HAZARD POTENTIAL CLASSIFICATION, STRUCTURAL STABILITY, AND SAFETY FACTOR ASSESSMENTS

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Bottom Ash CCR Surface Impoundment
Stanton Station
Great River Energy

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Table of Contents

List of Tables

List of Figures

List of Appendices

1.0 INTRODUCTION

1.1 Purpose

The Environmental Protection Agency's (EPA's) Coal Combustion Residual (CCR) Rule, 40 Code of Federal Regulations (CFR) Part 257, promulgated April 17, 2015 and effective October 19, 2015, requires that existing CCR surface impoundments meeting the requirements of §257.73(a) and §257.73(b) conduct initial and periodic:

- Hazard potential classification assessments in accordance with §257.73(a)(2),
- Structural stability assessments in accordance with §257.73(d), and
- Safety factor assessments in accordance with §257.73(e).

This report documents the hazard potential classification assessment, structural stability assessment, and safety factor assessment for Great River Energy's (GRE's) Stanton Station (SS) Bottom Ash CCR Surface Impoundment (Bottom Ash Impoundment).

1.2 Site Background

SS is located in Sections 16 and 21, Township 144N, Range 84W of Mercer County, approximately three miles southeast of Stanton, North Dakota. The Bottom Ash Impoundment is composed of three composite-lined cells (north, south, and center cells). The north and south cells are active cells used to temporarily store and dewater bottom ash and the center cell functions as a retention cell. Bottom ash is sluiced into one of the active cells until the cell reaches capacity. Once capacity is reached, bottom ash deposition is directed to the other active cell and the filled cell is dewatered by decanting water through the outlet structure to the center cell. Bottom ash remaining in the filled cell is excavated and hauled to the adjacent Bottom Ash CCR Landfill for disposal.

Materials permitted for deposition in the Bottom Ash Impoundment include hydraulically conveyed bottom ash, water from the plant storm water retention pond, water from the coal unloading pit sump, mineralizer reject water, boiler blowdown and overflow water, and water from miscellaneous plant drains.

1.3 Geological Conditions

Stanton Station is located in the Missouri Slope district of the glaciated Missouri Plateau of the Great Plains physiographic province (NDDH 2005). The Bottom Ash Impoundment is constructed in Missouri River alluvial deposits. The alluvial deposits have two distinct subunits: upper and lower. The upper subunit consists of a silty sand and clay and the lower subunit is an outwash sand and gravel (Barr 2010).

1.4 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 Bottom Ash Impoundment was issued Construction Permit 918 in September 1994.

The North Dakota Department of Health (NDDH) Division of Waste Management is the environmental regulatory body for the CCR facilities at SS. The three Bottom Ash Impoundment cells are permitted as a surface water impoundment under NDDH permit SP-043.

1.5 Previous Evaluations

The following evaluations were previously performed on the Bottom Ash Impoundments at SS:

- Golder Associates Inc., Stability Evaluation of Bottom Ash Surface Impoundment Report, Great River Energy Stanton Station, dated May 16, 2011 (Golder 2011a)
- Golder Associates Inc., Stability Evaluation of Bottom Ash Surface Impoundment Addendum Letter, Great River Energy Stanton Station, dated December 22, 2011 (Golder 2011b)
- Golder Associates Inc., Seismic Stability Evaluation Addendum to Stability Evaluation of Bottom Ash Surface Impoundment Addendum, Great River Energy Stanton Station, dated March 16, 2012 (Golder 2011b)
- Kleinfelder, Coal Ash Impoundment Site Assessment Final Report, Stanton Station, October 26, 2012 (Kleinfelder 2012)

In this study, 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 §257.73(e)(1)(i)-(iv).

2.0 HAZARD POTENTIAL CLASSIFICATION ASSESSMENT – §257.73(A)(2)

The CCR rules require conducting initial and periodic hazard potential classification assessments by a qualified professional engineer to document the hazard potential classification of CCR surface impoundments and the basis for the classifications. Hazard potential classifications include the following listed in §257.73(a)(2):

- High hazard potential CCR surface impoundment
- Significant hazard potential CCR surface impoundment
- Low hazard potential CCR surface impoundment

These hazard classifications are defined under §257.53 as:

> **High hazard potential CCR surface impoundment** – means a diked surface impoundment where failure or mis-operation will probably cause loss of human life.
>
> **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.
>
> **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.

2.1 Hazard Potential Classification Assessment

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

- The impoundment volume 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 cells is unlikely to have a significant environmental impact and is not likely to leave the owner's property.
- The economic impacts associated with a failure will primarily be to the owner's property. A failure could potentially disrupt rail service that provides coal to the power generation facility.
- An EPA consultant performed a site assessment of Stanton Station in 2011 under EPA supervision. Following the EPA dam hazard classification system, the Bottom Ash Surface Impoundment was given a "Less Than Low Hazard" classification (Kleinfelder 2012).

2.2 Emergency Action Plan – §257.73(a)(3)

The CCR 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 impoundment. The Bottom Ash Impoundment has been categorized as a low hazard potential CCR surface impoundment and no EAP is required.

3.0 STRUCTURAL STABILITY ASSESSMENT – §257.73(D)

The CCR rules (§257.73(d)(1)) require conducting initial and periodic structural stability assessments 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."

3.1 Foundation – §257.73(d)(1)(i)

The foundation soils of the Bottom Ash Impoundment consist of native soils (silty sand and clay) and some clay fill. Geologic and hydrogeologic information about Stanton Station was first presented in the *Reports of Soil Explorations for the Steam Generator Addition* and *for the Supplemental Steam Generator*, prepared by the Soil Exploration Company and dated October 14, 1977 and May 10, 1978, respectively.

The locations of the historic ash disposal ponds (i.e., Ash Ponds A, B, and C) and the closed special waste landfill were later characterized by Braun Intertec in the report titled *Hydrogeological Assessment – Stanton Station Ash Ponds* (Braun 1993). The Bottom Ash Impoundment is located within the historic limits of Ash Pond A.

According to the *Hydrogeological Assessment*, the Stanton Station site is located in the Missouri Slope district of the glaciated Missouri Plateau section of the Great Plains physiographic province. The Bottom Ash Surface Impoundment is constructed in Missouri River alluvial deposits. The alluvial deposits have two distinct subunits: upper and lower. The upper subunit consists of a silty sand and clay and the lower subunit is an outwash sand and gravel (Barr 2011). The natural soils are described as predominantly silty sands (SM) with layers of fat clay (CH).

Based on historic site information and observations, the Bottom Ash Impoundment is not built over wet ash or other unsuitable materials, and the foundation soils are stable.

3.2 Slope Protection – §257.73(d)(1)(ii)

The downstream slope of the Bottom Ash Impoundment embankments 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. Based on site experience and observations, the high-density polyethylene (HDPE) liner on the upstream slopes and two feet of soil protective cover on the bottom of each cell provides sufficient protection from deterioration due to wave action.

3.3 Dikes (Embankment) – §257.73(d)(1)(iii)

The Bottom Ash Impoundment perimeter embankments and the two interior embankments have a top elevation of 1720 feet. The bottom elevation of the cells varies between 1700 and 1704 feet, based on original construction drawings. The perimeter embankments along the north, east, and south sides of the impoundment consist of an historic embankment to elevation 1715 feet, with an embankment extension to elevation 1720 feet constructed from 1994 through 1995. The west perimeter embankment and two interior embankments were constructed in 1994 and 1995. The upstream and downstream slopes of the embankments are 3H:1V.

Historic embankment fill properties are from test boring results from September 1993 (Stone & Webster 1993). The historic embankment fill materials were classified as clean to silty sands (SP, SM) with some layers of lean to fat clays (CL, CH) and some silts (ML). Test borings were advanced through the historic embankment and laboratory analyses were performed on eight soil samples to determine dry density, moisture content, Atterberg Limits, specific gravity, and gradation. Dry unit weight values ranged from 104 to 121 pounds per cubic foot (pcf) with moisture contents between 12% and 28% (Stone & Webster 1993).

Additional embankment slopes were constructed from clayey soil from the Glenharold Mine site. Construction testing of the new embankment fill placed between 1994 and 1996 indicate that the material is predominantly fat clay (CH) with some lean clay (CL) (UPA 1996). In-Situ dry density of the constructed embankment ranged between 94 and 107 pcf with an average of 99 pcf. The in-situ moisture content of the constructed embankment ranged between 18% and 27% with an average of 23%. The moist unit weight from these averages is approximately 120 pcf.

Standard Proctor testing of site soils during construction indicated maximum dry densities of between 91 and 115 pcf, with optimum moisture contents of between 14% and 28% (UPA 1996). Based on this information, both historic and new embankment fill materials appear to have been compacted to densities sufficient for loading conditions expected at the impoundment cells.

3.4 Vegetated Slopes – §257.73(d)(1)(iv)

The Bottom Ash Impoundment 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 the bottom ash impoundment cells 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 of impoundments.

3.5 Spillways – §257.73(d)(1)(v)

There are no spillways associated with the Bottom Ash Impoundment. Existing controls are in place to monitor water levels in the Bottom Ash Impoundment and limit potential overtopping of the impoundment. Flow between cells is managed via weir discharge structures with removable stop logs, preventing the active cells from reaching water levels above elevation 1718 feet. A weir discharge structure with removable stop logs at the east end of the center cell is set so that the cell continuously decants water to a discharge pipe with a maximum water level set at approximately 1717.5 feet. The design crest of the soil embankments surrounding the Bottom Ash Impoundment are approximately 15 to 20 feet above surrounding topography (elevation 1720 feet), preventing stormwater run-on into the Bottom Ash Impoundment.

Existing controls in place to monitor the water levels in the Bottom Ash Impoundment include weekly observations of water levels by SS personnel, and daily observations by SS operations personnel.

Additional observations are noted by GRE employees familiar with site CCR units. After large storm events, SS 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 the north or south cells reach the maximum design elevation (1718 feet), GRE has operating procedures to switch the piped inflows to another cell for short-term water management or adjust the level of water within the cells using the stop logs in the concrete weir structures.

Bottom ash impoundment cells are operated with a minimum freeboard of between 2 feet and do not have substantial precipitation capture area beyond the cell footprint areas since they are designed above grade. Therefore, each bottom ash impoundment cell can adequately contain the precipitation associated with the 24-hour, 100-year storm (4.95 inches) event (assuming 100% of precipitation is captured) with at least 1.6 feet of additional capacity prior to overtopping.

3.6 Hydraulic Structures – §257.73(d)(1)(vi)

Hydraulically conveyed bottom ash and other water streams enter either the north cell or south cell of the Bottom Ash Impoundment via two polyvinyl chloride (PVC) pipes (Retention Pond Inlet Pipe and Coal Pit Sump Inlet Pipe) and two steel pipes (Bottom Ash Pipes) through the eastern embankment near the northeast corner of each cell.

Water from the north cell is routed to the center cell via a concrete outfall structure and 36-inch diameter reinforced concrete pipe (RCP) through the interior embankment between the north cell and center cell (near the west end of the embankment). Water from the south cell is routed to the center cell via a concrete outfall structure and 36-inch diameter RCP through the interior embankment between the south cell and center cell (near the west end of the embankment). Water is discharged from the center cell via a concrete outfall structure and 18-inch diameter piping through the eastern embankment (approximately at the midpoint between the northeast and southeast corners of the center cell).

No significant deterioration, deformation, distortion, bedding deficiencies, sedimentation, or debris were observed that may negatively affect the operation of the hydraulic structures.

3.7 Downstream Slopes Adjacent to Water Body – §257.73(d)(1)(vii)

There are no water bodies adjacent to the downstream slopes of the Bottom Ash Impoundment. The structural stability of the downstream slope is not impacted by the presence or sudden drawdown of this water.

3.8 Structural Stability Deficiencies – §257.73(d)(2)

No structural stability deficiencies were identified during this assessment.

4.0 SAFETY FACTOR ASSESSMENT – §257.73(E)

The CCR rules require that safety factors need 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 v7.0 (Rocscience 2016). Factors of safety were computed for circular failure surfaces using Spencer's method for force and moment equilibrium. 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

A critical cross section for the Bottom Ash Impoundment 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 the Bottom Ash Impoundment is located across the south embankment (Figure 1). The south embankment was surveyed in October 2014 (Interstate 2014) and has a crest at an elevation of 1720 feet, with 3H:1V downstream slopes to approximately 1700 feet (Figure 2).

4.3 Material Properties

Soil properties were calculated based on geotechnical materials testing performed on the soil samples collected during the installation of the piezometers P-1 and P-2 on October 4, 2011. Boring logs and lab analyses are provided in Appendix A. The piezometer borings were advanced from the embankment crest at elevation 1720 feet, through the base of the historic embankment. Soils were tested for moisture, density, grain-size distribution, and shear strength. The material properties for each soil type included in the static and pseudostatic stability analyses of the Bottom Ash Impoundment are provided in Table 1. Additional details regarding the development of these material properties can be found in subsequent sections. Results of the lab analyses used in the development of these material properties are provided in Appendix A.

Table 1 Material Properties

Material	Moisture/Density					Shear Strength			
						Static		Pseudostatic	
	γdry	ω	γwet	ω (sat)	γsat	ϕ'/δ	c'/a	ϕ/δ	c/a
	pcf	%	pcf	%	pcf	degrees	psf	degrees	psf
Historic Embankment Fill – Silt	112.0	11.0	124	19.0	133	30.0	100	14.0	630
Natural Soil – Silt/Clay	104.0	-	-	19.0	124	30.0	100	Sheer/Normal Fx	
New Embankment Fill – Clay from Glenharold Mine	100.0	23.0	123	28.0	128	30.0	200	15.5	470
Compacted Fill – Clay from Glenharold Mine	100.0	23.0	123	28.0	128	30.0	200	15.5	470
Smooth HDPE/Clay	NA	NA	NA	NA	NA	7.5	190	7.5	190
Protective Cover – Clay from Glenharold Mine	100.0	23.0	123	28.0	128	30.0	200	15.5	470

4.3.1 Static Material Properties

4.3.1.1 Historic Embankment Fill

Historic embankment fill properties were developed from soil samples collected from borings done for installation of the piezometers P-1 and P-2 drilled in October 2011 (see Appendix A). The historic embankment fill was classified as a silty sand (SM). Piezometer borings were advanced through the historic embankment and laboratory analyses were performed on three soil samples to determine dry density, five soil samples to determine moisture content, and two soil samples to determine plasticity and gradation.

Calculated dry unit weight values were 108.8, 111.0, and 114.9 pounds per cubic foot (pcf) with moisture contents varying between 9.0% and 13.6%. The average dry unit weight of 112 pcf and the average moisture content of 11% are used in the stability analysis to account for the material variability in the historic embankment fill. This results in a moist unit weight of 124 pcf.

The saturated unit weight was determined by applying the saturated moisture content of the silty sand from the natural soil. The natural soils were assumed to be saturated because the soil was observed near or below the water table. The saturated unit weight of the historic embankment fill is 133 pcf when using the saturated water content of 19% from the natural soil.

The predominant soil in the embankment fill is silty sand (SM). A three-point triaxial shear test was performed to determine the effective stress shear strength parameters. An effective stress friction angle of 30 degrees and an effective cohesion intercept of 100 pounds per square foot (psf) were selected for this analysis.

4.3.1.2 Natural Soil

The natural soils are predominantly silty sands (SM) with layers of fat clay (CH). The dry unit weight of the natural soil is 104 pcf based on the dry unit weight of a silty sand layer collected 22 feet below ground surface (bgs) from the boring for Piezometer P-1 (see Appendix A). The moist unit weight of 124 pcf was determined by averaging the moisture content of five silty sand samples collected from the borings for piezometers P-1 and P-2. The average moisture content was calculated as 19%.

The friction angle and cohesion were also based on the silty sand from the historic embankment fill described in Section 4.3.1.1.

4.3.1.3 New Embankment Fill

The new embankment was constructed from clayey soil from the Glenharold Mine site. The material is predominantly fat clay (CH). The dry density of the constructed embankment ranges between 94 and 108.8 pcf with an average of approximately 100 pcf. The moisture content of the constructed embankment ranges between 20.4% and 26% with an average of 23%. The calculated moist unit weight from these averages is approximately 123 pcf.

The saturated unit weight of the new embankment fill is based on the saturated water content of the Glenharold Mine clay from hydraulic conductivity tests performed in 2014. Two soil samples of the Glenharold Mine clay had saturated moisture contents of approximately 28%. Based on this moisture content, the saturated unit weight is 128 pcf.

Soil strength values are based on effective stress shear strength parameters determined from a three-point triaxial shear test performed on the clay collected from the new embankment fill in a Shelby tube in 2011. An effective stress friction angle of 30 degrees and an effective cohesion intercept of 200 psf were selected for this analysis.

4.3.1.4 Compacted Fill

The compacted fill was assumed to be taken from the Glenharold Mine site. Therefore, the compacted fill is assumed to be fat clay and has the same material properties as the new embankment fill material described above.

4.3.1.5 Geomembrane

Geomembrane interface inputs are based on Golder's previous experience working with similar types of material and published values. The interfaces of interest are a smooth HDPE against compacted fill (Glenharold Mine clay) and smooth HDPE against bottom ash. The geomembrane/clay interface is more critical than the geomembrane/bottom ash interface; therefore, the geomembrane/clay interface was used in this stability analysis. Golder conservatively assumed an interface residual friction angle of 7.5 degrees and a residual adhesion intercept of approximately 190 psf for the stability analysis.

4.3.1.6 Protective Cover

The protective cover was assumed to be fat clay taken from the Glenharold Mine site; therefore, it is assumed to have the same material properties as the new embankment fill.

4.3.2 Pseudostatic Material Properties

The material properties for each soil type included in the pseudostatic stability analysis of the Bottom Ash Impoundment 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* (EPA 1995). Fine-grained soils (historic embankment fill, natural soil, new embankment fill, compacted fill, and protective cover) were assigned strength parameters corresponding to 80% of the total stress (undrained) strength parameters.

- The historic embankment fill was assigned strength parameters based on laboratory testing of the silty sand (SM) predominantly found in the fill. The reduction of the strength parameters to 80% of the total stress strength parameters resulted in cohesion intercept of 630 psf and a friction angle of 14 degrees.
- The natural soil was observed to contain clay, silt, and sand. Strength parameters for this material were based on laboratory testing of clay (CH) material collected from the site with the strength envelope intercepting at zero with zero confining pressure to conservatively model the effects of sand layers. A shear/normal function (with corresponding reduction to 80% strength parameters) was used to describe the shear strength of the material and is shown below:

  Assumed Dynamic Strength Envelope

Normal Stress (psf)	Shear Strength (psf)
0	0
2,188	1,094
14,400	2,879

- The new embankment fill and compacted fill were assumed to have the same pseudostatic material properties. This clay material was assigned strength parameters based on laboratory testing of a fat clay (CH) sample collected from the site in 2011. The reduction of the strength parameters to 80% of the total stress strength parameters resulted in cohesion of 470 psf and a friction angle of 15.5 degrees.
- The geomembrane 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* (EPA 1995).

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

For the Bottom Ash Impoundment at SS, the impoundment pool elevation for the long-term maximum storage pool loading condition has been defined as the design maximum pool elevation for the facility, which is 1718 feet (2 feet of freeboard). The facility's composite liner extends up to the embankment crest elevation at elevation 1720 feet. The impoundment pool elevation for the maximum surcharge pool loading condition has been defined as the maximum pool elevation for the facility, which is 1720 feet (zero feet freeboard). The likelihood of the Bottom Ash Impoundment pool elevation reaching an elevation of 1720 feet or overtopping the embankment is very low, based on the following reasoning:

- Primary inflows to the north and south cells of the facility are via pipelines from the plant, which include two bottom ash pipelines, the Retention Pond pipeline, and the Coal Pit Sump pipeline. These inflows are actively managed per the operations plan and can be shut off at any time.
- Additional inflow may come from precipitation events. However, since the impoundment does not have a substantial contributing area upstream of the facility, the amount of water directed to the pond is not more than the precipitation event.
- Outflow from each of the cells occurs through weir outfall structures and stop logs. If necessary, stop logs can be removed to lower water levels in the impoundment cells.

The onsite operations plan discusses the operation and contingency plans associated with the Bottom Ash Impoundment cells (Stone & Webster 1994).

4.5 Subsurface Water Conditions

The three cells of the Bottom Ash Impoundment at SS are lined with composite geomembrane/clay liners. Based on recent readings from the piezometers and monitoring wells installed in the embankment (P-1, P-2, PZ-BA-B, and MW-100), groundwater elevations near the Bottom Ash Impoundment are below the base elevation of the embankments (approximate maximum elevation 1700 feet). Consequently, the embankments are not modeled to be hydraulically connected to surrounding groundwater.

Readings from the piezometers and monitoring wells installed near the Bottom Ash Impoundment demonstrate that groundwater generally flows north under the facility toward the Missouri River and that groundwater elevations vary from approximately 1690 to 1700 feet under the facility. Based on these data, the stability analyses were performed with a groundwater elevation of approximately 1700 feet.

4.6 Seismic Loading Conditions

Stanton 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 2016).

4.6.1 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 0.02g (1g equals 32.2 ft/sec^2) and 0.04g using the *US Seismic Hazard 2014 Map* (USGS 2014) and the *US Seismic Hazard 2008 Map* (USGS 2008); for purposes of this analysis, the bedrock PGA was estimated to be 0.03g (Appendix B). The peak ground acceleration at Stanton 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* (EPA 1995, see Appendix B).

4.6.2 Horizontal Seismic Coefficient Determination

The horizontal seismic load coefficient (k_s), for use in the 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* (EPA 1995). Based on Step 2 of the Seismic Stability and Deformation and 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 4.6.1). As a result, a horizontal seismic load coefficient of 0.025g (0.5 * 0.05g = 0.025g) was used in the pseudostatic analysis.

