and 56 days prior to testing to evaluate long-term behavior. The results of the test provided an average cohesion of 1610 psf with a friction angle of 32 degrees. These properties were used in the stability analyses.

2.8 Geosynthetic Interfaces

Geomembrane Interface inputs are based on lab work performed by Golder and published values (GRI 2005). The interfaces of interest are:

- Smooth HDPE against clay liner.
- Smooth HDPE against sand or bottom ash.

Geosynthetic interface shear strengths are based on interface shear test results and consider both peak and residual shear strengths. Two large-scale direct shear interface friction tests were performed between a 40-mil smooth high density polyethylene (HDPE) liner and site specific clays representative of those used in liner construction. The HDPE liner used in testing was excavated from site facilities and is similar to what was used during construction of Ash Pond 91. Results of the two tests indicate a friction angle of about 7.5 degrees and a residual adhesion intercept of approximately 200 psf for smooth HDPE against clay liner (Figure 2).

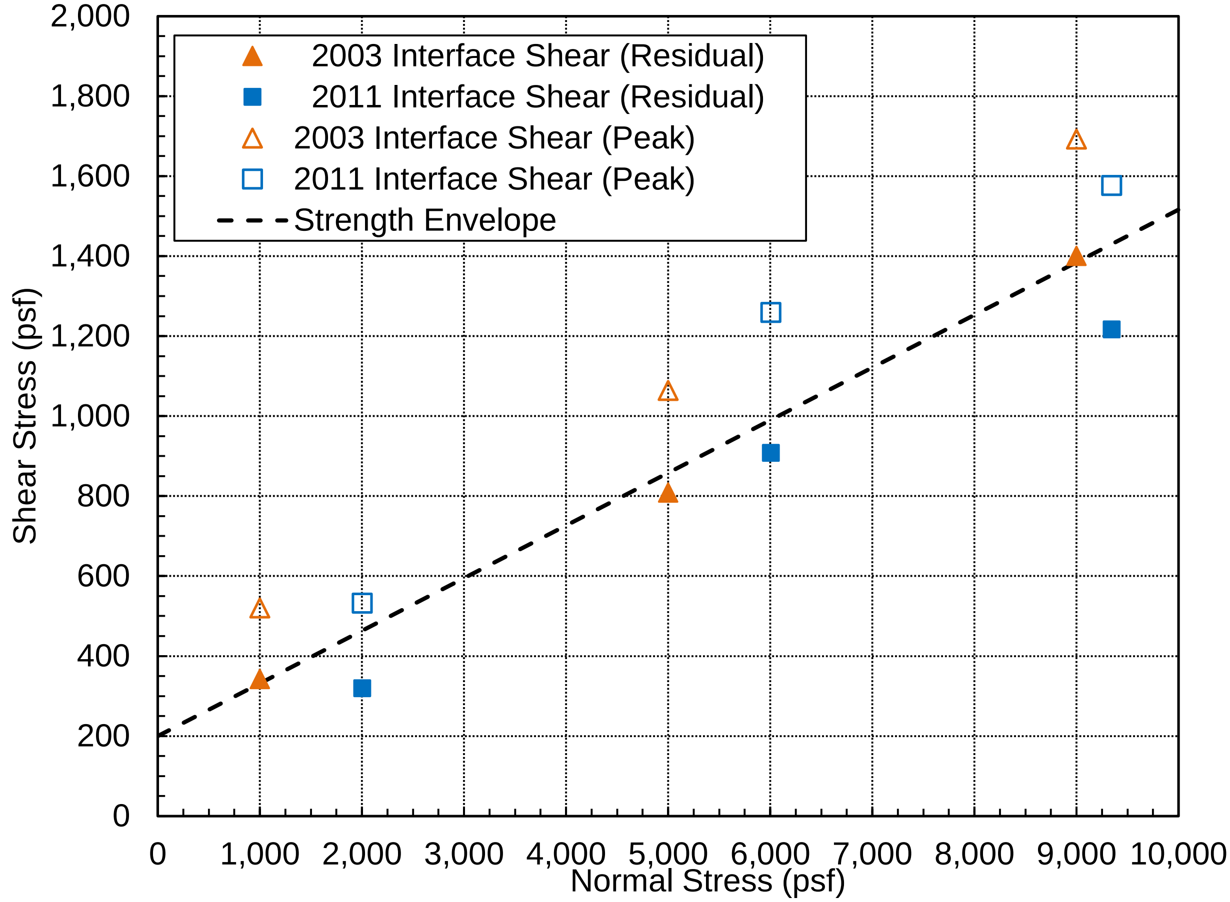


Figure 3: Interface Shear Tests for Smooth HDPE against Site-Specific Clay

Golder lab experience for smooth HDPE against sand indicates a residual friction angle between 13.4 and 20 degrees (average of 16.7 degrees) and a residual adhesion intercept between 0 and 72 psf (average of 36 psf). The Geosynthetic Research Institute (GRI 2005) published a residual friction angle of 17 degrees and a residual adhesion intercept of 0 psf for smooth HDPE against granular soil based on 128 tests. Based on Golder lab experience and GRI (2005), a friction angle of 17 degrees with no adhesion intercept was chosen for use in engineering analysis.

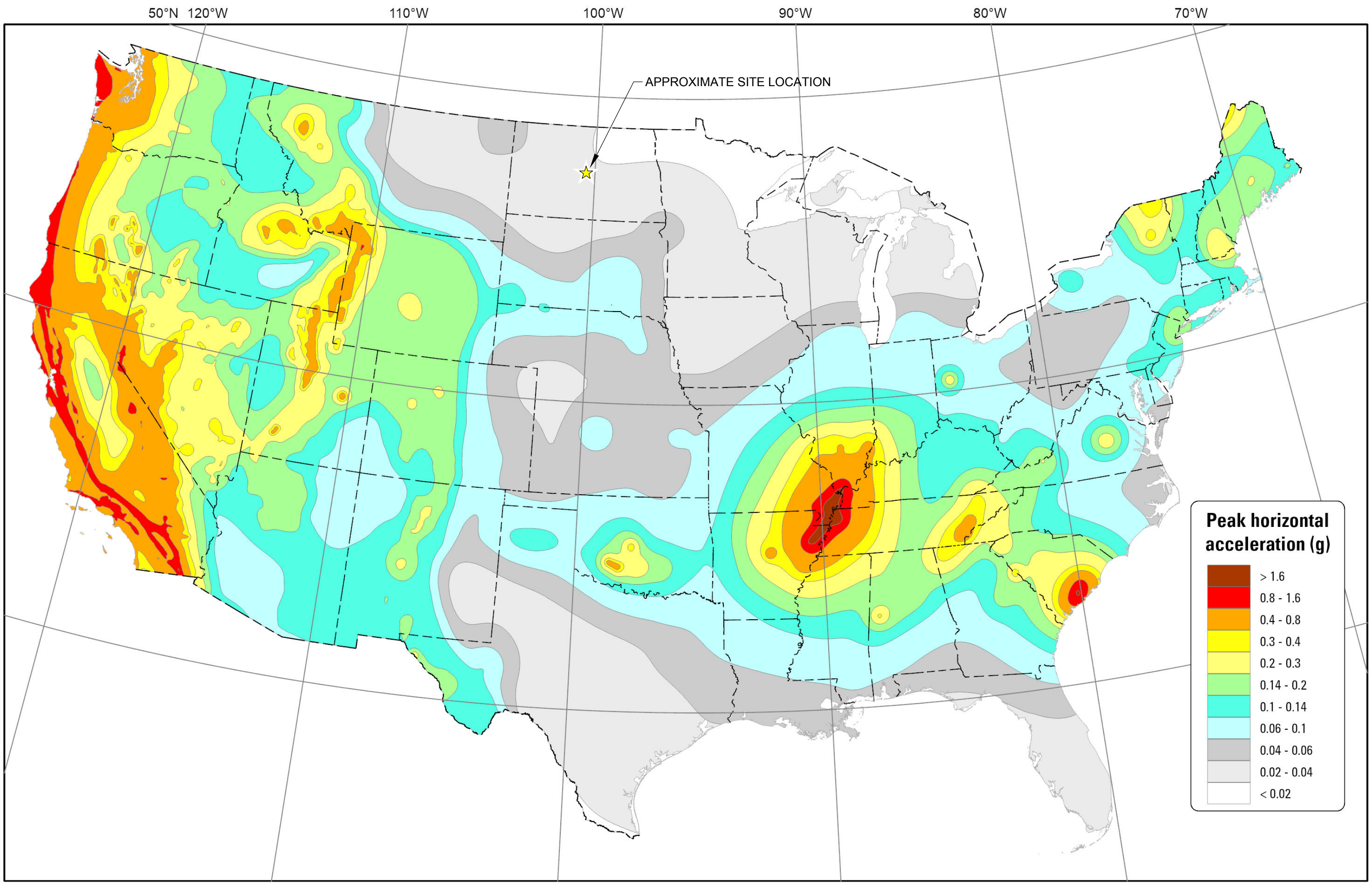
3.0 REFERENCES

Foundations and Earth Structures, Design Manual 7.02, Naval Facilities Engineering Command, September 1996 (NAVFAC 7.02).