4.7 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 it to behave like a liquid. The phenomenon is most often observed in saturated, loose (low density or uncompacted), sandy soils. The historic and new embankment soils of the Bottom Ash Impoundment are composed of silty/clayey materials with substantial fines. Soils immediately below the constructed embankments are also composed of soils with substantial fines. Based on these observations, the onsite soils are not anticipated to be susceptible to liquefaction.

4.8 Stability Analysis Results

Slope stability analyses were performed under static and pseudostatic conditions for the critical section (Figures 3 and 4). The results of the slope stability analyses are presented in Table 2. The results indicate that the Bottom Ash Impoundment slopes comply with the required safety factors per Section 257.73(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. 1718 ft)	1.50	2.31	3
	Max Surcharge (el. 1720 ft)	1.40	2.31	3
Pseudostatic	Max Storage (el. 1718 ft)	1.00	1.83	4
	Max Surcharge (el. 1720 ft)	1.00	1.83	4

5.0 CERTIFICATION

Based on the review of the information provided by GRE and on-site observations, we have classified the Stanton Station Bottom Ash 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 §257.73(e)(1)(i)-(iv).

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)

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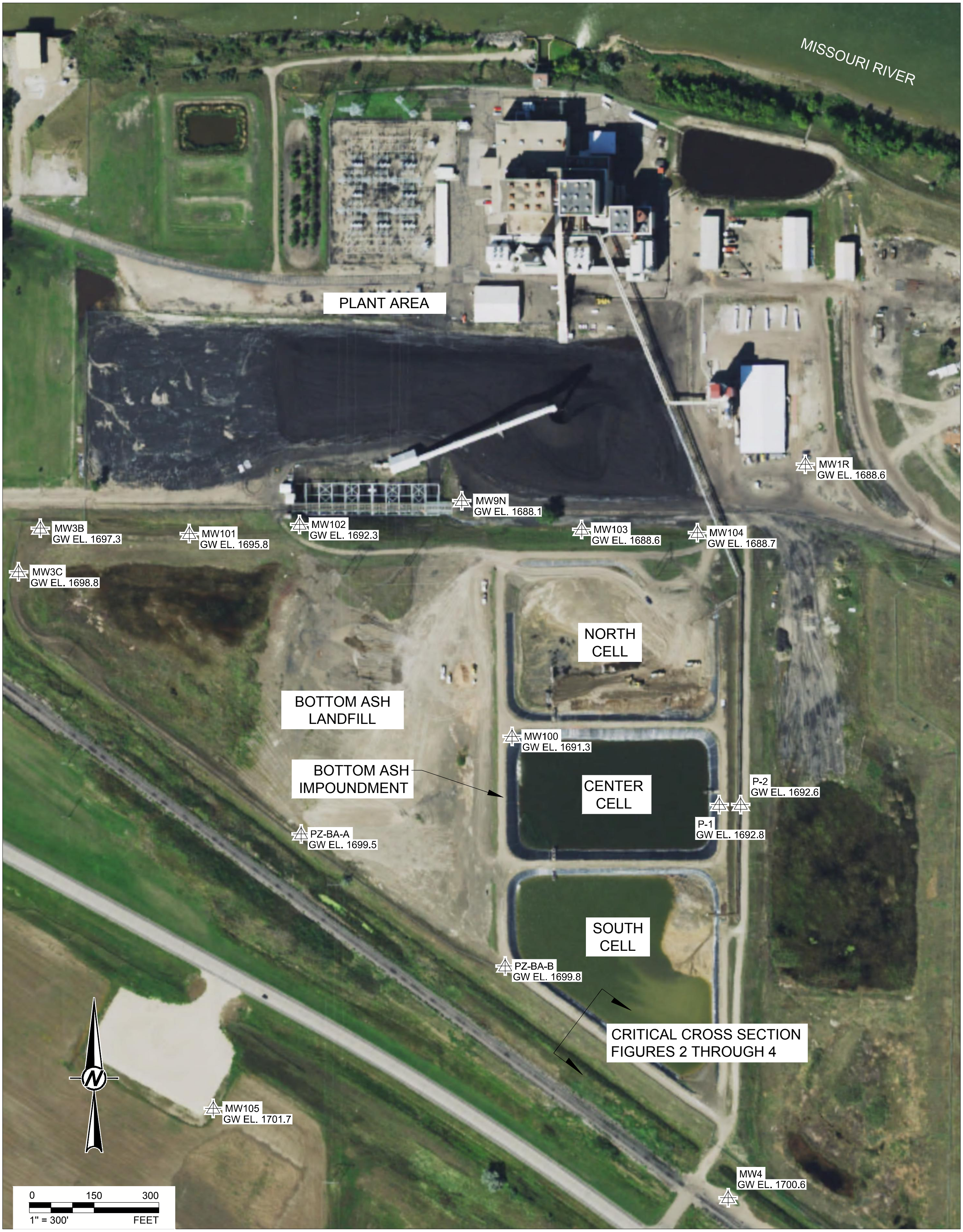
USGS. 2016. *United Station Geological Survey – Earthquake Archives*, http://earthquake.usgs.gov/earthquakes/search/, Retrieved June 13, 2016.

FIGURES



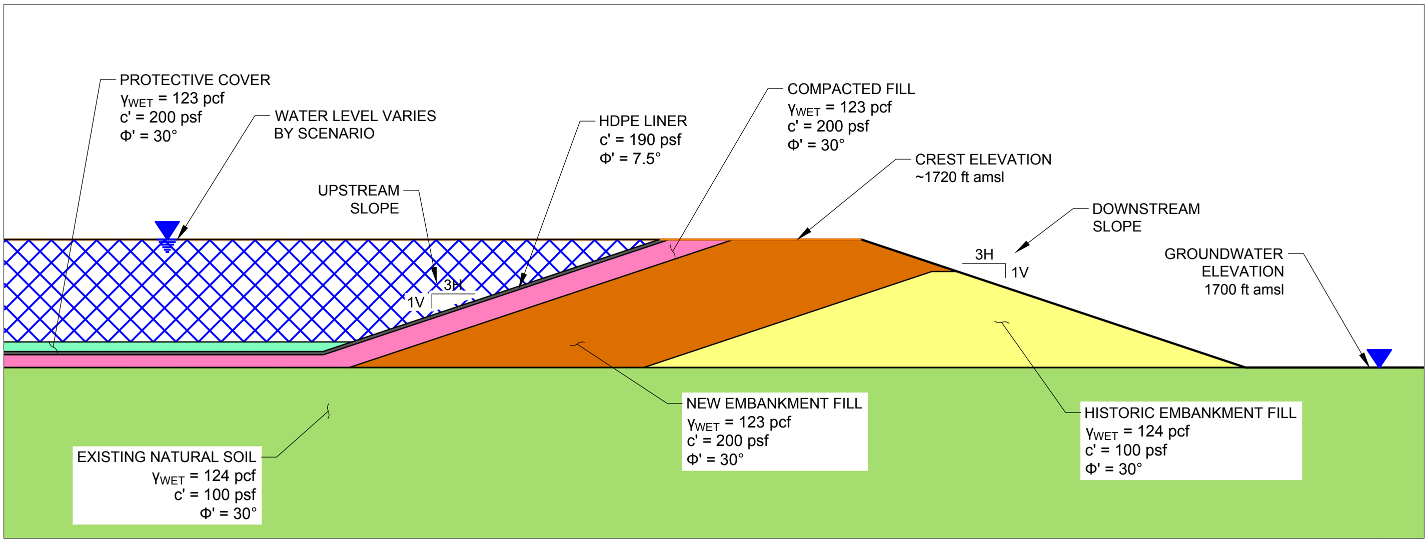
GREAT RIVER ENERGY
STANTON STATION
PLANT OVERVIEW

Golder Associates

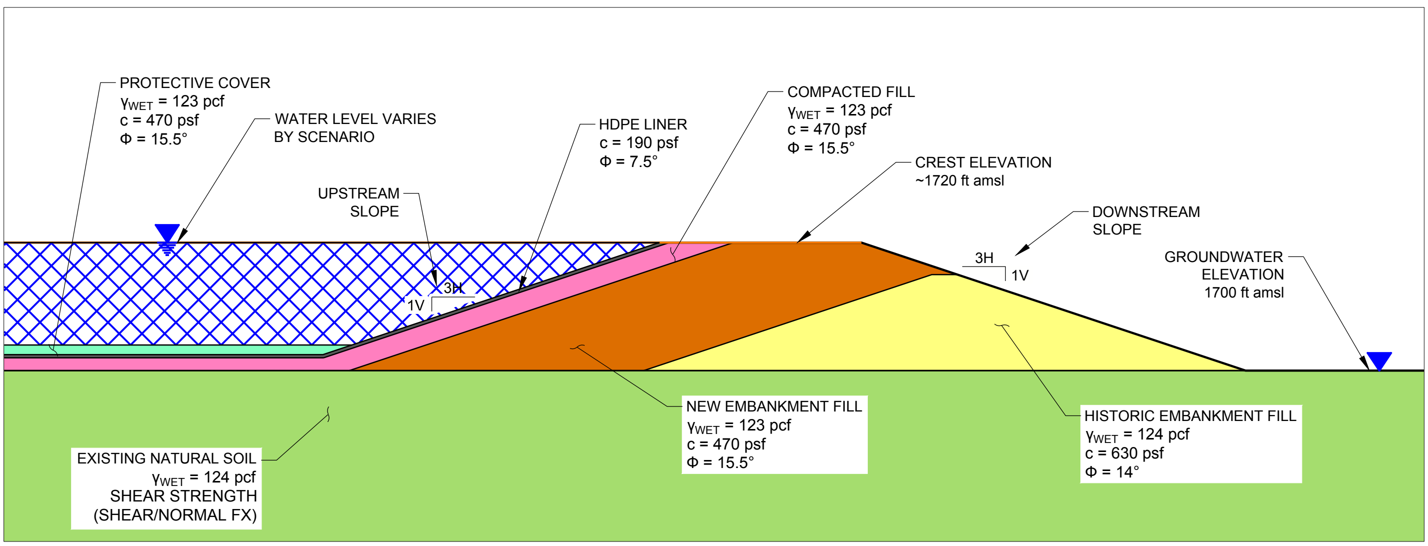
PROJECT No. 1649580 CADD: AMS DATE: 09/29/2016 FILE No. Aerial Overview Figure.dwg

FIGURE 1

Path: J:\16JOBS\1649580 GRE SS\CCR REGULATIONS\STABILITY\SLIDE\DRAFT | File Name: Aerial Overview Figure.dwg



CRITICAL CROSS SECTION - SOUTH EMBANKMENT
(STATIC LOADING MATERIAL PROPERTIES)



CRITICAL CROSS SECTION - SOUTH EMBANKMENT
(PSEUDOSTATIC LOADING MATERIAL PROPERTIES)

0 15 30
1" = 30' FEET

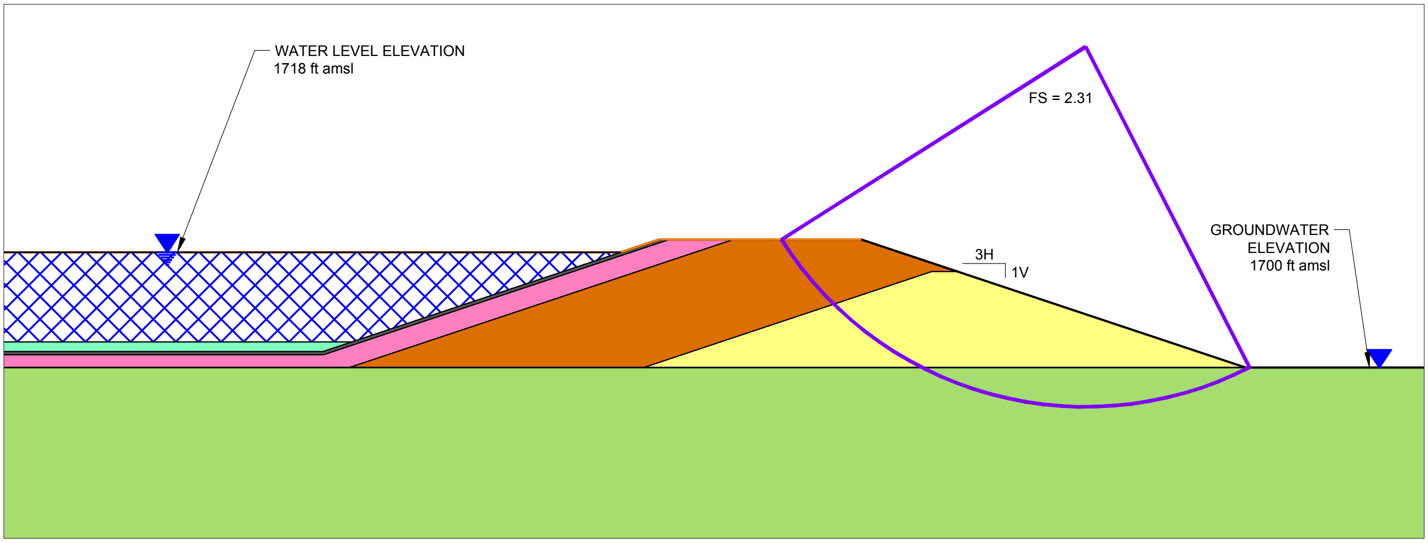
Path: J:\16JOBS\1649580 GRE SS\CCR REGULATIONS\STABILITY\SLIDE\DRAFT | File Name: Cross Section Figures_Final.dwg

Golder Associates

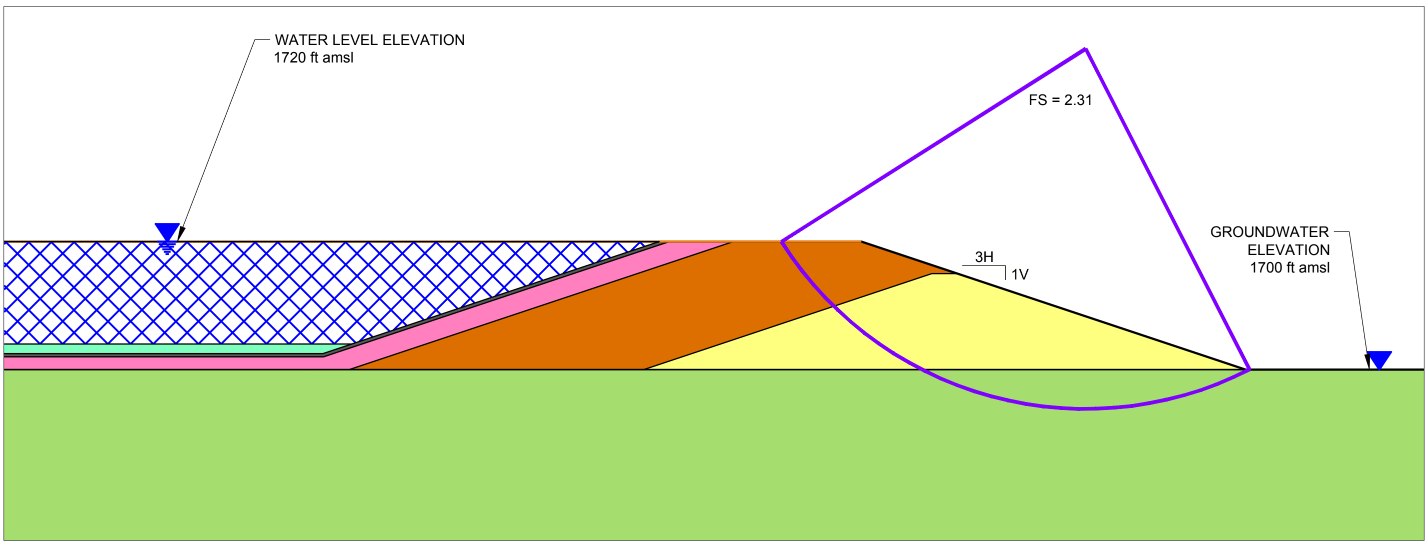
GREAT RIVER ENERGY
STANTON STATION
SLOPE GEOMETRY AND MATERIAL PROPERTIES

PROJECT No. 1649580 CADD: AMS DATE: MM/DD/2016 FILE No. Cross Section Figures_Final.dwg

FIGURE 2



STABILITY ANALYSIS RESULTS (MAXIMUM STORAGE POOL)



STABILITY ANALYSIS RESULTS (MAXIMUM SURCHARGE POOL)

0 15 30
1" = 30' FEET

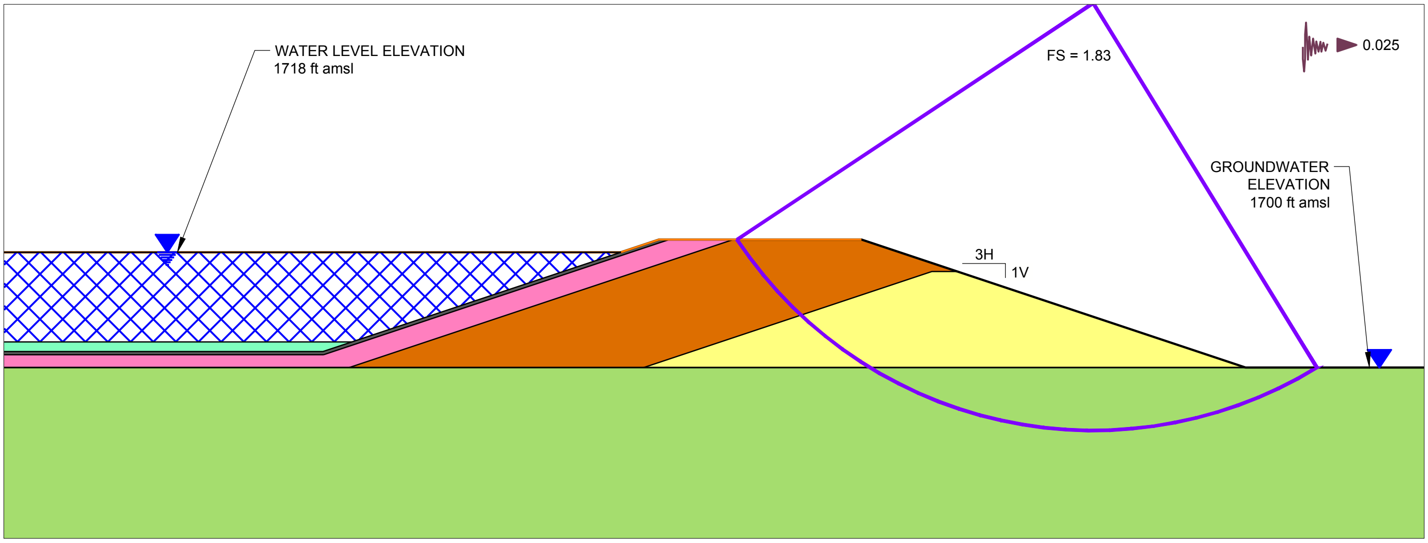
Golder Associates

GREAT RIVER ENERGY
STANTON STATION
STATIC STABILITY RESULTS

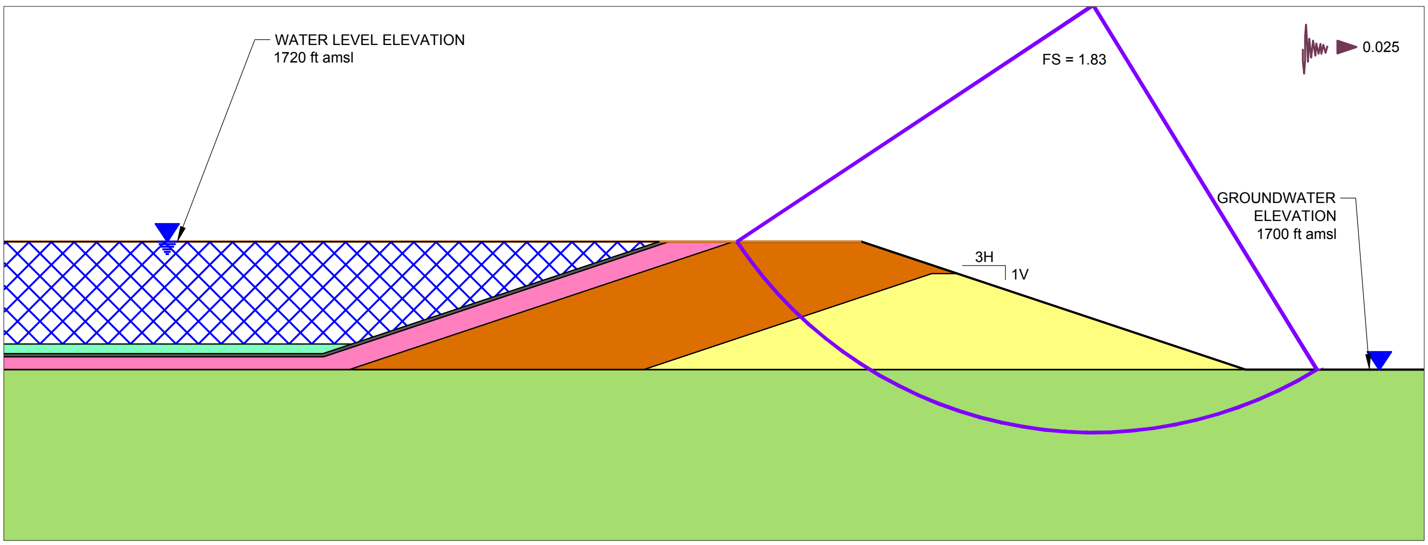
PROJECT No. 1649580 CADD: AMS DATE: MM/DD/2016 FILE No. Cross Section Figures_Final.dwg

FIGURE 3

Path: J:\16JOBS\1649580 GRE SS\CCR REGULATIONS\STABILITY\SLIDE\DRAFT | File Name: Cross Section Figures_Final.dwg



STABILITY ANALYSIS RESULTS (MAXIMUM STORAGE POOL)



STABILITY ANALYSIS RESULTS (MAXIMUM SURCHARGE POOL)

0 15 30
1" = 30' FEET

Golder Associates

GREAT RIVER ENERGY
STANTON STATION
SEISMIC (PSEUDOSTATIC) STABILITY RESULTS

PROJECT No. 1649580 CADD: AMS DATE: MM/DD/2016 FILE No. Cross Section Figures_Final.dwg

FIGURE 4

Path: J:\16JOBS\1649580 GRE SS\CCR REGULATIONS\STABILITY\SLIDE\DRAFT | File Name: Cross Section Figures_Final.dwg

APPENDIX A
BORING LOGS AND LABORATORY SOIL TEST RESULTS

Midwest Testing LABORATORY, INC. A Terracon COMPANY

October 19, 2011

Golder Associates, Inc
44 Union Blvd, Suite 300
Lakewood, CO 80228-1856

Attn: Todd Stong

RE: Piezometer Installations
Great River Energy – Stanton Station
Stanton, North Dakota
GAI #113-81645, MTL Project M2115282

Dear Todd:

Attached please find logs of the two borings advanced for the above-referenced project. These borings were converted to piezometers installations.

Piezometer construction diagrams and the state of North Dakota required monitoring well reports will be sent to you after the appropriate ownership information and survey data has been obtained.

As requested, several soil samples were selected for laboratory analysis. The testing program included determining moisture content, dry density, Atterberg limits and grain-size distribution. Additionally, moisture-density relationships were determined on three bag samples. All test results are included on the attached boring logs opposite of the samples on which they were performed and on the attached summary reports.

Should you have any questions or require further assistance, please contact us.

Sincerely,

MIDWEST TESTING LABORATORY

Steven S. Smith

Steven S. Smith, P.E.
Senior Project Engineer

SSS/cb

Attachments: test boring logs (2)
tests of soils (2)
grain-size distributions (6)
moisture-density relationships
soil classifications
descriptive terminology

cc: Golder Associates, Inc\Tammy Rauen

Midwest Testing
LABORATORY, INC.
A Terracon COMPANY

1805 Hancock Dr, PO Box 2084
Bismarck, ND 58502-2084
Phone (701) 258-2833 Fax (701) 258-2857

JOB NO.: M21115282 LOG OR TEST BORING NO.: 1 VERTICAL SCALE: 1"=5'

PROJECT: Piezometer Installations, Great River Energy-Stanton Station, Stanton, North Dakota

DEPTH IN FEET	SOIL DESCRIPTION SURFACE ELEVATION:	SAMPLE NO.	SAMPLE TYPE	N	LABORATORY TESTS MOISTURE	DENSITY	LL/PL	Q_u (psf)
2"	GRAVEL							
	FILL-FAT CLAY-grayish brown, soft to stiff, moist (CH)	1	SS	3				
		2	SS	4	26	94		
		3	TW	--				
		4	SS	9	24	98		
		5	TW	--				
		6	SS	13	24	97		
12	FILL-FAT CLAY WITH SAND-brown, stiff, coal inclusions (CH)							
14	FILL-FAT CLAY-grayish brown, stiff, coal inclusions (CH)	7	TW	--				
		8	SS	14	22	102		
		9	TW	--				
20	SILTY SAND-brownish gray, fine-grained, medium dense, waterbearing @ 23' (SM)	10	SS	10	19	104		
		11	SS	13	18			
		12	SS	12	17			
27½	FAT CLAY-grayish brown, mottled, stiff, moist (CH)	13	SS	10	30	94		
28½	SILTY SAND-grayish brown, fine to medium-*							
29½	FAT CLAY-grayish brown, mottled, stiff, moist (CH)							
31	END OF BORING *grained, waterbearing (SM)							

WATER LEVEL DATA

DATE	TIME	CAVE IN DEPTH	WATER LEVEL
10-4-11	11:25	HSA 22½'	23'
10-4-11	11:35	HSA 29½'	28'
10-4-11	12:00	30'	None

BORING DATA

STARTED: 10-4-11 **COMPLETED:** 10-4-11 @ 11:35

METHOD USED: 3¼" ID HSA 0-29½'

CREW CHIEF: M. Roberts

Midwest Testing
LABORATORY, INC.
A Terracon COMPANY

1805 Hancock Dr, PO Box 2084
Bismarck, ND 58502-2084
Phone (701) 258-2833 Fax (701) 258-2857

JOB NO.: M21115282 LOG OR TEST BORING NO.: 2 VERTICAL SCALE: 1"=4'

PROJECT: Piezometer Installations, Great River Energy-Stanton Station, Stanton, North Dakota

DEPTH IN FEET	SOIL DESCRIPTION SURFACE ELEVATION:	SAMPLE NO.	SAMPLE TYPE	N	MOISTURE	DENSITY	LL/PL	Q_u (psf)
4"	FILL-FAT CLAY WITH SAND-brown, coal *	1	SS	22				
	FILL-SILTY SAND-brown, fine to medium-grained, medium dense (SM)	2	SS	9	9			
2	SILTY SAND-dark brown to brown, fine-grained, loose to medium dense to very loose (SM) *inclusions (CH)	3	TW	--				
		4	SS	20	7			
		5	TW	--				
		6	SS	3	11	111		
		7	TW	--				
17	SAND WITH SILT-light brown, fine-grained, medium dense to loose, waterbearing @ 19½' (SP-SM)	8	SS	11	18			
		9	SS	8	21			
		10	SS	11	29	94		
22½	FAT CLAY-grayish brown, mottled, stiff, moist (CH)	11	TW	--				
26	END OF BORING							

WATER LEVEL DATA

DATE	TIME	CAVE IN DEPTH	WATER LEVEL
10-4-11	14:00	HSA 19½'	19½'
10-4-11	14:15	HSA 24½'	20'
10-4-11	14:30	25'	23'

BORING DATA

STARTED: 10-4-11 **COMPLETED:** 10-4-11 @ 14:15

METHOD USED: 3¼" ID HSA 0-24½'

CREW CHIEF: M. Roberts

MIDWEST TESTING LABORATORY

1805 Hancock Dr / PO Box 2084 / Bismarck, North Dakota 58502
Telephone (701) 258-2833 / Fax (701) 258-2857

REPORT OF: TESTS OF SOILS

PROJECT: Piezometer Installations
Great River Energy – Stanton Station
Stanton, North Dakota
GAI #113-81645

DATE: October 19, 2011

REPORTED TO: Golder Associates, Inc
Attn: Todd Stong
44 Union Blvd, Suite 300
Lakewood, CO 80228-1856

COPIES: Golder Associates, Inc
Attn: Tammy Rauen

PROJECT NO: M2115282

SAMPLE NUMBER:	1	2	3
LOCATION:	Test boring 1, depth 1-17'	Test boring 2, depth 1-15'	Test boring 2, depth 1-15'
CLASSIFICATION:	FAT CLAY (CH)	SILTY SAND (SM)	SILTY SAND (SM)
COLOR:	Grayish brown	Grayish brown	Grayish brown
PARTICLE DISTRIBUTION (see attached curves):			
Gravel (%)			
Coarse (plus 30 mm)			
Fine (30-5 mm)			
Sand (%)			
Coarse (5-2 mm)			
Medium (2-.44 mm)	2	3	3
Fine (.44 mm-.074)	6	70	67
Fines (%)			
Silt (.074-.005 mm)	42	14	17
Clay (.005-.001 mm)	17	4	4
Colloids (less than .001 mm)	33	9	9
ATTERBERG LIMITS:			
Liquid Limit	63	NP	NP
Plastic Limit	17	NP	NP
Plasticity Index	46	NP	NP
LABORATORY MOISTURE-DENSITY RELATIONS (see attached curves):			
Method	Standard proctor, ASTM:D698, Method "A"		
Optimum Moisture (%)	20.9	11.4	12.2
Maximum Density (pcf)	103.0	119.0	117.4

REMARKS: Samples were obtained at the locations shown above by Midwest Testing Laboratory on October 4, 2011.

Signed: Steven S. Smith

MIDWEST TESTING LABORATORY

Project No.: M2115282

Sample Source: 1, Borehole #1, Depth: 1'-17'

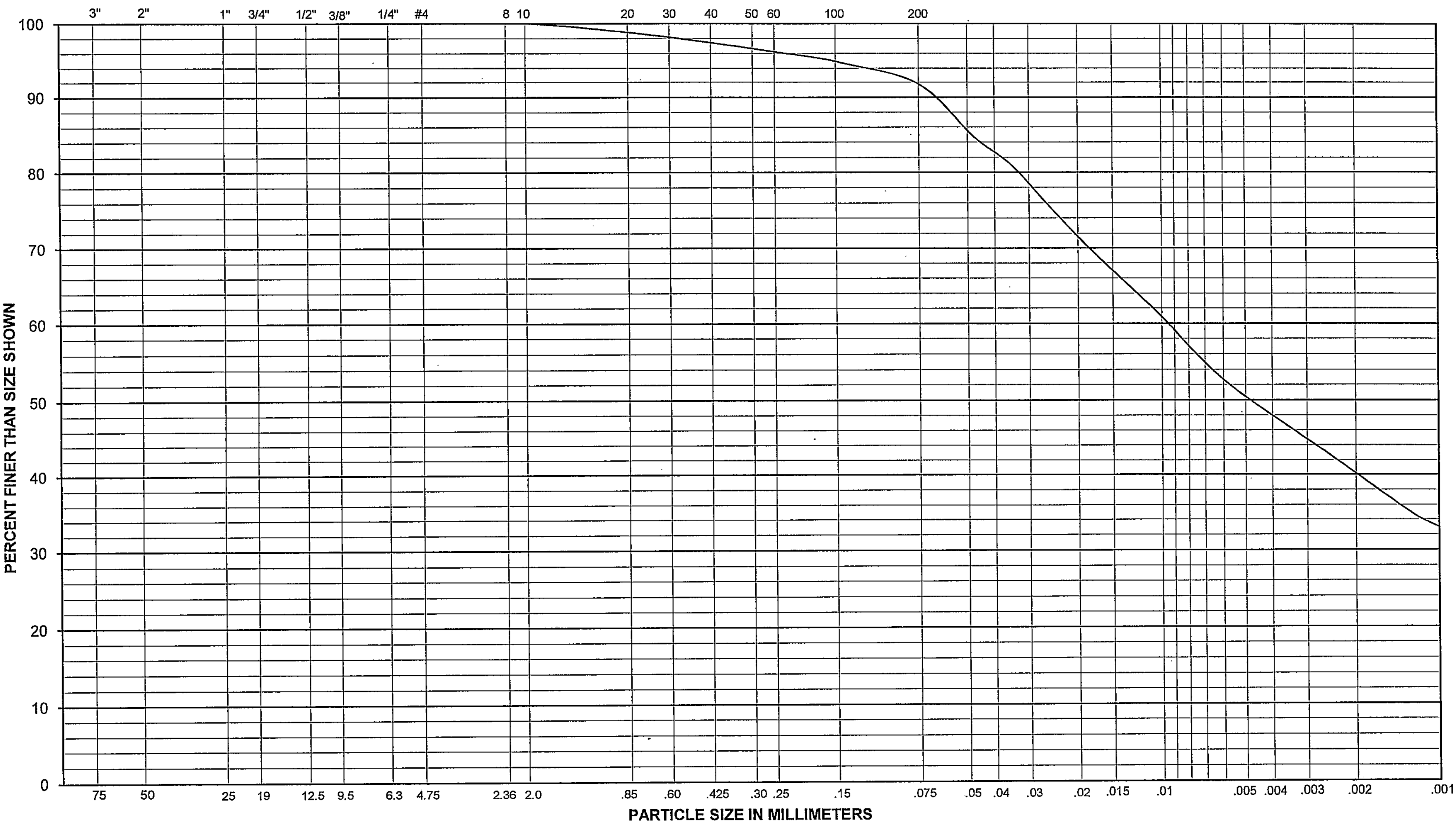
Classification: FAT CLAY - grayish brown (CH)

Project: Piezometer Installations
GRE - Stanton Station
Stanton, ND

Reported to: Golder Associates, Inc

GRAIN SIZE DISTRIBUTION CURVE

U.S. Standard Sieve Sizes



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

MIDWEST TESTING LABORATORY

1805 Hancock Dr. / P.O. Box 2084 / Bismarck, North Dakota 58502
Phone (701) 258-2833 / Fax (701) 258-2857

REPORT OF: MOISTURE-DENSITY RELATIONS OF SOIL

PROJECT: Piezometer Installations
Great River Energy – Stanton Station
Stanton, North Dakota
GAI #113-81645

DATE: October 19, 2011

REPORTED TO: Golder Associates, Inc
Attn: Todd Stong
44 Union Blvd, Suite 300
Lakewood, CO 80228-1856

COPIES: Golder Associates, Inc
Attn: Tammy Rauen

PROJECT NO: M2115282

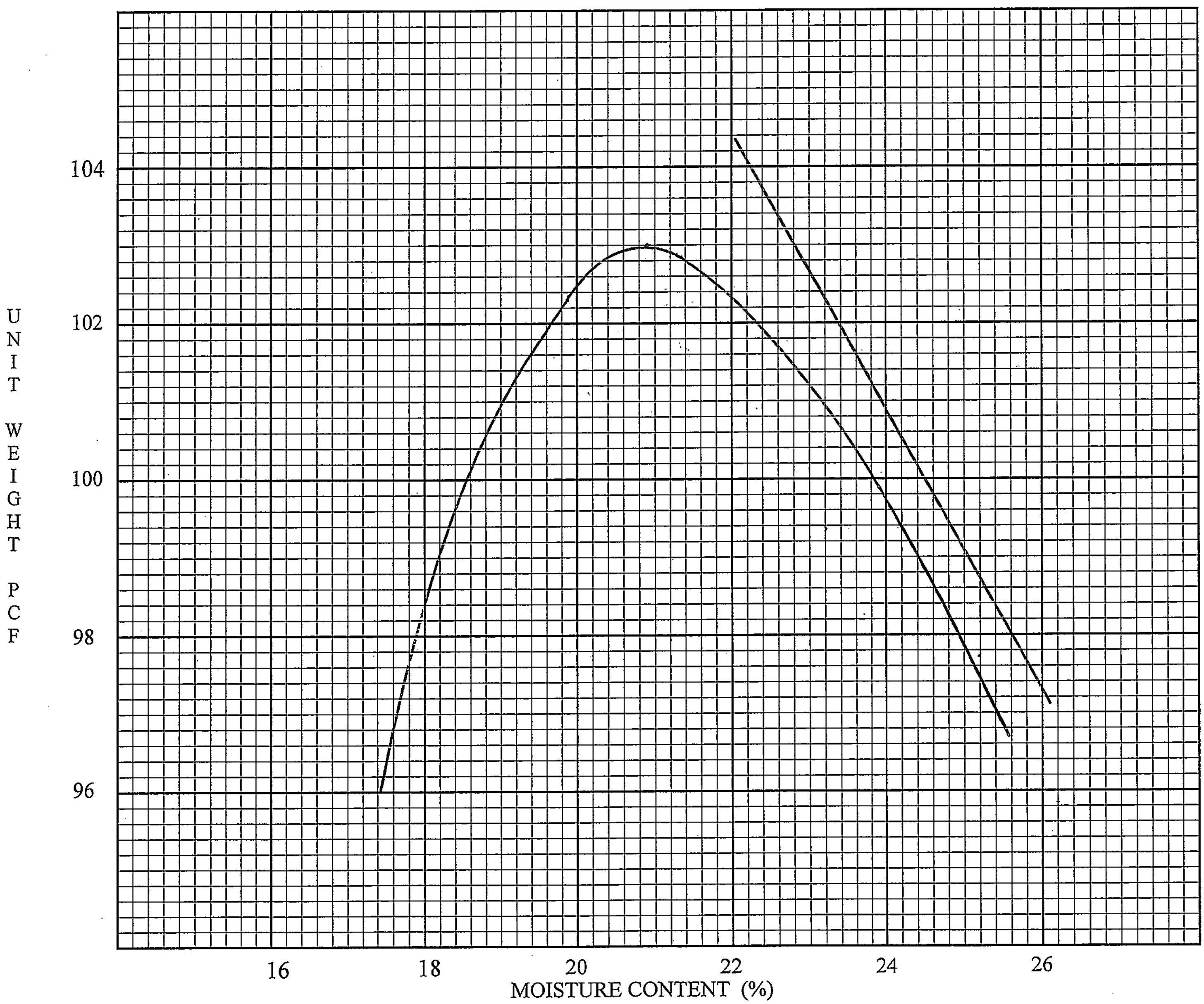
SAMPLE NUMBER: 1 (Test boring 1, depth 1-17')

METHOD: Standard proctor, ASTM:D698, Method "A"

MAXIMUM DENSITY: 103.0 pcf

SOIL TYPE: FAT CLAY-grayish brown (CH)

OPTIMUM MOISTURE: 20.9%

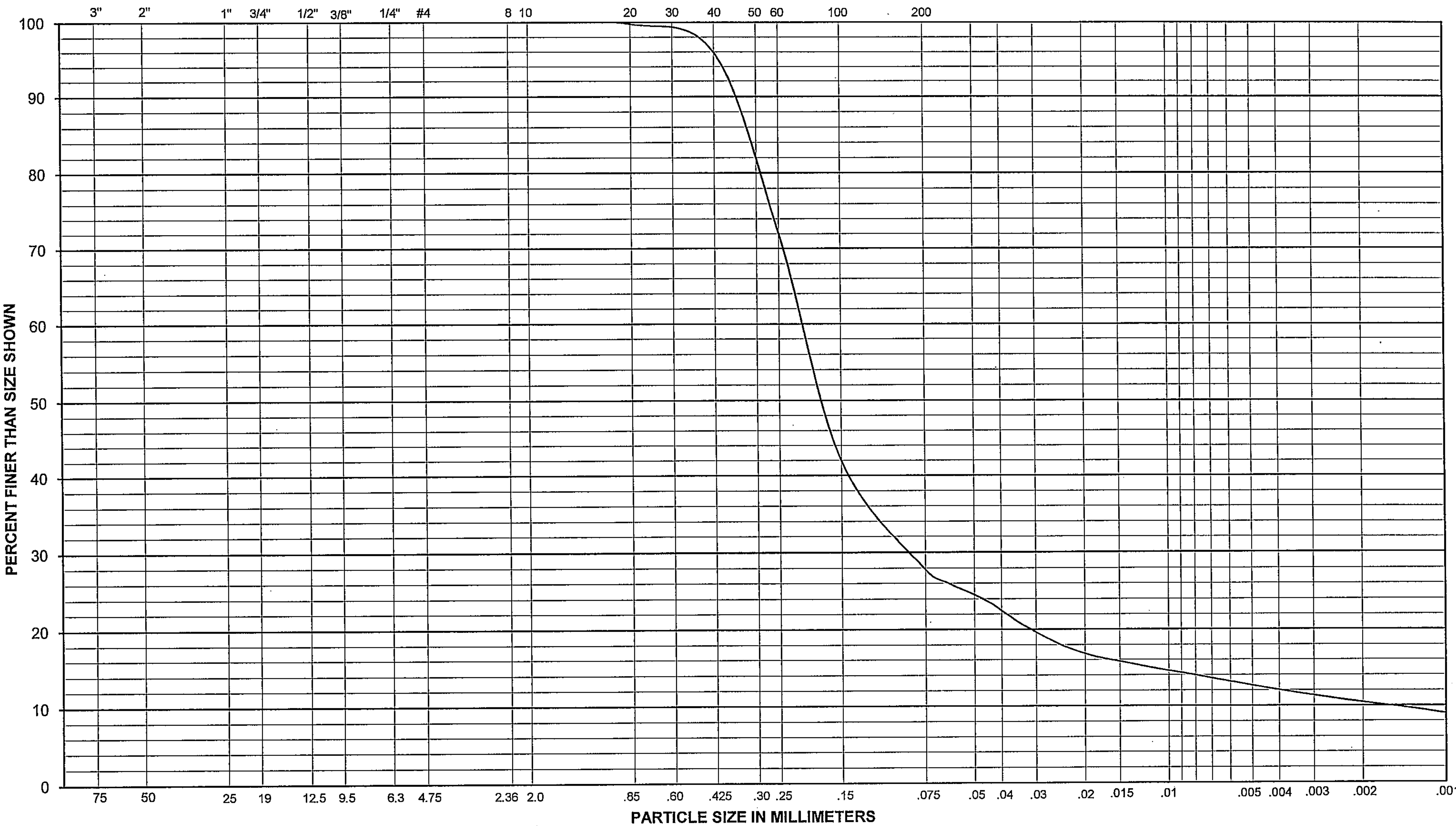


MIDWEST TESTING LABORATORY

Project No.:	M2115282	Project:	Piezometer Installations GRE - Stanton Station Stanton, ND
Sample Source:	2, Borehole #2, Depth: 1'-15'		
Classification:	SILTY SAND - grayish brown (SM)	Reported to:	Golder Associates, Inc

GRAIN SIZE DISTRIBUTION CURVE

U.S. Standard Sieve Sizes



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

MIDWEST TESTING LABORATORY

1805 Hancock Dr. / P.O. Box 2084 / Bismarck, North Dakota 58502
Phone (701) 258-2833 / Fax (701) 258-2857

REPORT OF: MOISTURE-DENSITY RELATIONS OF SOIL

PROJECT: Piezometer Installations
Great River Energy – Stanton Station
Stanton, North Dakota
GAI #113-81645

DATE: October 19, 2011

REPORTED TO: Golder Associates, Inc
Attn: Todd Stong
44 Union Blvd, Suite 300
Lakewood, CO 80228-1856