Koerner, G.R. and Narejo, D. (2005). *Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces, Geosynthetic Research Institute Report #30.* June 14 2005 (GRI 2005).

APPENDIX B

Seismic Loading Conditions



2018 National Seismic Hazard Model for the conterminous United States
Peak horizontal acceleration
with a 2% probability of exceedance in 50 years
NEHRP site class B/C (V_{s30} = 760 m/s)

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2% PROBABILITY OF EXCEEDANCE IN 50 YEARS MAP OF PEAK GROUND ACCELERATION
USGS SEISMIC HAZARD MAP (2018)

FIGURE B-1

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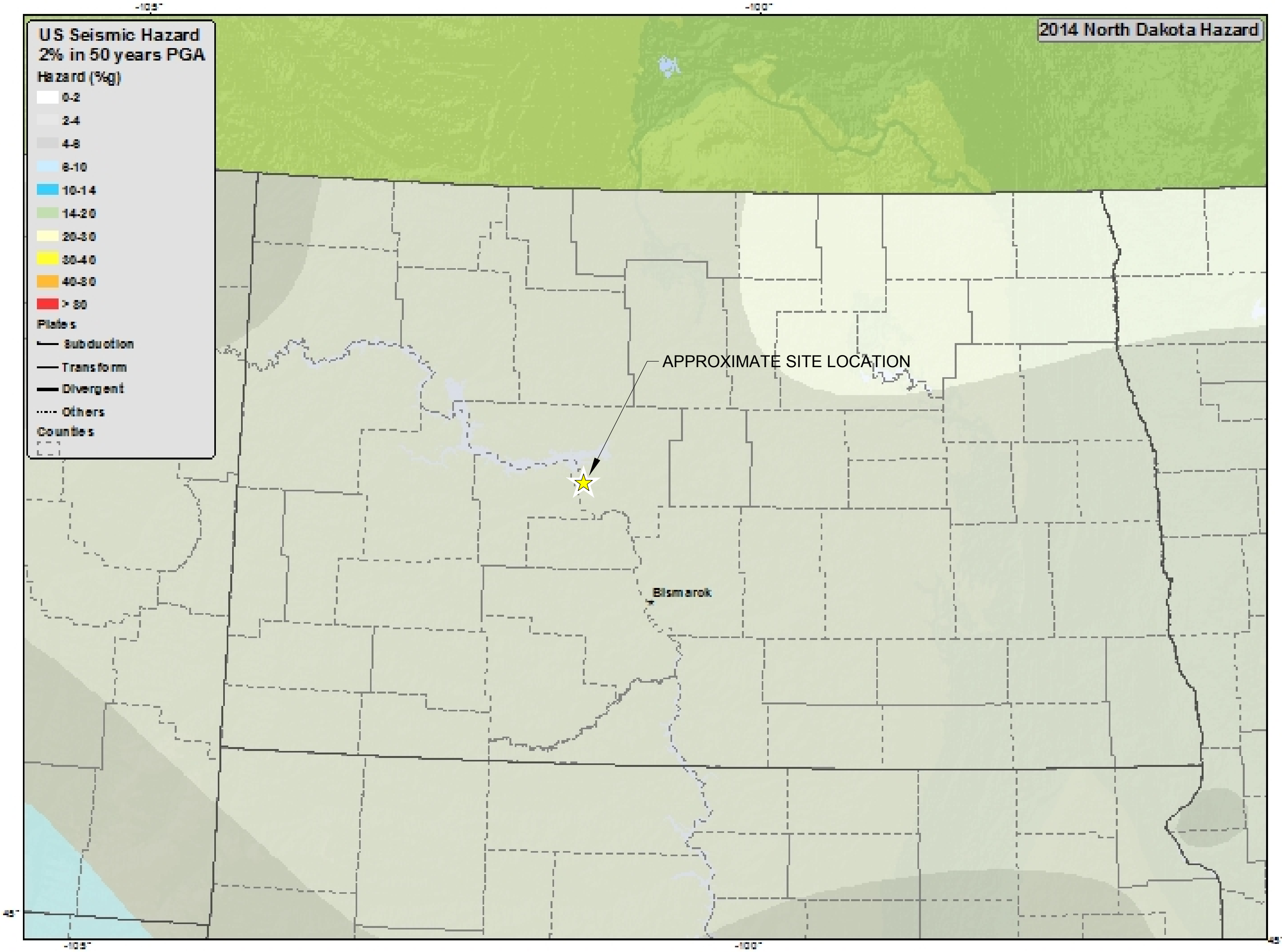