COPIES: Golder Associates, Inc
Attn: Tammy Rauen

PROJECT NO: M2115282

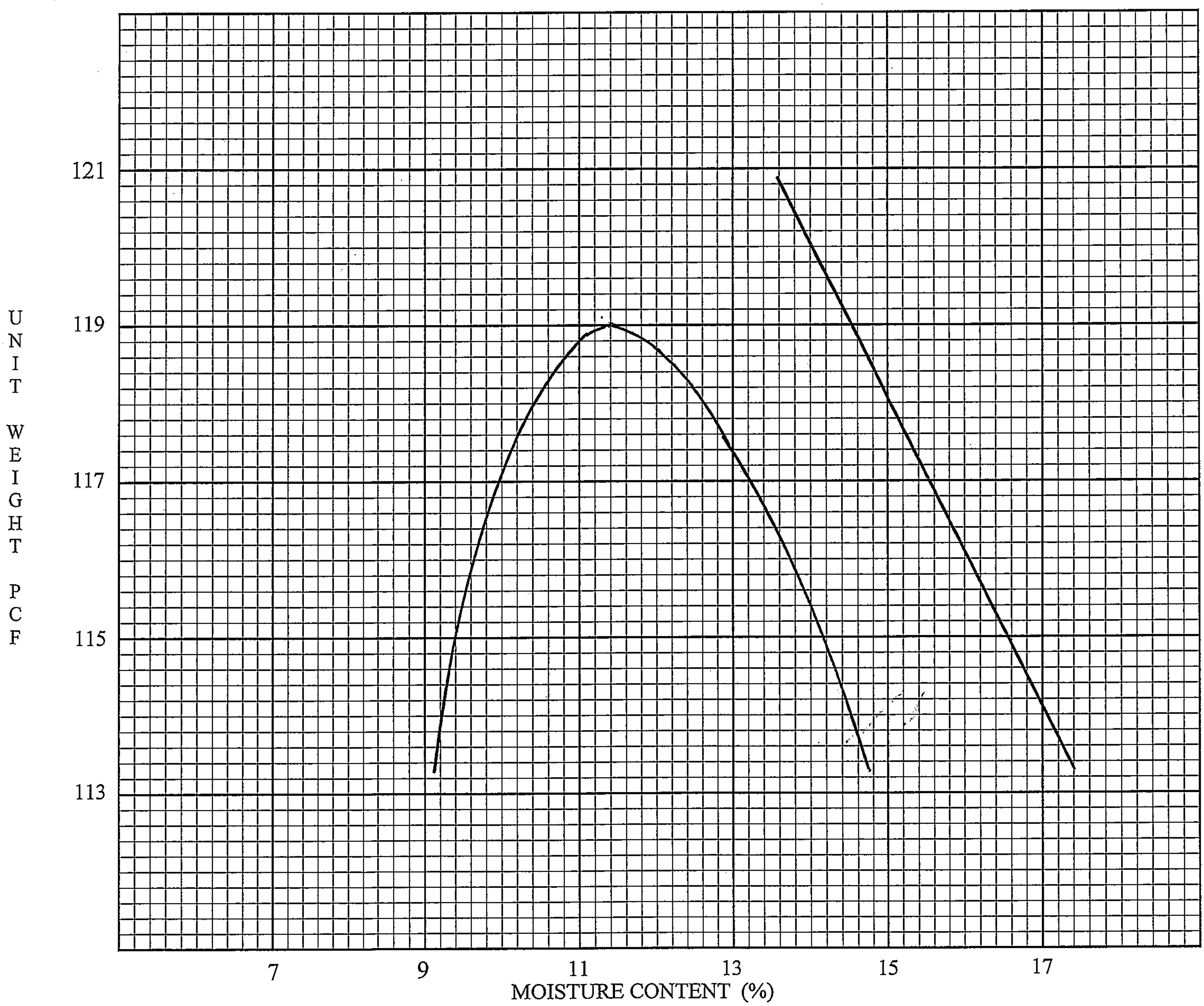
SAMPLE NUMBER: 2 (Test boring 2, depth 1-17')

METHOD: Standard proctor, ASTM:D698, Method "A"

MAXIMUM DENSITY: 119.0 pcf

SOIL TYPE: SILTY SAND-grayish brown (SM)

OPTIMUM MOISTURE: 11.4%



MIDWEST TESTING LABORATORY

Project No.: M2115282

Sample Source: 3, Borehole #2, Depth: 1'-15'

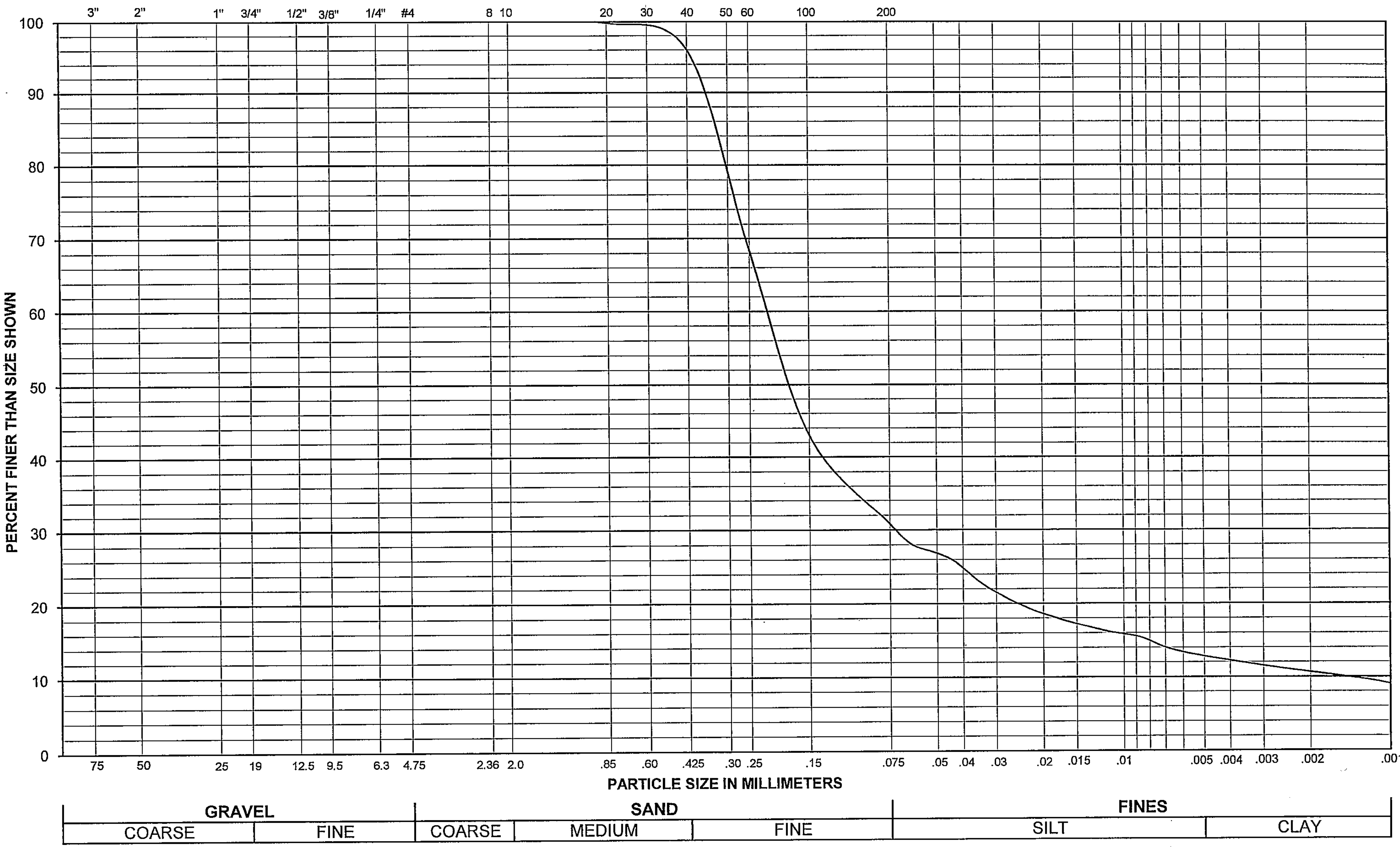
Classification: SILTY SAND - grayish brown (SM)

Project: Piezometer Installations
GRE - Stanton Station
Stanton, ND

Reported to: Golder Associates, Inc

GRAIN SIZE DISTRIBUTION CURVE

U.S. Standard Sieve Sizes



MIDWEST TESTING LABORATORY

1805 Hancock Dr. / P.O. Box 2084 / Bismarck, North Dakota 58502
Phone (701) 258-2833 / Fax (701) 258-2857

REPORT OF: MOISTURE-DENSITY RELATIONS OF SOIL

PROJECT: Piezometer Installations
Great River Energy – Stanton Station
Stanton, North Dakota
GAI #113-81645

DATE: October 19, 2011

REPORTED TO: Golder Associates, Inc
Attn: Todd Stong
44 Union Blvd, Suite 300
Lakewood, CO 80228-1856

COPIES: Golder Associates, Inc
Attn: Tammy Rauen

PROJECT NO: M2115282

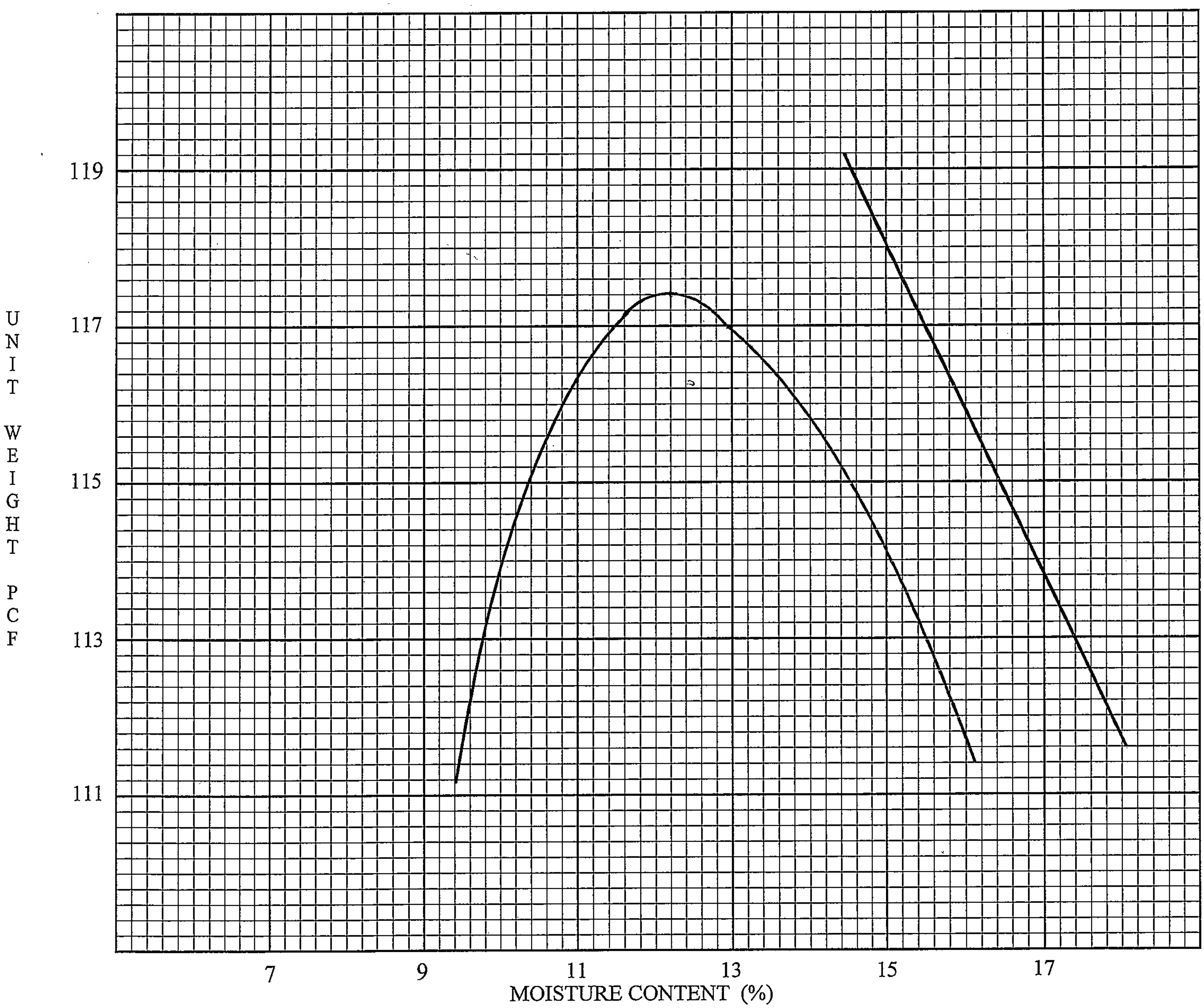
SAMPLE NUMBER: 3 (Test boring 2, depth 1-17')

METHOD: Standard proctor, ASTM:D698, Method "A"

MAXIMUM DENSITY: 117.4 pcf

SOIL TYPE: SILTY SAND-grayish brown (SM)

OPTIMUM MOISTURE: 12.2%



MIDWEST TESTING LABORATORY

1805 Hancock Dr / PO Box 2084 / Bismarck, North Dakota 58502
Telephone (701) 258-2833 / Fax (701) 258-2857

REPORT OF: TESTS OF SOILS

PROJECT: Piezometer Installations
Great River Energy – Stanton Station
Stanton, North Dakota
GAI #113-81645

DATE: October 19, 2011

REPORTED TO: Golder Associates, Inc
Attn: Todd Stong
44 Union Blvd, Suite 300
Lakewood, CO 80228-1856

COPIES: Golder Associates, Inc
Attn: Tammy Rauen

PROJECT NO: M2115282

SAMPLE NUMBER:	C1	C2	C3
LOCATION:	Test boring 1, Composite of Samples 2, 4, 6 & 8	Composite of Test boring 1, Samples 10, 11 & 12 and test boring 2, Samples 8 & 9	Composite of Test boring 1, Sample 13 & test boring 2, Sample 10
CLASSIFICATION:	FAT CLAY (CH)	POORLY-GRADED SAND WITH SILT (SP-SM)	FAT CLAY (CH)
COLOR:	Grayish brown	Brown	Brown
PARTICLE DISTRIBUTION (see attached curves):			
Gravel (%)			
Coarse (plus 30 mm)			
Fine (30-5 mm)			
Sand (%)			
Coarse (5-2 mm)			
Medium (2-.44 mm)	3	4	1
Fine (.44 mm-.074)	6	85	7
Fines (%)			
Silt (.074-.005 mm)	46	4	14
Clay (.005-.001 mm)	15	3	21
Colloids (less than .001 mm)	30	4	57
ATTERBERG LIMITS:			
Liquid Limit	56	NP	79
Plastic Limit	18	NP	18
Plasticity Index	38	NP	61

REMARKS: Samples were obtained at the locations shown above by Midwest Testing Laboratory on October 4, 2011.

Signed: Steven S. Smith

MIDWEST TESTING LABORATORY

Project No.: M2115282

Sample Source: C1, Boring 1, Samples 2, 4, 6, 8

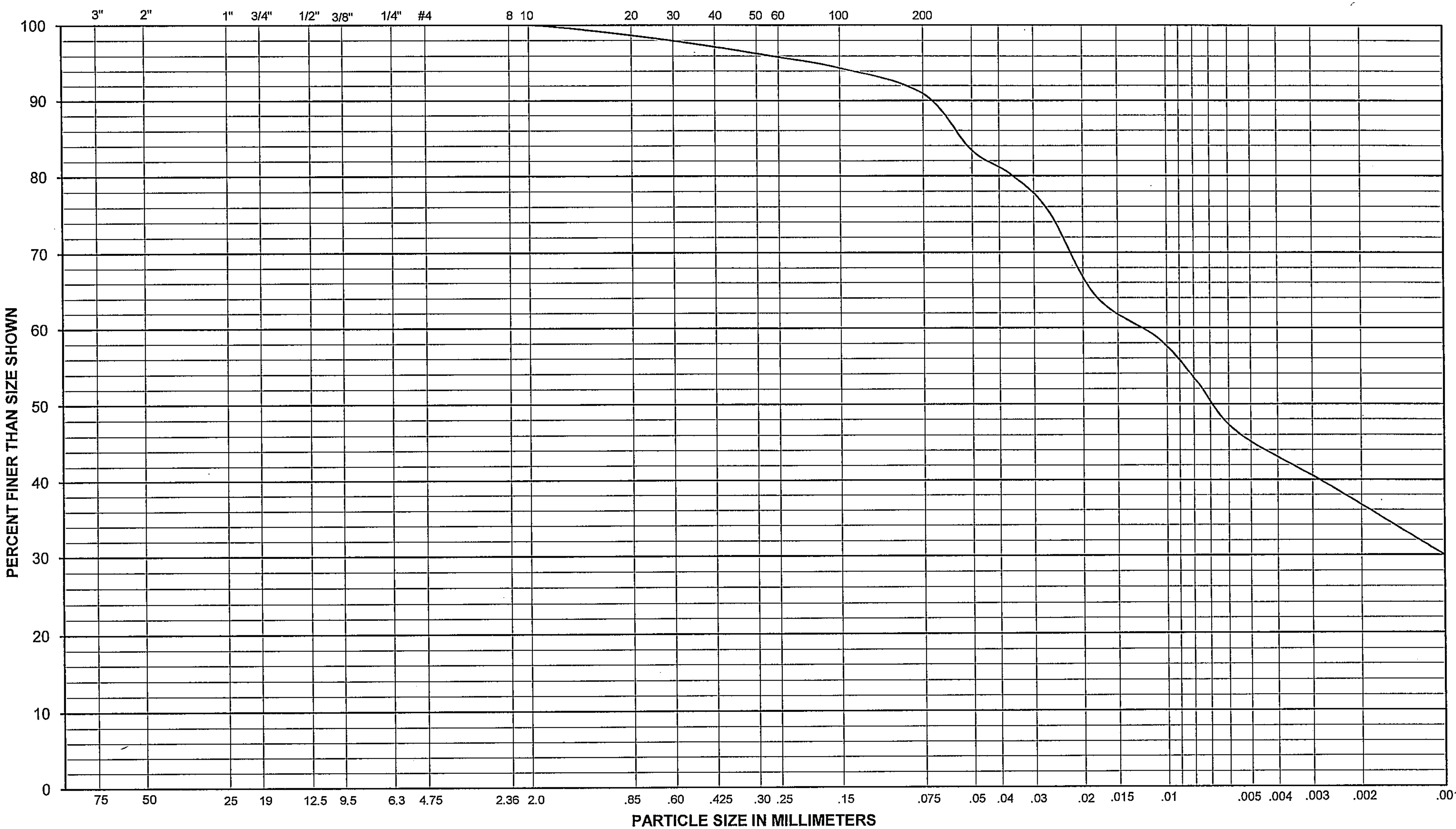
Classification: FAT CLAY - grayish brown

Project: Piezometer Installations
GRE - Stanton Station
Stanton, ND

Reported to: Golder Associates, Inc

GRAIN SIZE DISTRIBUTION CURVE

U.S. Standard Sieve Sizes



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

MIDWEST TESTING LABORATORY

Project No.: M2115282

Sample Source: C2, Boring 1, Samples 10,11,12; and Boring 2, Samples 8, 9

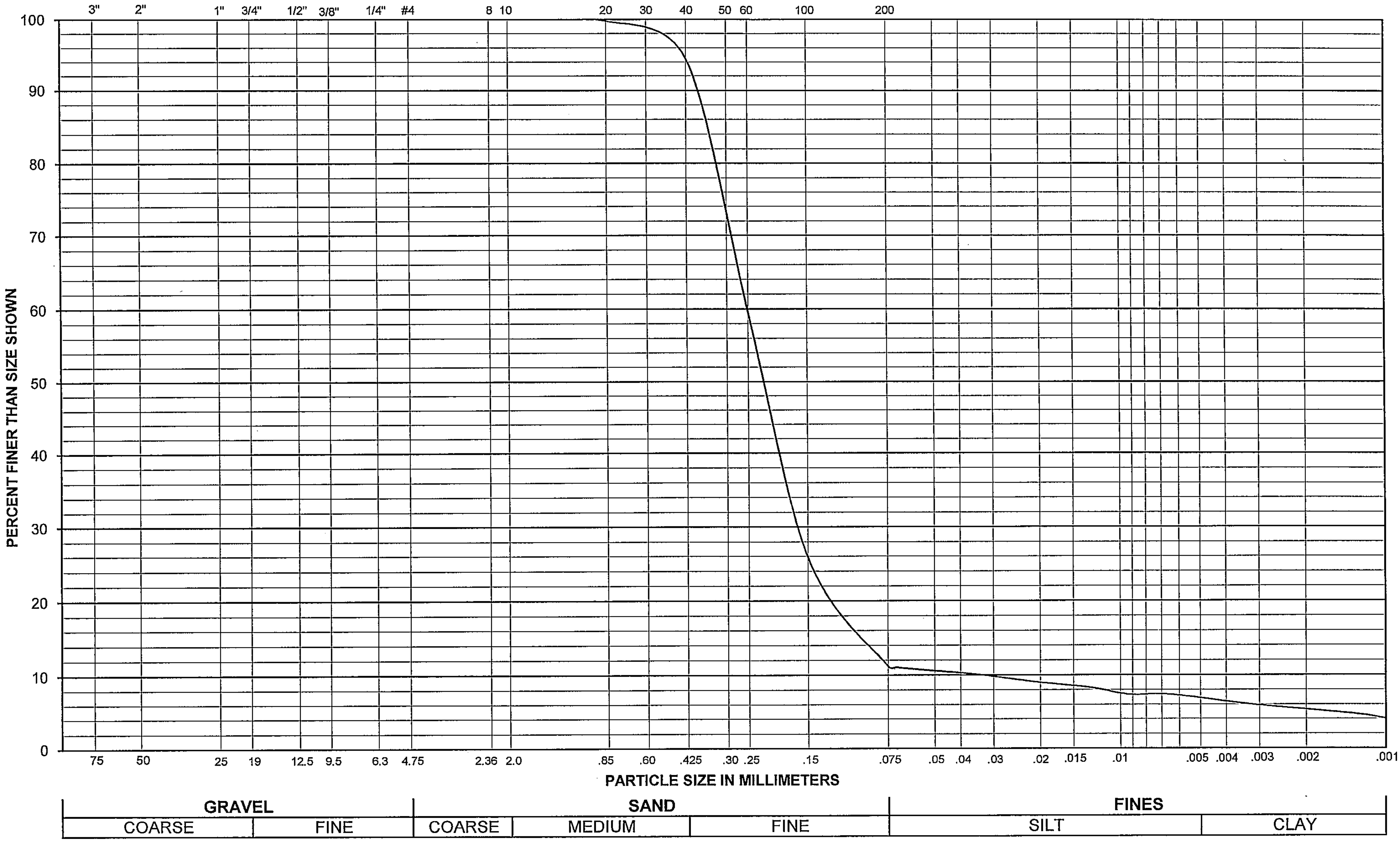
Classification: POORLY GRADED SAND WITH SILT - brown (SP-SM)

Project: Piezometer Installations
GRE - Stanton Station
Stanton, ND

Reported to: Golder Associates, Inc

GRAIN SIZE DISTRIBUTION CURVE

U.S. Standard Sieve Sizes



MIDWEST TESTING LABORATORY

Project No.: M2115282

Sample Source: C3, Boring 1, Sample 13; and Boring 2, Sample 10

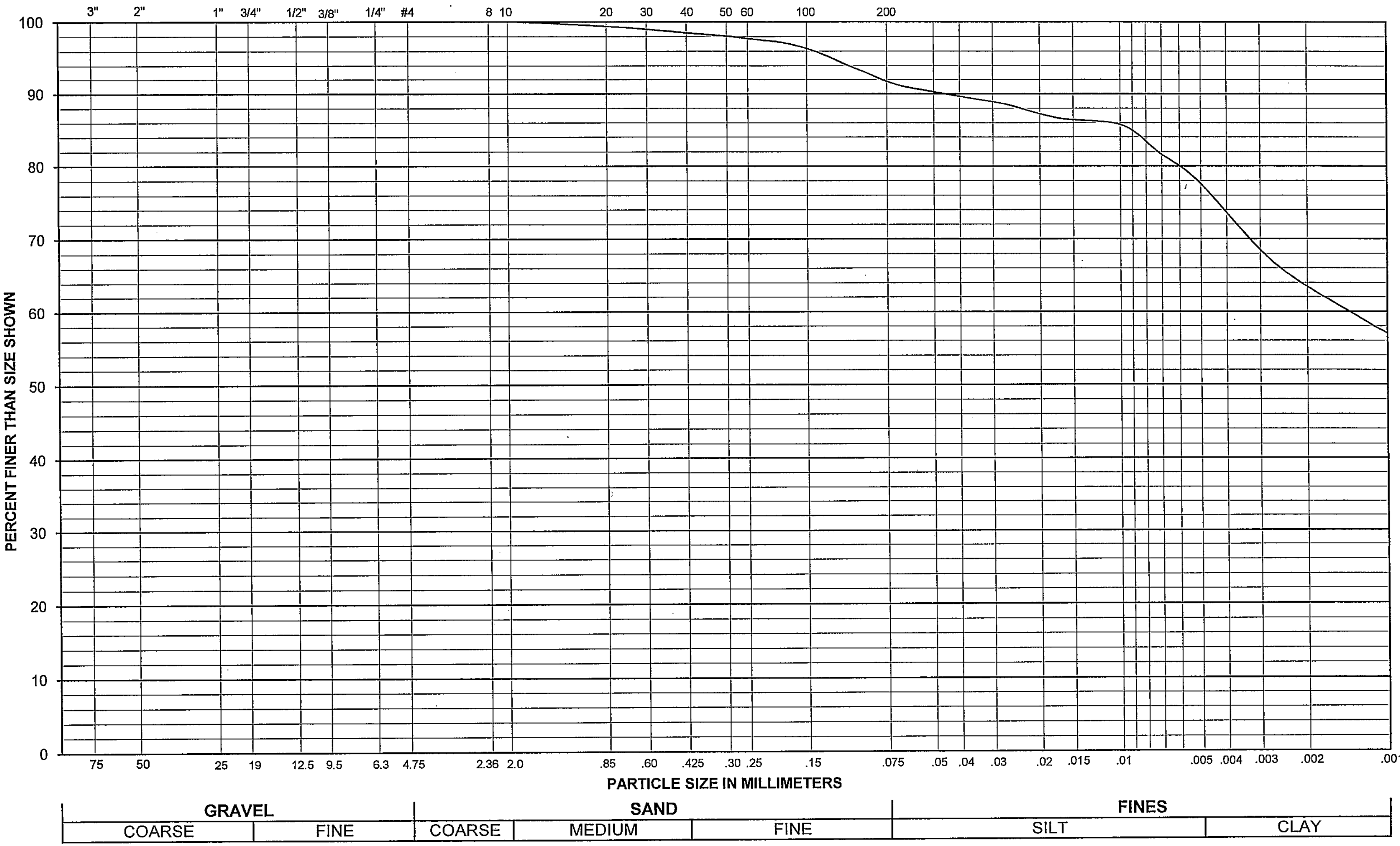
Classification: FAT CLAY - brown

Project: Piezometer Installations
GRE - Stanton Station
Stanton, ND

Reported to: Golder Associates, Inc

GRAIN SIZE DISTRIBUTION CURVE

U.S. Standard Sieve Sizes



Classification of Soils For Engineering Purposes

ASTM:D 2487-98

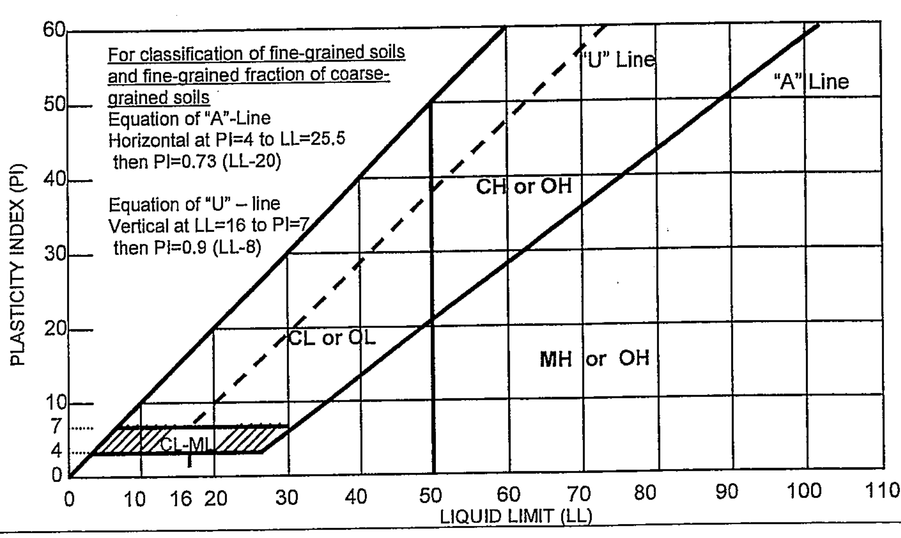
Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification: Group Symbol	Soil Classification: Group Name[B]
Coarse-Grained Soils More than 50% retained on No. 200 Sieve	Gravels More than 50% coarse fraction retained on No. 4 Sieve	Clean Gravels Less than 5% fines[C]	Cu≥4 and 1≤Cc≤ 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	Cu≥6 and 1≤Cc≤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 Fibric Peat > 67% Fiber	Primary organic matter, dark in color, and organic odor Hemic Peat 33%-67% Fibers			PT	Peat Sapric Peat < 33% Fibers

[A]Based on the material passing the 3-in. (75mm) 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 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] $Cu = D_{60} / D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[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 hatched area, soil is 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.



DESCRIPTIVE TERMINOLOGY

RELATIVE DENSITY

Term	"N" Value
Very Loose	0-4
Loose	5 – 9
Medium Dense	10 – 30
Dense	31 – 50
Very Dense	Greater than 50

THICKNESS OF SOIL INTRUSIONS

Term	Range
Lense / Lamination	0 – 1/8"
Seam	1/8" – 1"
Layer	1" – 12"

CONSISTENCY OF COHESIVE SOILS

Term	"N" Value
Very soft	Less than 2
Soft	2 – 4
Medium stiff	5 – 8
Stiff	9 – 15
Very Stiff	16 – 30
Hard	Greater than 30

RELATIVE PROPORTIONS

Term	Range
Trace	0 – 5%
A Little	5 – 15%
With	15 – 50%

PARTICLES SIZES

Term	Range
Boulders	Over 12"
Cobbles	3"– 12"
Gravel	
Coarse	3/4" – 3"
Fine	#4 – 3/4"
Sand	
Coarse	#4 - #10
Medium	#10 - #40
Fine	#40 - #200
Silt	#200 – 0.005 mm
Clay	Less than 0.005 mm

Note: Sieve sizes shown are U.S. Standard

DRILLING & SAMPLING SYMBOLS

Symbol	Definition
FA	Flight Auger
SS	Split Spoon
TW	Thin-Walled Tube
HSA	Hollow Stem Auger
N	Penetration Resistance: blows required to drive a two-inch OD split spoon sampler one foot by means of a 140-pound hammer falling 30 inches

LABORATORY TEST SYMBOLS

Symbols	Definition
LL	Liquid Limit, %
PL	Plastic Limit, %
Q_u	Unconfined Compressive Strength, psf

Additional insertions in Q_u column

Symbols	Definition
G	Specific Gravity
SL	Shrinkage Limit, %
pH	Hydrogen Ion Content-Meter Method
O	Organic Content, % - Combustion Method
M.A.	Grain Size Analysis - Mechanical Method
Hyd.	Grain Size Analysis - Hydrometer Method
C	One-Dimensional Consolidation
Q_c	Triaxial Compression
K	Coefficient of Permeability

WATER LEVEL INFORMATION

Water levels shown on the boring logs are levels measured in the borings at the time and under the conditions noted. In sand, the indicated levels can be considered reliable. In clay soil, it is not possible to determine the ground water level within the normal scope of a test boring investigation, except where lenses or layers of more pervious water-bearing soils are present. Even then, a long period of time may be necessary to reach equilibrium. Therefore, the position of the water level noted on the boring logs for cohesive or mixed-texture soils may not indicate the true level of the ground water table.

SOIL STRATIFICATION BOUNDARIES

The soil stratification lines shown on the boring logs indicate the approximate boundary between different soil types. In the field, the transition between soil types may be gradual.

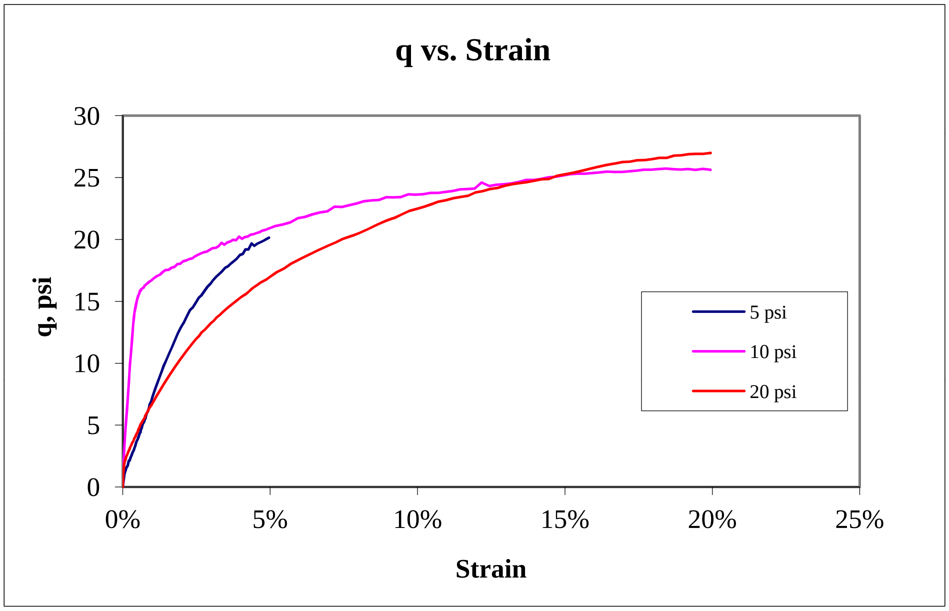
CLAY FILL

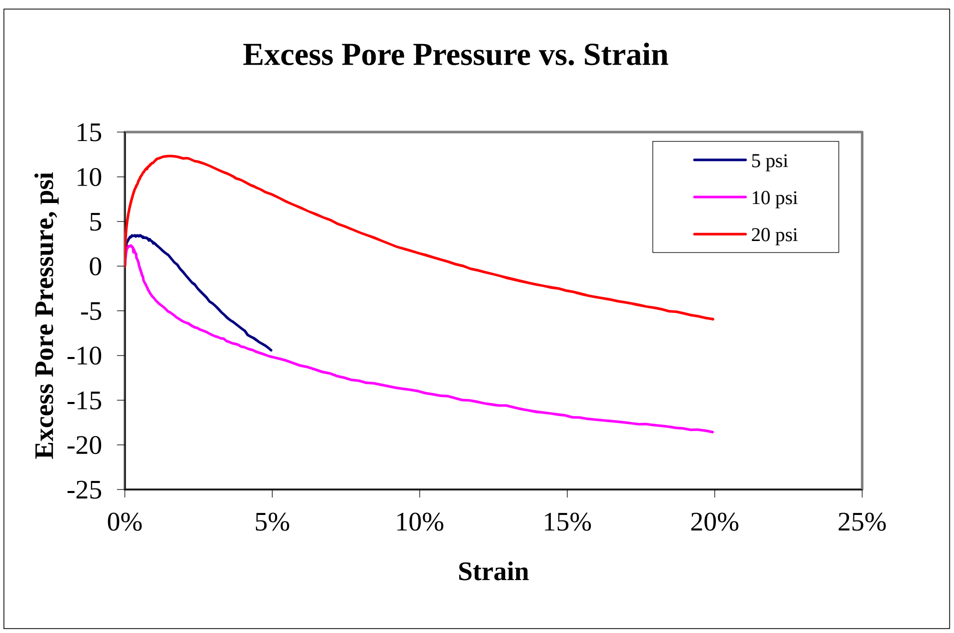
Sample # =	Clay Fill 2A		Sample # =	Clay Fill 2A		Sample # =	Clay Fill 2B	
Point # =	1 (staged)		Point # =	2 (staged)		Point # =	3	
	Initial			**Initial**			**Initial**	
Length =	14.74	cm	Length =	14.74	cm	Length =	17.58	cm
Diameter =	7.28	cm	Diameter =	7.28	cm	Diameter =	7.29	cm
Wet Weight =	1231.20	g	Wet Weight =	1231.20	g	Wet Weight =	1540.40	g
Area =	41.6	cm^2	Area =	41.6	cm^2	Area =	41.7	cm^2
Sample Area =	6.45	in^2	Sample Area =	6.45	in^2	Sample Area =	6.47	in^2
Volume =	613.5	cm^3	Volume =	613.5	cm^3	Volume =	733.8	cm^3
Moisture Content =	21.4%		Moisture Content =	21.4%		Moisture Content =	20.4%	
Specific Gravity =	-		Specific Gravity =	-		Specific Gravity =	-	
Dry Weight of Solids =	1014.17	g	Dry Weight of Solids =	1014.17	g	Dry Weight of Solids =	1279.40	g
Wet Unit Weight =	2.01	g/cm^3	Wet Unit Weight =	2.01	g/cm^3	Wet Unit Weight =	2.10	g/cm^3
Dry Unit Weight =	1.65	g/cm^3	Dry Unit Weight =	1.65	g/cm^3	Dry Unit Weight =	1.74	g/cm^3
Wet Unit Weight =	125.2	pcf	Wet Unit Weight =	125.2	pcf	Wet Unit Weight =	131.0	pcf
Dry Unit Weight =	103.1	pcf	Dry Unit Weight =	103.1	pcf	Dry Unit Weight =	108.8	pcf
Cell Pressure =	65	psi	Cell Pressure =	70	psi	Cell Pressure =	90	psi
Back Pressure =	60	psi	Back Pressure =	60	psi	Back Pressure =	70	psi
Confining Pressure =	5	psi	Confining Pressure =	10	psi	Confining Pressure =	20	psi

Notes: Intact sample with ends trimmed flush
Sample 2A was staged (points 1 & 2)

Golder Associates, Inc. **Denver, Colorado**	**Title:** **TRIAXIAL SHEAR TEST REPORT** **SAMPLE DATA AND CALCULATIONS**
Job Short Title: **GRE/2011 STANTON STAT ENG SRVC/ND**	

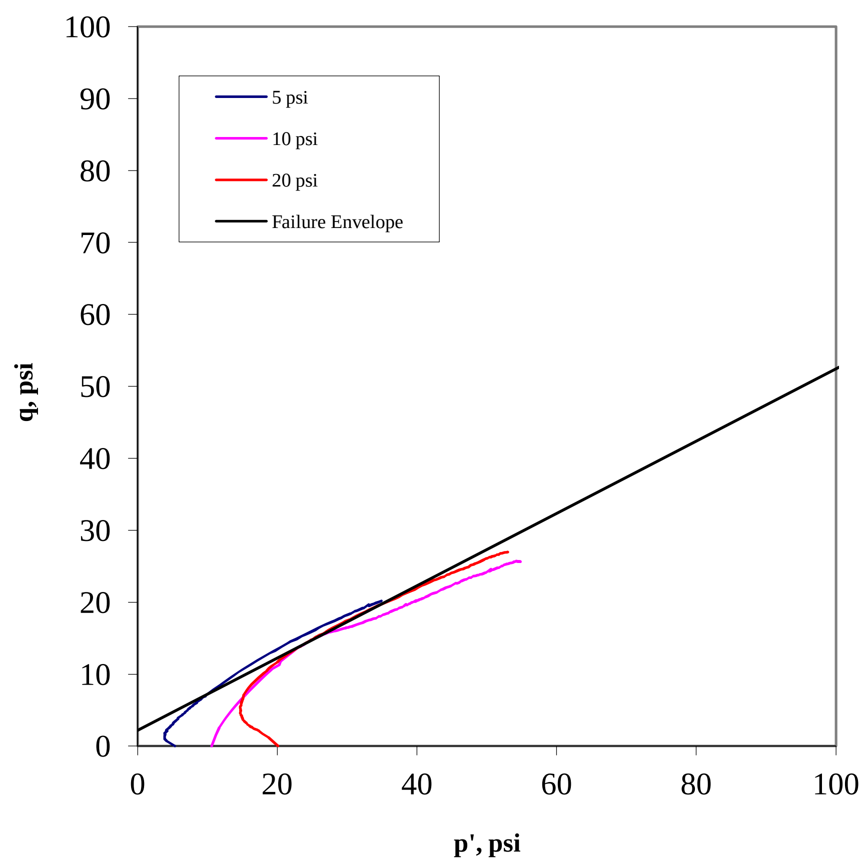
Sample Number:	Reviewed:	Date:	Job Number:	Figure:
Clay Fill 2	**TJS**	**11/21/11**	**113-81645**	**1**





Golder Associates, Inc. **Denver, Colorado**		**Title:** C-U TRIAXIAL SHEAR DATA q AND EXCESS PORE PRESSURE PLOTS		
Job Short Title: GRE/2011 STANTON STAT ENG SRVC/ND				
Sample Number: Clay Fill 2	**Reviewed:** TJS	**Date:** 11/21/11	**Job Number:** 113-81645	**Figure:** 2

Stress Path (p'-q) Plot

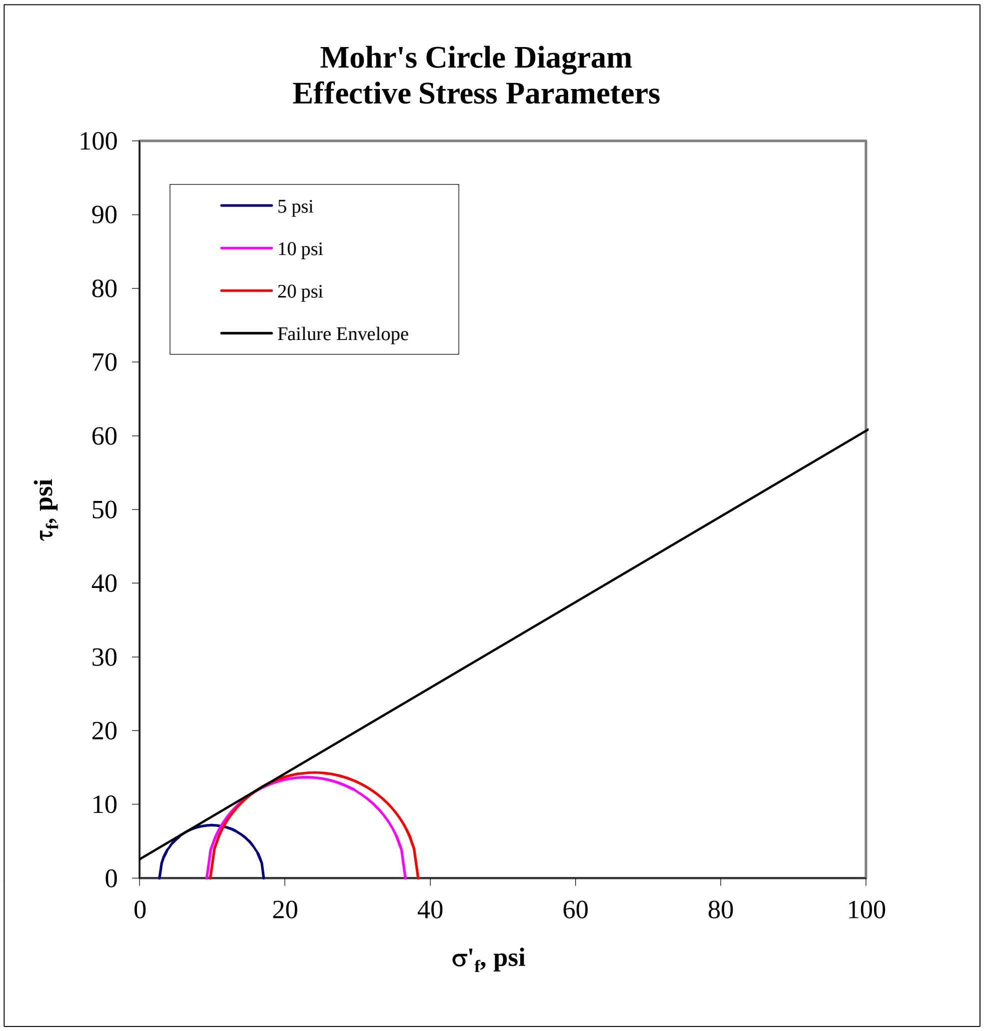


Stress Path Parameters

ψ' = 26.7 degrees

a' = 2.2 psi

Golder Associates, Inc. Denver, Colorado	Title: C-U TRIAXIAL SHEAR DATA STRESS PATH PLOT			
Job Short Title: GRE/2011 STANTON STAT ENG SRVC/ND				
Sample Number: Clay Fill 2	Reviewed: TJS	Date: 11/21/11	Job Number: 113-81645	Figure: 3