2% PROBABILITY OF EXCEEDANCE IN 50 YEARS MAP OF PEAK GROUND ACCELERATION
USGS SEISMIC HAZARD MAP (2014)

FIGURE B-2

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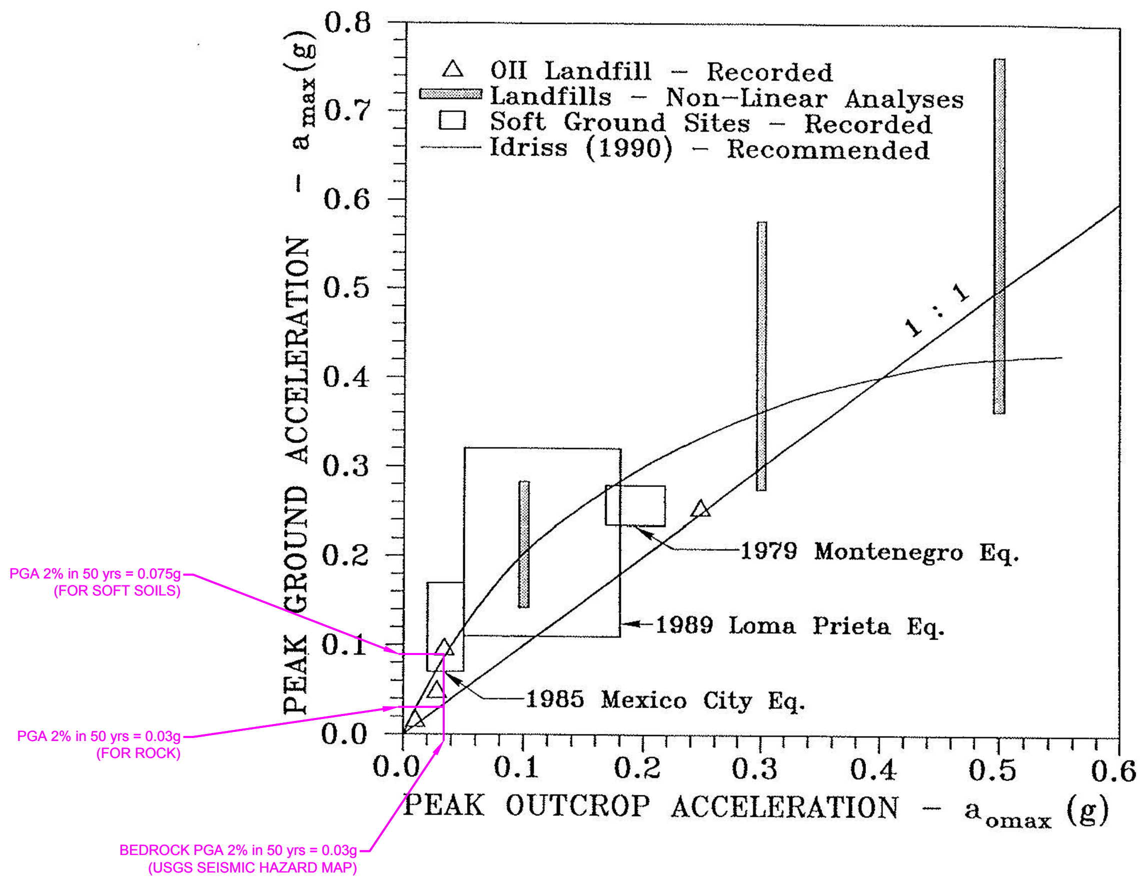

Figure 4.5 Observed Variations of Peak Horizontal Accelerations on Soft Soil and MSW Sites in Comparison to Rock Sites (Kavazanjian and Matasović, 1994).

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COMPARISON OF PEAK GROUND ACCELERATION AND PEAK OUTCROP ACCELERATION

FIGURE B-3

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