Effective Stress Shear Strength Parameters

$\phi' =$ **30.2 degrees**

c' = 2.5 psi

Golder Associates, Inc. **Denver, Colorado**		**Title:** C-U TRIAXIAL SHEAR DATA MOHR'S CIRCLE DIAGRAM		
Job Short Title: GRE/2011 STANTON STAT ENG SRVC/ND				
Sample Number: Clay Fill 2	**Reviewed:** TJS	**Date:** 11/21/11	**Job Number:** 113-81645	**Figure:** 4

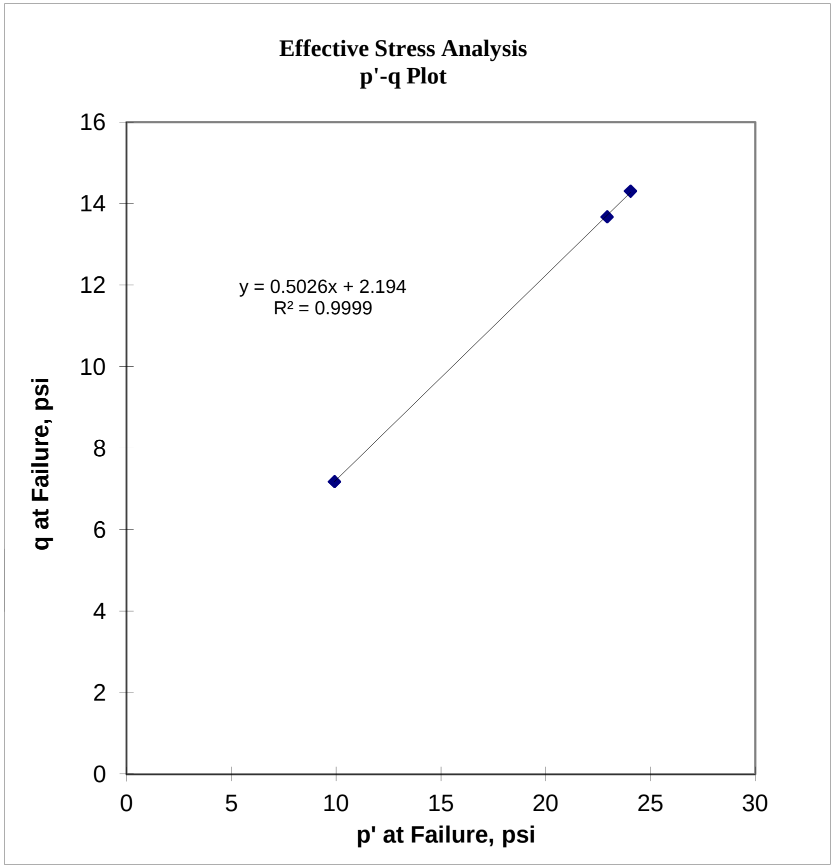
Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: **GRE/2011 STANTON STAT ENG SRVC/ND**
Project Number: **113-81645**

Sample Number	Clay Fill 2
Effective Stress Analysis	

Point Number	p' (psi)	q (psi)
1 (staged)	9.9	7.2
2 (staged)	22.9	13.7
3	24.0	14.3

tan(ψ') = **0.50**
a' = **2.2** **psi**

ϕ' = **30.2** **degrees**
c' = **2.5** **psi**



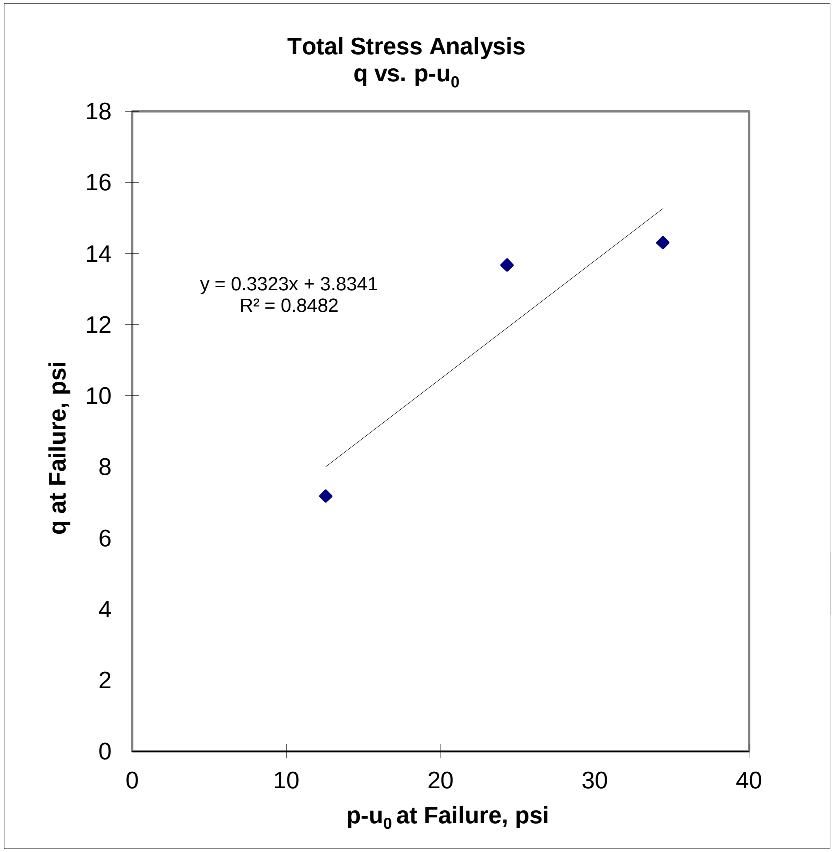
Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: **GRE/2011 STANTON STAT ENG SRVC/ND**
Project Number: **113-81645**

Sample Number	Clay Fill 2
Total Stress Analysis	

Point Number	$p-u_o$ (psi)	q (psi)
1 (staged)	12.5	7.2
2 (staged)	24.3	13.7
3	34.4	14.3

tan(ψ) = **0.33**
a = **3.8** **psi**

ϕ = **19.4** **degrees**
c = **4.1** **psi**



Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: GRE/2011 STANTON STAT ENG SRVC/ND
Project Number: 113-81645

Mohr-Coulomb Failure Criteria:

$$\tau_{ff} = c' + \sigma'_{ff} \tan(\phi')$$
$$\tau_{ff} = c + \sigma_{ff} \tan(\phi)$$

Where:
c', c = effective and total stress cohesion intercepts
ϕ', ϕ = effective and total stress friction angles
τ_{ff} = shear strength on the failure surface at failure
σ'_{ff}, σ_{ff} = effective and total normal stresses on the failure surface at failure

Stress Path Space:

$$q = \frac{\sigma_1 - \sigma_3}{2} \qquad p' = \frac{\sigma'_1 + \sigma'_3}{2} \qquad p = \frac{\sigma_1 + \sigma_3}{2}$$

Where:
q = maximum shear stress
p', p = mean effective and total stresses
σ'_1, σ_1 = effective and total axial stresses
σ'_3, σ_3 = effective and total confining stresses

Stress Path Failure Criteria:

$$q = a' + p' \tan(\psi')$$
$$q = a + (p - u_0) \tan(\psi)$$

Where:
a', a = intercepts of the q-axis in effective stress and total stress spaces
ψ', ψ = angles of the failure envelopes in effective stress and total stress spaces
q = maximum shear stress at failure
p' = mean effective stress at failure
$p-u_0$ = mean total stress at failure minus the initial pore pressure

The relationships between ψ and ϕ and a and c are as follows:

$$\tan(\psi) = \sin(\phi)$$
$$a = c \cos(\phi)$$

The relationships between ψ' and ϕ' and a' and c' are as follows:

$$\tan(\psi') = \sin(\phi')$$
$$a' = c' \cos(\phi')$$

Sample 2A
Staged 5 & 10psi

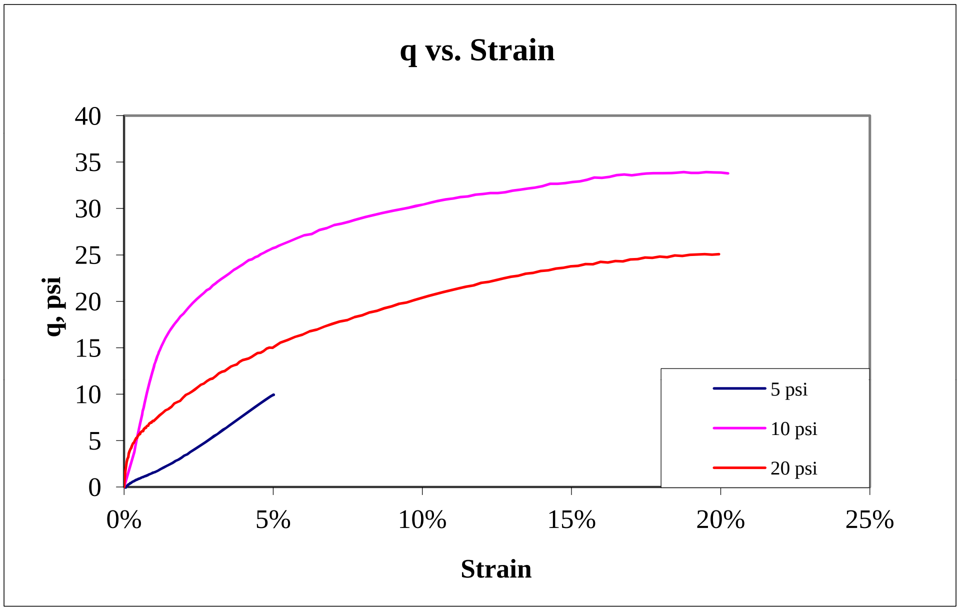
Sample 2B
20psi

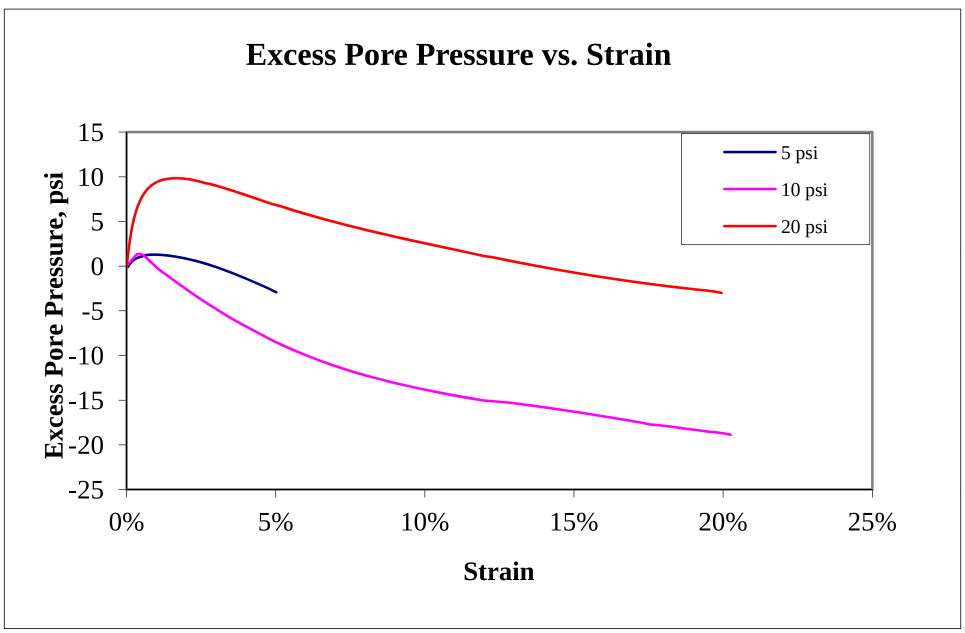
SILT FILL

Boring =	# 2		Boring =	# 2		Boring =	# 2	
Sample # =	4A		Sample # =	4A		Sample # =	3A	
Depth (ft) =	5.0-7.0		Depth (ft) =	5.0-7.0		Depth (ft) =	10.0-12.0	
Point # =	1 (staged)		Point # =	2 (staged)		Point # =	3	
	Initial			**Initial**			**Initial**	
Length =	15.50	cm	Length =	14.71	cm	Length =	17.10	cm
Diameter =	7.30	cm	Diameter =	7.47	cm	Diameter =	7.27	cm
Wet Weight =	1356.60	g	Wet Weight =	1356.60	g	Wet Weight =	1398.00	g
Area =	41.9	cm^2	Area =	43.8	cm^2	Area =	41.5	cm^2
Sample Area =	6.49	in^2	Sample Area =	6.79	in^2	Sample Area =	6.43	in^2
Volume =	648.7	cm^3	Volume =	644.7	cm^3	Volume =	709.8	cm^3
Moisture Content =	13.6%		Moisture Content =	13.6%		Moisture Content =	13.0%	
Specific Gravity =	na		Specific Gravity =	na		Specific Gravity =	na	
Dry Weight of Solids =	1194.19	g	Dry Weight of Solids =	1194.19	g	Dry Weight of Solids =	1237.17	g
Wet Unit Weight =	2.09	g/cm^3	Wet Unit Weight =	2.10	g/cm^3	Wet Unit Weight =	1.97	g/cm^3
Dry Unit Weight =	1.84	g/cm^3	Dry Unit Weight =	1.85	g/cm^3	Dry Unit Weight =	1.74	g/cm^3
Wet Unit Weight =	130.5	pcf	Wet Unit Weight =	131.3	pcf	Wet Unit Weight =	122.9	pcf
Dry Unit Weight =	114.9	pcf	Dry Unit Weight =	115.6	pcf	Dry Unit Weight =	108.8	pcf
Cell Pressure =	65	psi	Cell Pressure =	70	psi	Cell Pressure =	90	psi
Back Pressure =	60	psi	Back Pressure =	60	psi	Back Pressure =	70	psi
Confining Pressure =	5	psi	Confining Pressure =	10	psi	Confining Pressure =	20	psi

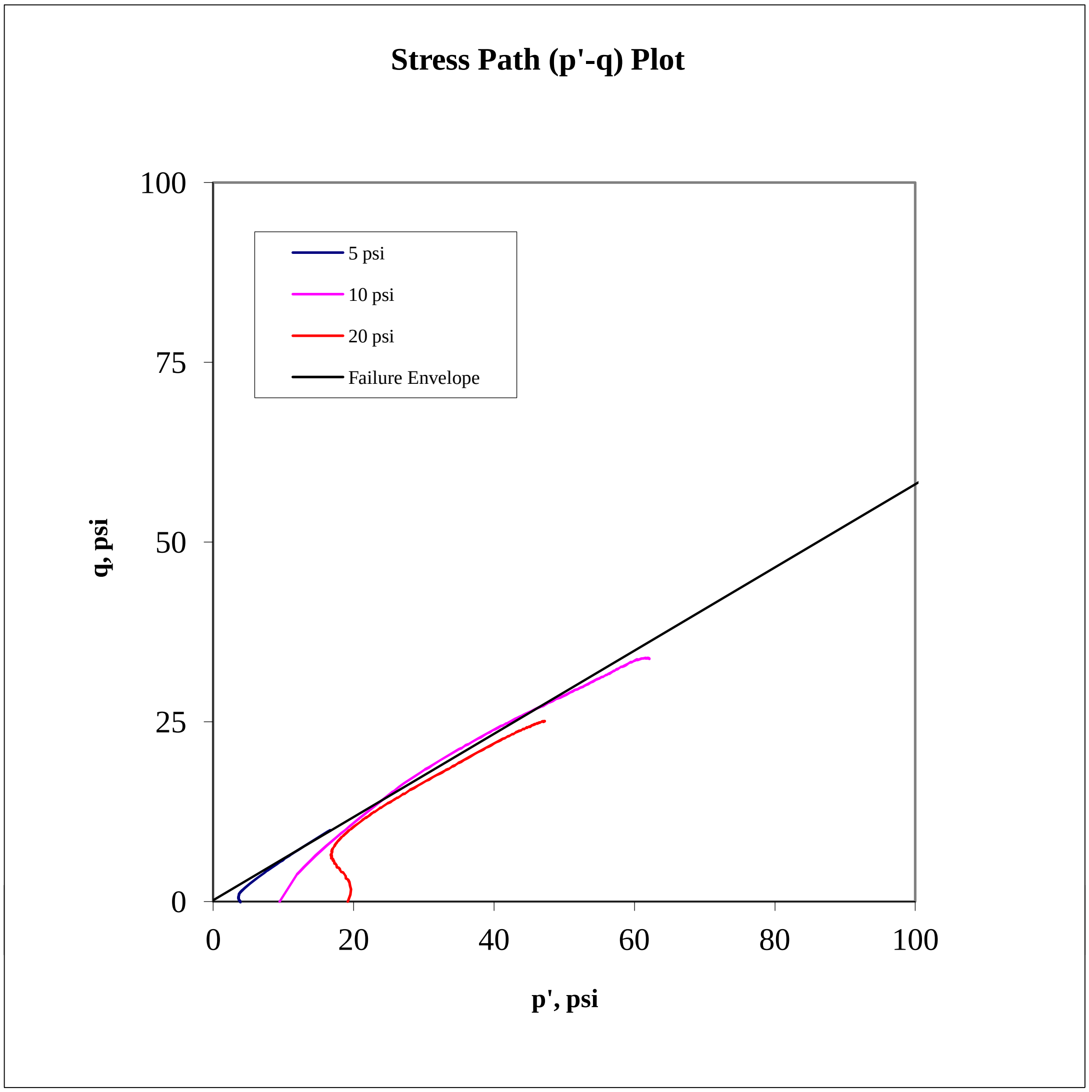
Notes: Undisturbed core sample
Points 1 and 2 were staged

Golder Associates Inc. **Denver, Colorado**	**Title:** **TRIAXIAL SHEAR TEST REPORT** **SAMPLE DATA AND CALCULATIONS**			
Job Short Title: **GRE/2011 STANTON STAT ENG SRVC/ND**				
Sample Number: **Silt Fill**	**Reviewed:** **TJS**	**Date:** **12/7/11**	**Job Number:** **113-81645**	**Figure:** **1**





Golder Associates Inc. **Denver, Colorado**		**Title:** C-U TRIAXIAL SHEAR DATA q AND EXCESS PORE PRESSURE PLOTS		
Job Short Title: GRE/2011 STANTON STAT ENG SRVC/ND				
Sample Number: Silt Fill	**Reviewed:** TJS	**Date:** 12/7/11	**Job Number:** 113-81645	**Figure:** 2

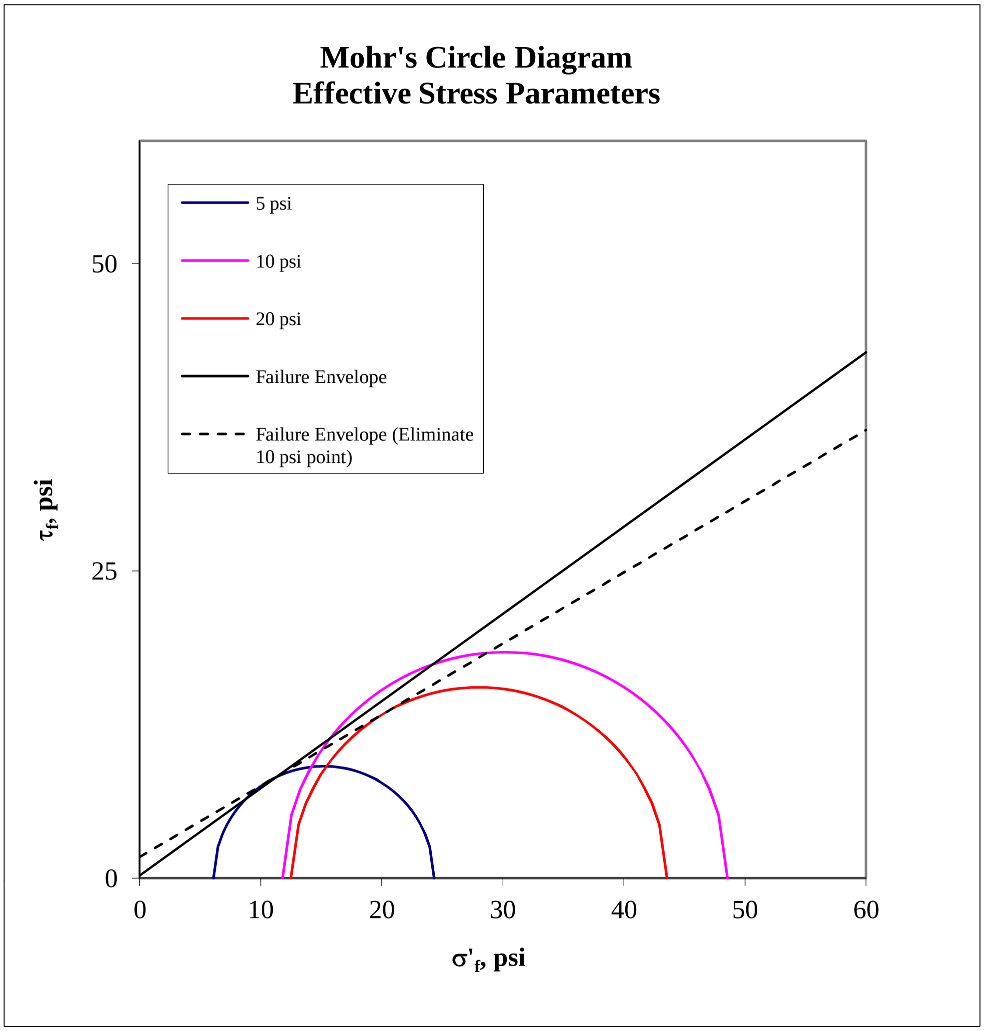


Stress Path Parameters

ψ' = 30.1 degrees
a' = 0.2 psi

Golder Associates Inc. Denver, Colorado	Title: C-U TRIAXIAL SHEAR DATA STRESS PATH PLOT
Job Short Title: GRE/2011 STANTON STAT ENG SRVC/ND	

Sample Number:	Reviewed:	Date:	Job Number:	Figure:
Silt Fill	TJS	12/7/11	113-81645	3



Effective Stress Shear Strength Parameters

ϕ' = 35.4 degrees

c' = 0.2 psi

Golder Associates Inc. **Denver, Colorado**		**Title:** **C-U TRIAXIAL SHEAR DATA** **MOHR'S CIRCLE DIAGRAM**		
Job Short Title: **GRE/2011 STANTON STAT ENG SRVC/ND**				
Sample Number: **Silt Fill**	**Reviewed:** **TJS**	**Date:** **12/7/11**	**Job Number:** **113-81645**	**Figure:** **4**

Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: **GRE/2011 STANTON STAT ENG SRVC/ND**
Project Number: **113-81645**

Sample Number	Silt Fill
Effective Stress Analysis	

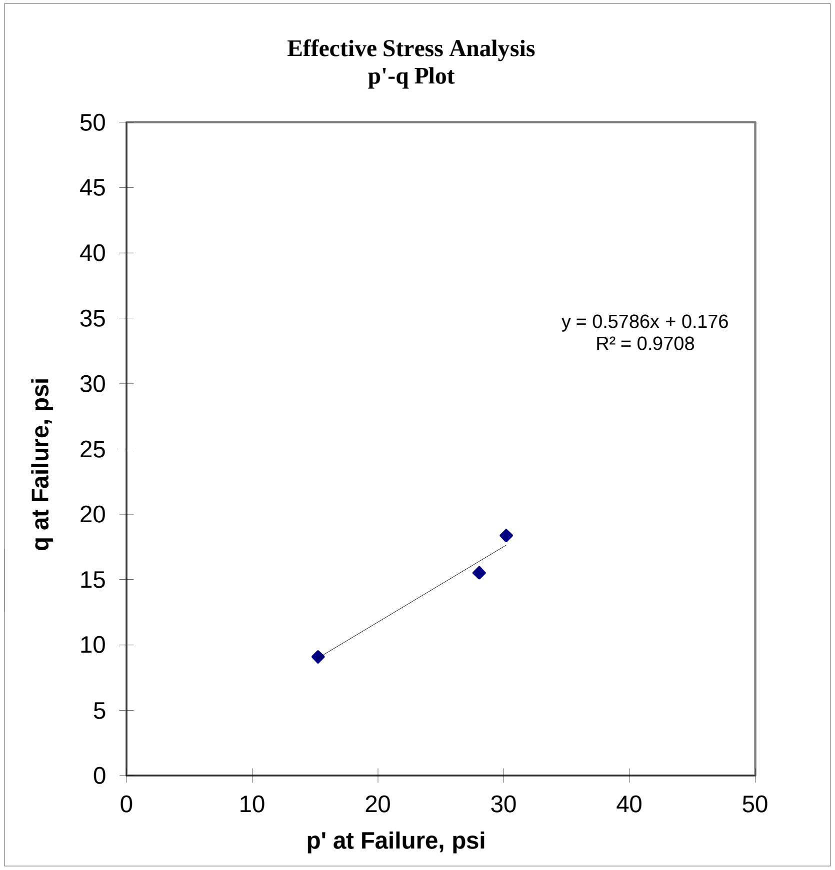
Point Number	p' (psi)	q (psi)
1 (staged)	15.2	9.1
2 (staged)	30.2	18.4
3	28.0	15.5

tan(ψ') = 0.58
a' = 0.2 psi

ϕ' = 35.4 degrees
c' = 0.2 psi

If eliminate the second point at 10 psi.
tan(ψ') = 0.50
a' = 1.5 psi

ϕ' = 30.1 degrees
c' = 1.7 psi



Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: **GRE/2011 STANTON STAT ENG SRVC/ND**
Project Number: **113-81645**

Sample Number	Silt Fill
Total Stress Analysis	

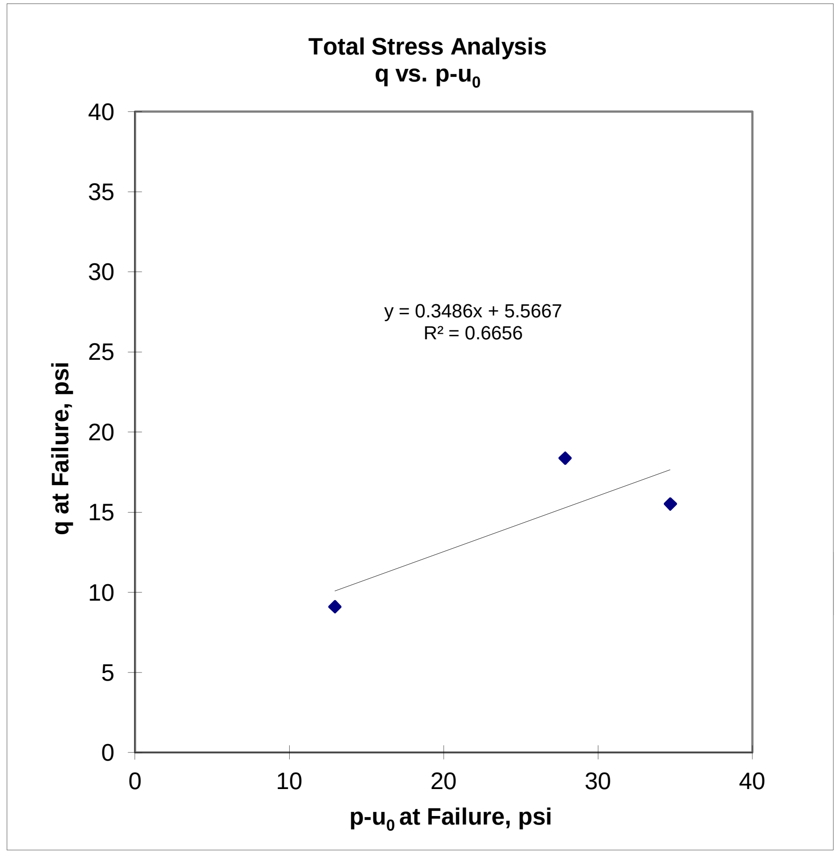
Point Number	$p-u_o$ (psi)	q (psi)
1 (staged)	12.9	9.1
2 (staged)	27.9	18.4
3	34.7	15.5

tan(ψ) = 0.35
a = 5.6 psi

ϕ = 20.4 degrees
c = 5.9 psi

If eliminate the second point at 10 psi.
tan(ψ) = 0.30
a = 5.3 psi

ϕ = 17.2 degrees
c = 5.5 psi



Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: GRE/2011 STANTON STAT ENG SRVC/ND
Project Number: 113-81645

Mohr-Coulomb Failure Criteria:

$$\tau_{ff} = c' + \sigma'_{ff} \tan(\phi')$$
$$\tau_{ff} = c + \sigma_{ff} \tan(\phi)$$

Where:
c', c = effective and total stress cohesion intercepts
ϕ', ϕ = effective and total stress friction angles
τ_{ff} = shear strength on the failure surface at failure
σ'_{ff}, σ_{ff} = effective and total normal stresses on the failure surface at failure

Stress Path Space:

$$q = \frac{\sigma_1 - \sigma_3}{2} \qquad p' = \frac{\sigma'_1 + \sigma'_3}{2} \qquad p = \frac{\sigma_1 + \sigma_3}{2}$$

Where:
q = maximum shear stress
p', p = mean effective and total stresses
σ'_1, σ_1 = effective and total axial stresses
σ'_3, σ_3 = effective and total confining stresses

Stress Path Failure Criteria:

$$q = a' + p' \tan(\psi')$$
$$q = a + (p - u_0) \tan(\psi)$$

Where:
a', a = intercepts of the q-axis in effective stress and total stress spaces
ψ', ψ = angles of the failure envelopes in effective stress and total stress spaces
q = maximum shear stress at failure
p' = mean effective stress at failure
$p-u_0$ = mean total stress at failure minus the initial pore pressure

The relationships between ψ and ϕ and a and c are as follows:

$$\tan(\psi) = \sin(\phi)$$
$$a = c \cos(\phi)$$

The relationships between ψ' and ϕ' and a' and c' are as follows:

$$\tan(\psi') = \sin(\phi')$$
$$a' = c' \cos(\phi')$$

Sample 4a
Staged 5 & 10psi

Sample 3a
20psi

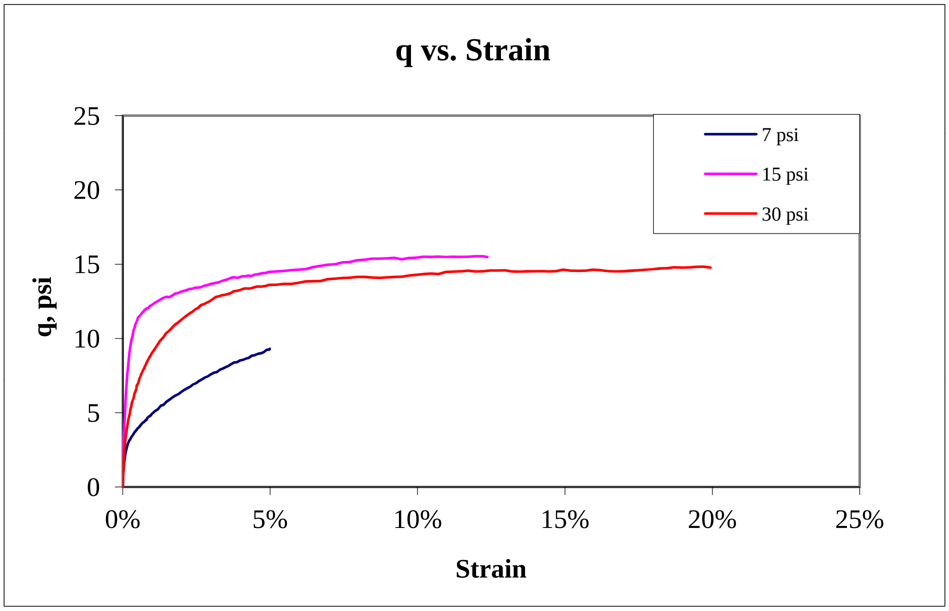
NATURAL CLAY

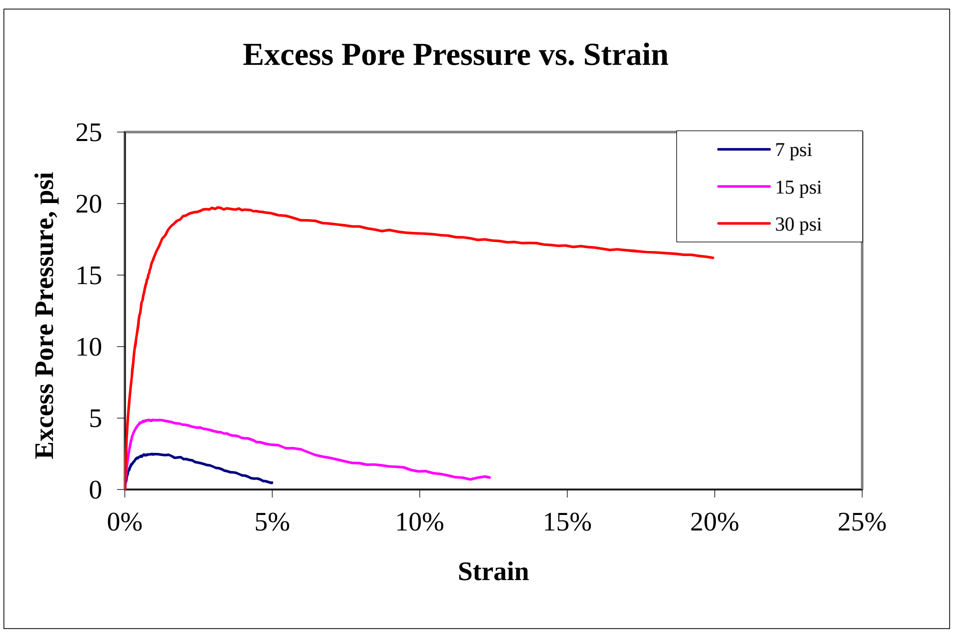
Boring # =	Natural Clay # 1		Boring # =	Natural Clay # 1		Boring # =	Natural Clay # 1	
Sample # =	5A		Sample # =	5A		Sample # =	5B	
Depth (ft) =	20.0-22.0		Depth (ft) =	20.0-22.0		Depth (ft) =	20.0-22.0	
Point # =	1 (staged)		Point # =	2 (staged)		Point # =	3	
	Initial			**Initial**			**Initial**	
Length =	15.15	cm	Length =	15.15	cm	Length =	15.40	cm
Diameter =	7.29	cm	Diameter =	7.29	cm	Diameter =	7.28	cm
Wet Weight =	1247.40	g	Wet Weight =	1247.40	g	Wet Weight =	1216.30	g
Area =	41.7	cm^2	Area =	41.7	cm^2	Area =	41.6	cm^2
Sample Area =	6.47	in^2	Sample Area =	6.47	in^2	Sample Area =	6.45	in^2
Volume =	632.3	cm^3	Volume =	632.3	cm^3	Volume =	641.0	cm^3
Moisture Content =	25.8%		Moisture Content =	25.8%		Moisture Content =	25.8%	
Specific Gravity =	-		Specific Gravity =	-		Specific Gravity =	-	
Dry Weight of Solids =	991.57	g	Dry Weight of Solids =	991.57	g	Dry Weight of Solids =	966.85	g
Wet Unit Weight =	1.97	g/cm^3	Wet Unit Weight =	1.97	g/cm^3	Wet Unit Weight =	1.90	g/cm^3
Dry Unit Weight =	1.57	g/cm^3	Dry Unit Weight =	1.57	g/cm^3	Dry Unit Weight =	1.51	g/cm^3
Wet Unit Weight =	123.1	pcf	Wet Unit Weight =	123.1	pcf	Wet Unit Weight =	118.4	pcf
Dry Unit Weight =	97.9	pcf	Dry Unit Weight =	97.9	pcf	Dry Unit Weight =	94.1	pcf
Cell Pressure =	37	psi	Cell Pressure =	45	psi	Cell Pressure =	70	psi
Back Pressure =	30	psi	Back Pressure =	30	psi	Back Pressure =	40	psi
Confining Pressure =	7	psi	Confining Pressure =	15	psi	Confining Pressure =	30	psi

Notes:
Visually observed as dark olive brown silty clay.
Intact sample; ends trimmed flush
Rate of strain = 4.364%/hour; t_{50} = 5.5 minutes.
Points 1 and 2 were staged.
Point 2 lost cell pressure during test; results shown to 12.4% strain

Golder Associates, Inc. **Denver, Colorado**	**Title:** **TRIAXIAL SHEAR TEST REPORT** **SAMPLE DATA AND CALCULATIONS**
Job Short Title: **GRE/2011 STANTON STAT ENG SRVC/ND**	

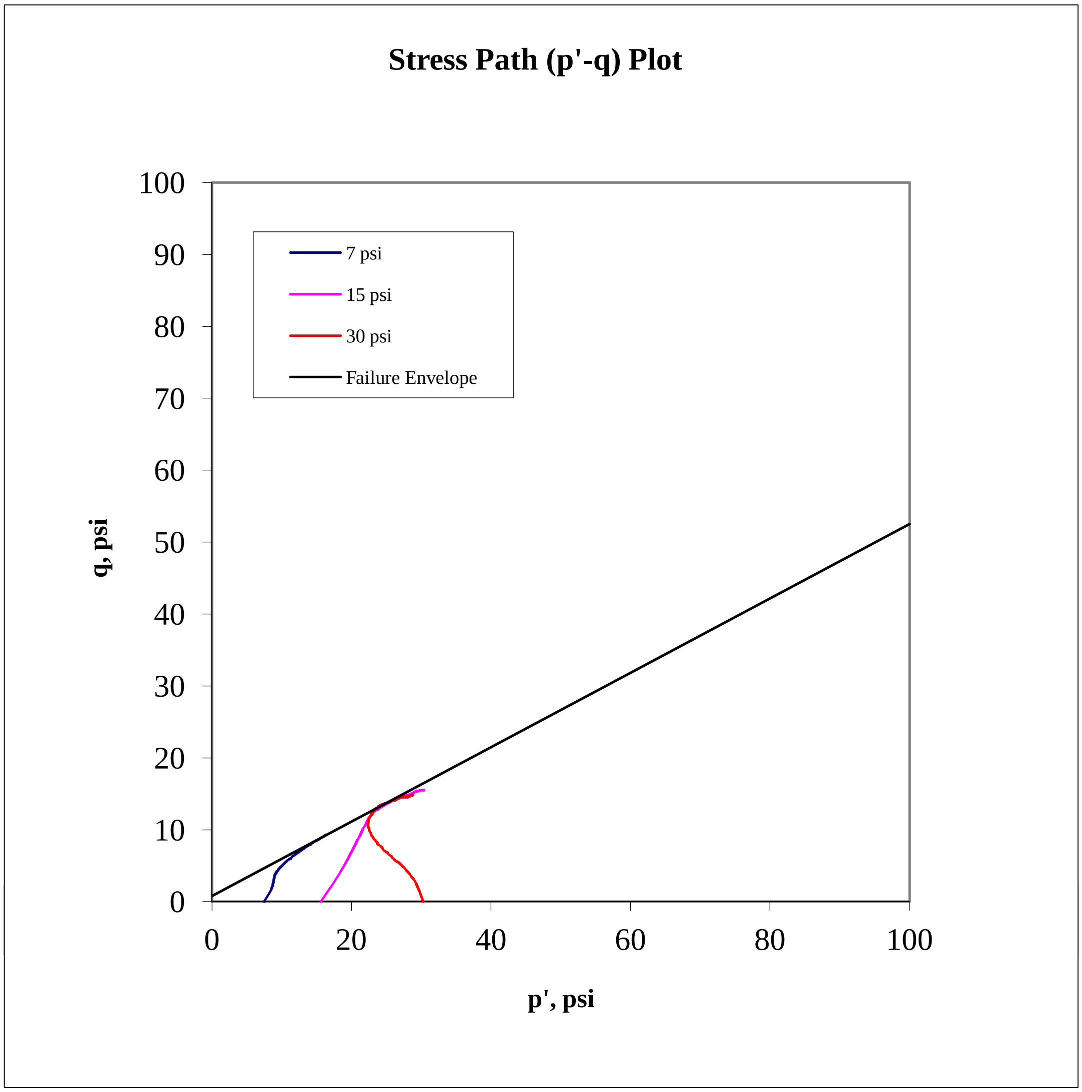
Sample Number:	**Reviewed:**	**Date:**	**Job Number:**	**Figure:**
Natural Clay # 1 / 5A and 5B	**TJS**	**12/13/11**	**113-81645**	**1**





Golder Associates, Inc. **Denver, Colorado**	**Title:** **C-U TRIAXIAL SHEAR DATA** **q AND EXCESS PORE PRESSURE PLOTS**
Job Short Title: **GRE/2011 STANTON STAT ENG SRVC/ND**	

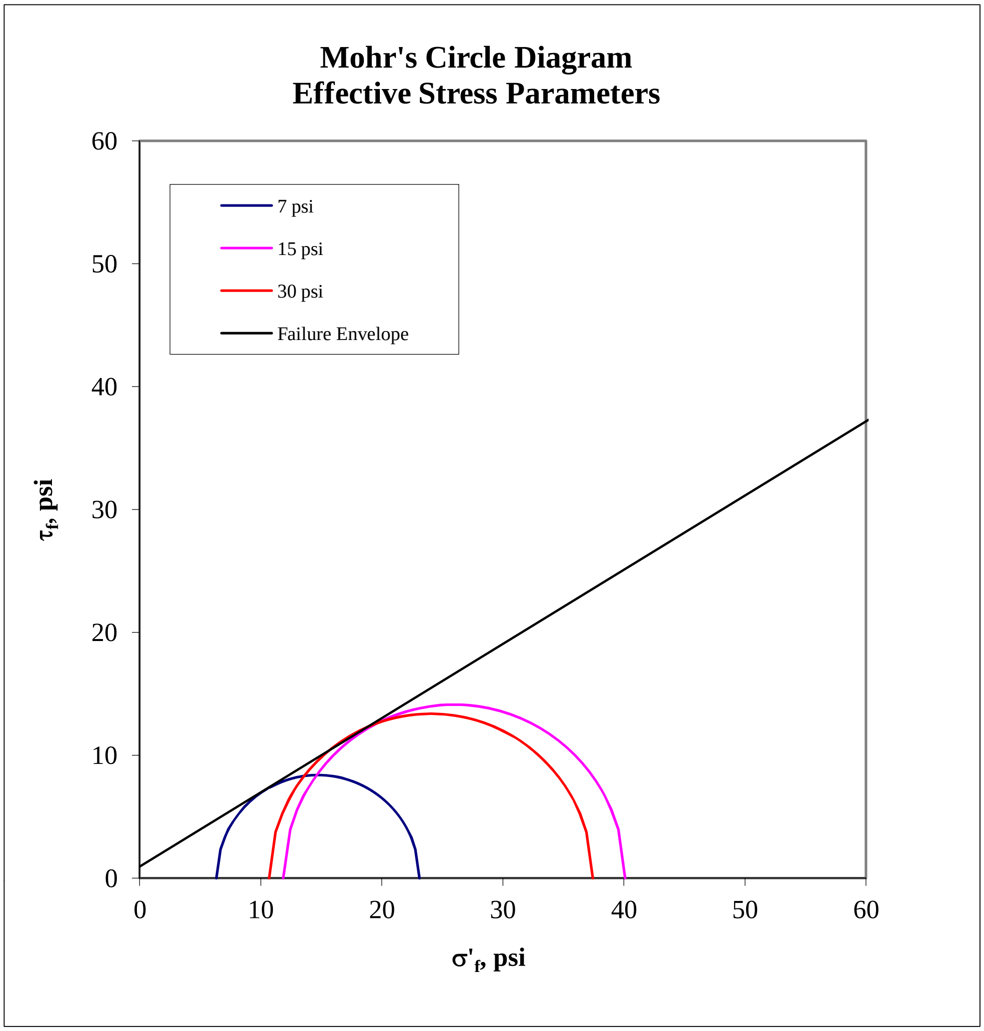
Sample Number:	**Reviewed:**	**Date:**	**Job Number:**	**Figure:**
Natural Clay # 1 / 5A and 5B	**TJS**	**12/13/11**	**113-81645**	**2**



Stress Path Parameters

ψ' = 27.3 degrees
a' = 0.8 psi

Golder Associates, Inc. Denver, Colorado	**Title:** C-U TRIAXIAL SHEAR DATA STRESS PATH PLOT			
Job Short Title: GRE/2011 STANTON STAT ENG SRVC/ND				
Sample Number: Natural Clay # 1 / 5A and 5B	**Reviewed:** TJS	**Date:** 12/13/11	**Job Number:** 113-81645	**Figure:** 3



Effective Stress Shear Strength Parameters

ϕ' = 31.1 degrees
c' = 0.9 psi

Golder Associates, Inc. **Denver, Colorado**	**Title:**
Job Short Title: **GRE/2011 STANTON STAT ENG SRVC/ND**	**C-U TRIAXIAL SHEAR DATA** **MOHR'S CIRCLE DIAGRAM**

Sample Number:	Reviewed:	Date:	Job Number:	Figure:
Natural Clay # 1 / 5A and 5B	TJS	12/13/11	113-81645	4

Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: **GRE/2011 STANTON STAT ENG SRVC/ND**
Project Number: **113-81645**

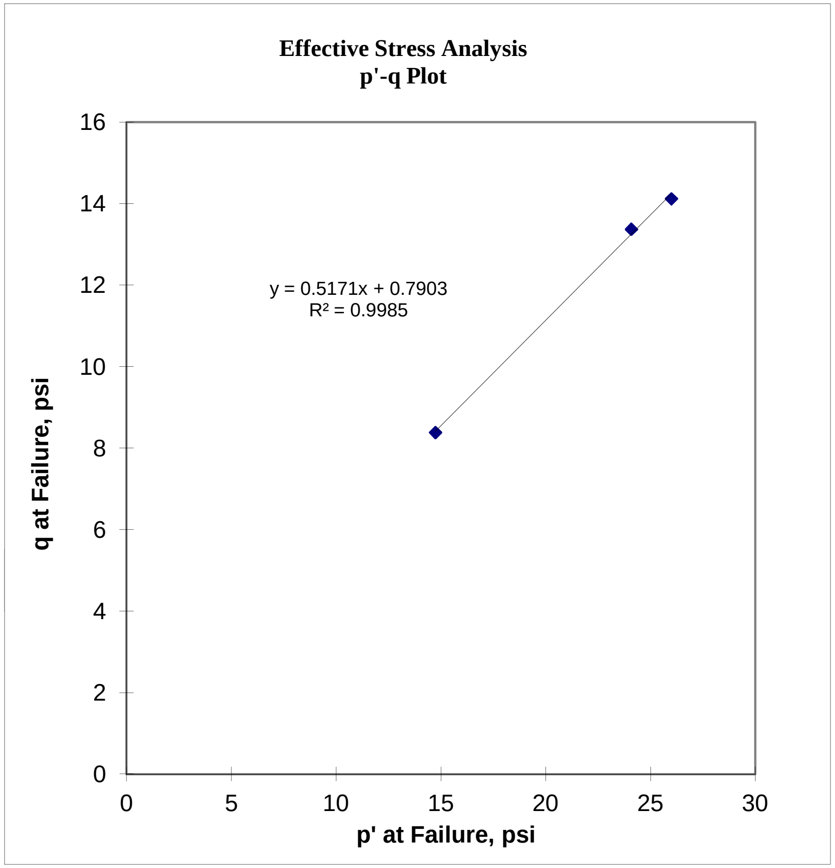
Sample Number Natural Clay # 1 / 5A and 5E

Effective Stress Analysis

Point Number	p' (psi)	q (psi)
1 (staged)	14.7	8.4
2 (staged)	26.0	14.1
3	24.1	13.4

tan(ψ') = **0.52**
a' = **0.8** **psi**

ϕ' = **31.1** **degrees**
c' = **0.9** **psi**



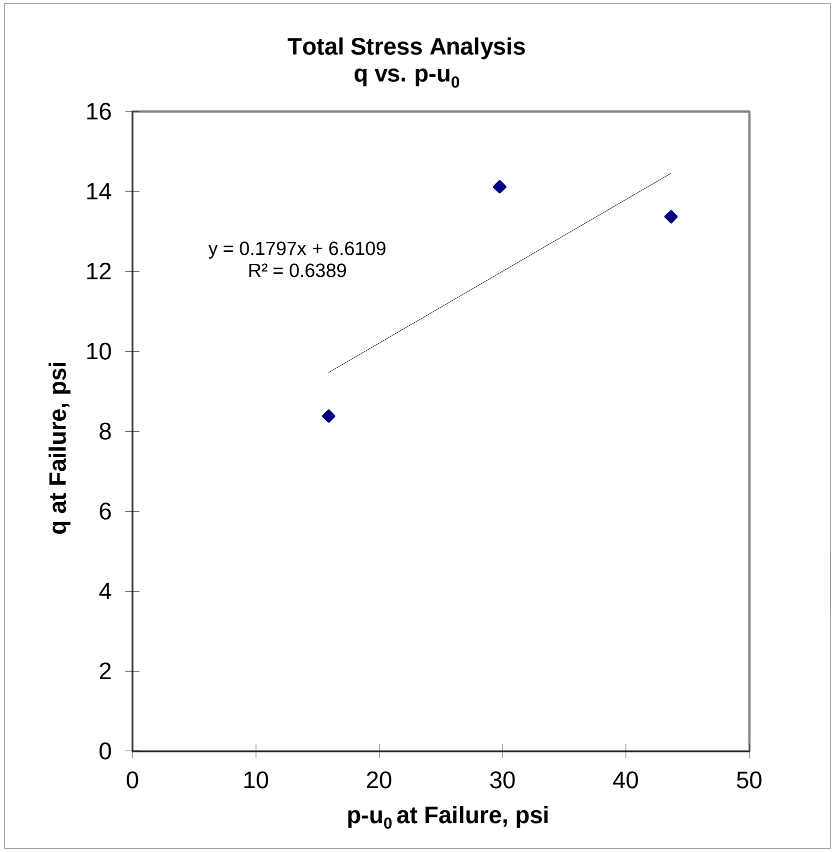
Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: **GRE/2011 STANTON STAT ENG SRVC/ND**
Project Number: **113-81645**

Sample Number	Natural Clay # 1 / 5A and 5B
Total Stress Analysis	

Point Number	$p-u_o$ (psi)	q (psi)
1 (staged)	15.9	8.4
2 (staged)	29.7	14.1
3	43.6	13.4

tan(ψ) = 0.18
a = 6.6 psi

ϕ = 10.4 degrees
c = 6.7 psi



Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
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Mohr-Coulomb Failure Criteria:

$$\tau_{ff} = c' + \sigma'_{ff} \tan(\phi')$$
$$\tau_{ff} = c + \sigma_{ff} \tan(\phi)$$

Where:
c’, c = effective and total stress cohesion intercepts

ϕ', ϕ = effective and total stress friction angles

τ_{ff} = shear strength on the failure surface at failure

σ'_{ff}, σ_{ff} = effective and total normal stresses on the failure surface at failure

Stress Path Space:

$$q = \frac{\sigma_1 - \sigma_3}{2} \qquad p' = \frac{\sigma'_1 + \sigma'_3}{2} \qquad p = \frac{\sigma_1 + \sigma_3}{2}$$

Where:
q = maximum shear stress

p’, p = mean effective and total stresses
σ'_1, σ_1 = effective and total axial stresses

σ'_3, σ_3 = effective and total confining stresses

Stress Path Failure Criteria:

$$q = a' + p' \tan(\psi')$$
$$q = a + (p - u_0) \tan(\psi)$$

Where:
a’, a = intercepts of the q-axis in effective stress and total stress spaces

ψ', ψ = angles of the failure envelopes in effective stress and total stress spaces

q = maximum shear stress at failure

p’ = mean effective stress at failure

$p-u_0$ = mean total stress at failure minus the initial pore pressure

The relationships between ψ and ϕ and a and c are as follows:

$$\tan(\psi) = \sin(\phi)$$
$$a = c \cos(\phi)$$

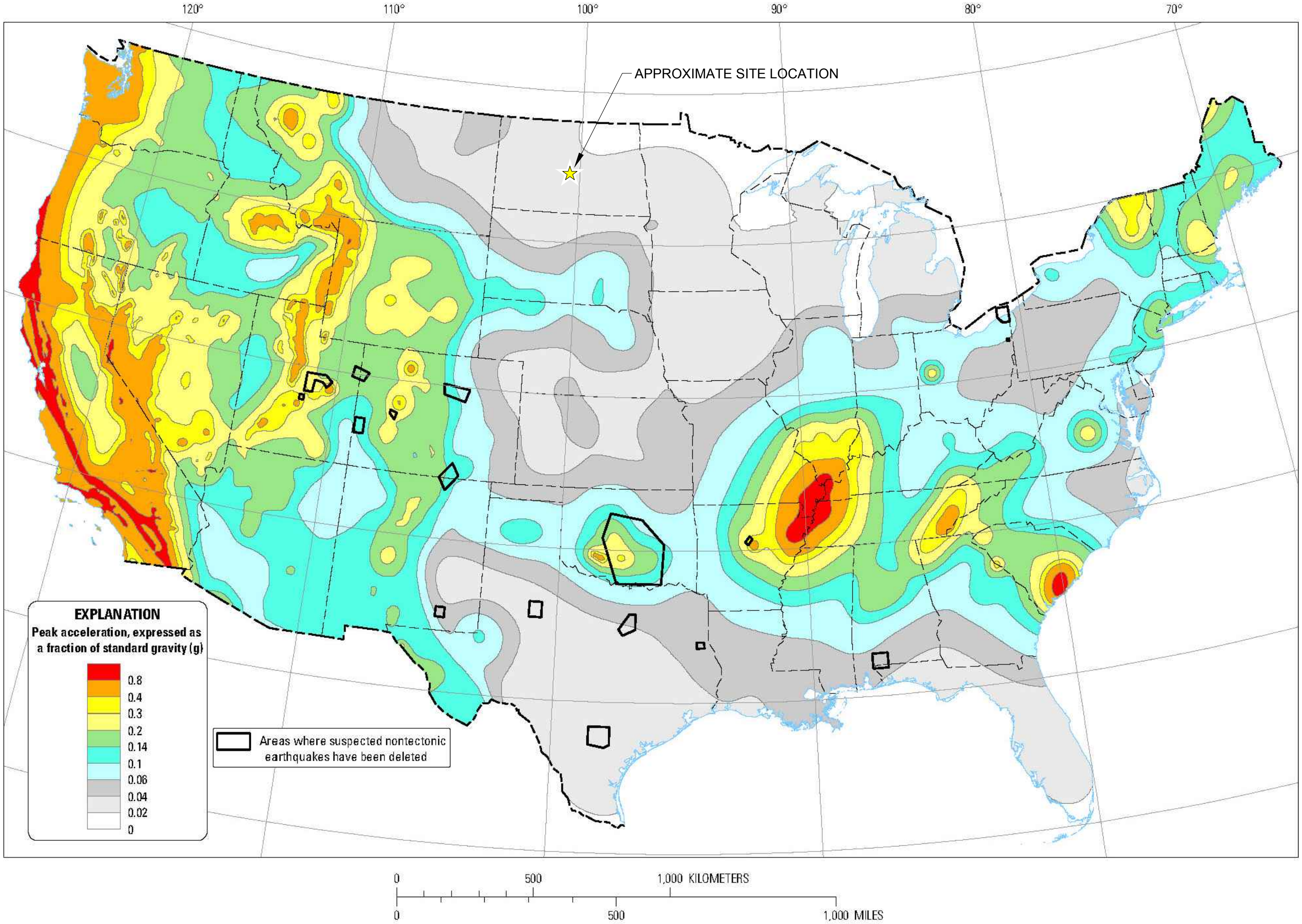
The relationships between ψ' and ϕ' and a’ and c’ are as follows:

$$\tan(\psi') = \sin(\phi')$$
$$a' = c' \cos(\phi')$$

Sample 5A
Staged 7 & 15psi

Sample 5B
30psi

APPENDIX B
SEISMIC LOADING CONDITIONS



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

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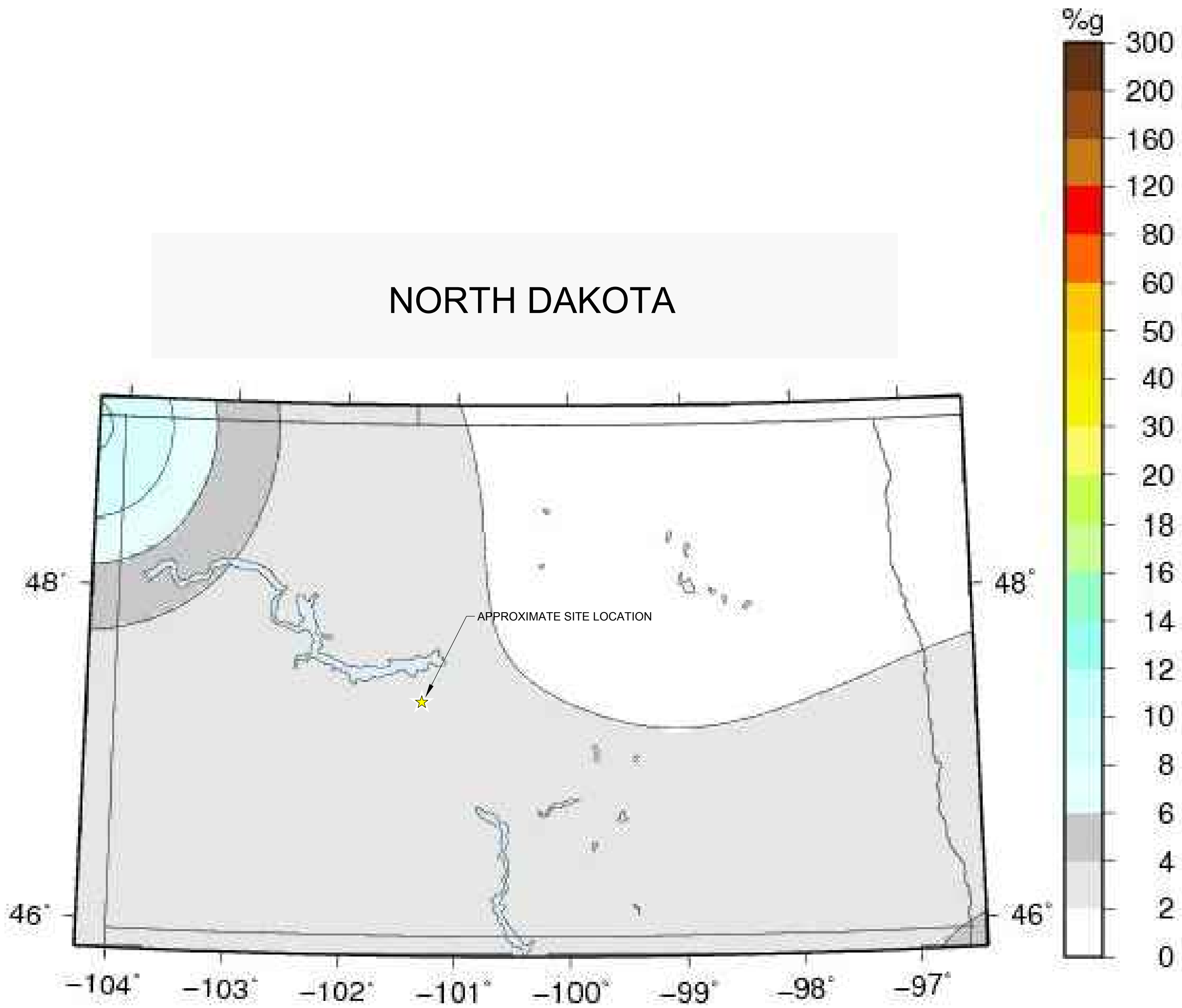

2% PROBABILITY OF EXCEEDANCE IN 50 YEARS MAP OF PEAK GROUND ACCELERATION
USGS SEISMIC HAZARD MAP (2008)

Golder Associates

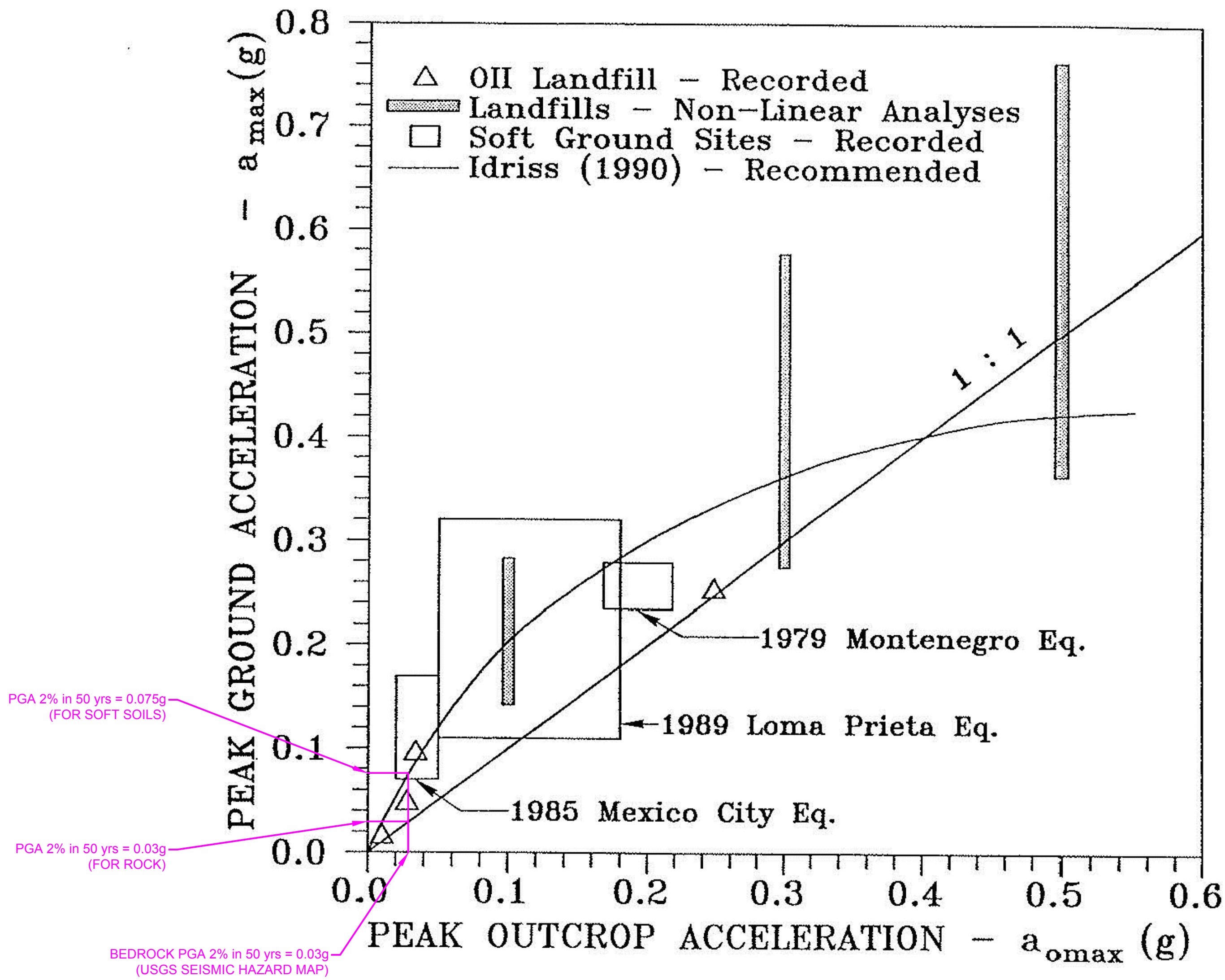

Figure 4.5 Observed Variations of Peak Horizontal Accelerations on Soft Soil and MSW Sites in Comparison to Rock Sites (Kavazanjian and Matasović, 1994).

COMPARISON OF PEAK GROUND ACCELERATION AND PEAK OUTCROP ACCELERATION

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FIGURE B-3

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